SMITH CANAL GATE

HYDRODYNAMIC MODELING

ALIGNMENT AND GATE WIDTH EVALUATION

FINAL REPORT

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LIST OF ACRONYMS

CDEC	California Data Exchange Center
CDWR	California Department of Water Resources
cfs	Cubic foot per second
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute
DSM2	Delta Simulation Model II
FEMA	Federal Emergency Management Agency
fps	Feet per second
KSN	Kjeldsen, Sinnock, Neudeck, Inc.
MHHW	Mean Higher High Water
MHW	Mean High Water
MLLW	Mean Lower Low Water
MLW	Mean Low Water
MTL	Mean Tide Level
NAVD88	North American Vertical Datum of 1988
NFIP	National Flood Insurance Program
NRMSE	Normalized Root Mean Square Error
PBI	Peterson Brustad, Inc.
RMSE	Root-Mean-Square Error
RRI	Rough and Ready Island
RRI Gage	Rough and Ready Island Gage
SDWSC	Stockton Deep Water Ship Channel
SGCC	Stockton Golf and Country Club
SJG Gage	San Joaquin River Gage
USGS	United States Geological Service
UTM	Universal Transverse Mercator
WSE	Water Surface Elevation

EXECUTIVE SUMMARY

Smith Canal is located in Stockton, CA off the Stockton Deep Water Ship Channel (SDWSC). Levees along Smith Canal and Atherton Cove serve to prevent back flooding from the San Joaquin Delta during periods of elevated runoff occurring at high tides. Some sections of the levee are encroached upon by existing property along the Canal and Atherton Cove. This precludes the levee sections from being accredited by Federal Emergency Management Agency (FEMA) according to today's standards, which require access to and along the levee for inspection and maintenance purposes.

In order to develop a solution capable of meeting the minimum management criteria of the National Flood Insurance Program (NFIP), FEMA has acknowledged that incorporation of a closure structure at the entrance to Smith Canal would enable the adjoining area to meet FEMA flood protection requirements.

Three closure structure alternative alignments are proposed. These alternatives are:

- Alignment 1: This alignment incorporates a floodwall between Dad's Point and the Stockton Golf and Country Club (SGCC), tying into the existing FEMA-accredited levee. The floodwall will consist of a sheet-pile wall featuring a 50-ft wide gate for access to Atherton Cove and Smith Canal. An additional levee raising would be located between Dad's Point Levee and Louis Park (Figure 1-5).

- Alignment 2: The improvements would consist primarily of bringing the existing levee and terrain up to the elevation required for flood protection. The levee improvements would start at the termination of the existing FEMA-accredited levee along the shore of the SGCC, and terminate at the shore of Smith Canal. A floodwall with a 50-ft wide gate would be incorporated across Smith Canal in order to provide flood protection of the areas surrounding Smith Canal (Figure 1-7).

- Alignment 3: This alternative incorporates a floodwall across the entrance to Atherton Cove between the western tip of Atherton Island and across to the SGCC, where the double sheet-pile floodwall would tie into the existing FEMA-accredited levee along the shore of the SGCC. A second double sheet-pile floodwall and gate would be incorporated across the entrance to Smith Canal. This solution requires the incorporation of two gate structures, one at the entrance to Atherton Cove, and another at the entrance to Smith Canal (Figure 1-8).

Four Water Quality Improvement Options for the Alignment Alternative 1 were also considered and analyzed. These four options are:

- Option 1: Culverts under Moreing Road to Atherton Island – propose to install three 4-ft diameter culverts with an invert elevation of -2 ft NAVD88 under Moreing Road.

- Option 2: Dredging Atherton Cove - propose to dredge the bed elevation in the Cove to an elevation of -4 ft NAVD88.

- Option 3: Combination of Configuration Options #1 and #2 - with both dredging of Atherton Cove and culverts under Moreing Road.

- Option 4: Culverts in Floodwall - Propose to install five 4-ft diameter culverts with an invert elevation of -4 ft NAVD88 within the floodwall.

The objective of the present work focuses on hydrodynamic evaluation of the three proposed closure structure alignments, sensitivity evaluation of gate widths, and effective assessment of the Water Quality Improvement Options. Analysis included evaluation of the tidal flow velocities at the proposed gate locations, flow patterns within Smith Canal and Atherton Cove, and potential impacts to residence time.

MIKE21-FM modeling suite was selected for this study. The hydrodynamic model (MIKE21-FM-HD) was successfully calibrated with stages and discharges measured at gages located in SDWSC and San Joaquin River. The transport model (MIKE21-FM-TR) was calibrated with a dye test conducted in Smith Canal in 2004. The calibrated models were then applied for alternative alignment analyses.

Overall, the proposed closure structures do not create any significant change to the stage in Smith Canal and Atherton Cove and discharge through the entrances to Smith Canal and Atherton Cove. However, the hydrodynamic model results indicate that introduction of the closure structure leads to localized changes in the flow field within the vicinity of the gate. For Alternative Alignment 1 with a 50-ft wide gate, peak spring tidal velocities through the gate reach approximately 2.4 ft per second (fps). However, the spring tide high velocity only occurs a few times a year and the duration of high tidal velocity is relatively short. The duration of the peak velocity exceeding 2.0 fps is less than 3% of the time and that exceeding 1.5 fps is about 10% of the time. For remaining alternatives, the peak tidal velocity reaches about 1.3 fps. Away from the gates, flow patterns within Smith Canal and Atherton Cove are largely unaffected by incorporation of the closure structures. Below is a summary of hydrodynamic modeling results for each alignment:

- For Alternative 1, on the flood tide, the gate redistributes the flow from the SDWSC and creates a jet flow toward the west corner of Atherton Island resulting in a counter-clockwise eddy in the entrance to Atherton Cove and a clockwise eddy in Smith Canal, which would slightly improve the circulation in Atherton Cove. The sensitivity analysis indicates a wider gate opening would typically result in lower flow velocities toward Atherton Cove.

- For Alignment 2, the sheet pile wall within Atherton Cove and gate structure within Smith Canal does not significantly change the hydrodynamics; however, it slightly reduces the water volume in Atherton Cove. Locally around the gate structure, there is an increase in the flow velocity on both ebb and flood tides, but this effect is largely decreased with distance into and out of Smith Canal.

- For Alignment 3, there is a significant change to the flow pattern due to the convergence of the two jets from the gate structures. On the ebb tide, this convergence creates an area of slightly increased flow velocity at the entrance to Smith Canal from the Deep Water Shipping Channel. Changes to the flow field on the flood tide are localized to the gate structures.

Gate width sensitivity analyses indicate a wider gate would reduce the peak tidal velocity through the gate and localized hydraulic impact in the vicinity of the gate; however, it has no significant benefit in terms of tidal circulation or residence time in the hydraulic system modeled.

With regards to siltation, model results show regional changes in peak tidal velocity around the gate structure and indicate a potential for areas of deposition and erosion. With the heightened flow velocities through the gate, there is a potential for mobilization of bottom sediments that will need to be addressed during final design. Siltation would also occur in areas with reduced velocity. No siltation or erosion is expected in the inner Atherton Cove or Smith Canal away from the structures for all alignments. Sedimentation potentials for each alternative alignment are summarized below:

- For Alignment 1, velocity increases at the immediate vicinity of the gate structure, west corner of Atherton Island, north bank of the entrance to Atherton Cove, and the Smith Canal side of the proposed southern floodwall. Siltation may occur on both sides of the northern floodwall away from the gate opening, and flood shoals may form in the entrance to the Smith Canal. Gate width sensitivity analyses indicate that the pattern of the flows with increased gate width remains largely unchanged. Rather, it is the changes to the flow intensity that decreases with increased gate width. Therefore, the sedimentation changes are expected to be similar, but less severe.

- For Alignment 2, erosion may occur in the immediate vicinity of the gate structure along the center of the channel. However, siltation may occur on both sides of the new floodwalls away from the gate opening.

- For Alignment 3, erosion may occur in the immediate vicinity of the gate structures along the center of the channel and the Canal side of the proposed southern floodwall. Siltation may occur on both sides of the new floodwalls away from the gate opening and flood shoal may form in the entrance of Smith Canal.

A literature review indicated the earliest available survey data was established in 1934. Since then, the canal has not been resurveyed and depths indicated on navigation charts to this date reflected the originally surveyed water depths. Bathymetric surveying provided by Kjeldsen, Sinnock, Neudeck, Inc. (KSN) for the current project development indicates that water depths in Smith Canal to the east of I-5 are substantially lower than in past times, whereas water depths in the portion of the canal west of I-5 have increased. Water depths in Atherton Cove have also decreased due to siltation. The historical average siltation rate is around 0.2 to 0.3 ft per year in the past 80 years. More development has occurred in the past 80 years, so it is expected that the future average siltation rate will be slightly reduced.

The model results indicate that residence time is largely unaffected by the introduction of the closure structure for any of the alternatives, as the gate opening is still sufficiently large enough to enable tidal flow to propagate into Smith Canal and Atherton Cove without causing tidal muting. Because of the narrow and elongated extent of Smith Canal, the model results clearly demonstrate that residence time varies significantly from the mouth of Smith Canal, through the canal, and into Yosemite Lake. Typical residence times near the mouth of Smith Canal are around one day, whereas residence times approximately midway into the canal approach 30 days. Residence times in the back of the canal and into Yosemite Lake exceed 40 days. The residence time is very short, less than 2 days in Atherton Cove, indicating good flushing and circulation in the area.

The modeling results of the Water Quality Improvement Options indicate these Options would not change the overall hydrodynamics, circulation efficiency, and sedimentation pattern. The localized impacts and/or benefits are:

- Installing culverts under Moreing Road would slightly reduce residence at the end of Atherton Cove.

- Dredging of Atherton Cove would slightly increase residence time at Atherton Cove as dredging increases the volume of the Cove, but the residence time at Atherton Cove is still very short, less than 2 days.

- Installing culverts in the floodwall would slightly reduce tidal velocity through the 50-ft gate, but would not change the overall circulation. This is consistent with results from the gate width sensitivity analyses.

1. INTRODUCTION

1.1 PROJECT SETTING

The Smith Canal Closure Structure project setting is in the Central California Delta. Figure 1-1 provides a map of the State of California indicating the project location at Stockton, CA within the Central Valley, comprised of the Sacramento Valley and the San Joaquin Valley extending north and south from Stockton.

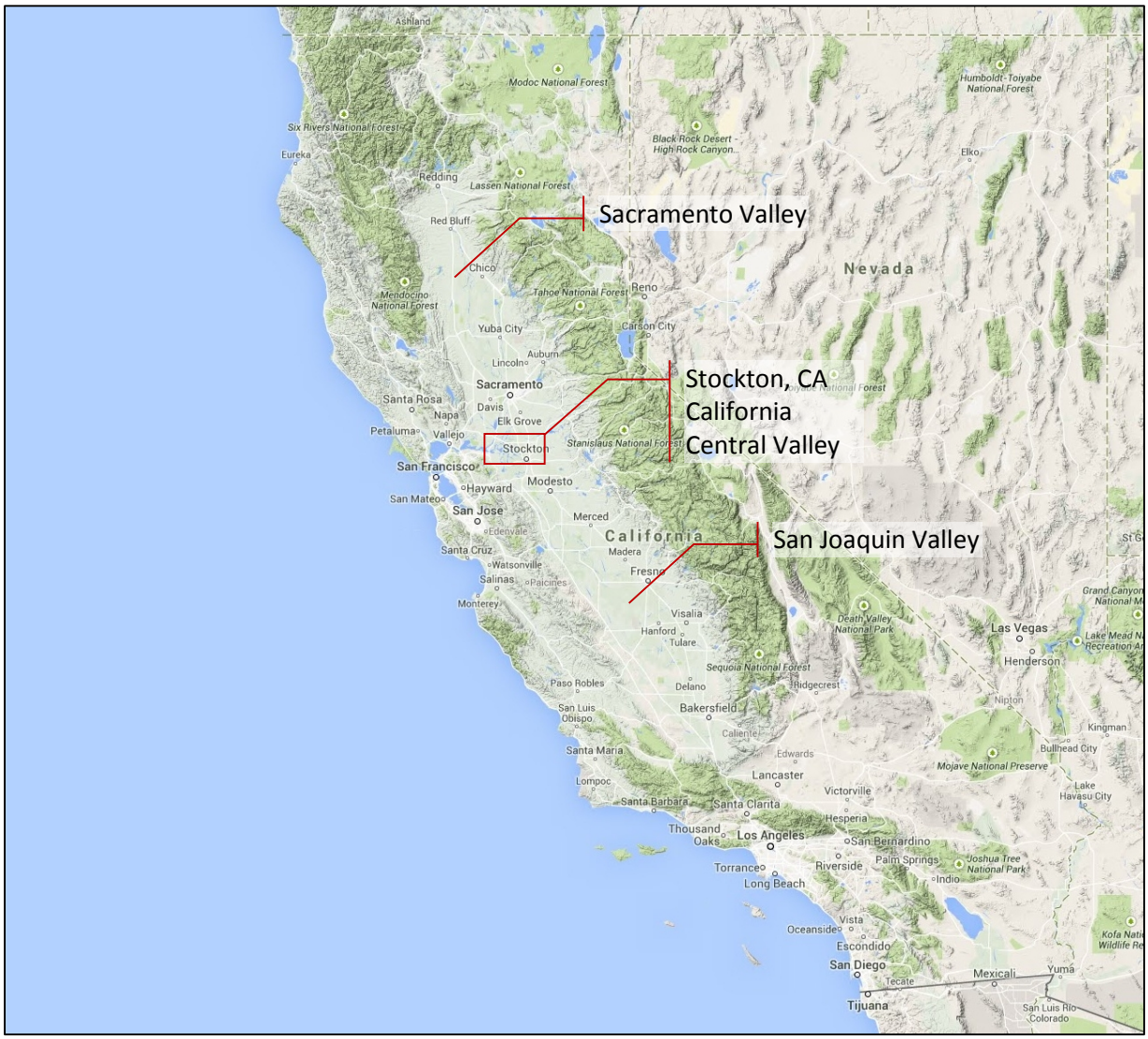


Figure 1-1: Project Location Map

The California Delta is an expansive inland river delta and estuary connected to the Pacific Ocean via the Suisun, San Pablo, and San Francisco Bays. The main sources of fresh water include the Sacramento River from the north, the San Joaquin River from the south, and the Calaveras and Mokelumne Rivers from the east. The total area of the Delta, including both land and water, is approximately 1,100 square miles. The population of the delta exceeds 500,000. The city of Stockton (Figure 1-2) is located on the San Joaquin River on the eastern edge of the Delta with a population of 297,984 (2012).

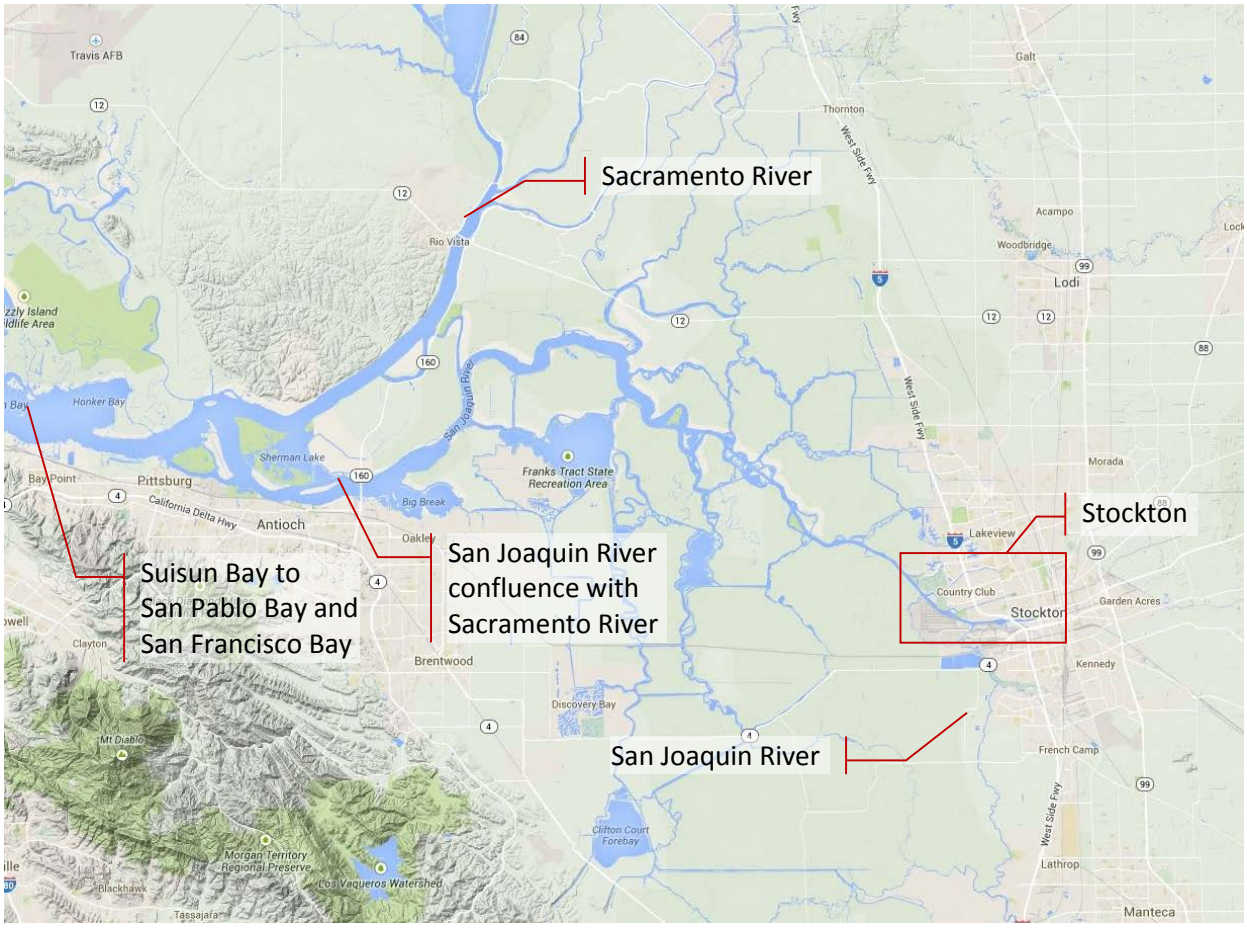


Figure 1-2: Location of Stockton in the California Central Delta

In a geologic setting, the Delta has existed for about 10,000 years. The Delta was formed due to rising sea level following the glaciations during the last Ice Age, which led to accumulation of sediment from the Sacramento and San Joaquin Rivers behind the narrow Carquinez Strait, the only outlet from the Central Valley. Sediment deposition, coupled with tidal action, results in the formation of expansive islands.

In its natural state, the California Delta was a large freshwater marsh, consisting of shallow channels and sloughs surrounding low islands of peat and tule. Since the mid-19th Century, most of the region has been reclaimed for agriculture. In the process of developing islands for agricultural use, levees were built along their shores for flood protection purposes. Today, the Delta consists of approximately 57 reclaimed islands and tracts surrounded by 1,100 miles of levees that border 700 miles of waterways.

Excavation of materials for levee construction, wind erosion and oxidation of the organic carbon of peat soils have led to widespread subsidence on the Central Delta islands so that much of the Delta region today sits below sea level.

Smith Canal is located in Stockton, CA off the Stockton Deep Water Ship Channel (SDWSC). Figure 1-3 shows the geographical location of the canal and surrounding areas with waterways annotated in red and land areas annotated in black.

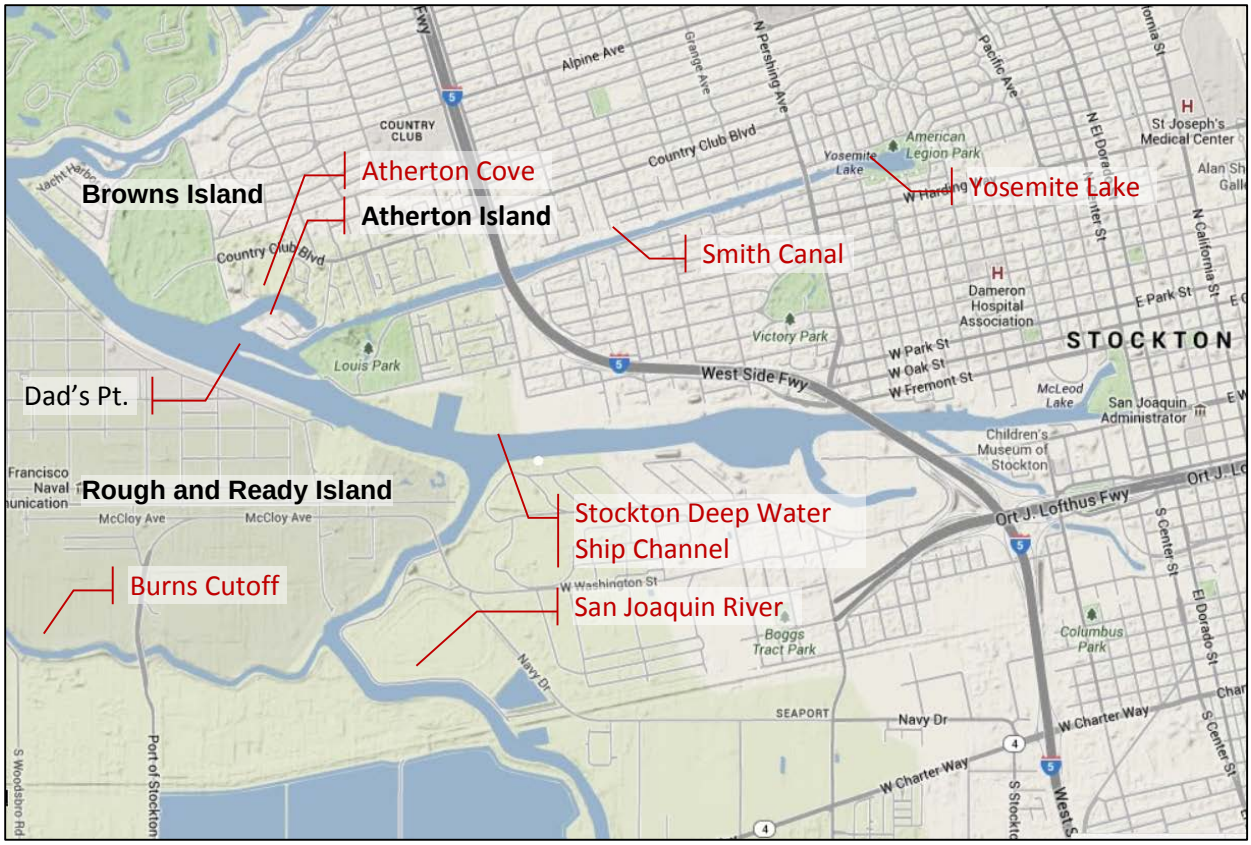


Figure 1-3: Location of Smith Canal Relative to Stockton Deep Water Ship Channel

Smith Canal is approximately 2.6 miles long and terminates at Yosemite Lake. Atherton Cove, Atherton Island, and Dad's Point Levee, adjacent to the SDWSC, form the entrance to Smith Canal.

Levees along Smith Canal and Atherton Cove serve to prevent back flooding from the San Joaquin Delta during periods of elevated runoff occurring at high tides. Some sections of the levee are encroached upon by existing property along the Canal and Atherton Cove. This precludes the levee sections from being accredited by Federal Emergency Management Agency (FEMA) by today's standards, which require access to and along the levee for inspection and maintenance purposes. Approximate flood depths are provided in Figure 1-4.

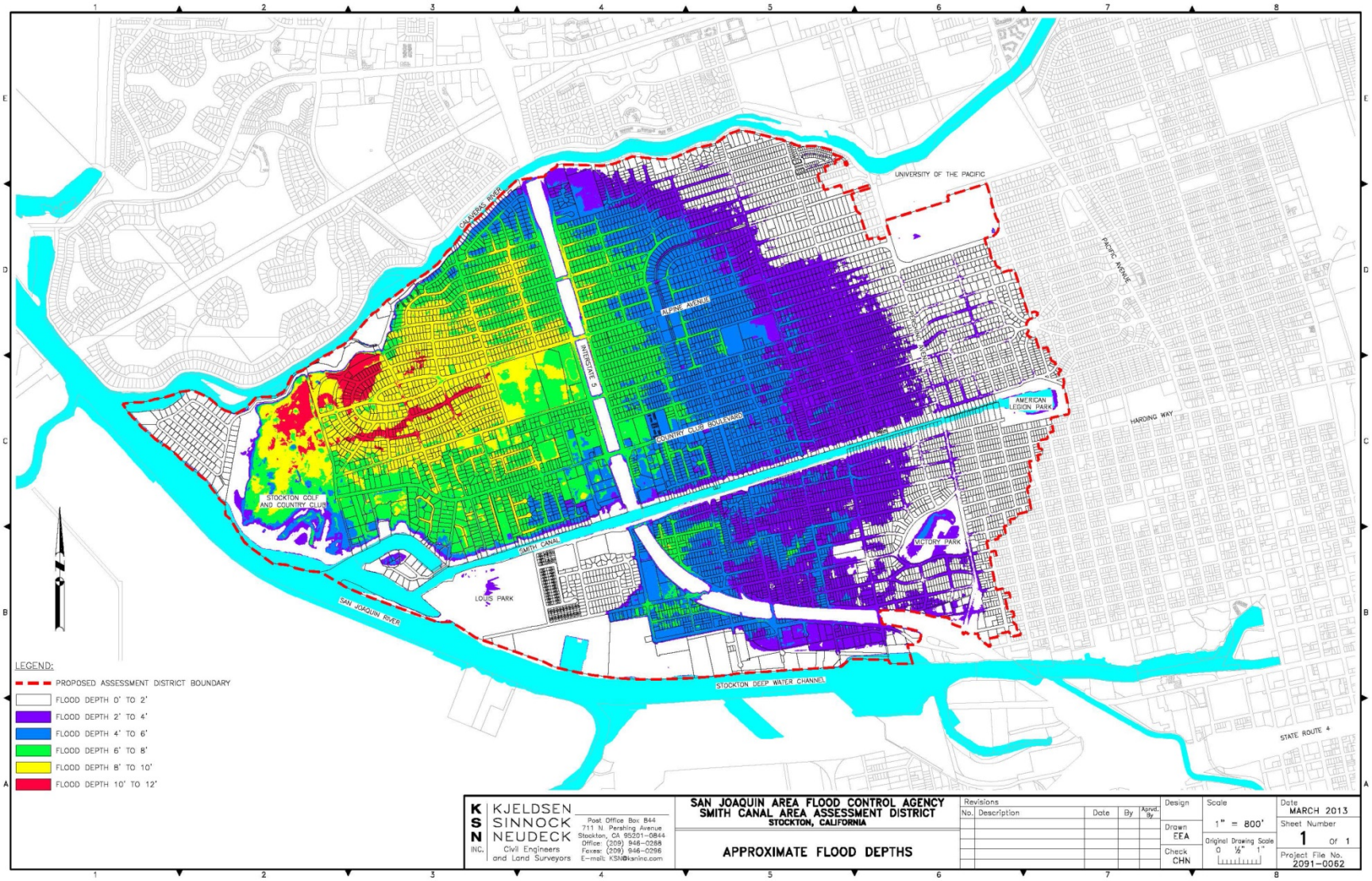


Figure 1-4: Approximate Flood Depths in Stockton Area (San Joaquin Area Flood Control Agency)

In order to develop a solution capable of meeting the minimum management criteria of the National Flood Insurance Program (NFIP), FEMA has acknowledged that incorporation of a closure structure at the entrance to Smith Canal would enable the adjoining area to meet current FEMA flood protection requirements.

1.2 OBJECTIVE

The objective of the present work focuses on hydrodynamic evaluation of three proposed closure structure alignments, sensitivity evaluation of gate width for Alignment 1, and evaluation of four Water Quality Improvement Options.

The scope of work includes the following items:

- Calibration of MIKE21 model to include residence times estimated for Smith Canal based on a dye-tracer test reported in *Jones & Stokes 2005*.
- Development of a hydrodynamic model in the computer code MIKE21 for each of the three proposed Alternative Alignments. These three closure wall and gate layouts are illustrated in Figure 1-5 through Figure 1-8. The numerical modeling includes evaluation of the three alternatives to determine flow velocities at the proposed gate locations, flow patterns, and potential impacts to residence time. The results from the revised alignments are compared and differences highlighted.
- Sedimentation evaluation to identify effects of the project on sedimentation within Smith Canal and Atherton Cove. Currently, water levels are too low in Atherton Cove at low tide for recreational boating, as reported by Atherton Cove homeowners. Historical sedimentation rates within Smith Canal are reviewed based on a comparison of past and present bathymetry. Potential sedimentation within Atherton Cove is estimated for the existing conditions and the proposed condition with the floodwall and gate in place (gate open). Results are compared to assess the impact of the project features (presence of floodwall and gate).
- Sensitivity analysis of Alternative Alignment 1 with respect to gate width, in order to assess the response of the hydrodynamic model to three gate widths: 100-ft and 200-ft openings are compared to the proposed 50-ft gate width.
- Assessment of the following four Water Quality Improvement Options:
    - Culverts under Moreing Road to Atherton Island;
    - Dredging Atherton Cove;
    - Combination of culverts under Moreing Road and dredging of Atherton Cove; and
    - Culverts in floodwall.

1.3 CLOSURE WALL ALIGNMENT ALTERNATIVES

The three proposed alternative closure wall alignments are presented in the following sections.

1.3.1 Alternative Alignment 1

Figure 1-5 shows Alignment 1. This alignment incorporates a floodwall between Dad's Point and the Stockton Golf and Country Club (SGCC) tying into the existing FEMA-accredited levee along

that shore. The floodwall will consist of a double sheet-pile wall featuring a 50-ft wide gate for access to and from Atherton Cove and Smith Canal. The gate control building is envisioned to be located at Dad's Point. A levee raising is proposed along the toe of Dad's Point Levee at Louis Park in order to meet the required elevation for flood protection in this area.

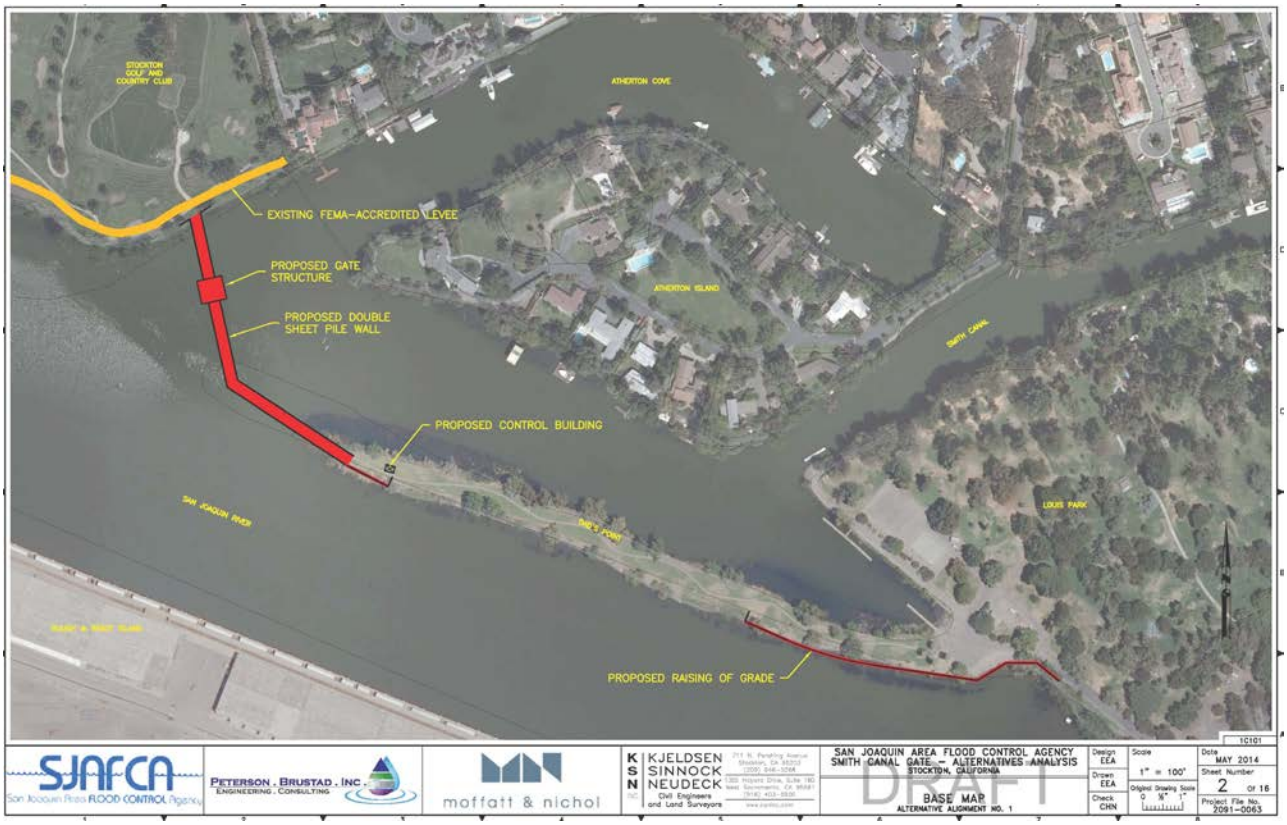


Figure 1-5: Alternative Alignment 1

The elevation of the gate opening is shown in Figure 1-6. The opening is 50-ft-wide with the sill located at elevation -6.0 ft NAVD88. The closure wall extends to elevation +14.9 ft NAVD88. The great diurnal tide range from Mean Lower Low Water (MLLW) to Mean Higher High Water (MHHW) is indicated in the figure by the two blue lines across the gate opening. The design water surface elevations (Peterson Brustad, Inc. [PBI] 2014a) on the San Joaquin River at Smith Canal are as follows:

- 100-yr WSE = +9.4 ft (NAVD88)
- 200-yr WSE = +9.5 ft (NAVD88)

The top of floodwall/gate elevation was determined as follows:

+ 9.5 ft (NAVD88) - 200-yr WSE
+ 3.0 ft for freeboard
+ 1.0 ft for hydraulic uncertainty
<u>+ 1.4 ft for sea level rise through 2050 (median projection)</u>
+14.9 ft (NAVD88) for top of wall/top of gate

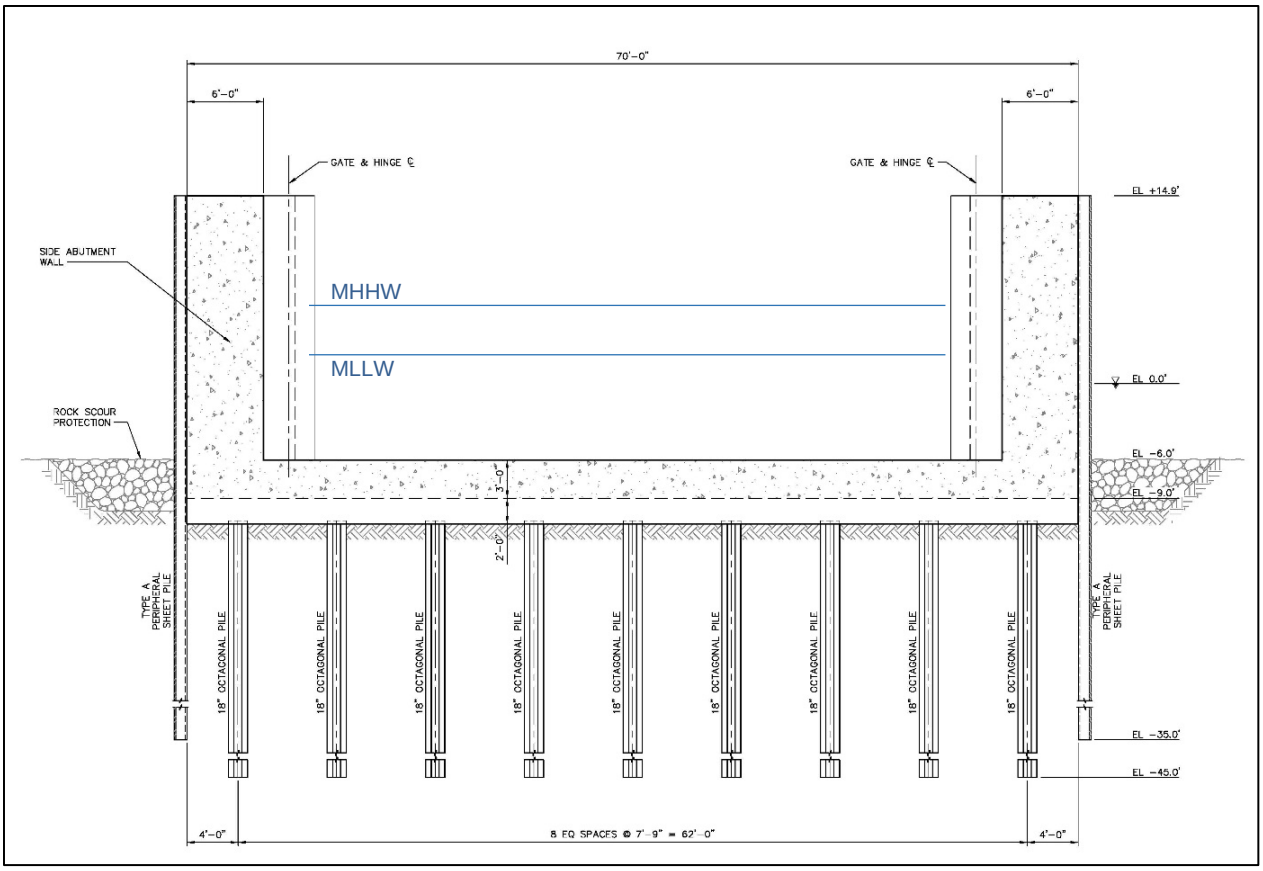


Figure 1-6: Gate Elevation Diagram

1.3.2 Alternative Alignment 2

Alternative Alignment 2 is shown in Figure 1-7. This alternative is based on a solution where the existing levee along the shore of Atherton Cove is improved to the level where it can meet the FEMA-required level of flood protection. The improvements would consist primarily of extending the existing levee and terrain to provide access for inspection required for FEMA accreditation. The levee improvements would start at the termination of the existing FEMA-accredited levee along the shore of the SGCC, and terminate at the shore of Smith Canal. Here, a floodwall with a 50-ft wide gate would be incorporated across Smith Canal in order to provide flood protection of the areas surrounding Smith Canal. The floodwall would consist of a double wall sheet-pile wall. The gate control building is proposed to be located a short distance within Louis Park, where an existing road/trail would provide access. The control building would need to be located at an elevation sufficient to provide an unobstructed view of Smith Canal upstream and downstream of the gate.

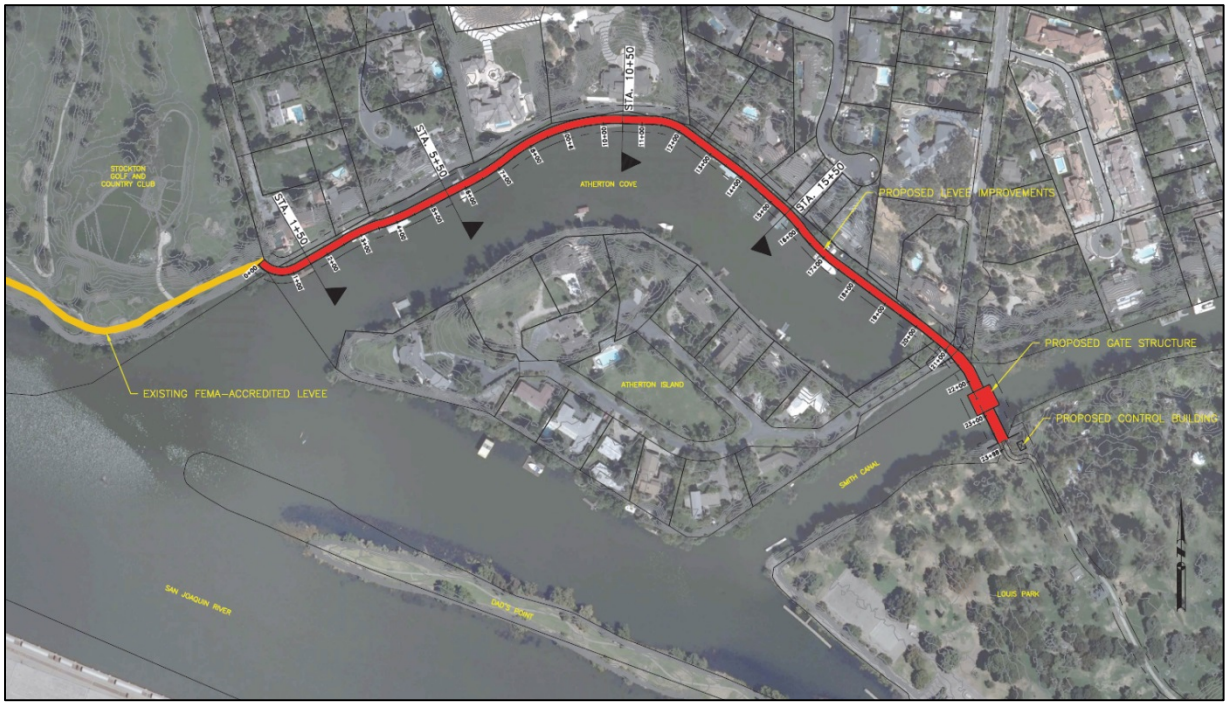


Figure 1-7: Alternative Alignment 2

1.3.3 Alternative Alignment 3

Alternative Alignment 3 is shown in Figure 1-8. This alternative is based on a solution where the amount of floodwall to be incorporated is minimized to the extent possible. This alternative incorporates a floodwall across the entrance to Atherton Cove between the western tip of Atherton Island and across to the SGCC, where the double sheet-pile floodwall would tie into the existing FEMA-accredited levee along the shore of the SGCC. A short distance southeast along the Shore of Atherton Island, a short section of double wall sheet-pile is proposed. This will provide access to the floodwall via the existing roads on Atherton Island. The gate control building would be located on the opposite shore at the corner of the SGCC, where a short road would be incorporated to enable access via the existing streets of the adjoining neighborhood. The control building would be located to provide an unobstructed view of Atherton Cove and the SDWSC in order to monitor vessel traffic on approach to and departure from the gate.

A second double sheet-pile floodwall and gate would be incorporated across the entrance to Smith Canal. Roads would be incorporated to enable access from Atherton Island and via Louis Park. The gate control building is proposed to be located at the western tip of Louis Park, at an elevation sufficient to provide an unobstructed view of Smith Canal upstream of the gate and along Dad's Point Levee in order to monitor vessels on approach to and departure from the gate.

While this solution minimizes the amount of floodwall (new structures) to be incorporated, it requires the incorporation of two gate structures, one at the entrance to Atherton Cove and another at the entrance to Smith Canal.

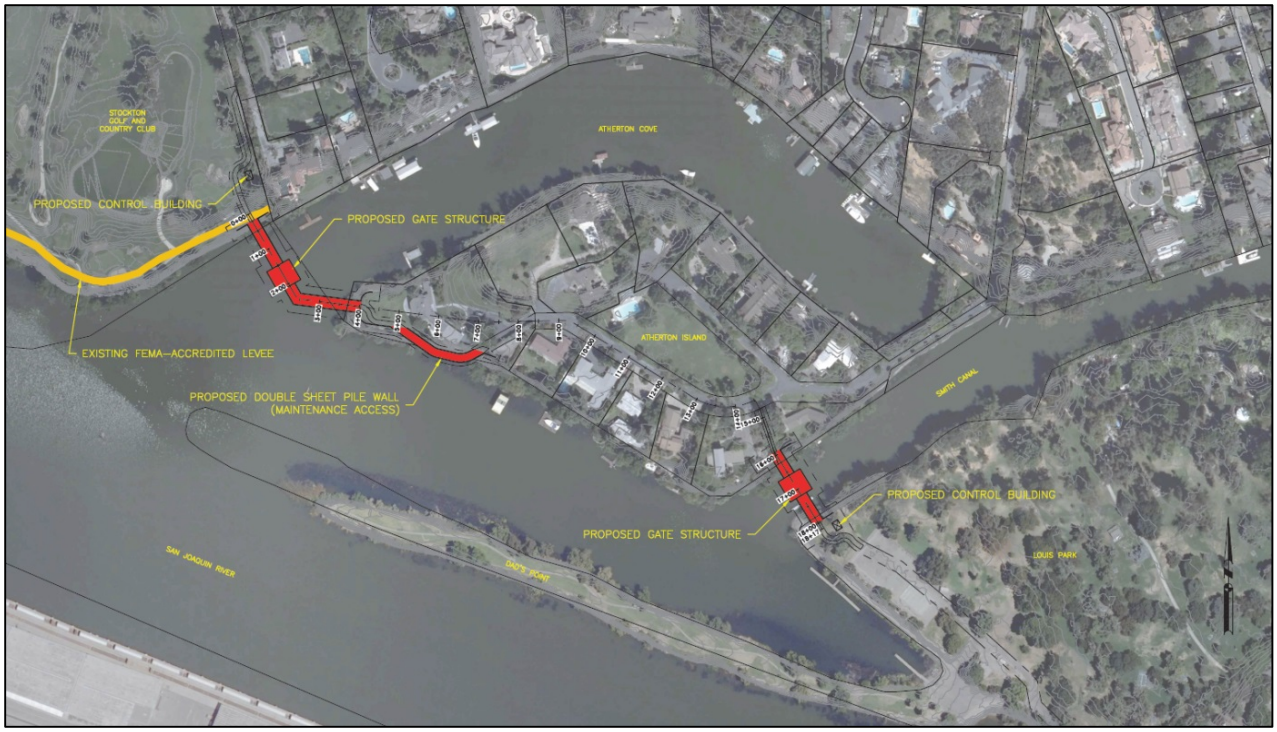


Figure 1-8: Alternative Alignment 3

1.4 WATER LEVELS AND TIDAL DATUMS

The river stage at Smith Canal is affected by ocean tides propagating into San Francisco Bay and up the San Joaquin River into the Delta. Tides in the Delta are mixed semidiurnal, having two high tides and two low tides of differing magnitude each (lunar day) tidal cycle. Table 1-1 summarizes the California Data Exchange (CDEC) stream gage stations within the region, (California Department of Water Resources [CDWR] 2014c). The locations of these two gages are illustrated in Figure 1-9. The nearest CDEC stream gage to Smith Canal is Rough and Ready Island (RRI), which is on the San Joaquin River approximately 3/4 of a mile north of the mouth of Smith Canal.

Table 1-1: Stream Gage Locations

Station ID	Station Name	Operator	Latitude	Longitude	Data Collection
RRI	ROUGH AND READY ISLAND	Cal DWR/DES	37.9630°N	121.3650°W	SATELLITE
SJG	SAN JOAQUIN RIVER AT GARWOOD BRIDGE	USGS	37.9350°N	121.3290°W	DATA XCHG-USGS



Figure 1-9: Locations of CDEC Stream Gages in Proximity to Smith Canal

Figure 1-10 shows the historical water level at RRI Gage. In March 2011, the river stage reached 8.2 ft NAVD88, which was a case of elevated river stage occurring during the wet year of 2011. It's interesting to note that the river stage reached an elevation of 8.0 ft NAVD88 in January 2008, nearly as high as the March 2011 peak, although this event was not specifically related to major runoff, but was more of an effect due to a series of higher tides.

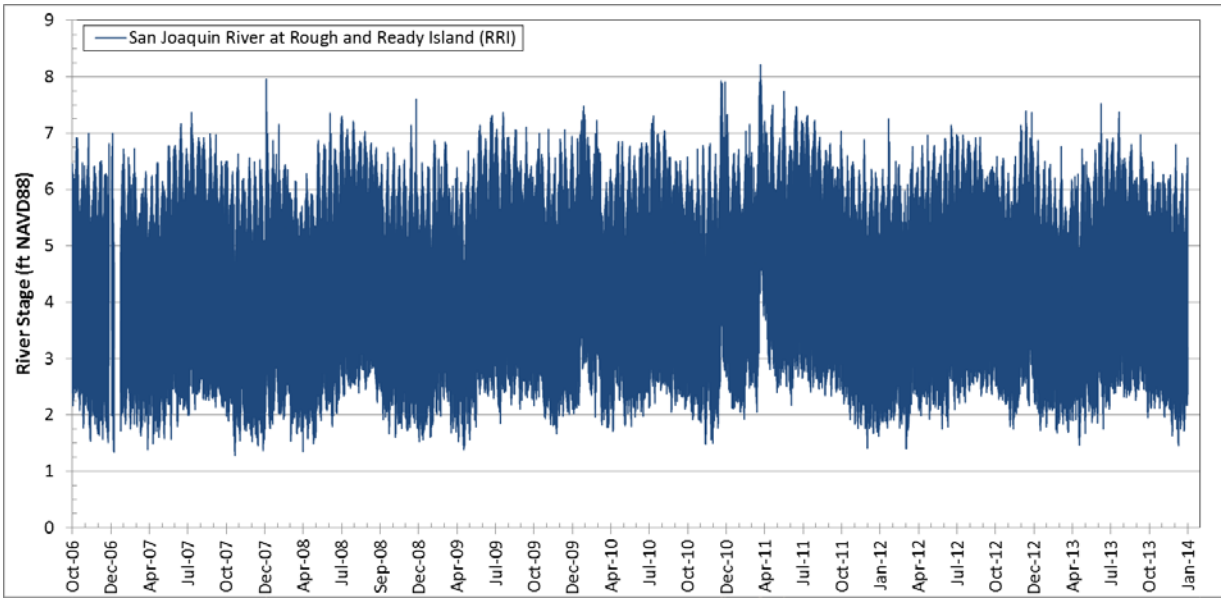


Figure 1-10: Historical River Stage of San Joaquin River at RRI Gage

Tide-range statistics for RRI Gage are summarized in Table 1-2. These statistics are based on gage data from the period of 10/1/2006 – 1/1/2014. Data before October 2006 was not used in the analysis since DWR transitioned to reporting all tide data collected in the Delta to the NAVD88 vertical datum on October 1, 2006.

Table 1-2: Tide elevations at CDEC gage RRI

Datum	Elevation (feet, NAVD88)
Highest Observed Water Level (3/23/2011)*	+8.21
Mean Higher High Water (MHHW)	+6.11
Mean High Water (MHW)	+5.64
Mean Tide Level (MTL)	+4.25
Mean Low Water (MLW)	+2.87
Mean Lower Low Water (MLLW)	+2.37
Lowest Observed Water Level (10/22/2007)*	+1.28
North American Vertical Datum of 1988 (NAVD88)	0.00

An example of the typical tidal variation (10/1/08 to 11/30/08) is shown in Figure 1-11 with the Table 1-2 tidal datums overlain. Note that MHHW is an average of the higher high tides and, therefore, does not reflect the peak (highest) tide levels; similarly MLLW is an average of the lower low tides and does not indicate the lowest tide levels.

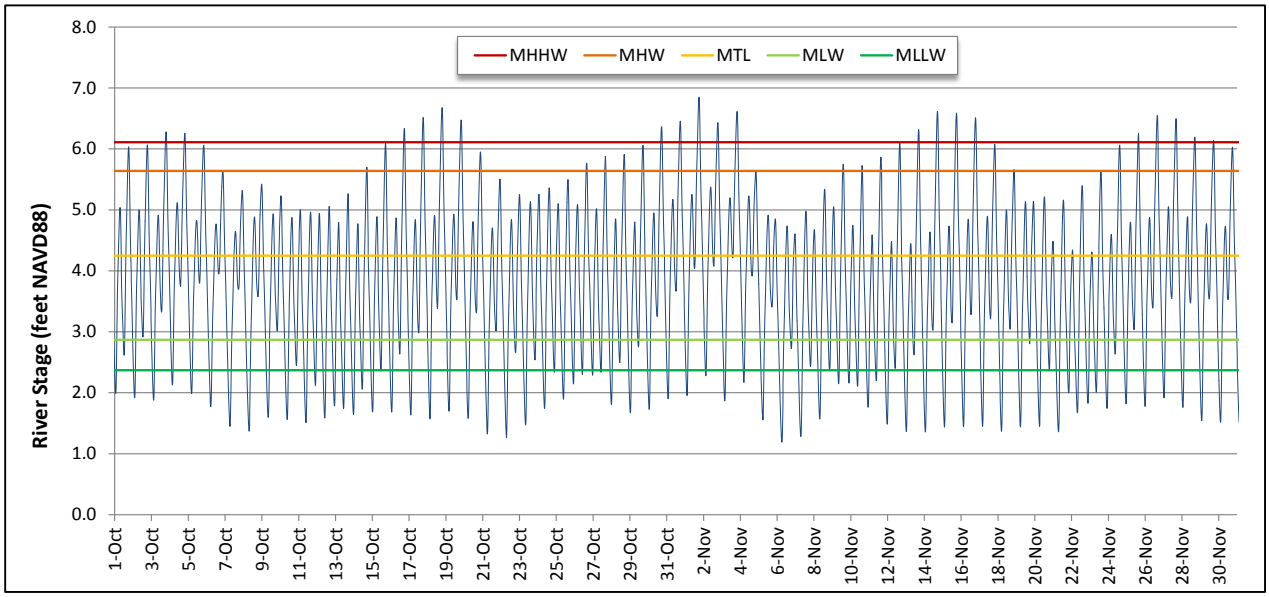


Figure 1-11: Example of Tidal Variations

1.5 RIVER FLOWS

Flow data obtained for numerical modeling purposes spans the time horizon from July 2005 through March 2012. This time frame was selected because tidal variations are known to be referenced accurately to NAVD88 datum, and the period contains wet, dry, and typical years of outflow. Figure 1-12 shows discharge data at RRI gage in San Joaquin River and the Calaveras River over the above time frame. It can be seen that typical outflow from the Calaveras is within ±1,200 cubic foot per second (cfs) and the average outflow is around 200 cfs. The typical outflow from the San Joaquin River is much larger having a typical discharge within approximately ±8,000 cfs. The negative discharges reflect daily flood tide conditions where water pushes up the San Joaquin and Calaveras. The higher positive discharges indicate periods of elevated outflow from the rivers, typically seen during months of November through April due to increased precipitation and snowmelt runoff from the mountains. Two pronounced cases of elevated discharges occurred from 12/2005 to 06/2006 and from 01/2011 to 09/2011 that are characteristics of wet years with above-normal precipitation and snow melt.

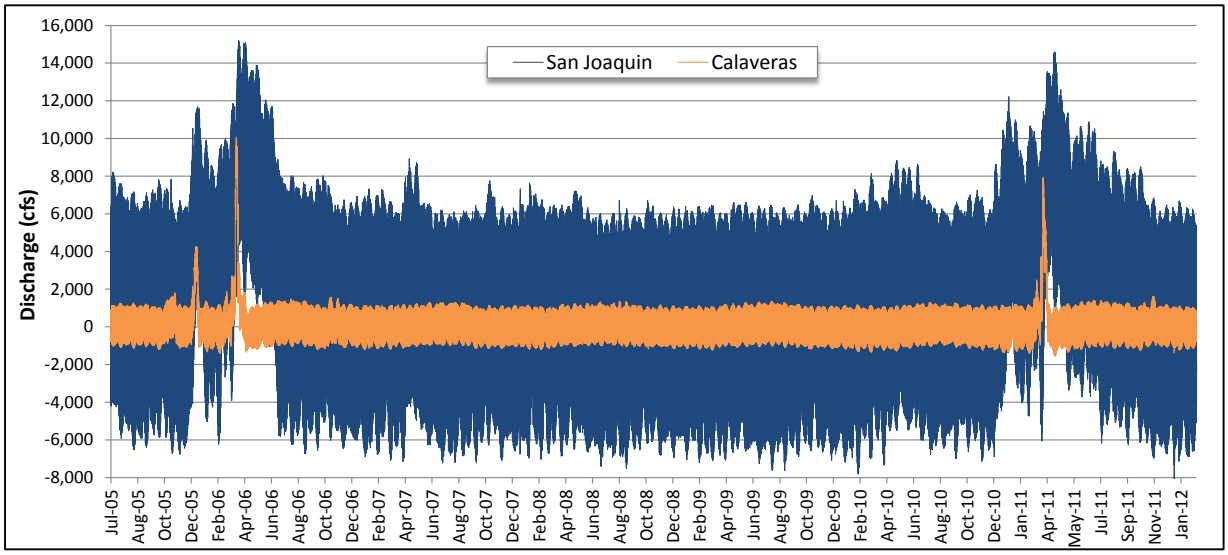


Figure 1-12: Comparison of Discharge from San Joaquin and Calaveras Rivers

2. MODEL SELECTION AND DESCRIPTION

The commercially available MIKE21 Flow Model Flexible Mesh (FM) modeling suite, developed by the Danish Hydraulic Institute (DHI) has been adopted to study tidally-driven water exchange within Smith Canal. MIKE21-FM is a modeling system based on a flexible mesh approach for applications within oceanographic, coastal, and estuarine environments. The model can use a wide range of forcing terms including water level variations, discharge, bottom friction, wind shear stresses, Coriolis force, barometric pressure gradients, and wave radiation stresses, among others. The following model components were used:

- Two-dimensional, depth-averaged Hydrodynamics (MIKE21-FM-HD) component to model current and tidal hydraulics throughout Smith Canal, using an unstructured mesh. The model output includes water levels and depth-averaged flow velocities, among other parameters, which can be extracted at any location of interest and/or summarized over the model domain. The results can be directly used to assess potential hydraulic impact.

- Two-dimensional, depth-averaged Transport Model (MIKE21-FM-TR) is linked to the MIKE21-FM-HD model and was used to perform advection/dispersion modeling and to evaluate dye tracer mixing and circulation efficiency.

MIKE21-FM-HD and MIKE21-FM-TR models provide details of how water is exchanged between Smith Canal and the SDWSC, where the flow is a combination of oceanic tide and runoff from the San Joaquin River. Tidal exchange plays an important role in water exchange and residence time, and flow patterns will have an effect on siltation and may also impact navigation. The focus of the present study is to replicate existing conditions and subsequently determine how incorporation of a closure structure with a gate will affect tidal exchange and flow patterns within Smith Canal and Atherton Cove.

3. MODELING APPROACH

Tidal exchange and flow patterns within Smith Canal and impacts to water quality and siltation associated with incorporation of a closure structure have been determined using the MIKE 21-FM models. The modeling approach and procedure for this study were:

1. Numerical model setup of the existing hydraulic system for model calibration and validation to determine numerical parameters and for establishing a baseline condition for assessing the potential impacts of proposed project alternatives.

2. Model calibration and validation of the MIKE21-FM-HD model for determining numerical model parameters such as roughness coefficients and eddy viscosity. The MIKE21-FM-HD model was first calibrated by matching model-predicted tidal elevations and flow discharges with those measured within the model domain at Rough and Ready Island (RRI) Gage and San Joaquin River (SJG Gage) for a period from November 19-24, 2008, then validated for a separate two-week period from January 1-15, 2009.

3. Model calibration of the MIKE21-FM-TR model for determining numerical modeling parameter of dispersion coefficient. The MIKE21-FM-TR model was calibrated against a dye tracer test (Litton 2004) performed in the Smith Canal during a rainfall event in February 2004.

4. Creating new models to incorporate the proposed flood control structures into the numerical model. Five different models (modeling scenarios) created include three alternatives and two gates, with variations of Alternative 1.

5. Performing hydrodynamic model runs of each of the five modeling scenarios for a 2-month period from June through July 2007, which is selected to represent large spring-neap tidal cycles for the velocity and sedimentation potential analysis. As the MIKE21-FM-TR model is driven by the MIKE21-FM-HD model and the residence time in Smith Canal is very long, a 2-month period was run for both HD and TR models. The results of the HD modeling runs include dry weather peak flood and ebb velocity to assess potential impacts, flow discharge, and tidal elevations.

6. Performing residence time analyses for each of the five modeling scenarios. The modeling results are residence time in Atherton Cove and Smith Canal.

7. Performing hydrodynamic model runs of each of the four Water Quality Improvement Options. As the MIKE21-FM-TR model is driven by the MIKE21-FM-HD. The results of the HD modeling runs include dry weather peak flood and ebb velocity to assess potential impacts, flow discharge, and tidal elevations.

8. Performing residence time analyses for each of the four Water Quality Improvement Options to assess effectiveness of these options. The modeling results are residence time in Atherton Cove and Smith Canal.

Numerical model runs performed are listed in

Table 3-1 for the hydrodynamic model and Table 3-2 for the transport model.

Table 3-1: Summary of Hydrodynamic Model Runs

Run No.	Alternative	Gate Width	Time Period	Purpose
1	Existing Condition	n/a	Nov 19-24, 2008	HD model calibration
2	Existing Condition	n/a	Jan 1-15, 2009	HD model validation
3	Existing Condition	n/a	Feb 2-14, 2004	TR model calibration
4	Existing Condition	n/a	June - July 2007	Alternative comparison
5	Alternative 1	50-ft	June - July 2007	Alternative comparison
6	Alternative 2	50-ft	June - July 2007	Alternative comparison
7	Alternative 3	50-ft	June - July 2007	Alternative comparison
8	Alternative 1	100-ft	June - July 2007	Gate width sensitivity
9	Alternative 1	200-ft	June - July 2007	Gate width sensitivity
10	Alternative 1, WQ Option1	50-ft	June 2007	WQ improvement
11	Alternative 1, WQ Option2	50-ft	June 2007	WQ improvement
12	Alternative 1, WQ Option3	50-ft	June 2007	WQ improvement
13	Alternative 1, WQ Option4	50-ft	June 2007	WQ improvement

Note: n/a = not applicable; WQ = Water quality

Table 3-2: Summary of Transport Model Runs

Run No.	Alternative	Gate Width	Time Period	Purpose
1	Existing Condition	n/a	Feb 2-14, 2004	TR model calibration
2	Existing Condition	n/a	June - July 2007	Alternative comparison
3	Alternative 1	50-ft	June - July 2007	Alternative comparison
4	Alternative 2	50-ft	June - July 2007	Alternative comparison
5	Alternative 3	50-ft	June - July 2007	Alternative comparison
6	Alternative 1	100-ft	June - July 2007	Gate width sensitivity
7	Alternative 1	200-ft	June - July 2007	Gate width sensitivity
8	Alternative 1, WQ Option1	50-ft	June 2007	WQ improvement
9	Alternative 1, WQ Option2	50-ft	June 2007	WQ improvement
10	Alternative 1, WQ Option3	50-ft	June 2007	WQ improvement
11	Alternative 1, WQ Option4	50-ft	June 2007	WQ improvement

Note: n/a = not applicable; WQ = Water quality

4. NUMERICAL MODEL SETUP

This section describes the MIKE21-FM numerical model setup including model domain, bathymetry used, resulting model mesh, as well as numerical models calibration and validation.

4.1 MODEL SETUP

4.1.1 Model Domain

The model domain includes Smith Canal, Atherton Cove, Burns Cutoff, 3 miles of the Calaveras River, SDWSC, and the surrounding parts of the City of Stockton. The hydrodynamic portion of the model is outlined in Figure 4-1, which captures Smith Canal, the adjacent levees, and the adjoining waterfront portions along the SDWSC. This model domain was carefully determined by considering the following factors:

- data availability (shown in Table 4-1; data gaps are not included) such as discharge and/or stage at model open boundaries;
- open boundaries that are set far enough away without incurring boundary effects to the study area (Smith Canal); and
- capturing the tidal prism of the modeling area to ensure accuracy of tidal currents in the study area.

Table 4-1: Hydraulic Data Inventory

Station	Longitude	Latitude	Ageny	Parameter	Interval	Data Type	Record Period
Rough and Ready Island (RRI)	121.365000°W	37.963000°N	CA Data Exchange Center (CDEC)	Stage	15 min	Measured	08/28/2000 - 10/07/2014
				Discharge	15 min	Measured	08/28/2000 - 10/07/2014
			CA Water Library (CAWL)	Stage	15 min	Measured	10/01/1982 - 10/01/2012
				Discharge	15 min	Measured	10/01/2006 - 09/05/2013
San Joaquin River At Garwood Bridge (SJG)	121.329000°W	37.935000°N	CDEC	Stage	15 min	Measured	08/27/2003 - 12/05/2013
				Discharge	15 min	Measured	08/27/2003 - 12/05/2013
			US Geological Service (USGS)	Stage	15 min	Measured	10/01/2007 - 10/18/2014
Stockton Fire Station	121.319000°W	37.999000°N	CDEC	Percipitation	Daily Average	Measured	10/01/1996 - 11/04/2014
Legion Park Pump Station	121.306939°W	37.967907°N	City of Stockton	Pump Outflow	Varies	Measured	02/01/2004 - 02/15/2004
RCAL009	Varies	Varies	State of California	Discharge	Daily Average	Synthetic	10/01/1989 - 05/19/2013

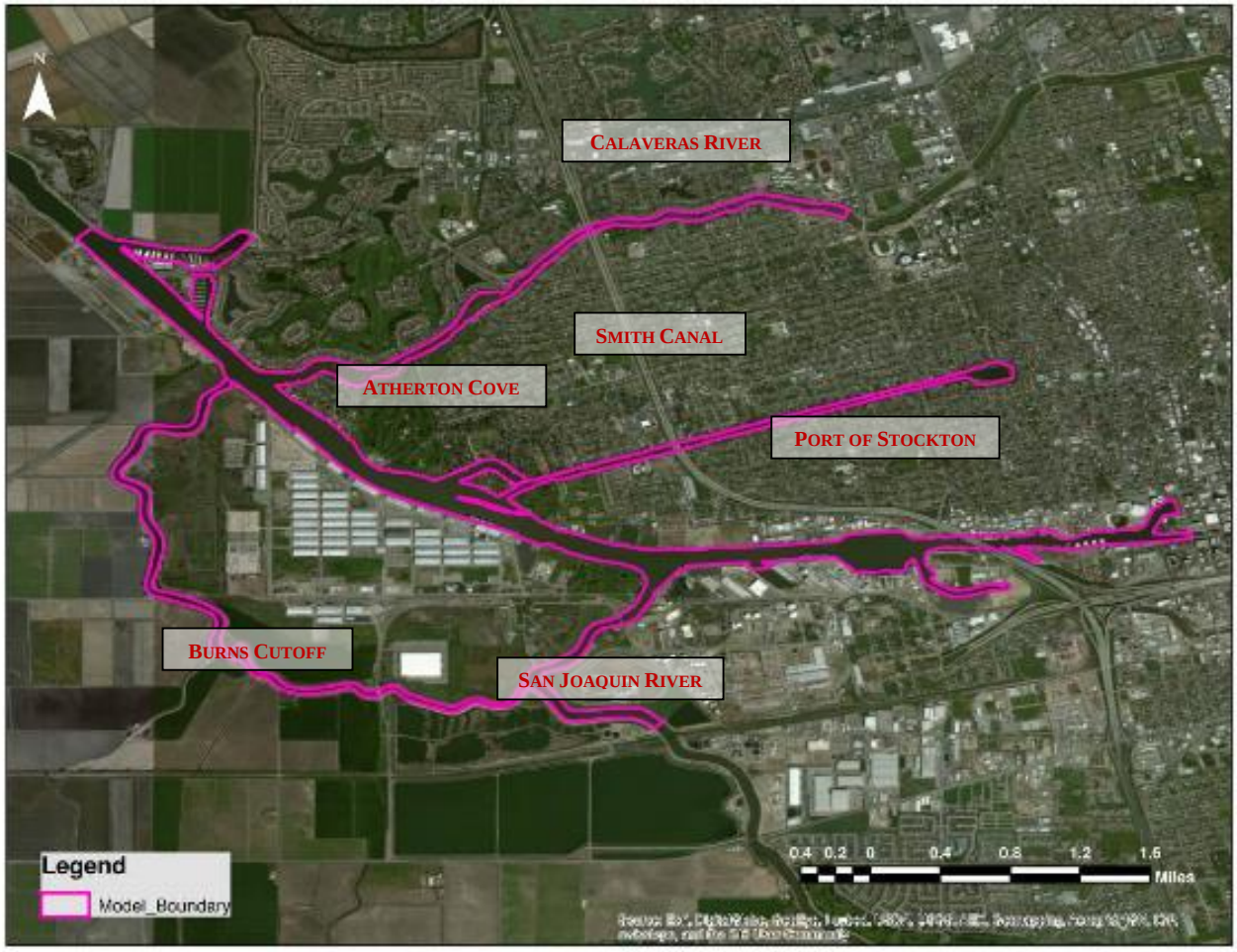


Figure 4-1: MIKE21-FM Model Domain

4.1.2 Model Bathymetry

The overall modeling domain was defined through model bathymetry. For the flexible mesh models used here, the model bathymetry was set up through a flexible mesh with triangular and rectangular elements, with a bed elevation defined at each element of the mesh. The digital elevation model (DEM) adopted for the hydraulic analysis has been developed based on 2014 bathymetric survey performed by Kjeldsen, Sinnock, Neudeck, Inc. (KSN) for this project, and topographic LiDAR data acquired by CDWR during flights of the Sacramento-San Joaquin Delta conducted in 2007. Bathymetric data for the San Joaquin River and SDWSC adopted for the digital elevation model reflects survey data acquired for CDWR's Delta Simulation Model II (DSM2) (CDWR 2014a). Figure 4-2 illustrates the CDWR's LiDAR topographic data in the vicinity of the Smith Canal.

The model bathymetry used NAVD88, and the horizontal coordinate system used in the model were Universal Transverse Mercator (UTM) Zone 10. All units in the MIKE21-FM models are in metric.



Figure 4-2: CDWR LiDAR Topographic Data in the Vicinity of the Smith Canal

4.1.3 Model Mesh

The model domain is represented with a number of elements, and the elements can be either triangular or rectangular. Figure 4-3 shows the model mesh for the entire model domain, which consists of 38,531 elements. Because the modeling area is quite large, individual elements are not visible. Figure 4-4 through Figure 4-7 show mesh details at Atherton Cove and Smith Canal as well as connections of SDWSC and Burns Cutoff.

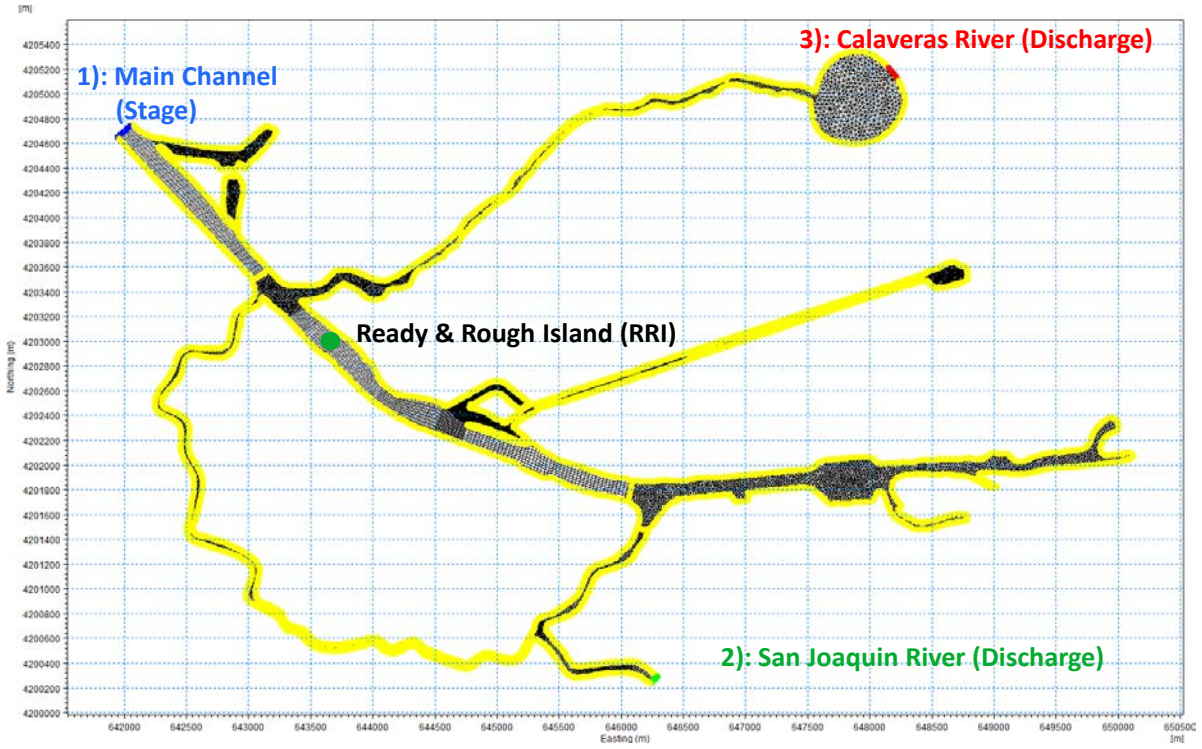


Figure 4-3: MIKE21-FM Mesh and Boundaries

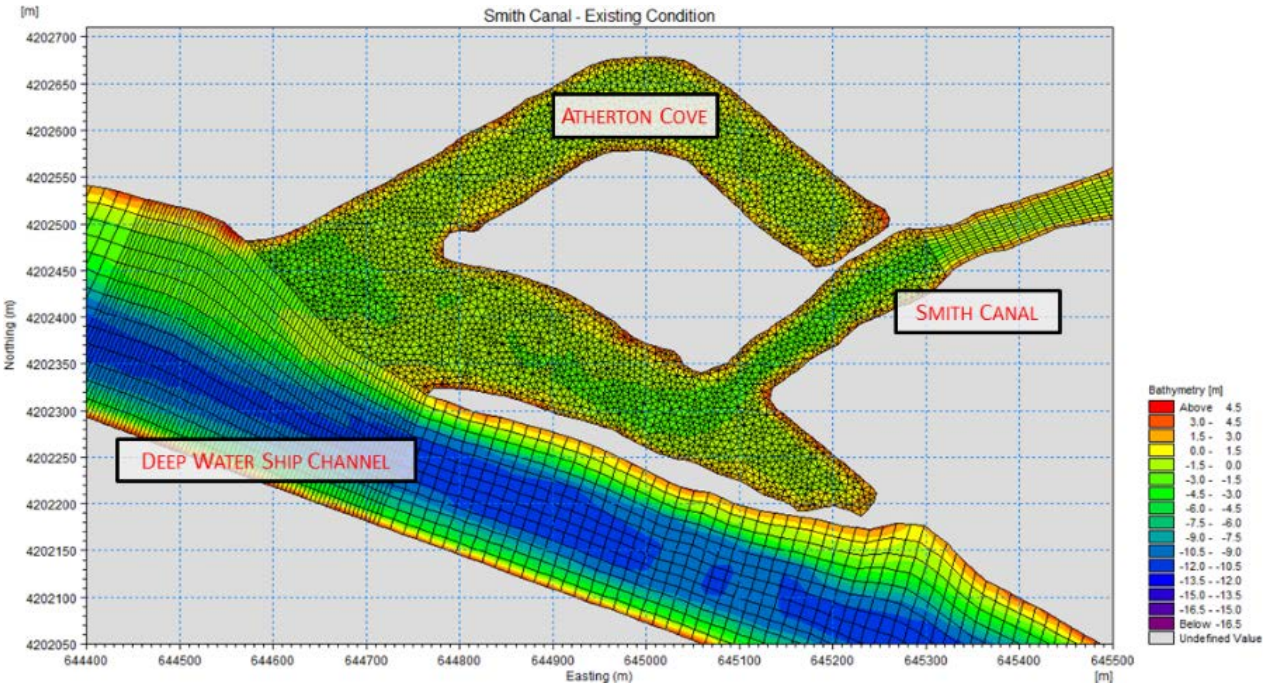


Figure 4-4: MIKE21-FM Mesh of Atherton Cove and Smith Canal

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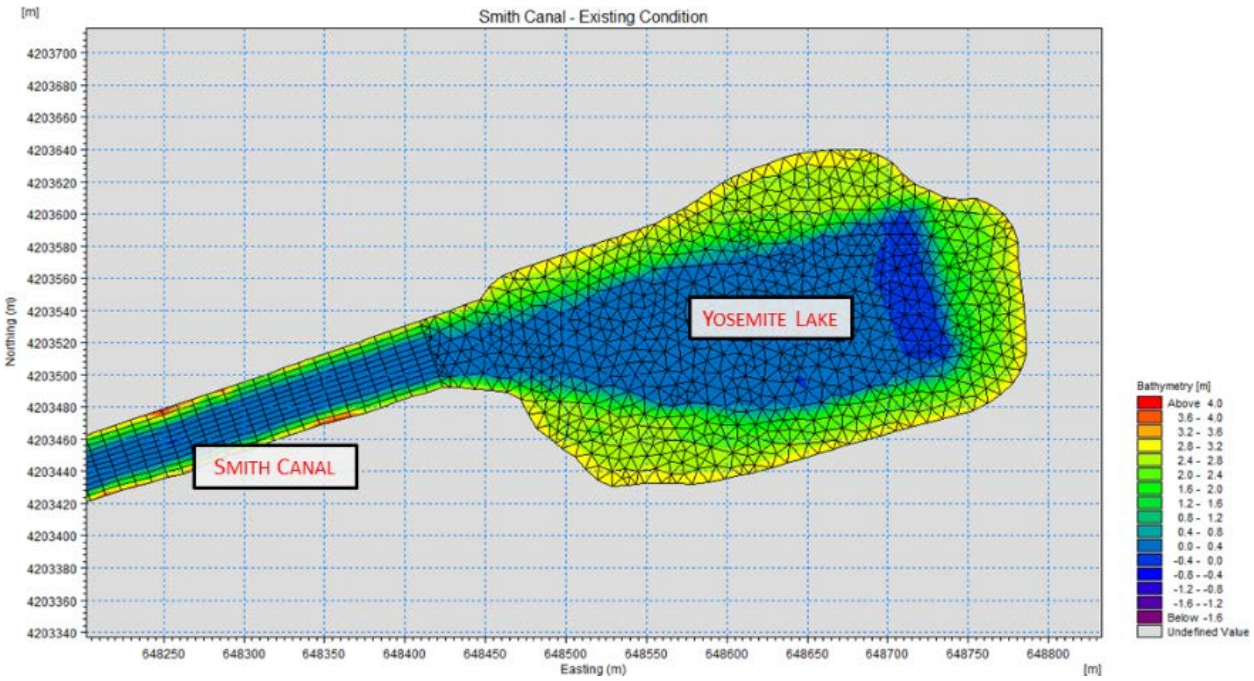


Figure 4-5: MIKE21-FM Mesh of Yosemite Lake

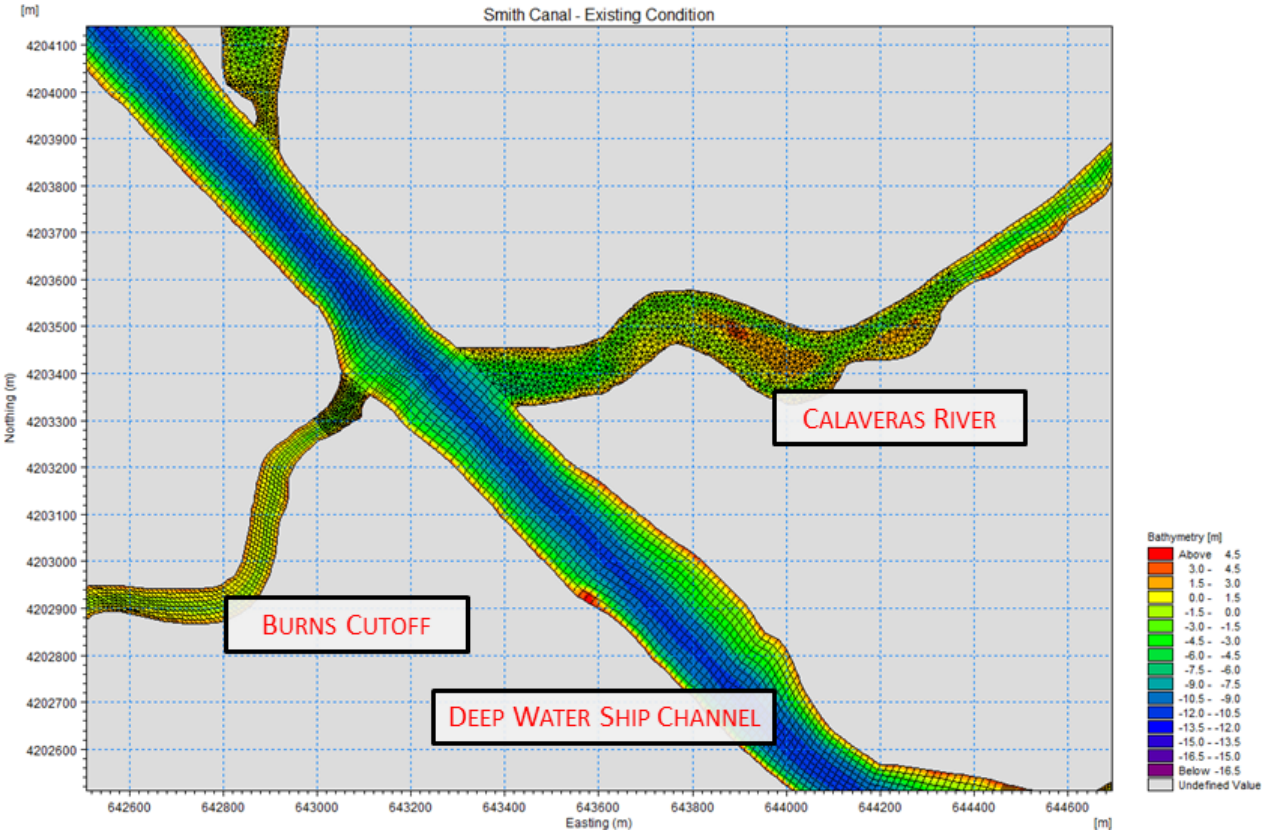


Figure 4-6: MIKE21-FM Mesh at Intersection of Burns Cutoff and Deep Water Ship Channel

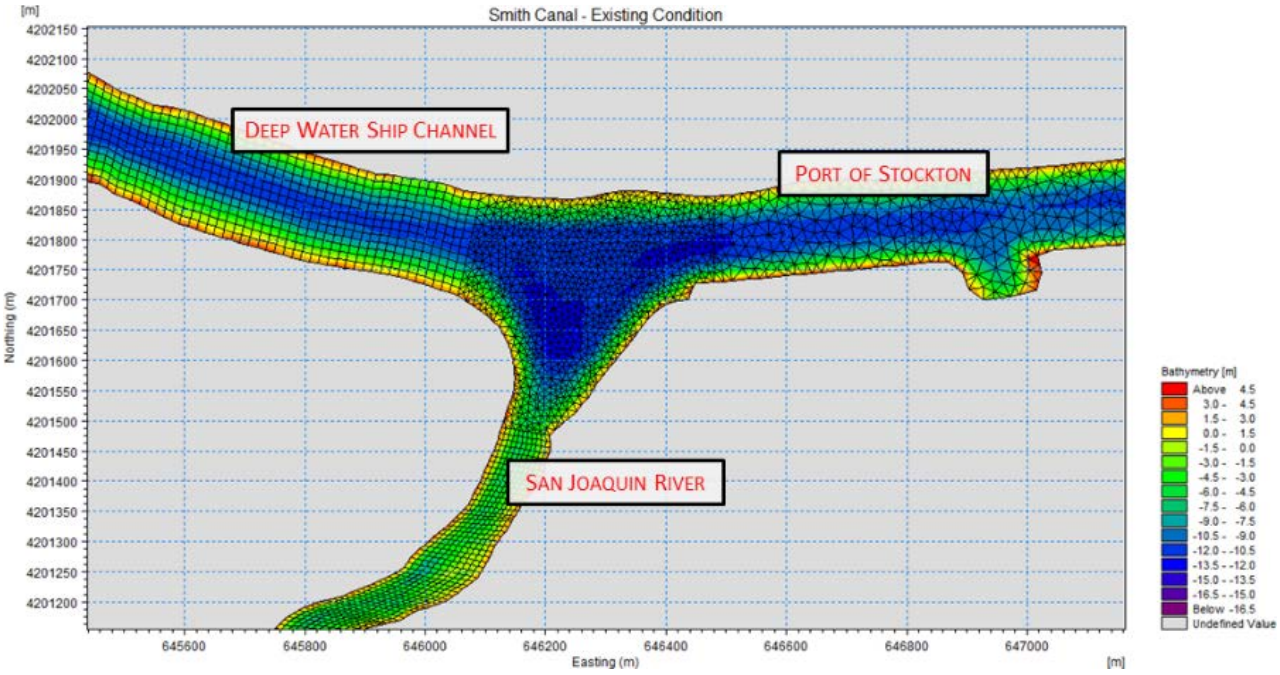


Figure 4-7: MIKE21-FM Mesh at Connection of San Joaquin River and Deep Water Ship Channel

4.2 HYDRODYNAMIC MODEL

4.2.1 Boundary Conditions

As shown in Figure 4-3, there are three open model boundaries:

1. Main Channel (SDWSC) – Measured instantaneous stage data (15-min interval) from RRI Gage, downloaded from California Water Library (CDWR 2014b), and California Data Exchange Center (CDWR 2014c), were applied at this open boundary. The RRI Gage is located between Nodes 20 and 21 in the DSM2 model shown in Figure 4-9. A comparison of DSM2 data provided by the State of California (Bay-Delta Office 2014) between Node 22 and average of Nodes 20 and 21 indicates that the stage difference between the two locations is less than 0.04 ft or 1%, as shown in the lower plot of Figure 4-8. Essentially, there is a negligible tidal muting between these two locations. Note that the DSM2 data are simulation results of the DMS2 model of the current condition.

2. San Joaquin River – Measured instantaneous discharges (15-min interval) at the Garwood Bridge of the San Joaquin River (SJG Gage) were applied at this open boundary. The data was downloaded from the United States Geological Service (USGS) web site (USGS 2014) and the California Data Exchange Center (CDWR 2014c) website.

3. Calaveras River - Daily average synthetic discharges provided by the State of California (Bay-Delta Office 2014) were applied in this boundary. In order to capture the full tidal prism of the Calaveras River without extending the model domain too far, a reservoir was added to capture the missing tidal prism of the river beyond the model boundary. The size of reservoir is determined based on estimation of the missing tidal prism.

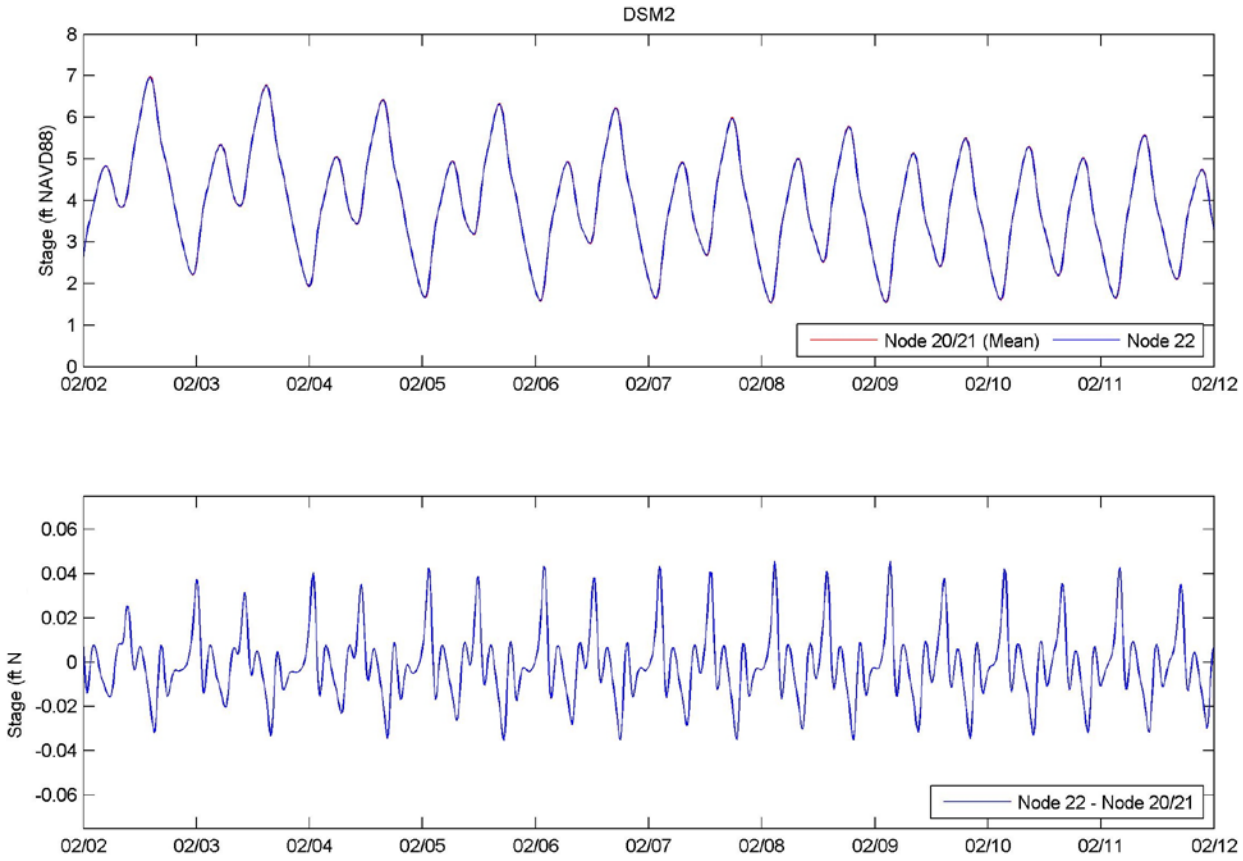


Figure 4-8: Stage Comparison between Node 22 and Average of Nodes 20 and 21

4.2.2 Model Calibration and Validation

The MIKE21-FM-HD was calibrated for a period in November 2008 and validated for a period in January 2009. These two time periods were selected because:

- measured stage and discharge data at both RRI and SJG Gages were available to compare model output against; and
- this is a dry period so discharge data from pump stations within the Smith Canal are not required.

The RRI and SJG Gage locations are shown in Figure 4-9 as green dots. Ideally, the model should have been calibrated to stage and flow data collected within Atherton Cove or Smith Canal, but there are no data available.

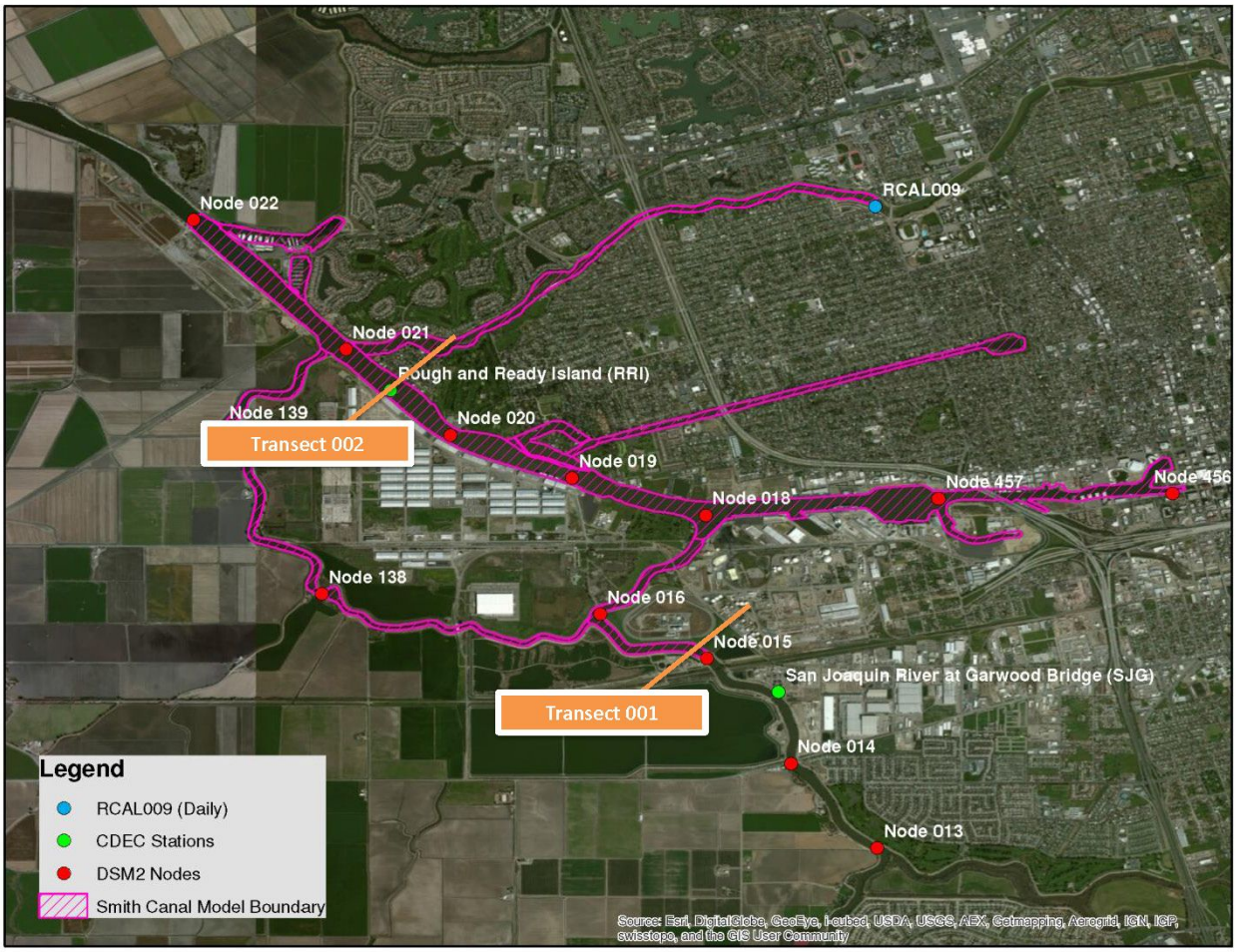


Figure 4-9: Gage and Transect Locations of HD Calibration

The calibration coefficients of the MIKE21-FM-HD model are roughness coefficient and eddy viscosity. The roughness coefficients are the primary calibration parameter in the calibration process, which involves matching the model stage and discharge with those measured at Transect 002 (RRI Gage), which is shown in Figure 4-9. Comparison of the model-predicted stage and discharge at Transect 001 (see Figure 4-9) with those measured at the SJG Gage was also made. Although the SJG Gage is not exactly located at Transect 001, there are no tributaries between the modeled location and gauge location, so the stage and discharge should be very similar between these two locations.

The horizontal eddy viscosity was based on the Smagorinsky formulation, with a constant Smagorinsky coefficient value selected for the entire model domain. The recommended value for the Smagorinsky coefficient ranges from 0.25 to 1.0, while the default value in the MIKE21-FM-HD model is 0.28. A sensitivity analysis (M&N 2014) was performed for the eddy viscosity coefficient and the results indicate the default value of 0.28 used in the last phase of modeling for the project is reasonable and is continuously being used in this phase of study.

Manning's *M* bed roughness was used as the primary parameter for refining the hydrodynamic model during the hydrodynamic calibration process. A constant roughness value was used for the entire modeling domain. The best match was found with a Manning's *M* value of 60, which corresponds to a Manning's *n* value of 0.0167. Model calibration results are presented in the following section.

4.2.2.1 Model Calibration Results

A period from November 19-24, 2008 was selected as the hydrodynamic model calibration period. Comparison of model-predicted stage and discharge data were compared with those measured at RRI Gage during the model calibration process while varying the roughness coefficient. The best match was achieved with a roughness coefficient *M* value of 60. Figure 4-10 and Figure 4-11 show the model calibration results at Transect 001, and Figure 4-12 and Figure 4-13 show the model calibration results at Transect 002. These figures only show three days of data, as the first two days of the model warm-up period are excluded. The DSM2 results are also included in the time series plot for information.

Two statistical parameters: the R^2 correlation coefficient and normalized root mean square error (NRMSE) were used to assess model calibration and validation. The R^2 correlation is a measure of the linear correlation (dependence) between two variables. These two variables are measured data and model-predicted data in this case. The R^2 correlation provides a measure of how well measured data are replicated by the model. A value of 1 means two variables are perfectly replicated. A value of 0 implies there is no correlation between the two variables. The root-mean-square error (RMSE) is also a frequently used measure of the differences between values predicted by a model and the values actually observed. Basically, the RMSE represents the sample standard deviation of the differences between predicted values and observed values. RMSE is a good measure of accuracy. The NRMSE is the RMSE divided by the range of observed values of a variable being predicted. The value is often expressed as a percentage and called the relative error, where lower values indicate less residual variance.

The result comparison indicates:

- At Transect 001 (SJG Gage), the model-predicted discharge matches the measurement better than that at Transection 002. Good discharge comparison was expected since the imposed discharge boundary condition is close to this transect. The stage comparison indicates that the model-predicted stage is slightly (1 to 2 in.) higher than those measured. The excellent agreement of the model with measured stage at this transect is very encouraging since stage was not an imposed boundary condition at the SJR boundary, and the imposed stage boundary condition of the Main Channel is at the other end of the model domain.

- At Transect 002 (RRI Gage), the model-predicted tidal phase and range (stage) match almost perfectly with those measured. Good model agreement with measured stage at this transect is expected since a stage boundary condition was imposed near this transect. The model-predicted discharge matches the measurement quite well with the exception of the brief peak flow spikes and wiggles shown in the measured data. The cause for this could be fourfold: (1) accuracy of the measurements; (2) limitations in the modeling of wetting/drying processes; (3) bathymetry accuracy of the SDWSC; and (4) measurement accuracy of the discharge as discharges measured at RRI Gage have always had accuracy issues, based on personal conversation with Russ Brown at ICF International. Metadata for measured stage and discharge is not available on websites of the CDWR and USGS. Regardless of the causes of the peak flow wiggles and the inability of the model to capture them, the computed flows agree well with measured. The primary differences in the discharge scatter plot of Figure 4-13 are due to the peak flow oscillations in the measured data. These differences should not affect the accuracy of project alternative assessment and comparison as all alternatives, including the existing conditions, will be affected equally.

Comparison with the DSM2 results indicates the MIKE21-FM-HD model performed better than the DSM2 model in both tidal range and phase. The DSM2 model tends to over-predict the tidal

range more so than MIKE21-FM-HD model and has a phase shift issue. The statistic error analyses, shown in Table 4-2, indicate that the correlation coefficient is nearly 1 for both stage and discharge; hence, the model-predicted variables well replicated the measurements. The NRMSE are also very low. The relatively large NRMSE value of discharge at Transect 002 may be the result of spikes in the discharge measurements.

Table 4-2: Statistic Error Analyses for Hydrodynamic Model Calibration

Variable	Transect	R^2 correlation coefficient	NRMSE (%)
Stage	001	0.996	1.8
	002	0.999	0.7
Discharge	001	0.9996	6.8
	002	0.923	30.7

Overall, the MIKE21-FM-HD model predicted very satisfactory results for the purpose of project alternative evaluation and potential impact analyses.

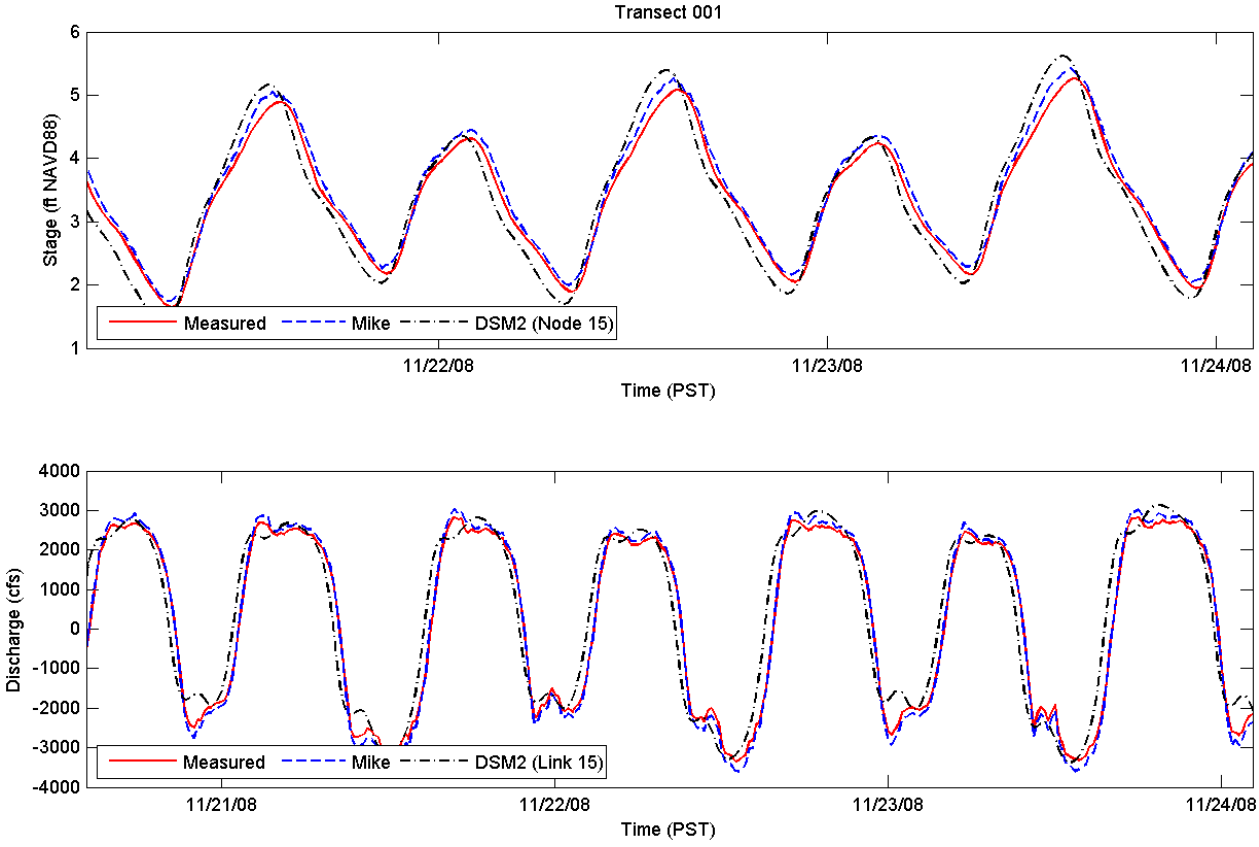


Figure 4-10: HD Model Calibration Time Series Plots at Transect 001 (SJG Gage)

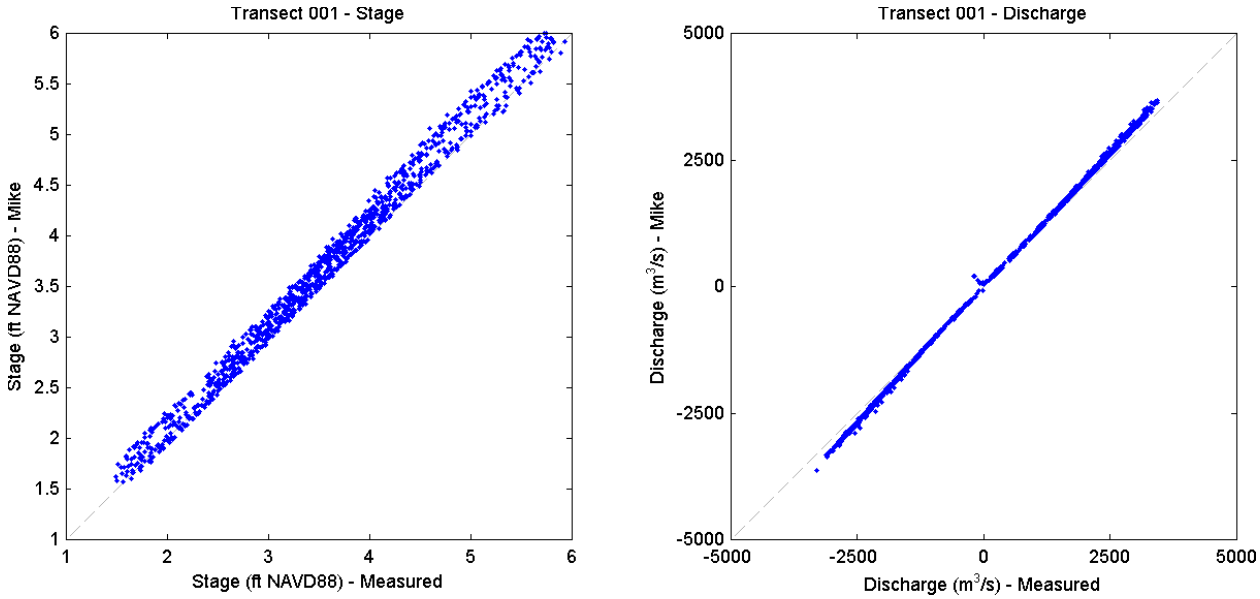


Figure 4-11: HD Model Calibration Scatter Plots at Transect 001 (SJG Gage)

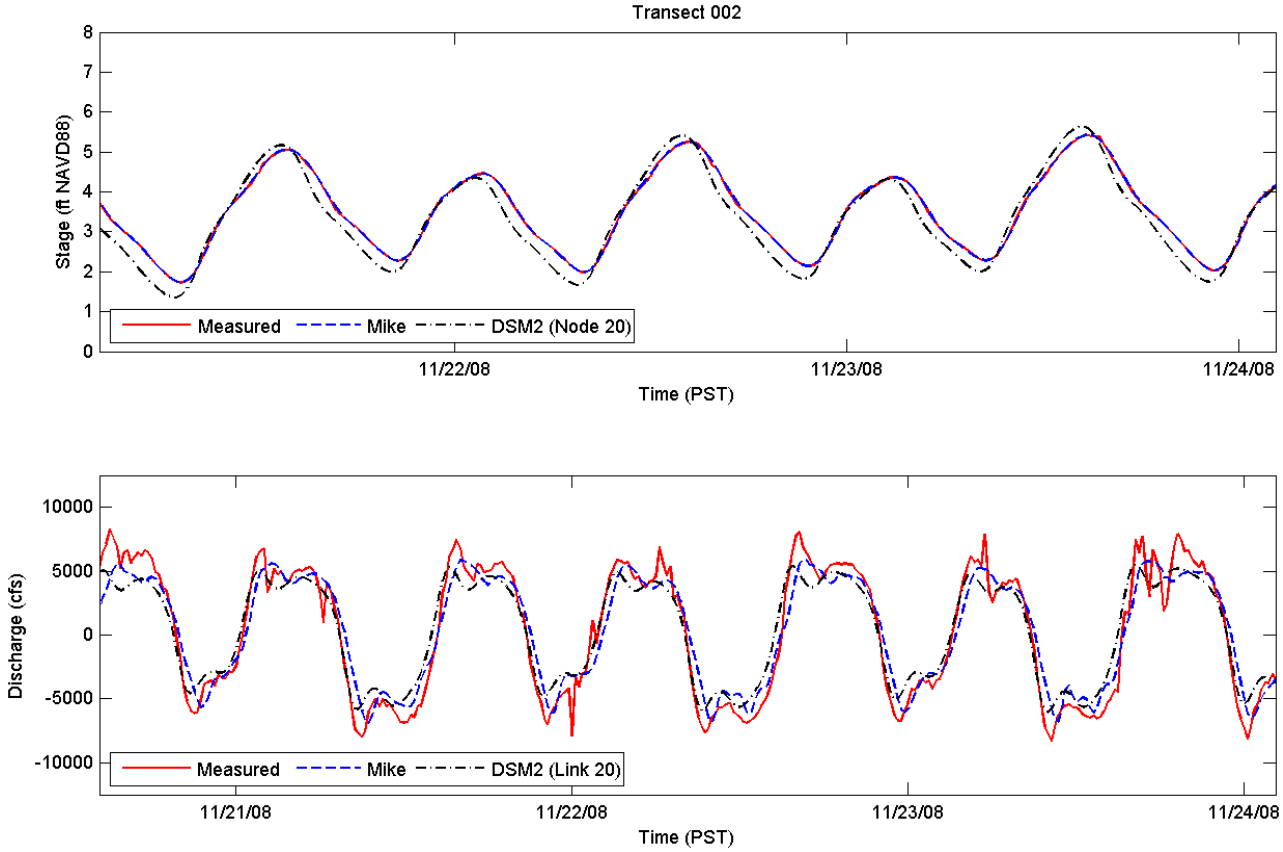


Figure 4-12: HD Model Calibration Time Series Plots at Transect 002 (RRI Gage)

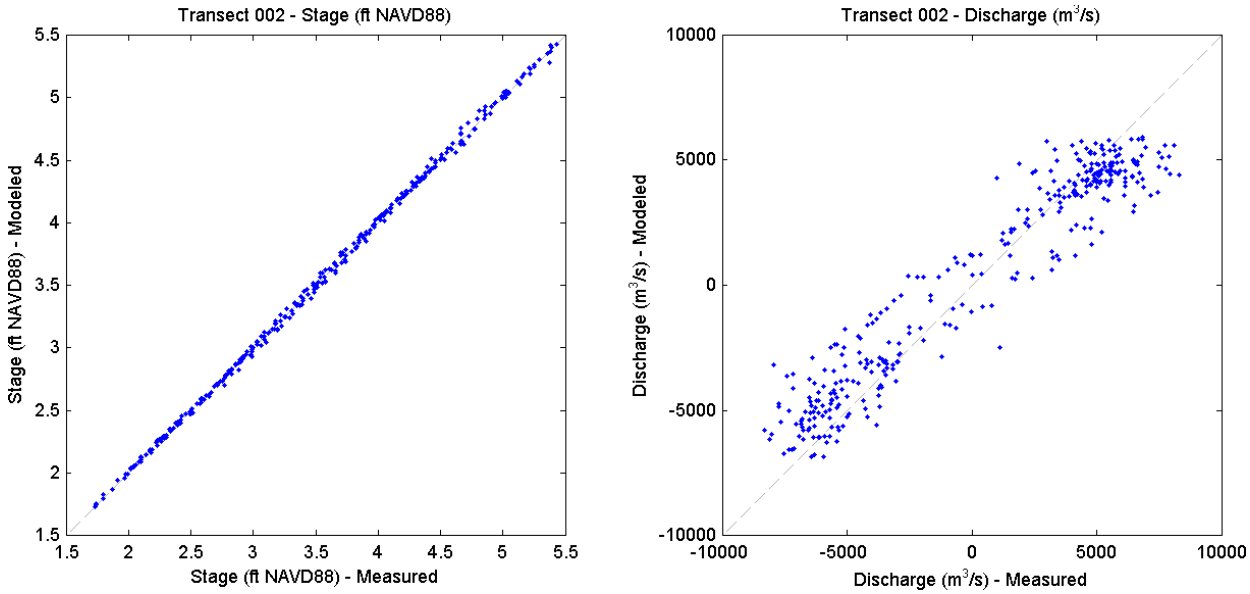


Figure 4-13: HD Model Calibration Scatter Plots at Transect 002 (RRI Gage)

4.2.2.2 Model Validation Results

A separate 2-week period from January 1-15, 2009, which is also a dry weather period, was selected for MIKE21-FM-HD model validation. The purpose of the validation was to test the model with another input data set while using the calibrated roughness coefficients and eddy viscosity. The modeling results from January 3-14, 2009, excluding the first two days of model warm-up, are shown in Figure 4-14 and Figure 4-15 for Transect 001 and Figure 4-16 and Figure 4-17 for Transect 002. The results are consistent with findings from the model calibration.

- At Transect 001 (SJG Gage), the model-predicted discharge agrees very closely with measured data in terms of both phase and magnitude. The model-predicted tidal elevation and phase also match measurements very closely. The model-predicted tidal elevation is perched 1 to 2 in. higher than the measurements.

- At Transect 002 (RRI Gage), the model-predicted tidal range and phase match almost perfectly with those measured. The model-predicted discharge matches with the measurement quite well in terms of both phase and general magnitude. The model did not predict the brief peak-flow spikes shown in the measured data the same as those shown in the calibration comparison. It is noted that both measurement and model-computed results have limitations, and perfect model-measurement agreement for discharge is rarely, if ever, achieved in hydrodynamic modeling. The discharges measured at RRI Gage have always had accuracy issues based on personal conversation with Russ Brown at ICF International. Dr. Brown is the author of the dye studies performed in Smith Canal (Jones & Stokes 2005) The statistic error analyses for the model validation, shown in Table 4-3, indicate that the correlation coefficient similar to that of the model calibration is nearly 1 for both stage and discharge; hence, the model-predicted variables well replicated the measurements. The NRMSE are also very low. The relatively large NRMSE value in discharge at Transect 002 is the result of spikes in the discharge measurements, which should not affect the relative comparison between alternatives and those with the existing conditions.

Table 4-3: Statistic Error Analyses for Hydrodynamic Model Validation

Variable	Transect	R^2 correlation coefficient	NRMSE (%)
Stage	001	0.991	3.1
	002	0.999	0.8
Discharge	001	0.999	6.8
	002	0.958	28.7

Overall, the model performed very satisfactorily for the validation period. The hydrodynamic model is considered well calibrated and validated for use in assessing potential impacts of the proposed project.

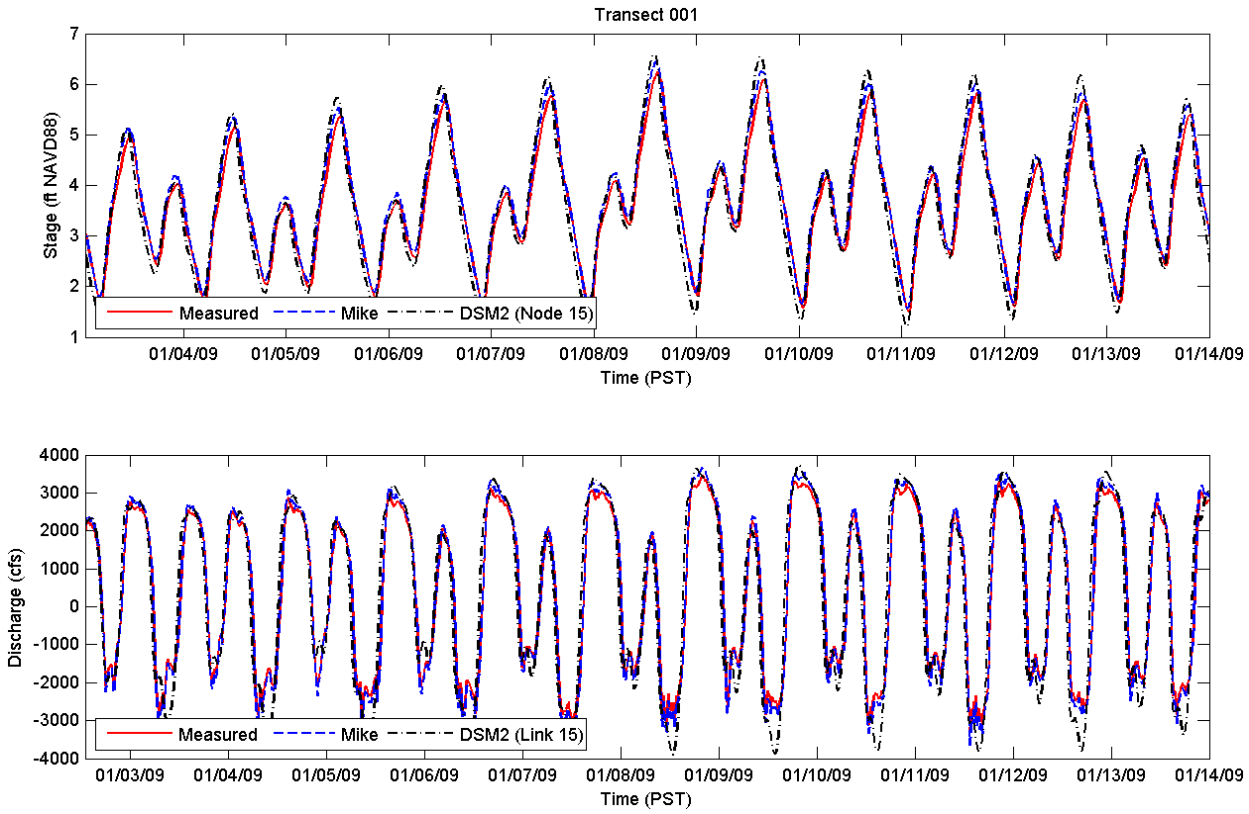


Figure 4-14: HD Model Validation Time Series Plots at Transect 001 (SJG Gage)

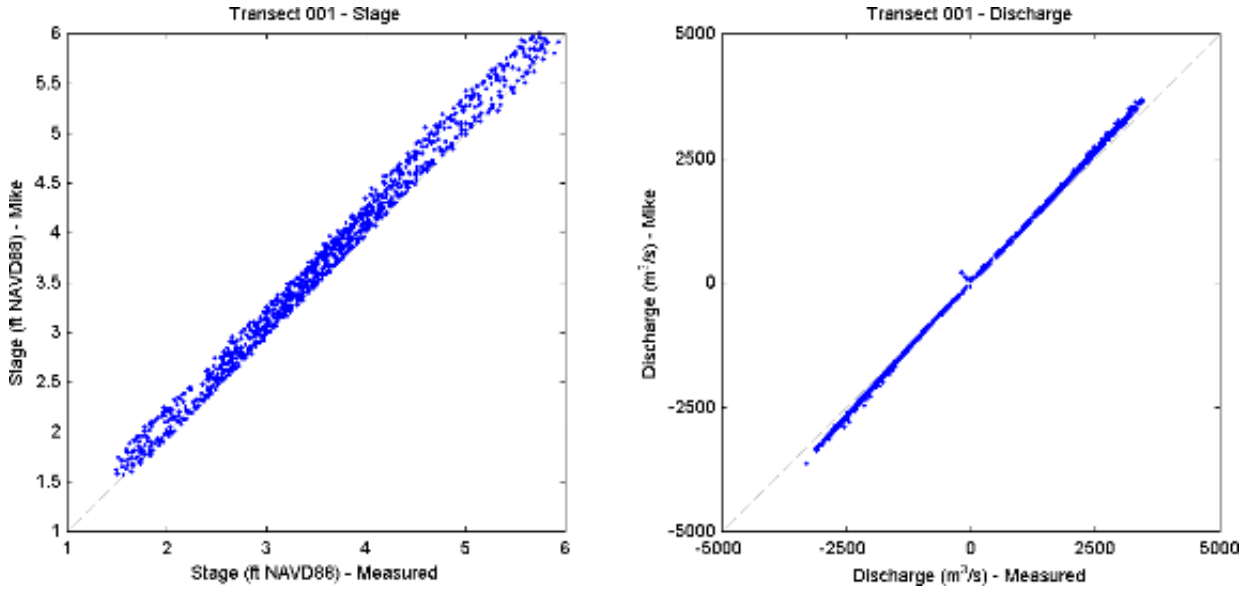


Figure 4-15: HD Model Validation Scatter Plots at Transect 001 (SJG Gage)

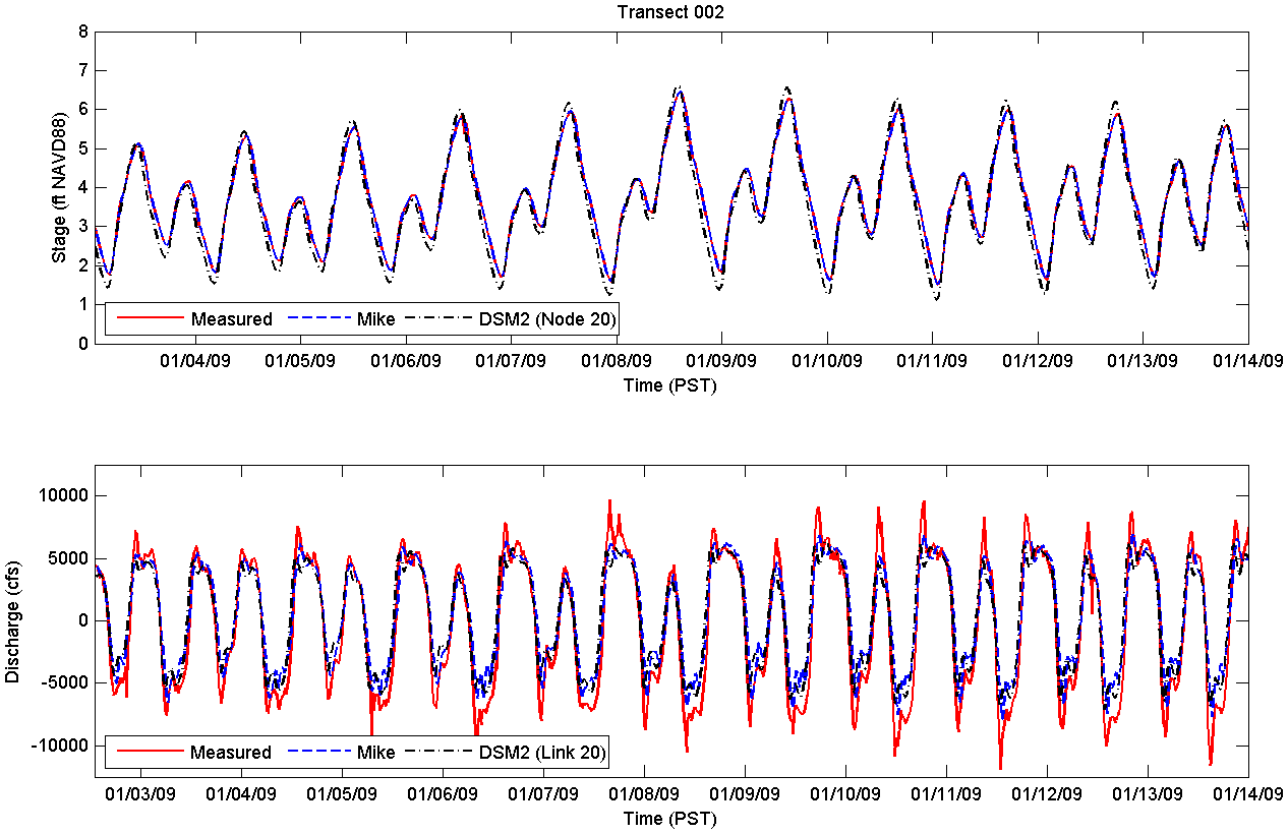


Figure 4-16: HD Model Validation Time Series Plots at Transect 002 (RRI Gage)

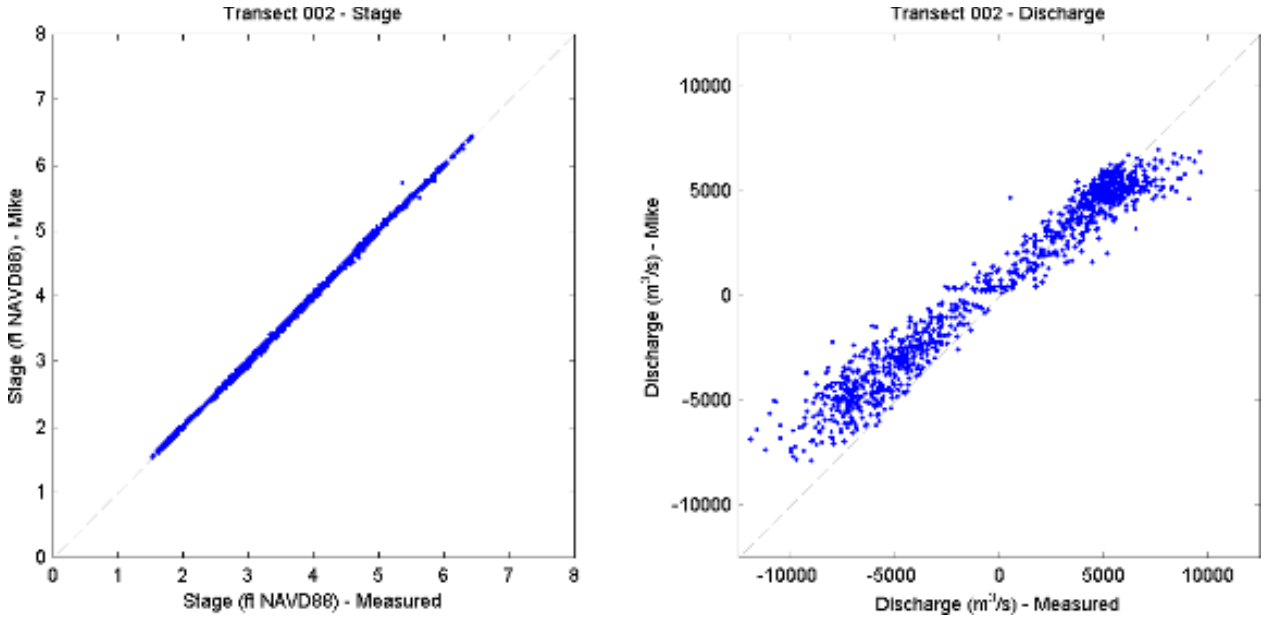


Figure 4-17: HD Model Validation Scatter Plots at Transect 002 (RRI Gage)

4.3 TRANSPORT MODEL

Transport Module (MIKE21-FM-TR) simulates the spreading and fate of dissolved or suspended substances under the influence of advective and dispersive fluid transport processes. The transport modeling is useful as it directly simulates the pollutant transport as comparing to hydrodynamic modeling, which calculates the tidal variation and velocity. The initial conditions are the spatial distribution of the component concentration throughout the computational domain at the beginning of the simulation. The resulting transport of the conservative materials is based on the flow conditions provided by the hydrodynamic calculations (MIKE21-FM-HD).

4.3.1 Transport Model Calibration

The MIKE21-FM-TR model was calibrated against a dye tracer test (Litton 2004) performed during a rainfall event in February 2004. In the study, the dye tracer was injected directly into the effluent from the Legion Park Pump Station, which discharges into Yosemite Lake located at the upstream end of the Smith Canal. The Smith Canal watershed consists of 13 subareas; Yosemite Lake is the largest and most upstream subarea. These subareas are shown in Figure 4-18 and their drainage surface areas are listed in Table 4-4. Urban runoff affected the transport of dye tracer in the Smith Canal during the dye-tracer study; therefore, the estimated urban runoff during the rainfall event was incorporated into the model as a point-source runoff. Pump outflows of the Legion Park Pump Station during the dye tracer test (see Figure 4-19) was provided by the City of Stockton via ICF International. Pump data for the other outflows was requested from the City of Stockton but no data was available from any of the remaining 12 pump stations. Therefore, to account for the additional flow into Smith Canal, the outflows from the Legion Park Pump Station was increased by a factor of 2 based on drainage areas of the additional subareas relative to the Legion Park (Yosemite Lake) Pump Station subarea. The precipitation data shown in Figure 4-19 are recorded at Stockton Fire Station No. 4. The latitude and longitude of the station are 38º 00' and 121º 19', respectively.

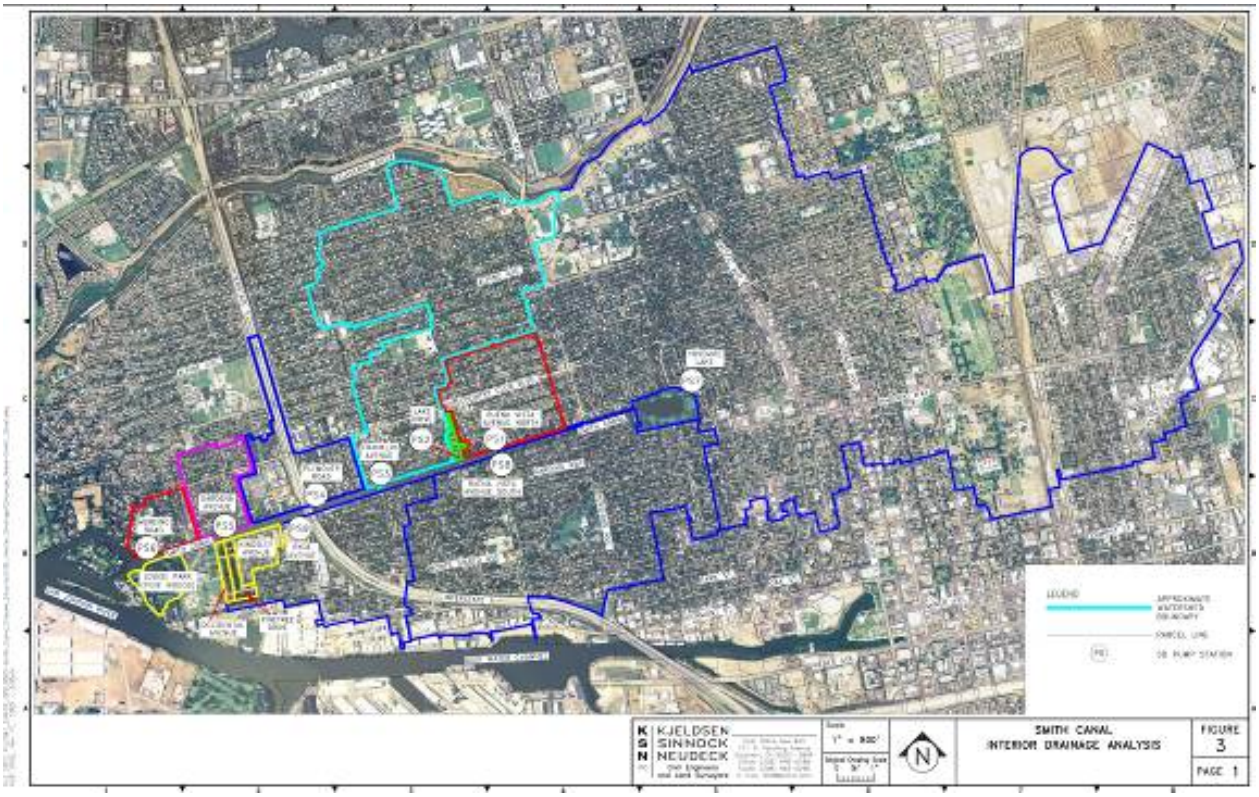


Figure 4-18: Smith Canal Pump-Outfall Locations and Drainage Subareas (Conrad 2010)

Table 4-4: Smith Canal Drainage Subarea Surface Area (Conrad 2010)

Drainage Subarea	Drainage Subarea Name	Drainage Subarea Area (Ac)
1	Buena Vista North	122
2	Lake Dr.	4
3	Franklin Ave.	425
4	Plymouth Rd.	90
5	Gardena PS	55
6	Moreing Rd.	36
7	Yosemite Lake	1936
8	Buena Vista South	477
9	Ryde Ave.	167
10	Kingsley Ave.	18
11	Pinetree Dr.	8
12	Occidental Ave.	6
13	Louis Park	21

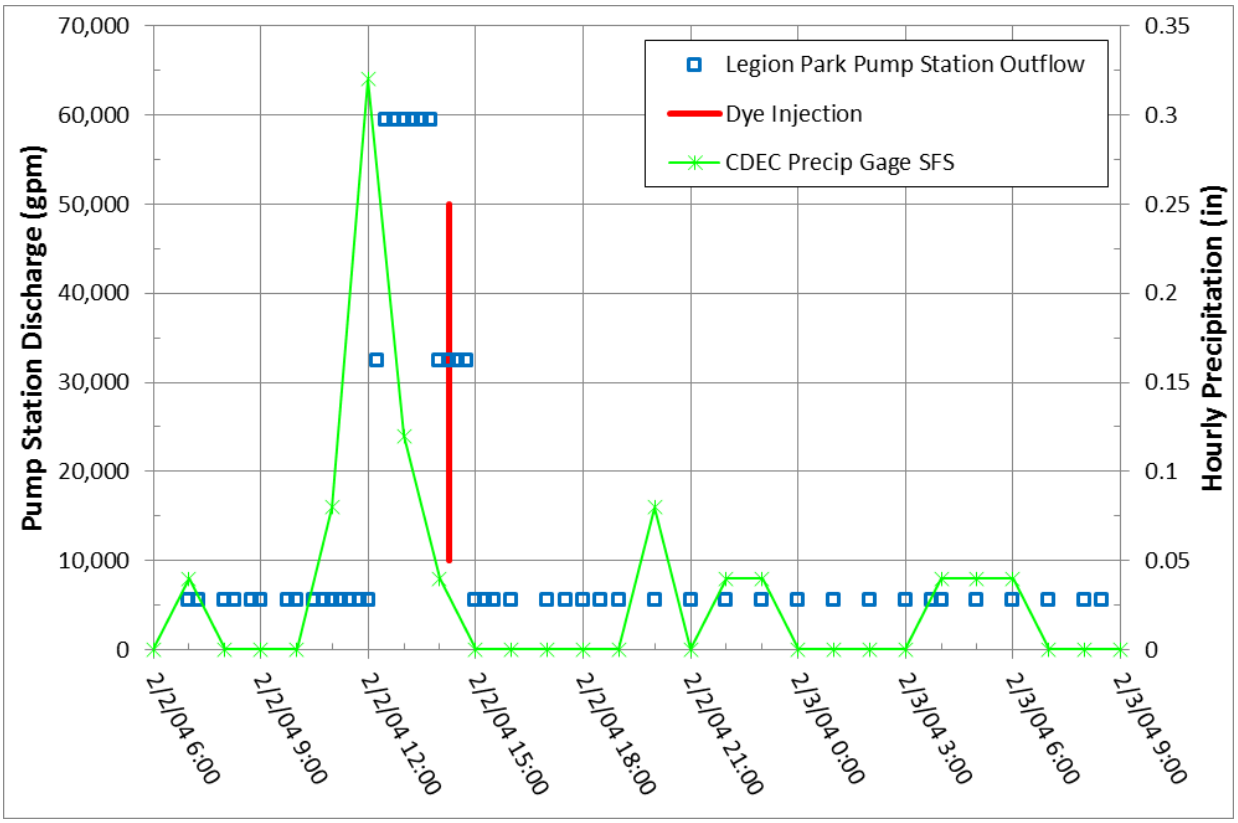


Figure 4-19: Legion Park Pump-Station Outflow (Litton 2004) and Precipitation Data at Stockton Fire Station for February 2004 Wet-Weather Event

The concentration of dye added to the transport model was based on information presented in the dye tracer study (Litton 2004). The dye tracer study details a mass of 912-g of Rhodamine that was added to the pump-station effluent over a period of 30 minutes, during which the pump-rate from the station was 30,500-gpm. Therefore, based on the documented mass of Rhodamine and pump rate, a dye concentration of 250 µg/L was used in the model. This dye was added as a point source for a 30-minute period from the Yosemite Lake Pump Station.

The calibration parameter of the transport model is the horizontal dispersion coefficient, which is required for the dispersion term in the transport equation. A constant value was applied for the entire model domain. The calibration process involved matching the model-predicted dye tracer concentration profiles with those presented in the dye tracer study by Litton (2004).

4.3.2 Calibration Results

The discharge for the MIKE21-FM-TR model calibration period in February 2004 was compared between model-predicted and those measured at SJG gage. The discharge data for RRI gage were not compared due to the lack of data consistency. The stage for the MIKE21-FM-TR model calibration period in February 2004 was also compared between model-predicted and those measured at RRI gage. The stage data for SJG gage was not available for comparison. The model-predicted data closely matched the measured data, results are shown in Figure 4-20 and Figure 4-21 for SJG and RRI gages, respectively.

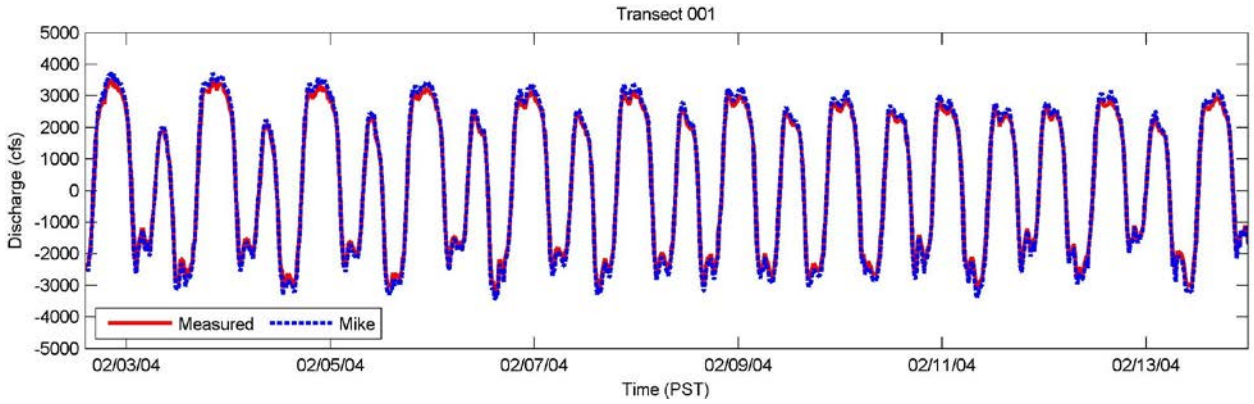


Figure 4-20: Comparison of Discharge at SJG Gage during the Dye Tracer Study

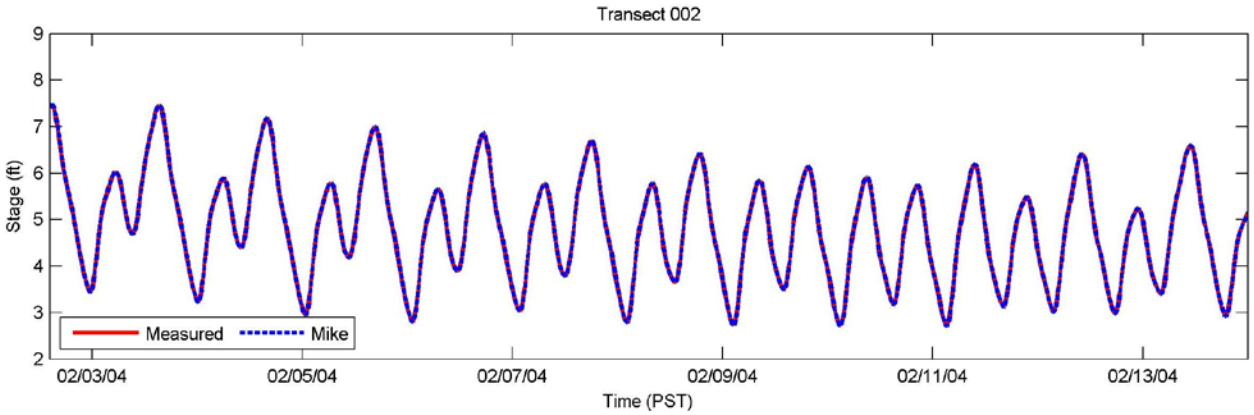


Figure 4-21: Comparison of Stage at RRI Gage during the Dye Tracer Study

A range of dispersion coefficients from 0.1 to 53.8 ft^2/s (0.01 to 5 m^2/s) was tested. The best match was achieved with a dispersion coefficient of 12.9 ft^2/s (1.2 m^2/s). The final dye tracer calibration results are illustrated in Figure 4-22. The results indicate:

- the model-predicted peak dye concentrations agree reasonably well with monitored peak concentrations for all five monitoring dates; and
- the model-predicted dye concentration profiles also match reasonably well with the measurements for all five monitoring dates. The best match occurs over the February 13, 2004 measurements.

Overall, the MIKE21-FM-TR model predicted acceptable dye concentration results. The model is considered to be acceptable for the purpose of alternative comparison and evaluation of the proposed project.

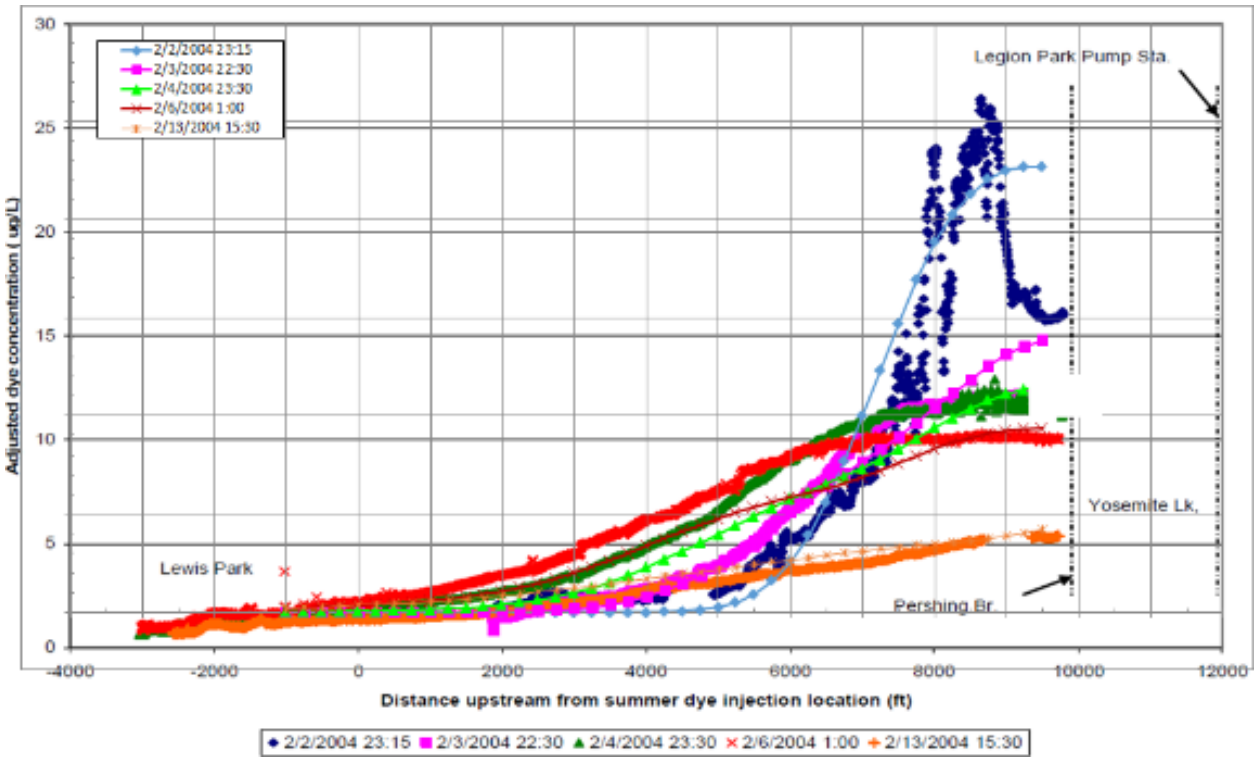


Figure 4-22: Comparison of Dye Concentrations Predicted by MIKE21-FM-TR Model versus Those Measured in the Dye-Tracer Study (Litton 2004)

4.4 CONCLUSIONS

The stage and discharge, including their phases and magnitudes predicted by the hydrodynamic model, match closely with those measured. The dye tracer peak concentrations and concentration profiles predicted by the transport model also match well with those measured. These results indicate both the hydrodynamic and transport models are well calibrated for use in alternative comparison and evaluating potential impacts of the proposed projects.

5. ANALYSES OF VELOCITY AND SEDIMENTATION POTENTIAL

A two-month period from June through July 2007 was selected for the velocity and sedimentation potential analysis for the following reasons:

- It is a dry summer period, therefore eliminating the need for inflow data from the Smith Canal pump stations, which are often not available;
- It (June through July) has larger than average spring tides, which is advantageous for assessing tidal muting in the study area; and
- It has measured data for model boundary input.

In addition to the existing condition, five proposed alternative scenarios were modeled. These five scenarios include the three proposed Alternative Alignments discussed in Section 1.3 and two additional gate widths of Alternative 1. Five numerical models were created by editing meshes within Atherton Cove and Smith Canal to incorporate the proposed closure alignment structures into the baseline existing condition model. Outside of this area, the model mesh remained unchanged from the existing condition. Sample model meshes within Atherton Cove and Smith Canal are presented in Appendix A.

5.1 HYDRODYNAMIC MODELING RESULTS IN SMITH CANAL

In order to assess impacts on tidal exchange and circulation in Atherton Cove and Smith Canal, two transects, one within Atherton Cove and one within Smith Canal, were selected and shown in Figure 5-1. Modeling results of stage and discharge were extracted as changes in stage and discharge are indications of tidal exchange and circulation changes. The stage is defined as the average surface elevation across transect. The discharge is the total flow rate through the cross-section where transects are located. The discharge is defined as positive if the water is flowing into Smith Canal/Atherton Cove or negative if it is flowing out. In other words, the flood tide discharge is positive and ebb tide discharge is negative.

The stage and discharge under each proposed alignment scenario were compared to those under the existing conditions. Similar to the methods used in model calibration and validation, the correlation and NRMSE, defined in Section 4.2.2, were calculated for each comparison.

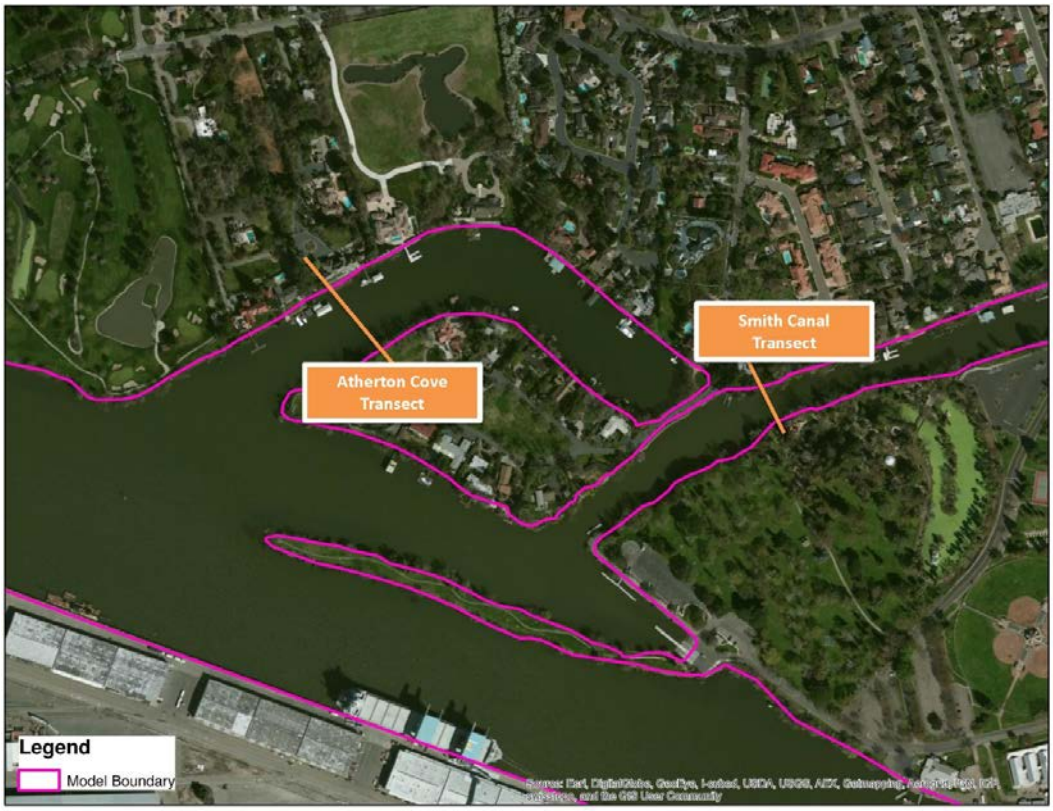


Figure 5-1: Location of Transects for Comparison of Hydrodynamic Results.

5.1.1 Alignments Comparison

At the Smith Canal Transect, the comparison of stage and discharge are shown in Figure 5-2 and Figure 5-3. The time series plots show that stage and discharge under the proposed alternatives overlap those under the existing condition. The scatter plots in Figure 5-3 also show a great match between existing and proposed alternative project conditions. The statistical analysis of the comparison is summarized in Table 5-1. These results indicate:

- For the stage comparison, the R^2 correlation coefficients are nearly equal to 1, indicating perfect matches and NRMS difference is less than 1%. Therefore, there is very little change in stage from the existing condition for all alignments.
- For the discharge comparison, the R^2 correlation coefficients are nearly equal to 1 and the NRMS difference is less than 5%. The largest difference in discharge is Alignment 1 and the smallest is Alignment 2. Therefore, there is a very small amount of change to discharge for all alignments when comparing to existing conditions.

Table 5-1: Statistic Analyses for Smith Canal

Variable	Alternative	R^2 Correlation Coefficient	NRMS Difference (%)
Stage	1	1.000	0.35%
	2	1.000	0.04%
	3	1.000	0.05%
Discharge	1	0.993	3.81%
	2	1.000	0.91%
	3	0.999	1.04%

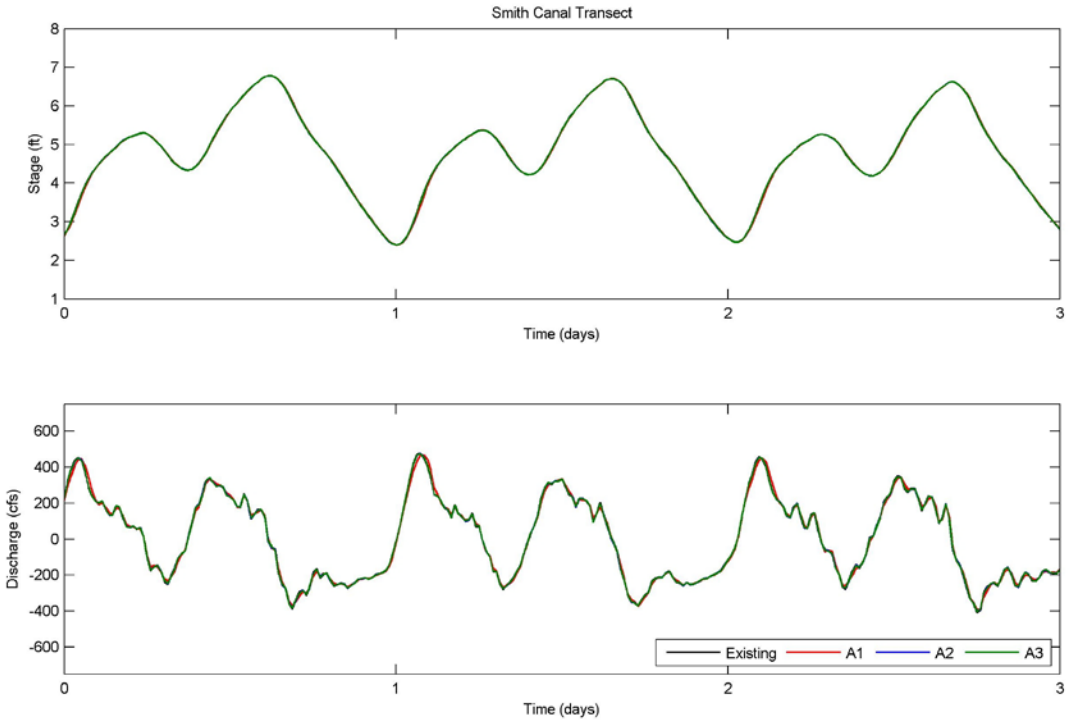


Figure 5-2: Sample Time Series of Stage (Top) and Discharge (Bottom) for Smith Canal

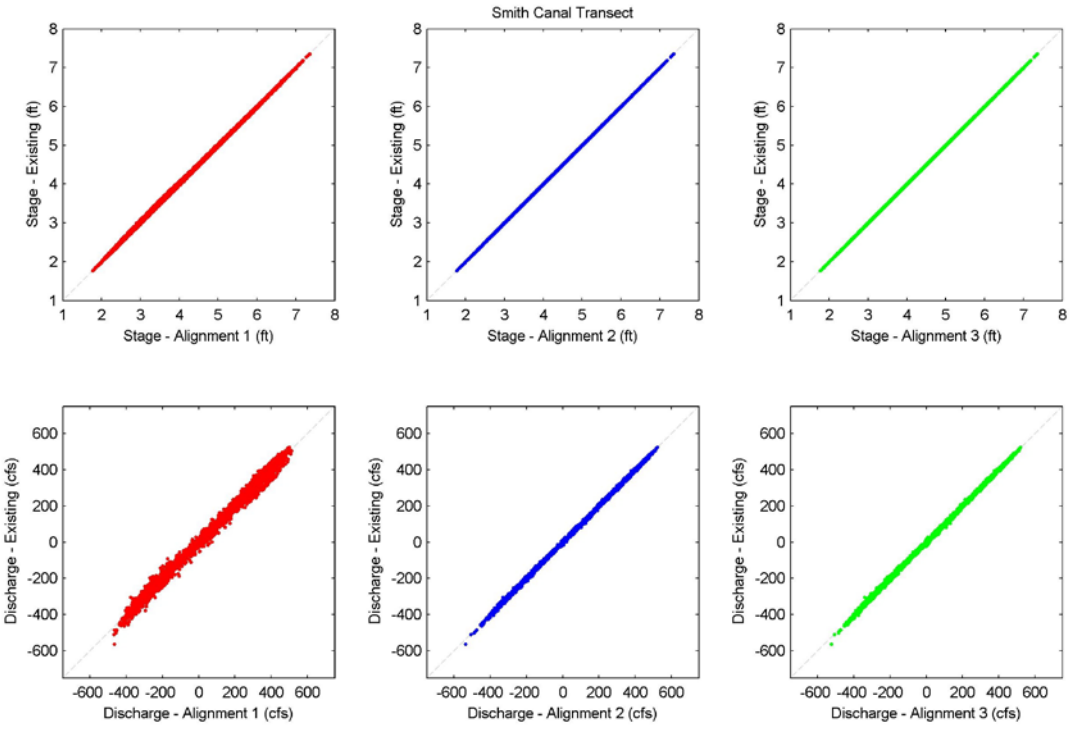


Figure 5-3: Stage (Top) and Discharge (Bottom) Correlation between Existing and Alterative Alignments for Smith Canal

At the Atherton Cove Transect, the comparison of stage and discharge are shown in Figure 5-4 and Figure 5-5. The time series plots show that stage under the proposed alternatives overlap those under the existing conditions. The scatter plots in Figure 5-5 also show a great match between existing and proposed alternative project conditions. The statistical analysis of the comparison is summarized in Table 5-2. These results indicate:

- For the stage comparison, the R^2 correlation coefficients are nearly equal to 1, indicating perfect matches and NRMS difference is less than 1%. Therefore, there is very little change in stage for all three alignments when comparing to the existing conditions.
- For the discharge comparison, there are slight changes from the existing conditions for all three alignments, especially for Alternative Alignment 2. For Alternative Alignment 2, the discharge is slightly lower than that under the existing conditions as the floodwall may have slightly reduced the tidal prism. For Alternative Alignment 1, the slight increase in peak discharge is a result of the narrow gate creating a jet flow toward Atherton Cove, which slightly alters the flow distribution between Atherton Cove and Smith Canal. But, the overall change in discharge is very small, as shown in Figure 5-4 and statistical analyses.

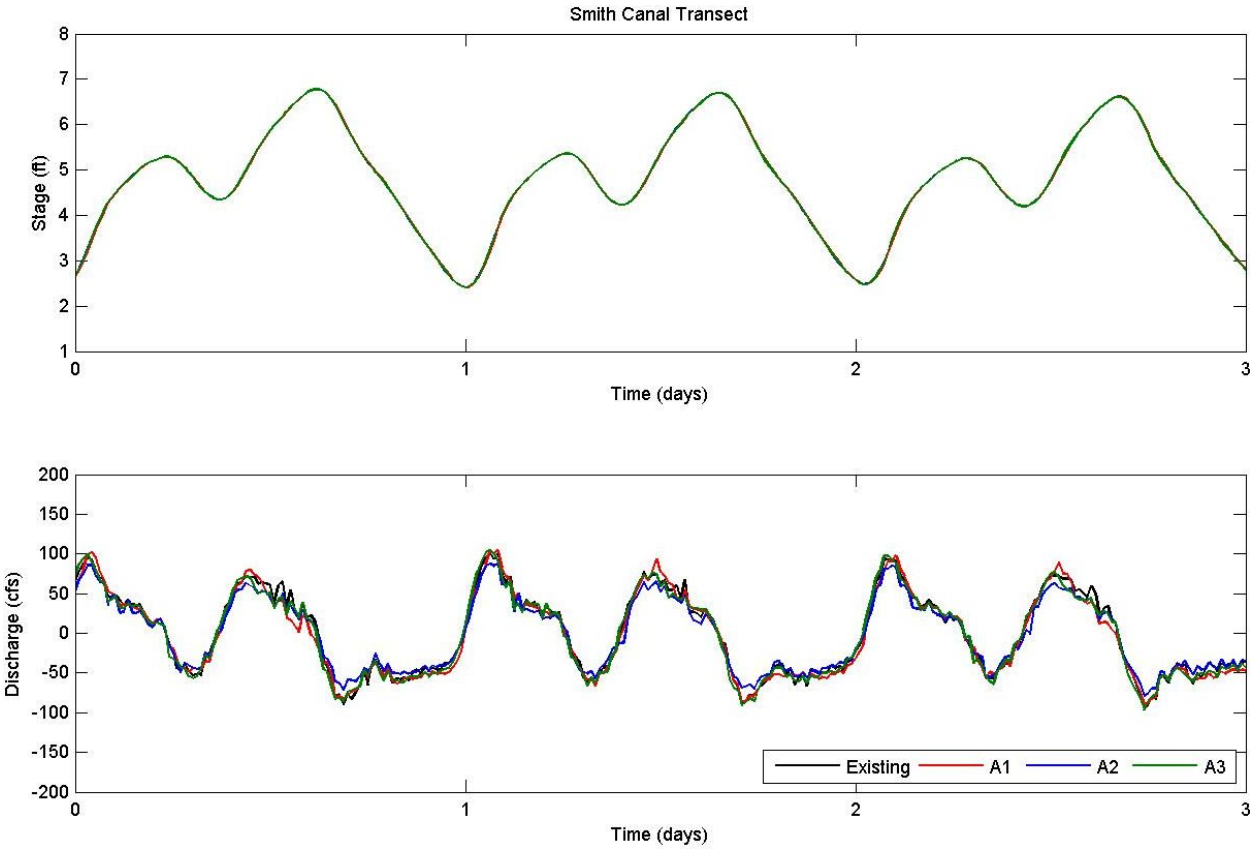


Figure 5-4: Sample Time Series of Stage (Top) and Discharge (Bottom) for Atherton Cove

Table 5-2: Statistic Analyses for Atherton Cove

Variable	Gate Width	R^2 Correlation Coefficient	NRMS Difference (%)
Stage	1	1.000	0.34%
	2	1.000	0.01%
	3	1.000	0.03%
Discharge	1	0.975	7.67%
	2	0.988	7.73%
	3	0.983	6.40%

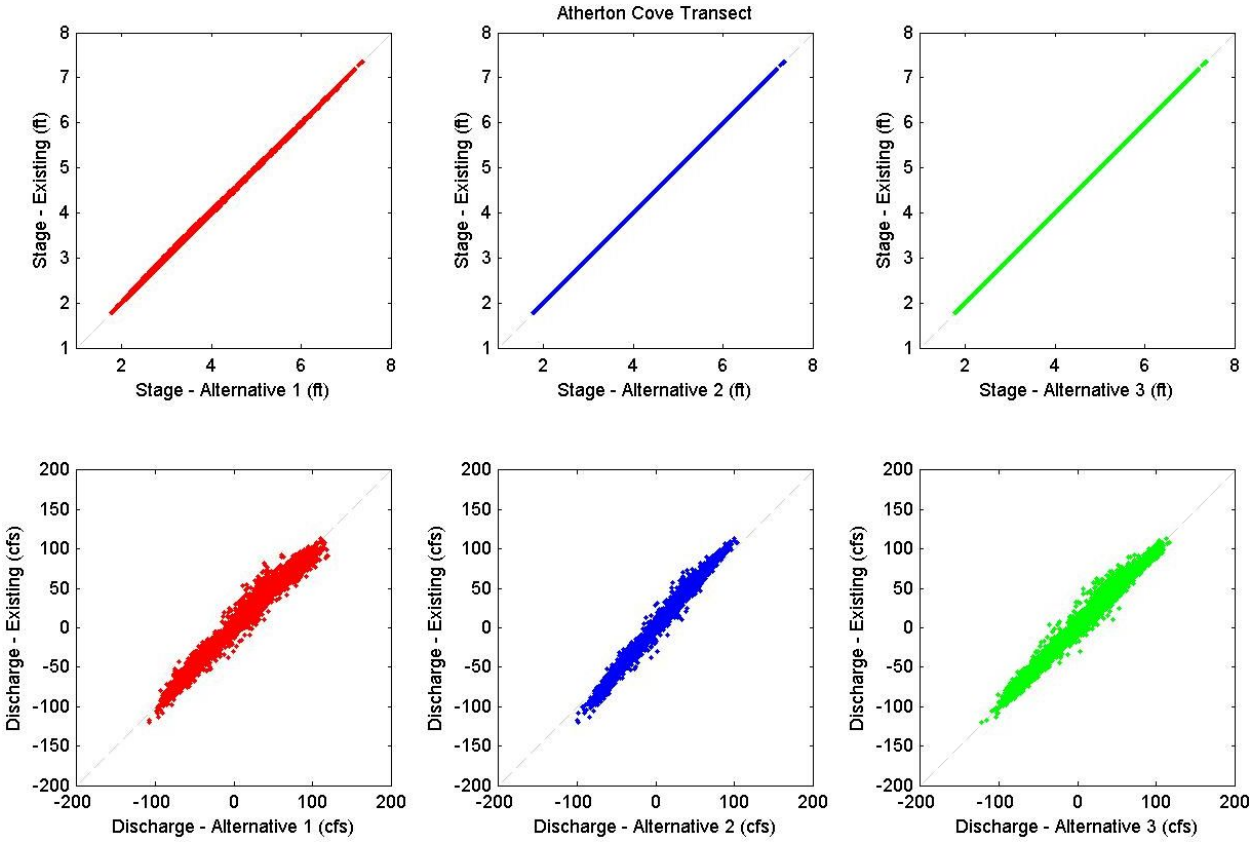


Figure 5-5: Stage (Top) and Discharge (Bottom) Correlation between Existing and Alterative Alignments for Atherton Cove

5.1.2 Alignment 1 Gate Width Sensitivity

To test gate width sensitivity, three gate widths of 50-ft, 100-ft, and 200-ft were modeled for Alternative Alignment 1. The resulting stage and discharge at the two transects shown in Figure 5-1 are compared between different gate width and existing conditions.

For the Smith Canal Transect, the comparison of stage and discharge between three gate widths and existing conditions are shown in Figure 5-6 and Figure 5-7. The statistical analysis of the comparison is summarized in Table 5-3. The comparison indicates:

- For stage comparison, the R^2 correlation coefficients are nearly equal to 1, indicating perfect matches and the NRMS difference is less than 1%. There is very little change in stage between the three gate widths.

- For discharge comparison, the R^2 correlation coefficients are nearly equal to 1, indicating perfect matches and the NRMS difference decreases with increased gate width but is less than 4% for all cases. Therefore, there is a small amount of change in discharge between different gate widths. For this location, the largest difference is the 50-ft gate and the smallest is the 200-ft gate.

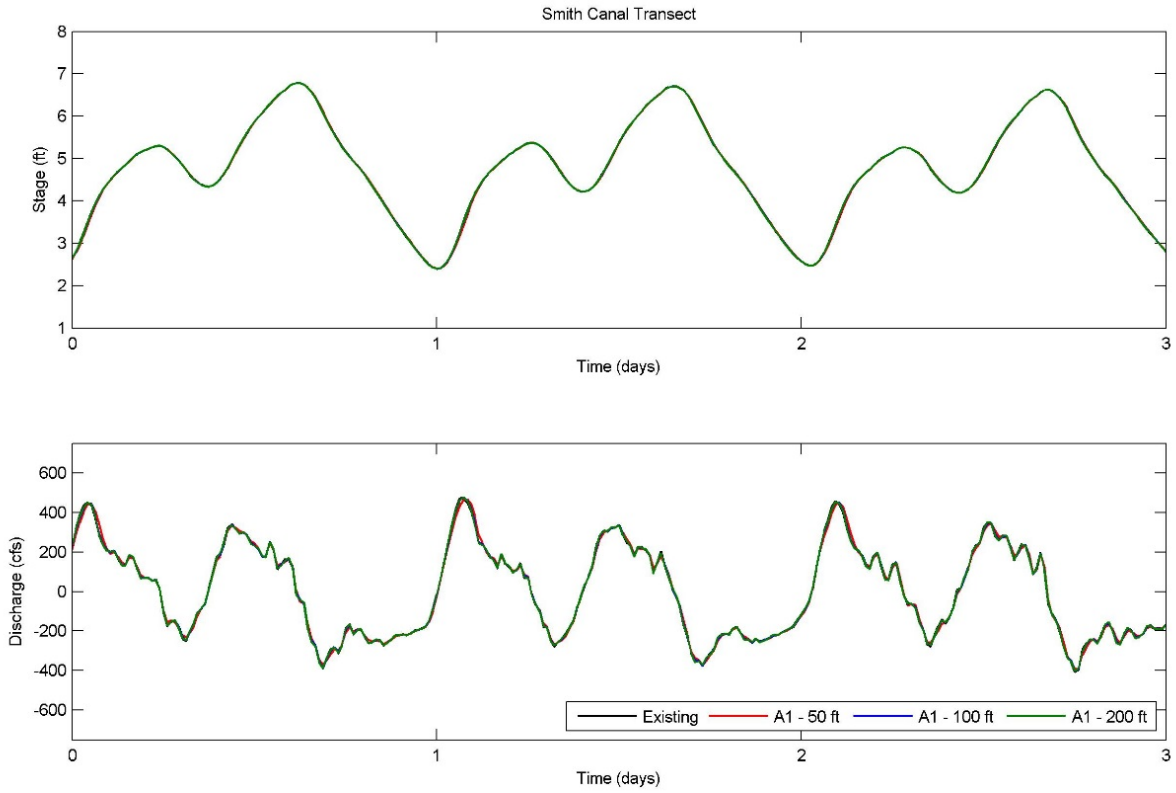


Figure 5-6: Sample Time Series of Stage (Top) and Discharge (Bottom) for Smith Canal

Table 5-3: Statistic Analyses for Smith Canal

Variable	Gate Width	R^2 Correlation Coefficient	NRMS Difference (%)
Stage	50 ft	1.000	0.35%
	100 ft	1.000	0.09%
	200 ft	1.000	0.03%
Discharge	50 ft	0.993	3.81%
	100 ft	0.999	1.39%
	200 ft	1.000	0.96%

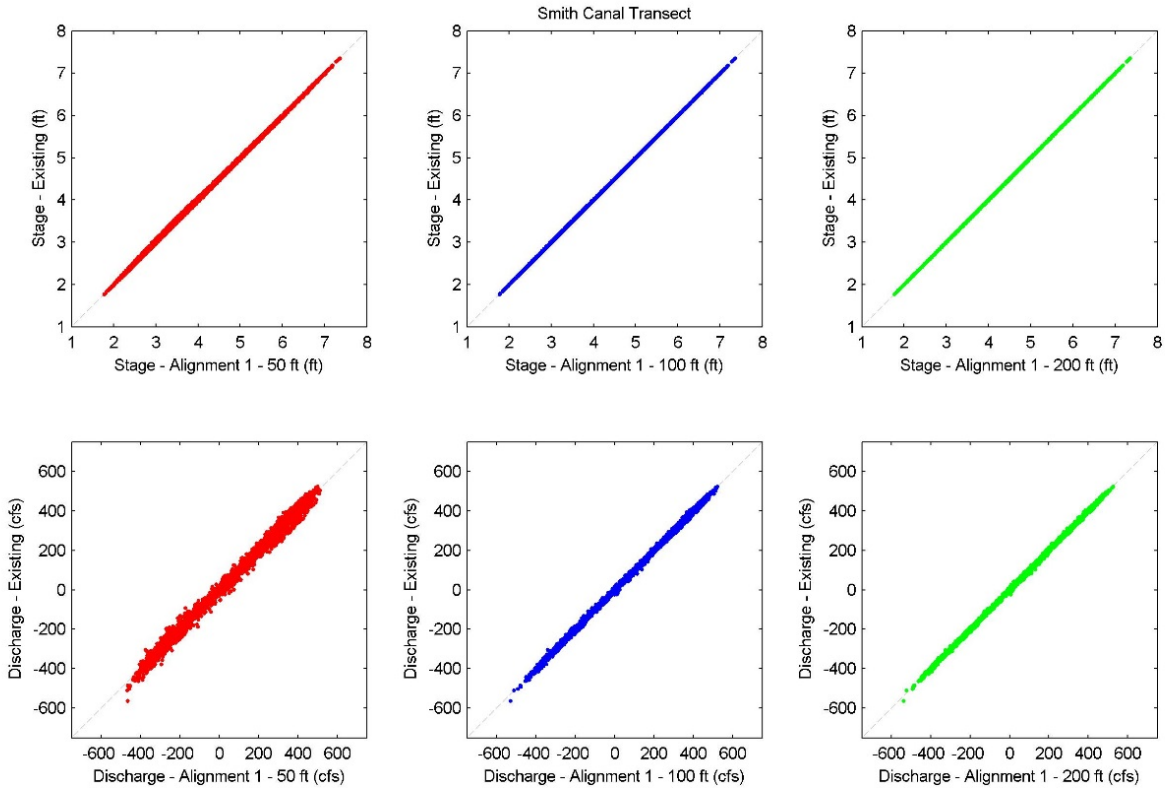


Figure 5-7: Stage (Top) and Discharge (Bottom) Correlation between Existing and Three Gate Widths of Alignment 1 for Smith Canal

For the Atherton Cove Transect, the comparison plots of stage and discharge are shown in Figure 5-8 and Figure 5-9. The statistical analysis of comparison is summarized in Table 5-4. The comparison indicates:

- For the stage comparison, there is very little change in stage between all gate widths. The R^2 correlation coefficients are nearly equal to 1 and the NRMS difference is less than 1%.
- For discharge comparison, there is also very little change in discharge between the three gate widths at this location. The R^2 correlation coefficients are nearly equal to 1 and the NRMS difference is less than 8%.

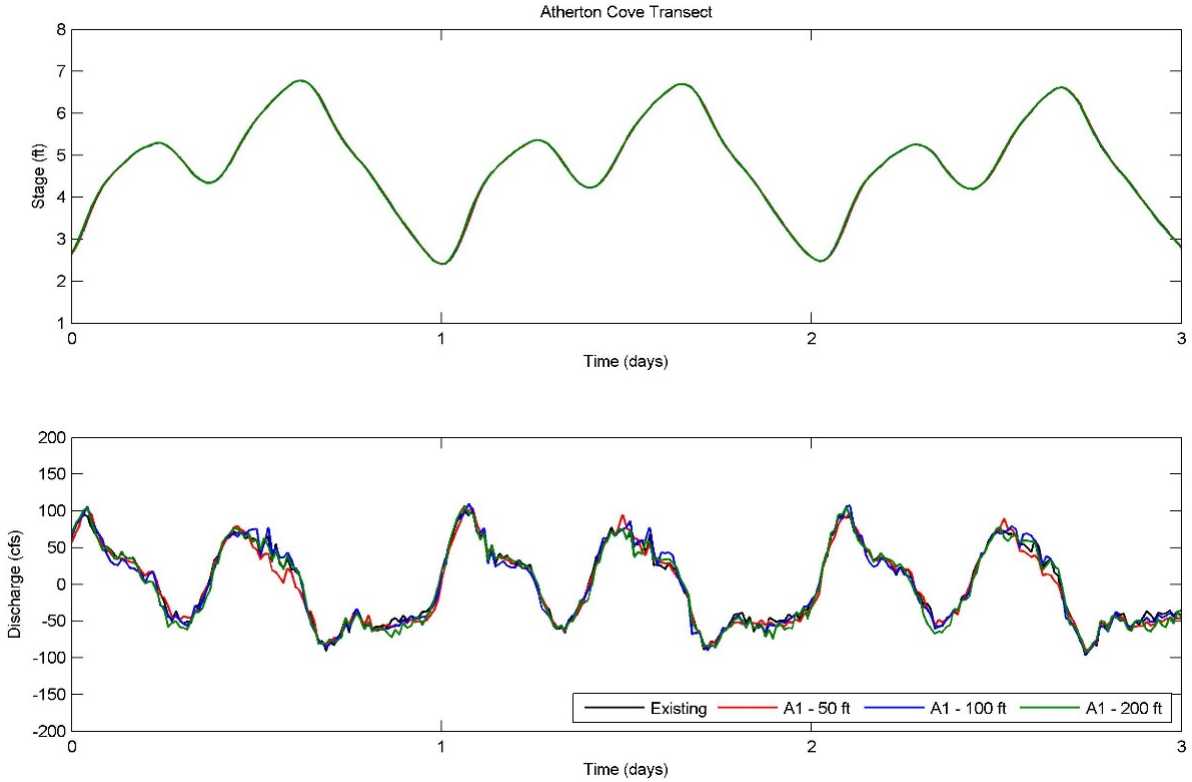


Figure 5-8: Sample Time Series from Smith Canal Sensitivity Study of Stage (Top) and Discharge (Bottom) for Atherton Cove

Table 5-4: Statistic Analyses for Atherton Cove

Variable	Gate Width	R^2 Correlation Coefficient	NRMS Difference (%)
Stage	50	1.000	0.34%
	100	1.000	0.08%
	200	1.000	0.02%
Discharge	50	0.975	7.67%
	100	0.982	6.62%
	200	0.979	7.51%

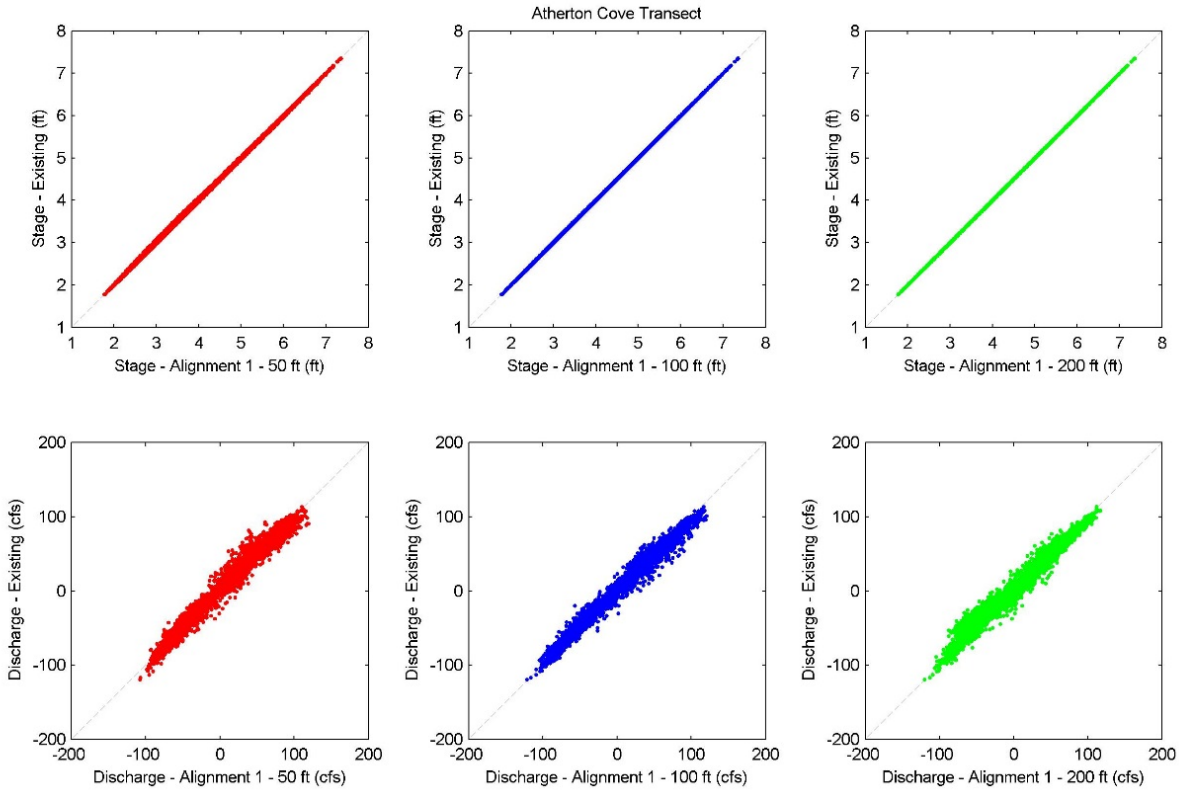


Figure 5-9: Stage (Top) and Discharge (Bottom) Correlation between Existing and Three Gate Widths of Alignment 1 for Atherton Cove

5.2 VELOCITY RESULTS AT PROPOSED GATE

This section summarizes modeling results of tidal velocity through the proposed gates. The maximum tidal velocity through the gate under the proposed project condition and that at the same location under the existing conditions are summarized in Table 5-5. Figure 5-10 through Figure 5-13 show velocity time series during the spring tidal condition. The spring tide occurs twice a month, approximately at the full and new moon when the difference between high and low tide is greatest. The results indicate the flood tidal velocity is generally higher than that of the ebb tidal velocity.

- For Alignment 1, the peak tidal velocity is 2.4 fps with a 50-ft gate, 1.3 fps with a 100-ft gate and 0.54 fps with a 200-ft gate compared to existing velocity of 0.2 fps at the same section location. The velocity decreases with increase in the gate width.

- For Alignment 2, the peak tidal velocity is 1.1 fps at the gate in Smith Canal compared to the existing conditions of 0.53 fps. The peak velocity is lower than that under Alignment 1 as the tidal prism through the Alignment 2 gate is smaller than that through the Alignment 1 gate.

- For Alignment 3, the peak tidal velocity increases from 0.89 fps to 1.2 fps for the gate in Smith Canal, and from 0.2 fps to 0.6 fps for the gate in Atherton Cove. The velocity at the Smith Canal gate is higher than that under Alignment 2 as the tidal prism is slighter larger for Alignment 3.

- For the existing conditions, the velocity in Smith Canal at the gate location of Alignment 3 is higher than that of Alignment 2 as the channel is narrower and tidal prism is larger at the Alignment 3 gate location.

Table 5-5: Comparison of Maximum Tidal Velocity at Gate

Alternative	Gate Location and Dimension	At Gate Location under Existing Condition (ft/s)	At Gate Location under Proposed Condition (ft/s)
Alignment 1	Smith Canal – 50 ft	0.20	2.38
Alignment 1	Smith Canal – 100 ft	0.20	1.26
Alignment 1	Smith Canal – 200 ft	0.20	0.54
Alignment 2	Smith Canal – 50 ft	0.53	1.11
Alignment 3	Smith Canal – 50 ft	0.89	1.20
Alignment 3	Atherton Cove – 50 ft	0.18	0.60

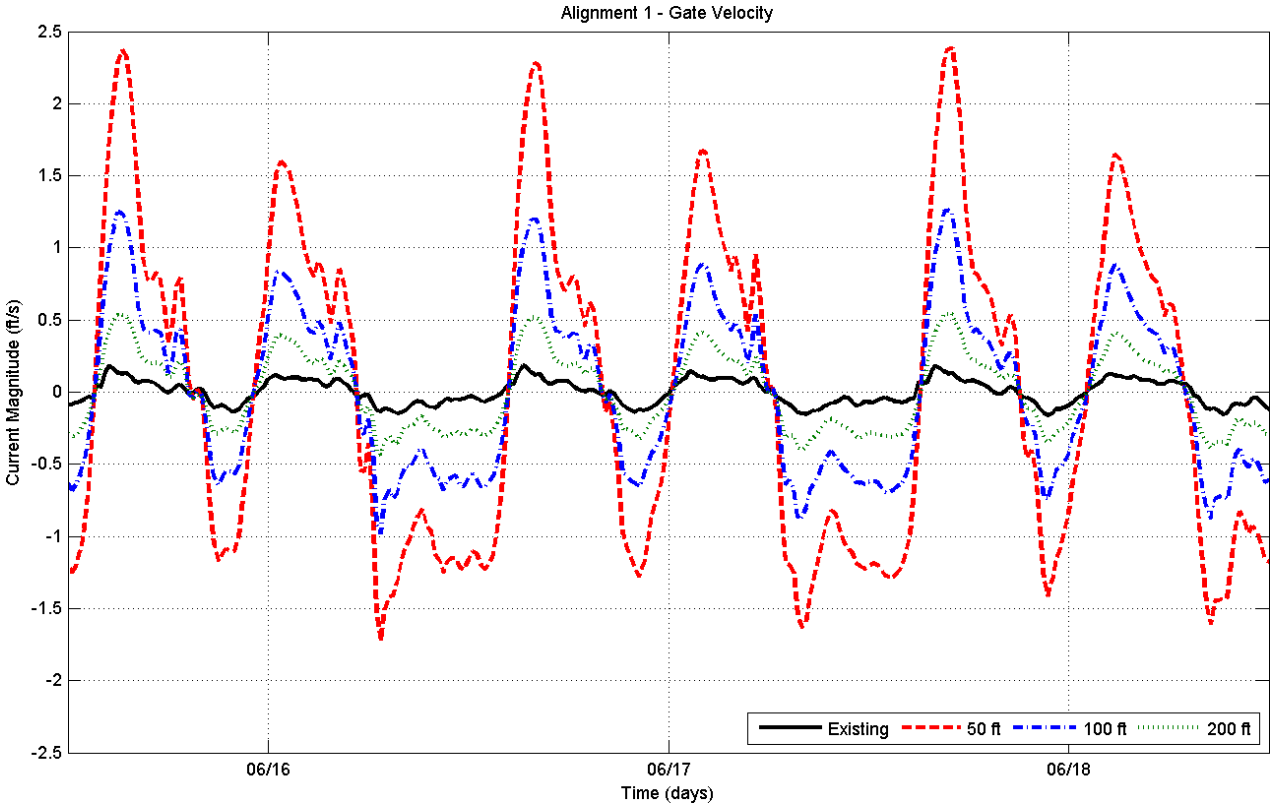


Figure 5-10: Velocity at Gate of Alignment 1

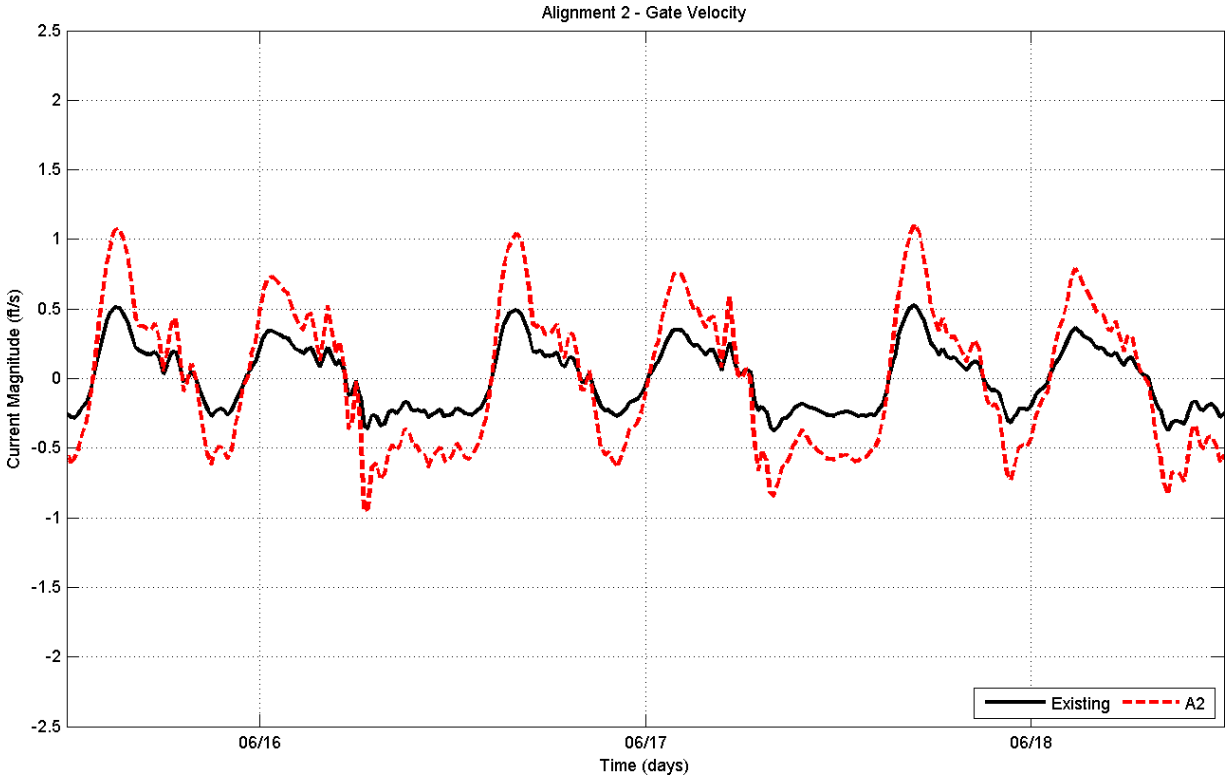


Figure 5-11: Velocity at Gate of Alignment 2

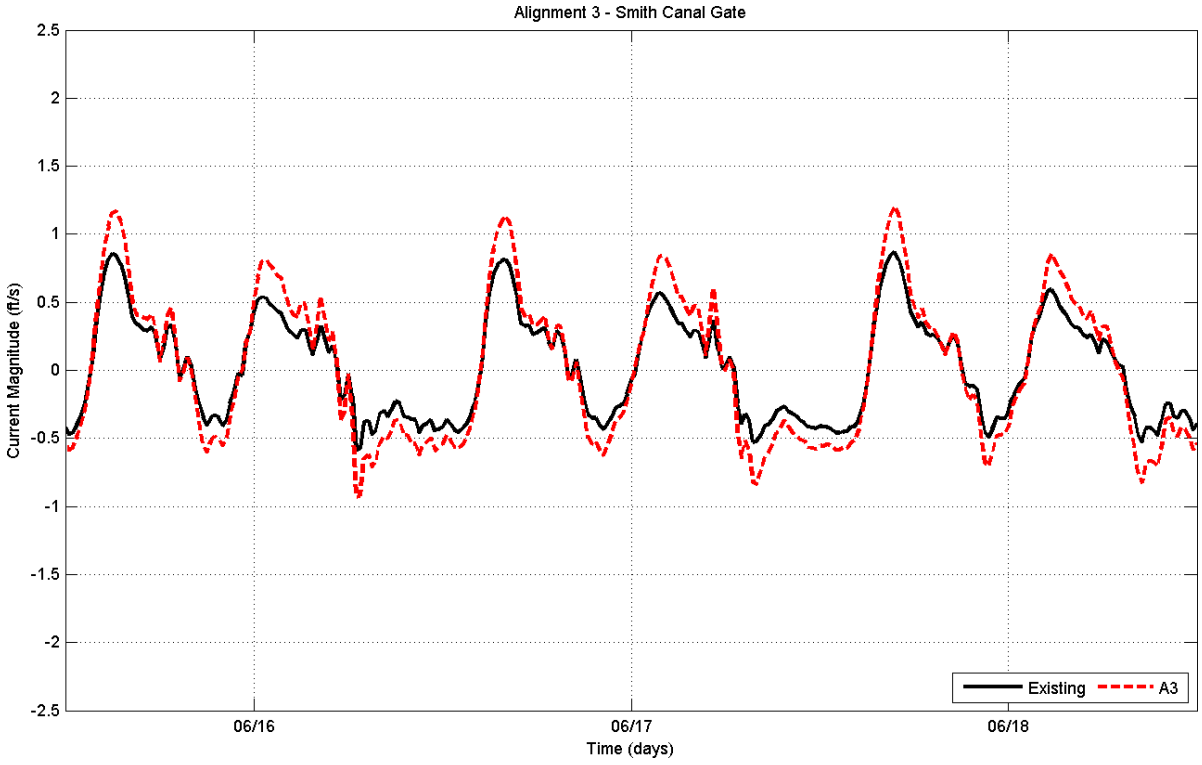


Figure 5-12: Velocity at Gate on Smith Canal of Alignment 3

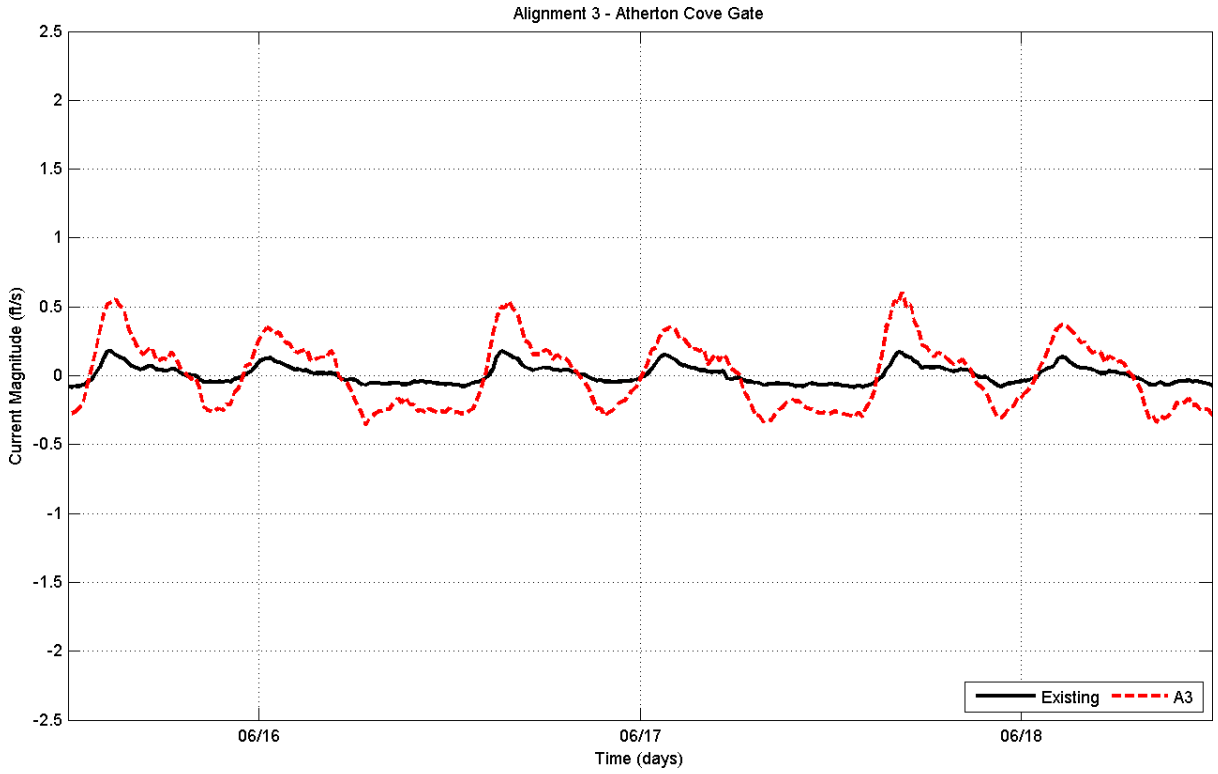


Figure 5-13: Velocity at Gate on Atherton Cove of Alignment 3

Ocean tides change daily and follow a semi-diurnal pattern in California. The semi-diurnal range varies in a two-week cycle. Approximately twice a month, around new moon and full moon when the Sun, Moon, and Earth form a line, the tidal force due to the sun reinforces that due to the Moon. The tide's range is then at its maximum, called the spring tide. When the Moon is at first quarter or third quarter, the sun and Moon are separated by 90° when viewed from the Earth, and the solar tidal force partially cancels the Moon's. At these points in the lunar cycle, the tide's range is at its minimum, called the neap tide. Table 5-5 summarizes the spring tide maximum velocity through proposed tidal gate and velocity varies over time, as shown in Figure 5-10 through Figure 5-13. Therefore, statistical exceedance analyses of current speeds are performed based on two spring and neap tidal cycles and results are summarized in Table 5-6. The results indicate that the current speed through the 50-ft gate of Alignment 1 would exceed 2.0 fps for 2.7% of the time and 1.5 fps for about 10% of the time. For the 100-ft gate of Alignment 1, the current speed would exceed 1.0 fps for 3.7% of the time. Figure 5-14 through Figure 5-17 show exceedance curves of current speed through the gate.

Table 5-6: Current Speed Exceedance Percentages

Current Speed (ft/s)	Alignment 1			Alignment 2	Alignment 3	
	50 ft	100 ft	200 ft		Smith Canal	Atherton Cove
1.0	44.0%	3.7%	0.0%	0.9%	2.4%	0.0%
1.5	10.2%	0.0%	0.0%	0.0%	0.0%	0.0%
2.0	2.7%	0.0%	0.0%	0.0%	0.0%	0.0%
2.5	0.0%	0.0%	0.0%	0.0%	0.0%	0.0%

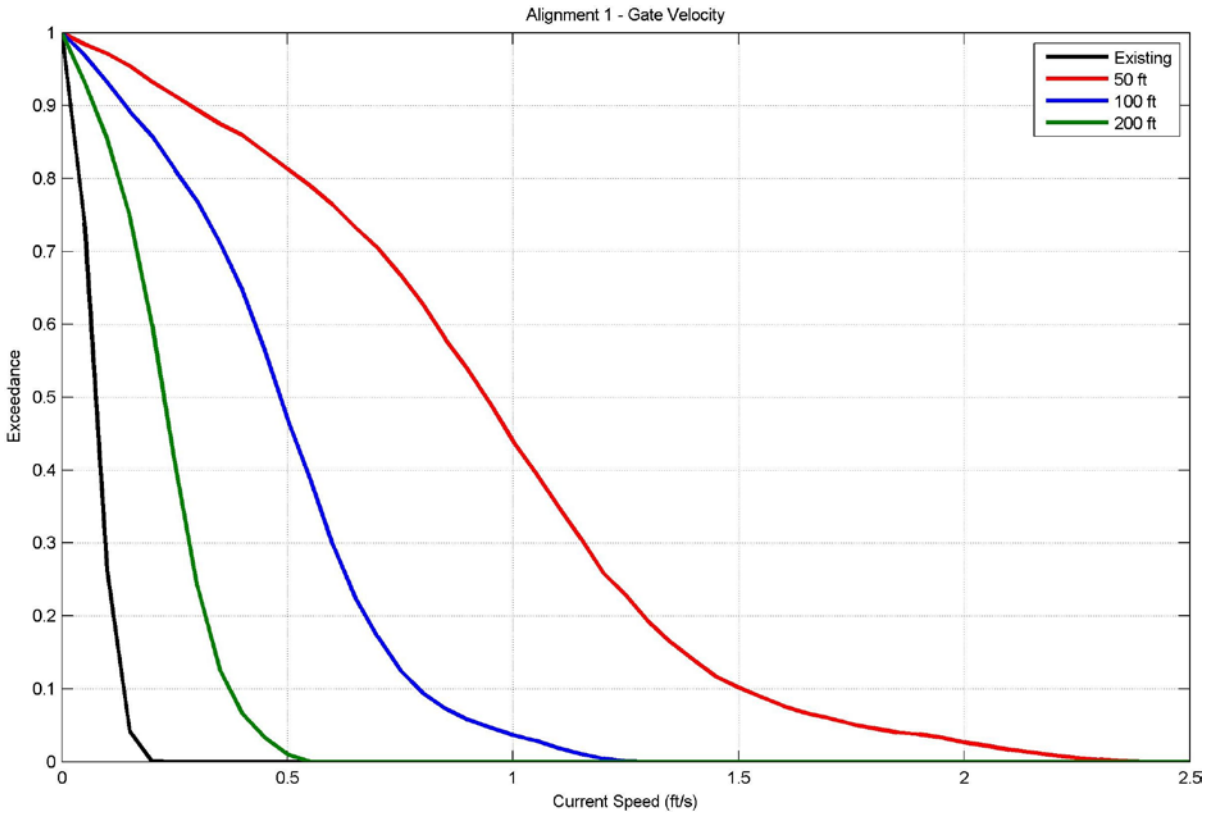


Figure 5-14: Current Speed Exceedance Curves at Gate of Alignment 1

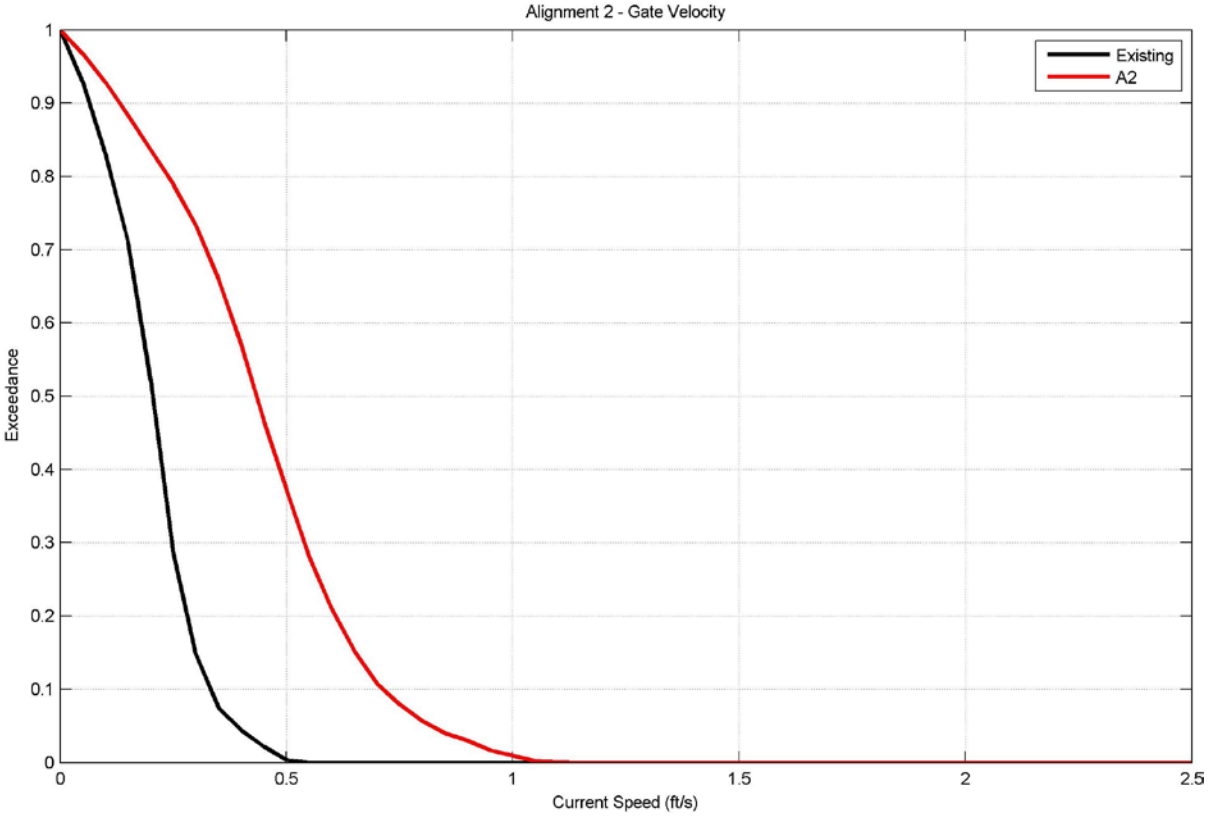


Figure 5-15: Current Speed Exceedance Curve at Gate of Alignment 2

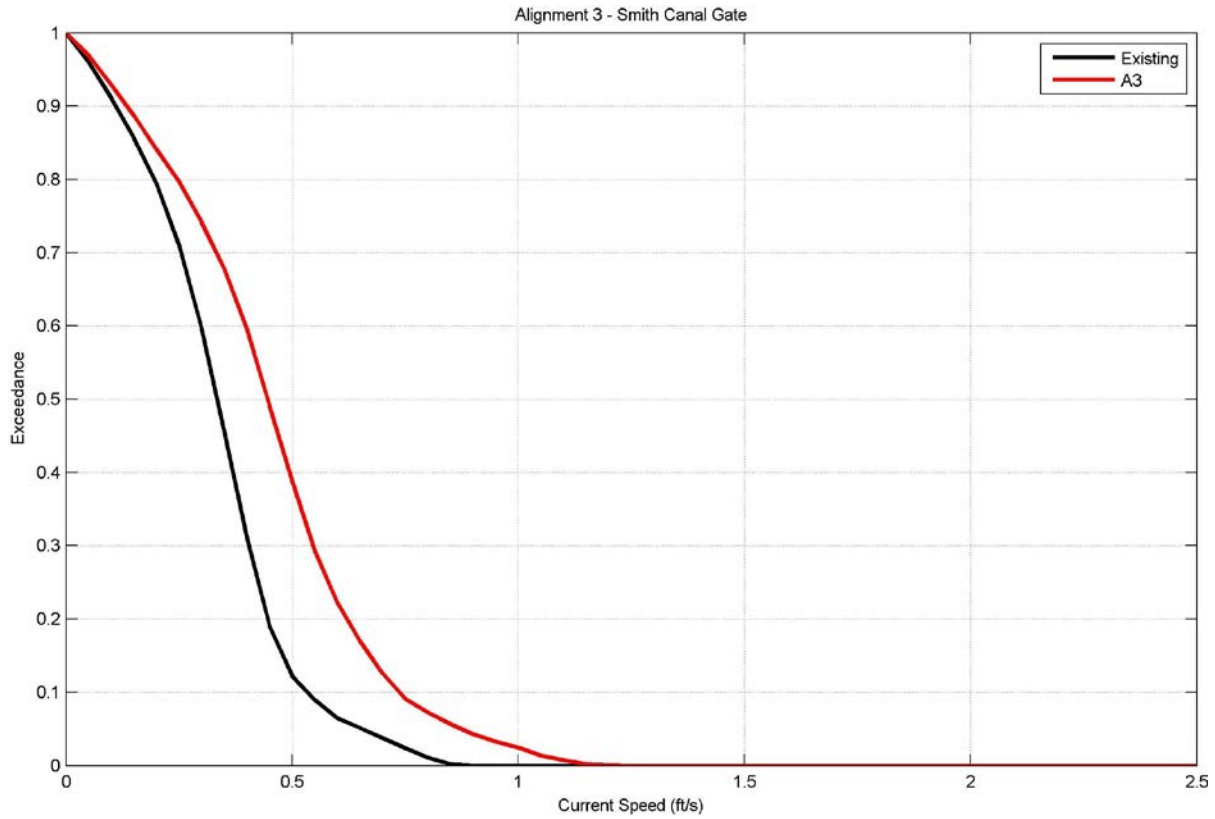


Figure 5-16: Current Speed Exceedance Curve at Gate on Smith Canal of Alignment 3

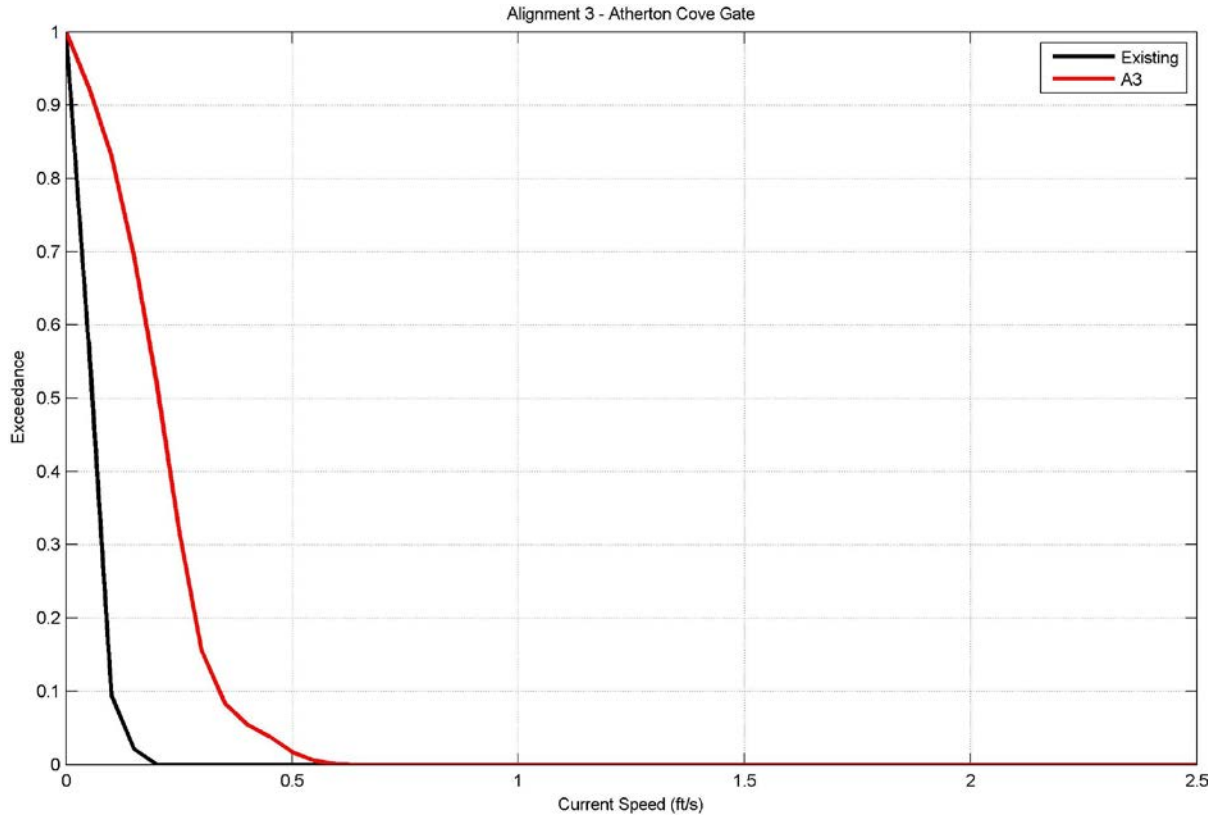


Figure 5-17: Current Speed Exceedance Curve at Gate on Atherton Cove of Alignment 3

5.3 VELOCITY PATTERN AND ANALYSES OF SEDIMENTATION POTENTIAL

To evaluate the sedimentation and erosion potential for each of the Alternative Alignments, the flow field output from the numerical model was used as a proxy. The maximum spring tide cycle (or tidal day) was first determined, and then the time stamp of both peak ebb and flood flow fields was determined for each Alternative modeled. The selected spring tide peak ebb and flood flow fields for each of the proposed alignments were compared against those of the existing conditions. The alternative peak ebb and flood velocity fields were transformed onto a common grid and subtracted by those under the existing conditions to show changes in velocity spatial distribution. For each Alignment, peak flood velocity distribution, peak ebb velocity distribution, and changes in velocity are compared and plotted. The mobilization velocity for sand sized material is 0.6 fps; therefore, the local scour may occur if the velocity exceeds 0.6 fps. On the other hand, if the current velocity drops to below 0.15 fps, the sediment in the water column may settle down.

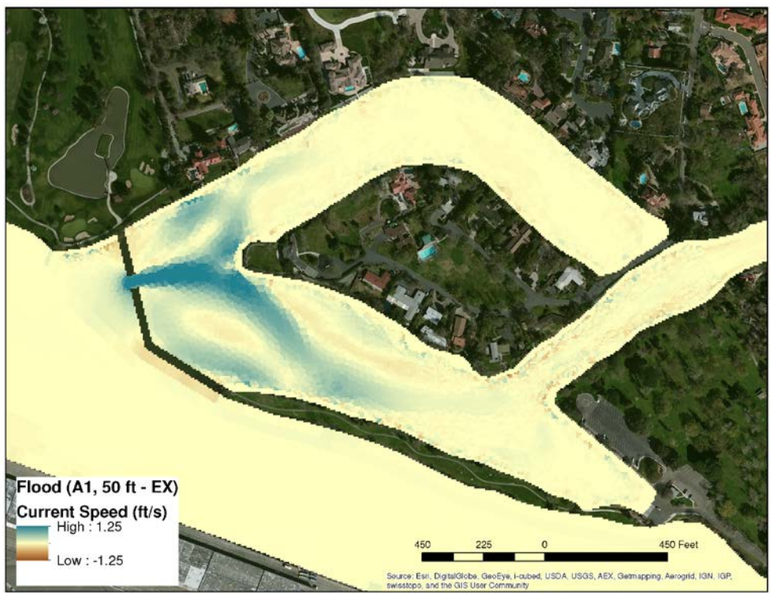
5.3.1 Alignments Comparison

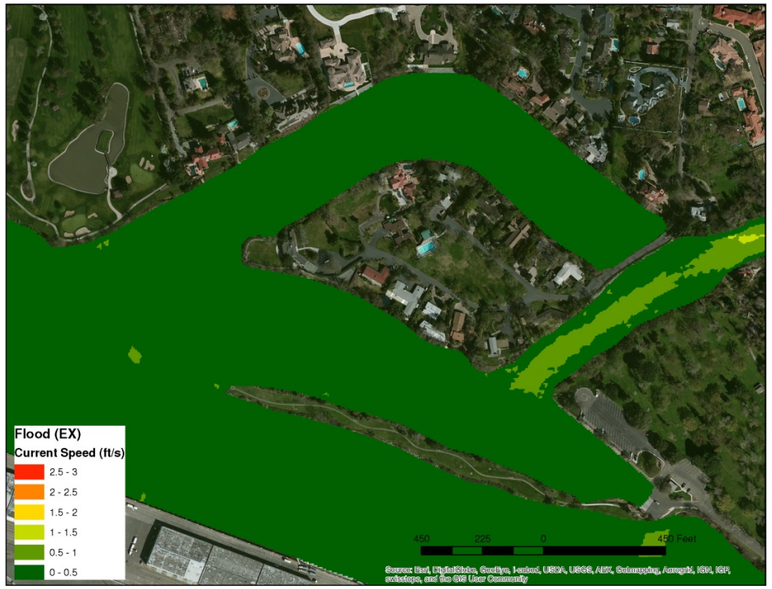
The spring tide peak ebb and flood velocity of each proposed alignment are subtracted by the corresponding peak ebb and flood velocity under the existing conditions, and the resulting difference in velocity is shown in Figure 5-18, and Figure 5-21 for Alignment 1, Figure 5-22 and Figure 5-23 for Alignment 2, and Figure 5-24 and Figure 5-25 for Alignment 3. Figure 5-19 and Figure 5-20 are velocity vector plots for Alignment 1 with a 50-ft gate. The velocity patterns are similar for 100-ft and 200-ft gates of Alignment 1, but the eddies are weaker.

Alignment 1 during the flood tide:

- Velocity: there is an increase in the flow velocity in the vicinity of the gate structure (on the order of 2.0 fps at the peak). On the flood tide, the gate creates a localized counter-clockwise eddy in Atherton Cove entrance and a clockwise eddy in Smith Canal, as shown in Figure 5-19, which promotes the localized circulation. However, the jet flow resulting from the gate also increases velocity at the west corner of Atherton Island.
- Sedimentation: The flood flow may result in a flood shoal at the entrance to Smith Canal near Dad's End and near the north shore. Sedimentation is also expected on both sides of the north floodwall away from the gate. No sedimentation change is expected in areas away from the proposed structure.
- Impacts to SDWSC: No hydraulic impacts are expected from the proposed project to the SDWSC.

ALIGNMENT 1 (50 FT GATE) - EXISTING





EXISTING



ALIGNMENT 1 (50 FT GATE)

Figure 5-18: Existing and Alignment 1 (50 ft Gate) Flood Tide Velocity Difference

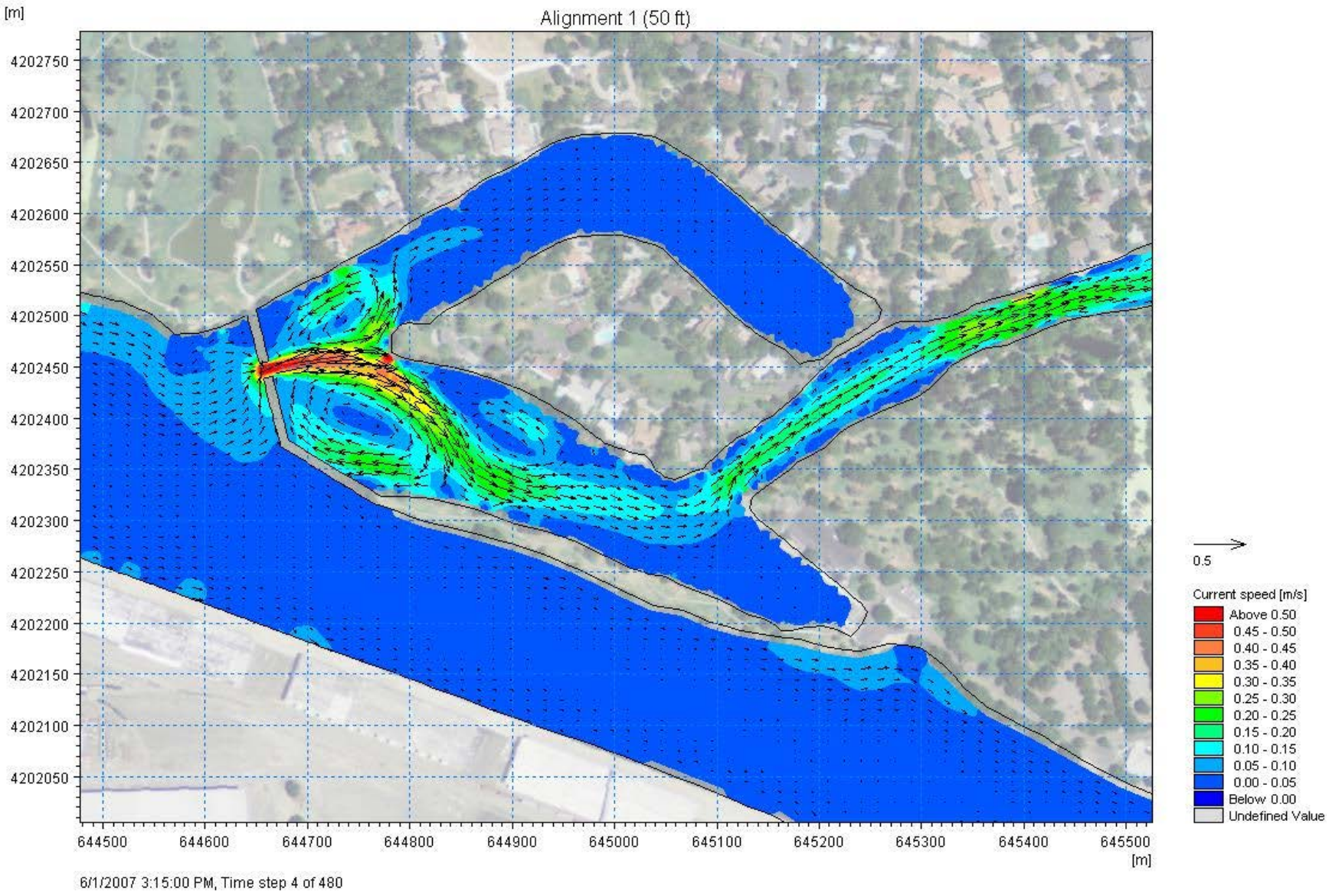


Figure 5-19: Alignment 1-50 ft Gate Velocity Vector Plot during Flood Tide

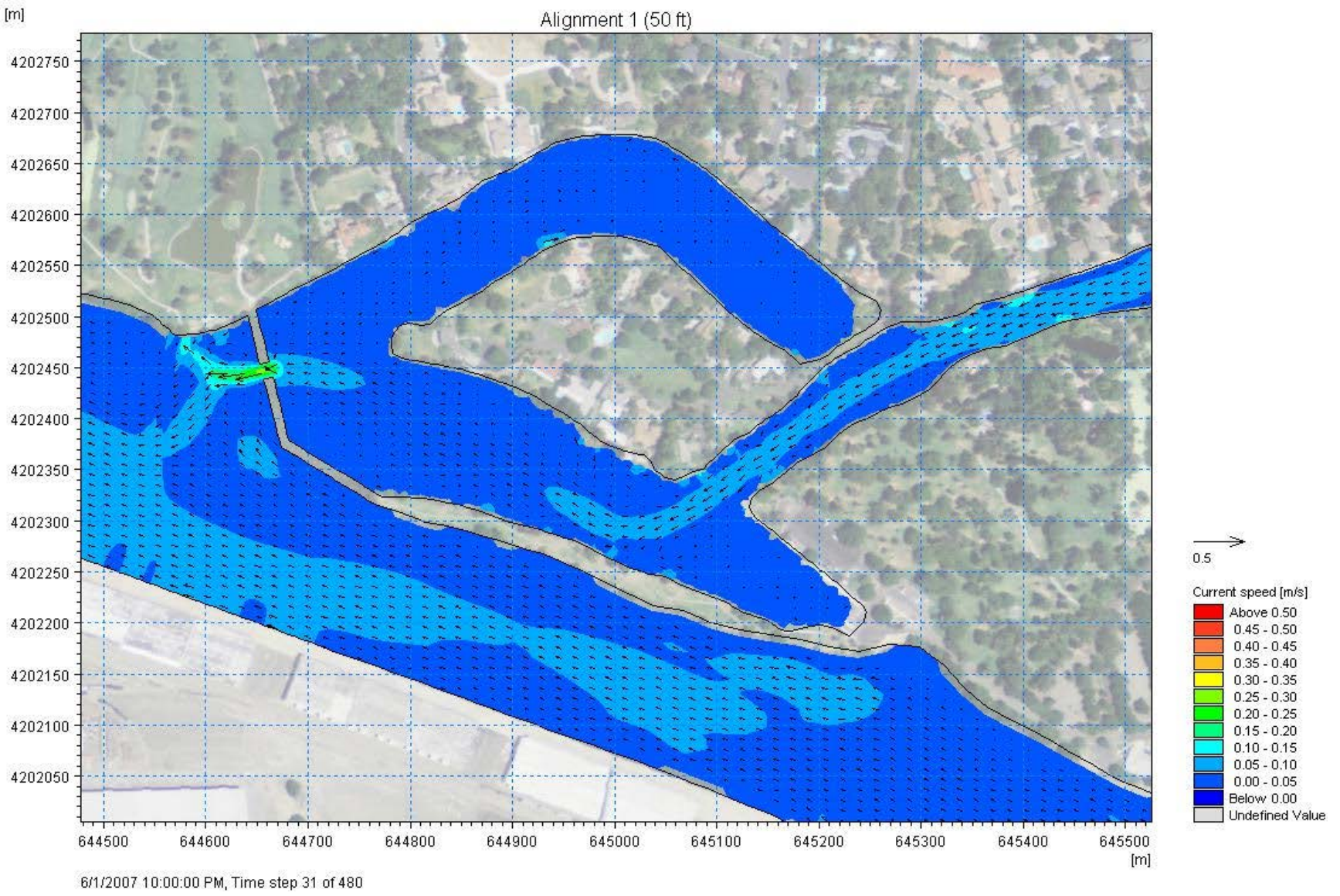
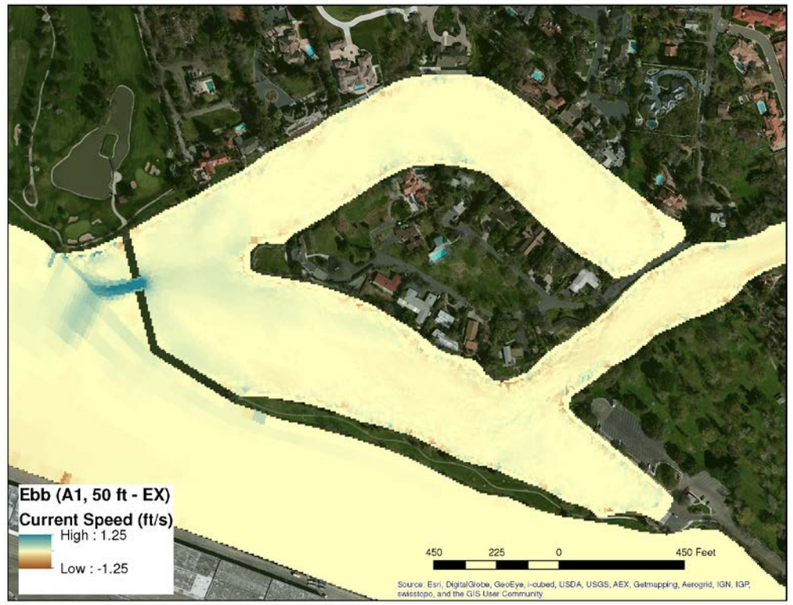


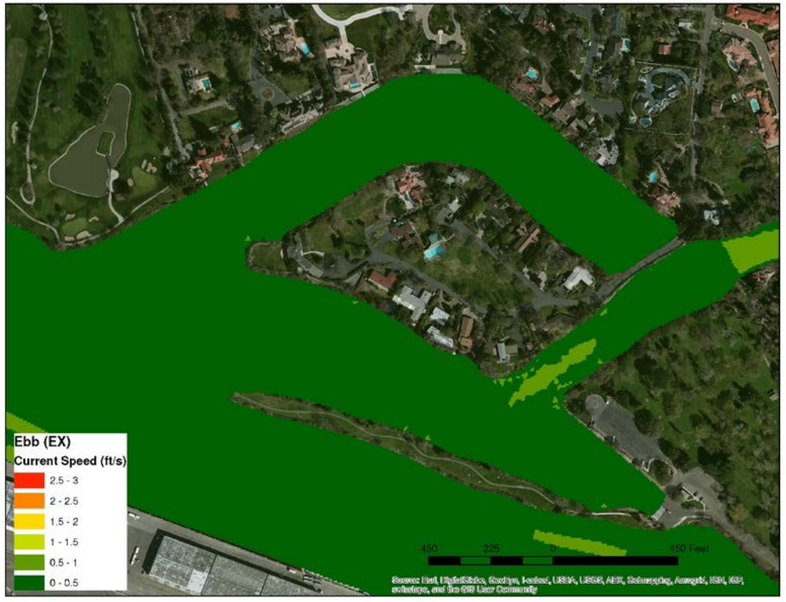
Figure 5-20: Alignment 1-50 ft Gate Velocity Vector Plot during Flood Tide

Alignment 1 during the ebb tide:

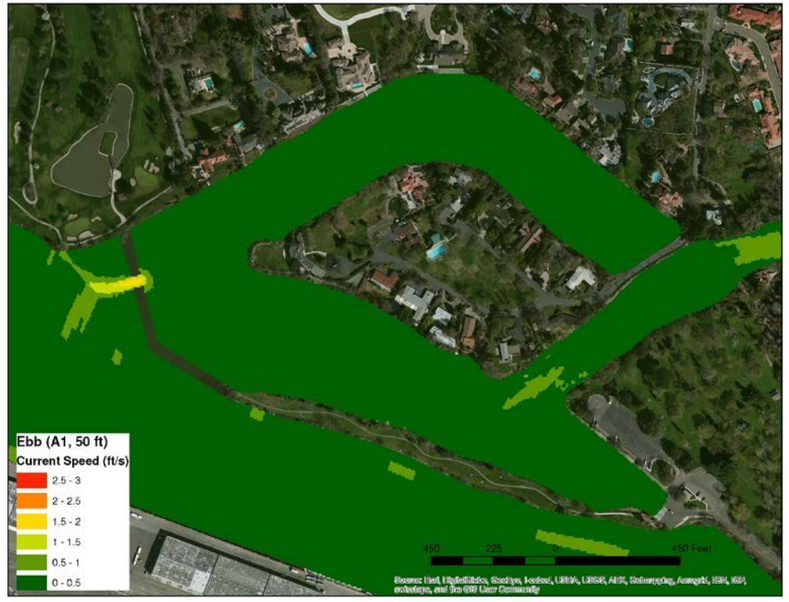
- Velocity: The velocity increase through the gate structure is not as significant as that during the flood tide (on the order of 1 fps at the peak), and it is much more localized around the gate structure. Away from the structure, the velocity pattern and magnitude largely remains unchanged.

ALIGNMENT 1 (50 FT GATE) - EXISTING





EXISTING



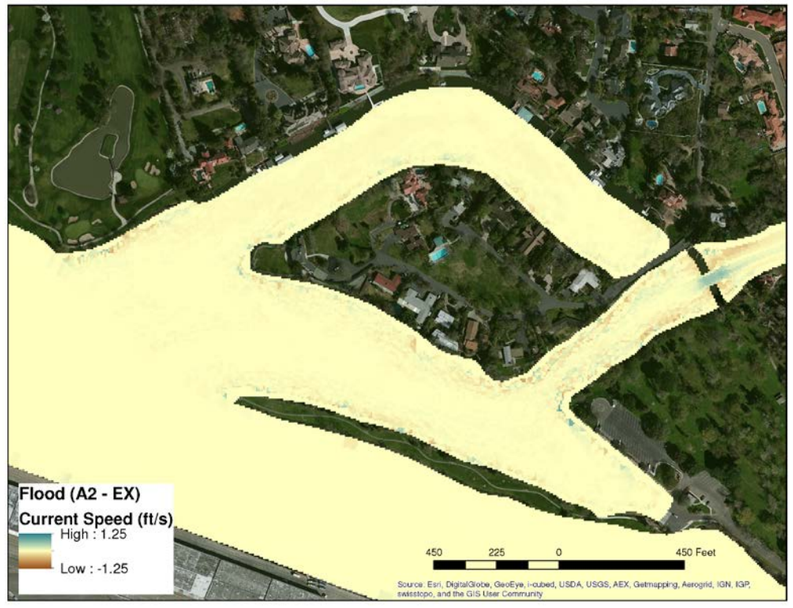
ALIGNMENT 1 (50 FT GATE)

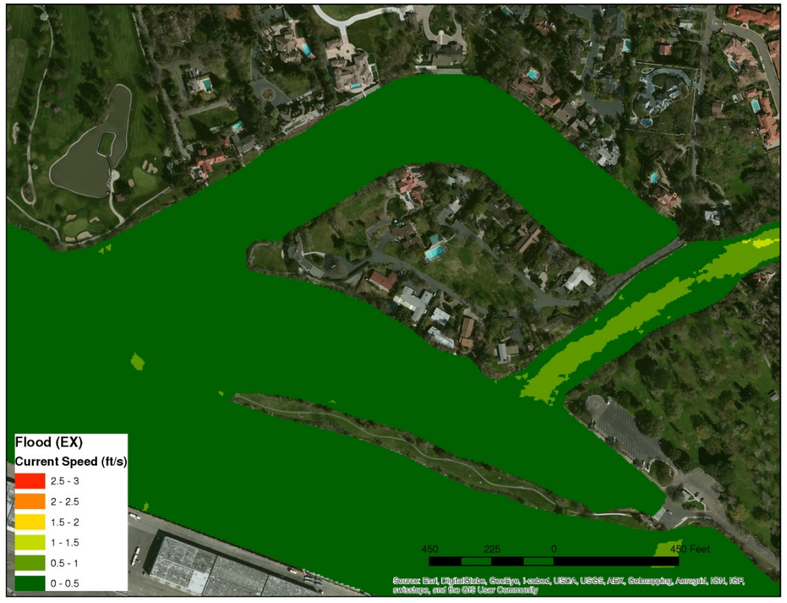
Figure 5-21: Existing and Alignment 1 (50 ft Gate) Ebb Tide Velocity Difference

Alignment 2 during the flood tide:

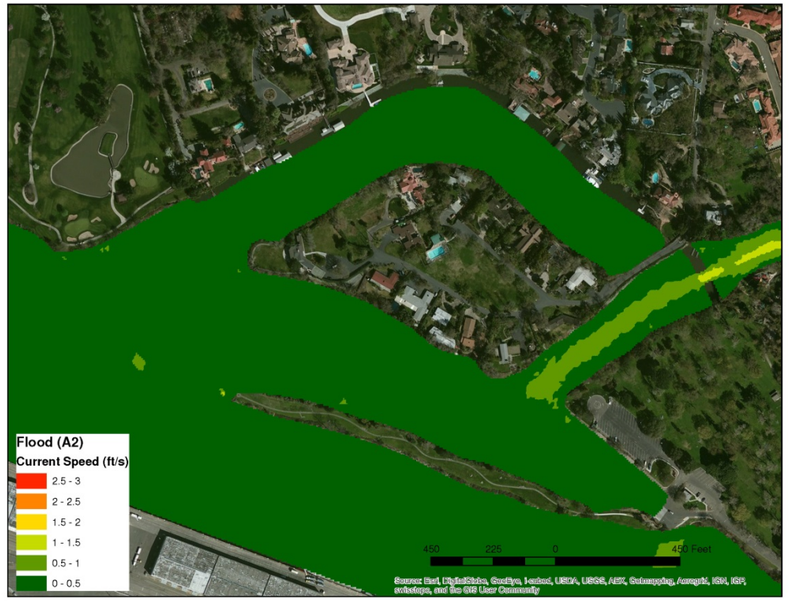
- Velocity: Sheet pile wall within Atherton Cove and the gate structure within Smith Canal does not significantly change the hydrodynamics. Locally around the gate structure, there is an increase in the flow velocity on both ebb and flood tides, but this effect is largely in the middle of the canal and decreases with increase in distance from the gate.
- Sedimentation: The scour potential is largely located in the middle of the canal adjacent to the gate structure. At the gate section, potential aggradation along the shore is expected on both sides of the gate away from the center of the gate opening.

ALIGNMENT 2 - EXISTING





EXISTING



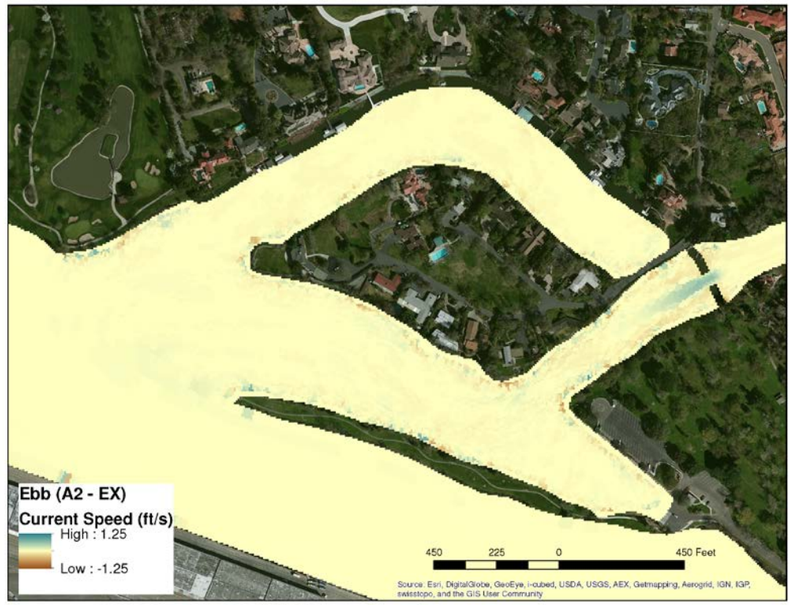
ALIGNMENT 2

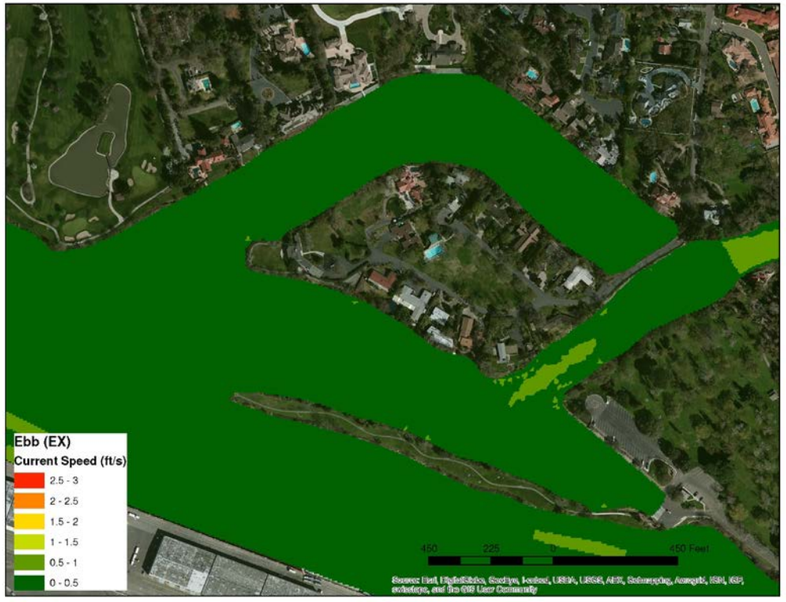
Figure 5-22: Existing and Alignment 2 Flood Tide Velocity Difference

Alignment 2 during the ebb tide:

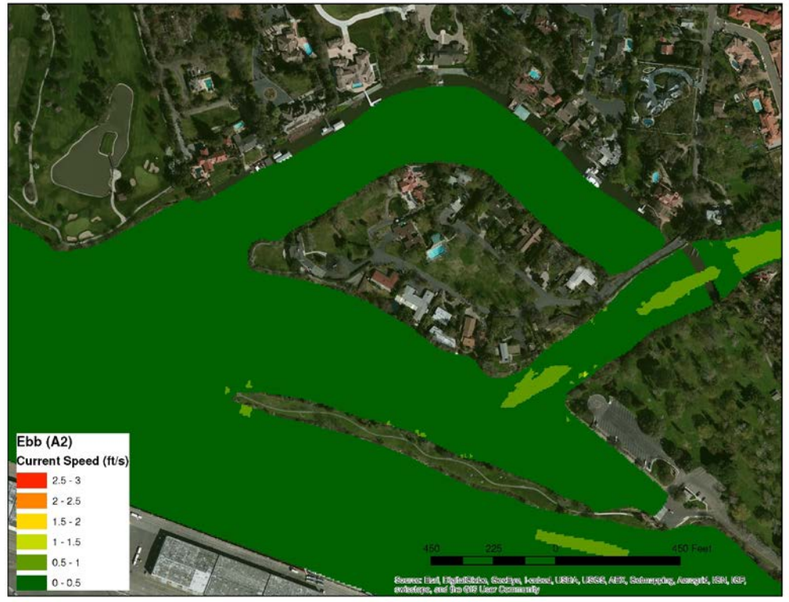
- Velocity: The velocity increase through the gate structure is not as significant as that during the flood tide (on the order of 0.5 fps at the peak), and it is much more localized around the gate structure. Away from the structure, the velocity pattern and magnitude largely remains unchanged.

ALIGNMENT 2 - EXISTING





EXISTING



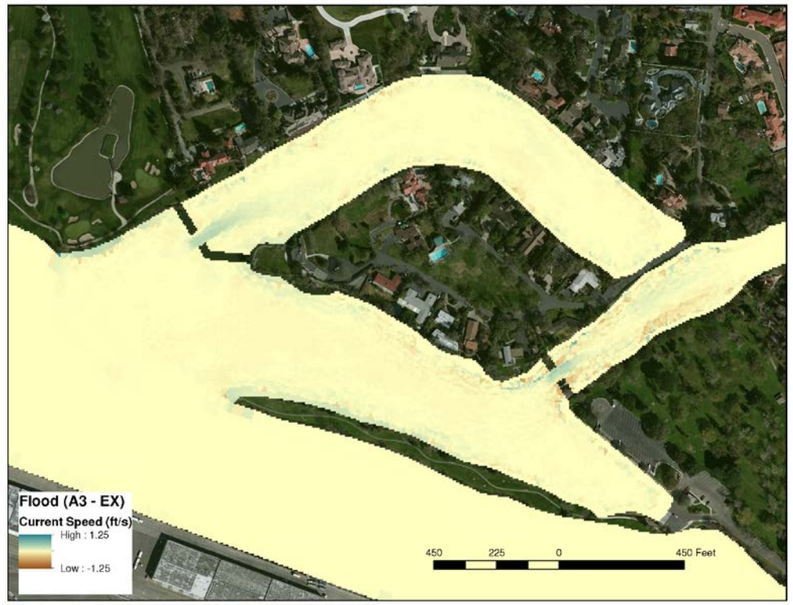
ALIGNMENT 2

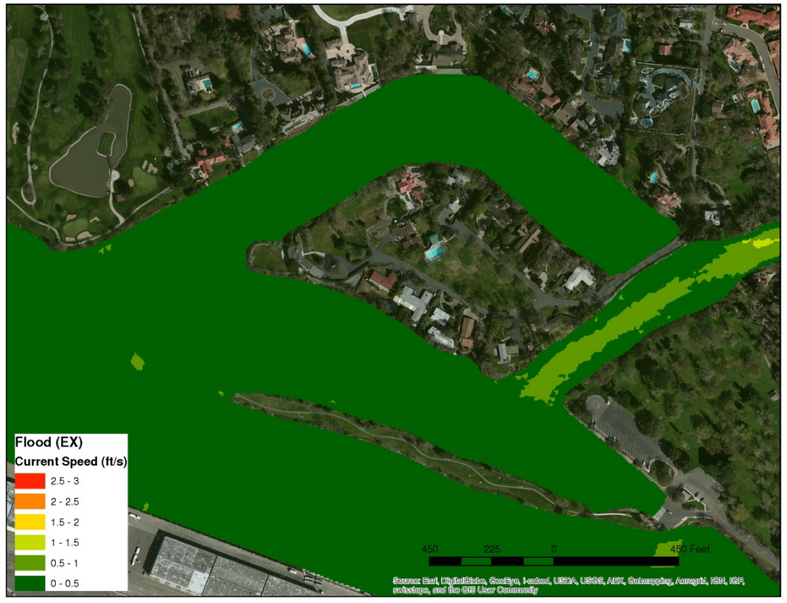
Figure 5-23: Existing and Alignment 2 Ebb Tide Velocity Difference

Alignment 3 during the flood tide:

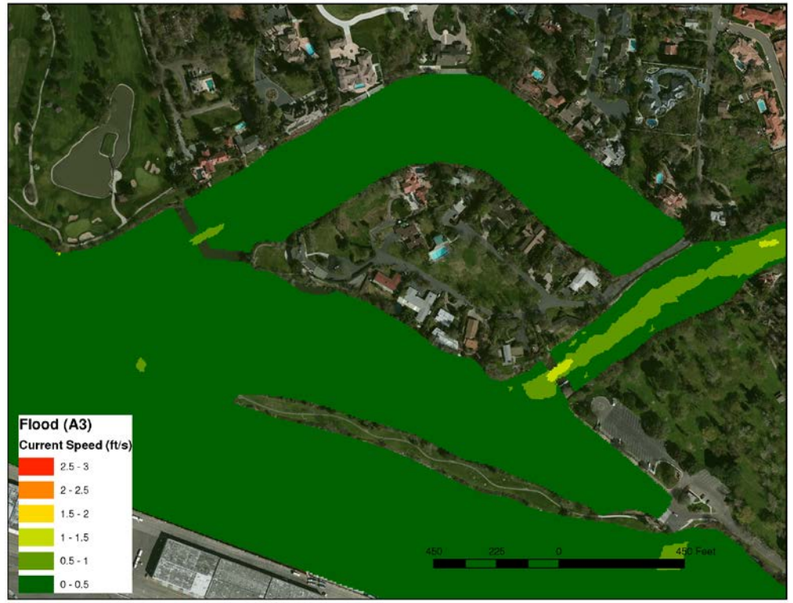
- Velocity: the velocity change is much more localized around the two gates, and the velocity increase is relatively small, less than 0.5 fps at both gates.
- Sedimentation: The scour potential is largely located in the middle of the canal adjacent to the gate structures. The potential erosion hot spots are along the center of the channel near the two gates. At the gate section, potential aggradation along the shore is expected on both sides of the floodwall away from the center of the gate opening.

ALIGNMENT 3 - EXISTING





EXISTING



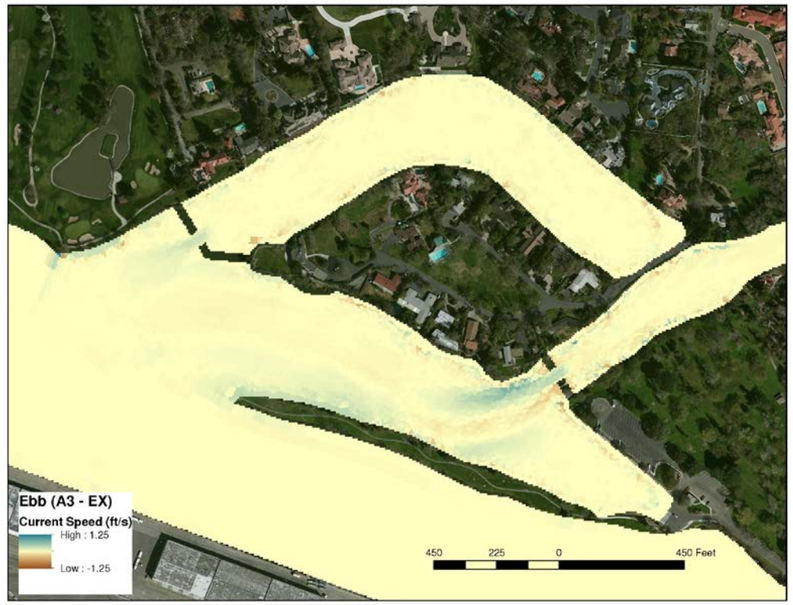
ALIGNMENT 3

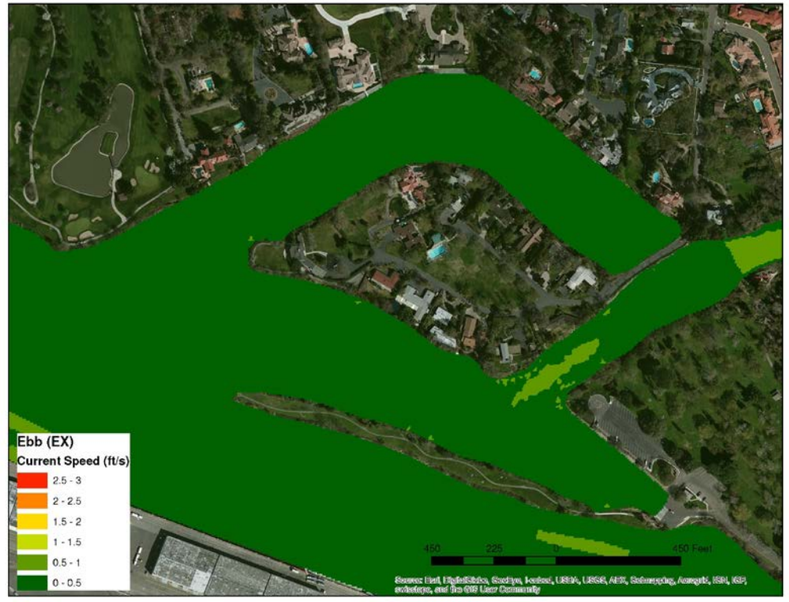
Figure 5-24: Existing and Alignment 3 Flood Tide Velocity Difference

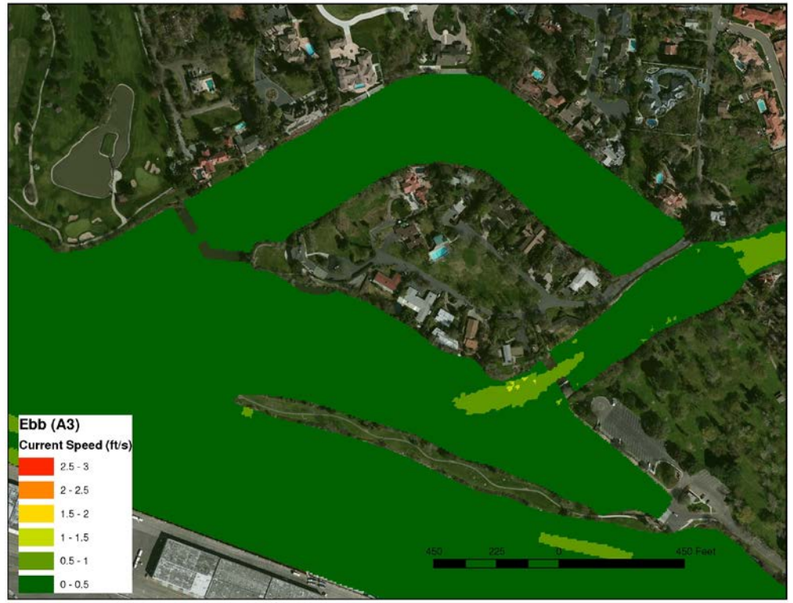
Alignment 3 during the ebb tide:

- Velocity: The velocity increase through the gate structure is slightly less than that during the flood tide (on the order of 0.5 fps at the peak), and it is much more localized around the gate structures. Away from the structure, the velocity pattern and magnitude largely remains unchanged.

ALIGNMENT 3 - EXISTING







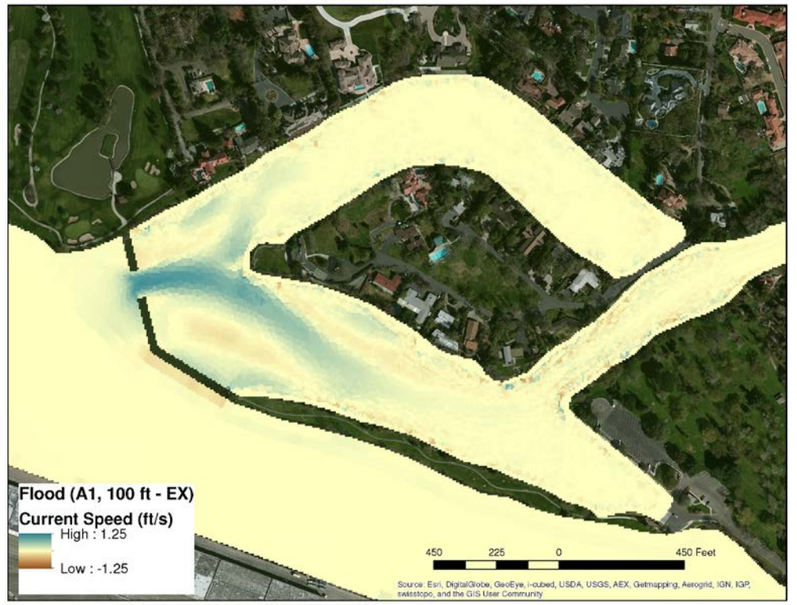
EXISTING ALIGNMENT 3

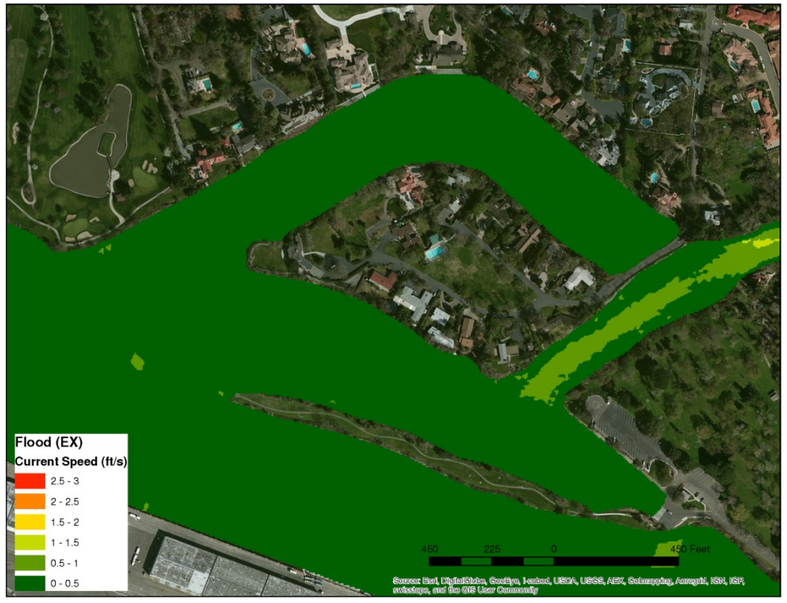
Figure 5-25: Existing and Alignment 3 Ebb Tide Velocity Difference

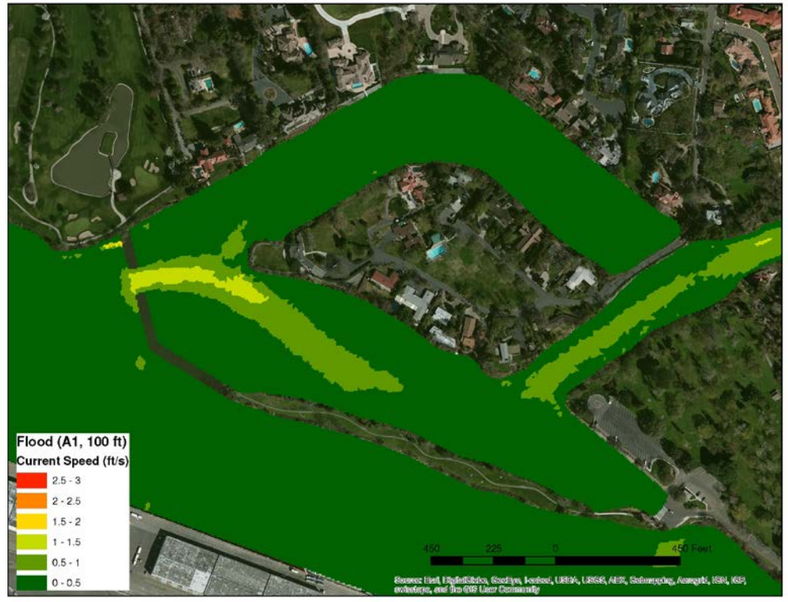
5.3.2 Alignment 1 Gate Width Sensitivity

The spring tide peak ebb and flood velocity of each proposed gate width are subtracted by the corresponding peak ebb and flood velocity under the existing conditions. Figure 5-18, Figure 5-21, and Figure 5-26 through Figure 5-29 show comparisons of peak tidal flow velocity for Alignment 1 with 50-ft, 100-ft, and 200-ft gate widths. The comparisons indicate that between different gate widths, the pattern of the flows with increased gate width remains largely unchanged. Rather, it is the changes to the flow velocity that decreases linearly with increased gate width. The velocity through the gate decreases to nearly half when the gate width doubles from 50-ft to 100-ft. On the flood tide, the gate creates a localized counter-clockwise eddy at the entrance to Atherton Cove and a clockwise eddy in Smith Canal as shown in Figure 5-19. The gate slightly increases the discharge into Atherton Cove, while slightly decreasing the discharge into Smith Canal. The extent of the flow redistribution reduces with increase in gate width. For the 200-ft wide gate, the flood shoal is expected to be along the inside of the south floodwall.

ALIGNMENT 1 (100 FT GATE) - EXISTING





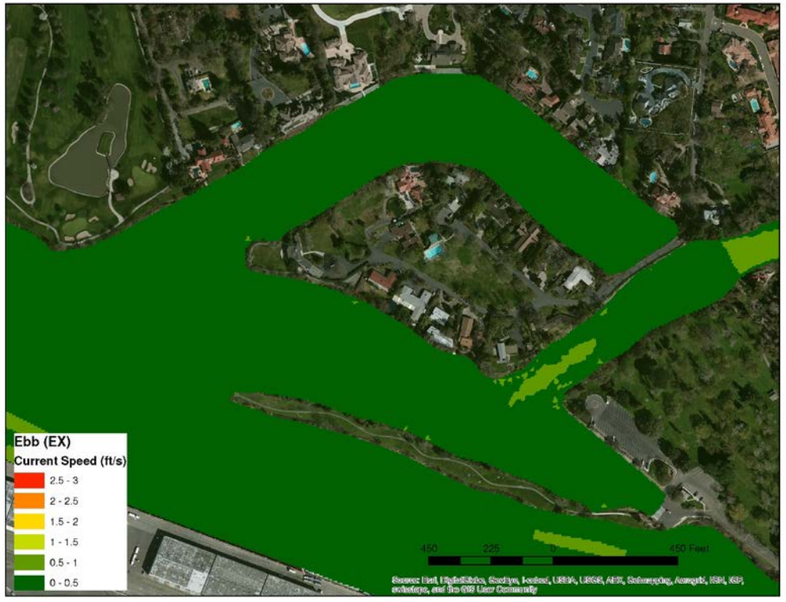


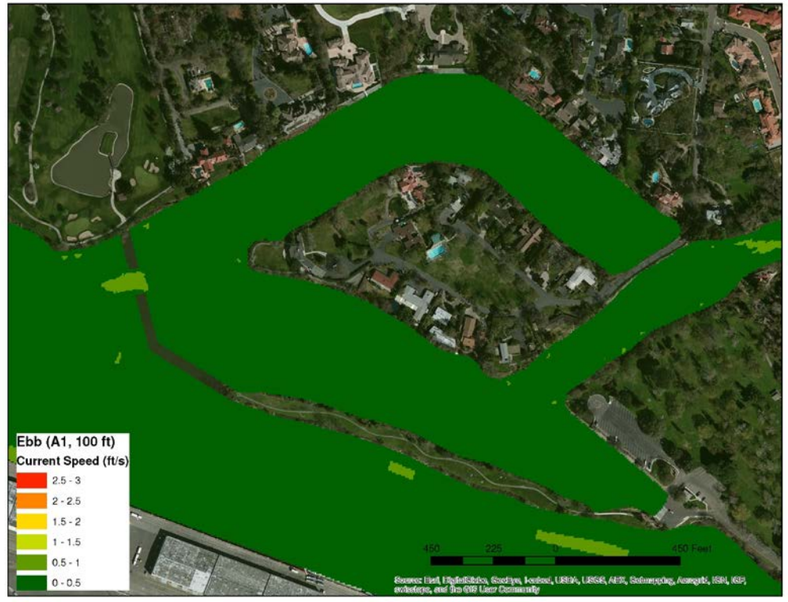
EXISTING ALIGNMENT 1 (100 FT GATE)

Figure 5-26: Existing and Alignment 1 (100 ft Gate) Flood Tide Velocity Difference

ALIGNMENT 1 (100 FT GATE) - EXISTING



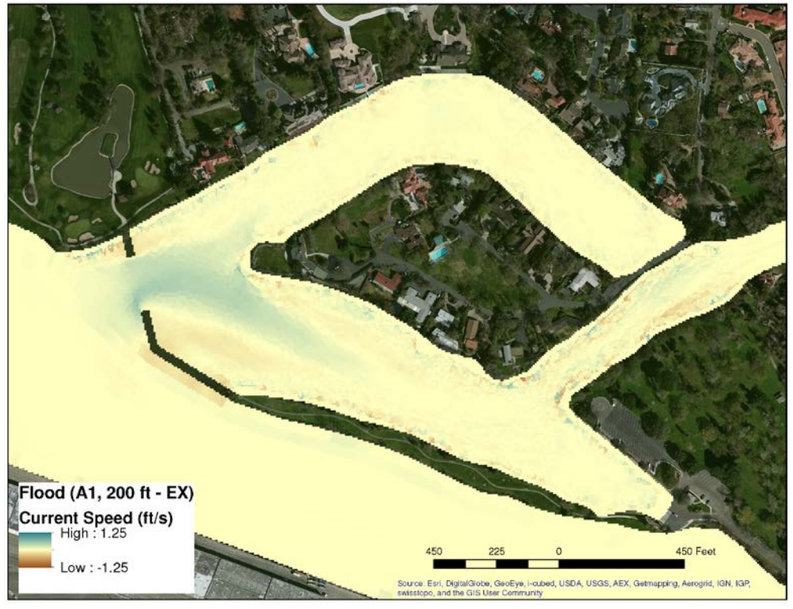


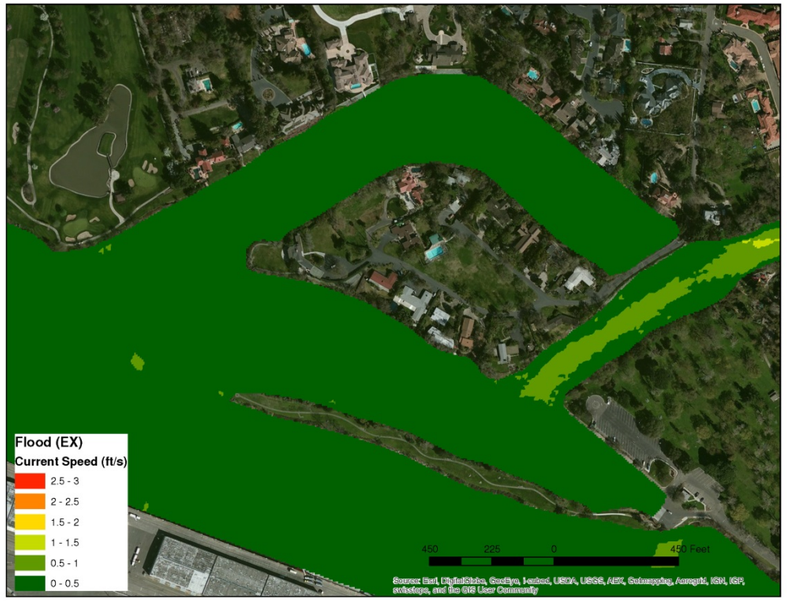


EXISTING ALIGNMENT 1 (100 FT GATE)

Figure 5-27: Existing and Alignment 1 (100 ft Gate) Ebb Tide Velocity Difference

ALIGNMENT 1 (200 FT GATE) - EXISTING





EXISTING

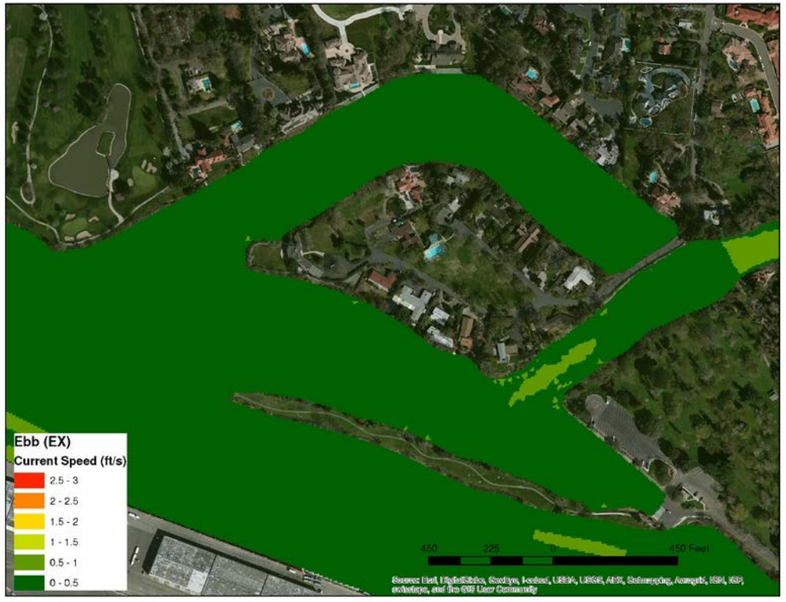


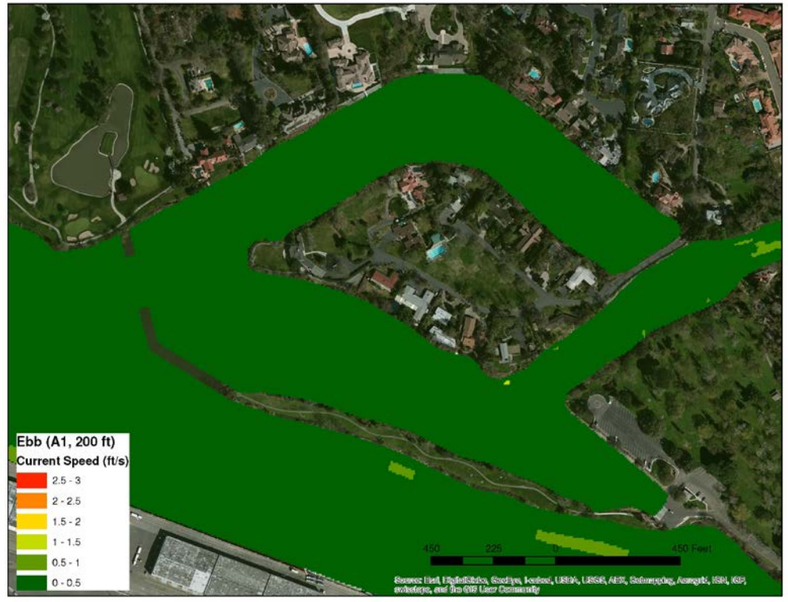
ALIGNMENT 1 (200 FT GATE)

Figure 5-28: Existing and Alignment 1 (200 ft Gate) Flood Tide Velocity Difference

ALIGNMENT 1 (200 FT GATE) - EXISTING







EXISTING ALIGNMENT 1 (200 FT GATE)

Figure 5-29: Existing and Alignment 1 (200 ft Gate) Ebb Tide Velocity Difference

5.3.3 Siltation Rates

5.3.3.1 Historical Siltation Rates

Construction of Smith Canal was planned in 1889 for the purpose of discharging storm water and sewage, and as a supply route for barging of goods as part of the development of the City of Stockton. The canal was first identified on survey maps from 1894 and the exact completion date is not documented.

The earliest available survey data was established in 1934. Since then, the Canal has not been resurveyed and depths indicated on navigation charts to this date reflected the originally surveyed water depths.

Figure 5-30 provides an excerpt of a hydrographic chart from 1948, indicating water depths in Smith Canal ranging from 3 to 7 ft relative to MLLW. Figure 5-31 is an excerpt from the current day NOAA nautical chart reflecting the same water depths as the 1948 chart. The nautical chart for this area has not been updated since 1948.

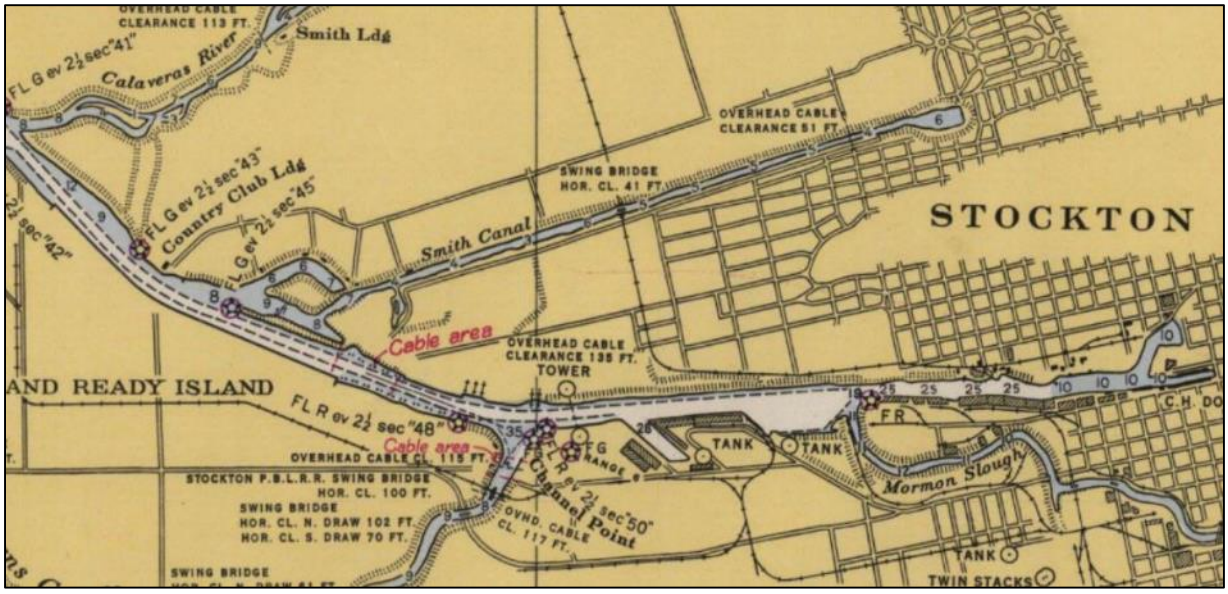


Figure 5-30: Smith Canal, excerpt from Chart No. 5527 – San Joaquin River, U.S. Coast and Geodetic Survey (1948)

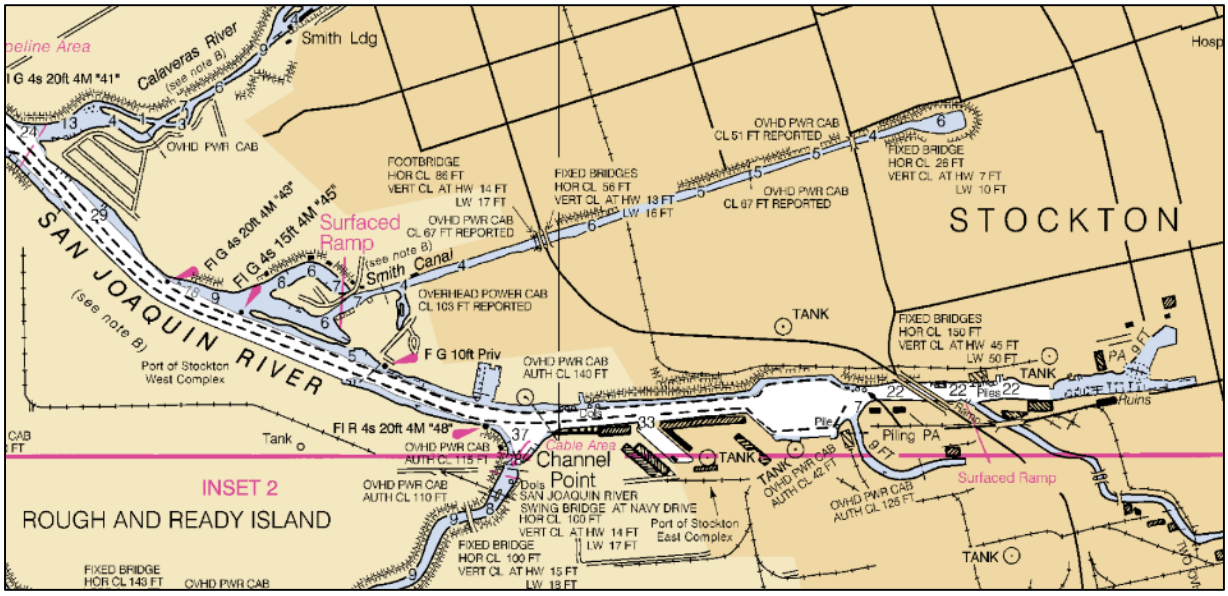


Figure 5-31: Smith Canal, excerpt from Nautical Chart No.18661 – Sacramento and San Joaquin Rivers, NOAA (2009)

Bathymetric surveying provided by KSN for the current project development indicate that water depths in Smith Canal to the east of I-5 are substantially lower than in past times, whereas water depths in the portion of the canal west of I-5 have increased. Water depths in Atherton Cove have also decreased due to siltation. The water depth increase west of I-5 may be a result of channel dredging.

Figure 5-32 shows an example comparison of the historical (red) canal cross-section with recent survey data (blue curve). The section is located approximately midway between the I-5 freeway overpass and Yosemite Lake. The example shows that the originally deeper center portion of the channel has silted up, and also that the embankment side slopes have been built out slightly, so the canal has become wider.

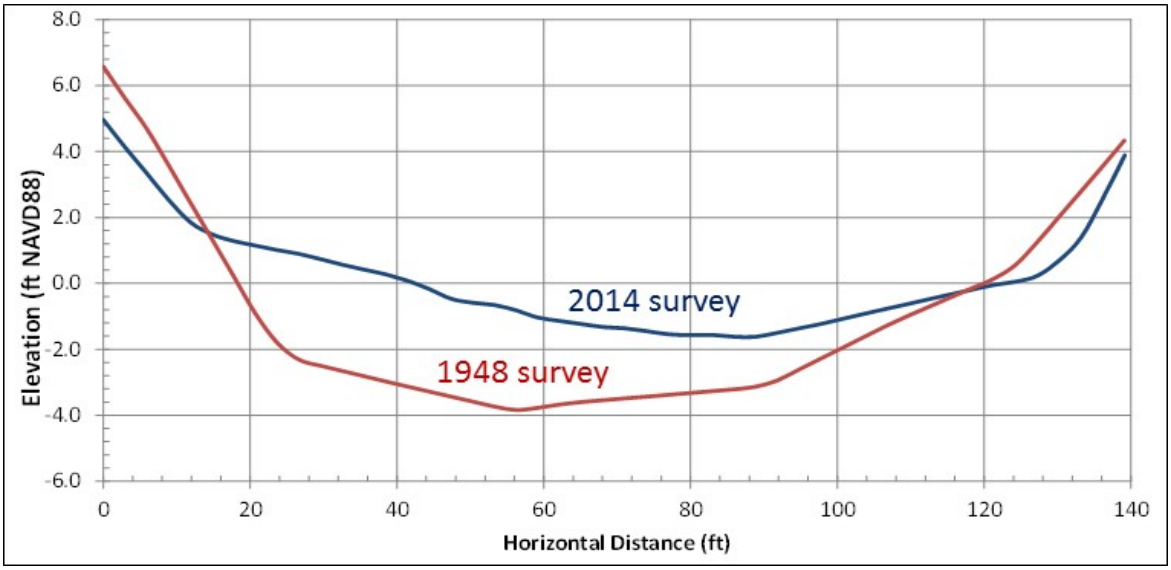


Figure 5-32: Example Siltation in Smith Canal

If a scenario of steady sedimentation of the canal over the past 80 years is assumed with no maintenance dredging, the average rate of siltation in Smith Canal on both sides of I-5 Bridge ranges from 0.2 to 0.3 in. per year, while the average siltation rate in Atherton Cove has been around 0.3 in. per year. This corresponds to siltation of approximately 1.5 to 2.0 ft.

The deepening of the canal ranges from around 1 ft at the I-5 Bridge to around 3 ft near the mouth of the Smith Canal, which is likely a result of dredging.

5.3.3.2 Future Siltation Rates

The hydrodynamic modeling results indicate that the proposed project would not change overall siltation rates in the study area except the area immediately adjacent to the gate. In Smith Canal, the historical average siltation rate ranges from 0.2 to 0.3 inches per year, as discussed in the section above. The sedimentation in the system is largely contributed by the sediment yield from the watershed as the existing peak tidal velocity, as shown in Table 5-5, is less than 0.2 fps, which is too low to mobilization sediment. The watershed has been further developed in the past 80 years and sediment yield from the watershed is expected to be reduced, therefore, the future average siltation rate is expected to be less than 0.2 to 0.3 inches per year. For similar reasons, the future average siltation rate in Atherton Cover is expected to be less than 0.3 inches per year.

6. RESIDENCE TIME ANALYSIS

The general water quality of Smith Canal can be characterized using residence time modeling. In residence time modeling, the constituent concentrations in a water body reflect a balance between the rate of constituent supply and the rate of constituent removal by tidal flushing and clean inflow. Residence time provides a useful measure of the rate at which waters in the hydraulic system are renewed. For example, if there are 100 particles released in a hydraulic system such as Yosemite Lake, the residence time is the average time it takes for the 100 particles to leave the hydraulic system. Accordingly, residence times provide a means for assessing the water quality of the hydraulic system, such as whether changes to a hydraulic system are likely to improve or degrade water quality in the system.

6.1 METHODOLOGY

Water quality is affected by a wide range of variables including salinity, flow exchange, water temperature, oxygen levels, and concentrations of nutrients in the water column. It is not within the scope of the present modeling effort to provide a description of the full range of variables attributing to water quality, but the two dimensional modeling effort does provide insight into two of the primary variables affecting water quality related to flow dynamics, which are flow velocities and flow exchange. These are combined into residence time, which describes the time it takes for water particles to move through the project area. For example, in areas with high flow velocities and substantial flow exchange, residence time may be as short as hours to days, whereas in stagnant areas having little flow exchange, typical residence time may be measured in weeks to months.

The model utilized to evaluate residence time is the MIKE21 MT (Material Transport) model. Consider the reduction of a contaminant concentration in a tidal embayment due to flushing after being released, in which C_0 is initial concentration, K is a reduction coefficient, and $C(t)$ is the concentration at time t:

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

The residence time of the contaminant (T_r) in the embayment is determined by:

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

Since the concentration at $t = T_r$ is:

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

T_r can be calculated from the relative contaminant concentration distribution computed by the model. T_r corresponds to the time the relative concentration, $C(T_r)/C_0$, reaches a value of 1/e or 0.37. The variable e is the base of the natural logarithm and has a value of 2.718.

The general procedure for computing residence times for different parts of a tidal embayment is as follows:

- Assign an initial contaminant concentration of unity within the study area (Smith Canal and Atherton Cove) and a value of zero for the rest of water body. Also, a zero concentration is assigned at model open water boundaries.
- Run the model for a number of tidal cycles until substantial reductions of contaminant concentrations have occurred due to tidal flushing at the locations of interest.
- Extract the computed relative concentration time series at the selected gage locations, as shown in Figure 6-2.
- Determine residence times corresponding to the locations of interest from the distribution curves.

Figure 6-1 and Figure 6-2 show an example of how the method works. It is assumed that the water in the ambient environment outside of the study area is clean and an initial contaminant concentration of zero is assigned in the model, as shown in blue in Figure 6-1. An initial contaminant concentration of unity is specified within the study area, which is Atherton Cove and Smith Canal, as illustrated in Figure 6-1 as red.

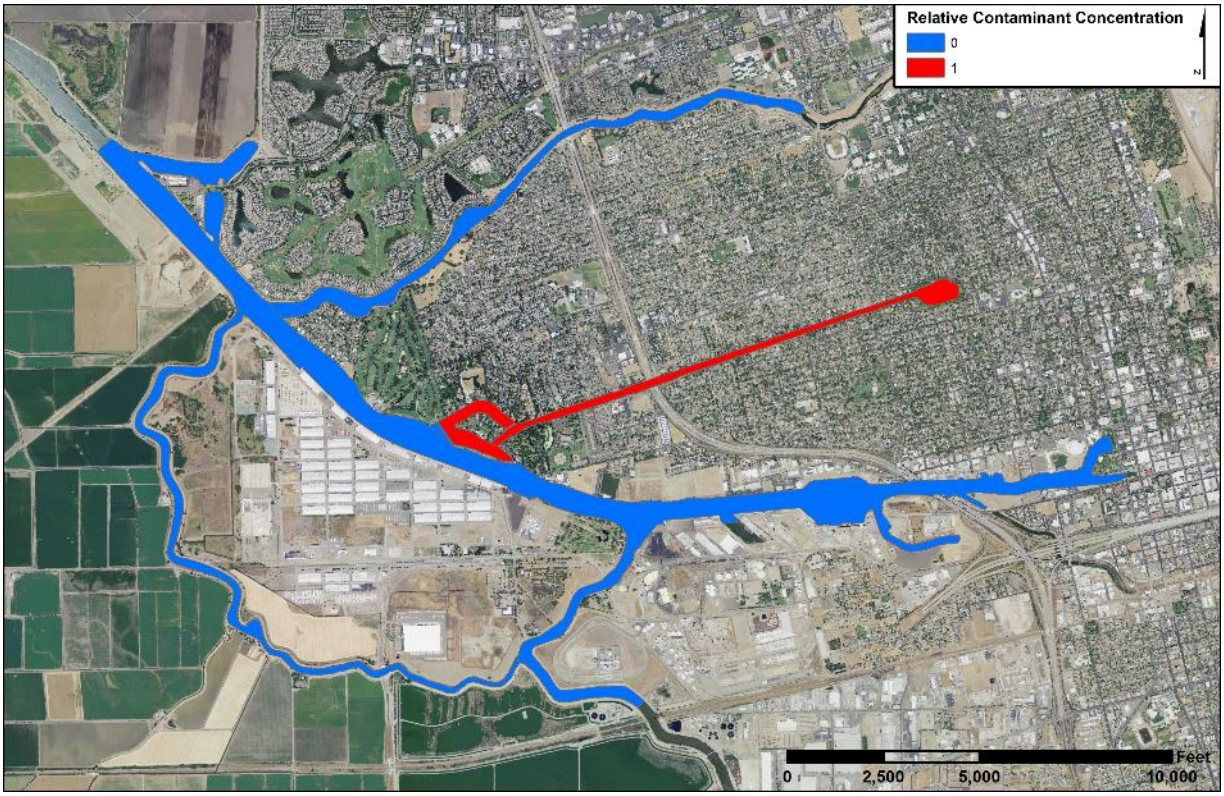


Figure 6-1: Initial Contaminant Concentrations (Blue: Zero; Red: 100% Relative Concentration)

The solid blue line in Figure 6-2 shows an example of results derived from the model. In this example, the red line shows the 25-hour running average (one full diurnal tidal cycle). The red line (average) drops to a value of 1/*e*, or approximately 0.37 after approximately 250 hours, and the blue line drops to 1/e after approximately 268 hours. In this study, the residence time is based on the instantaneous method (blue line) as the residence time in Atherton Cove is very short and less than a day if the moving average method is used. This method is applied to a number of

locations selected throughout Smith Canal and Atherton Cove and the results are compared for "without-project" and "with-project" conditions. The main purpose of transport modeling is to predict whether there are significant changes to residence times, and, therefore, to the water quality within Smith Canal with the proposed closure structure configurations.

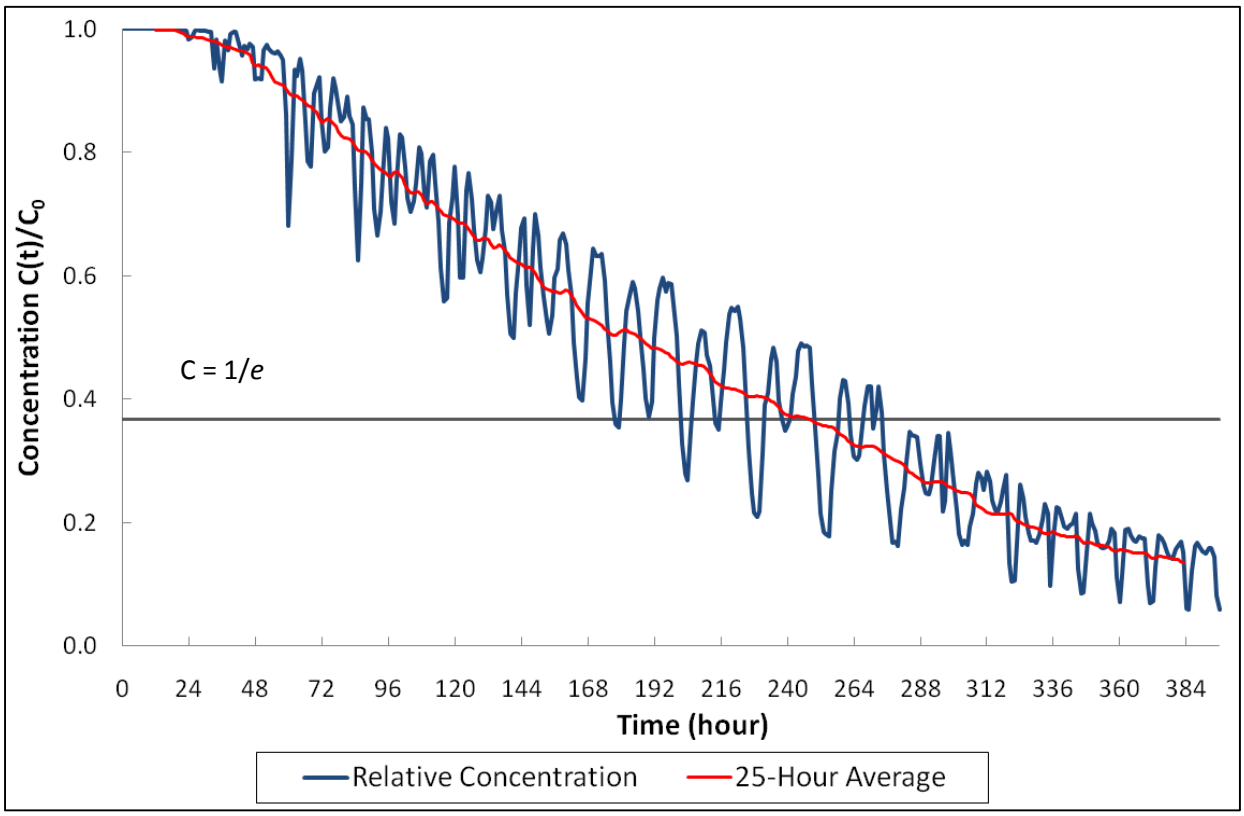


Figure 6-2: Example Residence Time Plot

6.2 RESIDENCE TIME RESULTS

The locations of the residence time reference sites selected in this analysis are presented in Figure 6-3 and Figure 6-4. There are eight sites within Smith Canal and seven within Atherton Cove.



Figure 6-3: Residence Time Locations for Smith Canal



Figure 6-4: Residence Time Locations for Atherton Cove

6.2.1 Alignments Comparison

A comparison of residence time between existing and proposed alignments is summarized in Table 6-1. The results indicate:

- For Alignment 1, there is a slight increase in residence time at Smith Canal 7 and 8. The increase in Smith Canal 7 and 8 are relatively small, about 2%, compared to an overall residence time of 41 days. Residence time at Atherton Cove 5 through 7 is reduced as a result of higher flow toward Atherton Cove.

- For Alignment 2, there is a decrease in residence time of 3.7 days at Smith Canal 4. Examing the relative concentration time series shown in Figure 6-5 indicates that the relative concentration for this alignment perched just below the threshold line of 0.37 while those for existing and Alignment 1 perched just above 0.37 for three consecutive days. This slight improvement of Alignment 2 is largely a result of the locally increased flushing, as model results are extracted from the gage located in the center of the Canal where flushing is better than the sides of the Canal due to its proximity to the gate opening. This improvement is very localized. The overall circulation remains the same when it is away from the gate, as shown in Smith Canal 5 through 8. The proposed closure structure has little effect to the residence time at Atherton Cove, as shown in the results.

- For Alignment 3, there is a similar decrease in residence time of 3.7 days at Smith Canal 4 as in Alignment 2 due to the same reason. The only increase in residence time is at Atherton Cove 5 and 6; again, the difference is very small - less than 4%.

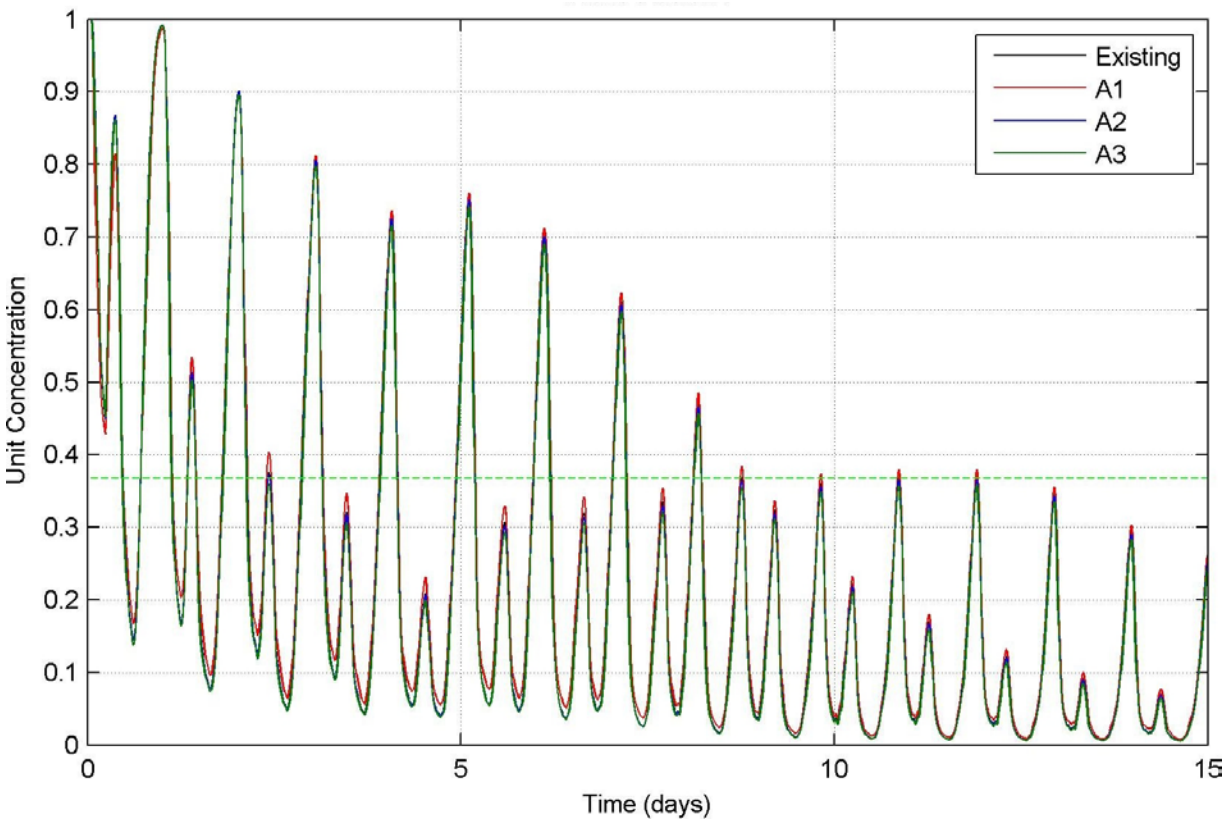


Figure 6-5: Relative Concentration Time Series at Smith Canal #4

Similar to the results of the hydrodynamic modeling and velocity comparisons, comparison of residence times between proposed alignments and the existing conditions did not show any significant changes to the global flushing characteristics in both Atherton Cove and Smith Canal. In fact, the circulation may be slightly improved at Atherton Cove with proposed Alignment 1.

Table 6-1: Residence Time Comparison Between Different Alignments (in days)

Station	Existing	Alignment 1	Alignment 2	Alignment 3
Smith Canal 1	1.0	1.0	1.0	1.0
Smith Canal 2	6.1	6.1	6.1	6.1
Smith Canal 3	7.2	7.2	7.2	7.2
Smith Canal 4	11.9	11.9	8.2	8.2
Smith Canal 5	23.8	23.8	23.8	23.8
Smith Canal 6	36.1	36.1	36.1	36.1
Smith Canal 7	41.0	42.0	41.0	41.0
Smith Canal 8	41.5	42.2	41.5	41.5
Atherton Cove 1	1.0	1.0	1.0	1.0
Atherton Cove 2	1.0	1.0	1.0	1.0
Atherton Cove 3	1.1	1.1	1.1	1.1
Atherton Cove 4	1.1	1.1	1.1	1.1
Atherton Cove 5	1.4	1.2	1.3	1.5
Atherton Cove 6	1.5	1.4	1.5	1.6
Atherton Cove 7	1.6	1.5	1.6	1.6

6.2.2 Alignment 1 Gate Width Sensitivity

Similar to Alternative Alignment comparison, a comparison between different gate widths for Alternative Alignment 1 was performed and results are presented in

Table 6-2.

- As discussed in the prior section, for the 50-ft gate width, there is a slight reduction in residence time in Atherton Cove and increases at Smith Canal 7 and 8 are very small, less than 2%.
- The 100-ft and 200-ft gate widths show very similar results compared with those of the 50-ft gate width. The slight reduction of residence time at Atherton Cove decreases with the increase in gate width. The residence time in Smith Canal generally remained the same with different gate widths.

Overall, comparison of residence times between different gate widths did not show any significant changes to the global flushing characteristics in both Atherton Cove and Smith Canal. In other words, residence times in both Atherton Cove and Smith Canal are not sensitive to the gate width.

Table 6-2: Residence Time Comparison Between Different Gate Widths for Alignment 1 (in days)

Station	Existing	50-ft Gate	100-ft Gate	200-ft Gate
Smith Canal 1	1.0	1.0	1.0	1.0
Smith Canal 2	6.1	6.1	6.1	6.2
Smith Canal 3	7.2	7.2	7.2	7.2
Smith Canal 4	11.9	11.9	11.9	11.9
Smith Canal 5	23.8	23.8	23.8	23.8
Smith Canal 6	36.1	36.1	36.1	36.1
Smith Canal 7	41.0	42.0	42.0	42.0
Smith Canal 8	41.5	42.2	42.2	42.2
Atherton Cove 1	1.0	1.0	1.0	1.0
Atherton Cove 2	1.0	1.0	1.0	1.1
Atherton Cove 3	1.1	1.1	1.1	1.1
Atherton Cove 4	1.1	1.1	1.1	1.1
Atherton Cove 5	1.4	1.2	1.2	1.4
Atherton Cove 6	1.5	1.4	1.5	1.5
Atherton Cove 7	1.6	1.5	1.6	1.6

7. ANALYSES OF WATER QUALITY IMPROVEMENT OPTIONS

The Alignment 1 with a 50-ft gate hydrodynamic model was used to assess the effectiveness of four different Water Quality Improvement Options on hydrodynamics and residence times. Each improvement Option is modeled independently, and compared to Alignment 1. This section describes the Water Quality Improvement Options and hydrodynamic and water quality modeling results.

The same modeling suite of MIKE21-FM used in the Alignment Alternative evaluation is used. However, the Water Quality Improvement Options include culverts and the model execution time with culvert structures is very long. The model execution time is almost equivalent to the real time. With this limitation, the simulation is reduced from 45-day for Alignment Alternative to 7 days for Water Quality Improvement Options. However, a 7-day simulation time is sufficient to assess effectiveness of Water Quality Improvement Options.

7.1 DESCRIPTION OF WATER QUALITY IMPROVEMENT OPTIONS

The four Water Quality Improvement Options are:

- *Option 1: Culverts under Moreing Road to Atherton Island* - propose to install culverts under the road. The function of the culverts is primarily as open conduits, enabling exchange of water between Atherton Cove and Smith Canal. Prior to numerical modeling, culverts were sized by hand-calculations to determine the likely number of culverts and size in order to facilitate flow exchange. The calculations indicated that 3, 4-ft diameter culverts should be used with inverts at both ends placed at -2 ft NAVD88. The culverts were assumed to be 180 ft (55 m) long and modeled using a Manning's n of 0.0167 $s/m^{1/3}$.

- *Option 2: Dredging Atherton Cove* - propose to dredge the bed elevation in the Cove to an elevation of -4 ft NAVD88. The average dredging depth is 2 ft. The mesh from Alignment 1 was revised to reflect the dredged condition in Atherton Cove.

- *Option 3: Combination of Configuration Options 1 and 2* - with both dredging of Atherton Cove and culverts under Moreing Road.

- *Option 4: Culverts in Floodwall* - Propose to install culverts within the floodwall. The culverts provide continuous water exchange on flood and ebb tides. Prior to numerical modeling, culverts were sized by hand-calculations to determine the likely number of culverts and size in order to facilitate flow exchange. The calculations indicated that 2 4-ft diameter culverts on the north side of the gate and three 4-ft diameter culverts on the south side of the gate should be used. Inverts at both ends were placed at -4 ft NAVD88. The culverts were assumed to be 29.5 ft (9 m) long and modeled using a Manning's *n* of 0.0167 $s/m^{1/3}$. This roughness coefficient is relatively conservative for new culverts, but is reasonable with some marine growth.

An overview of Water Quality Improvement Options 1, 3 and 4 is provided in Figure 7-1.

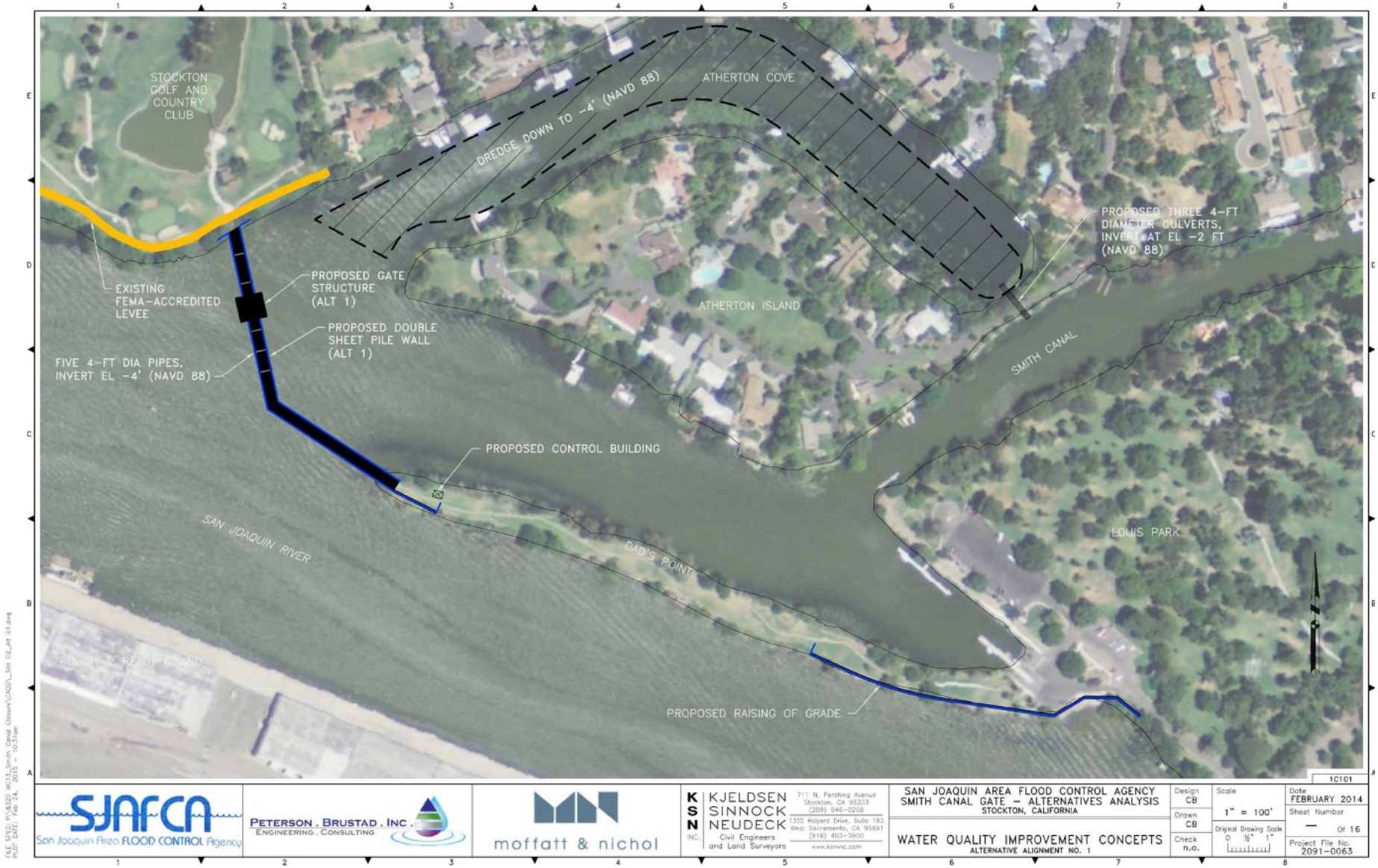


Figure 7-1: Overview of Water Quality Improvement Options 1, 3, and 4

7.2 HYDRODYNAMIC ANALYSIS OF WATER QUALITY IMPROVEMENT OPTIONS

Water Quality Improvement Options 1, 3 and 4 included culvert structures. The dimensions of culverts are much smaller than the model element size used in the computational grid. Therefore, the effect of structures is modeled by a subgrid technique. The flow past a structure is driven by differential head between the upstream and downstream not the current velocity.

7.2.1 Hydrodynamic Modeling Results

To assess effectiveness of Water Quality Options on tidal exchange and circulation in Atherton Cove and Smith Canal, two transects, one within Atherton Cove and one within Smith Canal, were selected and shown in Figure 5-1. Modeling results of stage and discharge were extracted as changes in stage and discharge are indications of tidal exchange and circulation changes. The stage is defined as the average surface elevation across transect. The discharge is the total flow rate through the cross-section where transects are located. The discharge is defined as positive if the water is flowing into Smith Canal/Atherton Cove or negative if it is flowing out. In other words, the flood tide discharge is positive and ebb tide discharge is negative.

The stage and discharge under each proposed Water Quality Option were compared to those under Alignment 1 conditions. Similar to the methods used in model calibration and validation, the correlation and NRMSE, defined in Section 4.2.2, were calculated for each comparison.

At the Smith Canal Transect, the comparison of stage and discharge are shown in Figure 7-2 and Figure 7-3. The time series plots show that stage and discharge under the proposed water Quality Improvement Options overlap those under Alignment 1. The scatter plots in Figure 7-3 also show a great match between Alignment 1 and proposed Water Quality Options. The statistical analysis of the comparison is summarized in Table 7-1. These results indicate:

- For the stage comparison, the R^2 correlation coefficients are nearly equal to 1, indicating perfect matches and NRMS difference is less than 0.1%. Therefore, there is very little change in stage from the Alignment 1 for all Water Quality Options.
- For the discharge comparison, the R^2 correlation coefficients are nearly equal to 1 and the NRMS difference is less than 0.5%. Therefore, there is a very small amount of change to discharge at this location with Water Quality Options when compared to Alignment 1.

Table 7-1: Statistical Analysis of Hydrodynamic Results at Smith Canal for Water Quality Improvement Options

Variable	Water Quality Improvement Option	R^2 Correlation Coefficient	NRMS Difference (%)
Stage	1	1.000	0.04%
	2	1.000	0.01%
	3	1.000	0.01%
	4	1.000	0.07%
Discharge	1	1.000	0.54%
	2	1.000	0.39%
	3	1.000	0.37%
	4	1.000	0.80%

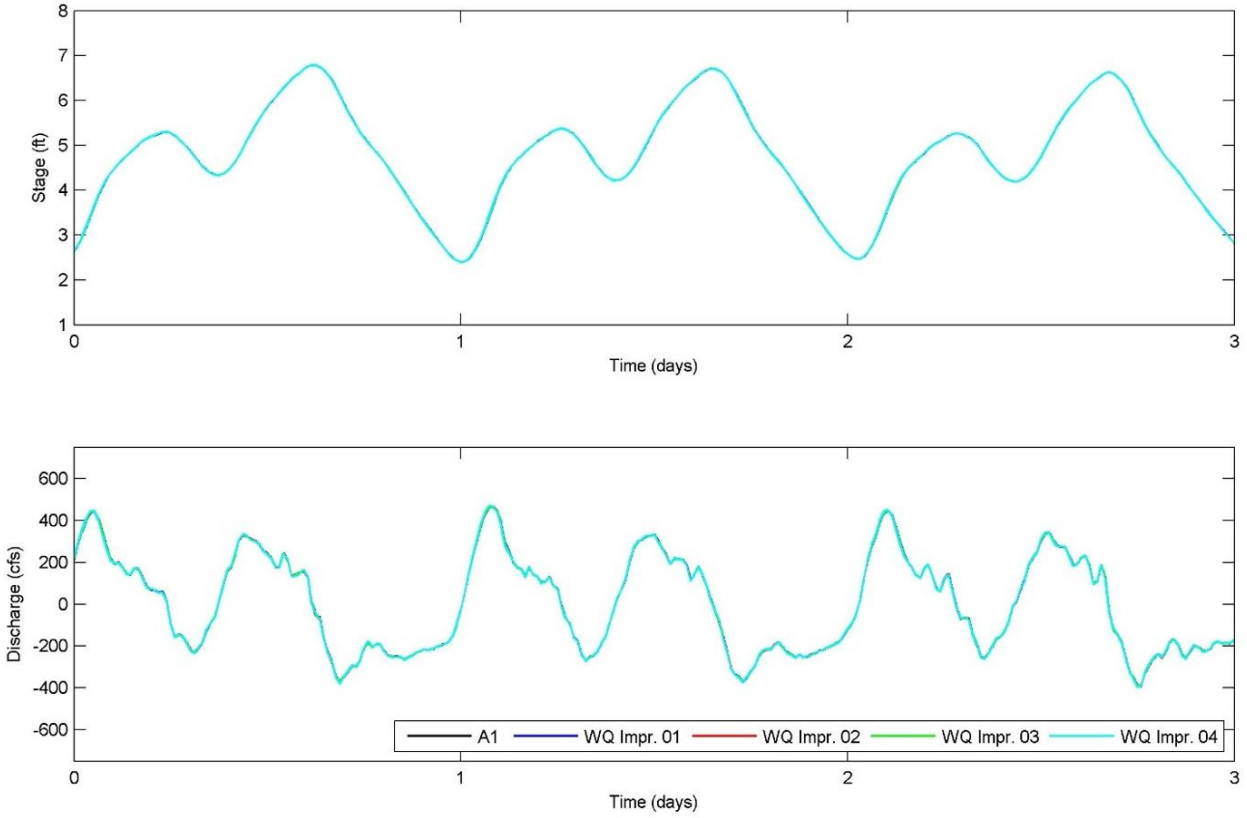


Figure 7-2: Time Series of Stage (Top) and Discharge (Bottom) at Smith Canal for Water Quality Improvement Options

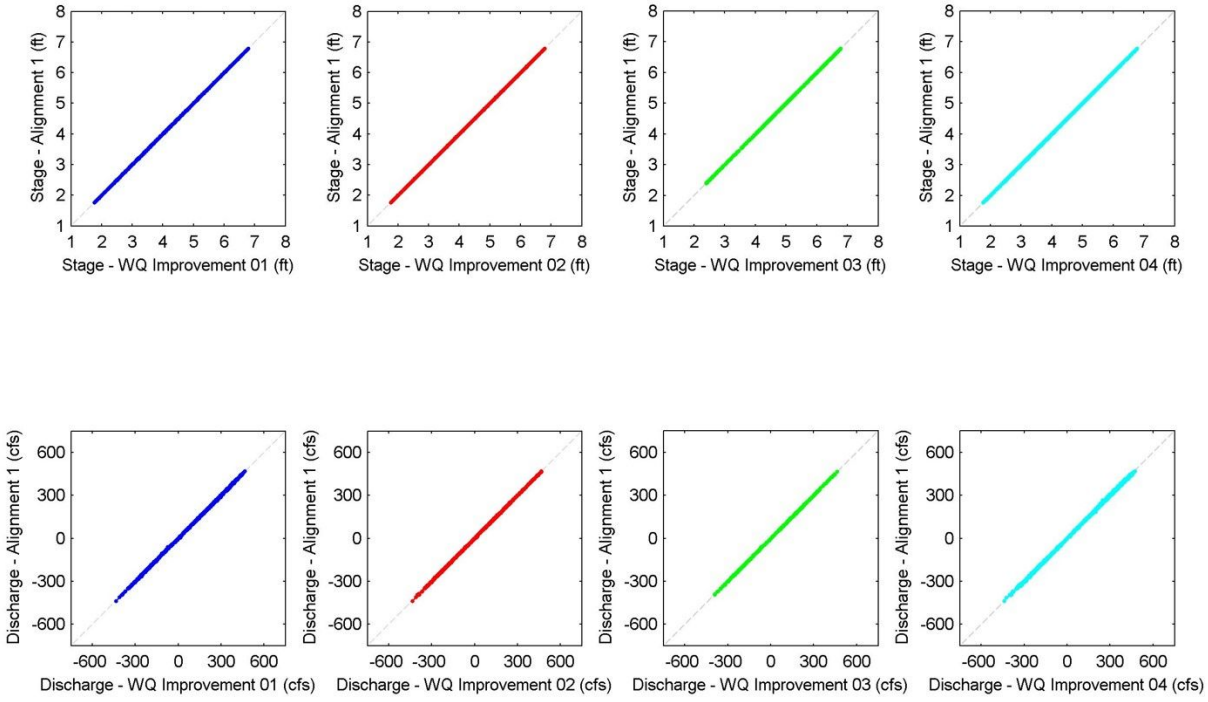


Figure 7-3: Stage (Top) and Discharge (Bottom) Correlation between Water Quality Options and Alignment 1 at Smith Canal

At the Atherton Cove Transect, the comparison of stage and discharge is shown in Figure 7-4 and Figure 7-5. The time series plots show that stage and discharge under the proposed Water Quality Improvement Options overlap those under Alignment 1. The scatter plots in Figure 7-5 also show a great match between Alignment 1 and proposed Water Quality Options. The statistical analysis of the comparison is summarized in Table 7-2. These results indicate:

- For the stage comparison, the R^2 correlation coefficients are nearly equal to 1, indicating perfect matches and NRMS difference is less than 0.1%. Therefore, there is very little change in stage from the Alignment 1 for all Water Quality Options.
- For the discharge comparison, the R^2 correlation coefficients are equal to 0.95 and the NRMS difference is between 11 and 16%. Therefore, there is a small amount of change to discharge at this location with Water Quality Options when compared to Alignment 1 as these Water Quality Improvement Options are located in or very close to the Atherton Cove. Slightly higher discharge of Option 2 is associated with increased Cove volume as a result of dredging.

Table 7-2: Statistical Analysis of Hydrodynamic Results at Atherton Cove for Water Quality Improvement Options

Variable	Water Quality Improvement Option	R^2 Correlation Coefficient	NRMS Difference (%)
Stage	1	1.000	0.03%
	2	1.000	0.01%
	3	1.000	0.01%
	4	1.000	0.07%
Discharge	1	0.964	11.18%
	2	0.929	16.08%
	3	0.953	13.74%
	4	0.952	11.65%

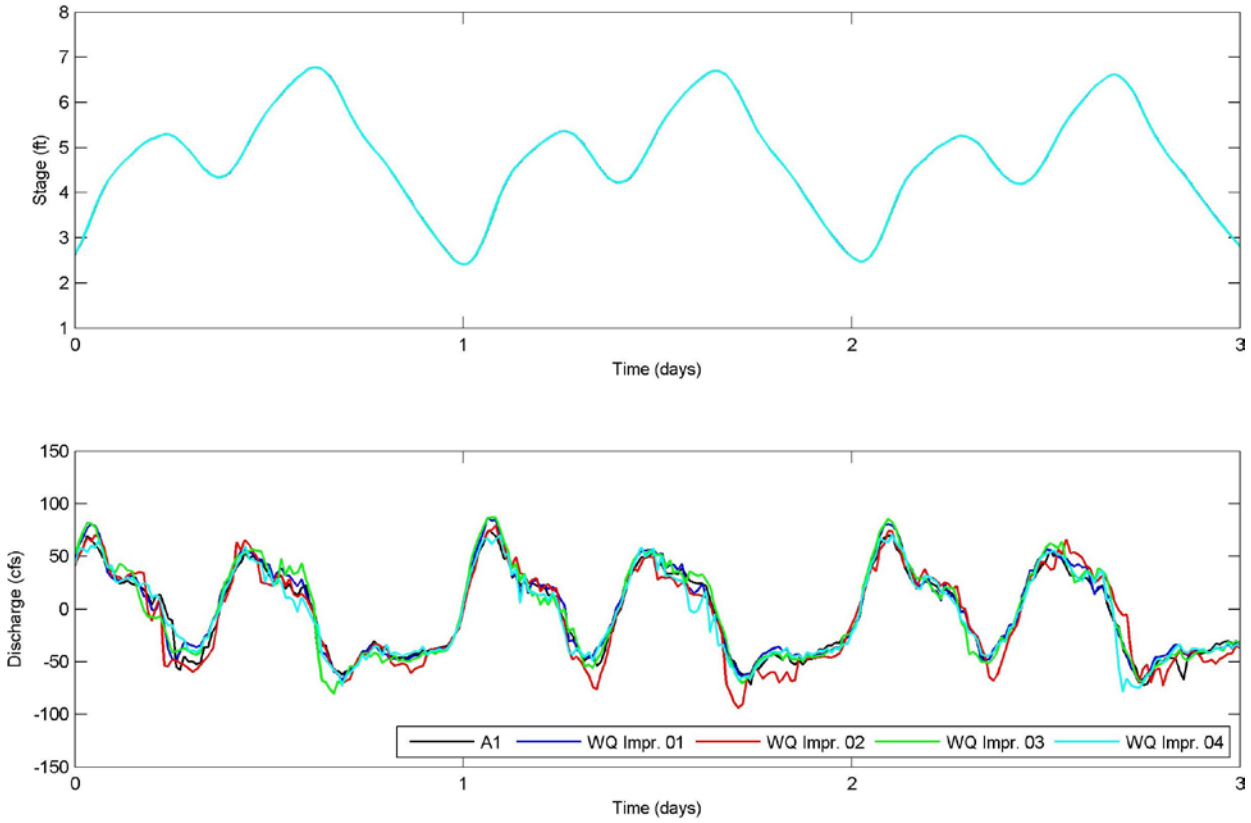


Figure 7-4: Time Series of Stage (Top) and Discharge (Bottom) at Atherton Cove for Water Quality Improvement Options

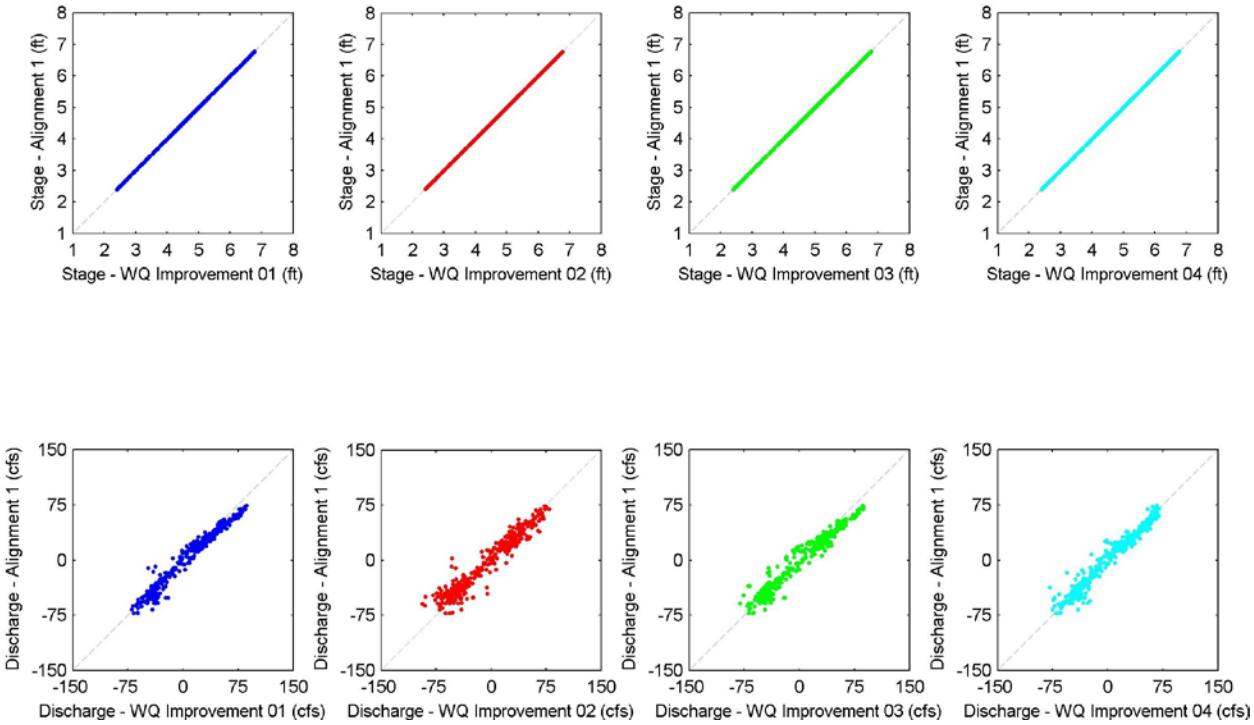


Figure 7-5: Stage (Top) and Discharge (Bottom) Correlation between Water Quality Options and Alignment 1 at Atherton Cove

7.2.2 Velocity at Proposed Gates

This section summarizes modeling results of tidal velocity through the proposed 50-ft gate under the four water Quality Improvement Options. The sample tidal velocity through the gate under the proposed Water Quality Improvement Options and that at the same location under the Alignment 1 are shown in Figure 7-6. The results indicate that the velocity through the 50-ft gate does not change much under the water Quality Improvement Options, except that for Option 4. Option 4 proposes to install five 4-ft diameter culverts through the floodwall. The culverts increase flood conveyance, hence slightly lowering the velocity through the gate. Similarly, the percentage of current speed exceedance does not change except for Option 4, as summarized in Table 7-3. Table 7-3 provided to show the relative comparison among the water Quality Improvement Options and was based on results from the 7-day modeling period; hence, the percentages were slightly different from those shown in Table 5-6, which was based on two spring-neap tidal cycles. Figure 7-7 shows comparison of the current speed exceedance curves. The exceedance curves are very similar except for Option 4.

Table 7-3: Current Speed Exceedance Percentages for Water Quality Improvement Options

Current Speed (ft/s)	Alignment 1	Water Quality Improvement			
		Option 1	Option 2	Option 3	Option 4
1.00	37.6%	38.8%	37.6%	37.9%	30.3%
1.50	7.3%	7.9%	7.6%	7.6%	5.1%
2.00	2.5%	2.5%	2.5%	2.5%	0.0%
2.50	0.0%	0.0%	0.0%	0.0%	0.0%

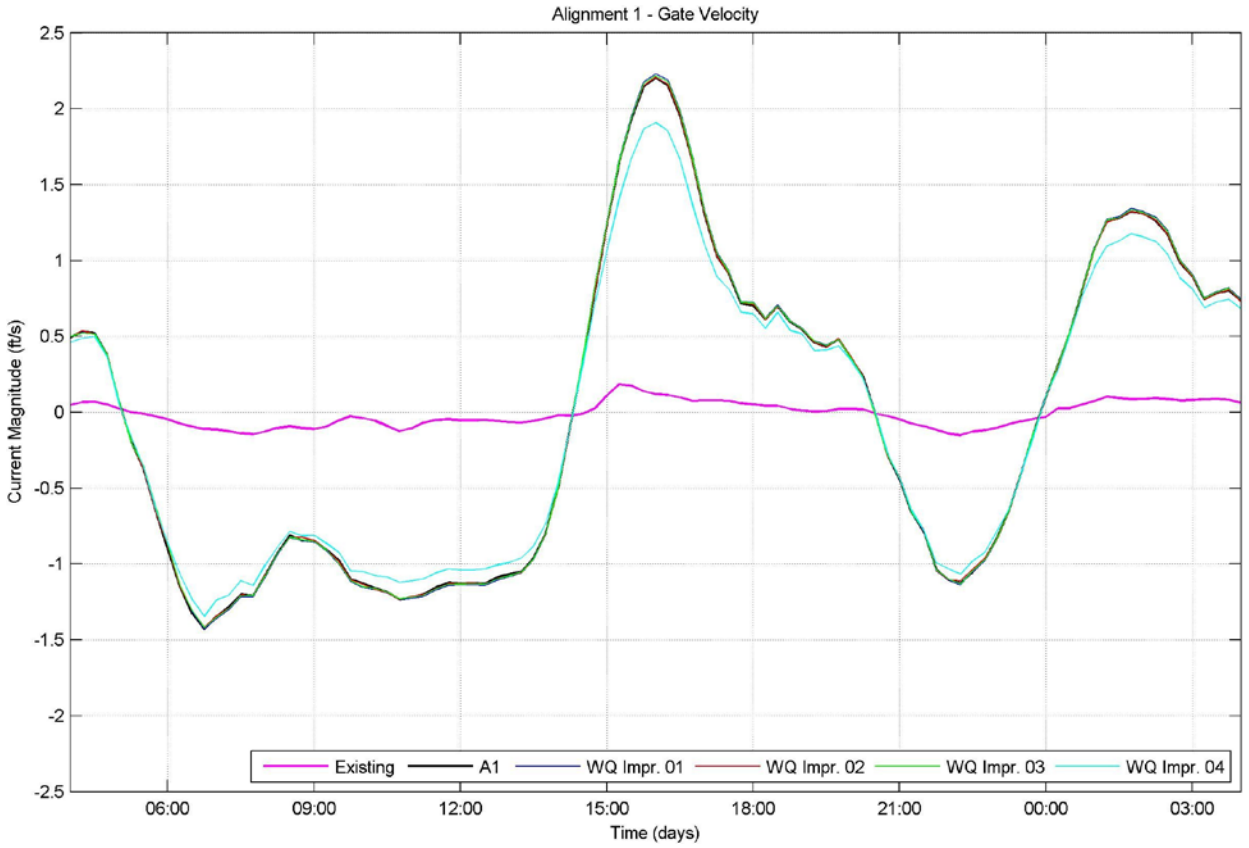


Figure 7-6: Comparison of Velocity at Gate of Alignment 1 with Water Quality Improvements

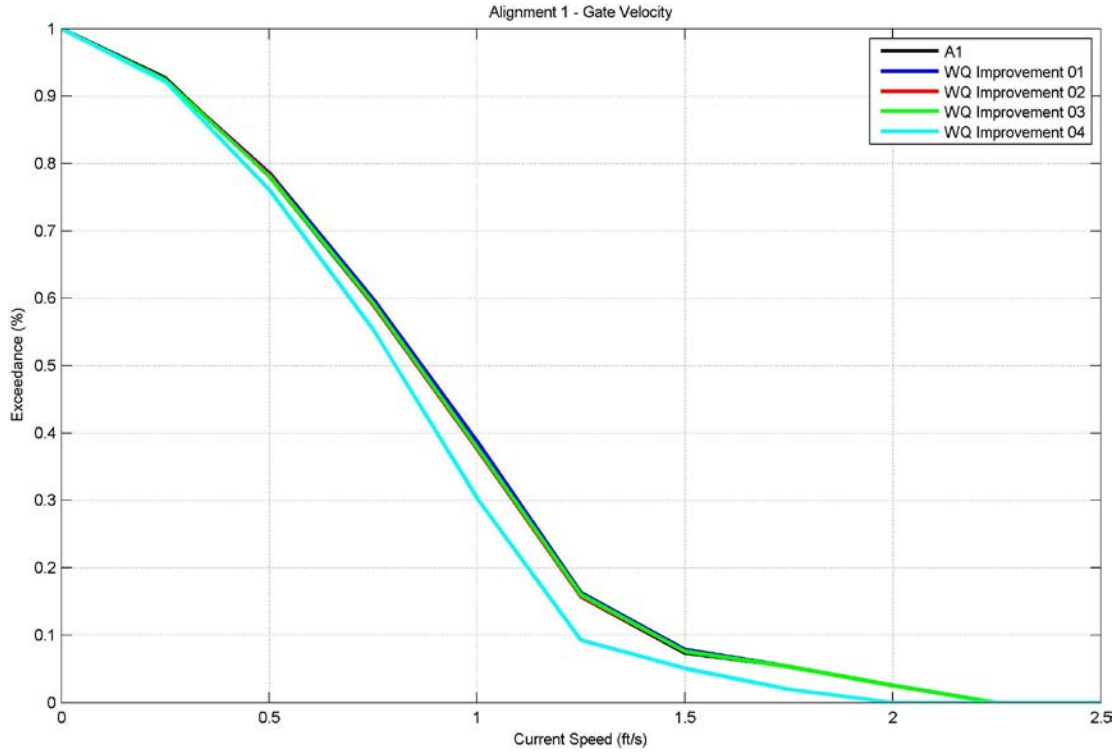


Figure 7-7: Current Speed Exceedance Curves at Gate of Alignment 1 with Water Quality Improvements

The velocity patterns of Alignment 1 with Water Quality Improvement Options are also very similar to those under the Alignment 1 condition. The comparison is included in Appendix B. Therefore, no appreciable differences in sedimentation pattern and trend are expected if Water Quality Improvement Options are incorporated into the project.

7.3 RESIDENCE TIME ANALYSIS OF WATER QUALITY IMPROVEMENT OPTIONS

The same methodology used in the Alignment Alternative analyses is used in assessment of residence time for water Quality Improvement Options. The residence times at the same gage locations in both Smith Canal and Atherton Cove, shown in Figure 6-3 and Figure 6-4 respectively, are analyzed and results are summarized in Table 7-4. For gages in Smith Canal, only Gages 1 and 2 results are included as the simulation duration is not long enough to extract results in the remaining gages. However, the simulation duration is long enough to assess the potential benefit of the Water Quality Improvements Options. These results indicate:

- In Smith Canal, the residence time is not affected by the Water Quality Improvement Options.
- In Atherton Cove, the residence time at Gages 6 and 7 is slightly reduced with the culverts under Moreing Road, but residence time will be slightly longer with proposed dredging at Atherton Cove due to increased tidal prism.

In general, marinas with a residence time of 1-2 days are considered well circulated. This modeling exercise assesses circulation and does not address other water quality issues and benefits; for example, dredging could remove contaminated soil if the soil is contaminated and may improve water quality in terms of dissolved oxygen, temperature, etc.

Table 7-4: Residence Time Comparison Between Different Water Quality Improvements for Alignment 1 (in days)

Location	Existing	Alignment 1	Water Quality Improvement			
			Option 1	Option 2	Option 3	Option 4
Smith Canal 1	1.0	1.0	1.0	1.0	1.0	1.0
Smith Canal 2	6.2	6.2	6.2	6.2	6.2	6.2
Atherton Cove 1	1.0	1.0	1.0	1.0	1.0	1.0
Atherton Cove 2	1.0	1.0	1.0	1.1	1.0	1.0
Atherton Cove 3	1.1	1.1	1.1	1.1	1.1	1.1
Atherton Cove 4	1.1	1.1	1.1	1.2	1.2	1.1
Atherton Cove 5	1.4	1.2	1.2	1.5	1.5	1.2
Atherton Cove 6	1.5	1.5	1.3	1.6	1.5	1.5
Atherton Cove 7	1.6	1.5	1.3	1.7	1.6	1.5

7.4 CONCLUSIONS OF WATER QUALITY IMPROVEMENT OPTIONS

The hydrodynamic and residence time analyses results indicate that proposed Water Quality Improvement Options do not change the overall hydrodynamic circulation, residence time, and sedimentation patterns. The impacts of individual options are:

- Water Quality Improvement Option 1 - the residence time at the far end of Atherton Cove is slightly reduced;

- Water Quality Improvement Option 2 - the residence time at the far end of Atherton Cove is slightly increased due to increased water volume in the Cove resulting from dredging;
- Water Quality Improvement Option 3 – the residence time results of Option 3 are between that of Options 1 and 2;
- Water Quality Improvement Option 4 – there is no discernable improvement to the residence time, however, it slightly reduces the tidal velocity through the gate with addition of culverts in the floodwall.

8. FINDINGS AND CONCLUSIONS

The MIKE21-FM model suite was applied in this study. The study focuses on hydrodynamic evaluation of the three proposed closure structure alignments, sensitivity evaluation of three gate widths and Water Quality Improvement Options for Alternative Alignment 1. For each proposed alternative alignment, gate width and Water Quality Improvement Option, analyses within Smith Canal and Atherton Cove included evaluation of the flow velocities at the proposed gates, potential impacts to tidal hydrodynamics, and changes in residence time.

The model results indicate that introduction of the closure structure has very little impact to tidal range and tidal prism of both Atherton Cove and Smith Canal for all three proposed alignments, gate widths and Water Quality Improvement Options. Large-scale flow patterns within Smith Canal and Atherton Cove in areas away from the closure structures are also largely unaffected. However, it does lead to localized changes to the flow velocity and pattern within the vicinity of the gates. There is an increase in flow velocity on both ebb and flood tides, but this effect is largely decreased with distance away from the gate. The peak flood tide velocity through the gate reaches approximately 2.4 fps for Alternative Alignment 1 with a 50-ft wide gate and less than 1.3 fps for remaining gate widths and alternatives. Below is a brief summary of flow field changes in the vicinity of the closure structures:

- For Alternative 1, on the flood tide, the gate redistributes the flow from the SDWSC and creates a localized counter-clockwise gentle eddy at the entrance to Atherton Cove. The sensitivity analysis indicates a wider gate opening would result in less flow redistribution and peak discharge changes.

- For Alignment 2, the sheet pile wall within Atherton Cove and gate structure within Smith Canal do not significantly change flow patterns, but slightly reduces the discharge into the Atherton Cove.

- For Alignment 3, there is a change to the flow field due to the convergence of the two jets from the gate structure. On the ebb tide, this convergence creates an area of slightly increased flow velocity at the entrance to Smith Canal from the SDWSC. Changes to the flow field on the flood tide are localized to the gate structures.

A literature review indicated the canal has not been resurveyed since the earliest available survey data in 1934. Bathymetric surveying provided by Kjeldsen, Sinnock, Neudeck, Inc. (KSN) for the current project indicates that water depths in Smith Canal to the east of I-5 are substantially lower than in past times due to siltation, whereas water depths in the portion of the canal west of I-5 have increased, which is likely the result of dredging. Water depths in Atherton Cove have also decreased due to siltation. The historical average siltation rate is around 0.2 to 0.3 ft per year in the past 80-years. More development has occurred in the past 80 years, so it is expected that the future average siltation rate shall be slightly reduced.

With regards to siltation, model results show regions of increased and decreased peak tidal velocities, which indicate a potential for areas of deposition and erosion. With the high flow velocities through the gate, there is a potential for mobilization of bottom sediments. However, changes in sedimentation are limited to the immediate vicinity of the structures and to the entrance area to Atherton Cove and Smith Canal. Scour protection may be required for some embankments.

Transport model results indicate that residence time is largely unaffected by introduction of the closure structures for any of the alternatives. Typical residence times near the mouth of Smith Canal are around one day, whereas residence times approximately midway into the canal

approach 30 days. Residence times in the back of the canal and into Yosemite Lake exceed 40 days. The residence time is very short, less than 2 days in Atherton Cove, indicating good tidal flushing and circulation in the area.

Gate width sensitivity analyses indicate a wider gate would reduce the peak tidal velocity through the gate; however, it has no significant benefit in terms of tidal circulation or residence time in the hydraulic system modeled.

The modeling results of the Water Quality Improvement Options indicate these options would not change the overall hydrodynamics, circulation efficiency, and sedimentation pattern. The localized impacts and/or benefits are:

- Installing culverts under Moreing Road would slightly reduce residence at the end of Atherton Cove.
- Dredging of Atherton Cove would slightly increase residence time at Atherton Cove as dredging increases the water volume in the Cove, but the residence time at Atherton Cove is still very short, less than 2-days.
- Installing culverts in the floodwall would slightly reduce tidal velocity through the 50-ft gate, but would not change the overall circulation. This is consistent with results from the gate width sensitivity analyses.

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APPENDIX A

MODEL MESH AROUND GATES

A.1 Alternative Alignment Comparison

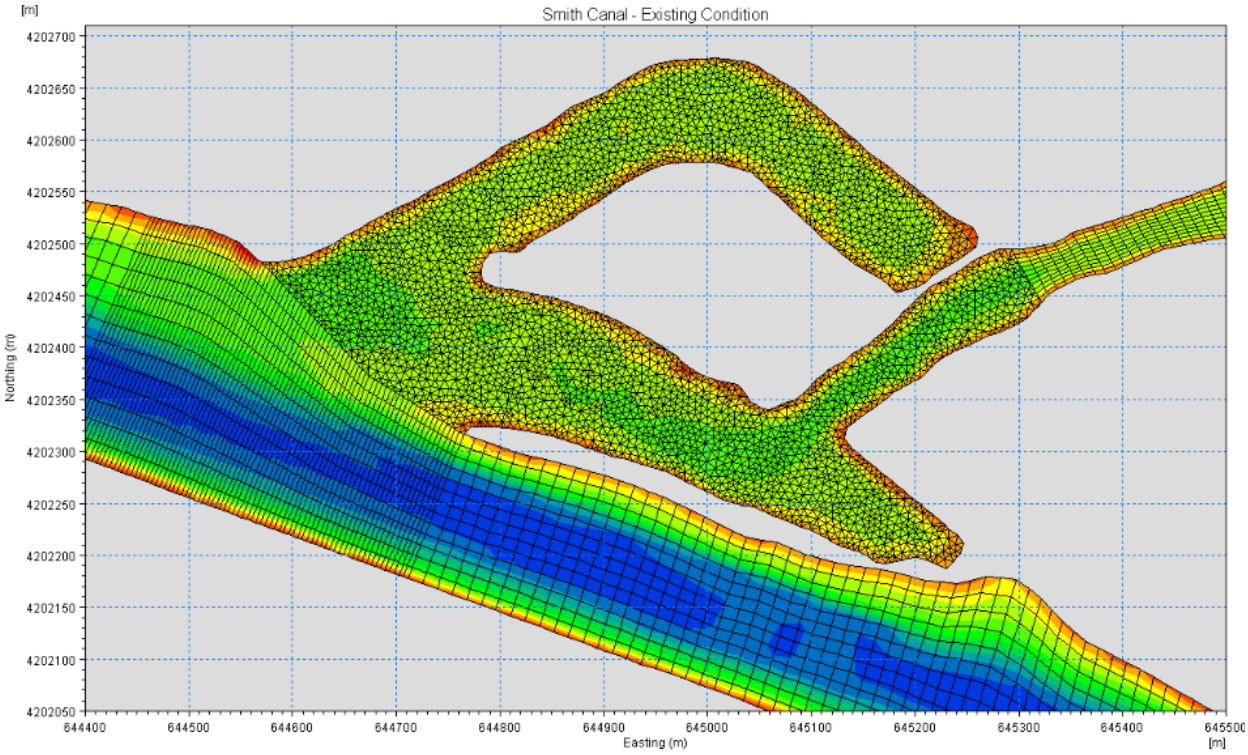


Figure A-1: Existing Condition Mesh

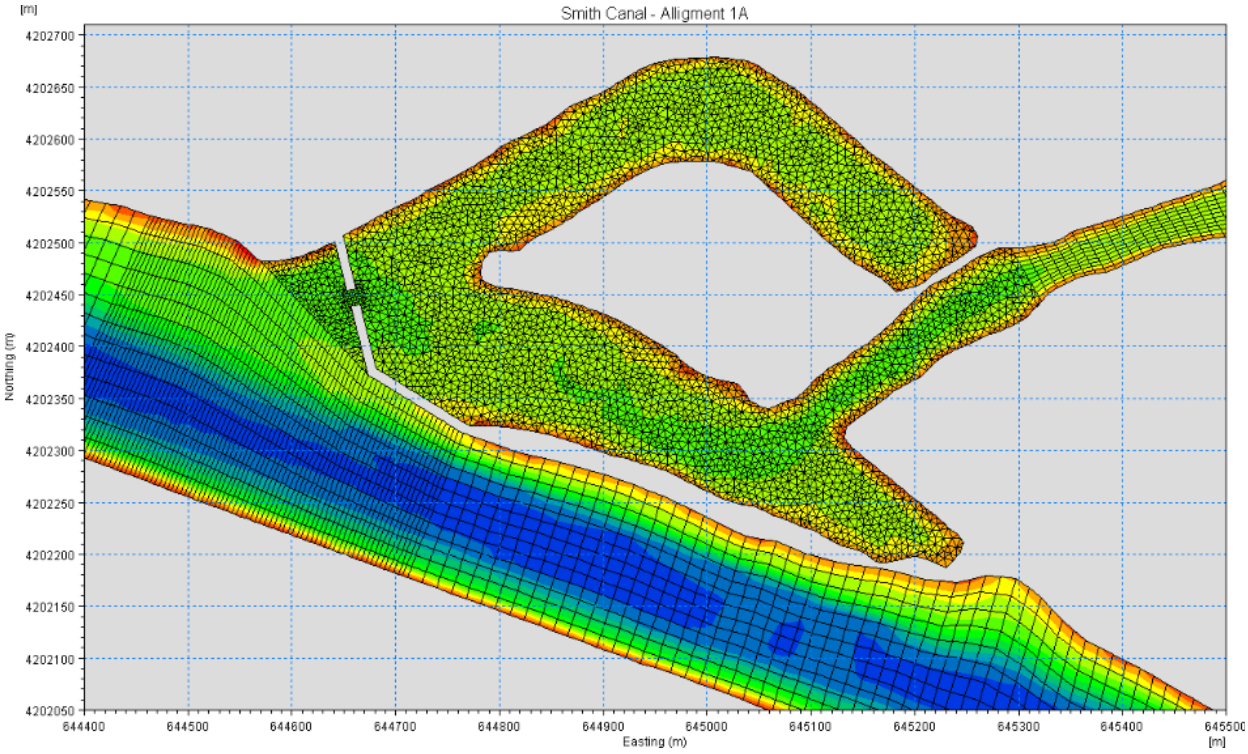


Figure A-2: Alignment 1 Mesh

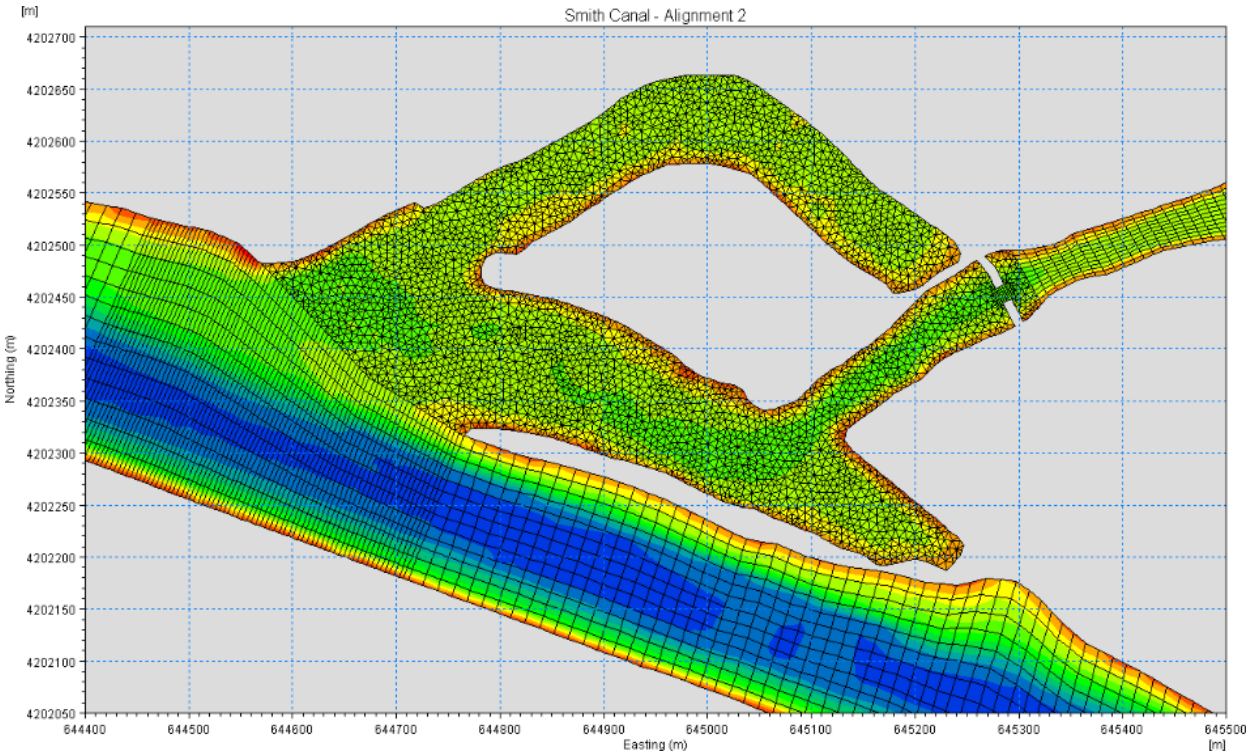


Figure A-3: Alignment 2 Mesh

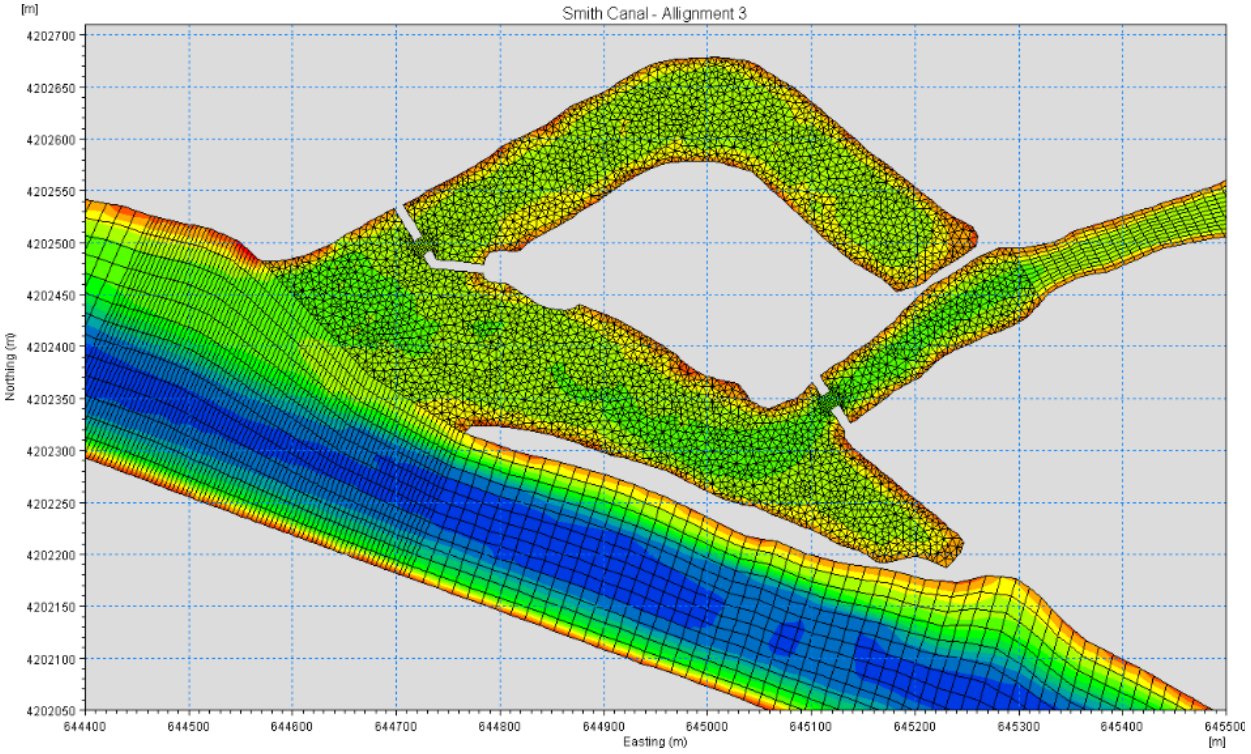


Figure A-4: Alignment 3 Mesh

A.2 Alignment 1A Gate Width Comparison

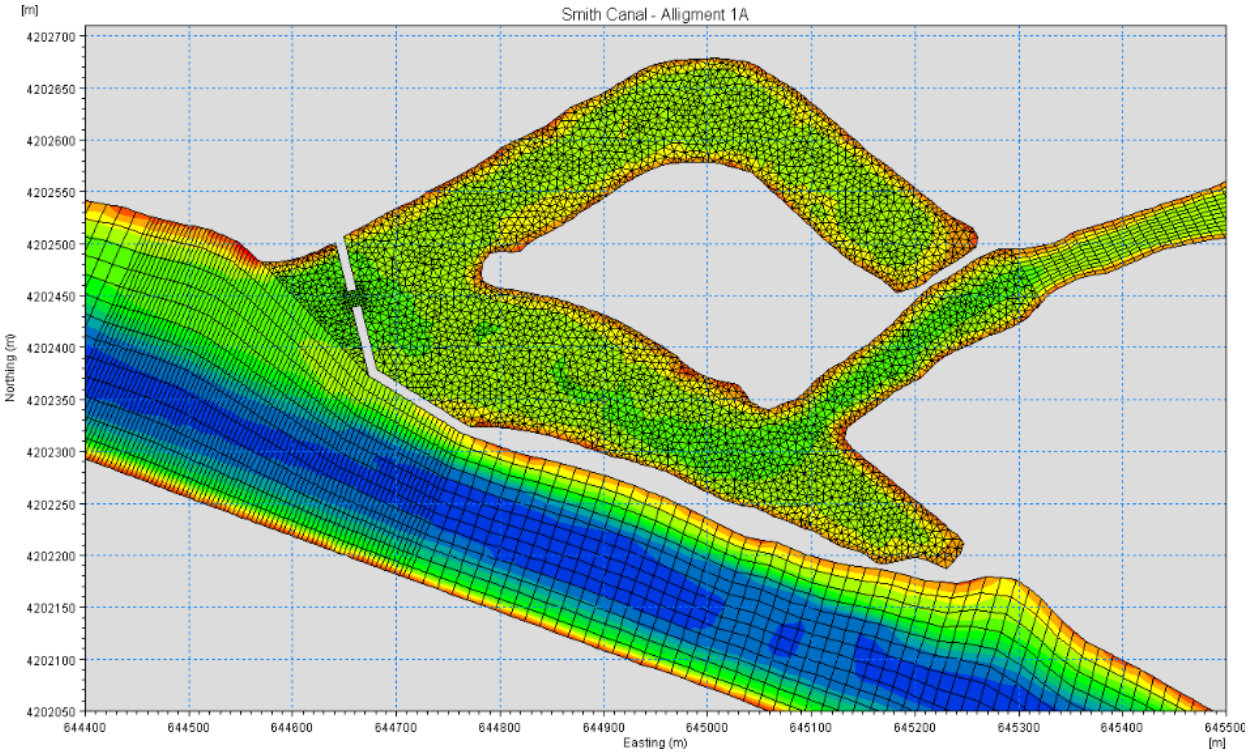


Figure A-5: Alignment 1 Mesh (50 ft Gate Width)

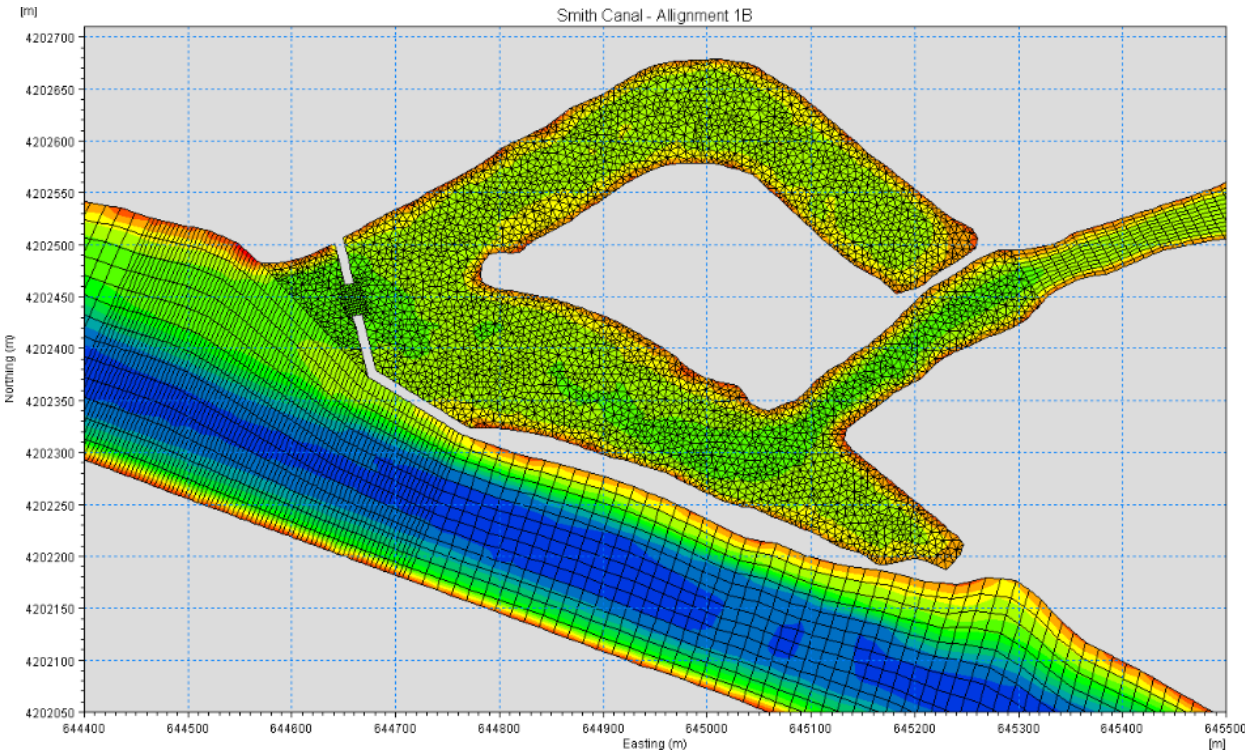


Figure A-6: Alignment 1 Mesh (100 ft Gate Width)

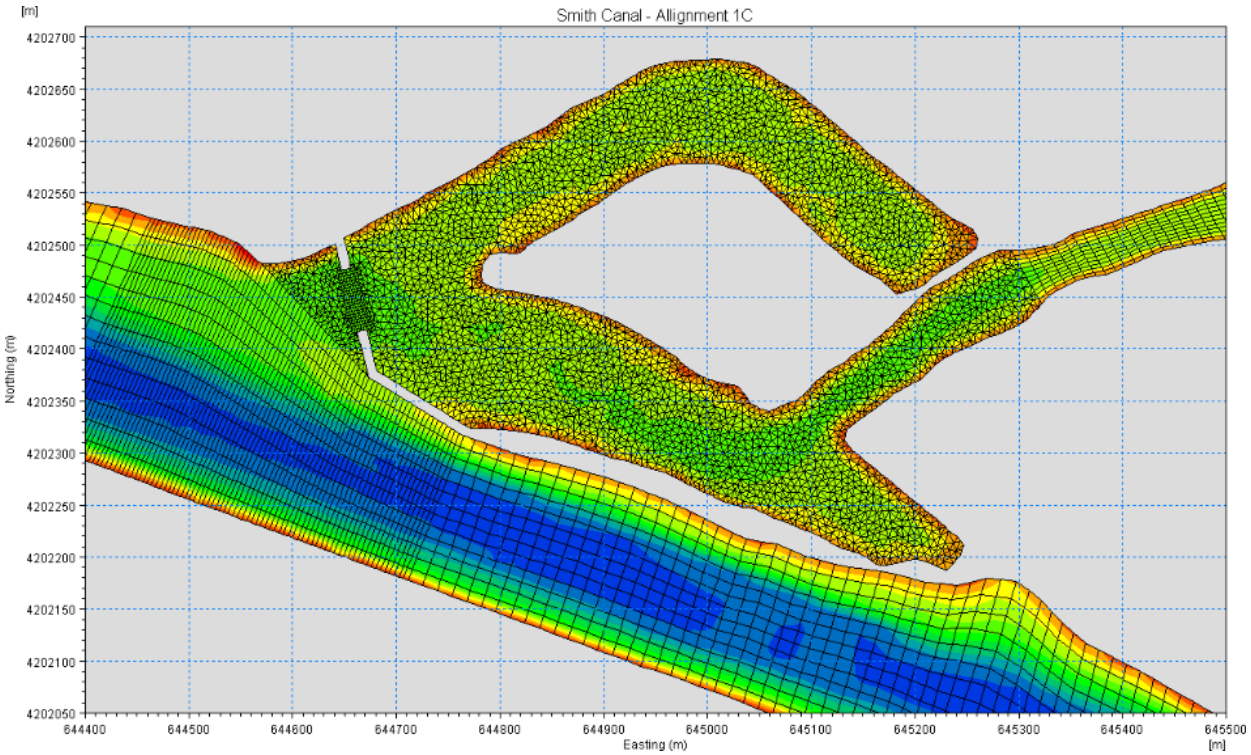


Figure A-7: Alignment 1 Mesh (200 ft Gate Width)

APPENDIX B

VELOCITY PATTERN AND DIFFERENCE PLOTS FOR WATER QUALITY IMPROVEMENT OPTIONS

Water Quality Improvement Option 1 during the flood tide:

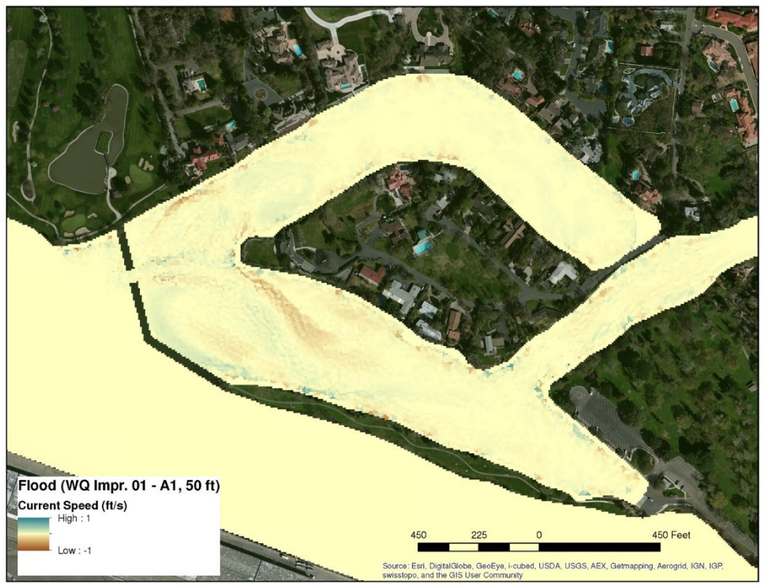
ALIGNMENT 1 (50 FT GATE)	WATER QUALITY IMPROVEMENT OPTION 1 - ALIGNMENT 1 (50 FT GATE)
	
	

Figure B-1:Alignment 1 (50 ft Gate) and Water Quality Improvement Option 1 Flood Tide Velocity Difference

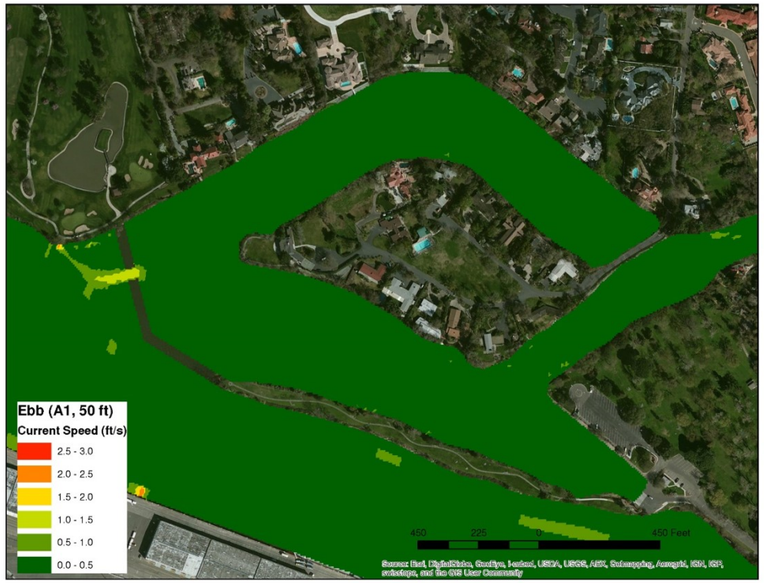
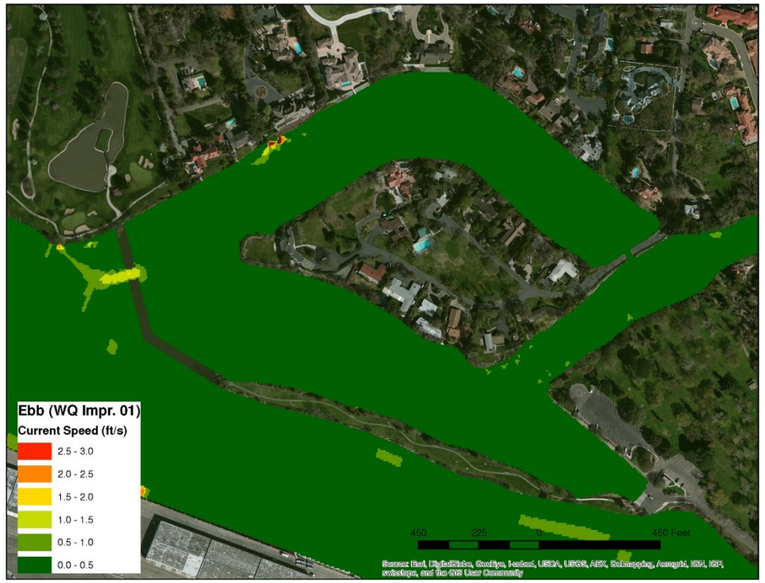
Water Quality Improvement Option 1 during the ebb tide:	WATER QUALITY IMPROVEMENT OPTION 1 - ALIGNMENT 1 (50 FT GATE)
	
ALIGNMENT 1 (50 FT GATE)	WATER QUALITY IMPROVEMENT OPTION 1

Figure B-2: Alignment 1 (50 ft Gate) and Water Quality Improvement Option 1 Ebb Tide Velocity Difference

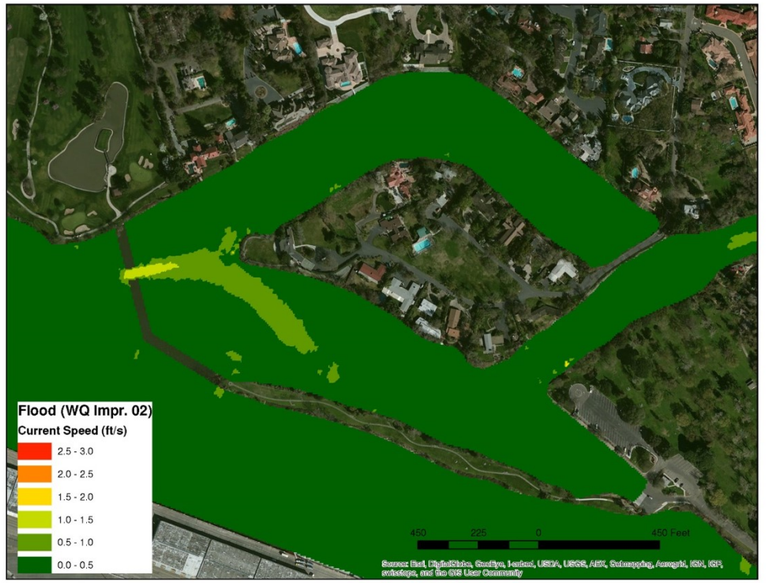
Water Quality Improvement Option 2 during the flood tide:	WATER QUALITY IMPROVEMENT OPTION 2 - ALIGNMENT 1 (50 FT GATE)
	
ALIGNMENT 1 (50 FT GATE)	WATER QUALITY IMPROVEMENT OPTION 2

Figure B-3: Alignment 1 (50 ft Gate) and Water Quality Improvement Option 2 Flood Tide Velocity Difference

Water Quality Improvement Option 2 during the ebb tide:

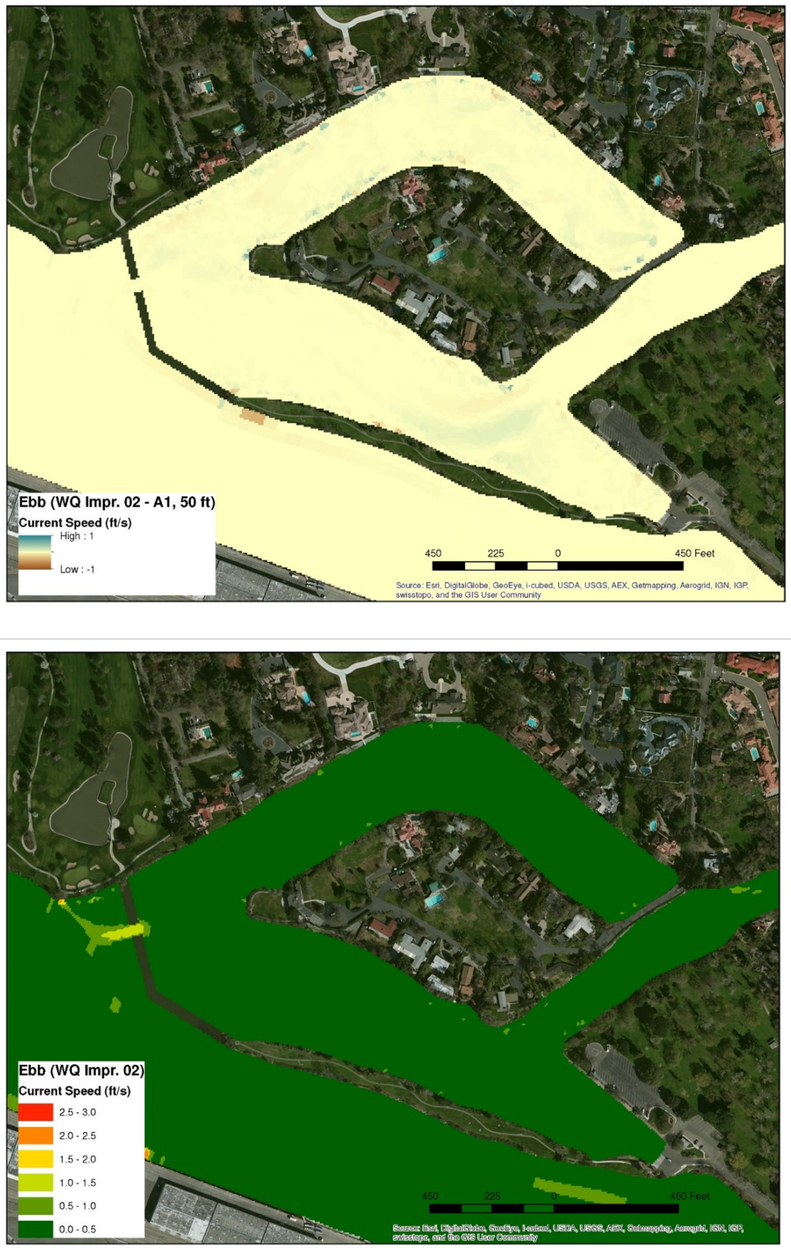
WATER QUALITY IMPROVEMENT OPTION 2 - ALIGNMENT 1 (50 FT GATE)	
	
ALIGNMENT 1 (50 FT GATE)	WATER QUALITY IMPROVEMENT OPTION 2

Figure B-4: Alignment 1 (50 ft Gate) and Water Quality Improvement Option 2 Ebb Tide Velocity Difference

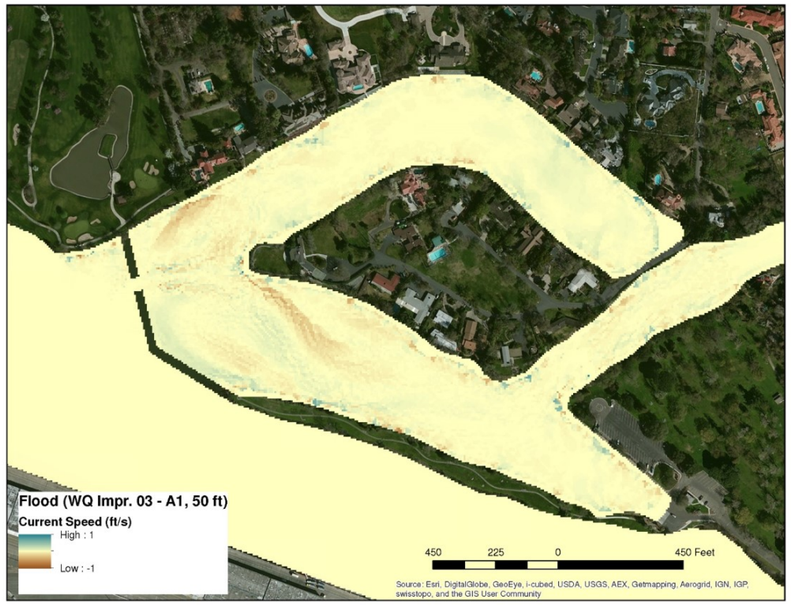
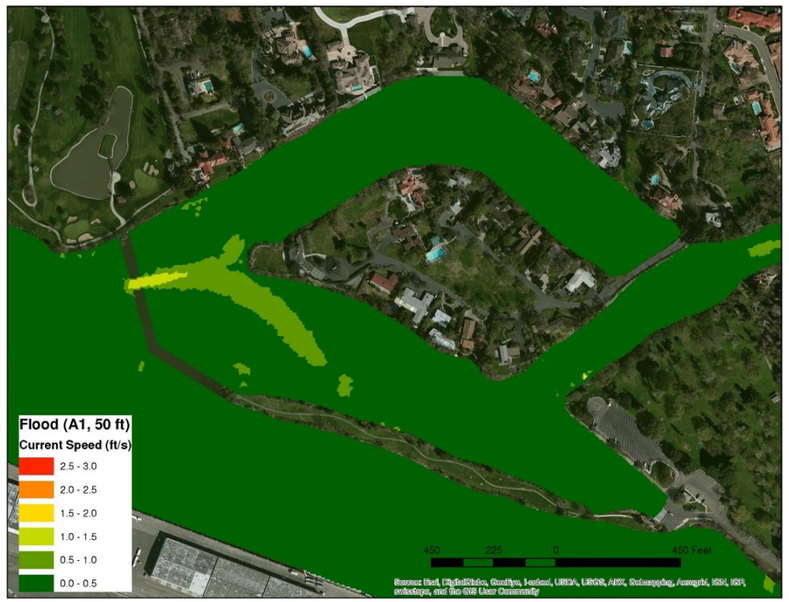
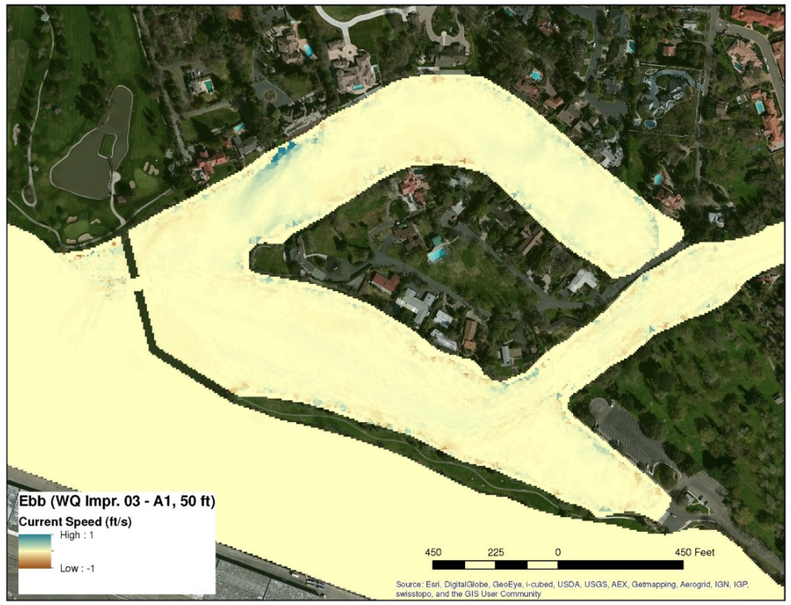
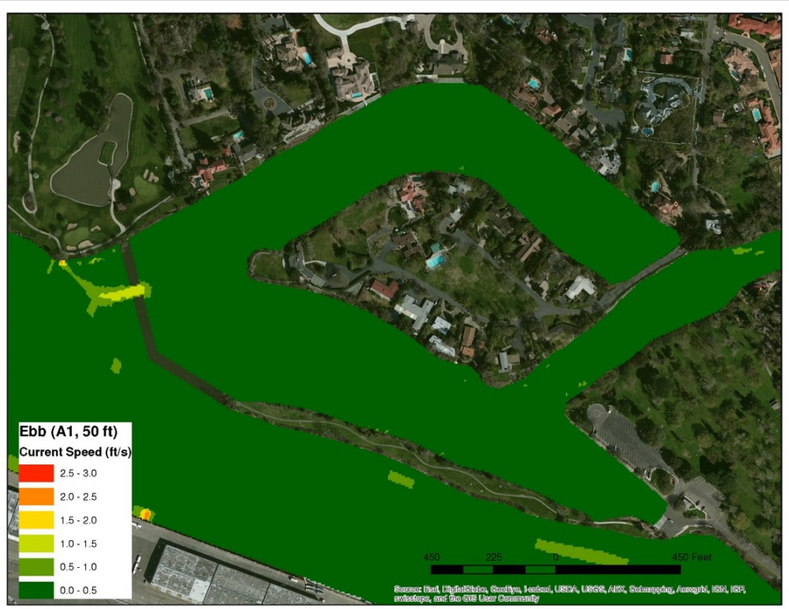
Water Quality Improvement Option 3 during the flood tide:	WATER QUALITY IMPROVEMENT OPTION 3 - ALIGNMENT 1 (50 FT GATE)
	
	
ALIGNMENT 1 (50 FT GATE)	WATER QUALITY IMPROVEMENT OPTION 3

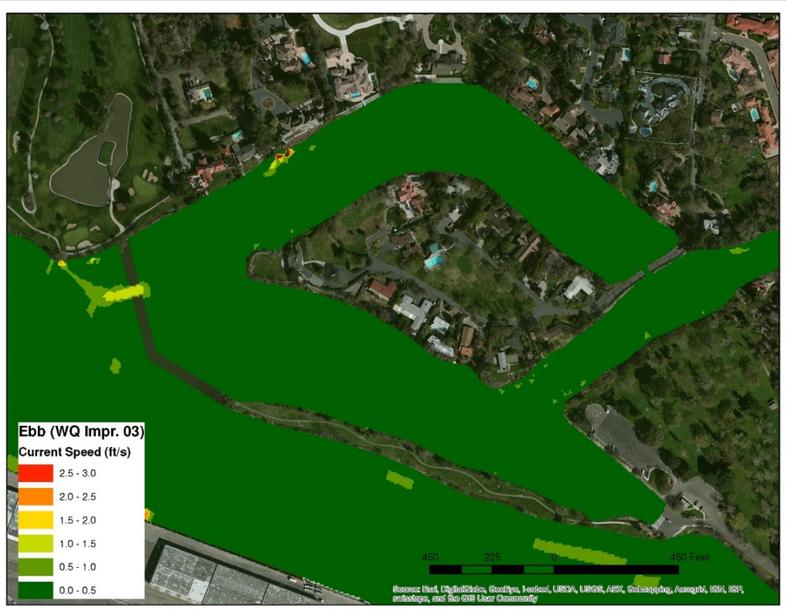
Figure B-5:Alignment 1 (50 ft Gate) and Water Quality Improvement Option 3 Flood Tide Velocity Difference

Water Quality Improvement Option 3 during the ebb tide:

WATER QUALITY IMPROVEMENT OPTION 3 - ALIGNMENT 1 (50 FT GATE)







ALIGNMENT 1 (50 FT GATE) | WATER QUALITY IMPROVEMENT OPTION 3

Figure B-6: Alignment 1 (50 ft Gate) and Water Quality Improvement Option 3 Ebb Tide Velocity Difference

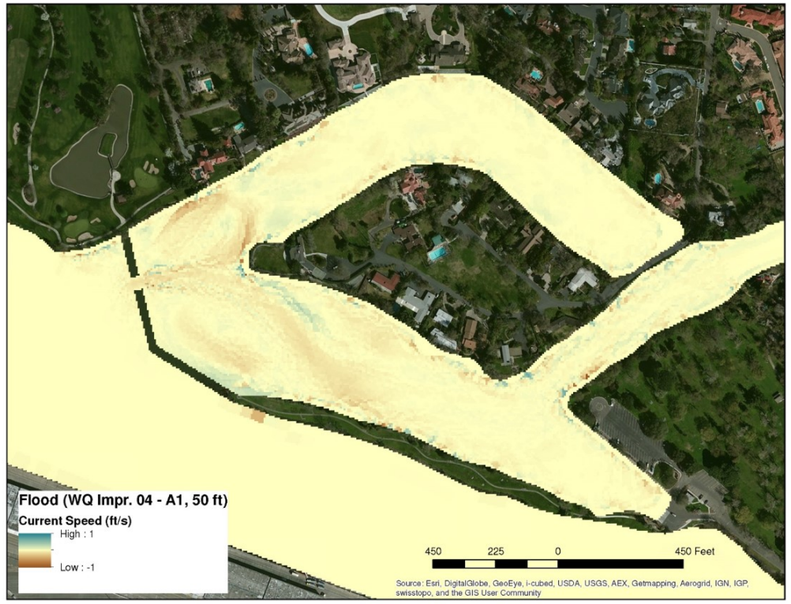
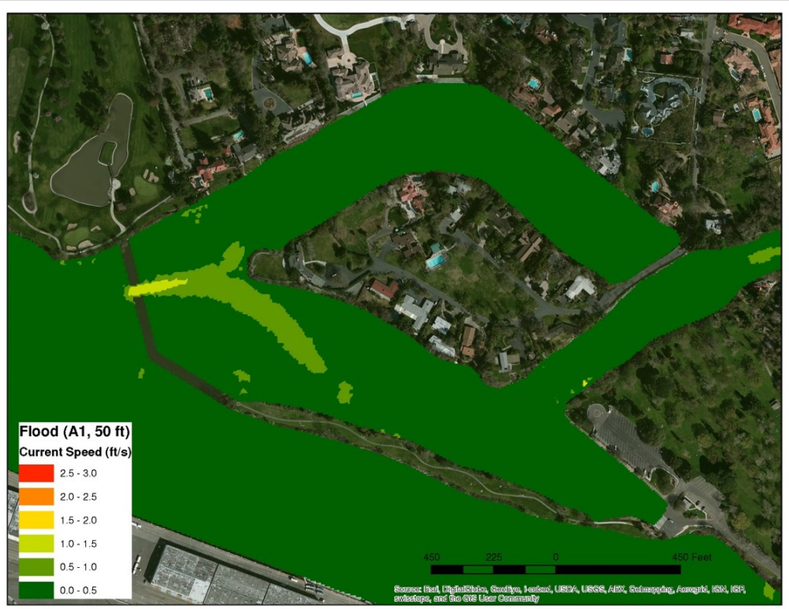
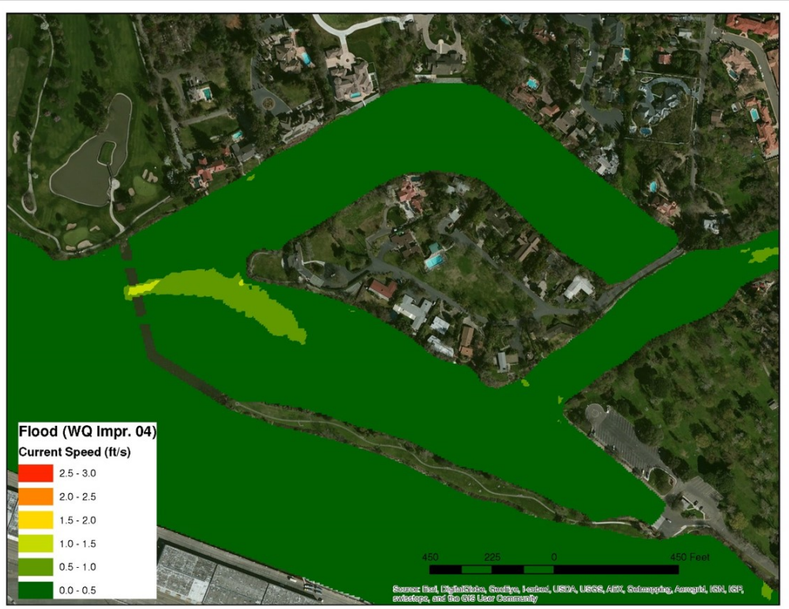
Water Quality Improvement Option 4 during the flood tide:	WATER QUALITY IMPROVEMENT OPTION 4 - ALIGNMENT 1 (50 FT GATE)
	
	
ALIGNMENT 1 (50 FT GATE)	WATER QUALITY IMPROVEMENT OPTION 4

Figure B-7:Alignment 1 (50 ft Gate) and Water Quality Improvement Option 4 Flood Tide Velocity Difference

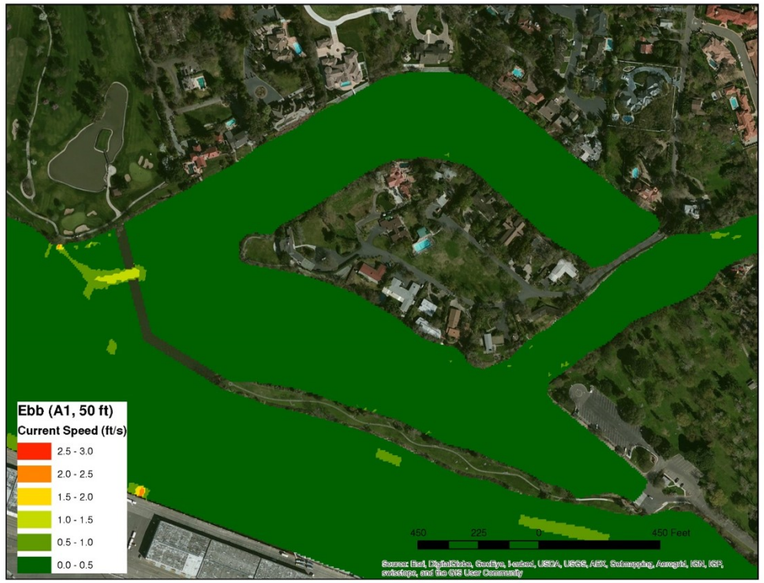
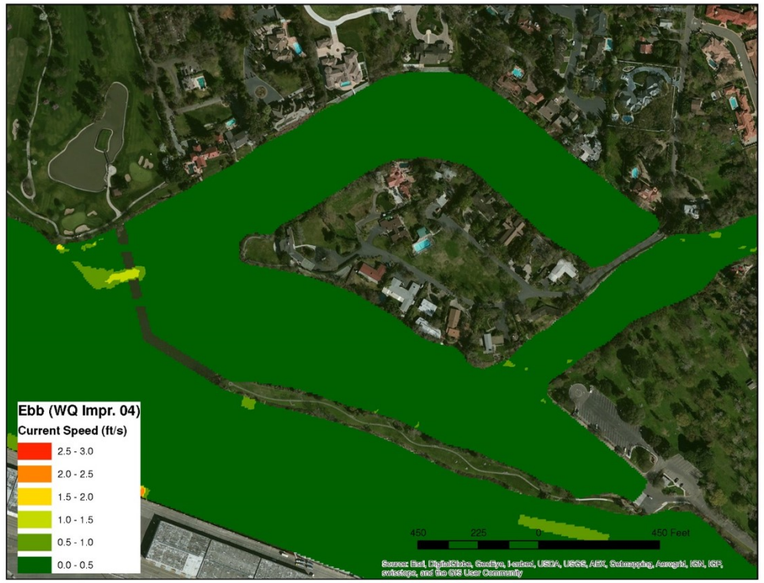
Water Quality Improvement Option 4 during the ebb tide:	WATER QUALITY IMPROVEMENT OPTION 4 - ALIGNMENT 1 (50 FT GATE)
	
ALIGNMENT 1 (50 FT GATE)	WATER QUALITY IMPROVEMENT OPTION 4

Figure B-8: Alignment 1 (50 ft Gate) and Water Quality Improvement Option 4 Ebb Tide Velocity Differenc