



**MARINA ADVISORY COMMITTEE  
REGULAR MEETING 5:00 PM**



**FEBRUARY 6, 2025**

**COMMITTEE MEMBERS**

Brenda Plechaty (Chair)  
Ken Wright (Co-Chair)  
Bob Lockhart  
James Kale  
Michael Pimentelli  
VACANT  
VACANT

**HYBRID MEETING**  
[www.Cityofvallejo.net](http://www.Cityofvallejo.net)

**Council Chambers  
555 Santa Clara Street  
Vallejo, CA 94590**

<p>NOTICE: Members of the Public will be able to participate in-person or remotely via Zoom</p>	<p>City Hall and the Council Chambers will be open to members of the public 30 minutes prior to the start of the meeting.</p>
<p><b>PUBLIC COMMENT: Members of the Public may provide public comments during the City Council Meeting in person or via ZOOM (<a href="https://ZoomRegular.Cityofvallejo.net">https://ZoomRegular.Cityofvallejo.net</a>), or via phone, by dialing (669) 900-6833.</b></p>	<p><b>For additional instructions on how to speak remotely during public comment, please visit, <a href="http://www.cityofvallejo.net/publiccomment">www.cityofvallejo.net/publiccomment</a></b></p>
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<p>Any supplemental writing related to an agenda item for an open session of a regular meeting that is distributed to all or a majority of all members of the City Council less than 72 hours before the meeting will be posted concurrently on the City’s website at <a href="http://www.cityofvallejo.net/agendas">www.cityofvallejo.net/agendas</a> Written material distributed during the meeting, will be available at the meeting if prepared by the City or after the meeting if prepared by someone else. Such materials may be obtained from the City Clerk</p>	
	<p>Vallejo City Council Chambers ADA compliant. Devices for the hearing impaired are available from the City Clerk. Requests for disability related modifications or accommodations, aids or services may be made by a person with a disability to the City Clerk’s office no less than 72 hours prior to the meeting as required by Section 202 of the Americans with Disabilities Act of 1990 and the federal rules and regulations adopted in implementation thereof</p>

## AGENDA

**1 CALL TO ORDER**

**2 PLEDGE OF ALLEGIANCE**

**3 ROLL CALL**

**4 CONSENT CALENDAR AND APPROVAL OF AGENDA**

**A APPROVAL OF AGENDA**

Recommendation: By Motion approve February 6th, 2025, REGULAR meeting agenda for Marina Advisory Committee.

Contact: Melissa Tigbao, Executive Secretary (707) 648-4085

[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)

**B APPROVAL OF MEETING MINUTES MAY 9, 2024**

Recommendation: By motion approve meeting minutes from May 9th, 2024, SPECIAL meeting of the Marina Advisory Committee.

Contact: Melissa Tigbao, Executive Secretary (707) 648-4085

[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)

**C APPROVAL OF MEETING MINUTES OCTOBER 17, 2024**

Recommendation: By motion approve meeting minutes from October 17, 2024, SPECIAL meeting of the Marina Advisory Committee.

Contact: Melissa Tigbao, Executive Secretary (707) 648-4085

[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)

**D APPROVAL OF MEETING MINUTES NOVEMBER 7, 2024**

Recommendation: By motion approve meeting minutes from November 7, 2024, REGULAR meeting of the Marina Advisory Committee.

Contact: Melissa Tigbao, Executive Secretary (707) 648-4085

[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)

**5 ANNOUNCEMENTS**

**A REPORT ON MARINA IMPROVEMENTS AND PROJECTS**

**B REPORT ON MARINA SECURITY AND INCIDENTS**

**C REPORT ON MARINA OCCUPANCY, INSURANCE AND REGISTRATION**

**6 ACTION CALENDAR**

**A APPROVE SCHEDULE OF REGULAR MEETING DATES IN 2025 - FEBRUARY 6, MAY 8, AUGUST 7, NOVEMBER 6, AND FEBRUARY 6, 2026.**

Recommendation: Approve schedule of regular meeting dates in 2025 - February 6, May 8, August 7, November 6, and February 6, 2026.

Contact: Melissa Tigbao, Executive Secretary (707) 648-4085

[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)

**B DISCUSSION OF JANUARY 2025 DRAFT SEDIMENTATION STUDY REPORT BY FOTH & VAN DYKE**

Recommendation: Discussion of January

2025 Draft Sedimentation Study Report by Foth & Van Dyke

Contact: Melissa Tigbao, Executive Secretary (707) 648-4085

[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)

**7 NEXT MEETING DATE**

**A NEXT REGULAR MEETING DATE SHALL BE HELD ON MAY 8, 2025**

**8 REPORT OF THE CITY COUNCIL LIAISON**

**9 REPORT OF THE CHAIRPERSON AND MEMBERS OF THE COMMITTEE/BOARD**

**10 REPORT OF THE COMMITTEE/BOARD SECRETARY**

**11 FUTURE AGENDA ITEMS**

**12 ADJOURNMENT**

**ADDITIONAL CITY INFORMATION**

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I, Melissa Tigbao, Marina Advisory Committee Executive Secretary do hereby certify that I have caused a true copy of the above notice and agenda to be delivered to

Brenda Plechaty (Chair)  
Ken Wright (Co-Chair)  
Bob Lockhart  
James Kale  
Michael Pimentelli  
VACANT  
VACANT,

at the time and in the manner prescribed by law and that this agenda was posted at City Hall, 555 Santa Clara Street, CA at 5:00 P.M., January 31, 2025.

Dated: January 31, 2025

*Melissa Tigbao*

Marina Advisory Committee Executive Secretary

**MARINA ADVISORY COMMITTEE**  
**SPECIAL MEETING MINUTES**  
**May 9, 2024**

**1. CALL TO ORDER**

The meeting was called to order at 5:02pm

**2. PLEDGE OF ALLEGIANCE**

**3. ROLL CALL**

**Present:** Committee Chair Plechaty, Co-Chair Wright, Committee Members Lockhart, Pimentelli, and Kale.

**Absent:** Hildebrand

**Staff Present:** Public Works Director Tigbao, Attorney Zagaroli, Manager Travis Lund

**4. APPROVAL OF AGENDA**

A. Recommendation: By motion, approve the May 9, 2024, agenda

**ACTION:** *Approve agenda with correction of agenda title from “Regular Meeting” to “Special Meeting” Motioned by Lockhart, seconded by Co-Chair Wright. Motion carried unanimously by members present. (Absent – Hildebrand)*

**5. APPROVAL OF MEETING MINUTES**

A. Recommendation: By motion, approve the February 1, 2024 Meeting Minutes.

**ACTION:** *Motioned by Co-Chair Wright, seconded by Kale. Motion carried unanimously by members present. (Absent – Hildebrand)*

**6. INFORMATION/ACTIVITY REPORTS/ANNOUNCEMENTS**

Marina Manager Travis Lund provided reports

- A. Report on Marina Improvements and Projects
  - *Report received.*
- B. Report on Marina Security and Incidents
  - *Report received.*
- C. Report on Marina occupancy, insurance, and registration.
  - *Report received.*

**7. ACTION CALENDAR**

A. Receive an update from Foth & Van Dyke and Associates

*Wendy Rocha, working on Vallejo projects since 2009. Contracted by the City to complete a Sedimentation study. So far, Foth & Van Dyke has developed 6 alternatives with the goal to reduce sedimentation and deposition rates, reduce the frequency of dredging events, and improve efficiency of the Marina. 6 alternatives presented verbally*

*Still waiting for finalized data to complete modeling the 6 alternatives and potentially more.*

*Question and answer period.*

*A-Dock removal update – BCDC is citing the marina for the unusable dock. Foth submitted a proposal to provide services related to permitting, design, bid, and removal of the dock. PW bringing contract to Council in May.*

*Question and answer period.*

*Dredging – Permits have expired for maintenance dredging. Foth submitted proposal for preparation of permitting and coordination with regulatory agencies to get a 10-year permit approved.*

*Question and answer period.*

*Presentation to be continued at next meeting.*

B. Discuss Findings of Ad-Hoc Committee for jet ski parking locations  
*Attorney Zagaroli stated that the formation of the committee was not*

**8. CONFIRMATION OF NEXT MEETING DATE**

*Chair Plechaty announced the next regular meeting date of the committee will be held on August 1, 2024*

**9. REPORT OF THE CITY COUNCIL LIAISON – None.**

**10. REPORT OF CHAIRPERSON AND MEMBERS OF THE COMMITTEE – None.**

**11. REPORT FROM PUBLIC WORKS DIRECTOR, SECRETARY TO THE MARINA ADVISORY COMMITTEE – Reported that PWD and Travis Lund have been hosting tours during low tide at the marina, 4 have been hosted, and it has been informative for people. Tours included Council members, Senior Leadership, and city staff.**

**12. FUTURE AGENDA ITEMS**

A. Formation of the Ad-Hoc Committee to discuss jet ski access and parking

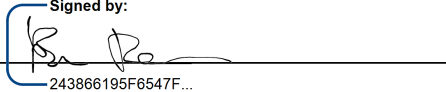
**ACTION:** *Motioned by Pimentelli, seconded by Chair Plechaty. Motion carried unanimously by members present. (Absent – Hildebrand)*

B. Update from Foth on their studies and work on Marina.

**13. ADJOURNMENT**

The meeting adjourned at 5:51 PM

Signed by:



A handwritten signature in blue ink, appearing to read 'Brenda Plechaty', is written over a horizontal line. To the left of the signature is a blue DocuSign signature box.

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**BRENDA PLECHATY, CHAIRPERSON**

**ATTEST:**

DocuSigned by:



A handwritten signature in blue ink, appearing to read 'Melissa L. Tigbao', is written over a horizontal line. To the left of the signature is a blue DocuSign signature box.

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**MELISSA TIGBAO, SECRETARY**

**MARINA ADVISORY COMMITTEE**  
**SPECIAL MEETING MINUTES**  
**October 17, 2024**

**1. CALL TO ORDER**

The meeting was called to order at 5:02 PM

**2. PLEDGE OF ALLEGIANCE**

**3. ROLL CALL**

**Present:** Committee Chair Plechaty, Co-Chair Wright, Committee Members Lockhart, Pimentelli, and Kale.

**Absent:** Marina General Manager Travis Lund

**Staff Present:** Public Works Director Tigbao, Attorney Zagaroli, Grants Coordinator Sidney Wilson

**4. APPROVAL OF AGENDA**

A. Recommendation: By motion, approve the October 17, 2024, agenda

**ACTION:** *Approve agenda. Motioned by Co-Chair Wright and seconded by Pimentelli. Motion carried and unanimously by members present.*

**5. INFORMATION/ACTIVITY REPORTS/ANNOUNCEMENTS**

Marina General Manager Travis Lund absent.

A. Report on Marina Improvements and Projects

- *No report received.*

B. Report on Marina Security and Incidents

- *No report received.*

C. Report on Marina occupancy, insurance, and registration.

- *No report received.*

**6. ACTION CALENDAR**

A. *Foth & Van Dyke presented Sedimentation Study; Sidney Wilson presented Seawall Rehab Grant scope of work; PWD Tigbao provided near, medium, and long term plans for the Marina.*

*Question and answer period*

*Committee requested more time to review the slide deck to formulate questions and bring back to next Committee Meeting.*

*Committee requested a follow up Special Meeting once Foth has received all the*

*data and completed full analysis.  
Committee supports utilizing the CNRA grant toward soft costs.*

B. Extend Jet-Ski Ad-Hoc Committee through first regular meeting in 2025.

**7. CONFIRMATION OF NEXT MEETING DATE**

*Chair Plechaty announced the next regular meeting date of the committee will be held on November 7, 2024*

**8. REPORT OF THE CITY COUNCIL LIAISON – None.**

**9. CHAIRPERSON'S AND COMMITTEE MEMBER'S REPORTS / AGENDA REQUESTS**

A. There are 2 vacancies in the Marina Advisory Committee, and would like to know when the positions will be advertised to be filled.

**10. REPORT FROM PUBLIC WORKS DIRECTOR, SECRETARY TO THE MARINA ADVISORY COMMITTEE –**


A. The MI Causeway Bridge is advertising for bid and the project is a maintenance project for the paint, railings, lighting, piers, but does not improve any mechanical issues on the lift. Project will run for approximately 15 months, and a presentation of the project will be given by the City Engineer at the October 22 Council Meeting.

**11. FUTURE AGENDA ITEMS**

**12. ADJOURNMENT**

The meeting adjourned at 6:24 PM

Signed by:




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BRENDA PLECHATY, CHAIRPERSON

ATTEST:

DocuSigned by:



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MELISSA TIGBAO, SECRETARY

**MARINA ADVISORY COMMITTEE**  
**REGULAR MEETING MINUTES**  
**November 7, 2024**

**1. CALL TO ORDER**

The meeting was called to order at 5:02 PM

**2. PLEDGE OF ALLEGIANCE**

**3. ROLL CALL**

**Present:** Committee Chair Plechaty, Co-Chair Wright, Committee Member Pimentelli.

**Absent:** Committee Members Lockhart and Kale

**Staff Present:** Public Works Director Tigbao, Attorney Zagaroli, Grants Coordinator Sidney Wilson, Marina General Manager Travis Lund

**4. LACK OF A QUORUM.**

A. Recommendation: Motion to adjourn meeting due to lack of a quorum.

**ACTION:** *Meeting was adjourned. Motioned by Chair Plechaty and seconded Pimentelli.*

**5. ADJOURNMENT**

The meeting adjourned at 5:06 PM

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BRENDA PLECHATY, CHAIRPERSON

ATTEST:

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MELISSA TIGBAO, EXECUTIVE SECRETARY



**DATE:** February 6, 2025  
**TO:** Marina Advisory Committee Chair & Committee Members  
**FROM:** Melissa Tigbao, Marina Advisory Committee Executive Secretary  
**SUBJECT:** **APPROVE SCHEDULE OF REGULAR MEETING DATES IN 2025 - FEBRUARY 6, MAY 8, AUGUST 7, NOVEMBER 6, AND FEBRUARY 6, 2026.**

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**RECOMMENDATION**

Approve schedule of regular meeting dates in 2025 - February 6, May 8, August 7, November 6, and February 6, 2026.

**BACKGROUND AND DISCUSSION**

Approve meeting dates for the committee.

**ATTACHMENTS**

None

**CONTACT**

Melissa Tigbao, Executive Secretary (707) 648-4085  
[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)



**DATE:** February 6, 2025  
**TO:** Marina Advisory Committee Chair & Committee Members  
**FROM:** Melissa Tigbao, Marina Advisory Committee Executive Secretary  
**SUBJECT:** **DISCUSSION OF JANUARY 2025 DRAFT SEDIMENTATION STUDY REPORT BY FOTH & VAN DYKE**

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**RECOMMENDATION**

Discussion of January 2025 Draft Sedimentation Study Report by Foth & Van Dyke

**BACKGROUND AND DISCUSSION**

Sedimentation study discussion by Foth & Van Dyke.

**ATTACHMENTS**

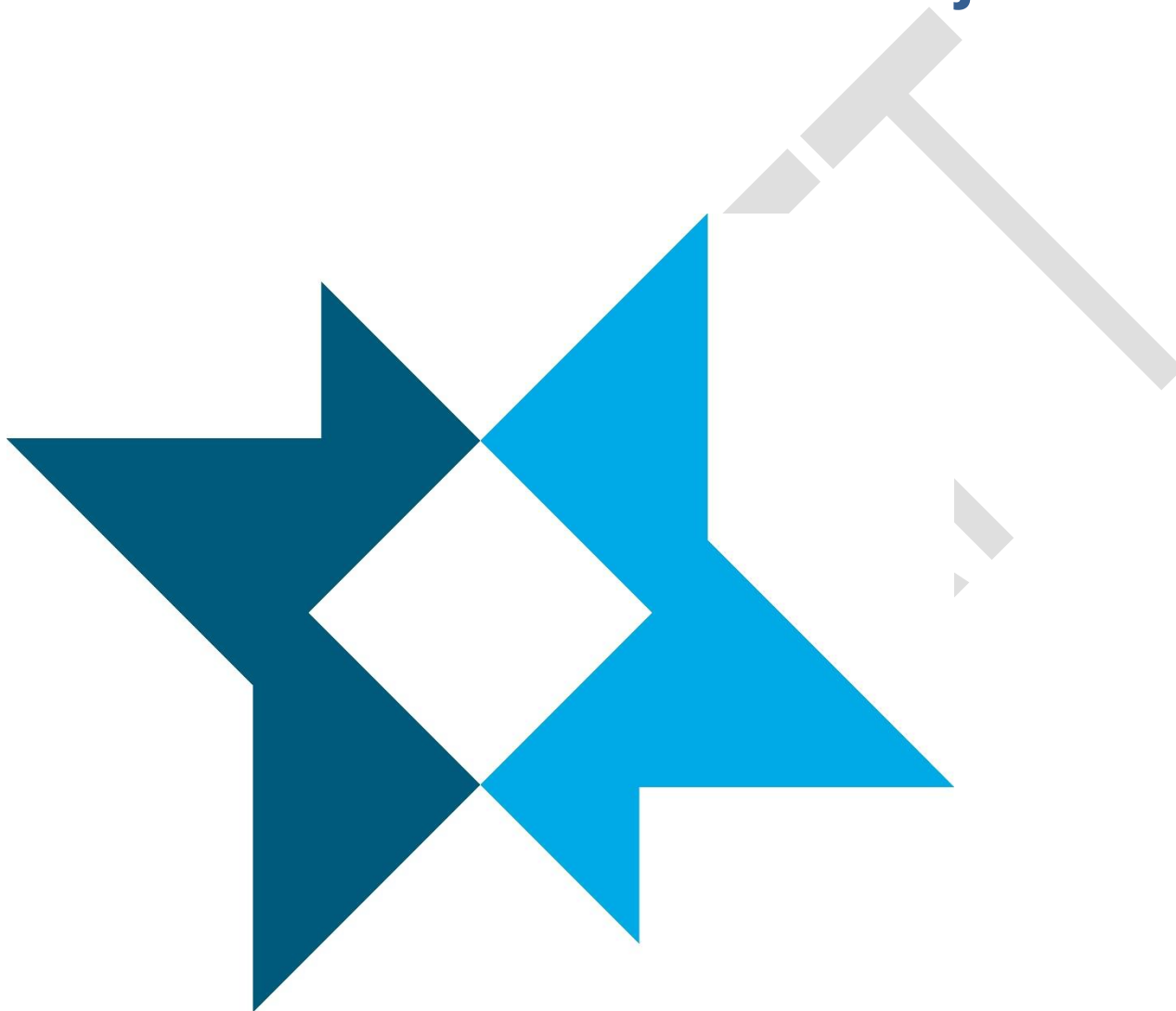
1. Vallejo Marina_Sedimentation Draft Study Report_January 2025
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**CONTACT**

Melissa Tigbao, Executive Secretary (707) 648-4085  
[Melissa.Tigbao@cityofvallejo.net](mailto:Melissa.Tigbao@cityofvallejo.net)



# Vallejo Marina Draft Project Study Report



**City of Vallejo, California**

January 2025

Project ID: 0023V013.00

**Solving our clients' toughest  
science and engineering challenges.**



755 Baywood Drive, Suite 228  
Petaluma, CA 94954  
(415)763-4274  
foth.com

January 16, 2025

Ms. Melissa Tigbao, P.E.  
Public Works Director  
City of Vallejo  
555 Santa Clara Street  
Vallejo, CA 94590

Re: Draft Vallejo Marina Project Study Report

Dear Ms. Tigbao:

Foth & Van Dyke and Associates, Inc. (Foth) is pleased to present the following Project Summary Report for the Vallejo Marina Sedimentation Study. Foth was tasked by the City of Vallejo to evaluate the Vallejo Marina and analyze the existing conditions, including sedimentation rates and operational concerns, and develop reconfiguration alternatives with the goal of reducing the frequency of dredge episodes and improving access within the Marina.

The following Project Summary Report discusses the purpose and goals of the project, reviews the potential for future dredge events, discusses the environmental impacts and the anticipated permitting processes, and also discusses the costs associated with each of the proposed alternatives.

Sincerely,

Foth & Van Dyke and Associates, Inc.

A handwritten signature in blue ink that reads "Wendy Rocha".

Wendy P. Rocha  
Senior Client Manager

Cody Flynn  
Project Engineer, PE  
Licensed in MA

Enclosure



# Vallejo Marina Sedimentation Study Report

Project ID: 0023V013.00

Prepared for  
**The City of Vallejo, California**  
555 Santa Clara Street  
Vallejo, CA 94590  
Prepared by

**Foth & Van Dyke and Associates, Inc.**

January 2025

#### **REUSE OF DOCUMENTS**

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# Vallejo Marina Sedimentation Study Report

## Table of Contents

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	Page
Executive Summary .....	iv
1. Existing Conditions .....	5
1.1 Overview .....	5
1.2 Historical Dredging .....	8
1.3 Project Description .....	10
1.4 Project Purpose .....	11
1.5 Hydrodynamic Model of the Strait.....	12
1.6 Sedimentation Study .....	13
2. Alternatives Analysis.....	13
2.1 Existing Marina Conditions .....	14
2.2 Alternative 1: Remove Center Wall .....	17
2.3 Alternative 2: Remove Center Wall, Add Entrance.....	17
2.4 Alternative 3: Remove Center Wall and Add Sump Areas.....	18
2.5 Alternative 4: Remove Center Wall, Add Mid Entrance & Sump Area .....	19
2.6 Alternative 5: Reduce Marina Size and Add Sump North of USCG .....	19
2.7 Alternative 6: Reduce Marina Size and Add Sump South of USCG .....	20
2.8 Alternative 7: Reduce Marina Size and Add Sump at Entrance.....	21
2.9 Alternative 8: Reduce Marina Size, Remove Middle Wall, Add Two Entrances ....	21
2.10 Alternative 9: Reduce Marina Size, Remove Middle Wall, Add Two Entrances and Two Sumps .....	22
2.11 Alternative 10: Reduce Marina Size, Remove Middle Wall, Add Two New Entrances and Remove Existing Entrances .....	23
2.12 Alternative 11: Reduce Marina Size, Remove Middle Wall, Add New Two Entrances, Remove Existing Entrances and Add Two Sumps.....	23
2.13 Deposition Rates.....	24
2.13.1 Phased Approach.....	25
2.13.2 Conceptual Alternative 12: Reduced Marina Size, 4 Total Entrances, 2 Channels .....	25
2.14 Alternatives Discussion and Further Recommendations .....	26
3. Regulatory Overview .....	27
3.1 Anticipated Permit Requirements .....	27

---

## Table of Contents (continued)

		Page
3.1.1	Bay Conservation and Development Commission (BCDC):.....	27
3.1.2	California Department of Fish and Wildlife (CDFW): .....	27
3.1.3	California State Land Commission (CSLC): .....	27
3.1.4	City of Vallejo.....	28
3.1.5	Consultations with National Marine Fisheries (NMFS) and US Fish and Wildlife Services (USFWS).....	28
3.1.6	Long Term Management Strategy (LTMS):.....	28
3.1.7	San Francisco Bay Water Quality Control Board (SFBWQCB): .....	29
3.1.8	U.S. Army Corps of Engineers (USACE):.....	29
3.2	Impact to Aquatic Resources .....	29
3.3	Endangered and Special Status Species.....	29
4.	Mitigation Measures .....	30
5.	Economic Analysis .....	31
5.1	Assumptions .....	31
5.2	Opinion of Probable Costs .....	32
5.3	Projected Dredging Costs .....	33
5.4	Constructability.....	35
6.	Stakeholder Input .....	36
6.1	Relevant Stakeholders and Interested Parties .....	36
7.	Evaluation of Alternatives .....	38
8.	Conclusion .....	38
10.	References.....	39

### Tables

Table 1: Vallejo Marina Historic Dredging Summary .....	10
Table 2: USCG Vallejo Station - Historic Dredging .....	10
Table 3: Deposition Rate Summary .....	24
Table 4: Listed Special Species Work Window.....	30
Table 5: Opinion of Probable Costs - Alternatives.....	32
Table 6: Opinion of Probable Costs – Phased Approach .....	33
Table 7: Dredge Return Frequency .....	34
Table 8: Projected Annual Dredge Costs .....	35
Table 9: Alternative Feature Comparison .....	38

## Figures

Figure 1: Vallejo Marina Project Location .....	5
Figure 2: Bay Trail Map near Project Site .....	5
Figure 3: Vallejo Marina Site Overview .....	6
Figure 4: Existing Condition of A-Dock .....	7
Figure 5: Sidescan Imagery of Existing Sheet Pile Wall (2017 Sidescan Survey) .....	8
Figure 6: Existing Conditions at Sheet Pile Wall .....	8
Figure 7: Typical Dredge Section .....	9
Figure 8: Existing Marina Layout .....	11
Figure 9: Sediment Accumulation in North Basin .....	12
Figure 10: SF Bay Bed Level .....	12
Figure 11: Bay Model bathymetry (NAVD88 m) and grid near Vallejo Marina study area. ....	13
Figure 12: Maximum (Left) and Average (Right) Sediment Velocity Model of Vallejo Marina .....	13
Figure 14: Existing Conditions Elevation Banding .....	16
Figure 15: Alternative 1 Layout .....	17
Figure 16: Alternative 2 Layout .....	18
Figure 18: Alternative 4 Layout .....	19
Figure 19: Alternative 5 Layout .....	20
Figure 20: Alternative 6 Layout .....	20
Figure 21: Alternative 7 Layout .....	21
Figure 22: Alternative 8 Layout .....	22
Figure 23: Alternative 9 Layout .....	22
Figure 22: Alternative 10 Layout .....	23
Figure 23: Alternative 11 Layout .....	23
Figure 23: Conceptual Alternative Layout (Note: Reorientation of North Basin Floats Not Shown) .....	26

## Appendices

Appendix A	Vallejo Marina Sedimentation Study, Dated July 18, 2024. Prepared by Integral Consulting, Inc.
Appendix B	Vallejo Marina Sedimentation Study Addendum, Dated October 22, 2024. Prepared by Integral Consulting, Inc.

# Vallejo Marina Sedimentation Study Report

## Executive Summary

---

Foth was contracted by the City of Vallejo to perform a study to reconfigure the Vallejo Marina located on the Mare Island Strait in Vallejo, California.

The Vallejo Marina (Marina) provides 621 slips for vessels from 20 to 110 feet length overall (LOA) and consists of two (2) basins, the North Basin and the South Basin. The North Basin contains covered slips and is home to the United States Coast Guard (USCG) Vallejo Station.

The Marina is subject to constant siltation which requires regular maintenance dredging. Currently, the accretion in the Marina is restricting access to slips and limiting activity within in the Marina. Additionally, the increase in siltation along Mare Island Strait has led to an accumulation of material along the backside of the existing sheet pile wall breakwater at the Marina, which has led to large amounts of stress in localized areas along the wall.

Seven (7) maintenance dredging events have been performed by the Vallejo Marina since 2001, with the most recent dredge event occurring in 2019. The USCG Vallejo Station has also conducted three (3) dredge events between 2008 and 2021 and is planning a maintenance dredging event for 2024. Previously, the Marina was permitted to be dredged to a depth of -10' MLLW plus 1' over dredge in the North Basin and to a depth of -8' MLLW plus 1' over dredge under the floats and to a depth of -10' plus 1' over dredge in the approaches and fairways in the South Basin. Dredge volumes averaged approximately 62,000 cubic yards of material removed per event, however the individual events do not represent a comprehensive dredge of the entire marina. A total volume of 432,500 cubic yards have been removed since 2001, through dredging efforts by the Vallejo Marina.

Foth has identified probable sources of sediment accretion and developed eleven (11) configuration alternatives, aiming to reduce frequency of maintenance dredging events by reducing the sedimentation rates within the Marina. The objectives of the reconfiguration alternatives also included maintaining a maximum current speed of 2 knots and analyzing sedimentation rates.

The following factors were considered for each of the proposed alternatives:

- Financial Implications;
- Sedimentation Rate;
- Environmental Impacts;
- Regulatory Agency Overview;
- Dredge Event Frequencies; and
- Stakeholder Review.

# 1. Existing Conditions

## 1.1 Overview

The Vallejo Marina, owned by the City of Vallejo and professionally managed by F3 Marina, is located on the Northeast shoreline of the Mare Island Strait, just southeast of the Mare Island Causeway Bridge at the north end of the Downtown Marina Waterfront.

The upland area of the Marina is bound by both the Mare Island Causeway and Mare Island Way. The San Francisco Bay Area Trail runs along the shoreline of the Marina. There are also multiple restaurants and Vallejo Boatworks, located within the upland area.



Figure 1: Vallejo Marina Project Location

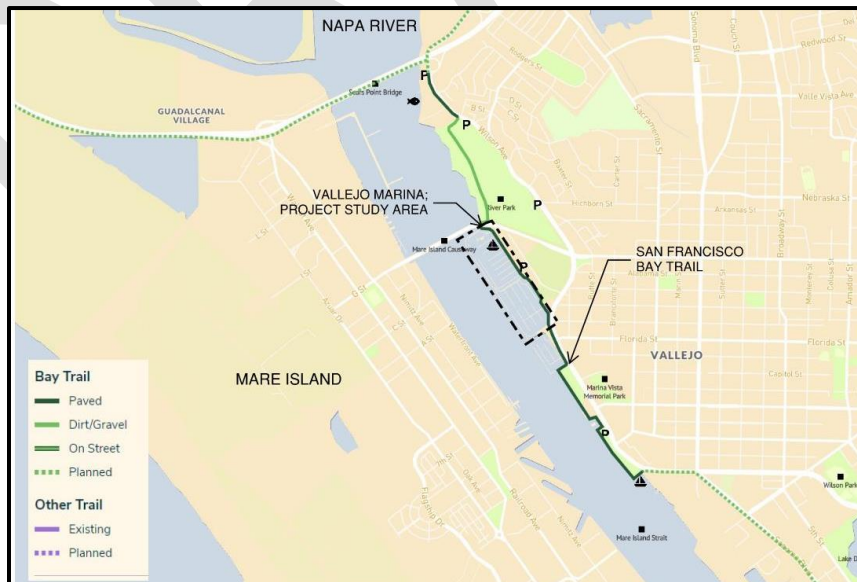


Figure 2: Bay Trail Map near Project Site

The Vallejo Marina provides 621 total slips for vessels from 20 to 110 feet length overall (LOA) and consists of two (2) basins, the North Basin and the South Basin. Covered slips are available inside the North Basin. The United States Coast Guard (USCG) Vallejo Station is located in the North Basin of the Marina as well.

The Marina is partially enclosed by an existing vinyl sheet pile wall, which extends for a length of approximately 2,600 linear feet around the perimeter of the Marina. The offshore face of this sheet pile wall is approximately 625 feet from the shoreline. The shoreline of the Marina consists of an armored rip rap shoreline.



Figure 3: Vallejo Marina Site Overview

The A-Dock is located on the north side of the North Basin and is currently in a severely deteriorated state. There is also significant accretion of sediment in the vicinity of the A-Dock, with areas of vegetation beginning to grow alongside the docks. Due to the deteriorated state that the A-Dock is in, each proposed alternative discussed throughout this report will include the removal of the docks and piles in this area.

Removal of the A-Dock will include the removal of at least fifty (50) timber piles and over 13,000 square feet of deteriorated timber floating docks.



Figure 4: Existing Condition of A-Dock

The existing Vinyl Breakwater structure, which is located along the perimeter of the Marina, was originally constructed in 1977. The sheet piles have experienced damage caused by winter storms, most notably in 2017, with the most severe damage of the wall located along the Northwest end. The breakwater structure is no longer functioning as designed in the North Basin and is acting to retain sediment inside the Marina. The original intent of the existing breakwater is not known, but it can be surmised that it was intended to provide protection from wakes, as well as to prevent silt accumulation within the marina. It is not known how deep the vinyl sheets originally extended, nor what the water depth was at the time of original construction.

While the current condition of the breakwater does provide protection from wakes, the reduced flow created by the breakwater also traps silt-laden tidal waters, effectively turning it into a very large settlement basin. Thus, the tidal exchange within the marina consists of a supply of turbid water in and out of the two entrances, causing a significant amount of sediment to accumulate within the marina basins. The result is that the reduced flow velocities allow for the suspended sediment to settle and create shoaling within the marina. The sedimentation rates vary depending on location, but the average rate is about one foot per year.

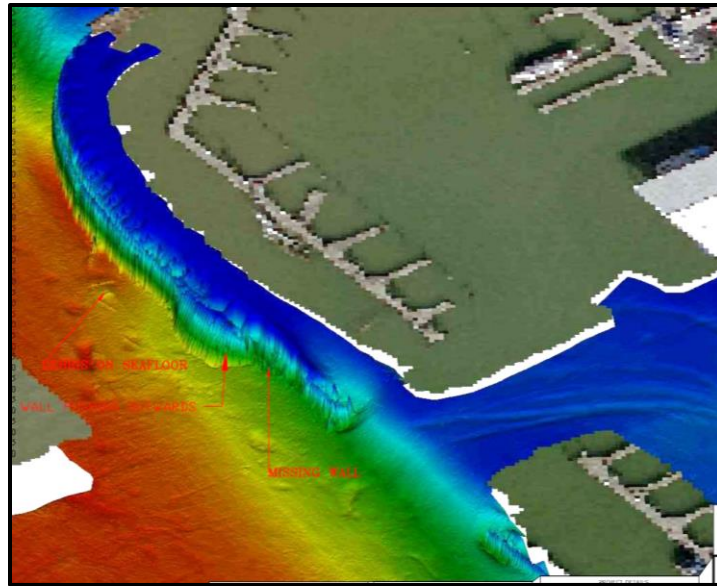


Figure 5: Sidescan Imagery of Existing Sheet Pile Wall (2017 Sidescan Survey)



Figure 6: Existing Conditions at Sheet Pile Wall

## 1.2 Historical Dredging

Seven (7) maintenance dredging events have been performed by the Vallejo Marina since 2001, with the most recent dredge event occurring in 2019. However, the frequency and location of dredge events has varied and lacked consistency within the Marina. The Marina had annual dredge events between 2001 and 2003, but no dredging occurred between 2003 and 2012. The inconsistency of historic events in the Marina makes it difficult to predict what dredge frequency and volumes are required to keep the Marina fully functioning. Additionally, the lack of consistent dredging in the same priority areas within the Marina has allowed for some areas of the Marina to become fully filled in with sediment and are rendered inaccessible.

The Marina was previously permitted to be dredged to a depth of -10' MLLW plus 1' over dredge in the North Basin and 8' MLLW plus 1' over dredge under the floats and to a depth of -10' plus 1' over dredge in the approaches and fairways in the South Basin. The average volume of material removed per the seven (7) dredge events conducted by the Marina between 2001 and 2019 is 62,000 cubic yards per event equaling

a total volume of approximately 432,500 cubic yards removed since 2001. It should be noted that the dredge events between 2001 and 2019 removed material from specific priority areas and that a comprehensive dredge event for the entire Marina did not occur during that time period.

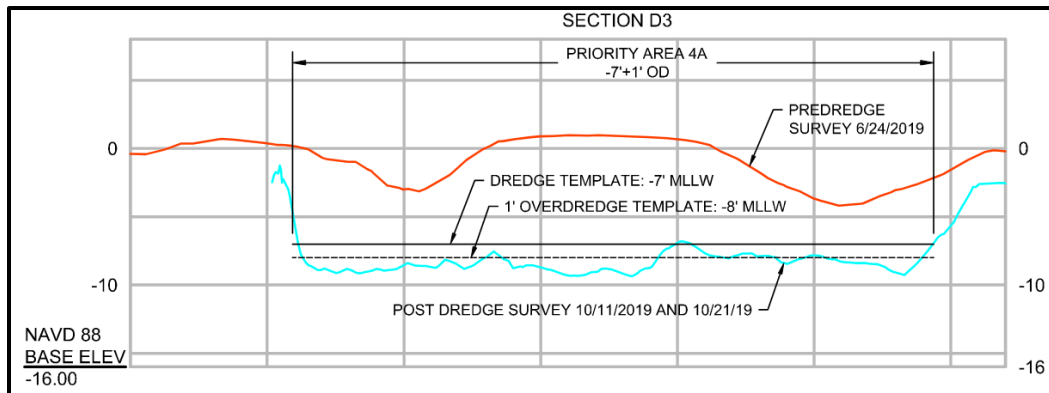


Figure 7: Typical Dredge Section

Prior to the 2019 dredge event, several areas within the Marina were shoaler than Mean Low Water and exposed during low tide events. The red line shown on Figure 7 above represents the existing grade in the Marina prior to dredging, and the blue line shows the post-dredge grade.

The USCG Vallejo Station has also conducted three (3) separate dredge events between 2008 and 2021 to provide access to their floating docks in the North Basin. Because Dredge events by the USCG were focused on providing access to their floats, these dredge events did not provide a significant benefit for slips in the Marina. Additionally, the USCG is planning on removal of approximately 20,000 cubic yards of material during the 2024 dredge season. Please see Tables 1 and 2 below for a summary of the Vallejo Marina Historic Dredging and the USCG Vallejo Station Historic Dredging, respectively.

**Table 1: Vallejo Marina Historic Dredging Summary**

Year	Areas	Dredge Depth (ft MLLW)	Volume Removed (cy)	Disposal Site
2019	North and South Basins	-10+1' OD	50,423	SF-9
2016	South Basin	-10+1' OD	13,125	SF-9
2014	North and South Basins	-10+1' OD	40,642	SF-9
2012	North and South Basins	-10.5+1' OD	61,539	SF-9
2003	South Basin	-10+1' OD	99,175	SF-9
2002	North and South Basins	-11+1' OD	76,980	SF-9
2001	North and South Basins	-11+1' OD	90,658	SF-9
<b>Total Volume Removed from 2001 to 2019 (cubic yards)</b>			<b>432,542</b>	
<b>Average Per Event (cubic yards / event)</b>			<b>61,792</b>	
<b>Total Volume per Year from 2001 to 2019 (cubic yards per year)</b>			<b>24,030</b>	

**Table 2: USCG Vallejo Station - Historic Dredging**

Year	Areas	Dredge Depth (ft MLLW)	Volume Removed (cy)	Disposal Site
2020-2021	USCG Access - North Basin	-10+2' OD	10,313	CRRP
2018	USCG Access/Fuel Dock - North Basin	-10+2' OD	15,600	SF-9 and CRRP
2008	USCG Access - North Basin	-10+2' OD	13,375	SF-9
<b>Total Volume Removed from 2008 to 2021 (cubic yards)</b>			<b>39,288</b>	
<b>Average Per Event (cubic yards / event)</b>			<b>13,096</b>	
<b>Total Volume per Year from 2001 to 2021 (cubic yards per year)</b>			<b>3,022</b>	

### 1.3 Project Description

Foth was tasked with assessing the sedimentation rates within the Marina and developing reconfiguration alternatives to ensure that the Vallejo Marina is suitable for the future. Foth subcontracted Integral Corporation, Inc. (Integral), to perform a sedimentation and accretion study of the site. Integral, along with Foth and Bay Marine Services (BMS), analyzed existing bathymetric survey data along with dredge records to develop estimates of sedimentation rates and approximate their special distribution.

Based on the results of the existing conditions modeling, Foth identified probable sources of sediment accretion and developed eleven (11) reconfiguration alternatives, aiming to reduce frequency of maintenance dredging events by reducing the sedimentation rates within the Marina.

Integral produced a sedimentation model of the site and analyzed the existing conditions along with each of the proposed alternatives to determine the anticipated accretion rates of each alternative, average current speed within the Marina, and the expected interval between dredge events.



Figure 8: Existing Marina Layout

## 1.4 Project Purpose

Due to a high sedimentation rate and lack of regular dredging, the water depths in the Vallejo Marina no longer allow for a sustainable level of access by patrons of the Marina. The purpose of the project was to use existing hydrographic/bathymetric survey data to conduct an in-depth analysis of the sediment accretion rates within the Marina and provide recommendations to assist the Marina in its goals of better serving the public through access to the navigable waters of San Francisco Bay through Mare Island Strait.

Each of the proposed alternatives was developed specifically to lower the sedimentation rates within the Marina, which will reduce the need for dredge events and allow for longer intervals of unimpeded access within the Marina. Due to large gaps in the sheet pile bulkhead along the perimeter of the Marina, sediment is currently depositing on the backside of the bulkhead during both the ebb and flood tides and the entire portion of the Marina adjacent to A-Dock has filled with sediment to nearly the Low Water Level. The center sheet pile wall, which separates the North and South Basins, also serves to trap sediment and lower the overall current in the Marina.

The proposed reconfiguration options provide potential layouts which will increase the tidal flow within Marina basins. Several of the reconfiguration options have also been designed to allow for continuous flow between the North and South Basin, which will prevent sediment from depositing in localized areas, as is occurring under the existing configuration.

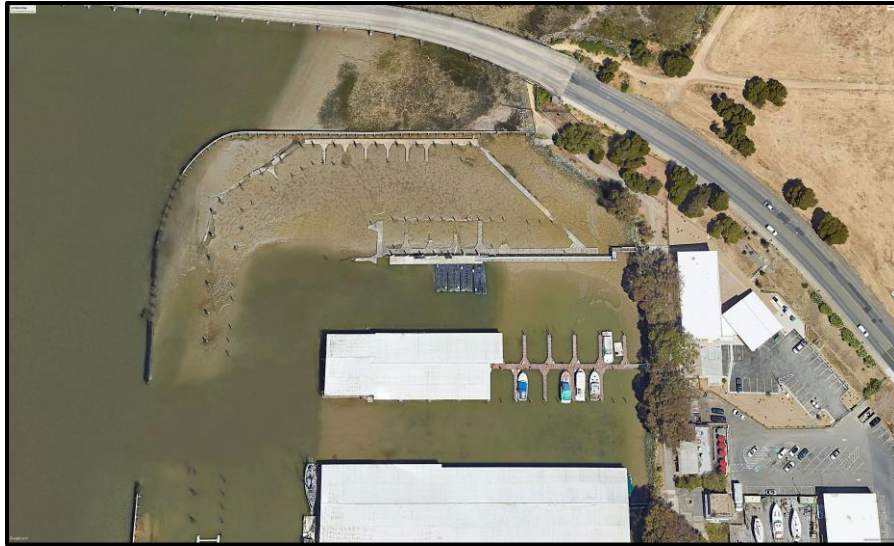


Figure 9: Sediment Accumulation in North Basin

## 1.5 Hydrodynamic Model of the Strait

With reference to creating a baseline condition, Foth contracted Integral Corporation to develop numerical models to investigate the potential for sedimentation in the existing pier configuration and to compare the eleven (11) proposed alternatives.

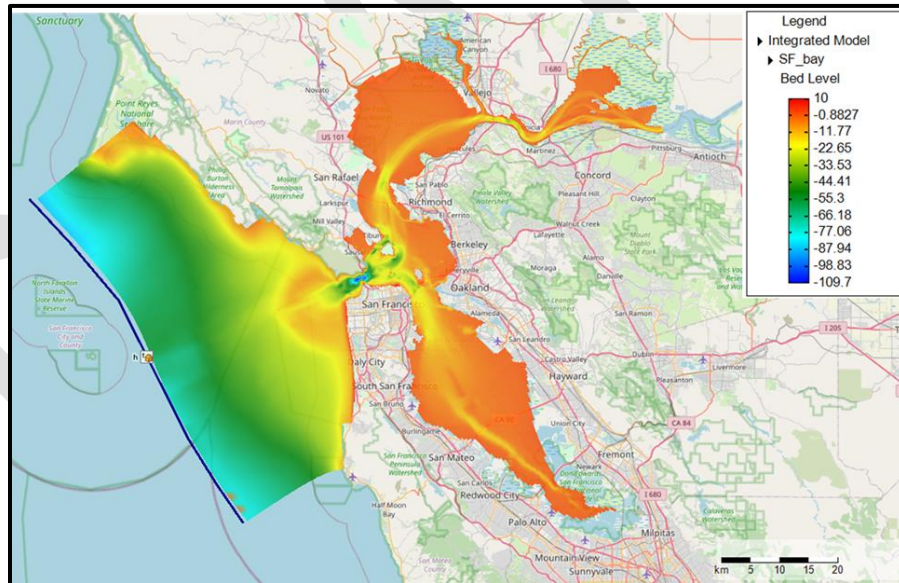


Figure 10: SF Bay Bed Level

The numerical model was used to predict bed shear stresses resulting from tidal currents within the Mare Island Strait and modified by the presence of the sheet pile wall around the Marina and the bridge piles and passage channel. The changes in shear stress patterns were compared with sedimentation rates, derived from a series of bathymetric surveys, to estimate sedimentation rates for each alternative.

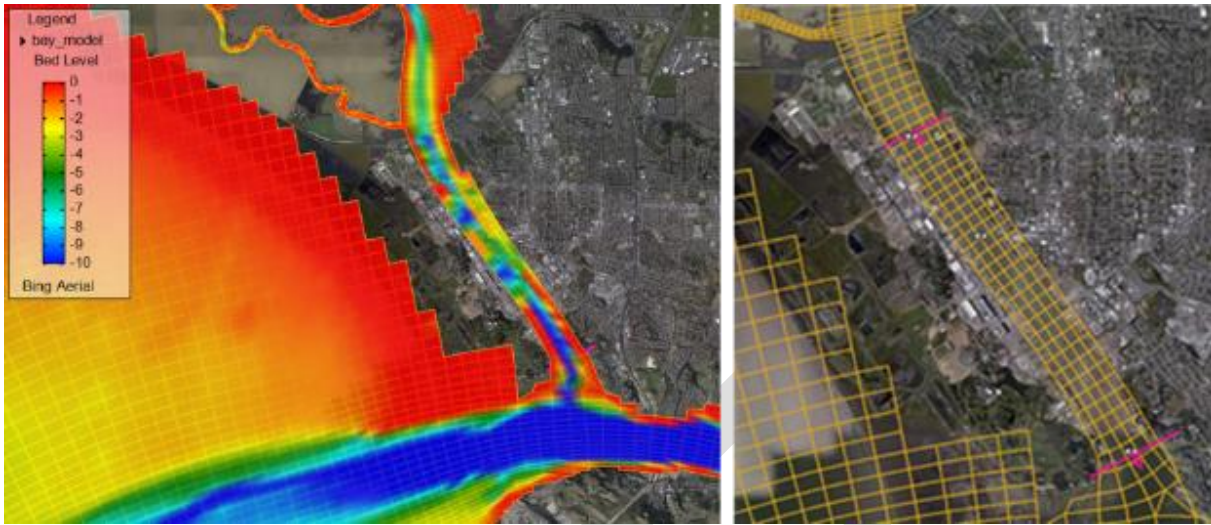


Figure 11: Bay Model bathymetry (NAVD88 m) and grid near Vallejo Marina study area. Pink Lines in right panel indicate locations of boundary conditions.

## 1.6 Sedimentation Study

The Sedimentation Study determined that the average velocity in Mare Island Strait is significantly higher than within the Marina. Maximum velocities in the Marina were isolated to areas adjacent to the entrances. The study also determined that areas with higher velocities correlated with areas of lower deposition. The velocity patterns suggest relatively low velocity within the Marina relative to channel. Maximum velocities indicate consistent eddy circulation, which prevents the sediment in the water column from depositing.

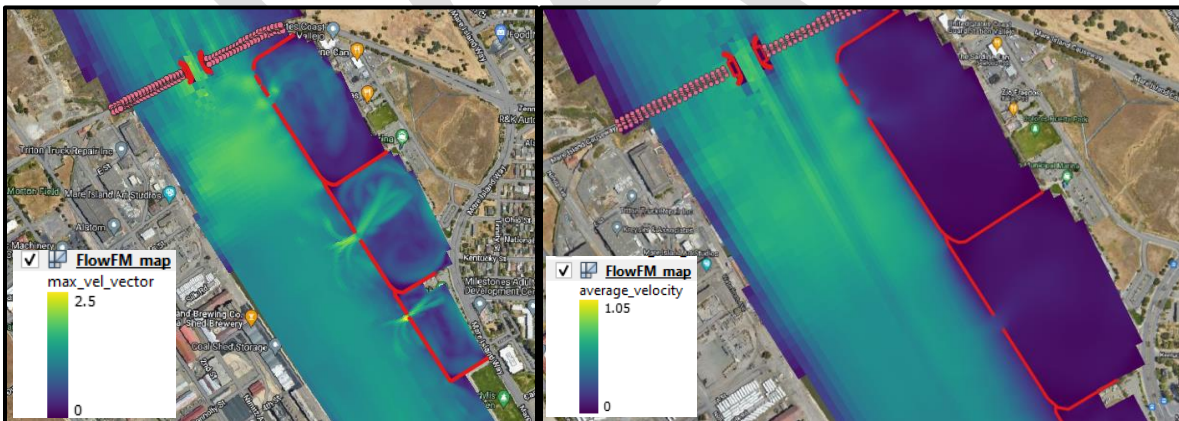


Figure 12: Maximum (Left) and Average (Right) Sediment Velocity Model of Vallejo Marina

In its current configuration, the mean deposition rate of sediment in the Marina in the dully-dredged state is approximately 0.99 ft/yr, averaged for both the North and South Basin. The mean deposition rates of the North and South Basin are approximately 0.98 and 1.02 feet per year, respectively.

## 2. Alternatives Analysis

Foth developed eleven (11) alternatives for the Marina reconfiguration with goals of minimizing the number of dredge events, extending the interval between dredge cycles, reducing the sedimentation rate in the Marina, improving efficiency within the Marina, and maintaining a current speed under two (2) knots. The

alternatives assume that the entire Marina is dredged to the permitted depth prior to construction of any alternative. Additionally, the A-Dock and associated piles are proposed to be removed prior to construction of each alternative.

Alternatives 1 and 2 were analyzed with geometric changes to the Marina, Alternatives 3 through 7 included geometric changes combined with sumps, intended to capture sediment in a concentrated area, and Alternatives 8 through 12 analyzed the Marina with added entrances which were oriented parallel to the direction of flow in Mare Island Strait to increase current flows within the Marina, both with and without sumps.

The sump areas would be dredged to a depth of 10' deeper than the design depth within the rest of the Marina and act to trap sediment which would otherwise deposit throughout the Marina. The sump areas were shown to have higher sediment accretion rates than the non-sump areas in the model, and therefore shown to be effective in capturing sediment which would have otherwise deposited in berthing areas and the Marina fairways. Additionally, the sumps offer a chance to provide long term cost savings by reducing the dredge frequency within the Marina and allowing for specific sump-only dredge events in the future. The cost for dredging the sump areas is anticipated to be lower than the cost for dredging in and around the Marina docks, as the sump configuration will allow for a more efficient dredge event, without digging around obstructions such as docks, piles, and other Marina infrastructure. Additional modeling, consisting of a sediment transport model, is recommended prior to finalization of the sump design to determine final dredge depth, size, and location.

Finally, it should be noted that the previously dredge permits authorize dredging to different depths within the Marina. Specifically, within the South Basin, the authorized dredge depth varies from -8' MLLW plus 1' overdredge underneath the docks to -10' MLLW + 1' overdredge in the approaches and fairways. It is recommended to amend all permits to allow for dredging to -10' MLLW + 1' overdepth throughout the Marina.

## **2.1 Existing Marina Conditions**

The sediment which has collected in the Marina since the previous dredge events, approximately 470,000 cubic yards, is currently restricting accessibility and maneuverability to the total 621 slips within the Marina. Even during high tide, only approximately 450 (72%) slips are accessible. At low tide, only approximately 65 (10%) of the Marina's slips are accessible, severely impacting the Marina's purpose of providing access to Mare Island Strait and the waters beyond. This limitation on access has had a negative effect on the public's perception of the Marina, as well as patronage.

Out of the 621 available slips, only approximately 235 (37%) are regularly occupied. The limited accessibility and maneuverability within the Marina is likely to make potential patrons wary of utilizing the Vallejo Marina as they may not be able to arrive and depart at their leisure, but rather are confined to specific tide windows. As shown in Figure 13 above, the high sedimentation rates are a concern throughout the Marina, with over 90% of the Marina inaccessible during low tides.

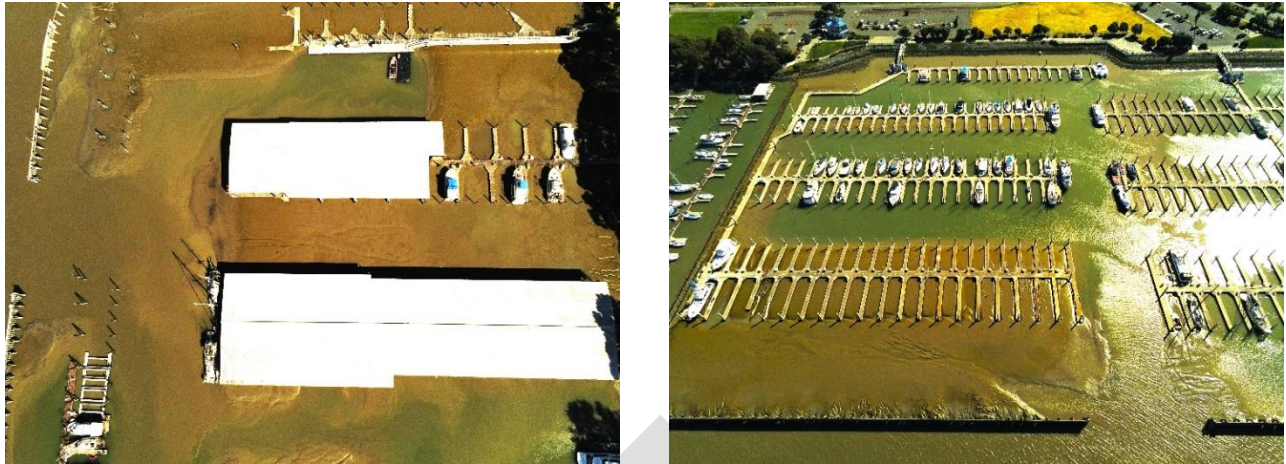


Figure 13: Drone Photo Showing Accumulated Sediment in North Basin (Left) and South Basin (Right)

The Northern basin of the Marina has fairways aligned in a perpendicular pattern to the tidal flow, however a 90-degree rotation of the fairways and boat slips will make it easier to keep the fairways clearer of sediment accretion. Fortunately, the Southern Basin already has the fairways aligned in the recommended direction of flow, so no significant changes there are needed. The recommendation, however, would be to align the northern floats so that they align with the southern floats to allow for straighter water flow through the Marina.

Foth also reviewed the most recent Marina survey data collected by Bay Mare Services from July of 2024 to determine the amount of sediment above the design dredge depth which is currently in the Marina. The analysis of this data yielded a total volume of approximately 410,000 cubic yards of sediment above the design depth and an additional 60,000 cubic yards of sediment within the 1-foot overdredge prism. The volumes analysis shows that there is a roughly even split of material in the North and South Basins, with the areas with the most sediment accumulating adjacent to the North breakwater structure and to the South of the middle dividing wall. An overview of this information is presented in Figure 14 below, with the orange/red color banding indicating higher elevations of sediment.

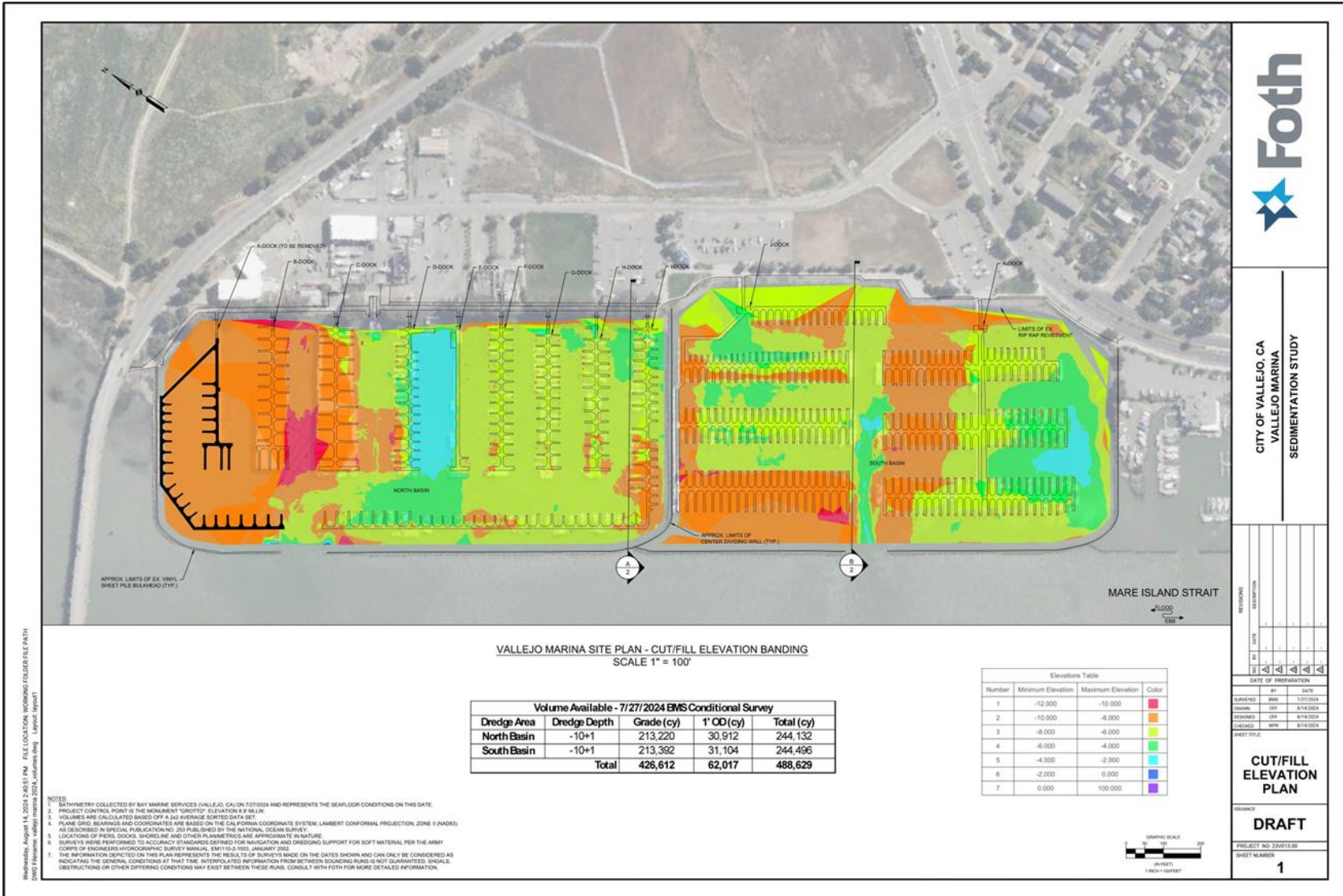


Figure 14: Existing Conditions Elevation Banding

## 2.2 Alternative 1: Remove Center Wall

Alternative 1 involves repositioning the North breakwater structure to be directly to the North of the USCG dock and removing the approximately 700 LF center wall between the North and South Basins. The removal of the center sheet pile wall was shown to allow for increased flow throughout Marina and result in lower deposition rates throughout compared to the existing configuration.

The removal of the center wall allows for a greater dispersion of current throughout the Marina, and North Basin was found to have a mean sediment deposition rate of approximately 0.78 feet per year and 0.47 feet per year in the South Basin. These results show that the removal of the center wall allows for a greater current flow throughout the Marina and overall lowers the deposition rates.

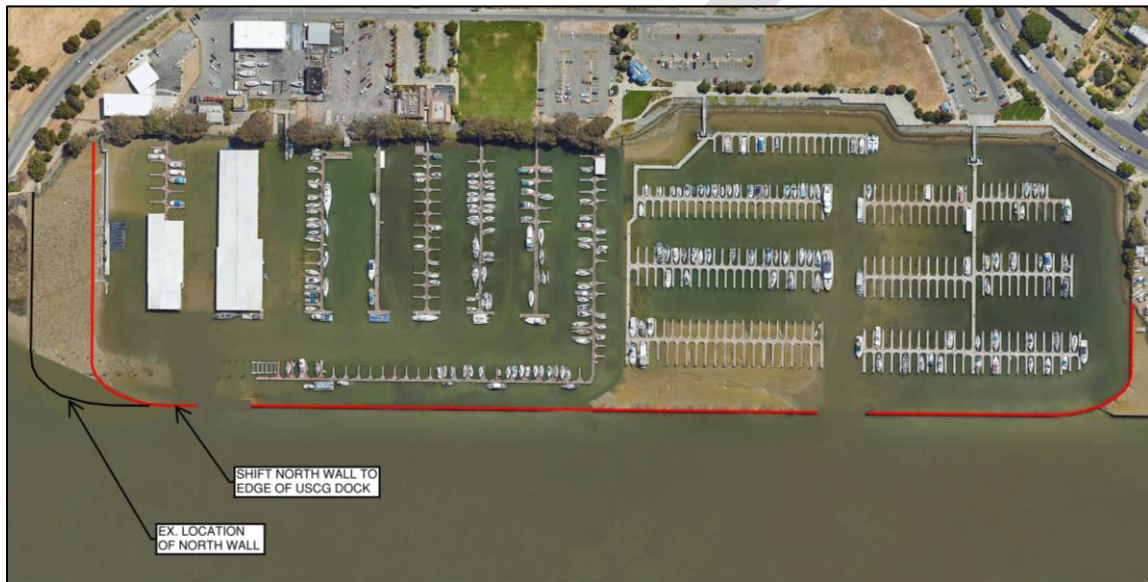


Figure 15: Alternative 1 Layout

## 2.3 Alternative 2: Remove Center Wall, Add Entrance

Alternative 2 is similar to the concept of Alternative 1 in that it involves shifting the North breakwater structure to be adjacent to the USCG dock and removing the approximately 700 LF center wall between the North and South Basins. However, a third entrance is added for this alternative which creates an additional inlet/outlet to increase the flow of water in the Marina. The model of Alternative 2 showed that current speeds increase in the area adjacent to the entrance, which reduces the likelihood of deposition in these areas.

The combination of the removal of the center wall and additional entrance allows for a greater dispersion of current throughout the Marina, with higher velocities adjacent to the new entrance. Deposition rates in the North Basin were found to be 0.82 feet per year and 0.60 feet per year in the South Basin. These deposition rates are slightly higher than the results of Alternative 1 however are still lower than the deposition rates in the current configuration.

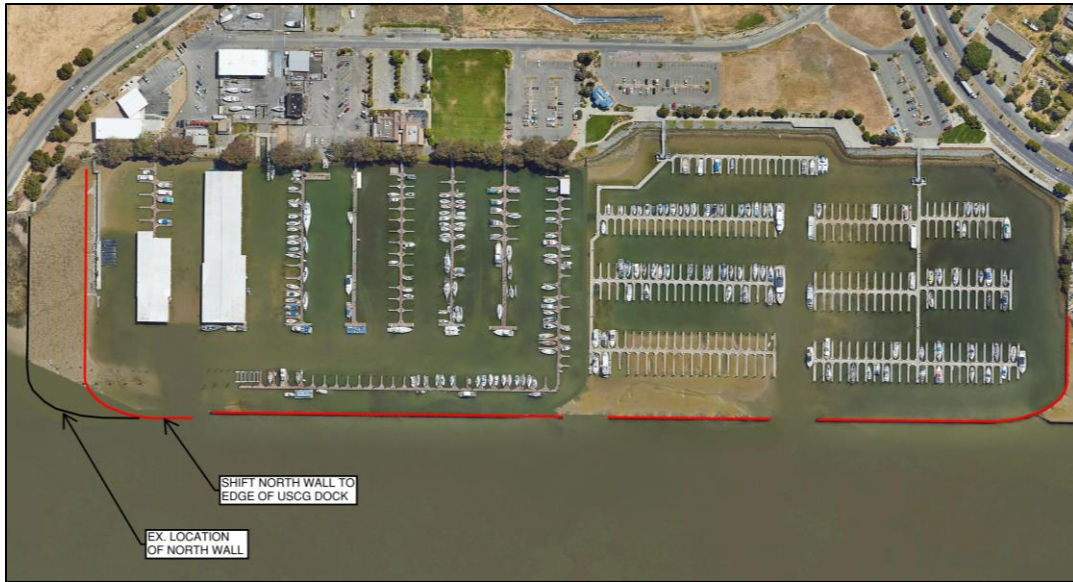


Figure 16: Alternative 2 Layout

## 2.4 Alternative 3: Remove Center Wall and Add Sump Areas

Alternative 3 involves shifting the North breakwater structure, removal of the center wall and the creation of a sacrificial sump area in the North Basin to be dredged to a depth of -20' (10' deeper than the adjacent dredge depth). The sump would be in the general vicinity of the USCG dock and would provide additional deep draft for the USCG in addition to acting as a trap for sediment in the Marina.

The removal of the center wall allows for a greater dispersion of current throughout the Marina, and the sump in the North Basin was found to have a mean sediment deposition rate of approximately 0.99 feet per year while lowering the deposition rates in the North and South Basins to 0.64 and 0.38 feet per year, respectively. These results show that the sump is effective in capturing sediment which would otherwise be dispersed throughout the Marina.

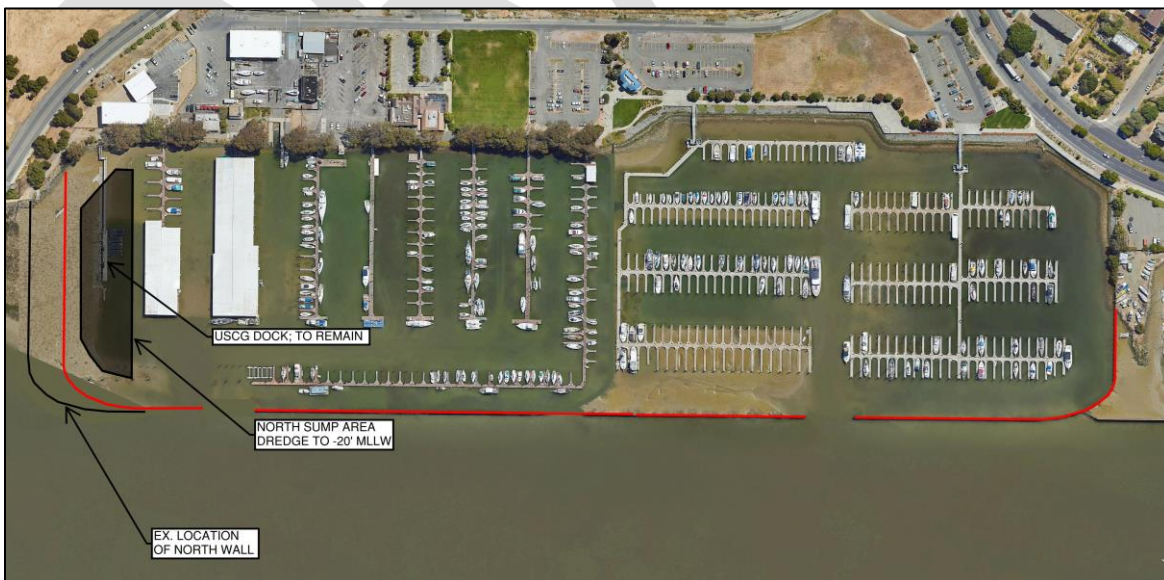


Figure 17: Alternative 3 Layout

## 2.5 Alternative 4: Remove Center Wall, Add Mid Entrance & Sump Area

Alternative 4 involves the removal of the center wall and the addition of a middle entrance as well as the creation of a sump area in the North Basin. Similar to Alternative 3, the sump would be dredged to a depth of -20', allowed to fill with sediment, and would reduce the dredge footprint during dredge events.

The combination of the removal of the center wall and the added entrance in the middle of the Marina allows for a greater dispersion of current throughout the Marina. The sump in the North Basin was found to have a mean sediment deposition rate of approximately 0.93 feet per year while lowering the deposition rates in the North and South Basins to 0.64 and 0.52 feet per year, respectively. These results show that the sump is effective in capturing sediment which would be otherwise dispersed throughout the Marina.

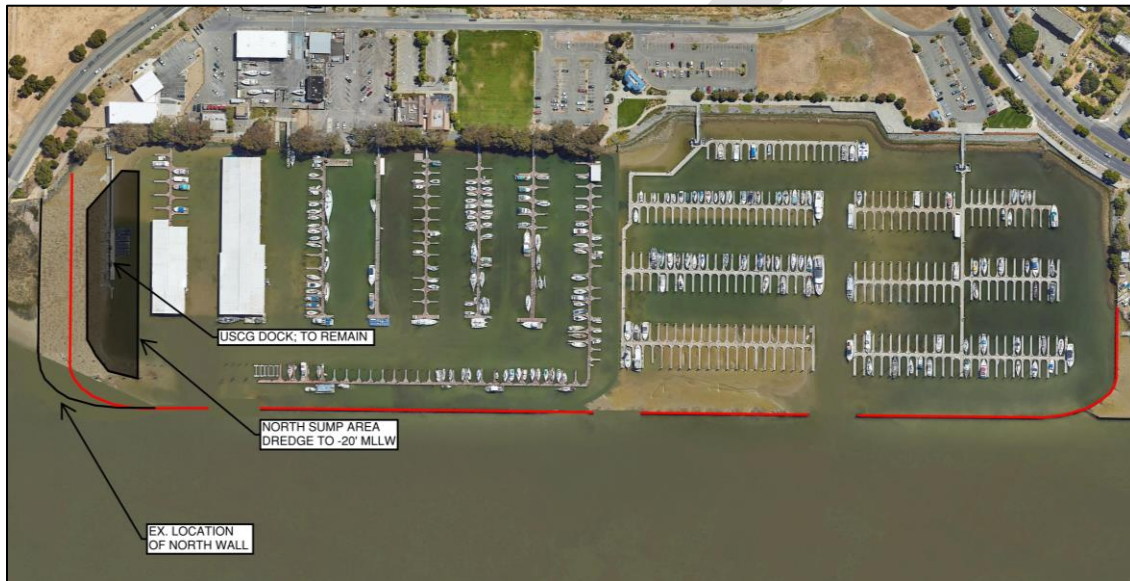


Figure 18: Alternative 4 Layout

## 2.6 Alternative 5: Reduce Marina Size and Add Sump North of USCG

Alternative 5 involves reducing the overall footprint of the Marina to a larger extent than Alternatives 1 through 4 by shifting the north wall by approximately 300 LF to the south. The two covered berthing areas on the north side of the Marina will be shifted to the south to accommodate these modifications. To facilitate access within the Marina and to provide an open location for future improvement options, the I-dock and H-Docks will be removed.

Two (2) sump locations will be incorporated as part of Alternative 5. The northern sump will be located adjacent to the relocated North breakwater structure and the center sump will be located in the open area created from the removal of the I and H docks. Deposition rates for this alternative were found to be an average of 0.98 feet per year in the north sump and 0.74 feet per year in the center sump while lowering the deposition rates in the North and South Basins to 0.32 and 0.49 feet per year, respectively. The downsizing of the Marina in this alternative will reduce the overall dredge footprint significantly and better allow the City and F3 to meet Marina needs and improve the quality of service and management for patrons of the Marina.

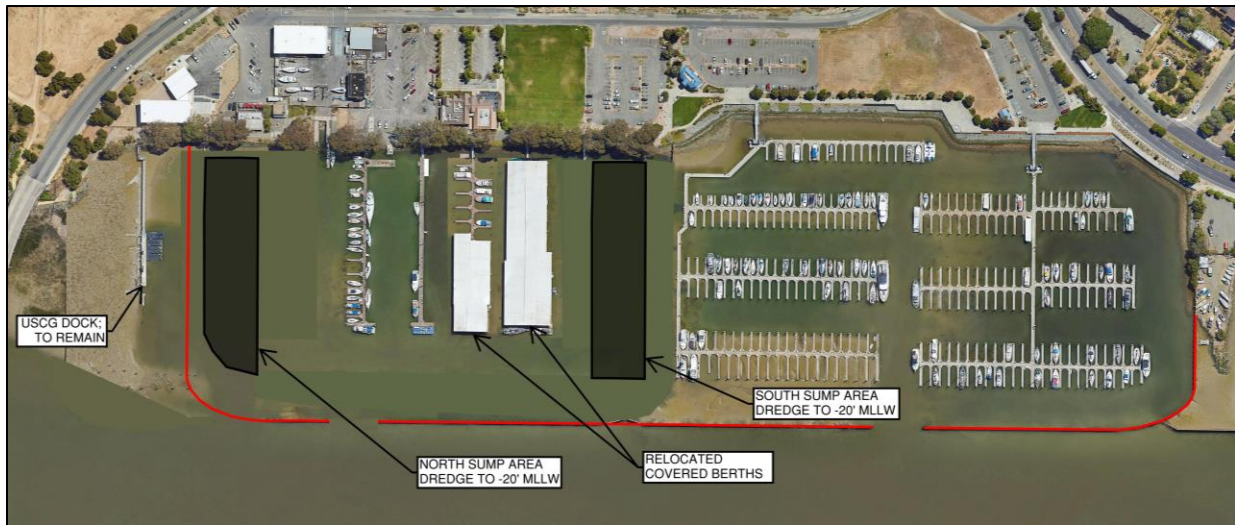


Figure 19: Alternative 5 Layout

## 2.7 Alternative 6: Reduce Marina Size and Add Sump South of USCG

Alternative 6 is similar in concept to Alternative 5, however it reduces the overall size of the Marina even further by shifting the location of the north wall by approximately 400 LF to the south. The covered berths sumps, and USCG dock will both be in the same locations as in Alternative 5. The location of this sump will provide benefit for Vallejo Boat Works by creating a deep-water access channel in their approach.

Deposition rates for this alternative were found to be an average of 1.18 feet per year in the north sump and 0.73 feet per year in the center sump. The deposition rate for the rest of the Marina was found to be 0.29 feet per year in the North Basin and 0.42 feet per year in the South Basin.

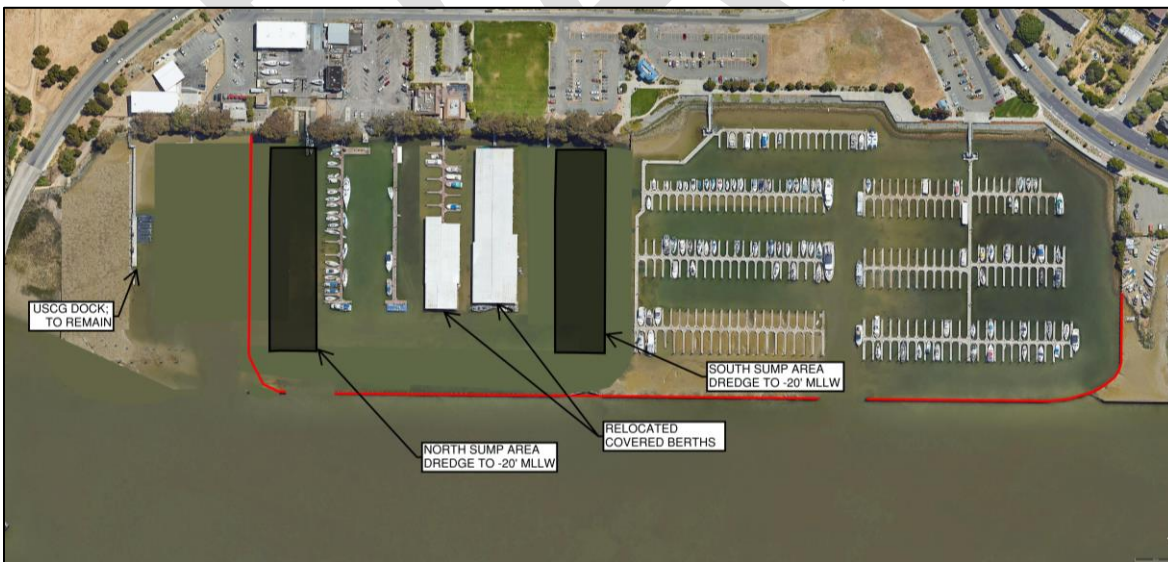


Figure 20: Alternative 6 Layout

## 2.8 Alternative 7: Reduce Marina Size and Add Sump at Entrance

Alternative 7 is identical to Alternative 6 but consists of orienting the north sump in the north/south direction, parallel to the direction of flow in the Marina. This orientation will provide the deeper sump area closer to the northern entrance to the Marina, providing a deep channel to access the reconfigured floating docks in the north basin.

The model showed that the north sump was not as effective in the north/south orientation as the east/west orientation of Alternative 6, however, the sumps still proved effective in lowering the overall deposition rate within the Marina. Deposition rates for this alternative were found to be an average of 1.01 feet per year in the north sump and 0.73 feet per year in the center sump. The deposition rate for the rest of the Marina was found to be 0.32 feet per year in the North Basin and 0.43 feet per year in the South Basin.

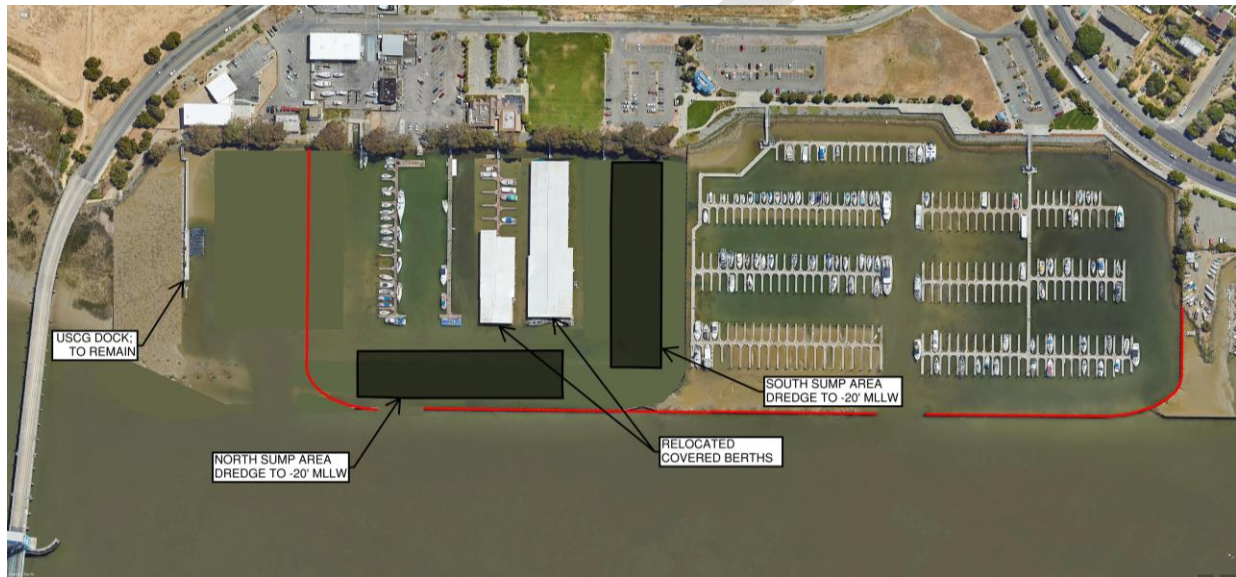


Figure 21: Alternative 7 Layout

## 2.9 Alternative 8: Reduce Marina Size, Remove Middle Wall, Add Two Entrances

Alternatives 8 through 11 were added to the analysis after the November meeting with the Marina Advisory Board and upon further review of Alternatives 1 through 7. Alternative 8 consists of moving the existing North wall towards the South by approximately 160 feet and adding an additional entrance in both the North and South Basins. The added entrances will be oriented to be parallel with the direction of current flow to facilitate tidal influence within the Marina.

Alternative 8 was found to result in an overall erosional deposition rate within the North Basin, which for the purposes of this report will be considered non-depositional on average. The deposition rate in the South Basin was found to be 0.55 feet per year. It should be noted that while the North Basin was found to be non-depositional on average, there will still be areas within the North Basin where sediment accumulates and dredging will eventually be required in isolated areas.



Figure 22: Alternative 8 Layout

## 2.10 Alternative 9: Reduce Marina Size, Remove Middle Wall, Add Two Entrances and Two Sumps

Alternative 9 is similar to Alternative 8, with the implementation of two sumps. The North Sump is located adjacent to the existing USCG dock in the North Basin and a Center Sump is located in the center of the Marina, oriented parallel to the direction of flow. This orientation was selected to facilitate current flow between the two basins.

Deposition rates for this alternative were found to be an average of 0.72 feet per year in the north sump and 0.70 feet per year in the center sump. Additionally, Alternative 9 was also found to result in an overall erosional deposition rate within the North Basin, which for the purposes of this report will be considered non-depositional on average. The deposition rate in the South Basin was found to be 0.44 feet per year for an overall marina deposition rate of 0.17 feet per year.



Figure 23: Alternative 9 Layout

## 2.11 Alternative 10: Reduce Marina Size, Remove Middle Wall, Add Two New Entrances and Remove Existing Entrances

Alternative 10 is similar to Alternative 8 and consists of moving the existing North wall towards the South by approximately 160 feet, adding an additional entrance in both the North and South Basins, and closing the existing entrances. The added entrances will be oriented to be parallel with the direction of current flow to facilitate tidal influence within the Marina.

Alternative 10 was found to result in an overall erosional deposition rate within the North Basin, which for the purposes of this report will be considered non-depositional on average. The deposition rate in the South Basin was found to be 0.63 feet per year.



Figure 24: Alternative 10 Layout

## 2.12 Alternative 11: Reduce Marina Size, Remove Middle Wall, Add New Two Entrances, Remove Existing Entrances and Add Two Sumps

Alternative 11 is similar to Alternative 10, with the implementation of two sumps. The North Sump is located adjacent to the existing USCG dock in the North Basin and a Center Sump is located in the center of the Marina, oriented parallel to the direction of flow. This orientation was selected to facilitate current flow between the two basins.

Deposition rates for this alternative were found to be an average of 0.79 feet per year in the north sump and 0.85 feet per year in the center sump. Additionally, Alternative 11 was also found to result in an overall erosional deposition rate within the North Basin, which for the purposes of this report will be considered non-depositional on average. The deposition rate in the South Basin was found to be 0.51 feet per year.



Figure 25: Alternative 11 Layout

## 2.13 Deposition Rates

Table 3 below provides a summary of the deposition rates for the existing conditions and each alternative based on the post-dredge model conditions.

**Table 3: Deposition Rate Summary**

Configuration	Description	Deposition Rate (ft/year)				
		Mean (Entire Marina)	North Basin	South Basin	North Sump	Center Sump
Existing Conditions	Baseline	0.99	0.95	1.02	-	-
Alternative 1	Remove Middle Wall	0.63	0.78	0.47	-	-
Alternative 2	Remove Middle Wall, Add Entrance	0.74	0.82	0.60	-	-
Alternative 3	Remove Middle Wall, Add N. Sump	0.51	0.64	0.38	0.99	-
Alternative 4	Remove Middle Wall, Add Mid Entrance & N. Sump	0.58	0.64	0.52	0.93	-
Alternative 5	Move N. Wall South, Add N. Sump	0.41	0.32	0.49	0.98	0.74
Alternative 6	Move N. Wall South, Add Sump South of USCG	0.35	0.29	0.42	1.18	0.73
Alternative 7	Same as #6, N. sump parallel to current	0.38	0.32	0.43	1.01	0.73
Alternative 8	Move N. Wall South, Remove Middle Wall, Add 2 Entrances	0.26	0 <sup>A</sup>	0.55	-	-
Alternative 9	Same as #8, Add N. and Center Sumps	0.17	0 <sup>A</sup>	0.44	0.72	0.70
Alternative 10	Move N. Wall South, Close Ex. Entrances, Add 2 New Entrances	0.24	0 <sup>A</sup>	0.63	-	-
Alternative 11	Same as #10, Add N. and Center Sumps	0	0 <sup>A</sup>	0.51	0.79	0.85

<sup>A</sup>: Erosional deposition rates from model are presented as 0 ft/yr accumulation, to remove influence of erosion from calculations of Marina as a whole.

Despite the average deposition rates for each of the Alternatives being significantly lower than the existing Marina configuration, these rates represent the average deposition rate across the entire Basin (North or South), and there will inevitably be some areas where the flow rates remain low.

Deposition rates are decreased by removing the dividing wall and adding the mid entrance (as applicable) in Alternatives 1 and 2. The addition of the sumps in Alternatives 3 through 7 caused a significant decrease in the mean deposition rates in the non-sump areas of the Marina. The overall intent of the sump areas is to capture sediment which would otherwise settle in other areas of the Marina. By capturing sediment in the sumps, that sediment is unavailable to distribute throughout the rest of the Marina, therefore lowering the deposition rates of the non-sump areas.

Alternatives 8 through 11 were found to have slightly erosional average deposition rates within the North Basin, which suggests that the added entrances are effective in increasing the flows throughout the Marina and reducing deposition rates. However, it should be noted that while some areas of the North Basin may experience erosion (deepening), there will be areas where sediment accretion continues and localized dredging will be required to address these areas when exceed allowable depths in the Marina.

### **2.13.1 Phased Approach**

Foth also evaluated a Phased Approach to the Marina reconfiguration which consists of two phases of work. The first phase would include the removal of the middle wall and dredging of the South Basin to the design depth. The second phase would then consist of dredging of the North Basin, including sumps, and any repairs/relocation of the North breakwater structure. For this approach to be most efficient, the two phases would be completed as close together as possible, as it is likely that the circulation patterns within the Marina will be influenced by the elevation differences between the fully dredged South Basin and the un-dredged North Basin, causing the Marina to act as though a middle wall was present.

A model of this phased approach was not developed, however, deposition rates were inferred from similar alternatives and expected circulation patterns. During the first phase of this approach, the deposition rates for the North and South Basins are expected to be 0.992 and 1.08 feet per year, respectively. The deposition rate for the North Basin is expected to be similar to the current rate. For the South Basin, the deposition rate is expected to be similar to the predicted rate for the fully dredged condition with the center wall in place. Upon completion of the second phase, it is expected that the deposition rates would decrease and reflect the rate of the selected alternative.

### **2.13.2 Conceptual Alternative 12: Reduced Marina Size, 4 Total Entrances, 2 Channels**

During the final review of the eleven (11) alternatives analyzed to date, and in an effort to provide the City with a lower cost and an additional phased option, Foth developed Conceptual Alternative 12, which has not been modeled by Integral.

Conceptual Alternative 12 consists of moving the existing North wall towards the South by approximately 160 feet and adding an additional entrance in both the North and South Basins. The added entrances will be oriented to be parallel with the direction of current flow to facilitate tidal influence within the Marina.

This conceptual alternative proposes to reorientate the slips in the North Basin to be perpendicular to the current direction, to match the South Basin. Additionally, this alternative will include new channels matching the two (2) eastern most fairways of the South Basin, extended throughout the Marina. These channels serve as an alternative to the sump options which would be intended to produce a scouring effect in the higher water velocity zones and improve waver circulation throughout the rest of the Marina.

These channels would provide options for phasing, as the diagonal primary channels could be dredged in the first project phase, followed by the longer through channels in later phases. These channels could also be dredged to a shallower overall depth than the proposed sumps, which would reduce the overall costs of implementation. The one aspect of this conceptual alternative which would be an additional cost, not required by the other alternatives is the reorientation of the floats in the North Basin by 90 degrees to align the through channels with the fairways. It is possible that the existing North Basin floats could be reused for this purpose, however, an in-depth structural condition assessment should be performed to determine the overall condition of these floats.



Figure 26: Conceptual Alternative Layout (Note: Reorientation of North Basin Floats Not Shown)

## 2.14 Alternatives Discussion and Further Recommendations

Each of the proposed alternatives were designed with the goal of reducing the frequency of dredge events within the Marina. The implementation of the sumps in the Marina were strategically located to collect sediment where the highest accumulation rates were found in order to capture that sediment and prevent it from settling elsewhere in the Marina. Additionally, several alternatives were considered which reduce the overall size of the Marina, reducing long term maintenance costs on the Marina infrastructure and reducing the dredge footprint for future dredging events.

The removal of the wall between the north and south basins, which is featured in several alternatives, will allow for more access within the Marina and also open up new berthing for larger vessels on the north side of J-dock. Removing this wall also was shown to be a key component in increasing the flow throughout the Marina, as circulation will no longer be isolated between the basins.

Alternatives 5, 6 and 7 include the removal of the H and I docks to facilitate creation of the southern sump. This new, open area will allow for future improvements in the Marina, as it will no longer be used for berthing and could be used for the location of a public boat ramp in the future, as the City is currently without a functioning boat ramp in the area.

It should be noted that it is recommended that a more in-depth sediment transport model is recommended to be performed on the alternatives which utilize sumps in an effort to justify the additional cost, permitting, and logistics associated with their installation. The sedimentation model which has been performed on these alternatives calculates the probability of deposition for each alternative and does not have the ability to account for sediment removed from the system by the sumps, nor take into account the influence of the sumps on the remainder of the Marina. Assumptions were therefore made in adjusting the deposition rates presented in this report to account for the sump influence, however these assumptions should be validated by a sediment transport model prior to their installation.

With regards to budgetary constraints, permitting, and development of a more detailed model, it is recommended to utilize any initial dredging expenditures on ways to promote better through-flows through the Marina. This could be performed in parallel with a sediment transport model, which could be adjusted to reflect the conditions and accretion patterns observed after initial improvements are made. Additionally, the initial improvements will help to determine the necessity of sumps and if required help to select the ideal sump locations and depths.

An alternative that may be worthy of considering, would be installing "SEDCON" units in some of the lower flow areas. The SEDCON technology has been around for a considerable amount of time and has proven

quite successful in many installations. SEDCON units consist of large, “L” shaped underwater pumps, that rotate like a floor mounted house fan. These act to blow sediments from stagnant areas into other areas of higher flow. Once the settled sediments become resuspended, they are then usually carried away. A site-specific study and evaluation is required for them to be successfully deployed, but in general, in many cases have proven to be cheaper than sumps and dredging. The SEDCON areas still require some level of remedial dredging however, the quantities are much lower. This will require additional discussion with the regulatory agencies.

Finally, it should be noted that with the constant flows of highly turbid water flowing through Mare Island Strait and the surrounding areas, sediment related shoaling is virtually impossible to stop completely. Even under the best of circumstances the best efforts can still only reduce volumes of shoaling. However, though careful planning and the ability to adapt plans to better fit the specific site conditions - generally improved results can be expected.

### **3. Regulatory Overview**

#### **3.1 Anticipated Permit Requirements**

The dredging activities within the Marina have been previously permitted by BCDC, CEQA, CDFW, USACE, and the Water Quality Control Board. The current dredge permits have expired with the exception of the BCDC and CDFW permit, which are set to expire in 2026 and 2025. The below list provides a summary of the permits and consultations which will likely be required for rehabilitation and sump dredging within the Marina.

##### **3.1.1 Bay Conservation and Development Commission (BCDC):**

BCDC’s jurisdiction of the Bay and the certain named waterways, including the Mare Island Strait, extends to the mean high tide line in areas that do not contain tidal marsh and up to five feet above mean sea level in areas of tidal marsh and a shoreline band of land extending inland for 100 feet from the shoreline of the Bay.

California has a federally approved Coastal Management Program, which includes the California Coastal Act and the McAtter-Petris Act. The program established the BCDC as the coastal management and regulatory agency responsible for governing coastal resources in San Francisco Bay. In accordance with its role in implementing CZMA, the BCDC is responsible for conducting Federal consistency reviews for projects along the San Francisco Bay segment of the California coastal zone.

The San Francisco Bay Plan (Bay Plan), adopted in 1969, is BCDC’s policy document specifying goals, objectives, and policies for BCDC jurisdictional areas (BCDC, 2008). Federal lands are outside the coastal zone, but Federal activities on land outside the coastal zone that affect resources of the coastal zone must be evaluated for their consistency, to the maximum extent practicable, with the Bay Plan and related policies. Because the landside portion is not on Federal lands, the City is required to obtain BCDC approval for its waterside Proposed Action; the project will be subject to a full BCDC review process. A Major Amendment to current permit is anticipated.

##### **3.1.2 California Department of Fish and Wildlife (CDFW):**

It is anticipated that an Amendment to current (or new) Lake/Streambed Alteration Agreement will be required. A meeting with the regional CDFW representative regarding the project will occur prior to final design to determine if an Incidental Take Permit will be required to cover pile driving activities on potentially affected state-listed species (longfin smelt and delta smelt).

##### **3.1.3 California State Land Commission (CSLC):**

The California State Lands Commission (CSLC) has jurisdiction and management authority over all ungranted tidelands, submerged lands, and the beds of navigable lakes and waterways. The Commission

also has certain residual and review authority for tidelands and submerged lands legislatively granted in trust to local jurisdictions (PRC §6301 and §6306). All tidelands and submerged lands, granted or ungranted, as well as navigable lakes and waterways, are subject to the protections of the Common Law Public Trust Doctrine.

As general background, the State of California acquired sovereign ownership of all tidal and submerged lands upon its admission to the United States in 1850. The State holds these lands for the benefit of all people of the State for statewide Public Trust purposes, which include but are not limited to waterborne commerce, navigation, fisheries, water-related recreation, habitat preservation, and open space. On tidal waterways, the State's sovereign fee ownership extends landward to the mean high tide line, except for areas of fill or artificial accretion or where the boundary has been fixed by agreement or a court. The Commission has determined that the project is located within lands the State granted to the City of Vallejo pursuant to Chapter 588, Statutes of 2004, and as amended, with minerals reserved to the State.

New legislation that took effect January 1, 2014, Chapter 104, Statutes of 2013 (AB 727, Stone), eliminated the requirement of a lease from the CSLC for dredging on granted public trust lands where minerals are reserved to the State, if the dredged material is disposed at an approved on or offshore disposal site, is not sold, and is consistent with the proper management of the granted lands. Consultation with CSLC should be initiated to confirm concurrence with the preferred alternative.

### **3.1.4 City of Vallejo**

The City of Vallejo is one of the major stakeholders involved in the project, as the Owner of the site. The rehabilitation and optimization align with the long-term goals of the City. "Propel Vallejo", the City's General Plan for 2040, states that the development of the central waterfront area around the Marina will foster a walkable environment and improve open spaces. It also proposes the potential inclusion of the Marina in the San Francisco Bay Area Water Trail, a network of non-motorized, human-powered boat launch and landing sites throughout the region. Improving the existing Marina site through the rehabilitation project will enhance Vallejo's waterfront culture, provide access to nature, and contribute to driving the local economy. The rehabilitation project will also make the Marina a more attractive option for boaters looking for a facility that best fits the needs for their vessel.

### **3.1.5 Consultations with National Marine Fisheries (NMFS) and US Fish and Wildlife Services (USFWS)**

The Endangered Species Act (ESA) is administered by the USFWS and NMFS. Section 7 of the ESA requires formal consultation with the USFWS or NMFS for projects that may affect those species that are either listed as, or proposed for listing as endangered or threatened, to ensure that the Proposed Action will not jeopardize listed species or destroy or adversely modify designated critical habitat for such species.

The Magnuson-Stevens Fishery Conservation and Management Act (MSA) requires consultation with NMFS on activities that may adversely affect Essential Fish Habitat (EFH), or when NMFS independently learns of a Federal activity that may adversely affect EFH.

### **3.1.6 Long Term Management Strategy (LTMS):**

The project's dredging activities will be subject to review by the LTMS, as the Vallejo Marina was previously deemed to fall under the Small Dredger Programmatic Alternatives Analysis. A small dredger project is defined by having a project depth of 12 feet Mean Lower Low Water (MLLW) or less, and having an annual dredging/disposal volume of 50,000 cubic yards or less. Due to the volume of sediment which has accumulated in the Marina, and the need for a comprehensive dredge event to remove this material, the Vallejo Marina will no longer qualify as a Small Dredger. Additionally, the proposed Alternatives which include a sump area would be dredged beyond the depths allowed by the LTMS Small Dredger Program. Therefore, an Integrated Alternatives Analysis will need to be performed to determine placement options for the removed sediment. Per the LTMS goals, a minimum of 80% of the disposal volume will be required to be placed at an upland beneficial reuse site.

### **3.1.7 San Francisco Bay Water Quality Control Board (SFBWQCB):**

A Pre-Filing Meeting will be conducted to review the project with SFBWQCB prior to submission of a section 401 Water Quality Certification (WQC) to cover project activities. Upon completion of the Pre-Filing Meeting, an application for a WQC will be submitted to the Water Board for review and approval for the proposed project activities.

### **3.1.8 U.S. Army Corps of Engineers (USACE):**

Section 10 of the Rivers and Harbors Act of 1899 (33 USC Section 401 et seq.) requires a permit from the Corps for creating obstructions (including excavation and fill activities) to the navigable waters of the U.S. Navigable waters are defined as those water bodies subject to the ebb and flow of the tide, and/or that are used, in their natural condition or by reasonable improvements, as a means to transport interstate or foreign commerce. Construction of structures in, under, or over navigable water; deposition or excavation of material in navigable waters; and all work affecting the location, condition, course, or capacity of navigable water are covered by Section 10 of the Rivers and Harbors Act.

Section 404 will be required to comply with the Clean Water Act. Reconfiguration will not qualify for a standard nation-wide permit. A Regional General Permit application would need to be approved for this project.

## **3.2 Impact to Aquatic Resources**

The total area impacted by the reconfiguration of the Marina varies between alternatives. The majority of the alternatives propose to reduce the overall size of the Marina and each alternative will reduce the frequency of dredging within the Marina. Additionally, the removal of A-Dock, which is proposed for all options, will remove approximately derelict timber floating docks and approximately the associated creosote treated timber piles from the waters of San Francisco Bay, providing a long term benefit to aquatic resources in the area. The removal of A-Dock should be considered during the project permitting process as a mitigation measure to offset any potential impacts.

Additionally, for alternatives proposing to move the location of the north sheet pile wall to the south, there is the potential for the creation of a natural tidal salt marsh in the area previously bound by the north wall. This area is anticipated to naturally collect sediment over time and has potential to return to its natural state, providing a suitable environment for wildlife.

## **3.3 Endangered and Special Status Species**

The existing regulatory permits define work windows for the covered activities, based on anticipated impacts to Endangered and Special Status Species. Most of the sensitive fish species that have the potential to be present in the project area are likely to be present only during certain seasons; however, Green Sturgeon could be present at the project site at any time of year.

The environmental work window, based on the existing permits for the project activities, is August 1 through October 15 in order to minimize impacts to the species listed in the table below. It should be noted that the environmental work window restrictions for in-water construction activities vary by resource agency and species. The windows shown in the table below have been combined to represent the most restrictive windows allowed by the permits. Work windows listed in individual permit authorizations may have longer durations than those listed in Table 4 below.

**Table 4: Listed Special Species Work Window**

Species	Work Window
Steelhead Trout ( <i>Oncorhynchus mykiss</i> )	August 1- October 15
Chinook Salmon ( <i>Oncorhynchus tshawytscha</i> )	August 1- November 30
North American (Southern Distinct Population Segment) Green Sturgeon ( <i>Acipenser medirostris</i> )	August 1- November 30
Delta smelt ( <i>Hypomesus transpacificus</i> )	August 1- November 30
Longfin Smelt ( <i>Spirinchus thaleichthys</i> )	August 1- November 30
Tidewater goby ( <i>Eucyclogobius newberryi</i> )	August 1- November 30
Sacramento splittail ( <i>Pogonichthys macrolepidotus</i> )	August 1- November 30
River lamprey ( <i>Lampetra ayresii</i> )	August 1- November 30

#### 4. Mitigation Measures

The two major components of the proposed Alternatives which are anticipated to require the most mitigation are pile driving activities associated with repairs to the existing breakwater and the new-work dredging required for the sump areas. If an Alternative which utilizes a sump is selected, the permits listed in Section 3 will be amended for the new-work dredging of the sumps.

The CDFW Bios Online Mapping tool and the BCDC Open Data Portal tool were reviewed and it was confirmed that there is no eelgrass anticipated in the proposed project area. USACE regulatory public notices from nearby projects such as Kiewit Infrastructure West Company Maintenance Dredging (dated February 4, 2015), Mare Island Shipyard Maintenance Dredging (dated August 6, 2020), and US Coast Guard Station Vallejo Maintenance Dredging (dated October 23, 2020), state that eelgrass is not expected to be established in the area.

Construction of the waterside improvements to the sheet pile wall could result in indirect impacts to common and special-status fish species if they are present in Mare Island Strait during construction activities. Listed anadromous salmonids are not expected to be in the project area during the in-water construction period of August 1 through October 15, and therefore would not be impacted by construction activities.

Pile driving could cause disturbance of bottom sediment and increased turbidity, and disturbed sediments could contain contaminants. When a vibratory hammer is used, very little turbidity is expected; however, a small amount of temporary turbidity disturbance would occur in close proximity to the pile but is expected to quickly dissipate.

If an impact hammer is used—and consequently, a bubble curtain is installed to reduce sound waves; turbidity could occur in a larger area surrounding the activity. However, the potential impact from turbidity and any released contaminants would be localized and short term and is not expected to have a lasting impact on the common and special-status species, designated critical habitat, or Essential Fish Habitat (EFH).

Similarly, construction activities are not expected to generate levels of turbidity that would be harmful to benthic invertebrates. Standard Best Management Practices would be implemented during construction to minimize potential impacts on water quality, such as training workers to identify and prevent releases of pollutants, using containment booms to capture floating demolition debris, and removing solid waste from the site regularly. Implementation of these measures would reduce the potential impact of degraded water quality to benthic invertebrates and fish during construction.

The construction of any of the proposed reconfigurations will result in cover (shadows) of estuarine habitat. Shadowing of estuarine waters may result in alteration of benthic habitat and a reduction in benthic or planktonic productivity due to reduction in solar energy.

For all pile driving associated with the construction of the selected alternative, the following mitigation measures will apply:

- A qualified marine contractor will mobilize a barge at high tide to access the Marina.
- The new piles and miscellaneous equipment will be transported onsite by trailer or barge, dependent on size and weight.
- Work will be performed by barge mounted crane and supporting equipment as required.
- Removal of existing piles shall be performed with a vibratory hammer. If a pile breaks during the removal process, the contractor will cut the pile below mudline.
- The proposed piles are to be installed by a barge mounted crane. All pile driving will take place using a vibratory hammer in lieu of an impact hammer, unless unforeseen difficult driving conditions are encountered. If the proposed piles are unable to be driven to the design depth with a vibratory hammer, an impact hammer may be utilized to complete the installation.
- The Contractor selected to perform the work will reduce noise and vibration by furnishing, installing, operating, and maintaining a sound attenuation system to reduce noise generated by impact driving of piles into the stream if sound pressure levels (SPL) and sound exposure limits (SEL) thresholds may be exceeded. Vibratory driving sound level will conform to CalTrans specifications regarding dBs as described below;
  - 206 dB peak –the onset of injury is expected in fish exposed to peak SPLs at or above 206 dB.
  - 187 dB cumulative SEL for fish > 2 grams –fish > 2 grams will experience the onset of injury after a cSEL at or above 187 dB over the accumulation period.
  - 183 dB cumulative SEL for fish < 2 grams – fish smaller than 2 grams will experience the onset of injury after a cSEL at or above 183 dB over the accumulation period.
- The practice of soft start pile driving will be utilized. The initial strikes of all in-water piles, will occur at less than full impact force for a period of 15 seconds followed by 30 seconds of no activity. This action will be repeated two (2) additional times and impact will be gradually brought up to full force blows to allow aquatic species adequate time to leave the project area.
- The duration of the pile driving will not exceed eight (8) hours of continuous driving per day and the noise levels shall not exceed the Caltrans specifications regarding decibel levels.
- No heavy equipment will be stored, maintained, or refueled in the resource areas.

All Alternatives will involve the removal of the A-Dock, which will improve water quality and circulation in the area the dock currently occupies. It is anticipated that the removal of the A-Dock will count as a mitigation credit for the overall rehabilitation of the Marina.

## **5. Economic Analysis**

### **5.1 Assumptions**

The following Opinion of Probable Costs be considered for general order of magnitude cost purposes only. This estimate was prepared in July-August 2024, and in light of recent global supply chain issues, labor shortages, and other volatile market conditions, the costs associated with each of the proposed alternatives may change significantly between when the cost estimate was prepared and when the project

moves into the construction phase. Foth recommends that an additional cost estimate be performed during final design, prior to the project moving into the construction phase.

## 5.2 Opinion of Probable Costs

Foth provided the following preliminary Opinion of Probable Costs for the proposed reconfiguration alternatives, which can be found in Table 5 below. The estimate was developed based on material quantities required for the construction of each alternative. The estimate includes the costs to remove the existing A-Dock and associated timber piles, initial dredging of the Marina to permitted depths, as this work will be common to all alternatives. It should be noted that regulatory recommendations may impact the project design and result in changes to materials quantities, sizes, and locations, all of which will impact the total project cost. Therefore, the total project costs include a 20% contingency and should be considered an approximate budgetary range for long term capital planning.

This Opinion of Probable Costs does not include costs for the re-routing of utilities for the proposed reconfigurations, as this cost will be dependent upon the City's existing infrastructure and projected needs. A cost estimate for this work will be developed in the final design phases once utility design is finalized. Costs for maintenance to the existing sheet pile wall beyond the north wall were also omitted from the estimate, as an updated inspection of the wall should be conducted in order to confirm repair locations, size and quantity. General infrastructure upgrades and repairs, such as repairs to floating docks, gangways, and ramps were also omitted, as these consist of items which fall outside the scope of this study.

Additionally, the initial start-up dredging and A-dock removal were common costs which were shared among all alternatives. It is recommended that the A-dock is removed whether or not an alternative is selected to proceed, as it is causing a hazard to safe navigation in its current condition.

**Table 5: Opinion of Probable Costs - Alternatives**

	Existing Configuration	Alternative 1	Alternative 2	Alternative 3
Dredge North/South Basins	\$ 16,375,000	\$ 15,675,000	\$ 15,675,000	\$ 15,675,000
Additional Sump Dredging	\$ -	\$ -	\$ -	\$ 1,205,000
Demo/Repairs/Reconfiguration	\$ 885,000	\$ 5,900,000	\$ 5,965,000	\$ 5,900,000
20% Contingency	\$ 3,452,000	\$ 4,315,000	\$ 4,328,000	\$ 4,556,000
<b>Total</b>	<b>\$ 20,712,000</b>	<b>\$ 25,890,000</b>	<b>\$ 25,968,000</b>	<b>\$ 27,336,000</b>
	Alternative 4	Alternative 5	Alternative 6	Alternative 7
Dredge North/South Basins	\$ 15,675,000	\$ 14,995,000	\$ 13,405,000	\$ 13,405,000
Additional Sump Dredging	\$ 1,205,000	\$ 2,250,000	\$ 2,250,000	\$ 2,250,000
Demo/Repairs/Reconfiguration	\$ 5,965,000	\$ 6,505,000	\$ 6,505,000	\$ 6,505,000
20% Contingency	\$ 4,569,000	\$ 4,750,000	\$ 4,432,000	\$ 4,432,000
<b>Total</b>	<b>\$ 27,414,000</b>	<b>\$ 28,500,000</b>	<b>\$ 26,592,000</b>	<b>\$ 26,592,000</b>
	Alternative 8	Alternative 9	Alternative 10	Alternative 11
Dredge North/South Basins	\$ 15,675,000	\$ 15,675,000	\$ 15,675,000	\$ 15,675,000
Additional Sump Dredging	\$ -	\$ 2,250,000	\$ -	\$ 2,250,000
Demo/Repairs/Reconfiguration	\$ 6,570,000	\$ 6,570,000	\$ 6,700,000	\$ 6,700,000
20% Contingency	\$ 4,449,000	\$ 4,899,000	\$ 4,475,000	\$ 4,925,000
<b>Total</b>	<b>\$ 26,694,000</b>	<b>\$ 29,394,000</b>	<b>\$ 26,850,000</b>	<b>\$ 29,550,000</b>

It should be noted that a large portion of the costs shown above are consist of the initial dredging costs and that these costs are shared among each of the alternatives. The estimated initial dredging costs, not inclusive of the sump dredging, range from 50% to 70% of the total initial construction costs, based on the alternative.

As previously discussed in Section 2.10.1, a Phased Approach was also reviewed to lower the initial project costs and complete the construction of a preferred alternative in two sequences of work. Table 7 provides a rough order of magnitude for the Phased Approach, assuming that Phase 1 consists of South Basin dredging and removal of the middle wall. Phase 2 would then consist of dredging the North Basin along with the removal of the A-dock, relocation of the North Breakwater, and other miscellaneous repairs to the seawall.

**Table 6: Opinion of Probable Costs – Phased Approach**

Phase 1	
Mobilization	\$ 135,000.00
Dredge South Basin	\$ 8,560,000.00
Remove Middle Wall	\$ 370,000.00
20% Contingency	\$ 1,786,000.00
<b>Phase 1 Total</b>	<b>\$ 10,851,000.00</b>
Phase 2	
Mobilization	\$ 135,000.00
Dredge North Basin	\$ 7,115,000.00
Additional Sump Dredging	\$ 1,205,000.00
Breakwater Repairs/Reconfiguration/Demo	\$ 5,395,000.00
20% Contingency	\$ 2,770,000.00
<b>Phase 2 Total</b>	<b>\$ 16,620,000.00</b>
<b>Total Phases 1 and 2</b>	<b>\$ 27,471,000.00</b>

### 5.3 Projected Dredging Costs

Dredging in the Marina has occurred seven (7) times since 2001, although the interval between dredge events and the coverage within the basins have been irregular. The average volume removed over these episodes of 62,000 cubic yards per event does not reflect the volume which is required to be removed to keep the Marina fully operational, as the entire Marina was never fully dredged to the permitted design depth during this time.

Considering the current state of the Marina, a comprehensive dredge event would be required to restore access and navigation and would consist of the removal of approximately 470,000 cy of material, based on the most recent survey of the Marina. Material removed from the Marina has historically been disposed of at the in-Bay disposal site, SF-9. However, as discussed in Section 3.1.6, the Marina will not be considered a Small Dredger per the LTMS protocols, as the volume required to be removed from the Marina exceeds LTMS limits. Therefore, the removed sediment will be required to be placed at a beneficial reuse site. The projected cost per cubic yard of material removed is anticipated to range between \$30.00 to \$40.00, based off historical bid prices and current estimates.

In order to compare each configuration on an even basis, the projected dredging costs assumed that dredging would be triggered when the average depth in the Marina reached -7' MLLW, the frequency of

dredge events per alternative is shown below in Table 6. To find the dredge return period for each alternative, additional analysis was required beyond the scope of the sediment deposition model. The model created for the Marina calculates the average shear stress as a function of velocity and predicts the probability of sediment deposition from those values. The Marina model does not model sediment transportation within the Marina and therefore it was necessary to account for the sediment removed from the system by the sumps separately.

Foth assumed that the amount of sediment available was a fixed value within the Marina and calculated the total volume available for distribution. The volume which accumulated in the sumps was then subtracted from the total available volume, and the remainder of sediment dispersed throughout the non-sump areas in the Marina was then calculated. A relationship between the volume and average depth was reviewed to determine when the average depth would exceed the -7' MLLW threshold and require a dredge event. Finally, Foth assumed that the entire Marina would be dredged to a depth of -10' MLLW for this analysis. The 1' allowable over dredge was not included in the return time. Table 7 below shows the dredge frequency values based on this analysis.

**Table 7: Dredge Return Frequency**

	Dredge Frequency (years)			
	North Basin	South Basin	North Sump	Center Sump
<b>Existing Configuration</b>	3.2	2.9	-	-
<b>Alternative 1</b>	3.8	6.4	-	-
<b>Alternative 2</b>	3.7	5.0	-	-
<b>Alternative 3</b>	4.7	7.9	10.1	-
<b>Alternative 4</b>	4.7	5.8	10.8	-
<b>Alternative 5</b>	9.3	6.1	10.2	13.5
<b>Alternative 6</b>	10.4	7.2	8.5	13.7
<b>Alternative 7</b>	9.3	7.0	9.9	13.7
<b>Alternative 8</b>	N/A <sup>1</sup>	5.5	-	-
<b>Alternative 9</b>	N/A <sup>1</sup>	6.7	13.9	14.3
<b>Alternative 10</b>	N/A <sup>1</sup>	4.8	-	-
<b>Alternative 11</b>	N/A <sup>1</sup>	5.9	12.7	11.8

<sup>1</sup> While the model indicates that the average deposition rate in the North Basin is erosional for Alternatives 8 through 11, there will be areas within the North Basin where sediment accretion will occur. Dredging will be required when these areas begin to prevent vessel access. These areas can be more accurately defined through a sediment transport model.

Table 8 shows the anticipated maintenance dredging costs for each alternative as an annual capital cost. The annual costs shown are based on the estimated accretion volume to be removed in the dredge event at the given return frequency from Table 7. These costs also account for a mobilization cost for each dredge event and should be considered to be a budgetary value for annual planning purposes.

**Table 8: Projected Annual Dredge Costs**

Annual Savings - Alternative vs. Existing		
Configuration	Annual Dredging Cost	Annual Savings vs. Existing
Existing Conditions	\$ 2,035,484.69	\$ -
Alternative 1	\$ 1,251,977.96	\$ 783,506.73
Alternative 2	\$ 1,422,726.05	\$ 612,758.63
Alternative 3	\$ 1,117,658.54	\$ 917,826.15
Alternative 4	\$ 1,236,117.63	\$ 799,367.06
Alternative 5	\$ 959,325.72	\$ 1,076,158.96
Alternative 6	\$ 838,724.58	\$ 1,196,760.11
Alternative 7	\$ 838,608.50	\$ 1,196,876.19
Alternative 8	\$ 567,521.06	\$ 1,467,963.62
Alternative 9	\$ 676,477.17	\$ 1,359,007.51
Alternative 10	\$ 650,069.58	\$ 1,385,415.10
Alternative 11	\$ 775,503.38	\$ 1,259,981.31

A breakeven analysis was not performed for this study, as the value of the additional revenue which will be generated by the Marina in each of the alternatives is unknown and under investigation by F3. Foth and F3 will work together during the next phases of this project to add these costs to the projections. Additionally, Foth is performing a market study for the Marina to provide an understanding of the demands of the local economy. Foth can refine the alternatives per the results of this study and utilize the results of the market study to further develop the preferred alternative.

## 5.4 Constructability

In addition to the costs associated with each of the proposed reconfigurations, each reconfiguration alternative was evaluated for the anticipated level of constructability. Potential issues and conflicts which were anticipated to arise during construction are described below. It should be noted that all alternatives will require repairs to the existing perimeter wall and that these repairs should be carried out even if the Marina is not reconfigured to provide a functioning wall and address existing damage. Additionally, the A-dock and associated infrastructure should be removed for all alternatives.

Alternative 1 slightly reduces the boundaries of the existing Marina and will consist of shifting the location of the north sheet pile wall to the south. Because the location of the north wall will be changed for each alternative, they will require the construction of an entirely new section of sheet pile wall. Additional demolition of the center wall between the basins will be required to allow flow throughout the Marina. Alternative 2 will require the same base level of effort as Alternative 1, with additional demolition required to create the new center entrance.

Alternatives 3 and 4 require the same initial level of effort as Alternatives 1 and 2, respectively, but also include additional dredging in order to create a sump. This new dredging will be considered as improvement dredging by the regulatory agencies and will require additional permit modifications. However, the sumps will offer a long-term benefit to the Marina by providing areas away from the floating docks which can be dredged at a lower cost to the Marina than the dredging between docks.

Alternatives 5, 6 and 7 consist of reducing the size of the Marina more significantly by shifting the location of the north sheet pile wall further to the south which will also include the removal of two (2) of the existing floating docks and the associated support piles in the north basin and the reconfiguration of the covered berthing docks. The work associated with these activities will add additional time to the initial construction.

Alternatives 8 through 11 will introduce two (2) new entrances to the Marina, in addition to the work described in Alternatives 5 through 6. As this study assumes that a new North Wall is to be constructed, the new entrance on the North side of the Marina will not incur additional costs, as it can be installed as a part of the new wall. However, the added South entrance and the closure of the two (2) existing entrances in Alternatives 10 and 11 will be additional work items unique to these alternatives.

## **6. Stakeholder Input**

### **6.1 Relevant Stakeholders and Interested Parties**

As the City moves forward with the selection of a preferred alternative, Foth recommends that the following entities be consulted to gain further guidance and input for a successful project. A Stakeholder Engagement workshop is recommended with the following relevant stakeholders and potential interested parties:

- Zio Fraedos
  - An Italian restaurant located in the upland area directly adjacent to the Marina, Zio Fraedos would benefit from the rehabilitation of the Marina as it is anticipated to increase traffic in the Marina and improve waterside access to Vallejo.
- Sardine Can
  - Another restaurant in the upland area adjacent to the Marina, an increase in Marina patronage and the availability of slips would be a benefit to the Sardine can.
- Vallejo Marine Craft
  - Located within the boundaries of the Marina, Vallejo Marine Craft offers vessel repair services and vessel haul-out services via their marine travel lift. Reduction of the sedimentation rate in the Marina will allow for continued operation of their travel lift. Additionally, an increase in the number of usable slips in the Marina will likely result in a direct increase in business, as the need for professional vessel repair services will be directly correlated to an increase in vessels within the Marina.
- USCG
  - The proposed project is anticipated to benefit the U.S. Coast Guard Station within the Marina. The dredging included as part of the proposed work will delay the need for the Coast Guard to conduct their own dredging in the area, particularly if a sump alternative is chosen, which would involve a deepened area below the coast guard docks.
- Tenants/boaters
  - The day-to-day patrons of the Marina will see the benefits of a fully dredged Marina through an increase in their availability to utilize their vessels throughout the tide cycle, as access to a large majority of slips is currently limited to high tides. Additionally, patrons will benefit from the reconfigured alternatives through an increase in access throughout the Marina via additional entrances, removal of derelict structures, a fully functioning perimeter wall, and the ability to freely navigate throughout the Marina. The reconfigured alternatives will also have lower sedimentation rates, allowing the Marina patrons to enjoy these benefits for a longer time than would be allowed by the existing Marina configuration.

- San Francisco Bay Trail:
  - Coordination will be conducted with the Metropolitan Transportation Commission in regards to potential impacts to the San Francisco Bay Trail. The existing Marina configuration contains approximately 700 linear feet of transient docking space at the side tie dock located at Gate E. This transient docking area may be used by members of the community as an access point to the Bay Trail.
- Whale Boat Society
- Commercial Companies (Bay Marine, MSRC, Moose Boats, Lind Marine, DRS, Dutra, PG&E, Dolphin Charters)
  - Commercial companies, including Bay Marine, MSRC, Moose Boats, Lind Marine, DRS, Dutra, PG&E, and Dolphin Charters, are all anticipated to benefit due to the improved access to Vallejo that will result from the Marina rehabilitation project. In addition, access to the travel lift at the Marina will allow commercial companies ease of access to load and unload any necessary vessels to conduct their work and complete any needed services.

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## 7. Evaluation of Alternatives

Table 9 below presents a comparison of Alternatives 1 through 7 based on the major components associated with each alternative.

**Table 9: Alternative Feature Comparison**

Alternative	Start-Up Dredging	Demolition of A-Dock	Removal of Middle Wall	Relocation of North Wall	Add Entrance	Add Sumps	Remove Entrances
1	✓	✓	✓	✓			
2	✓	✓	✓	✓	✓		
3	✓	✓	✓	✓		✓	
4	✓	✓	✓	✓	✓	✓	
5	✓	✓	✓	✓		✓	
6	✓	✓	✓	✓		✓	
7	✓	✓	✓	✓		✓	
8	✓	✓	✓	✓	✓		
9	✓	✓	✓	✓	✓	✓	
10	✓	✓	✓	✓	✓		✓
11	✓	✓	✓	✓	✓	✓	✓

## 8. Conclusion

After review and analysis of the proposed reconfigurations and consideration of feedback from Marina and City Staff, each of the proposed Alternatives are anticipated to meet the Marina's goals of reducing the frequency of dredge events, as supported by data from Integral's sedimentation study model. Reconfiguration of the Marina will also improve the efficiency and accessibility throughout the Marina through the removal of derelict structures and added entrances or middle wall removal.

The next steps for the City and Marina staff, will include finalizing stakeholder input, selection of a preferred alternative, performing a market study and searching for grant opportunities and/or additional funding to facilitate construction of the preferred alternative. Foth recommends that once the Market Study is finalized, that a preferred alternative is selected. If the preferred alternative involves the implementation of sumps or other significant changes from the alternatives presented in this report, an in-depth sediment transport model is preformed to justify the additional expenditures incurred by the sumps and to account for any modifications resulting from Marina changes recommended by the market study.

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**Appendix A**

**Vallejo Marina Sedimentation Study, Dated July 18, 2024. Prepared by  
Integral Consulting, Inc.**

**DRAFT**

# Vallejo Marina Sedimentation Study

*Prepared for*  
Foth & Van Dyke and Associates



601 Montgomery Street  
Suite 888  
San Francisco, CA 94111

July 18, 2024

# CONTENTS

<b>LIST OF FIGURES</b> .....	<b>iv</b>
<b>LIST OF TABLES</b> .....	<b>viii</b>
<b>ACRONYMS AND ABBREVIATIONS</b> .....	<b>ix</b>
<b>EXECUTIVE SUMMARY</b> .....	<b>x</b>
<b>1 Introduction</b> .....	<b>1-1</b>
1.1 SITE DESCRIPTION .....	1-1
1.2 PROJECT TASKS .....	1-2
<b>2 Numerical Modeling</b> .....	<b>2-1</b>
2.1 SAN FRANCISCO BAY-DELTA COMMUNITY MODEL.....	2-1
2.1.1 Boundary Conditions.....	2-3
2.1.2 Bay Model Evaluation.....	2-4
2.2 VALLEJO MARINA MODEL .....	2-6
2.2.1 Hydrodynamic Forcing Conditions.....	2-9
2.3 VALLEJO MARINA MODEL EVALUATION.....	2-9
2.3.1 Model Performance Metrics.....	2-11
2.3.2 Vallejo Model Calibration .....	2-13
2.3.3 Vallejo Model Validation.....	2-16
2.4 MODEL UNCERTAINTY .....	2-17
<b>3 Sedimentation Study</b> .....	<b>3-1</b>
3.1 EXISTING DATA SETS .....	3-1
3.2 DEPOSITION RATE CALCULATION .....	3-3
<b>4 Alternative Marina Configurations: Modeling and Analysis</b> .....	<b>4-1</b>
4.1 ALTERNATIVE MARINA CONFIGURATION MODEL EVALUATIONS.....	4-2
4.2 VELOCITY .....	4-3
4.3 SHEAR STRESS .....	4-13
4.4 PROBABILITY OF DEPOSITION .....	4-22
4.4.1 Sediment Deposition.....	4-23
4.5 DEPOSITION RATES .....	4-26
4.6 IMPLICATIONS ON DREDGING.....	4-27

**5 Conclusions .....5-1**  
**6 References .....6-1**

## LIST OF FIGURES

Figure 1-1.	Vallejo Municipal Marina, in Mare Island Strait. North basin extents in red. South Basin extents in yellow.....	1-2
Figure 2-1.	San Francisco Bay-Delta Community Model Domain and Bed Level Elevation Referenced to NAVD88 meters.....	2-2
Figure 2-2.	Bay Model bathymetry (NAVD88 m) and grid near Vallejo Marina study area. Pink Lines in right panel indicate locations of boundary conditions.....	2-2
Figure 2-3.	San Francisco Bay-Delta Community Model Water Level and Napa River Discharge Boundary Conditions. The selected calibration period is shaded in gray. ....	2-3
Figure 2-4.	Bay Model Comparison Stations and Location of Mare Island Strait Model Domain. ....	2-4
Figure 2-5.	San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9414863 at Richmond, California. ....	2-5
Figure 2-6.	San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9415102 at the Martinez-Amorco Pier, California.....	2-6
Figure 2-7.	Overview of Vallejo Marina Model Grid.....	2-7
Figure 2-8.	Vallejo Marina Grid refinement around marina. ....	2-8
Figure 2-9.	Overview of Vallejo Marine model bottom elevations.....	2-8
Figure 2-10.	Vallejo Terminal Model Discharge and Water Level Boundary Conditions Extracted from the Bay Model. The selected calibration and validation periods are shaded in gray.....	2-9
Figure 2-11.	ADCP depth averaged current velocity used for calibration and validation of the hydrodynamic model. The selected calibration and validation time periods are shaded in gray. ....	2-11
Figure 2-12.	Discharge and water level boundary conditions. The selected calibration and validation periods are shaded in gray. ....	2-14
Figure 2-13.	Vallejo Model calibration to depth averaged velocity from the Mare Island Strait ADCP.....	2-15
Figure 2-14.	Vallejo Model Validation to Depth Averaged Velocity from the Mare Island Strait ADCP.....	2-16
Figure 3-1.	Spatial occurrence of available surveys .....	3-2
Figure 3-2.	Vallejo Marina bathymetry referenced to NAVD88. ....	3-3

Figure 3-3. Inter-dredge north basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text). .....3-4

Figure 3-4. Inter-dredge south basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text). .....3-5

Figure 3-5. Inter-dredge deposition rate for the north basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text). .....3-6

Figure 3-6. Inter-dredge deposition rate for the south basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text). .....3-7

Figure 3-7. Calculated average (left), median (middle) and time-weighted (right) deposition rate. ....3-8

Figure 3-8. Time-Weighted Average Deposition Rate (top) and Maximum Deposition Rate (bottom). .....3-8

Figure 4-1. Bathymetry and marina wall configuration of the alternative analysis evaluation. NAVD88 CA State Plane Zone 2 with units of meters with vertical datum of NAVD88 with units of feet. Red lines indicate marina walls. ....4-3

Figure 4-2. Maximum grid-cell velocity during the model simulation. Red lines indicate marina walls. ....4-4

Figure 4-3. Maximum grid-cell velocity difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-5

Figure 4-4. Maximum grid-cell velocity during flood-tide. Red lines indicate marina walls. ....4-6

Figure 4-5. Maximum grid-cell velocity difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-6

Figure 4-6. Maximum grid-cell velocity during ebb-tide. Red lines indicate marina walls. ....4-7

Figure 4-7. Maximum grid-cell velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls. ....4-8

Figure 4-8. Mean velocity during the model simulation. Red lines indicate marina walls. ....4-9

Figure 4-9. Mean velocity difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-9

Figure 4-10. Mean velocity during the model simulation during flood-tide. Red lines indicate marina walls. ....4-10

Figure 4-11. Mean velocity difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-10

Figure 4-12. Mean velocity during the model simulation during ebb-tide. Red lines indicate marina walls. ....4-11

Figure 4-13. Mean velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls. ....4-12

Figure 4-14. Map showing regions where the velocities exceeds 2 kt shown in indigo (indigo) for maximum velocities. Red lines indicate marina walls.....4-13

Figure 4-15. Maximum grid-cell shear stress during the model simulation. Red lines indicate marina walls. ....4-14

Figure 4-16. Maximum grid-cell shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-15

Figure 4-17. Maximum grid-cell shear stress during flood-tide. Red lines indicate marina walls. ....4-16

Figure 4-18. Maximum grid-cell shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-16

Figure 4-19. Maximum grid-cell shear stress during ebb-tide. Red lines indicate marina walls...4-17

Figure 4-20. Maximum grid-cell shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls. ....4-17

Figure 4-21. Mean shear stress during the model simulation. Red lines indicate marina walls. ....4-18

Figure 4-22. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-19

Figure 4-23. Mean shear stress during flood-tide. Red lines indicate marina walls.....4-20

Figure 4-24. Mean shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-20

Figure 4-25. Mean shear stress during ebb-tide. Red lines indicate marina walls.....4-21

Figure 4-26. Mean shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls. ....4-21

Figure 4-27. Mean sediment deposition rate during the model simulation. Red lines indicate marina walls. ....4-24

Figure 4-28. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-24

Figure 4-29. Mean sediment deposition rate during flood-tide. Red lines indicate marina walls.4-25

Figure 4-30. Mean sediment deposition rate difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-25

Figure 4-31. Time till bed attains a depth of -7ft MLLW. ....4-28

## LIST OF TABLES

Table 2-1.	Model Skill Performance Metrics Set for This Study .....	2-12
Table 2-2.	Calibration Performance Metrics for Vallejo Model .....	2-15
Table 2-3.	Validation Performance Metrics for Vallejo Model .....	2-17
Table 3-1.	Bathymetry survey dates considered in sedimentation analysis .....	3-1
Table 3-2.	First post-dredge surveys .....	3-4
Table 4-1.	Marina configuration model simulations .....	4-2
Table 4-2.	Maximum (95 <sup>th</sup> percentile) north and south basin velocity (ft/s) .....	4-12
Table 4-3.	Mean north and south basin velocity (ft/s) .....	4-13
Table 4-4.	Maximum (95 <sup>th</sup> percentile) shear stress (Pa) .....	4-22
Table 4-5.	Mean shear stress (Pa) .....	4-22
Table 4-6.	Mean sediment deposition rate (ft/yr) .....	4-26
Table 4-7.	Mean flood sediment deposition rate for flood-tide (ft/yr) .....	4-26
Table 4-8.	Median estimated years before required dredging based on mean deposition rate .....	4-28

## ACRONYMS AND ABBREVIATIONS

ADCP	acoustic Doppler current profiler
DAV	depth-averaged velocity
EPA	U.S. Environmental Protection Agency
Foth	Foth & Van Dyke and Associates
Integral	Integral Consulting Inc.
MLLW	mean lower low water
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
RMSE	root mean square error
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey

## EXECUTIVE SUMMARY

Integral Consulting Inc. (Integral) supported Foth & Van Dyke and Associates (Foth) in the assessment of sedimentation rates and reconfiguration alternatives in the Vallejo Municipal Marina. The Vallejo Marina is located in the Mare Island Strait that connects the Napa River to San Francisco Bay where sediment is transported on tidal currents and may circulate through the marina. Sediment depositing in the marina from this tidal circulation has impacted navigable waterways and berthing areas that require sediment removal via dredging to maintain safe operation and access. Efforts are underway to evaluate alternative configurations of the marina that may reduce the need for dredge activities and

Foth has been tasked by the City of Vallejo and the Marina operators, F3 Marina, to develop alternative configurations and design of the Marina structures to reduce the frequency of maintenance dredge activities. Integral leveraged a hydrodynamic numerical model of the Mare Island Strait to evaluate changes in circulation patterns and bed forces within the Marina due to proposed changes in entrance configuration, marina extents, and bathymetric changes.

Bed shear stress patterns were compared with sedimentation rates, derived from a series of bathymetric surveys, to estimate sedimentation rates for the existing and proposed alternatives. This report outlines the development of numerical models, observed sedimentation rates, and comparison of existing and alternative marina configurations.

The alternatives evaluated in this study incorporated post-dredge sediment bed elevations, the potential for over-dredged areas, or sumps, and altered the configuration and location of marina walls. Removal of the center wall dividing the two marinas had a positive impact on circulation patterns, increasing the velocity relative to the existing case and indicating a potential decrease in sediment deposition potential. Addition of the sump areas, where it is theorized sediment would deposit and remain, also showed a benefit to the system. Addition of another marina entrance may also improve the ability for sediment to redistribute or exit the system on ebb tides.

# 1 INTRODUCTION

Integral Consulting Inc. (Integral) conducted a study of sediment deposition potential in support of ongoing redesign efforts at the Vallejo Marina. The work informed Foth, the City of Vallejo, and marina operators on the potential for sediment bed response due to the reconfiguration of the marina. Integral relied on numerical models of hydrodynamic conditions and surveys of sediment bed elevations to compare potential changes to sediment deposition patterns and computed dredge interval times.

## 1.1 SITE DESCRIPTION

The Vallejo Municipal Marina (the Site) is located within the Mare Island Strait, to the east of San Francisco Bay's San Pablo Bay subregion. The marina is located on the eastern side of the Strait, south of the Mare Island Causeway, and includes two basins which hold a number of vessel berths (Figure 1-1). Discharge from the Napa River as well as tidal fluctuations within San Francisco Bay drive currents that can mobilize, transport, and redistribute sediment. The Marina is surrounded by walls that extend above the high tide line to and into the sediment bed. While some areas have degraded, this evaluation did not consider permeability of this outer wall in its analysis. The two basins are separated by a wall and both marinas have one entrance. The United States Coast Guard operates a dock in the north basin of the marina.

Bedded sediment within the Site is exposed to natural and anthropogenic forces from tidal currents, and traversing commercial ferries and personal watercraft. The natural forces influencing the hydrodynamic conditions and associated sediment accretion at the Site must be investigated to determine how sediment may be transported under potential marina configurations. Currents move through the Site predominantly in a north-south direction and are associated with tidal exchanges between San Francisco Bay and the Pacific Ocean. Wind waves were not considered due to the limited fetch in Mare Island Strait and orientation of the Strait relative to the rest of San Francisco Bay.

Sedimentation rates within the Mare Island Strait and within the marina are governed by hydrodynamic conditions, sediment loads from the Napa River and San Francisco Bay, and anthropogenic activity including vessel induced scour and wake affects, and dredging events. Due to the lack of data and range of uncertainty in sediment loads within the system, this modeling study does not include direct sediment transport loads or transport to the system. Rather, the study uses a scaling approach of modeled bed shear stress to infer the potential for deposition or erosion. In addition, while it is assumed that the vessel activity modifies bed accretion and erosion potential due to vessel induced wakes and prop scour, the model is not equipped to handle these processes and it was not considered in this analysis.



Figure 1-1. Vallejo Municipal Marina, in Mare Island Strait. North basin extents in red. South Basin extents in yellow.

## 1.2 PROJECT TASKS

The technical tasks performed by Integral are outlined below. The analysis and findings of these tasks are presented in the following sections.

### Task 1. Set Up and Refine Hydrodynamic Model

Foth developed a hydrodynamic model in D-Flow FM and used it to simulate hydrodynamic conditions within the Mare Island Strait and around the Site. D-Flow FM is a numerical modeling suite that resolves hydrodynamic and sediment transport conditions on an unstructured grid. Integral developed boundary conditions to simulate water levels and tidal currents within the system concurrent with the period of hydrodynamic instrument deployment.

**Task 2. Calibrate Hydrodynamic Model**

The hydrodynamic model was calibrated by comparing model results to velocity data collected within the model domain. This ensures the model recreated hydrodynamic conditions and resulting forces that govern sediment mobility. Velocity data collected during a ~45-day deployment of an acoustic Doppler current profiler (ADCP) was used for model calibration and validation of depth averaged currents.

**Task 3. Assess Sedimentation Rates**

The estimation of sedimentation rates was conducted using available bathymetric surveys and dredge records collected since 2013. Analysis of the surveys produced estimates of rates of accretion within each basin. Results from this study were incorporated with the model results to infer the effects of marina alterations by scaling these deposition rates with changes in shear stress.

**Task 4. Evaluate Marina Alternatives**

Foth and Integral iterated through a set of alternative marina configurations to evaluate which may increase the time between maintenance dredging events. These alternatives modify the location of marina outer walls, add or alter the entrances to the Marina, and incorporate updated bathymetry including over-dredged areas for sediment accretion. Integral applied the layouts within the hydrodynamic model domain compare and computed changes in bed shear stress, deposition rates, and dredge return intervals.

## 2 NUMERICAL MODELING

A coupled hydrodynamic model was constructed for this study, using the San Francisco Bay-Delta Community model and a site-specific model generated by Foth, employing the open-source D-Flow FM modeling suite. D-Flow FM is a state-of-the-science model that can simulate aquatic systems in 1, 2, and 3 dimensions. It solves the 3-dimensional equations of motion in a water body with variable fluid density using free-surface and hydrostatic conditions. Water column turbulence, which generates small scale mixing, is modeled with a k-epsilon turbulence closure scheme at each model grid cell and at each time step. D-Flow FM was selected as the model for this Site based on the following capabilities:

- The model is publicly available, technically rigorous and defensible, and has been used widely to evaluate a broad range of systems.
- The model incorporates complex bathymetry using an unstructured grid, which allows for more accurate representation of bottom topography.
- The model allows for time-varying inputs of water levels and discharges.

Efforts by a team of investigators including Deltares, the U.S. Geological Survey (USGS), and San Francisco Estuary Institute developed the San Francisco Bay-Delta Community model that extended from the Pacific Ocean through the bay and into the delta (Deltares et al. 2021). This community-developed calibrated and validated model provided more refined hydrodynamic, or flow, information around the Site in the absence of measured data. The Mare Island Strait Model with model resolution refinements within and around the Marina, was used to simulate hydrodynamic conditions based on inputs from the Community model.

### 2.1 SAN FRANCISCO BAY-DELTA COMMUNITY MODEL

The San Francisco Bay-Delta Community model, or Bay model, domain consists of discrete grid cells ranging in size and shape, encompassing all of San Francisco Bay and its deltas (Figure 2-1). This model domain utilizes an unstructured grid, or flexible mesh, that allows cells to have any number of sides, typically three or four, with sides varying in length. The flexibility in size and shape allows complex shoreline geometries to be better represented in the model, and resolution can be increased in areas of interest. Each model grid cell has an associated depth, derived from a 2015 bathymetric survey conducted by USGS (2017). While meant to supply hydrodynamic boundary conditions to the Vallejo Marina model, the Bay model was setup with multiple grid cells across the Mare Island Strait to effectively resolve the hydrodynamics. The Bay model was set up with one vertical layer because of the relatively shallow depths of the Site and assumed negligible effect of any salinity or temperature driven circulation.

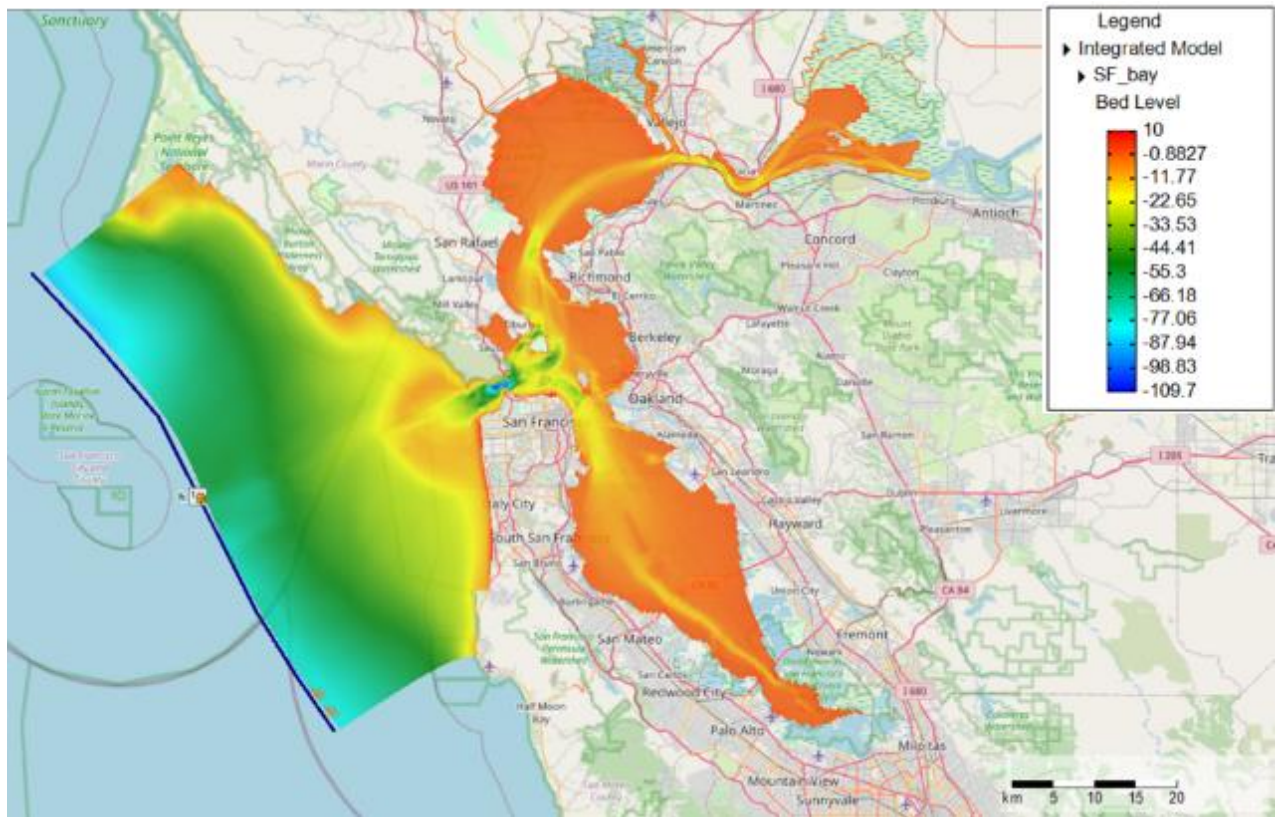


Figure 2-1. San Francisco Bay-Delta Community Model Domain and Bed Level Elevation Referenced to NAVD88 meters.

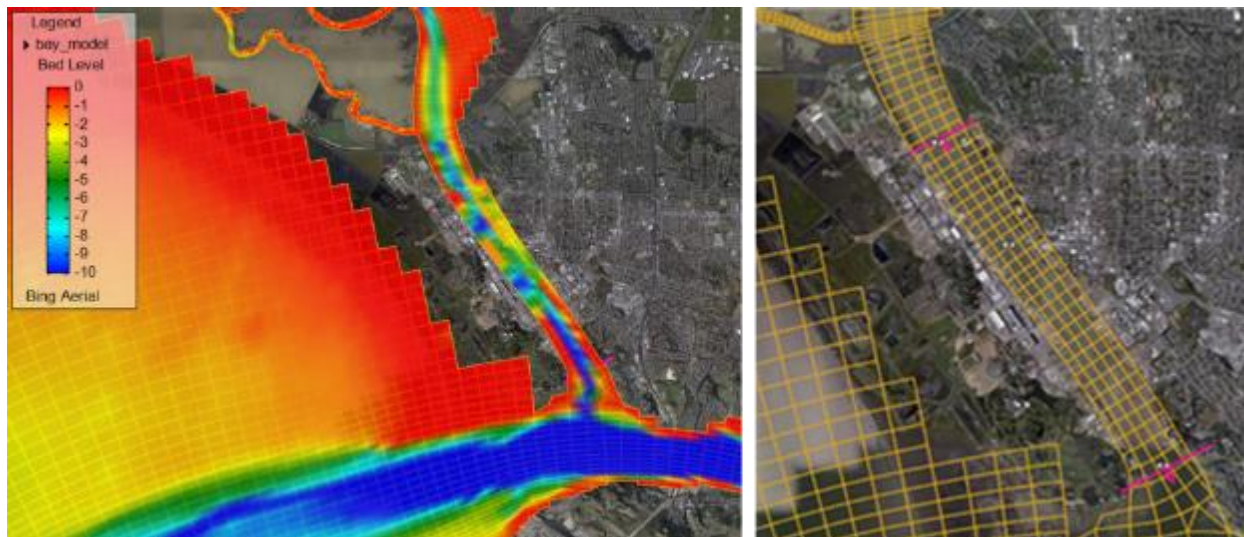


Figure 2-2. Bay Model bathymetry (NAVD88 m) and grid near Vallejo Marina study area. Pink Lines in right panel indicate locations of boundary conditions.

### 2.1.1 Boundary Conditions

Boundary conditions provide the forcing that drives the water mass within the model domain and usually consists of the water level at a boundary of the model domain, or discharge in to or out of the domain, or a combination of both. Two boundary conditions were utilized to drive circulation within San Francisco Bay, tidal fluctuations from the ocean, and riverine discharge from the Napa River. Tidal fluctuations at open ocean boundary, denoted by the blue line in Figure 2-1, vary water levels and drive currents through the mouth of San Francisco Bay (Figure 2-3). These data were sourced from measurements at National Oceanic and Atmospheric Administration (NOAA) station 9414290 at the Golden Gate Bridge. Due to the location of the model boundary in the coastal waters to the west of the station, a 42-minute offset was applied to temporally align boundary conditions with the measured water levels. The phase offset was determined by comparing measurements at the NOAA station location within the model with model results and adjusting accordingly. The discharge data from the Napa River was pulled from the USGS Streamflow database<sup>1</sup>. A majority of the Napa River record showed no discharge, including during the selected model calibration period (Figure 2-3). As a result, the ocean water level boundary would provide the forcing for hydrodynamics within San Francisco Bay, which included Mare Island Strait.

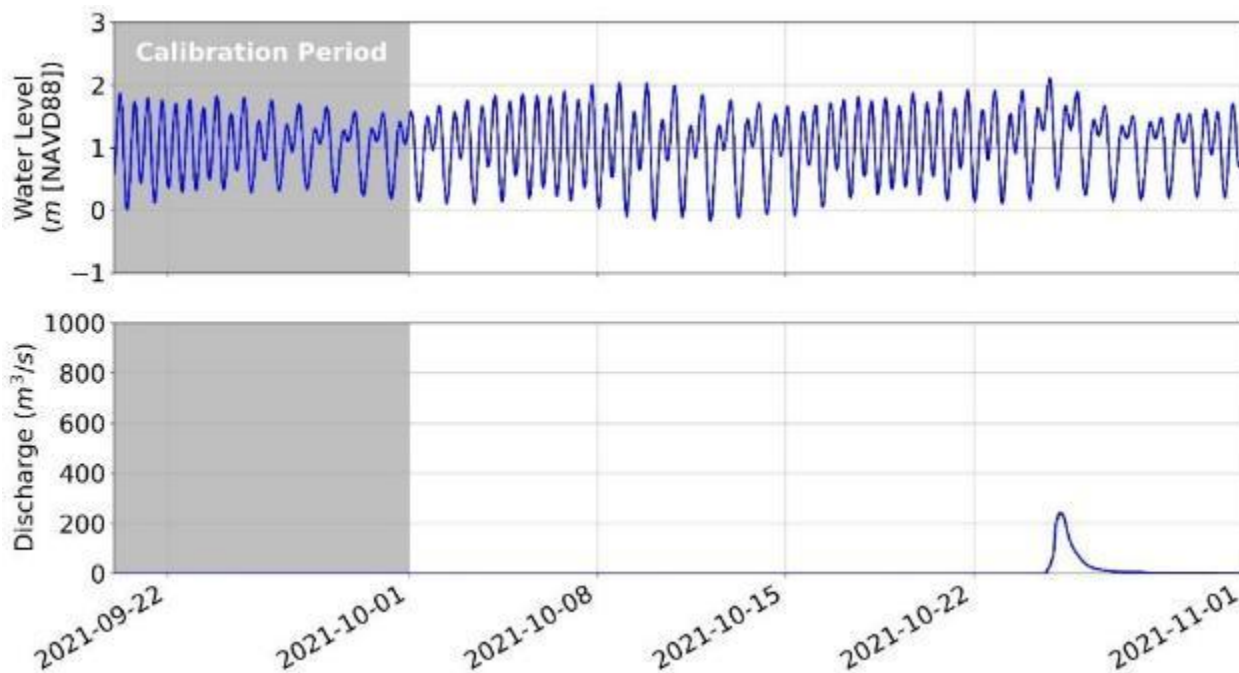


Figure 2-3. San Francisco Bay-Delta Community Model Water Level and Napa River Discharge Boundary Conditions. The selected calibration period is shaded in gray.

<sup>1</sup> <https://waterdata.usgs.gov/nwis/uv?11458000>

## 2.1.2 Bay Model Evaluation

Given that the Bay model would be providing boundary conditions to the Mare Island Strait model, it was important to show that the larger scale model accurately represents hydrodynamic conditions around San Francisco Bay.

The Bay model was assessed over a spring-neap tidal cycle during the evaluation period (September 20 to October 1, 2021) to ensure the model could adequately capture the full range of tidal elevations (Figure 2-3). Two NOAA-managed tide stations were selected to compare to the model predictions, Station 9414863 at Richmond, California, a site northeast of the Golden Gate Bridge, and Station 9415102 at the Martinez-Aramco Pier, California, southeast of the entrance to the Mare Island Strait (Figure 2-4).

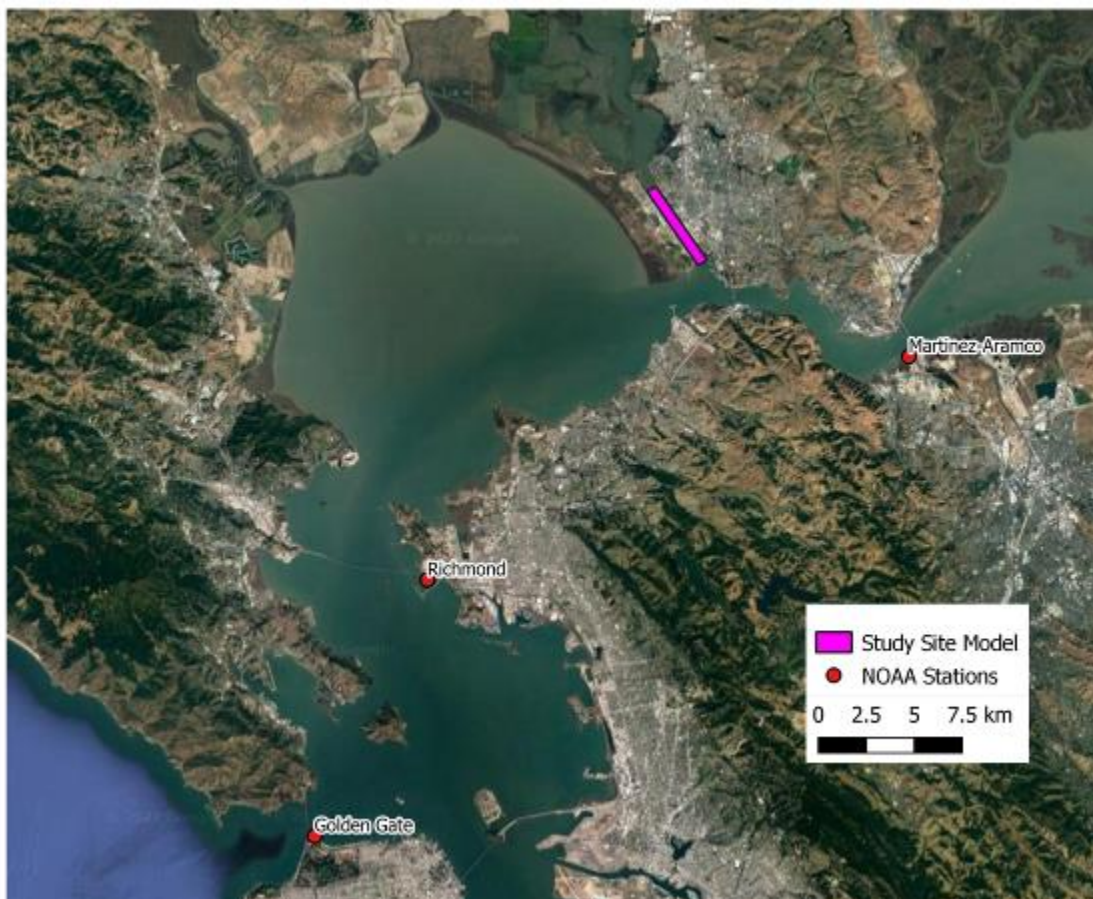


Figure 2-4. Bay Model Comparison Stations and Location of Mare Island Strait Model Domain.

The Bay model was modified in the Mare Island Strait portion of the domain by deepening some cells to align with the higher fidelity Foth provided bathymetry to better align the

modeled velocities with the measured ADCP data. The bathymetry with the original model limited flow and by deepening some areas, the boundary conditions derived from this model required less calibration for application than the Mare Island Strait model.

The Bay model does well in predicting the measured water levels at the two NOAA stations within San Francisco Bay. The Bay model best predicts water level at the Richmond station, which is closer to the Golden Gate Bridge and the primary water level boundary condition (Figure 2-5). The timing of the flood and ebb tides, and high and low tides, is correctly predicted; however the Bay model does overpredict the height of the high and low tides, though the maximum difference is 25 cm. This is likely a result of the ocean water level boundary condition. This difference in measured vs. modeled data will likely have a minimal impact on the hydrodynamics within the Mare Island Strait. The Bay model predictions at the Martinez-Amorco Pier station are similar in the accurate timing of the flood and ebb tides, and high and low tides. The overprediction of the high and low tides is increased moving northeast into the Bay, though the maximum difference is 33 cm. The overall agreement between the measured data and the Bay model predictions during the selected evaluation period supports the conclusion that the model is producing valid results at the scales of interest for the findings in this study.

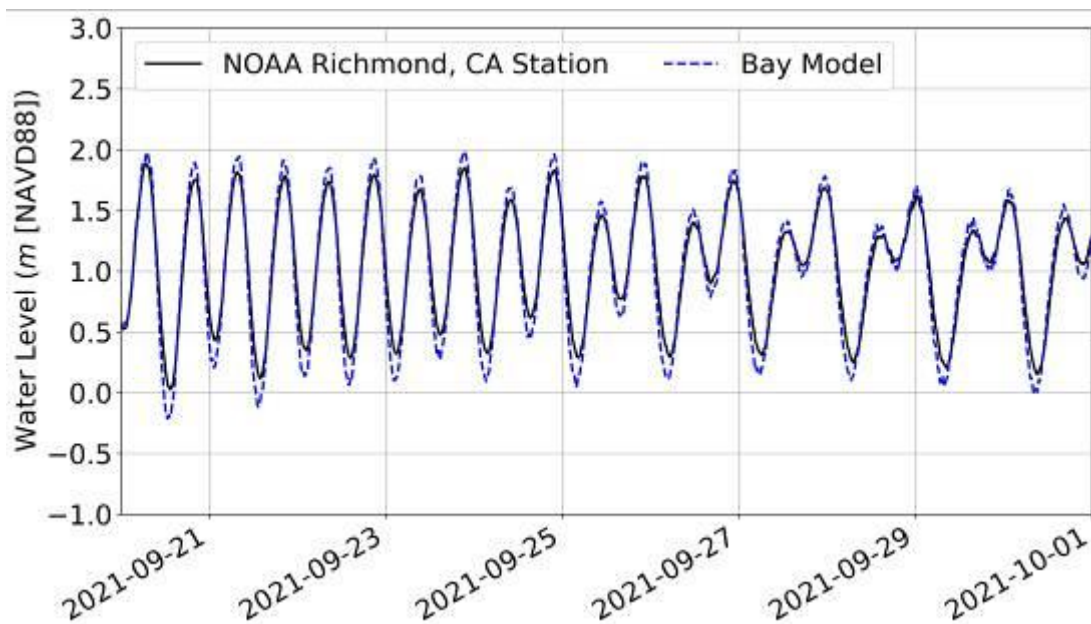


Figure 2-5. San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9414863 at Richmond, California.

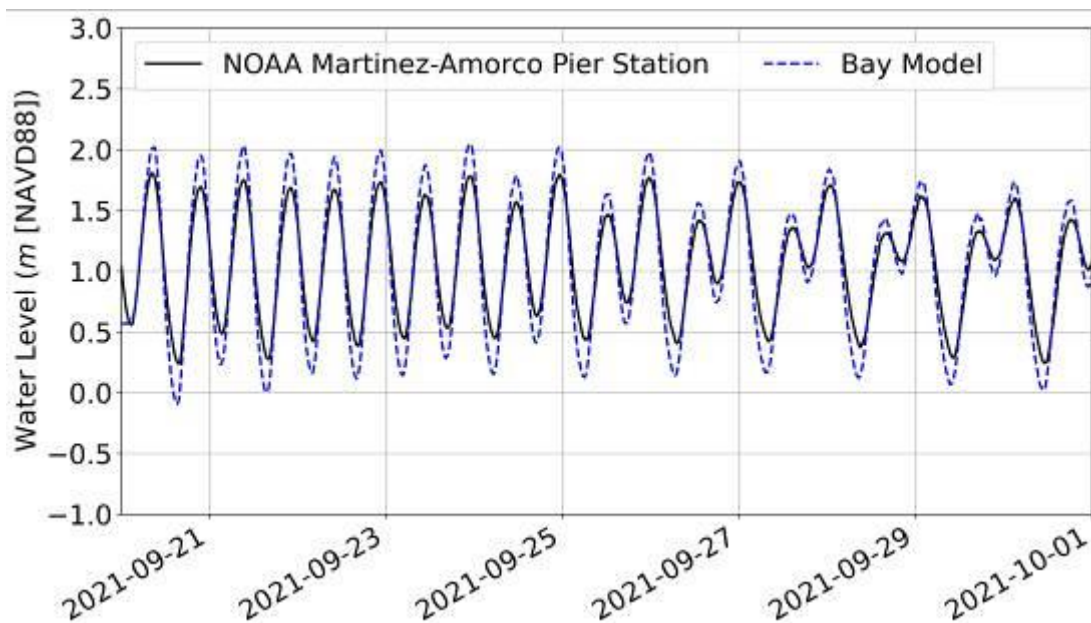


Figure 2-6. San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9415102 at the Martinez-Amorco Pier, California.

## 2.2 VALLEJO MARINA MODEL

A fundamental component and one of the first steps in the setup of a hydrodynamic model is constructing a model grid. The model grid discretizes the region of interest so that the parameters of interest (e.g., water level, velocity, shear stress) may be simulated accurately at a resolution appropriate for the project. For this study, Foth provided a previously developed D-Flow FM hydrodynamic model of the Mare Island Strait. The Vallejo Marina model domain encompasses the entire width of the Mare Island Strait and extends north to Sears Point Road and south to the mouth of the strait (Figure 2-7). The model resolution varied with the main stretch of river has a horizontal resolution of 22 m, and then two refinement areas with a transition zone of 9 m and 5 m within the marina.

The model was set up as 2-dimensional, meaning the water column is represented as a single depth-averaged layer. This assumption was made because of the relatively shallow depths within Mare Island Strait and assumed negligible effect of any salinity- or temperature-driven circulation given the limited flow input from the Napa River. The bathymetry used in the Vallejo Marina model was supplied by Foth (Figure 2-9).

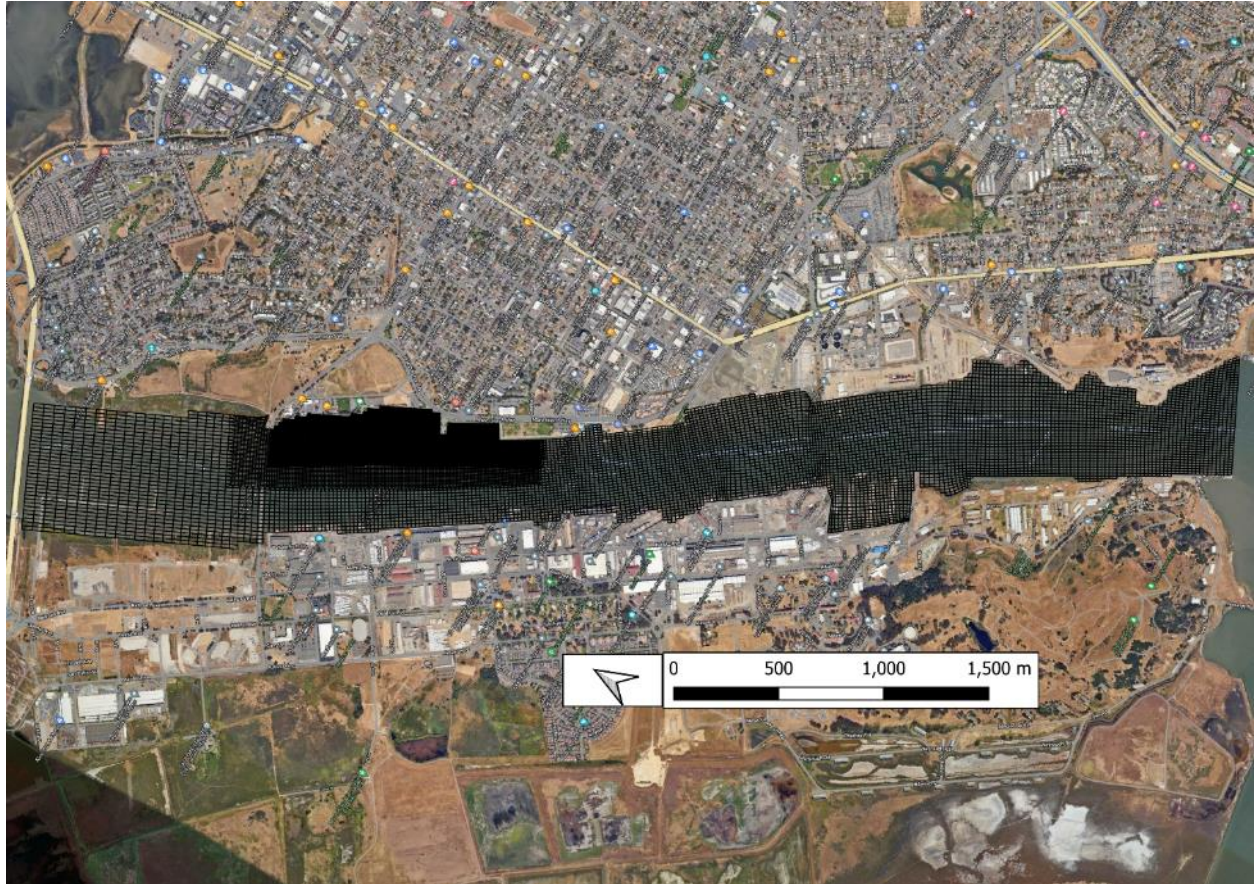


Figure 2-7. Overview of Vallejo Marina Model Grid

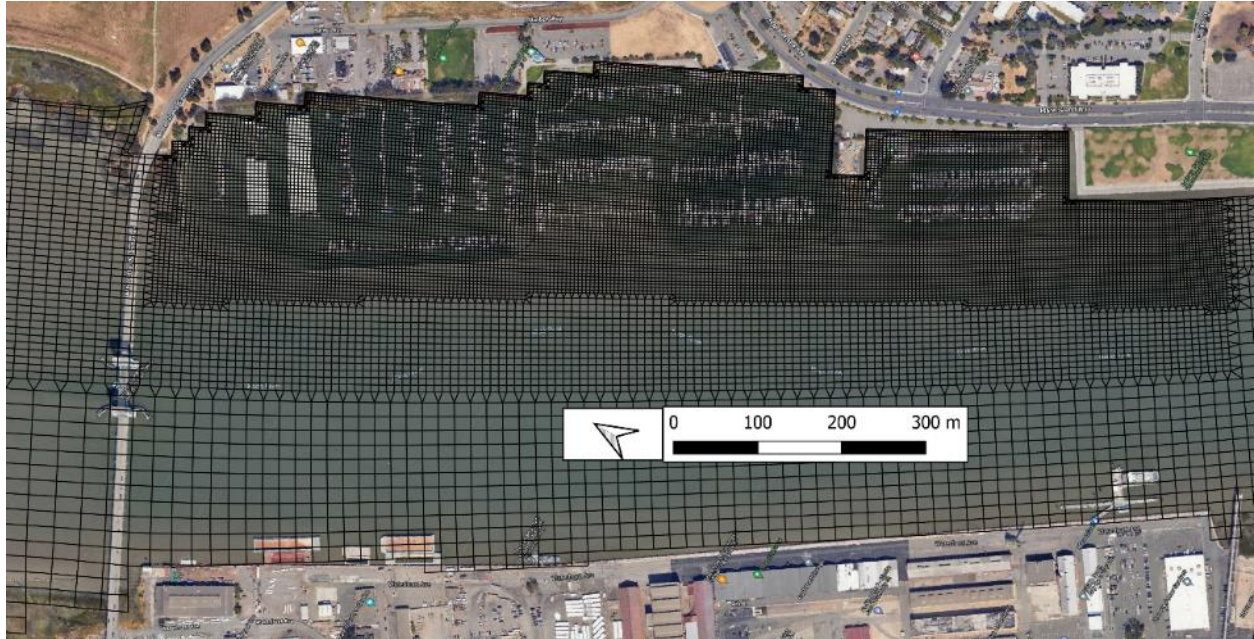


Figure 2-8. Vallejo Marina Grid refinement around marina.



Figure 2-9. Overview of Vallejo Marine model bottom elevations

The Vallejo Marina model was coupled with the Bay model offline to ensure the interaction of larger scale flows were included in estimations of bed shear stress. Offline coupling signifies that the San Francisco Bay-Delta Community model was run first, and then the outputs from that run were input into the Vallejo Marina model. The internal time-step of both models was

dynamic and changes to meet the model numerical stability criteria, allowing for a stable solution and potential decreases in computation time without sacrificing solution accuracy.

## 2.2.1 Hydrodynamic Forcing Conditions

Hydrodynamic conditions at the open northern and southern model boundaries provide the forcing that drives the water mass within the model domain and usually consists of the water level at a boundary of the model domain, or discharge into or out of the domain, or a combination of both. Two boundary conditions were applied to drive circulation within the Vallejo Marina model domain, riverine discharge at the north boundary and water level at the south boundary (Figure 2-10). These two boundary conditions for the model were extracted from the Bay model by means of defining output transects to save discharge and water level predictions where the model boundaries fall within the Bay model.

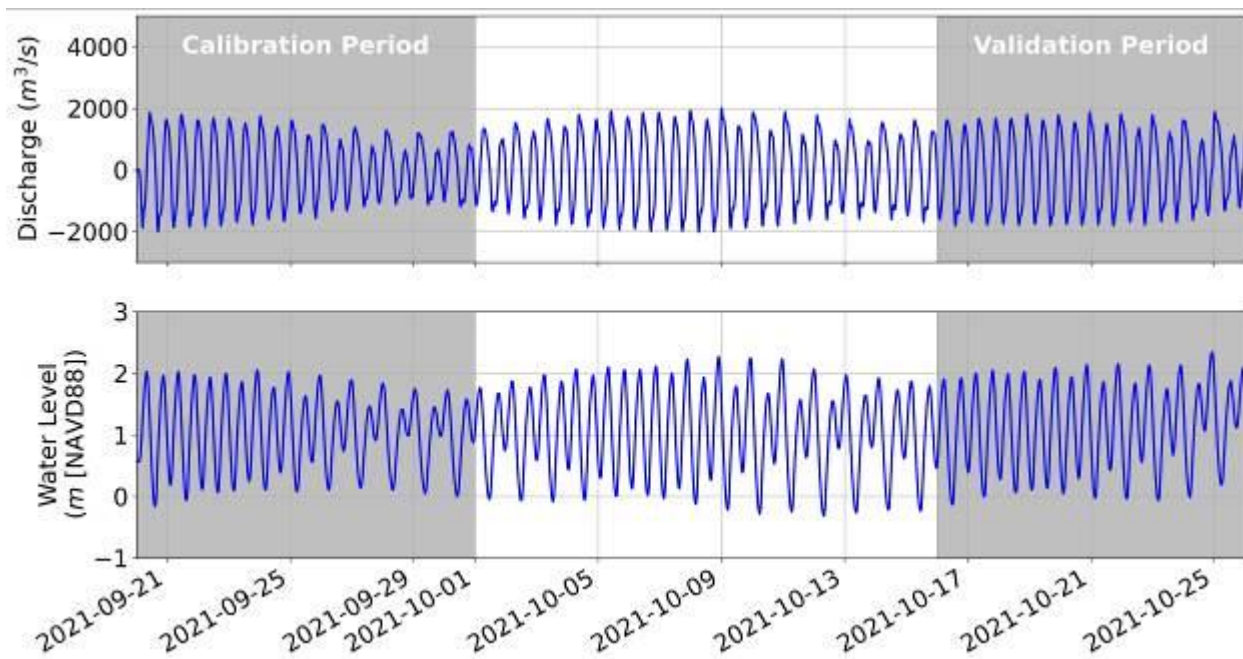


Figure 2-10. Vallejo Marina Model Discharge and Water Level Boundary Conditions Extracted from the Bay Model. The selected calibration and validation periods are shaded in gray.

## 2.3 VALLEJO MARINA MODEL EVALUATION

Evaluation of the Vallejo Marina model was performed to qualitatively and quantitatively assess how well it reproduced observed hydrodynamics within Mare Island Strait. Model evaluation consisted of two separate steps, model calibration followed by model validation. In addition, after the validation, the sensitivity of the model to various parameters was further

assessed. Model calibration and validation, as used in U.S. Environmental Protection Agency guidance (USEPA 2009), are defined as follows:

- **Model calibration**—Using site-specific information from a historical period to adjust model parameters to more accurately reflect measured site-specific conditions
- **Model validation**—Demonstrating that the calibrated model accurately reproduces known conditions over a period different from that used for calibration.

A single ADCP was deployed by Foth within Mare Island Strait, in a water depth of 5 m, in the main channel northwest of the Vallejo Ferry Terminal, to record current speed and direction. The ADCP was deployed in support of a project on the Vallejo Ferry Terminal, and the data was released for use in this project. Two deployments of the ADCP were made, the first, with a data record of September 20–October 26, 2021, and the second, with a data record of November 11–December 2, 2021. These measured current data were used to compute a depth-averaged velocity (DAV) and reviewed to identify time periods representative of the range of typical conditions observed at the Site (Figure 2-11). The negative values shown in the figure represent an ebb tide and the positive values are a flood tide. Based on the single Napa River flow event recorded during the first ADCP data record, two time periods were selected within this data record that encompassed spring and neap tidal cycles. No Napa River discharge events were recorded during the second ADCP deployment. The time period of September 20–October 1, 2021, was selected for the model calibration. This date range included a spring to neap tide transition and no flow from the Napa River. October 16–26, 2021, was selected for the model validation. This date range encompasses a neap to spring tide transition and a Napa River flow event, with a maximum discharge of 241 m<sup>3</sup>/s.

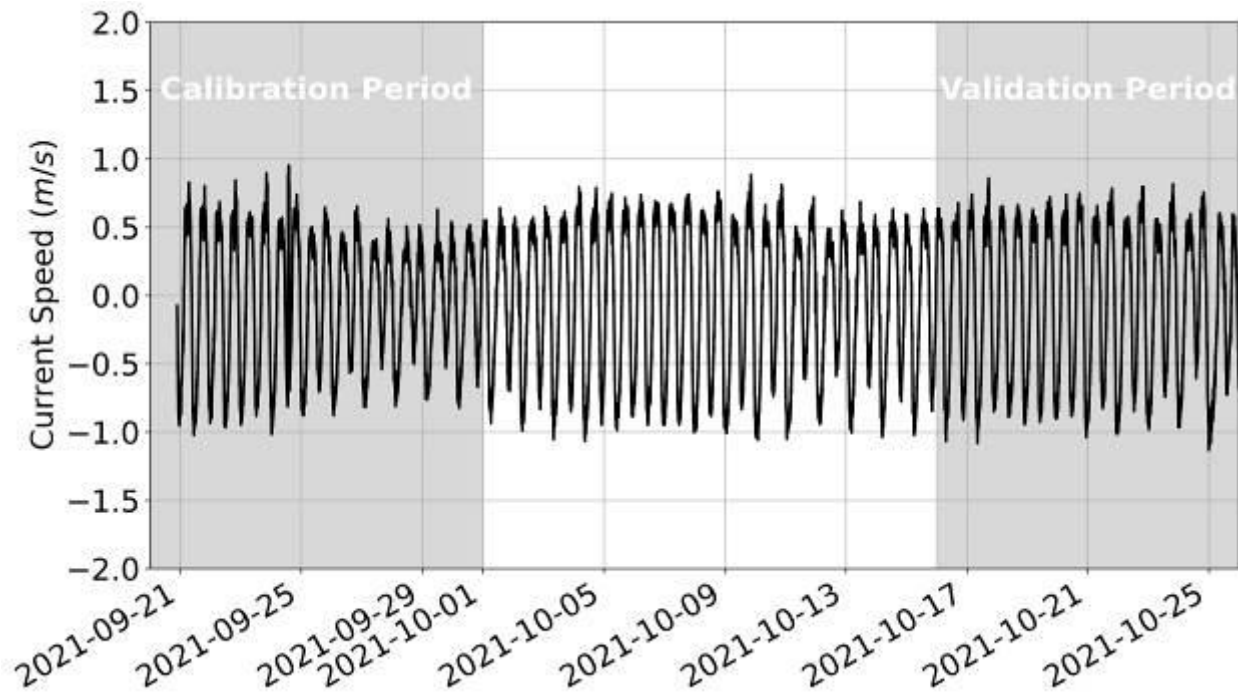


Figure 2-11. ADCP depth averaged current velocity used for calibration and validation of the hydrodynamic model. The selected calibration and validation time periods are shaded in gray.

The flow model was assessed over multiple spring-neap cycles during the calibration period and validation periods to ensure the model could adequately capture the full range of tidal elevations. In this system, successful calibration of the flow model relied on modifying the hydrodynamic bottom roughness within the domain, modifying the discharge boundary condition extracted from the Bay model. The bottom roughness around the Site was used as a calibration factor, to align velocity magnitudes with the measurements and a value of 0.026 m was applied uniformly throughout the domain. Within the area of flow grid refinement around the Site, the bottom roughness supplied by the Bay model was set to a constant value. The smaller scale processes resolved as a result of increased grid resolution, and the inclusion of structures at the Site necessitated the divergence from the values supplied in the Bay model.

### 2.3.1 Model Performance Metrics

Quantitative metrics were selected and used to evaluate the model calibration and validation performance, or the ability of the model to accurately reproduce measured water level and velocity at the Site. These metrics, combined with the qualitative evaluation (e.g., visual inspection of model results), provide an assessment of how well the model predicts the measured values at the study site. The metrics used to evaluate model performance include the model skill score (Murphy and Epstein 1989), root mean square error (RMSE), Pearson's

correlation coefficient, and skew. The model skill score served as the primary basis for evaluating model ability across a range of parameters and was computed as follows:

$$SS = 1 - \frac{\sum(x_{meas} - x_{model})^2}{\sum(x_{meas} - \bar{x}_{meas})^2}$$

Where  $x_{meas}$  is the measured value,  $x_{model}$  is the model-predicted value, and  $\bar{x}_{meas}$  is the average measured value.

The skill score varies from 0 to 1, with 1 representing perfect agreement between measured and modeled data. The model skill values, as well as Pearson's correlation coefficient, described below, are characterized in Table 2-1, based on values presented in Marechal (2004).

Table 2-1. Model Skill Performance Metrics Set for This Study

Skill Score	Performance
0–0.2	Poor
0.2–0.5	Good
0.5–0.65	Very Good
0.65–1	Excellent

The RMSE is a measure of the average difference between model-predicted and measured values and was computed as follows:

$$RMSE = \sqrt{(x_{model} - x_{meas})^2}$$

The computed RMSE was compared with measured values such that an acceptable RMSE value for current speed verification is  $\pm 30$  percent of the average measured values (USEPA 1990).

Finally, Pearson's correlation coefficient was used to ensure the model is qualitatively reproducing the temporal variability in measured values by computing the linear association of modeled and measured values. The correlation coefficient ( $r$ ) was computed as follows:

$$r = \frac{n \sum(x_{meas}x_{model}) - \sum(x_{meas})\sum(x_{model})}{\sqrt{[n \sum(x_{meas}^2) - (\sum x_{meas})^2][n \sum(x_{model}^2) - (\sum x_{model})^2]}}$$

Where  $n$  is the number of data points in the time series. Evaluation of Pearson's correlation coefficient depends on the number of samples compared; the values vary from 0 (no correlation) to 1 (perfect correlation). The Pearson's  $r$  value can be classified with the same performance values as used for the model skill score (Table 2-1).

Together, these metrics provide a robust evaluation of the model's ability to reproduce measured data. Model calibration is considered complete when the model skill score falls within the very good to excellent range, or if it is determined that available data are insufficient to support additional model calibration.

### **2.3.2 Vallejo Model Calibration**

The hydrodynamic model calibration consisted of systematic adjustments to model parameters within a reasonable range until measured Site conditions were accurately simulated by the hydrodynamic model. Calibration involved the following steps:

- Adjustment of model calibration parameters, including bottom roughness and the horizontal eddy viscosity coefficients.
- Adjustment of the north boundary discharge from the Bay model.

Upon initial analysis of the measured data versus modeled predictions, the Vallejo Marina model was underpredicting the ebb tide and flood tide velocities from the Mare Island Strait ADCP. The adjustment of bottom roughness and viscosity, calibration parameters that influence velocities, had minimal impact on the underpredictions. The bottom elevations of the hydrodynamic model were found to be accurate when compared to measured values. This led to the conclusion that the discharge boundary conditions, derived from the Bay model, were the primary factor in the model predictions. The Bay model was likely not discharging enough flow volume into the Mare Island Strait. The discharge boundary condition was iteratively adjusted and compared to the measured DAV. The final adjustment resulted in a doubling of the ebb tide discharge and a 20 percent increase in the flood tide discharge (Figure 2-12). The water level boundary condition derived from the Bay model was not adjusted. This adjustment of the discharge boundary condition was necessary to achieve the model calibration and validation presented below. In addition, the evidence of the shallow bathymetry of the Bay model within the Mare Island Strait presented in Section 2.1.2, which likely extends upstream into the Napa River, supports the need for adjustment of the discharge provided by the Bay model.

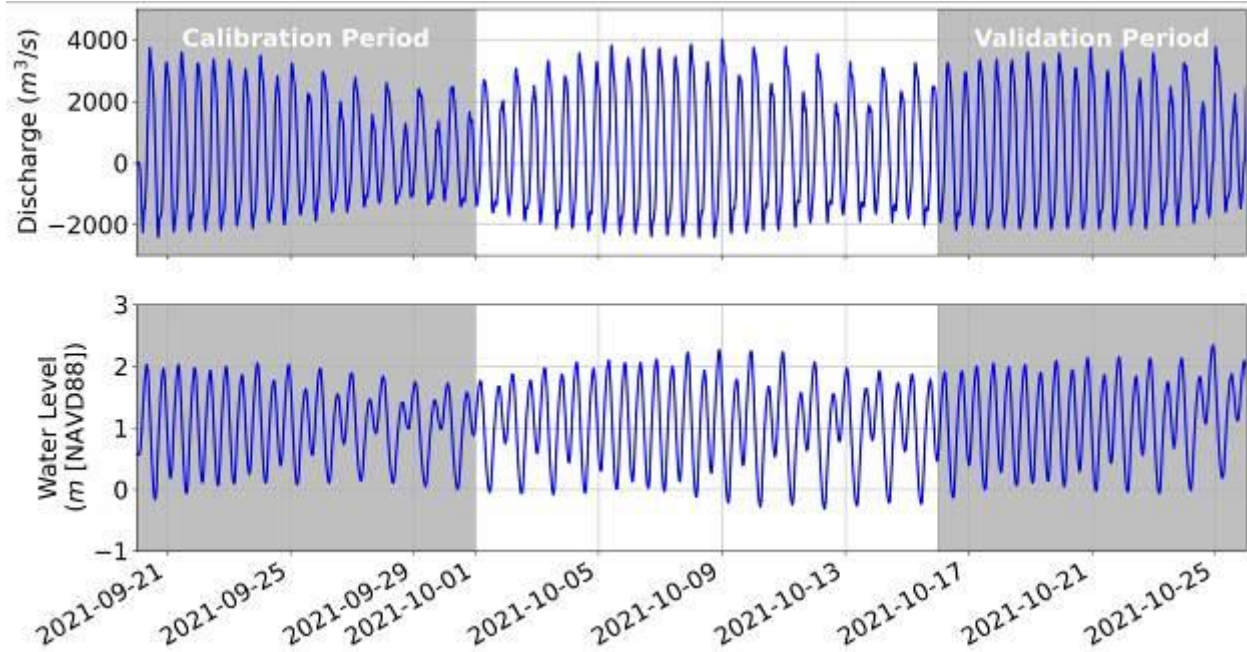


Figure 2-12. Discharge and water level boundary conditions. The selected calibration and validation periods are shaded in gray.

The measured versus modeled DAV, after incorporating the calibration changes described above, is presented in Figure 2-13. The negative values shown in the figure represent an ebb tide and the positive values are a flood tide. The model captured the timing and the magnitude of the depth averaged current for flood and ebb tides during the spring to neap tide transition. The model does underpredict the current speed on ebb tide moving into the neap tide at near the end of the calibration period, though the difference in the measured data and model predictions are small and would have a minimal impact on the predictions of bed shear stress.

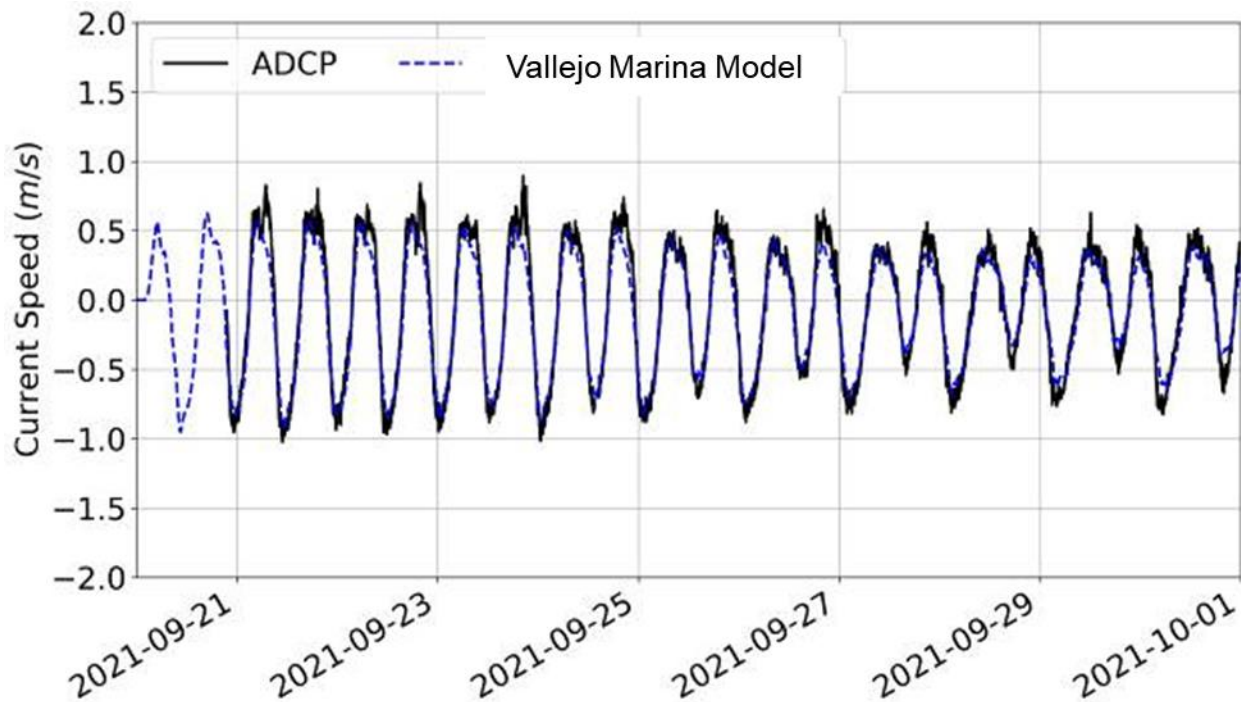


Figure 2-13. Vallejo Marina Model calibration to depth averaged velocity from the Mare Island Strait ADCP.

When the quantitative model performance metrics for the calibration period were computed, the DAV was in excellent agreement with the measured data, as shown by the skill and  $r$  values (Table 2-2). The Pearson’s  $r$  value, a performance metric used to evaluate how well the model is reproducing the temporal variability in measured values, showed an excellent correlation between the measured and modeled DAV. The RMSE value, representative of the difference between the measured and modeled values, was within 2 percent of the 30 percent limit defined by USEPA (1990). The quantitative metrics paired with the calibration figures support a good model calibration. Though there is under- and overprediction, it is likely a result of the uncertainty in the bottom elevations and the boundary conditions supplied by the Bay model. Model uncertainty is discussed further in Section 2.4.

Table 2-2. Calibration Performance Metrics for Vallejo Model

	Skill	RMSE	Pearson’s $r$	Performance
Depth Averaged Velocity	0.98	0.12 m/s (32%)	0.98	Excellent

### 2.3.3 Vallejo Model Validation

The model validation was performed to test the selected calibration parameters during hydrodynamic conditions that are different from the calibration conditions. Compared to the calibration conditions, the model validation conditions represent a higher flow rate period resulting from the single recorded discharge event from the Napa River.

Similar to the calibration period, the predicted DAV was compared to the measured DAV recorded by the ADCP (Figure 2-14). The model captured the timing and the magnitude of the depth averaged current for flood and ebb tides during the neap to spring tide transition. The model does underpredict the current speed on flood tide moving into the spring tide at near the end of the validation period, though the differences in the measured data and model predictions are small. The model does well at predicting the elevated ebb tide DAV on October 25, 2021 as a result of the Napa River discharge. The small underpredictions of the flood tide DAV would have a minimal impact on the predictions of bed shear stress.

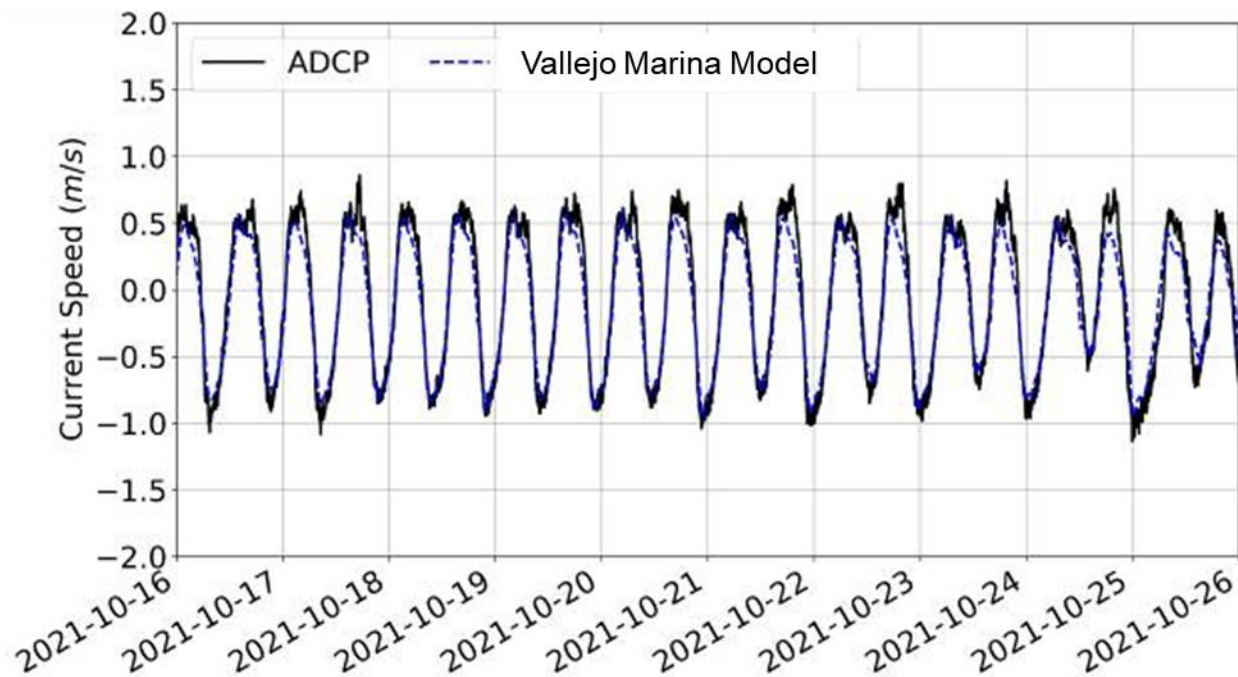


Figure 2-14. Vallejo Marina Model Validation to Depth Averaged Velocity from the Mare Island Strait ADCP.

When the quantitative model performance metrics for the validation period were computed, the DAV was in excellent agreement with the measured data, as shown by the skill and  $r$  values (Table 2-3). The RMSE value, representative of the difference between the measured and modeled values, was within 4 percent of the 30 percent limit defined by USEPA (1990).

Table 2-3. Validation Performance Metrics for Vallejo Model

	Skill	RMSE	Pearson's <i>r</i>	Performance
Depth Averaged Velocity	0.98	0.15 m/s (34%)	0.98	Excellent

Overall, the model calibration and validation are in good to excellent agreement with the variations in DAV. Given the objectives of this modeling effort, evaluating bed shear stress and possible sedimentation in and around the proposed Vallejo Marina modifications, the calibrated and validated model satisfactorily captures the temporal variations and magnitudes of the measured values, which are the primary forcing components of bed shear stress and associated sedimentation.

## 2.4 MODEL UNCERTAINTY

Overall, the hydrodynamic models adequately capture site dynamics that are important for effective remediation design. This is supported by the good fit between measured and simulated hydrodynamic data. The models are meant to be used as tools to aid remediation design, and the parameters used in the model are subject to uncertainty.

Typical sources of uncertainties in a hydrodynamic model arise from the model grid setup including bathymetry, boundary conditions, and choice of hydrodynamic parameterizations. Subgrid-scale processes, such as bottom topography variation within a grid cell, are a source of uncertainty in any hydrodynamic model. The Site also contains many structures that modify and direct flows, shelter areas from wave activity, and provide berthing areas for vessels that may have an influence on localized dynamics.

Errors in estimating the bottom topography used in the model arise from both measurement uncertainties and the finite grid scale. Errors from the finite grid scale occur due to the use of an average depth for the entire grid cell, whereas nature might exhibit more vertical variation in an area than can be represented by a single elevation in a model grid cell. Overall, the bottom topographic variation away from the banks of Mare Island Strait is minimal, and the high-resolution grid captures the variations in the Site well.

Boundary conditions for the model consisted of measured water levels from the Golden Gate Bridge. While the water level data are validated, the water levels were measured 40 km from the model boundary and were adjusted temporally to align model and measurement phases.

The instruments used to collect velocity data have a cited precision value of  $\pm 2.2$  cm/s. Maximum depth-averaged velocity magnitudes were relatively low, ranging from 0.5 to 1.0 m/s, which means the velocities could vary as much as 22 percent at the lower maximum value. The velocity data have uncertainty in interpretation and application for validation. The velocities are

measured at points that are smaller than the grid scale and include other sub-grid scale turbulent processes that contribute to uncertainty in model and data comparison.

### 3 SEDIMENTATION STUDY

#### 3.1 EXISTING DATA SETS

Foth provided existing bathymetric survey files of the Vallejo Marina in the form of delimited text files. Dates of surveys and whether they were used in subsequent analysis are shown in Table 3-1. All datasets were converted to a common vertical reference system, NAVD88 meters relative to the Martinez-Amorco Pier (9415102) NOAA tide gage station. All XYZ surveys were then mapped in GIS and projected into UTM 11 coordinates. Surveys were then interpolated from point locations to a raster grid. This was done in two ways, surveys with an evenly spaced grid were converted directly from points to raster. Surveys with unevenly spaced grids were interpolated into raster geotiffs using the nearest neighbor interpolation method at a resolution determined by the spacing of the survey locations. A post processing step was then conducted to fill small gaps in the raster surface, and the raster was clipped by the outer bounds of the survey. For consistency, all surveys were snapped to a common grid and resampled to standard 1-meter resolution.

Table 3-1. Bathymetry survey dates considered in sedimentation analysis

Date	Used in Sedimentation Analysis	Date	Used in Sedimentation Analysis
2013 June 1		2016 Oct. 17	
2014 Apr. 2	X	2019 June 24	X
2014 June 19	X	2019 June 26	X
2014 July 16	X	2019 Aug. 02	X
2014 July 21	X	2019 Aug. 29	X
2014 Oct.16	X	2019 Oct. 11	X
2014 Oct. 29		2019 Oct. 21	
2014 Oct. 29	X	2019 Oct 21	
2014 Oct. 31		2022 May 02	X
2016 Jan. 01	X	2022 Aug. 01	X
2016 June 01	X	2023 Sep. 22	X
2016 Oct. 17	X		

Each raster was further standardized to a uniform a 1-m grid with bounding coordinates of (563,665 m, 4,217,696m) and (564,382m, 4,218,510m) to account for variations in spatial extent and raster cell coverage using the *RegularGridInterpolator* routine of the Python package *scipy*. The spatial occurrence of surveys is shown in Figure 3-1. This shows that many surveys are

limited to the marina openings and along a few docks. Often a single survey covers some of the marina and the marina extents along the north and south walls have no coverage.

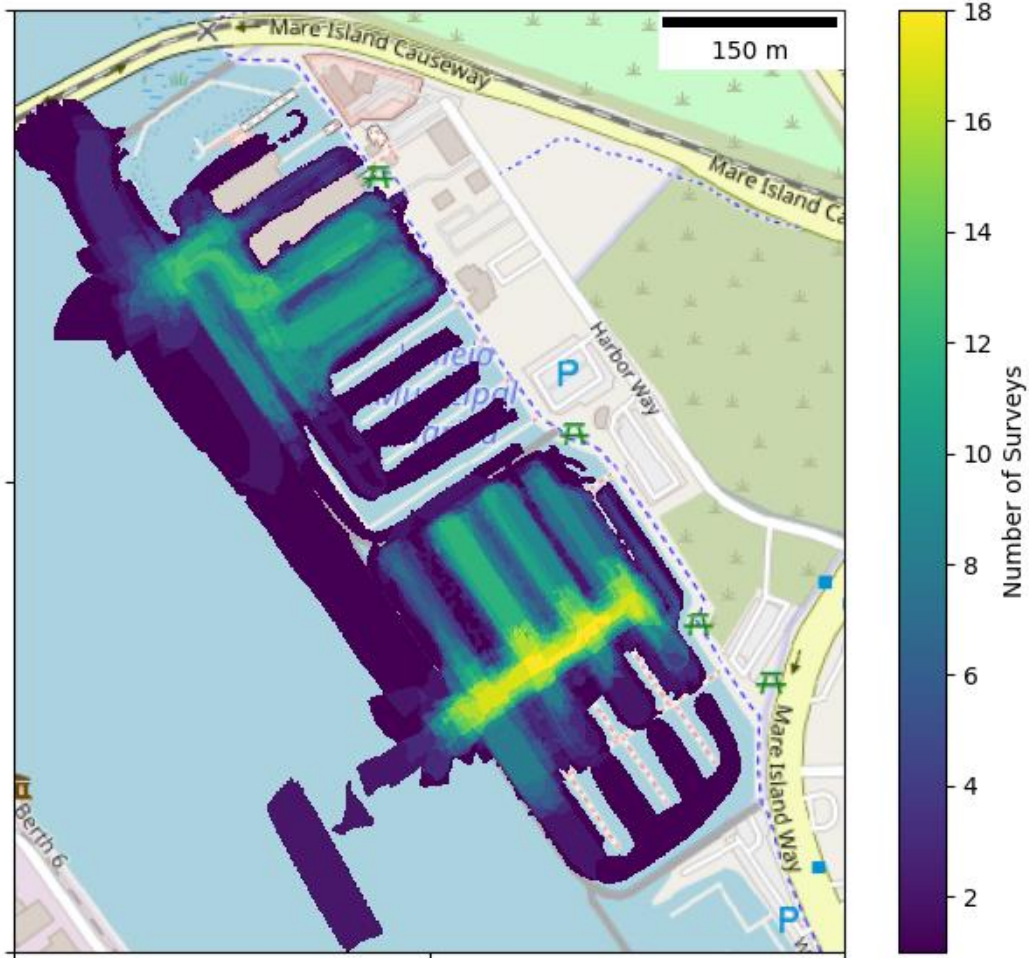


Figure 3-1. Spatial occurrence of available surveys

Some surveys either contained duplicate data or were only a few days apart, resulting in incorrect change rates and/or noise due to the short time period. For these survey files, only the earliest date and the survey with the largest spatial coverage were included. This resulted in 17

datasets ranging from 2014 to 2023 covering pre- and post-dredge marina conditions (Table 3-1).

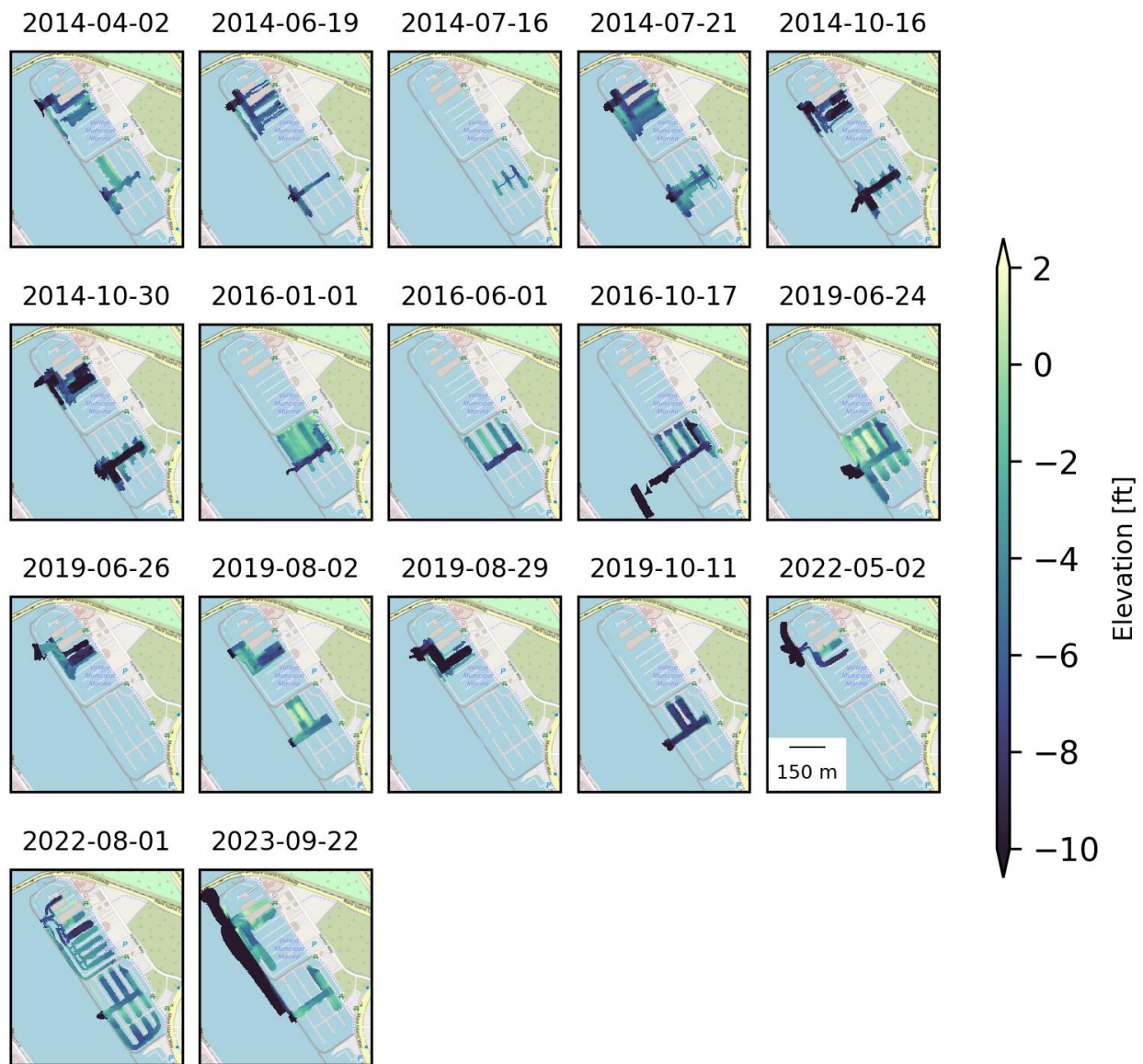


Figure 3-2. Vallejo Marina bathymetry referenced to NAVD88.

### 3.2 DEPOSITION RATE CALCULATION

The individual surveys were grouped for the north and south marina basins to determine sedimentation rates and between post- and pre-dredge surveys. The dates of dredging were inferred from documents provided by Foth and .xyz file names indicating pre-/post-dredge and are listed in Table 3-2. Due to the sparse spatial coverage of successive surveys, a direct elevation change between successive individual surveys was not possible for most cases. This

resulted in 4 estimates of sedimentation rate for the north basin and 6 for the south basin, each with limited spatial coverage due to preferential surveys near the marina entrances. The elevation changes for north basin are shown in Figure 3-3 and the south basin elevation change is shown in Figure 3-4. The bathymetric change maps indicate periods of accretion and erosion as well as infilling within the basins following dredge events. From the maps, the greatest depositional rates occur within the main navigable channels from the marina entrances to the north and south basin and decrease further within the basin. Erosion primarily occurs outside the marina in the straight.

Table 3-2. First post-dredge surveys

Date
2014 Oct. 16
2019 Aug. 2

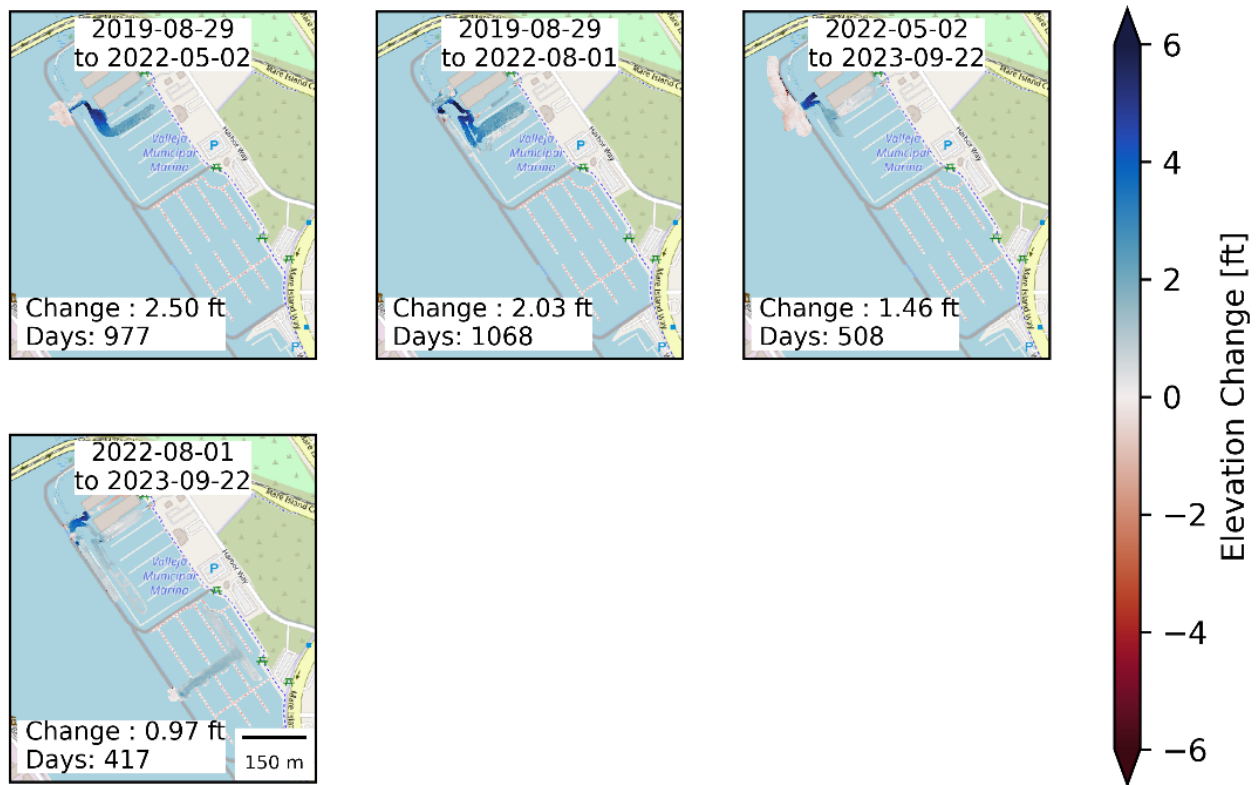


Figure 3-3. Inter-dredge north basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text).

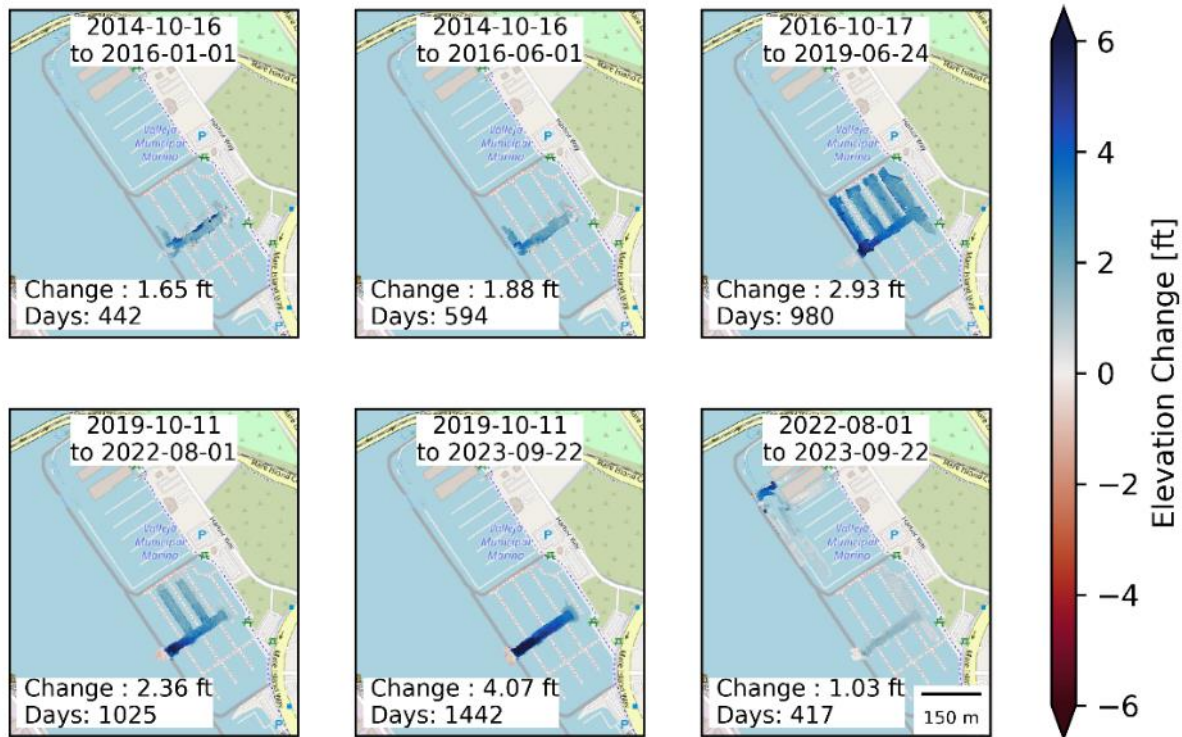


Figure 3-4. Inter-dredge south basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text).

The natural deposition rate was calculated from bathymetric change for each of these survey pairs for the north and south basins. The largest rates of sediment deposition occur within the main navigable channel from the marina entrances. The variation in average sediment deposition is attributed to the differences in spatial extent of the surveys with survey changes limited to the main channels resulting in larger accumulation rates than at more distal extents. This indicates a decrease in sediment availability due to sediment depositing out of the water column due to decreases in water velocity as the flow enters the marina. For the north marina basin (Figure 3-5), the deposition rate varies between 0.69 to 1.05 ft/yr. For the south marina basin (Figure 3-6), the deposition rate varies between 0.84 to 1.36 ft/yr.

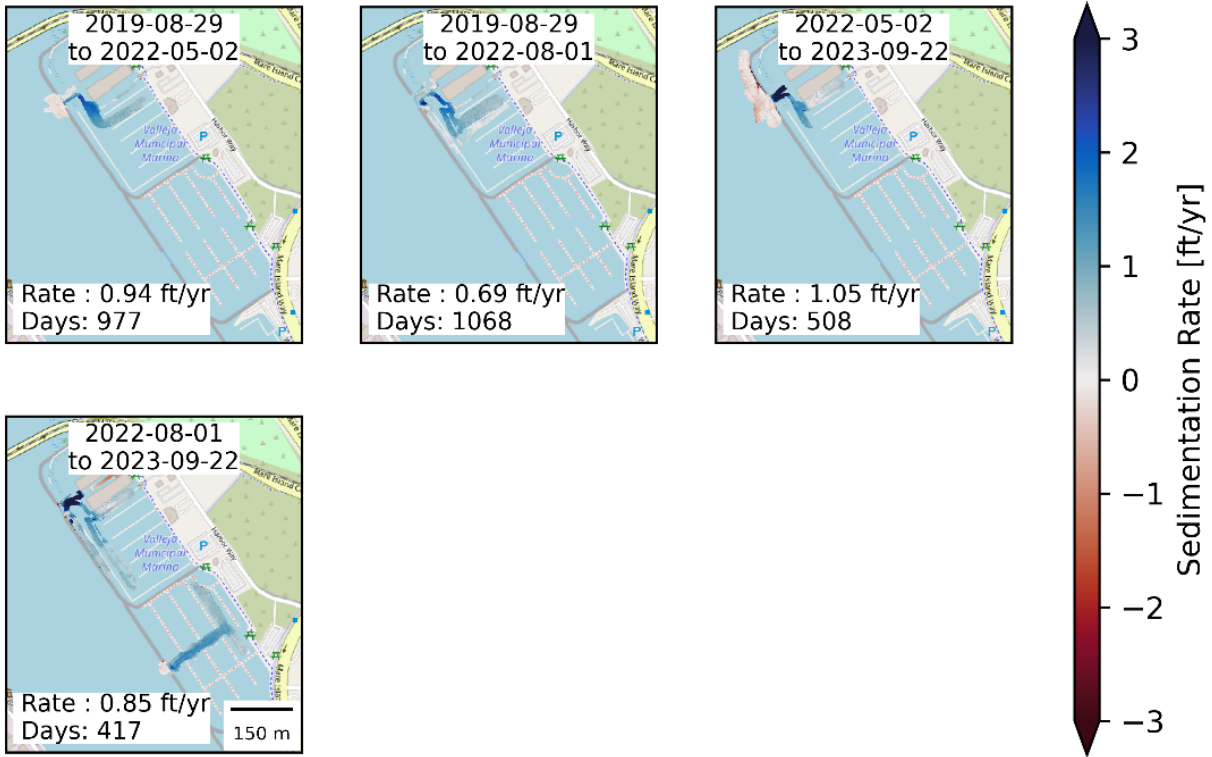


Figure 3-5. Inter-dredge deposition rate for the north basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text).



Figure 3-6. Inter-dredge deposition rate for the south basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text).

The average, median, and time-weighted average deposition rate, calculated for each 1-m grid cell is shown in Figure 3-7. The time weighted average results in a north basin sediment deposition rate of 0.85 ft/yr and a south basin deposition rate of 0.99 ft/yr. Due to the sparse survey data from which to calculate a sediment deposition rate, the marina with unknown rates was assumed to have the time-weighted mean deposition rate for the north basin and south basin. The resulting spatial depositional rate is shown in Figure 3-8.

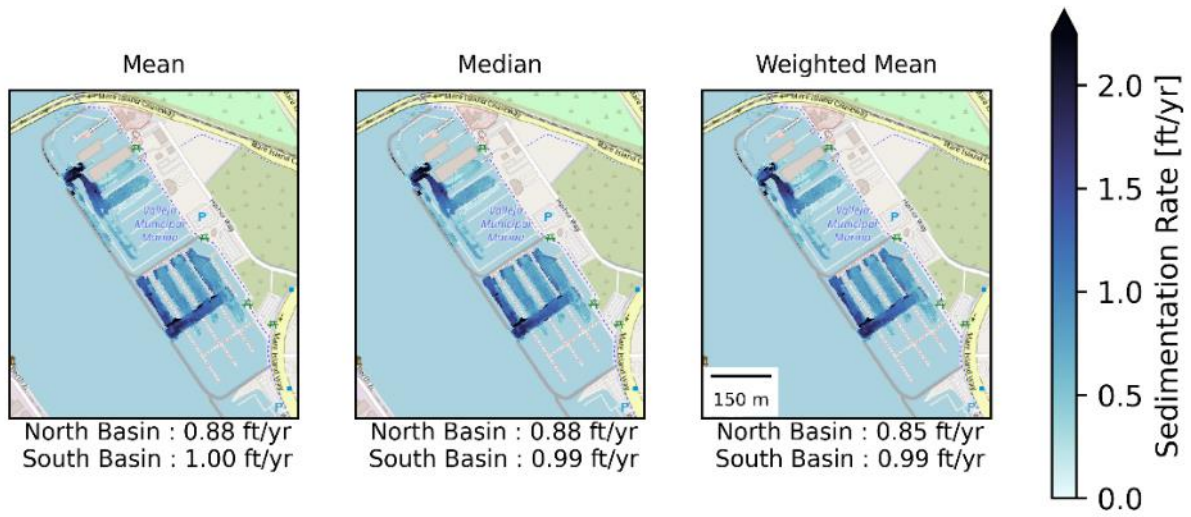


Figure 3-7. Calculated average (left), median (middle) and time-weighted (right) deposition rate.

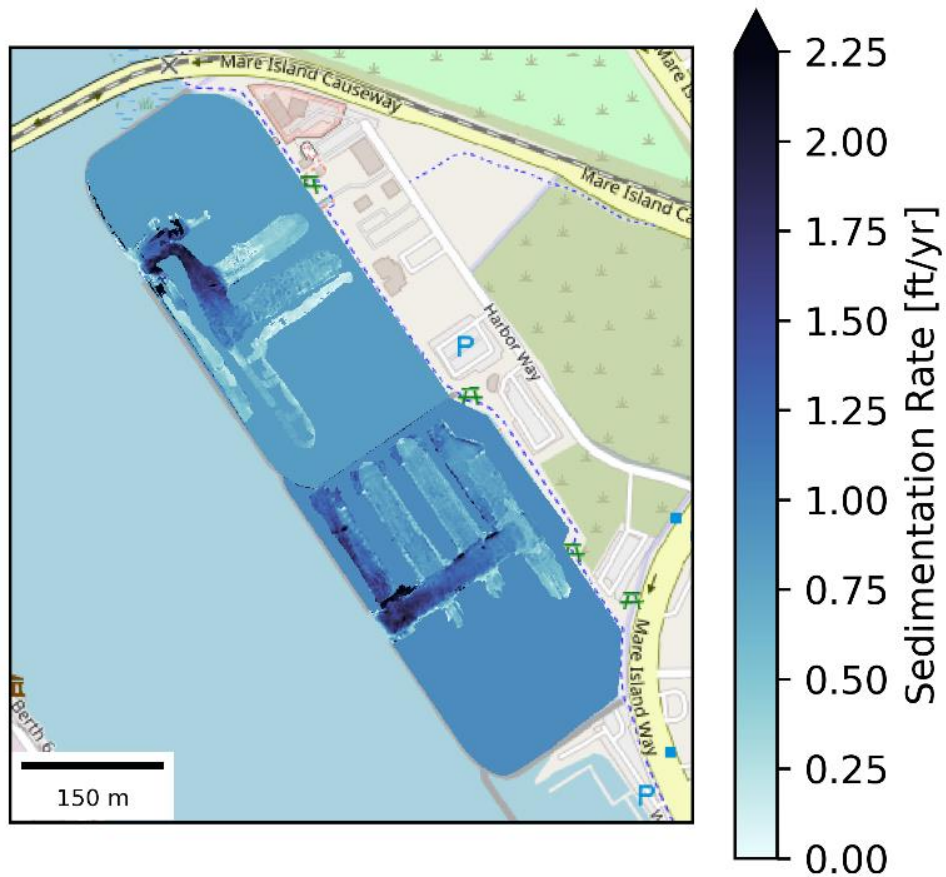


Figure 3-8. Time-Weighted Average Deposition Rate (top) and Maximum Deposition Rate (bottom).

## 4 ALTERNATIVE MARINA CONFIGURATIONS: MODELING AND ANALYSIS

The Vallejo Marina model was used to simulate typical site conditions over a 45-day period with alternative marina configurations. The alternative marina configuration modeling utilized the same model setup derived from the calibration and validation study. The sedimentation study produced estimates of the sedimentation rate in the system. The modeling analysis provides an estimate of maximum and average shear stress, the force that governs the ability for sediment to erode. The analysis performed here scales the measured deposition rates by the modeled shear stresses to estimate a new deposition rate within the marina for each alternative. The configuration of the alternative simulations and the results of the analysis are presented below.

The alternative marina configurations use an assumed post-dredge bathymetry. For this bathymetry, the north marina basin is dredged to an elevation of -10 ft MLLW and the south basin to an elevation of -8 ft MLLW. The sumps are dredged to a depth of -20 ft MLLW.

For Case 2, the present marina footprint is used with the middle wall separating the north and south basin removed. A sump is added to the north-basin oriented east-west along the north wall. For Case 3, an additional marina entrance is added near the existing wall separating the north and south basin.

The Alternative 1 concept was proposed by F3 (Case 4). This concept moves the north wall southward, removes the marina wall separating the north and south basins, adds a new north entrance to the marina, and adds a third entrance near the middle of the marina. Case 5 adds two sumps, one oriented east-west along the north wall and another located east-west at the middle marina entrance.

The Alternative 2 (Case 6) was proposed by F3. This concept moves the north wall further southward from the Alternative 1 location, removes the marina wall separating the north and south basins, adds a new north entrance to the marina, and adds a third entrance near the middle of the marina. Case 7 adds two sumps, one oriented east-west along the north wall and another located east-west at the middle marina entrance.

The Alternative 3 (Case 8) was proposed by F3. This alternative has the same marina footprint as Alternative 2 but orients the northern sump north-south along the eastern marina wall.

## 4.1 ALTERNATIVE MARINA CONFIGURATION MODEL EVALUATIONS

A total of seven marina model configurations are evaluated including the above alternatives are evaluated with and without sumps, the existing marina configuration pre-dredge and the existing marina configuration with the middle wall separating the north and south basin removed are evaluated. A list of the model scenarios evaluated, and the associated bathymetry and wall configurations are shown in Table 4-1 and Figure 4-1.

Table 4-1. Marina configuration model simulations.

Case Number	Bathy	Marina Walls	Sumps
1	Pre-Dredge	Existing	None
2	Post-Dredge	No mid-wall	Included North Basin Sump
3	Post-Dredge	No mid-wall, added mid-entrance	Included North Basin Sump
4	Post-Dredge	Shifted North Wall South and added mid-entrance	None
5	Post-Dredge	Same as Case 4	Two Sumps in North Basin
6	Post-Dredge	Shifted North Wall and added Mid entrance	None
7	Post-Dredge	Same as Case 6	Two Sumps in North Basin
8	Post-Dredge	Same as Case 6	Two Sumps in North basin, in nav channels

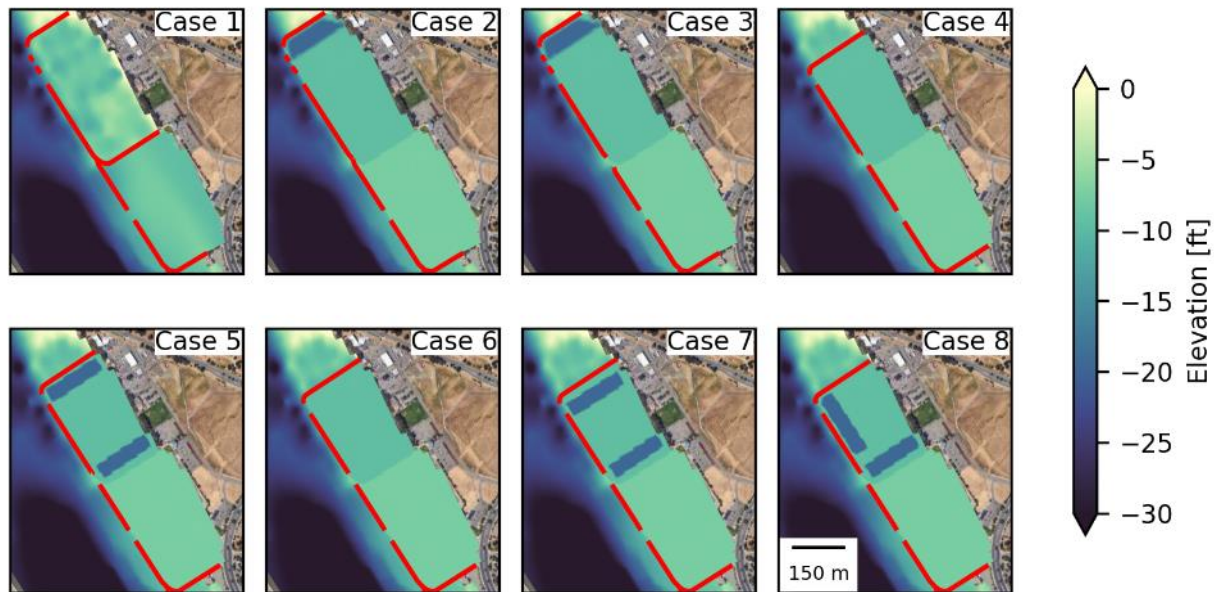


Figure 4-1. Bathymetry and marina wall configuration of the alternative analysis evaluation. NAVD88 CA State Plane Zone 2 with units of meters with vertical datum of NAVD88 with units of feet. Red lines indicate marina walls.

## 4.2 VELOCITY

Velocity within the marina is indicative of sediment flow pathways and the potential path of sediment redistribution. There are two primary tidal flows that are the focus of this study. Flood-tide results during north-westward directed flow due to higher water elevations in San Francisco Bay while ebb-tide results in south-eastward directed flows due to receding water levels in San Francisco Bay. Sediment is introduced to the marina on flood tides and then settles out as currents change direction and slack tide occurs. The ebb tide may not be strong enough to resuspend sediment and flush the system, resulting in deposition.

The maximum velocity within the marina is calculated as the maximum velocity per grid cell during the complete model simulation (Figure 4-2). This indicates that velocities can exceed 3 ft/s near the marina entrances with the fastest velocities often occur within the southern marina basin. For Case 1, faster velocities are confined to the northern section of the north and south basins. Removing the dividing wall in Case 2 results in velocities being more evenly distributed through the basin with slower velocities near the channel wall near the middle of the marina and along the southern most portion of the south basin. Slower velocities also occur within the sump in the north basin due to the increase water depth. In Case 3, adding the extra middle marina entrance results in slower velocities in the southern section of the south basin and the sump. Moving the north wall south in Case 4 results in faster velocities near the marina entrances with faster velocities extending further into the south basin. Adding sumps in Case 5 reduces velocities in the north basin as the faster velocities near the north entrance are

dampened by the deep water in the sump. This also occurs due to the sump near the middle entrance. Moving the north wall further south for Case 6 increases the extent of faster velocities into the marina in the south basin with similar velocities for the middle and north entrance to Case 4. Adding sumps in Case 7 and Case 8, dampen the velocities for the middle and north entrance, however the east-west oriented sump in Case 7 dampens the velocities through the north entrance more than the north-south sump in Case 8.

Comparing the maximum velocities for each alternative to Case 1 (Figure 4-3), Case 2 results in a slower velocity in the northern half of the north basin and an overall slower velocity in the south basin but faster velocity within the southern half of the north basin where flows were previously restricted. A similar pattern is observed for Case 3; however, there is increased flow velocities in the southern half of the north basin along the channel wall due to the middle entrance. Cases 4 through Case 8 result in faster velocities throughout most of the marina including the south basin south of the southern marina entrance. These cases also have reduced flow in the northeastern corner of the marina and areas of the northern half of the south basin, though less reduction than Cases 2 and 3.

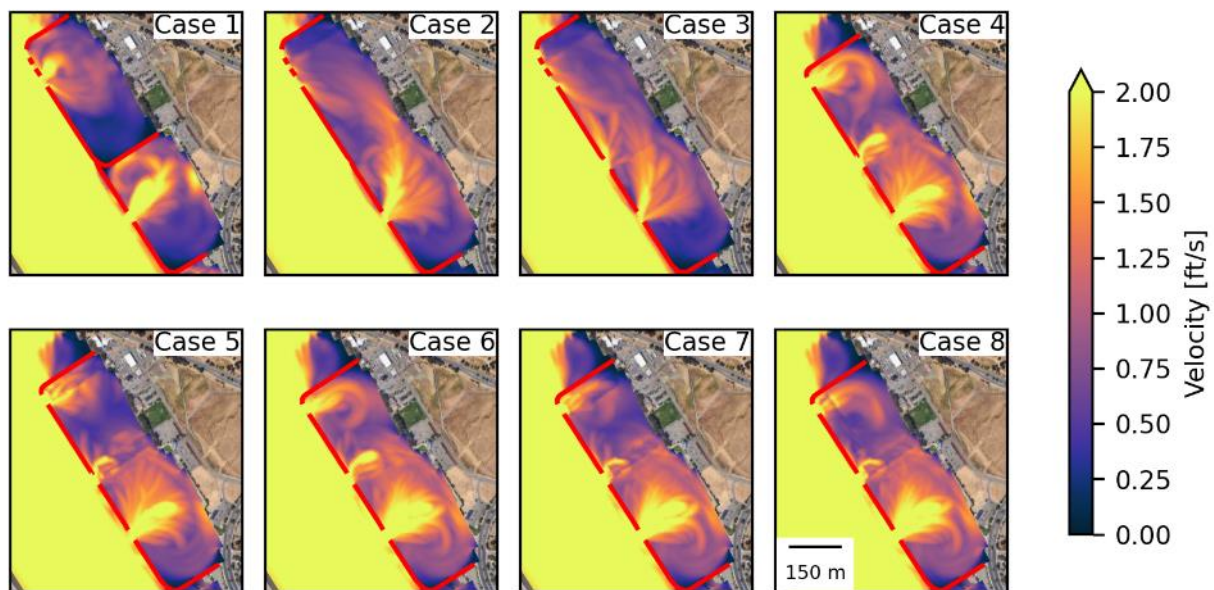


Figure 4-2. Maximum grid-cell velocity during the model simulation. Red lines indicate marina walls.

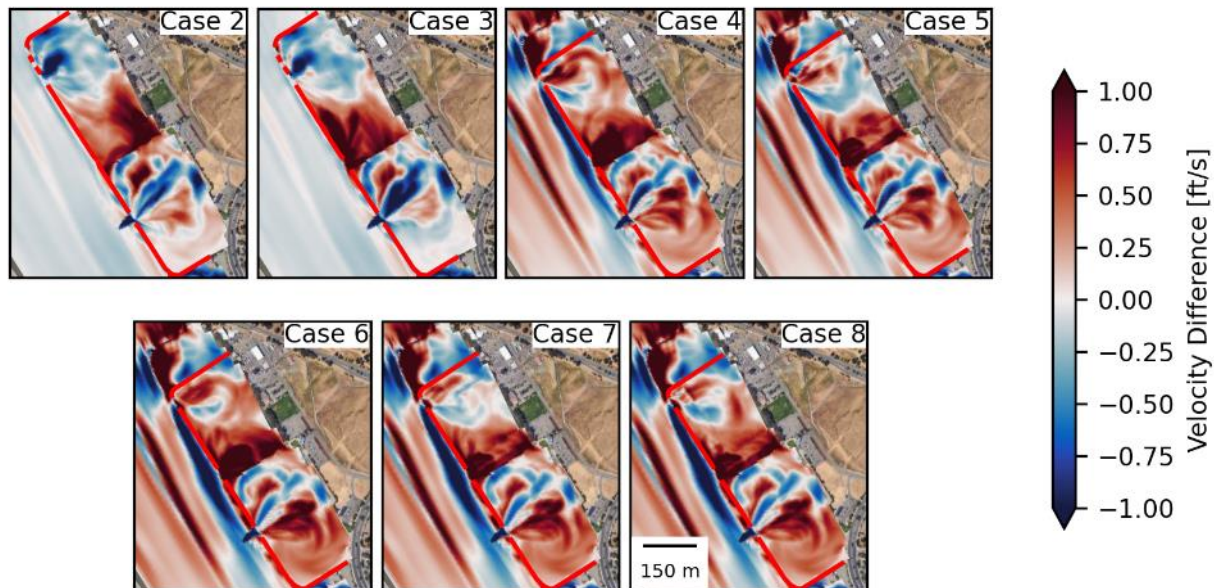


Figure 4-3. Maximum grid-cell velocity difference from Case 1 during the model simulation. Red lines indicate marina walls.

For flood-tide flows (Figure 4-4) the maximum velocity is directing north-eastward through the channel entrances. For Case 1, the flow is restricted from full marina circulation due to the middle marina wall in the south basin and the north wall in the north basin, this results in eddies forming, extending the faster velocity flow southward. In Case 2, the flow into the marina is restricted in the north entrance; however, the flow from the faster velocities from the south entrance extend through the marina from the south entrance to the north. Flow velocities are slowest in the south of the south basin entrance. In Case 3, the middle entrance increases flow velocities along the channel bounding marina wall. There is a reduction in flow velocities for Case 4 through the middle entrance with approximately 1.5 ft/s flows coming through the north entrance. In Case 5, these velocities are reduced by the north sump, and the middle sump reduces flows in the middle of the basin. Case 6, results in larger flows through all entrances with a clockwise eddy forming which directs flows to the south. Adding sumps in Case 7 and 8 reduce the flows through the north and middle entrances and limits the eddy formation. Only Case 6 results in faster flows in the southern part of the south basin.

Compared to Case 1 (Figure 4-5), maximum flood tide flows are primarily increased within the middle of the marina between the southern entrance and northern entrance for Case 2 and halfway between the northern and middle entrance for Cases 3 through 8. All cases result in a slight increase in velocity within the southern marina wall. Case 6 results in the largest marina wide increase in velocity with decreases primarily extending from the southern entrance towards land.

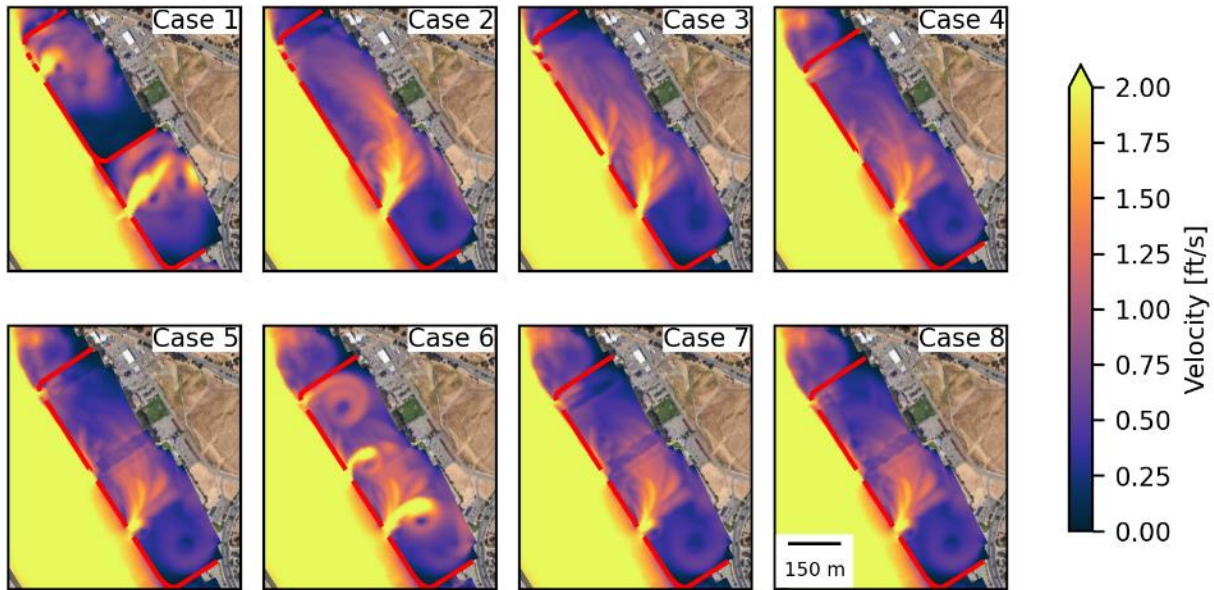


Figure 4-4. Maximum grid-cell velocity during flood-tide. Red lines indicate marina walls.

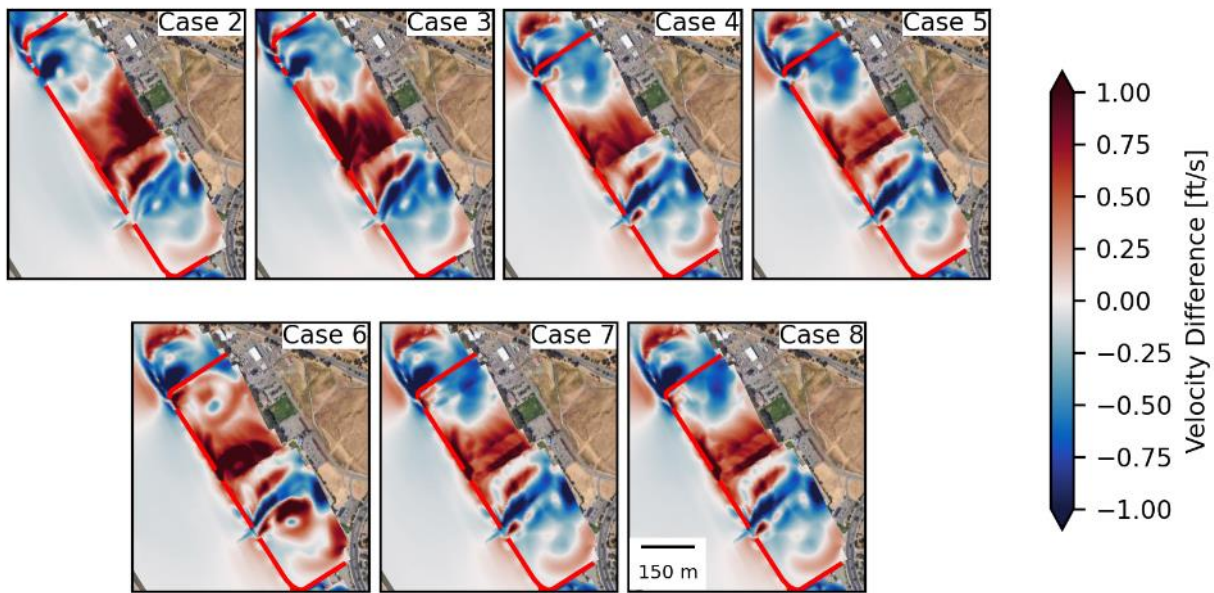


Figure 4-5. Maximum grid-cell velocity difference from Case 1 during flood-tide. Red lines indicate marina walls.

For maximum ebb-tide velocities (Figure 4-6), the flow within the north marina basin for Cases 1 through 3 is directed southward while for Cases 4 through 8 the flow is directed landward. For the middle entrance and the south entrance, the maximum flows are primarily directed landward. Increased maximum velocities occur with Cases 4 through 8. Comparing the cases to

Case 1 (Figure 4-7), Case 2 results in increased velocities in the south half of the north and south basin with a primary reduction in the northern half of each basin. For Case 3, the velocity is larger in the southern half of the north basin with a decrease in the north basin and northern half of the south basin. Negligible change in velocity occurs in the southern half of the south basin. Cases 4 through 5 result in nearly complete increase in velocity in the south basin and the southern half of the north basin and a decrease in maximum velocity in the northern half of the north basin.

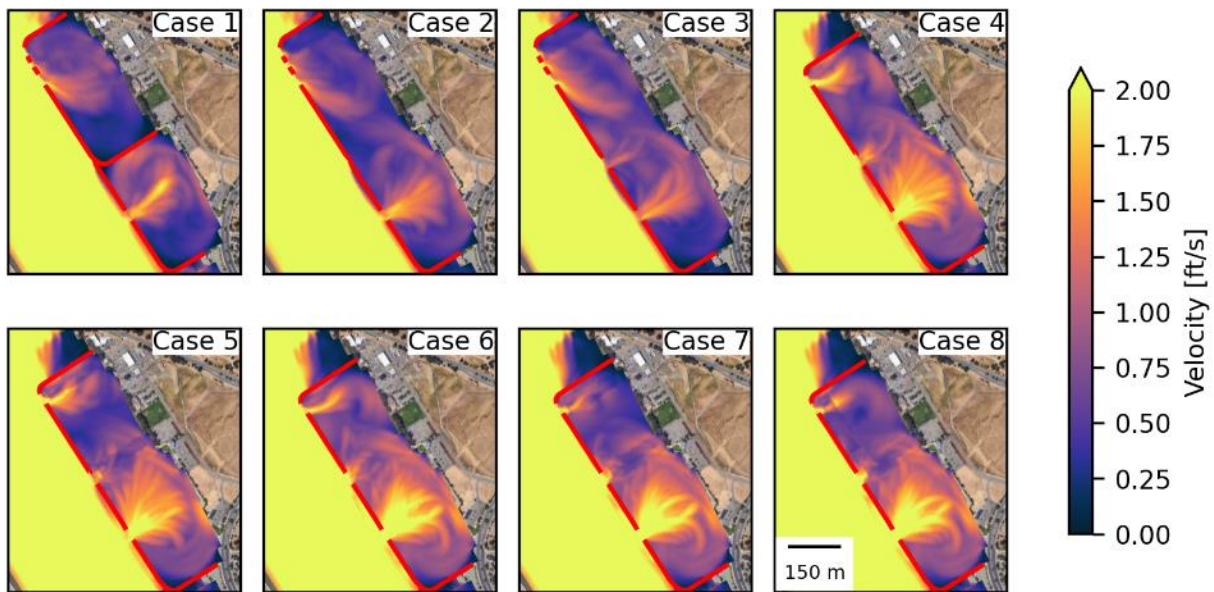


Figure 4-6. Maximum grid-cell velocity during ebb-tide. Red lines indicate marina walls.

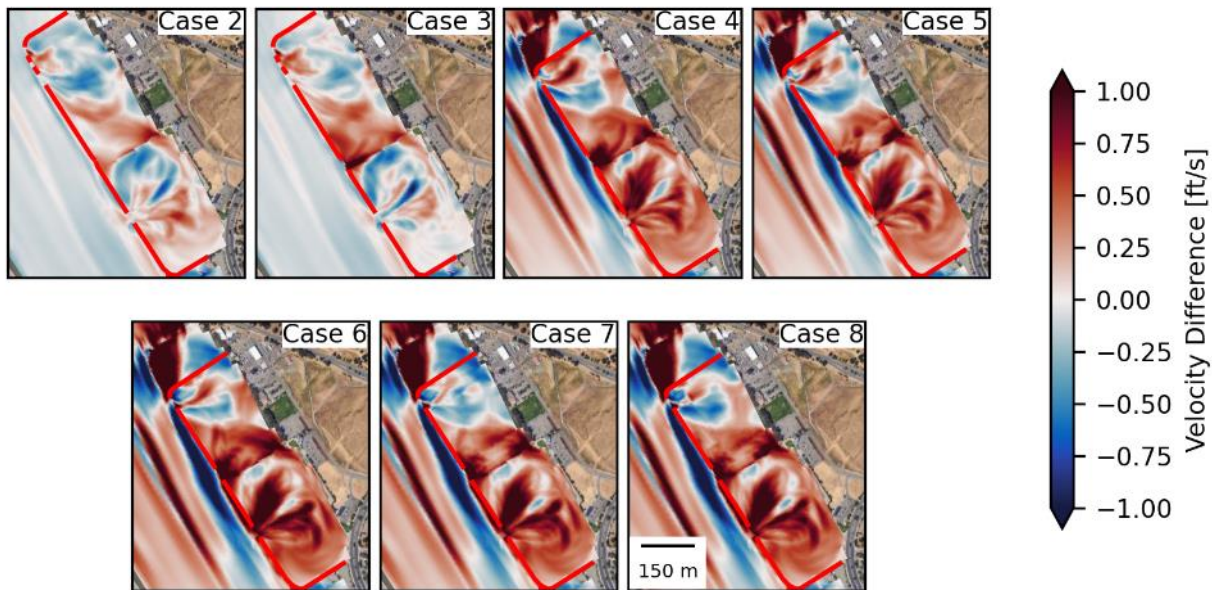


Figure 4-7. Maximum grid-cell velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls.

Mean velocity is calculated as the average velocity for each grid cell of the entire model simulation resulting (Figure 4-8). Average flow patterns are shown as arrows in the figures. Mean velocities are less than 1 ft/s for majority of the marina with the fastest velocities confined to the marina entrances. Mean flow from Case 1 indicate the north and south basins have two eddies forming with circulation extending first to the landward side of the marina and then diverting north and south towards the marina entrances. Removing the middle wall in Case 2 results in faster velocities through the southern marina entrance which extend towards the landward side of the marina and continue towards the north marina entrance. An eddy forms along the southern half of the south basin; however with lower velocities than the norther half of the basin. In Case 3 the velocity increases near the middle entrance with a primary south to north directed flow. Case 4 is similar, however an addition eddy forms in the north-east corner of the marina and the velocity increases in the southern half of the south basin. This eddy is present in Case 6 and Case 8 and to a weaker extent in Case 7. The northeastern eddy does not form for Case 5 with the flow directed towards the northern entrance. Comparing the cases to Case 1 (Figure 4-9), indicates a primarily basin with increase in mean flows with the exception of the northern most end of the north basin.

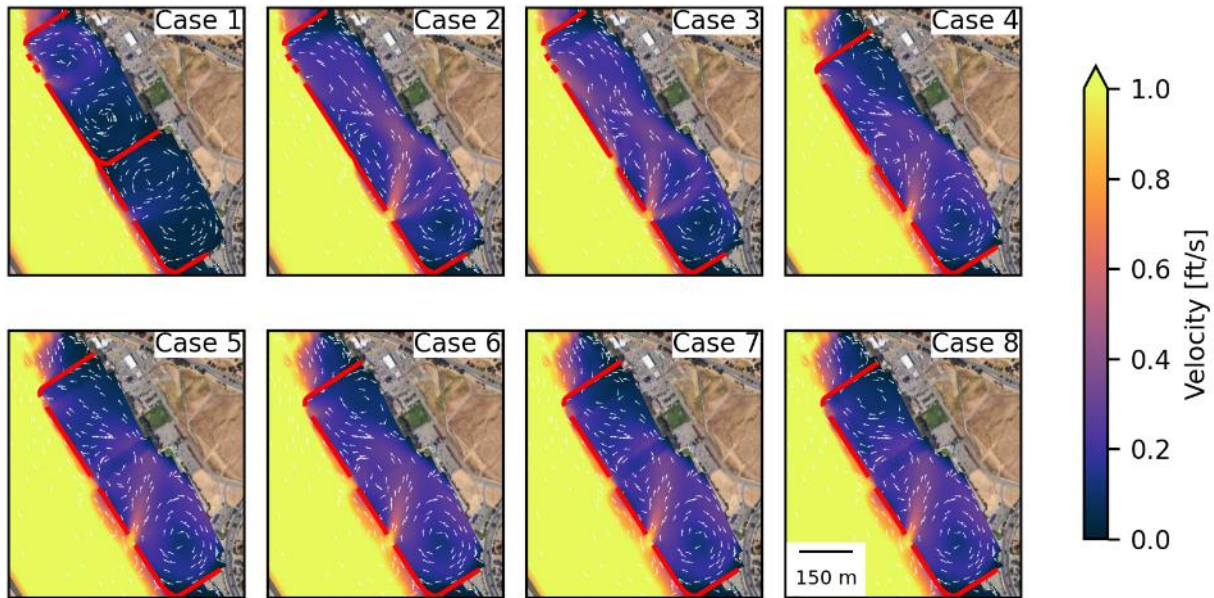


Figure 4-8. Mean velocity during the model simulation. Red lines indicate marina walls.

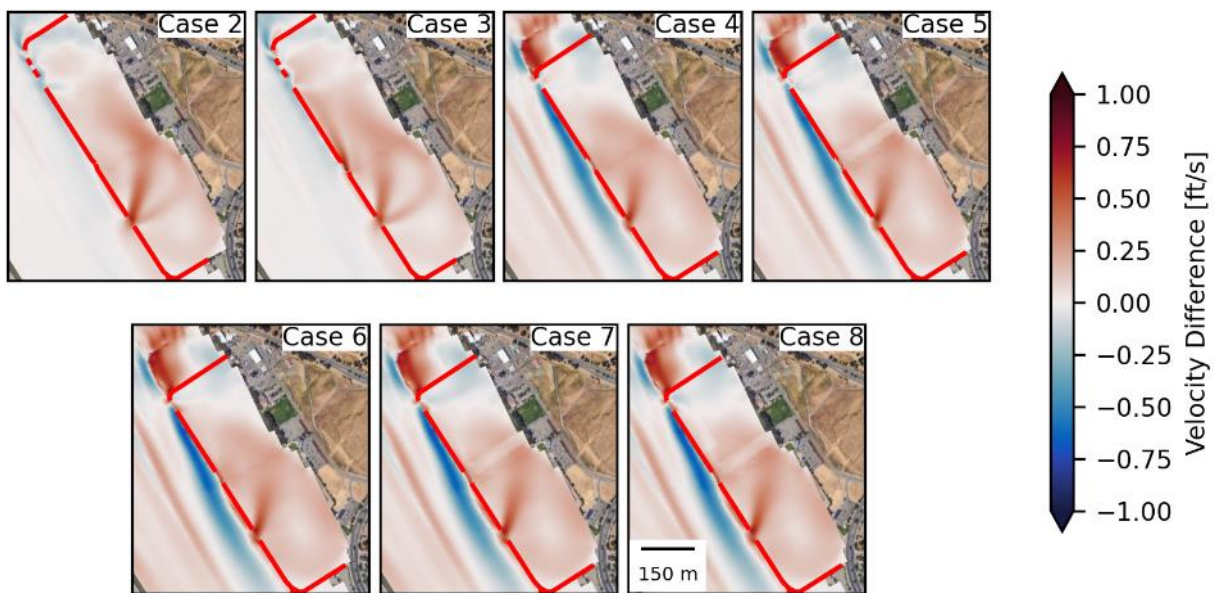


Figure 4-9. Mean velocity difference from Case 1 during the model simulation. Red lines indicate marina walls.

During flood-tide (Figure 4-10), the mean flow is directed from the south marina entrance to the north entrance. With the exception of Case 3, the middle entrance has a slower inward directed flow. Case 1 shows the flow enters the north entrance and produces a single clockwise oriented eddy extending through the north basin with flows less than 0.2 ft/s. A similar eddy forms in the south basin, also with flows less than 0.2 ft/s. For Cases 2 through 8, a clockwise eddy forms

in the southern half of the south basin, also with flows less than 0.2 ft/s. The middles sump in Cases 5, 7, and 8 reduces the maximum flow into the north basin. Compared to Case 1 (Figure 4-11) the mean flood-tide flows result in a nearly marina wide increase in flow velocity with a small reduction in the north-east corner of the marina and a minimal increase in the southern half of the south basin.

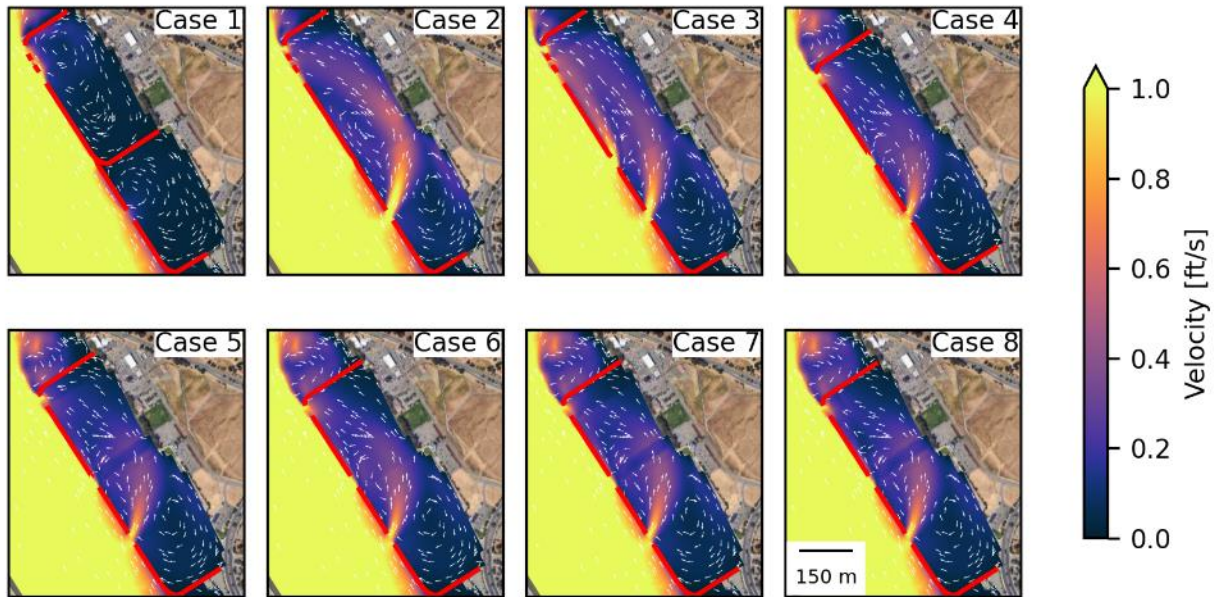


Figure 4-10. Mean velocity during the model simulation during flood-tide. Red lines indicate marina walls.

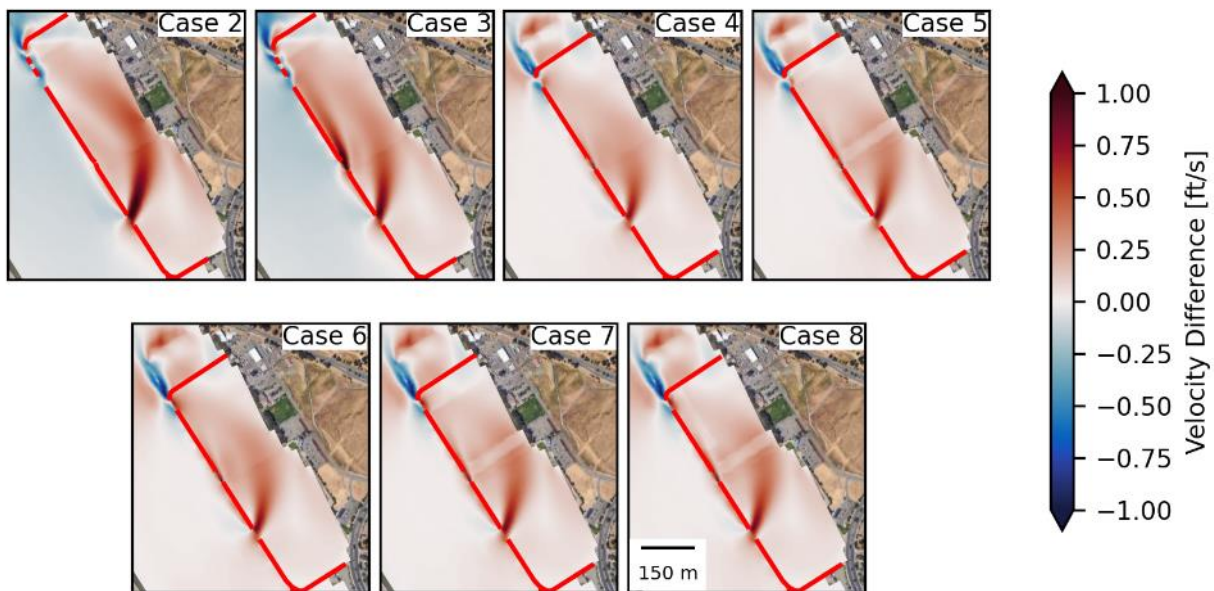


Figure 4-11. Mean velocity difference from Case 1 during flood-tide. Red lines indicate marina walls.

During ebb-flows (Figure 4-12Figure 4-12Figure 4-8), the mean velocity for Case 1 results in two eddies forming in the north and south basin. In the north basin, the flow through the north entrance is directed south-east towards the landward side of the marina. The fastest flow velocities extend northward and exit the northern entrance while smaller velocities extend southward and exit the northern entrance. The south basin has slower velocities than the north basin. For Cases 2 and 3 the fastest flow velocities enter through the southern entrance and extend landward. The fastest flow velocities extend northward with an eddy forming along the southern half of the south basin. Three addition eddies form with one being a counter-clockwise flow in the north basin, and a counter-clockwise flow in the northern half of the south basin. A small clockwise eddy forms along the landward side of the marina in the southern half of the north basin. Only the southern eddy remains in cases 4 through 8 with higher velocities. In Case 4 and 5 a counter clockwise eddy form along the middle entrance and a north-eastern counter clockwise eddy is present in Case 4, 6, 7, and 8. Compared with Case 1 (Figure 4-13), There is a decrease in velocity during ebb-tide int the northern half of the north basin and an increase through the south basin and lower half of the north basin.

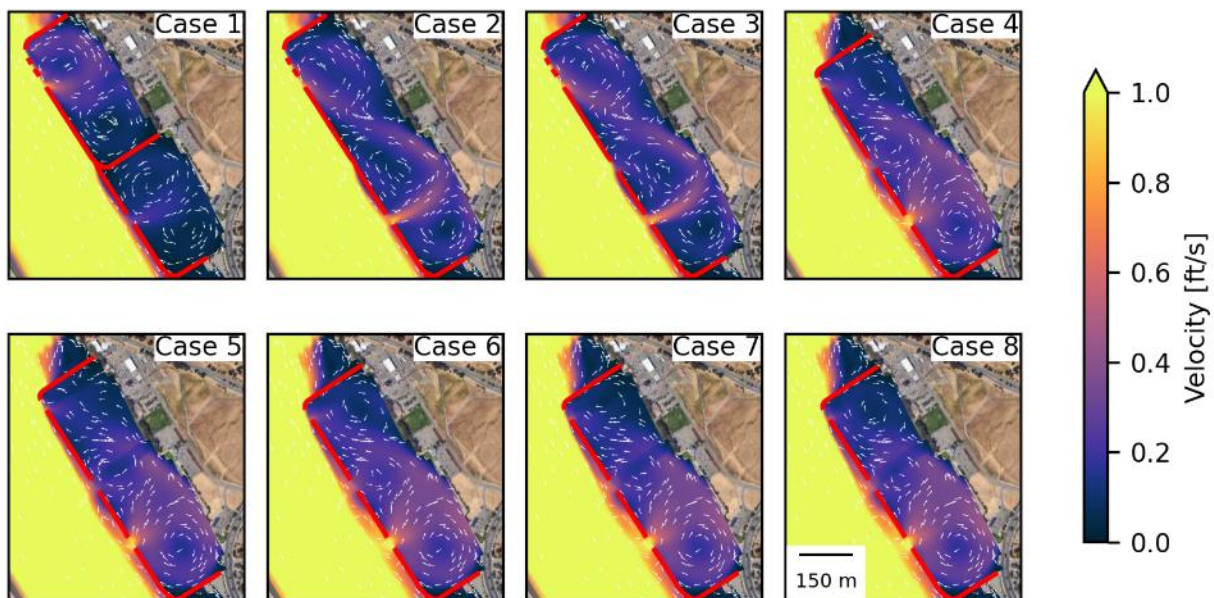


Figure 4-12. Mean velocity during the model simulation during ebb-tide. Red lines indicate marina walls.

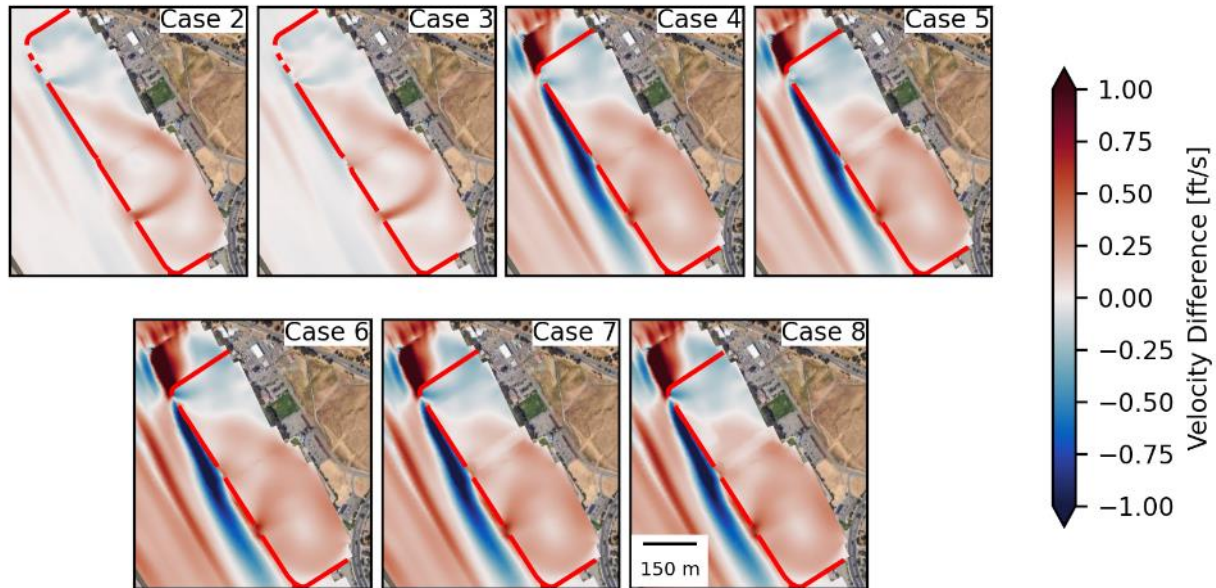


Figure 4-13. Mean velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls.

The maximum defined as the 95<sup>th</sup> percentile of maximum velocities in the north and south basin are shown in (Table 4-2). While the maximum values are sometimes observed Case 1, as shown above, the alternative cases often result in faster velocities over a greater extent of the marina the fastest velocities for Case 01 are limited to a confined region often near the entrances. This table shows that the fastest velocities are greater than 1ft/s for all cases with the largest velocities in the south basin. The increased extent of fastest velocities for the alternatives become apparent when looking at the mean velocities (Table 4-3). With the exception of ebb-tide for the north basin, all basin wide mean velocities are greater than Case 1. Based on model simulations it is unlikely that the velocity will reach the 2kt threshold mentioned by Forth for docked boats for the standard hydrodynamic conditions simulated (Figure 4-14).

Table 4-2. Maximum (95<sup>th</sup> percentile) north and south basin velocity (ft/s).

Marina Configuration	North Basin Velocity (ft/s)		South Basin Velocity (ft/s)	
	Ebb	Flood	Ebb	Flood
Case 01	1.20	1.47	1.46	2.00
Case 02	1.15	1.35	1.41	1.92
Case 03	1.30	1.39	1.31	1.61
Case 04	1.46	1.10	1.82	1.46

Case 05	1.37	1.05	1.86	1.46
Case 06	1.43	1.74	2.17	2.02
Case 07	1.25	1.11	2.16	1.49
Case 08	1.31	1.08	2.17	1.49

Table 4-3. Mean north and south basin velocity (ft/s).

Marina Configuration	North Basin Velocity (ft/s)		South Basin Velocity (ft/s)	
	Ebb	Flood	Ebb	Flood
Case 01	0.17	0.09	0.09	0.04
Case 02	0.17	0.25	0.21	0.22
Case 03	0.20	0.28	0.19	0.18
Case 04	0.18	0.18	0.27	0.15
Case 05	0.15	0.17	0.27	0.16
Case 06	0.18	0.20	0.30	0.16
Case 07	0.15	0.19	0.30	0.17
Case 08	0.16	0.18	0.30	0.16

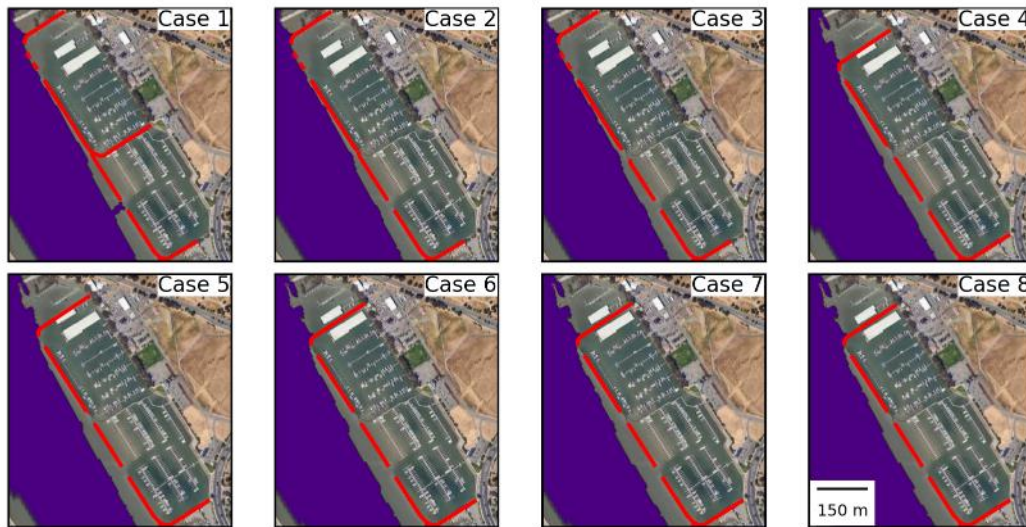


Figure 4-14. Map showing regions where the velocities exceeds 2 kt shown in indigo (indigo) for maximum velocities. Red lines indicate marina walls.

### 4.3 SHEAR STRESS

The amount of deposition or erosion experienced within the marina is inversely proportional to the bottom shear stress ( $\tau$ ). An increase in shear stress will result in a decrease in deposition or increase in erosion, whereas a decrease in  $\tau$  will result in a decreased rate or erosion or increased rate of deposition. To determine the deposition rate expected to occur in the marina

and around each alternative marina configuration, the mean  $\tau$  of the time series and the maximum  $\tau$  of the time series for flood, ebb, and full model simulation were calculated. Shear stress is directly proportional to velocity and as such increased shear stress is related to the increased velocities discussed in section 4.2.

The maximum shear stress in the marina during the simulation is shown in Figure 4-15. For Case 1, the largest shear stresses are near the north marina entrance and the south marina entrance. The larger shear stresses extend further into the northern half of the south basin than the north basin. The southern half of both the north and south basin have the smallest shear stress. For Case 2 and 3, the shear stress extends from the south marina entrance northward along the landward side of the marina. The south end of the marina and the sump have lower shear stress. For Cases 4 through 8 there is increased shear stress along at the marina entrances with a decreased shear stress at the middle and north entrances due to the presence of sumps in Cases 5, 7, and 8. Cases 4 through 8 have slightly greater shear stress in the southern half of the south basin. Compared to Case 1 (Figure 4-16), there is increased shear stress in the marina. Cases 2, 3, 5 have a decreased shear stress in the northern half of the north basin and a decrease near the entrances due to variation in the flow entering the marina.

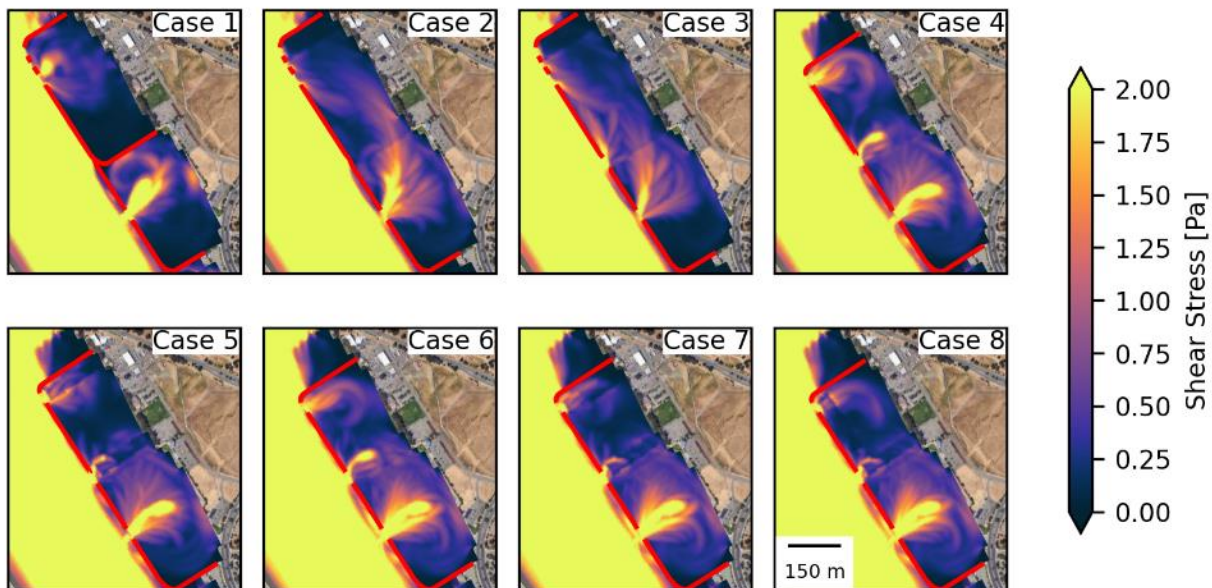


Figure 4-15. Maximum grid-cell shear stress during the model simulation. Red lines indicate marina walls.

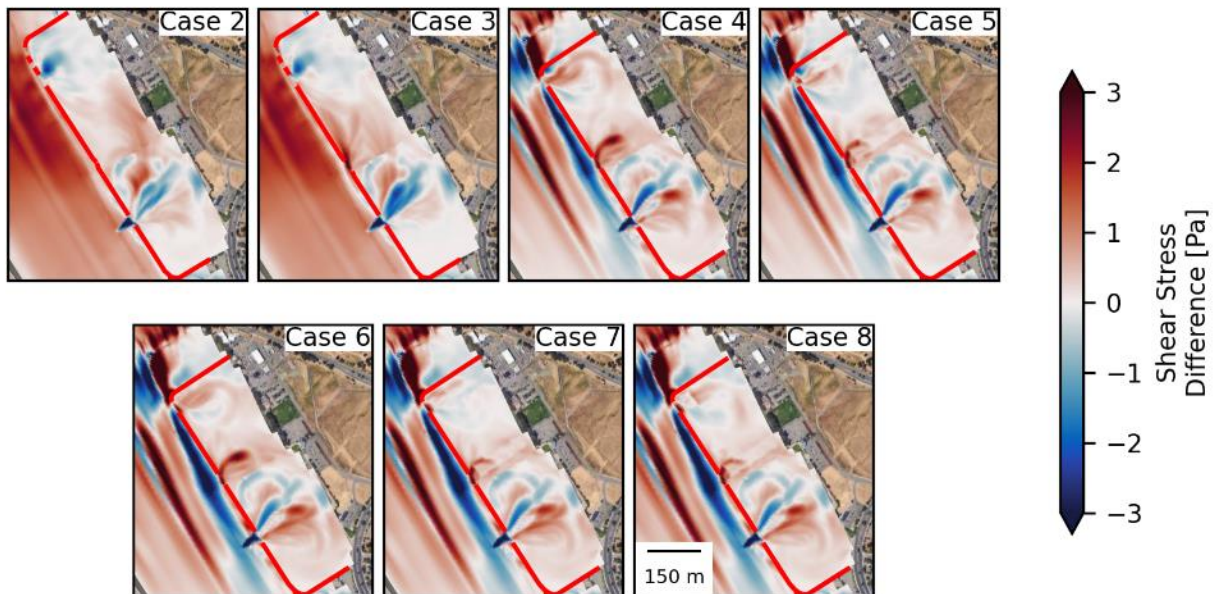


Figure 4-16. Maximum grid-cell shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls.

The maximum shear stress during flood-tide (Figure 4-17) indicates increased shear stress north of the entrances for Cases 1 through 5 and cases 7 through 8. Case 6 results in eddies forming as the flow enters the marina which results in a more direct flow into the marina. During flood-tide the greatest change from Case 1 (Figure 4-18) is an increase in shear stress in the center of the marina, a decrease in shear stress in the northern half of the north basin, and minimal change in the southern half of the south basin. The exception of Case 6 where there is an increase in shear stress in the north half of the north basin.

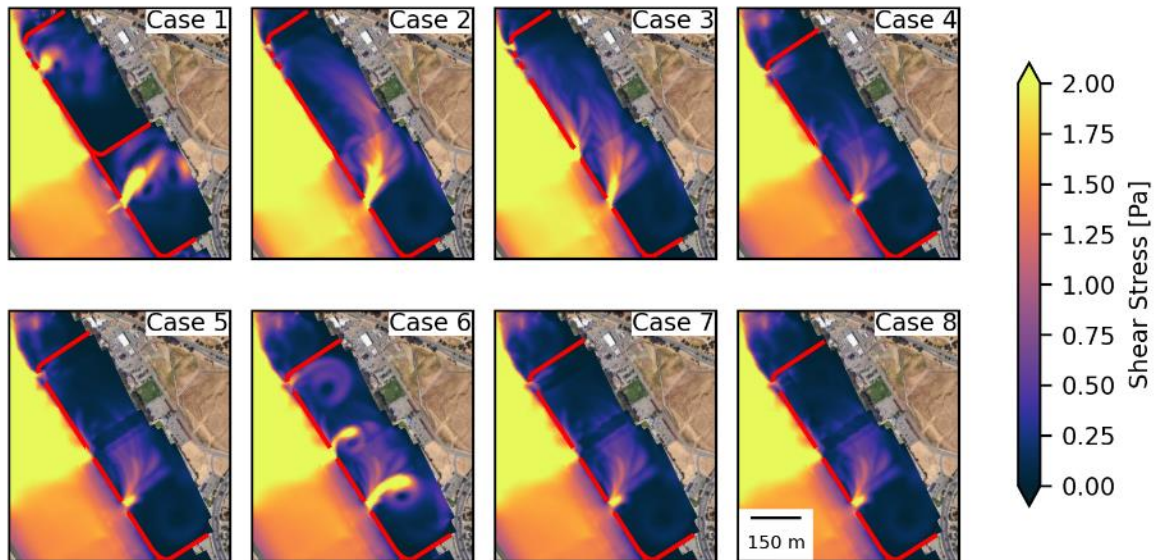


Figure 4-17. Maximum grid-cell shear stress during flood-tide. Red lines indicate marina walls.

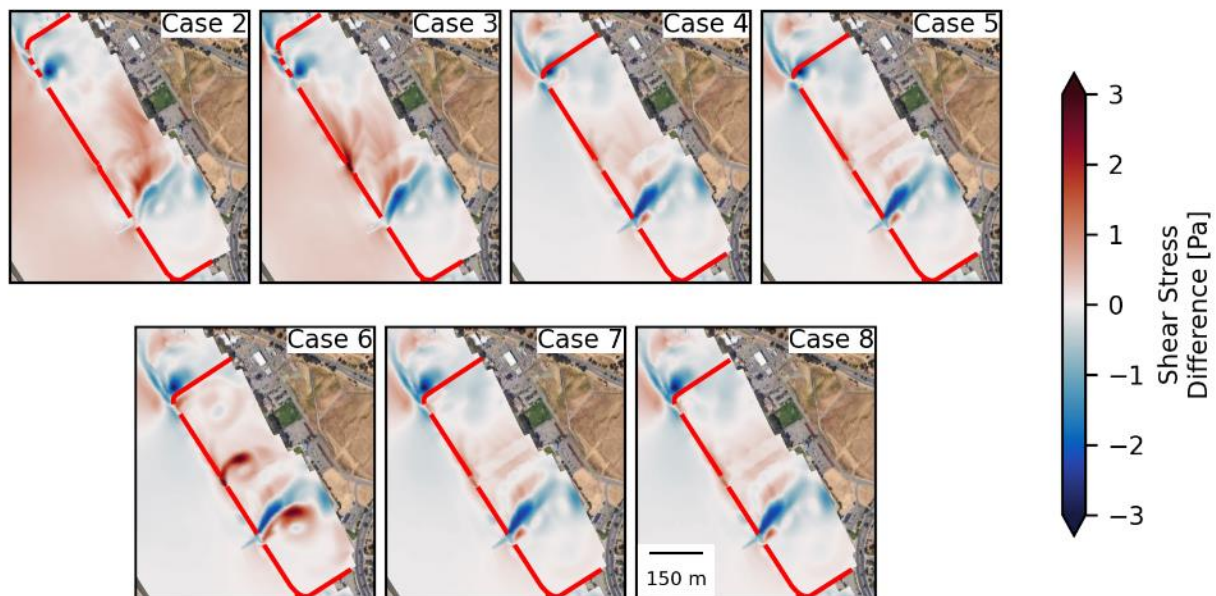


Figure 4-18. Maximum grid-cell shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls.

For maximum shear stress during ebb-flows (Figure 4-19), the largest shear stress, like flood tide, is confined to the marina entrances except for shear stress extending more landward than northward. The shear stress is greater for Cases 4 through 8 in the middle south basin. Shear stress is smaller in the north basin with higher shear stresses along the north wall for Cases 4 through 8. For Cases 2 and 3 the shear stress is directed southward through the north marina

entrance. Compared to Case 1 (Figure 4-20), Cases 4 through 8 result in an overall increase in shear stress for the south basin and the southern half of the north basin. There is a decrease in shear stress for half of the north basin depending on the orientation of the flow through the entrance. For Case 2 and 3 the difference is smaller with negligible change in the southern half of the south basin and along the north wall, an increase in the middle of the marina, and variable increases and decreases near the north and south marina entrances.

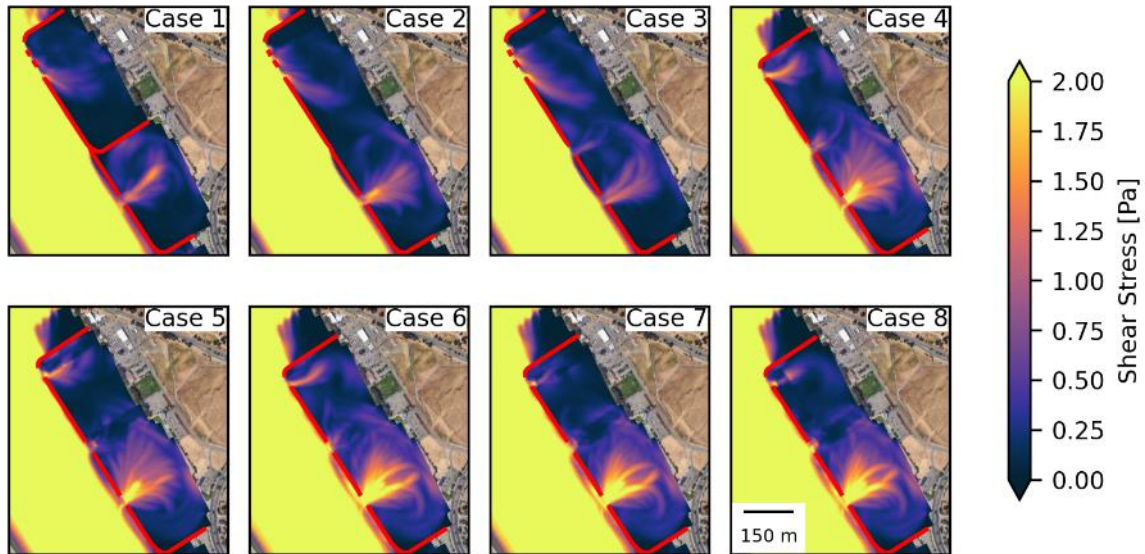


Figure 4-19. Maximum grid-cell shear stress during ebb-tide. Red lines indicate marina walls.

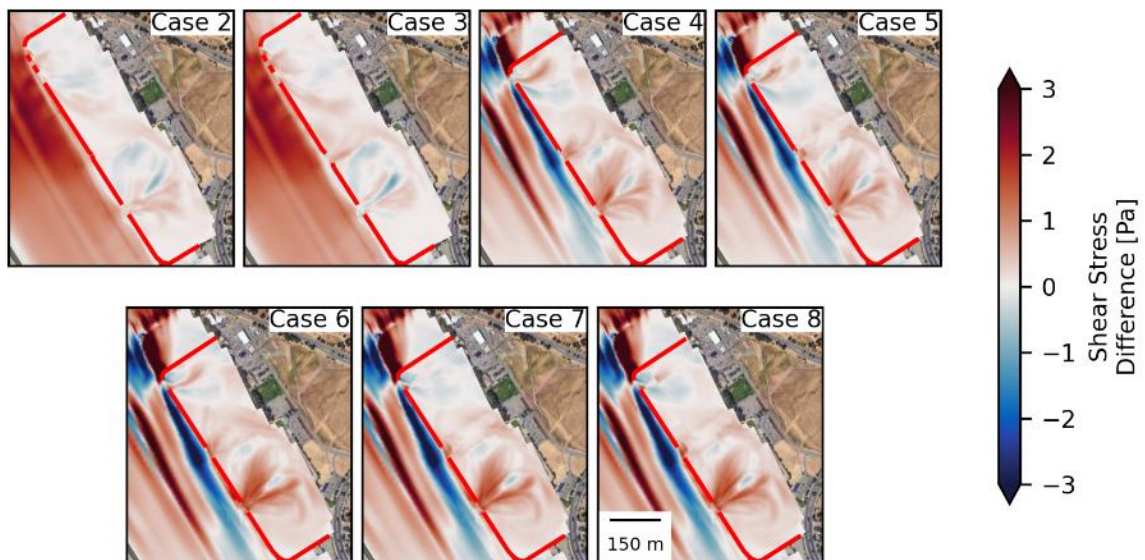


Figure 4-20. Maximum grid-cell shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls.

Mean shear stresses are shown in 4-16. The prevalent regions of higher shear stress extend from the south marina entrance towards the north marina entrance. The larger stresses extend further north for Cases 2 and 3, while the larger stresses decrease around the middle of the marina for Cases 4 through 8. Differences from Case 1 (Figure 4-22), indicate an overall increase in shear stress with negligible changes in the north basin and the largest increase confined to the velocities associated with mean flow near the south marina entrance.

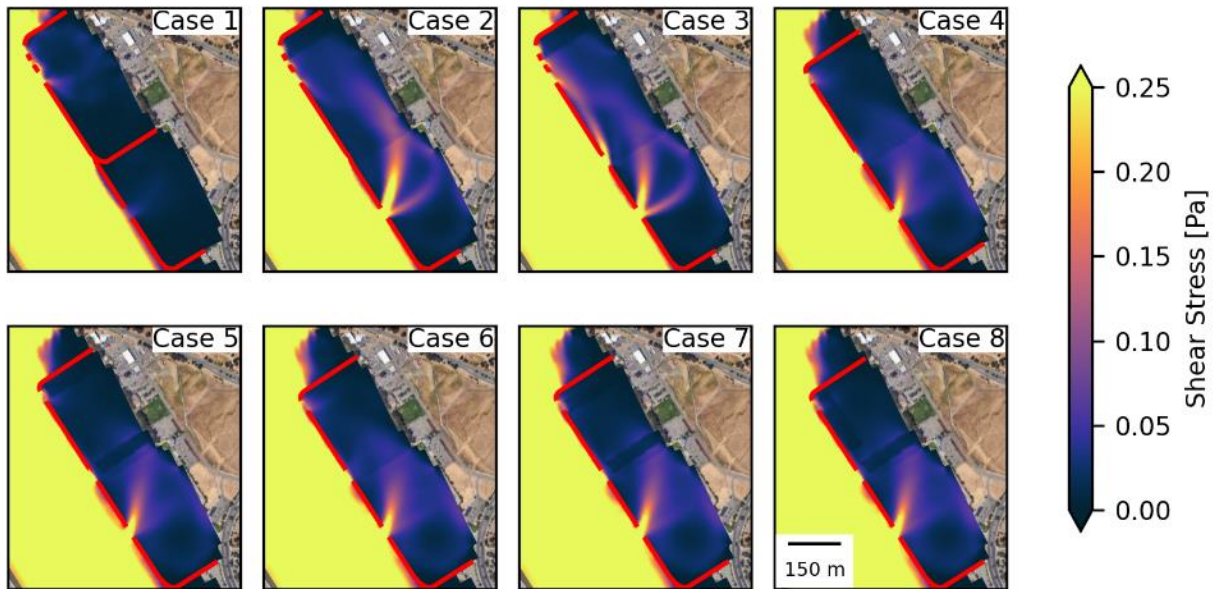


Figure 4-21. Mean shear stress during the model simulation. Red lines indicate marina walls.

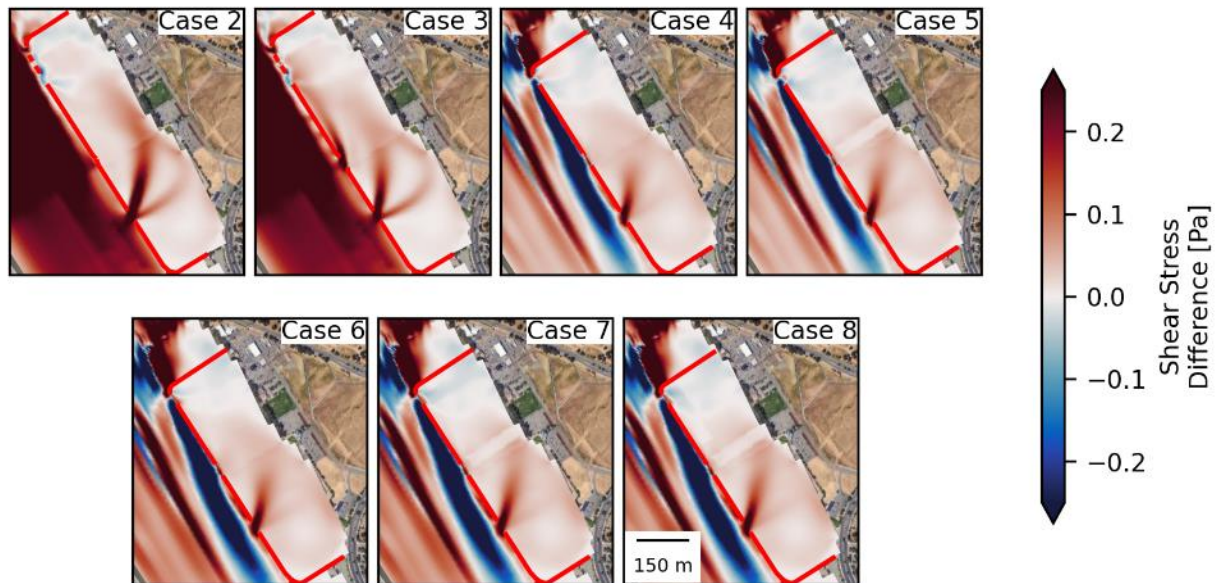


Figure 4-22. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls.

During mean flood-tidal flows (Figure 4-23), the largest shear stress extends from the south marina entrance northward into the marina. The addition of the middle entrance in Case 3 results in increased shear stress along the channel wall inside the marina. Overall there is little variation in the location of the largest shear stresses however, the extent of large shear stresses into the marina occur in Case 2 and 3. Compared to Case 1 (Figure 4-24), there is negligible changes in shear stress for cases 4-8 outside of the velocity influence from the south marina entrance. The largest increase in shear stress is for Case 3 due to the influence of the south and middle entrances.

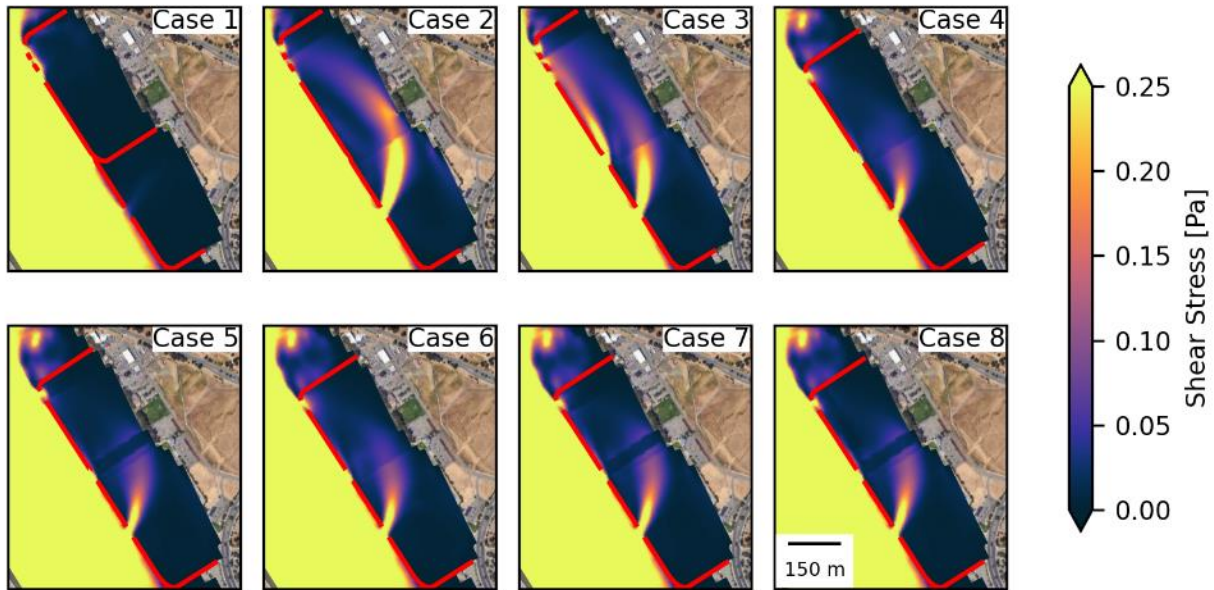


Figure 4-23. Mean shear stress during flood-tide. Red lines indicate marina walls

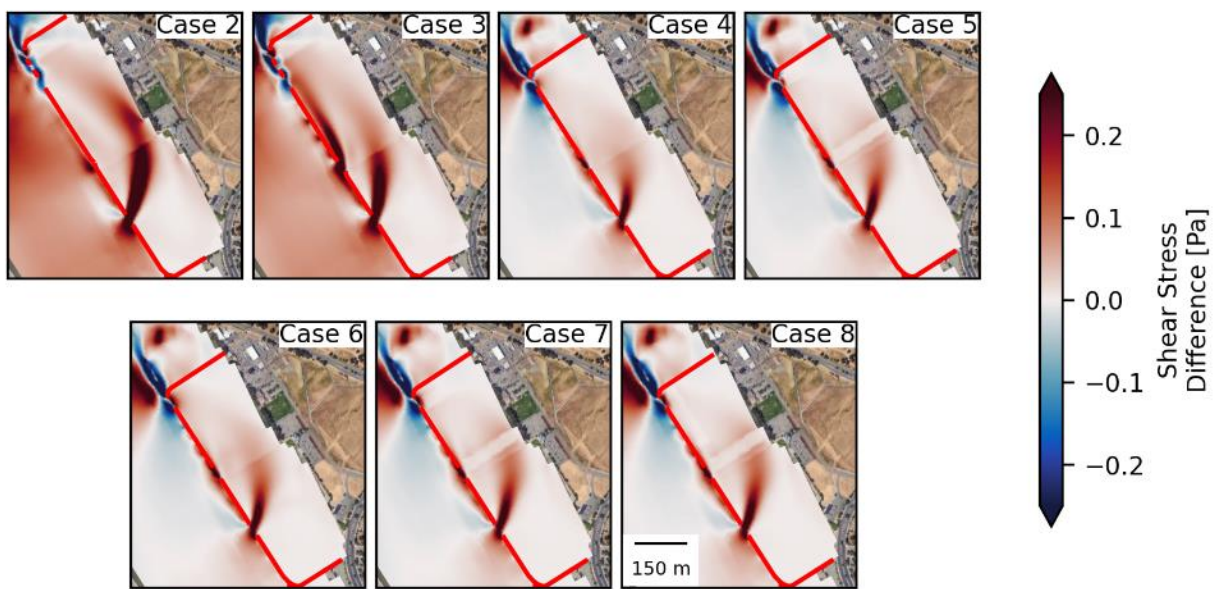


Figure 4-24. Mean shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls.

During mean ebb-tidal flows (Figure 4-25), the maximum shear stress extends from the south entrance towards the landward side of the marina and then extends towards the north marina entrance for Cases 2 and 3. For Cases 4 through 8, the maximum shear stress become more diffuse after entering the south marina entrance and extending landward. For Case 1 the maximum shear stress is in near the northern marina entrance and extends south-east.

Compared to case 1 (Figure 4-26), the mean shear stress increases in the south basin and the middle of the marina. There is a decrease in shear stress in the northern half of the north basin and extending along the landward side.

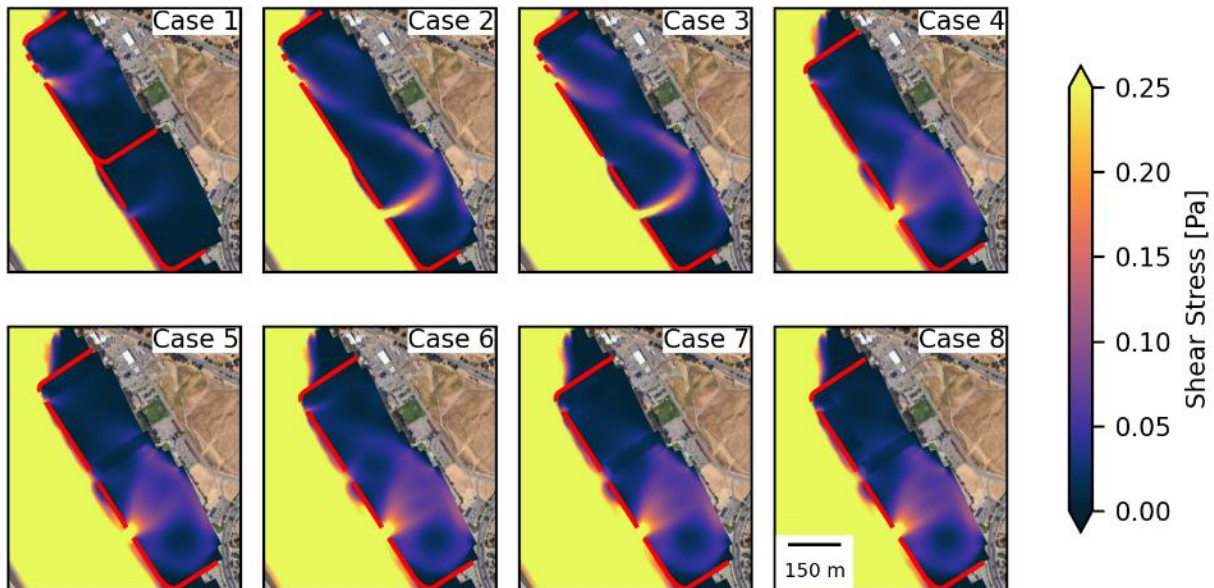


Figure 4-25. Mean shear stress during ebb-tide. Red lines indicate marina walls

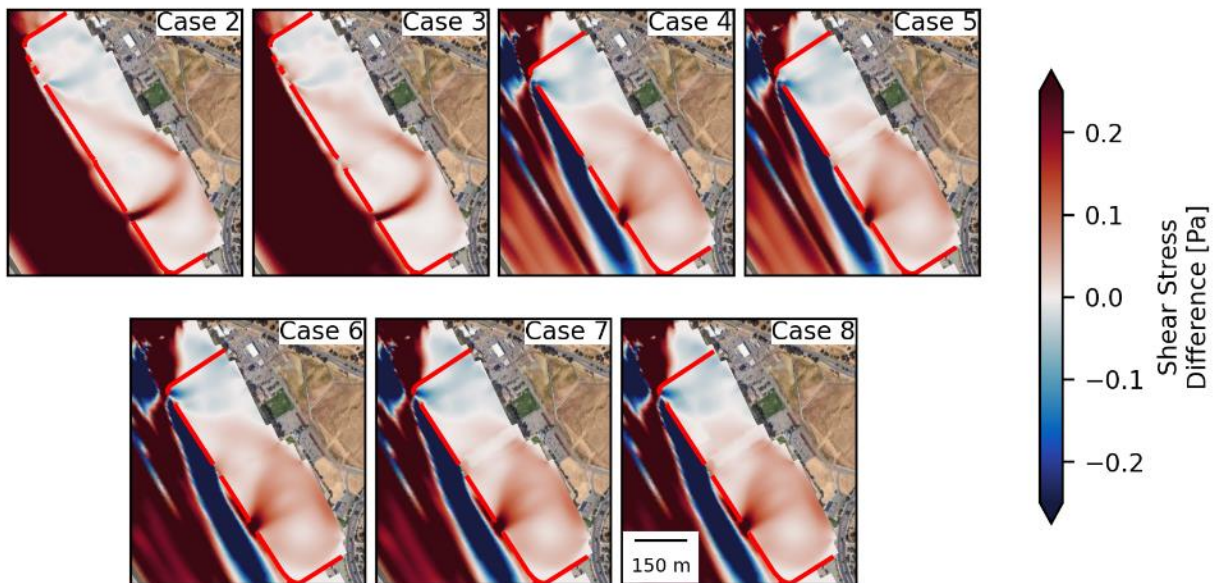


Figure 4-26. Mean shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls.

The maximum defined as the 95<sup>th</sup> percentile of maximum shear stress in the north and south basin are shown in (Table 4-4). The table shows the shear stress is variable in the North Basin

between the different Cases with the largest ebb-tide shear stress in Case 4 and smallest shear stress in Case 7. Flood-tide shear stress is smaller for the north basin in Cases 4, 5, 7 and 8. The south basin has larger shear stresses overall except for Case 3 which has the same maximum ebb shear stress. The mean shear stress (Table 4-5), shows that the shear stress if the south basin is consistently larger than Case 1, while the north basin shear stress is variable compared to Case 1.

Table 4-4. Maximum (95<sup>th</sup> percentile) shear stress (Pa)

Marina Configuration	North Basin		South Basin	
	Ebb	Flood	Ebb	Flood
Case 01	0.58	0.86	0.88	1.67
Case 02	0.65	0.86	0.99	1.80
Case 03	0.84	0.91	0.84	1.28
Case 04	0.85	0.46	1.32	0.85
Case 05	0.73	0.41	1.38	0.84
Case 06	0.80	1.20	1.88	1.87
Case 07	0.55	0.47	1.84	0.88
Case 08	0.62	0.44	1.86	0.88

Table 4-5. Mean shear stress (Pa)

Marina Configuration	North Basin		South Basin	
	Ebb	Flood	Ebb	Flood
Case 01	0.022	0.011	0.008	0.003
Case 02	0.024	0.051	0.038	0.066
Case 03	0.032	0.064	0.035	0.046
Case 04	0.020	0.021	0.050	0.026
Case 05	0.015	0.019	0.051	0.027
Case 06	0.021	0.027	0.060	0.029
Case 07	0.015	0.024	0.059	0.030
Case 08	0.016	0.022	0.059	0.030

#### 4.4 PROBABILITY OF DEPOSITION

The likelihood of sediment deposition for each case was calculated using a probability of deposition based on excess shear stress:

$$P_a = \left(1 - \frac{\tau}{\tau_{cr}}\right) \quad \text{Equation 1}$$

Where  $P$  is the probability of deposition and the subscript 'a' indicates the alternative marina configurations,  $\tau$  is the shear stress, and  $\tau_{cr}$  is the critical shear stress assumed to be 0.1 Pa based on typical young bay sediment characteristics in the channel. The deposition rate ( $D$ ) for these cases is then related to the current deposition rate (4-16) as:

$$D_a = D_e \left( \frac{P_a}{P_e} \right) \quad \text{Equation 2}$$

where the subscript "e" indicates existing marina configuration. The mean sediment deposition rate for the north and south basin was used for  $D_e$ .

#### 4.4.1 Sediment Deposition

The mean and mean flood-tide deposition rates ( $D_a$ ) are shown in Figure 4-27 and Figure 4-28, respectively. Case 1 shows the mean deposition rate used for  $D_e$  based on the sedimentation analysis in section 3. The representative deposition rate is assumed to be accounted for during the mean flow as maximum flows are limited in temporal occurrence. In addition, since flow coming from San Francisco Bay (flood-tide) will bring in more sediment the deposition rate for flood-tide conditions are also evaluated. For mean flow (Figure 4-27), erosion is predicted due to the flows entering the south marina entrance and limited to area around the north entrance. For Cases 2 through 8. For Case 3 erosion extends northward along the marina channel wall and along the north marina entrances. Compared to Case 1 (Figure 4-28), the mean deposition rate decreases or remains the same for the south basin, with a large decrease in deposition rate due to the south entrance flow. There is an increase in deposition rate along the north wall. For Cases 2, 3, 5, 7 this corresponds to infilling of the sump. For Case 8, there is infilling of the sump near the entrance, though the southern end of the sump indicates no change in deposition rate. For Case 4 and 6, there is an increase in deposition along the north wall despite the sump being present, corresponding to the lower shear stresses discussed in section 4.3. The middle sump experiences a decrease in sediment deposition rate compared to the average, however it is larger than the surrounding area, indicating that sediment might be preferentially deposited in the sump.

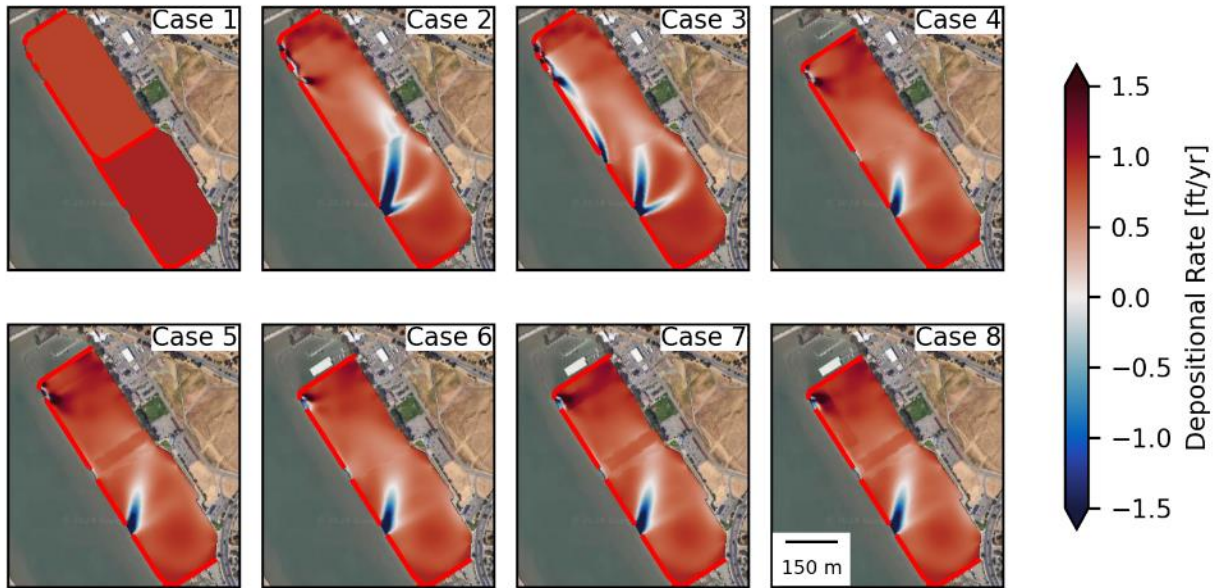


Figure 4-27. Mean sediment deposition rate during the model simulation. Red lines indicate marina walls.

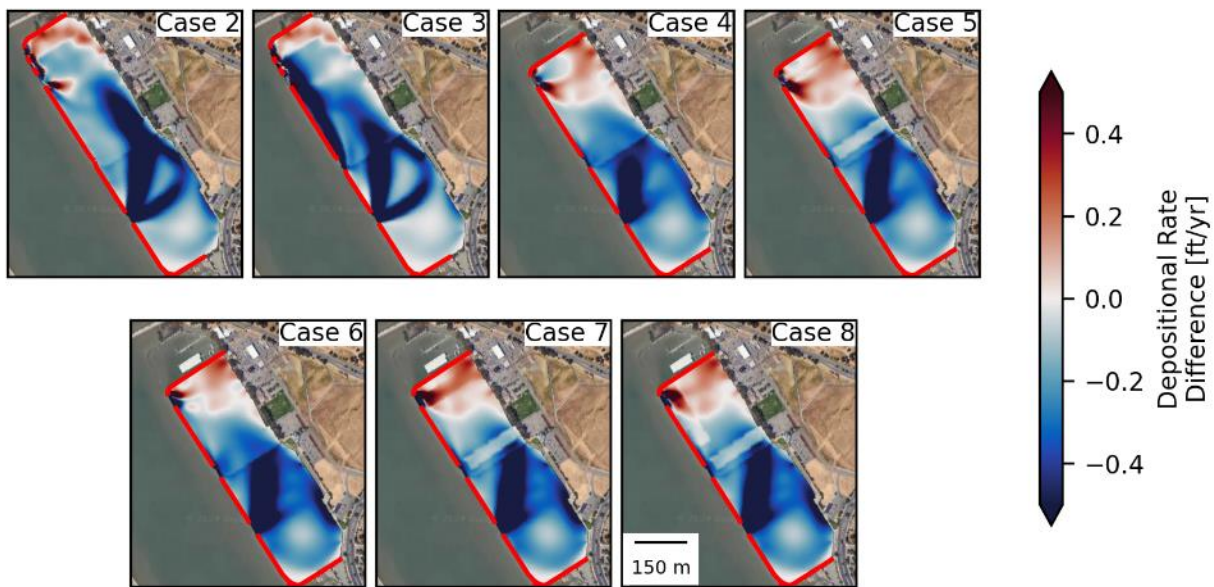


Figure 4-28. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls.

For flood-tide (Figure 4-29), the mean sediment deposition rate shows a similar erosion pattern to the mean rate with the exception of increase erosion along the channel wall in Case 3, and an increase in the erosion extent into the middle of the marina. Compared to Case 1 (Figure 4-30), there a is negligible change in deposition rate in the southern basin outside the flow influence

from the south marina entrance. The increased deposition rate for the mean flow along the north wall is smaller with no erosion along the north marina entrance. The deposition rate is still increased in the northeast corner of the marina.

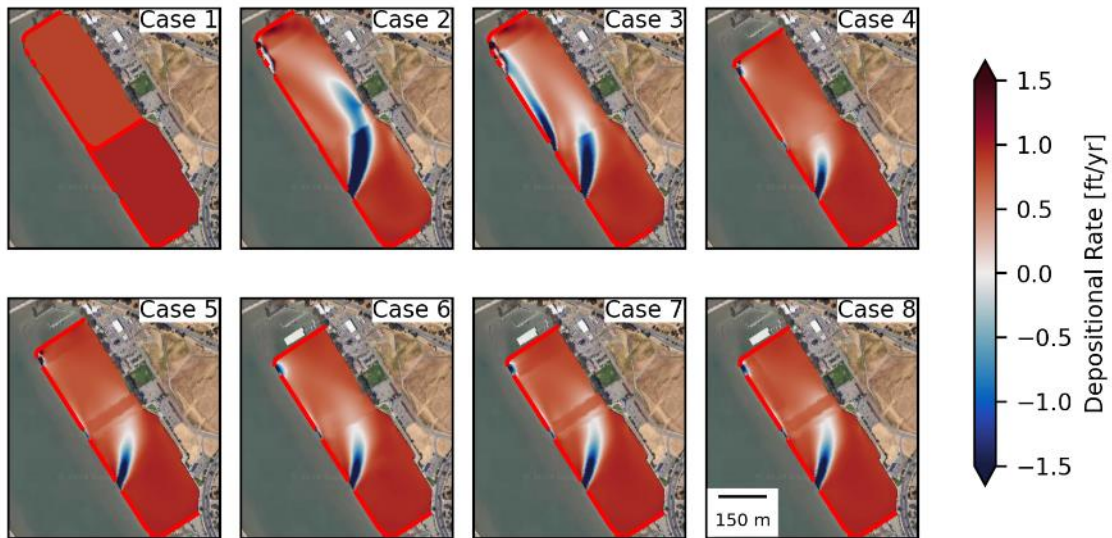


Figure 4-29. Mean sediment deposition rate during flood-tide. Red lines indicate marina walls.

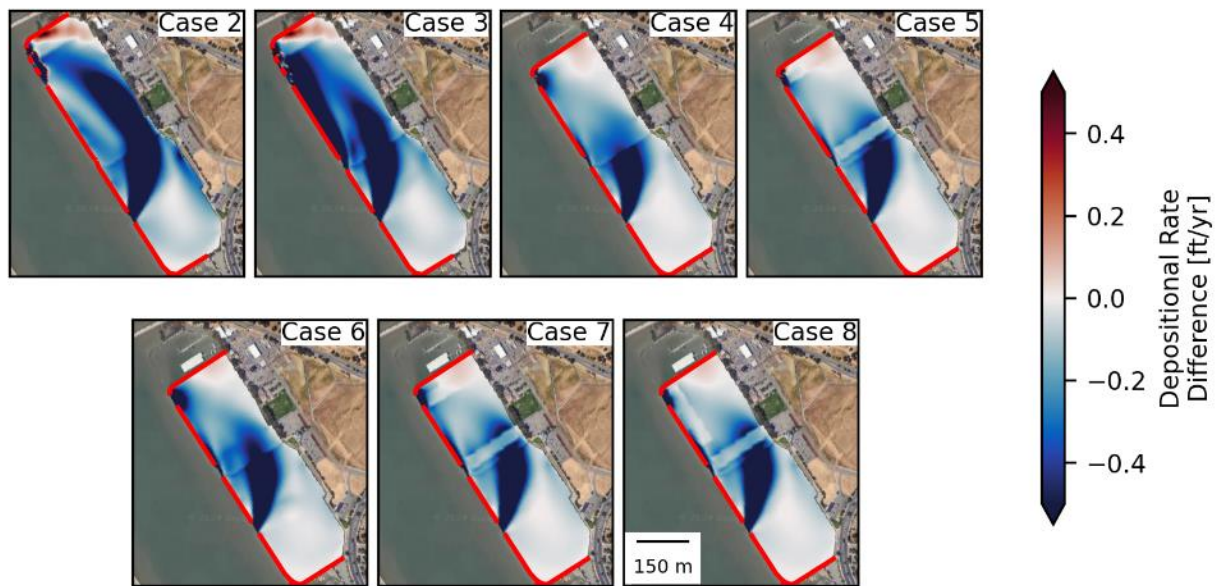


Figure 4-30. Mean sediment deposition rate difference from Case 1 during flood-tide. Red lines indicate marina walls.

## 4.5 DEPOSITION RATES

The mean deposition rate within the marina were calculated for mean (Table 4-6) and mean flood-tide (Table 4-7). For mean flows, the overall marina deposition rate is reduced for all Cases compared to Case 1 with Case 2 having the overall smallest deposition rate with Case 6 being similar. However, when excluding the sumps and only evaluating the remaining area, Case 7 and 8 have a similar overall deposition rate to Case 6. Deposition rate in the sumps is larger than the overall marina average indicating these might reduce the sediment deposited in other areas of the marina. Cases 2 through 8 show the greatest reduction in deposition rate in the south basin while the north basin exhibits less average reduction. Mean deposition rate during flood-tide is shown in (Table 4-7). The deposition rate for Cases 2 through 8 are smaller for the north and south basin than Case 1. This also indicates that the deposition rate is greater in the sumps than the basin. Case 2 results in the largest decrease in deposition rate for the north and south basin, reducing the marina wide deposition rate by 80% when excluding sump areas.

Table 4-6. Mean sediment deposition rate (ft/yr)

Case	Marina	North Basin	South Basin	North Sump	Middle Sump	Excluding Sump Area
Case 01	0.92	0.85	0.99	--	--	--
Case 02	0.63	0.80	0.46	0.99	--	0.62
Case 03	0.73	0.86	0.59	0.93	--	0.72
Case 04	0.69	0.77	0.63	--	--	--
Case 05	0.71	0.82	0.62	0.98	0.74	0.70
Case 06	0.64	0.75	0.57	--	--	--
Case 07	0.66	0.80	0.56	1.18	0.73	0.64
Case 08	0.67	0.81	0.56	1.01	0.73	0.64

Mean deposition rate during flood-tide is shown in (Table 4-7). The deposition rate for Cases 2 through 8 are smaller for the north and south basin than Case 1. This also indicates that the deposition rate is greater in the sumps than the basin. Case 2 results in the largest decrease in deposition rate for the north and south basin, reducing the marina wide deposition rate by 80% when excluding sump areas.

Table 4-7. Mean flood sediment deposition rate for flood-tide (ft/yr)

Case	Marina	North Basin	South Basin	North Sump	Middle Sump	Excluding Sump Area
Case 01	0.92	0.85	0.99	--	--	--
Case 02	0.40	0.50	0.30	1.19	--	0.36
Case 03	0.46	0.39	0.52	0.99	--	0.43
Case 04	0.72	0.68	0.74	--	--	--

Case 05	0.71	0.69	0.73	0.86	0.72	0.70
Case 06	0.67	0.63	0.71	--	--	--
Case 07	0.68	0.66	0.70	0.82	0.70	0.68
Case 08	0.69	0.68	0.69	0.76	0.70	0.68

## 4.6 IMPLICATIONS ON DREDGING

The predicted deposition rates in Vallejo Marina are derived from observed deposition rates based on survey data from 2014–2023 that include the current piling configuration and evidence of vessel-induced scour. The limited spatial coverage of these surveys prevents a full evaluation of the spatial variability of sediment deposition rates and as such a basin average was used here. Furthermore, these deposition rates are expected to change following the removal of the middle wall, addition of entrances, and alteration of vessel traffic due to change in dock design. Of note in this analysis, the sediment captured by sumps is not taken directly into account as the models do not include a finite sediment load tracked over time and then deposited, nor are changes in the deposition rates computed as sediment would be deposited over time.

Boats within the marina need at least 7 feet of water below MLLW to operate safely. Based on this water depth and bed elevation the estimated timing of for dredging based on the mean deposition rate is shown in Figure 4-31. The average dredge return interval using the mean deposition rates from Table 4-6 and the average pre-dredge (Case 1) and post-bathymetry (Cases 2 through 8) are shown in Table 4-8. While this analysis indicates sumps dredged to 20ft MLLW will take more than 10 years to fill in, this does not consider the expected entrapment of sediment within these sumps which will result in a faster rate of infill. The analysis indicates the south basin would require more frequent dredging than the north basin. This can be attributed to the fact that the south basin is only dredged to 8ft MLLW, necessitating only a foot of deposition prior to dredging. The north basin is dredged to a depth of 10ft MLLW requiring more time to infill. Over dredging is not considered in this analysis. In addition, changes to the south basin dredge depth could change the flow pattern and shear stress modeled in this study.

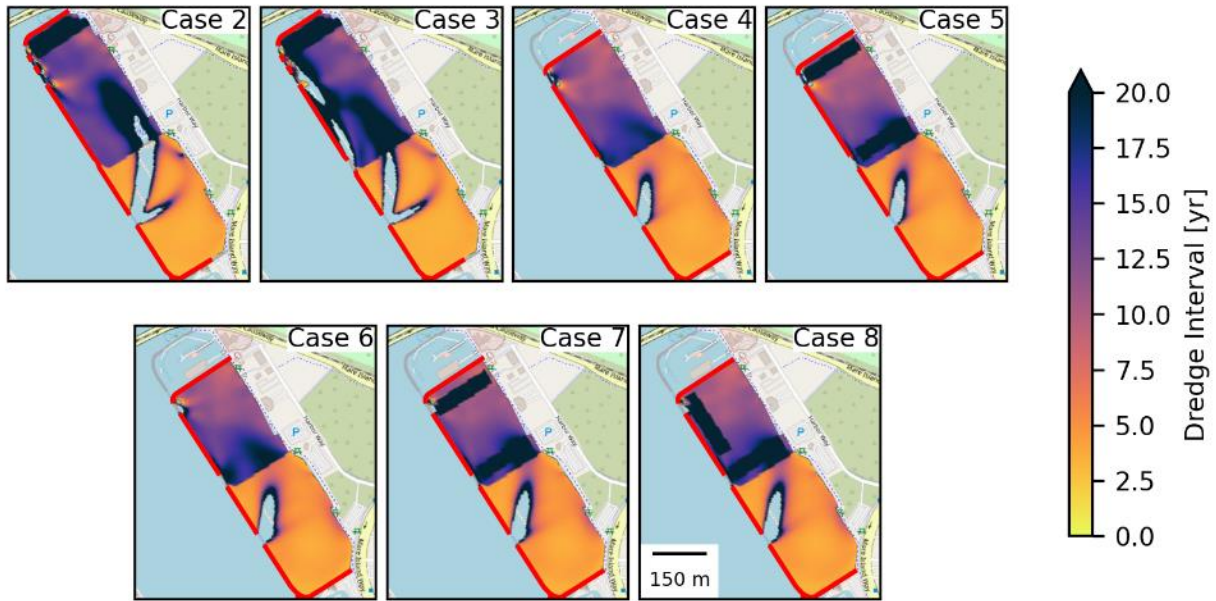


Figure 4-31. Time till bed attains a depth of -7ft MLLW.

C

Table 4-8. Estimated years before required dredging based on mean deposition rate.

Case	North Basin	South Basin	North Sump	Middle Sump	North Basin without Sumps
Case 1	0.6*	1.5*	--	--	--
Case 2	3.7	2.2	13.1	--	3.8
Case 3	3.5	1.7	14.0	--	3.5
Case 4	3.9	1.6	--	--	--
Case 5	3.7	1.6	13.2	17.6	3.7
Case 6	4.0	1.8	--	--	--
Case 7	3.7	1.8	11.0	17.9	3.9
Case 8	3.7	1.8	12.8	17.9	3.8

\*average bed level used.

## 5 CONCLUSIONS

The objective of this study is to support effective marina design through simulating bed shear stress and predicting the potential for deposition based on a combination of model results and survey data. The calibrated and validated Vallejo Marina model was shown to predict depth averaged velocities within the Mare Island Strait with good to excellent agreement, suggesting the results of forces within the marina are representative and can be used to compare alternatives configurations influence on deposition patterns.

Deposition rates experienced within the Marina were derived from a series of bathymetric surveys and accounted for periodic dredge activities within the Site. Due to limited spatial coverage of the surveys, a north and south basin average deposition rate were computed. Estimates of deposition rates aligned with observations by the marina operators. These rates formed the basis for evaluating changes in the system due to marina modifications by scaling relative to changes in modeled bed shear stress.

The team iterated through a number of alternatives that moved or removed marina walls, altered entrances, and added sacrificial areas to allow for additional deposition before dredging is required. The circulation patterns within the marina was helped most by removing the wall currently dividing the north and south basins. The increased circulation range elevated velocities within the system, thus increasing the potential for sediment suspension and ultimately the potential for sediment to be flushed during ebb tide. F3 also suggested reducing the overall footprint of the marina, by moving the north wall to the south. This modified the results of the study by eliminating an area presently experiencing significant deposition. It is expected that sediment would still deposit in the northern extent of any Marina configuration, based on the circulation patterns observed. The proposed northern sump could have added impact in reducing sediment availability to deposit elsewhere in the marina.

In addition, the alternatives increase the flow velocity and shear stress through much of the marina, likely assisting with flushing some of the sediment out during periods of higher flow velocities. If the marina exterior walls were kept the same, then removing the center wall (Case 2) results in the smallest deposition rate. Adjusting the north wall cases 4 through 8 give similar results. It is worth noting that the sumps will likely capture more sediment than this rate of change analysis predicts and may influence the availability of sediment to deposit elsewhere. This is true in regions like the Southern Basin region where we see minimal in the dredge return periods. Also, in the analysis, the average deposition rates within each were used, rather than a model cell-by-cell basis. This was done to provide a conservative estimate of results across the system as a whole. The results suggest that the sumps would have a positive effect on the system's ability to modify sedimentation patterns and extend the amount of time between dredge events.

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**Appendix B**

**Vallejo Marina Sedimentation Study Addendum, Dated October 22, 2024.**

**Prepared by Integral Consulting, Inc.**

**DRAFT**

# Vallejo Marina Sedimentation Study Addendum

*Prepared for*  
Foth Infrastructure and Environment, LLC



601 Montgomery Street  
Suite 888  
San Francisco, CA 94111

October 22, 2024

# CONTENTS

**LIST OF FIGURES** .....iv

**LIST OF TABLES** ..... viii

**ACRONYMS AND ABBREVIATIONS**.....ix

**EXECUTIVE SUMMARY** ..... 1-1

**1 Introduction**..... 1-2

    1.1 SITE DESCRIPTION ..... 1-2

    1.2 PROJECT TASKS ..... 1-3

**2 Numerical Modeling**..... 2-1

    2.1 SAN FRANCISCO BAY-DELTA COMMUNITY MODEL..... 2-1

        2.1.1 Boundary Conditions..... 2-3

        2.1.2 Bay Model Evaluation..... 2-4

    2.2 VALLEJO MARINA MODEL ..... 2-6

        2.2.1 Hydrodynamic Forcing Conditions..... 2-9

    2.3 VALLEJO MARINA MODEL EVALUATION..... 2-9

        2.3.1 Model Performance Metrics..... 2-11

        2.3.2 Vallejo Model Calibration ..... 2-13

        2.3.3 Vallejo Model Validation..... 2-16

    2.4 MODEL UNCERTAINTY ..... 2-17

**3 Sedimentation Study** ..... 3-1

    3.1 EXISTING DATA SETS ..... 3-1

    3.2 DEPOSITION RATE CALCULATION ..... 3-3

**4 Additional Alternative Marina Configurations: Modeling and Analysis**..... 4-1

    4.1 ALTERNATIVE MARINA CONFIGURATION MODEL EVALUATIONS..... 4-1

    4.2 VELCOCITY ..... 4-2

    4.3 SHEAR STRESS ..... 4-17

    4.4 PROBABILITY OF DEPOSITION ..... 4-27

        4.4.1 Sediment Deposition..... 4-28

    4.5 DEPOSITION RATES ..... 4-31

    4.6 IMPLICATIONS ON DREDGING ..... 4-31

**5 Conclusions .....5-34**  
**6 References .....6-1**

## LIST OF FIGURES

Figure 1-1.	Vallejo Municipal Marina, in Mare Island Strait. North basin extents in red. South Basin extents in yellow.....	1-3
Figure 2-1.	San Francisco Bay-Delta Community Model Domain and Bed Level Elevation Referenced to NAVD88 meters.....	2-2
Figure 2-2.	Bay Model bathymetry (NAVD88 m) and grid near Vallejo Marina study area. Pink Lines in right panel indicate locations of boundary conditions.....	2-2
Figure 2-3.	San Francisco Bay-Delta Community Model Water Level and Napa River Discharge Boundary Conditions. The selected calibration period is shaded in gray. ....	2-3
Figure 2-4.	Bay Model Comparison Stations and Location of Mare Island Strait Model Domain. ....	2-4
Figure 2-5.	San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9414863 at Richmond, California. ....	2-5
Figure 2-6.	San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9415102 at the Martinez-Amorco Pier, California.....	2-6
Figure 2-7.	Overview of Vallejo Marina Model Grid.....	2-7
Figure 2-8.	Vallejo Marina Grid refinement around marina. ....	2-8
Figure 2-9.	Overview of Vallejo Marine model bottom elevations.....	2-8
Figure 2-10.	Vallejo Marina Model Discharge and Water Level Boundary Conditions Extracted from the Bay Model. The selected calibration and validation periods are shaded in gray.....	2-9
Figure 2-11.	ADCP depth averaged current velocity used for calibration and validation of the hydrodynamic model. The selected calibration and validation time periods are shaded in gray. ....	2-11
Figure 2-12.	Discharge and water level boundary conditions. The selected calibration and validation periods are shaded in gray. ....	2-14
Figure 2-13.	Vallejo Marina Model calibration to depth averaged velocity from the Mare Island Strait ADCP.....	2-15
Figure 2-14.	Vallejo Marina Model Validation to Depth Averaged Velocity from the Mare Island Strait ADCP.....	2-16
Figure 3-1.	Spatial occurrence of available surveys .....	3-2
Figure 3-2.	Vallejo Marina bathymetry referenced to NAVD88. ....	3-3

Figure 3-3. Inter-dredge north basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text). .....3-4

Figure 3-4. Inter-dredge south basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text). .....3-5

Figure 3-5. Inter-dredge deposition rate for the north basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text). .....3-6

Figure 3-6. Inter-dredge deposition rate for the south basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text). .....3-7

Figure 3-7. Calculated average (left), median (middle) and time-weighted (right) deposition rate. ....3-8

Figure 3-8. Time-Weighted Average Deposition Rate (top) and Maximum Deposition Rate (bottom). .....3-8

Figure 4-1. Bathymetry and marina wall configuration of the alternative analysis evaluation. NAVD88 CA State Plane Zone 2 with units of meters with vertical datum of NAVD88 with units of feet. Red lines indicate marina walls. ....4-2

Figure 4-2. Maximum grid-cell velocity during the model simulation. Red lines indicate marina walls. ....4-4

Figure 4-3. Maximum grid-cell velocity difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-5

Figure 4-4. Maximum grid-cell velocity during flood-tide. Red lines indicate marina walls. ....4-6

Figure 4-5. Maximum grid-cell velocity difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-7

Figure 4-6. Maximum grid-cell velocity during ebb-tide. Red lines indicate marina walls. ....4-8

Figure 4-7. Maximum grid-cell velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls. ....4-9

Figure 4-8. Mean velocity during the model simulation. Red lines indicate marina walls. ....4-10

Figure 4-9. Mean velocity difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-11

Figure 4-10. Mean velocity during the model simulation during flood-tide. Red lines indicate marina walls. ....4-12

Figure 4-11. Mean velocity difference from Case 1 during flood-tide. Red lines indicate marina walls.....4-13

Figure 4-12. Mean velocity during the model simulation during ebb-tide. Red lines indicate marina walls. ....4-14

Figure 4-13. Mean velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls.....4-15

Figure 4-14. Map showing regions where the velocities exceed 2 kt shown in indigo (indigo) for maximum velocities. Red lines indicate marina walls.....4-17

Figure 4-15. Maximum grid-cell shear stress during the model simulation. Red lines indicate marina walls. ....4-18

Figure 4-16. Maximum grid-cell shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-19

Figure 4-17. Maximum grid-cell shear stress during flood-tide. Red lines indicate marina walls. ....4-20

Figure 4-18. Maximum grid-cell shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-20

Figure 4-19. Maximum grid-cell shear stress during ebb-tide. Red lines indicate marina walls...4-21

Figure 4-20. Maximum grid-cell shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls. ....4-22

Figure 4-21. Mean shear stress during the model simulation. Red lines indicate marina walls. ....4-23

Figure 4-22. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-23

Figure 4-23. Mean shear stress during flood-tide. Red lines indicate marina walls.....4-24

Figure 4-24. Mean shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-25

Figure 4-25. Mean shear stress during ebb-tide. Red lines indicate marina walls.....4-26

Figure 4-26. Mean shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls. ....4-26

Figure 4-27. Mean sediment deposition rate during the model simulation. Red lines indicate marina walls. ....4-29

Figure 4-28. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls. ....4-29

Figure 4-29. Mean sediment deposition rate during flood-tide. Red lines indicate marina walls.4-30

Figure 4-30. Mean sediment deposition rate difference from Case 1 during flood-tide. Red lines indicate marina walls. ....4-30

Figure 4-31. Time till bed attains a depth of -7ft MLLW. ....4-32

## LIST OF TABLES

Table 2-1. Model Skill Performance Metrics Set for This Study .....2-12

Table 2-2. Calibration Performance Metrics for Vallejo Model .....2-15

Table 2-3. Validation Performance Metrics for Vallejo Model .....2-17

Table 3-1. Bathymetry survey dates considered in sedimentation analysis .....3-1

Table 3-2. First post-dredge surveys .....3-4

Table 4-1. Marina configuration model simulations. ....4-1

Table 4-2. Maximum (95<sup>th</sup> percentile) north and south basin velocity (ft/s). ....4-16

Table 4-3. Mean north and south basin velocity (ft/s).....4-16

Table 4-4. Maximum (95<sup>th</sup> percentile) shear stress (Pa) .....4-27

Table 4-5. Mean shear stress (Pa) .....4-27

Table 4-6. Mean sediment deposition rate (ft/yr) .....4-31

Table 4-7. Mean flood sediment deposition rate for flood-tide (ft/yr).....4-31

Table 4-8. Estimated years before required dredging based on mean deposition rate.....4-33

## ACRONYMS AND ABBREVIATIONS

ADCP	acoustic Doppler current profiler
DAV	depth-averaged velocity
EPA	U.S. Environmental Protection Agency
Foth	Foth Infrastructure and Environment, LLC
Integral	Integral Consulting Inc.
MLLW	mean lower low water
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
RMSE	root mean square error
USACE	U.S. Army Corps of Engineers
USGS	U.S. Geological Survey

## EXECUTIVE SUMMARY

Integral Consulting Inc. (Integral) supported Foth & Van Dyke and Associates (Foth) in the assessment of sedimentation rates and reconfiguration alternatives in the Vallejo Municipal Marina. The Vallejo Marina is located in the Mare Island Strait that connects the Napa River to San Francisco Bay where sediment is transported on tidal currents and may circulate through the marina. Sediment depositing in the marina from this tidal circulation has impacted navigable waterways and berthing areas that require sediment removal via dredging to maintain safe operation and access. Efforts are underway to evaluate alternative configurations of the marina that may reduce the need for dredge activities and

Foth has been tasked by the City of Vallejo and the Marina operators, F3 Marina, to develop alternative configurations and design of the Marina structures to reduce the frequency of maintenance dredge activities. Integral leveraged a hydrodynamic numerical model of the Mare Island Strait to evaluate changes in circulation patterns and bed forces within the Marina due to proposed changes in entrance configuration, marina extents, and bathymetric changes.

Bed shear stress patterns were compared with sedimentation rates, derived from a series of bathymetric surveys, to estimate sedimentation rates for the existing and proposed alternatives. This report outlines the development of numerical models, observed sedimentation rates, and comparison of existing and alternative marina configurations.

The initial study of the potential marina reconfiguration, conducted by Foth and Integral, included post-dredge sediment bed elevations, the potential for over-dredged areas hereafter called sumps, and altered the configuration and location of marina walls and entrances. Removal of the center wall dividing the two marinas had a positive impact on circulation patterns, increasing the velocity relative to the existing case and indicating a potential decrease in sediment deposition potential. Addition of the sump areas, where it is theorized sediment would deposit and remain, also showed a benefit to the system. It was possible that another marina entrance may also improve the ability for sediment to redistribute or exit the system on ebb tides. In this study, four additional alternative marina designs are evaluated which include 2 additional entrances. The results indicate the addition of the new entrances increase velocities and reduce deposition potential within the system compared to the existing configuration.

# 1 INTRODUCTION

Integral Consulting Inc. (Integral) conducted a study of sediment deposition potential in support of ongoing redesign efforts at the Vallejo Marina. The work informed Foth, the City of Vallejo, and marina operators on the potential for sediment bed response due to the reconfiguration of the marina. Integral relied on numerical models of hydrodynamic conditions and surveys of sediment bed elevations to compare potential changes to sediment deposition patterns and computed dredge interval times.

## 1.1 SITE DESCRIPTION

The Vallejo Municipal Marina (the Site) is located within the Mare Island Strait, to the east of San Francisco Bay's San Pablo Bay subregion. The marina is located on the eastern side of the Strait, south of the Mare Island Causeway, and includes two basins which hold a number of vessel berths (Figure 1-1). Discharge from the Napa River as well as tidal fluctuations within San Francisco Bay drive currents that can mobilize, transport, and redistribute sediment. The Marina is surrounded by walls that extend above the high tide line to and into the sediment bed. While some areas have degraded, this evaluation did not consider permeability of this outer wall in its analysis. The two basins are separated by a wall and both marinas have one entrance. The United States Coast Guard operates a dock in the north basin of the marina.

Bedded sediment within the Site is exposed to natural and anthropogenic forces from tidal currents and traversing commercial ferries and personal watercraft. The natural forces influencing the hydrodynamic conditions and associated sediment accretion at the Site must be investigated to determine how sediment may be transported under potential marina configurations. Currents move through the Site predominantly in a north-south direction and are associated with tidal exchanges between San Francisco Bay and the Pacific Ocean. Wind waves were not considered due to the limited fetch in Mare Island Strait and orientation of the Strait relative to the rest of San Francisco Bay.

Sedimentation rates within the Mare Island Strait and within the marina are governed by hydrodynamic conditions, sediment loads from the Napa River and San Francisco Bay, and anthropogenic activity including vessel induced scour and wake affects, and dredging events. Due to the lack of data and range of uncertainty in sediment loads within the system, this modeling study does not include direct sediment transport loads or transport to the system. Rather, the study uses a scaling approach of modeled bed shear stress to infer the potential for deposition or erosion. In addition, while it is assumed that the vessel activity modifies bed accretion and erosion potential due to vessel induced wakes and prop scour, the model is not equipped to handle these processes and it was not considered in this analysis.



Figure 1-1. Vallejo Municipal Marina, in Mare Island Strait. North basin extents in red. South Basin extents in yellow.

## 1.2 PROJECT TASKS

The technical tasks performed by Integral are outlined below. The analysis and findings of these tasks are presented in the following sections.

### Task 1. Set Up and Refine Hydrodynamic Model

Foth developed a hydrodynamic model in D-Flow FM and used it to simulate hydrodynamic conditions within the Mare Island Strait and around the Site. D-Flow FM is a numerical modeling suite that resolves hydrodynamic and sediment transport conditions on an unstructured grid. Integral developed boundary conditions to simulate water levels and tidal currents within the system concurrent with the period of hydrodynamic instrument deployment.

### Task 2. Calibrate Hydrodynamic Model

The hydrodynamic model was calibrated by comparing model results to velocity data collected within the model domain. This ensures the model recreated hydrodynamic conditions and resulting forces that govern sediment mobility. Velocity data collected during a ~45-day deployment of an acoustic Doppler current profiler (ADCP) was used for model calibration and validation of depth averaged currents.

### **Task 3. Assess Sedimentation Rates**

The estimation of sedimentation rates was conducted using available bathymetric surveys and dredge records collected since 2013. Analysis of the surveys produced estimates of rates of accretion within each basin. Results from this study were incorporated with the model results to infer the effects of marina alterations by scaling these deposition rates with changes in shear stress.

### **Task 4. Evaluate Marina Alternatives**

Foth and Integral iterated through a set of alternative marina configurations to evaluate which may increase the time between maintenance dredging events. These alternatives modify the location of marina outer walls, add or alter the entrances to the Marina, and incorporate updated bathymetry including over-dredged areas for sediment accretion. Integral applied the layouts within the hydrodynamic model domain compare and computed changes in bed shear stress, deposition rates, and dredge return intervals.

## 2 NUMERICAL MODELING

A coupled hydrodynamic model was constructed for this study, using the San Francisco Bay-Delta Community model and a site-specific model generated by Foth, employing the open-source D-Flow FM modeling suite. D-Flow FM is a state-of-the-science model that can simulate aquatic systems in 1, 2, and 3 dimensions. It solves the 3-dimensional equations of motion in a water body with variable fluid density using free-surface and hydrostatic conditions. Water column turbulence, which generates small scale mixing, is modeled with a k-epsilon turbulence closure scheme at each model grid cell and at each time step. D-Flow FM was selected as the model for this Site based on the following capabilities:

- The model is publicly available, technically rigorous and defensible, and has been used widely to evaluate a broad range of systems.
- The model incorporates complex bathymetry using an unstructured grid, which allows for more accurate representation of bottom topography.
- The model allows for time-varying inputs of water levels and discharges.

Efforts by a team of investigators including Deltares, the U.S. Geological Survey (USGS), and San Francisco Estuary Institute developed the San Francisco Bay-Delta Community model that extended from the Pacific Ocean through the bay and into the delta (Deltares et al. 2021). This community-developed calibrated and validated model provided more refined hydrodynamic, or flow, information around the Site in the absence of measured data. The Mare Island Strait Model with model resolution refinements within and around the Marina, was used to simulate hydrodynamic conditions based on inputs from the Community model.

### 2.1 SAN FRANCISCO BAY-DELTA COMMUNITY MODEL

The San Francisco Bay-Delta Community model, or Bay model, domain consists of discrete grid cells ranging in size and shape, encompassing all of San Francisco Bay and its deltas (Figure 2-1). This model domain utilizes an unstructured grid, or flexible mesh, that allows cells to have any number of sides, typically three or four, with sides varying in length. The flexibility in size and shape allows complex shoreline geometries to be better represented in the model, and resolution can be increased in areas of interest. Each model grid cell has an associated depth, derived from a 2015 bathymetric survey conducted by USGS (2017). While meant to supply hydrodynamic boundary conditions to the Vallejo Marina model, the Bay model was setup with multiple grid cells across the Mare Island Strait to effectively resolve the hydrodynamics. The Bay model was set up with one vertical layer because of the relatively shallow depths of the Site and assumed negligible effect of any salinity or temperature driven circulation.

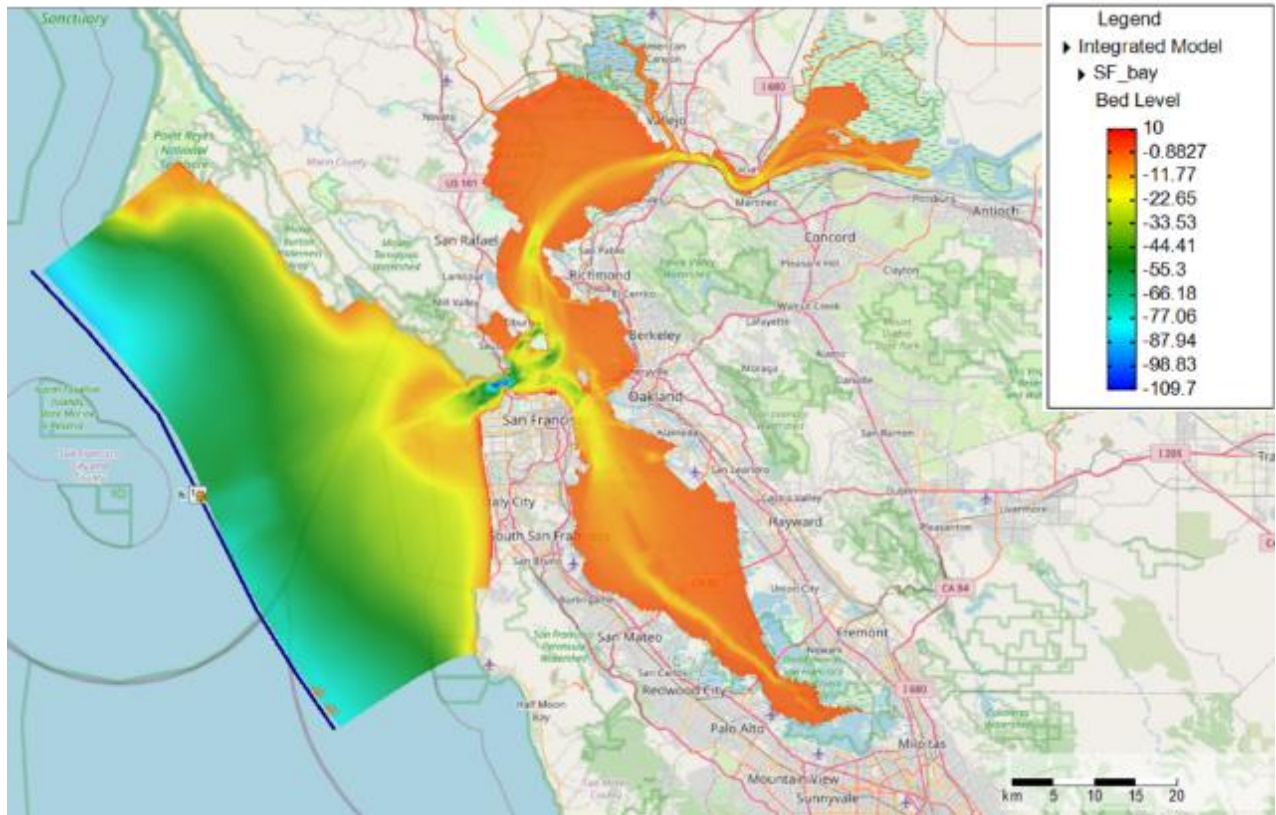


Figure 2-1. San Francisco Bay-Delta Community Model Domain and Bed Level Elevation Referenced to NAVD88 meters.

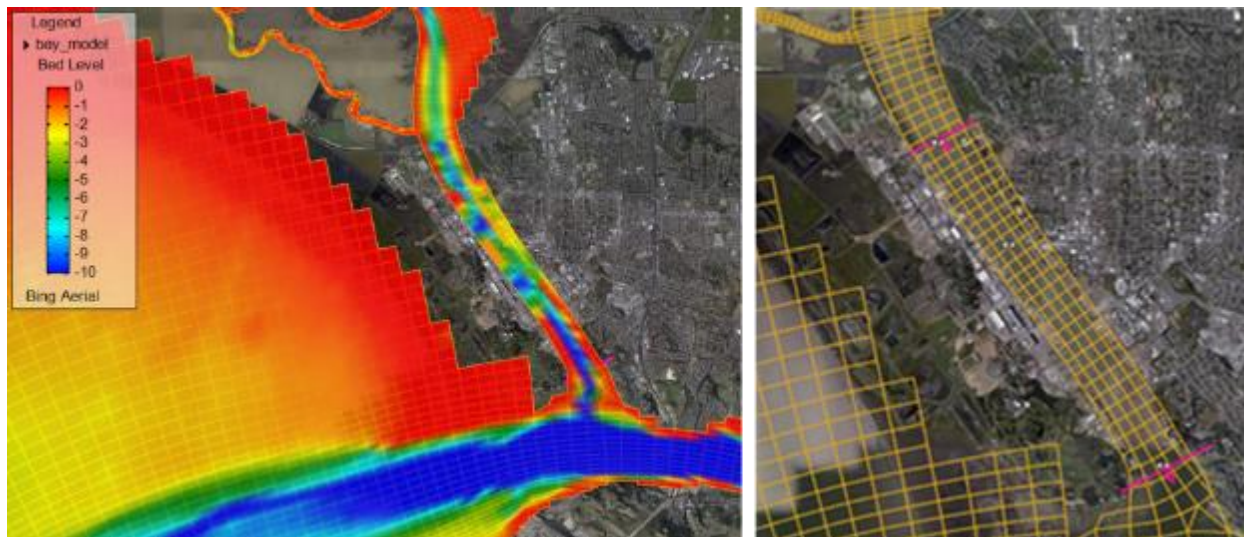


Figure 2-2. Bay Model bathymetry (NAVD88 m) and grid near Vallejo Marina study area. Pink Lines in right panel indicate locations of boundary conditions.

### 2.1.1 Boundary Conditions

Boundary conditions provide the forcing that drives the water mass within the model domain and usually consists of the water level at a boundary of the model domain, or discharge in to or out of the domain, or a combination of both. Two boundary conditions were utilized to drive circulation within San Francisco Bay, tidal fluctuations from the ocean, and riverine discharge from the Napa River. Tidal fluctuations at open ocean boundary, denoted by the blue line in Figure 2-1, vary water levels and drive currents through the mouth of San Francisco Bay (Figure 2-3). These data were sourced from measurements at National Oceanic and Atmospheric Administration (NOAA) station 9414290 at the Golden Gate Bridge. Due to the location of the model boundary in the coastal waters to the west of the station, a 42-minute offset was applied to temporally align boundary conditions with the measured water levels. The phase offset was determined by comparing measurements at the NOAA station location within the model with model results and adjusting accordingly. The discharge data from the Napa River was pulled from the USGS Streamflow database<sup>1</sup>. A majority of the Napa River record showed no discharge, including during the selected model calibration period (Figure 2-3). As a result, the ocean water level boundary would provide the forcing for hydrodynamics within San Francisco Bay, which included Mare Island Strait.

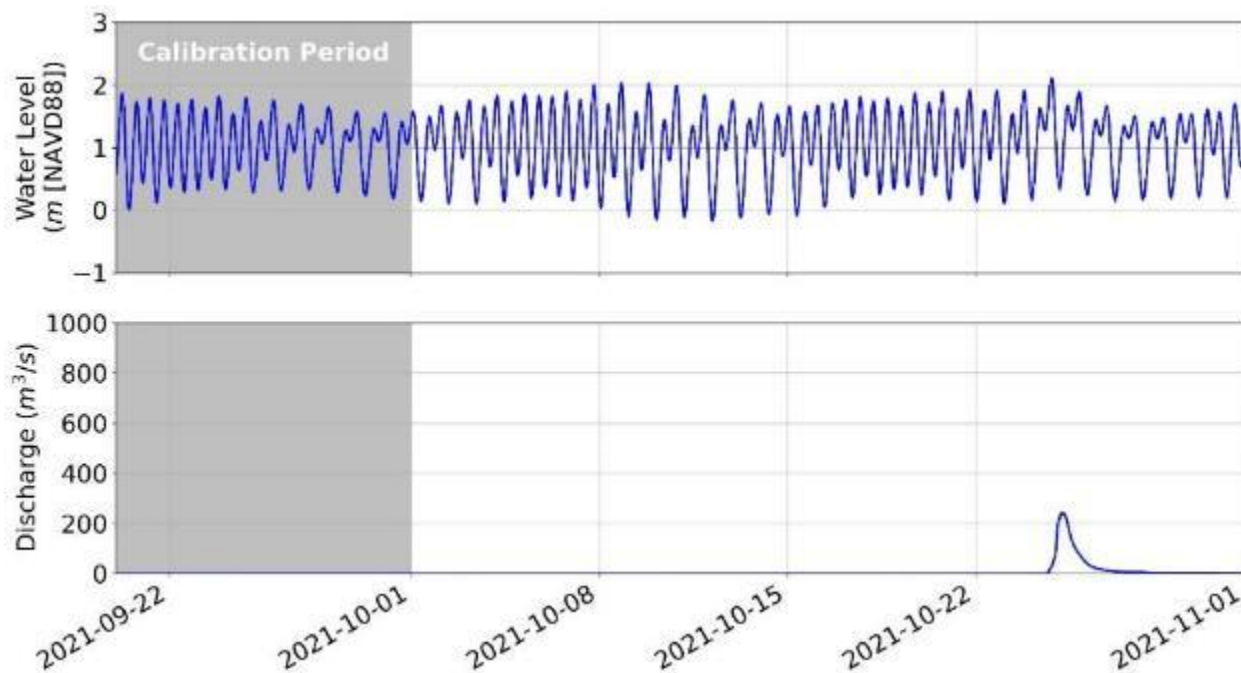


Figure 2-3. San Francisco Bay-Delta Community Model Water Level and Napa River Discharge Boundary Conditions. The selected calibration period is shaded in gray.

<sup>1</sup> <https://waterdata.usgs.gov/nwis/uv?11458000>

## 2.1.2 Bay Model Evaluation

Given that the Bay model would be providing boundary conditions to the Mare Island Strait model, it was important to show that the larger scale model accurately represents hydrodynamic conditions around San Francisco Bay.

The Bay model was assessed over a spring-neap tidal cycle during the evaluation period (September 20 to October 1, 2021) to ensure the model could adequately capture the full range of tidal elevations (Figure 2-3). Two NOAA-managed tide stations were selected to compare to the model predictions, Station 9414863 at Richmond, California, a site northeast of the Golden Gate Bridge, and Station 9415102 at the Martinez-Amorco Pier, California, southeast of the entrance to the Mare Island Strait (Figure 2-4).

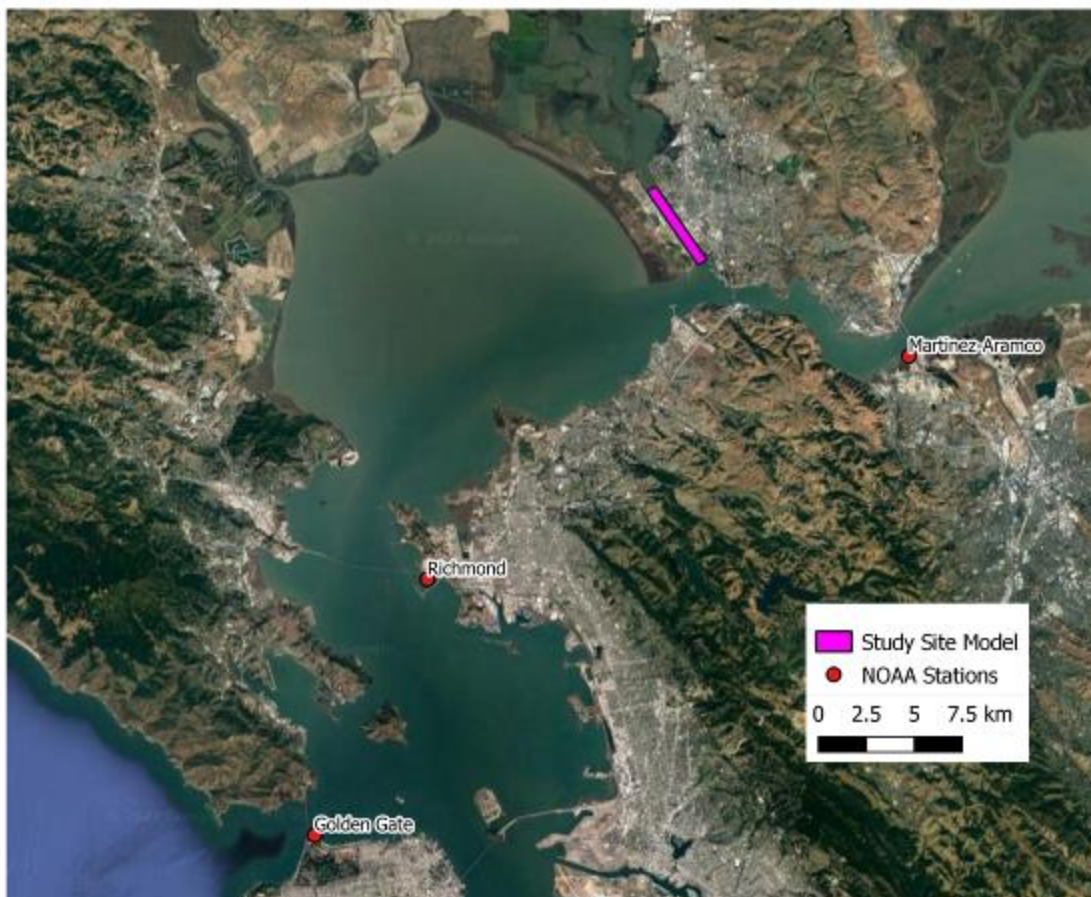


Figure 2-4. Bay Model Comparison Stations and Location of Mare Island Strait Model Domain.

The Bay model was modified in the Mare Island Strait portion of the domain by deepening some cells to align with the higher fidelity Foth provided bathymetry to better align the modeled velocities with the measured ADCP data. The bathymetry with the original model

limited flow and by deepening some areas, the boundary conditions derived from this model required less calibration for application than the Mare Island Strait model.

The Bay model does well in predicting the measured water levels at the two NOAA stations within San Francisco Bay. The Bay model best predicts water level at the Richmond station, which is closer to the Golden Gate Bridge and the primary water level boundary condition (Figure 2-5). The timing of the flood and ebb tides, and high and low tides, is correctly predicted; however the Bay model does overpredict the height of the high and low tides, though the maximum difference is 25 cm. This is likely a result of the ocean water level boundary condition. This difference in measured vs. modeled data will likely have a minimal impact on the hydrodynamics within the Mare Island Strait. The Bay model predictions at the Martinez-Amorco Pier station are similar in the accurate timing of the flood and ebb tides, and high and low tides. The overprediction of the high and low tides is increased moving northeast into the Bay, though the maximum difference is 33 cm. The overall agreement between the measured data and the Bay model predictions during the selected evaluation period supports the conclusion that the model is producing valid results at the scales of interest for the findings in this study.

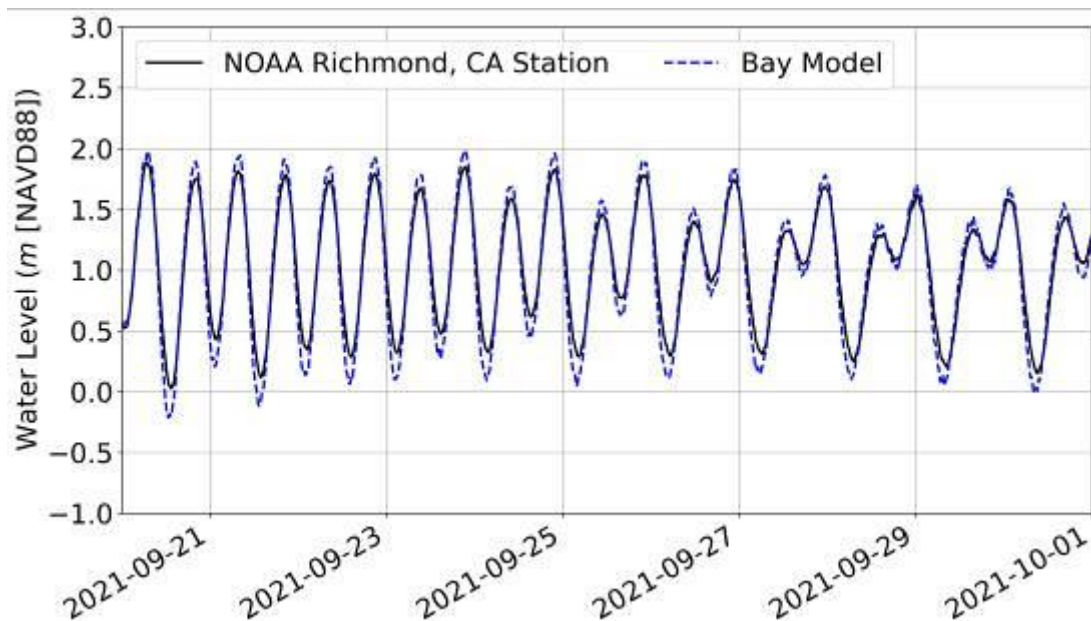


Figure 2-5. San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9414863 at Richmond, California.

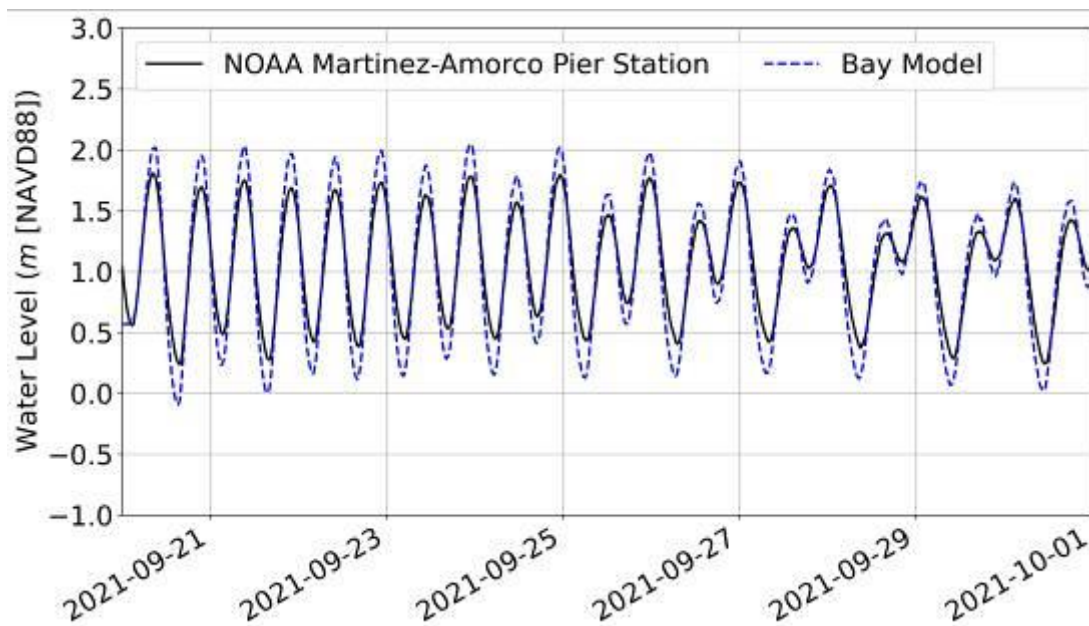


Figure 2-6. San Francisco Bay-Delta Community Model Water Level Predictions Compared to Measurements from NOAA Station 9415102 at the Martinez-Amorco Pier, California.

## 2.2 VALLEJO MARINA MODEL

A fundamental component and one of the first steps in the setup of a hydrodynamic model is constructing a model grid. The model grid discretizes the region of interest so that the parameters of interest (e.g., water level, velocity, shear stress) may be simulated accurately at a resolution appropriate for the project. For this study, Foth provided a previously developed D-Flow FM hydrodynamic model of the Mare Island Strait. The Vallejo Marina model domain encompasses the entire width of the Mare Island Strait and extends north to Sears Point Road and south to the mouth of the strait (Figure 2-7). The model resolution varied with the main stretch of river has a horizontal resolution of 22 m, and then two refinement areas with a transition zone of 9 m and 5 m within the marina.

The model was set up as 2-dimensional, meaning the water column is represented as a single depth-averaged layer. This assumption was made because of the relatively shallow depths within Mare Island Strait and assumed negligible effect of any salinity- or temperature-driven circulation given the limited flow input from the Napa River. The bathymetry used in the Vallejo Marina model was supplied by Foth (Figure 2-9).

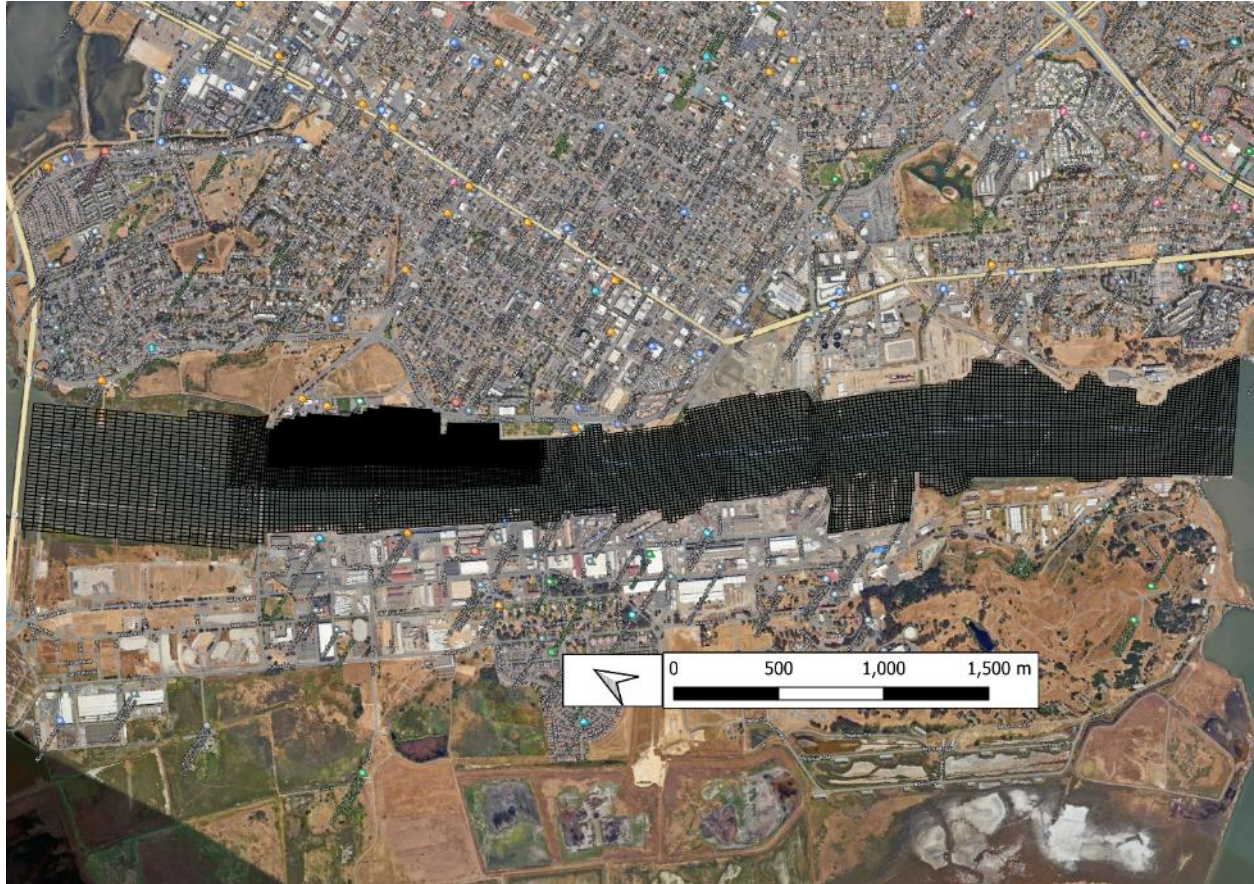


Figure 2-7. Overview of Vallejo Marina Model Grid

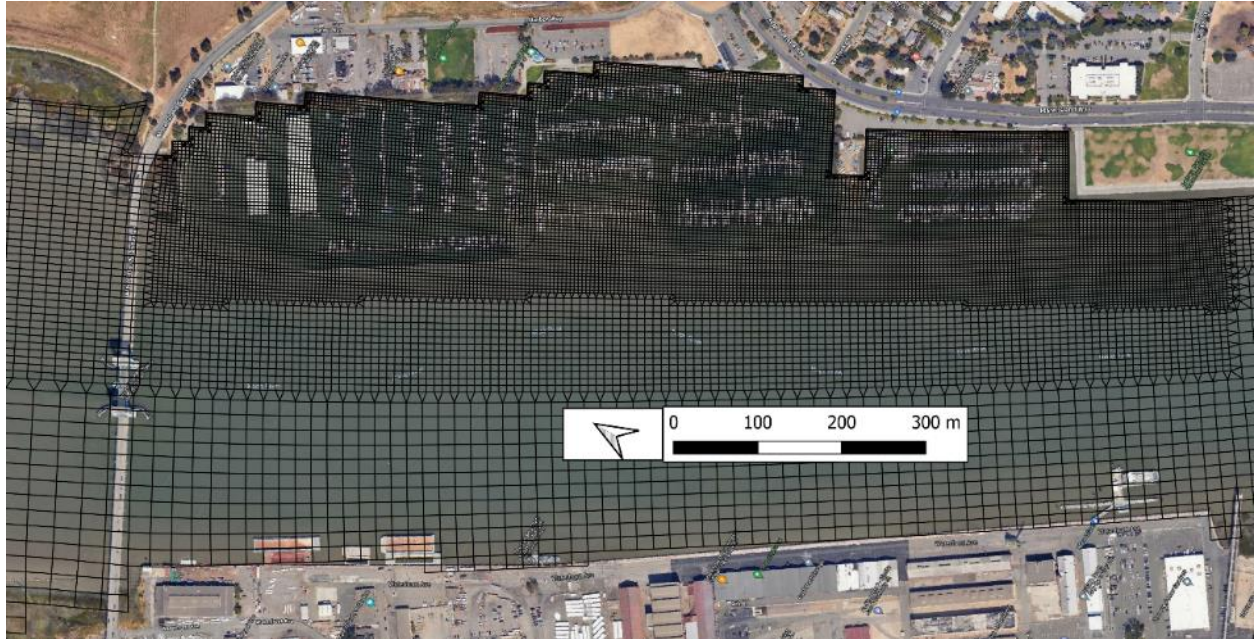


Figure 2-8. Vallejo Marina Grid refinement around marina.



Figure 2-9. Overview of Vallejo Marine model bottom elevations

The Vallejo Marina model was coupled with the Bay model offline to ensure the interaction of larger scale flows were included in estimations of bed shear stress. Offline coupling signifies that the San Francisco Bay-Delta Community model was run first, and then the outputs from that run were input into the Vallejo Marina model. The internal time-step of both models was

dynamic and changes to meet the model numerical stability criteria, allowing for a stable solution and potential decreases in computation time without sacrificing solution accuracy.

## 2.2.1 Hydrodynamic Forcing Conditions

Hydrodynamic conditions at the open northern and southern model boundaries provide the forcing that drives the water mass within the model domain and usually consists of the water level at a boundary of the model domain, or discharge into or out of the domain, or a combination of both. Two boundary conditions were applied to drive circulation within the Vallejo Marina model domain, riverine discharge at the north boundary and water level at the south boundary (Figure 2-10). These two boundary conditions for the model were extracted from the Bay model by means of defining output transects to save discharge and water level predictions where the model boundaries fall within the Bay model.

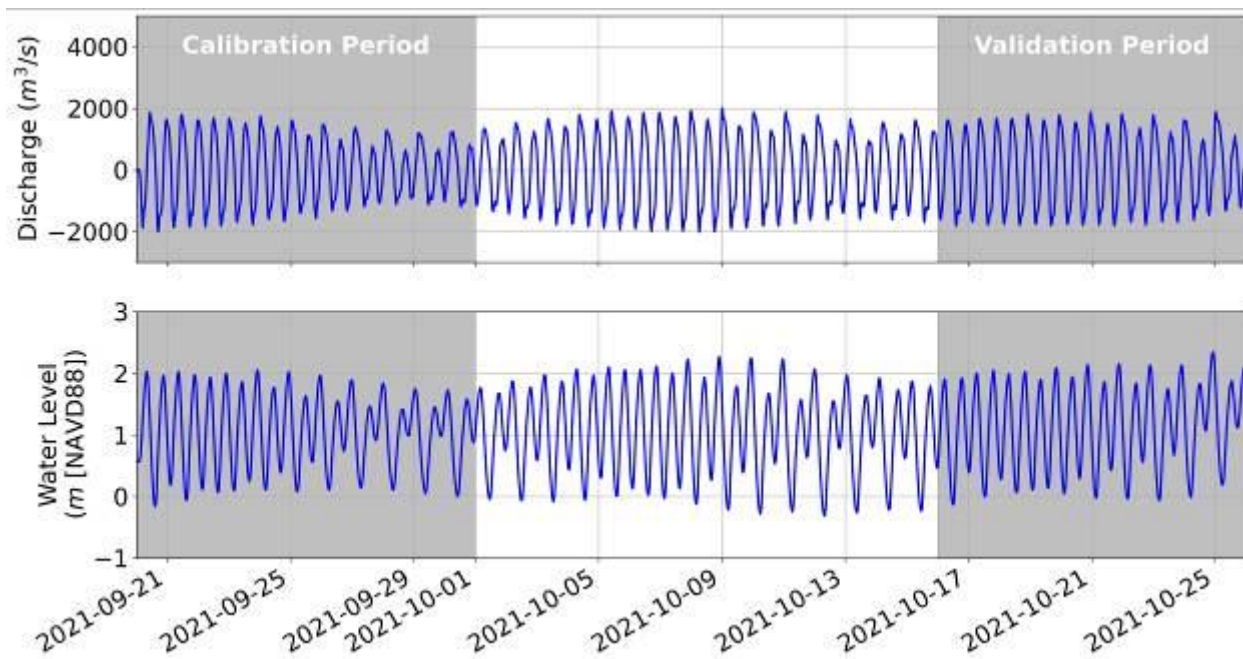


Figure 2-10. Vallejo Marina Model Discharge and Water Level Boundary Conditions Extracted from the Bay Model. The selected calibration and validation periods are shaded in gray.

## 2.3 VALLEJO MARINA MODEL EVALUATION

Evaluation of the Vallejo Marina model was performed to qualitatively and quantitatively assess how well it reproduced observed hydrodynamics within Mare Island Strait. Model evaluation consisted of two separate steps, model calibration followed by model validation. In addition, after the validation, the sensitivity of the model to various parameters was further

assessed. Model calibration and validation, as used in U.S. Environmental Protection Agency guidance (USEPA 2009), are defined as follows:

- **Model calibration**—Using site-specific information from a historical period to adjust model parameters to more accurately reflect measured site-specific conditions
- **Model validation**—Demonstrating that the calibrated model accurately reproduces known conditions over a period different from that used for calibration.

A single ADCP was deployed by Foth within Mare Island Strait, in a water depth of 5 m, in the main channel northwest of the Vallejo Ferry Terminal, to record current speed and direction. The ADCP was deployed in support of a project on the Vallejo Ferry Terminal, and the data was released for use in this project. Two deployments of the ADCP were made, the first, with a data record of September 20–October 26, 2021, and the second, with a data record of November 11–December 2, 2021. These measured current data were used to compute a depth-averaged velocity (DAV) and reviewed to identify time periods representative of the range of typical conditions observed at the Site (Figure 2-11). The negative values shown in the figure represent an ebb tide and the positive values are a flood tide. Based on the single Napa River flow event recorded during the first ADCP data record, two time periods were selected within this data record that encompassed spring and neap tidal cycles. No Napa River discharge events were recorded during the second ADCP deployment. The time period of September 20–October 1, 2021, was selected for the model calibration. This date range included a spring to neap tide transition and no flow from the Napa River. October 16–26, 2021, was selected for the model validation. This date range encompasses a neap to spring tide transition and a Napa River flow event, with a maximum discharge of 241 m<sup>3</sup>/s.

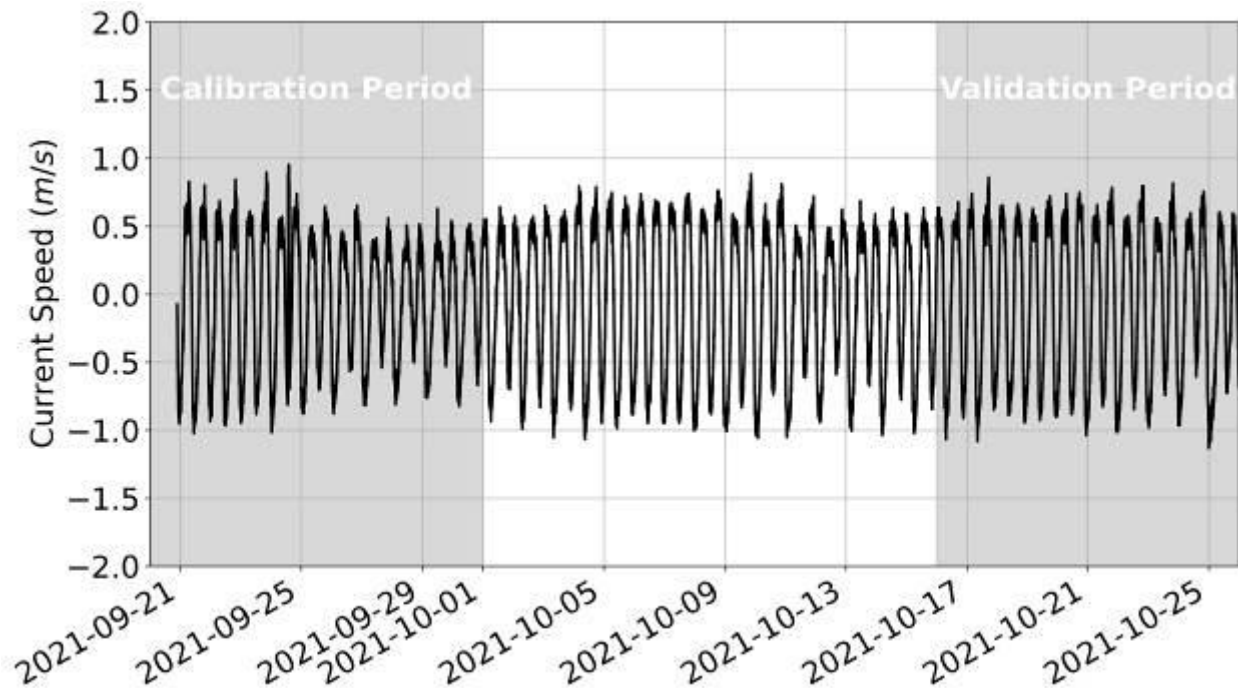


Figure 2-11. ADCP depth averaged current velocity used for calibration and validation of the hydrodynamic model. The selected calibration and validation time periods are shaded in gray.

The flow model was assessed over multiple spring-neap cycles during the calibration period and validation periods to ensure the model could adequately capture the full range of tidal elevations. In this system, successful calibration of the flow model relied on modifying the hydrodynamic bottom roughness within the domain, modifying the discharge boundary condition extracted from the Bay model. The bottom roughness around the Site was used as a calibration factor, to align velocity magnitudes with the measurements and a value of 0.026 m was applied uniformly throughout the domain. Within the area of flow grid refinement around the Site, the bottom roughness supplied by the Bay model was set to a constant value. The smaller scale processes resolved as a result of increased grid resolution, and the inclusion of structures at the Site necessitated the divergence from the values supplied in the Bay model.

### 2.3.1 Model Performance Metrics

Quantitative metrics were selected and used to evaluate the model calibration and validation performance, or the ability of the model to accurately reproduce measured water level and velocity at the Site. These metrics, combined with the qualitative evaluation (e.g., visual inspection of model results), provide an assessment of how well the model predicts the measured values at the study site. The metrics used to evaluate model performance include the model skill score (Murphy and Epstein 1989), root mean square error (RMSE), Pearson's

correlation coefficient, and skew. The model skill score served as the primary basis for evaluating model ability across a range of parameters and was computed as follows:

$$SS = 1 - \frac{\sum(x_{meas} - x_{model})^2}{\sum(x_{meas} - \bar{x}_{meas})^2}$$

Where  $x_{meas}$  is the measured value,  $x_{model}$  is the model-predicted value, and  $\bar{x}_{meas}$  is the average measured value.

The skill score varies from 0 to 1, with 1 representing perfect agreement between measured and modeled data. The model skill values, as well as Pearson's correlation coefficient, described below, are characterized in Table 2-1, based on values presented in Marechal (2004).

Table 2-1. Model Skill Performance Metrics Set for This Study

Skill Score	Performance
0–0.2	Poor
0.2–0.5	Good
0.5–0.65	Very Good
0.65–1	Excellent

The RMSE is a measure of the average difference between model-predicted and measured values and was computed as follows:

$$RMSE = \sqrt{(x_{model} - x_{meas})^2}$$

The computed RMSE was compared with measured values such that an acceptable RMSE value for current speed verification is  $\pm 30$  percent of the average measured values (USEPA 1990).

Finally, Pearson's correlation coefficient was used to ensure the model is qualitatively reproducing the temporal variability in measured values by computing the linear association of modeled and measured values. The correlation coefficient ( $r$ ) was computed as follows:

$$r = \frac{n \sum(x_{meas}x_{model}) - \sum(x_{meas})\sum(x_{model})}{\sqrt{[n \sum(x_{meas}^2) - (\sum x_{meas})^2][n \sum(x_{model}^2) - (\sum x_{model})^2]}}$$

Where  $n$  is the number of data points in the time series. Evaluation of Pearson's correlation coefficient depends on the number of samples compared; the values vary from 0 (no correlation) to 1 (perfect correlation). The Pearson's  $r$  value can be classified with the same performance values as used for the model skill score (Table 2-1).

Together, these metrics provide a robust evaluation of the model's ability to reproduce measured data. Model calibration is considered complete when the model skill score falls within the very good to excellent range, or if it is determined that available data are insufficient to support additional model calibration.

### **2.3.2 Vallejo Model Calibration**

The hydrodynamic model calibration consisted of systematic adjustments to model parameters within a reasonable range until measured Site conditions were accurately simulated by the hydrodynamic model. Calibration involved the following steps:

- Adjustment of model calibration parameters, including bottom roughness and the horizontal eddy viscosity coefficients.
- Adjustment of the north boundary discharge from the Bay model.

Upon initial analysis of the measured data versus modeled predictions, the Vallejo Marina model was underpredicting the ebb tide and flood tide velocities from the Mare Island Strait ADCP. The adjustment of bottom roughness and viscosity, calibration parameters that influence velocities, had minimal impact on the underpredictions. The bottom elevations of the hydrodynamic model were found to be accurate when compared to measured values. This led to the conclusion that the discharge boundary conditions, derived from the Bay model, were the primary factor in the model predictions. The Bay model was likely not discharging enough flow volume into the Mare Island Strait. The discharge boundary condition was iteratively adjusted and compared to the measured DAV. The final adjustment resulted in a doubling of the ebb tide discharge and a 20 percent increase in the flood tide discharge (Figure 2-12). The water level boundary condition derived from the Bay model was not adjusted. This adjustment of the discharge boundary condition was necessary to achieve the model calibration and validation presented below. In addition, the evidence of the shallow bathymetry of the Bay model within the Mare Island Strait presented in Section 2.1.2, which likely extends upstream into the Napa River, supports the need for adjustment of the discharge provided by the Bay model.

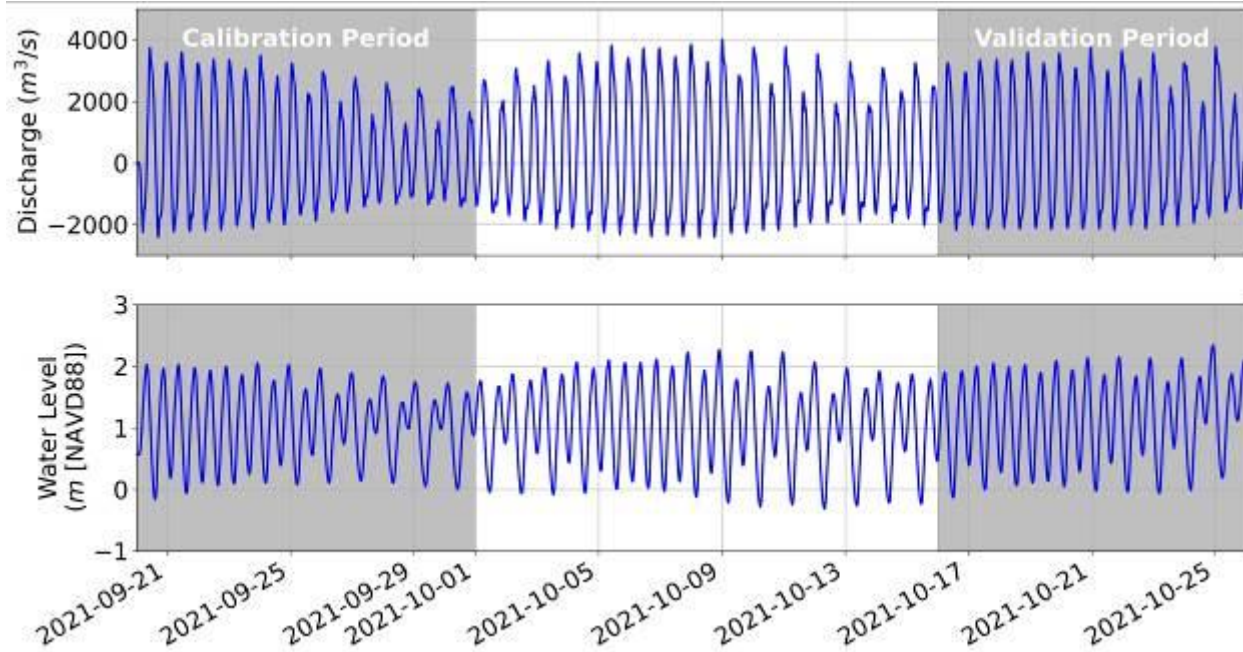


Figure 2-12. Discharge and water level boundary conditions. The selected calibration and validation periods are shaded in gray.

The measured versus modeled DAV, after incorporating the calibration changes described above, is presented in Figure 2-13. The negative values shown in the figure represent an ebb tide and the positive values are a flood tide. The model captured the timing and the magnitude of the depth averaged current for flood and ebb tides during the spring to neap tide transition. The model does underpredict the current speed on ebb tide moving into the neap tide at near the end of the calibration period, though the difference in the measured data and model predictions are small and would have a minimal impact on the predictions of bed shear stress.

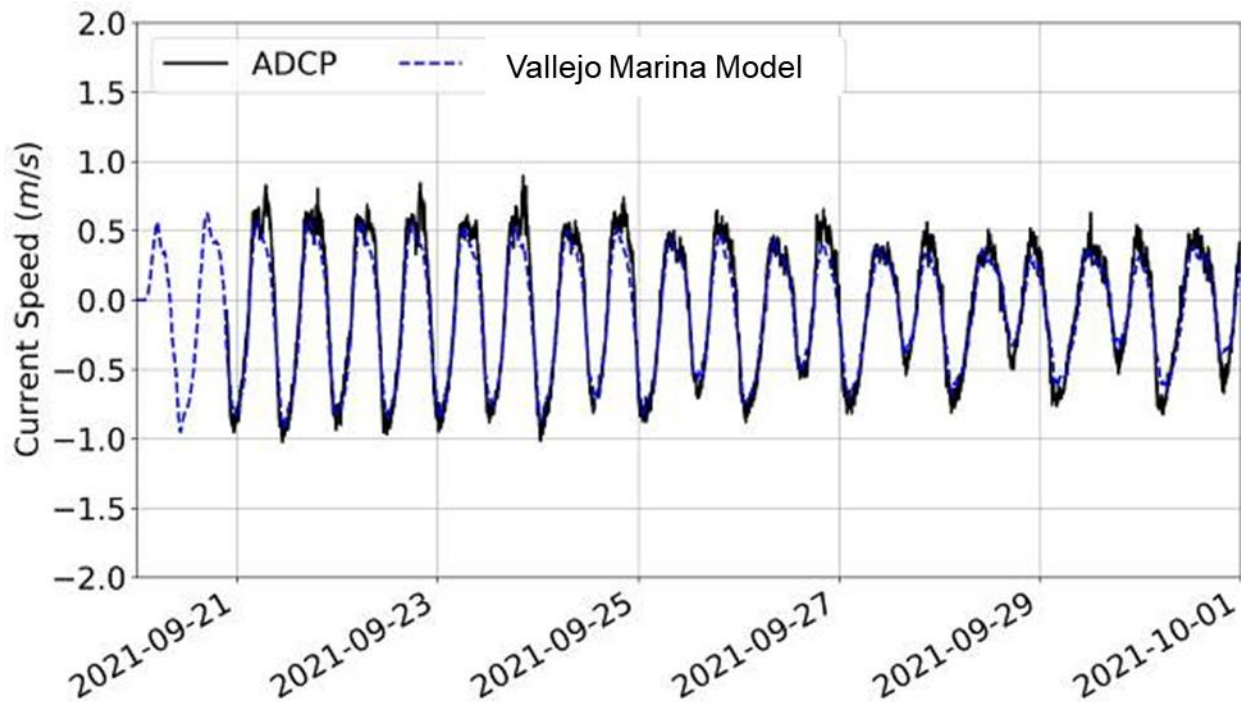


Figure 2-13. Vallejo Marina Model calibration to depth averaged velocity from the Mare Island Strait ADCP.

When the quantitative model performance metrics for the calibration period were computed, the DAV was in excellent agreement with the measured data, as shown by the skill and  $r$  values (Table 2-2). The Pearson’s  $r$  value, a performance metric used to evaluate how well the model is reproducing the temporal variability in measured values, showed an excellent correlation between the measured and modeled DAV. The RMSE value, representative of the difference between the measured and modeled values, was within 2 percent of the 30 percent limit defined by USEPA (1990). The quantitative metrics paired with the calibration figures support a good model calibration. Though there is under- and overprediction, it is likely a result of the uncertainty in the bottom elevations and the boundary conditions supplied by the Bay model. Model uncertainty is discussed further in Section 2.4.

Table 2-2. Calibration Performance Metrics for Vallejo Model

	Skill	RMSE	Pearson’s $r$	Performance
Depth Averaged Velocity	0.98	0.12 m/s (32%)	0.98	Excellent

### 2.3.3 Vallejo Model Validation

The model validation was performed to test the selected calibration parameters during hydrodynamic conditions that are different from the calibration conditions. Compared to the calibration conditions, the model validation conditions represent a higher flow rate period resulting from the single recorded discharge event from the Napa River.

Similar to the calibration period, the predicted DAV was compared to the measured DAV recorded by the ADCP (Figure 2-14). The model captured the timing and the magnitude of the depth averaged current for flood and ebb tides during the neap to spring tide transition. The model does underpredict the current speed on flood tide moving into the spring tide at near the end of the validation period, though the differences in the measured data and model predictions are small. The model does well at predicting the elevated ebb tide DAV on October 25, 2021 as a result of the Napa River discharge. The small underpredictions of the flood tide DAV would have a minimal impact on the predictions of bed shear stress.

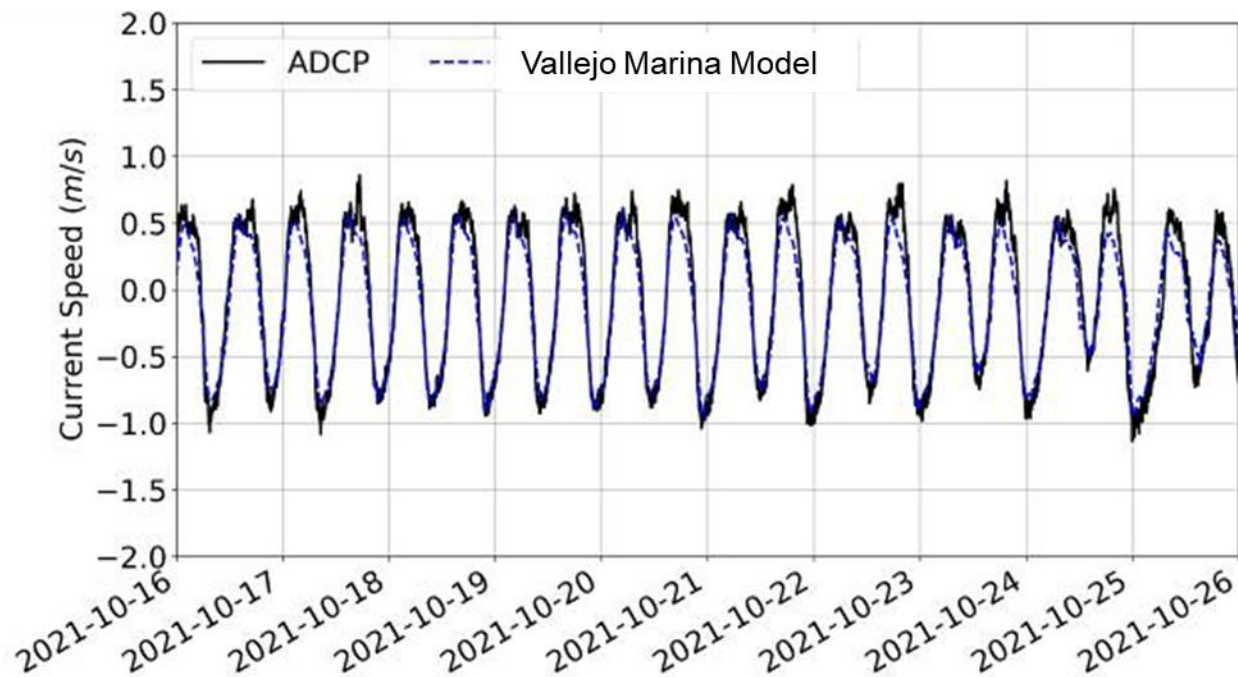


Figure 2-14. Vallejo Marina Model Validation to Depth Averaged Velocity from the Mare Island Strait ADCP.

When the quantitative model performance metrics for the validation period were computed, the DAV was in excellent agreement with the measured data, as shown by the skill and  $r$  values (Table 2-3). The RMSE value, representative of the difference between the measured and modeled values, was within 4 percent of the 30 percent limit defined by USEPA (1990).

Table 2-3. Validation Performance Metrics for Vallejo Model

	Skill	RMSE	Pearson's <i>r</i>	Performance
Depth Averaged Velocity	0.98	0.15 m/s (34%)	0.98	Excellent

Overall, the model calibration and validation are in good to excellent agreement with the variations in DAV. Given the objectives of this modeling effort, evaluating bed shear stress and possible sedimentation in and around the proposed Vallejo Marina modifications, the calibrated and validated model satisfactorily captures the temporal variations and magnitudes of the measured values, which are the primary forcing components of bed shear stress and associated sedimentation.

## 2.4 MODEL UNCERTAINTY

Overall, the hydrodynamic models adequately capture site dynamics that are important for effective remediation design. This is supported by the good fit between measured and simulated hydrodynamic data. The models are meant to be used as tools to aid remediation design, and the parameters used in the model are subject to uncertainty.

Typical sources of uncertainties in a hydrodynamic model arise from the model grid setup including bathymetry, boundary conditions, and choice of hydrodynamic parameterizations. Subgrid-scale processes, such as bottom topography variation within a grid cell, are a source of uncertainty in any hydrodynamic model. The Site also contains many structures that modify and direct flows, shelter areas from wave activity, and provide berthing areas for vessels that may have an influence on localized dynamics.

Errors in estimating the bottom topography used in the model arise from both measurement uncertainties and the finite grid scale. Errors from the finite grid scale occur due to the use of an average depth for the entire grid cell, whereas nature might exhibit more vertical variation in an area than can be represented by a single elevation in a model grid cell. Overall, the bottom topographic variation away from the banks of Mare Island Strait is minimal, and the high-resolution grid captures the variations in the Site well.

Boundary conditions for the model consisted of measured water levels from the Golden Gate Bridge. While the water level data are validated, the water levels were measured 40 km from the model boundary and were adjusted temporally to align model and measurement phases.

The instruments used to collect velocity data have a cited precision value of  $\pm 2.2$  cm/s. Maximum depth-averaged velocity magnitudes were relatively low, ranging from 0.5 to 1.0 m/s, which means the velocities could vary as much as 22 percent at the lower maximum value. The velocity data have uncertainty in interpretation and application for validation. The velocities are

measured at points that are smaller than the grid scale and include other sub-grid scale turbulent processes that contribute to uncertainty in model and data comparison.

### 3 SEDIMENTATION STUDY

#### 3.1 EXISTING DATA SETS

Foth provided existing bathymetric survey files of the Vallejo Marina in the form of delimited text files. Dates of surveys and whether they were used in subsequent analysis are shown in Table 3-1. All datasets were converted to a common vertical reference system, NAVD88 meters relative to the Martinez-Amorco Pier (9415102) NOAA tide gage station. All XYZ surveys were then mapped in GIS and projected into UTM 11 coordinates. Surveys were then interpolated from point locations to a raster grid. This was done in two ways, surveys with an evenly spaced grid were converted directly from points to raster. Surveys with unevenly spaced grids were interpolated into raster geotiffs using the nearest neighbor interpolation method at a resolution determined by the spacing of the survey locations. A post processing step was then conducted to fill small gaps in the raster surface, and the raster was clipped by the outer bounds of the survey. For consistency, all surveys were snapped to a common grid and resampled to standard 1-meter resolution.

Table 3-1. Bathymetry survey dates considered in sedimentation analysis

Date	Used in Sedimentation Analysis	Date	Used in Sedimentation Analysis
2013 June 1		2016 Oct. 17	
2014 Apr. 2	X	2019 June 24	X
2014 June 19	X	2019 June 26	X
2014 July 16	X	2019 Aug. 02	X
2014 July 21	X	2019 Aug. 29	X
2014 Oct.16	X	2019 Oct. 11	X
2014 Oct. 29		2019 Oct. 21	
2014 Oct. 29	X	2019 Oct 21	
2014 Oct. 31		2022 May 02	X
2016 Jan. 01	X	2022 Aug. 01	X
2016 June 01	X	2023 Sep. 22	X
2016 Oct. 17	X		

Each raster was further standardized to a uniform a 1-m grid with bounding coordinates of (563,665 m, 4,217,696m) and (564,382m, 4,218,510m) to account for variations in spatial extent and raster cell coverage using the *RegularGridInterpolator* routine of the Python package *scipy*. The spatial occurrence of surveys is shown in Figure 3-1. This shows that many surveys are

limited to the marina openings and along a few docks. Often a single survey covers some of the marina and the marina extents along the north and south walls have no coverage.

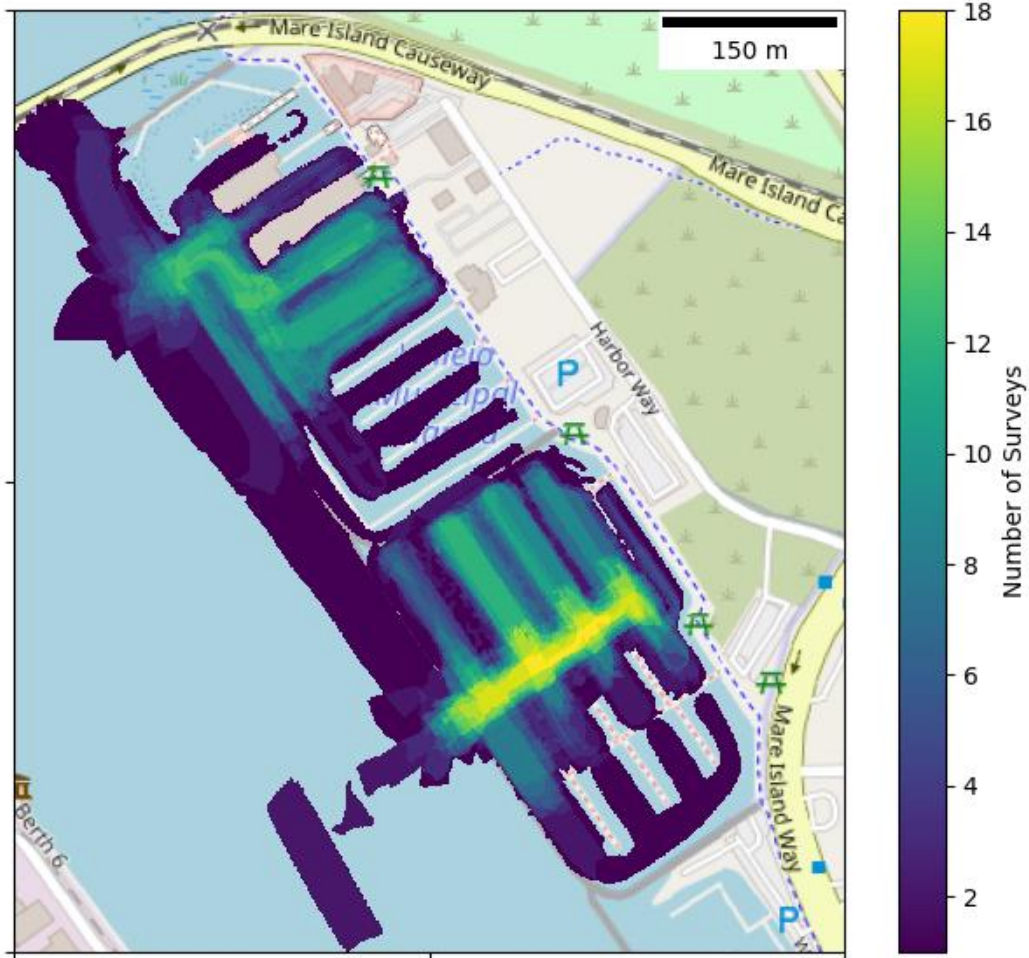


Figure 3-1. Spatial occurrence of available surveys

Some surveys either contained duplicate data or were only a few days apart, resulting in incorrect change rates and/or noise due to the short time period. For these survey files, only the earliest date and the survey with the largest spatial coverage were included. This resulted in 17

datasets ranging from 2014 to 2023 covering pre- and post-dredge marina conditions (Table 3-1).

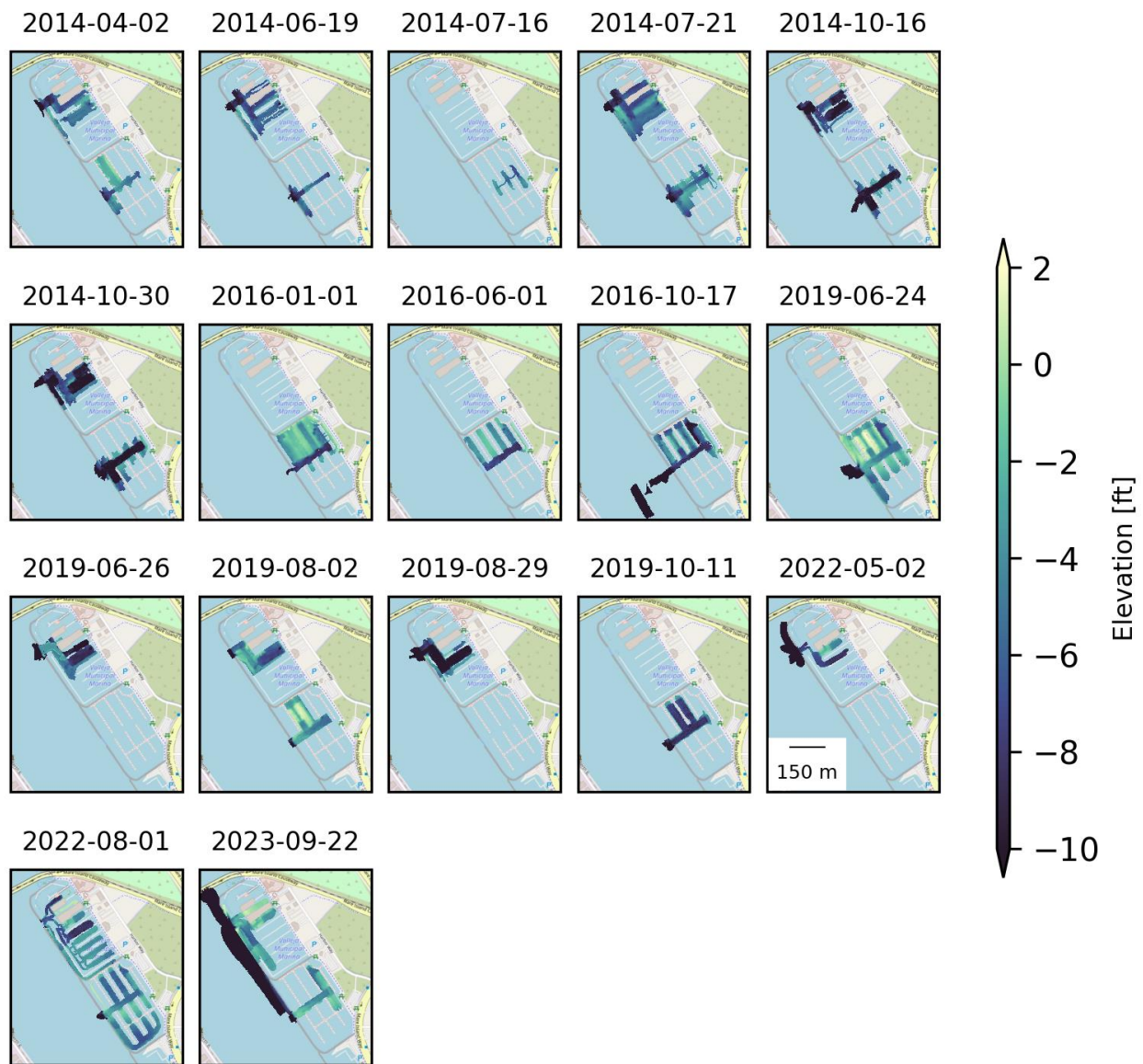


Figure 3-2. Vallejo Marina bathymetry referenced to NAVD88.

### 3.2 DEPOSITION RATE CALCULATION

The individual surveys were grouped for the north and south marina basins to determine sedimentation rates and between post- and pre-dredge surveys. The dates of dredging were inferred from documents provided by Foth and .xyz file names indicating pre-/post-dredge and are listed in Table 3-2. Due to the sparse spatial coverage of successive surveys, a direct elevation change between successive individual surveys was not possible for most cases. This

resulted in 4 estimates of sedimentation rate for the north basin and 6 for the south basin, each with limited spatial coverage due to preferential surveys near the marina entrances. The elevation changes for north basin are shown in Figure 3-3 and the south basin elevation change is shown in Figure 3-4. The bathymetric change maps indicate periods of accretion and erosion as well as infilling within the basins following dredge events. From the maps, the greatest depositional rates occur within the main navigable channels from the marina entrances to the north and south basin and decrease further within the basin. Erosion primarily occurs outside the marina in the straight.

Table 3-2. First post-dredge surveys

Date
2014 Oct. 16
2019 Aug. 2

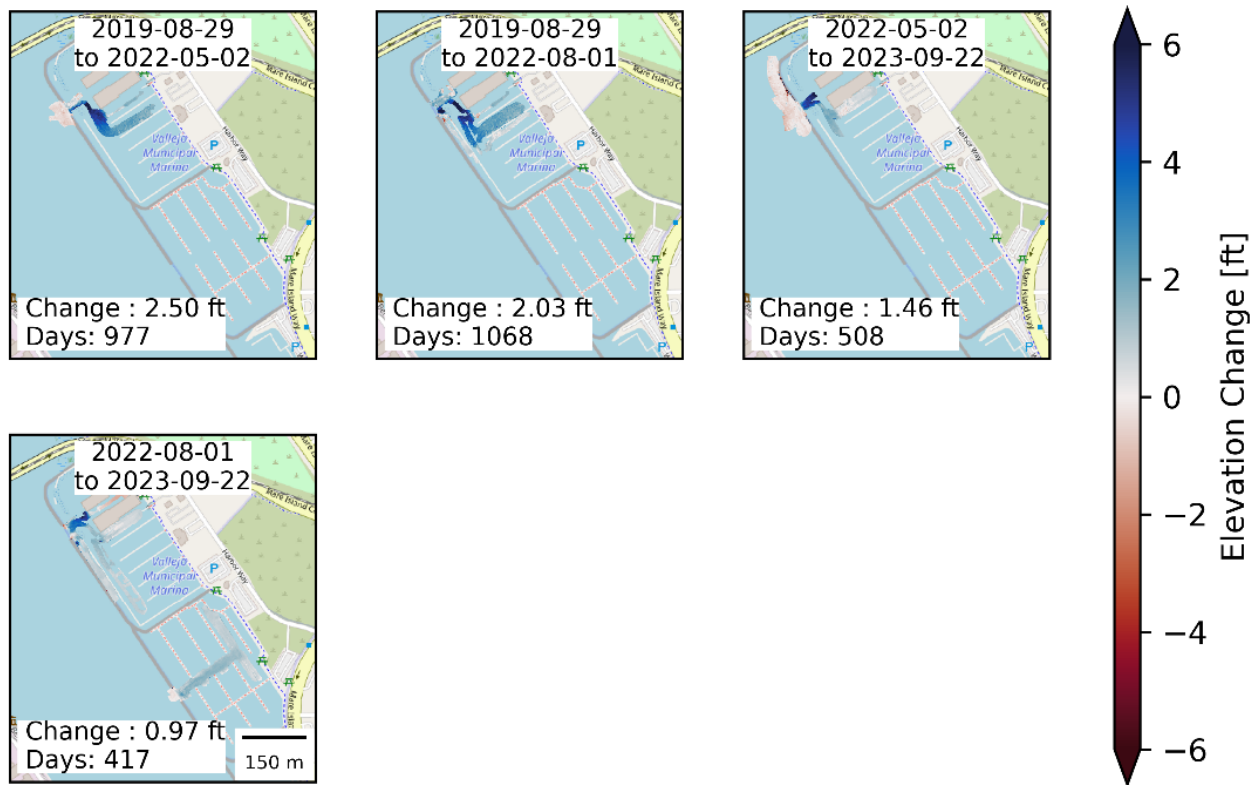


Figure 3-3. Inter-dredge north basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text).

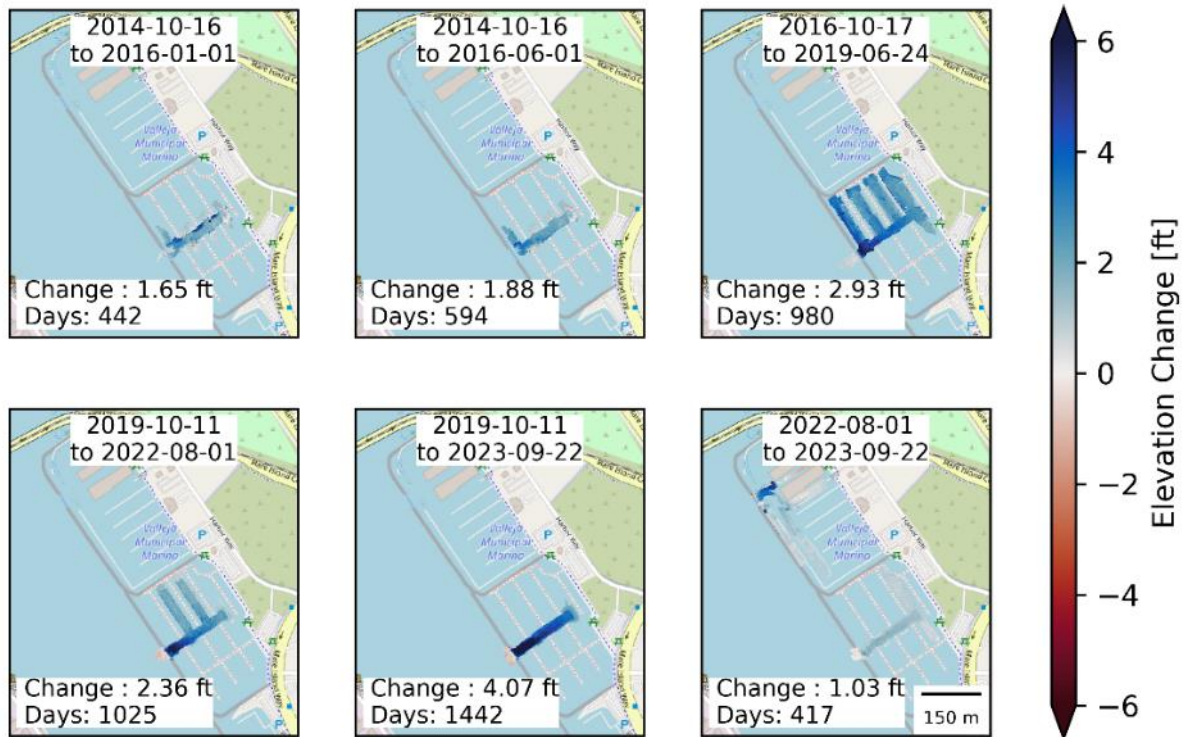


Figure 3-4. Inter-dredge south basin elevation change showing the date range to calculate each change (top text) and the average elevation change for each time period (bottom text).

The natural deposition rate was calculated from bathymetric change for each of these survey pairs for the north and south basins. The largest rates of sediment deposition occur within the main navigable channel from the marina entrances. The variation in average sediment deposition is attributed to the differences in spatial extent of the surveys with survey changes limited to the main channels resulting in larger accumulation rates than at more distal extents. This indicates a decrease in sediment availability due to sediment depositing out of the water column due to decreases in water velocity as the flow enters the marina. For the north marina basin (Figure 3-5), the deposition rate varies between 0.69 to 1.05 ft/yr. For the south marina basin (Figure 3-6), the deposition rate varies between 0.84 to 1.36 ft/yr.

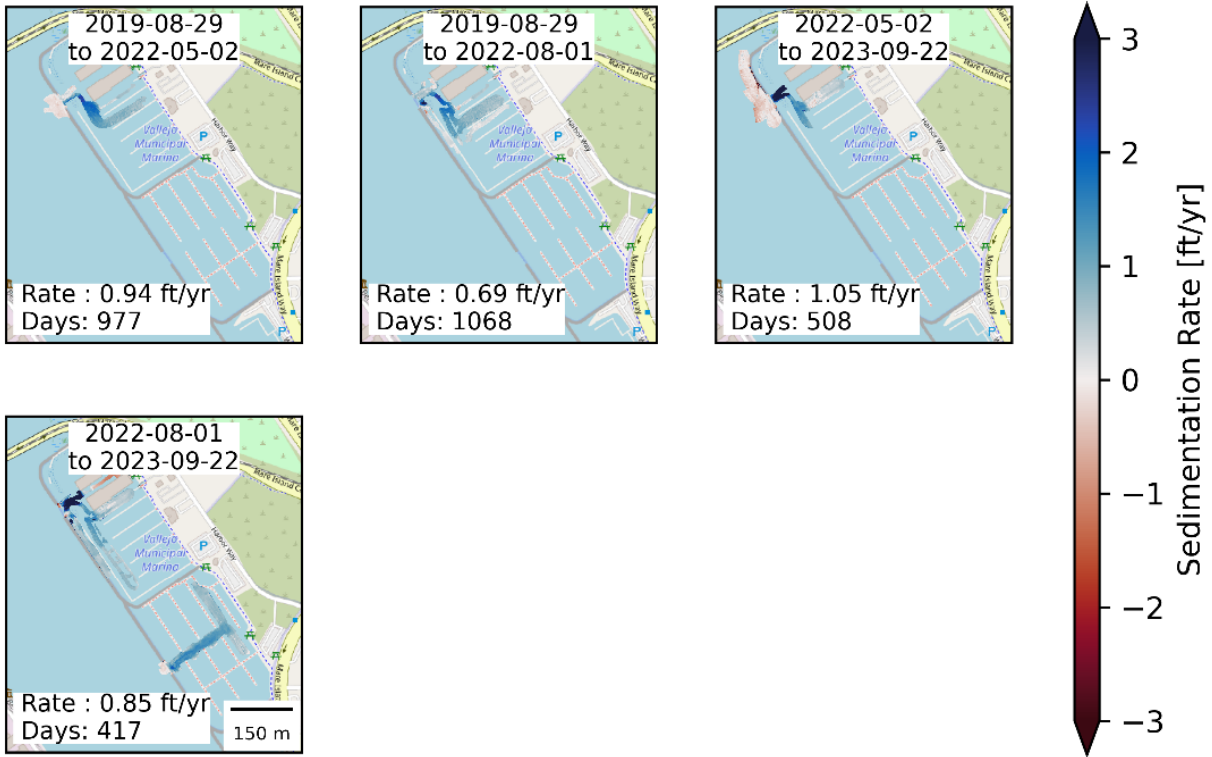


Figure 3-5. Inter-dredge deposition rate for the north basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text).

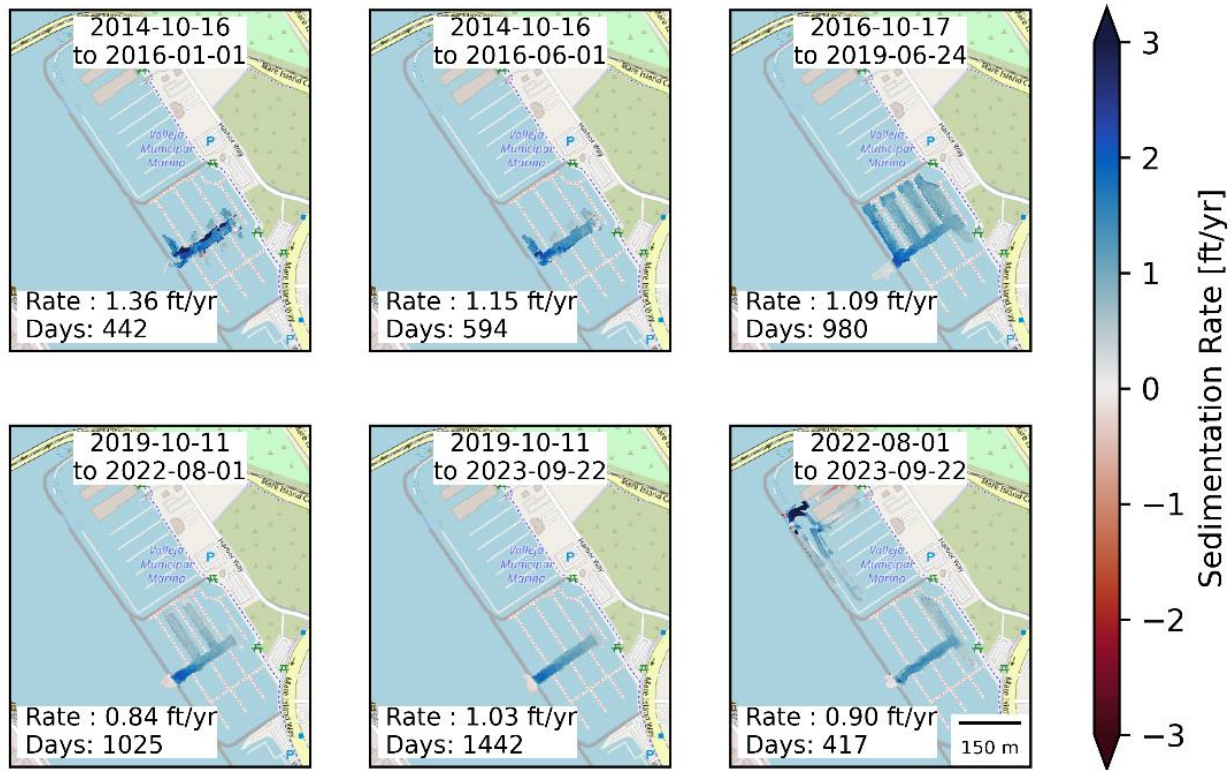


Figure 3-6. Inter-dredge deposition rate for the south basin showing the date range to calculate each change (top text) and the average sediment accumulation rate for each period (bottom text).

The average, median, and time-weighted average deposition rate, calculated for each 1-m grid cell is shown in Figure 3-7. The time weighted average results in a north basin sediment deposition rate of 0.85 ft/yr and a south basin deposition rate of 0.99 ft/yr. Due to the sparse survey data from which to calculate a sediment deposition rate, the marina with unknown rates was assumed to have the time-weighted mean deposition rate for the north basin and south basin. The resulting spatial depositional rate is shown in Figure 3-8.

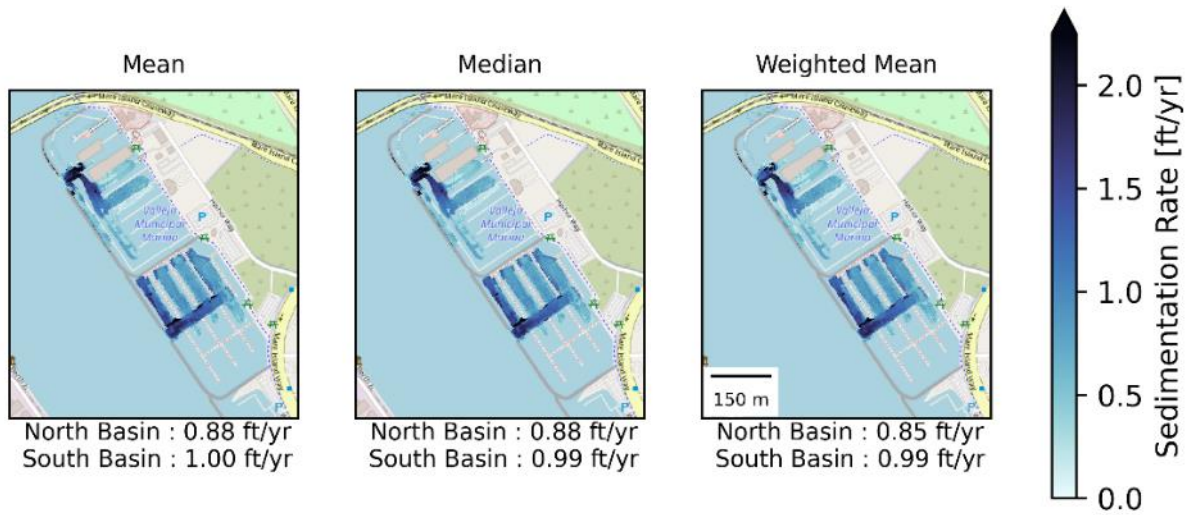


Figure 3-7. Calculated average (left), median (middle) and time-weighted (right) deposition rate.

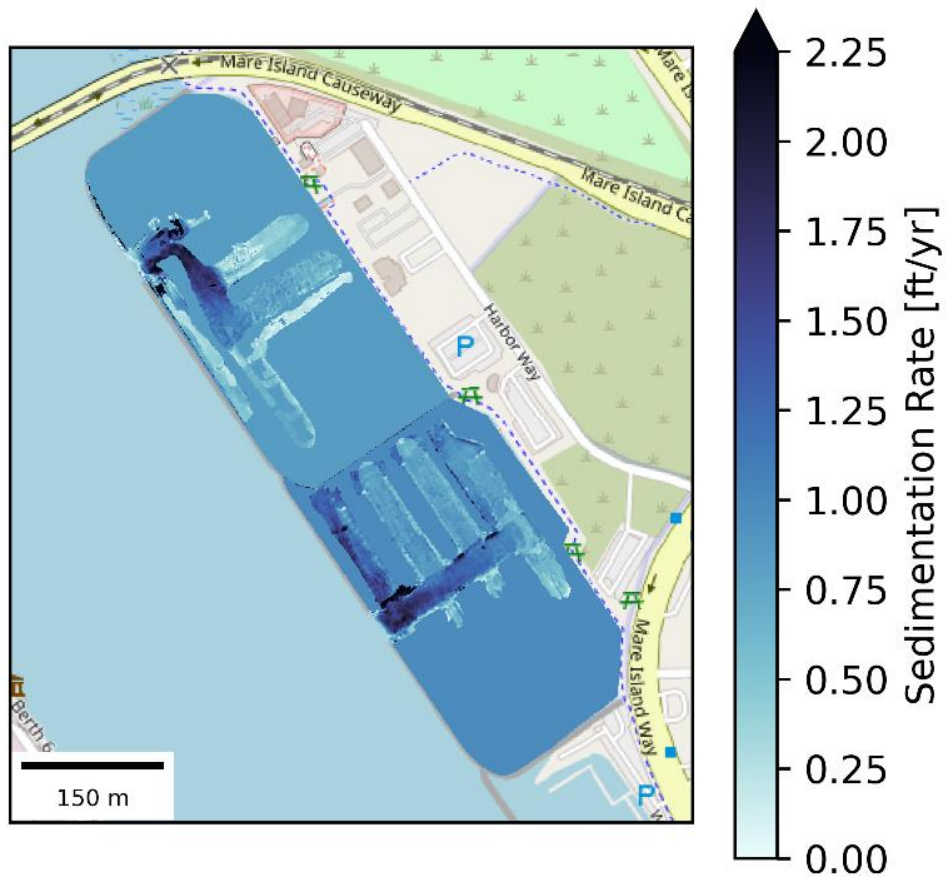


Figure 3-8. Time-Weighted Average Deposition Rate (top) and Maximum Deposition Rate (bottom).

## 4 ADDITIONAL ALTERNATIVE MARINA CONFIGURATIONS: MODELING AND ANALYSIS

The Vallejo Marina model was used to simulate typical site conditions over a 45-day period with alternative marina configurations. The alternative marina configuration modeling utilized the same model setup derived from the calibration and validation study. The sedimentation study produced estimates of the sedimentation rate in the system. The modeling analysis provides an estimate of maximum and average shear stress, the force that governs the ability for sediment to erode. The analysis performed here scales the measured deposition rates by the modeled shear stresses to estimate a new deposition rate within the marina for each alternative. The configuration of the alternative simulations and the results of the analysis are presented below.

The additional alternative marina configurations use an assumed post-dredge bathymetry. For this bathymetry, the north marina basin is dredged to an elevation of -10 ft MLLW and the south basin to an elevation of -10 ft MLLW. The sumps are dredged to a depth of -20 ft MLLW.

For Case 1, the present marina footprint is used without sumps and dredged to -10ft MLLW for both basins.

Case 2 was proposed by F3 and includes an alternative marina footprint with the north wall moved southward and 4 marina entrances (two existing and two proposed). For Case 3, the same marina footprint is used with the addition of a sump is added to the north-basin oriented east-west along the north wall and a northwest-southeast oriented central sump added between the north and south basin.

Case 4 was proposed by F3 and includes an alternative marina footprint with the north wall moved southward and with only the 2 proposed entrances. Case 5 adds two sumps, one oriented east-west along the north wall and another located northwest-southeast between the north and south basin.

### 4.1 ALTERNATIVE MARINA CONFIGURATION MODEL EVALUATIONS

A list of the model scenarios evaluated, and the associated bathymetry and wall configurations are shown in Table 4-1 and Figure 4-1.

Table 4-1. Marina configuration model simulations.

Case Number	Bathy	Marina Walls	Sumps
1	Post-Dredge	Existing	None

2	Post-Dredge	Shifted North Wall South, added north and south entrance	None
3	Post-Dredge	Shifted North Wall South, added north and south entrance	Includes two Sumps
4	Post-Dredge	Shifted North Wall South, removed existing entrance, added north and south entrance	None
5	Post-Dredge	Shifted North Wall South, removed existing entrance, added north and south entrance	Includes two Sumps

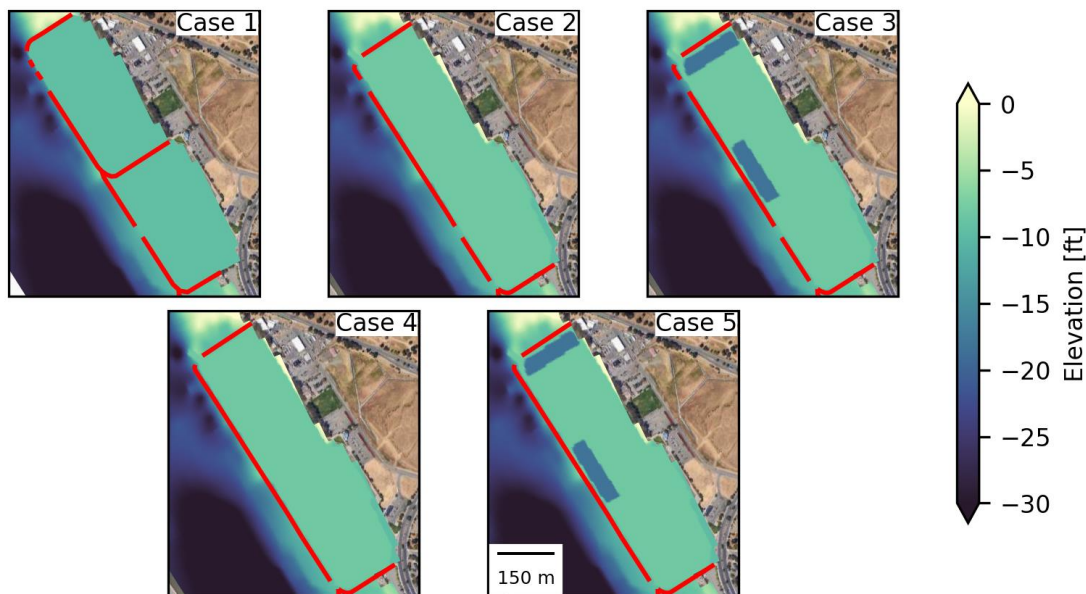


Figure 4-1. Bathymetry and marina wall configuration of the alternative analysis evaluation. NAVD88 CA State Plane Zone 2 with units of meters with vertical datum of NAVD88 with units of feet. Red lines indicate marina walls.

## 4.2 VELCOCITY

Velocity within the marina is indicative of sediment flow pathways and the potential path of sediment redistribution. There are two primary tidal flows that are the focus of this study. Flood-tide results during north-westward directed flow due to higher water elevations in San Francisco Bay while ebb-tide results in south-eastward directed flows due to receding water levels in San Francisco Bay. Sediment is introduced to the marina on flood tides and then settles out as currents change direction and slack tide occurs. The ebb tide may not be strong enough to resuspend sediment and flush the system, resulting in deposition.

The maximum velocity within the marina is calculated as the maximum velocity per grid cell during the complete model simulation (Figure 4-2). This indicates that velocities can exceed 3 ft/s near the marina entrances. For Case 1, faster velocities are confined to the northern portion of the north and south basins. Removing the dividing wall and adding the additional entrances in Case 2 results in increased velocities throughout the basins with the largest velocities along the channel oriented maria wall in the north basin. The lowest velocities are in the most northern part of the basin and the southernmost portion of the south basin. The additional southern entrance increases velocities in the south basin. Adding the two sumps in Case 3, results in similar flow patterns but with slower velocities in the sumps. In Case 4 compared to case 2, removing the two existing entrances and removing the sumps results in a slower velocity in the northern portion of the basin and a reduced velocity near the middle to south entrance near the channel wall. Add the tow sumps in Case 5 results in a reduction of the velocity near the sumps with little change in the rest of the maria compared to Case 4.

Comparing the maximum velocities for each alternative to Case 1 (Figure 4-3), Case 2 results in a increased velocity in majority of the marina with a decrease in the northeast corner of the marina and near the existing south entrance. A similar pattern is present for Case 3, with a notable decreased change in velocity within the sumps. For Case 3, there is a reduction in the velocity within the northeast corner of the marina as well as in the central section of the north basin compared to Case 1. There is also a reduction in velocity near the existing south entrance due to the closure in this case. Compared to Case 2/3, there is an increase in velocity along the marina's southern wall, due to an increase flow through the single south entrance. Case 5 is like Case 4 except for a reduced change in velocity within the sumps.

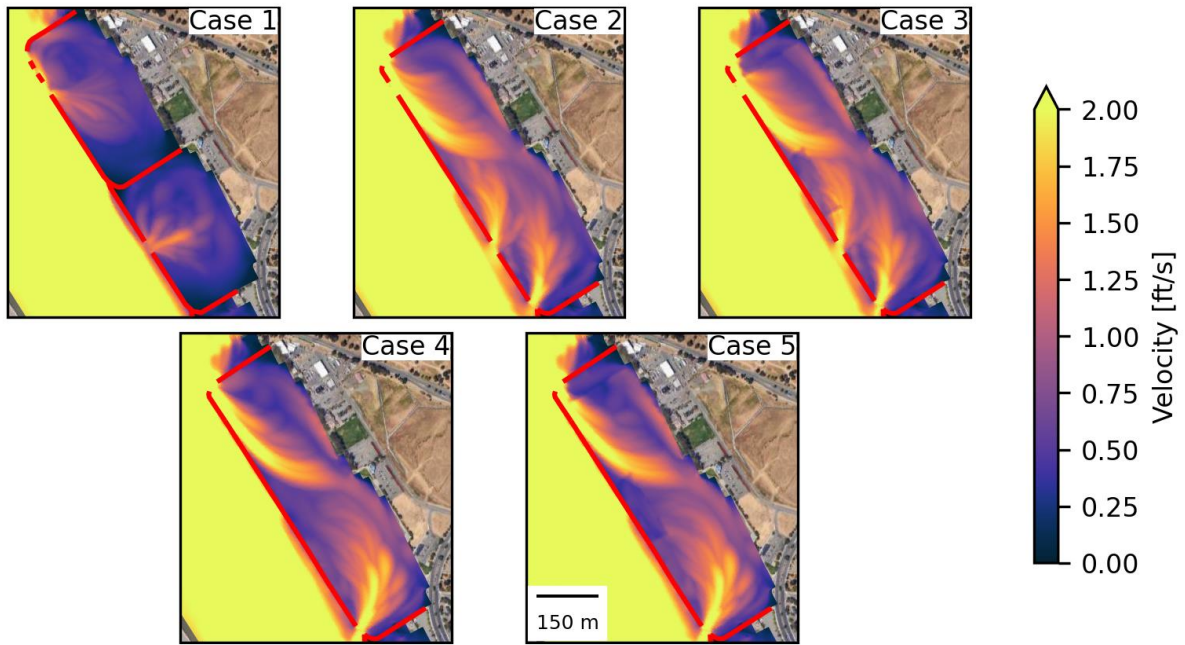


Figure 4-2. Maximum grid-cell velocity during the model simulation. Red lines indicate marina walls.

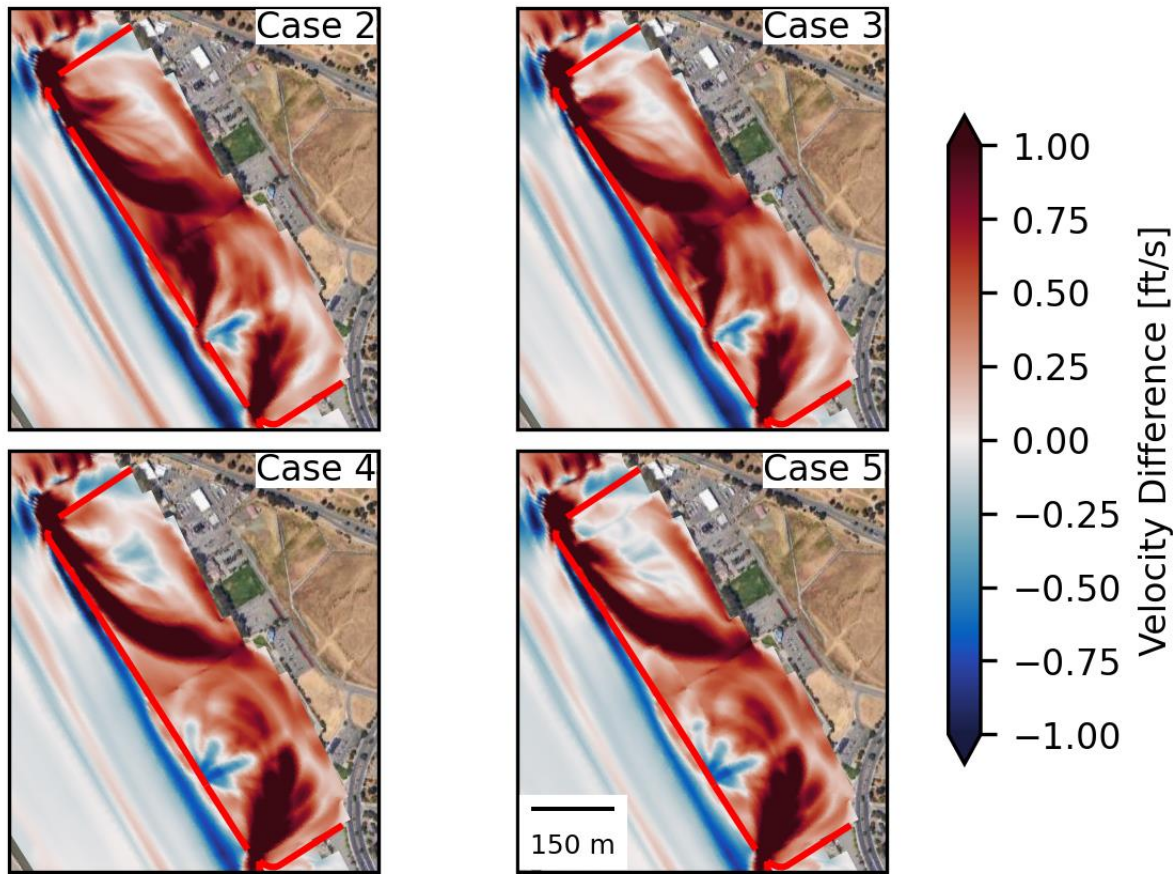


Figure 4-3. Maximum grid-cell velocity difference from Case 1 during the model simulation. Red lines indicate marina walls.

For flood-tide flows (Figure 4-4) the maximum velocity is directing north-eastward through the channel entrances. For Case 1, the flow is restricted from full marina circulation due to the middle marina wall in the south basin and the north wall in the north basin, resulting in clockwise eddies forming due to flow through the entrances. In Case 2, the flow into the marina is restricted in the north entrance; however, the flow from the faster velocities from the south entrance extend through the marina from the south entrances to the north. Flow velocities are slowest in the south of the south basin entrance and in the northeast corner of the marina. In Case 3, is like Case 2 except for a reduced velocity in the sumps. For Case 4, the velocity is reduced, compared to Case 2/3 in the central part of the marina due to a reduced flow through the single south entrance, with a further reduction in the sumps for Case 5.

Compared to Case 1 (Figure 4-5), maximum flood tide flows are primarily increased throughout the marina. The largest increase is obtained with Case 2 and 3 with the two southern entrances allowing greater flow into the marina. There is a decreased increase in the velocities in the central part of the marina due to the reduced flow the single south entrance. In addition, there is

decreased velocity compared to Case 1 near the existing entrances as well as in the northeast and central sections of the north basin. Adding the north sump in Case 5 further reduces the velocity increases in the north basin, near the north entrance.

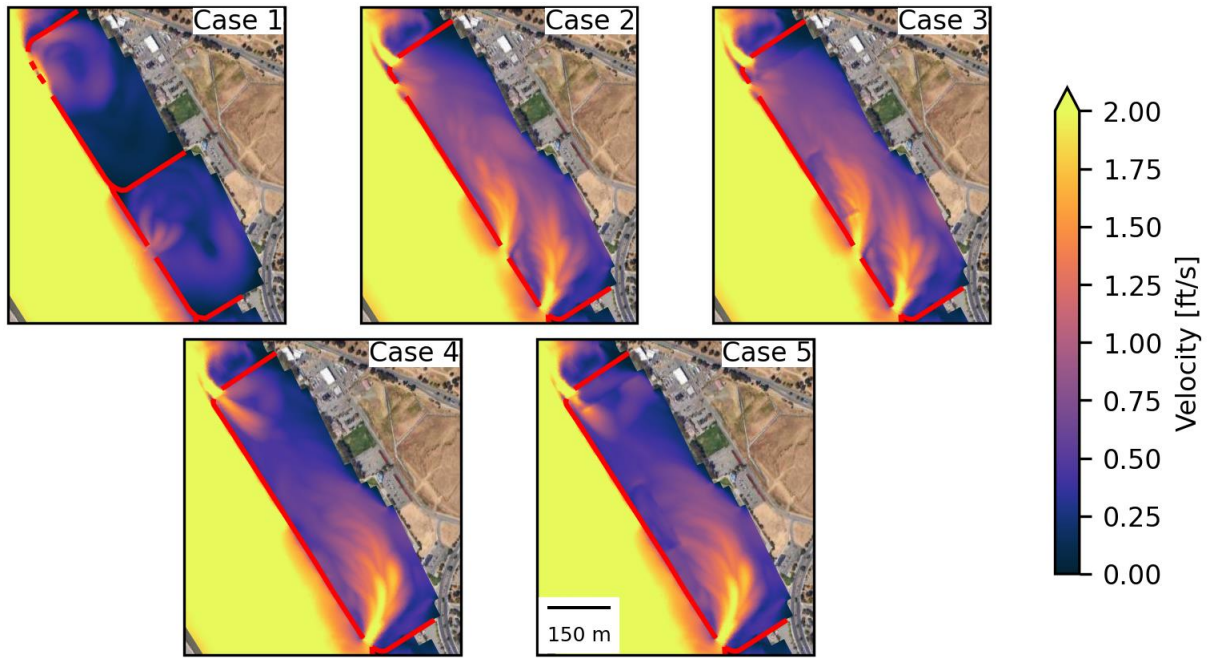


Figure 4-4. Maximum grid-cell velocity during flood-tide. Red lines indicate marina walls.

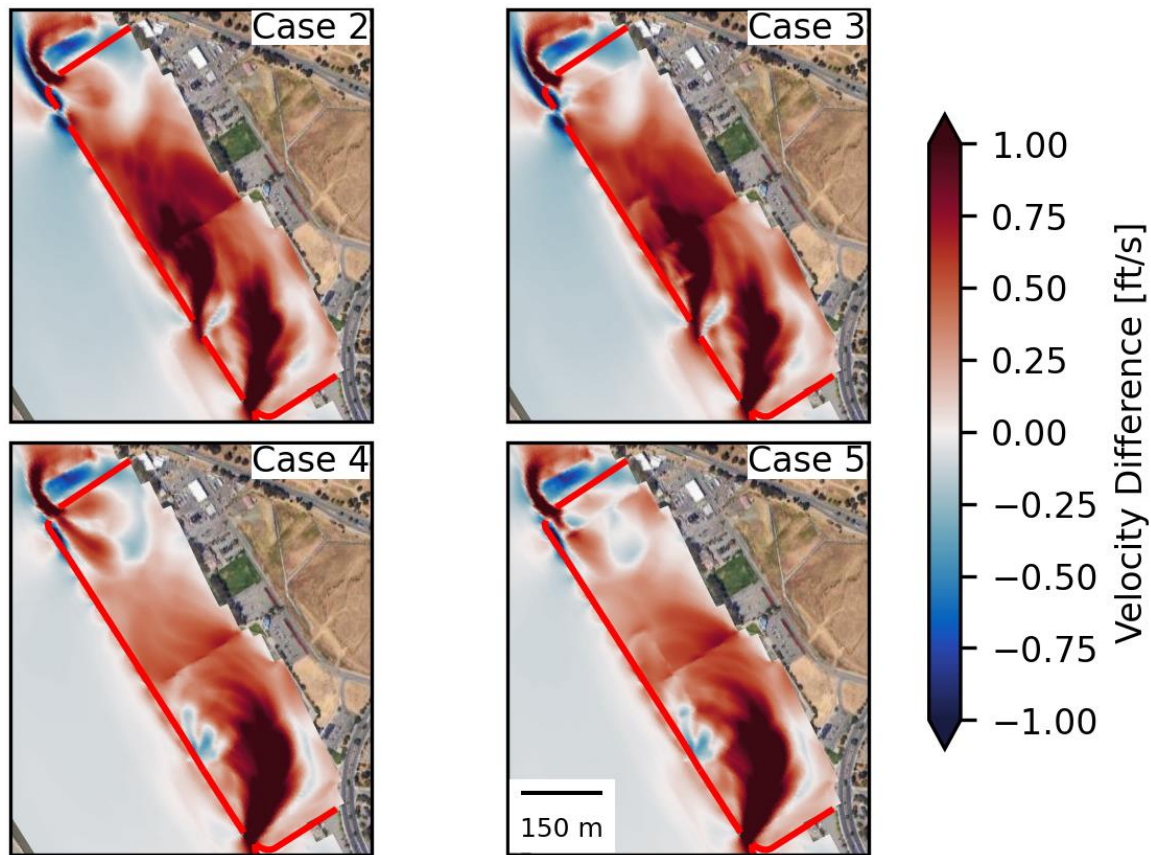


Figure 4-5. Maximum grid-cell velocity difference from Case 1 during flood-tide. Red lines indicate marina walls.

For maximum ebb-tide velocities (Figure 4-6), the flow within the north marina basin for all cases is directed southward through the northern entrance. For Case 1, the velocities are smaller in the marina than the alternative cases 2 through 5. For Case 1, the faster velocities are limited to near the entrances, while for Cases 2 through 5, the faster velocities extend from the north basin through the central part of the basin, with slower velocities in the south basin. For Case 3-5 the velocities are fastest near the channel wall in the north basin with reduced velocities towards the mainland. However, for cases 4 and 5 the single entrance in the south basin results in faster velocities only near the south marina wall with slower velocities near the central basin. The sumps in cases 3 and 5 result in slower velocities within the sumps compared to cases 2 and 4, respectively.

Comparing the cases to Case 1 (Figure 4-7), Cases 2 and 3 result in increased velocities for most of the north and central marina with increases along the south marina wall. There is a reuction in velocity in the north east corner and just sound of the existing southern entrance due to some flow directed through the additional southern entrance. Eliminating the existing entrances in Cases 4 and 5 lead to a reduction in velocity, compared to Case 1, near the existing south

entrance as well as the north and middle sections of the north basin. The larger velocities are primarily directed along the channel wall in the north basin and then the middle and mainland boundary for the southern half of the existing north basin.

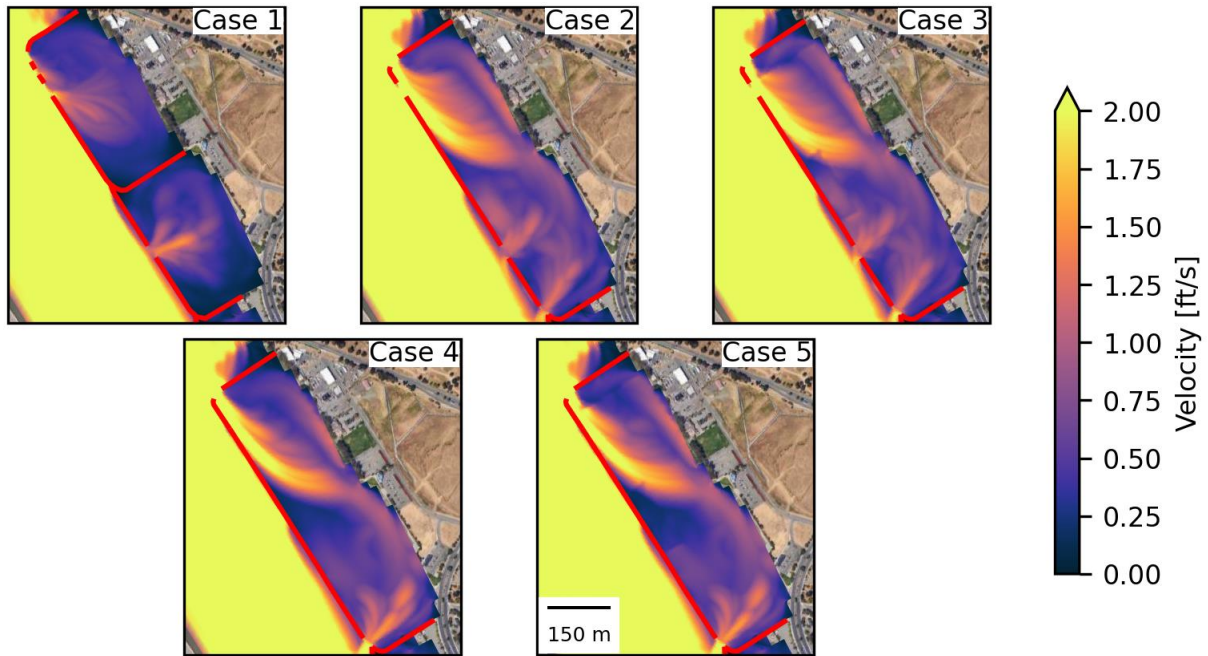


Figure 4-6. Maximum grid-cell velocity during ebb-tide. Red lines indicate marina walls.

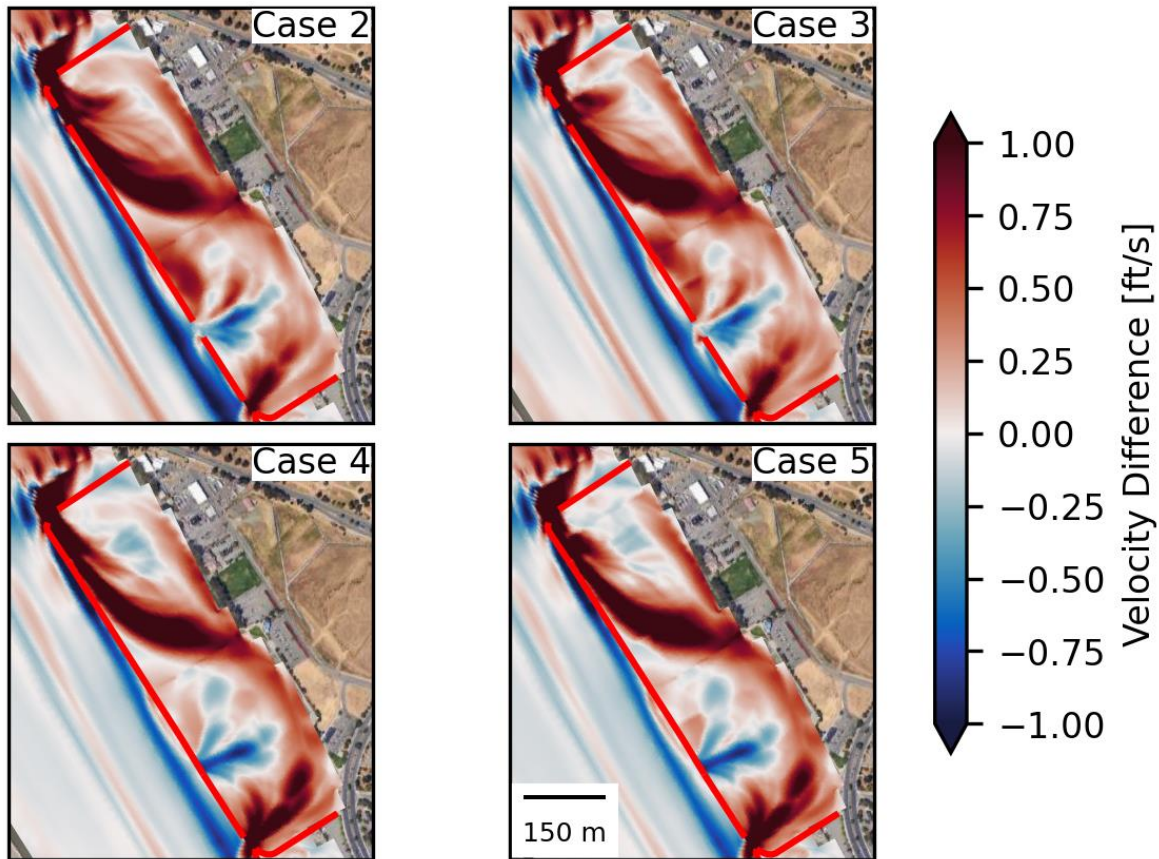


Figure 4-7. Maximum grid-cell velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls.

Mean velocity is calculated as the average velocity for each grid cell of the entire model simulation resulting (Figure 4-8). Average flow patterns are shown as arrows in the figures. Mean velocities are less than 1 ft/s for majority of the marina with the fastest velocities confined to the marina entrances. Mean flow from Case 1 indicate the north and south basins have two eddies forming with circulation extending first to the landward side of the marina and then diverting north and south towards the marina entrances. For Cases 2 through 5, mean flow is primarily directed from the two south marina entrances, towards to the two north entrances due to the stronger flood tide than ebb tide. A counterclockwise eddy forms in the north basin with a weaker clockwise eddy forming in the south basin along the southern marina wall. The 4 entrances for Cases 2 and 3 result in faster flow velocities in the southern and northern marina basins than Cases 4 and 5. Both sumps in Cases 3 and 5 result in slower velocities than the respective no sump cases.

Compared to Case 1 (Figure 4-9), Cases 2-5 indicate an overall increase in mean velocity throughout the marina with the largest increase extending from the southern marina entrances

towards the central section of the marina. There is a slight decrease in velocity between the new and existing north entrance and just outside the existing north entrance. Compared to the average velocities in the marina, there is a decreased velocity within the sumps.

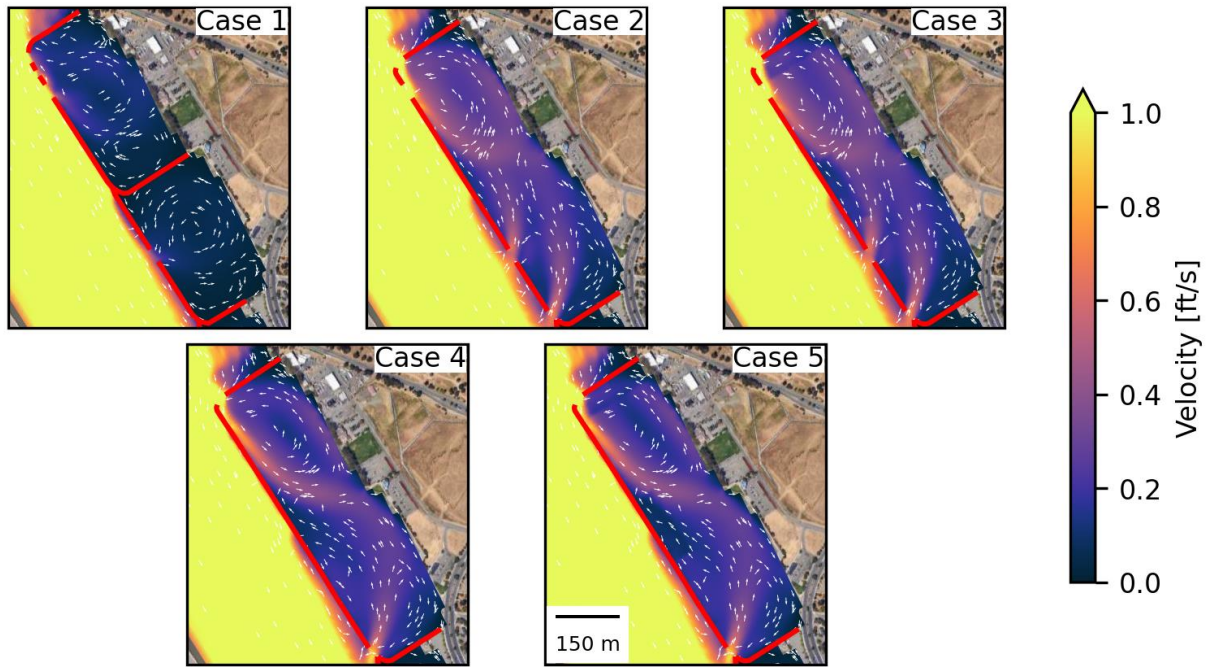


Figure 4-8. Mean velocity during the model simulation. Red lines indicate marina walls.

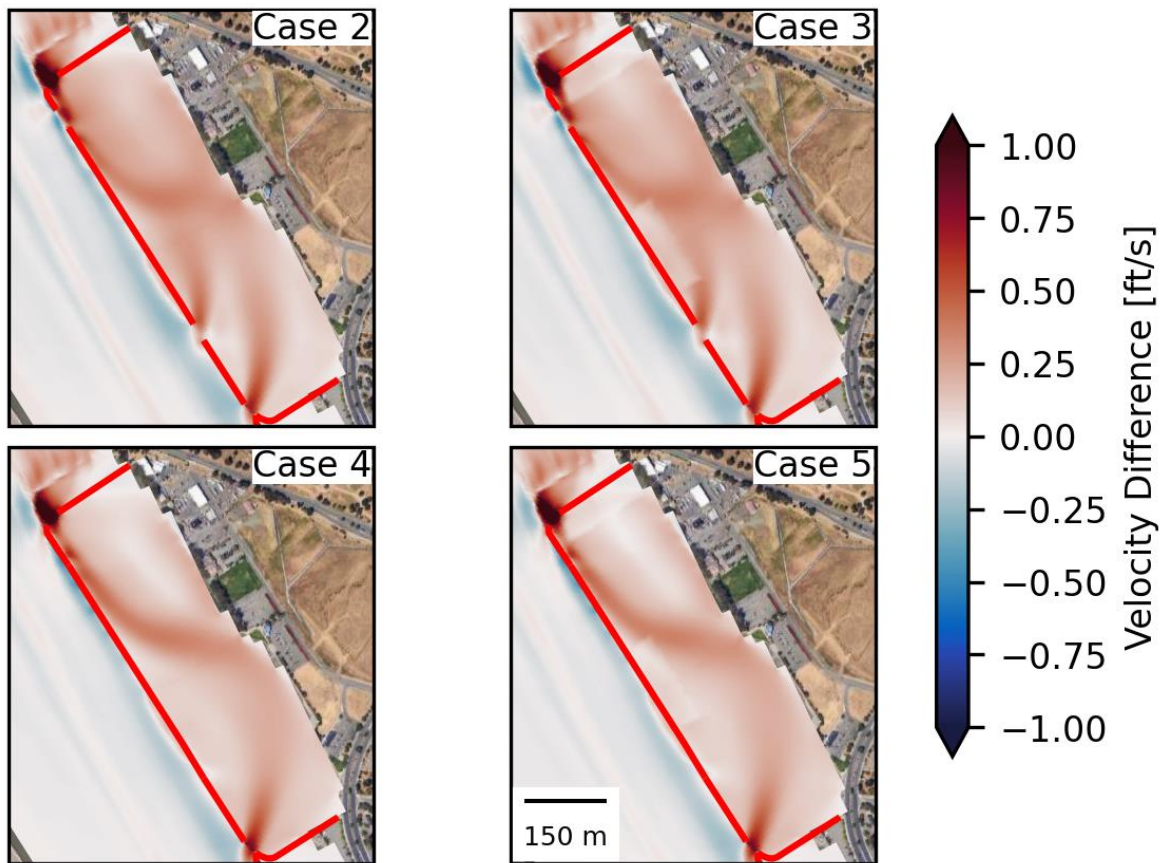


Figure 4-9. Mean velocity difference from Case 1 during the model simulation. Red lines indicate marina walls.

During flood-tide (Figure 4-10), the mean flow is directed from the south marina entrance to the north entrance. Case 1 shows the flow enters the north entrance and produces two clockwise oriented eddies extending through the north basin with flows less than 0.2 ft/s. Multiple eddies form in the south basin. For Cases 2 and 3, a small eddy clockwise eddy form along the south marina wall with majority of the flow extending from the sound marina entrances to the north entrances. For Cases 4 and 5, there is a smaller counterclockwise eddy that from just northwest of the south entrance and another eddy along the south marina wall. The flow pattern is from the sound entrance through the north entrance but with slower flow velocities through the marina than Cases 2 and 3. Flow velocities in the sumps are reduced compared to the no sump cases. Compared to Case 1 (Figure 4-11), Cases 2-5 mean flood-tide flows result in a nearly marina wide increase in flow velocity with a small reduction in the north-east corner of the marina.

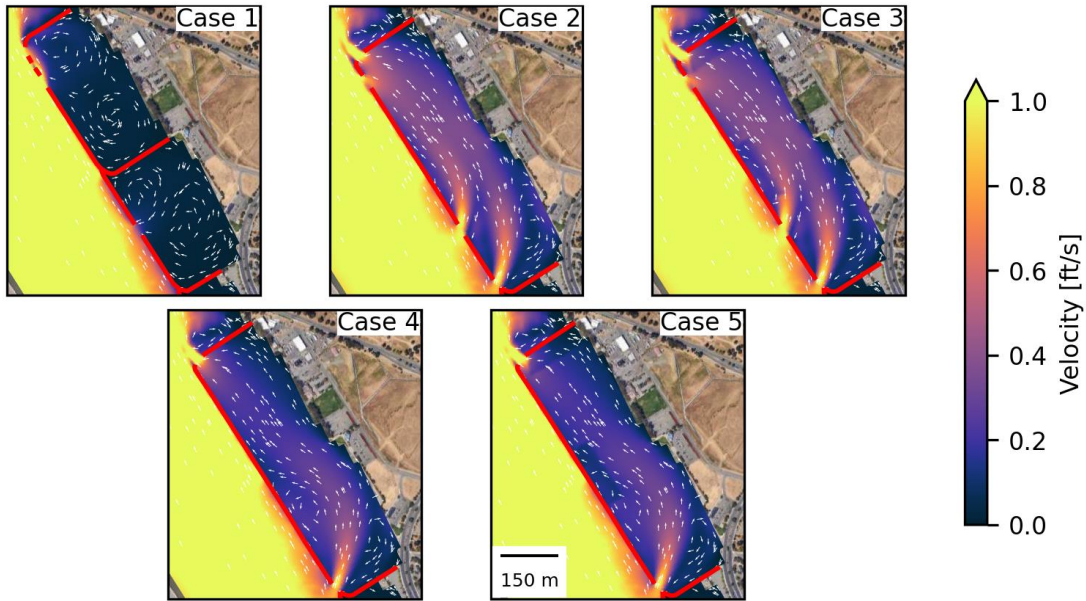


Figure 4-10. Mean velocity during the model simulation during flood-tide. Red lines indicate marina walls.

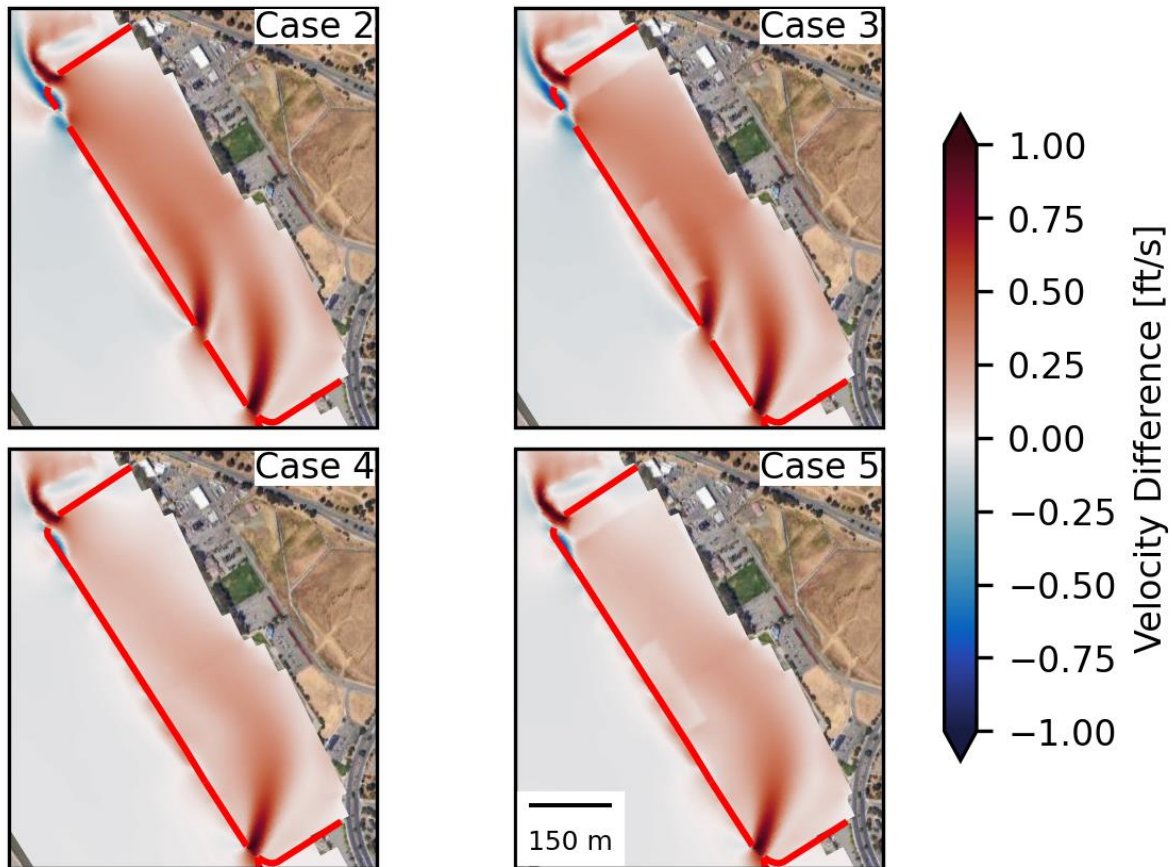


Figure 4-11. Mean velocity difference from Case 1 during flood-tide. Red lines indicate marina walls.

During ebb-flows (Figure 4-12), the mean velocity for Case 1 results in two eddies forming in the north and south basin. In the north basin, the flow through the north entrance is directed south-east towards the landward side of the marina. The fastest flow velocities extend northward and exit the northern entrance while smaller velocities extend southward and exit the northern entrance. The south basin has slower velocities than the north basin. For Cases 2 and 3 the fastest flow velocities enter through the northern entrance extend landward. The flow bifurcates in the central section of the marina, a counterclockwise eddy forming, with flow existing out the north marina entrances while the other fork continues along the landward marina section of the south basin and exits the two south entrances. In the south basin, a clockwise eddy forms northwest of the existing wall. For Cases 4 and 5, the north basin eddy is faster than Cases 2 and 3, with the flow through the south basin directed through the single south entrance. An eddy forms in the south basin along the channel wall and extend further south than Cases 2 and 3. For both Cases 3 and 5, the velocity is slower in the sumps than Cases 2 and 4, respectively. Compared with Case 1 (Figure 4-13), There is an increase in velocity during ebb-tide in the northern half of the north basin and an increase through the south basin and lower

half of the north basin. The change is negligible in the north basin in the center of the eddy and there is a slight decrease in velocity for Cases 4 and 5 near the existing entrances.

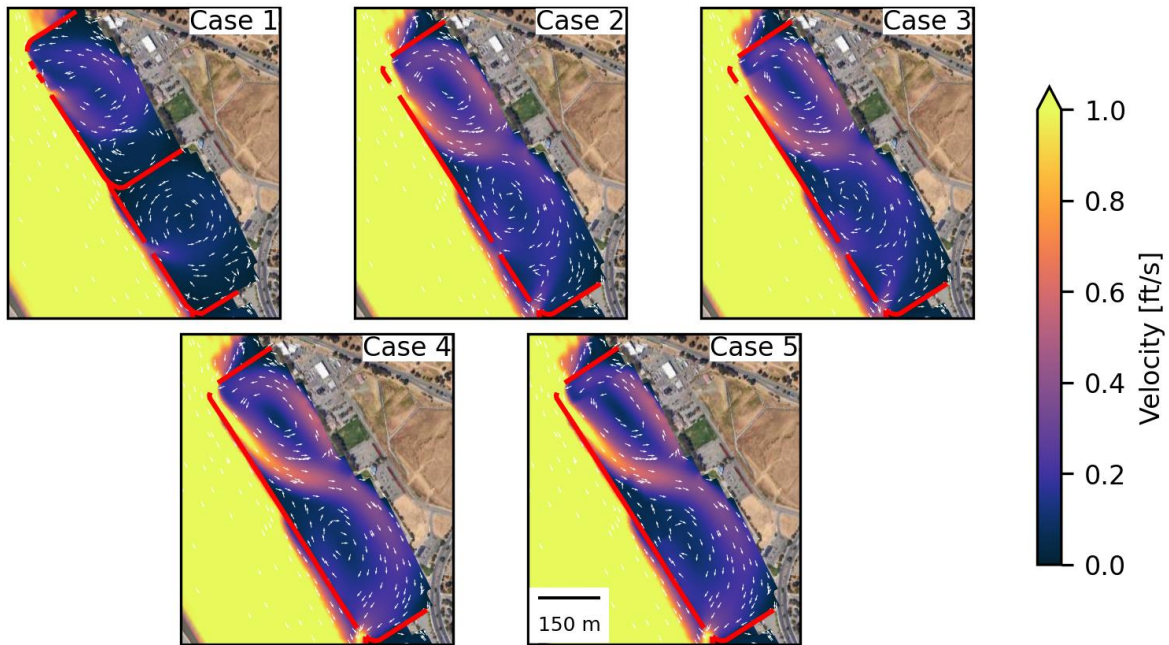


Figure 4-12. Mean velocity during the model simulation during ebb-tide. Red lines indicate marina walls.

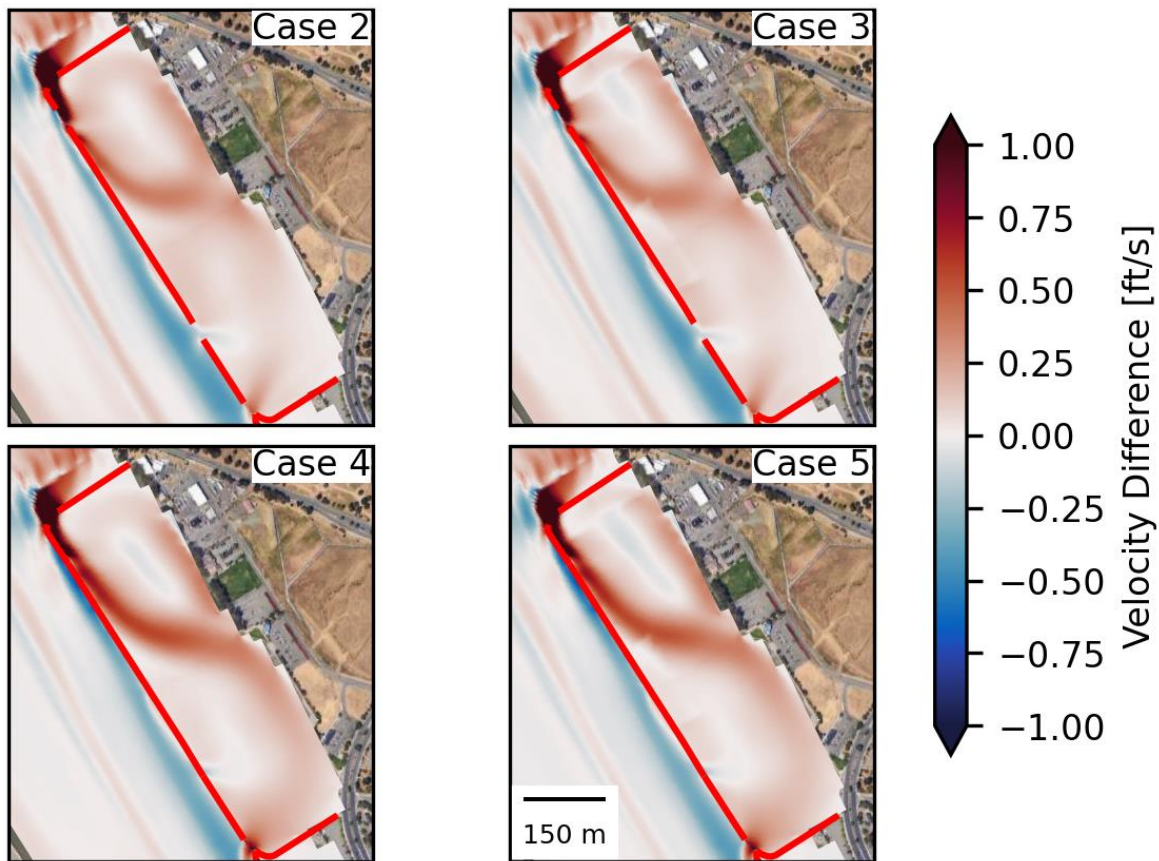


Figure 4-13. Mean velocity difference from Case 1 during ebb-tide. Red lines indicate marina walls.

The maximum defined as the 95<sup>th</sup> percentile of maximum velocities in the north and south basin are shown in (Table 4-2). The maximum velocities for the alternative cases (Case 2 – Case 5) result in faster velocities than Case 1 and over a greater extent of the marina. This table shows that the fastest velocities for ebb tidal flows are greater than 1ft/s for all cases with the largest velocities in the north basin and Case 3. For the South Basin the maximum velocity is for during the flood tidal flows and during Case 3 and Case 4. The increased extent of fastest velocities for the alternatives become apparent when looking at the mean velocities (Table 4-3). All of the mean velocities within the alternatives are at least twice as large as Case 1. Based on model simulations it is unlikely that the velocity will reach the 2kt threshold mentioned by Forth for docked boats for the standard hydrodynamic conditions simulated (Figure 4-14) as 2kt speeds are only likely to occur near the north entrances.

Table 4-2. Maximum (95<sup>th</sup> percentile) north and south basin velocity (ft/s).

Marina Configuration	North Basin Velocity (ft/s)		South Basin Velocity (ft/s)	
	Ebb	Flood	Ebb	Flood
Case 01	1.03	0.90	1.01	0.70
Case 02	2.24	1.13	1.10	1.68
Case 03	2.27	1.13	1.09	1.71
Case 04	2.22	1.02	1.18	1.73
Case 05	2.17	0.86	1.16	1.73

Table 4-3. Mean north and south basin velocity (ft/s).

Marina Configuration	North Basin Velocity (ft/s)		South Basin Velocity (ft/s)	
	Ebb	Flood	Ebb	Flood
Case 01	0.14	0.07	0.07	0.03
Case 02	0.33	0.31	0.15	0.28
Case 03	0.32	0.31	0.14	0.29
Case 04	0.35	0.18	0.19	0.23
Case 05	0.34	0.18	0.19	0.23

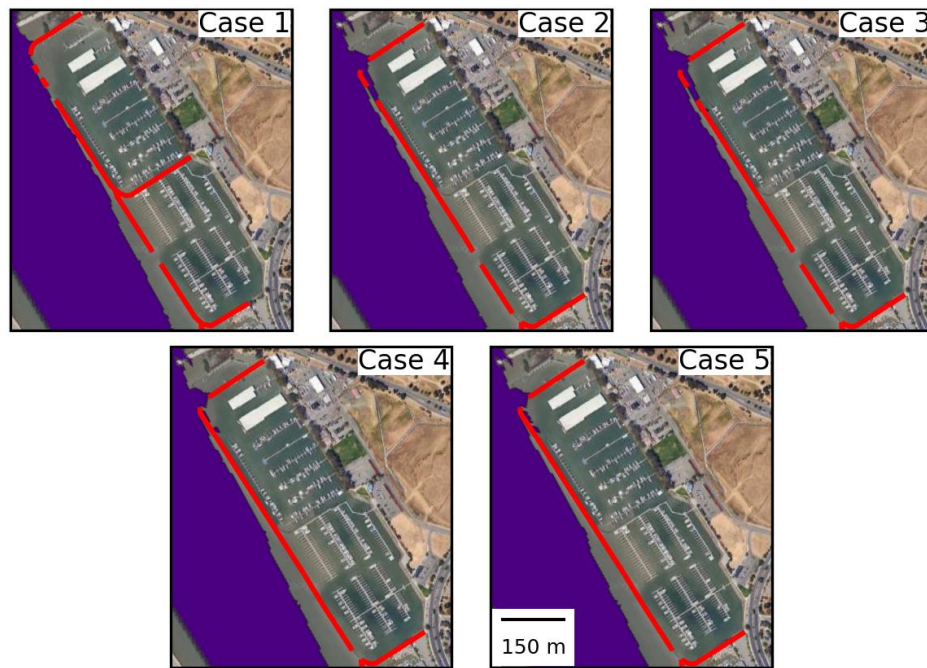


Figure 4-14. Map showing regions where the velocities exceed 2 kt shown in indigo (indigo) for maximum velocities. Red lines indicate marina walls.

### 4.3 SHEAR STRESS

The amount of deposition or erosion experienced within the marina is inversely proportional to the bottom shear stress ( $\tau$ ). An increase in shear stress will result in a decrease in deposition or increase in erosion, whereas a decrease in  $\tau$  will result in a decreased rate of erosion or increased rate of deposition. To determine the deposition rate expected to occur in the marina and around each alternative marina configuration, the mean  $\tau$  of the time series and the maximum  $\tau$  of the time series for flood, ebb, and full model simulation were calculated. Shear stress is directly proportional to velocity and as such increased shear stress is related to the increased velocities discussed in section 4.2.

The maximum shear stress in the marina during the simulation is shown in Figure 4-15. For Case 1, the largest shear stresses are near the north marina entrance and the south marina entrance. The larger shear stresses extend further into the northern half of the south basin than the north basin. The southern half of both the north and south basin have the smallest shear stress. For Case 2 and 3, the shear stress extends from the south marina entrance northward along the landward side of the marina and towards the center of the marina. The south end of the marina and the sump have lower shear stress. In the northern basin, the largest shear stresses are directed southward along the channel bounding marina wall towards the center of

the marina and extends landward. For Cases 4 and 5 exhibit similar shear stress, however the northern basin higher shear stress is less distributed in the center of the basin and the single south entrance results in lower shear stress in the central basin along the channel wall. Both Case 3 and 5 has lower shear stress within the sumps. Compared to Case 1 (Figure 4-16), there is overall increased shear stress within the marina for Case 2 through 5. There is a decrease in shear stress near the existing south entrance for Case 2 and 3, and a larger decrease when the entrance is removed for Case 4 and 5. The northeastern corner of the north basin indicates negligible change.

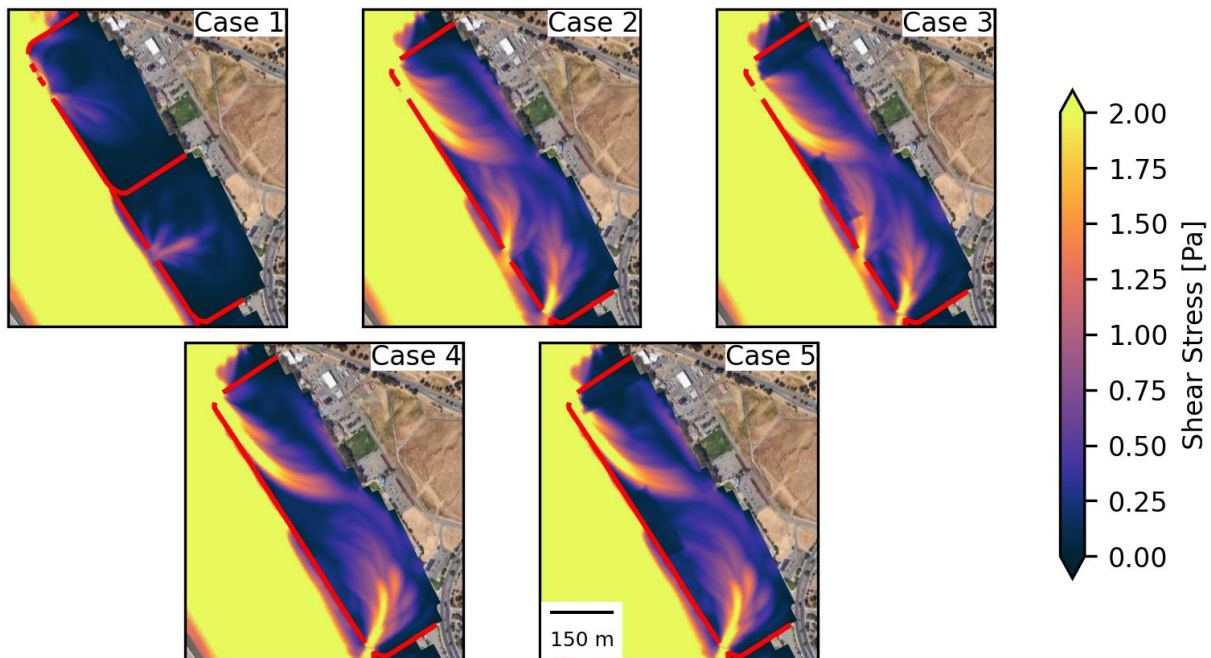


Figure 4-15. Maximum grid-cell shear stress during the model simulation. Red lines indicate marina walls.

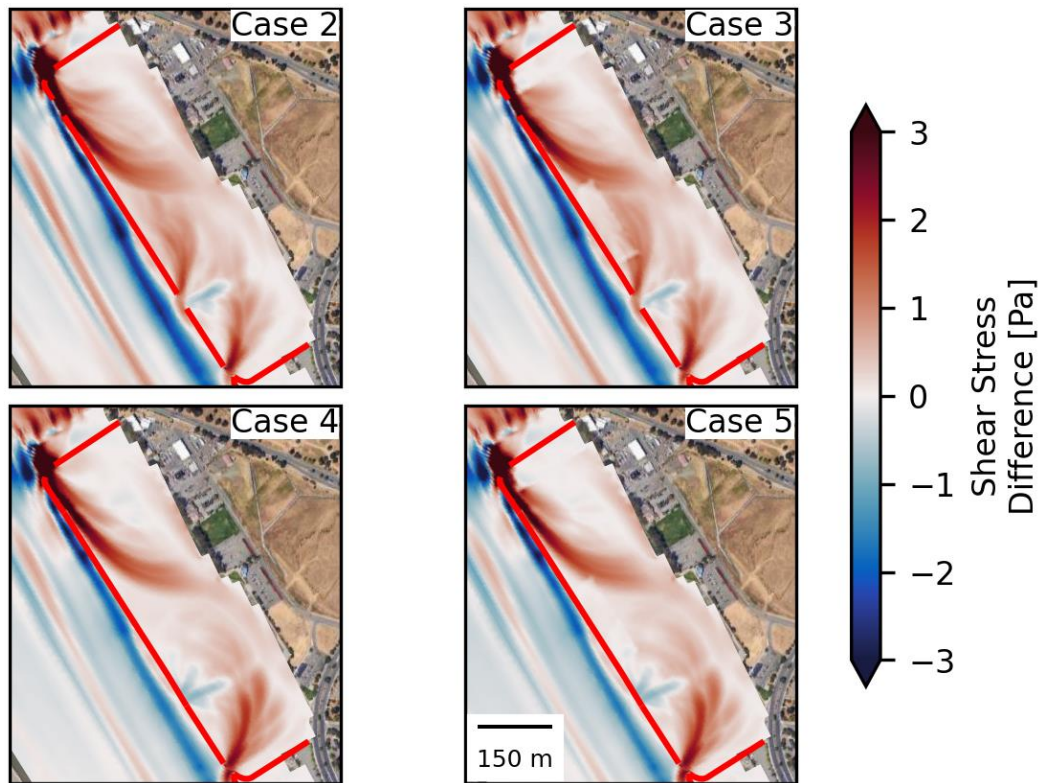


Figure 4-16. Maximum grid-cell shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls.

The maximum shear stress during flood-tide (Figure 4-17) indicates increased shear stress north of the entrances for Cases 1 through 5. For Case 1, an eddy forms in the northern half of the north basin, while the shear stress is more northward directed from the south entrances through the south basin and exiting the north basin for Case 2 through 5. During flood-tide the greatest change from Case 1 (Figure 4-18) is an increase in shear stress in the south basin (Case 2 and Case 3) and center of the marina (Case 4 and Case 5). Decreases occur near the existing entrances, with negligible changes in the north eastern section of the north basin for Case 2 and Case 3 and along the landward side of the north basin for Case 4 and Case 5.

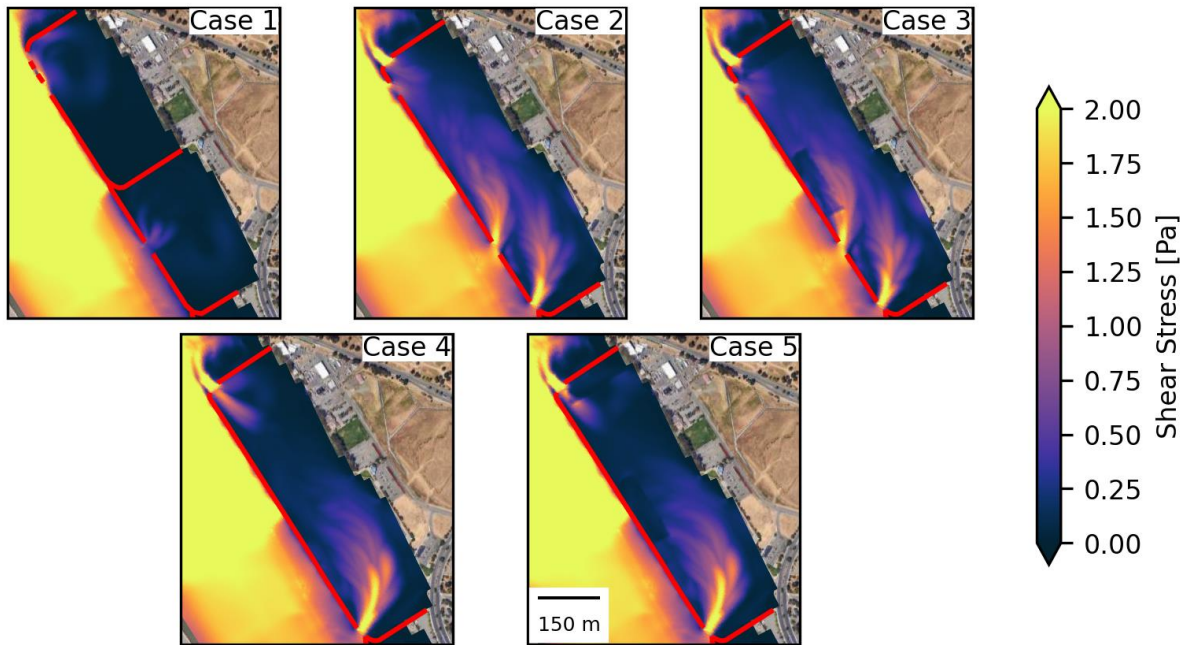


Figure 4-17. Maximum grid-cell shear stress during flood-tide. Red lines indicate marina walls.

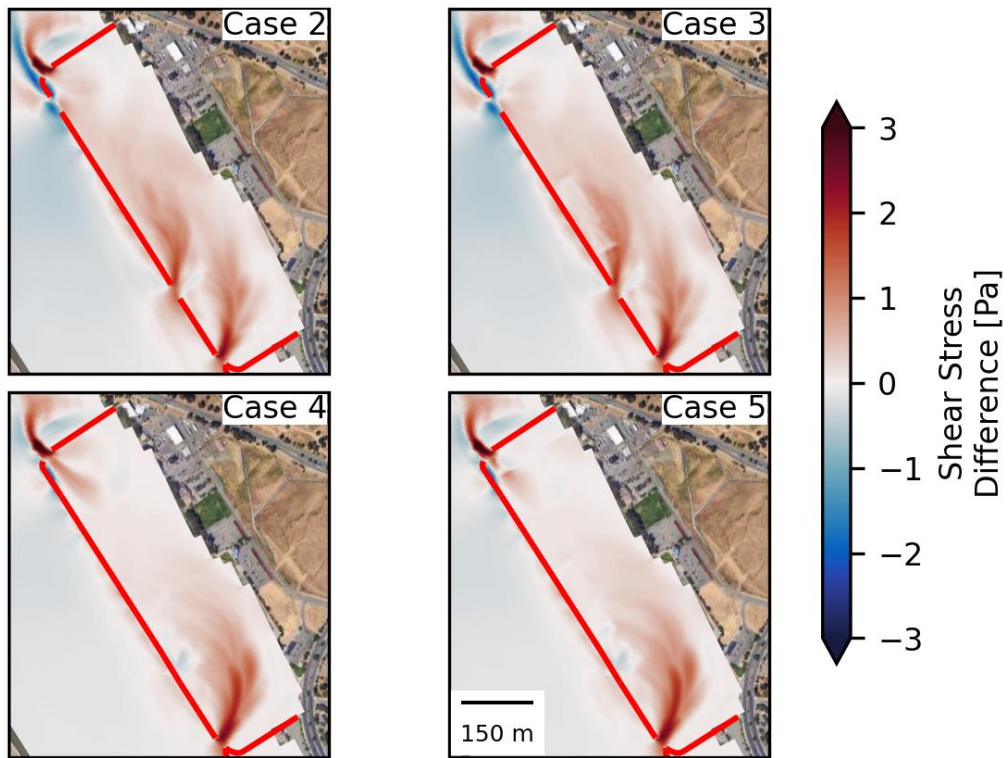


Figure 4-18. Maximum grid-cell shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls.

For maximum shear stress during ebb-flows (Figure 4-19), the largest shear stress for Case 1 is confined to the marina entrances. For Cases 2 through 5, the greatest shear stress is a result of flow through the north entrances, with the larger shear stresses extending along the channel wall of the north basin and extending towards the center of the marina. There is reduced shear stress in the south basin compared to the north basin with the larger shear stress near the entrances and extending landward. For Case 4 through Case 5 there is reduced shear north of the south entrance in the south basin with the larger shear stress extending along the south basin wall from the south entrance. Compared to Case 1 (Figure 4-20), Cases 2 through 5 result in an overall increase in shear stress for the north basin with a noticeable increase in the south basin along the south marina wall and the landward side of the marina. A decrease is observed south of the existing south entrance in Case 2 and Case 3 and over a larger area when this entrance is removed in Case 4 and 5. The norther section of the north basin has limited change in Case 2 and 3 with the change further limited in the central part of the north basin for Case 4 and Case 5.

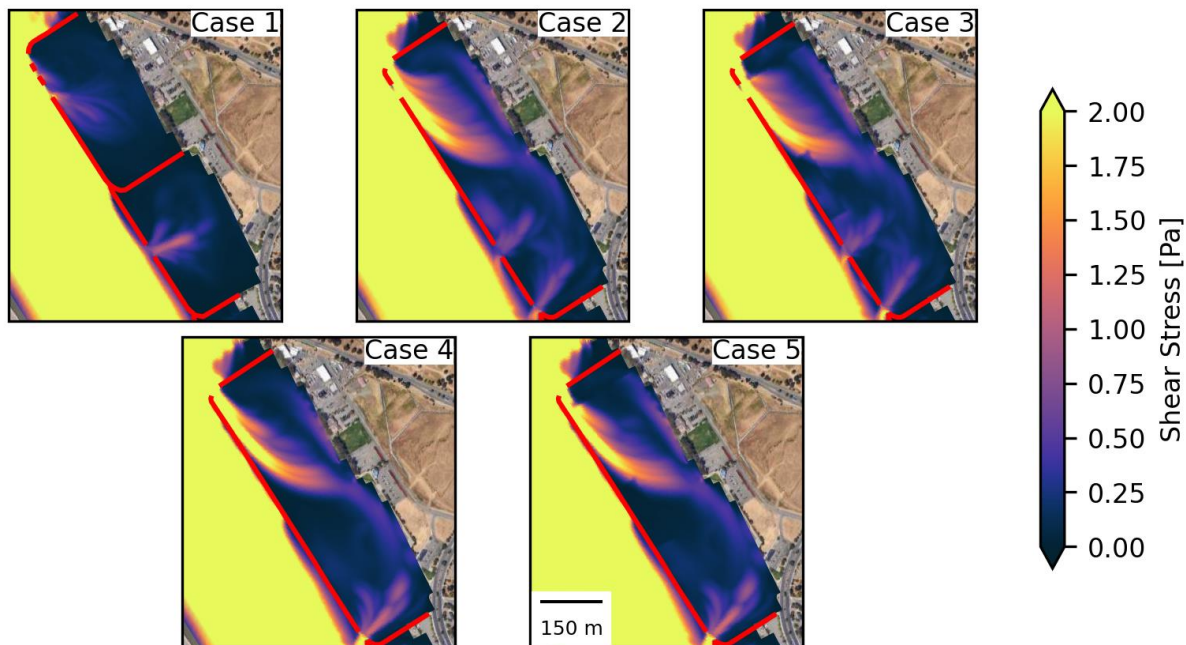


Figure 4-19. Maximum grid-cell shear stress during ebb-tide. Red lines indicate marina walls.

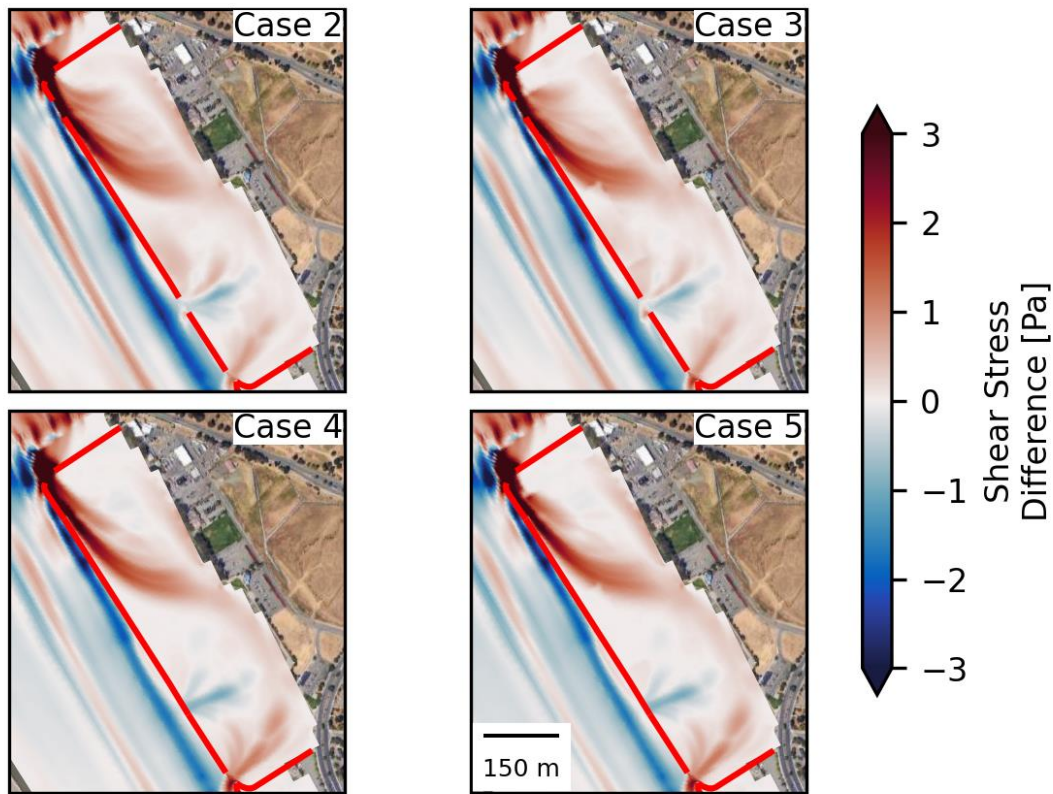


Figure 4-20. Maximum grid-cell shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls.

Mean shear stresses are shown in (Figure 4-21). For Case 1, there is generally low shear stress, mostly limited to the north marina entrances. For Case 2 through Case 5, the prevalent regions of higher shear stress extend from the south marina entrance towards the north marina entrance, and along the north marina entrances and canal wall towards the central section of the marina. The two south entrances and north entrances in Case 2 and 3 result in increased shear stresses in the marina, while the single entrances in Case 4 and 5 result in smaller overall shear stress in the south and central basin and the center of the north basin. Differences from Case 1 (Figure 4-22), indicate an overall increase in shear stress with negligible changes in the southwest corner of the south basin for Case 2 and Case 3, and in the south-central section of the basin and northwest corner of the north basin and middle of the north basin for Case 4 and Case 5.

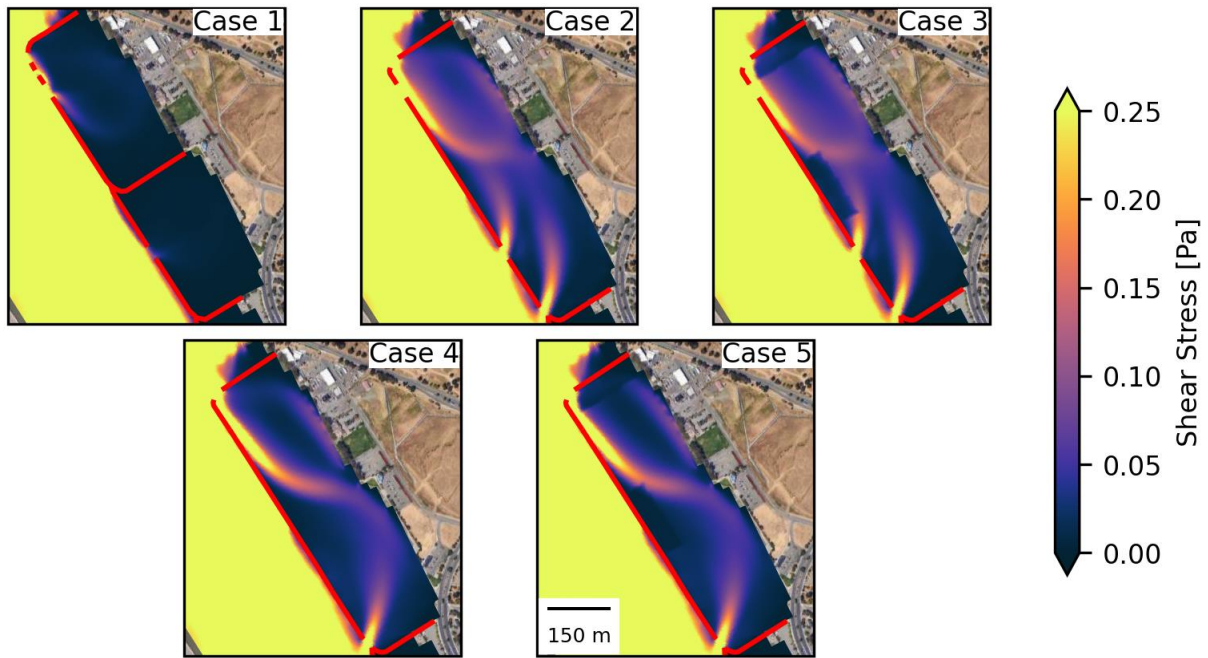


Figure 4-21. Mean shear stress during the model simulation. Red lines indicate marina walls.

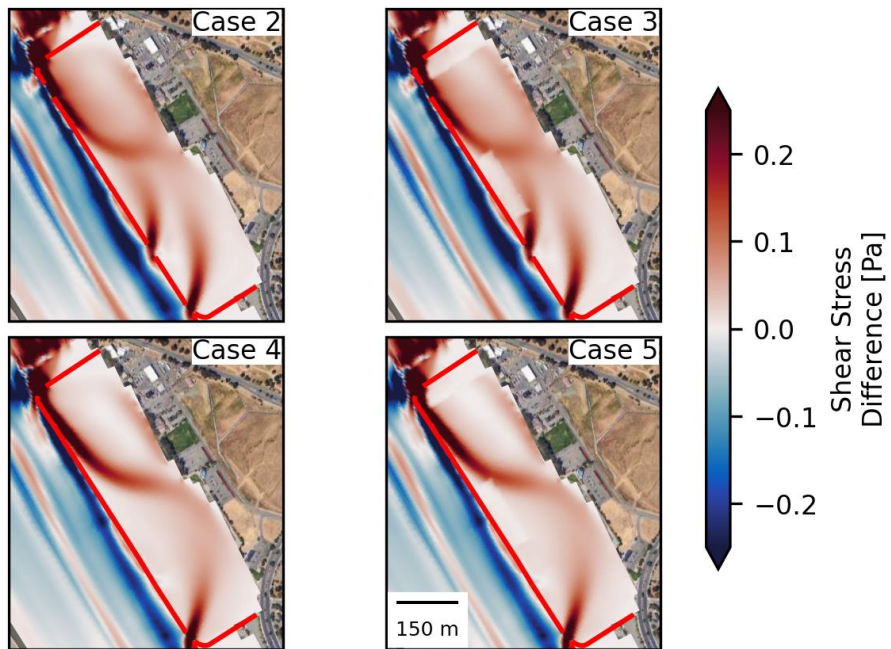


Figure 4-22. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls.

During mean flood-tidal flows (Figure 4-23), Case 1 exhibits small shear stress within both basins with higher shear stress limited to the norther entrances. Removing the middle wall and adding additional entrances in Case 2 and Case 3, results in increase shear stress extending from the south basin though the middle of the marina and towards the north entrances. For Case 4 and Case 5, the shear stress is less than Case 2 and Case 3 with the greatest shear stress near the south marina entrance. The shear stress is reduced in Case 3 and Case 5 in the location of the sumps. Compared to Case 1 (Figure 4-24), there is an overall increase in shear stresses in majority of the marina except for the northwestern corner of the north basin for Case 2 and the southwest corner of the south basin. The largest increase in shear stress is for Case 3 due to the influence of the south entrances with a larger increase with the presence of two entrances in the north and south basin.

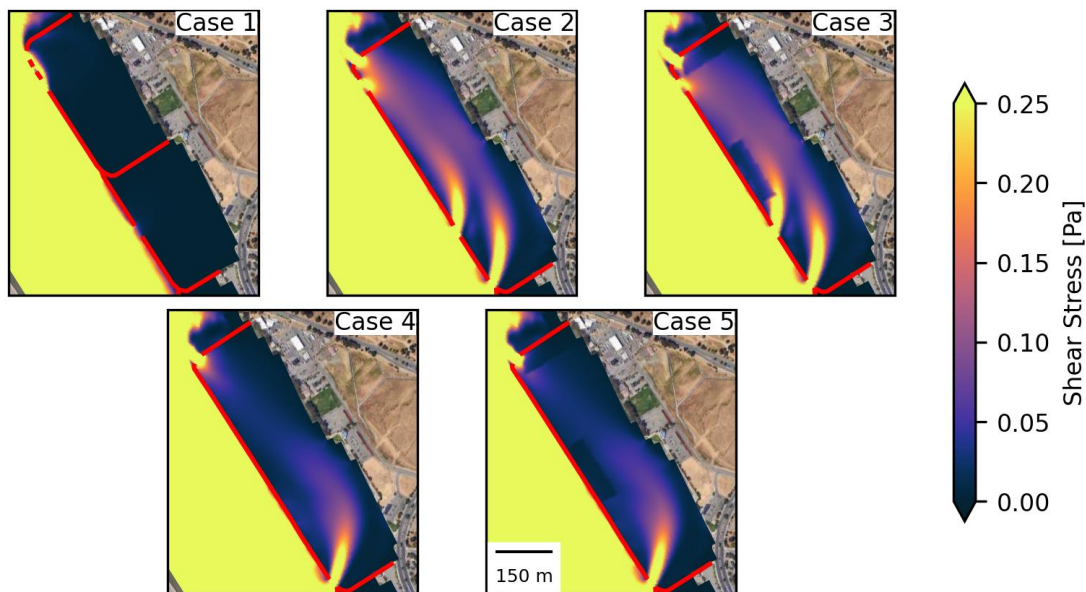


Figure 4-23. Mean shear stress during flood-tide. Red lines indicate marina walls

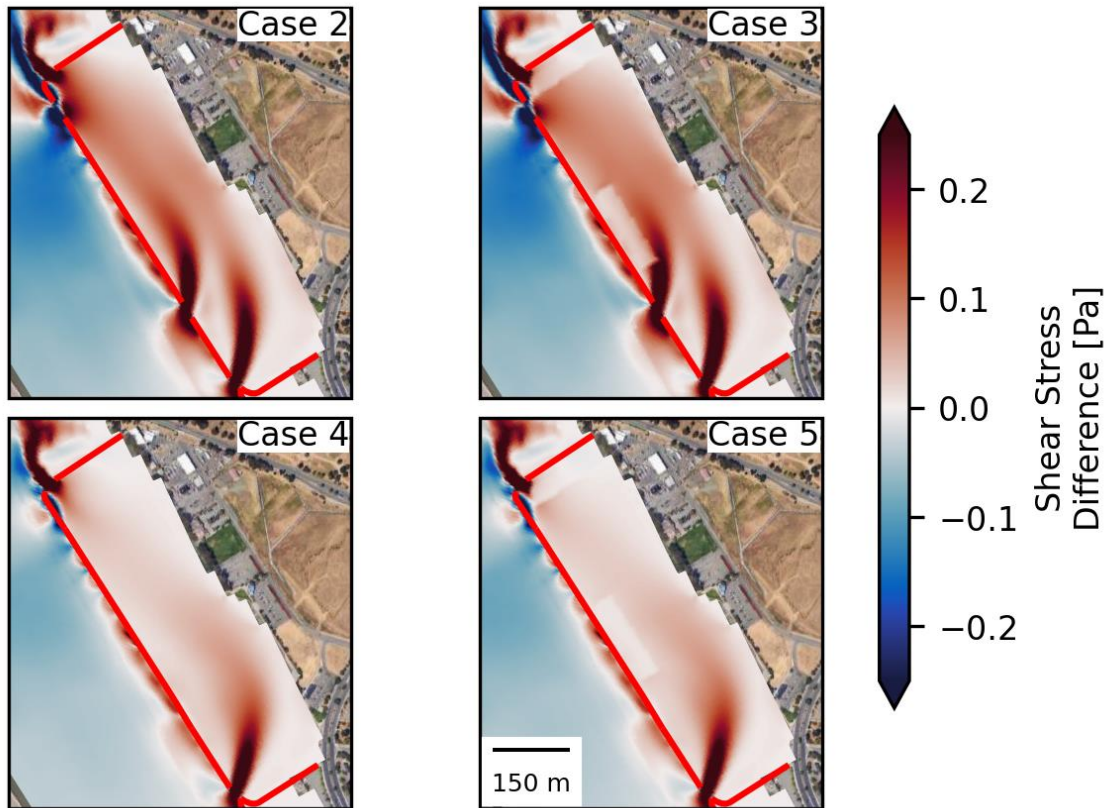


Figure 4-24. Mean shear stress difference from Case 1 during flood-tide. Red lines indicate marina walls.

During mean ebb-tidal flows (Figure 4-25), the maximum shear stress southward into the north basin for Case 1. For Case 2 and 3, the highest shear stress is in the north basin with highest stresses remaining near the land boundary in the upper south basin before extending towards the sough entrances. The lowest shear stresses are near the channel wall in the central basin and the southeast corner of the marina. For Cases 4 and 5 there is increased shear stress is larger in the north basin, compared to Case 2 and 3. Compared to Case 1 (Figure 4-26), the mean shear stress increases in the north basin and the landward boundary of the central marina. There is negligible change in shear stress in the northern central part of the north basin and extending along the channel side of the south and central basin.

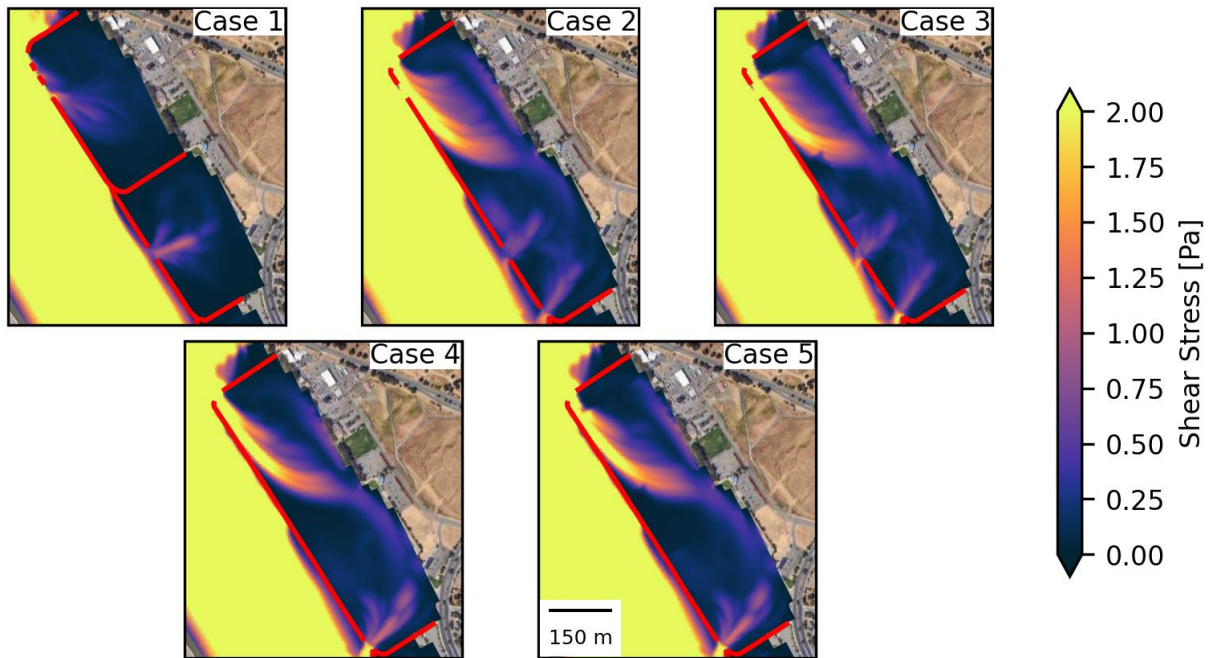


Figure 4-25. Mean shear stress during ebb-tide. Red lines indicate marina walls

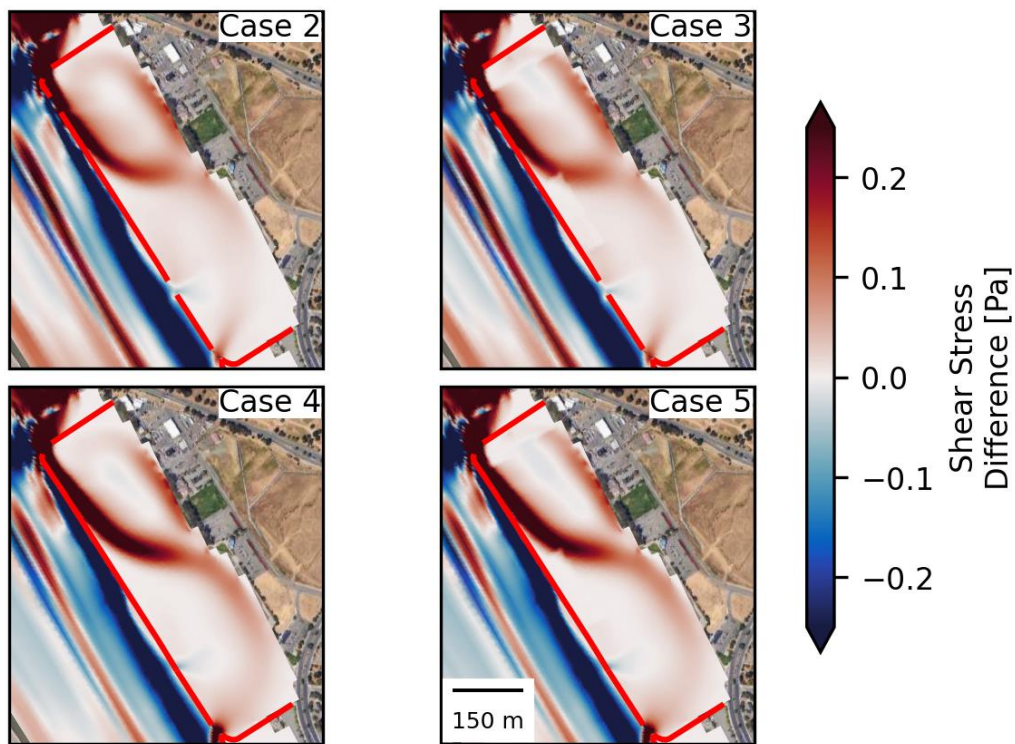


Figure 4-26. Mean shear stress difference from Case 1 during ebb-tide. Red lines indicate marina walls.

The maximum defined as the 95<sup>th</sup> percentile of maximum shear stress in the north and south basin are shown in (Table 4-4). The table shows the shear stress is variable in the North Basin between the different Cases with the largest ebb-tide shear stress in Case 3 and smallest shear stress in Case 1. The alternative cases are all several times greater than Case 1. Flood-tide shear stress is smaller for the north basin in Cases 1, 5, and largest for Case 2. The north basin has larger shear stresses overall. The mean shear stress (Table 4-5), shows that the shear stress in for the alternatives is consistently larger than Case 1, indicating an overall marina wide increase in shear stress.

Table 4-4. Maximum (95<sup>th</sup> percentile) shear stress (Pa)

Marina Configuration	North Basin		South Basin	
	Ebb	Flood	Ebb	Flood
Case 01	0.51	0.38	0.50	0.23
Case 02	2.47	0.64	0.59	1.37
Case 03	2.56	0.61	0.58	1.39
Case 04	2.44	0.56	0.67	1.41
Case 05	2.31	0.38	0.64	1.40

Table 4-5. Mean shear stress (Pa)

Marina Configuration	North Basin		South Basin	
	Ebb	Flood	Ebb	Flood
Case 01	0.15	0.10	0.12	0.08
Case 02	0.76	0.29	0.24	0.47
Case 03	0.76	0.27	0.23	0.47
Case 04	0.63	0.17	0.25	0.44
Case 05	0.63	0.15	0.24	0.45

#### 4.4 PROBABILITY OF DEPOSITION

The likelihood of sediment deposition for each case was calculated using a probability of deposition based on excess shear stress:

$$P_a = \left(1 - \frac{\tau}{\tau_{cr}}\right) \quad \text{Equation 1}$$

Where  $P$  is the probability of deposition and the subscript 'a' indicates the alternative marina configurations,  $\tau$  is the shear stress, and  $\tau_{cr}$  is the critical shear stress assumed to be 0.1 Pa based

on typical young bay sediment characteristics in the channel. The deposition rate ( $D$ ) for these cases is then related to the current deposition rate (Case 1) as:

$$D_a = D_e \left( \frac{P_a}{P_e} \right) \quad \text{Equation 2}$$

where the subscript “e” indicates existing marina configuration. The mean sediment deposition rate for the north and south basin was used for  $D_e$ .

#### 4.4.1 Sediment Deposition

The mean and mean flood-tide deposition rates ( $D_a$ ) are shown in Figure 4-27 and Figure 4-28, respectively. Case 1 shows the mean deposition rate used for  $D_e$  based on the sedimentation analysis in section 3. The representative deposition rate is assumed to be accounted for during the mean flow as maximum flows are limited in temporal occurrence. In addition, since flow coming from San Francisco Bay (flood-tide) will bring in more sediment the deposition rate for flood-tide conditions is also evaluated. For mean flow (Figure 4-27), erosion is predicted due to the flows entering the south marina entrance and limited to area around the north entrance. For Case 2 and 3, there is deposition extending from the north and south entrance towards the center of the marina with deposition outside of the main flow paths. Case 3 indicates deposition in the central sump where there was erosion in Case 2 and there is increased deposition in the north sump. Case 4 and 5 indicate slightly larger erosion in the north basin along the flow path from the north entrance, however there is decreased erosion in the south basin. The sumps in Case 5 indicates increased deposition compared to the surrounding area.

Compared to Case 1 (Figure 4-28), the mean deposition rate decreases or remains the same for the north and south basins, with a large decrease in deposition rate along the flow path between the north and south marina entrances. There is an increase in deposition rate along the northeast corner of the marina as well as in a portion of the north sump for Case 5.

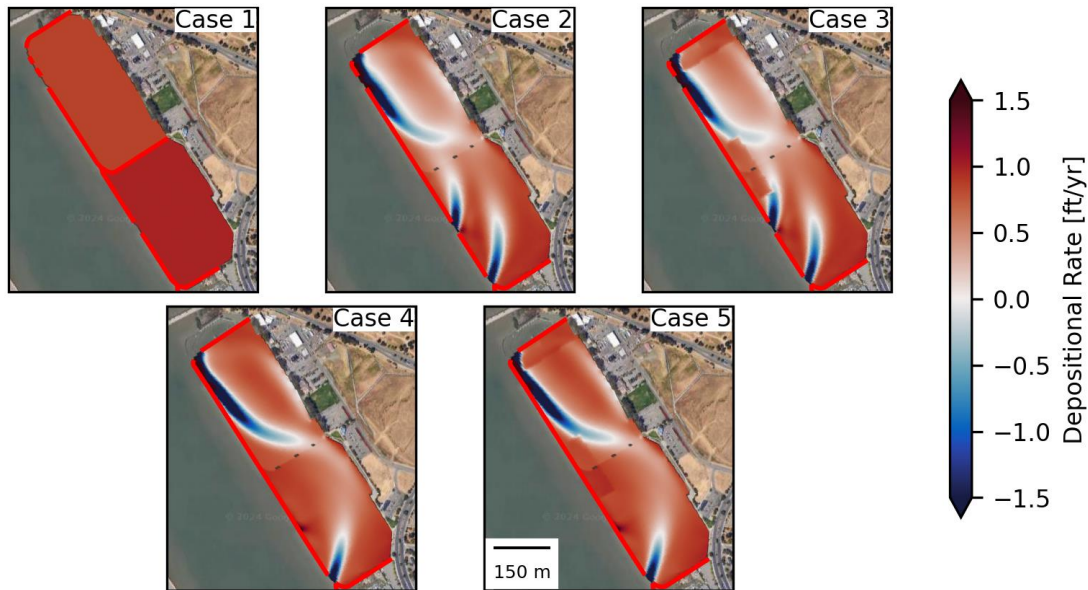


Figure 4-27. Mean sediment deposition rate during the model simulation. Red lines indicate marina walls.

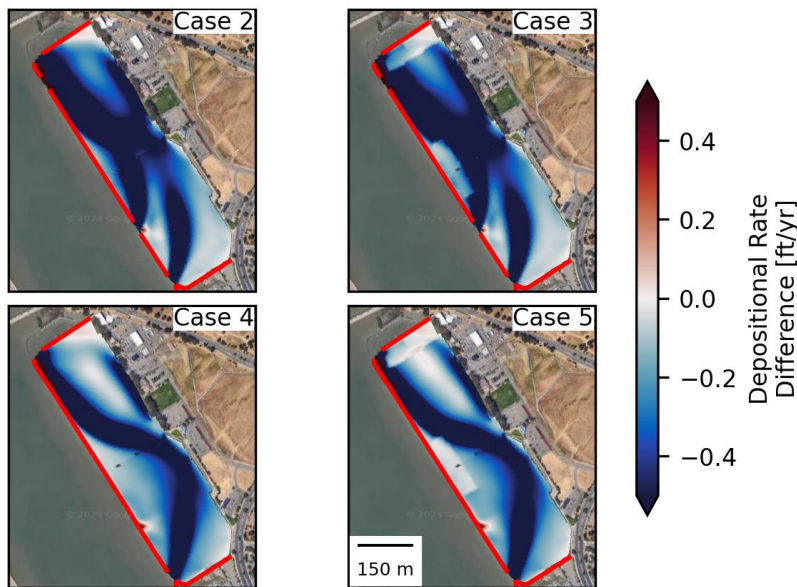


Figure 4-28. Mean shear stress difference from Case 1 during the model simulation. Red lines indicate marina walls.

For flood-tide (Figure 4-29), the mean sediment deposition rate shows a similar erosion pattern to the mean rate with the exception of increase deposition within the northern basin due to limited flow through the north entrances during flood-tide. Compared to Case 1 (Figure 4-30), there an overall decrease in deposition rate within the basin with the exception of the northeast

corner of the north basin. There is deposition with the north sump for Case 3. There is less of a decrease in deposition rate in the north basin for Case 4 and Case 5 compared to Case 2 and Case 3.

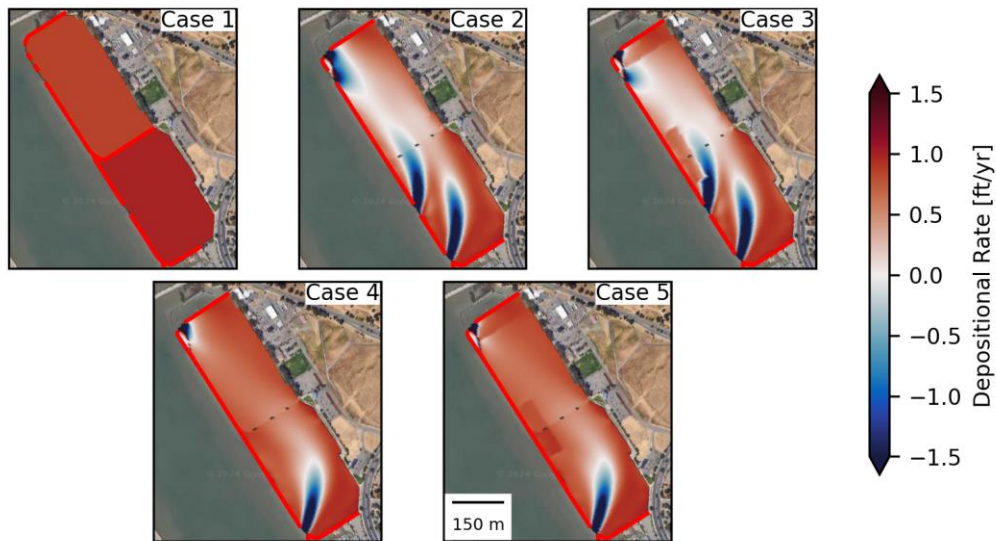


Figure 4-29. Mean sediment deposition rate during flood-tide. Red lines indicate marina walls.

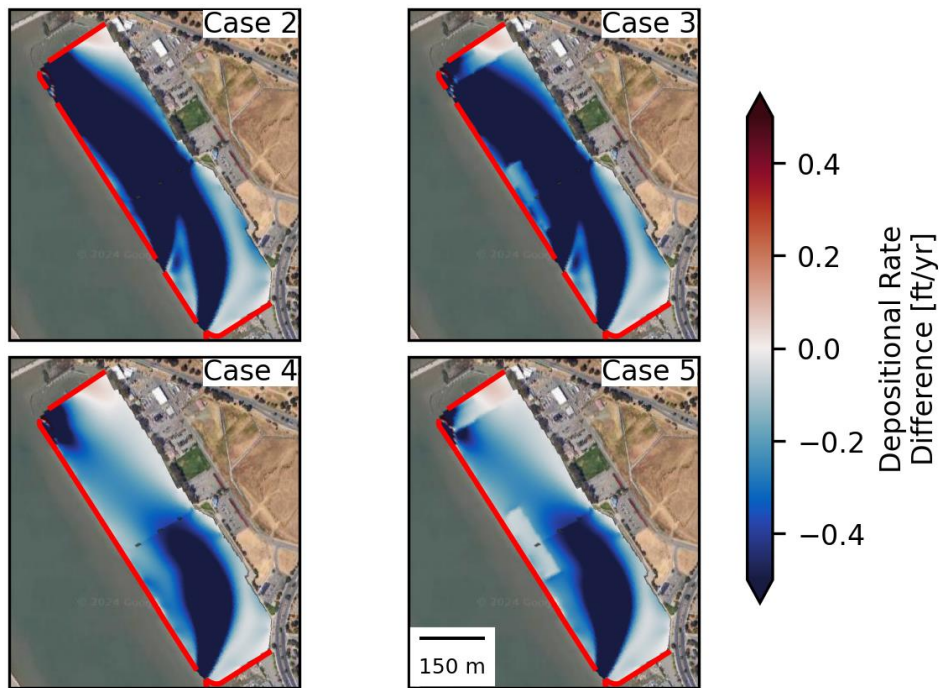


Figure 4-30. Mean sediment deposition rate difference from Case 1 during flood-tide. Red lines indicate marina walls.

## 4.5 DEPOSITION RATES

The mean deposition rate within the marina were calculated for mean (Table 4-6) and mean flood-tide (Table 4-7). For mean flows, the overall marina deposition rate is reduced for all Cases compared to Case 1 with Case 2 having the overall erosion in the north basin a deposition rate of 0.55 ft/yr deposition rate with Case 3 being similar. However, when excluding the sumps and only evaluating the remaining area, Case 5 has a smaller marian wide deposition rate of 0.24. Deposition rate in the sumps is larger than the overall marina average for Case 3 and Case 5, indicating these might reduce the sediment deposited in other areas of the marina. Mean deposition rate during flood-tide is shown in (Table 4-7). Case 2 and Case 3 exhibit smaller for the marina wide, as well as the north and south basin, deposition rates than Case 1, Case 4 and Case 5. This also indicates that the deposition rate is greater in the sumps than the basin. Case 2 results in the largest decrease in deposition rate overall and in the North basin, while Case 3 has the smallest deposition in the South basin and the overall area excluding sumps.

Table 4-6. Mean sediment deposition rate (ft/yr)

Case	Marina	North Basin	South Basin	North Sump	Middle Sump	Excluding Sump Area
Case 01	0.92	0.85	0.99	--	--	--
Case 02	0.28	-0.03	0.55	--	--	--
Case 03	0.30	0.02	0.55	0.72	0.70	0.27
Case 04	0.27	-0.15	0.63	--	--	--
Case 05	0.29	-0.11	0.63	0.79	0.85	0.24

Table 4-7. Mean flood sediment deposition rate for flood-tide (ft/yr)

Case	Marina	North Basin	South Basin	North Sump	Middle Sump	Excluding Sump Area
Case 01	0.92	0.85	0.99			
Case 02	0.19	0.16	0.22			
Case 03	0.20	0.20	0.20	0.68	0.53	0.17
Case 04	0.52	0.57	0.48			
Case 05	0.53	0.59	0.48	0.79	0.83	0.51

## 4.6 IMPLICATIONS ON DREDGING

The predicted deposition rates in Vallejo Marina are derived from observed deposition rates based on survey data from 2014–2023 that include the current piling configuration and evidence of vessel-induced scour. The limited spatial coverage of these surveys prevents a full evaluation of the spatial variability of sediment deposition rates and as such a basin average was used

here. Furthermore, these deposition rates are expected to change following the removal of the middle wall, addition of entrances, and alteration of vessel traffic due to change in dock design. Of note in this analysis, the sediment captured by sumps is not taken directly into account as the models do not include a finite sediment load tracked over time and then deposited, nor are changes in the deposition rates computed as sediment would be deposited over time.

Boats within the marina need at least 7 feet of water below MLLW to operate safely. Based on this water depth and bed elevation the estimated timing of for dredging based on the mean deposition rate is shown in Figure 4-31. The average dredge return interval using the mean deposition rates from Table 4-6 and post-bathymetry (Figure 4-1) are shown in Table 4-8. While this analysis indicates sumps dredged to 20ft MLLW will take more than 10 years to fill in, this does not consider the expected entrapment of sediment within these sumps which will result in a faster rate of infill. With both basins dredged to a depth of -10ft MLLW, each basin would require dredging at roughly the same interval for Case 1, Case 2, and Case 4. For Case 3 and Case 4, the north basin takes longer to fill. Over dredging is not considered in this analysis. In addition, changes to the south basin dredge depth could change the flow pattern and shear stress modeled in this study.

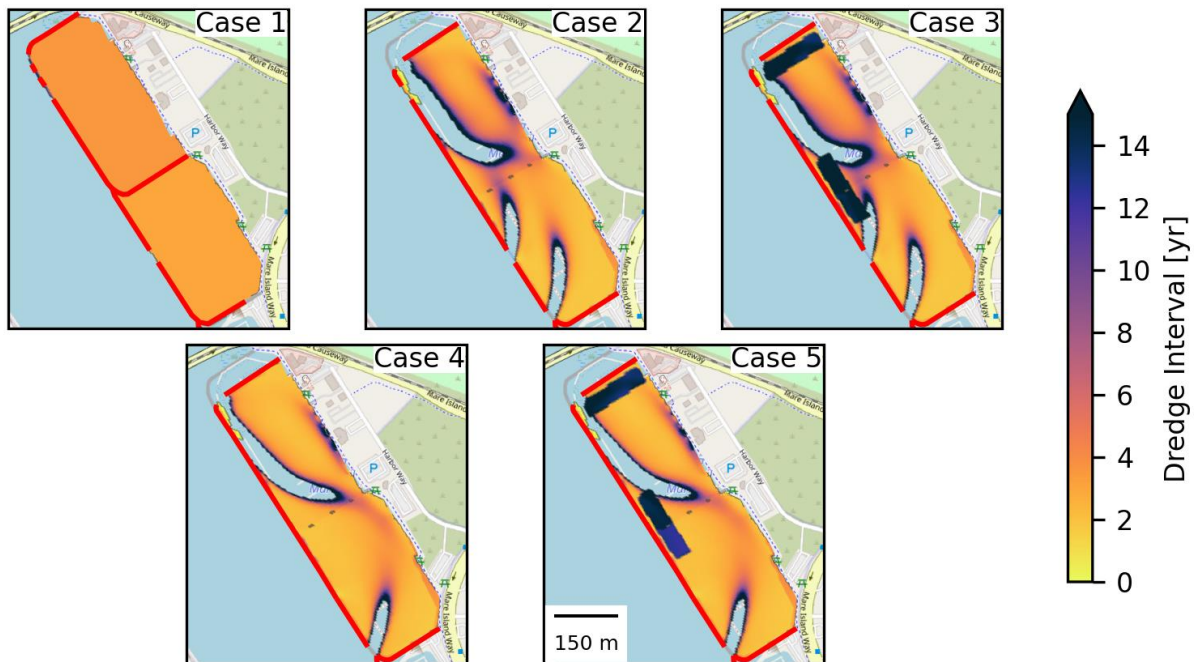


Figure 4-31. Time till bed attains a depth of -7ft MLLW.

Table 4-8. Estimated years before required dredging based on mean deposition rate.

Case	North Basin	South Basin	North Sump	Middle Sump	Maraina without Sumps	North Basin without Sumps
Case 1	3.3	3.3	--	--	--	--
Case 2	6.5	6.7	--	--	--	--
Case 3	10.9	7.5	38.8	38.8	6.6	6.9
Case 4	6.9	7.1	--	--	--	--
Case 5	11.5	7.9	40.9	40.8	6.9	7.2

## 5 CONCLUSIONS

The objective of this study is to support effective marina design through simulating bed shear stress and predicting the potential for deposition based on a combination of model results and survey data. The calibrated and validated Vallejo Marina model was shown to predict depth averaged velocities within the Mare Island Strait with good to excellent agreement, suggesting the results of forces within the marina are representative and can be used to compare alternatives configurations influence on deposition patterns.

Deposition rates experienced within the Marina were derived from a series of bathymetric surveys and accounted for periodic dredge activities within the Site. Due to limited spatial coverage of the surveys, a north and south basin average deposition rate were computed. Estimates of deposition rates aligned with observations by the marina operators. These rates formed the basis for evaluating changes in the system due to marina modifications by scaling relative to changes in modeled bed shear stress.

As a result of the previous study, which evaluated removing the marina wall dividing the north and south basin, reducing the marina's northern footprint, and adding sumps, F3 proposed 4 additional possible marina configurations (Cases 2-5) as well as proposed dredging of south basin to -10ft MLLW (Case 1). Results for these new designs, indicates velocities and shear stress increase in the marina with the presence of two north and south entrances, with the maximum velocity for standard conditions remaining below 2kts within the marina. Deposition rates are reduced from the original marina configuration with the same dredge depth. The addition of sumps results in increased deposition rate within the sumps as compared to deposition of the marina without sumps. It is worth noting that the sumps will likely capture more sediment than this rate of change analysis predicts and may influence the availability of sediment to deposit elsewhere. Also, in the analysis, the average deposition rates within each were used, rather than a model cell-by-cell basis. This was done to provide a conservative estimate of results across the system as a whole. The results suggest that the sumps would have a positive effect on the system's ability to modify sedimentation patterns and extend the amount of time between dredge events.

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