Technical Appendix E

Balboa West Marina Expansion Project Impact Analysis for Proposed Alternatives
Everest International Consultants, Inc.
March 2013

Balboa West Marina Expansion Project

Impact Analysis for Proposed Project Alternatives

Prepared for
URS Corporation

Prepared by
Everest International Consultants, Inc.

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Balboa West Marina Expansion Project
Impact Analysis for Proposed Project Alternatives

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Everest Project No. P2178

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1. INTRODUCTION

URS Corporation (URS) is assisting the Irvine Company in the development of the Balboa Marina West - an extension to an existing marina in the Lower Newport Bay, and has retained Everest International Consultants, Inc. (Everest) to provide coastal engineering support services for the proposed marina extension project. The coastal engineering study consists of two phases: Phase 1 for the evaluation of potential environmental concerns, and Phase 2 for the determination of design loadings of the selected marina extension alternative.

This report provides a summary of the approach and findings of the Phase 1 coastal engineering analyses conducted to evaluate potential impacts of the proposed marina extension on tidal currents, sedimentation and erosion to areas adjacent to the project site; in particular the impact of a proposed groin wall for one of the alternatives to the beach property just north of the marina and areas near and under the Pacific Coast Highway (PCH) Bridge.

Potential impacts associated with the proposed project were evaluated based on changes to hydrodynamic and transport conditions in the project vicinity. A two-dimensional (2D) hydrodynamic model of Newport Bay and Harbor was used to simulate tidal currents in the project vicinity under existing and with marina extension conditions. Changes in tidal currents were used to infer potential impact of the proposed marina alternatives to sedimentation near the PCH Bridge and adjacent beach area. Potential project impacts were evaluated for a range of conditions with varying tide levels and stormwater runoff.

Section 2 of this report provides a description of the project site and the proposed marina extension alternatives. The hydrodynamic model and model setup are summarized in Section 3. Model results and impact analyses are provided in Section 4. Lastly, a summary of the findings of this coastal engineering analyses is provided in Section 5.

2. PROJECT DESCRIPTION

2.1 Project Site

Newport Bay is comprised of Lower and Upper Newport Bay, separated by the Pacific Coast Highway (PCH) Bridge. Lower Newport Bay includes Balboa Island, Lido Island, and Newport Island Channels. The upper portion of the bay extends nearly four miles north from the PCH Bridge into the Upper Newport Bay Ecological Reserve and Upper Newport Bay State Marine Park. The proposed Balboa Marina extension is located in Lower Newport Bay just south of the PCH Bridge on the eastern edge of the main channel. A map showing the location of the project site is shown in Figure 2.1.

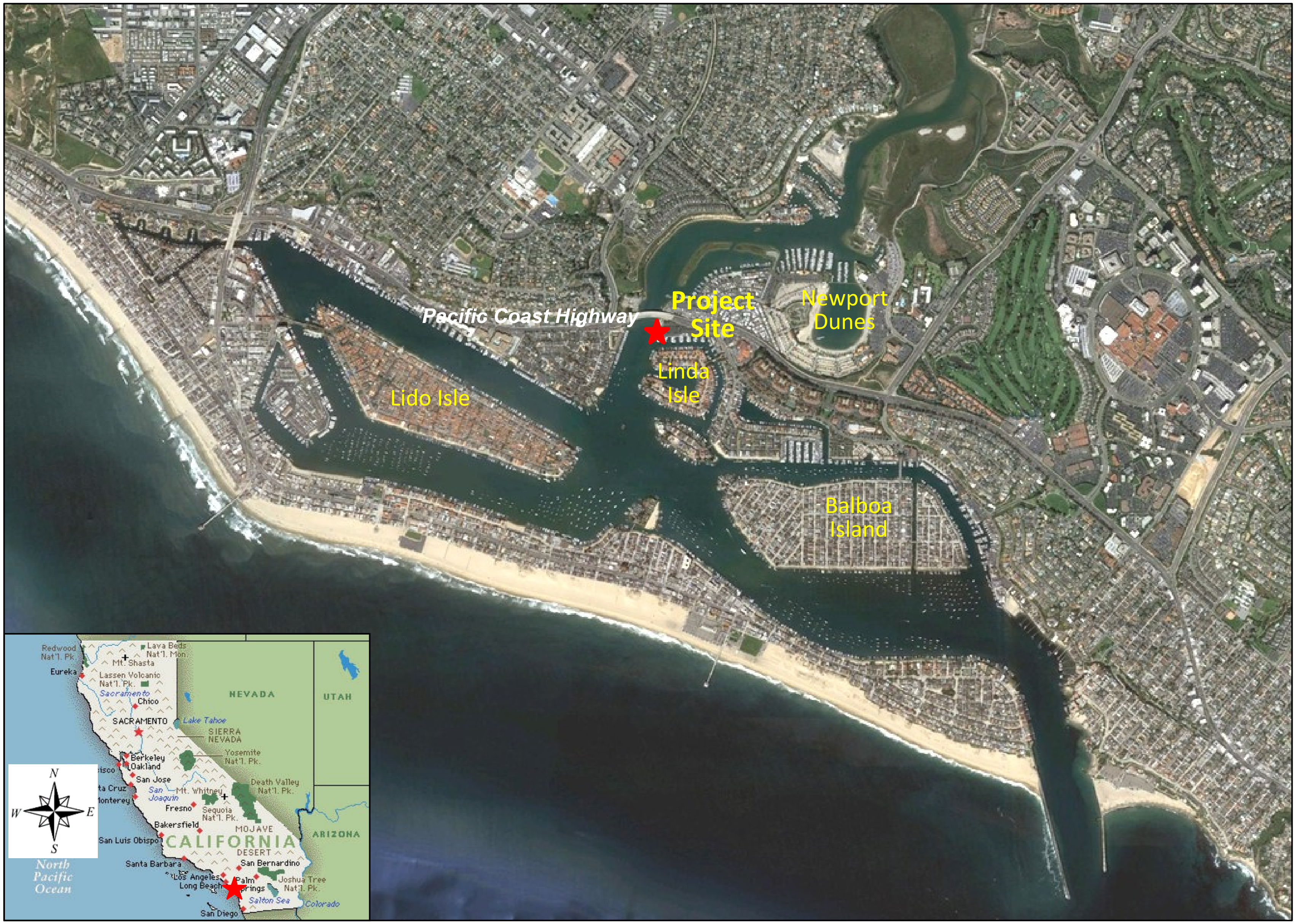
Site photos of the proposed project site are provided in Figure 2.2. The upper left picture shows the seawall of the existing marina looking downstream towards Linda Isle. Upstream from the proposed project site is the PCH Bridge, shown in the upper right picture. As shown in the picture, the bridge spans across the main channel and is supported by four concrete piers within the main channel. Between the proposed project site and the PCH Bridge, there is a sandy beach area adjacent to the existing seawall. Photos of the adjacent beach area looking upstream and downstream from the existing marina are provided in the two lower photos in Figure 2.2.

2.2 Proposed Marina Alternatives

There are three proposed marina extension alternatives, differ by the presence or absence of a groin wall between the proposed marina extension and the property to the north, and the use of a seawall or revetment for slope protection for the landside development of the marina property. In summary, these three proposed alternatives are:

- Alternative 1 - with seawall and groin wall
- Alternative 2 - with seawall and no groin wall
- Alternative 3 – with revetment, no seawall and no groin wall

For Alternatives 2 and 3, each has an Option 1 with a cutoff wall instead of a groin wall to separate the marina extension from the adjacent property to the north. In addition, for all the alternatives and option, each has two different dock layouts – Layout A with most of the boats docking perpendicular to the dominant flow along the main channel, and Layout B with most of the boats docking in line with the dominant flow along the main channel.



Source: Google Earth Pro

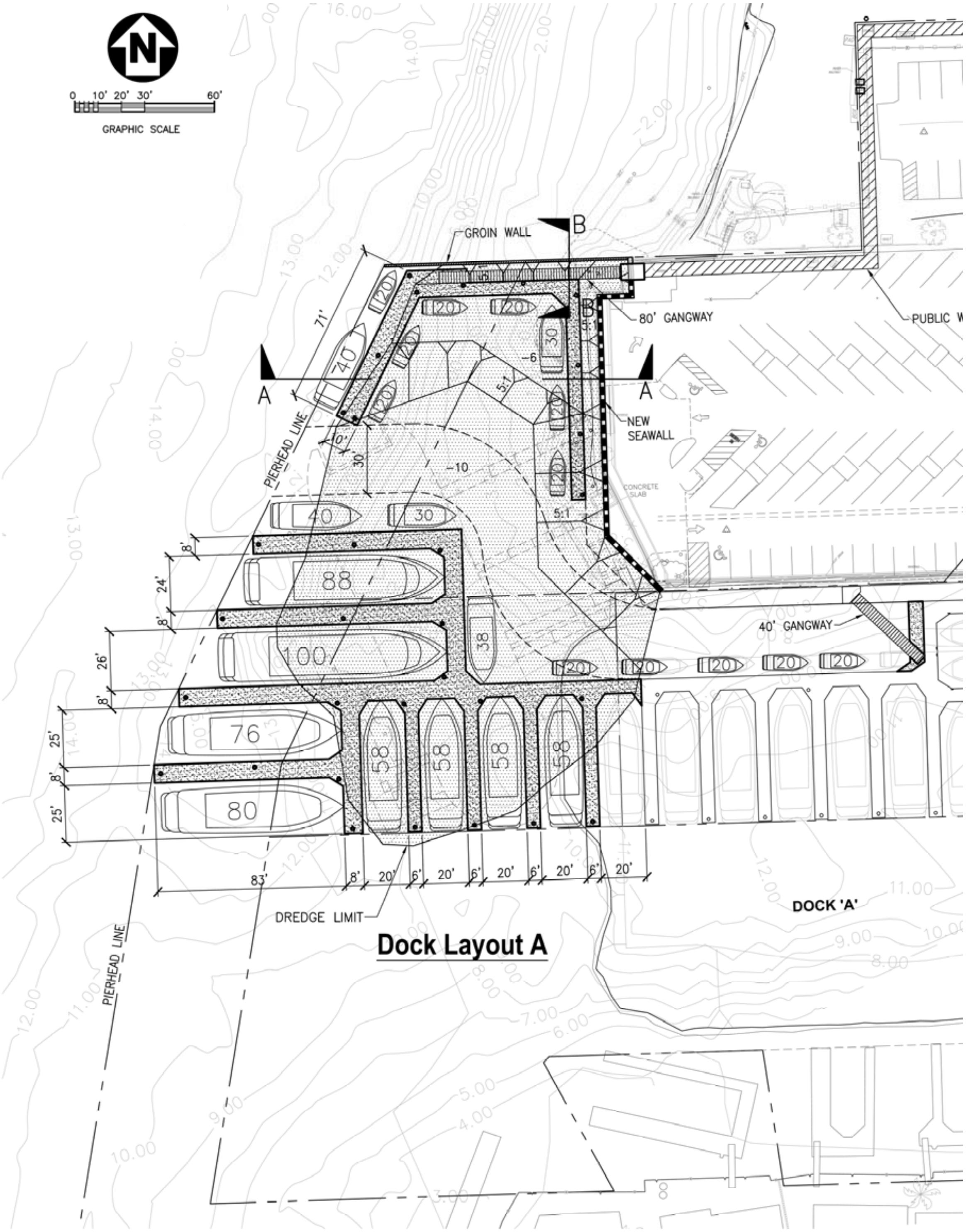
Figure 2.1 Project Site Location Map

Figure 2.2 Project Site Photos

The proposed Alternative 1, as illustrated in Figure 2.3, would replace the existing seawall with a new seawall along the existing marina edge. In addition, a groin wall would be constructed along the northern edge of the marina separating the marina from the adjacent beach area. The groin wall with a top elevation of +8.0 ft, MLLW would block tidal flows and enable dredging of the marina to deeper depths than the adjacent property.

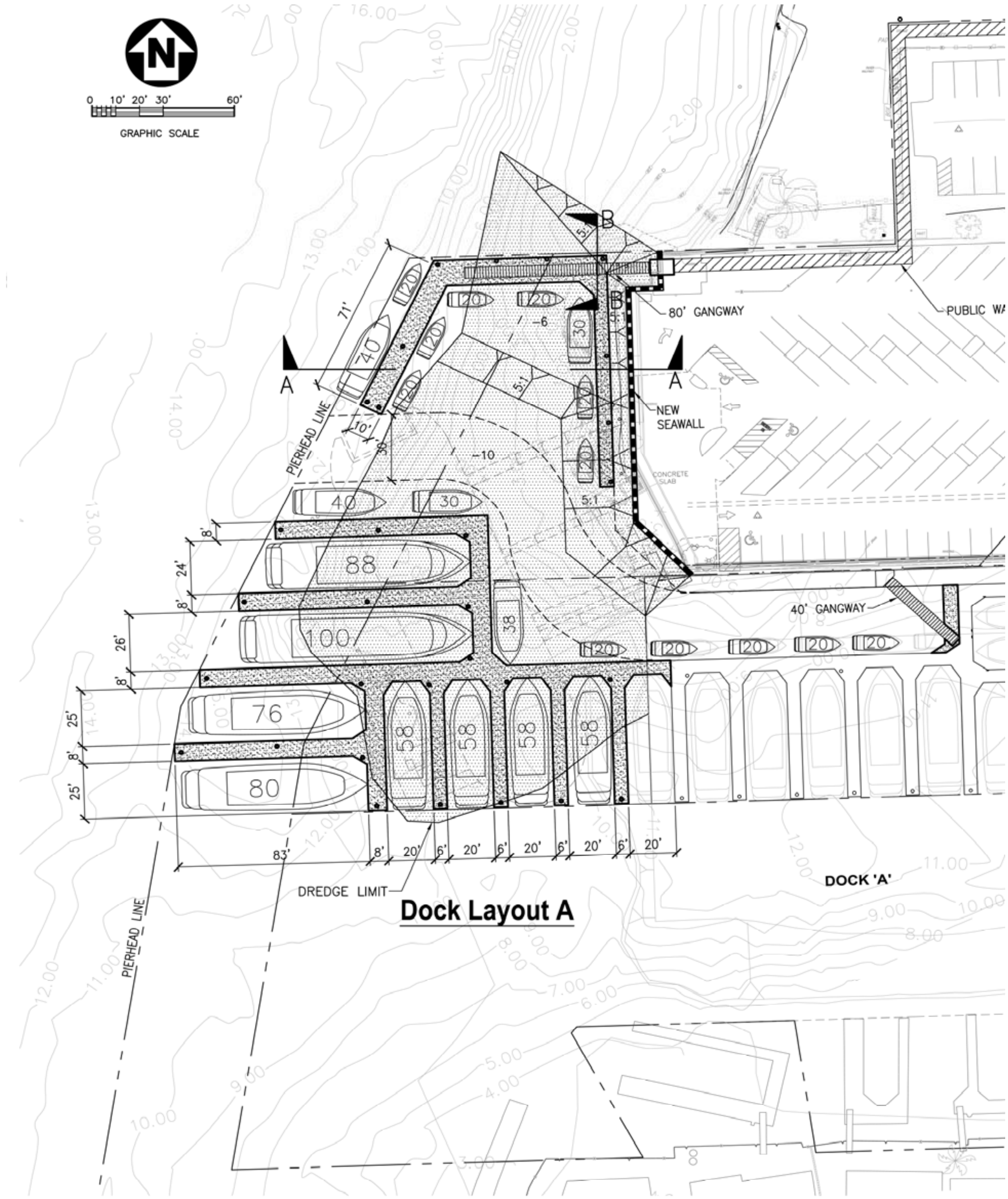
Similar to Alternative 1, the existing seawall would be replaced with a new seawall under Alternative 2. However, as shown in Figure 2.4, there is no groin wall under this alternative. Without the groin wall, the dredging area has to be extended northward from the marina into the adjacent property to provide smooth transition in grading from the marina area to existing bathymetry. Alternative 2 Option 1, which is shown in Figure 2.5, would add a cutoff wall instead of a groin wall along the northern marina edge. The cutoff wall is essentially a wall with the top elevation matching the existing bathymetry. This would maintain the beach area in the adjacent property while not blocking tidal flows between the marina and the adjacent property.

The proposed Alternative 3 is shown in Figure 2.6. Similar to Alternative 2, this alternative will not have a groin wall. However, under this alternative, the existing seawall will be replaced by a revetment, thereby expanding the intertidal area within the marina. Also similar to Option 1 of Alternative 2, Alternative 3 Option 1 will include a cutoff wall along the northern edge of the marina, as shown in Figure 2.7.



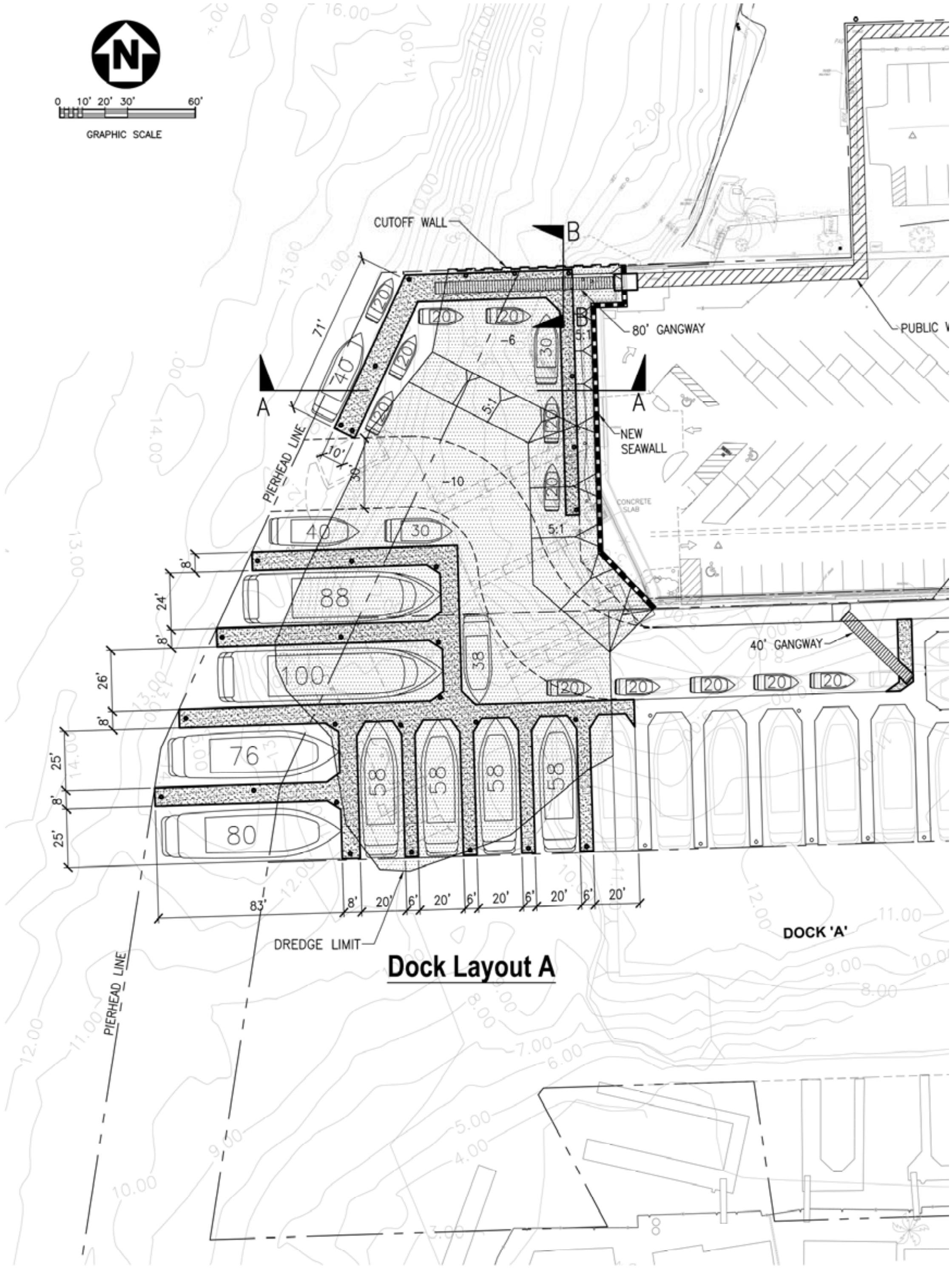
Source: Drawing prepared by URS

Figure 2.3 Dock Layout for Alternative 1A (With Seawall and Groin Wall)



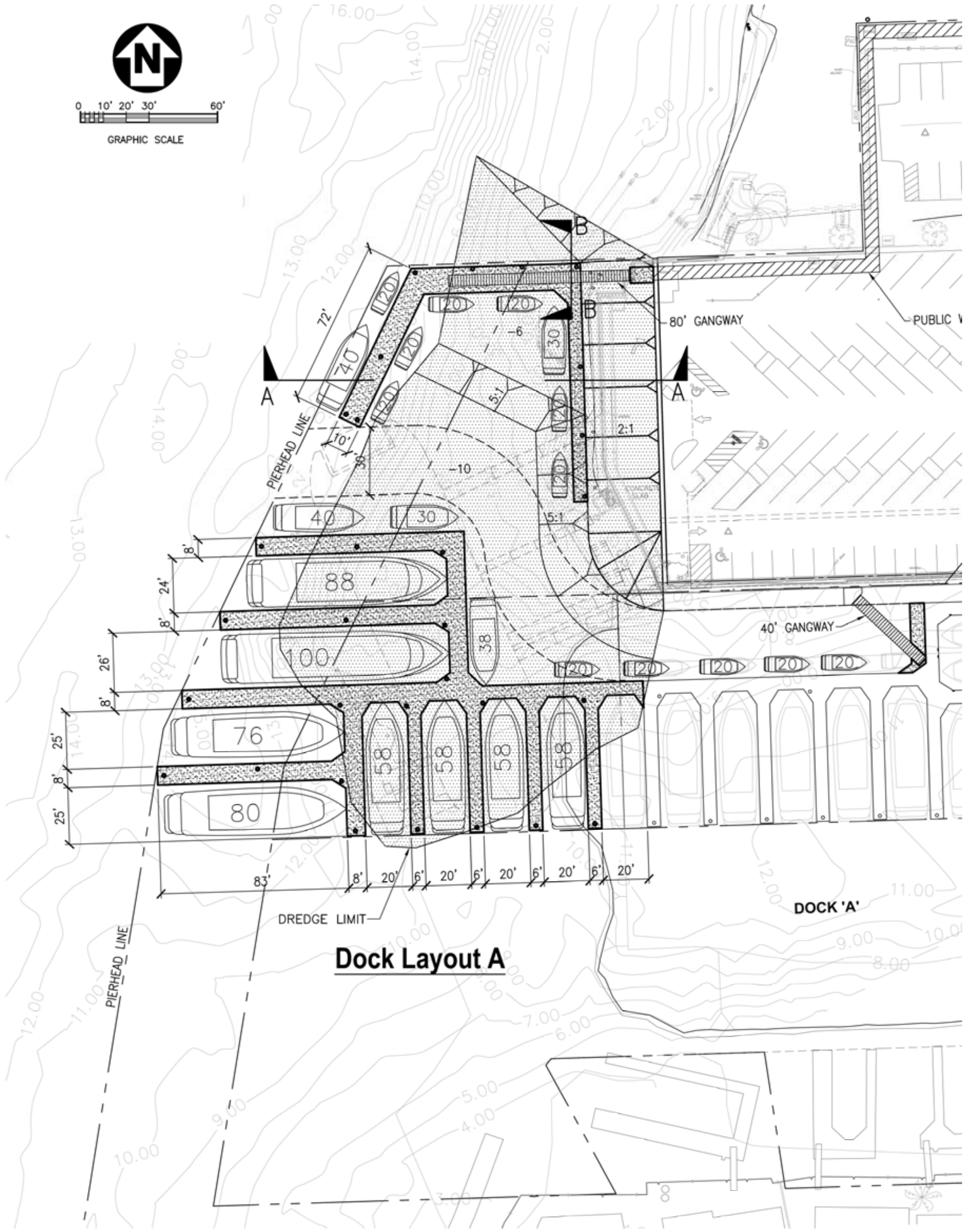
Source: Drawing prepared by URS

Figure 2.4 Dock Layout for Alternative 2A (With Seawall and No Groin Wall)



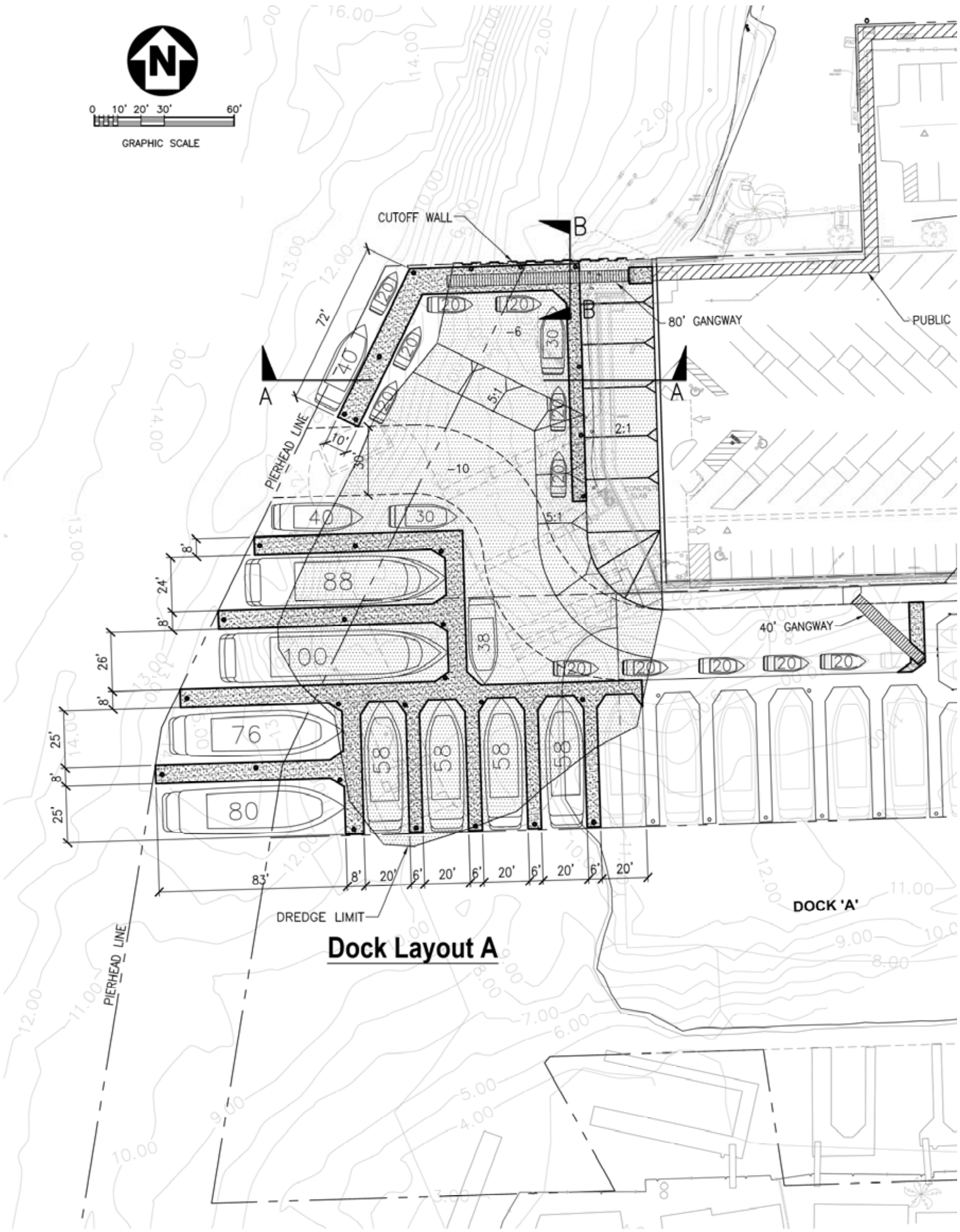
Source: Drawing prepared by URS

Figure 2.5 Dock Layout for Alternative 2A – Opt 1 (With Seawall and Cutoff Wall)



Source: Drawing prepared by URS

Figure 2.6 Dock Layout for Alternative 3A with Revetment (No Seawall and No Groin Wall)



Source: Drawing prepared by URS

Figure 2.7 Dock Layout for Alternative 3A with Revetment – Opt 1 (No Seawall with Cutoff Wall)

3. HYDRODYNAMIC MODELING

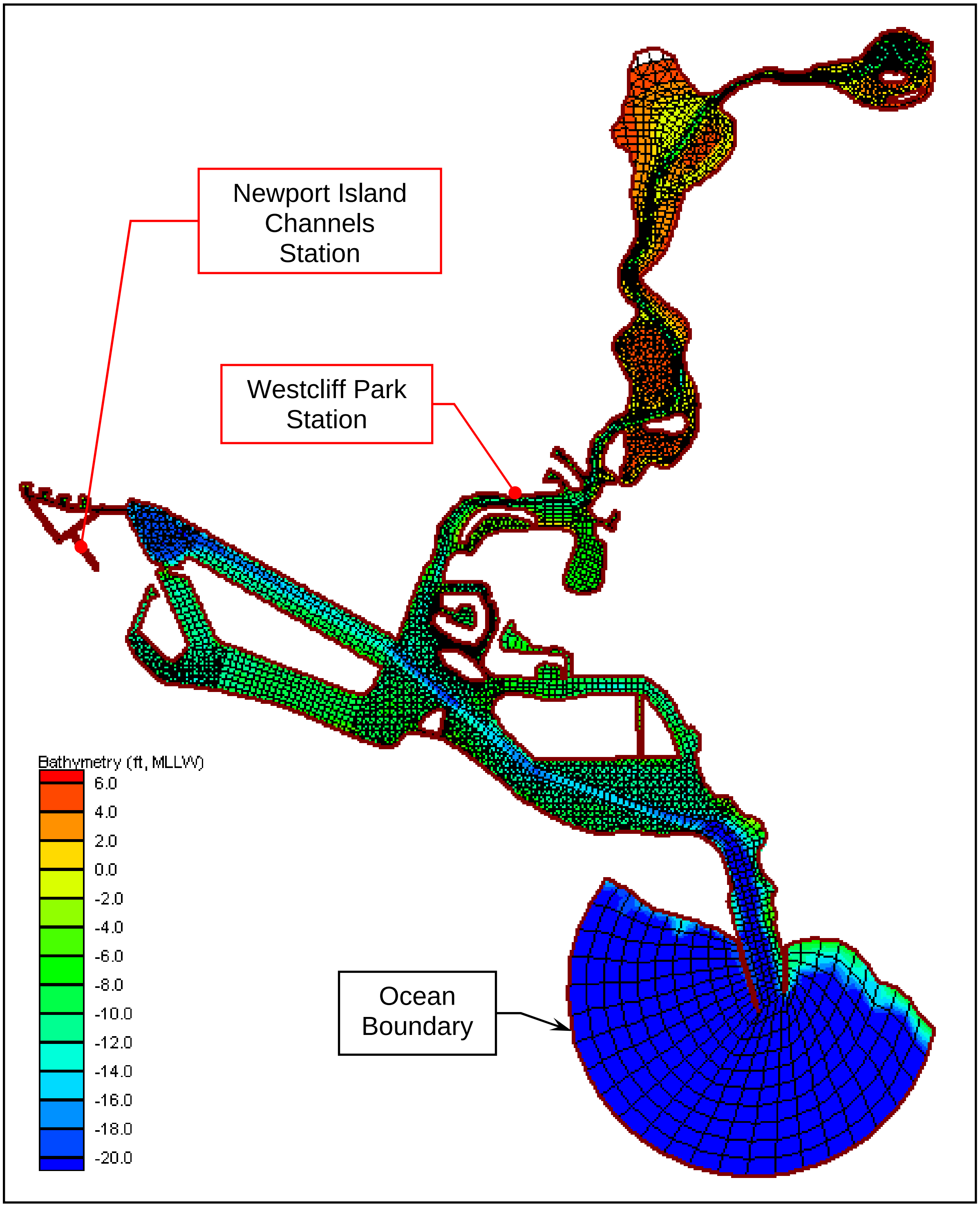
3.1 Hydrodynamic Model

A two-dimensional (2D) hydrodynamic model of Newport Bay that was previously developed for the City of Newport Beach (Everest 2005) was used for this study to simulate tidal currents in the project vicinity. The Newport Bay Model was developed based on the RMA2 Model; the later was developed by the U.S. Army Corps of Engineers (USACE). The RMA2 model is capable of simulating water elevations and depth-averaged velocities due to tidal conditions and storm water discharges. The Newport Bay Model has been verified with tide and velocity data collected at two different locations, as shown in Figure 3.1. One of the model verification locations (Westcliff Park Station) is near the project site. Model verification results at this location when the Newport Bay Model was developed are reproduced in Figure 3.2 to illustrate that the model can provide accurate simulation of tidal elevations and tidal currents at the vicinity of the project site. In Figure 3.2, the top panel shows a comparison of the model-predicted water elevations with those measured by a tide gage. It can be seen that the model-predicted water surface elevations perfectly match with the field data. The bottom panel of Figure 3.2 shows the comparison of model-predicted velocities with those measured with an ADCP (Acoustic Doppler Current Profiler). It can be seen that the model-predicted velocities in general match well with the field data.

3.2 Model Grids

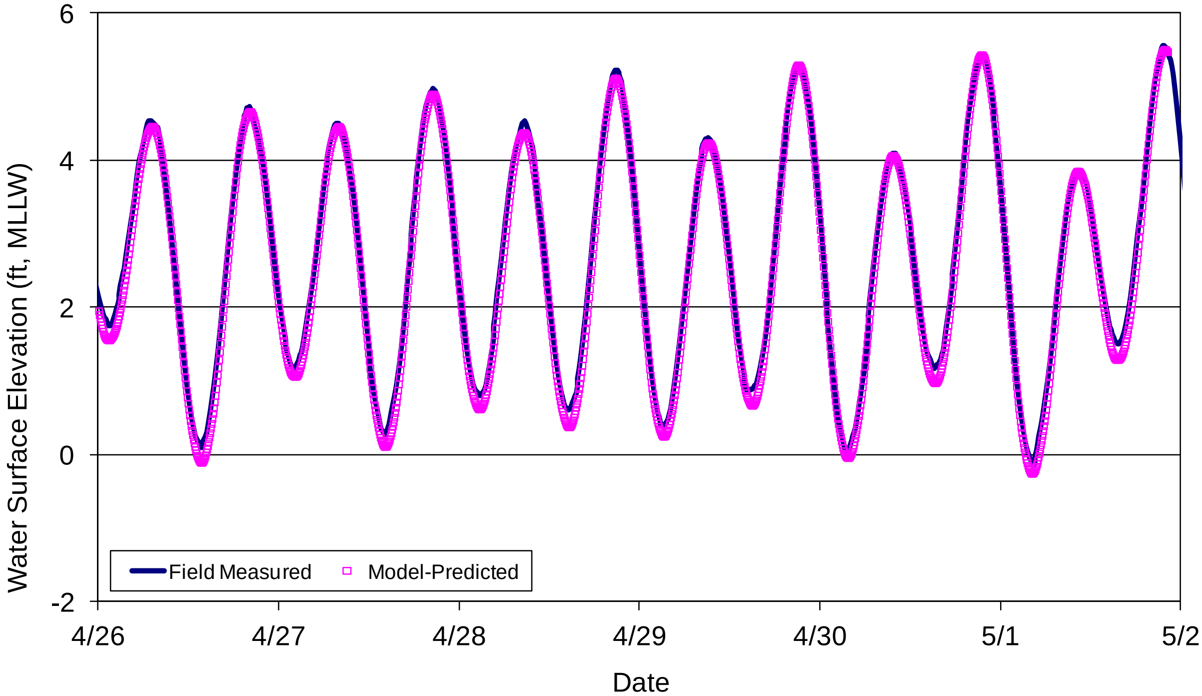
3.2.1 Model Grid for Existing Condition

The numerical model grid for existing condition developed for this study is shown in Figure 3.3. This model grid was developed by modifying the Newport Bay grid to reflect details of the project site and vicinity. As illustrated in the left panel of Figure 3.3, the model grid in the project vicinity was refined with smaller grid cells to improve model accuracy. The model grid extends approximately three-quarters of a mile offshore. The ocean inlet for Newport Bay is defined by two jetties that enable tidal exchange between the ocean and bay. The model grid of the bay itself includes both the intertidal areas and tidal channels of Upper Newport Bay, as well as Lower Newport Bay.

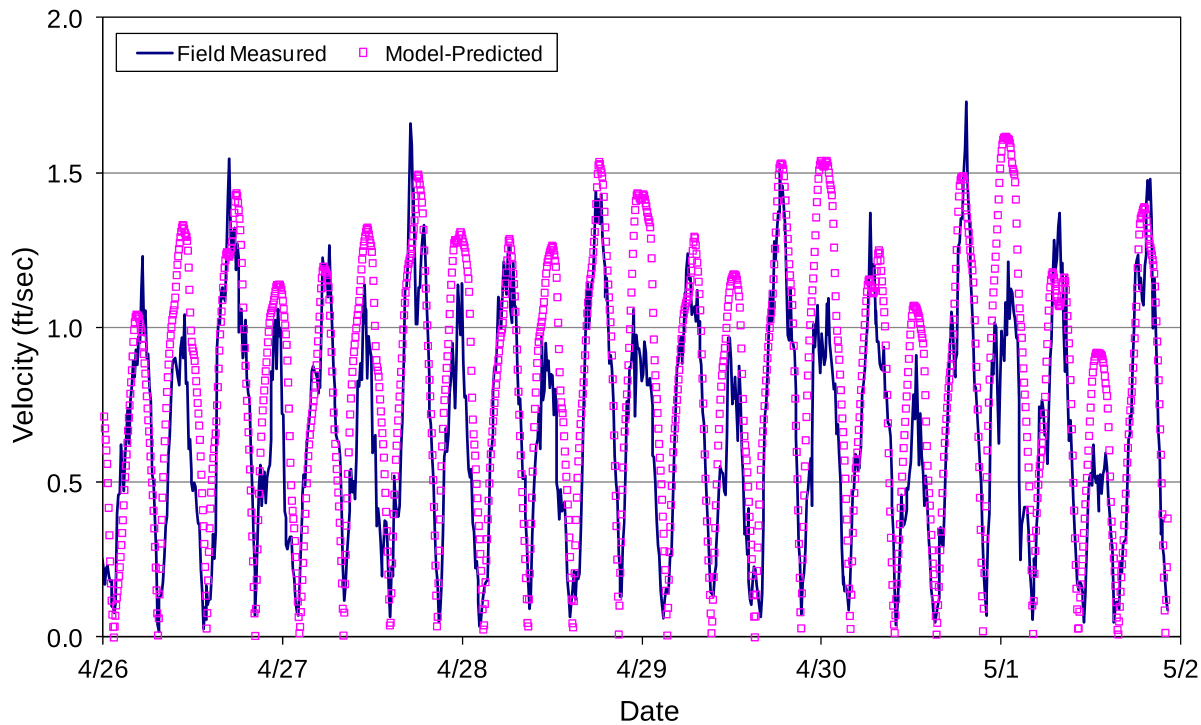


Source: Newport Bay Model Report, Everest 2005

Figure 3.1 RMA2 Model Grid for Newport Bay



(A) Water Surface Elevations



(B) Tidal Currents

Source: Newport Bay Model Report, Everest 2005

Figure 3.2 Comparison of Field Measured Data and Model-Predicted Results at Westcliff Park Station (4/2/2003 – 5/2/2003)

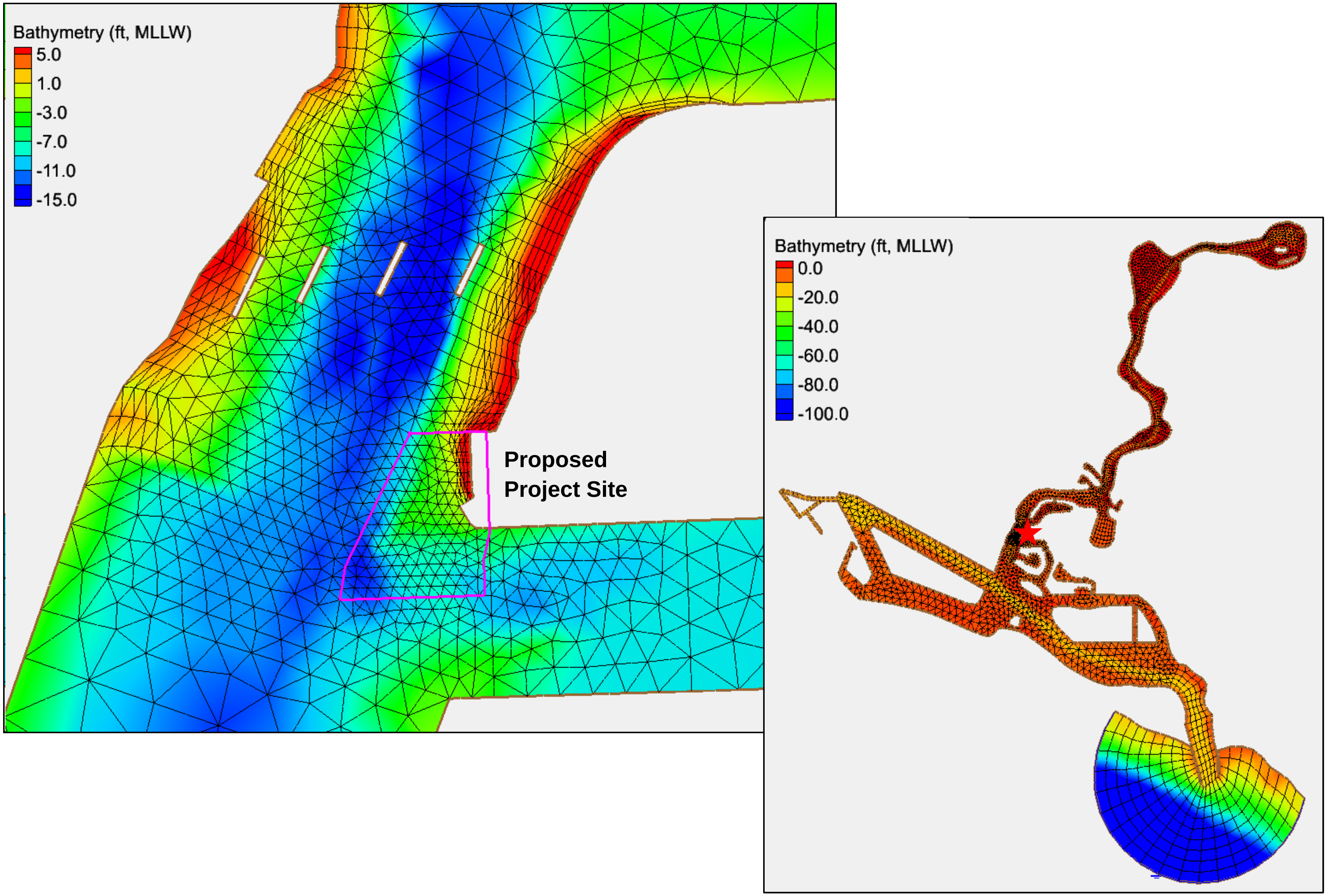


Figure 3.3 Numerical Model Grid for Existing Conditions

Bathymetry data used in setting up the model grid were based on several data sets. Majority of the Lower Newport Bay bathymetry was defined based on a USACE survey conducted in April 2002. Bathymetry for Upper Newport Bay was specified using a USACE survey from 2003. Bathymetry in the project area was established using more recent bathymetric surveys conducted in 2011 and 2012. The remaining areas where detailed bathymetric data were not available were based on the 1999 National Oceanic and Atmospheric Administration (NOAA) navigation chart (No. 18754).

3.2.2 Model Grids for Proposed Alternatives

For each of the proposed alternatives and option described in Section 2.2 and shown in Figures 2.3 through 2.7, there is a corresponding numerical model grid being developed. These corresponding model grids are shown in Figures 3.4 to 3.8. Each of these model grids was modified from the model grid for existing condition to include all the changes to the site for each particular alternative. For example, the model grid for Alternative 1 shown in Figure 3.4 has included the new seawall and groin wall, as well as changes in bathymetry at the project site.

Note that the dock facility (decks and piles) are not shown in the model grids. These features are not significant in changing the flows in the project vicinity. However, if the marina is fully occupied, the boats will block the tidal flow. Potential impacts to the adjacent flows when the marina is fully occupied were evaluated assuming the boats would block all tidal flows. For those simulations (discussed later in Section 4), the area for Dock Layouts A and B in the model grid is blocked off. An example of the modified grid for Alternative 1 with the blocked off area is shown in Figure 3.9.

3.3 Model Simulation Conditions

3.3.1 Tides

Tides off the coast of Newport Bay are characterized as mixed, semidiurnal with two daily highs and two daily lows. Tidal conditions were defined based on tidal datums at the NOAA tide gage at the Newport Bay Entrance (9410580). Tidal datums from the 1983-2001 tidal epoch for the Newport Bay Entrance are summarized in Table 3.1. Typical tide conditions were constructed based on the datums with two highs (MHHW and MHW) and two lows (MLW and MLLW) over a 24-hour period.

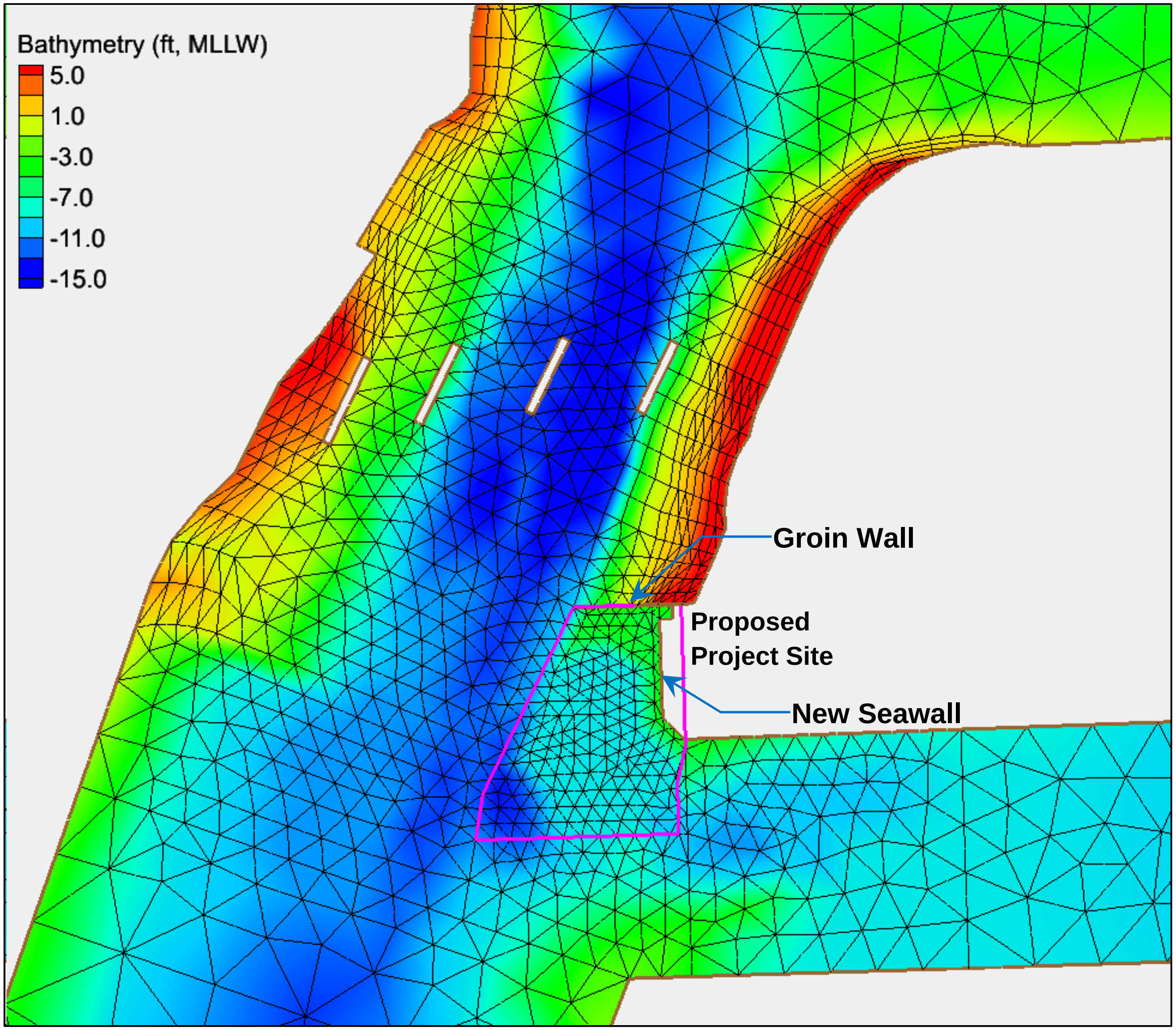


Figure 3.4 Model Grid for Alternative 1 (With Seawall & Groin Wall)

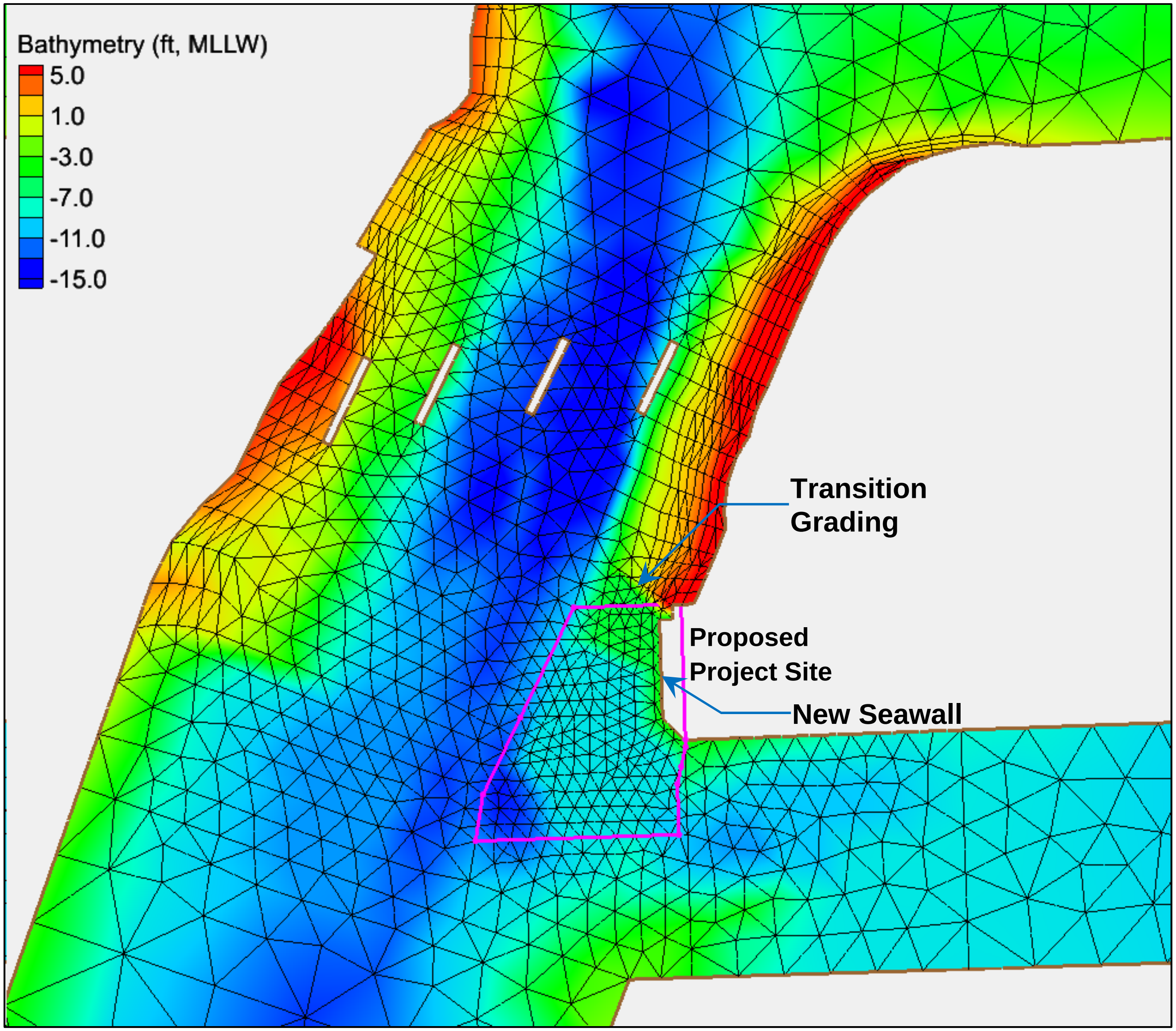


Figure 3.5 Model Grid for Alternative 2 (With Seawall and No Groin Wall)

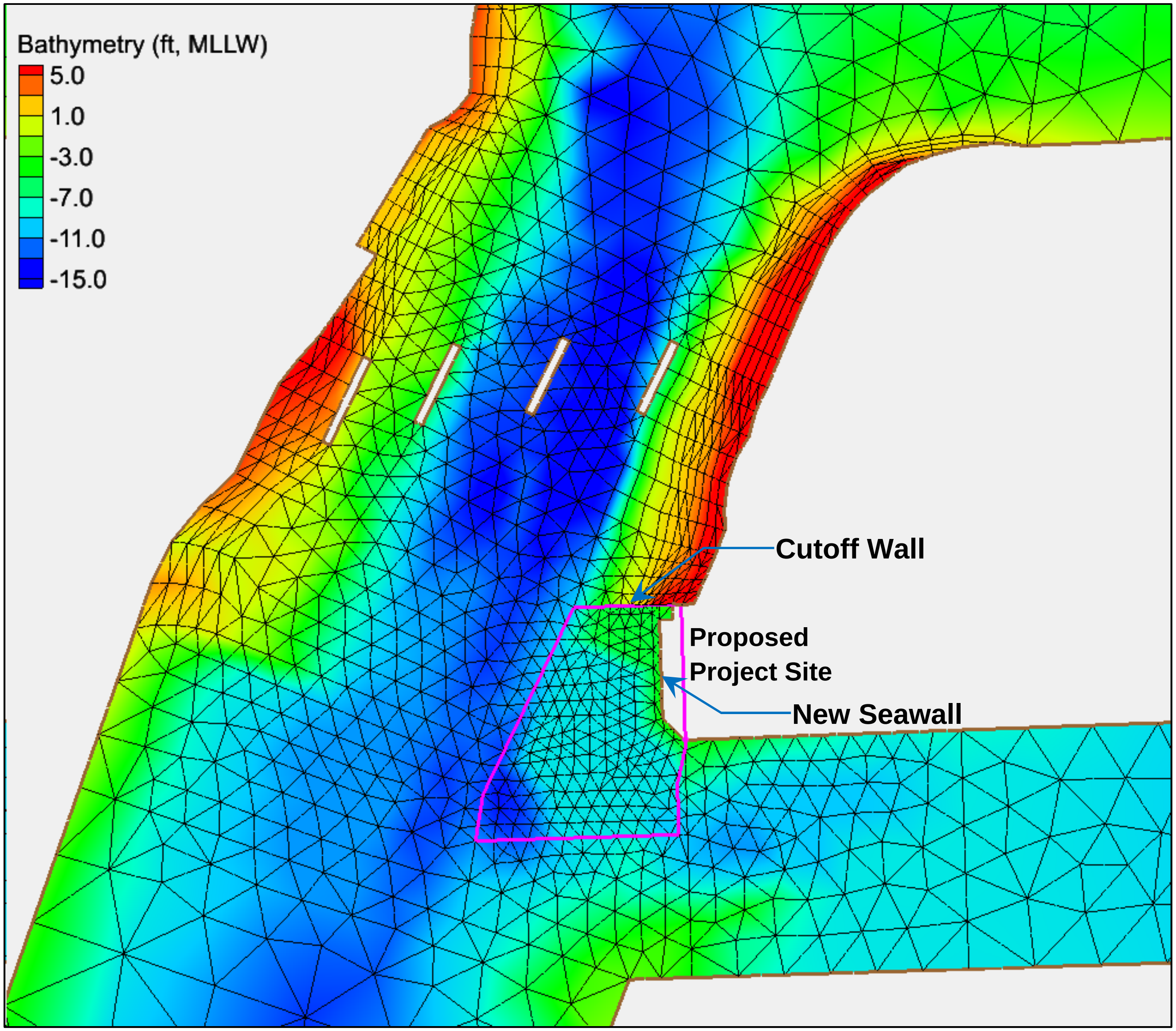


Figure 3.6 Model Grid for Alternative 2 – Opt 1 (With Seawall and Cutoff Wall)

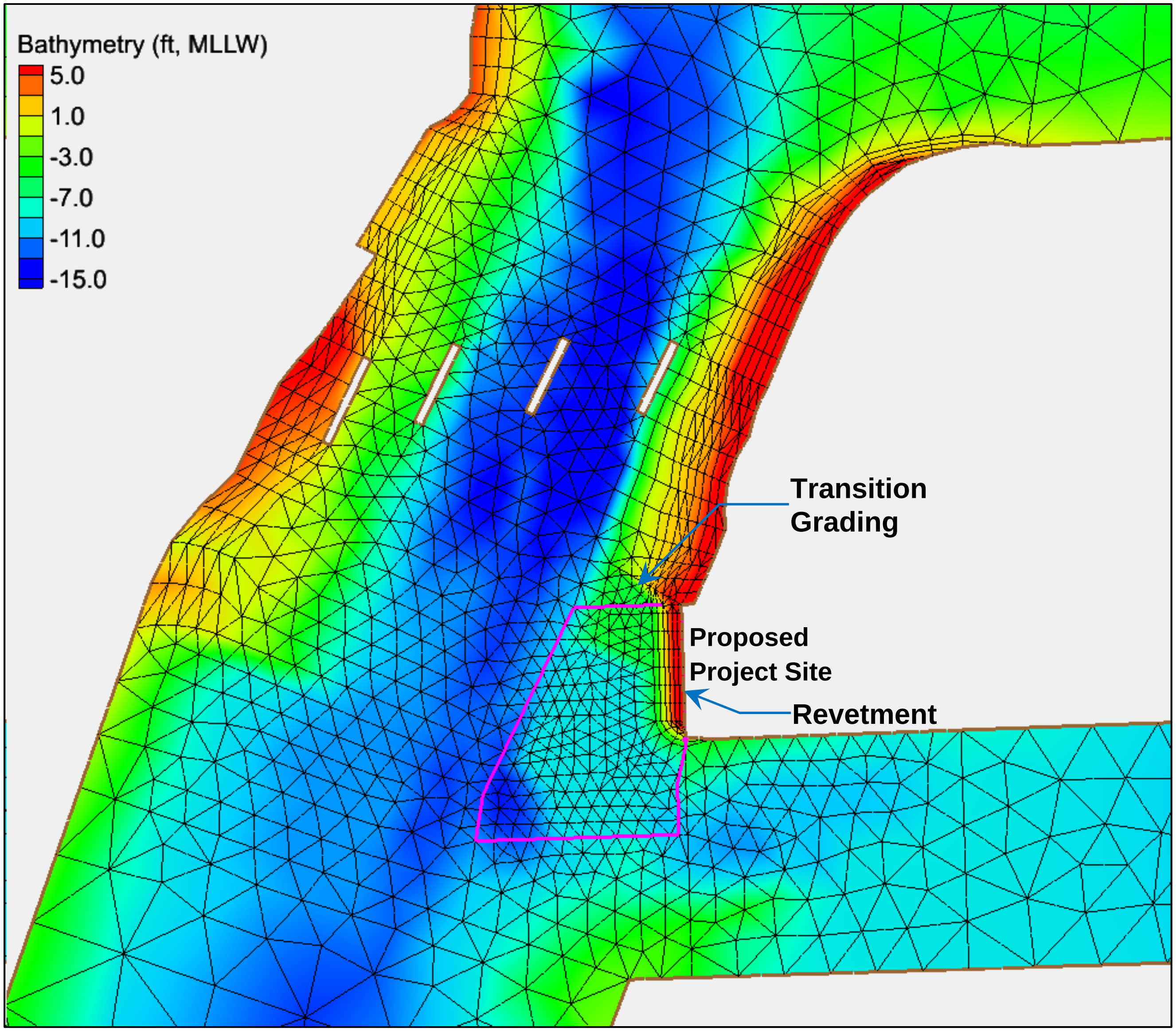


Figure 3.7 Model Grid for Alternative 3 (Revetment, No Seawall and No Groin Wall)

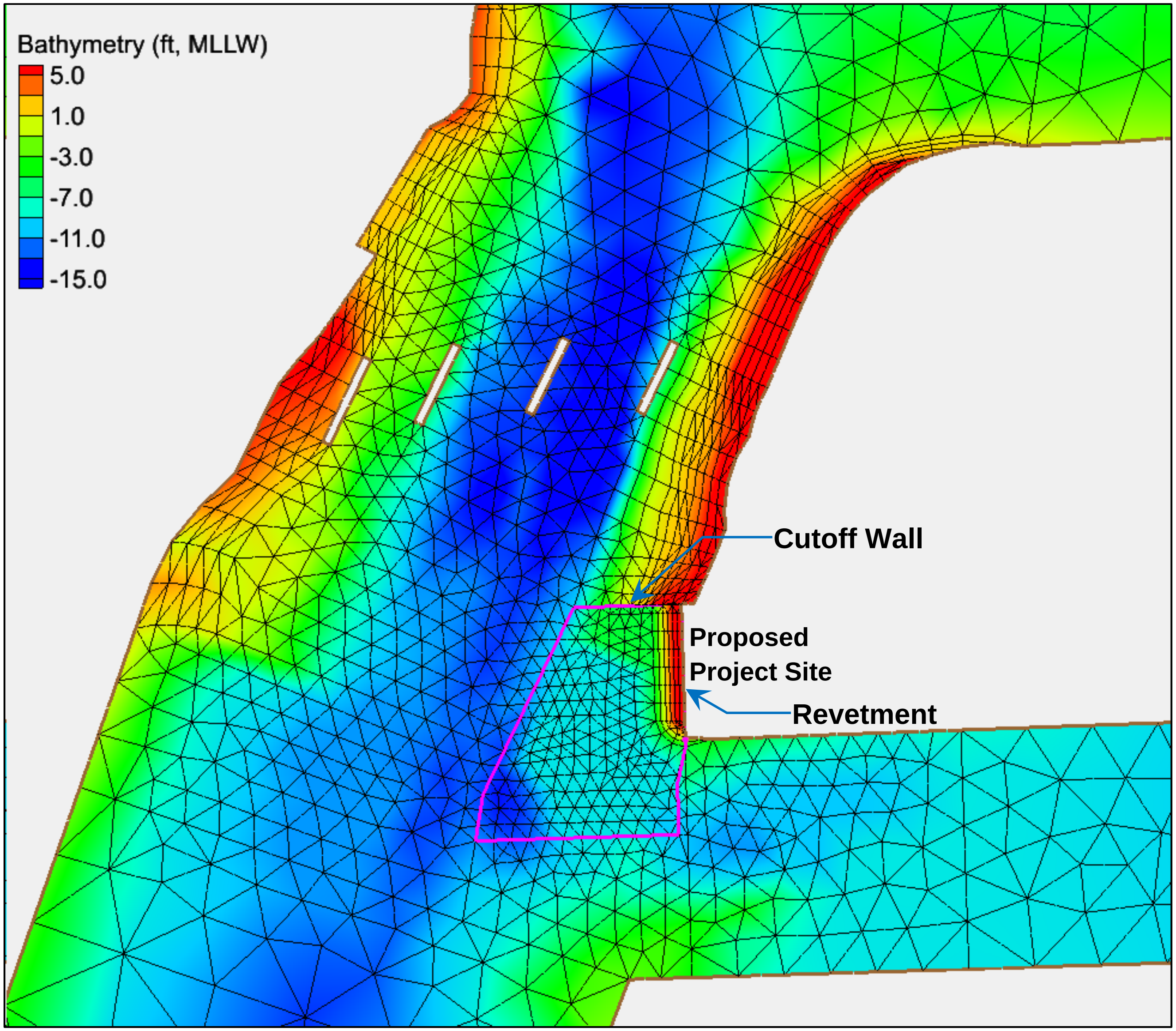


Figure 3.8 Model Grid for Alternative 3 – Opt 1 (Revetment, No Seawall with Cutoff Wall)

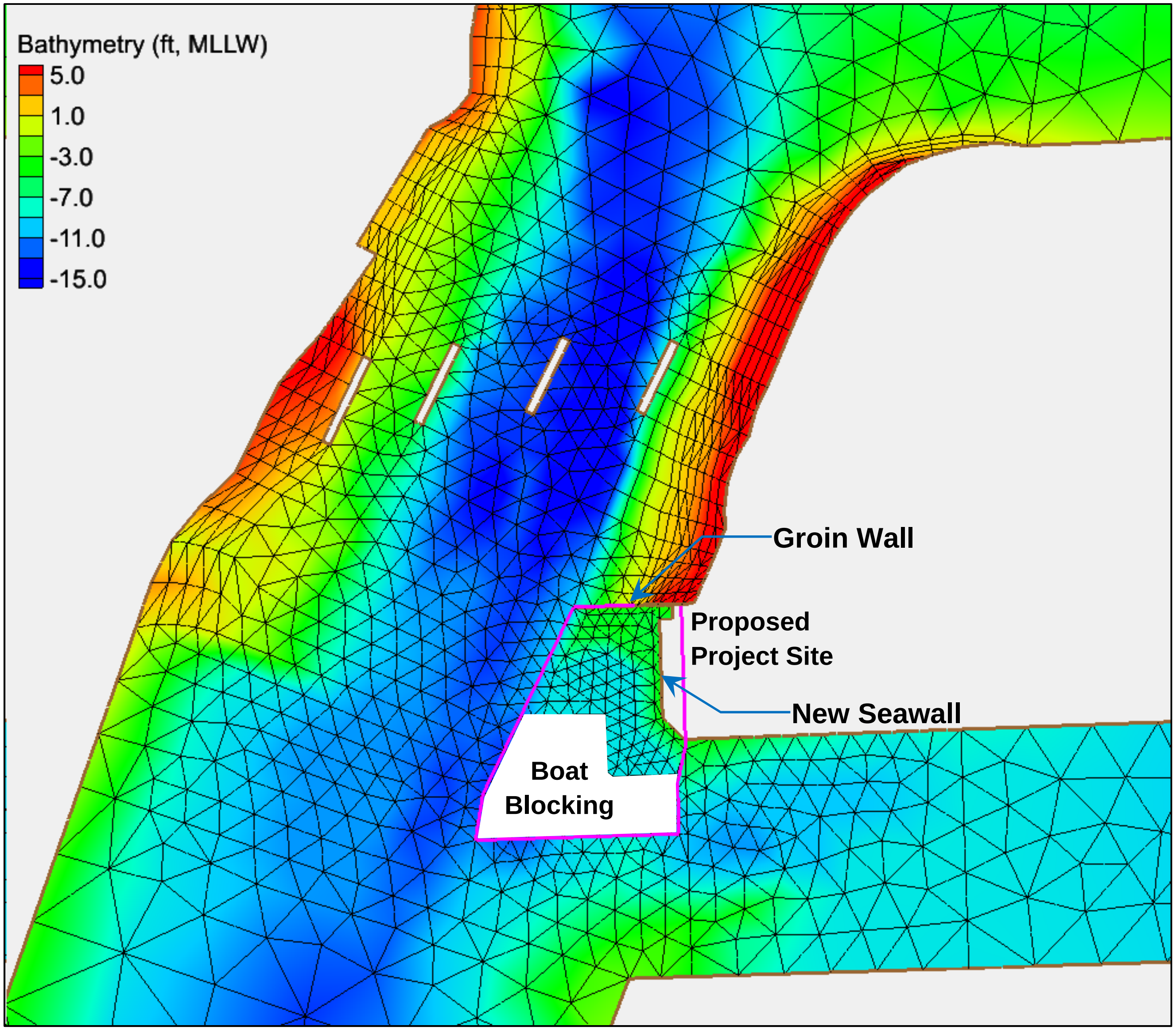


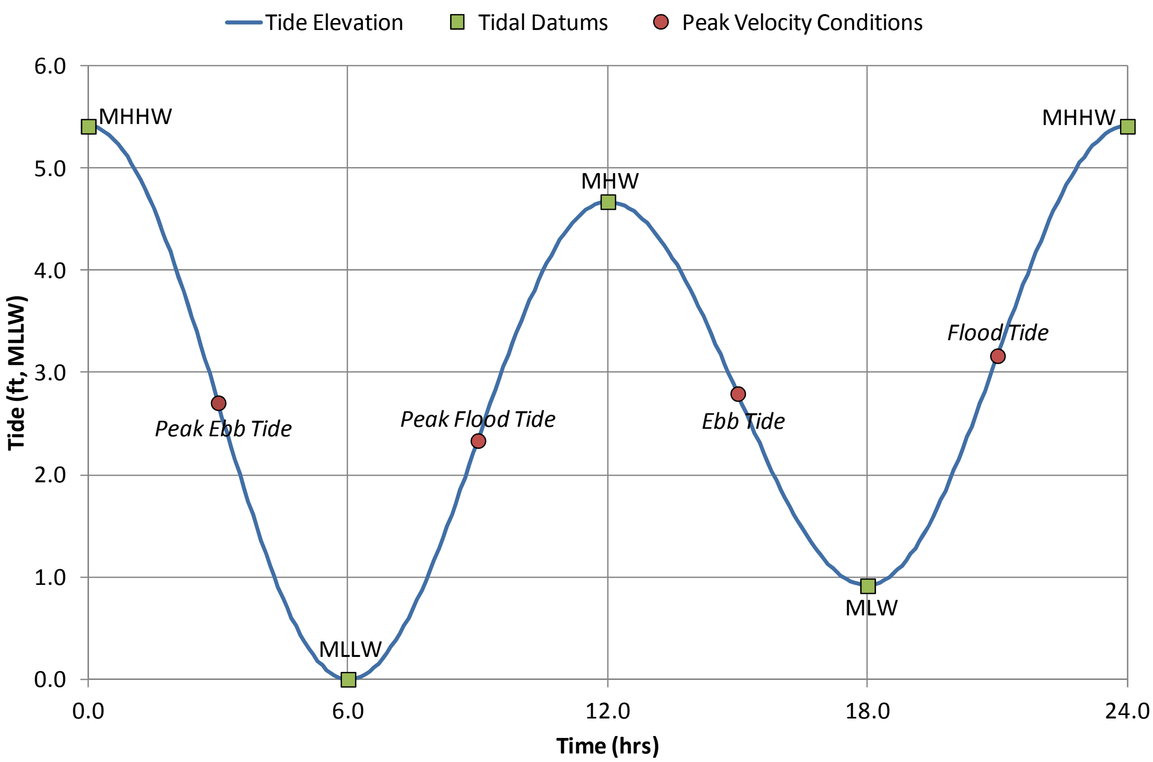
Figure 3.9 Modified Model Grid for Alternative 1 Boat Blocking

Table 3.1 Tidal Datums for Newport Bay Entrance

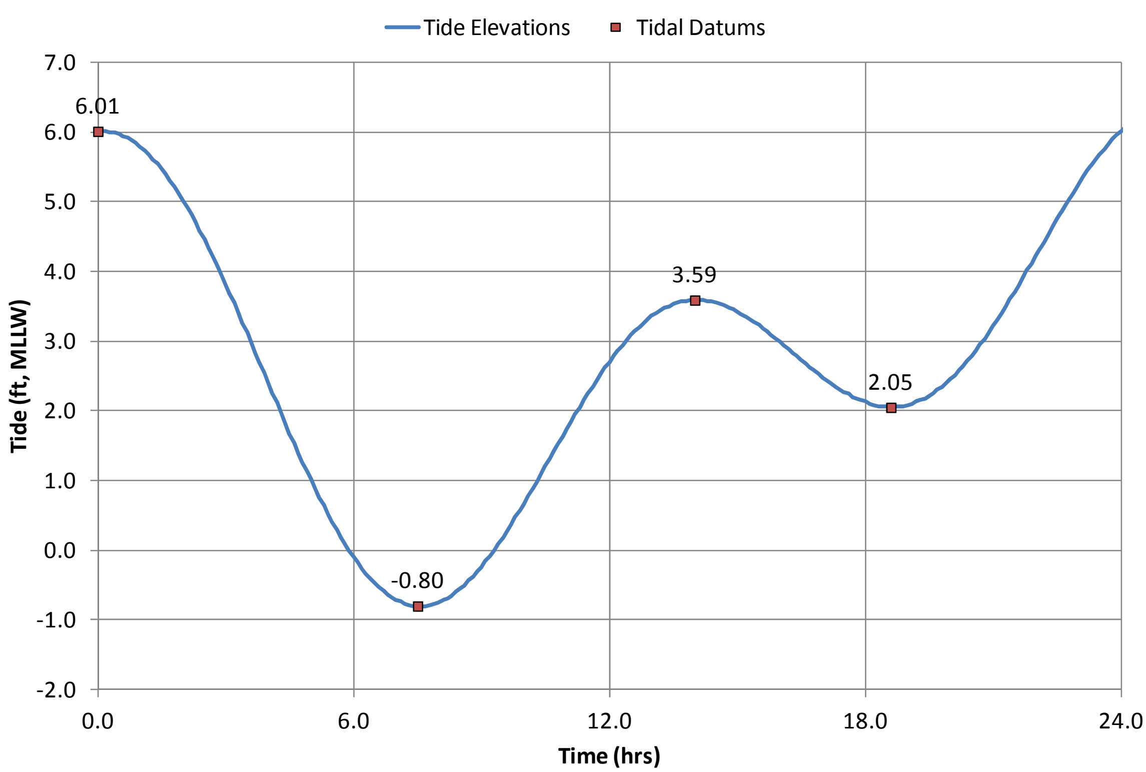
TIDAL DATUM	ELEVATION (FT, MLLW)
Highest Observed Water Level (1/28/83)	7.667
Mean Higher High Water (MHHW)	5.410
Mean High Water (MHW)	4.672
Mean Sea Level (MSL)	2.772
Mean Low Water (MLW)	0.915
Mean Lower Low Water (MLLW)	0.000
Lowest Observed Water Level (1/20/88)	-2.352

Source: NOAA 2003

Most of the model simulations were conducted with a mean tide to evaluate the potential long-term average impact of the proposed marina extension to adjacent tidal flows and sedimentation. However, limited simulations were also conducted using a spring tide condition to evaluate potential project impact during tidal conditions with a larger tidal range. The mean tide and spring tide used for model simulations are graphically illustrated in Figure 3.10.



(A) Mean Tide



(B) Spring Tide

Figure 3.10 Tidal Elevations for Newport Bay

4. IMPACT ASSESSMENT

4.1 Overview

The hydrodynamic model described in Section 3 is used to evaluate potential impact of the proposed marina alternatives to tidal currents in the project vicinity, in particular the currents between the PCH Bridge piers. The analyses include the impact to tidal currents (described in Section 4.2), and fresh stormwater flow from Upper Newport Bay during a 100-year flood event (described in Section 4.3). Potential impacts to sedimentation and erosion are inferred from the change in tidal currents (described in Section 4.4).

4.2 Tidal Currents

4.2.1 Existing Conditions

As mentioned in Section 3.2, mean tides were used to determine the average long-term hydrodynamic conditions in the project vicinity. The tidal oscillation of water levels results in tidal exchange between the ocean and Newport Bay. Tidal currents move out of the bay during ebb tide as water level drops, and move up into the bay during flood tide as water level rises. Tidal currents throughout the bay and at the project vicinity vary with the rise and fall of the water levels. In general, the highest tidal currents occur at a time somewhat halfway between the peak of the high and low tides, while slower tidal currents happen near the peak of the daily highs and lows. Throughout a twenty-four tidal cycle, the peak velocities take place in between the four tidal peaks (MHHW, MLLW, MHW, and MLW). The timing of the peak velocities are indicated by the red dots on the mean tidal cycle shown in Figure 3.10, and labeled as the peak ebb tide, peak flood tide, ebb tide, and flood tide.

The model simulated tidal velocities in the project vicinity for existing conditions are provided in Figure 4.1. In the figure, the velocity distributions during peak ebb, peak flood, ebb and flood tides are shown in separate panel. Arrows in the figure show the flow direction, and different colors (red for higher and blue for slower currents) are used to indicate different velocity magnitudes. As shown in the figure, tidal currents in the project vicinity are ebb-dominant, that is, ebb currents are higher than flood currents. As expected, the highest tidal currents occur during the peak ebb tide condition with the highest currents occur between the PCH Bridge piers. The area just north of the PCH Bridge also shows high ebb currents. In general, tidal currents higher than one foot per second may result in sediment erosion. Since the peak ebb currents in the area just north of and in between the PCH Bridge piers are higher than one foot per second, there may be some existing scouring at these locations.

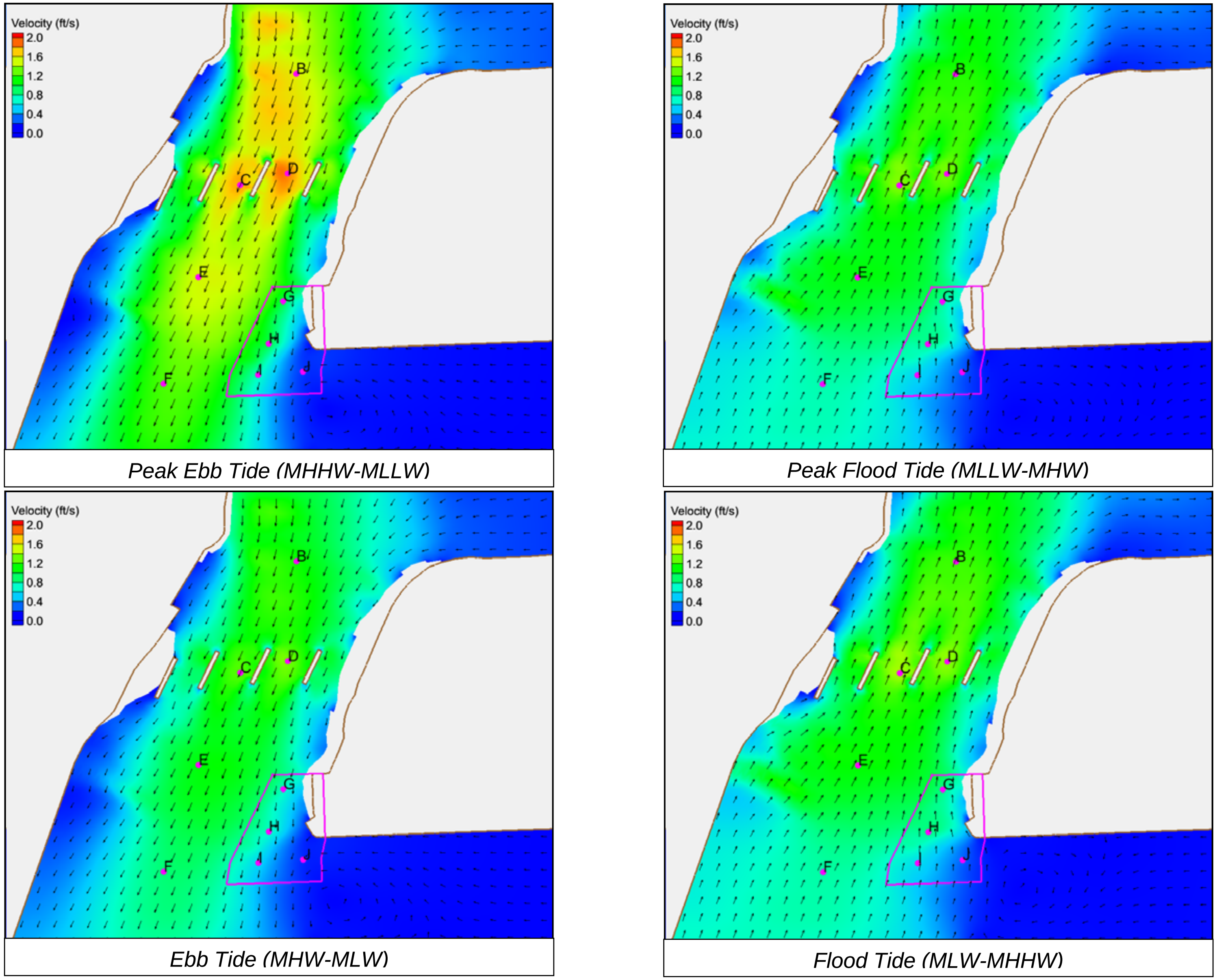


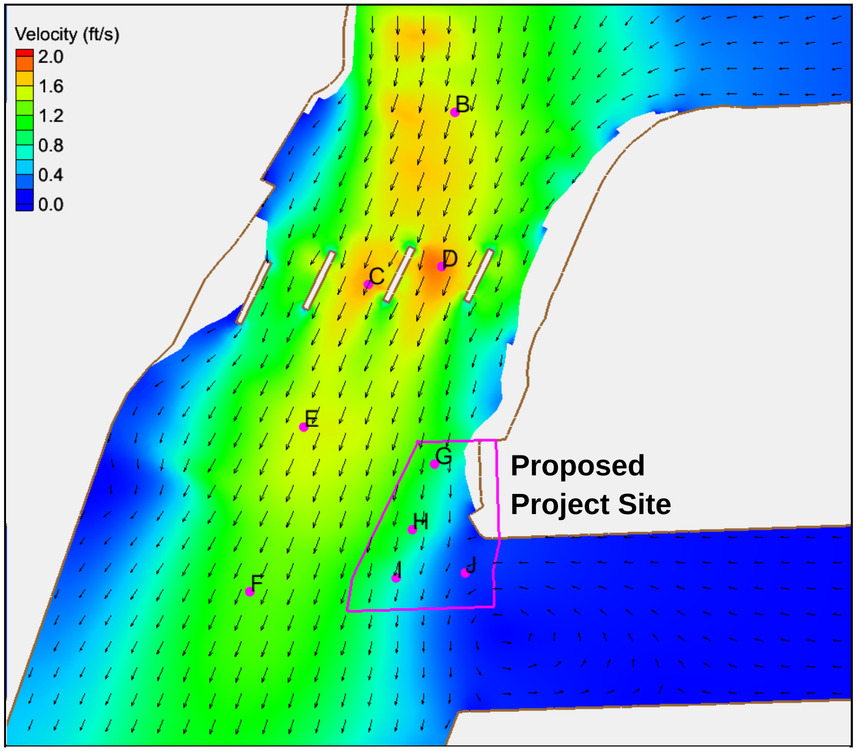
Figure 4.1 Velocity Distributions for Existing Conditions

The model predicted peak ebb currents were compared to the existing bathymetry in the project vicinity in Figure 4.2. As shown in the figure, the highest tidal currents occur just above the PCH Bridge and between the bridge piers as indicated by locations B, C, and D in the left panel. These high velocity areas correspond to the deepest areas along the main channel as indicated by the lowest bathymetry in the right panel. Areas of the lowest bathymetry would reflect areas of greater erosion corresponding to higher velocities. The higher velocities predicted along the western channel edge just north of the PCH Bridge (near B) also correspond to the bathymetric depression. Between the PCH Bridge piers, the model-predicted velocities show higher velocities at location D compared to location C. This comparison is also observed in the bathymetry data, which shows that the area near location D is deeper than the area near location C.

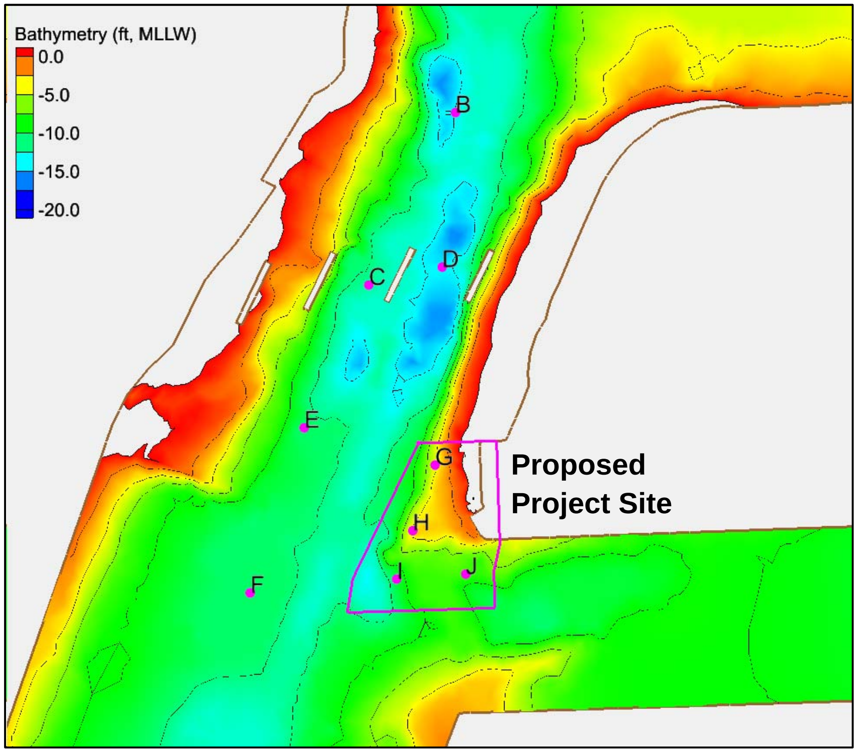
4.2.2 Potential Project Impact

Potential impact of the proposed marina alternatives were assessed by comparing existing and with-project tidal velocities in the project vicinity. These velocity comparisons during peak ebb, peak flood, ebb, and flood conditions are shown in Figures 4.3 to 4.6. As discussed earlier, the highest tidal currents occur during the peak ebb tide (between MHHW and MLLW). Under existing conditions (upper left panel), the highest velocities occur between the PCH Bridge piers in the center portion of the main channel. Velocities are smaller towards the channel sides and the project site.

As shown in Figure 4.3, among all the alternatives, the velocity distribution for Alternative 1 shows the greatest changes from existing conditions. For Alternative 1, the peak ebb velocities show changes between the PCH Bridge piers and areas just downstream of the bridge (locations E and F on the figure) compared to existing conditions. These changes in velocities are caused by the proposed groin wall that blocks the ebb flow; effectively reduces the channel width leading to higher velocities along the main channel. Changes in velocities also occur between the PCH Bridge piers with slightly higher velocities at location C and slightly lower velocities at location D. Under Alternative 1, tidal velocities within the proposed project site would be lower than existing conditions due to the groin wall. The velocity distributions for the other alternatives are similar to existing conditions.

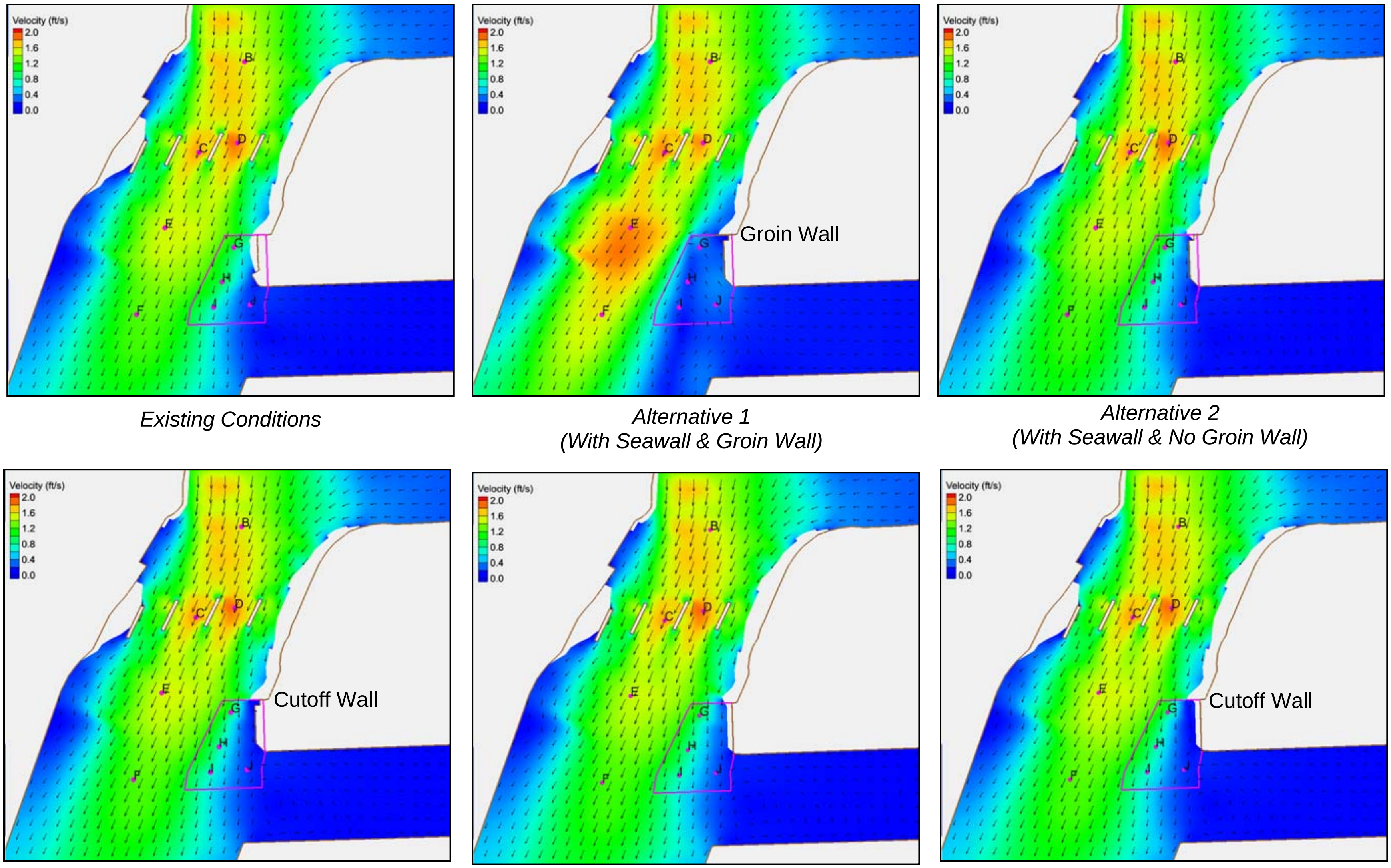


Peak Ebb Tide Velocity Distribution



Existing Bathymetry

Figure 4.2 Peak Ebb Tide Velocity Distribution Comparison with Bathymetry



Existing Conditions

Alternative 1
(With Seawall & Groin Wall)

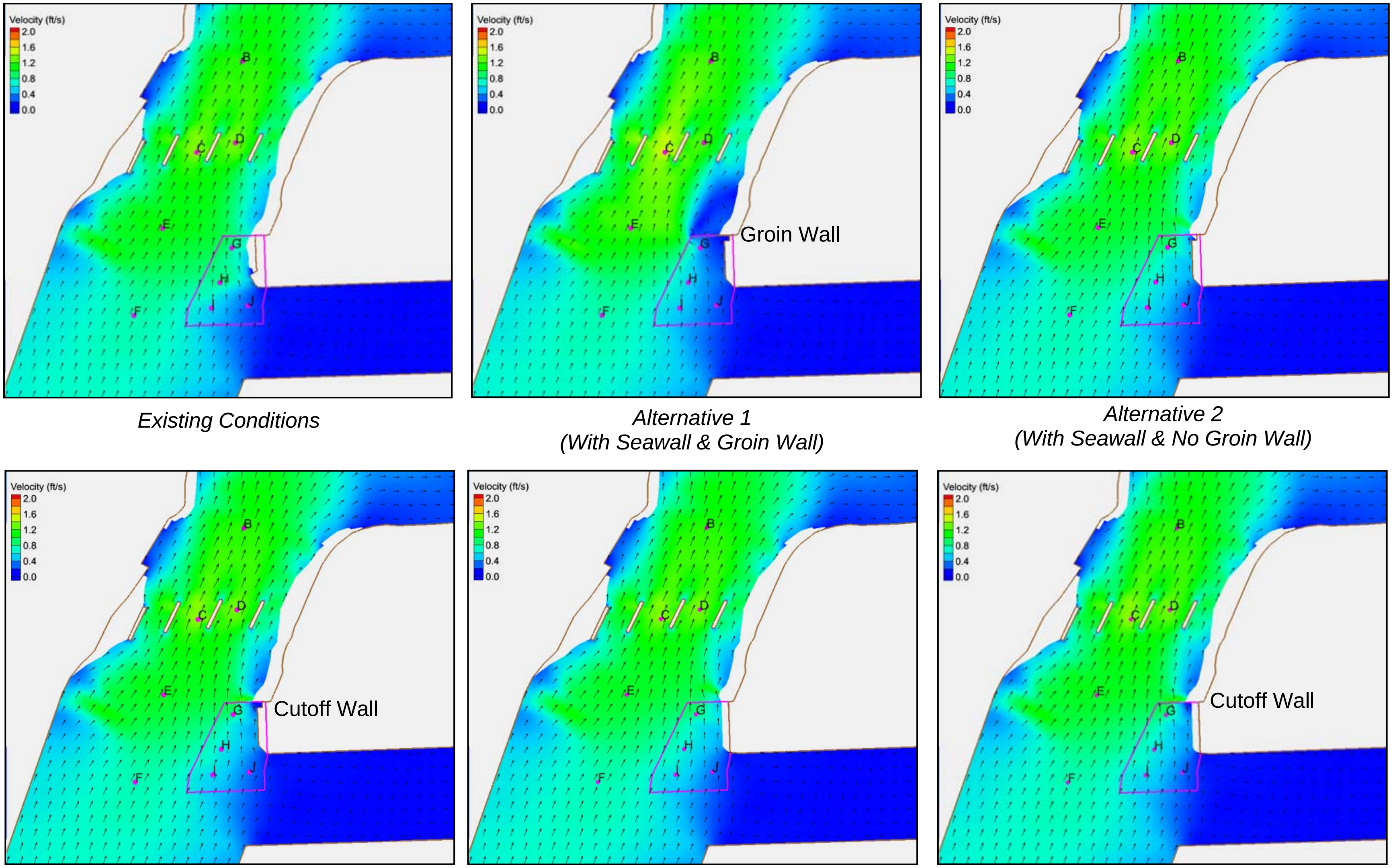
Alternative 2
(With Seawall & No Groin Wall)

Alternative 2 – Opt 1
(With Seawall & Cutoff Wall)

Alternative 3
(Revetment, No Seawall & No Groin Wall)

Alternative 3 – Opt 1
(Revetment, No Seawall with Cutoff Wall)

Figure 4.3 Velocity Distributions during Peak Ebb Tide



Existing Conditions

Alternative 1
(With Seawall & Groin Wall)

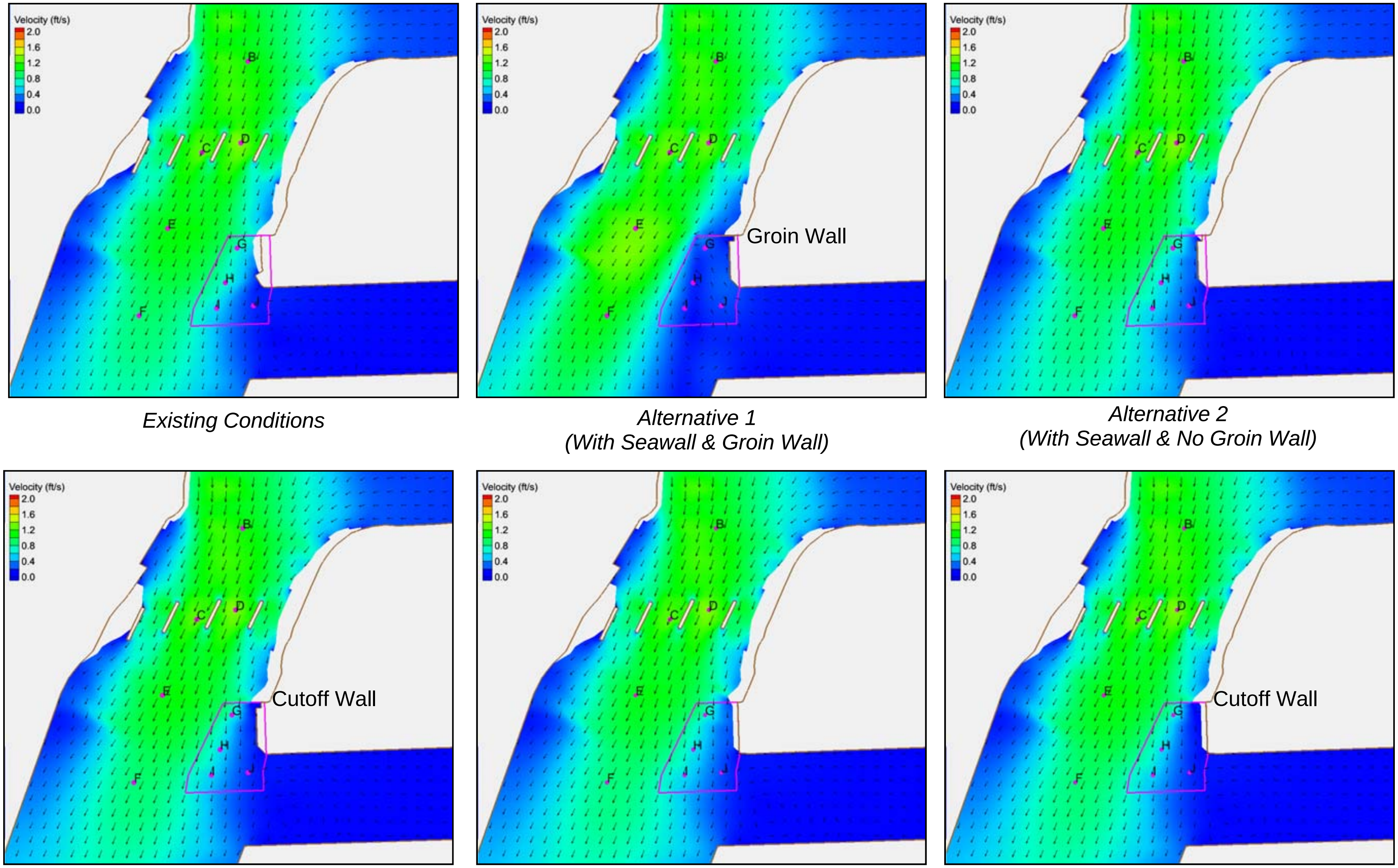
Alternative 2
(With Seawall & No Groin Wall)

Alternative 2 – Opt 1
(With Seawall & Cutoff Wall)

Alternative 3
(Revetment, No Seawall & No Groin Wall)

Alternative 3 – Opt 1
(Revetment, No Seawall with Cutoff Wall)

Figure 4.4 Velocity Distributions during Peak Flood Tide



Existing Conditions

Alternative 1
(With Seawall & Groin Wall)

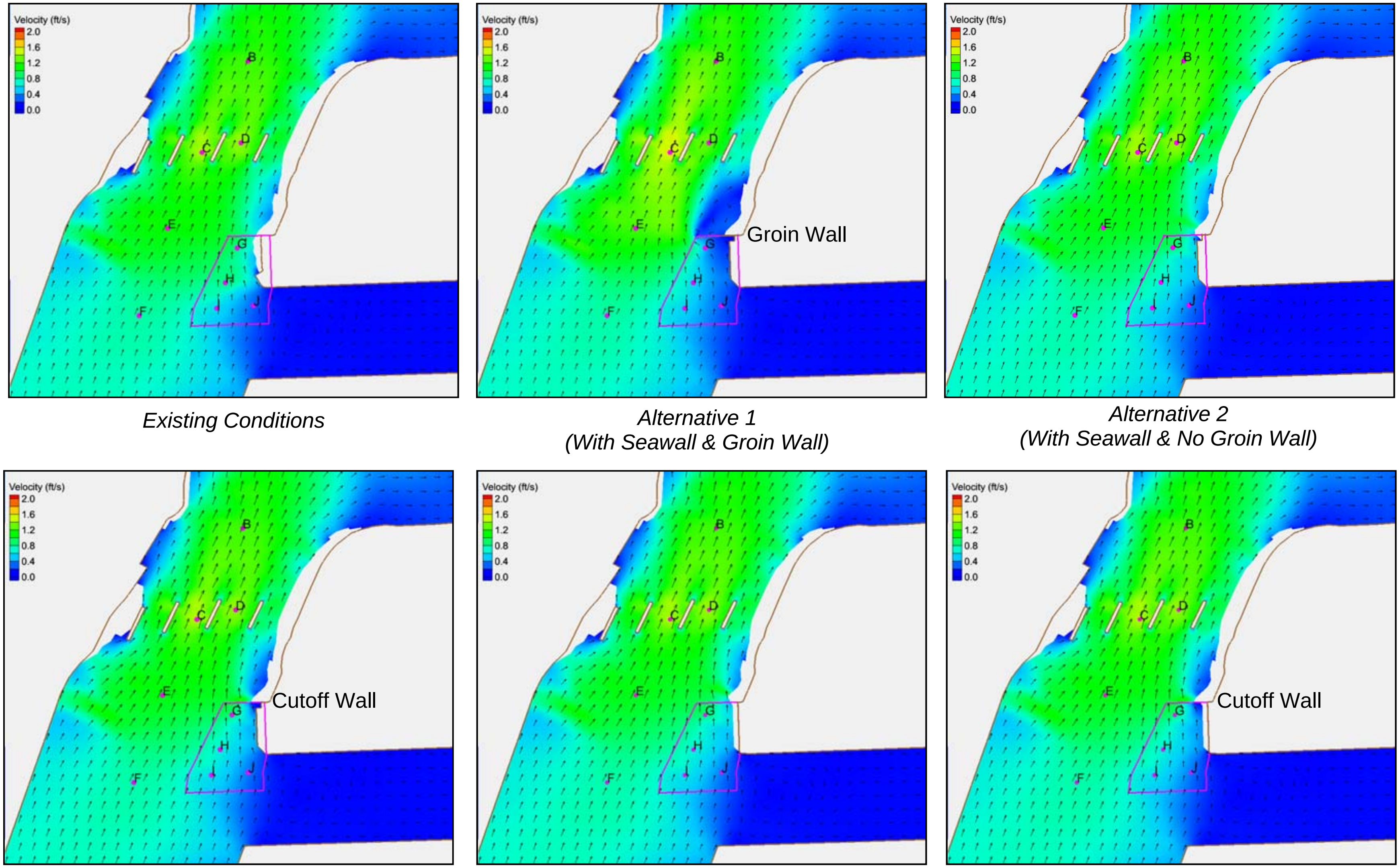
Alternative 2
(With Seawall & No Groin Wall)

Alternative 2 – Opt 1
(With Seawall & Cutoff Wall)

Alternative 3
(Revetment, No Seawall & No Groin Wall)

Alternative 3 – Opt 1
(Revetment, No Seawall with Cutoff Wall)

Figure 4.5 Velocity Distributions during Ebb Tide



Existing Conditions

Alternative 1 (With Seawall & Groin Wall)

Alternative 2 (With Seawall & No Groin Wall)

Alternative 2 – Opt 1 (With Seawall & Cutoff Wall)

Alternative 3 (Revetment, No Seawall & No Groin Wall)

Alternative 3 – Opt 1 (Revetment, No Seawall with Cutoff Wall)

Figure 4.6 Velocity Distributions during Flood Tide

Velocity distributions for the peak flood tide that occurs when the tide rises from MLLW to MHW are compared in Figure 4.4. Existing peak flood tide conditions shows a rather uniform velocity distribution across the main channel, with slightly higher velocities between the PCH Bridge piers. Similar to during the peak ebb condition, velocity distributions for the with-project conditions show the greatest change from existing conditions for Alternative 1, especially in the adjacent area north of the groin wall. During flood tide, the groin wall blocks the flows directed northward into Upper Newport Bay, resulting in lower velocities in the proposed project site and adjacent area. Velocity changes for Alternative 1 are also observed near locations C, D, and E. Velocity distributions for the other four alternatives were similar to existing conditions with some changes shown within the proposed project site and adjacent property.

The velocities during a smaller ebb tide condition (from MHW to MLW) are shown in Figure 4.5. As expected, the velocities are in general lower than velocities during the peak ebb tide (from MHHW to MLLW), but the velocity patterns are similar with higher velocity along the center portion of the main channel. In addition, there are not much change in velocities between the with-project and existing conditions. Among the alternatives, Alternative 1 shows the greatest change from existing conditions, primarily in the proposed project site and adjacent property. Comparisons of velocity distributions during the flood tide (from MLW to MHHW) are shown in Figure 4.6. Velocity distributions generally have similar patterns as velocities during the peak flood tide (from MLLW to MHW). Again, Alternative 1 results in the greatest change from existing conditions along the center portion of the main channel, as well as the proposed project site and adjacent property. Velocity distributions of the remaining alternatives are similar to existing conditions.

Figures 4.3 to 4.6 show snapshots of tidal velocities in the project vicinity for existing and with-project conditions, providing spatial assessment of the potential impact of the proposed marina alternatives to tidal velocities near the project site. To quantify the impact of the proposed alternatives, time series velocities were compared at eleven locations shown in Figure 4.7 in the project vicinity. Locations A and K are located approximately half a mile upstream and a quarter-mile downstream of the proposed project site. Location B is located about 300 feet upstream from the PCH Bridge, while locations C and D are between the center bridge piers. Locations E and F were selected along the main channel adjacent to the project site and downstream of the PCH Bridge. Four locations for the velocity comparisons are located within the proposed project site (G-J). Comparisons of velocity magnitude time series during mean tide conditions at these locations are provided in Figures 4.8 to 4.10.

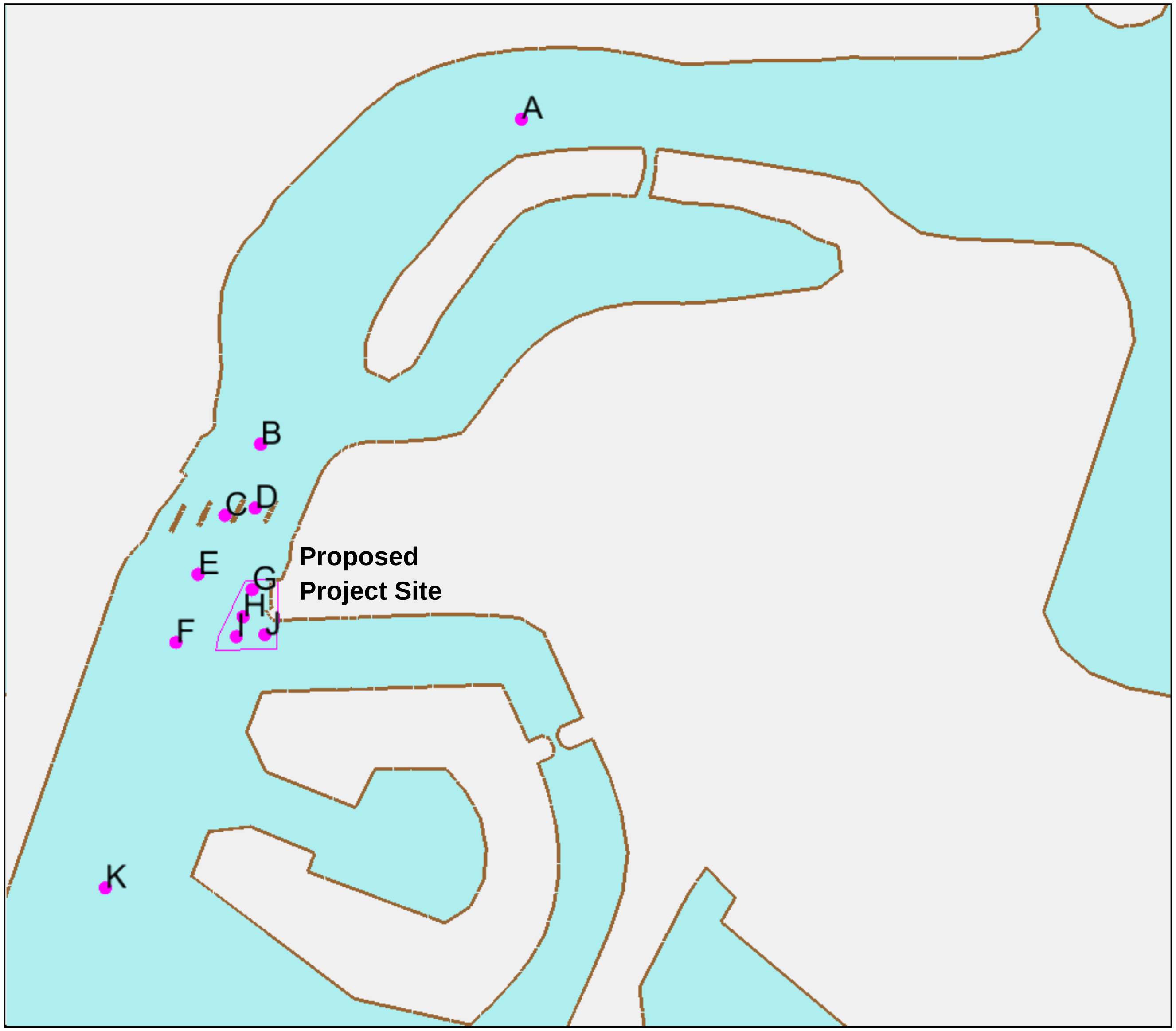


Figure 4.7 Locations for Velocity Comparisons

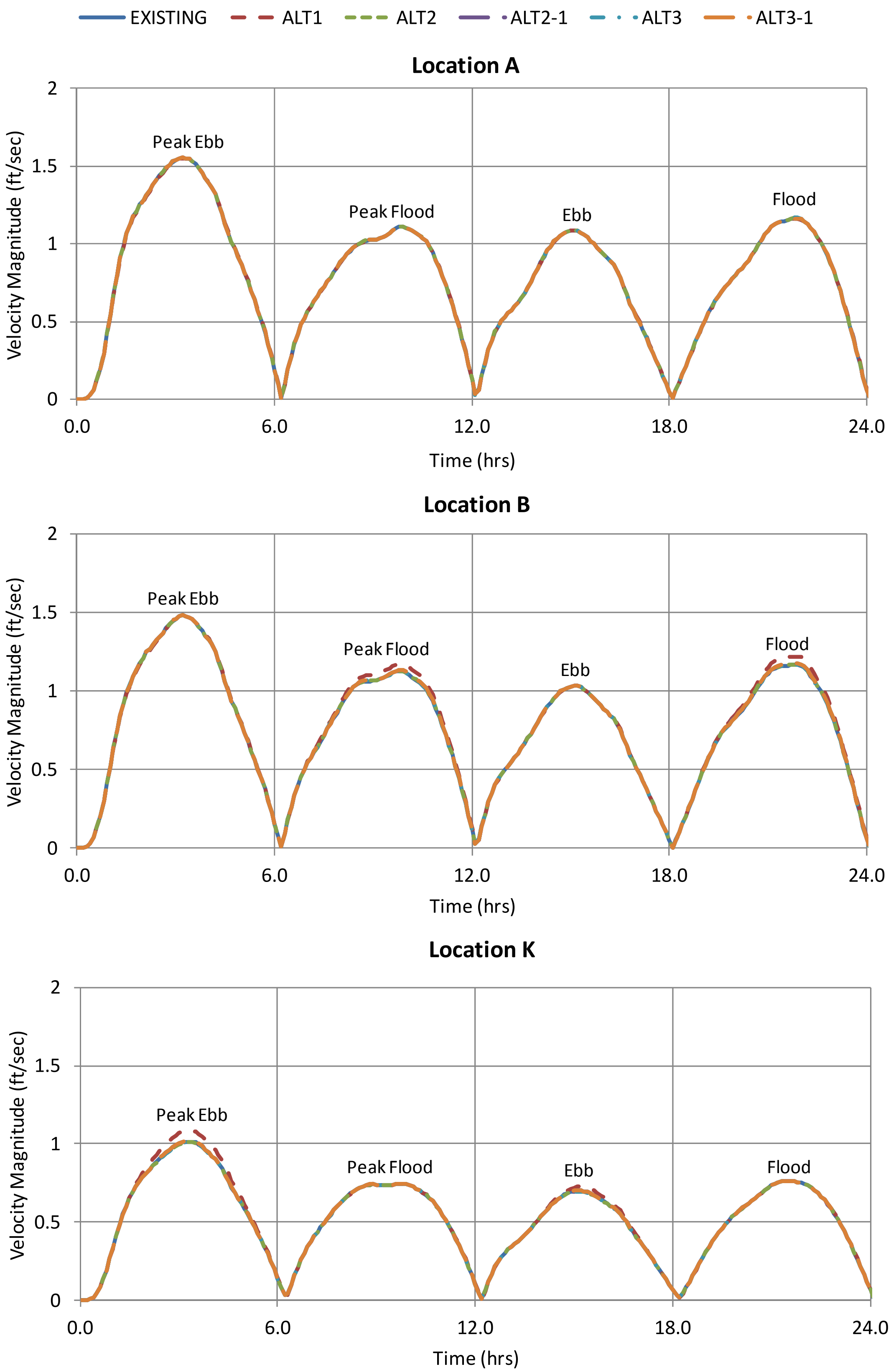


Figure 4.8 Velocity Comparisons at Locations A, B and K

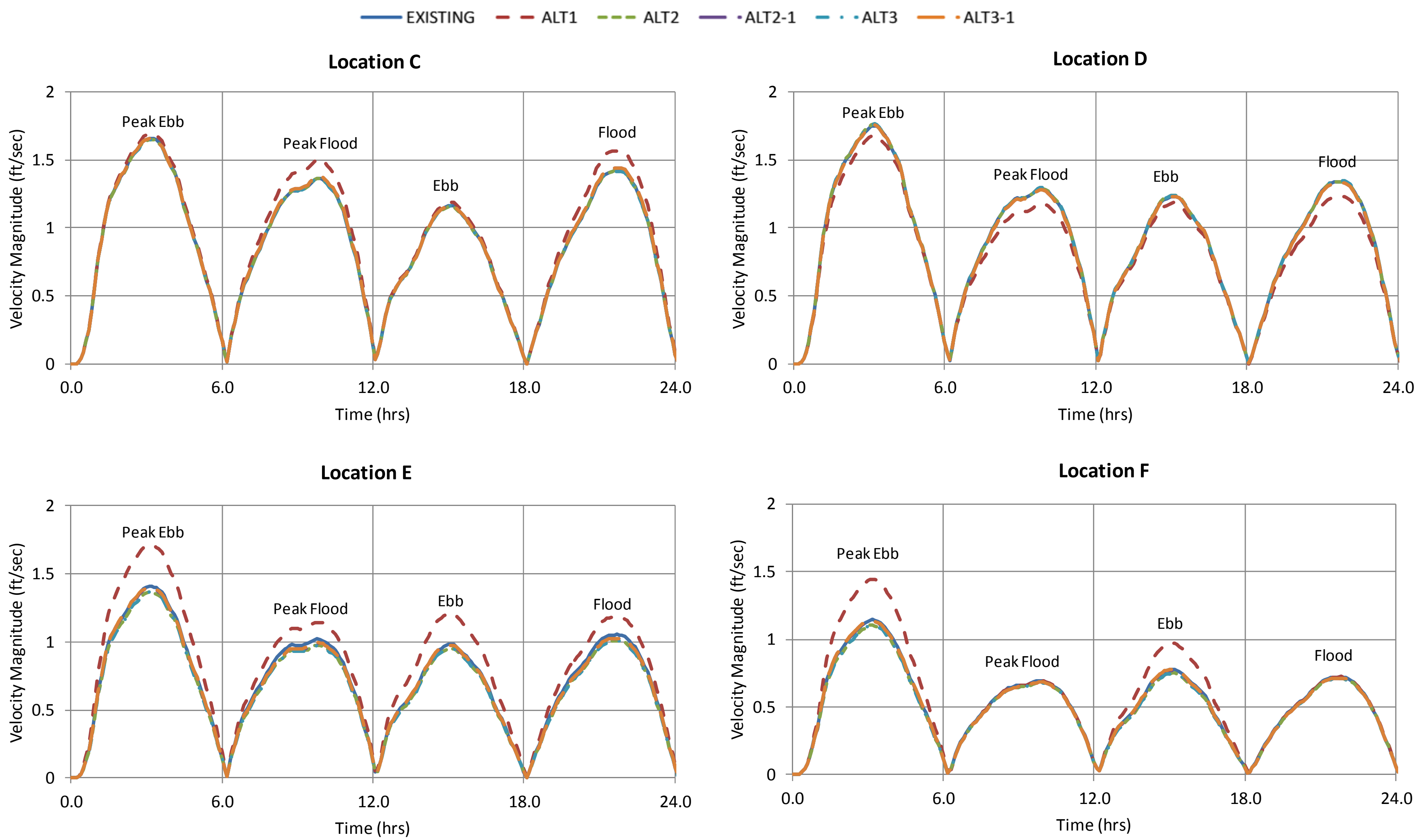


Figure 4.9 Velocity Comparisons at Locations C, D, E and F

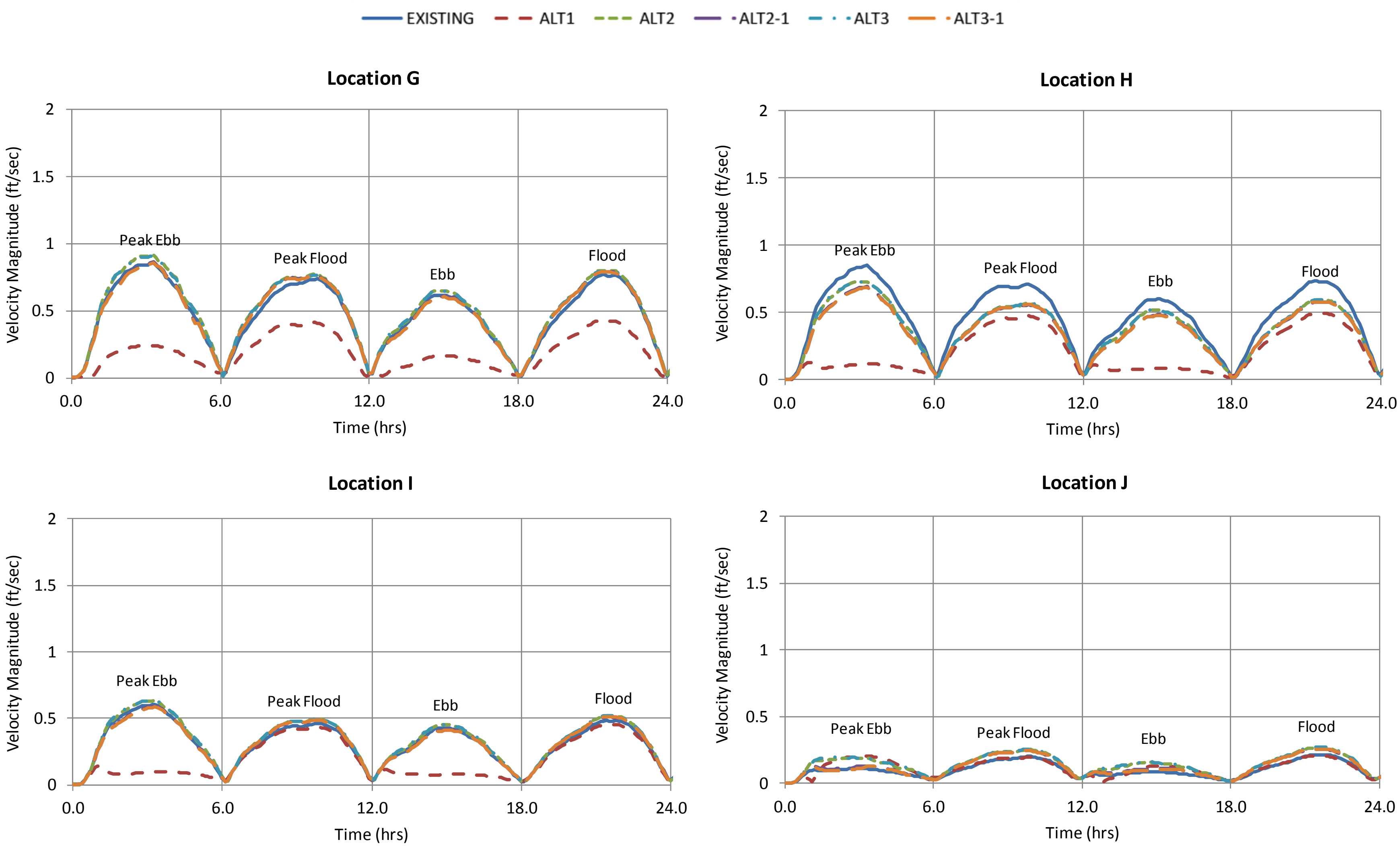


Figure 4.10 Velocity Comparisons at Locations G, H, I and J

Velocities at locations furthest away from the proposed project site (A, B and K) are compared in Figure 4.8. The velocities show the oscillation with the tides with four daily peak velocities during peak ebb, peak flood, ebb and flood tide conditions. Velocities at location A are the same between existing conditions and the with-project conditions. The highest velocity of approximately 1.6 ft/sec occurs during the peak ebb tide is similar to the measured flow at Westcliff Park Station previously discussed in Section 3.1. At location B, Alternative 1 is the only alternative that will slightly change velocities from existing conditions during flood tides. With-project velocities at location K are the same as existing conditions with the exception for Alternative 1 which shows a slight increase during ebbing tides.

Velocities in between the PCH Bridge piers are compared at locations C and D in the upper two panels of Figure 4.9. Existing tidal velocities indicated that velocities differ between the center bridge piers with slightly higher ebb velocities at location D compared to location C. Peak velocities under Alternative 1 show slight increase in velocities at location C and slight decrease in velocities at location D compared to existing conditions. The proposed groin wall for Alternative 1 would have greater impact to tidal currents under PCH Bridge during flood tide than during ebb tide. During flood tide, the groin wall constricts flows moving into Upper Newport Bay and redirect flows through the western portion of the channel, thereby increasing velocities at location C and reducing velocities at location D. At location C, peak velocities would increase slightly during ebb tide but by approximately 10% during flood tides. Peak velocities at locations C and D for the other proposed project alternatives were essentially the same as existing conditions.

Velocity comparisons along the main channel at locations E and F between existing and with-project conditions are shown in the lower two panels of Figure 4.9. Among all the proposed alternatives, Alternative 1 would result in the greatest changes in velocities at these two locations compared to existing conditions. At location E, for Alternative 1, the peak velocities during ebb tide conditions would increase by 21% (from 1.4 ft/sec to 1.7 ft/sec) and 23% (0.98 to 1.21 ft/sec). Peak velocities would increase by 12% during both flood tide conditions. Changes in velocities for the other alternatives indicate a reduction in peak velocities ranging from 1% to 5%. At location F, no changes to existing velocities were observed for flood tide conditions. There would be Increase in ebb tide velocities for Alternative 1, while there would be small reductions in ebb tide velocities for all the other alternatives. The peak ebb velocities for Alternative 1 would be about 25% higher than those under existing conditions.

Time series velocity comparisons within the proposed project site are provided in Figure 4.10 for locations G, H, I and J. Overall, tidal currents within the project site would be smaller than tidal current within the main channel and be less than 1 ft/sec for existing and with-project conditions. Tidal velocities for Alternative 1 were typically less than existing conditions due to the blocking of flows by the proposed groin wall. In general, tidal velocities for the other alternatives would be similar to existing conditions. At location G, peak tidal velocities were within 5% of existing conditions. Reductions in tidal velocities for all alternatives were observed at location H. Tidal velocities at locations I and J were the lowest compared to the other locations. Tidal currents for any of the alternatives did not indicate a large change in tidal velocities compared to existing conditions.

To assess potential impacts to the property just north of the proposed marina extension, velocities in the adjacent area were compared between existing and with-project conditions. Impacts to velocities in the adjacent area were evaluated at 10 locations shown in Figure 4.11. These locations were selected to represent different water depths. Locations 1, 2, and 3 are located near the edge of the sub-tidal area, while locations 4 and 5 are located along the edge of the intertidal area. The remaining locations are all within the intertidal area; hence these areas are intermittently dry during low tide.

Velocity comparisons in the adjacent area are provided in Figures 4.12 and 4.13. For locations within the intertidal area (locations 6 through 10), zero velocity indicates when the locations become dry during low tide. Overall, tidal currents in the adjacent area are small, with the peak less than about 0.7 ft/sec. In general, velocities within the adjacent area would be reduced under Alternative 1, especially during flood tide conditions when the groin wall blocks off tidal currents moving toward Upper Newport Bay. Alternatives 2 and 3 would result in the greatest increase in tidal currents in the intertidal portion of the adjacent area, as indicated by the dashed green and light blue lines in the figures. As expected, the cutoff wall (Option 1 for Alternatives 2 and 3) would result in smaller changes to tidal currents in the adjacent property compared to the groin wall.

4.2.3 Dock Impact

The proposed marina expansion would accommodate both large and small boats ranging in size from 20-ft to 70-ft boats. Under each proposed alternative, two dock layouts are being considered. Dock layout A would accommodate larger boat sizes with the boat slips aligned perpendicular to tidal currents. Dock layout B would accommodate a greater number of mid-sized boats (40 to 60-ft) with the boat slips aligned parallel to tidal currents. With the marina fully occupied, the boat draft would block a substantial portion of tidal flows especially during low tides. An assessment was conducted to determine potential impacts from boats docked in the proposed marina. These potential impacts were evaluated assuming the boats would completely block the tidal flows, thus representing the greatest impacts attributed to the proposed marina extension.

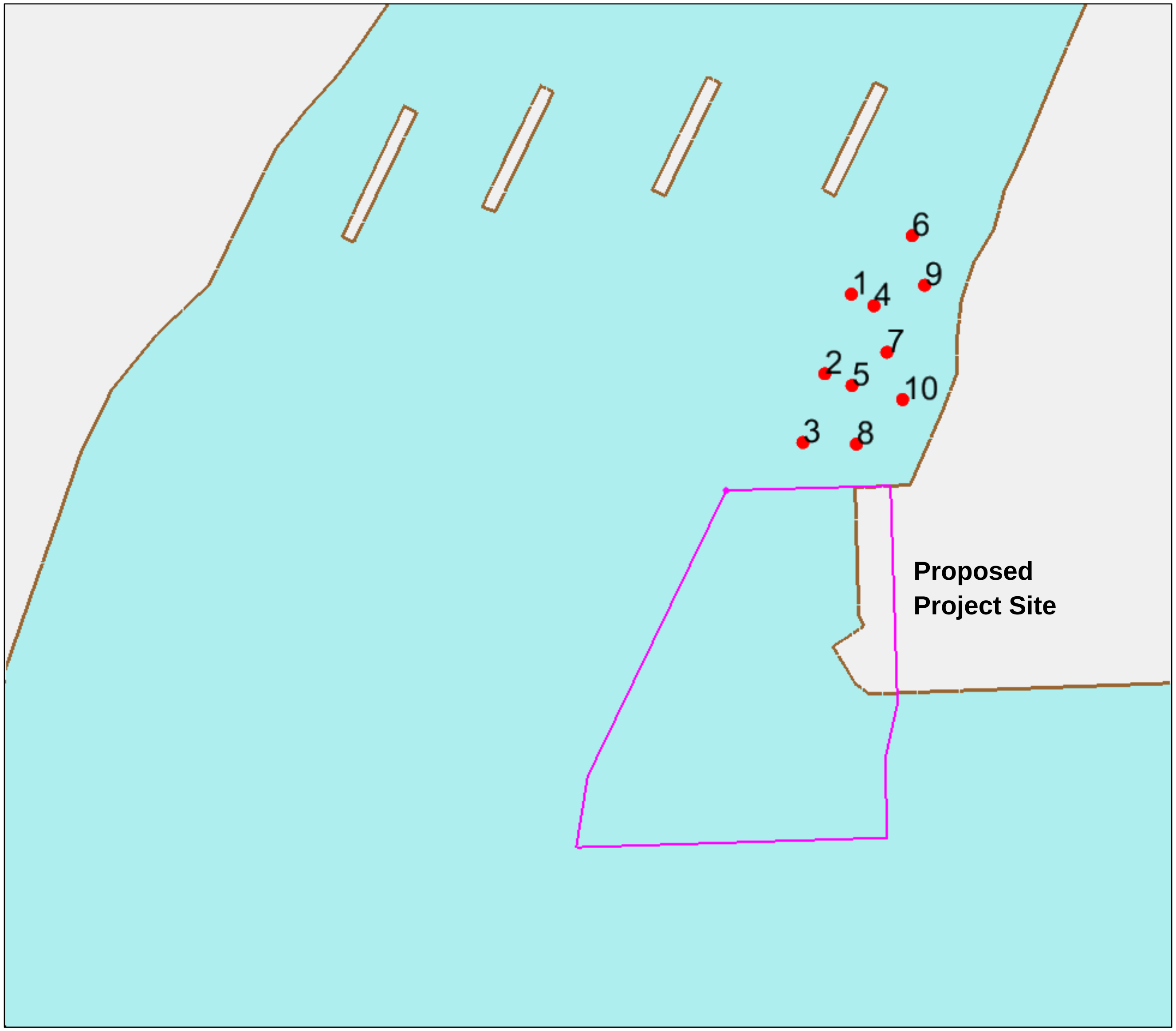


Figure 4.11 Locations for Velocity Comparisons in Adjacent Area

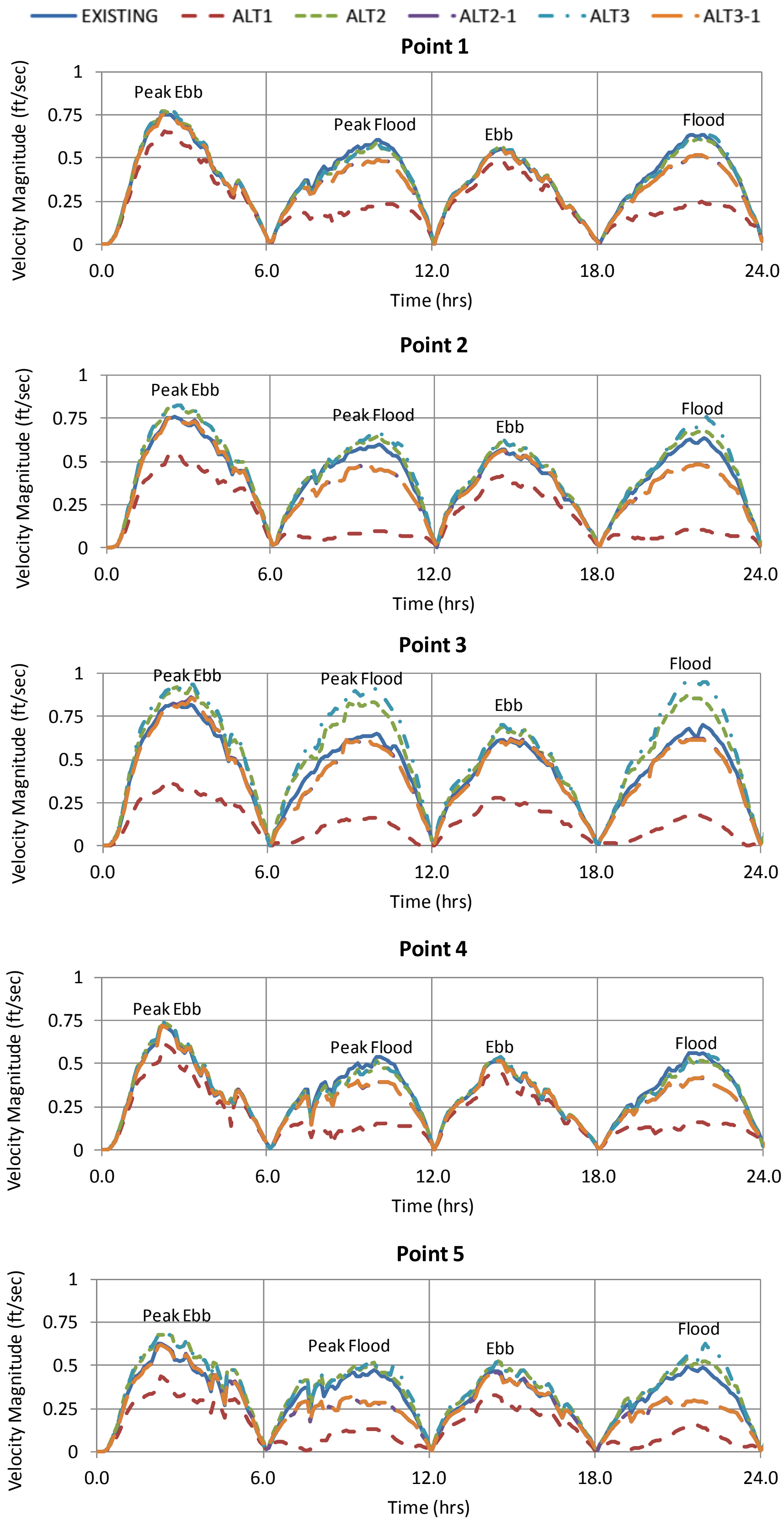


Figure 4.12 Velocity Comparisons in Adjacent Area at Points 1 to 5

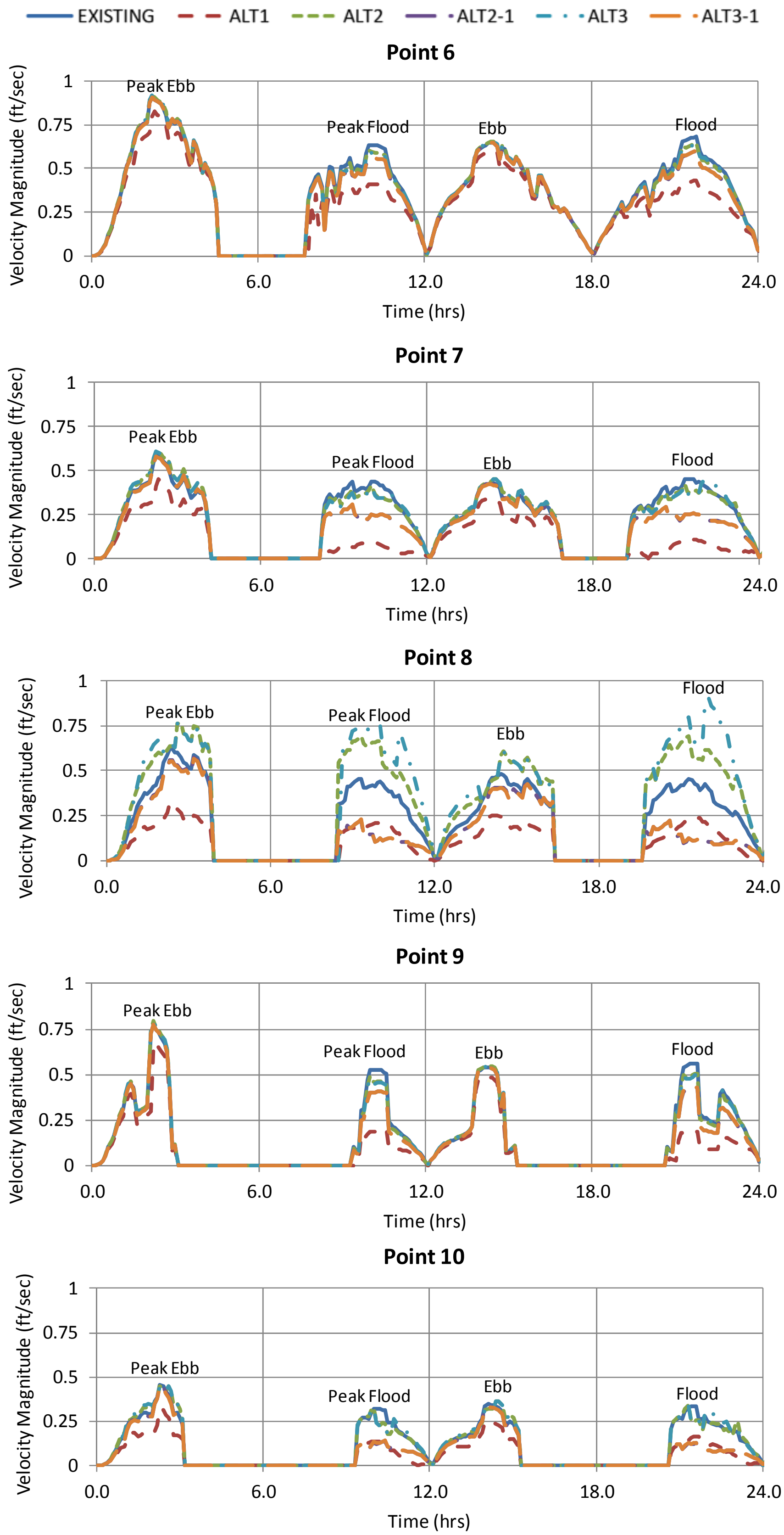


Figure 4.13 Velocity Comparisons in Adjacent Area at Points 6 to 10

Velocity distributions with blocking of the dock area by boats for the proposed project alternatives are compared with existing conditions in Figures 4.14 to 4.17. The white area shown in the figures indicates the location of the dock area. Comparisons during the peak ebb tide are shown in Figure 4.14. In general, the blocking of tidal flows by the dock area would result in increases in tidal velocities along the main channel near locations E and F. Velocity distributions during peak flood tide conditions are illustrated in Figure 4.15. For all project alternatives, the blocking of tidal flows by the dock area effectively reduces the width of the main channel leading to increase in velocities along the center portion of the channel as illustrated near locations E and F. In addition, the blocking by the docking area would result in higher velocities between the PCH Bridge piers at location C and a decrease in velocities at location D. As shown in Figures 4.16 and 4.17, velocity distributions for typical ebb and flood tide conditions show similar changes as those during peak ebb and peak flood tide conditions. Overall, boats docked within the proposed marina would increase velocities along the main channel of the bay and beneath the PCH Bridge.

Velocity time series with blocking of the dock area for the proposed project alternatives are compared to existing conditions in Figures 4.18 to 4.19. Changes in velocities at locations upstream (locations A, and B), and downstream (location K) of the PCH Bridge are provided in Figure 4.18. Essentially, upstream of the PCH Bridge, no changes were determined at location A while small increases in velocities were observed during flood tide at location B. Downstream of the PCH Bridge, small increases in velocities were found at location K during ebb tide.

Impacts to velocities at the PCH Bridge (locations C and D) and along the main channel (locations E and F) are shown in Figure 4.19. As shown in the bottom panels of the figure, all proposed alternatives would increase tidal currents along the main channel compared to existing conditions. At location E, peak tidal velocities would be increased by 24% from existing conditions for Alternative 1, and about 8% during ebb tide and 16% during flood tide for Alternative 2. Similar increases were observed for Alternative 2 – Option 1 and Alternative 3 with increases of 9% and 17% during ebb and flood tide. Tidal velocity increases for Alternative 3 – Option 1 were estimated to be 8% and 10% during ebb tide and 17% during flood tide. Higher increases in velocities were observed at location F. Peak velocities at location F were shown to increase by 26% during ebb tide and 22% during flood tide for Alternative 1, and between 19% and 22% for other alternatives.

For locations C and D between the PCH Bridge piers, all proposed alternatives show an increase in velocities at location C and decrease in velocities at location D. Changes under Alternative 1 are the highest with an increase of peak velocities by about 8% during flood tides and 2% during ebb tides. For the other alternatives, peak velocities during flood tides increase by about 3%, while increases during ebb tides are less than 1%.

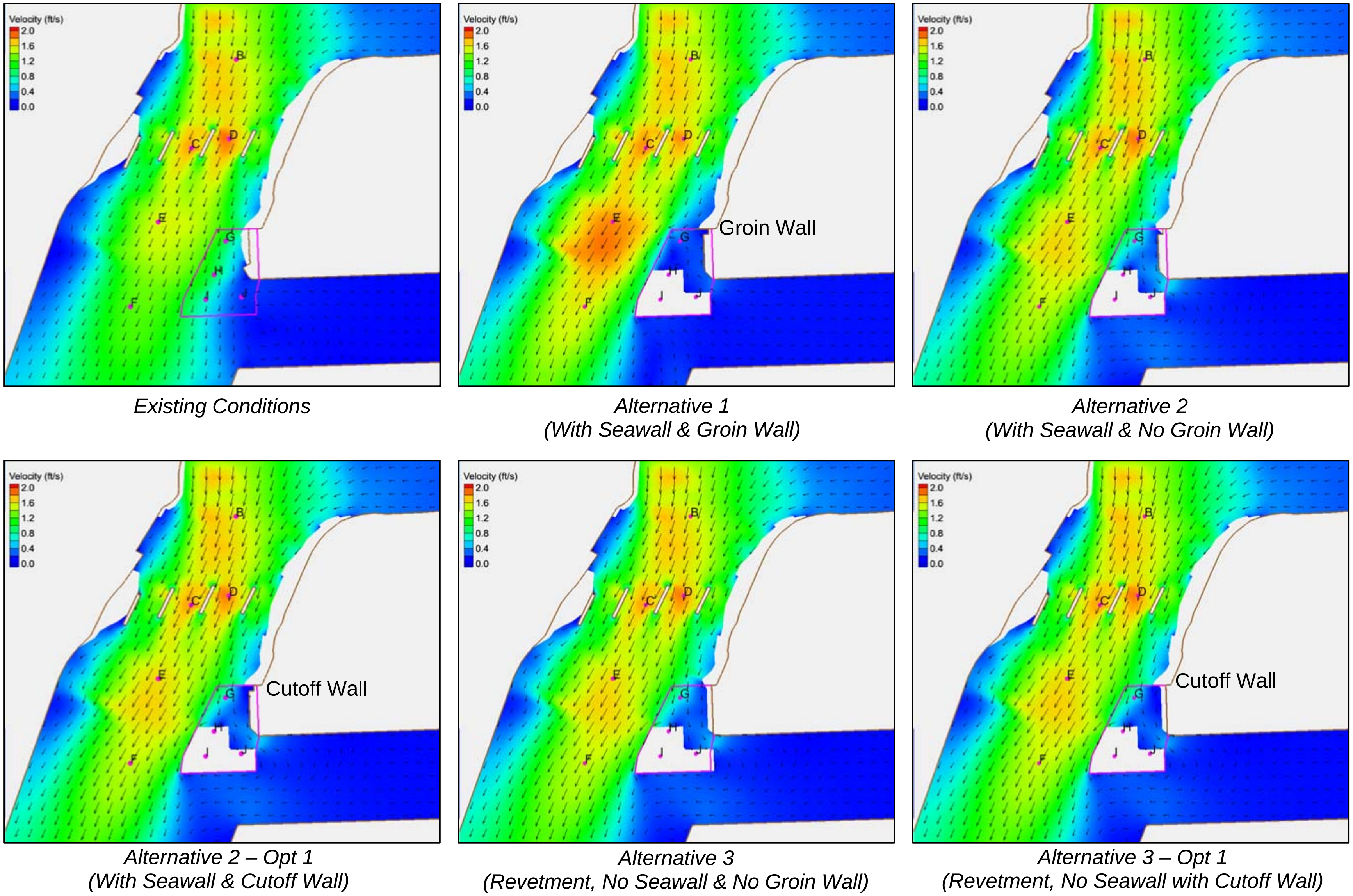


Figure 4.14 Velocity Distributions with Boat Blocking during Peak Ebb Tide

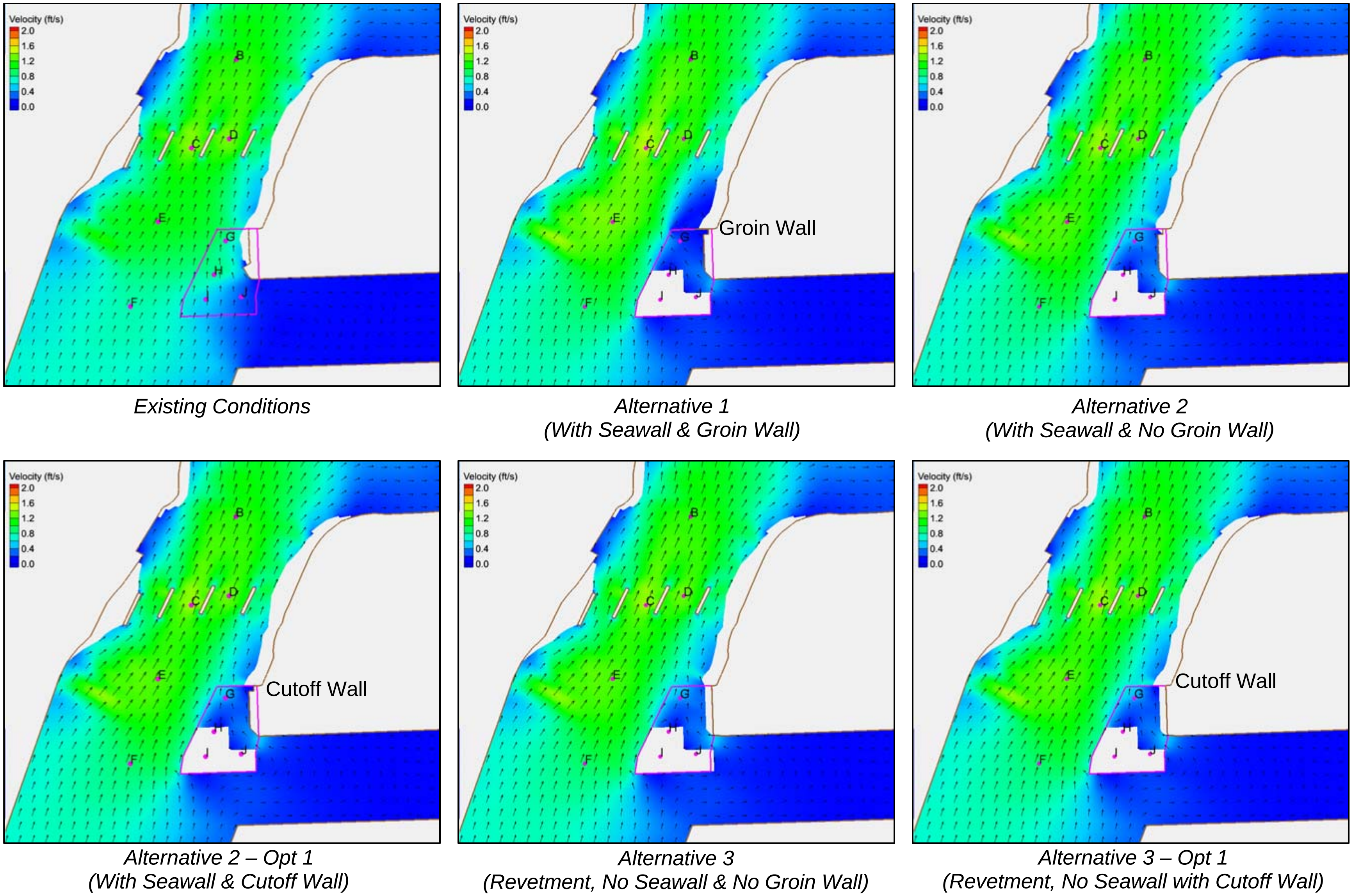
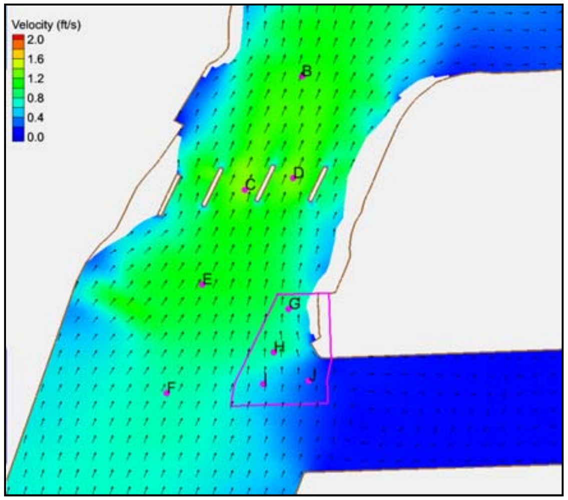
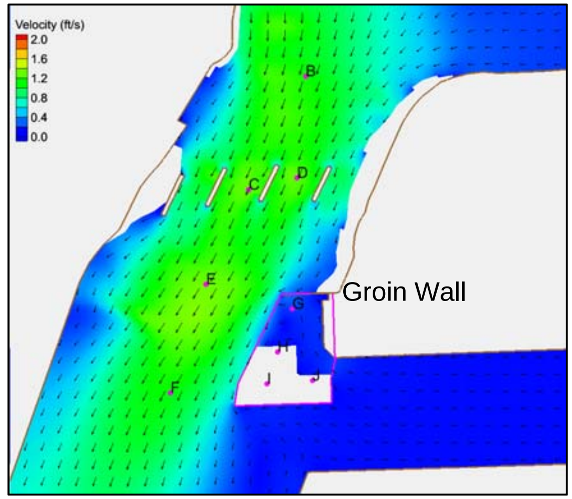


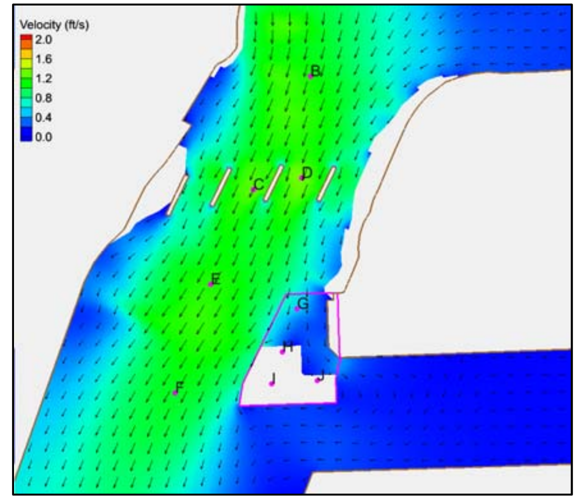
Figure 4.15 Velocity Distributions with Boat Blocking during Peak Flood Tide



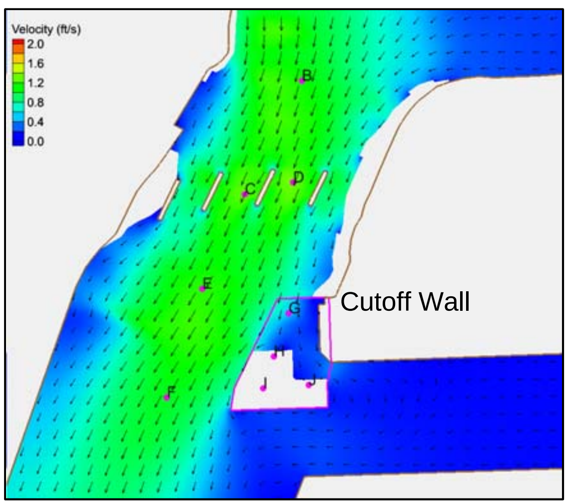
Existing Conditions



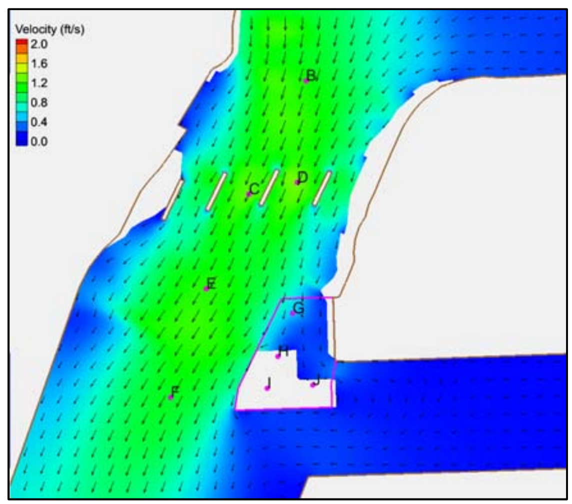
Alternative 1
(With Seawall & Groin Wall)



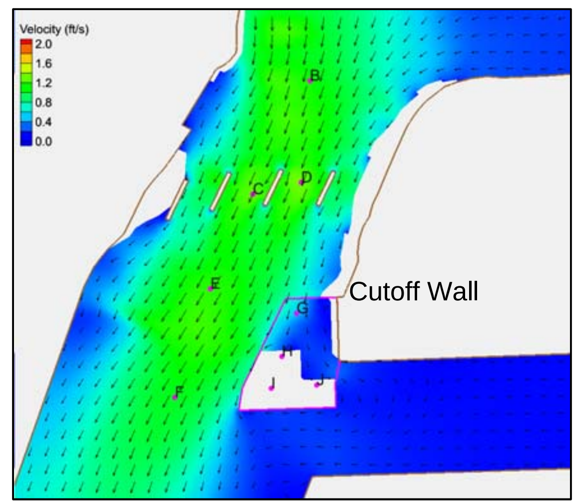
Alternative 2
(With Seawall & No Groin Wall)



Alternative 2 – Opt 1
(With Seawall & Cutoff Wall)

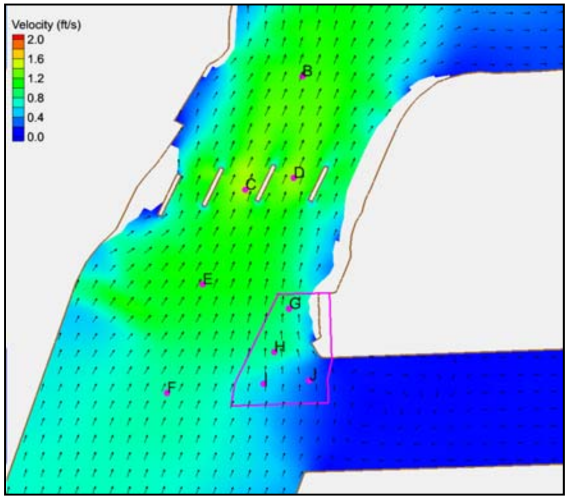


Alternative 3
(Revetment, No Seawall & No Groin Wall)

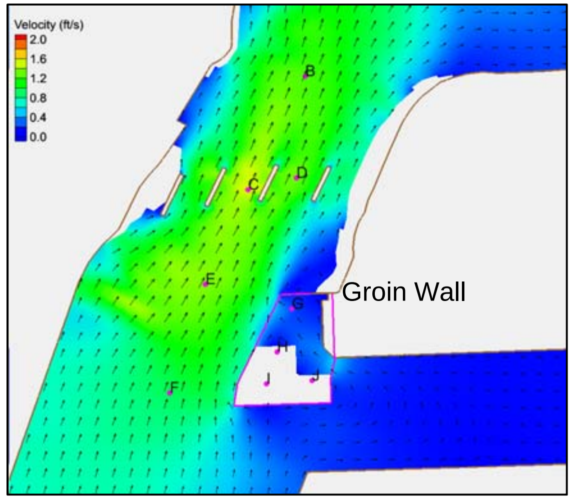


Alternative 3 – Opt 1
(Revetment, No Seawall with Cutoff Wall)

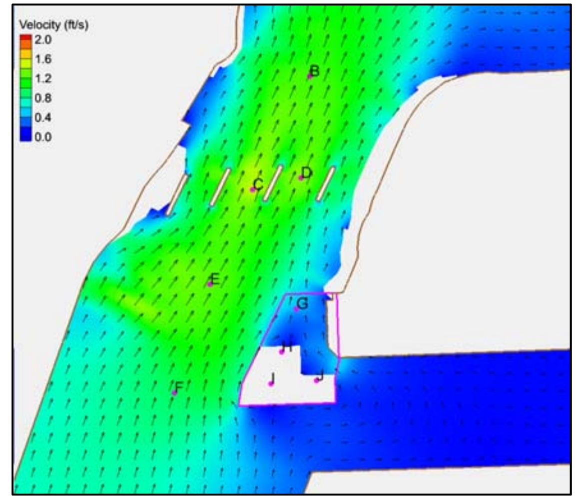
Figure 4.16 Velocity Distributions with Boat Blocking during Ebb Tide



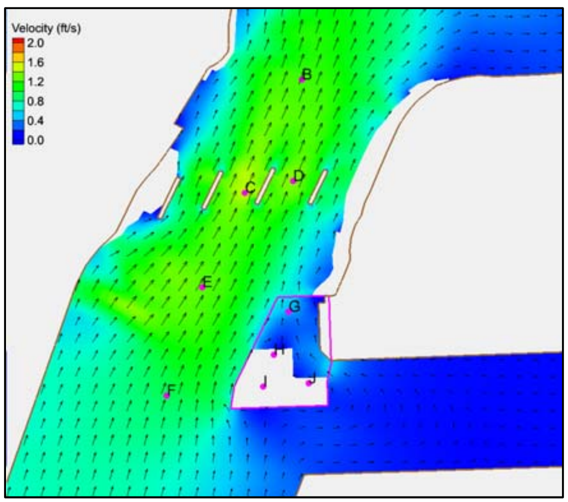
Existing Conditions



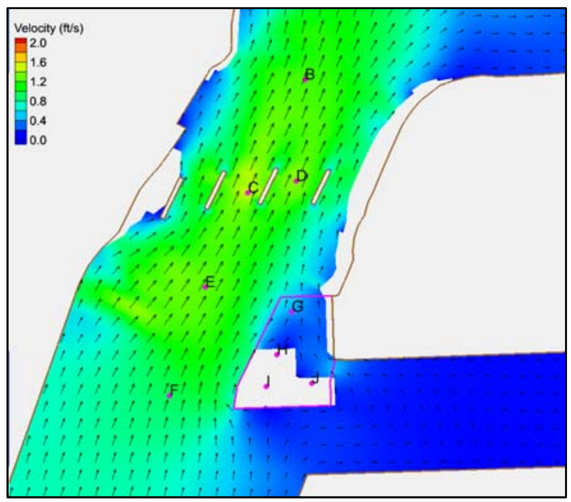
Alternative 1
(With Seawall & Groin Wall)



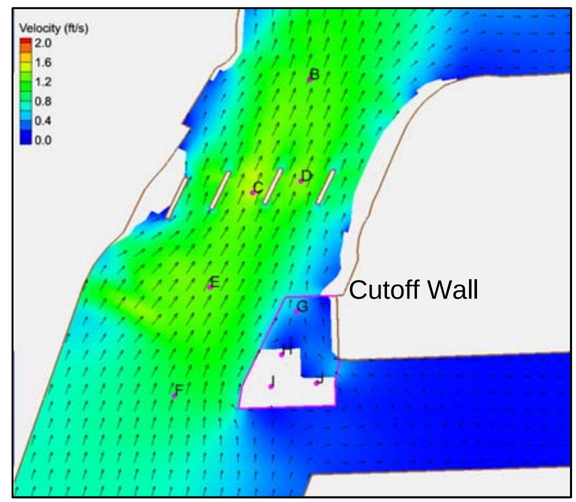
Alternative 2
(With Seawall & No Groin Wall)



Alternative 2 – Opt 1
(With Seawall & Cutoff Wall)



Alternative 3
(Revetment, No Seawall & No Groin Wall)



Alternative 3 – Opt 1
(Revetment, No Seawall with Cutoff Wall)

Figure 4.17 Velocity Distributions with Boat Blocking during Flood Tide

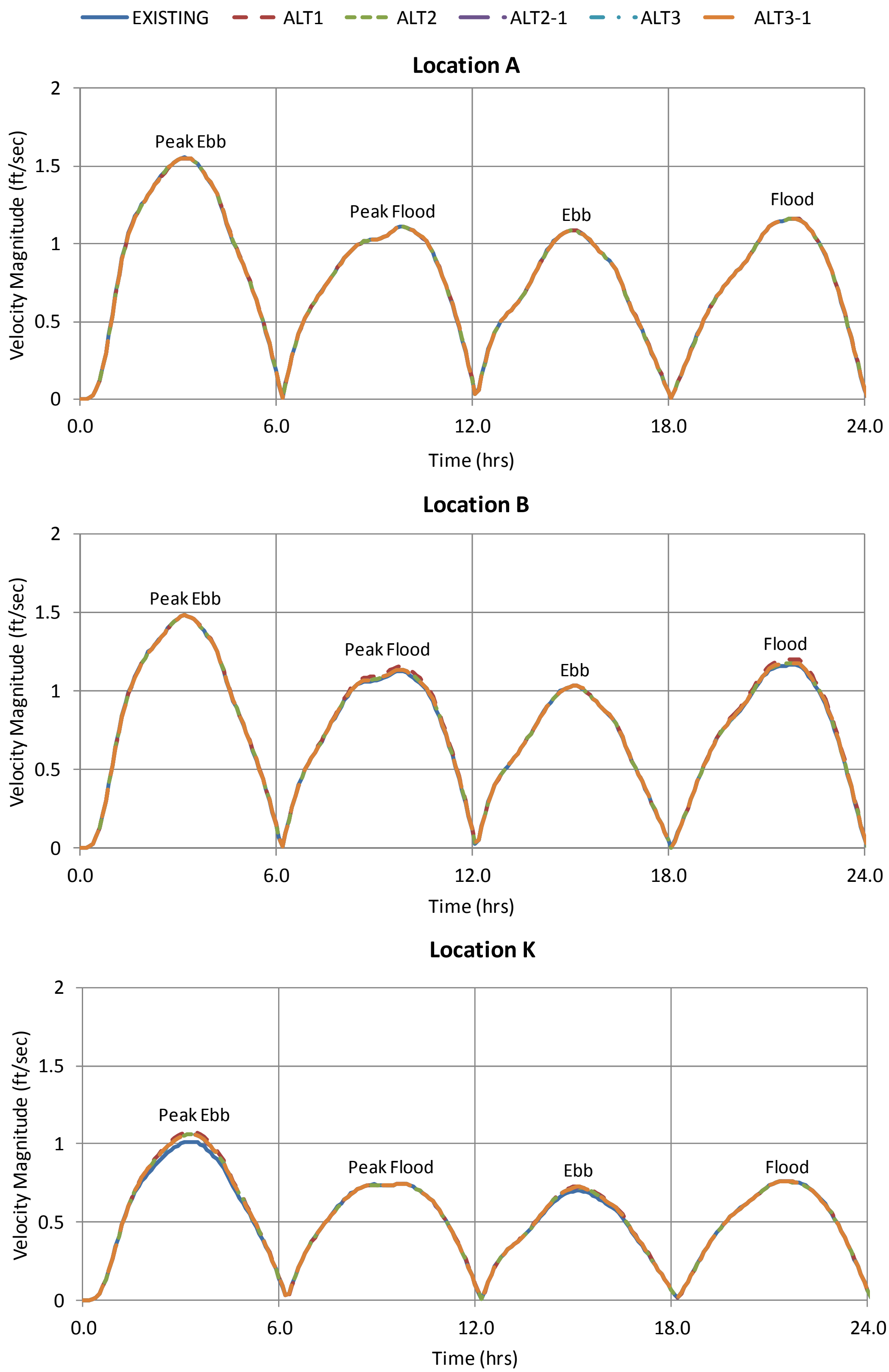


Figure 4.18 Velocity Comparisons with Boat Blocking at Locations A, B and K

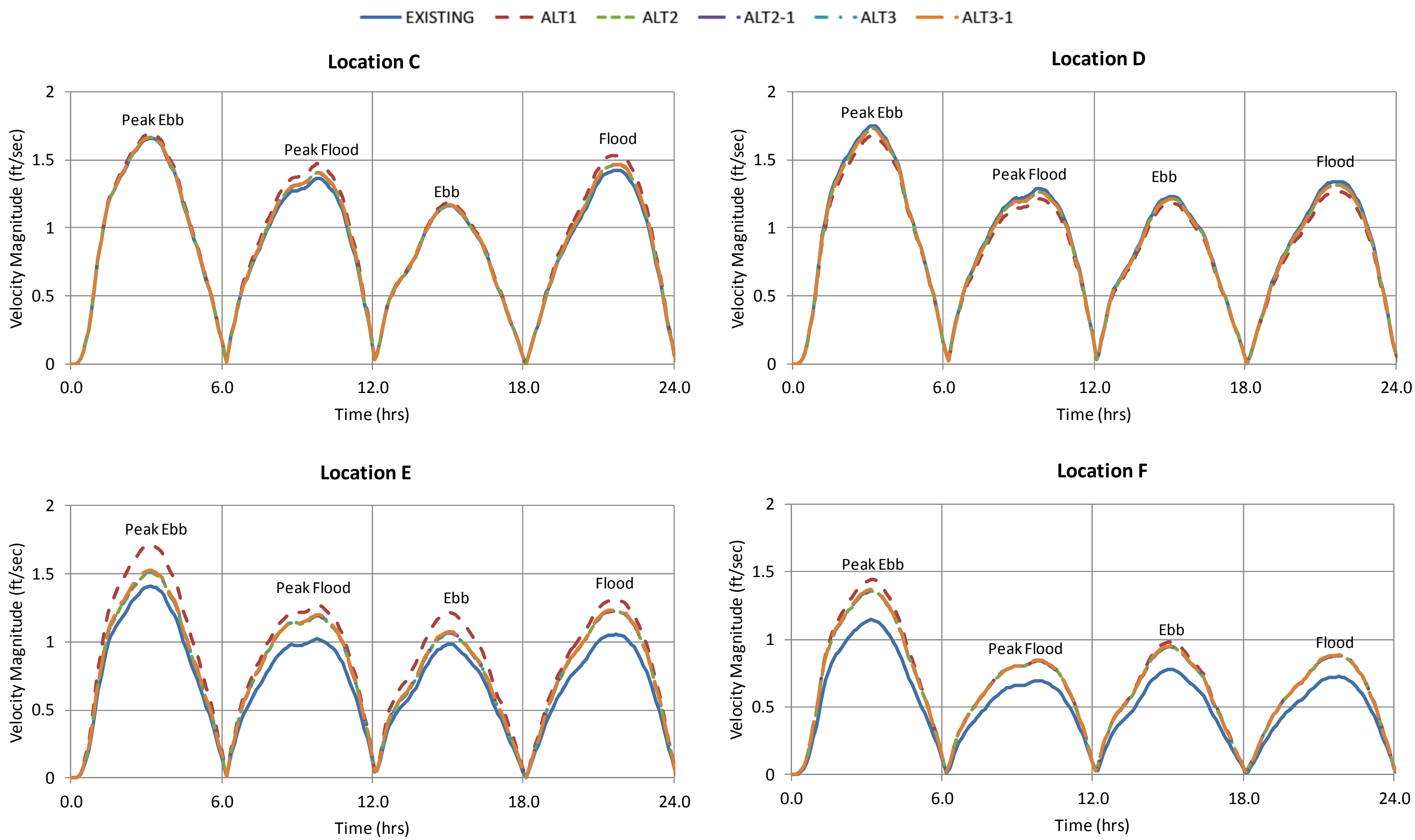


Figure 4.19 Velocity Comparisons with Boat Blocking at Locations C, D, E and F

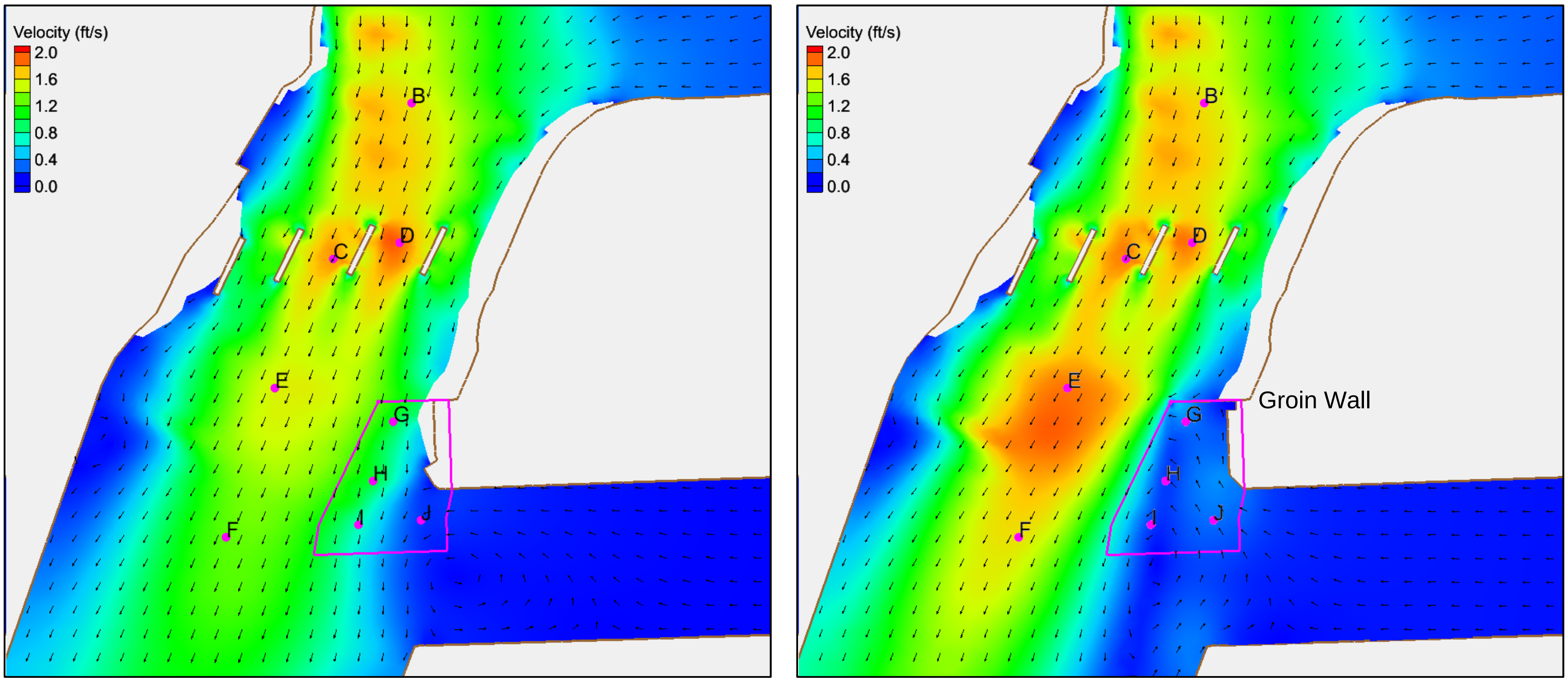
4.2.4 Spring Tide

Potential impact of the proposed marina alternatives to tidal currents in the project vicinity were extensively evaluated based on mean tide conditions, which represent long term average conditions. To illustrate potential impact to tidal velocities during spring tide (larger tidal range compared to mean tide), velocities during a spring tide were evaluated for existing conditions and Alternative 1. Potential proposed project impacts were only determined for Alternative 1 since this alternative shows the greatest impacts to tidal currents with mean tide conditions. The spring tide time series used for this study shown in Figure 3.10 has a tidal range of 6.81 ft compared to 5.41 ft for the mean tide.

Velocity distributions for existing conditions and Alternative 1 during the spring ebb and spring flood tides are provided in Figures 4.20 and 4.21. The changes in velocities between existing conditions and Alternative 1 are similar to the changes observed for mean tide conditions, with redistribution of velocities between the PCH Bridge piers at locations C and D and increase in velocities along the main channel at locations E and F. Velocity time series are compared between existing conditions and Alternative 1 in Figures 4.22 and 4.23. As shown in Figure 4.22, Alternative 1 would only result in minor velocity increases at locations upstream (location A and B) and downstream (location K) of the PCH Bridge. The largest velocity increases were found along the main channel (locations E and F), shown in bottom panels of Figure 4.23. As expected, the spring peak ebb velocities would be higher than the mean peak ebb velocities for both existing and with Alternative 1 conditions. Similar velocity changes beneath the PCH Bridge were found under the spring tide condition compared with the mean tide conditions – increase in tidal velocities were found at location C and decrease in velocities were found at location D.

4.3 Stormwater Runoff

Changes to velocities under proposed project conditions were also assessed for an extreme flood event when hydrodynamic conditions are dominated by fresh water flows. A 100-year flood event capturing all major stormwater discharges into Upper Newport Bay was simulated under existing and with Alternative 1 conditions. The largest discharge into Upper Newport Bay is the San Diego Creek, which accounts for approximately 80% of flows entering the upper portion of the bay. The San Diego Creek discharge location and other major stormwater discharge locations to Upper Newport Bay are illustrated in Figure 4.24. The Santa Ana Delhi Channel, Santa Isabella Channel, and Big Canyon Wash discharge into the middle portion of Upper Newport Bay; along with two other storm drains at the lower end - East Costa Mesa Channel and a storm drain from Dover Drive. For the flood flows, a 100-year flood hydrograph was estimated for San Diego Creek and the corresponding hydrographs for the other discharges were scaled based on their drainage areas. The 100-year flood hydrograph for San Diego Creek is shown in Figure 4.25 along with the timing of



Existing Conditions

Alternative 1 (With Seawall & Groin Wall)

Figure 4.20 Velocity Distributions during Spring Ebb Tide

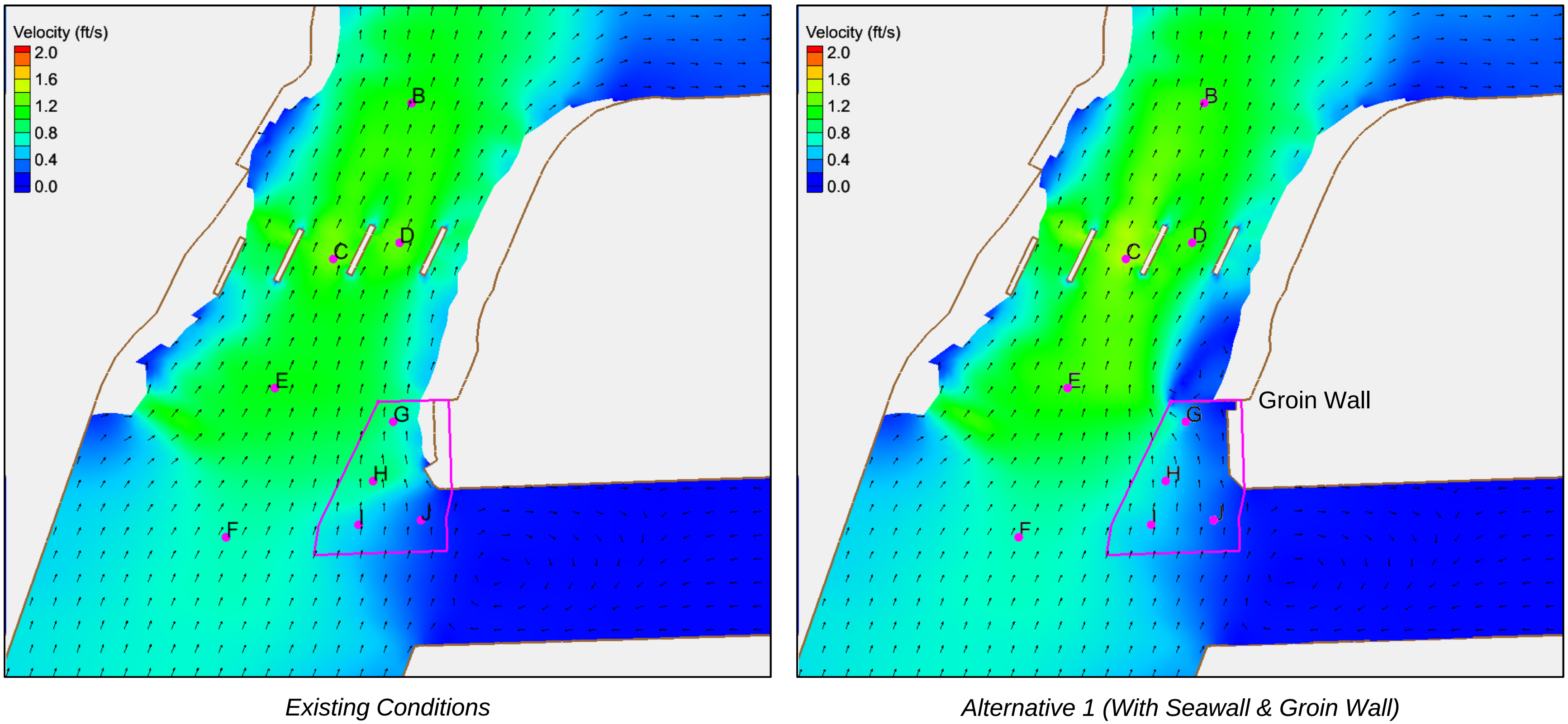


Figure 4.21 Velocity Distributions during Spring Flood Tide

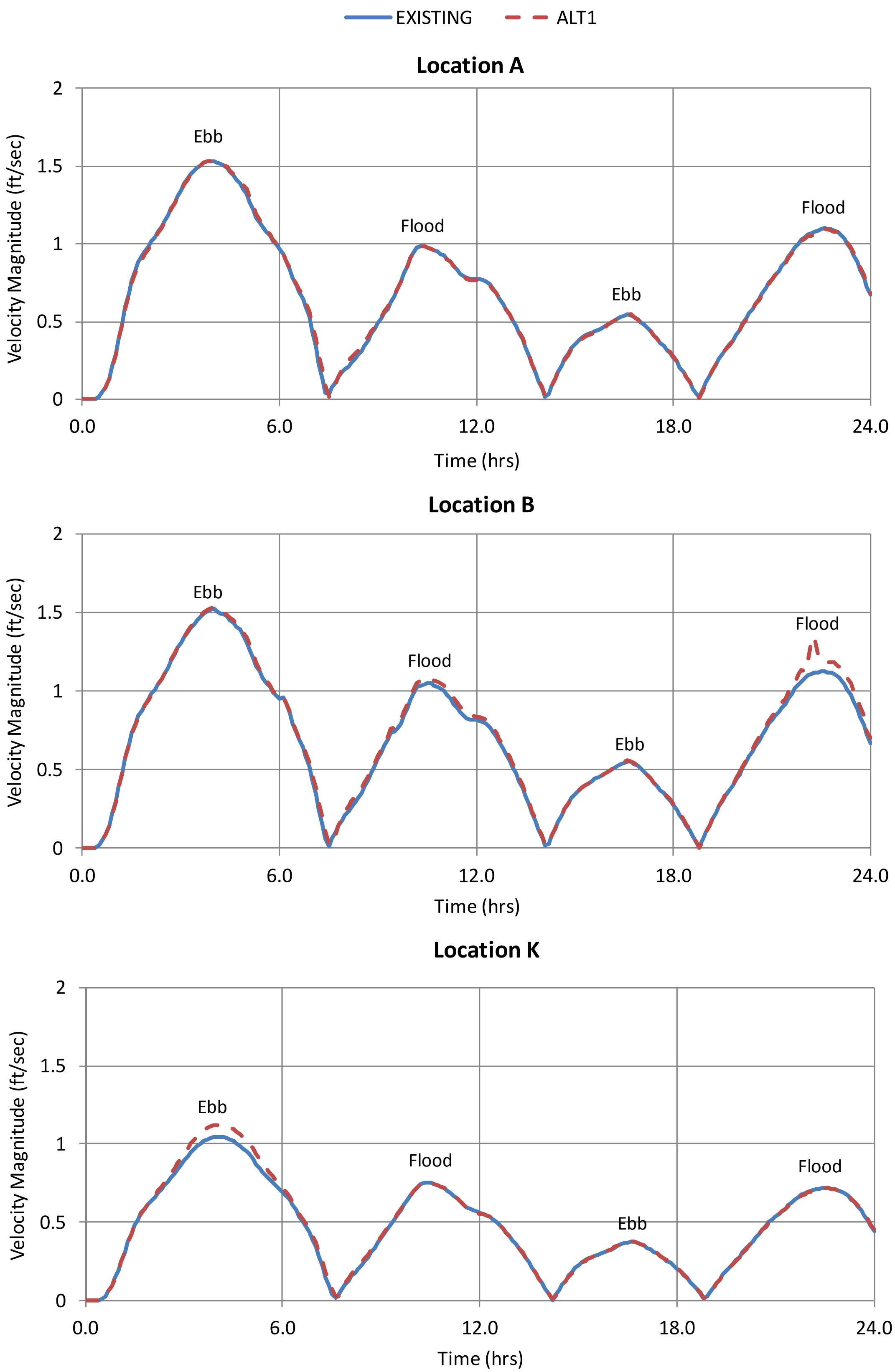


Figure 4.22 Spring Tide Velocity Comparisons at Locations A, B and K

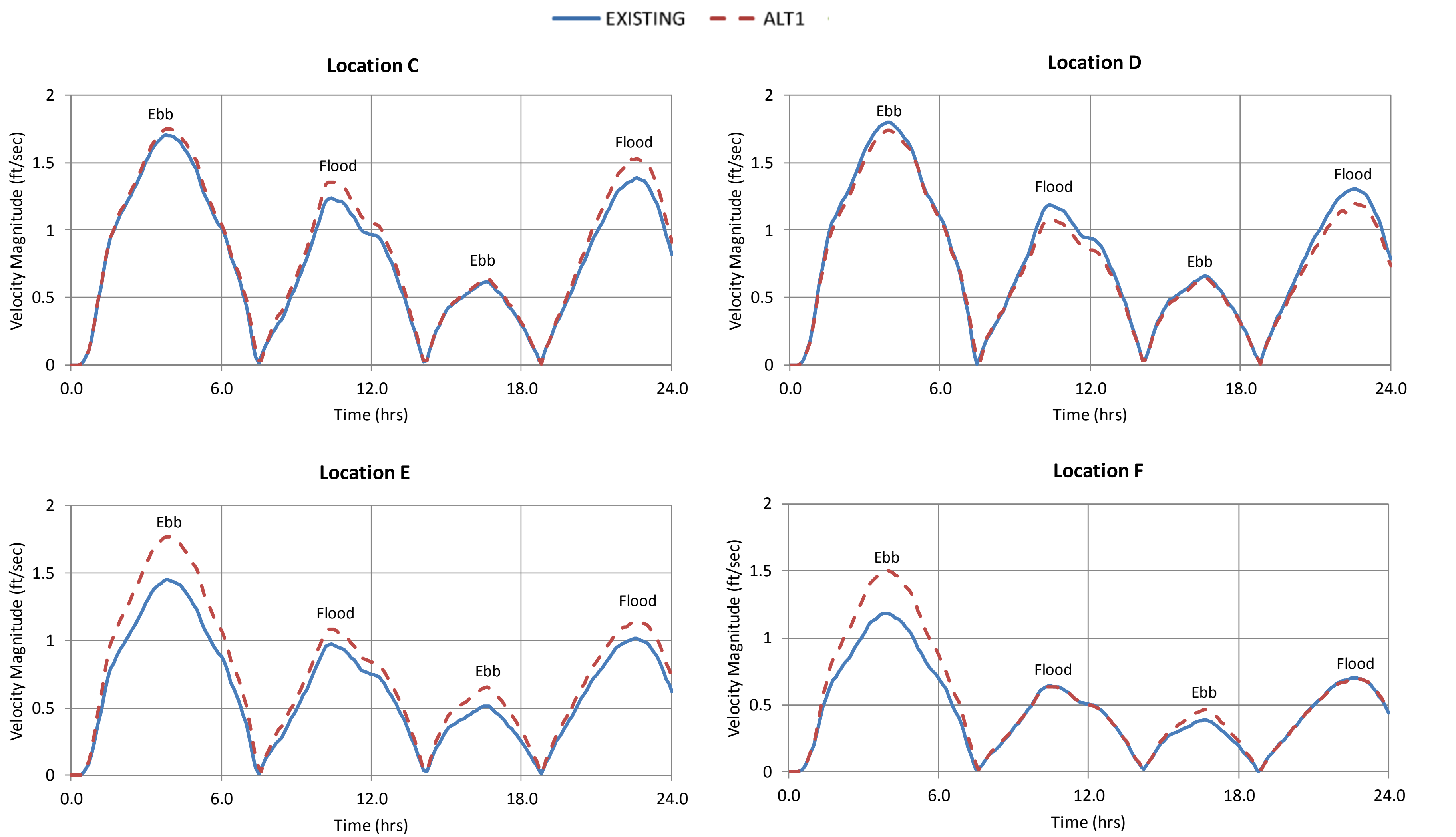


Figure 4.23 Spring Tide Velocity Comparisons at Locations C, D, E and F

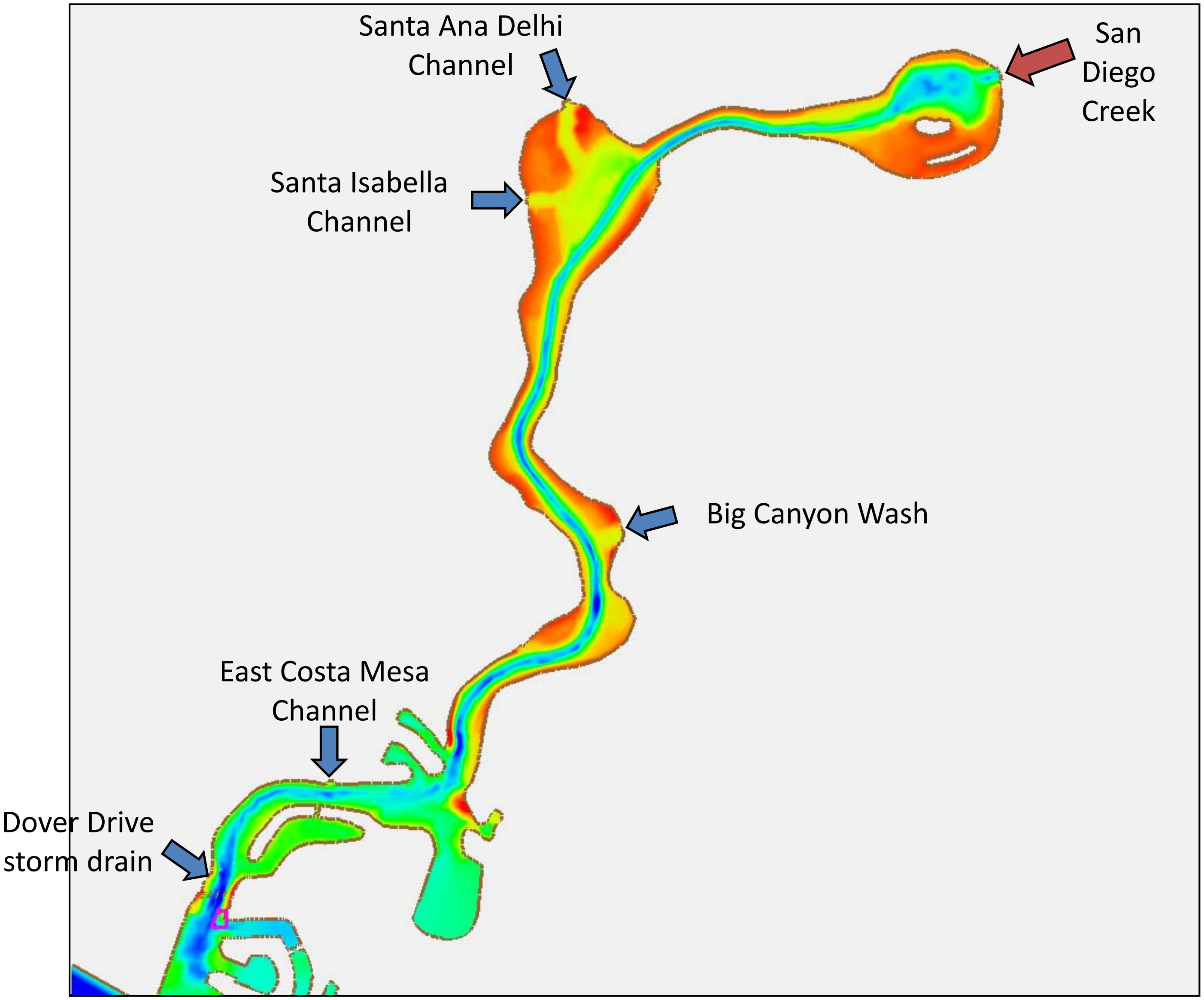


Figure 4.24 Major Discharges into Upper Newport Bay

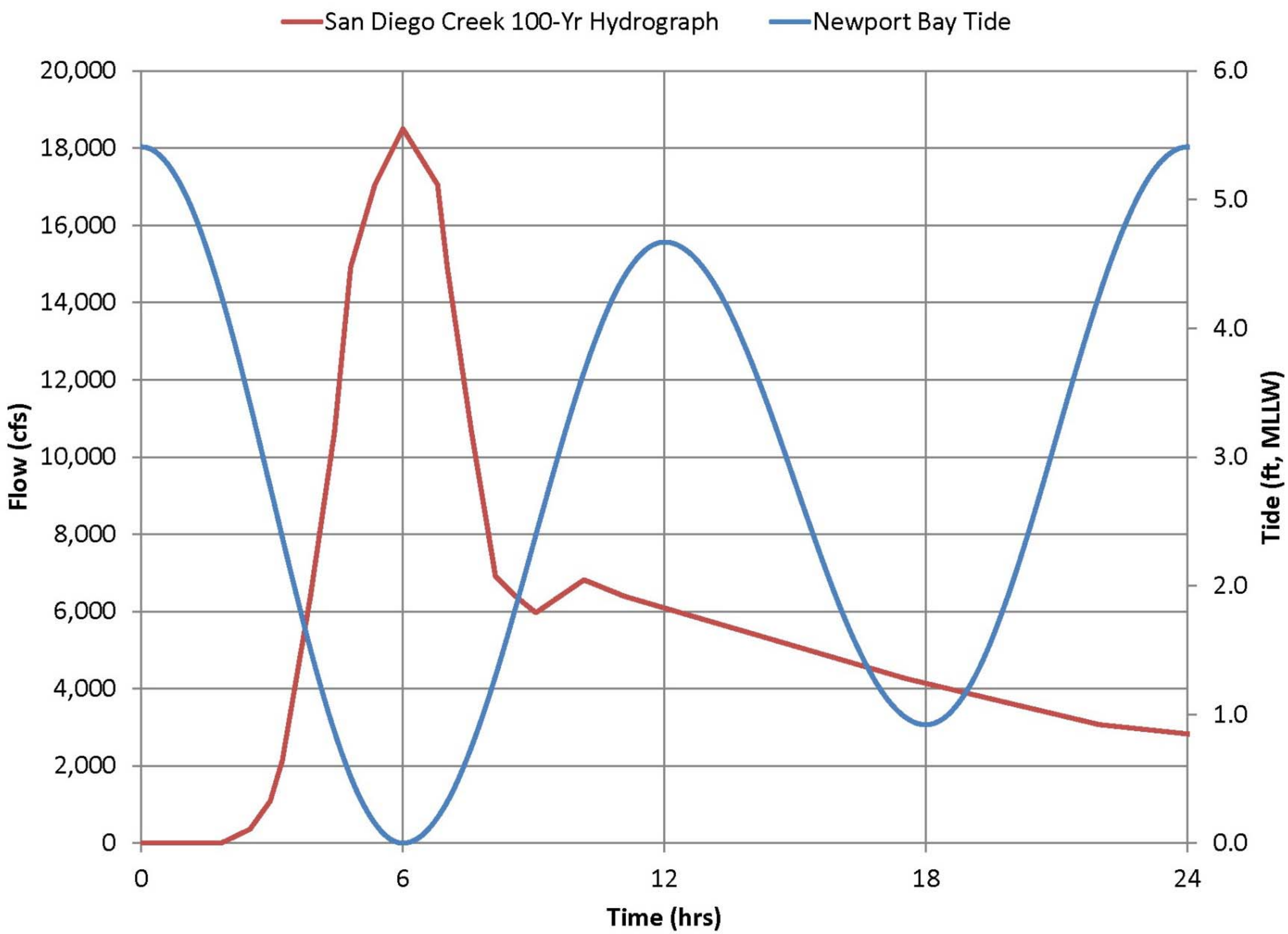


Figure 4.25 100-Year Flood Hydrograph for San Diego Creek

the peak flow coinciding with MLLW tide level. The shape of the hydrograph was obtained from flow data taken at the downstream end of San Diego Creek at Campus Drive (USACE 2000). The 100-year period peak flow was obtained from Federal Emergency Management Agency (FEMA) flood insurance study for Orange County (FEMA 2004).

The peak velocity distributions during the 100-year flood for existing conditions and Alternative 1 are compared in Figure 4.26. Note that the velocity magnitude color scale in this figure is different from those used in previous velocity distribution figures because the velocities under the 100-year event are much higher than the tidal velocities. In general, the areas of high velocities - along the center portion of the main channel and between the PCH Bridge piers, are similar to those under tide only conditions.

Velocity time series comparisons are provided in Figures 4.27 and 4.28. Upstream of the PCH Bridge (locations A and B), velocities under Alternative 1 and existing conditions are almost the same. Downstream of the PCH Bridge (location K), velocities under Alternative 1 were found to be slightly higher than those under existing conditions. For velocities at the PCH Bridge, as shown in the top panels of Figure 4.28, velocities would be similar at location C and smaller at location D. As shown in the bottom panels of Figure 4.28, for locations E and F along the main channel adjacent to the project site, Alternative 1 would result in higher velocities compared to existing conditions. The peak velocity at location E would increase from 6 ft/sec under existing condition to about 6.9 ft/ sec, a change of about 15%. At location F, the peak velocity would increase from 4.9 ft/sec under existing condition to 6.2 ft/sec under Alternative 1, an increase of about 27%.

4.4 Sedimentation and Erosion

Potential impact of the proposed marina alternatives to sedimentation and erosion could be qualitative evaluated based on the changes in tidal velocities in the project vicinity. In general, in areas currently experiencing scouring, an increase in tidal velocities due to the proposed marina could result in additional scouring; and areas currently not experiencing scouring could start to scour if the increased velocity is high enough to cause scouring. On the contrary, in areas currently having sedimentation problem, an increase in tidal velocities may help to reduce sedimentation. In addition, if the proposed project is creating a stagnant area, sedimentation may occur if there is a supply of sediments to the stagnant area. Understanding the existing sedimentation and erosion characteristics in the vicinity of the project site, hence, is the first step to assess potential impact of the proposed project alternatives.

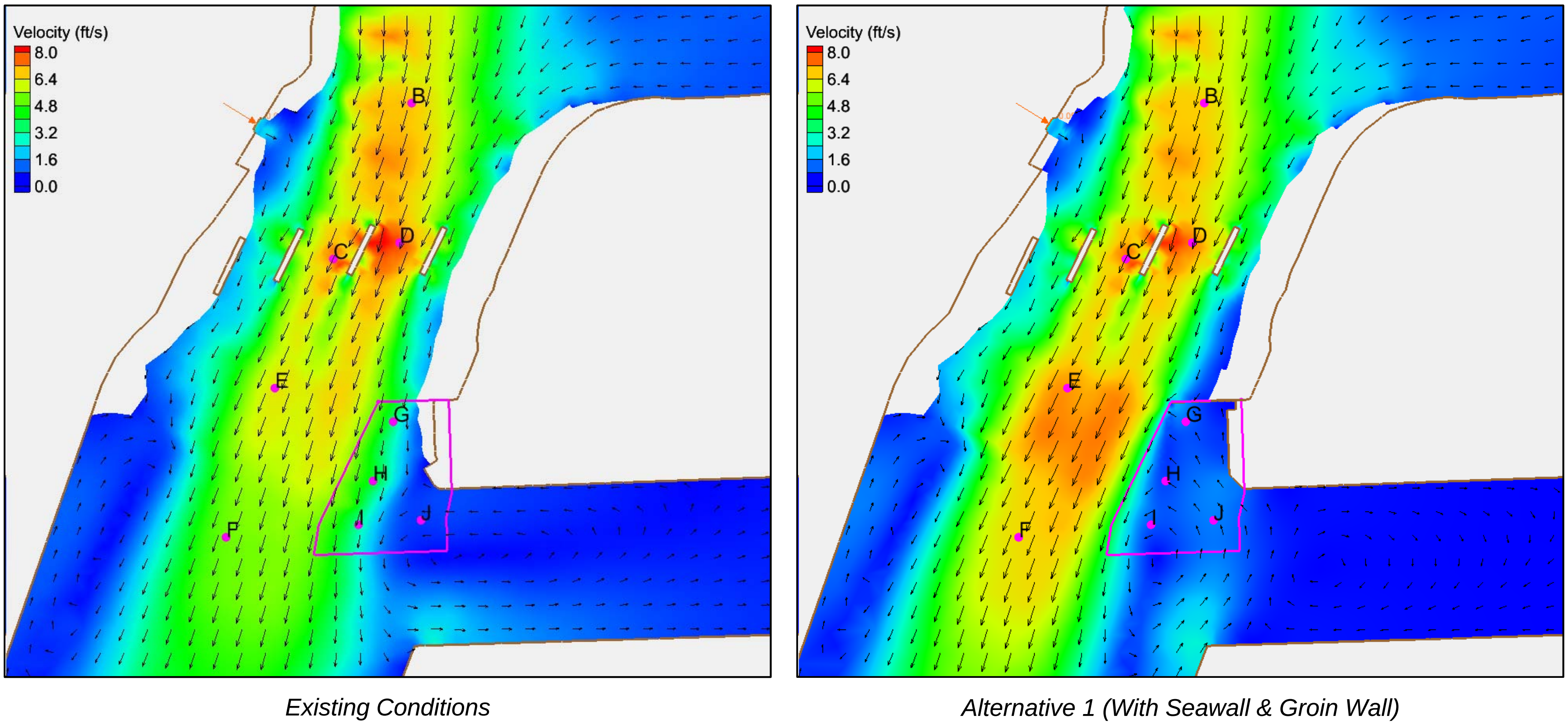


Figure 4.26 Velocity Distributions during 100-Year Flood

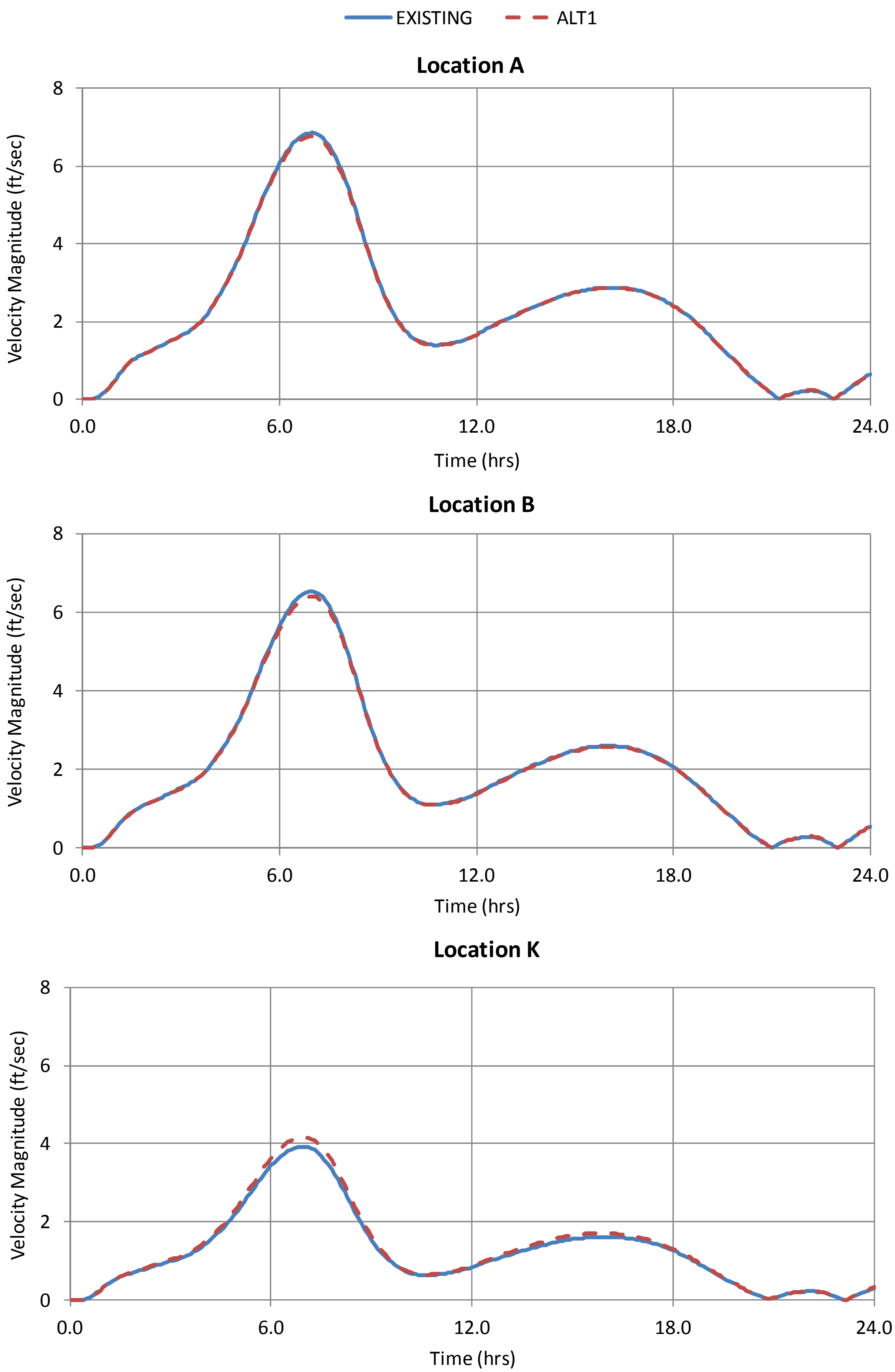


Figure 4.27 100-Year Flood Velocity Comparisons at Locations A, B and K

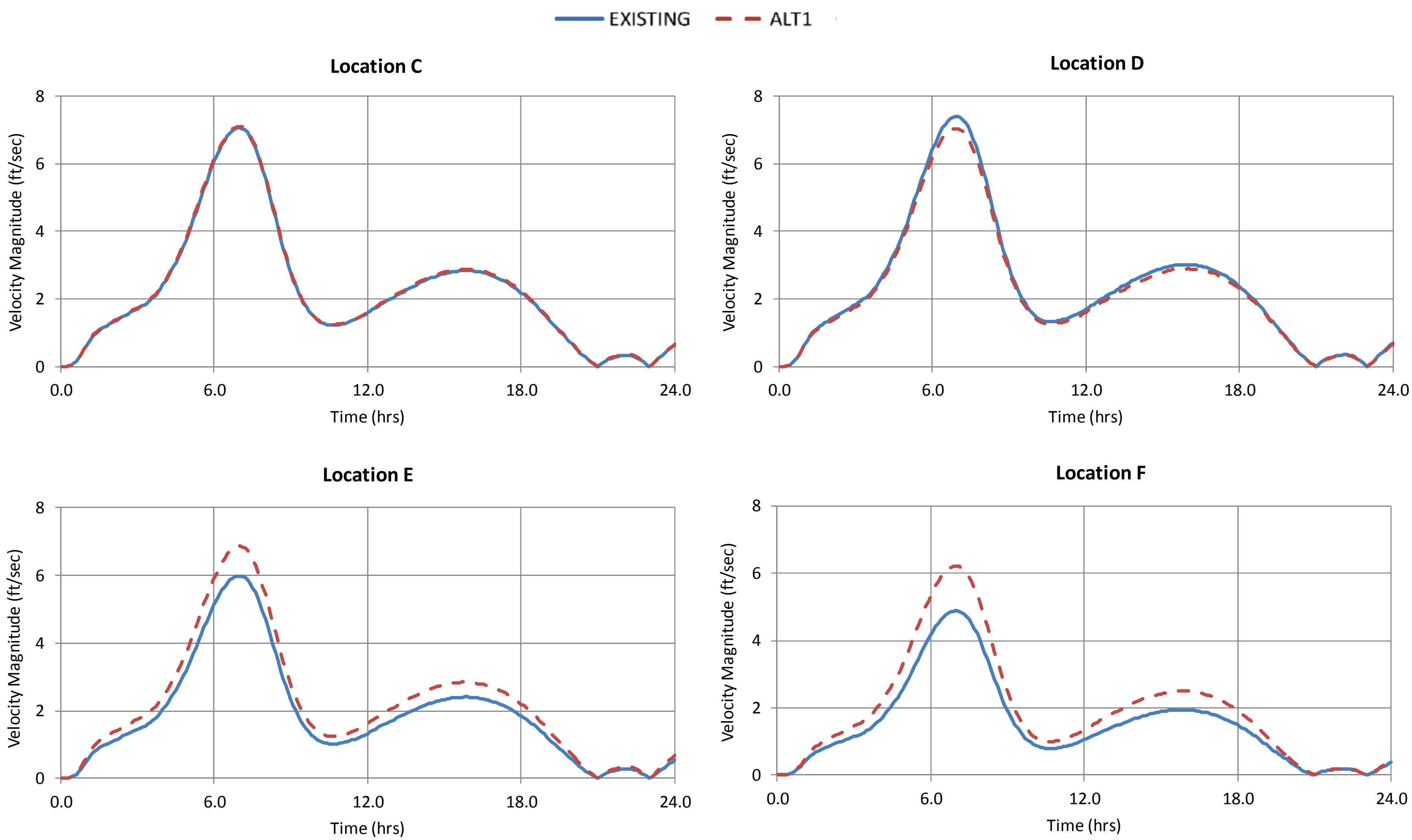


Figure 4.28 100-Year Flood Velocity Comparisons at Locations C, D, E and F

Bathymetry survey data were available for Years 2003 and 2011 for areas just upstream of the PCH Bridge near the proposed project site. These data were analyzed and processed to produce the bathymetric maps shown in Figure 4.29. As can be seen from the figure, the bathymetry in 2011 and 2003 are very similar, indicating that the area has been relatively stable (i.e. without significant sedimentation or erosion in the area) for over eight years. The scour areas indicated by the deeper contours (blue areas) remain in approximately the same locations - between the PCH Bridge piers and north of the bridge, from Year 2003 to 2011. These existing scour areas are likely caused by high tidal currents which are predicted by the hydrodynamic model as discussed in Section 3.1. Comparisons between the 2003 and 2011 contours also did not reveal any observable sedimentation in the project vicinity. The lack of changes in the bathymetry between 2003 and 2011 suggest that not much sedimentation or erosion occurred in the vicinity of the proposed marina extension for over an eight- year period.

Based on the hydrodynamic model results, the impact of the proposed marina alternatives to tidal and flood velocities would be localized, limited to within a few hundred feet downstream of the PCH Bridge along the main channel and beneath the bridge. Hence, any potential impact to erosion and sedimentation would also be localized. At the PCH Bridge, the model results show redistribution of velocities between the center piers, with an increase in velocity at one location but decrease in another location. Among the proposed project alternatives, Alternative 1 would lead to the most change in velocities between the PCH Bridge piers and downstream along the main channel. This may result in localized scouring between the piers at location C, and along the main channel at locations E and F. The increased velocities at these locations would be similar to velocities already occurring at location D. Hence, Alternative 1 would likely result in a scour pattern near location C between the bridge piers and locations E and F along the main channel similar to the existing scour pattern near location D as illustrated by depressions based on the current bathymetry shown in Figure 4.2. For all other alternatives, velocities along the main channel (locations E and F) would be similar to existing conditions without boat dockings. When the docks are completely occupied, velocities along the main channel at locations E and F would be increased, but the increased velocities would be lower than the velocities under Alternative 1. Hence, among all the proposed alternatives, potential scouring along the main channel would be highest under Alternative 1.

Based on the hydrodynamic model results, potential project impacts to the adjacent property just north of the proposed marina extension would be limited if there is any. Velocity comparisons made at ten locations in the adjacent beach area, as previously discussed in Section 4.1, shows that overall tidal currents in the area are small under existing and with-project conditions thus unlikely to cause any erosion.

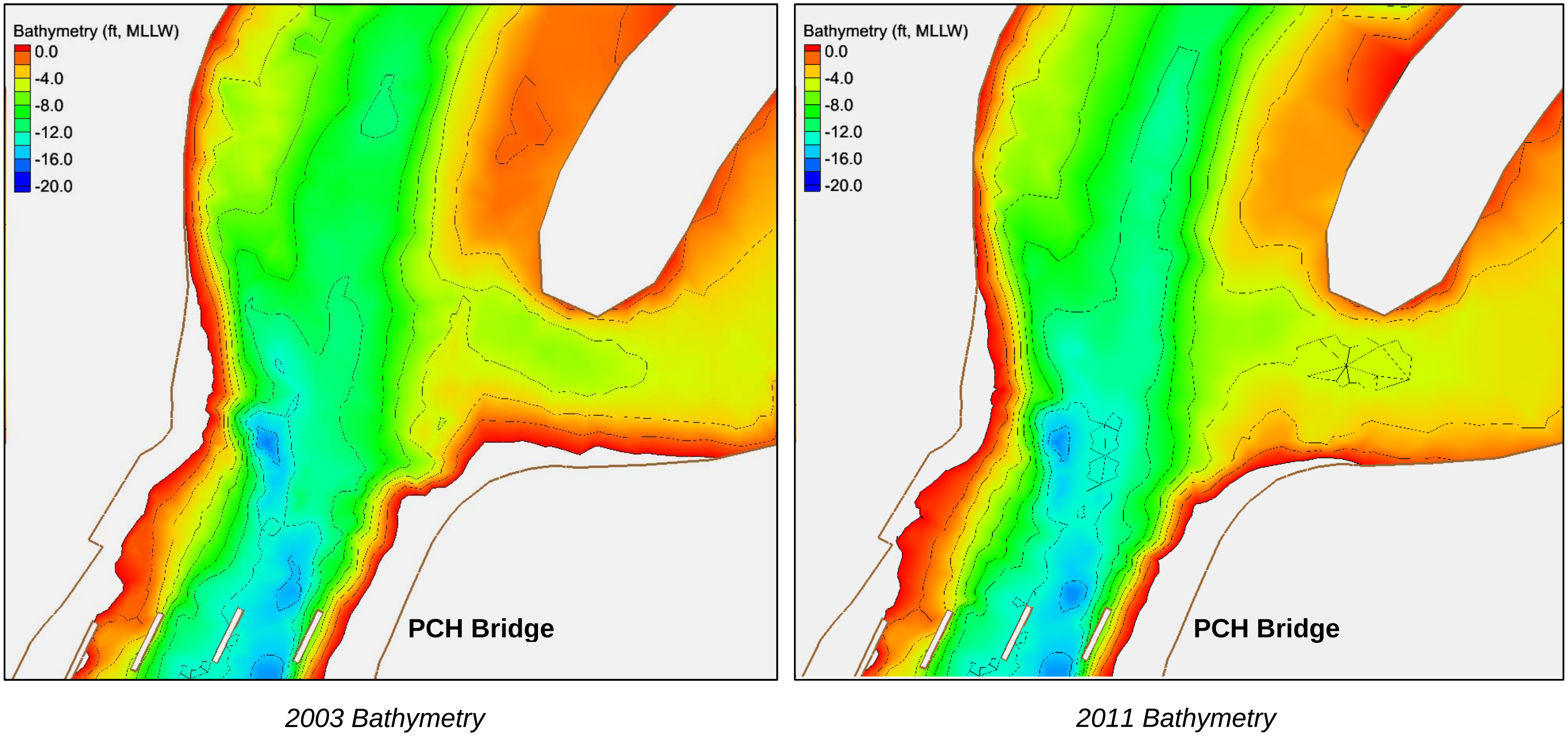


Figure 4.29 Bathymetry Contour Comparisons for 2003 and 2011

5. SUMMARY OF FINDINGS

This report presents the methods and findings of a coastal engineering study conducted for the proposed Balboa Marina West - an extension to an existing marina in the Lower Newport Bay in the City of Newport Beach. The coastal engineering study evaluates potential impacts of the proposed marina extension on tidal currents, sedimentation and erosion to areas in the vicinity of the project site. Potential impacts associated with the proposed project were evaluated based on changes to tidal currents using a two-dimensional (2D) hydrodynamic model. The changes in tidal currents were then used to infer potential impact of the proposed marina extension to sedimentation and erosion. Potential project impacts were evaluated for both the mean and spring tide conditions, as well as with a 100-year flood event.

Hydrodynamic model simulations were conducted for the following conditions:

- Existing Conditions
- Alternative 1 - with seawall and groin wall
- Alternative 2 - with seawall and no groin wall
- Alternative 2 Option 1 - with seawall and cutoff wall
- Alternative 3 – with revetment, no seawall and no groin wall
- Alternative 3 Option 1 – with revetment, no seawall and cutoff wall

Under existing conditions, tidal velocities in areas approaching the Pacific Coast Highway (PCH) Bridge and between the bridge piers along the main channel are high with peak ebb and peak flood tidal velocities of between 1.2 to 1.7 ft/sec. During a 100-year flood event, peak velocities at these areas can be as high as 7 to 7.5 ft/sec. In general, tidal velocities higher than 1 ft/sec may cause some erosion of the sediment bed, and review of existing bathymetry confirms there are scouring at these high-velocity areas. By comparing bathymetry data taken in Year 2003 and Year 2011, it is revealed that there were little changes in bathymetry in these scour areas for over an eight-year period, indicating scouring in these areas has been stabilized. In addition, there was little observable changes in the bathymetry at the project vicinity between Year 2003 and Year 2011, suggesting that not much sedimentation or erosion occurred in the vicinity of the proposed marina extension for this period.

Based on the hydrodynamic model results, impacts to tidal and flood velocities of the proposed marina extension to neighboring areas would be localized, mainly within a few hundred feet downstream of the PCH Bridge. Hence, any potential impact to erosion and sedimentation would also be localized. The greatest impacts to tidal and flood velocities would occur along the main channel just west of the project site and between the bridge piers. Among all the proposed alternatives, Alternative 1 with a groin wall will have the most impact. Under this alternative, the peak tidal velocities at the two locations under the bridge where existing tidal velocities are high will be changed such that velocities at one location will slightly increase while velocities at the other location will slightly decrease. These changes may result in small changes in the existing scouring pattern under the bridge. The most impact for Alternative 1 will occur along the main channel approximately 300 feet downstream of the PCH Bridge just west of the proposed marina extension. The peak tidal velocities in this area can be increased by approximately twenty percent from existing conditions; hence may result in additional scouring in these impacted areas. Nevertheless, these increased tidal velocities will be similar to the existing tidal velocities under the PCH Bridge; hence any additional scouring along the main channel would likely result in a similar scour pattern that is currently observed between the bridge piers.

All other alternatives beside Alternative 1 have very little impact to tidal velocities in the neighboring area of the proposed marina extension when the marina is not fully packed with boats. When the marina is fully occupied, the boats can block tidal flows. With an assumption that the boats would completely block off tidal flows, the proposed alternatives will then impact tidal velocities along the main channel in a similar manner as Alternative1, resulting in higher tidal currents along the main channel just downstream of the PCH Bridge and to the west of the proposed marina extension. However, the increase in tidal velocities at the impacted areas along the main channel will be less than the increase in velocities caused by Alternative 1; hence, any potential additional scouring along the main channel due to boats docked in the proposed marina extension will be less than that caused by Alternative 1.

Based on the hydrodynamic model results, potential project impacts to the adjacent property just north of the proposed marina extension would be limited if there is any. Velocity comparisons made at different locations in the adjacent beach area shows that overall tidal currents in the area are small under existing and with-project conditions; thus unlikely to cause any erosion.

6. REFERENCES

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