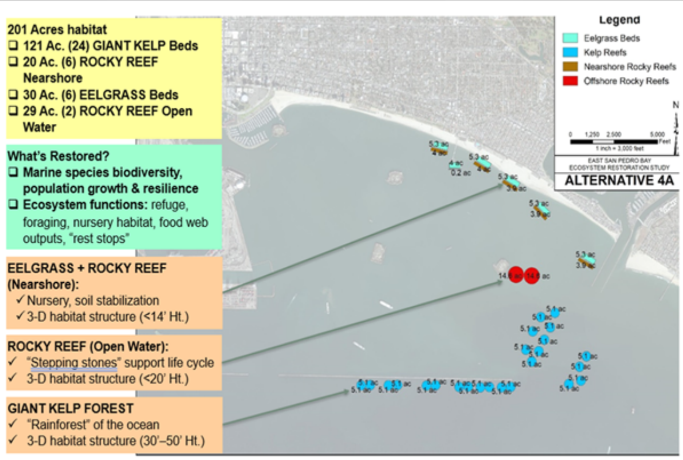
FINAL INTEGRATED FEASIBILITY REPORT (IFR) AND ENVIRONMENTAL IMPACT STATEMENT / ENVIRONMENTAL IMPACT REPORT (EIS/EIR)

APPENDIX A: COASTAL ENGINEERING AND DESIGN

EAST SAN PEDRO BAY
ECOSYSTEM RESTORATION STUDY
Long Beach, California

January 2022



US Army Corps of Engineers ®

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1 GENERAL

To support decisions made relating to the East San Pedro Bay (ESPB) Ecosystem Restoration Study, the following three-tiered approach was applied to the Coastal Engineering analysis to be discussed in this appendix. All numeric models are at a feasibility level developed to support plan formulation and selection of alternatives. Further refinement is required to ensure results remain consistent with more complex analysis procedures. A short summary of the findings and results are as follows.

1.1 HYDRODYNAMIC MODELING

A coupled model using EFDC (Environmental Fluid Dynamics Code) and CMS-Wave is used to provide hydrodynamic parameters that feed into the habitat evaluation model. The hydrodynamic modeling includes structural changes to the Long Beach Breakwater (breakwater), a training wall for the Los Angeles River and other habitat measures that alter the circulation patterns and magnitude within the bay.

Changes to the breakwater does not significantly affect the average amount of time a typical particle remains in East San Pedro Bay (ESPB). Although, the change is dependent on the size of the breakwater modification; i.e. that larger the opening, the less time particle remains in the location. Small openings show practically no change in duration while lowering of the entire breakwater, on average, decreases in time from the existing conditions. The training wall has the largest decrease on the average particle time during the winter months but also correspond to flow events from the Los Angeles and San Gabriel Rivers. Other structural changes do not significantly alter the circulation patterns within ESPB.

Bouss-2d is used to model more complex wave patterns and propagation into the study area and guide the design parameters for habitat measures, breakwater modifications and corresponding protective measures. Breakwater modifications allow for wave height increases of more than 100% in extreme events but generally only increase heights by 25% from existing conditions. Various protective measures will be required to safeguard existing infrastructure from any breakwater modification.

1.2 STRUCTURE DESIGN AND QUANTITY ESTIMATES

Each habitat measure is evaluated for stability, sustainability and longevity based on the hydrodynamic conditions and wave environment. The estimated quantities of materials are shown in the following table:

Table 1-1: Approximate Quantities for Alternatives

	Stone - Armor	Stone – Filter	Stone - Core	Fill	Sand	Concrete
	ton	Ton	ton	yd^3	yd^3	yd^3
Alternative 2	137,000	55,000	252,000	-	100,000	-
Alternative 4A	359,000	55,000	266,000	-	100,000	-
Alternative 8	2,079,000	92,000	300,000	2,990,000	796,000	48,000
Western BW Notching Plan	429,000	230,000	361,000	-	600,000	4,000
Eastern BW Removal Plan	361,000	149,000	106,000	-	600,000	4,000

Material sources have been identified and analyzed to ensure adequate quantity for construction of the measures. All materials, besides large armor stone, are relatively easy to obtain and can be procured at a rate to keep up with construction activities. Large armor stone production may lag the construction due to the sizeable quantity required and the need from other maintenance projects within the Southern California area. Beneficial re-use of material from adjacent large federal projects (Port of Long Beach Deepening Study and Naval Weapons Station, Seal Beach Pier Expansion) can be incorporated depending on the construction timeframe but should not be relied upon as a primary source.

1.3 IMPACTS TO COASTAL PROCESSES

Oyster reefs, kelp reefs and offshore rocky reefs will have little effect on wave and current patterns within ESPB. The tidal salt marsh will be designed to limit wave reflection within the bay so any affect will be negligible. The emergent island will shelter the shoreline and reduce the sediment transport potential in the lee of the structure.

The nearshore reefs will cause waves to shoal and break causing a lower sediment transport potential in the lee of the structures. This lower sediment transport will produce a small salient behind the structures and allow for the creation of a perched shoreline; an increase of bottom elevation caused by the lower sediment transport potential and a fixed impermeable seaward boundary. Current sand management operations employed by the City of Long Beach is expected to continue.

Modifications to the breakwater will allow for more wave energy within the bay. This increase of energy will cause more sediments to be available for transport; finer sediments will be mobilized and transported by the underlying tidal currents leaving more coarse materials than are currently not present in the top layers of sediment. Changes to the shoreline configuration is expected as a result of a breakwater modification. Lowering of the eastern end of the breakwater will widen the zone of erosion to a more western extent; while notching of the western side will cause localized pockets of erosion from the Shoreline Marina to Belmont Pier. In both cases, the total amount of sediment within the littoral cell will not change due to the lateral boundaries of stone jetties and groins. The change of sediment transport and runup height will need to be controlled by protective structures that will reduce the incident wave energy on the shoreline. Future analysis, if carried forward, will be required to determine the optimal structure placement for the nearshore reef and protective measure locations to best match the runup and sediment transport patterns that currently exist. A physical model of the project area will need be developed and analyzed before the final design can be developed and finalized.

Potential sea level rise effecting project elements can be countered by an increase in structure height during periods of maintenance. Significant runup and overtopping effects on the shoreline and existing shoreline structures is expected if more wave energy is allowed to enter the bay which will be exacerbated by increased water levels. Historic records show that without the protection of the Long Beach Breakwater, the shoreline recedes during storm events leading to a potential increase of coastal storm damage of waterfront structures.

2 INTRODUCTION

2.1 HISTORY

The location of the project lies near the port complex of Los Angeles/Long Beach. The history of the study area follows closely the development of the ports starting with the San Pedro Breakwater in the 1890's. Before this time, the precursor to the port complex was in the originally open western area of San Pedro Bay, sheltered by Point Fermin and with the bygone Wilmington Lagoon. Many improvements were made over the years and resulted in harden structures to maintain a navigation channel which allowed the budding port complex to flourish. With the increase in capacity, the port looked to expand and the San Pedro Breakwater was constructed from 1889 – 1910 to provide safe harbor refuge. Originally, there was a small gap between the shoreline and the detached breakwater but was closed in 1912 and truly signifies the development of the semi-enclosed basin that is known today. In 1928, a small rocky island named Deadman's Island was completely removed; this area is now directly within the main navigation channel leading to the Port of Los Angeles.

The Port of Long Beach development began around the 1900's; the city of Long Beach obtained rights to direct this development. By 1923, the Los Angeles River flood control project was completed which fixed the outflow location of the river. A rubble mound breakwater, beginning at the mouth of the new flood control project and extending 4,300 ft. into the ocean and then extended 3,500 ft. to the southwest 10 years later, was constructed and has been since extended and removed to even more expansion of the port complex. This earlier breakwater provided calm waters to the Port of Long Beach. The Middle Breakwater was constructed in phases from 1932 – 1942 for the port complex and provided a gap between the San Pedro Breakwater of approximately 2,000 ft. which is now known as Angeles Gate. Following this activity, the Long Beach Breakwater was constructed during World War II and completed shortly after in 1949. The 1,700 ft. gap between the Middle and Long Beach Breakwaters is now known as Queen's Gate. The total length of breakwater now exceeds 8 miles (Wiegel, 2009).

Development along the eastern side of Long Beach follows a similar timeline as the development of the port complex. After the channelization and damming of the Los Angeles River, the natural sediment supply to Long Beach quickly diminished and the shoreline eroded to the cliff face from the old Rainbow Harbor to Belmont Pier. Not until the construction of the Long Beach Breakwater and other sediment management measures after World War II did the shoreline conform to the current geometry (Lillevang, 1986).

2.2 UNITS OF MEASURE

All final units of measurement will be reported in the United States Customary System (USCS). Some intermediate units will be shown in the metric system. All numerical models were computed in the metric system then converted to the USCS. When appropriate, conversion to the USCS will be rounded for convenience.

3 PRESENT CONDITIONS

3.1 STUDY AREA

The study area for this project contains the entirety of San Pedro Bay from the Palos Verdes Peninsula to the east of Anaheim Bay outward to a water depth of approximately 70 feet. The study area includes the Ports of Los Angeles and Long Beach, Alamitos Bay and Anaheim Bay as well as the tidally influenced portions of the Dominguez Channel, Los Angeles and San Gabriel Rivers. For a further description of the study area, see the Main Report.

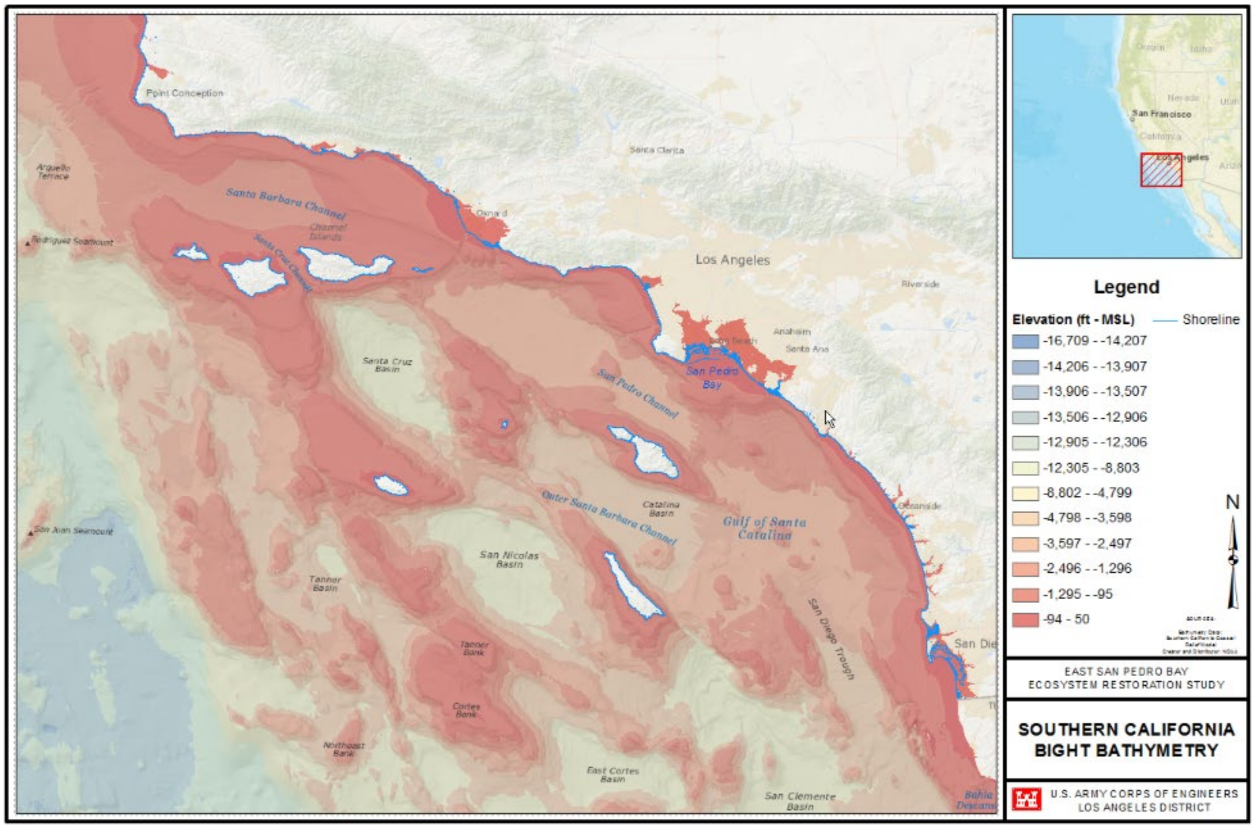
3.1.1 PHYSICAL SETTING

The coastal plain of the Los Angeles Basin, which includes the lower San Gabriel River, extends about 50 miles to the southeast along the shoreline from the Santa Monica Mountains and from about 12 to 20 miles inland from the ocean. The coastal plain is a region of low relief except for the Palos Verdes Hills and a nearly straight row of disconnected low hills and coastal mesas. The broad surface of the plain slopes gently seaward from its eastern boundary, where the land-surface elevations range from 200 to 300 ft. above mean sea level. The row of hills and mesas separates the seaward section of the plain (on the southwest) from the large inland part (on the northwest), which is known as the Downey Plain. The study area lies on the seaward edge of the Downey Plain.

3.1.2 BATHYMETRY & TOPOGRAPHY

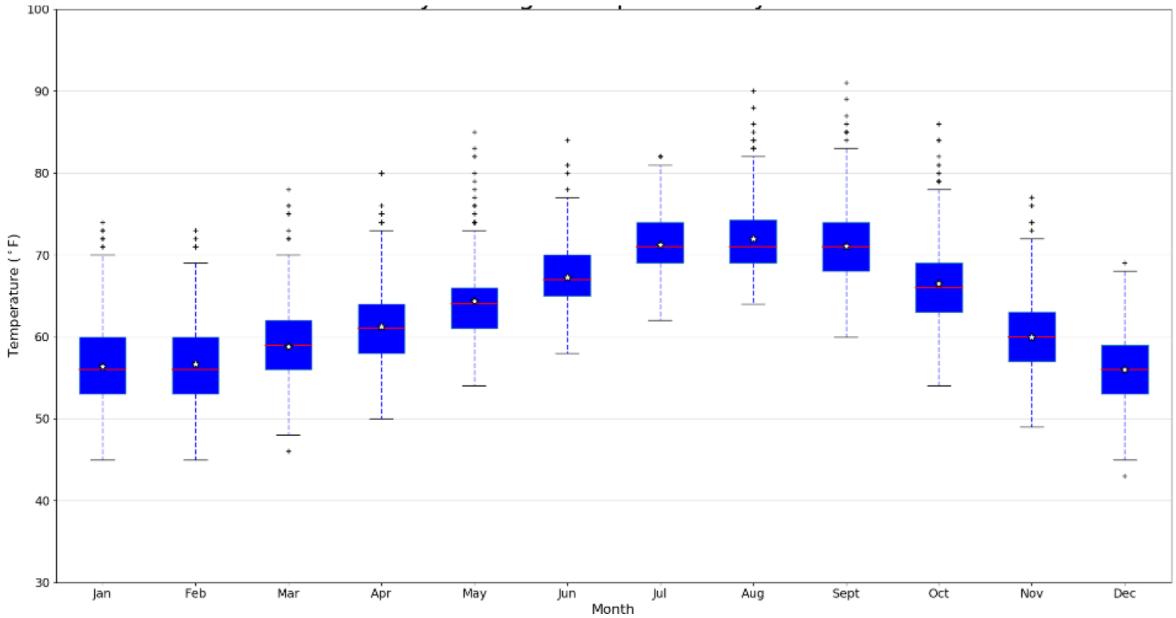
The Southern California Bight, or curved coastline that forms an open bay, extends from Point Conception to the north and the U.S. – Mexico border in the south. Two prevalent features are important to the coastal processes within the California Bight; Point Conception and the Channel Islands. Together, these features limit the amount of wave energy that eventually impact the shoreline. The bathymetry within the study area is generally parallel to the shoreline with a few exceptions. Structures has been built from the existing seabed and will be discussed later in this appendix. Two large borrow pits were created during the construction of the offshore energy islands and comprise of water depths upward of 60 feet. The gentle nearshore slope of 1V:100H meets the shoreface and rises at a steep slope in the east of 1V:8H and smooths to a milder 1V:20H slope in the west. The steepness of the shoreface is dependent on the incident wave energy and sediment supply.

Figure 3-1 shows the offshore bathymetry near the study area. From deep to shallow water, numerous seamounts, undersea basins, channels, shelves and islands are present. The most prominent features that significantly affect the study area are the Channel Islands (specifically Catalina Island), the San Pedro Channel and Shelf.

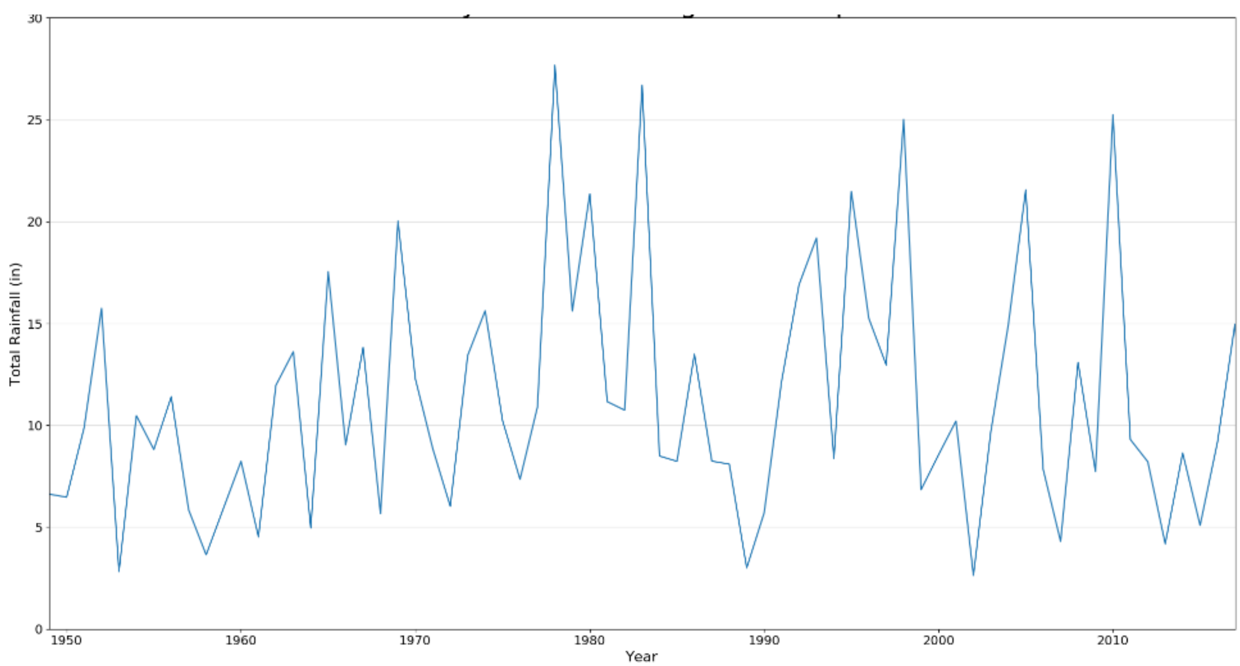


3.1.3 CLIMATE

The climate of San Pedro Bay is characterized by warm, dry summer and mild winters. A daily average temperature box plot arrange by month is shown in Figure 3-2 and shows the yearly temperature trend. Generally, the warmest month is August; averaging in the low seventies. However, afternoon temperatures from July to September average in the low eighties with a recorded extreme high of 111°F at the Long Beach Airport both in 1961 and 2010. January is typically to coolest month with minimum average temperatures in the middle fifties. Average annual high and low temperatures at the Port of Los Angeles is 67°F and 56°F.



Precipitation within the study area is in the form of rainfall. The Mediterranean climate characteristically has dry summer and wet winter months. Nearly 90% of the annual precipitation falls within the months of November to April. The mean annual precipitation at the airport (~4 miles from study area) is 11.2 inches and is typically caused by the influence of extratropical storms in the winter. Rainfall is generally heaviest in February. The total precipitation per year is shown in Figure 3-3. Humidity is generally higher in the harbor than inland areas due to its proximity to the ocean. In comparison to inland locations, fog is more frequent and persistent and overcast skies tend to burn off later in the day.

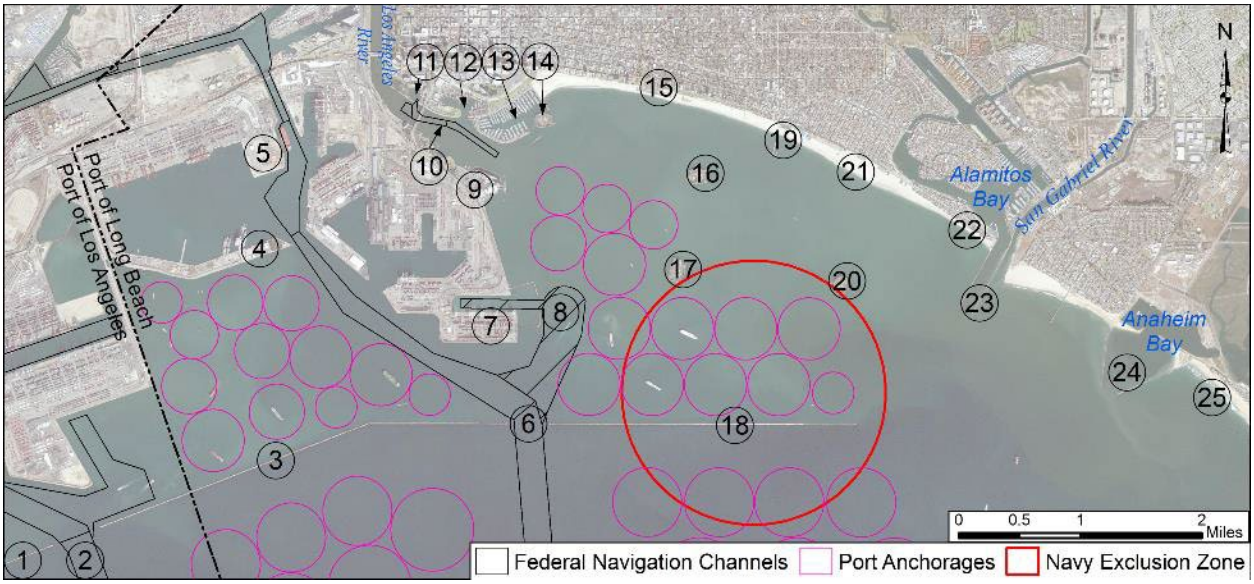


Winds near the study area are generally weak in the morning hours slowly picking up speed until the late afternoon and flow from the southwest. Winter months generally produce larger wind magnitudes than the summer months with hourly averages from 4 knots in the summer up to 7 knots in the winter. Peak values in wind speed can be much greater than the hourly averages; 20 knot gusts are not uncommon. Occasionally, during the fall and winter months, strong winds may develop during a Santa Ana condition and may persist for a few weeks.

3.1.4 PROPOSED PROJECT AREA

A proposed project area is shown in Figure 3-4 and described further in the Main Report.

All references to specific areas within the larger study area are shown in Figure 3-5 with numbers referenced below.



1.	San Pedro Breakwater	6.	Queen's Gate	11.	Catalina Express	16.	White Island	21.	Belmont Shore
2.	Angeles Gate	7.	Pier J	12.	Rainbow Harbor	17.	Freeman Island	22.	East (Peninsula) Beach
3.	Middle Breakwater	8.	Tentative Pier J Approach	13.	Shoreline Marina	18.	Long Beach Breakwater	23.	Alamitos Bay Jetties
4.	Navy Mole	9.	Carnival Cruise Lines	14.	Grissom Island	19.	Belmont Pier	24.	Naval Weapons Station, Seal Beach
5.	Pier T	10.	LARE	15.	Junipero Beach Parking Lot	20.	Chaffee Island	25.	Surfside-Sunset

3.1.5 FEDERAL PROJECTS

There are various federal projects within the study area; some have already been identified. The following is a short list and description of these current and previous federal projects identified on Figure 3-5.

3.1.5.1 SAN PEDRO BREAKWATER

The San Pedro Breakwater, completed in 1910, is the oldest federal project within the study area. This cut stone structure rests on a rubble mound base and is approximately 2 miles long and provides protection for the Port of Los Angeles from incoming wave energy. The structure is periodically maintained and repaired to provide an adequate level of protection.

3.1.5.2 LOS ANGELES FEDERAL NAVIGATION CHANNEL

The Los Angeles navigation channel is approximately 8 miles long and maintained at various depths from 53 ft. to 81 ft. MLLW depending on the location. The channel is typically dredged approximately every 10 years. According to the Waterborne Commerce Statistics Center, approximately $145 trillion of cargo utilized this channel in 2017.

3.1.5.3 LONG BEACH FEDERAL NAVIGATION CHANNEL

The Long Beach navigation channel is approximately seven miles long and maintained at a depth of 76 ft. MLLW. This channel is typically dredged every 10 years or when needed to remove high spots that interfere with safe navigation. A deepening project is currently under design to deepen the main channel to 80 ft, ease some of the channel bends and to create a new channel, at 55 ft, from Queen Gate to Pier J. According to the Waterborne Commerce Statistics Center, approximately $125 trillion of cargo utilized this channel in 2017.

3.1.5.4 MIDDLE BREAKWATER

The Middle Breakwater was completed in 1942 and serves as a protective structure for both the Port of Los Angeles and Port of Long Beach. It is a rubble mound structure that is approximately 3.5 miles long. This structure was recently repaired, completed in 2018, due to damage received during a strong southern swell generated by Hurricane Marie in August 2014.

3.1.5.5 LONG BEACH BREAKWATER

The Long Beach Breakwater was completed in 1949 and is the main protective structure within the proposed project area. This rubble mound structure is approximately 2.5 miles long. This structure will be discussed in more detail.

3.1.5.6 LOS ANGELES RIVER ESTUARY (LARE) FEDERAL NAVIGATION CHANNEL

This channel is considered shallow draft and utilized primarily by Catalina Cruise Lines, a local company that offers transportation to the nearby Channel Islands, and other recreational boaters. This channel is authorized to be maintained at a depth of 20 ft. and is typically dredged every 3-5 years or when needed to remove high spots.

3.1.5.7 SURFSIDE/SUNSET STORM DAMAGE REDUCTION PROJECT

Although not directly in the study area, this periodic beach nourishment project has been conducted successfully since the 1960’s. Sandy material is dredged from an offshore area and deposited to create a feeder beach that slowly nourishes the down-drift coastline of Seal Beach, Huntington Beach and Newport Beach. Typically, nourishment is authorized for a recurrence interval of 5 years or as funding allows. The last placement event was in 2009 and another planned event may occur within the next few years.

3.1.5.8 NAVAL WEAPONS STATION, SEAL BEACH MUNITIONS TRANSFER

The nearby Naval Weapons Station, Seal Beach uses the area protected by the Long Beach breakwater to transfer munitions and other supplies. An exclusion zone, shown in Figure 8-10, limits access when a military vessel is loading or unloading munitions. No vessels or persons are allowed within this exclusion zone when work is occurring.

3.2 TOPOGRAPHY AND BATHYMETRY

3.2.1 DATA SOURCES AND PROCESSING

Various data sources were used to obtain accurate bathymetric and topographic conditions of the project site. These sources consist of depth soundings from the National Oceanographic and Atmospheric Administration (NOAA) for the intermediate and near shore environment, aerial light detecting and ranging (LiDAR) flight paths from USACE's Joint Airborne LiDAR Bathymetry Technical Center of Expertise (JALBTCX) for shallow water and the emergent environment and a conglomeration of data compiled by NOAA's National Geophysical Data Center (NGDC) called the Southern California Coastal Relief for the deep water environment. All data sets were obtained or converted in a horizontal and vertical datum of NAD 83 California State Plane, Zone 5 and NAVD88 measured in meters.

3.2.1.1 NOAA SOUNDINGS (NOAA, 2013)

Local NOAA sounding were obtained for intermediate and shallow water areas. The reported horizontal and vertical datum was NAD83 in degrees and MLLW in meters with respect to station ID 9410660. Horizontal re-projection was performed to shift to California State Plane, Zone 5 in meters. A vertical shift of 0.062m was applied to shift the vertical datum to NAVD88 according to the tidal station.

3.2.1.2 USACE PERIODIC CONDITION SURVEY, L.A. RIVER ESTUARY (USACE, 2016)

In support of periodic dredging of the Los Angeles River Estuary, surveys are typically conducted yearly. The last to be perform by SPL was in June 2016. The horizontal and vertical datum are California State Plane, Zone 5 and MLLW both measured in feet. This data set was then converted into metric units and the vertical shift of 0.062m from MLLW to NAVD88 was applied based on tidal station 9410660. The survey data was collected using a multibeam sounder and depths were averaged over a 1 m grid.

3.2.1.3 JALBTCX LIDAR (USACE, 2014)

LiDAR data was obtained during flights in September and October of 2014. These datasets were obtained from NOAA's Data Access Viewer with a horizontal and vertical datum of California State Plane System, Zone 5 and NAVD88 respectively; both measurement in meters. This *.LAS dataset has a horizontal accuracy of 1 m at a 95% confidence interval and a varying vertical accuracy dependent on the water depth but at a 95% confidence interval. For above water, shallow water and deep water points the vertical accuracy is 0.095, 0.125 and 0.2 m respectively. NOAA's software converts the raw data into a raster file with a cell size of 2 m.

3.2.1.4 SOUTHERN CALIFORNIA COASTAL RELIEF (CALSBEEK ET AL., 2013; NGDC, 2003)

The Southern California Coastal Relief data includes various sources including multibeam swath sonar, U.S. Geological Survey topographical datasets and other federal government agencies and academic institutions. The data is assembled in a raster format with a 1 arc-second grid with a geographic horizontal datum of North America Datum of 1983 (NAD 83) and a vertical datum of MSL. The reported horizontal and vertical accuracies are approximately 30 m and 0.1 m to 5% of water depth depending on the data source. Because validated NOAA soundings were previously collected for near shore and LiDAR of coastal regions, this coastal relief model will only be used for deep water locations that remain seaward of the verified NOAA soundings location. For modeling purposes, the difference between the vertical datum of MSL and NAVD88 are less than the vertical accuracy of the relief map. Additionally, at these deep water depths, a shit of even 3 ft. will have a legible effect on the waves, tides and currents.

3.2.2 COASTAL FEATURES

With the development of the port complex as well as the nearby coastal communities, man-made coastal features are prevalent within the study area. The following is a brief description of the features that may alter the wave and current patterns

3.2.2.1 DETACHED BREAKWATERS

Most noticeably, the detached breakwaters (San Pedro, Middle and Long Beach) have the greatest effect on the nearshore environment. The San Pedro Breakwater is a randomly placed rubble mound structure covered by regularly cut stone. This structure has little direct effect on the proposed project area, so no more discussion will be provided. The Middle and Long Beach Breakwaters have the same basic design, as seen in Figure 3-5. The core of the structures consists of clay and silt that is covered in two sizes of armor stone. The smaller stone, Class “B”, protects the lower portions of the structure from the sea bed to a depth of 10 ft. MLLW. The larger, Class “A”, stone caps the structure and provides stability during the large wave events. The final design crest elevation of the structures is +14 ft. MLLW but due to progressive damage over the period of service, the crest elevation slightly meanders above and below the design elevation. Based on the original specifications, the Class “A” and “B” stones have a median weight of 10 tons and 500 lbs. respectively. Subsequent repairs of these structures have allowed for heavier individual stones to be added and increase the total stability of the structures.

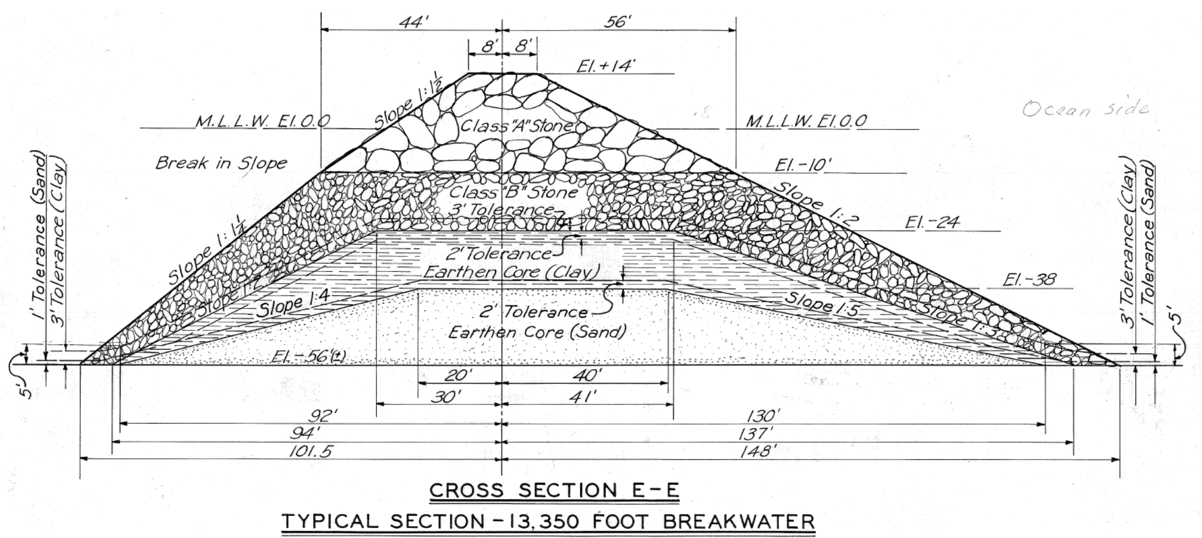


Figure 3-6: Original Cross-Section for the Long Beach Breakwater

3.2.2.2 OFFSHORE ENERGY ISLANDS

The offshore energy, or THUMS, islands consist of fill material covered in revetted slope for protection from the waves. They were created in 1965 for oil production and have been in continuous operation since. Original designs show the revetment crest elevation to be 15 ft. MLLW with a stone size of 5 tons on the seaward side and 1 ton on the leeward side. Subsequent repairs and modifications have altered the structure to consist of 5 ton stones everywhere with a crest elevation of 20 ft. MLLW and side slope of 1V:1.5H. The current design wave height for the structures is 11.3 ft (Moffatt & Nichol, 1983).

3.2.2.3 SHORELINE STRUCTURES

The Port of Long Beach complex within the proposed project area consist of reclaimed land surrounded by revetted slopes and harbor berths. Pier J terminal would be the most effected location of the port due to the proposed project. From observations, revetment stone sizes range from cobbles up to 5 ft. in diameter. The condition of the structure is dependent on the location and ranges from poor to very good.

Carnival Cruise Lines terminal is adjacent to the Queen Mary within the port complex. The terminal is the main location for the loading and offloading of passengers for this fleet of vessels. This location is highly susceptible to southern swells due to its location and orientation within the port; a swell coming from the south typically impacts the terminal un-interrupted and may lead to excessive ship motions that that are unsafe for both passengers and crew.

The Golden Shores Reserve, Queens Way Marina, Rainbow Harbor and Shoreline Marina are all marina-like facilities on the east side of the Los Angeles River. Vessels that depart from these facilities are both commercial and recreational. Shoreline Marina is also protected by a short detached breakwater that protects the marina entrance.

Belmont Pier approximately half the distance along the shoreline between Shoreline Marina to the west and Alamitos Bay to the east. This timber pile structure, originally constructed in 1915 and subsequently rebuilt in 1967, plays a minor role in altering the longshore transport of sediments. The structure alters the wave conditions and locally slows the transport rate. Due to the low wave environment provided by the offshore breakwaters and oil islands, this phenomenon is seldom observed.

The East Beach Bulkhead is a buried seawall with a crest elevation of 15 ft. MLLW that was originally constructed in the 1920's with the purpose of separating existing shoreline homes from the beach. The city of Long Beach currently maintains the beach in-front of the bulkhead with a backpassing operation that consists of mechanically moving sediments from the west to replace the transported sands.

The Alamitos Bay Jetties were originally constructed in 1933 and extended in the 1960's to their current length. These rubblemound structures are an almost perfect barrier to the littoral transport and delineate the boundaries to the San Pedro littoral sub-cells to be described later.

3.3 WATER LEVELS, TEMPERATURE AND CURRENTS

3.3.1 <u>DATUMS AND TIDES</u>

The west coast of the United States experiences mixed semi-diurnal tides. These tides are described by two high and two low tides of different levels which repeat approximately every 24 hours. Typically, a full cycle lasts 14 days when the higher high and lower low water levels are at their maximum.

Tide levels and vertical datums are taken from NOAA station 9410660, Los Angeles CA due to its proximity to the study area and long record of service (1923 – present). Table 3-1 shows the vertical datums calculated from the 1983 – 2001 tidal epoch; elevations are based on a standard station datum. The tidal range for the study area is approximately 5.5 ft.

Table 3-1: Tidal Levels and Datums Referenced from NOAA Station 9410660; Los Angeles, CA (NOAA, 2017a)

Datum	Value		Description
MHHW	9.32	ft	Mean Higher-High Water
MHW	8.58	ft	Mean High Water
MTL	6.67	ft	Mean Tide Level
MSL	6.65	ft	Mean Sea Level
DTL	6.57	ft	Mean Diurnal Tide Level
MLW	4.77	ft	Mean Low Water
MLLW	3.83	ft	Mean Lower-Low Water
NAVD88	4.03	ft	North American Vertical Datum of 1988
GT	5.49	ft	Great Diurnal Range
MN	3.81	ft	Mean Range of Tide
Maximum	11.75	ft	Highest Observed Water Level
Max Date & Time	1/10/2005 16:12		Highest Observed Water Level Date and Time

Expected water levels will exceed the stated elevations defined above over time. Using a GEV probability distribution, NOAA has calculated water level exceedance probabilities for both the high and low still water elevations as shown in Figure 3-7.

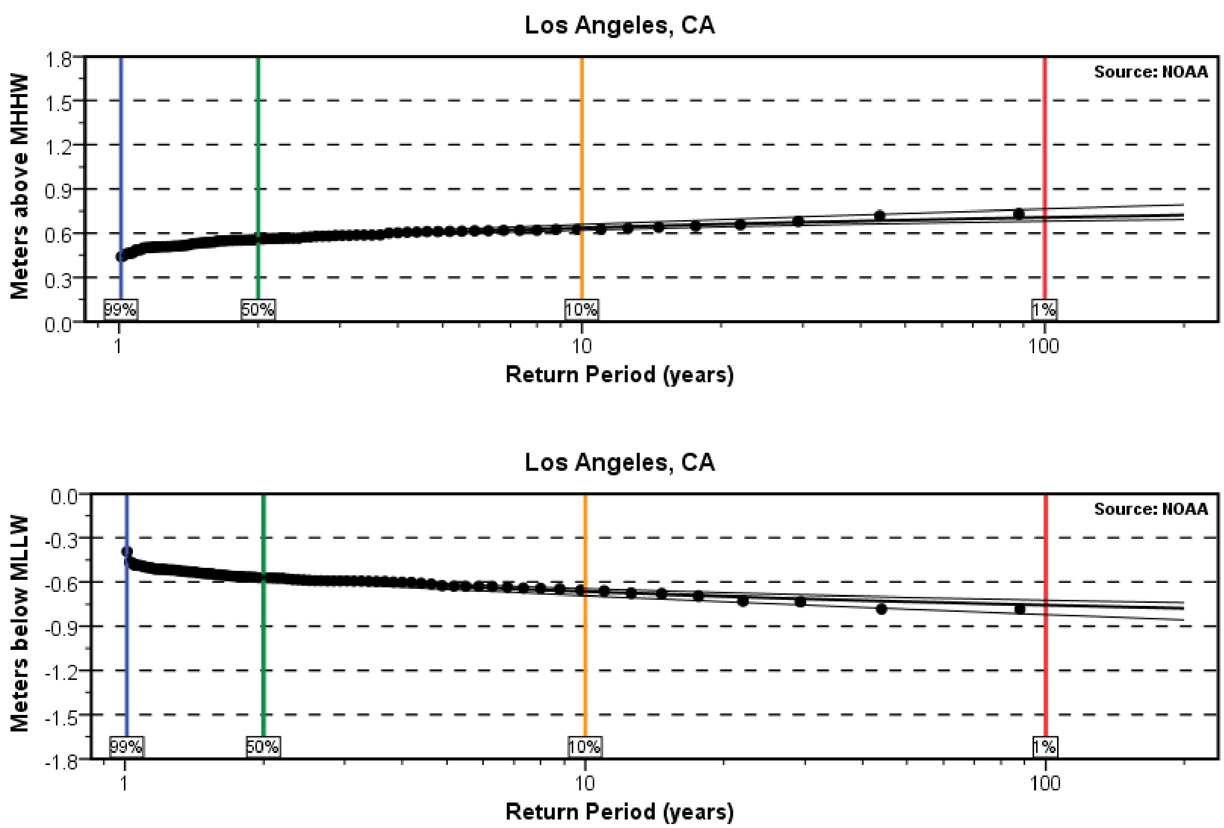


Figure 3-7: Exceedance Probabilities for High and Low Still Water Elevations (NOAA, 2017a). Note: units in meters

3.3.2 SEA LEVEL CHANGE

The sea level rise in the southern California region associated with ocean thermal expansion and the meltwater generated from continental glaciers and the Antarctic ice sheet is estimated to be 0.1 to 0.2 ft in a time span of 25 years (Church et al., 2004; USACE, 1991). This correlates to an approximate potential increase of 0.004 to 0.008 feet of mean sea level elevation per year. However, the historic trend within the harbor suggest the actual rate is on the low end of this estimate as shown by Figure 3-6, which provides a historic rate of sea level change to be 0.0031±0.0007 ft/yr for the period between 1923 and 2017 (NOAA, 2017a).

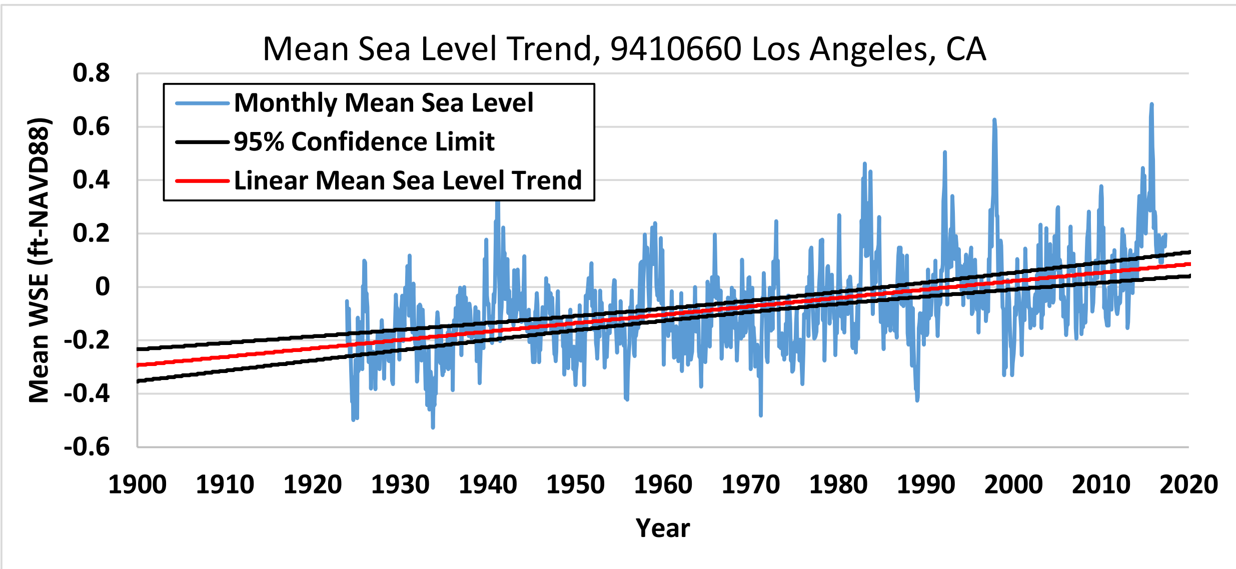


Figure 3-8: Mean Water Surface Elevation Measured at NOAA Station 9410660

The U.S. Army Corps of Engineers considers potential relative sea level change in every feasibility study undertaken in the coastal zone. ER-1100-2-8162 (USACE, 2013) provides a guideline to determine the potential sea level changes to consider. A sensitivity analysis should be conducted to determine what effect, if any, changes in sea level would have on plan evaluation and selection. The sea level increase scenarios can be described by:

$$E(t_1) - E(t_2) = 0.0031(t_2 - t_1) + b({t_2}^2 - {t_1}^2) \qquad (1)$$

where $E(t)$ is the eustatic sea level, t_1 is the time between the project's start date and 1992[1], t_2 is the time between a future date of interest and 1992 and b is a constant that changes depending on the rate of sea level rise considered; 0, $2.71\text{x}10^{-5}$ and $1.13\text{x}10^{-4}$ for low (historic), intermediate and high respectively. The projected relative sea level change, based on the local observations at the NOAA

[1] 1992 is the midpoint of the National Tidal Datum Epoch from 1983 – 2001.

station, is shown in Figure 3-7. Over a 50-year time span, approximately 2080, the low, intermediate and high scenario would produce a relative sea level rise of 0.14, 0.7 and 2.5 feet from the mean sea level at a base year of 2030, respectively. Similarly, over the 100-year planning horizon of the project, approximately 2130, the low, intermediate and high scenarios will produce sea levels increases of 0.3, 1.8, and 6.8 feet from the mean sea level observed in 2030. Although sea level changes are typically calculated from the time the baseline datum is analyzed, it is more simple to show the change from the base year of the project, otherwise there would be three different sea levels at the start of the project which would not be true in reality. Design water levels, to be discussed in the DESIGN CONDITIONS chapter, are based on predicted absolute sea level change from the 1992 datums.

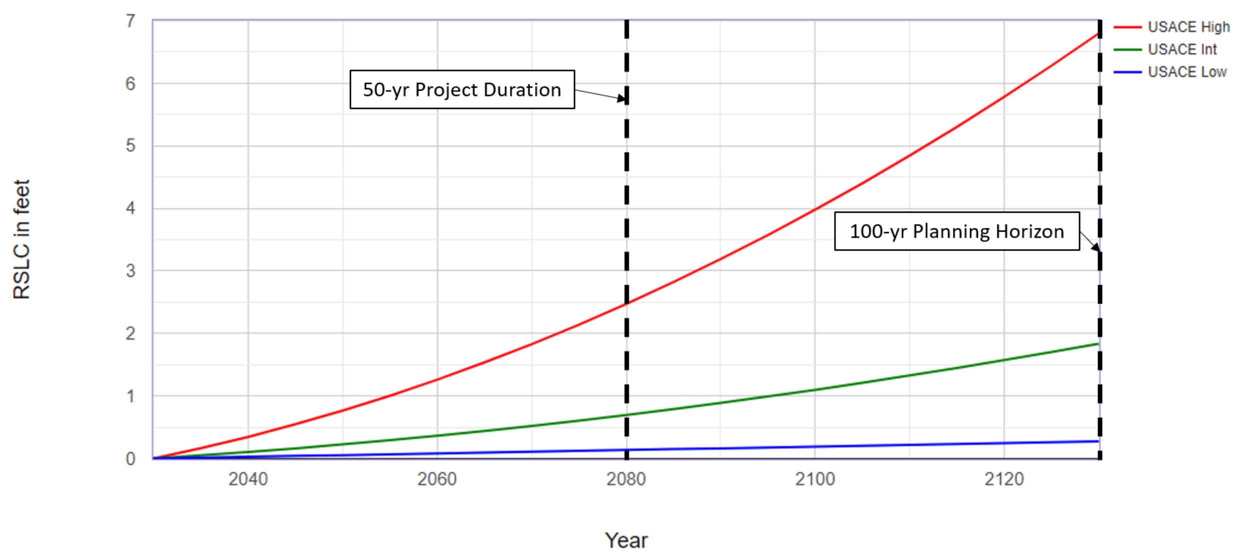


Figure 3-9: Relative Change in Sea Level for Various Scenarios

3.3.3 <u>EL NIÑO SOUTHERN OSCILLATION</u>

El Niño Southern Oscillation (ENSO) is a large scale periodic event in the Pacific Ocean that changes the conditions over the entire ocean basin. A detailed description is beyond the scope of this report, but the cycle typically repeats every 4 – 7 years. The effect on the study area will temporarily raise both the sea surface temperature (SST) and still water level (SWL) during a el Niño event and lower the SST for a la Niña event, each typically lasting less than a year. The rise in sea surface can be up to 1 ft. as shown in the comparison between the predicted and actual measurements at the Los Angeles Harbor tidal gauge shown in Figure 3-8.

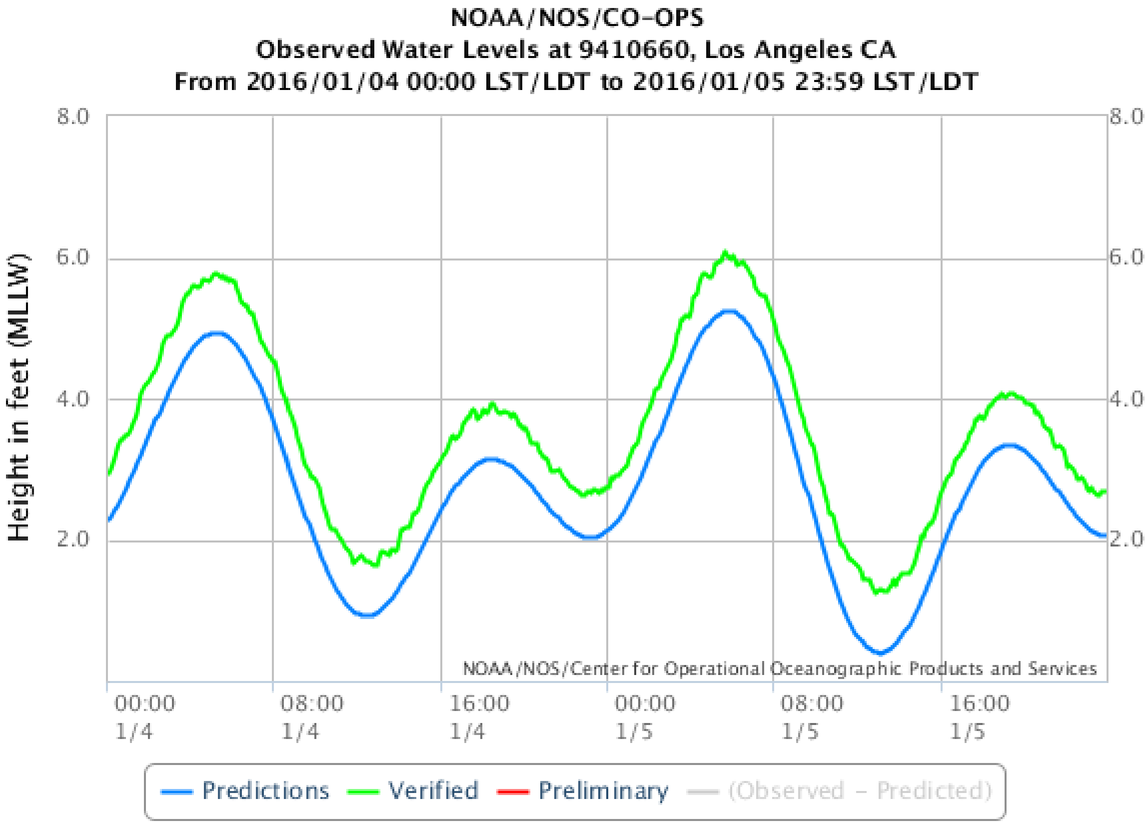


Figure 3-10: Increase of Water Level due to ENSO Event

The change in temperature as a running three month mean sea surface temperature (SST) anomaly is also known as the Oceanic Niño Index (ONI). Events with five consecutive three month periods with SST anomalies at or greater than 0.5°C (0.9°F) are considered el Niño while SST anomalies less than 0.5°C are considered la Niña events.

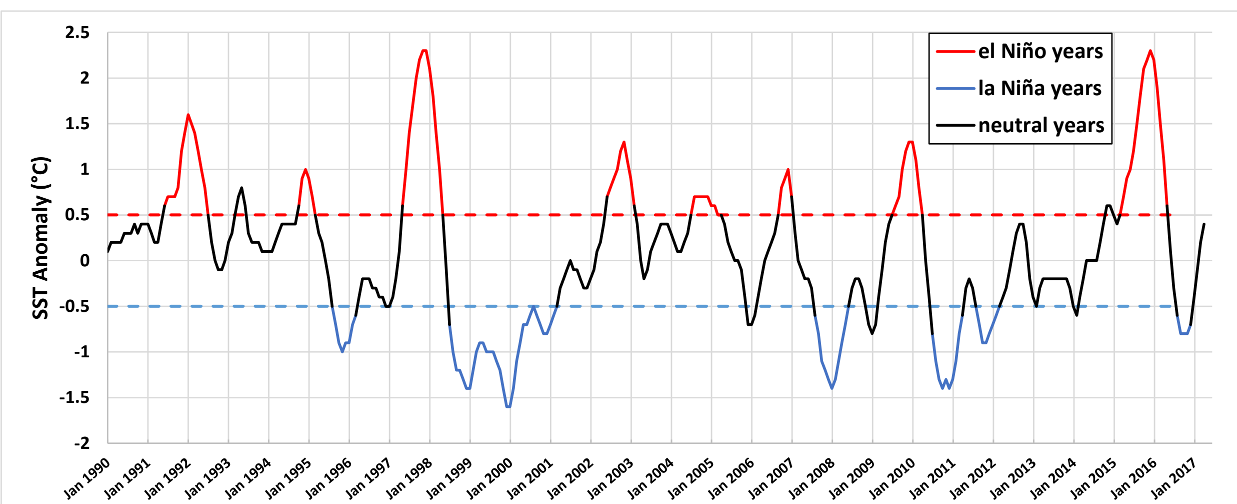


Figure 3-11: Oceanic Nino Index (NOAA, 2017b)

It can be seen from Figure 3-9 that strong el Niño years since the 1990's are 1997-1998 and 2015-2016 which cause the largest increase in SWL. La Niña years are less impactful to the study area since the change in water level is minor. The ENSO cycle effects much more than the SST and SWL such as a change in weather patterns throughout the Pacific Ocean and adjacent continents. This cycle is considered a coupled ocean-atmospheric phenomenon (NOAA, 2017b).

3.3.4 WATER TEMPERATURE

Water temperature in the Southern California Bight is primarily caused by two factors:

1. Air temperature
   - The difference between the water temperature and air temperature causes a transfer of heat.
2. Upwelling
   - This complex process is caused by winds blowing from the North. The North-South direction of the wind blowing over the oceans causes a phenomenon known as Ekman transport where surface waters are 'pushed' to the west causing cold and deep, nutrient rich waters to replace it.

From the local NOAA tidal gauge (9410660), the average water temperature can be determined for each month. From Figure 3-10, it can be seen that temperatures are generally lower in the winter months. Due to the two above listed processes, extreme water temperatures typically stay in a 10°F band with the occasional outlier. This data does include ENSO events, which can account for temperature swings of up to 7°F from one year to the next.

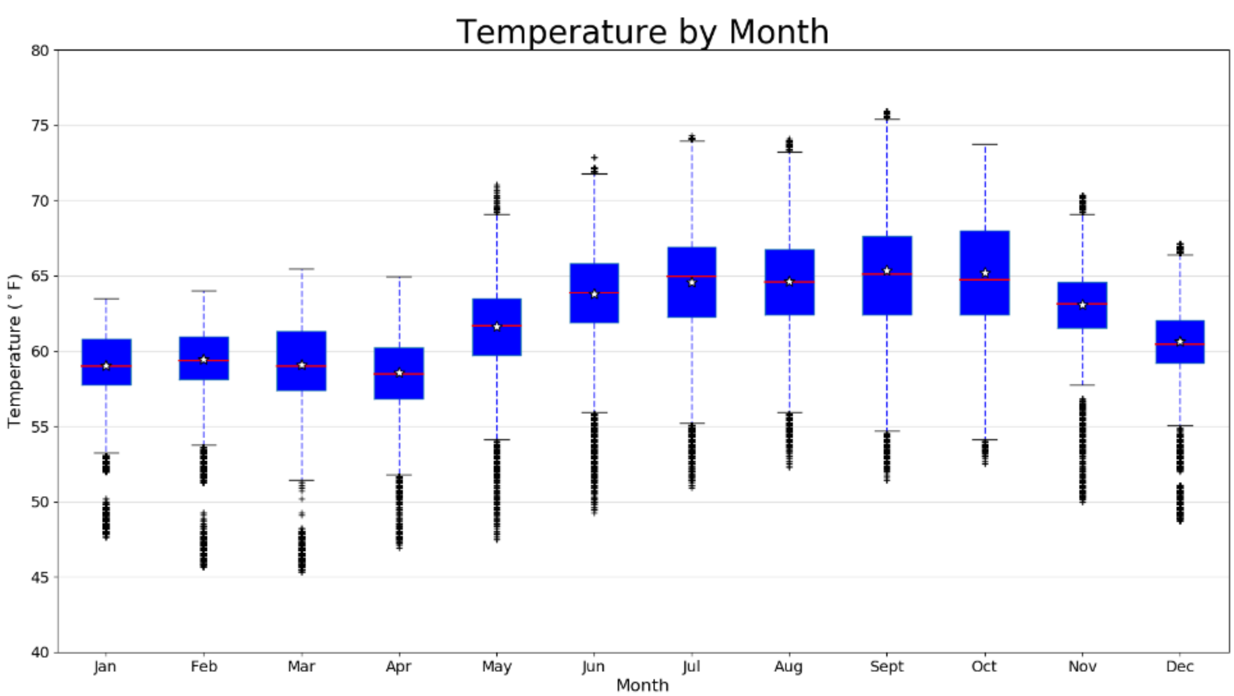


Figure 3-12: Monthly Water Temperatures at Los Angeles NOAA Gauge from 1995 - 2017

The offshore currents consists of major, large scale coastal currents which constitute the average seasonal circulation and tidal (and event) fluctuations of 3 – 10 days which are expected to be superimposed on the seasonal circulations. Hickey (1979) defines the constituents of the large scale California coastal currents:

1. The California Current: the southward flow of water off the coast with a mean speed of approximately 0.25 – 0.5 knots (Schwartzlose and Reid, 1972).
2. The California Undercurrent: A subsurface northward flow that occurs below the main pycnocline and seaward of the continental shelf typically on the order of 0.1 – 0.2 knots (Schwartzlose and Reid, 1972).
3. The Davidson Current: A northward flowing nearshore current associated with winter wind patterns north of Point Conception. From drib bottle records, the Davidson current attained speeds as high as 0.3 – 0.6 knots (Schwartzlose and Reid, 1972).
4. The Southern California Countercurrent (also known as the Southern California Eddy): A northward flow in the Southern California Bight south of Point Conception and inshore of the Channel Islands. This strong current can have speeds during winter months as high as 0.7 – 0.8 knots (Maloney and Chan, 1974)

Local structures do not play a role in the large-scale currents. The study area is generally sheltered from the offshore currents by Point Fermin. It is assumed that anything leaving San Pedro Bay will be transported away from the study area by the Southern California Countercurrent.

3.4 WAVE CLIMATE

3.4.1 WAVE ORIGINS

In Southern California, waves typically originate from five basic meteorological patterns (USACE, 1996) as illustrated in Figure 3-11 and described below. Much of the study area is sheltered from deep ocean waves by the protective breakwaters (San Pedro, Middle and Long Beach) and the port complex itself.

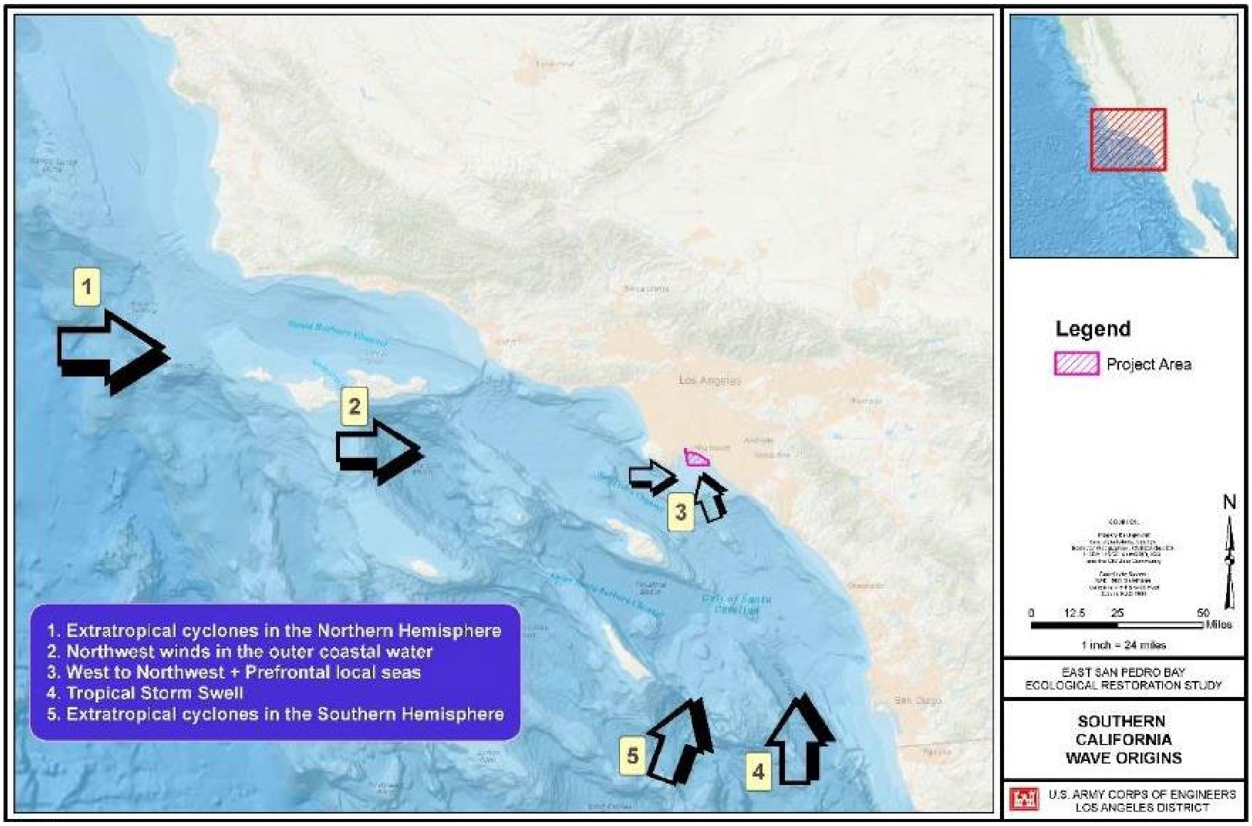


Figure 3-13: Southern California Wave Origins

3.4.1.1 EXTRATROPICAL CYCLONES IN THE NORTHERN HEMISPHERE

Low pressure centers which develop along the polar front are the source of the predominant wave action along the entire coast during the winter months. Storm swell is generated at some distance from the coastline in the North Pacific. Most commonly these storms will traverse the mid-Pacific before turning northeastward toward the Gulf of Alaska with swell decaying during the more than 1,500 miles of propagation distance to the coast of Southern California. However, under some meteorological conditions, storms can move in much closer to the coast; and on rare occasions these storms may move directly across Southern California, following either a northeast, east or southeast trajectory.

In general, the modal deep water approach direction of these waves to Southern California is between 275° and 285°. However, these North Pacific low pressure systems exhibit great variations from year to year such that wave approach directions and amplitudes will show a corresponding degree of variation. Years when the storm centers follow a more northerly route in the eastern Pacific will result in extremely quiet conditions in the study area, whereas more southerly storm tracks through the mid and/or eastern Pacific will result in frequent periods of high wave conditions.

Meteorological characteristics of this weather pattern within the eastern Pacific may be broadly separated into five types defined by the position and amplitude of troughs and ridges, air mass characteristics, split flow in the jet stream and position of blocking high pressure centers. These five types are further delineated as follows.

- Type A:

This weather type occurs frequently during the late winter and spring and may be identified by a strong mid-latitude high centered near 135° to 140° West with storms tracking southeastward from the Gulf of Alaska. These storms normally move inland well to the north with low pressure forming in the lee of the Sierra combining with the offshore high to give strong northwesterly winds. Depending on the position of the jet stream, these winds may extend all the way to the study area coastline or be limited to the outer coastal waters. On occasion, the storm center will drop southward off the coast giving a southeaster rather than a northwester.

During the summer months this type of meteorological pattern is also prevalent, but the surface low is usually not present and is reflected only by a cold trough in the upper air. As marine air advents into the San Joaquin Valley and the thermal low shifts to the lee of the mountains, strong northwest winds develop in the outer coastal waters.

- Type B:

A variety of sub-types exists within this pattern, some with a zonal or east – west flow and others with a more meridional flow. Common to them all, however, is the mean ridge position which is along or near the west coast. Swells which reach Southern California will have been generated within the mean trough position which is located 1,500 to 2,000 miles away. This is the most common type during the fall and early winter. It is associated with the clear and sometimes warm Southern California days. Most of the swells generated during this weather pattern arrive within the deep water direction sector of 285° to 300°. As such, island sheltering significantly reduces wave energy arriving in the study area. The more severe mid-Pacific storms, however, will produce some larger waves even from this semi-protected sector.

- Type C:

This type is associated with a split jet stream, one branch lying far to the north, the other extending west from Southern California. In between lies a broad belt of high pressure which covers the Pacific Northwest, the southern Gulf of Alaska and further on to the west-southwest. Storms which move eastward at a rather slow rate in the southern branch of the jet are often major rain producers in Southern California. These storms, however, are usually small and rarely intense, although, westerly swells are usually not high, their approach direction is very often optimum for impacting the study area.

This type of meteorological pattern tends to be infrequent in most winter seasons. In some winters, this becomes one of the predominant patterns; but is apparently el Niño dependent. This was observed in both the 1991 – 1992 and 1992 – 1993 seasons.

- Type D:

As a wave producer, this is the least important weather type for Southern California in the context of wave energy. It is marked by a large quasi-stationary low centered off the Pacific Northwest capped by an extensive, crescent-shaped high covering much of the Gulf of Alaska and western Canada. Swells generated in the southwest sector of the storm are too northwesterly for any appreciable effect on the study area. The front associated with the low will usually reach Southern California, giving southeast winds which are rarely very strong.

- Type E:

This type is associated with the major wave events of Southern California. There are actually four sub-types, two of which produce tracks far to the south. The storms of this type are often linked to a strong blocking high in the Gulf of Alaska, which feeds cold air southward to phase with a storm which has penetrated the ridge at low latitudes.

This rapidly intensifying system moves eastward with associated strong westerly winds pointing directly at Southern California. The low pressure center usually turns northeastward toward northern California, and waves decay about 600 to 800 miles off the coast of the study area.

3.4.1.2 NORTHWEST WINDS IN THE OUTER COASTAL WATERS (WIND SWELL)

Annually, the predominant wave action in Sothern California coast is due to the prevailing northwest winds located to the north and west of the study area. This is particularly true during the spring and summer months. Wave heights are usually low, less than 3 feet; but on occasion, with superposition of a strong surface high and an upper level trough, the northwesterlies increase, becoming very strong from about Pt. Sal to San Nicolas Island. The inner waters of Southern California very often remain unaffected under the influence of the Catalina eddy circulation. Waves traveling at a variance to the mean wind direction reach the study area with periods on the order of 6 to 10 seconds. Moderate northwesters will produce breaker heights of 4 to 6 feet, while strong events can give heights ranging from 6 to 9 feet.

3.4.1.3 WEST TO NORTHWEST (WEST SEAS) AND PRE-FRONTAL (SOUTHEAST SEAS) LOCAL SEAS

<u>West Seas:</u>

Westerly winds can be divided into two types: (1) temperature-induced sea breezes, and (2) gradient winds. The former exhibits a pronounced seasonal and diurnal variation. The strongest sea breezes occur during the late spring and summer months, while the lightest winds are during December and January. The summer sea breezes usually set in during the late morning and peaks in the mid-afternoon. In winter months, sea breeze conditions are limited to a few hours during early afternoon with wind speed on the order of 10 knots. The summer sea breezes, on the other hand, will average about 15 knots and can occasionally reach 20 knots or more.

Gradient winds are confined largely to the months of November through May with the peak in March and early April. They typically occur following a frontal passage or with the development of a cold low pressure area over the southwestern United States. The latter produces the strongest winds with durations of up to 3 days. Under such conditions, locally generated wind waves combine with components of the northwest swell generated off the California coast.

<u>Southeast Seas:</u>

The study area is particularly vulnerable under storm conditions prior to frontal passage winds blowing strongly from the southeast along the coast but turning toward the south-southeast to south a short distance offshore. Wind waves, with peak energy averaging between 6 and 8 seconds, reach shore with minimal loss due to island sheltering or refraction. Significant wave heights are generally in the range of 4 to 8 feet. Extreme wave heights are rare, because the fetch and often the duration of these wind waves are short. Most of the westerly swells tend to follow the frontal passage, when southeast seas are well on the way down. On occasion there will be some overlap, but it would be an extremely rare event for the two to peak concurrently.

3.4.1.4 TROPICAL STORM SWELL

Tropical cyclones form regularly along the intertropical convergence zone (ITCZ) west of Mexico from early July to early October. On the average, about 15 to 20 of them are to be expected each year. Most of them take a westerly track, and swells generated by these storms will have little or no effect on Southern California. Some, however, take a northwest track, thereby lengthening the effective fetch over which swells traveling toward Southern California can be generated. The critical direction of approach for the study area is 155°; anything east of that direction is blocked by Baja California, while anything west of that direction reaches the study area unimpeded. San Clemente Island becomes a factor at about 190° to 200°, but usually by the time a tropical storm has progressed that far northwestward it has long since begun to weaken or is moving on a westward track and is no longer generating swell events that propagate to the study area.

3.4.1.5 EXTRATROPICAL CYCLONE OF THE SOUTHERN HEMISPHERE (SOUTHERLY SWELL)

From the months of April through October, and to a lesser extent the remainder of the year, large South Pacific storm systems traversing the ocean between 40° and 60° south latitude from Australia to South America send swells northward to the west coast of Central and North America. Great circle approach directions off Southern California range from about 215° for storms near New Zealand to 170° for South American storm systems. The decay distance ranges from about 4,500 to 7,000 miles. Wave heights in deep water are usually low, on the order of 1 to 3 feet; however, since these waves are nearly monochromatic, their capacity for all waves shoaling in a similar manner is enhanced. Breakers of 8 to 10 feet on beaches to the south of the study area are not at all uncommon. Refraction effects are important as well, accounting for a high degree of selectivity in some areas.

3.4.2 <u>DEEP WATER WAVE CHARACTERISTICS</u>

Natural sea waves are random in nature. In order to represent these natural waves, an energy spectrum is considered. A spectrum can be thought of as a sum of many harmonic waves, each with a constant amplitude and phase, randomly chosen for each observance of a true record. Typically, a spectrum is constructed from a time series of recorded waves lasting 20 – 30 minutes. A Discrete Fourier Transformation is applied to the time series data which allows for wave statistics to be extracted from the record. These statistics can adequately describe the current sea state as a series of single discrete values, rather than an entire time series and are simpler to use for design/prediction purposes.

No new wave data will be collected during the feasibility stage of this study. Wave information will be collected from existing sources, as described below, and conclusions will be drawn based on this previously collected data.

Offshore wave conditions are furnished by the Coastal Data Information Program (CDIP), Integrative Oceanography Division, operated by the Scripps Institution of Oceanography, under the sponsorship of USACE and the California Department of Parks and Recreation. The San Pedro, CA (092) CDIP buoy, that lies approximately 12 miles offshore of the project area in 1500 ft. of water is a Datawell direction buoy which measures displacement in three directions and reports zeroth moment wave height (H_{mo}), peak and mean wave period (T_p and T_m) and peak direction (D_p) from the calculated directional energy spectrum. The zeroth moment wave height is very close the significant wave height (H_s) in deep water but is calculated from the directional energy spectrum. As the wave propagates into shallower water, these two values begin to deviate. Half hour wave conditions have been reported by this CDIP buoy since 1998 with sporadic non-directional data points going back the 1980's that does not include direction.

A numeric Wave Information Studies (WIS) buoy, 83101, is also in a near the study area in a similar position and water depth. Data produced by this program utilizes hindcast wind data recorded and various locations around the world and is used to develop a distant wave field. This wave field then propagates away from the origin and interacts with other wave and storm systems until finally reaching the virtual buoy. Similar wave data to the CDIP buoys is reported, but for a longer timeframe since hindcasted data is used. Continuous hourly sea state conditions have been calculated from 1980 – 2011.

Both of these wave records report significant events that allow for two separate analysis; the typical conditions within the project area and the extreme values that may be observed during the project duration. For brevity, only CDIP data will be used throughout this study, but similar conclusions would be obtained by using WIS data. The CDIP system records actual sea states, rather than calculating from the hindcasted wind field and propagating the energy to the project area, leading to more accurate data. Although the WIS system has been validated many times over, the length of record of the CDIP buoy, 18 years, is enough to conduct an extremal analysis and draw conclusions for the expected project life of 50 years.

Figure 3-12 to Figure 3-14 present the wave height, period and directional joint probability for the CDIP buoy, 092. Wave data is averaged over a three hour running mean and grouped into bins of a specific width. The probability of occurrence corresponding to the bins is plotted as shown in the figures. The upper left and lower right plot in each figure show the individual frequency of occurrence for the stated parameter and the lower left plot shows a heat map based on the total number of occurrences.

Figure 3-12 illustrates the occurrence of wave height in relation to period. It can be seen that the majority of wave heights range from 2.5 to 3.5 ft and closely follow the Raleigh Probability Density Function since the buoy is considered to be in deep water for all but the longest period waves. From the period plot on the bottom right, the dominate period range is from 12 – 18 seconds, which indicates long period swell waves that are generated away from the project site. The lower period values, 5 – 8 seconds suggest the presence of locally generated wind waves that are produced near the buoy. Even though these waves generally have a similar, but overall lower, wave height to the longer period swell waves, the short period indicates a great reduction in wave power and ability to produce large velocities and sediment transport along the sea floor.

Figure 3-13 demonstrates the relationship of wave height to direction. All the same aspects described above hold true for the wave height. Wave origins consist two predominate directions, the south (~180°) and the northwest (~280°); as seen by the two large spikes in the direction plot. These locations correspond to the tropical storms generated near the equator and extra-tropical storms in the Southern Hemisphere during the summer months and extratropical storms generated near the Aleutian Island during the winter months. It can also be seen that the waves generate from the northwest are typically larger due to the increased storm energy, duration and lower decay potential due to the shorter propagation distance.

Figure 3-14 shows the distribution of period and direction. The waves generated from the south typically have a longer period due to the increased propagation distance from the southern oceans to the project site. Two main components originate from the northwest; the extratropical events and locally generated wind waves. The extratropical swells have a significantly longer period than the locally generated waves.

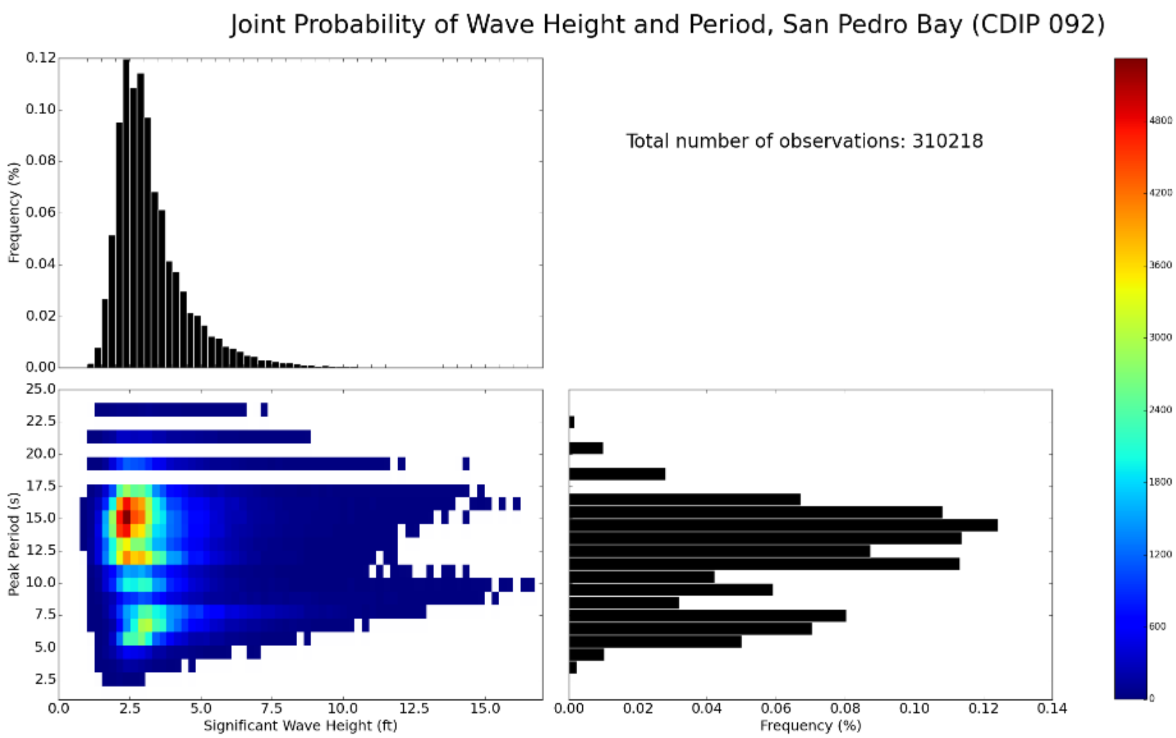


Figure 3-14: Joint Probability for Wave Height and Period

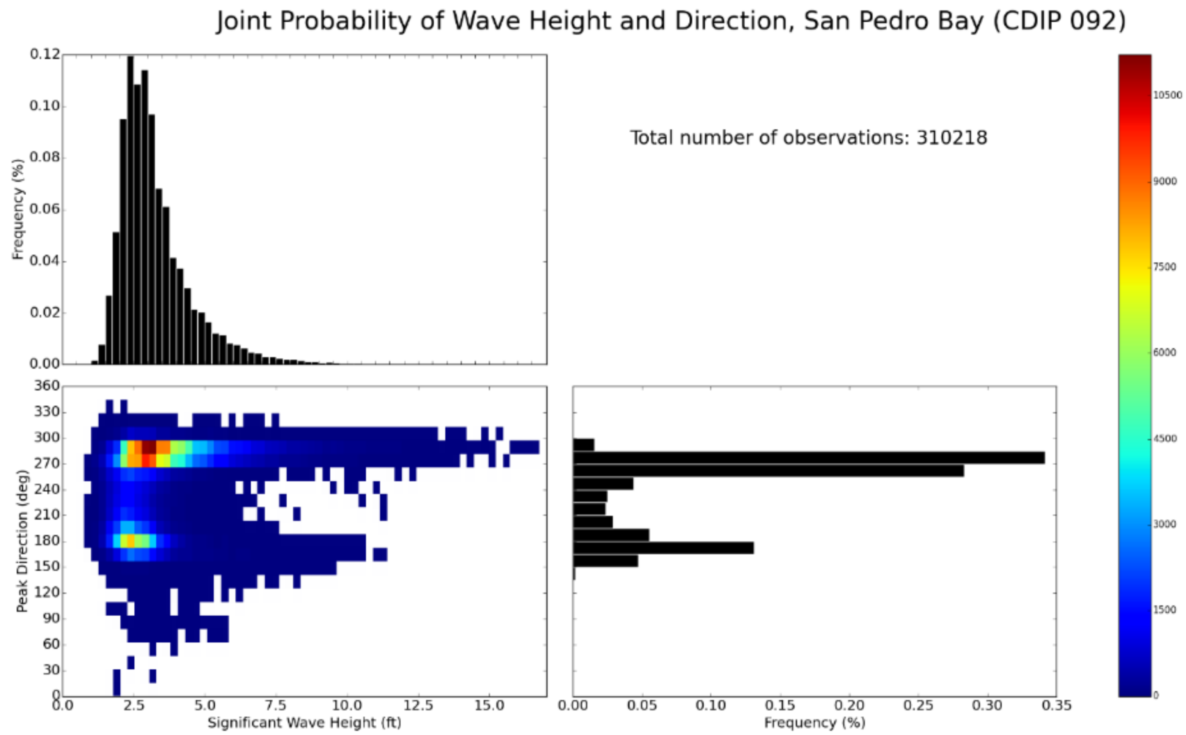


Figure 3-15: Joint Probability for Wave Height and Direction

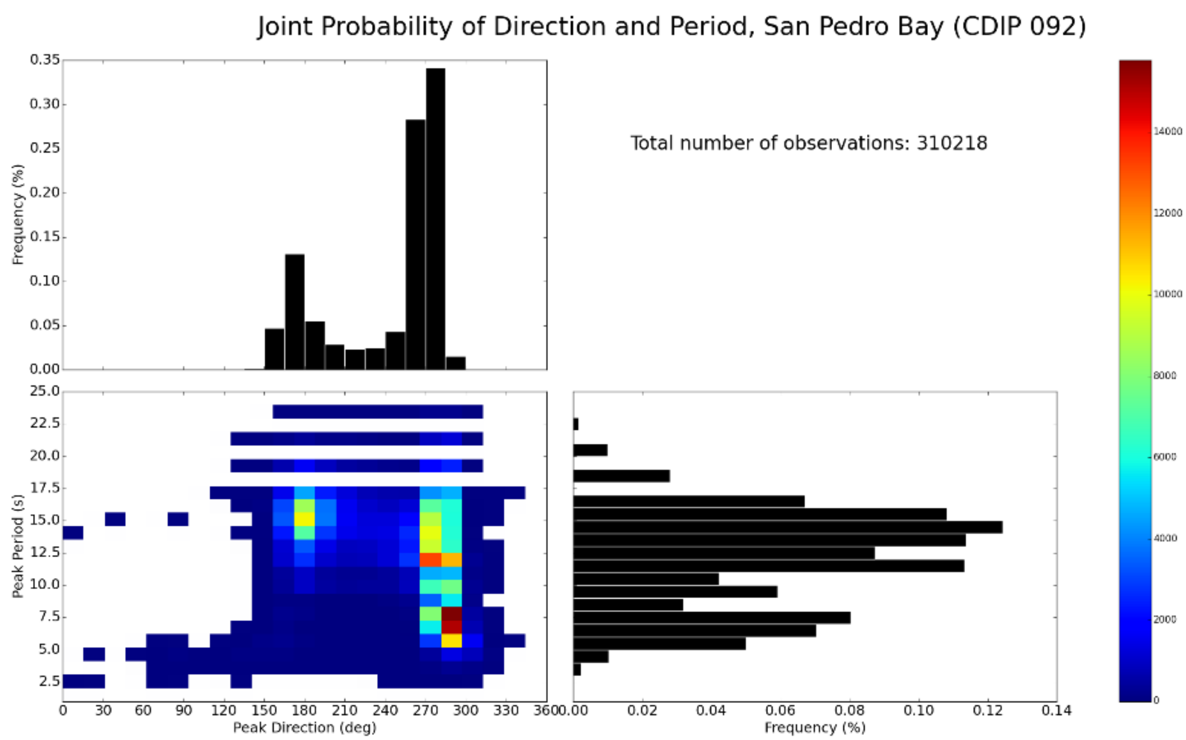


Figure 3-16: Joint Probability for Period and Direction

A summary of the recorded waves at the buoy location is shown in Table 3-2. This summary provides a good overview but does not provide adequate information to determine the yearly distribution of these events or the return period of the recordings.

Table 3-2: Wave Summary Statistics

Total number of wave records	310218	
Average wave height	3.3	ft
Standard deviation of wave height	1.4	ft
Average wave period	11.9	sec
Standard deviation of wave period	3.6	sec
Maximum wave height	16.8	ft
Period associated w/ max wave height	9.1	sec
Direction associated w/ max wave height	274	°
Date associated w/ max wave height	Feb 1, 2016	

For this study, two different wave analyses must be considered; extreme conditions to ensure structural stability of any design and typical conditions to indicate how the system is reacting on a daily basis. Figure 3-15 to Figure 3-17 present a box plot of the entire length of buoy record sorted by month. These plots show the mean and median value for each month represented by the star and red line respectively. The blue box represents one standard deviation from the mean on each side, the black bars show three standard deviations away from the mean and the black crosses show the outlier data points.

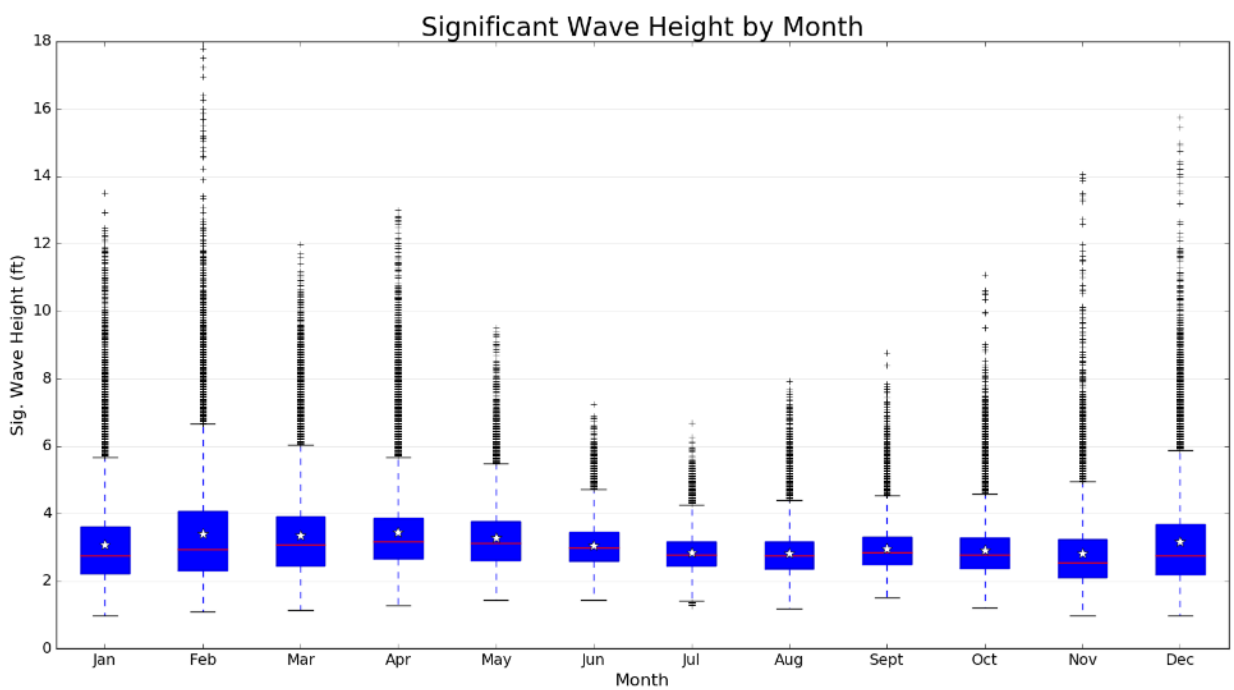


Figure 3-17: Average and Outliers of Significant Wave Height of Buoy Record

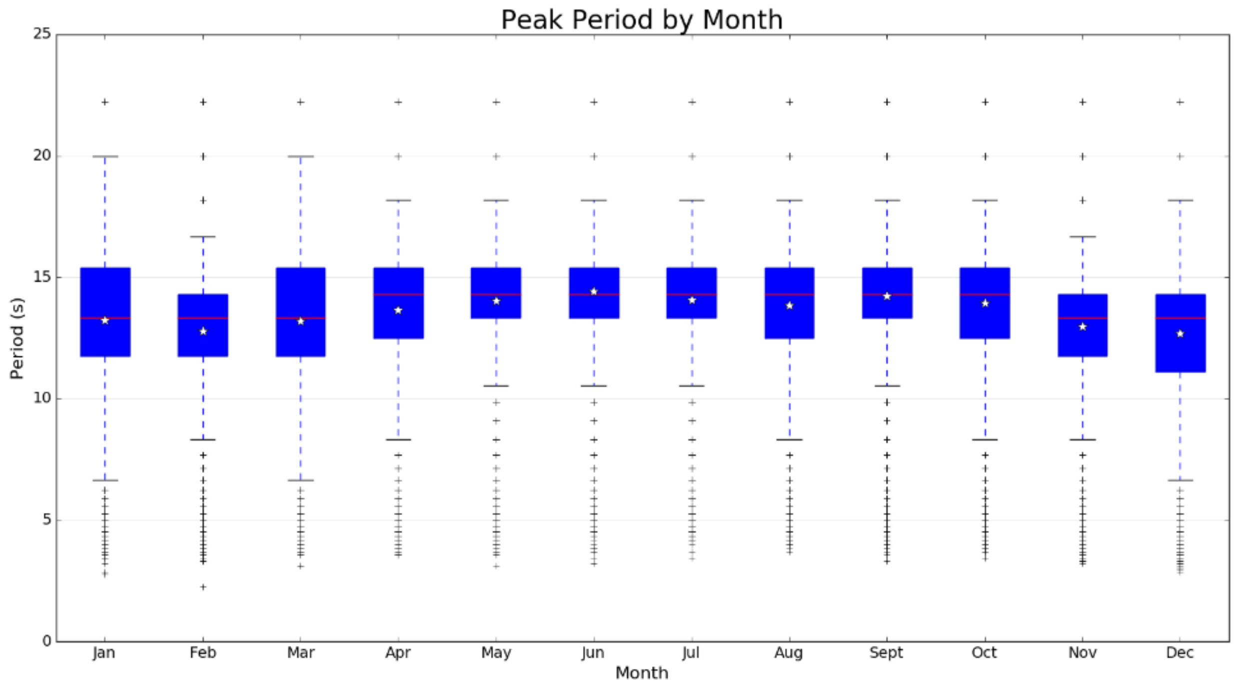


Figure 3-18: Average and Outliers of Peak Period of Buoy Record

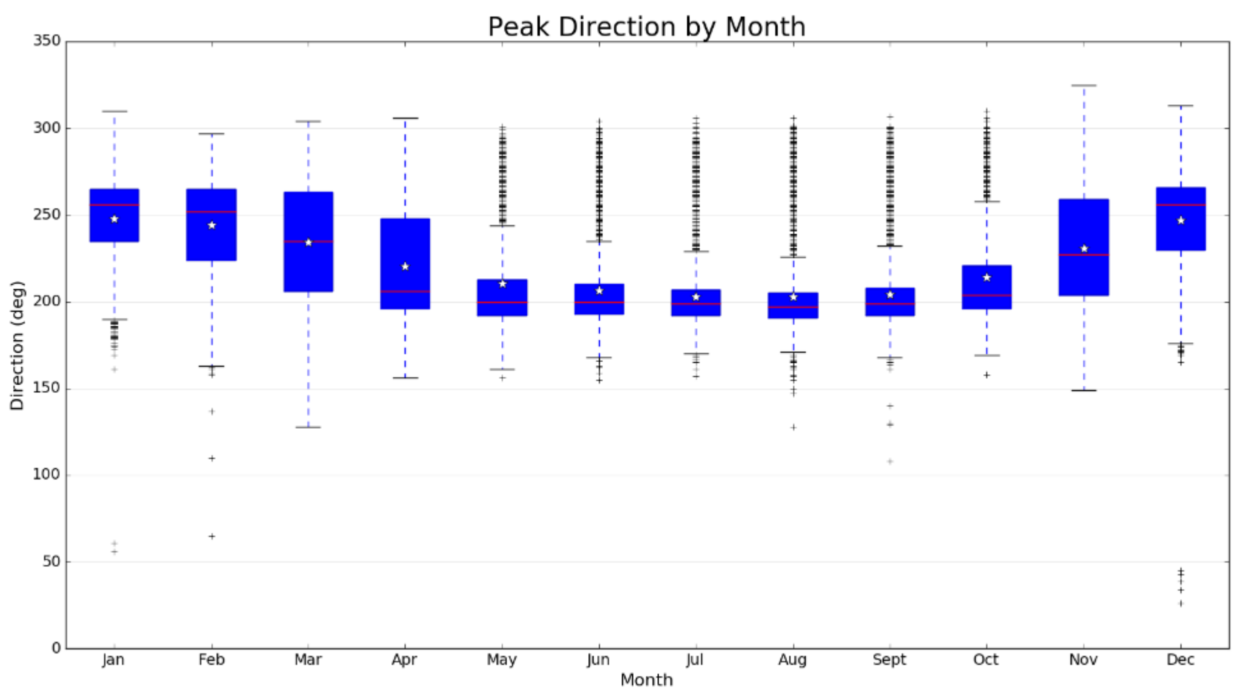


Figure 3-19: Average and Outliers of Peak Direction of Buoy Record

Using the averages of the above parameters lead to a determination of typical conditions for both the summer (May – October) and winter (November – April) months and shown in Table 3-3.

Table 3-3: Typical Wave Conditions for Summer and Winter Months

Condition	H_s (ft.)	T_p (sec)	D_p (°)
Summer	2.9	11.3	225
Winter	3.7	12.4	260

3.4.3 LONG TERM WAVE STATISTICS

Long term wave statistics are developed for the recorded waves at San Pedro Buoy to determine the return period of the extreme wave events. Although there are numerous classifications of waves in Southern California, as discussed previously, determining the exact type based on just the buoy record, that includes effects such as island sheltering, is tedious and may lead to grouping errors. For this study, the waves are subdivided only into a Northwest swell, South swell and local seas with parameters shown in Table 3-4.

Table 3-4: Parameters for Wave Events Division

Event	Peak Period	Peak Direction
Northwest Swell	> 8 sec	> 245°
South Swell	> 8 sec	< 245°
Local Seas	< 8 sec	Any

Events with a peak wave height larger than 6.5 ft. are extracted from the wave buoy recordings and grouped into one of the three wave conditions. Discrete events are not expected to last more than 2 days. A Weibull distribution is utilized to determine the return periods and confidence intervals of extreme storm events (Goda, 2010). Figure 3-18 to Figure 3-19 show the return period of the above-mentioned wave conditions along with the 95 percent confidence interval. A comparison of the CDIP and WIS data is shown in these figures. The CDIP records suggest more intense events from the north and less intense event from the south. The difference from the WIS calculations is most likely from insufficient sheltering effects from the offshore Channel Islands. The model used by the WIS system does not account for all the physics of wave transformation; so, some energy in deep water is not correctly propagated to the continental shelf. Although, these two data sources produce similar extremal results, the more conservative during the more extreme northern swell events along with the actual in-situ record, the CDIP data source will be solely used. Additionally, the shorter recording interval of CDIP, 30 minutes, provides additional information that is not reported with the 1 hour, WIS timestep.

The Northwest swell event produces the largest wave height followed by the South swell event then local seas. A large south swell event is rare as indicated by the small number of records, but since the study area is susceptible to this direction, it is important to include separately. Local seas will be ignored for this analysis since the longer period swells will have a greater effect on the study area. In this location, locally generated waves rarely cause damage.

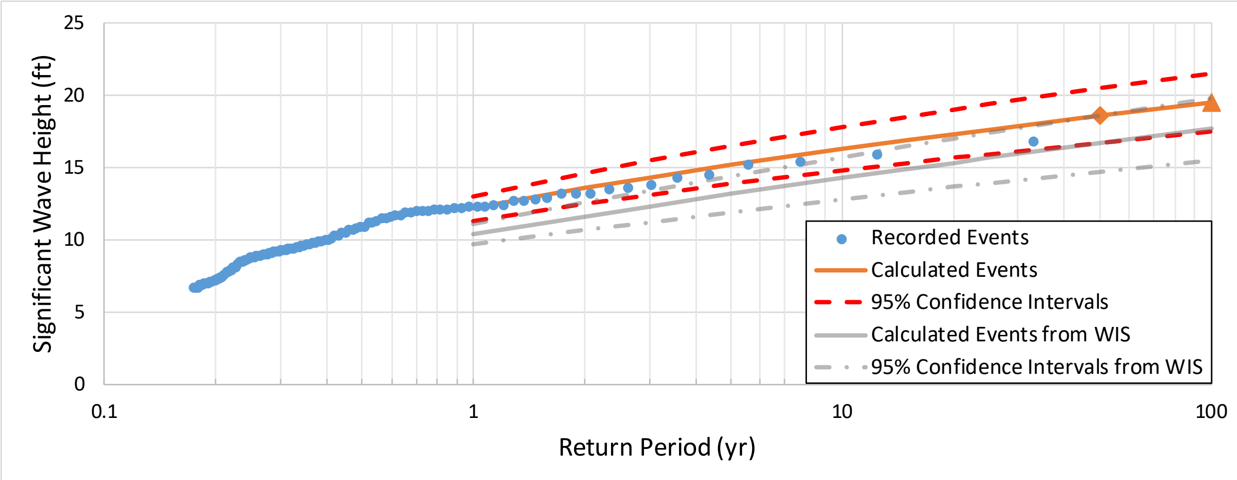


Figure 3-20: Return Period of a Northwest Swell

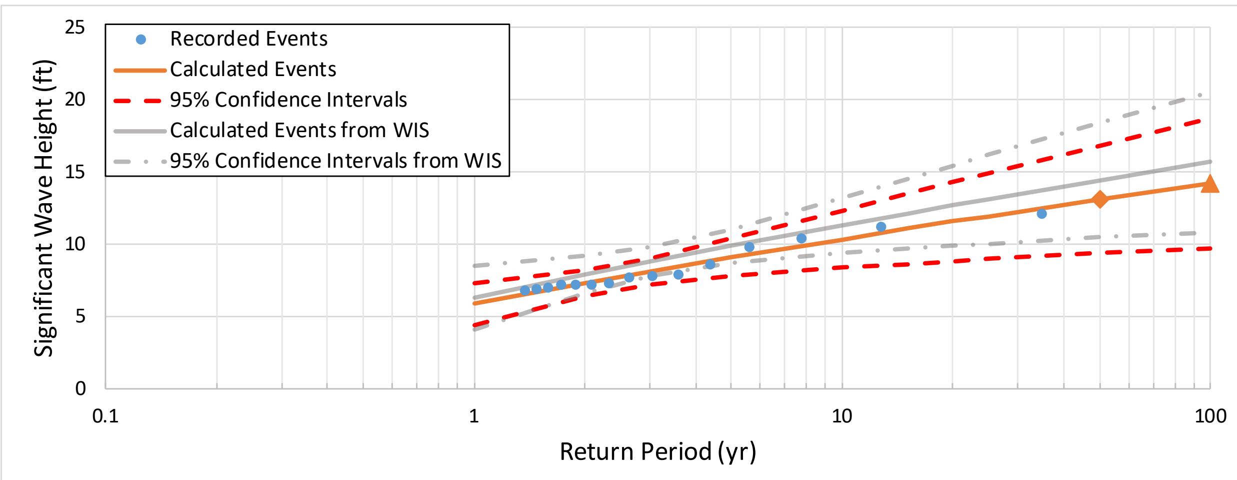


Figure 3-21: Return Period of a South Swell

Table 3-5 summarizes the return periods of the different event groups. The wave period and direction is assumed by investigating discrete events over the buoy record; the most common values for these storm events are used.

Table 3-5: Extreme Events Based on Recorded Buoy Data

Event Type	T_p (sec)	D_p (°)	1-yr Return H_s (ft.)	50-yr Return H_s (ft.)	100-yr Return H_s (ft.)
Northwest Swell	18	270	12.4	18.6	19.5
South Swell	16	180	5.8	13.1	14.2

3.5 LITTORAL PROCESSES & SEDIMENT COMPOSITION

The San Pedro littoral cell stretches approximately 31 miles from the headlands at Palos Verdes to Corona del Mar, near the entrance at Newport Bay. Due to the anthropogenic activities, such as the construction of the port of Los Angeles/Long Beach complex, oil islands and protective breakwaters, the natural sediment transport regime has been impacted which leads to the discretization of the littoral cell into three semi-independent sub-cells:

1. Los Angeles/Long Beach sub-cell
    - 9 miles from Point Fermin to the Alamitos Bay East Jetty
2. Seal Beach sub-cell
    - 1 mile from Alamitos Bay East Jetty to Anaheim Bay West Jetty
3. Huntington Beach sub-cell
    - 19 miles from Anaheim Bay West Jetty to the rocky headlands at Corona del Mar.

For this study, only the LA/LB and Seal Beach sub-cells will have a direct effect on the project area. The sediment transport potential is greatest for the Huntington Beach sub-cell due to the orientation and lack of protective structures, followed by the Seal Beach then finally the LA/LB sub-cells.

3.5.1 SEDIMENT SOURCES

3.5.1.1 RIVERS

Two main sediment sources, the Los Angeles and San Gabriel Rivers, lie within the study area. Extensive modification to the watershed and channel, such as the creation of upstream dams and hard substrate channelization, has reduced the amount of sediment that enters the littoral system. Under natural conditions, the Los Angeles River historically transported 233,000 yd^3/yr of sediment to the coast. After development, the river now only carries 33% of its original capacity or 77,000 yd^3/yr. Further south, the San Gabriel River originally allowed for the delivery of approximately 182,000 yd^3/yr in its natural state. Now, reduced a similar percentage as the Los Angeles River, the San Gabriel River only deposits 59,000 yd^3 of sediment into the littoral cell annually (Patsch & Griggs, 2007).

3.5.1.2 NOURISHMENT/OPERATIONAL ACTIVITIES

Although not directly a source of sediment, periodic nourishment events have been conducted at the project site which increases the amount of sediment that is available in the littoral zone. This sediment typically comes from dredging one of the three federal navigation channels within the study area or other areas of beneficial use. Beach nourishment has occurred over the years, but exact values are unknown. This type of nourishment is considered sand of opportunity, where sediments are placed due to excavation or dredging of other new work or O&M projects.

The city of Long Beach conducts annual backpassing of sediments from the beach near Belmont Pier (Belmont Shore) to the east near Alamitos Bay (East Beach). Typically, heavy machinery is able to move approximately 70,000 yd^3/yr along the beach (City of Long Beach, 2018), in the opposite direction from the net longshore transport from near 54th Pl. to East Beach. The process of backpassing is accomplished with the utilization of tractor-scrapers and bulldozers and typically takes about 2 weeks to move the quantity of sediment.

3.5.2 SEDIMENT SINKS

There are no major sinks for sediment within the project area. Some sediments are lost to the energy island borrow pits during large wave events. This amount is assumed to be negligible due to the protection provided by the breakwater.

The withdrawal of oil from the Wilmington Oil Field, dating back to the 1940's, has led to large magnitudes of subsidence of the seafloor. This lowering of the seafloor causes the sediments to shift to re-achieve an equilibrium profile. Since the initiation of a water injection process in the 1960's, this subsidence has since halted. Although, not being lost anymore, the redistribution of sediments into deeper waters limits the amount of sediment available for transport. Throughout the entire littoral cell, it is estimated that more than 6.5 million yd^3 was lost over the 20-year period of excessive subsidence.

Sediment shoaling occurs along the west of the Alamitos Bay West Jetty which withholds up to 20% of sediment lost from the study area. This quantity is moved offshore, but due to the limitation of wave energy produced by the protective structures; the sediment can no longer be transported landward by the waves, except in the most extreme conditions.

3.5.3 LONGSHORE SEDIMENT TRANSPORT

Longshore currents in the coastal zone are driven primarily by waves breaking near the shoreline at oblique angles. This wave generated current and turbulence due to the breaking process is the major factor in littoral transport. Due to the construction of the breakwaters in the study area, the longshore transport of sediments has basically come to a halt, although a small stretch of shoreline near East Beach remains active as indicated by the City of Long Beach's backpassing operations. This gradient of longshore flow within the Los Angeles/Long Beach sub-cell, only transports an estimated 49,000 yd^3/yr. (Patsch & Griggs, 2007). The Seal Beach sub-cell is much smaller in size and, in turn, more difficult to estimate the actual transport rate. The numerous nourishment volumes, inconsistent backpassing operations and significantly altered shoreline leads to an unknown sediment transport rate of the 1 mile section.

3.5.4 CROSS-SHORE SEDIMENT TRANSPORT

Cross-shore currents exist throughout the study area, particularly at time of high surf. These currents tend to concentrate at river mouths and structures but can occur anywhere along the shoreline in the form of rip currents which are the return flows of the complex nearshore circulation cell. With the protection and calm water created by the detached breakwaters, the cross-shore transport has been decreased also. It is estimated that only 1,500 yd^3/yr. of sediment is lost to the offshore boundary (Patsch & Griggs, 2007).

3.5.4.1 DEPTH OF CLOSURE

Sediments that are mobilized by waves are typically limited by the depth of closure (DoC). The depth is closure can be defined as the most landward depth seaward of which there is no significant change in bottom elevation and no significant net exchange between the nearshore and the offshore (Kraus et. al, 1999). Little data from the proposed project area exists corresponding to this parameter. From previous studies, the average depth of closure from Seal Beach to Huntington Beach was found to be ~21.5 ft. (6.5 m) (USACE, 2002). Although, this DoC value is unlikely to occur within the protection of the breakwaters, a consistent value must be used for modeling purposes with and without the breakwater present. Without the breakwater, the wave climate would act similar to the Seal Beach sub-cell, so the larger DoC is justified for this analysis.

3.5.4.2 BEACH PROFILES

Figure 3-20 shows three typical shoreline profiles based on locations shown in Figure 3-21. Two LiDAR surveys were used to determine the representative profiles within the proposed project area. Starting at

Shoreline Marina, the berm height increases from around 8 ft. to nearly 15 ft. near Peninsula Beach (which is an artificial berm created from backpassing operations) and the foreshore slope increases from a 1V:20H at areas sheltered by the breakwater to 1V:10H near the partially sheltered East (or Peninsula) Beach.

Figure 3-22: Long Beach Profile Transects

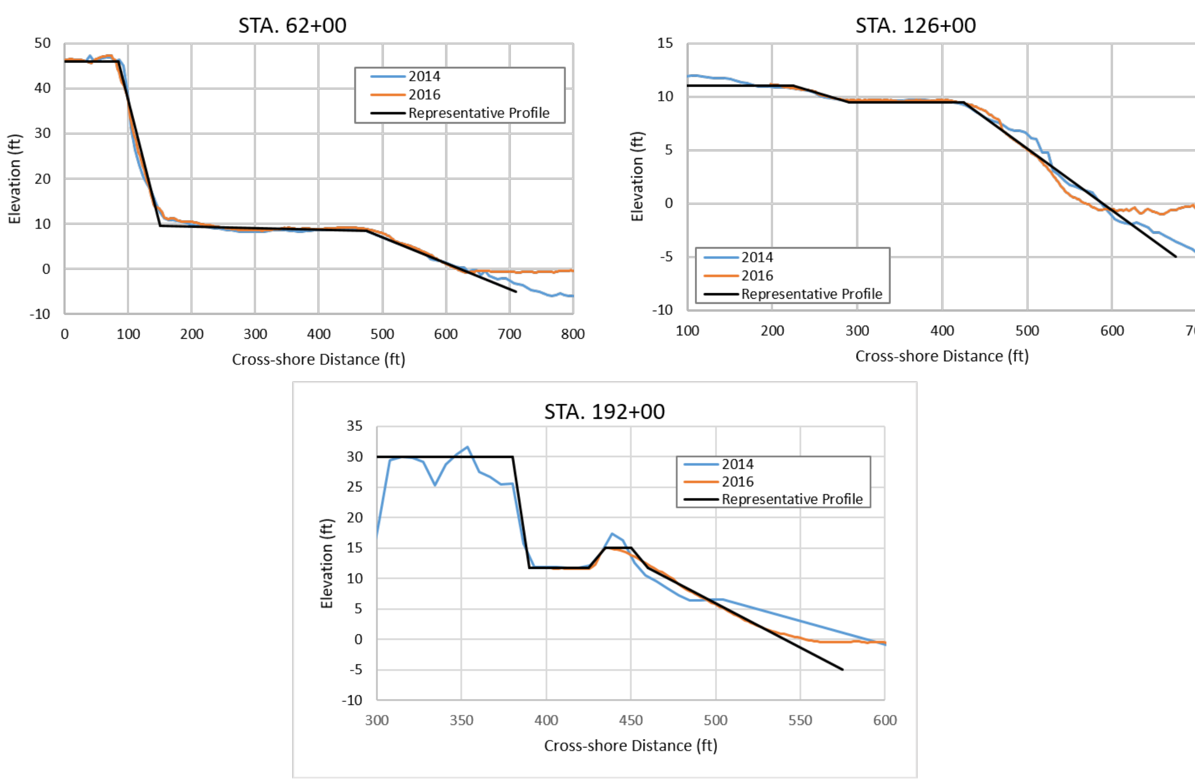


Figure 3-23: Shoreline Profiles along Long Beach

The low berm height in the west, starting near station 0+00, is a product of the relatively low energy wave climate as compared to the eastern side. The berm height slowly increases moving eastward. A bluff is present from the beginning station until 36th Pl. and provides a landward boundary of the beach. West of here, a line of private residences boarders the beach.

3.5.4.3 BRUUN RULE

The Bruun Rule relates the amount of sea level rise (*S*) with the potential shoreline recession (*R*) and provides a good approximation of the recession (Bruun, 1988). This relationship includes the contribution from the submerged and emergent profile using the depth of closure (*DoC*), the berm height (Ht_{berm}, taken as 10 ft.) and the active profile lengths from the berm crest to the depth of closure (*L*, assumed to be 2000 ft.)

$$R = \frac{SL}{(Ht_{berm} + DoC)} \quad (2)$$

The expected shoreline recession, corresponding to the low, intermediate and high sea level rise curves, is near 9, 45 and 160 ft, respectively.

3.5.5 SEDIMENT BUDGET

A sediment budget provides a conceptual model of littoral processes by accounting for volume changes and sediment fluxes within the cells and across cell boundaries. The sediment budget for the two sub-cells described above is mainly driven by the transport of sediments from the two rivers and can be seen in Figure 3-22. Due to the offshore breakwaters and other man-made structures within the port complex, the majority of sediment is kept within the sub-cells.

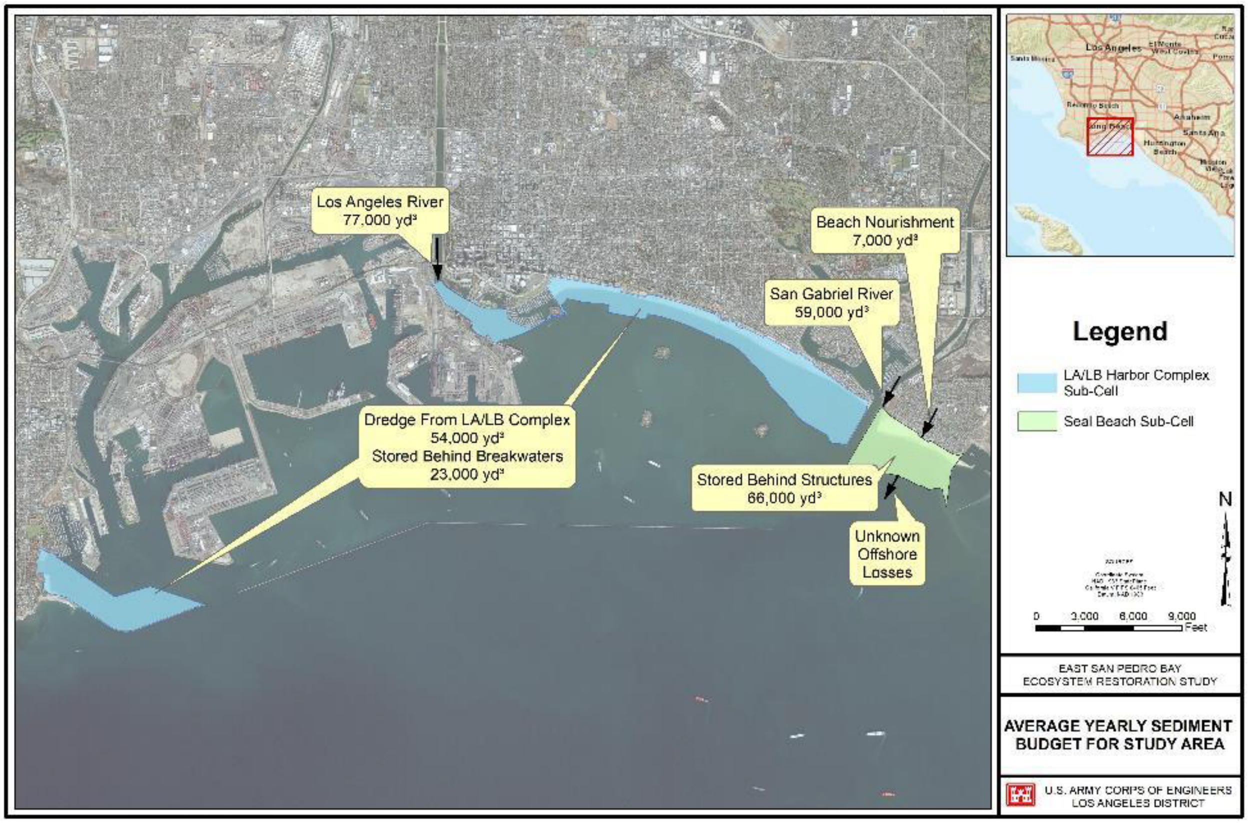


Figure 3-24: Sediment Budget within the Study Area, Patsch & Griggs (2007)

3.5.6 SEDIMENT MOBILITY

The ability for sediments to mobilize outside of the surf zone is entirely dependent on the wave height, water depth and sediment grain size. Using a similar analysis to McFall et. al. (2016), the potential for sediment mobility can be calculated. This analysis uses the combined current and wave shear stress theory that describes the mobilization of the non-cohesive sediments off the seabed. Since much of the sediment within the study area are on the order of 0.1 and 0.2 mm, these values will be used as bounding limits. Sediments can be considered mobilized if the combined wave and current bottom shear stress is larger than the critical shear stress calculated from the Shields parameter. This analysis is based solely on linear wave theory and produces less accurate results as a wave becomes more depth limited; although it is assumed that sediments are *always* mobilized within the surf zone.

The sediment mobility is determined by the mean mobility score calculated using the combined wave and current shear stress, calculated from the numeric models to be discussed, along with the critical shear stress at the same location. Some modeling results will be discussed which provide inputs for this sediment mobility analysis, for a complete description, see NUMERIC MODELS AND RESULTS section.

At each cell location, the maximum velocity at the seabed is extracted from the bottom layer of EFDC along with the wave conditions (wave height, period and direction) from CMS-Wave; with the bottom velocity proportional to the horizontal particle excursion along the seabed over the wave period with a factor of 1.25 to account for the spectral definition of the waves. The combined wave-current shear stress is calculated for each bin covering the entire probability of record and a weighted average of the mean mobility score is found. For further information of the mean mobility score, see McFall et. al. (2016).

Figure 3-23 shows the weighted mean mobility score for each wave bin to be described in the GROUPED DATA ENCOMPASSING ENTIRE RECORD section and the percent occurrence of that specific wave bin. The simplification provides information about the entire wave record in a single value that is spatially explicit. This calculated value gives the probably of sediment movement as a function of the mean sediment grain size. Since the mean mobility score only provides information on how often the sediments will be mobilized, the scale is normalized using the peak calculated value and describes the relative probability of potential mobility.

In the nearshore environment, the sediment will always be mobilized. Mobility of sediments from the seaward modeling boundary to the depth of closure is expected. Although Figure 3-25 shows movement near hard substrates, such as the Long Beach Breakwater, the analysis assumes all sediments are of the same size, so the potential mobility is not applicable in areas with large stone protection. The probability of sediment movement decreases with the distance from the shoreline. Finer sediments are mobilized at a higher probability than more coarse sediments, but the extent is not significantly different when comparing the coarser 0.2 mm and finer 0.1 mm grain sizes. In the proposed project area, the limits of the sediment mobility are more dependent on the breakwater configuration than on the sediment size.

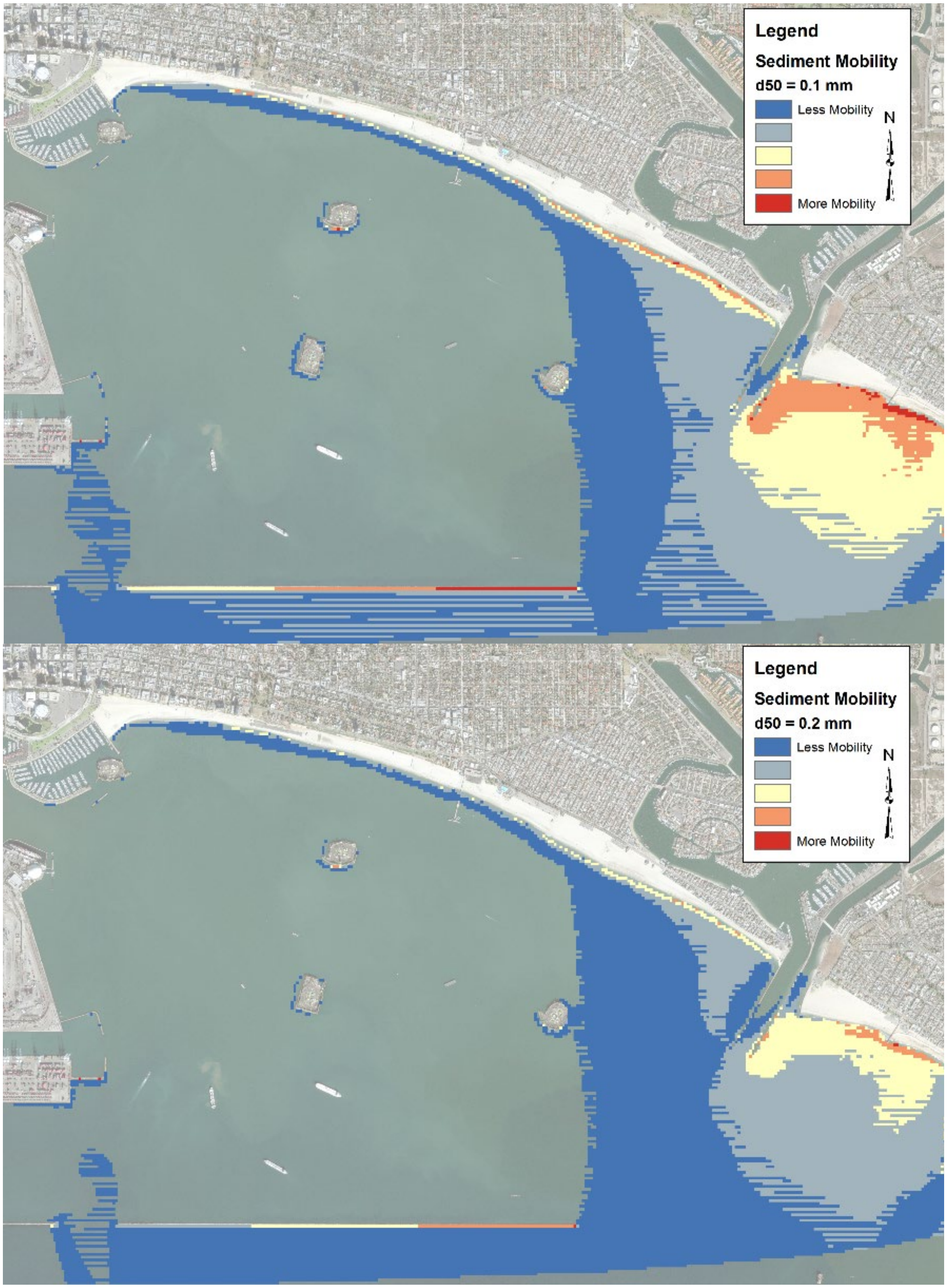


Figure 3-25: Potential Mobility of Sediments

3.5.7 SEDIMENT COMPOSITION

Sediment bed properties are based on a compilation of sediment data from multiple studies taken at different times to provide sufficient spatial coverage throughout the study area. Sediment data for the proposed project area, shown as percent fines, is illustrated in Figure 3-24. Due to the collection manner, the data only applied to the top-most layer of sediment and does not represent sub-surface conditions. For a further discussion of sediment types within the study area, see the GEOTECHNICAL APPENDIX. These sediment sampling results are only used for numeric modeling which is only concerned with skin friction, and not the underlying sediments; therefore, the use of grab sample data is acceptable for this purpose.

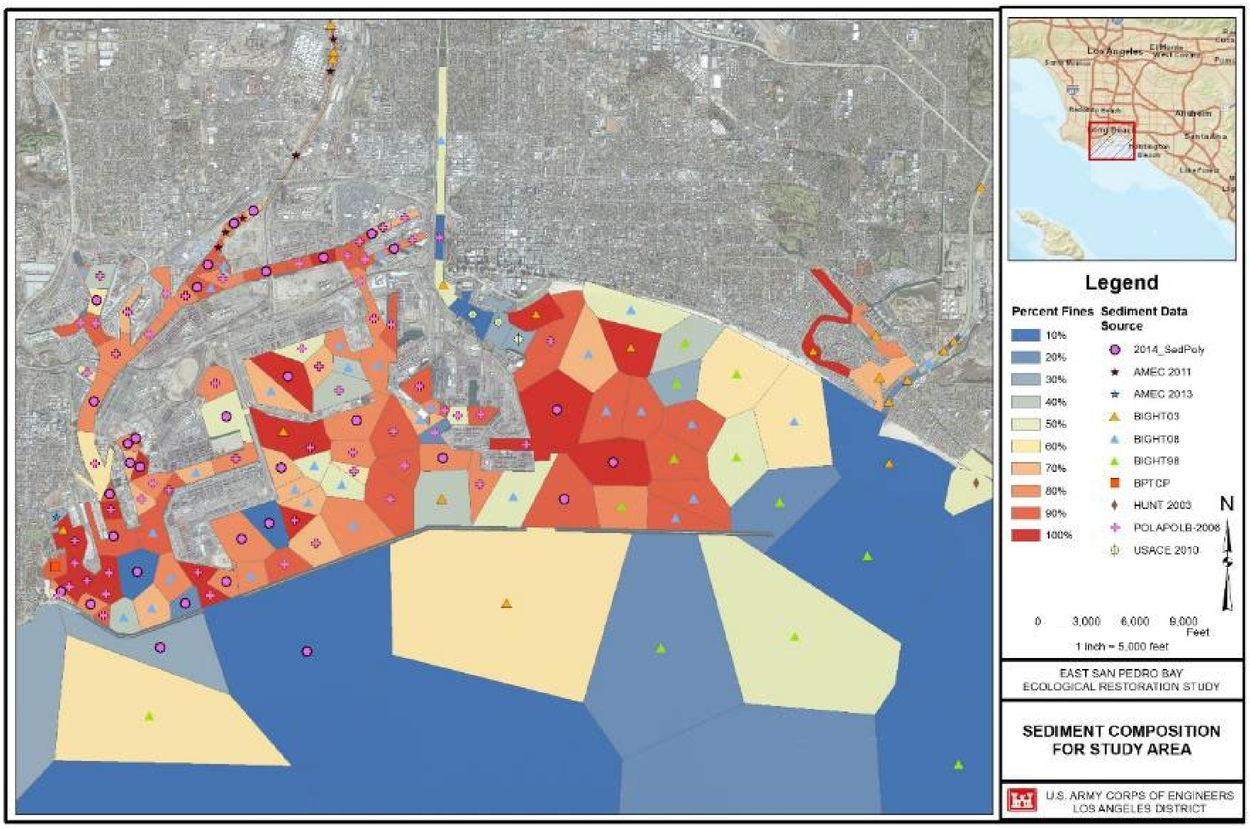


Figure 3-26: Sediment Types in Study Area

Generally, the sediment samples were collected with a Ponar grab sampling tool (or similar) which only collects sediments in the top 3-4 inches. This sampling regime is not adequate for complete sediment characterization and determination of the underlying sediments but assists with the numerical modeling aspect of this study by providing for a slight variation of bed friction throughout the domain.

Outside of the protection of the breakwaters, coarse sediments are more prevalent. Inside of the breakwaters, high energy waves are limited so fine-grained sediments are dominate. The sediment composition closely follows the locations of sediment mobility. In areas with higher mobility, the top layer of sediment is dominated by more coarse grain materials and the finer sediment are transported away. In areas with lower or no mobility, fine grains are present indicating little to no wave induced transport.

4 DESIGN CONDITIONS

4.1 WATER LEVELS

Total water levels are a combination of the tides, wave set-up, storm surge and sea level rise. The design water depths are shown in Table 4-1. The tides and other short term increases to the water surface elevation are superimposed onto each other while the increase in water surface elevation due to sea level rise are assumed to be independent from the preceding values. Sea level changes are calculated from the baseline NOAA defined datums in 1992.

Table 4-1: Design Water Level Elevations from 1992 Tidal Datum

Level	Elevation ft.
MLLW	0.00
MSL	2.82
MHHW	5.49
50-YR EWL	7.64
EL NINO	8.64
LOW SLR	8.88
MED. SLR	9.57
HIGH SLR	11.75

Tidal elevations are taken directly from the calculated datums defined by NOAA. The 50-yr extreme water level (EWL) is taken from Figure 3-7. The el Nino event water level is approximated from previous measurements with an example shown in Figure 3-10. Finally, the three sea level change elevations, during the 50-yr project duration, are calculated using the USACE SLC calculator discussed in the SEA LEVEL CHANGE section.

4.2 WAVES & HYDRODYNAMICS

As waves move from an offshore location to the study area they change in size, shape and direction due to shoaling, refraction and diffraction. The following discussion is based on outputs from both wave models to be described in the NUMERIC MODELS & RESULTS section.

4.2.1 NEAR-SHORE WAVE & HYDRODYNAMIC CHARACTERISTICS

Waves act differently when in deep water verses shallow water. In deep water, individual water particles produce the more circular pattern shown on the right side of Figure 4-1. Individual water particles move in a circular pattern; and in one wavelength, return to their original position. Theoretically, there is no net movement of water due to waves at these depths. Energy is propagated due to momentum transfer between the water particles and not net displacements. Moving from deep water to more shallow water, waves begin to “feel” the seabed. The influence from the bottom constricts the vertical direction of motion but does not significantly constrain the horizontal motion, as shown on the left side of Figure 4-1. The horizontal movement continues to have a more pronounced effect moving to more shallow waters, until the individual partial velocities overcome the natural wave propagation speed causing it to break and loose energy before running up on the beach and reflecting a portion of the energy back into the ocean.

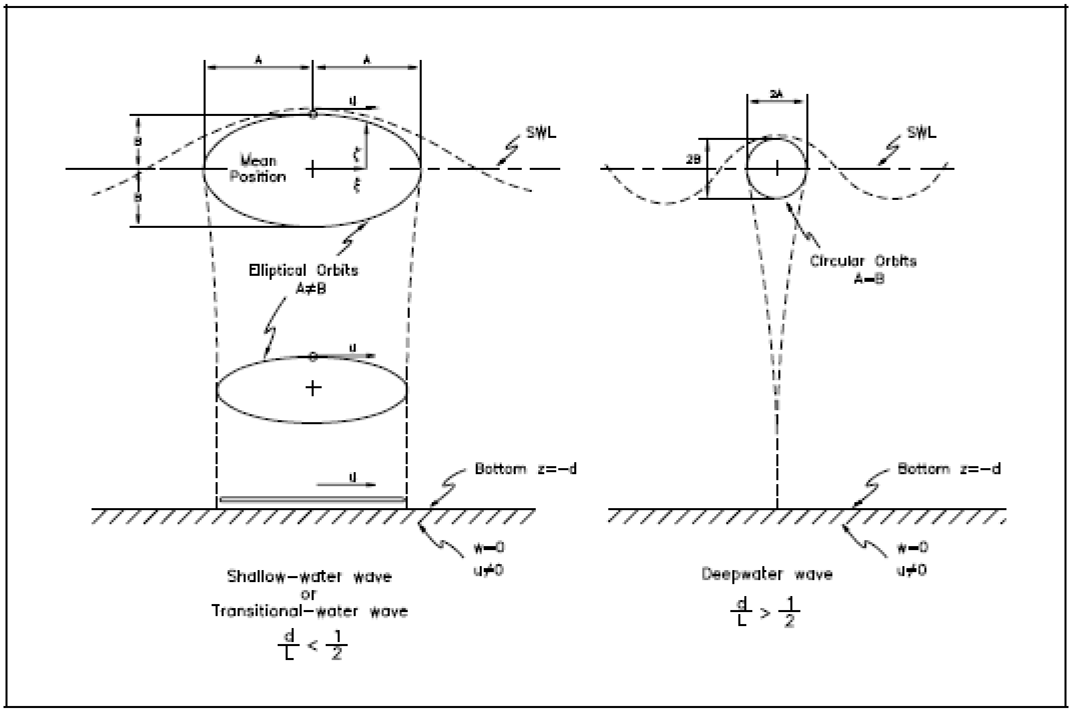


Figure 4-1: Wave Particle Motions (CEM, 2002)

The individual particle motions have an interesting effect in deep (and somewhat transitional) water; there is no net transport of mass (i.e. no circulation). As the waves transform to the nearshore, they become more skewed and asymmetric and begin to drive currents. Only when near the shoreline do the waves play more important role in circulation, producing a wave generated net velocity. More information of this can be found in the NUMERIC MODELS & RESULTS section

4.2.1.1 LOCAL BREAKWATER EFFECTS

The breakwaters within the project area are not completely impervious to wave effects. Transmission through the structures allows for energy to be transferred to the lee which will cause increased wave heights along the shoreline as compared to an impermeable structure. A study performed on a scale model of the Middle Breakwater (Hales, 1976) shows that the transmission coefficient, the ratio of the transmitted wave height and incident wave height, is on the order of 0.3 and also a function of the wave period. This means that only about 10% of the incident wave energy is transmitted through the structure. To verify this coefficient, the experimental data is compared with an empirical equation from Goda (2010) that utilizes parameters for overtopping and flow through a rubble-mound structure as a function of the peak period. Results and comparisons are presented in Figure 4-2, with the values calculated using Goda (2010) labeled as "Calc.". Two water levels are considered to investigate the contribution of the freeboard, the dry height from the local water level to the crest of the structure. The increase in water level allow for more of the highly porous upper sections of the structure to be subject to the transmission. Future increases in mean sea level will allow for even more transmission and overtopping.

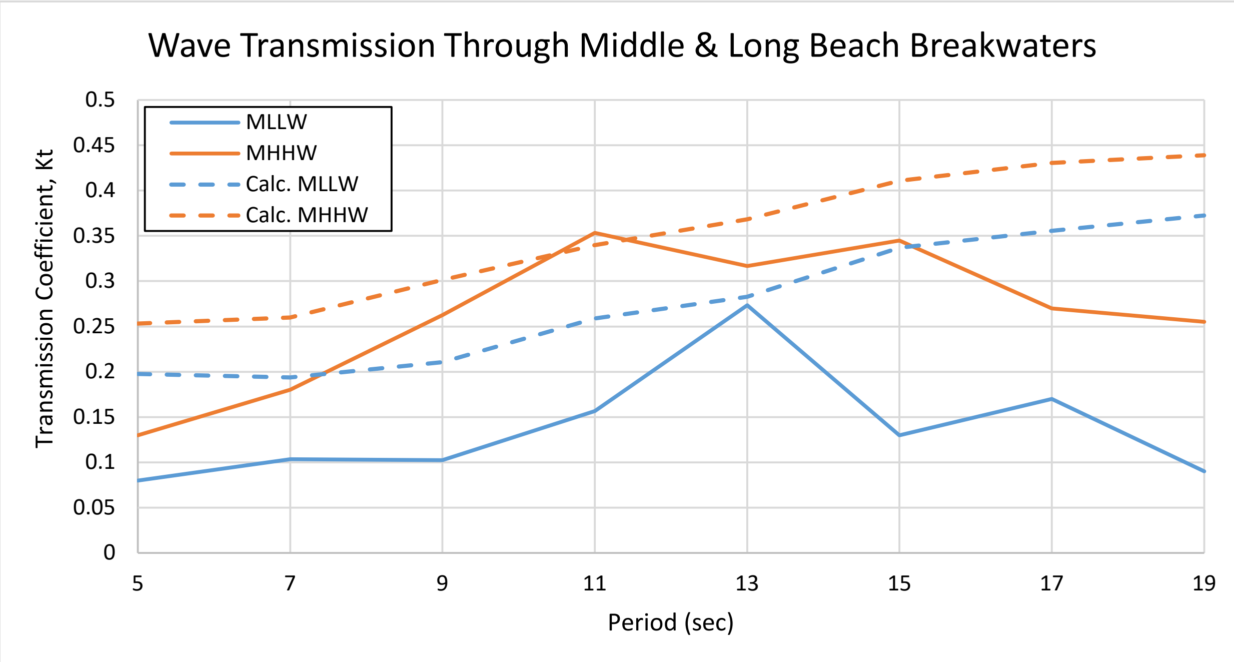


Figure 4-2: Transmission Coefficient as a Function of Peak Wave Period

Tidal effects from the breakwater are much less apparent. From Madsen and White (1976), the transmission through a porous structure is proportional to $(1 + \lambda)^{-1}$ with:

$$\lambda = \frac{klf}{2n} \tag{3}$$

where *k* is the wave number, *l* is the structure width, *f* is a friction factor and n in the porosity of the structure. If the tide is thought of as a long wave with a period of 12 hours, the wave number is found to be on the order of 10^{-5} m^{-1} which produces a transmission coefficient near unity. Therefore, it is assumed that the breakwaters have a negligible effect on a tidal gradient within the bay as nearly all the energy is transmitted. Although, currents are larger in magnitude, when transporting uninterrupted through the channels through the breakwaters, such as Queen's and Angel's Gate

4.2.2 WAVE RUNUP

Wave runup and overtopping occur both at a structure and along the shoreline. To determine runup at the shoreline, the Stockdon et al. (2006) formula is used. This empirical method, shown below, can give relatively accurate results and is developed using experimental data from around the world. Like most other empirical runup methods, wave setup and breaking are included in the results due to the data collection limitations. Required parameters included to utilize this method are the foreshore beach steepness, β_f, taken from nearshore transects of survey information discussed in BEACH PROFILES and the deep water wave height and wavelength, H_0 and L_0 respectively. The deep water wavelength is taken strictly from the model, as it is assumed that the wave period does not significantly change. Due to the presence of the breakwater, the original deep water wave height cannot be used directly. Modeled transformation coefficients are used in these cases. Model results are to be discussed in the NUMERIC MODELS AND RESULTS chapter.

$$R_{2\%} = 1.1\left(0.35\beta_f\sqrt{H_0L_0}\right) + \frac{\sqrt{H_0L_0\left(0.563\beta_f^2 + 0.004\right)}}{2} \quad (4)$$

Table 4-2 shows the expected two percent runup elevations for the suite of storm events previously described at discrete locations along Long Beach. The two percent runup, $R_{2\%}$, is the elevation at which the top two percent of waves reach during an event and is typically used in coastal inundation mapping. Beach foreshore slopes and transformation coefficients, K, are presented in Table 4-3 for comparison. Boundary conditions are defined in the WAVE CLIMATE section. For this study, the increases in wave height due to greater water depths caused by sea level rise is not included; only a linear superposition of the total water level is used to find the runup elevation.

Table 4-2: Expected Runup Elevations within Long Beach

2% Runup Elevation[1] (ft.)																
Event Type	**Peninsula Beach**				**Belmont Shores**				**Belmont Pier**				**Junipero Beach Parking Lot**			
	MHHW	**Sea Level Rise**			**MHHW**	**Sea Level Rise**			**MHHW**	**Sea Level Rise**			**MHHW**	**Sea Level Rise**		
		Low	**Int.**	**High**		**Low**	**Int.**	**High**		**Low**	**Int.**	**High**		**Low**	**Int.**	**High**
Typical Winter	11.0	14.5	15.0	17.0	8.0	11.5	12.0	14.5	8.0	11.5	12.0	14.0	7.5	11.0	11.5	13.5
Typical Summer	10.5	14.0	14.5	17.0	8.0	11.5	12.0	14.5	8.0	11.0	12.0	14.0	7.5	10.5	11.5	13.5
1-yr NW Swell	18.5	22.0	22.5	25.0	12.5	15.5	16.5	18.5	11.5	15.0	15.5	18.0	10.0	13.5	14.0	16.0
1-yr S Swell	17.0	20.5	21.0	23.5	11.0	14.5	15.5	17.5	11.0	14.5	15.0	17.0	9.5	13.0	13.5	15.5
50-yr NW Swell	20.5	23.5	24.5	26.5	13.5	17.0	17.5	19.5	12.5	16.0	16.5	19.0	10.5	14.0	14.5	17.0
50-yr S Swell	20.0	23.5	24.0	26.5	14.0	17.0	18.0	20.0	13.5	16.5	17.5	19.5	11.5	15.0	15.5	18.0
100-yr NW Swell	20.5	24.0	24.5	26.5	13.5	17.0	17.5	20.0	12.5	16.0	17.0	19.0	10.5	14.0	15.0	17.0
100-yr S Swell	20.5	23.5	24.5	26.5	14.0	17.5	18.0	20.5	13.5	17.0	17.5	19.5	12.0	15.5	16.0	18.0
[1] Values rounded to the nearest 0.5 ft.																

Table 4-3: Wave Transformation Coefficients for Modeled Events at Stated Locations

Event Type	**Peninsula Beach, β_f = cot(6.0)**	**Belmont Shores, β_f = cot(10.0)**	**Belmont Pier, β_f = cot(10.0)**	**Junipero Beach Parking Lot, β_f = cot(16.0)**
Typical Winter	0.53	0.30	0.25	0.28
Typical Summer	0.75	0.43	0.36	0.40
1-yr NW Swell	0.43	0.27	0.22	0.21
1-yr S Swell	0.89	0.51	0.45	0.44
50-yr NW Swell	0.39	0.26	0.21	0.21
50-yr S Swell	0.63	0.47	0.41	0.49
100-yr NW Swell	0.39	0.26	0.21	0.21
100-yr S Swell	0.59	0.45	0.40	0.48

Due to the presence of the breakwater, the largest runup events are in the eastern section of Long Beach. As waves diffract to equalize the energy distribution in the lee of the breakwater, the total runup is reduced to the west. Generally, the northern swell events produce larger runup elevations in the higher probability events (1-yr return period), while as the southern swell events produce larger wave heights within the project area during the rarer events. The southern swells are generally smaller in magnitude, but due to the orientation of the bay, a large southern swell event will cause the largest runup elevations along the shoreline.

Runup on a structure is calculated using a slightly different empirical relationship between the wave parameters and the structure characteristics calculated using the standard Delft Hydraulics formula of the form:

$$\frac{R_{2\%}}{H_s} = \begin{cases} A\,\xi_{om} & for\ 1.0 < \xi_{om} \le 1.5 \\ B\,(\xi_{om})^C & for\ 1.5 < \xi_{om} \le \left(\frac{D}{B}\right)^{\frac{1}{C}} \\ D & for\ \left(\frac{D}{B}\right)^{\frac{1}{C}} \le \xi_{om} < 7.5 \end{cases} \tag{5}$$

Where ξ_{om} is the Iribarren number based on the mean spectral period. The coefficients; A, B, C and D, are as defined in the CEM (USACE, 2002) for the two percent runup elevation. Results are shown in Table 4-3 for expected runup elevations relating to the MHHW and low, intermediate and high sea level rise water levels. Also, increases in wave height due to deeper water at the project site was not included. The structure slopes are assumed to be 2:1 (H:V). No reduction based on structure roughness was taken to be conservative in the runup sentiment. Similarly to the run-up along the shoreline, input transformation coefficients are presented in
Table 4-5.

Table 4-4: Expected Runup Elevations at Structures within ESPB

2% Runup Elevation[1] (ft.)												
Event Type	**Freeman Island**				**Chaffee Island**				**White Island**			
	MHHW	**Sea Level Rise**			**MHHW**	**Sea Level Rise**			**MHHW**	**Sea Level Rise**		
		Low	**Int.**	**High**		**Low**	**Int.**	**High**		**Low**	**Int.**	**High**
Typical Winter	7.5	11.0	11.5	14.0	7.5	11.0	12.0	14.0	7.5	11.0	11.5	14.0
Typical Summer	8.0	11.0	12.0	14.0	8.0	11.5	12.0	14.5	7.5	11.0	12.0	14.0
1-yr NW Swell	12.5	16.0	16.5	18.5	12.0	15.0	16.0	18.0	12.5	15.5	16.5	18.5
1-yr S Swell	11.5	14.5	15.5	17.5	12.0	15.0	16.0	18.0	11.0	14.5	15.0	17.5
50-yr NW Swell	15.5	18.5	19.5	21.5	14.0	17.5	18.0	20.0	15.0	18.5	19.0	21.5
50-yr S Swell	20.5	24.0	24.5	26.5	18.0	21.5	22.0	24.5	20.0	23.0	24.0	26.0
100-yr NW Swell	15.5	19.0	19.5	22.0	14.0	17.5	18.0	20.5	15.5	18.5	19.5	21.5
100-yr S Swell	21.5	25.0	25.5	28.0	19.0	22.0	23.0	25.0	21.0	24.5	25.0	27.0
Event Type	**Grissom Island**				**Shoreline Marina Breakwater**				**Pier J South Jetties**			
	MHHW	**Sea Level Rise**			**MHHW**	**Sea Level Rise**			**MHHW**	**Sea Level Rise**		
		Low	**Int.**	**High**		**Low**	**Int.**	**High**		**Low**	**Int.**	**High**
Typical Winter	6.5	10.0	10.5	13.0	6.5	10.0	10.5	12.5	8.0	11.5	12.0	14.5

Typical Summer	7.0	10.5	11.0	13.0	6.5	10.0	11.0	13.0	8.5	12.0	12.5	15.0
1-yr NW Swell	8.5	12.0	12.5	15.0	8.0	11.5	12.0	14.0	12.0	15.5	16.0	18.5
1-yr S Swell	9.5	13.0	13.5	16.0	9.0	12.0	13.0	15.0	13.0	16.5	17.0	19.5
50-yr NW Swell	9.5	13.0	13.5	16.0	8.5	12.0	13.0	15.0	14.5	17.5	18.5	20.5
50-yr S Swell	15.5	19.0	19.5	22.0	13.0	16.5	17.0	19.5	23.0	26.5	27.0	29.0
100-yr NW Swell	10.0	13.0	14.0	16.0	9.0	12.0	13.0	15.0	14.5	18.0	18.5	21.0
100-yr S Swell	16.5	20.0	20.5	23.0	13.5	17.0	17.5	20.0	24.5	28.0	28.5	30.5

[1] Values rounded to the nearest 0.5 ft.

Table 4-5: Wave Transformation Coefficients for Structure Runup Estimations

Event Type	**Freeman Island**	**Chaffee Island**	**White Island**	**Grissom Island**	**Shoreline Marina Breakwater**	**Pier J South Jetties**
Typical Winter	0.29	0.30	0.29	0.16	0.13	0.36
Typical Summer	0.40	0.45	0.39	0.25	0.21	0.54
1-yr NW Swell	0.29	0.26	0.28	0.13	0.10	0.28
1-yr S Swell	0.50	0.54	0.48	0.35	0.29	0.67
50-yr NW Swell	0.29	0.25	0.28	0.12	0.10	0.27
50-yr S Swell	0.58	0.49	0.55	0.39	0.29	0.68
100-yr NW Swell	0.29	0.25	0.28	0.12	0.10	0.27
100-yr S Swell	0.58	0.48	0.55	0.39	0.29	0.68

Similar to the shoreline runup elevations, the largest runup is encountered during the less frequent storm events. The offshore locations experience the largest runup elevations due to the relatively deep water allowing for large, unbroken waves. The protected structures, such as the Shoreline Marina Breakwater or Grissom Island, see the lowest runup elevations.

4.3 LONG TERM SHORELINE CHANGE

Periodic backpassing or nourishment is assumed to be continuously performed by the city in a similar manner to that currently undertaken. Therefore, the shoreline is assumed to remain in the current configuration throughout the period of analysis representing the without project conditions.

5 PLAN FORMULATION AND DESCRIPTION OF ALTERNATIVES

5.1 DESCRIPTION OF MEASURES AND BASIS FOR DESIGN

The following sections will describe a basis of design for the ecosystem restoration measures. Combined, these measures create the plan alternatives. For a more complete discussion of the measures, see the MAIN REPORT and APPENDIX D, MODEL DOCUMENTATION FOR THE SOUTHERN CALIFORNIA COASTAL BAY ECOSYSTEM MODEL. For this appendix, focus is given to the engineering design and quantity estimation of all the measures. All designs are preliminary and require additional analysis to verify the assumptions, conduct detailed design and the inclusion of the more accurate information during the Pre-construction Engineering and Design (PED) phase. Final array alternatives including placement location and preliminary cross-sections are provided in the PLAN SELECTION section.

5.1.1 ROCKY REEFS

The design for submerged reefs involves providing for sufficient voids to provide refuges for smaller fishes as well as substrate for different forms of algae and invertebrates. Creation of the rocky reef habitat can be accomplished with stone mined from a local quarry or a product called Reef Balls. For this study, local quarry stone will be used to determine costs and construction techniques. All guidelines within NOAA (2007) are to be followed.

5.1.1.1 DESIGN

Since the rocky reef measures are in two distinct areas, nearshore and open water, the basic design is fundamentally different. For the nearshore and higher profile offshore reefs, the armor stone size calculation can utilize van der Meer’s (1991) stability equation relating to submerged or partially submerged reef breakwaters with two layers of armor stones.

$$\frac{h'_c}{h} = (2.1 + 0.1S)\exp(-0.14N_s^*) \tag{6}$$

where h'_c is the height of the structure off the sea bed, h is the local water depth, S is the relative eroded area and N^*_s is the spectral stability number defined as:

$$N_S^* = \frac{H_s}{\Delta \mathrm{D}_{50}} s_p^{-1/3} \tag{7}$$

Such that Δ is the ratio of the density of armor stone to the density of water minus one and s_p is the local wave steepness corresponding to the peak of the wave energy. For this analysis, the relative eroded area is assumed to be zero to lessen the O&M costs as well as to provide a conservative, stable armor size. The stone size is highly dependent on the wave height and therefore, the location of the submerged reef. Starting at Alamitos Bay and moving from east to west, the reefs can become more sheltered, reducing the incident wave height; therefore, the design armor stone weight can be reduced. Typically, rubble mound structures in the coastal environment consist of three layers, an armor layer made of large stone to withstand the wave climate, an core layer typically consisting of quarry run material less than 1,000 lbs in individual size and an intermediate layer, or filter, to provide a boundary so as not to loose the internal core stone.

Before the stone weight calculation is made, the crest elevation is determined. The purpose of these reefs, aside from providing primary habitat benefits from the structure itself, is to reduce the velocity of the surrounding fluid as to provide for adequate eelgrass habitat. According to the APPENDIX D, MODEL DOCUMENTATION FOR THE SOUTHERN CALIFORNIA COASTAL BAY ECOSYSTEM MODEL, the optimal velocity for eelgrass is 0.4 knots with a maximum of around 3 knots. To provide these velocities in the lee of the submerged structure, the shoreline area is decomposed into four wave height "zones", as shown on Figure 5-1. The total bottom velocity will then be a superposition of the orbital wave velocity and the tidal current velocity, both measured at the seabed. For further information on the eelgrass beds, see section 5.1.3 EELGRASS BEDS.

Figure 5-1: Submerged Reef Required Crest Elevations

After the crest elevation is found, the stone size can be determined using Eq. (4) & (5) and shown in Table 5-1.

Table 5-1: Stone size for submerged reef breakwaters

Crest Elevation ft. (MLLW)	$H_{s,Design}$ ft.	$T_{p,Design}$ sec	W_{50} ton	D_{50} ft.
0	14.0	18	11.5	6.4
-3	12.0	18	6.0	5.2
-5	9.0	18	4.0	4.5
-7	7.0	18	2.5	3.9
-10	4.0	18	1.75	3.4

The van der Meer equation, eq. (4), is only applicable when the structure's crest height from the seabed is greater than half of the water depth. Since the offshore reefs are more deeply submerged than 10 feet, and in an even greater water depths, by observation, the required individual stone weight will also be substantially smaller. The actual stone size will need to be larger to achieve the void spaces required for the habitats described in APPENDIX D, MODEL DOCUMENTATION FOR THE SOUTHERN CALIFORNIA COASTAL BAY ECOSYSTEM MODEL. Further study is required to find the optimal stone size and shape that maximizes the void spaces between individual units as well as the placement techniques needed to ensure adequate void spaces. For cost and availability analysis, a stone size of 10 tons is assumed. This large stone size will ensure stability of the open water reefs and eliminate the required maintenance after a large storm which is difficult due to the limited visibility. Remaining habitat would also be further disturbed if maintenance activities take place; as these habitats would be recovering also. Overall, a narrow stone gradation is needed to optimize for the void spaces. For the feasibility stage, it is assumed that the locally produced quarry stone can be placed by a crane operator in a manner to achieve a porosity of 0.6.

Quantities for the nearshore rocky reefs are obtained through CAD software using the difference between two Triangulated Irregular Network (TIN) surfaces; the existing seafloor elevations and the proposed reefs. To convert from volume to total stone weight,

$$W_T = V_T(1-n)\gamma_a \tag{8}$$

where W_T and V_T are the total stone weight and volume, n in the porosity (0.4 for typical rubble mound structures or 0.6 for reef habitat creation) and γ_a is the unit weight of the stone, 165 lb/ft³. The final estimated quantity of stone is shown in Table 5-2 for each measure and location; the nearshore reefs are dependent on the location and exact depth of placement, whereas the offshore reefs have the same quantity independent of the location. The nearshore reef numbering convention begins with the nearest to Alamitos Bay and increasing moving west.

Table 5-2: Estimated Reef Stone Quantities and Sizes

Measure/Location		Fully Submerged			Increased Crest Elevation for Protective Measures			
		W_{armor}	W_{core}	$W_{a,50}$	W_{armor}	W_{filter}	W_{core}	$W_{a,50}$
		Ton	Ton	Ton	Ton	Ton	Ton	Ton
Nearshore[2]	Reef 1	50,700	65,200	6	74,500	-	49,700	11.5
	Reef 2	46,200	46,000	4	63,200	-	52,200	11.5
	Reef 3	16,200	2,700	2.5	64,800	-	53,700	11.5
	Reef 4	14,200	-	1.75	66,200	-	54,300	11.5
	Reef 5[1]	9,500	6,500	1.75	21,500	54,500	58,300	13
	Reef 6	38,700	13,400	1.75	65,100	-	59,200	11.5
	Reef 7	42,200	28,000	6	69,900	-	42,100	11.5
	Reef 8	43,000	30,200	4	59,800	-	48,800	11.5
	Reef 9	44,300	36,600	1.75	62,100	-	46,900	11.5
Offshore	Complex	91,300	-	10	-	-	-	-

[1] Submerged reef is replaced with emergent breakwater when needed for Belmont Pier protection
[2] Reef numbers are shown in Figure 5-2

The open water reefs are individual modules that vary in height between 3 feet to 12 feet above the seabed and are grouped into a reef complex. The distribution of these reefs are as follows and are defined by the crest height above the existing seabed:

- 3 ft. – 20%
- 6 ft. – 25%
- 9 ft. – 35%
- 12ft. – 20%

This distribution will offer a variety of habitats for different species. Higher reefs will be placed furthest away from any marine navigation (commercial and recreational) as possible. The highest crest elevation will be set no more than -15 ft. MLLW. Using the more conservative eq. (4), a medium stone weight of 10 tons will provide for sufficient stability.

In the absence of any energy dissipation (wave breaking, bottom friction, structure porosity, etc.), the reflection coefficient for the offshore reef complexes is determined to be on the order of 10 percent of the incident wave height calculated using Dean and Dalrymple (1991):

$$K_r = \frac{1 - \frac{C_2}{C_1}}{1 + \frac{C_2}{C_1}} \tag{9}$$

where K_r is the reflection coefficient and C_1 and C_2 is the celerity corresponding to the total water depth and height of the water column directly over the reef, respectively. Furthermore, the energy dissipating processes, such as wave breaking and flow through the porous structure, will further reduce the reflected (and transmitted) wave heights but is neglected in this analysis. Maximum results are presented in Table 5-3 and are experienced when the height of the water column is the smallest; 15 feet in a total water depth of 27 feet.

Table 5-3: Maximum Expected Reflection Coefficients without Energy Dissipation

Period sec	K_r
8	0.126
10	0.133
12	0.137
14	0.140
16	0.141
18	0.142
20	0.143

Locations for reef placement are primarily determined based on the absence of existing eelgrass and hard substrate. Along with these factors, additional considerations such as shoreline protection and impacts to recreational activities plays a role in site selection. The final locations resulting from the feasibility analysis of the reefs are shown in Figure 5-2. Additional analysis is required to validate the above assumptions and preliminary design.

Figure 5-2: Nearshore Reef Locations

5.1.1.2 CONSTRUCTION AND MAINTENANCE

The construction of the nearshore rocky reefs will be accomplished by a barge and crane with appropriate support vessels. Fill material may be dumped from a barge using a front loader or bulldozer. Armor stones must be specially placed by a crane to obtain the specific armor layer thickness. Construction of the offshore reefs require more complex placement techniques. For this measure, stone cannot be dumped from a barge and must be specially placed in order to obtain the required void spaces. This technique leads to a much longer duration of construction due to the single stone placement. Construction activities will be limited during the winter months (December – April) due to large wave events. At the completion of the construction period, a verification survey using a multibeam sounder to gather full bottom coverage, will be required.

Typical maintenance of the reefs would encompass re-setting displaced stones after large storm events or when needed to justify the mobilization costs. From other projects near the study area, maintenance typically occurs every 10 years. To plan for the cost of repair, 1% of the total construction cost per year is used for emergent breakwaters and jetties. The deeply submerged open water reefs will not experience any maintenance cost due to the large armor stone size required for sufficient large void spaces and stability. Since the placement will be entirely submerged in at least 15 ft. of water, maintenance after a failure will be nearly impossible as limited visibility would hinder such a repair and impact existing habitats.

5.1.1.3 RELATIVE SEA LEVEL RISE

The nearshore and open water reefs will respond differently to the increase in water level due to potential sea level rise, with the nearshore reefs being more susceptible to changes. Since the function of the nearshore reefs is to break waves and produce areas of calm for additional habitats to thrive and increase in water level would make them less effective. A specific submergence is required to break a portion of the waves while as allowing for transmission so as to not completely stop the sediment transport on the lee of the structures, so proactive measures to provide for water level increases cannot be performed during initial construction. Instead, during times of maintenance, when equipment is already mobilized, additional stone can be added to raise the height of the structures to obtain the required submergence. The relatively wide structure of 175 ft will provide a stable base for the expansion. Open water reefs will not experience the same issues as the nearshore reefs relating to sea level rise. As the water level potentially rises, these measures will become more deeply submerged and will cause less of a navigation impact, since more vessels can transit directly over the reefs.

5.1.2 <u>KELP REEFS</u>

A successful kelp reef has been constructed in the Southern California Bight. Offshore San Clemente, CA., approximately 40 miles from the study area a single layer of relatively small stones is specially placed on the seabed. This substrate was naturally colonized naturally by kelp and other organisms. Findings from this successful pilot study show that the artificial reefs grew and sustained kelp communities and are similar and somewhat greater in density of the existing habitat (Elwany et al, 2011).

5.1.2.1 DESIGN

The referenced pilot study is analogous to the restoration within the study area for the improvement of kelp reefs and can be accomplished with the same methods and materials. The optimal stone size used was 24 in. x 18 in. x 6 in. with a nominal weight of around 300 lbs. and no stone larger than 3.3 ft. in a single dimension. When deeply submerged, a structure is not exposed to the same conditions as a structure above or at the water surface. Accelerations play a lesser role and generally, when deeply submerged, inertial forces can be ignored. For these bottom situated structures, the forces experienced are similar to flow in a uniform field. The stone size can then be found using a general riprap design:

$$\frac{D_{30}}{h} = S_F C_s \left[\sqrt{\frac{\gamma_w}{\gamma_a - \gamma_w}}\left(\frac{\bar{u}}{\sqrt{K_1 g h}}\right)\right]^{5/2} \tag{10}$$

Where D_{30} is the diameter of stone where 30% of the gradation is finer, h is the local water depth, γ is the unit weight of water and the armor stone, K_1 takes into account the angle of repose for the stone as well as the coastal or channel slope defined as 0.5 in this case. S_F is a safety factor, taken as 3.0 to provide for a conservative result, and C_s is a factor accounting for the angularity of the stone taken as 0.3 for angular stone. The mean velocity, u, is calculated using linear wave theory, which provides an estimation of the flow near the seabed, but also must include the contribution due to the tidal currents that can be up to 2 knots. For a wave event with a peak period of 18 seconds, the minimum wave height to cause motion of a 300 lb. stone in 30 feet of water is 20 ft. From the wave analysis to be discussed in the NUMERIC MODELS AND RESULTS chapter, the maximum wave height in the locations proposed for kelp reefs is found to be 15 ft. Therefore, the kelp reefs can be placed anywhere within the proposed project area while still meeting the ecological parameters such as wave exposure and depth as long as the minimum stone size is 300 lbs. The stone required for kelp reefs will be referred to as quarry stone throughout this appendix.

Stone placement densities from the San Clemente pilot study ranged from 17% to 67% of stone coverage at the seabed. It was found that a coverage by weight of around 800 tons/acre is sufficient to achieve the required kelp density, which corresponds to a 20% seabed coverage. Allowing for a wider gradation of stone size, which can utilize slightly larger stone, 1075 tons/acre will be used for this feasibility analysis for quantity and cost estimates.

5.1.2.2 CONSTRUCTION AND MAINTENANCE

Kelp reefs can be constructed using a push-off method with the use of a barge and a front loader or dozer. The barge will be positioned directly over the proposed kelp reef location and the front loader or dozer will push off the material to achieve the required density of stone. No maintenance costs are expected for this measure. Burial by natural sediments is not expected due to the exposed wave climate that will limit the buildup of additional fine grain sediment. Increases in beach grooming is expected due to the quantity of kelp that may become dislodged from the substrate and wash up along the shoreline.

5.1.2.3 RELATIVE SEA LEVEL RISE

Kelp reefs will not be affected by sea level rise.

5.1.3 <u>EELGRASS BEDS</u>

Eelgrass thrives in calm, shallow waters. From APPENDIX D, MODEL DOCUMENTATION FOR THE SOUTHERN CALIFORNIA COASTAL BAY ECOSYSTEM MODEL, the optimal range of velocities is from zero to 3 knots.

5.1.3.1 DESIGN

To ensure a conservative design, the maximum target velocity will be on the order of 1 knot. As stated earlier, protection from incident wave energy will be provided by the nearshore rocky reefs. To determine the fluid velocities in the lee of the submerged structure, the transmitted wave height must be found. From Seabrook and Hall (1998) the ratio of incident, H_i, and transmitted, H_t, wave heights for a submerged breakwater can be found using the empirical relationship of:

$$K_t = \frac{H_t}{H_i} = 1 - \left[e^{-0.65\left(\frac{h_s}{H_i}\right) - 1.09\left(\frac{H_i}{B}\right)} + 0.047\left(\frac{B\,h_s}{L\,D_{50}}\right) - 0.067\left(\frac{h_s\,H_i}{B\,D_{50}}\right) \right] \quad (11)$$

where h_s is the level of submergence, B is the structure width at the crest taken as 175 feet, L is the local wavelength calculated from design wave period of 18 seconds. The stone size has been previously shown in Table 5-1.

Aside from designing for adequate stability of the protective structure, the eelgrass should not be uprooted or damaged during a typical storm event. Therefore, the 1-year event is used as a design condition for the structure crest height. In the event that the 1-year storm is exceeded, the eelgrass may be damaged and would need to re-grow.

To achieve the required velocity for eelgrass growth, the wave height must be lower than 1.5 ft on the lee of the structure. Although this does not account for the diffraction of waves around the submerged breakwater, the results provide an adequate estimation of the velocities encountered near the eelgrass beds.

Table 5-4: Transmission of Waves across Submerged Breakwaters during Typical Conditions

h_s ft	H_i ft	H_t ft
0	2.75	1.10
3	2.46	1.23
5	1.74	1.30
7	1.51	1.16
10	1.02	0.72

Due to the submergence of the protective structures, the effects on the small, short period waves is negligible. These waves pass directly over the structure, but do not shoal enough to break and dissipate energy. Sea level rise will have the greatest effect on the eelgrass measures than any other measure. With an increase in total water level, the protective submerged reefs will become more ineffective at dissipating energy from the small waves. Raising the crest elevation of these structures in advance would decrease the nearshore wave energy but may cause water to "pile up" on the lee of the structure and hinder the return flow. Therefore, future maintenance on the nearshore reefs is needed to counter the effects of sea level rise while keeping the structures at the design crest elevation. Additional stone can be added during a period of routine maintenance so as to not incur an additional mobilization cost.

Eelgrass also prefers shallow water where light penetration is abundant, generally from a 3 ft. to a 10 ft. depth range. The nearshore reefs will both create calmer waters in the lee of the structure but will also aid in the creation of a perched beach, similar to Figure 5-3. More analysis will need to be performed in

PED to optimize the design of the perched beach. Sediments may naturally accrete in the lee of these structures but to ensure the required depths for eelgrass to thrive, borrow may be required. As little as none and as much as 600,000 yd³ may be needed to obtain the required elevation. For cost estimating purposes and environmental consideration, 100,000 yd³ of beach quality sand will be obtained from the Surfside/Sunset borrow area and placed in the lee of the nearshore reefs.

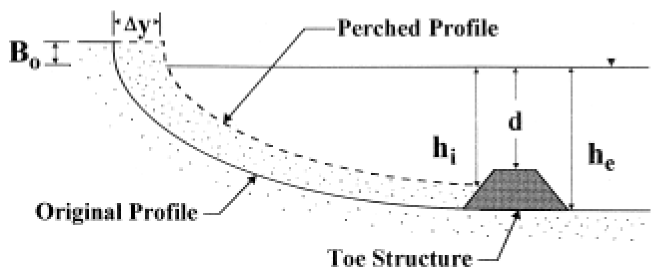


Figure 5-3: Perched Beach Example (González et al, 1999)

5.1.3.2 CONSTRUCTION AND MAINTENANCE

Planting of transplanted eelgrass will be required to begin the colony. For further descriptions, see the MAIN REPORT and APPENDIX D – MODEL DOCUMENTATION FOR THE SOUTHERN CALIFORNIA COASTAL BAY ECOSYSTEM MODEL. The required sand will be dredged from the Surfside/Sunset borrow area and dumped by a scow in as shallow as possible nearshore depths of 10-15 ft (MLLW). It is assumed that dredging at the borrow area can occur 24 hours a day and 7 days a week with two scows would working in tandem; allowing the placement of 4,000 yd³ per day. Expected downtime and shift changes will limit the working time to 22 hours per day but engines would remain idle during this time. Natural processes will re-distribute the sediment along the profile, allowing the perched shoreline to be held in place by the nearshore reefs. No maintenance is expected to be performed on the eelgrass beds after the monitoring and management.

5.1.3.3 RELATIVE SEA LEVEL RISE

As the water level rises, the eelgrass beds will become less productive. Since the suitability of the bed is dependent on the water level, even with a small increase of only 2.5 ft in the high case, the eelgrass will need to migrate to an area of shallower water depth. This will occur naturally since the expected sea level change over time will occur throughout multiple growing seasons allowing to the habitat to shift in the landward direction. If in the event that the landward migration is not occurring to keep up with sea level change, thin-layer placement techniques involving spreading successive layers of sediment over the existing eelgrass beds to raise the seabed height. It is expected that not more than 20,000 yd³ will be required to raise the individual beds by 1 ft in elevation. The exact quantity will be based on the realized sea level rise when this management technique is needed but the primary adaptation would be to allow the eelgrass to naturally migrate. Thin-layer placement would occur over multiple growing seasons to give the habitat sufficient time to adapt to the excess thin layer of sediment. Natural sedimentation caused by the nearshore reefs will also aide in the eelgrass's adaptation to sea level rise Model tests during the PED phase will assist in determining the level of expected sedimentation and the need for excess sediments.

5.1.4 OYSTER REEFS

5.1.4.1 DESIGN

Oyster reefs will be constructed using shell hash and placed in the areas indicated. The only true design condition is the placement depth and the substrate type. The placement depth must be within the intertidal zone, -4 ft. to 1.5 ft. MLLW. Within the proposed project area, the only locations where this depth is encountered is on a rock revetment (or breakwater), within the Los Angeles River Estuary and along the sandy shoreline. To limit the harvesting of oyster from individuals, the Los Angeles River Estuary and sandy shoreline areas have been excluded, leaving only locations within the slopes of existing revetments. Furthermore, these locations must be sheltered from the incoming wave energy to limit the scour that may dislodge the oysters from the reef structure. Additionally, since the design location of the reefs is above the natural seabed, sediment burial will not occur; which is detrimental to the oysters (Pauley et al, 1988). The suitable locations within the proposed project area is shown in Figure 5-4 and total an area of 0.4 acres. The single Oyster Reef near Shoreline Marina has been excluded due to the extremely low habitat value and ease of public access, separation from other restoration measures and close proximity to anthropogenic activities. The final size of the oyster reefs are 0.27 acres.

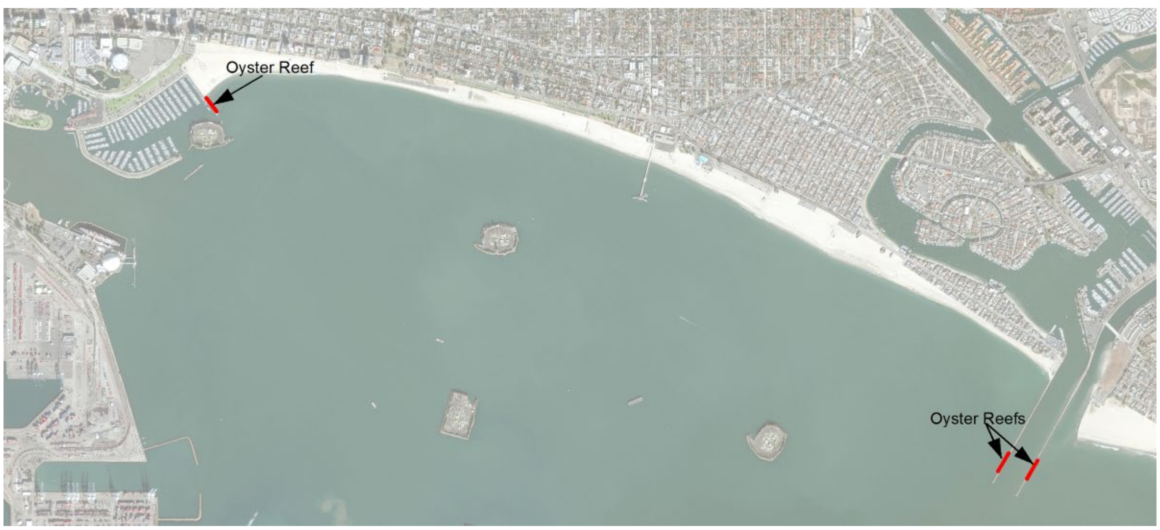


Figure 5-4: Oyster Reef Placement Locations

5.1.4.2 CONSTRUCTION AND MAINTENANCE

Shell hash will be distributed within the elevation bounds along the placement areas shown using an excavator mounted on a barge. An oyster platform can also be utilized. These floating platforms are submerged to the required depth and attached to the seabed using an anchor and cable system. Seeding of juveniles will be required directly after construction of the substrate and no additional seeding will be required after the adaptive management period.

5.1.4.3 RELATIVE SEA LEVEL RISE

Since the oyster beds are dependent on the submergence depth, sea level rise will influence this habitat. Due to the relatively small range of acceptable depths, mitigative measures may be required to keep the beds at a suitable depth. If oyster platformed are utilized, the raising of the oyster bed is easy and straight-forward; the platform will just need to be adjusted. Otherwise, if using shell hash as a base, succeeding generations of oysters will naturally colonize the previous substrate and naturally migrate to more shallow water depths.

5.1.5 TIDAL SALT MARSH

To simulate the function of a natural tidal salt marsh, placement must be near a source of fresh water inflow. Within the proposed project area, only two sites are applicable; the Los Angeles and San Gabriel Rivers. Both locations are in relatively shallow water but are adjacent to areas of commercial and recreational navigation.

5.1.5.1 DESIGN

To limit the impact on navigation, a vertical caisson structure will contain the newly created wetland area. The interior would consist of fill to approximately -5 ft. MLLW and a sand cap at the surface. Channelization would be required to simulate the wetland's natural processes and encourage adequate flushing of the system.

An assumed structural cross-section is used to estimate quantities and costs. The design includes a hollow, pre-cast concrete structure that can be filled with ballast stone to obtain the required weight for stability. From Goda (1974), parameters to calculate the horizonal forces on an impermeable wall are shown in Figure 5-5 where h_s is the water depth taken five times the design wave height from the structure, *h'* is the water total submerged structure height, *d* is the water depth at the toe of the structure, h_w is the total structure height and B_m is the berm width for scour protection. The p values and *η** are calculated values used to determine the total horizonal and uplift forces on the structure. See CEM (2001), Table VI-5-53 for a description of these equations and calculation process.

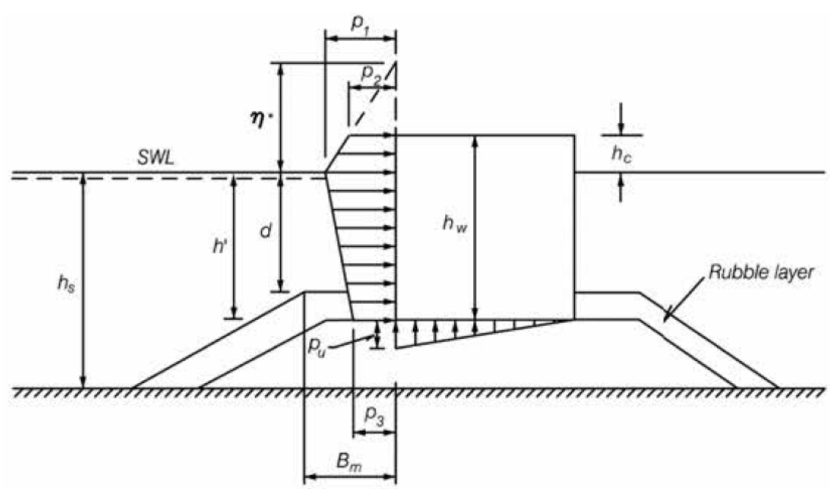


Figure 5-5: Parameters for Forces on a Vertical Structure (CEM, 2001)

The resultant approximate horizontal force is near 270 tons/ft. of caisson. The structure weight plus the resistance from the interior sediments will provide stability to the wetland complex. To protect against the design wave of 12 feet with a period of 16 seconds, approximately 22 tons/ft. of ballast would be required. The toe protection berm width is assumed to be 20 feet and the total water depth is 40 feet. Waves are assumed to be normal to the structure face to be conservative. Additional analysis would be required during the PED phase to ensure adequate stability.

Since the operation of port activities should not be impacted, the caisson structure will provide for internal wave damping to limit the wave reflection from the structure. Interaction between reflected and incoming waves can produce a wave height that is up to two times greater than the incoming wave alone. A multi-chamber perforated caisson structure, example shown in Figure 5-5, can be utilized to limit this reflection. This pre-cast unit is composed of concrete and can be fabricated both on- or off-site and transported by water and sequentially placed to create the outer edge of the proposed salt marshes. The dissipation mechanism of the perforated caisson structure is to trap waves within the units and allow turbulent processes to dissipate the wave energy, reducing the magnitude of the reflected waves. A physical model will be required in the PED phase to determine the exact structure width and the number of chambers to provide for the limited reflection. These types of reflection reducing structures have been successfully used across the Pacific and can lower the reflected wave to less than 10 percent of the incident wave height (Goda, 2010).

Figure 5-6: Example of Multi-Chamber Perforated Caisson Structure, Ko et. al. (2009)

The interior of the tidal salt marsh will require contouring of the sediment fill material to create adequate channels which, in turn, will allow for flushing of the system and limit the sediment loss over time. The design of the interior sections of the tidal salt marshes will be conducted during the PED phase.

5.1.5.2 CONSTRUCTION AND MAINTENANCE

Construction of the enclosed marsh begins by placing the foundational underlayment material. The quarry run foundation is placed en masse with a barge-mounted crane with an appropriate bucket. Individual concrete caisson structures can be cast offsite and floated to the project site and placed in the final position. Ballast is be added to the structure until it sinks onto the prepared foundation and in the correct location. Ballast stone is assumed to be quarry run for cost estimating purposes. This process is repeated until the entire perimeter of the salt marsh is defined. Fill material is then added to the interior with a tug and scow until the area becomes draft limited and the scow cannot transit any longer. The remaining fill material and the sand cap is hydraulically pumped into the structure. Interior contouring is required to define the channels and can be accomplished with a series of earth moving equipment such as excavators, dozers, scrapers and other support vehicles. The remaining ballast is placed within the interior of the caisson structure to achieve the required weight. A concrete cap is placed over the voids leaving a walkway/path that can be utilized for recreational purposes. Planting of natural wetland flora will take place soon after construction.

Maintenance is required both for the tidal salt marsh interior and structural components. Maintenance of the hard-structural components (caisson and foundation) will consist of repairing damages caused by large waves; such as replacing stone scoured out at the toe of the caisson or replacing individual caisson units that may have shifted during a storm event. Interior maintenance consists of monthly landscaping, cleaning and removal of unwanted species as well as replacement of the sediment lost from the system by tidal currents. For a conservative estimate, it is assumed that 25% of the sandy material will be lost and need to be replenished every 10 years.

5.1.5.3 RELATIVE SEA LEVEL RISE

Sea level rise will have a two-fold effect on the tidal salt marsh. The interior will become less suitable due to the increase in water depths requiring additional fill material to be added throughout the project duration. The exterior portion will experience increase rates of overtopping due to the waves super-imposed over the higher water level. Due to the surrounding depths, the design wave height will be depth-limited and the additional 2.5 feet of water surface elevation corresponding to a wave height increase of 1.5 feet will have a negligible effect of the wave force. No additional stability will be required for the tidal salt marsh perimeter.

5.1.6 <u>SANDY ISLANDS</u>

5.1.6.1 DESIGN

Sandy islands must include a naturally shaped beach to bolster the designed habitat function of the island. The core of the structure may consist of any easily obtainable material; such as unsuitable material dredged from the Ports of Los Angeles or Long Beach, quarry run, offshore dredged material from the Surfside/Sunset borrow area or any other source that may be available during construction. The core material must be capped with at least 5 feet of sand. The beach slope will be constructed with a 10 horizontal to 1 vertical slope that will eventually be re-distributed by local wave and current processes to create a more natural beach face. The color of sediment will be investigated further during the PED phase.

To protect the beach, the seaward side of the island must be armored. The elevation of the seabed at the toe of the structure is -20 ft. MLLW and the crest height of the armoring will be +16 ft. to be on par with the height of the energy islands. The maximum wave height near the structure for a 100-year southern swell event was found to be 12.5 ft. This wave height requires a median stone weight of 11 tons as determined by van der Meer (1988) for adequate stability with some wave overtopping shown in Eq (12).

$$\frac{H_s}{\Delta(f_i D_{n50})} = 6.2\, S^{0.2}\, P^{0.18}\, N_z^{-0.1}\, \xi_m^{-0.5} \text{ for } \xi_m < \xi_{mc}$$

$$\frac{H_s}{\Delta(f_i D_{n50})} = 1.0\, S^{0.2}\, P^{-0.13}\, N_z^{-0.1}\, \cot(\alpha)\, \xi_m^{P} \text{ for } \xi_m > \xi_{mc} \tag{12}$$

where *Δ* is the ratio of the unit weight of stone to the unit weight of water minus one, f_i is a reduction factor when wave overtopping occurs, D_{n50} is the nominal diameter of stone, *S* is the allowable damage area taken as 2, *P* is the structure permeability coefficient taken as 0.4, N_z is the expected number of incident waves during a storm event assumed to be 2,700 during a 12 hour event and α is the structure slope set at 2:1 (H:V). The Iribarren numbers are calculated using the mean wave periods. The reduction factor allows for a force reduction when wave overtopping or overwash occurs, which is expected. For a full explanation of the input parameters, see CEM (2001), Table VI-5-23.

With the addition of sea level rise, the increase in water surface elevation alone does not increase the incident waves, as this is not a depth limited condition, but raises the potential for excessive overtopping due to the elevated water surface. During storm events, overtopping is expected, calculated using the van der Meer and Janssen (1995) formulation, shown in Eq. (13), structure slope of 2:1 (H:V) and shown in Table 5-4. A reduction factor of 0.55 is used to simulate the rubble mound roughness. As the water level increase the run-up and overtopping also increases.

$$\frac{q}{\sqrt{gH_s^3}}\sqrt{\frac{s_{op}}{\tan(\alpha)}} = 0.06\exp\left(-5.2\frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{tan(\alpha)}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right)\text{ for }\xi_{op} < 2$$

$$\frac{q}{\sqrt{gH_s^3}} = 0.2\exp\left(-2.6\frac{R_c}{H_s}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right)\text{ for }\xi_{op} > 2 \tag{13}$$

In the van der Meer and Janssen formulation, *q* is the discharge per linear foot, s_{op} is the wave steepness using the peak period, R_c is the structure freeboard and α is the structure slope. The γ values are reduction factors for structure roughness, berm influence, shallow water and wave angle. ξ_{op} is the Iribarren number also calculated using the peak period. The 2 percent runup elevation is calculated using Eq. (4).

Table 5-5: Run-Up and Overtopping for Nearshore Emergent Islands

					Average Discharge, *q* (van der Meer & Janssen, 1995)				
Event	**H_s**	**T_p**	**W_{50}**	**$R_{2\%}$**	**MLLW Rc = 16 ft**	**MHHW Rc = 10.5 ft**	**Low SLR Rc = 7.1 ft**	**Med. SLR Rc = 6.4 ft**	**High SLR Rc = 4.2 ft**
	ft	**sec**	**ton**	**ft**	**ft³/s/ft**				
1-yr	6	16	N/A	11.8	1.6E-05	0.002	0.035	0.063	0.419
50-yr	11.5	16	N/A	22.7	0.032	0.384	1.769	2.416	6.475
100-yr	12.5	16	11	24.6	0.064	0.636	2.593	3.455	8.557

5.1.6.2 CONSTRUCTION AND MAINTENANCE

Construction of the sandy islands is accomplished in a similar manner as that of the original construction of the offshore energy islands. The interior fill material will be placed in lifts throughout the footprint. Material can be placed by scow and tug until reaching a limiting depth where the scow can no longer safely transit over the mound. Typically, this depth is near 10 ft. but the 6 ft. tidal range can be utilized to reduce the down-time during construction. After this limiting depth is reached, material will be pumped hydraulically into place. Armoring of the seaward side must be concurrent with the placement of the fill material to limit losses due to sediment transport. Filter and armor stones are individually placed with a barge mounted crane to achieve the required stone placement, interlocking and structure slope needed for stability. After all the fill material and armoring is placed, the sand cap can be constructed to achieve the design elevation and meet sediment color requirements. Construction equipment will include a dredge (hydraulic and/or mechanical), scows to transport and place material, a derrick crane and rock barge for stone placement and various support vessels. Earth moving equipment will be required to groom the sand cap and create the constructed beach slope.

Yearly maintenance will be required to clean and groom the sand along with weeding and grubbing to limit the vegetative cover and invasive species. Since this feature is not natural to the area, the sand cap is expected to be lost over time through natural processes. Period nourishment will be required; it is estimated that 50% of the sand material will need to be added every 10 years. Maintenance of the armored slope will occur approximately every 10 years or when needed. For rubble mound structures, 1% of the initial construction cost per year is estimated to account for repair costs.

5.1.6.3 RELATIVE SEA LEVEL RISE

Like the tidal salt marsh, sea level rise will affect both the beach and stone sides of the islands. As water levels rise, the depth-limited wave may be increased by 1.5 feet which is negligible for stone stability. Losses of sand predicted by the Bruun Rule suggest that more than 100 feet of beach could potential be lost if sea levels increase by 2.5 feet over the 50-year project duration. This shoreline recession corresponds to approximately 200,000 yd^3 of additional sand required to maintain the shoreface and slope.

5.1.7 <u>TRAINING WALL FOR L.A. RIVER</u>

A rubble mound training wall could be used to guide the outflows from the Los Angeles River (LAR) to a direction where it would be directed into the open ocean.

5.1.7.1 DESIGN

Stone weights would vary depending on the exact placement location and structure length but will not be larger than 12 tons at head section. The 3-layered structure with a length of approximately 3,500 ft. would be connected to the existing detached breakwater at Shoreline Marina and extend in a southerly direction into the bay. The total quantity of stone is estimated to be 500,000 tons of various sizes from quarry run for the core material up to 12 tons for the largest armor stones assumed from experience in the project area.

In conjuncture with the training wall, a deeper channel expanding on the federal channel coming from the Los Angeles River Estuary (LARE) would be needed to ensure a similar discharge rate from the LAR and limit impacts of increase water surface elevation upstream. Approximately 400,000 yd^3 of sediment would need to be dredged. This material could be used for fill in other restoration measures. No sediment is expected to be disposed of off-site.

Future design work would be needed to ensure excess reflections coming from the rubble mound structure are kept to a minimum to lessen impacts to port operations. More detailed modeling will be required to simulate the altered sediment transport patterns from material existing the LAR and the location of the final deposition.

5.1.7.2 CONSTRUCTION AND MAINTENANCE

Construction of the rubble mound structure would be performed with a derrick crane and barge as well as transport scows and support vessels such as tug boats. Core material (quarry run) would be pushed off the material transport scow to create the required footprint. The derrick crane would then place filter and armor stones individually to obtain proper interlocking of the stone. Dredging of the channel extension could be performed by either a mechanical or hydraulic dredge. Material would be excavated from the seabed and sieved into a scow to transport the material to the other habitat feature location.

Maintenance of the rubble mound structure will be conducted approximately every 10 years or as needed after a large wave event and would consist of replacing ejected stones and resetting existing stones to bring the structure back to the design condition. Dredging would need to be performed every 3 years but can be combined with the current federal LARE dredging. Each dredging event will remove around 250,000 yd^3 of material but is dependent on rainfall and the discharge of the LAR. This material can be beneficially reused for other habitat measures or placed in the nearshore environment. In the event that material is un-suitable for re-use; it can be placed at LA-2, an offshore EPA regulated placement site, approximately 11 miles away in 1000 ft. of water.

5.1.7.3 RELATIVE SEA LEVEL RISE

The training wall will only be suspectable to sea level change since the breaking wave height is increased by around 1.5 ft. This increase in wave height does not have a substantial effect on the stone stability and will be neglected. Since this structure is directing a flow and not designed for protection, runup and overtopping are not important processes and increases will not have a significant effect on structure stability as the structure is to be designed to permit overtopping.

5.1.8 BREAKWATER MODIFICATIONS

For this study, three modifications to the Long Beach Breakwater are considered that may increase wave energy within the bay. It is assumed that all material removed from the breakwater can be used create protection measures that limit the impact on the surrounding infrastructure. The three modification types are: (1) Removal of a single end section, (2) removal of a section(s) within the breakwater and (3) lowering of the entire breakwater. Further investigation must be performed to determine as-built actual cross-section of the breakwater so exact quantities can be computed before construction. For this study, it is assumed that the original construction drawings (Long Beach Breakwater, shown in Figure 3-5) are applicable and provide a general basis for cost estimation. No breakwater modification will be undertaken until extensive numerical and physical model studies are conducted to illustrate all the potential effects of such a modification.

5.1.8.1 END SECTION REMOVAL

If a single end section of the Long Beach Breakwater is removed, a head section must be re-built and armored corresponding to the design conditions. Stones will be removed individually using a derrick crane and barge system. The core material (sand and silt) will either be dredged and used as fill material for other habitat measures or left in place to transport naturally. Two cases are considered in this appendix; removal of the eastern 4,500 ft. (*EastLop*) and removal of the western 4,500 ft. (*WestLop*).

5.1.8.2 INTERMEDIATE SECTION(S) REMOVAL

If a single or combination of intermediate sections is removed in a similar manner, a head section must be re-built on any newly created end section of the breakwater that would not otherwise provide for the required stability. Stones removed from the sections are re-used for new head construction or other protection measures. A few different configurations are considered, such as; two 1000 ft. notches on the eastern side (*EastNotch*), two 1000 ft. notches on the western side (*WestNotch*), a single 1000 ft. notch on the western side (*SingleWestNotch*) and a single 1000 ft. notch directly in the center (*CenterNotch*).

5.1.8.3 LOWER ENTIRE STRUCTURE

If the entire breakwater is lowered, a cap of “B” stone is required to limit the extent of erosion of the sand and silt mound. For this study, analysis is performed relating to lowering the structure to -30 ft. MLLW (Low)

5.1.8.4 PROTECTIVE MEASURES DUE TO BREAKWATER MODIFICATIONS

The breakwater modifications will require other protective measures to limit the impacts of increased waves within the study area. These measures are dependent on the breakwater modification to be employed but will consist of various rubble mound structures that provide a similar level of protection to infrastructure that is currently available within the proposed project location. Major areas of impact will be the offshore energy islands, the long Beach shoreline and port features such as the jetties at Pier J South and the revetment surrounding the port complex. Areas identified in this appendix are preliminary to suggest the cost of such a measure. Final design is dependent on the physical and numerical models to be developed upon approval of this measure.

Creation of the protective measures along existing stone revetments will be accomplished by using material removed from the breakwater. Stones will be placed individually by a crane to ensure proper placement and interlocking of adjacent stones. Interlocking with the existing structure may require shifting or re-placement of existing stones to ensure adequate stability. Nearshore submerged breakwaters can be constructed in a similar manner as typical rubble mound structure as described in the ROCKY REEFS section.

5.2 HABITAT SUITABILITY MODELING

Inputs for the habitat suitability model are calculated from numerical model outputs and other derived quantities. For further descriptions of these parameters, see APPENDIX D, MODEL DOCUMENTATION FOR THE SOUTHERN CALIFORNIA COASTAL BAY ECOSYSTEM MODEL.

5.2.1 <u>ROCKY REEF</u>

- **Connectivity:** The spatial distance between distinct reefs. Determined by placement.
- **Reef Relief:** Elevation of top of reef from seabed. Determined by design.
- **Particle Duration:** Average particle time in domain to be an indicator of residence time, see the PARTICLE TRACKING sections in the following chapters.
- **Substrate:** Type of underlying material. Determined by presented conditions and design. Either present/absent.

5.2.2 KELP REEF

- **Temperature:** Water temperature given by Figure 3-12. Does not changed due to breakwater modification or habitat feature creation.
- **Substrate:** Type of underlying material. Determined by presented conditions and design. Either present/absent of the hard substrate.
- **Depth:** Water depth from MLLW datum. Determined by existing and design conditions.

5.2.3 EELGRASS BED

- **Circulation:** Water velocity at the seabed. Determined using the superposition of:

$$\overline{U} = \overline{U_W} + \overline{U_C} \tag{14}$$

where $\overline{U_W}$ is the velocity due to the cyclic waves and $\overline{U_C}$ is the tidal current velocity at the bottom layer from the EFDC modeling. The wave velocity at the seabed is calculated using linear theory from the wave modeling to be discussed:

$$\overline{U_W} = \frac{\pi H}{T \cdot \sinh\left(\overline{k}h\right)} \tag{15}$$

with *H* as the wave height, *T* as the wave period, $\overline{k}$ as the wave number and *h* as the local water depth. Velocities are expected to be slightly higher in magnitude near the sea bed since the calculation uses the significant wave height, and not the individual waves, but gives a good overall approximation.

- **Depth:** Water depth from MLLW datum. Determined by presented conditions and design.
- **Substrate:** Type of underlying material, described in terms of % fines. Percent fines is the percentage of material that passes the No. 200 sieve. Determined by present and design conditions.
- **Temperature:** Water temperature given by Figure 3-12. This value is independent of any breakwater modification.

5.2.4 OYSTER REEF

- **Salinity:** Calculated quantity from EFDC modeling output, to be discussed in the ENVIRONMENTAL FLUID DYNAMICS CODE (EFDC) section.
- **Depth:** Water depth from MLLW datum. Determined by presented conditions and design.

5.2.5 TIDAL SALT MARSH

- **Elevation:** Elevation above MLLW datum. Determined by presented conditions and design.
- **Salinity:** Calculated quantity from EFDC modeling output.
- **Size:** Spatial size of the marsh. Determined by design condition.
- **Substrate Grain Size:** Size of underlying material, separated by more coarse or more fine. More coarse represents gravels and cobbles and more fine contains all material more fine than gravel. Determined by design and existing conditions.
- **Connectivity:** Degree of connectivity between distinct tidal marshes.

5.2.6 SANDY ISLAND

- **Vegetation Cover:** Percent of vegetation cover. Determined by design.
- **Effective Size:** Spatial size of the island. Determined by design.
- **Elevation:** Surface elevation from MLLW. Determined by design and current conditions.
- **Sediment Grain Size:** Size of sediment. Determined by existing conditions and design. As shown in Figure 3-25.
- **Distance:** Spatial distance from other islands or the mainland. Determined by design.

The spatially explicit inputs for the HEM was provided as a geo-referenced database with the above calculated parameters.

5.3 CONSTRUCTION ACTIVITIES

The total construction duration for each measure varies depending on the complexity and size of the measures. A land-based staging and storage area is required for contractor's use including access to the water. A location near Pier T within the Port of Long Beach has been tentatively identified with adequate area and water access and is shown in Figure 5-6. Typically, work involving a dredge can be conducted 24 hours a day, 7 days a week. Work involving stone placement is limited to daylight hours. Due to the location of the staging area well within the commercial port complex, no access limitations are expected.

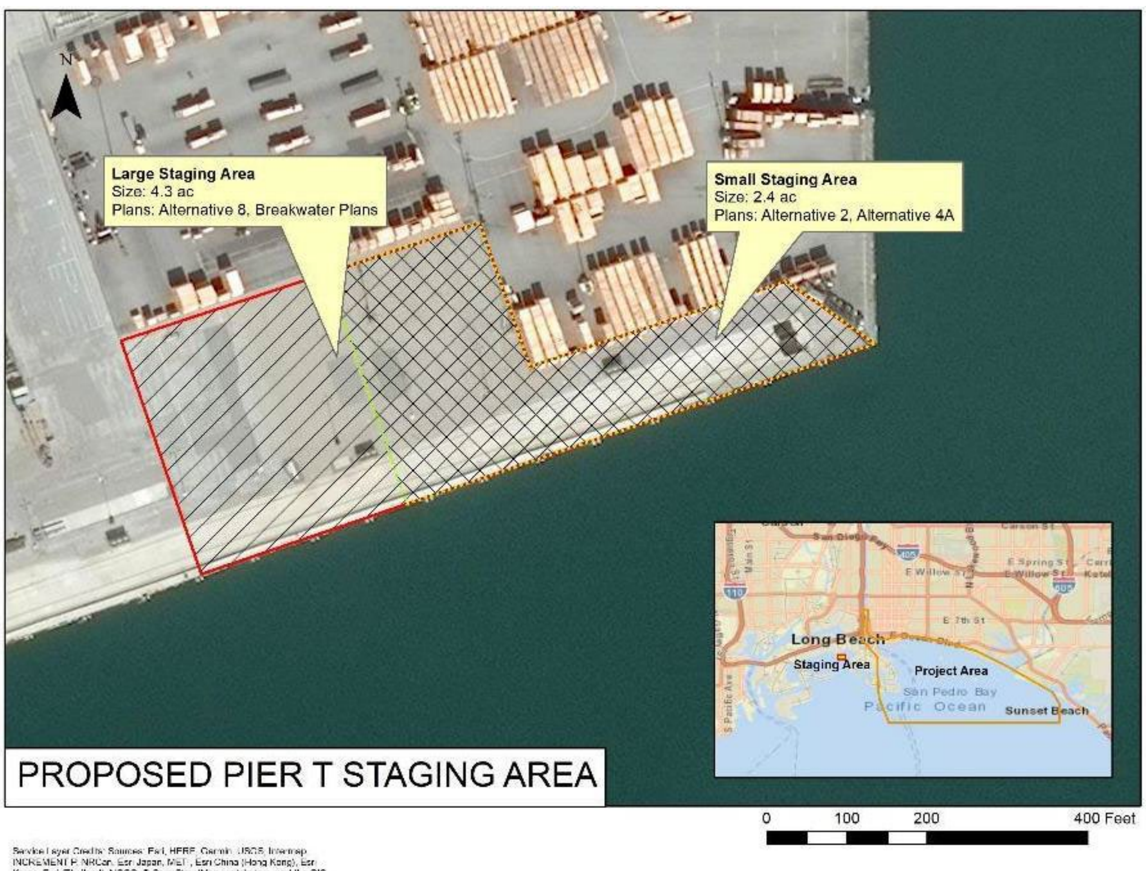


Figure 5-7: Proposed Staging and Storage Area at Pier T.

6 NUMERIC MODELS AND RESULTS

Various numerical models are developed to assist with the decision-making process and to provide quantifiable parameters or input into the HEM. The following is a description of the model set-up and results for the existing conditions.

6.1 CMS-WAVE

CMS-Wave is a steady-state, or phase-averaged, spectral wave model that calculates the transformation of waves from deep water to the nearshore environment. The formulation includes effects of energy dissipation due to processes such as bottom friction and wave breaking. Diffraction of waves around structures is handled explicitly within the model's governing equations. Although operating in a half-plane mode (waves only come from a single 180° band), the model can include back-reflections from structures and the interactions with the incident wave field (USACE, 2008). CMS-Wave has been validated and verified at numerous sites both on the West and East coasts of the U.S. and the phase-average model has also been applied to a separate USACE study within the project area (USACE, 1997). Due to the type of model, only wave periods from 3 to 25 seconds are included in the transformation; longer period waves are not specifically included, but the total energy is represented in the spectrum but distributed throughout the included energy bands.

6.1.1 MODEL SETUP

The CMS-Wave model is a component of the Surface-Water Modeling System (SMS, 2018) suite of software but can also be used independently. For this study, the SMS interface is used for bathymetry processing and model parameter setup. To transform the waves from the offshore buoy to the study area, a constant grid with ~325 ft. (100 m) cells is oriented in a manner shown in Figure 6-1, with an angle of 45° from true North as to include events from both the southern and northern directions without creating separate and independent grids.

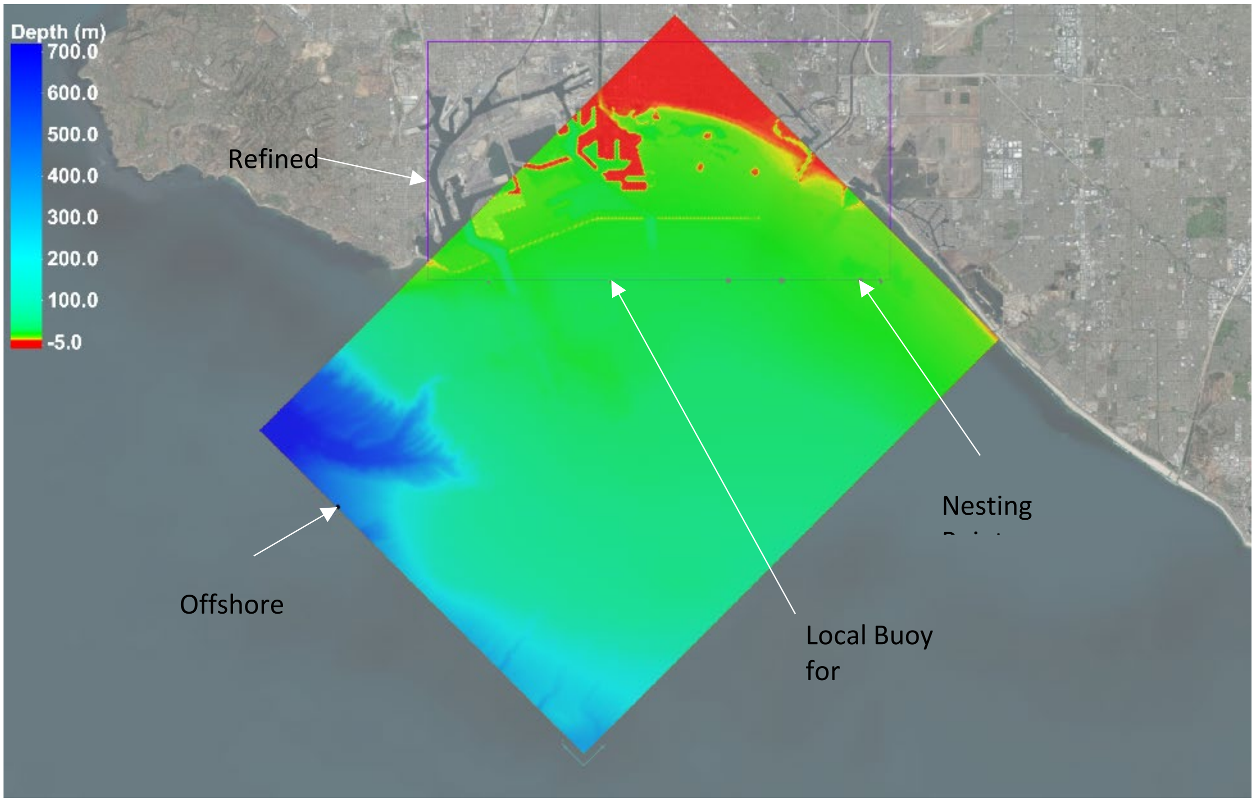


Figure 6-1: CMS-Wave Coarse Grid

For this coarse grid, the port study area does not need to be well defined. The bathymetric details will be considered in the finer grid. Nesting points to transfer the results from the coarse to fine grid are set along the grid interface as shown in Figure 6-1. These points define an initial condition for the refined model domain with constant cell spacing of ~65 ft. (20 m) angled to the North as shown in Figure 6-2. CMS-Wave allows for a structure cell, shown as brown, that can simulate the effects of rubble mound porosity and reflection.

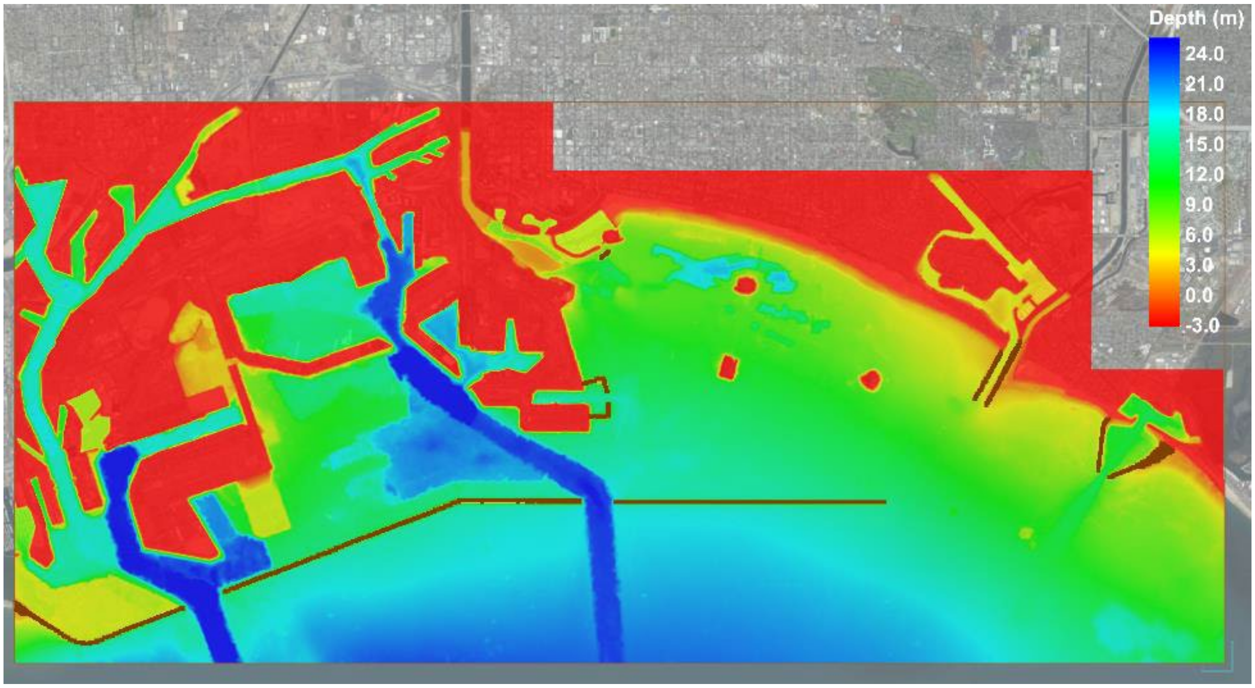


Figure 6-2: CMS-Wave Refined Grid

Since CMS-Wave is physics-based, little calibration is required to adequately represent real processes. A constant Manning's value of 0.025 is applied to the entire domain and simulates frictional effects as well as the presence of bedforms that may impact local wave transformation. Transmission through the breakwater is tuned using the parameters discussed in the LOCAL BREAKWATER EFFECTS section. The JONSWAP wave spectrum is used to drive the deep water ocean boundary with the conditions to be discussed in the succeeding sections.

6.1.2 VERIFICATION

To verify the accuracy of the wave model, waves are transformed from the offshore buoy (CDIP 092) in deep water to a local wave buoy (CDIP 215) nearer to the study area. Locations of the two buoys are shown in Figure 6-1. The local buoy is not used in the previously described extremal analysis since the duration of record is less than two years; much less time than required to make any sort of statistically valid categorizations or predictions of future events. Input conditions are taken directly from the measured spectrum at the offshore buoy. Figure 6-3 shows a time series relating the CMS-Wave modeled wave parameters with the local buoy measurements. Although the time series is not exactly aligned, the wave height trend and magnitude is on the same order and will assist with the decision making process during the feasibility stage of the study and provide insight on potential structural modifications. There are other effects, such as sheltering by the Channel Islands that make the complex wave field difficult to fully describe in a more detailed manner. The role of the wave modeling is to allow for comparisons of the various alternatives. More detailed physical and numerical modeling will be required for the selected plan.

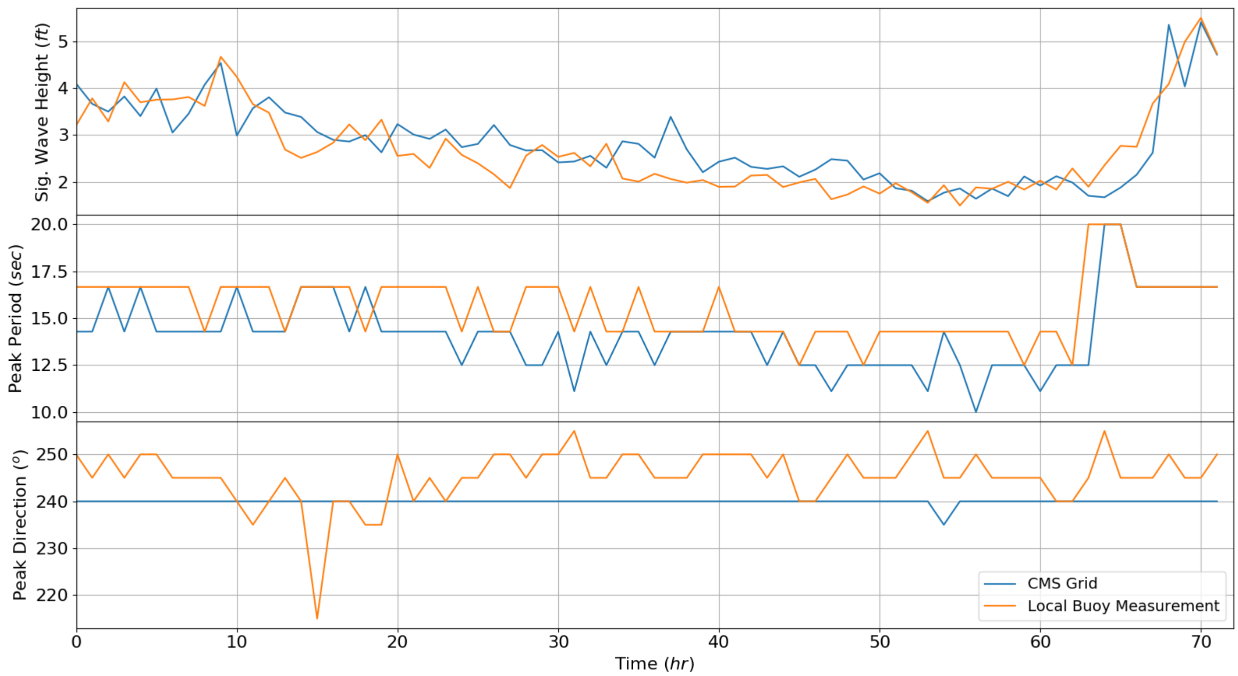


Figure 6-3: Comparison of Measured and Calculated Wave Parameters

To facilitate the environmental and engineering analysis, three sets of wave conditions are transformed from the deep water wave buoy location into the study area and described in the proceeding sub-sections.

6.1.3 SINGLE EVENTS

Using the extremal analysis presented in the LONG TERM WAVE STATISTICS section, a series of independent discrete events are created to aide in formulation and design of the restoration measures and shown in Table 6-1.

Table 6-1: Modeled Single Event Wave Conditions

Wave Condition	H_s ft.	T_p sec	D_p °
Typical Winter	3.7	12.4	259
Typical Summer	2.9	11.3	225
1-yr Northwest Swell	12.4	18.0	270
1-yr South Swell	5.8	16.0	180
50-yr Northwest Swell	16.9	18.0	270
50-yr South Swell	13.1	16.0	180
100-yr Northwest Swell	17.4	18.0	270
100-yr South Swell	14.2	16.0	180

Each wave condition is shown in APPENDIX A-2, WAVE CONDITIONS. Overall, the southern swells produce the largest wave heights within the study area due to the alignment of the detached breakwaters as well as the port complex itself. Waves from a northwest swell are greatly influenced by the offshore bathymetry which cause the incoming waves to refract to a more shore normal direction. Figure 6-4 and Figure 6-5 show the difference between the southern and northwest swells.

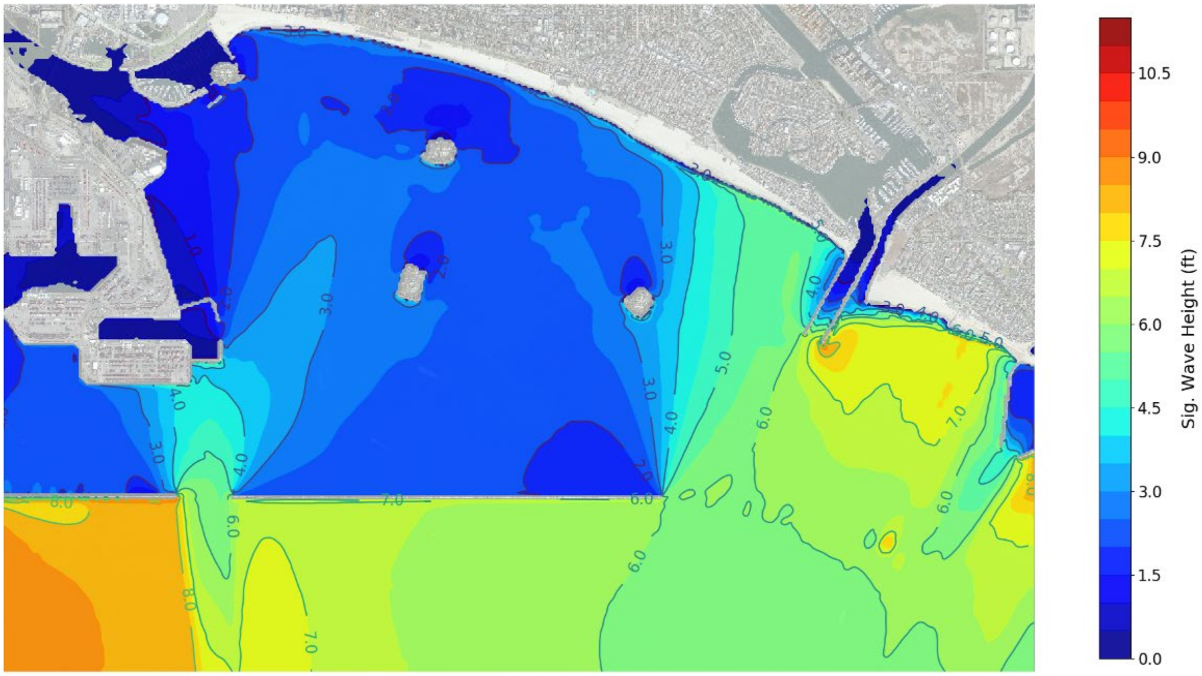


Figure 6-4: Significant Wave Height for 1-Year South Swell Event

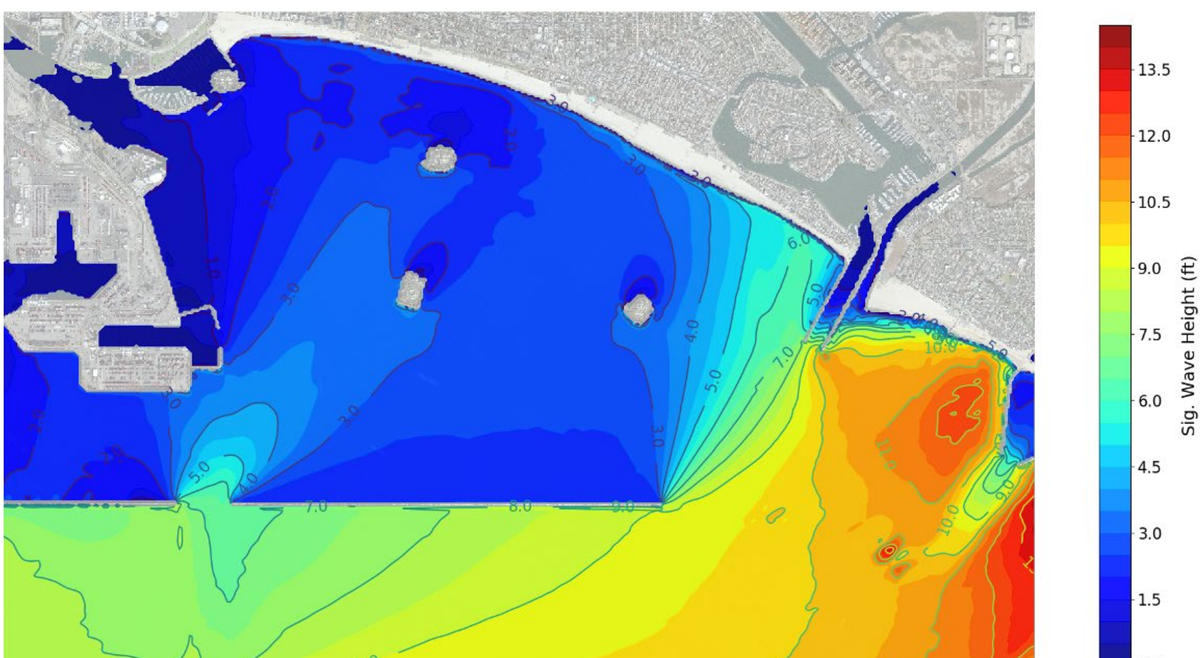


Figure 6-5: Significant Wave Height for 1-Year Northwest Swell Event

Even though northwest swells generally produce larger offshore wave heights, the sheltering provided by the detached breakwaters, Channel Islands and the Port complex itself, cause the impact on the local facilities to be smaller in most cases than when waves are generated to the south impact the study area. This is more true for the larger and more rare events.

6.1.4 TWO WEEK SEASONAL TIME SERIES

Including the effects from waves into a more detailed hydrodynamic model requires a full time series of waves. To describe the seasonality of the system, a two week time series during the summer and winter months, similar to the hydrodynamic model, is required with parameters shown in Table 3-3. The summer and winter conditions chosen are the two week periods starting on 15 July 2005 and 8 March 2013, respectively. Time series plots are shown in Figure 6-6 and Figure 6-7 for both conditions with the dashed line representing the average condition. The summer conditions are chosen since the wave height is fairly constant and the spectrum generally consists of locally generated wind-waves with a small swell in the background. The winter condition begins with a somewhat significant yearly wave event that lasts for day, followed by a fairly constant wave height with much of the energy originating from the northwest.

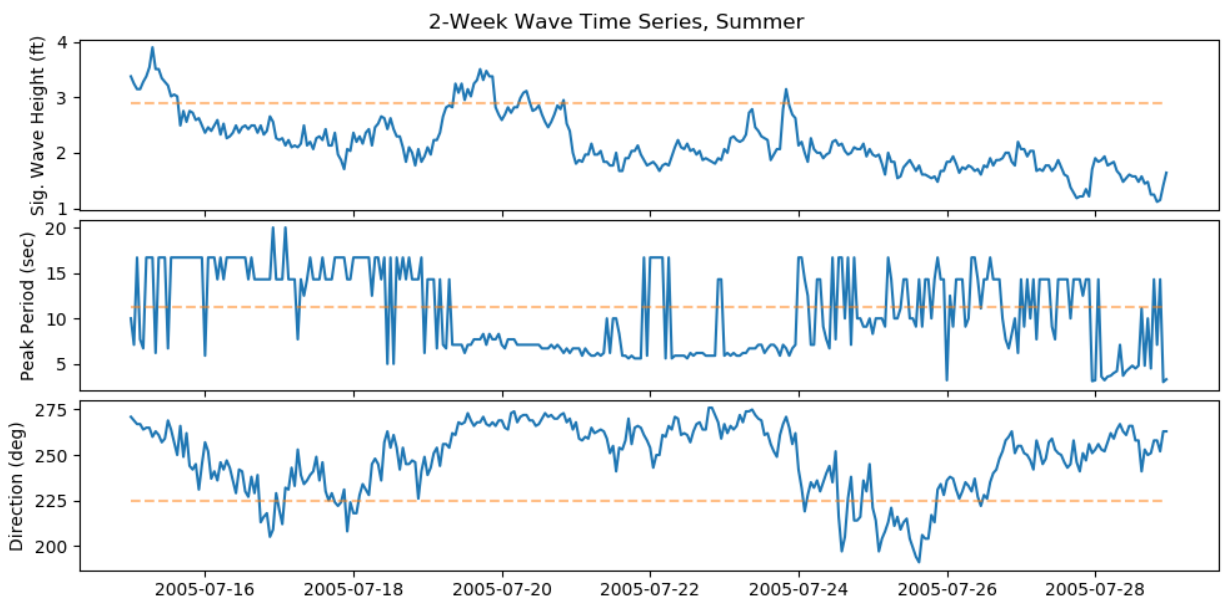


Figure 6-6: Two Week Time Series to Simulate Typical Summer Conditions

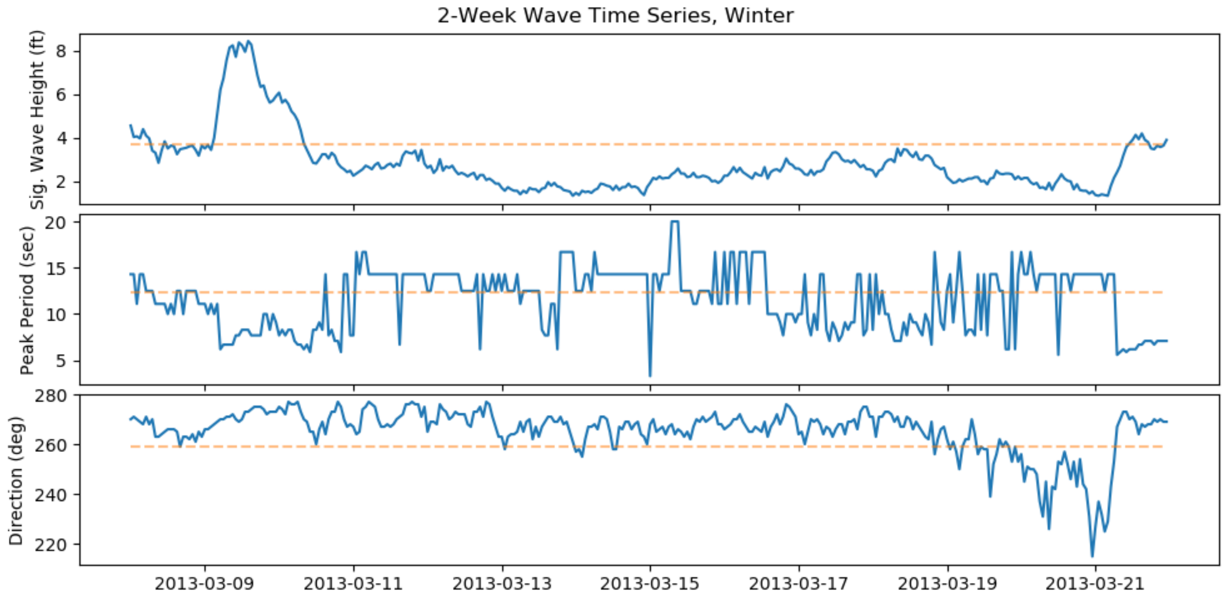


Figure 6-7: Two Week Time Series to Simulate Winter Conditions

Results of the wave transformation is similar to the results for the single events and will not be shown due to the number of runs (14 days x 6 transformations/day x 2 seasons = 112 transformations per bathymetry configuration). The output is incorporated into the EFDC hydrodynamic modeling in the form of the radiation stress gradients as defined by Longuet-Higgins and Stewart (1964). The formulation of the radiation stress gradient is based on linear theory and only applies in cases with energy dissipation; i.e. within the surf zone or a very rough surface.

6.1.5 GROUPED DATA ENCOMPASSING ENTIRE RECORD

With the inclusion of the entire wave record, the probability of occurrence of a specific wave condition can be estimated. The wave record is divided into bins in a similar manner as the joint probability analysis stated earlier. Ultimately, the wave record is decomposed into 526 distinct bins then transformed into the study area as before. After the wave transformation, the calculated probability is applied to each wave condition. The plots shown in Figure 6-8 and can be thought of as the approximate number of days per year that the threshold sea state will be exceeded; but the events need not be sequential. The reported value is the cumulative reoccurrence throughout a typical year. Although the plot is similar to a map of wave heights, the northwest swell component is the most dominate of the sea states; the maps resemble the waves originating from the northwest but show the probability instead.

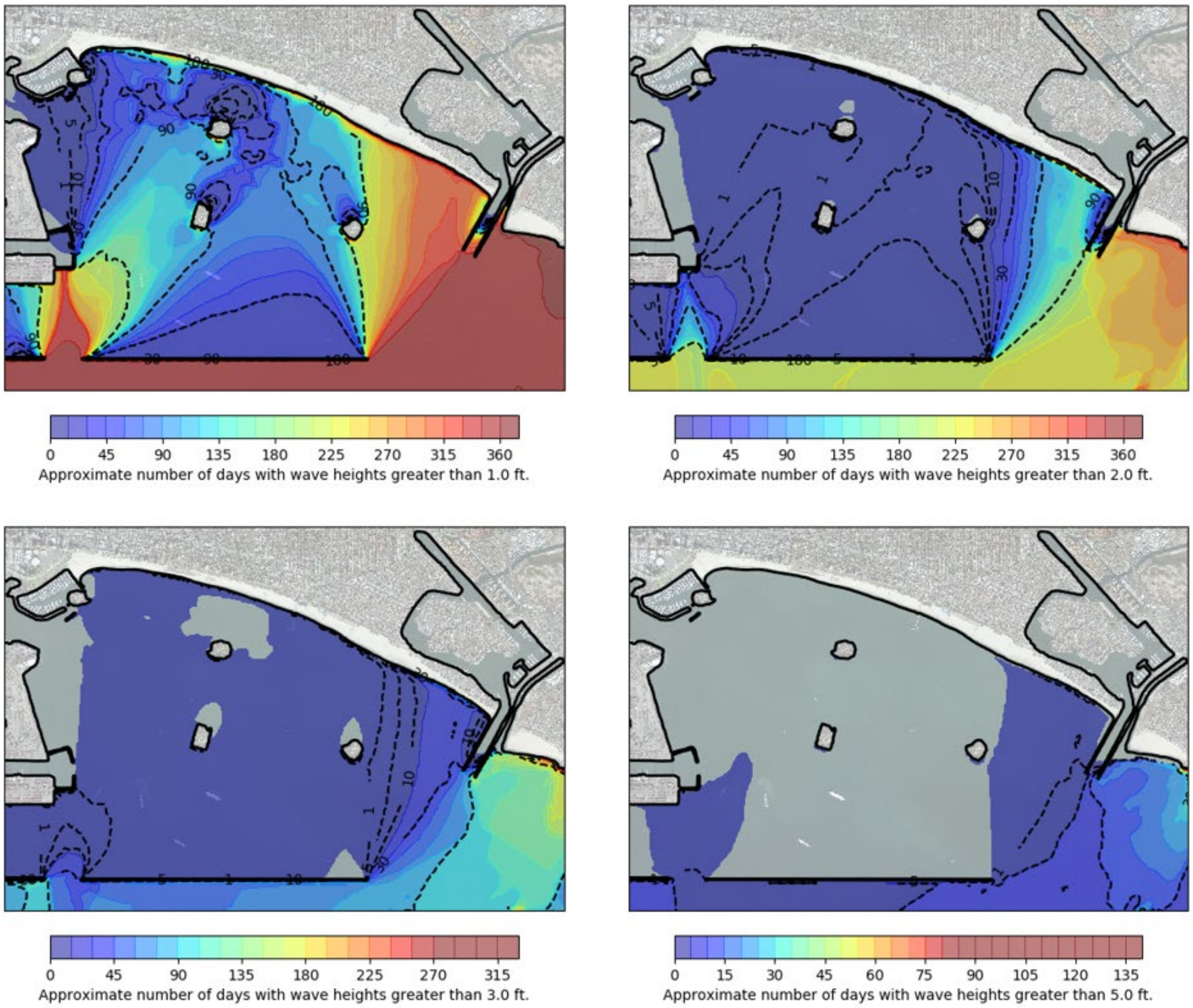


Figure 6-8: Approximate Days of Exceeded Wave Heights

It can be seen that large waves are not prevalent within the bay; they are typically on the order of 0 – 2 feet.

6.2 ENVIRONMENTAL FLUID DYNAMICS CODE (EFDC)

A three-dimensional (3D) hydrodynamic, sediment transport, and water quality model for ESPB (ESPB Model) is developed to provide hydrodynamic and water quality parameter prediction and will be used for the HEM. The ESPB Model simulations are conducted to provide currents, salinity, and total suspended solids (TSS) conditions to be used for habitat evaluation. In addition, the ESPB Model is also used for a numeric tracer tracking study, which is conducted to evaluate potential sediment (and associated contaminants) transport from the Los Angeles and San Gabriel Rivers into ESPB. Further information and validation of this model can be found in Everest (2017a & 2017b). Further discussion regarding the ESPB Model can be found in APPENDIX A-1, EFDC MODELING.

EFDC solves the 3D conservation of mass and momentum equations for the water surface elevation and the x-, y-, and z-components of the velocity at discrete locations or layers throughout the modeled water body; for this application, five vertical layers are used. Using the full 3D equations provides for a solution even if nothing is known about the geometrical constraints or physical simplifications that are applicable to the water body being modeled. In this case, all terms in the governing equations might be important, and no a priori scale analysis can be performed. The 3D solution remains applicable event when vertical gradients in temperature, salinity, and suspended sediment concentrations are present. Therefore, vertical and horizontal internal pressure gradients, due to density differences between different water masses, will be important for the acceleration of the fluid body in these cases.

The ESPB model calculates the water surface elevation, velocity field, salinity concentration and total suspended solids concentration throughout the modeling domain for a specific set of modeling scenarios. A 14-day timeseries of typical summer and winter conditions was developed that considered a range of tidal elevations, wave conditions and river inflow rates (during the winter months). Flow conditions exiting the Los Angeles River are selected from a year with the average annual precipitation. A timeseries with two flow events is chosen to simulate a smaller first flush event followed by a larger secondary flow event. The first flush is typically not indicative of typical conditions, since the episodic flows from the Los Angeles River do not occur too often; first flush events usually contain large concentration anthropogenic materials.

The calculated velocity field for the existing conditions is shown in Figure 6-9 corresponding to the surface and bottom layers of the ESPB Model. Within the majority of the modeled area, currents are typically small with velocities around 0.02 knots. Currents are typically larger in magnitude at the surface during wet weather flows and between constriction points such as Queen’s Gate. Timing of the flow conditions show in the following figures is show in Figure 6-10.

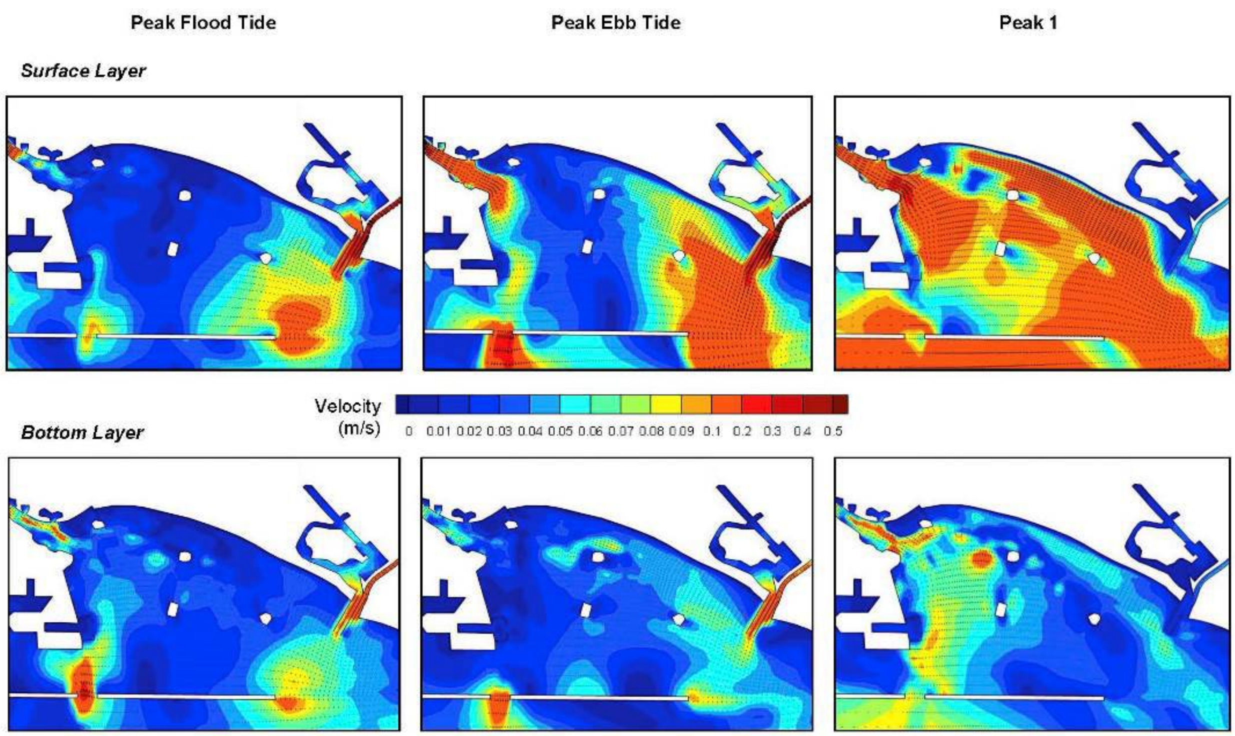


Figure 6-9: Existing Conditions Velocity during Winter Months
Note: units in metric, 1 m/s = 1.94 knots

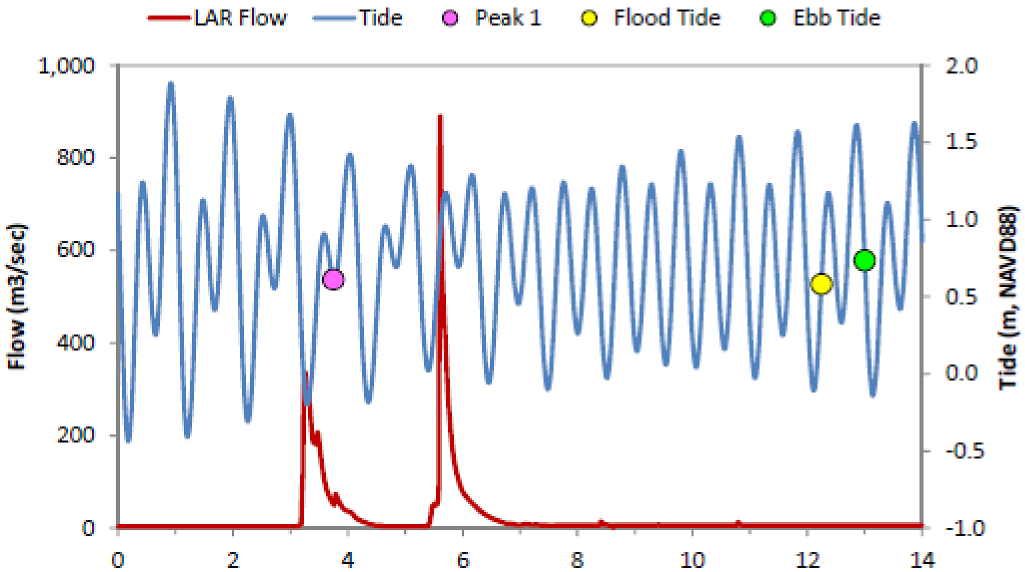


Figure 6-10: Timing of Flow Conditions during the Winter Season for EFDC Modeling
Note: units in metric

Salinity concentrations are presented in Figure 6-10 and show the greatest decrease during fresh water discharge events from the rivers in the study area. Lower salinity concentrations propagate out from the two rivers and into the bay during this time. Besides during times of wet weather, there is minimal stratification of salinity concentrations within the five levels which remain fairly constant. Salinity levels generally return to normal concentrations within 24 hours after the primary flow from the rivers has ceased.

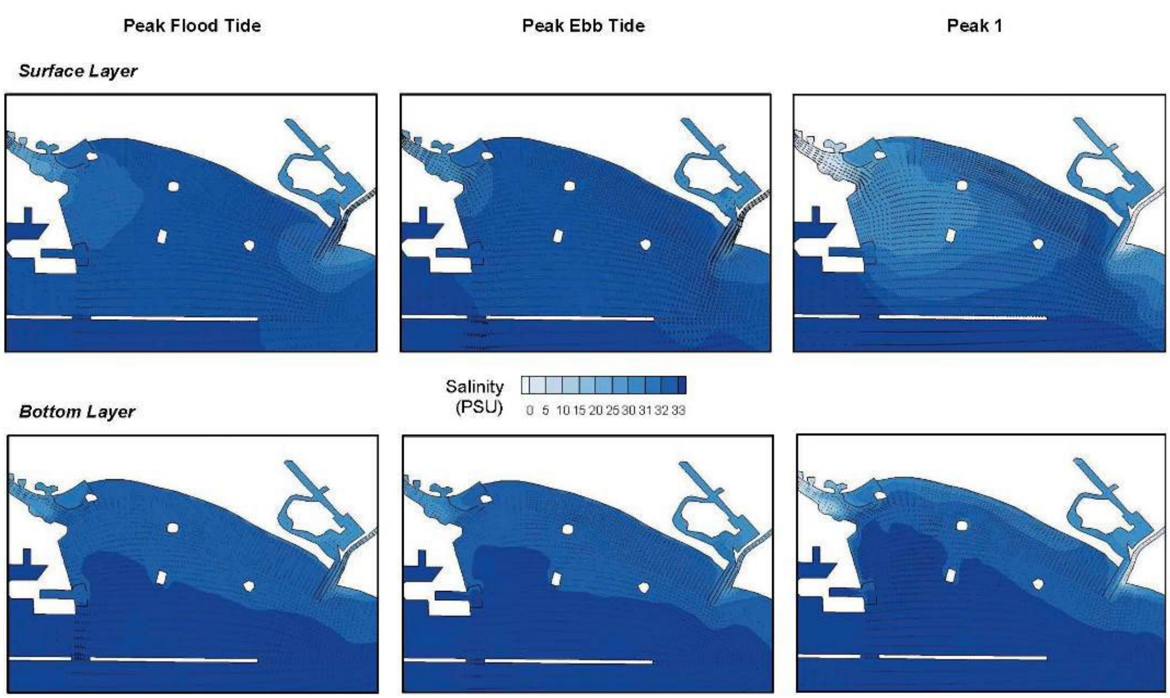


Figure 6-11: Existing Conditions Salinity Concentrations during the Winter Season

The suspended sediment concentration, shown in Figure 6-11, is highest near the river outflows, especially during wet weather flows. Surface and bottom concentrations are similar throughout the bay. Although with the inclusion of waves, the oscillatory effect is not modeled with EFDC, which allows for the resuspension of sediments after being initially deposited. The transport patterns would remain similar regardless of this assumption, but the concentration my increase in the shallower areas.

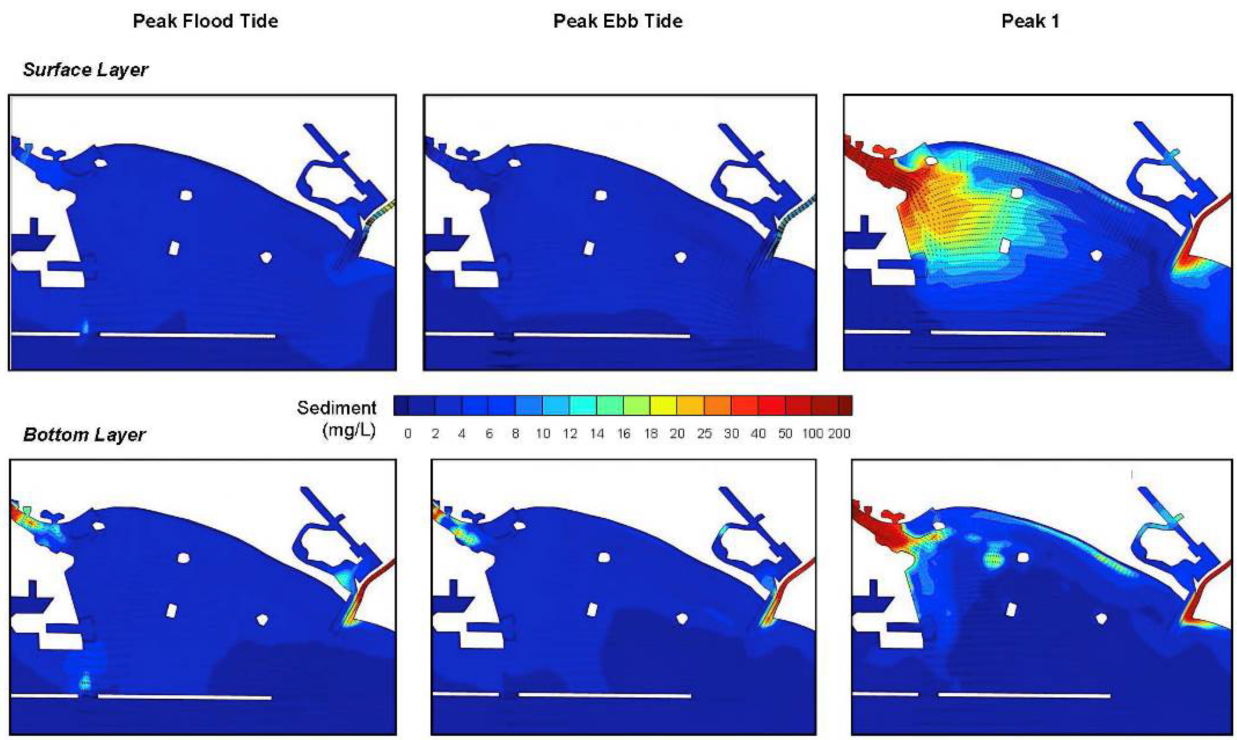


Figure 6-12: Existing Conditions Total Suspended Sediments

Five working scenarios were developed to show how various structural changes alter the velocity field throughout the bay, listed in Table 6-2. Results of these scenarios can be found in IMPACTS TO COASTAL PROCESSES and APPENDIX A-1.

Table 6-2: EFDC Modeling Scenarios

EFDC Model Name	Habitat Features	Structural Changes
Scenario 1	Nearshore Reefs, Tidal Salt Marsh	N/A
Scenario 2	Nearshore Reefs, Tidal Salt Marsh	2x Eastern Notches
Scenario 3	Nearshore Reefs, Tidal Salt Marsh	Eastern 1/3 Removal
Scenario 4	Nearshore Reefs, Tidal Salt Marsh, Emergent Islands	LAR Training Wall
Scenario 5	Nearshore Reefs, Tidal Salt Marsh, Emergent Islands	Eastern 1/3 Removal
Scenario 6	N/A	Complete Lowering

6.3 PARTICLE TRACKING

As part of the EFDC modeling work, presented in APPENDIX A-1, a numeric particle tracking study was performed on the initial set of modeling scenarios to investigate potential structural changes and impacts to the circulation or transport within the bay. Although not true residence time of the project area, this analysis is used to develop the representative duration of a typical mass of water within the bay.

Based on the five release locations, representative areas are defined by polygons shown in Figure 6-13 with the release times shown in Figure 6-14. The particle duration is determined by the start time of the release, progressing until the numeric particle reaches the edge of the originating polygon. Values at each release time and layer are averaged to give the representative duration within the polygon, shown in Table 6-3 for the existing conditions. Only the winter months are presented in the bottom and surface layers, since concerns of water quality and contaminate transport are typically express after a flow event when large amounts of sediments are flushed from the upland watersheds. Intermediate layers are limited by the boundary layers and will not experience durations shorter or longer the bottom or surface layers, respectivly.

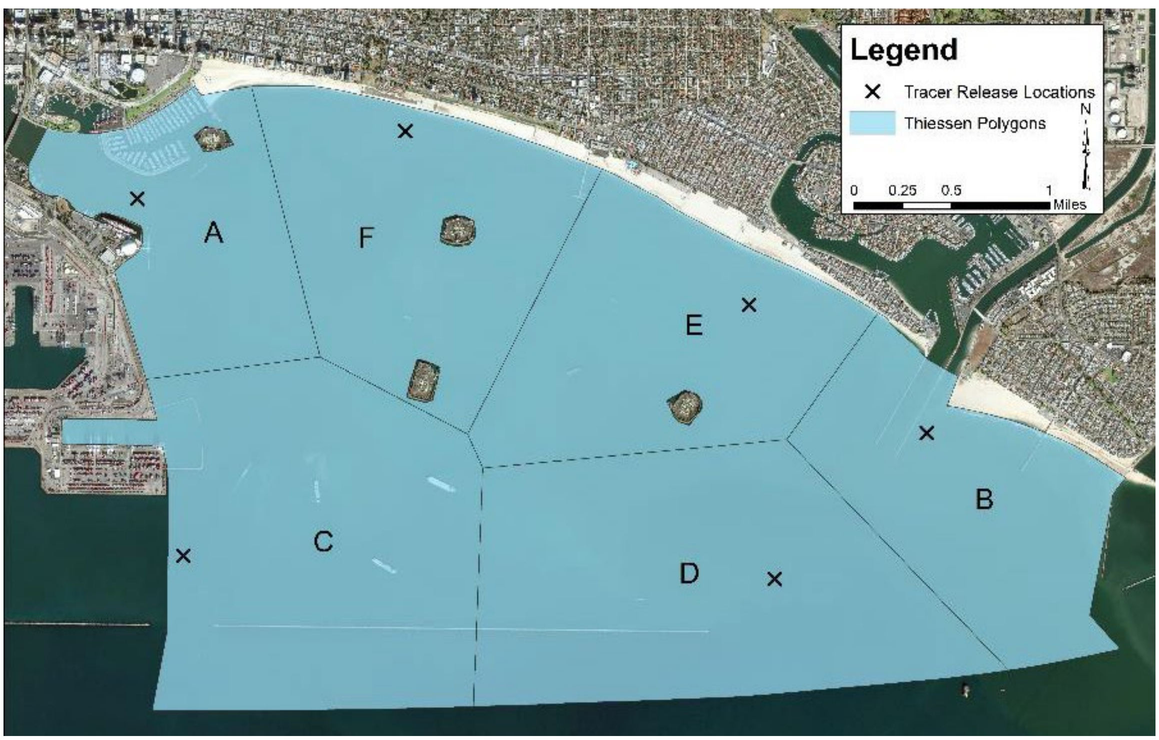


Figure 6-13: Domain Decomposition for Particle Tracking Time Calculations

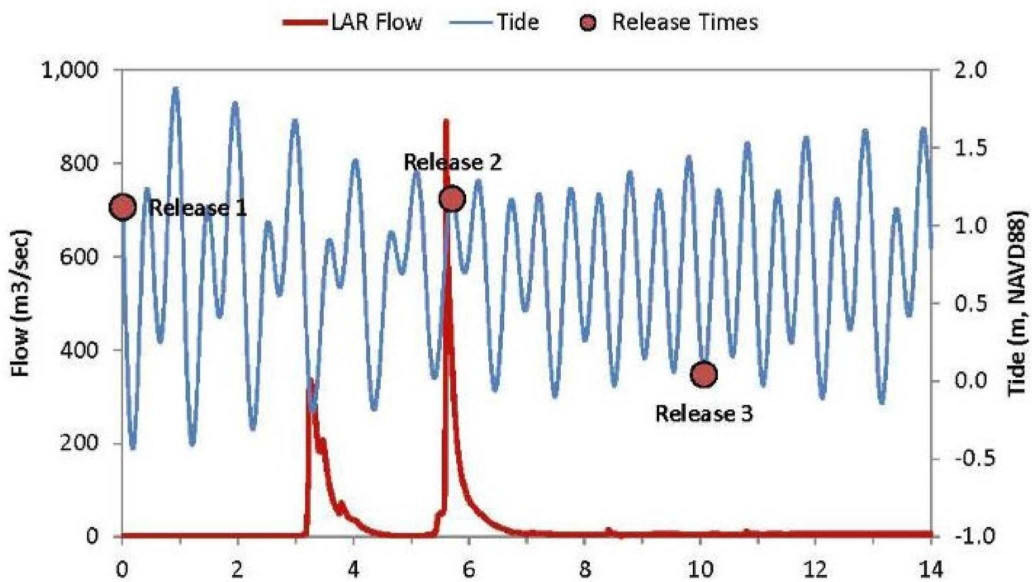


Figure 6-14: Release Time, in Days, of Numeric Tracers
Note: units in metric.

The representative duration is highly dependent on the location within the proposed project area and the time of particle release as it relates to the tidal cycle. Averaged together, the various release times and locations shows how the system will act due to a structural change such as breakwater modifications. Average particle duration for each area is provided for habitat modeling purposes. Plots of the results of the numeric tracker study can be found in PARTICLE TRACKING AND TRANSPORT or APPENDIX A-1.

Table 6-3: Representative Particle Duration, in Days, for Existing Conditions

Layer	Area						Average	Sum
	A	B	C	D	E	F		
Bottom	8.73	1.55	1.02	0.55	1.24	1.26	2.39	14.34
Surface	0.34	0.99	3.64	0.36	0.86	0.88	1.18	7.07
Average	4.53	1.27	2.33	0.45	1.05	1.07	1.78	10.71

6.4 BOUSS-2D

Bouss-2D is a phase-resolving wave model that utilizes a mild-slope version of the Boussinesq-type equations for more complex wave transformations within the proposed project area. This phase-resolving model utilizes more complex governing equations than the above describe CMS-Wave (phase-averaged) model. Although formulated to include, the CMS-Wave model cannot accurately resolve rapid variations that occur at sub-wavelength scales due to wave reflection and diffraction; phase resolving models are better suited for problems involving the reflection/diffraction problem. Furthermore, the Boussinesq-type equations are valid for a wide range of water depths from transitional depths to the nearshore environment. Bouss-2D has been validated and verified on both U.S. coasts as well as the high energy environments of the Northern Pacific Ocean in both Hawaii and Alaska (USACE, 2001).

For this analysis, only discrete events are modeled to present the difference between the two models. Because of the long computation time, only the southern swell events are modeled due to the susceptibility of the harbor from waves originating from this direction. More analysis will be needed during pre-construction and engineering design to apply a more stringent calibration and to verify the permeability of the breakwater with respect to different wave conditions and water levels. A more detailed long wave analysis will be required to observe the effects near the port terminals, marinas and docks and the effects on ship motions while moored or transiting to/from berth.

For the Bouss-2D modeling, the numeric grid is composed of 1,031,711 nodes with a constant cell size of approximately 33 ft. (10 m). Numerical damping is employed at open boundaries and structures to reduce the unrealistic reflection within the domain and presented in Table 6-4.

Table 6-4: Numerical Damping in Bouss-2D

Damping Type	Width	Coefficient
Open Boundaries	30	0.3
Structures and Breakwaters	60	0.3
Shorelines	30	0.6

Bottom friction is applied using a Chezy factor calculated from,

$$C_z = 18\log\left(\frac{12h}{k_s}\right) \tag{16}$$

where h is the local water depth and k_s is roughness height and can be approximated as $3*D_{90}$. The seaward boundary is artificially defined with a constant depth that slowly slopes to match the existing bathymetry. The total simulation time is 3600 seconds (1 hour), but the first 30 minutes are discarded and assumed to be the ramp-up period to fully develop the wave field within the bay. The time step to produce a Courant Number around 0.5 is set to 0.25 sec. Other parameters were left as default values. The bathymetry for the Bouss-2D simulations area shown in Figure 6-15. An internal numeric wavemaker near the open ocean boundary, defined from the TMA spectrum outputted by CMS-Wave, and is used to provide boundary conditions. Waves propagate away from the wavemaker source in both a southern and northern direction. Waves traveling south are quickly absorbed by the damped boundary that is approximately 500 ft. away. Figure 6-14 shows a sample output of the water surface elevation generated by Bouss-2D during a 1-yr southern swell event.

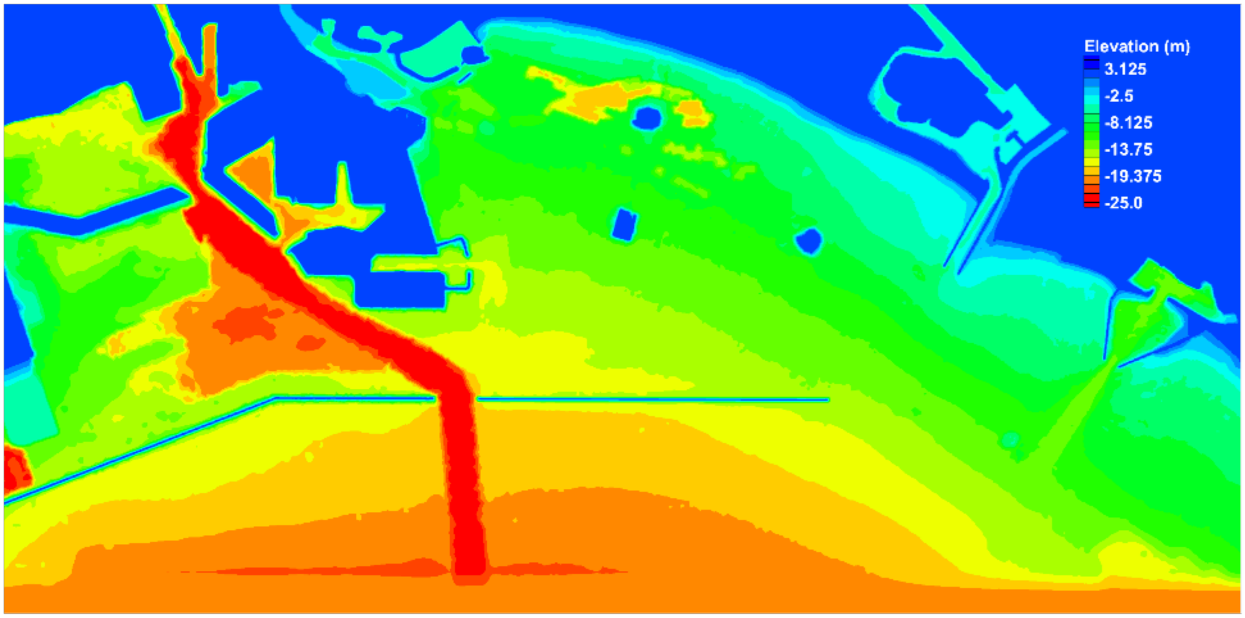


Figure 6-15: Bouss-2D Grid

As shown in LOCAL BREAKWATER EFFECTS, the porosity of the breakwaters allow for energy to be transferred through the structures. Bouss-2D allows for the protective breakwaters to be modeled as permeable or impermeable. Through calibration and suggestions by the model developers, the porosity parameters employed are shown in Table 6-5. This combination of values most closely resembles the results from the historic porosity experiments on the breakwaters in the 1970's. The difference between modeling the breakwater as porous using factors from Table 6-5 and impermeable using damping factors shown in Table 6-4 is presented in Figure 6-17. Since no artificial damping is employed when the structure is modeled as porous, the reflection of incident waves become larger than if waves were absorbed; the majority of this energy reflected should be dissipated by wave breaking and other turbulent processes along the rubble mound structure but is not accounted for in this model. The area of interest is not directly affected by these excessive reflections and will be ignored for this analysis. The inclusion of porosity has the largest effect within the project area near Bixby and Belmont Pier (see Figure 7-6 for naming conventions) where reported wave heights can increase more than 1 ft. without any breakwater permeability, for the 1-yr southern swell event.

Table 6-5: Porosity Coefficients and Factors used as Inputs

Bouss-2D Porosity Parameters	
Width	30 m
Porosity	0.4
Stone Size	1.5 m
f_{lam}	5000
f_{turb}	1
M_{add}	0

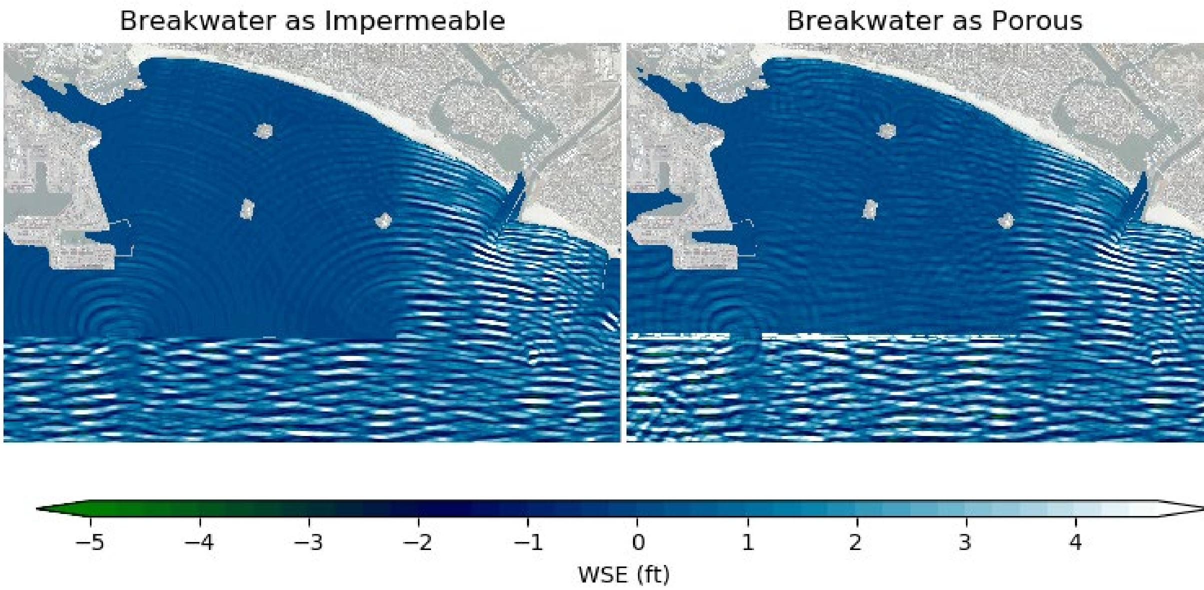


Figure 6-16: Water Surface Elevation from a 1-Year Southern Swell Event with the Current Breakwater Configuration Modeled as Impermeable (left) and with Porosity (right)

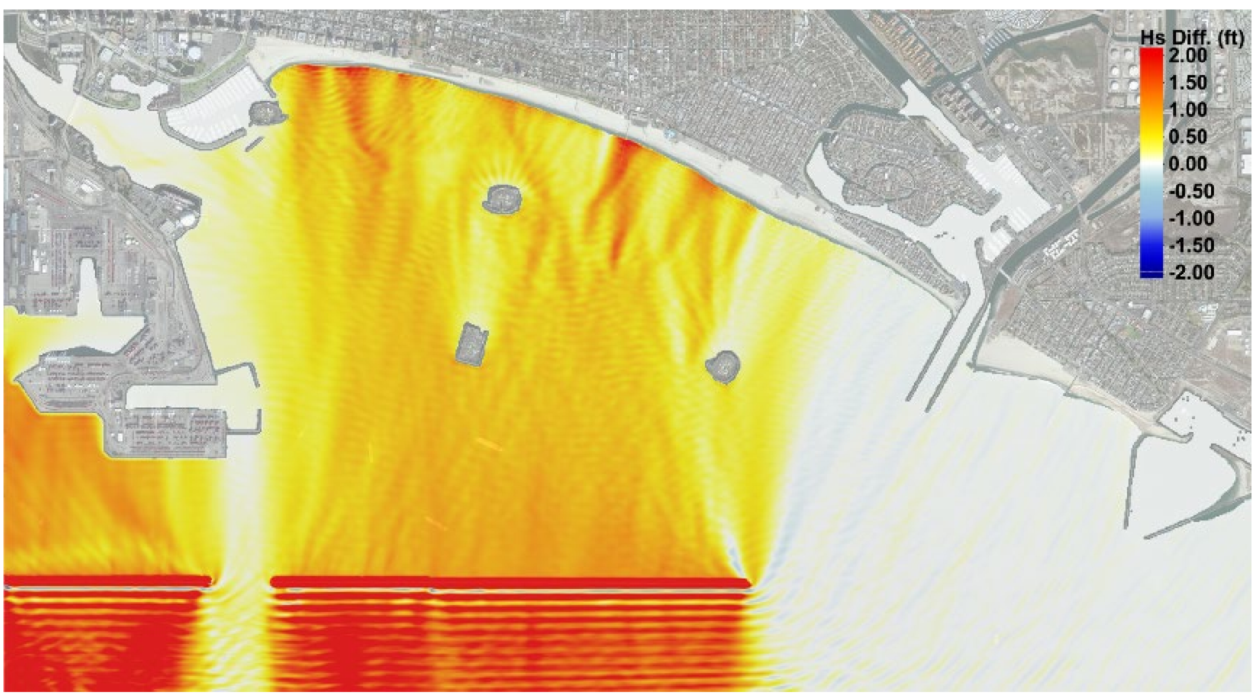


Figure 6-17: Difference in Sig. Wave Height from Breakwaters Modeled as Porous vs. Impermeable

Diffractive patterns can be seen in the water surface elevation plot especially near the tips of the Middle and Long Beach Breakwaters. The wave energy spreads to areas of lower wave height causing a redistribution in the lee of the breakwater. The offshore energy islands cause local sheltering effects on the shoreline and other features within the bay. Due to the proximity to relatively deep water (Energy Island Borrow Pits, > 40 ft.) and the position of White Island, a superposition of wave heights on the lee of the island causes a local increase in wave height. The typical conditions provide a similar wave pattern as the extreme events, only smaller in magnitude. Figure 6-18 shows the wave heights and velocity vectors for the typical summer conditions. The interference pattern near White Island is present in all wave conditions. Large wave heights reported at the breakwater interface are a numerical artifact of the implementation of porosity within the structure.

Figure 6-19 shows the resulting wave heights and wave generated currents from a 1 year southern swell event under the existing breakwater configuration. There are only minor differences when comparing this result to that of the CMS-Wave solution, mainly near structures and directly behind the breakwater. Due to the deeper water of the federal navigation channel as compared to the rest of the model domain, wave heights are generally lower within the channel than in adjacent area. This is a combination of wave shoaling, refraction and diffractive processes. A wave propagates faster in deeper water than in shallow water. As the wave propagates parallel with the navigation channel, refractive processes tend to turn the wave at the edge of the channel producing re-distribution of energy along the crest to equilibrate and spread the energy laterally. This causes a local decrease in wave height within the navigation channel. Wave generated velocities are relatively small, typically in the order of 0.01 knot but is greatly increase within the surf zone with the generation of the longshore current which can be on the order of 1 knot for small stretches.

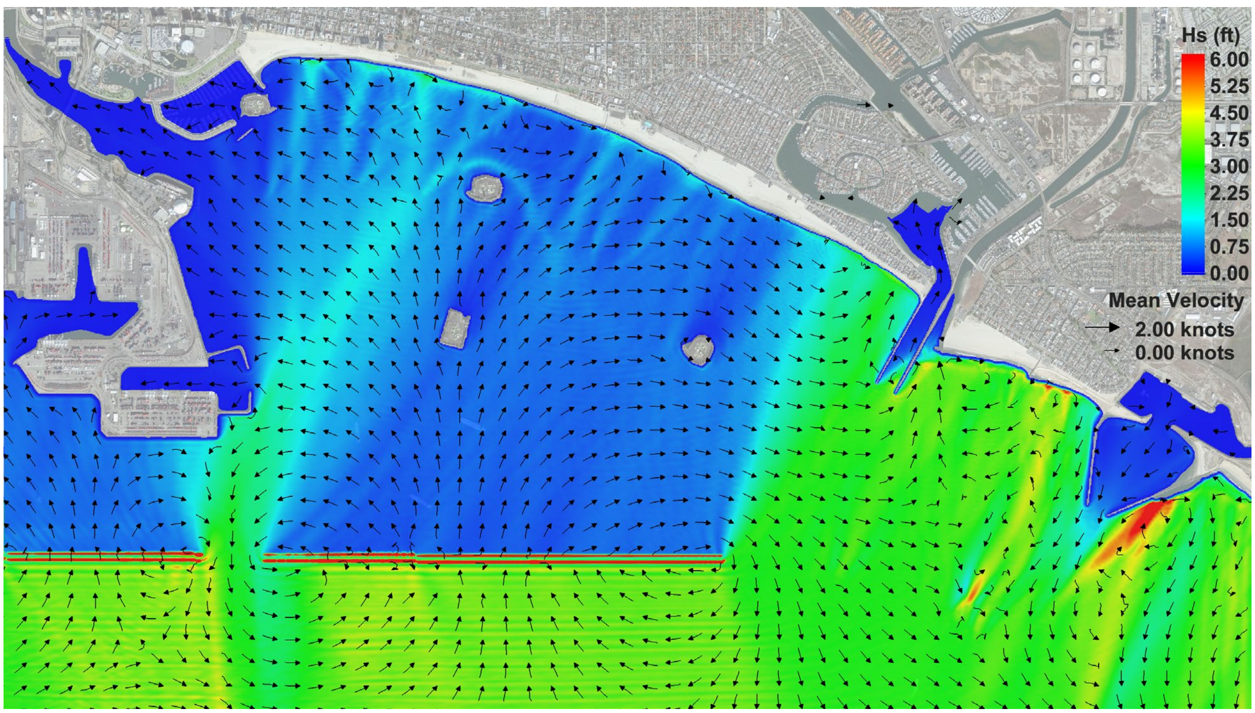


Figure 6-18: Wave Heights and Velocity Vectors for Typical Summer Conditions with Existing Breakwater Configuration

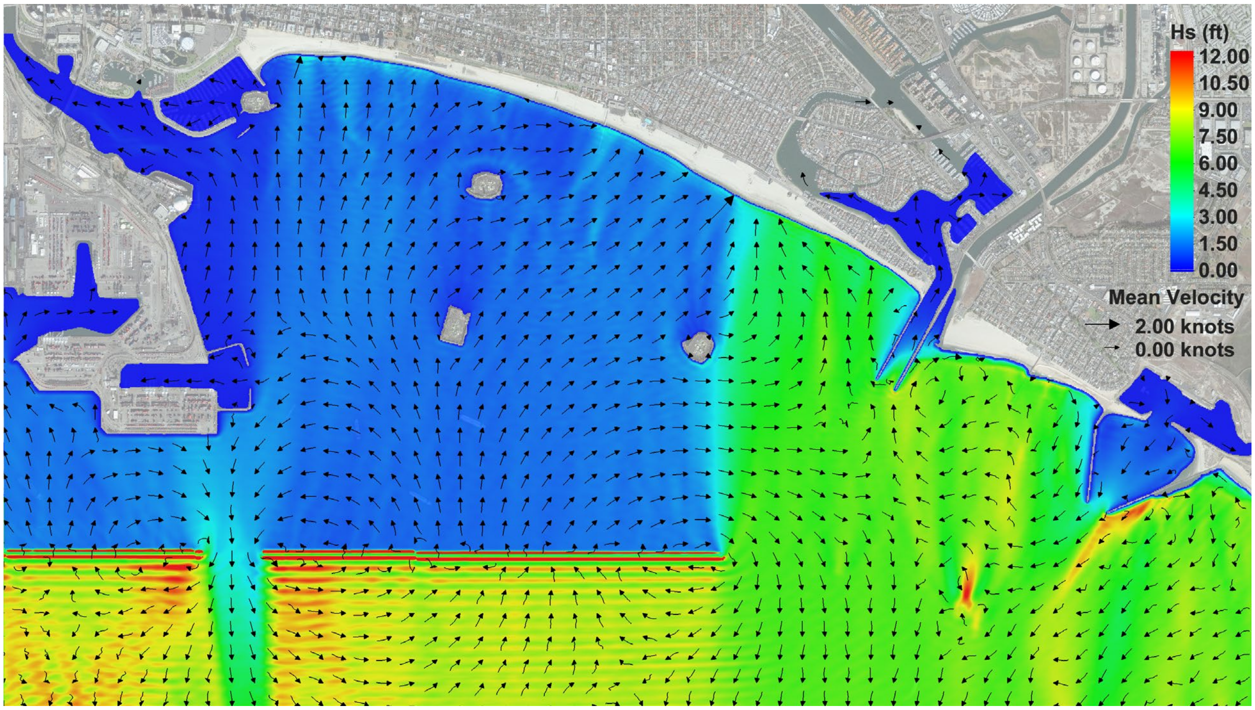


Figure 6-19: Significant Wave Height and Velocity Vectors under Existing Conditions due to a 1-Year Southern Swell Event

6.5 GENCADE

GenCade is a long-term shoreline change model developed by the Coastal Inlets Research Program which calculates shoreline change, wave-induced longshore sediment transport and morphology changes on a local to regional scale. GenCade, and the previous version named Genesis, has been extensively used near the study area; the Surfside/Sunset sand nourishment project has successfully modeled the fate of materials using GenCade (Gravens, 1990).

6.5.1 MODEL SETUP AND EXISTING CONDITIONS

From previous studies (USACE, 1997), the annual net longshore transport along the Long Beach shoreline is approx. 49,000 yd^3/yr. to the west. To calibrate GenCade, LiDAR surveys conducted in 2009 and 2014 are used as the initial and final shoreline positions for the calibration phase. To simulate the effect of the backpassing that the city currently employs, an average of ~72,000 yd^3 (55,000 m^3) is backpassed from the locations in Figure 6-19. Through cross-shore and other losses near (and through) Alamitos Bay Jetties, ~5,000 yd^3 (3,600 m^3) of sediment is removed from the system.

The reference shoreline and regional contour are chosen as the Mean Higher High Water (MHHW) line and the 6.5 ft depth, respectively. Gated lateral boundary conditions are used at both the western and eastern boundaries. A fixed seawall at non-erodible locations is employed; such as sidewalks, bluff faces, buildings, etc.

Wave condition are applied at the 26.25 ft. (8 m.) contour line and transformed to shore using the internal transformation model. A total of 13 wave input locations are utilized to capture the diffractive effects of the breakwater and offshore energy islands. This allows for the modifications of the Long Beach Breakwater to be easily implemented into this shoreline change model. To create the artificial wave time series, the actual record at the offshore wave buoy is extracted, binned using the same joint probability analysis, then transformed to the proposed project area. The binned data is then mapped back to the recorded time series to create the artificial timeseries at the GenCade input locations.

Based on an analysis of the Long Beach shoreline, the average berm height is found to be on the order of 10 feet (3 m.) and the depth of closure, already discussed, is determined to be ~21.5 ft. (6.5 m). Although these values are greater near Alamitos bay due to the larger waves, for simplicity, the same berm height and depth of closure will be used throughout the domain and for all conditions.

After model calibration using the 2009 and 2014 shoreline positions, the K1 and K2 constants are determined to be 0.11 and 0.1 respectively which produces an RMS error of ~22.0 ft. (6.7 m). The longshore transport rate near Peninsula Beach is near the observed average of 49,000 yd³/yr. but varies dependent on the wave conditions. All GenCade parameters can be found in Table 6-7 and a listing of critical features along the shoreline in Table 6-6. Sensitivity of this shoreline change model can be seen by varying the input parameters and comparing the results against the previously calibrated model.

Table 6-8 shows the change of shoreline position due to the sensitivity of the input parameters; the largest change is due to the increase in the K1 value.

Table 6-6: Critical features along shoreline

Location	Cells	
Peninsula Beach	1	44
Belmont Pier	106	
Junipero Beach parking lot	156	172

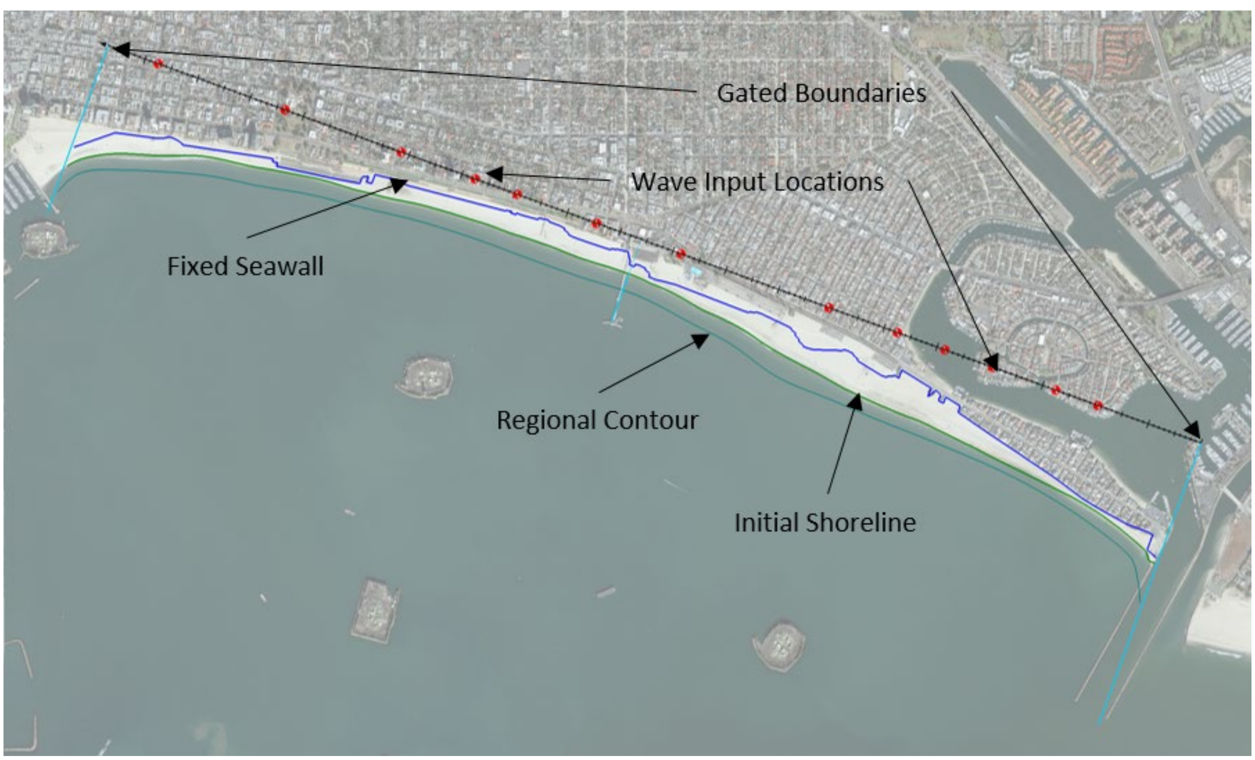


Figure 6-20: GenCade Initial Setup

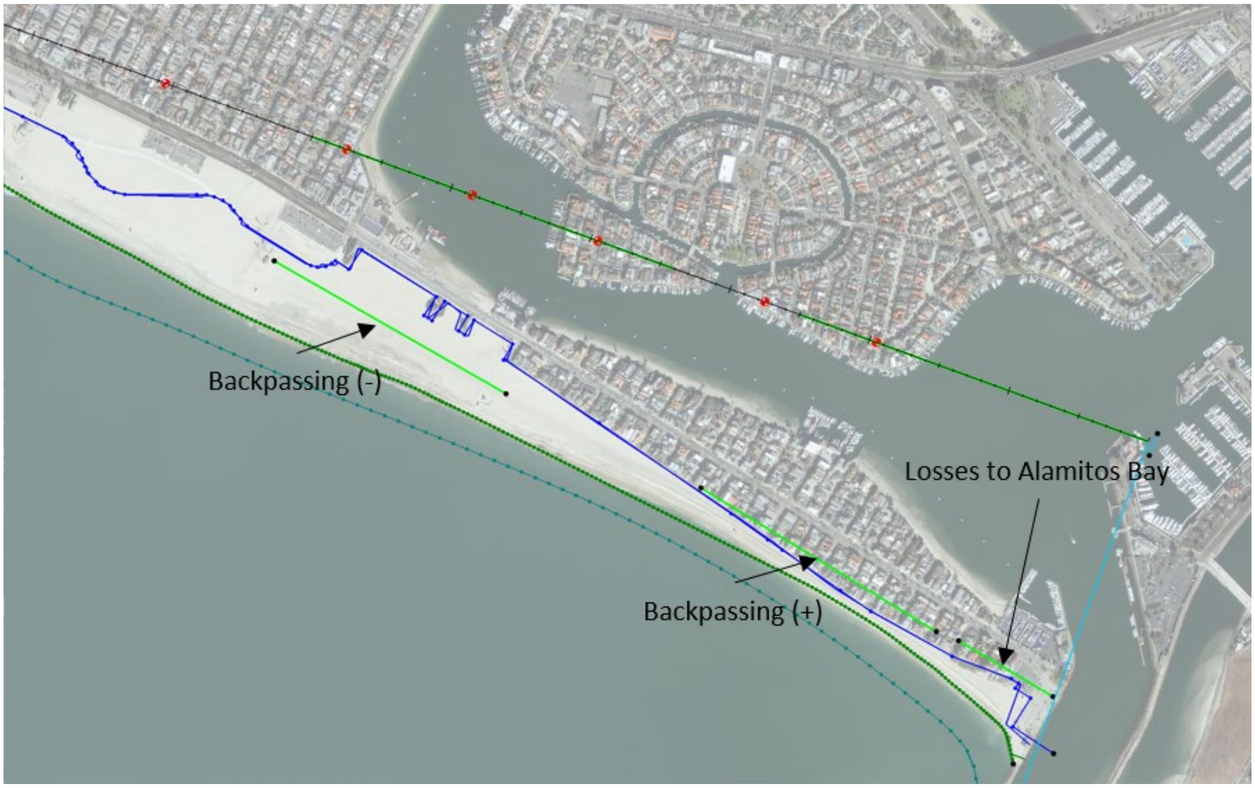


Figure 6-21: GenCade Backpassing Operations

Figure 6-22: GenCade Cell Numbering and Locations from Baseline

Table 6-7: GenCade Input Parameters
(note: units in metric)

Parameter	Value
Total Length	6210 m
Calibration Duration	10/1/2009 - 9/19/2014
Model Duration	10/1/2030 - 9/30/2040
K1	0.11
K2	0.10
# Wave Inputs	13
# of Cells	207
Cell Size	30 m
Time Step	30 min
Berm Height	3 m
Depth of Closure	6.5 m
D_{50}	0.2 mm

Table 6-8: Sensitivity of GenCade

Parameter	Calibrated Value	Sensitivity Range	RMS Error (m)
K1	0.11	0.05	14.58
		0.2	15.34
		0.5	31.50
K2	0.1	0.05	1.96
		0.2	4.54
		0.5	15.54
Berm Height	3	2.5	1.22
		3.5	1.13
Depth of Closure	6.5	5.5	2.56
		7.5	2.17
D50	0.2	0.15	2.09
		0.25	1.05

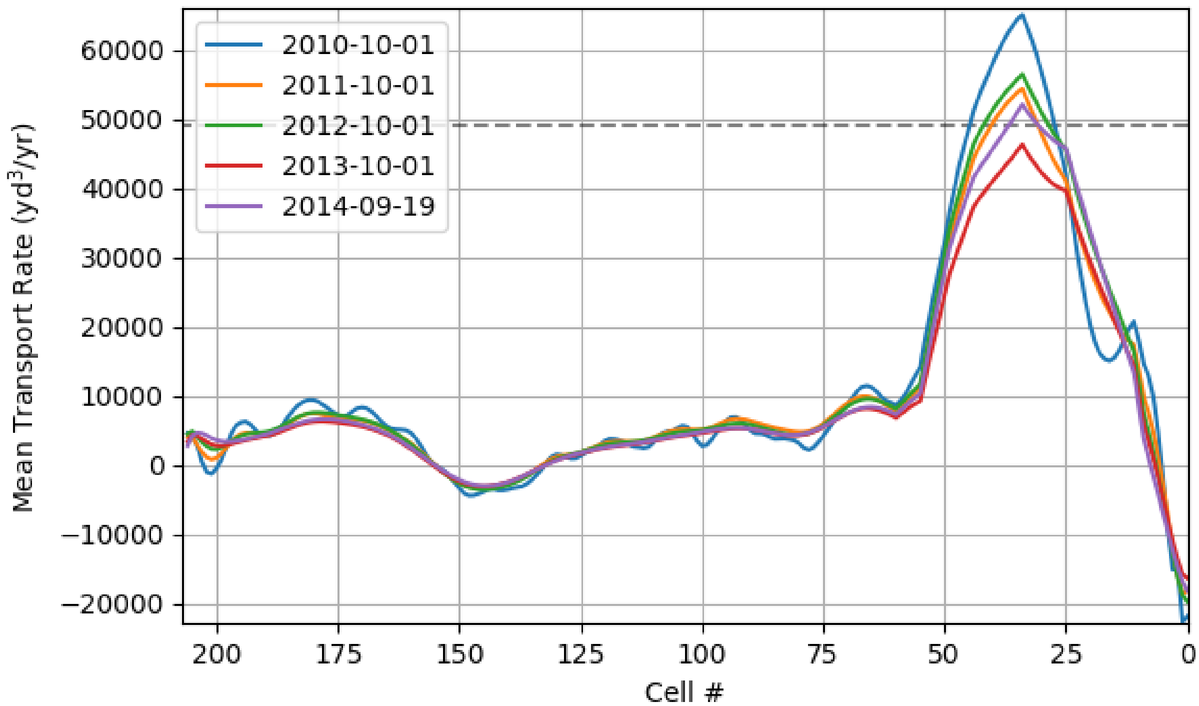


Figure 6-23: Mean Yearly Sediment Transport Rate from GenCade Calibration

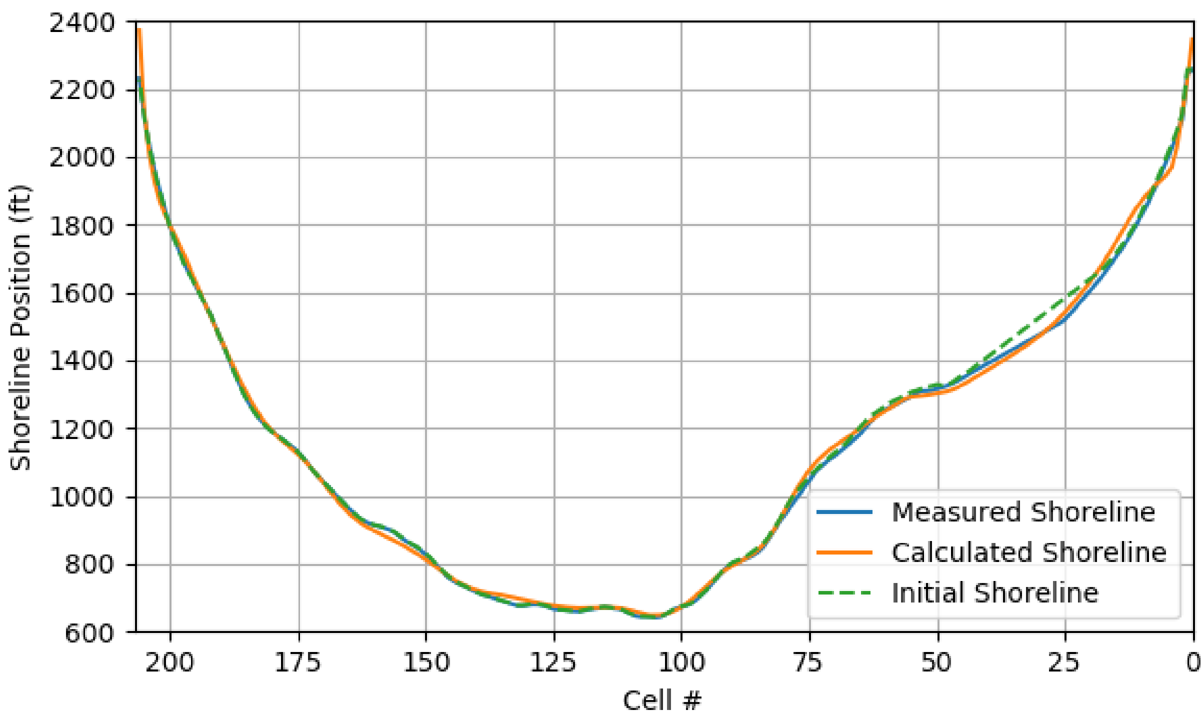


Figure 6-24: Final Shoreline Position from Baseline for GenCade Calibration

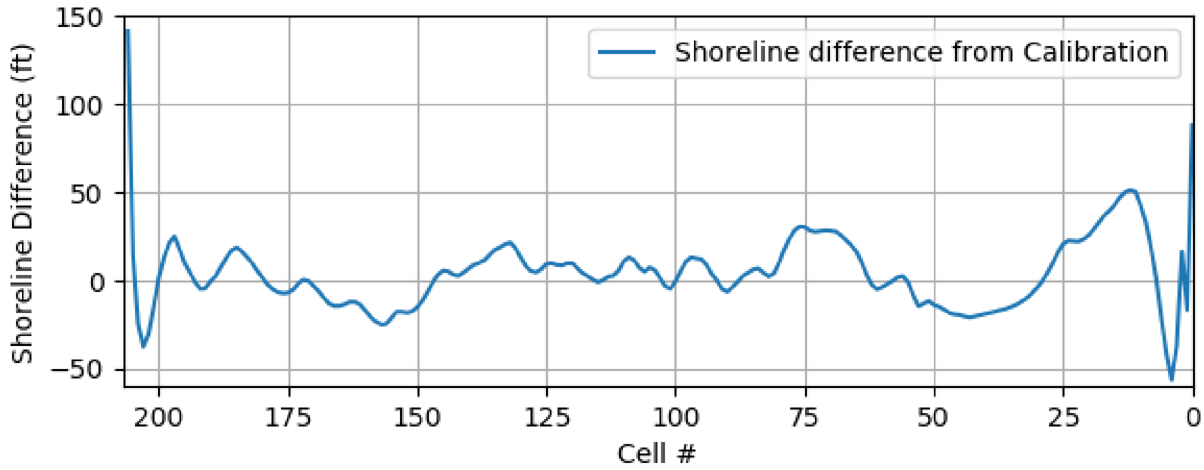


Figure 6-25: Shoreline Difference from Calibration Run (Measured vs. Calculated)

7 IMPACTS TO COASTAL PROCESSES

7.1 FLUID VELOCITIES

The nearshore rocky reefs do little to alter the velocities within the bay; a local decrease in velocity is present near the reefs. Additionally, the wetland complexes shelter the measure's projected area from incoming waves. A local increase of velocity is present at the Los Angeles River outflow but the total discharge is not altered. Figure 7-1 show the difference in fluid velocities from the existing conditions with the nearshore reefs and wetland complexes. For further discussion of these modeling results, see APPENDIX A-1, EFDC MODELING. Only a select combination of breakwater modifications and other restoration measures are included in the hydrodynamic modeling.

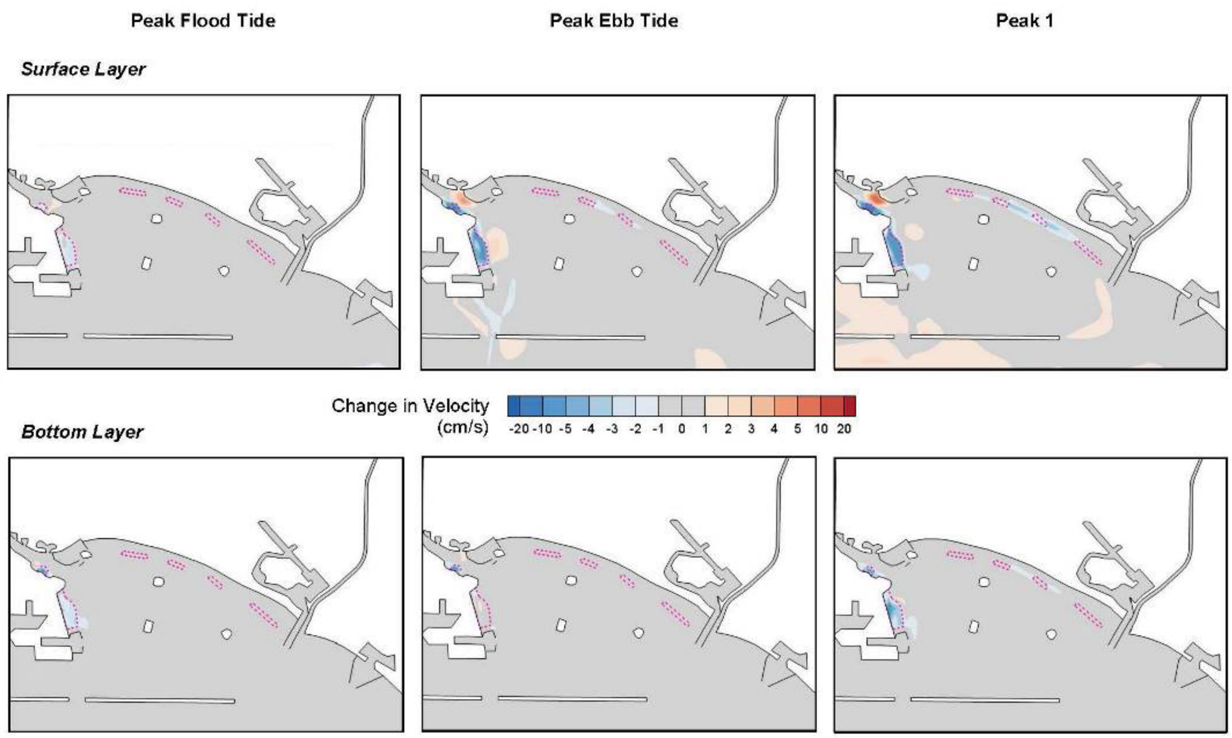


Figure 7-1: Changes to Fluid Velocities from Nearshore Reefs and Wetland Complexes

7.1.1 REMOVAL OF EASTERN 1/3

Figure 7-2 shows the velocity difference from existing condition and the eastern 1/3 removal (EastLop) of the Long Beach Breakwater. Local velocity magnitudes increase near the structural modifications, but adjacent areas are developed with a corresponding decrease in velocity magnitude. These changes are most prevalent during times with large flow magnitudes such as the ebb and flood tides or during wet weather flow events. Overall, the tidal prism is not altered resulting from a structural modification of the Long Beach Breakwater; the volume of water that enters or exits the bay remains the same. Corresponding currents are also not significantly altered as an effect of breakwater modification. For further information on currents and flow paths, see the PARTICLE TRACKING AND TRANSPORT section.

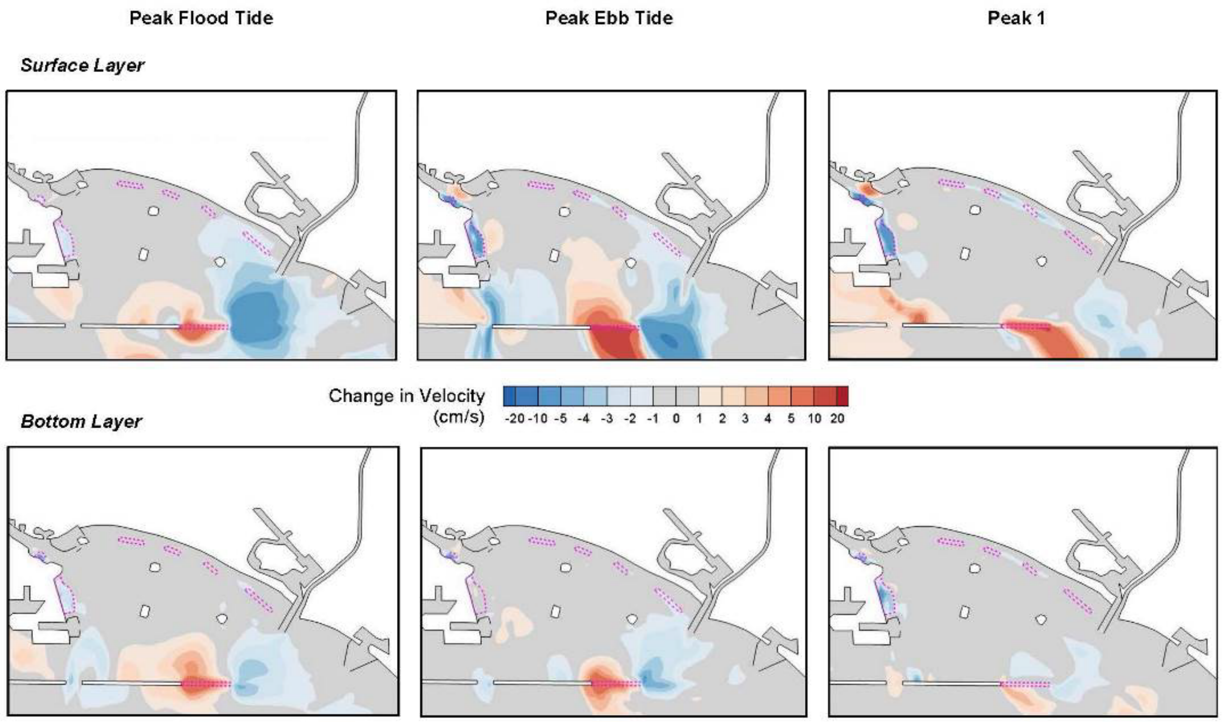


Figure 7-2: Changes to Fluid Velocities due to Removal of Eastern Third
Note: units in metric.

7.1.2 2 X 1000 FT. NOTCHES ON THE EASTERN SIDE

Results corresponding to notching on the eastern side of the breakwater is shown in Figure 7-3. Velocities are altered locally, but do not impact the bay as a whole.

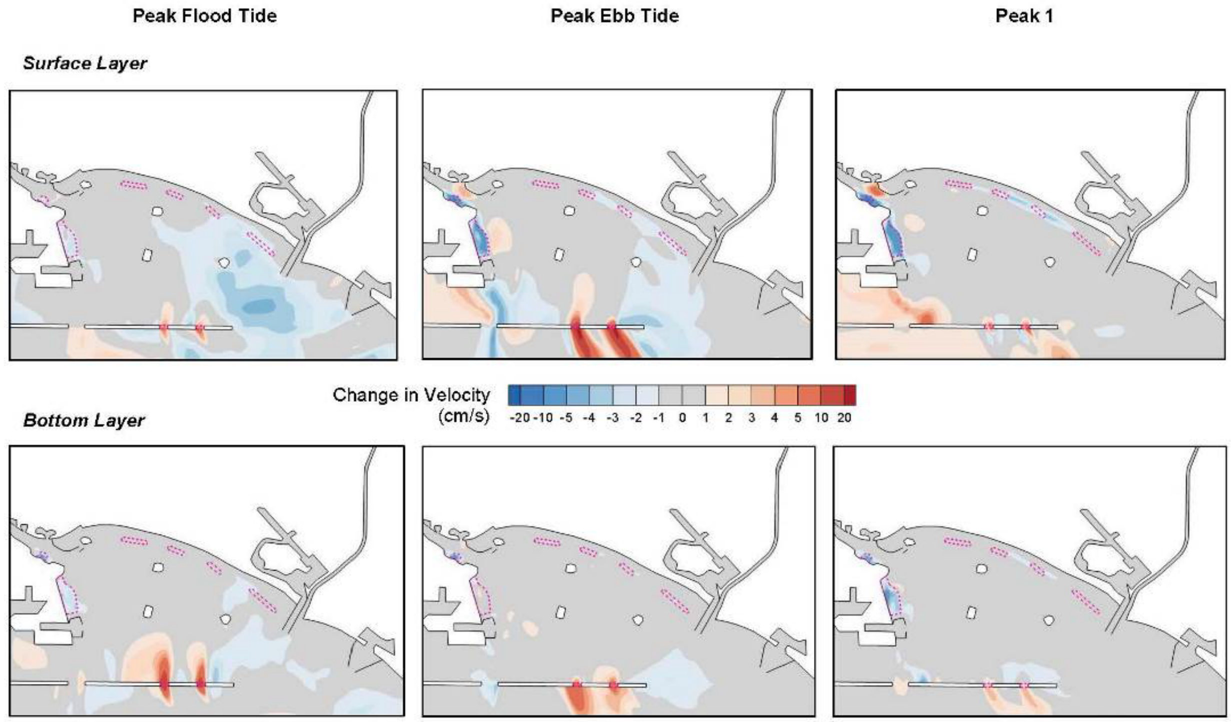


Figure 7-3: Changes to Fluid Velocities due to Two 1000 ft. Notches on Eastern Side
Note: units in metric.

7.1.3 <u>LOWERING OF ENTIRE STRUCTURE TO -30 FT. MLLW</u>

Lowering of the entire breakwater has a much more noticeable effect than the previous modifications, as shown in Figure 7-4. No other measures are included in this scenario and the full effects of the breakwater can be seen. Surface currents are more impacted than the bottom currents as magnitude changes extend further into the bay.

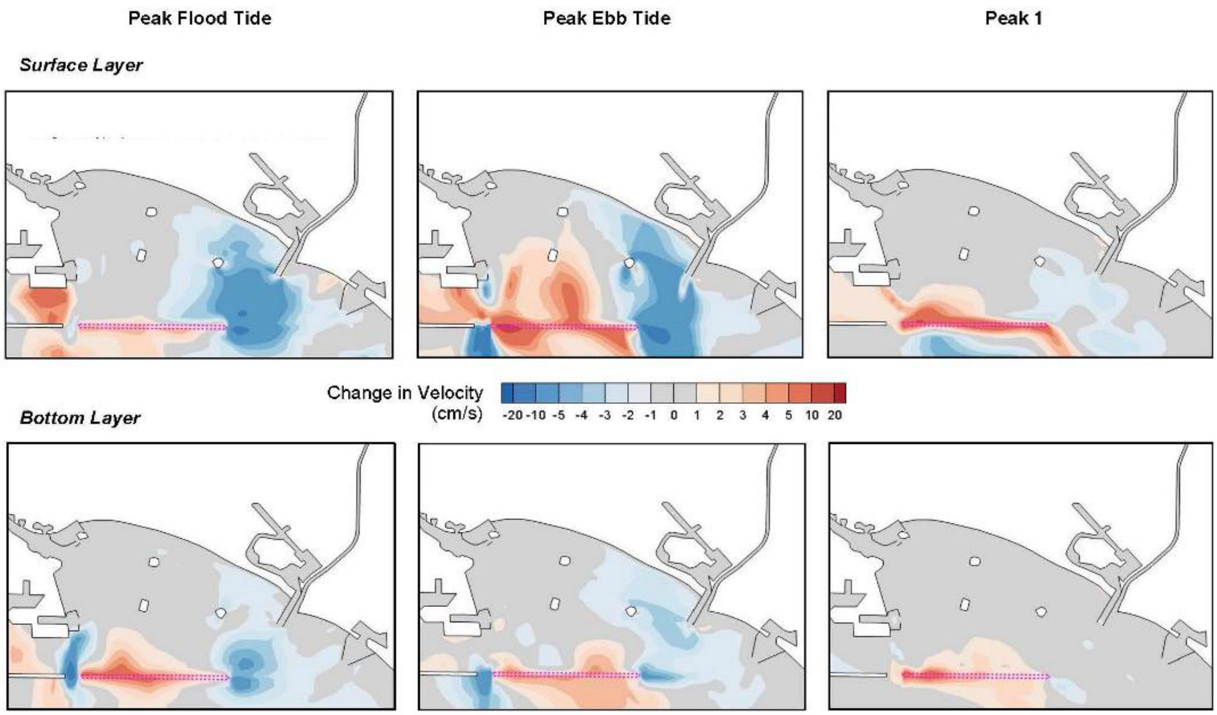


Figure 7-4: Changes to Fluid Velocities due to Lowering of the Entire Breakwater
Note: units in metric.

7.1.4 LOS ANGELES TRAINING WALL

The inclusion of the training wall, shown in Figure 7-5, only alters velocities significantly during wet weather events and is limited to the upper layers. The discharge from the Los Angeles River is deflected by the structure causing local velocities to increase and extend further into the bay. Water surface elevations during times of peak flow from the Los Angeles River produces a rise of fractions of an inch as compared to the existing condition. Therefore, even with the combination of the training wall and the tidal wetland that constricts the discharge flow, there will be little to no effect on the river's drainage and potential upstream flooding from these measures.

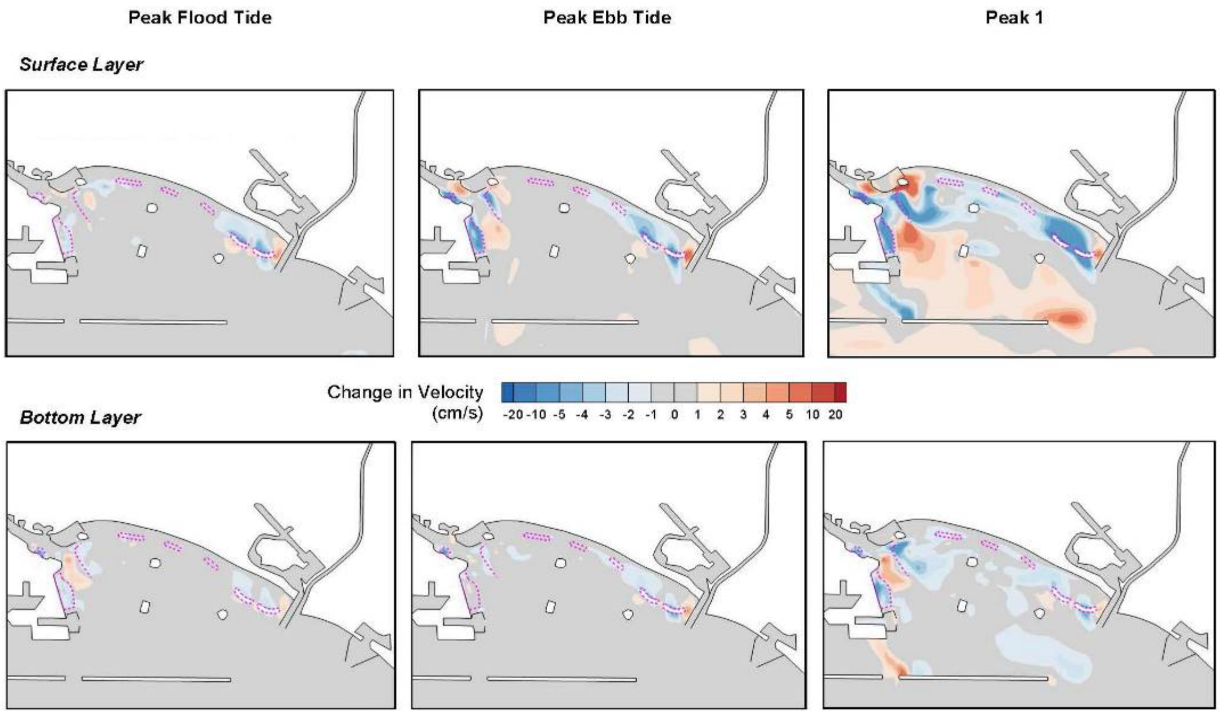


Figure 7-5: Changes to Fluid Velocities due to Los Angeles River Training Wall
Note: units in metric.

7.2 NEARSHORE WAVES

The nearshore wave environment may be altered by three structural measures under consideration; the nearshore rocky reefs, emergent islands and the breakwater modifications. Other measures will not directly or significantly impact the nearshore wave environment and will be neglected for this analysis.

7.2.1 BREAKWATER MODIFICATIONS

Modifications to the breakwater allow for larger waves within the bay. Changes in wave conditions are shown in the following sections. For output of all the modeled wave conditions, see APPENDIX A-2, WAVE CONDITIONS. Specific locations are extracted to show the effect of the potential breakwater modifications. A summary of conditions due to the breakwater modifications is shown in Table 7-1 and Table 7-2 for a 50 year event with wave origins from the northwest and south respectively. These tables show the change of wave heights as a percentage of the existing condition near locations shown in Figure 7-6.

Table 7-1: Percent Increase from Existing Conditions for a 50-Year Northwest Swell Event

	Removal of Eastern 1/3	Removal of Western 1/3	Eastern Notches	Western Notches	Single Western Notch	Single Center Notch	Complete Lowering
Pier J Approach Channel	0%	16%	0%	1%	1%	0%	17%
Pier J South	0%	22%	1%	4%	4%	1%	24%
Freeman Island	1%	84%	5%	25%	18%	8%	106%
Chaffee Island	111%	28%	46%	11%	3%	12%	173%
White Island	9%	36%	11%	14%	7%	8%	73%
Carnival Cruise Lines	3%	62%	7%	25%	20%	8%	80%
Shoreline Marina	4%	54%	5%	20%	16%	6%	73%
Bixby	1%	50%	1%	16%	12%	3%	60%
Belmont Pier	26%	67%	27%	34%	14%	22%	154%
Belmont Shore	38%	60%	30%	27%	12%	20%	140%
East Beach	50%	4%	14%	2%	0%	2%	57%

Table 7-2: Percent Increase from Existing Conditions for a 50-Year South Swell Event

	Removal of Eastern 1/3	Removal of Western 1/3	Eastern Notches	Western Notches	Single Western Notch	Single Center Notch	Complete Lowering
Pier J Approach Channel	-2%	49%	-1%	4%	4%	-1%	50%
Pier J South	-1%	54%	0%	9%	8%	1%	58%
Freeman Island	5%	77%	13%	35%	20%	17%	124%
Chaffee Island	103%	7%	30%	4%	1%	3%	123%
White Island	32%	42%	22%	18%	6%	10%	104%
Carnival Cruise Lines	6%	73%	9%	28%	21%	11%	103%
Shoreline Marina	9%	72%	8%	27%	9%	21%	104%
Bixby	1%	73%	2%	20%	16%	4%	88%
Belmont Pier	62%	47%	40%	31%	11%	22%	147%
Belmont Shore	58%	34%	33%	19%	8%	14%	101%
East Beach	16%	1%	4%	0%	0%	0%	18%

Figure 7-6: Wave Output Locations for Comparison

7.2.1.1 REMOVAL OF EASTERN 1/3

The removal of the eastern third (4500 ft.) of the Long Beach Breakwater allow for increases in wave heights shown in Figure 7-7 and Figure 7-8 for the 1-year southern and northern swell events. Only differences greater than 4 in. (0.1 m) is presented in the succeeding figures. Generally, local wave height increases are present where the breakwater no longer provides protection. Aside from the local direct effects, diffractive processes produce larger waves in the western section of the bay; although generally lower than a 1 ft. increase. Design wave heights for the offshore energy islands increase from 11 ft. to 14.5 ft. for the 100-year southern swell event.

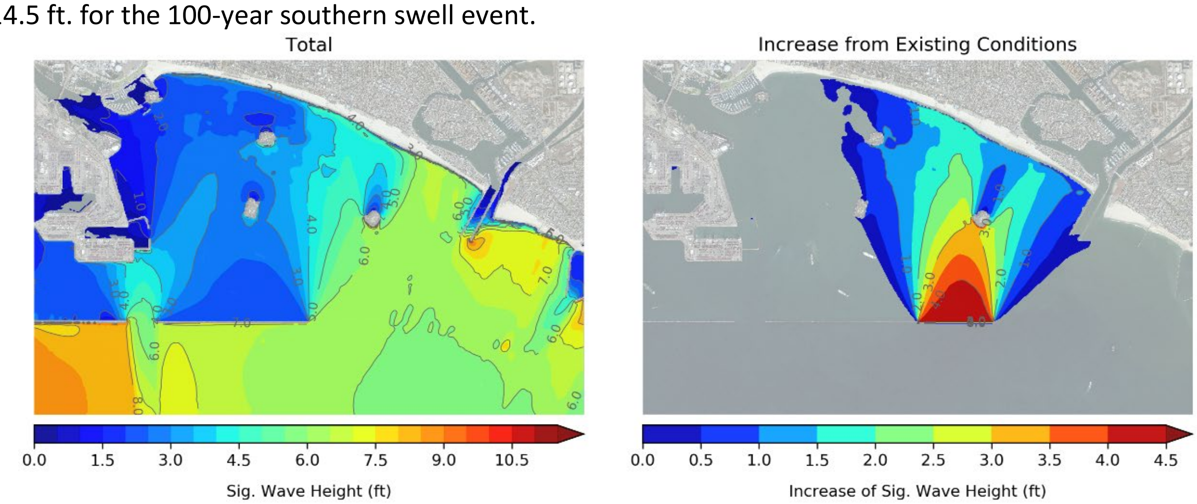


Figure 7-7: Wave Heights Resulting from Eastern 1/3 Breakwater Removal for a 1-Year Southern Swell Event

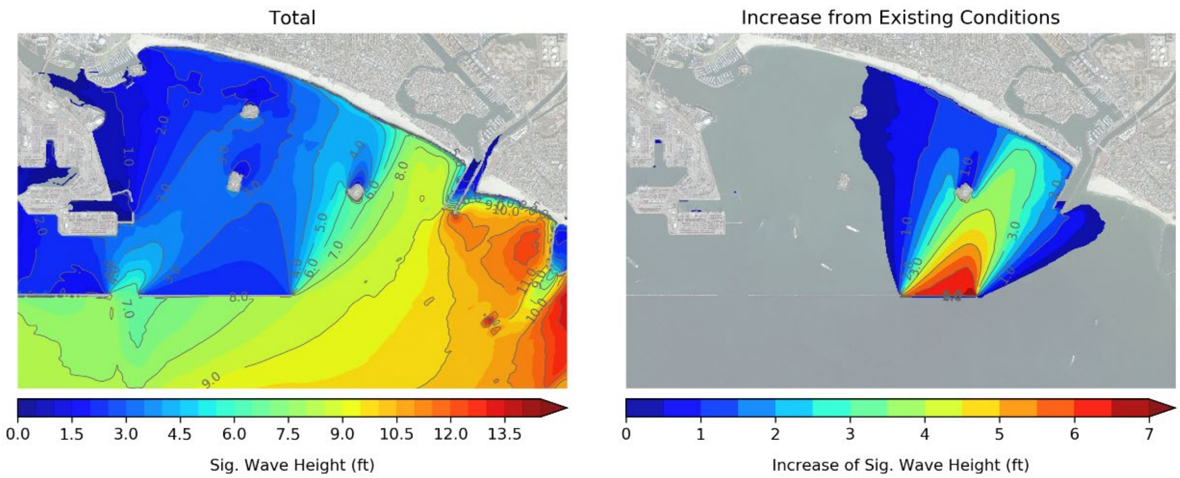


Figure 7-8: Wave Heights Resulting from Eastern 1/3 Breakwater Removal for a 1-Year Northwestern Swell Event

Protection measures for the existing infrastructure would include reinforcing the existing revetments on the energy islands by increasing the armor stone size and raising the structure height to reduce the expected increase in wave run-up and overtopping. Nearshore reefs will be used to assist in breaking the waves during large events before reaching the shoreline, thus reducing the magnitude of potential sediment transport and shoreline recession.

The probability of specific wave height is shown in Figure 7-9. Due to the modification, larger waves are present within the project area at higher probabilities but only occur on the eastern side of the project area; increases of energy are contained to the areas east of Freeman and White Islands.

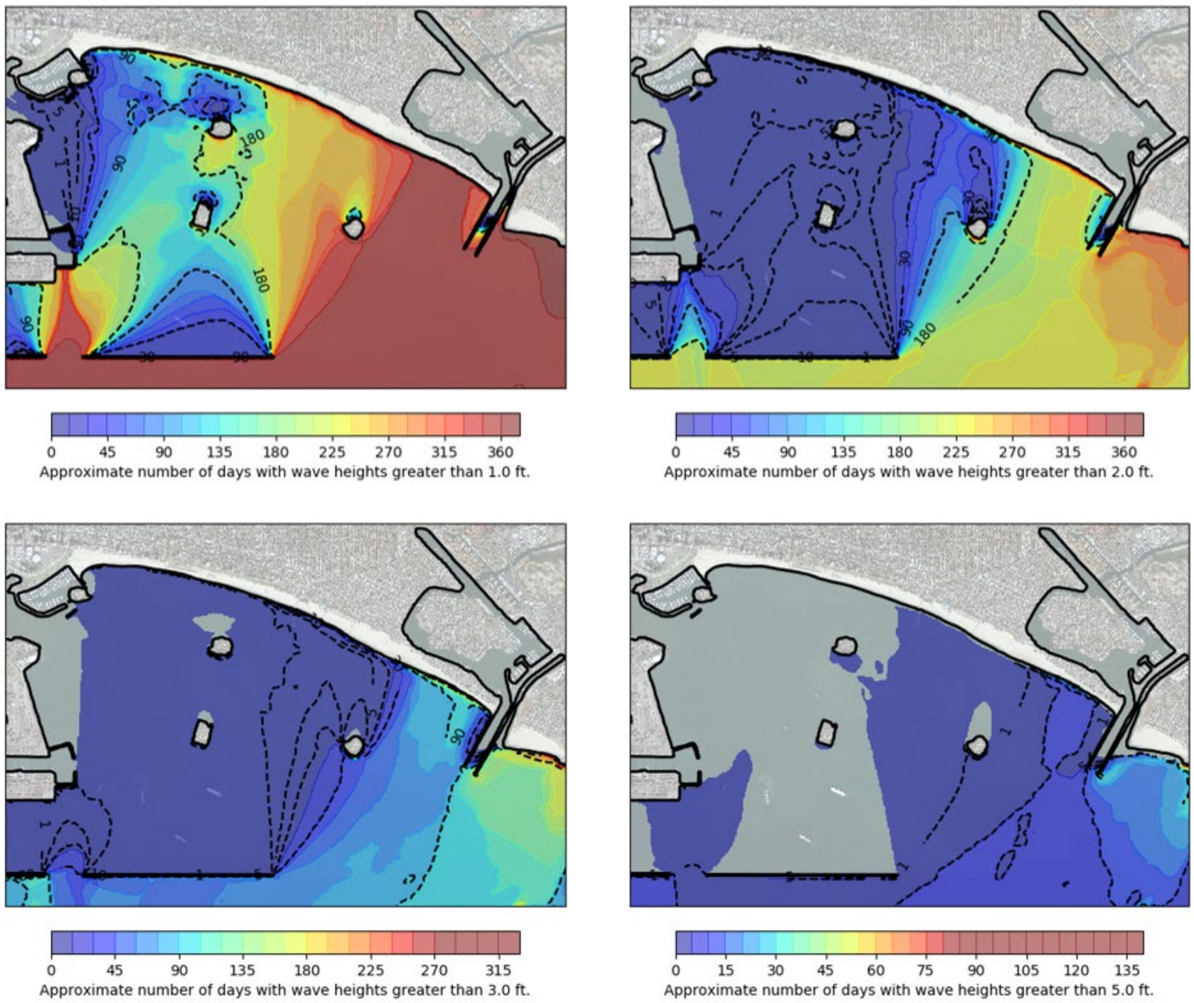


Figure 7-9: Probability of Occurrence due to Eastern Breakwater Removal

7.2.1.2 2 X 1000 FT. NOTCHES ON THE EASTERN SIDE

The removal of two 1000 ft. notches of the Long Beach Breakwater has a similar effect on the coastal processes as the removal of the third of the structure, only less drastic. Increased waves now enter through the newly created notches in the breakwater, as seen in Figure 7-10 and Figure 7-11 for the 1-year south and northwest swell events. These waves diffract in a similar manner as on the east side of the breakwater; only now, wave spreading occurs in multiple directions which greatly reduces the transmitted wave heights and spreads the wave energy throughout the bay. Local effects on the offshore energy islands and shoreline will produce increased waves, run-up and overtopping in these areas. Additional protection measures will be required to provide a wave environment similar to the current conditions and to armor existing infrastructure to withstand the more extreme condition.

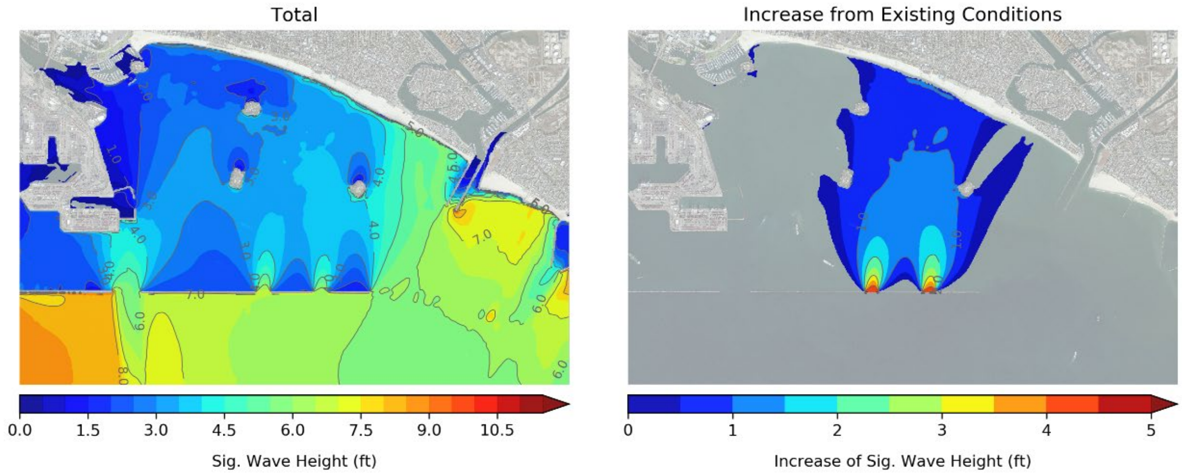


Figure 7-10: Wave Heights Resulting from 2x 1000 ft. Notches in Breakwater on Eastern Side for a 1-Year Southern Swell Event

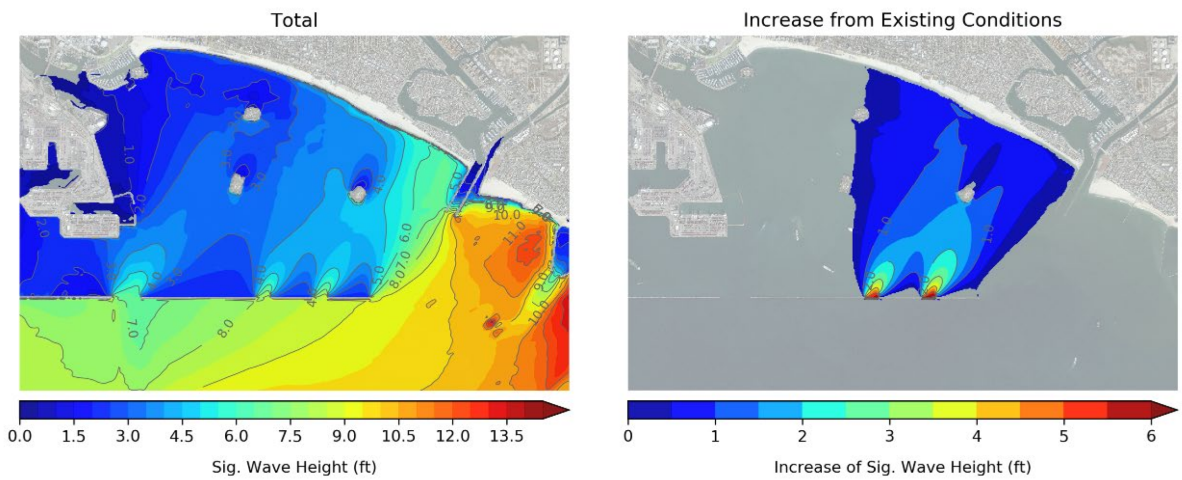


Figure 7-11: Wave Heights Resulting from 2x 1000 ft. Notches in Breakwater on Eastern Side for a 1-Year Northwestern Swell Event

The probability of a specific wave height is presented in Figure 7-12. Like the removal of the eastern third, the notches in the eastern side cause wave heights to increase on the eastern end of the project area. Increases in wave heights occur further west since the opening is shifted to the west in relation to the previous modification.

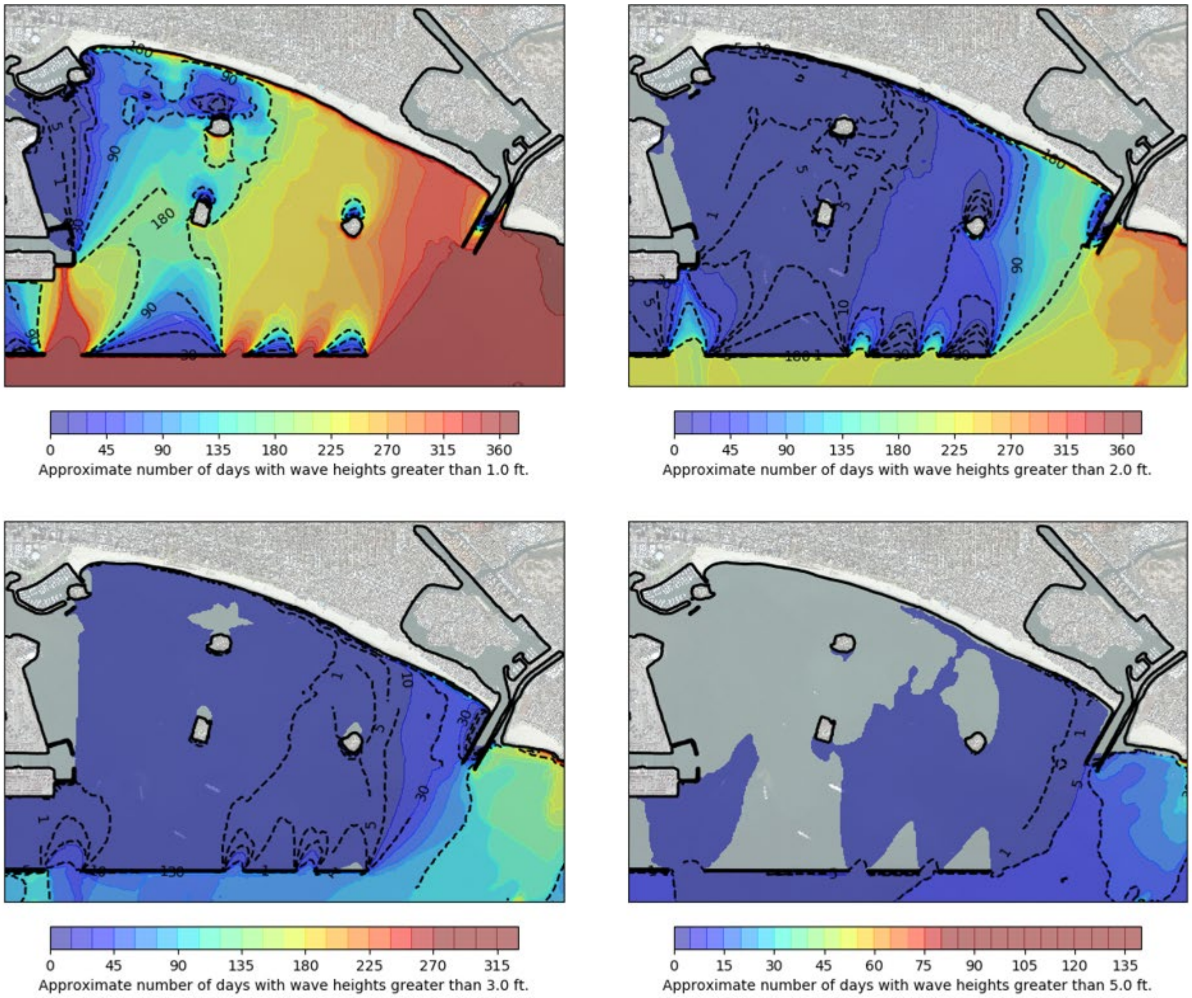


Figure 7-12: Probability of Occurrence due to Eastern Breakwater Notches

7.2.1.3 REMOVAL OF WESTERN 1/3

The removal of the western third (4,500 ft.) of the breakwater will cause similar wave height increases as the removal on the eastern side. Waves now propagate through the larger gap at Queen's Gate and greatly impact the Port of Long Beach, as seen in Figure 7-13 for a southern swell and Figure 7-14 for a northwestern swell. The Pier J jetties provide some protection from the increased waves, but diffractive effects produce waves within the Pier J South basin. This is particularly hazardous during a southern swell event, since port operators already experience both short and long wave problems with the existing breakwater configuration. Increased incident waves on the offshore and nearshore energy islands, Port of Long Beach revetment, Shoreline Marina and the shoreline itself will be required to reinforce the existing infrastructure. The increase of risk to port operations as well as the large extent of protective will violate the constraint to not impact port and navy operations. The increase of wave heights in the shipping channels will reduce the efficiency and effectiveness; vessels will have to transit slowly to reduce the risk of running aground.

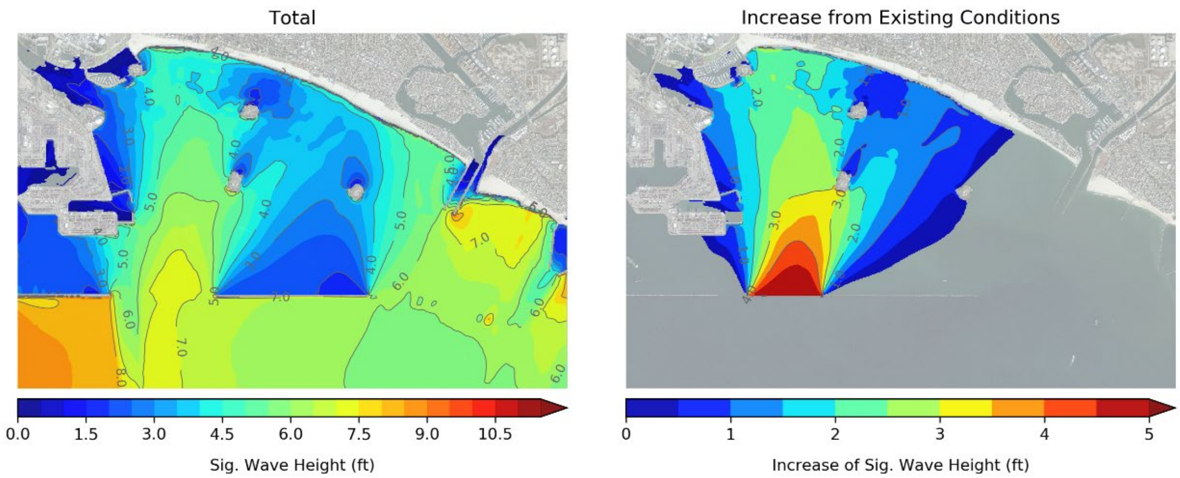


Figure 7-13: Wave Heights Resulting from Western 1/3 Breakwater Removal for a 1-Year Southern Swell Event

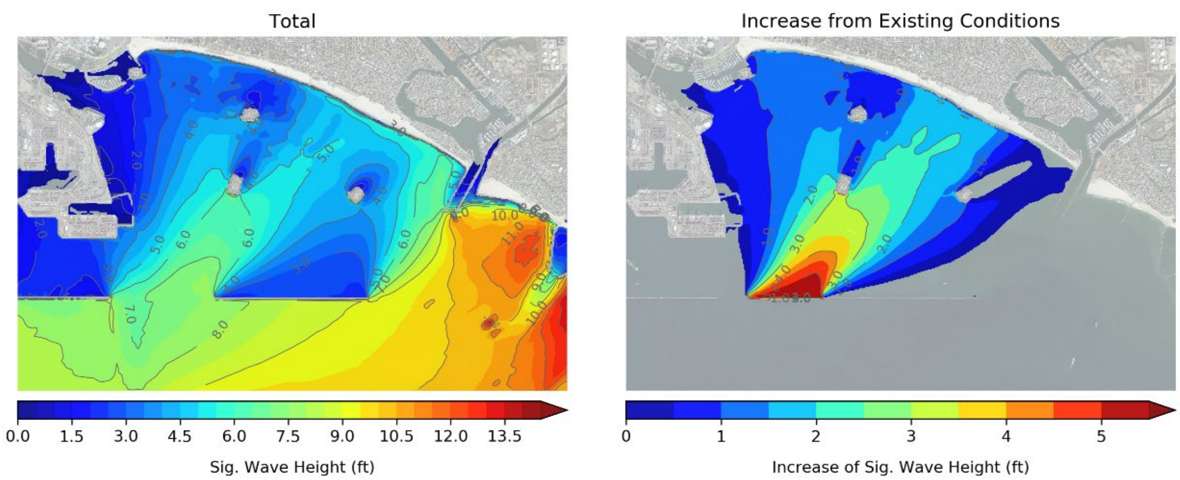


Figure 7-14: Wave Heights Resulting from Western 1/3 Breakwater Removal for a 1-Year Northwestern Swell Event

The changes to the probability of occurrence of a specific wave height due to the western third removal is shown in Figure 7-15.

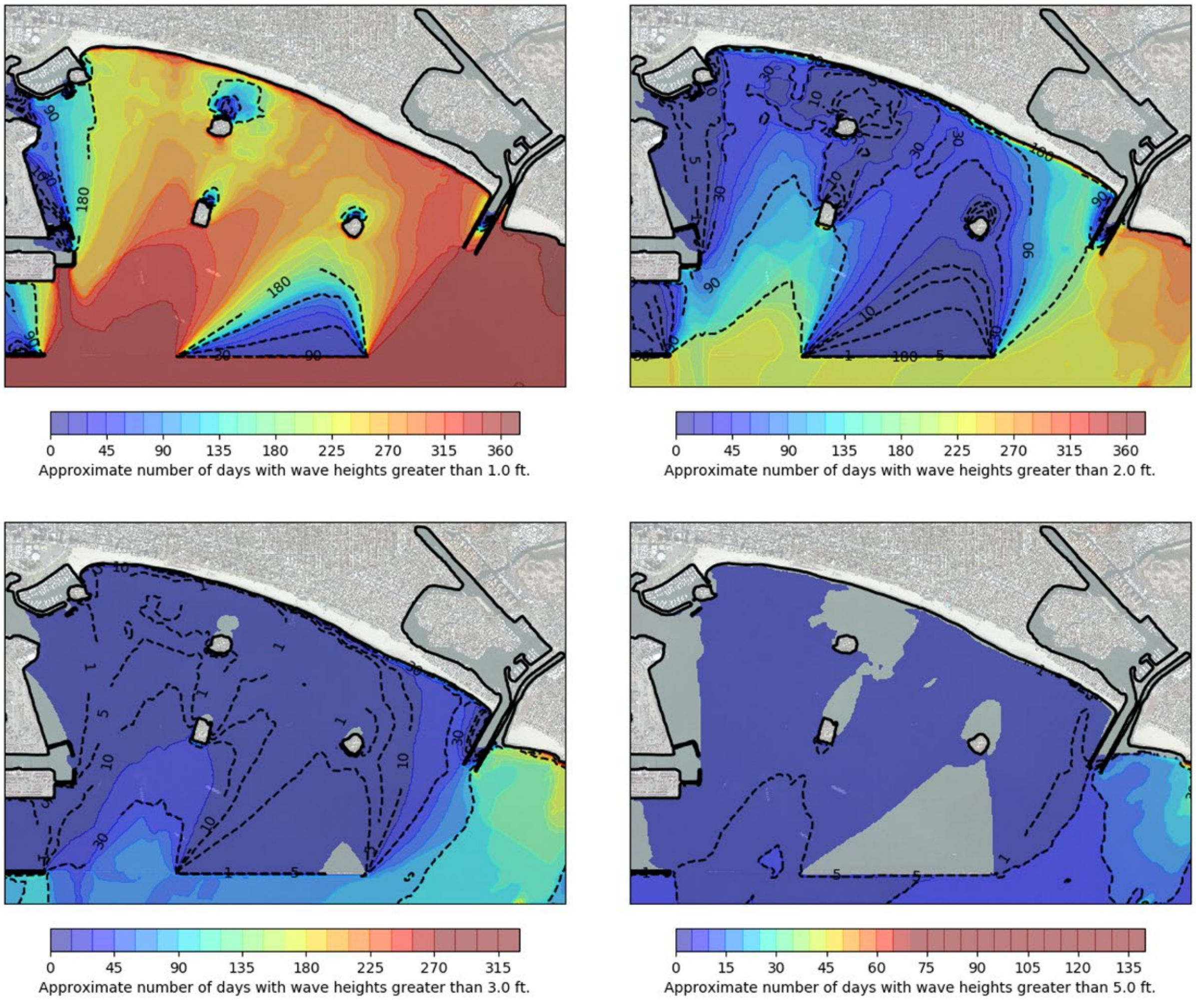


Figure 7-15: Probability of Occurrence due to Western Breakwater Removal

7.2.1.4 2 X 1000 FT. NOTCHES ON WESTERN SIDE

Unlike the removal of 4,500 ft. of breakwater, two 100 ft. notches in the western side of the structure provides slightly more protection to existing infrastructure. Figure 7-16 and Figure 7-17 present the wave conditions during a 1-year southern and northwestern swell event, respectively. Waves enter the bay through the notches, but diffractive processes cause the energy to spread and the wave height decreases soon after. Increase in the design wave heights of the offshore and nearshore energy islands and other port infrastructure are required, so the inclusion of various protective measures will also be required.

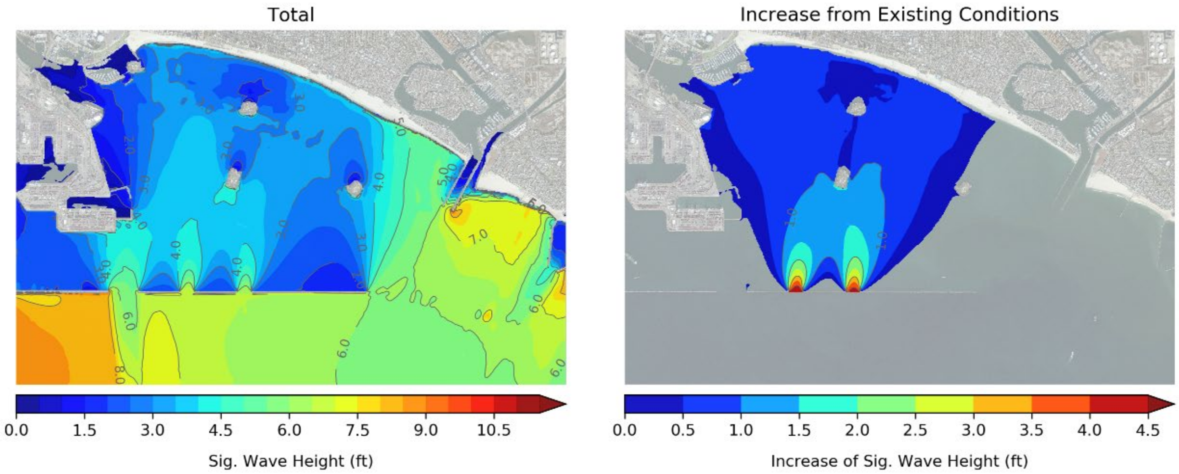


Figure 7-16: Wave Heights Resulting from 2x 1000 ft. Notches in Breakwater on Western Side for a 1-Year Southern Swell Event

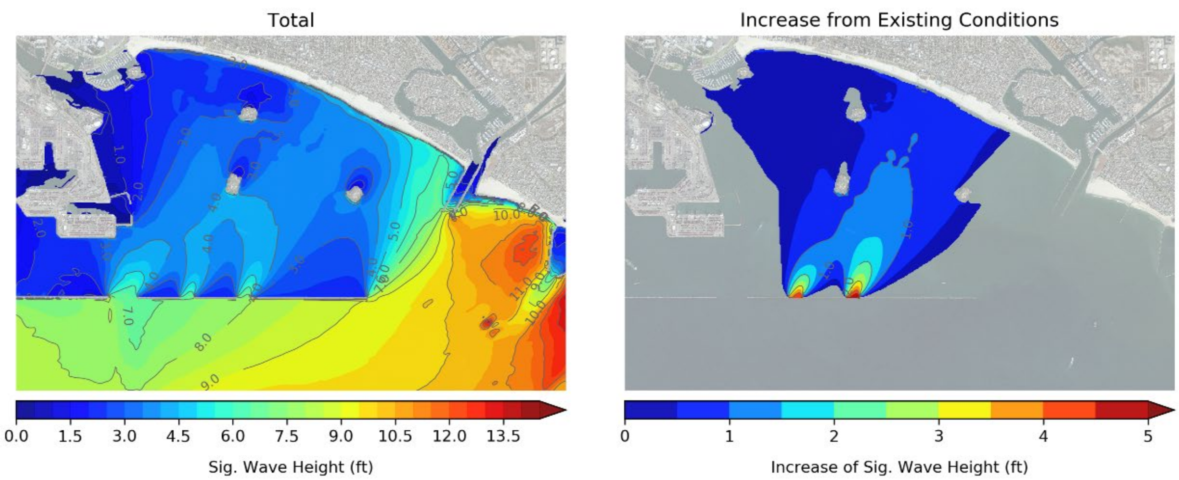


Figure 7-17: Wave Heights Resulting from 2x 1000 ft. Notches in Breakwater on Western Side for a 1-Year Northwestern Swell Event

The wave height probability of occurrence relating to the notching of the western side of the breakwater is shown in Figure 7-18. This modification allows for larger waves within the western side of the project area. Due to the western or north-western predominate swell direction, this modification also allows the larger waves to impact the eastern side of the project area.

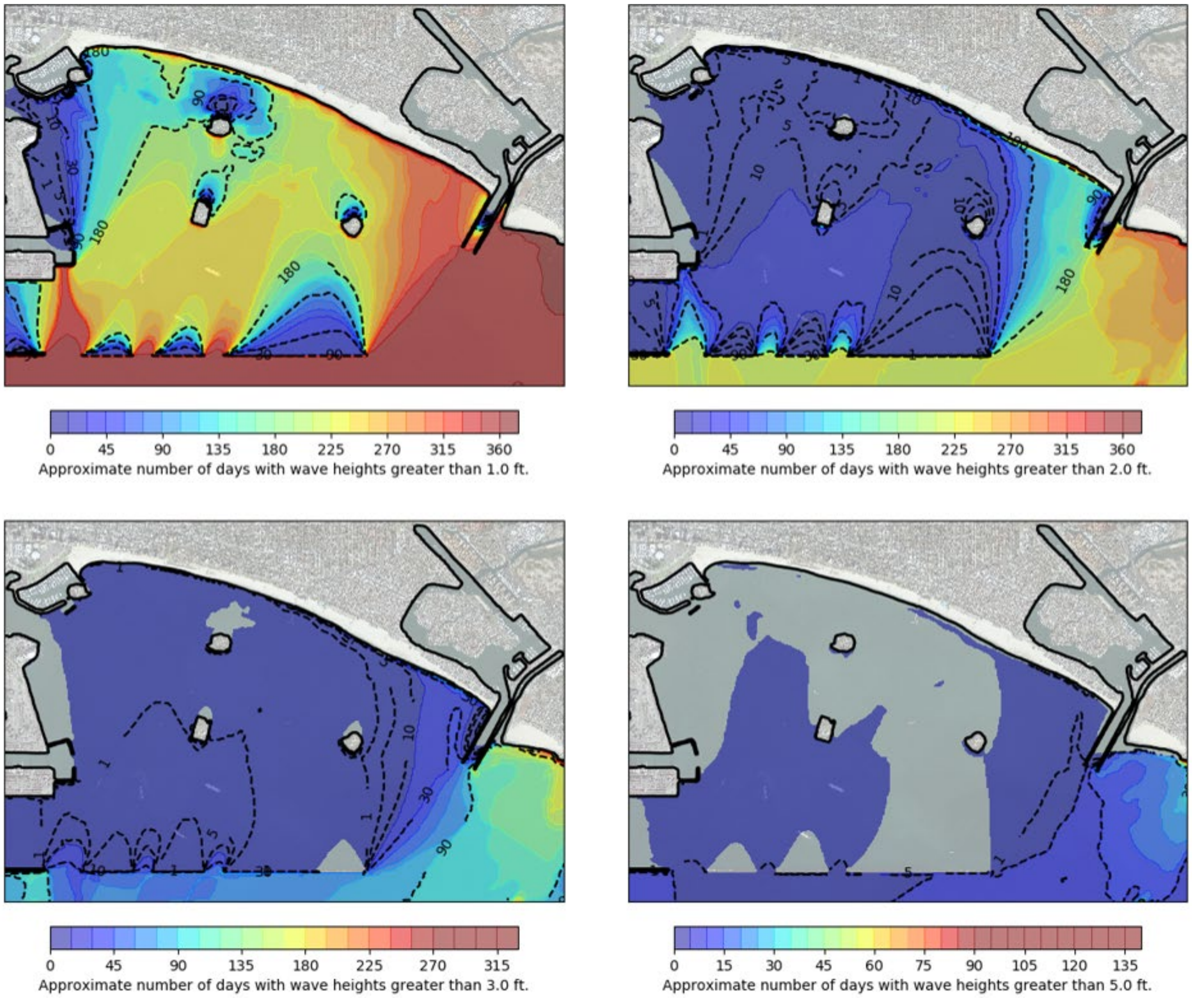


Figure 7-18: Probability of Occurrence due to Western Breakwater Notches

7.2.1.5 1 X 1000 FT. NOTCH ON WESTERN SIDE

A single 1000 ft. notch allows for a minimal amount of energy into the system and is shown in Figure 7-19 and Figure 7-20 for the 1 year swell events. Increase are generally limited to not more than 1 ft. even in the extreme 1 year event due to the relatively small opening and corresponding diffractive processes.

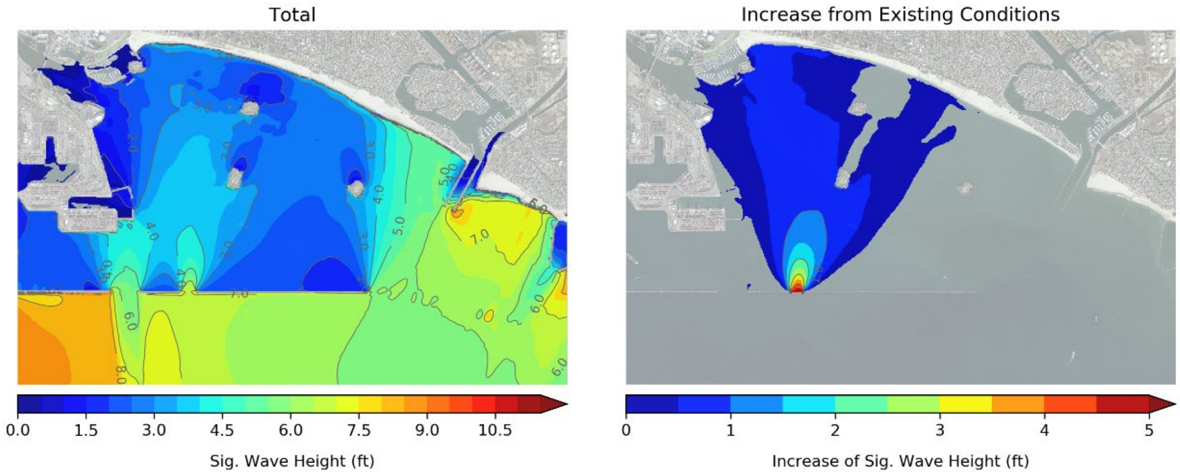


Figure 7-19: Wave Heights Resulting from Single 1000 ft. Notch in Western Side of the Breakwater for a 1-Year Southern Swell Event

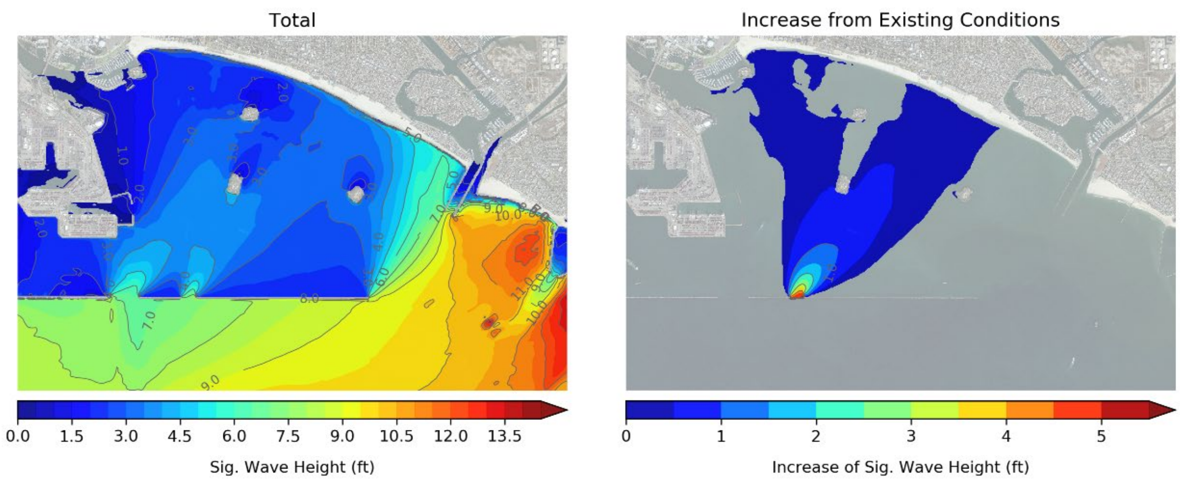


Figure 7-20: Wave Heights Resulting from Single 1000 ft. Notch in Western Side of the Breakwater for a 1-Year Northwestern Swell Event

7.2.1.6 1 X 1000 FT. NOTCH IN CENTER

A single notch in the center of the structure is shown in Figure 7-21 and Figure 7-22. A minimal amount of energy is transmitted through the newly created gap. Diffractive process quickly spread the energy and the wave heights are generally less than a 0.5 ft. increase from existing conditions, even in extreme events.

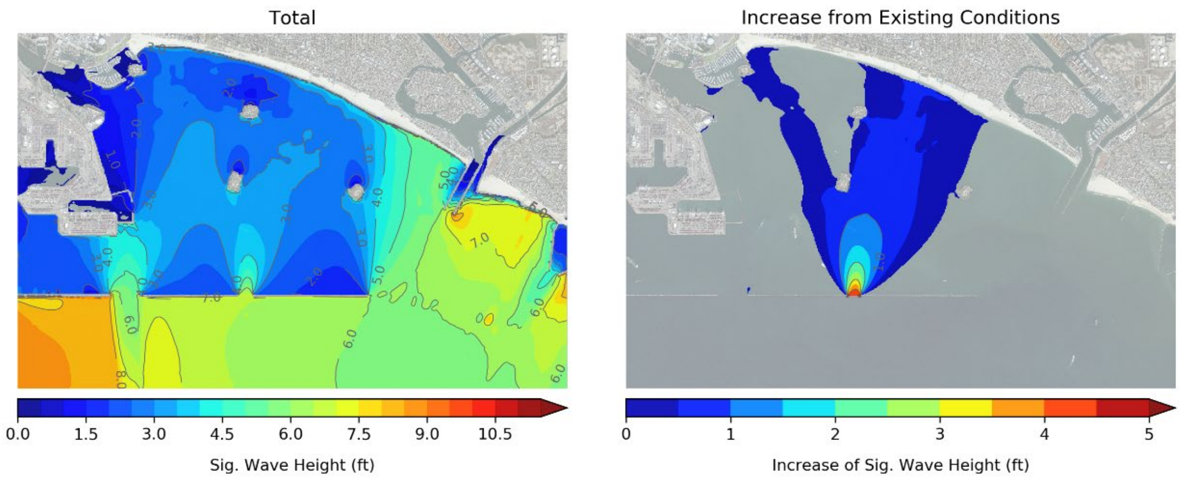


Figure 7-21: Wave Heights Resulting from Single 1000 ft. Notch in Center of Breakwater for a 1-Year Southern Swell Event

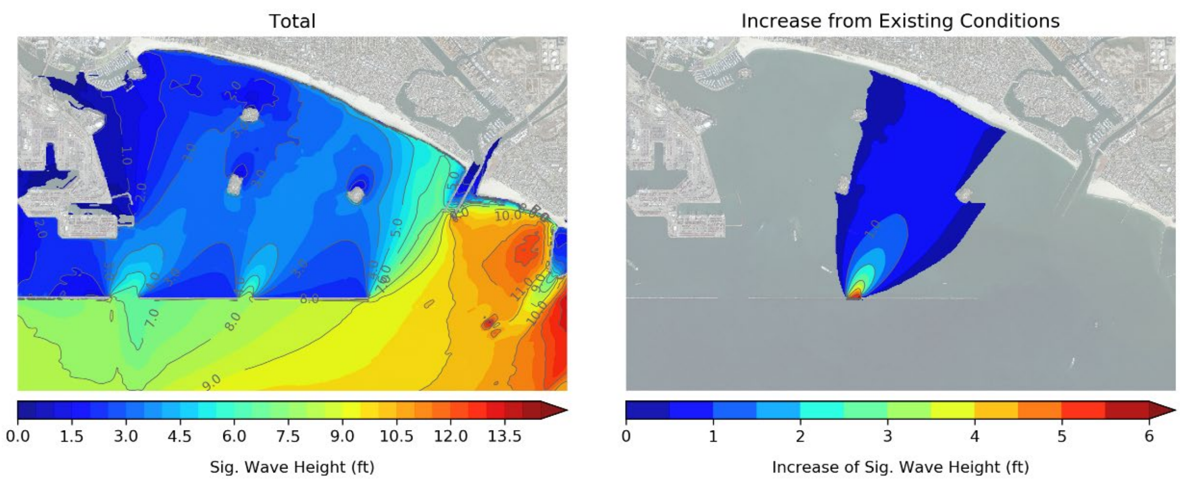


Figure 7-22: Wave Heights Resulting from Single 1000 ft. Notch in Center of Breakwater for a 1-Year Northwestern Swell Event

7.2.1.7 LOWERING OF ENTIRE STRUCTURE TO -30 FT. MLLW

Lowering of the entire breakwater has a drastic effect on all areas within the proposed project area. In most areas, wave heights more than double in size. The lowering does not modify the incident waves; most of the energy is transferred past the breakwater; only a small percentage of the incident energy is reflected by the remaining small seabed mound. Figure 7-23 and Figure 7-24 present the results for a 1-yr southern and northwestern swell events, respectively.

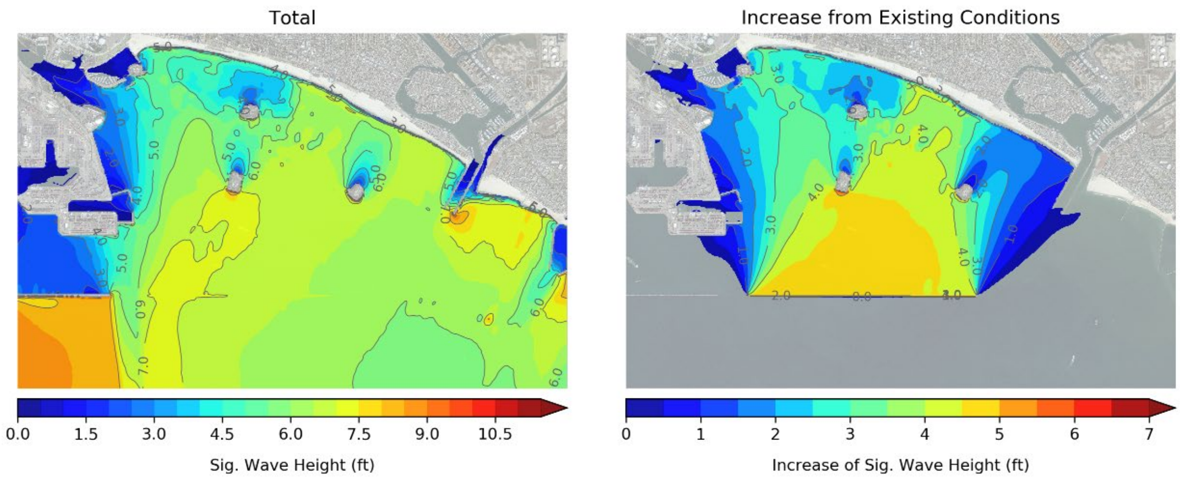


Figure 7-23: Wave Heights Resulting from Complete Breakwater Lowering to -30 ft. for a 1-Year Southern Swell Event

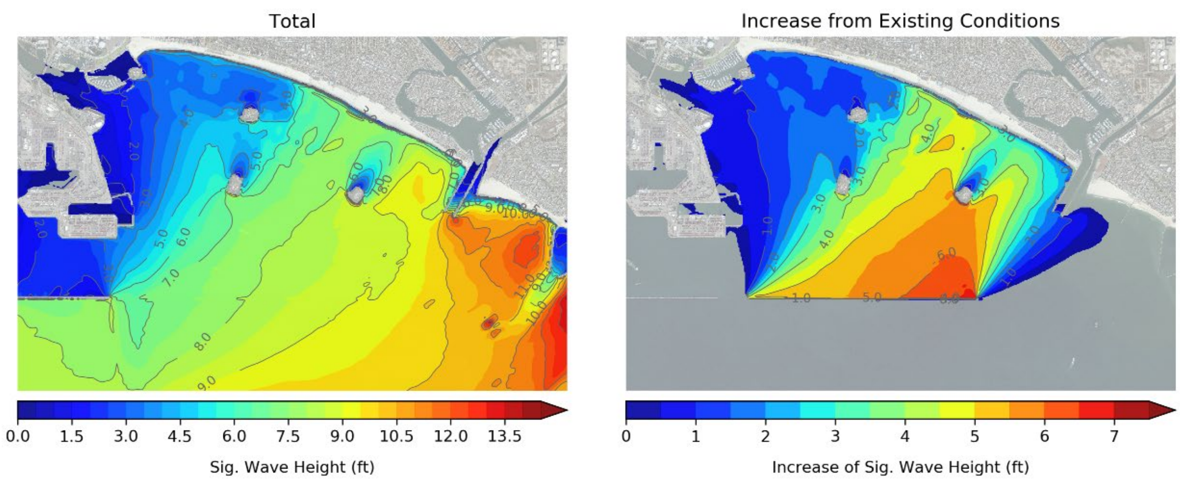


Figure 7-24: Wave Heights Resulting from Complete Breakwater Lowering to -30 ft. for a 1-Year Northwestern Swell Event

The probability of a specific wave height is shown in Figure 7-25 corresponding to lowering of the entire breakwater. Lowering of the structure allows for the largest waves into the project area. Although the breakwater is removed, the port complex provides moderate sheltering to the shoreline during the west or north-west swell events.

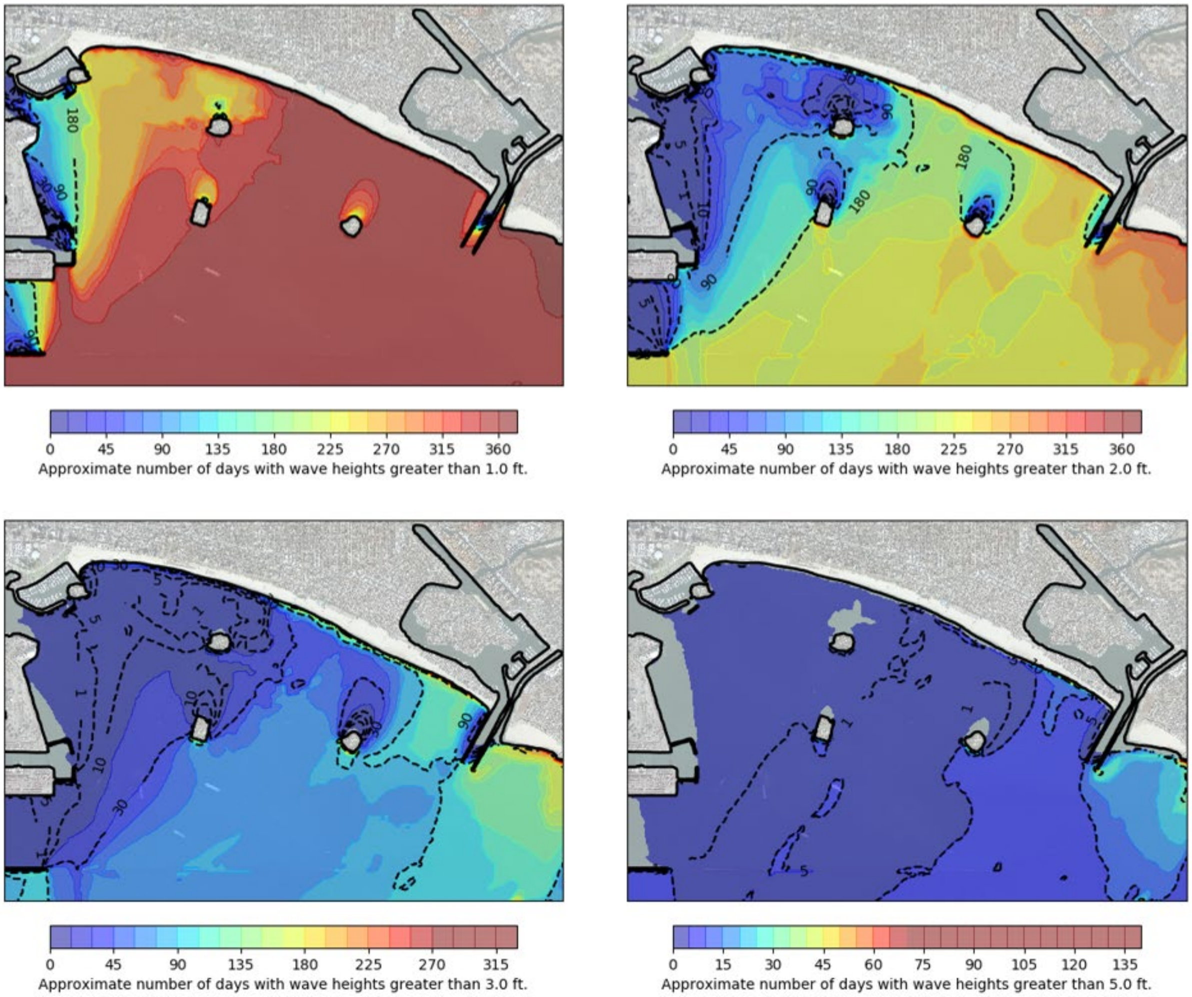


Figure 7-25: Probability of Occurrence due to Complete Breakwater Lowering

7.2.2 NEARSHORE SHOALS AND SUBMERGED BREAKWATERS AND EMERGENT ISLANDS

The nearshore shoals or submerged breakwaters cause waves to shoal and break thus reducing the energy transmitted to the shoreline. The emergent island provides an impermeable barrier to waves; completely reducing the transmitted wave height to zero on the lee of the structure. Diffractive processes allow for the energy transmitted in-between the structures to spread in areas of lower energy, furthermore reducing the incident wave height on the shore face. The following, Figure 7-26, shows the difference in wave heights during a 1-yr southern swell event for the four configurations of submerged breakwaters and emergent islands. Minor differences in the extent of the altered wave heights when comparing to results from the previous section are due to the 4 in. wave height cutoff that is not applied to these figures.

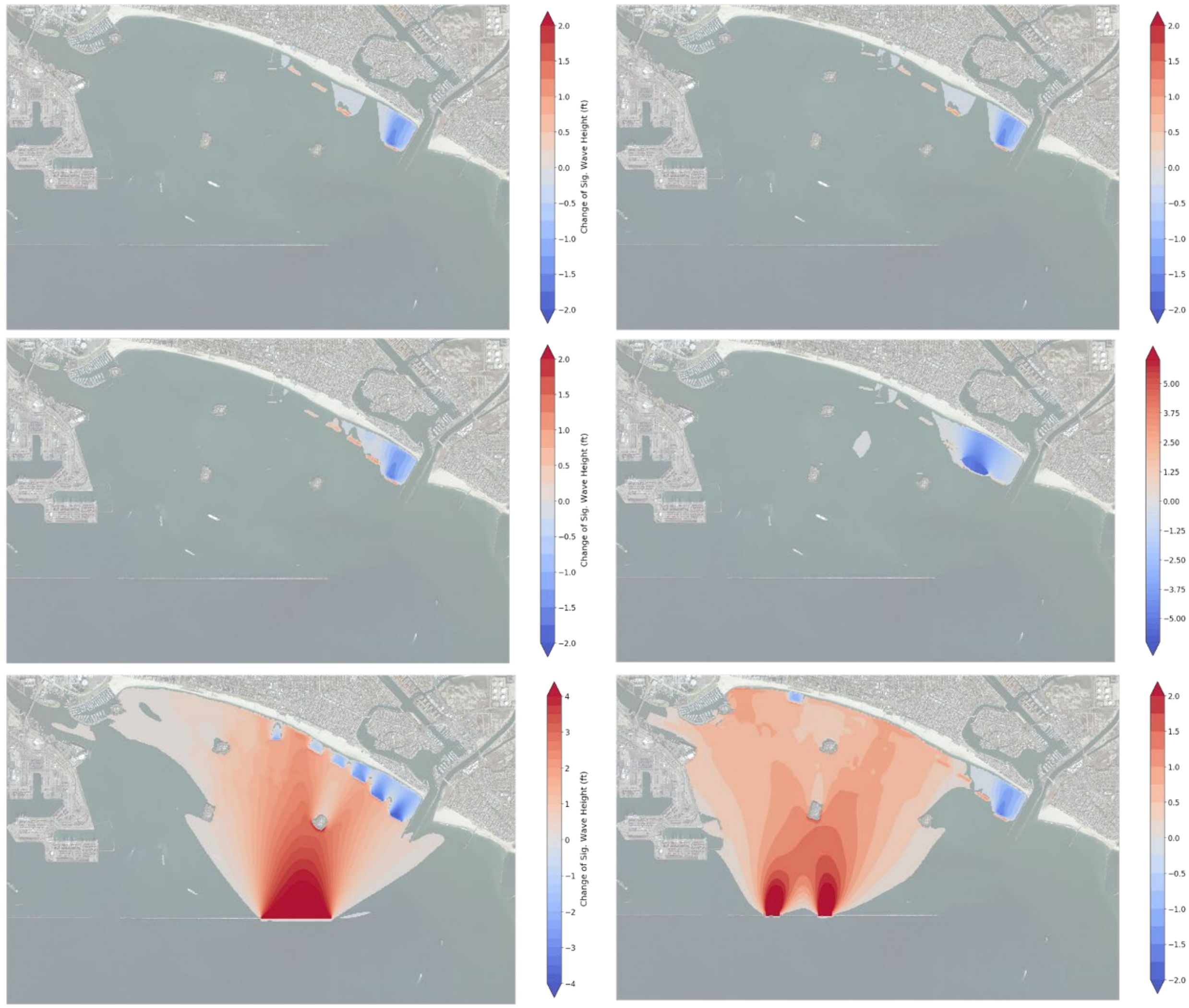


Figure 7-26: Change of Wave Conditions from Nearshore Reefs during a 1-Year South Swell Event Blue tint indicates decrease in expected wave heights. Top row: five reefs (left) and six reefs (right). Middle row: five reefs shifted (left) and six reefs with emergent island (right). Bottom row: Eastern removal of breakwater with submerged breakwater (left) and Western notches with nearshore reefs and Junipero Beach submerged breakwater (right)

7.2.3 WAVE PERIODS

The peak wave period is not expected to change significantly as a result of modifications to the breakwater. The output from Bouss-2D provides a way to capture the change of wave period. Recording stations are chosen within the Bouss-2D grid to output a time series of the water surface elevation at a sampling rate that is the same as the simulation time step. The time series is then processed using a Discrete Fourier Transformation (FFT algorithm) which converts the results from time space to frequency space. The frequency (or period) distribution is then examined to show the change caused by the breakwater modification. The final 30 minutes of the modeled water surface elevation data is utilized for each modeling alternative and a window of 512 sec (n=2048) is applied to reduce the associated error bounds corresponding to the long time series. The following figures show the energy distribution as a function of the wave period for various locations within the proposed project area. The peak period is displayed in the legend corresponding to the breakwater modification. The modifications that were included in this modeling exercise are:

- Existing breakwater configuration (Existing)
- Removal of the eastern 1/3 (EastLop)
- Removal of the western 1/3 (WestLop)
- 2x 1000' notches in the western side (WestNotch)
- 2x 1000' notches in the eastern side (EastNotch)
- Complete lowering to -30' (Low)

The mean spectrum within they bay is shown in Figure 7-27 and is calculated using the average energy within each frequency bin across the spatial domain for the 1-year southern swell event. Notice that the larger the breakwater modification, the more energy is transmitted into the bay.

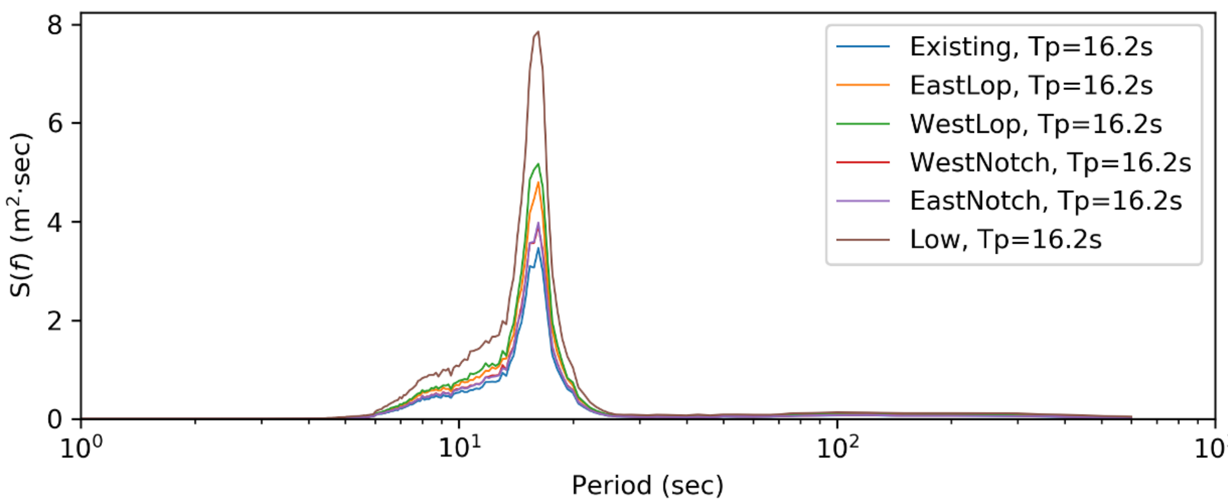


Figure 7-27: Mean Energy of Entire Bay for Various Breakwater Modifications

The influence of the breakwater on Anaheim Bay is minimal, both due to the proximity to the breakwater itself and the protection provided by the harbor's jetties. Figure 7-28 shows the energy distribution that is generally unchanging due to the breakwater modification. The peak periods do not deviate from the existing conditions. All the test cases for breakwater modification did not produce any change to the wave climate within Anaheim Bay.

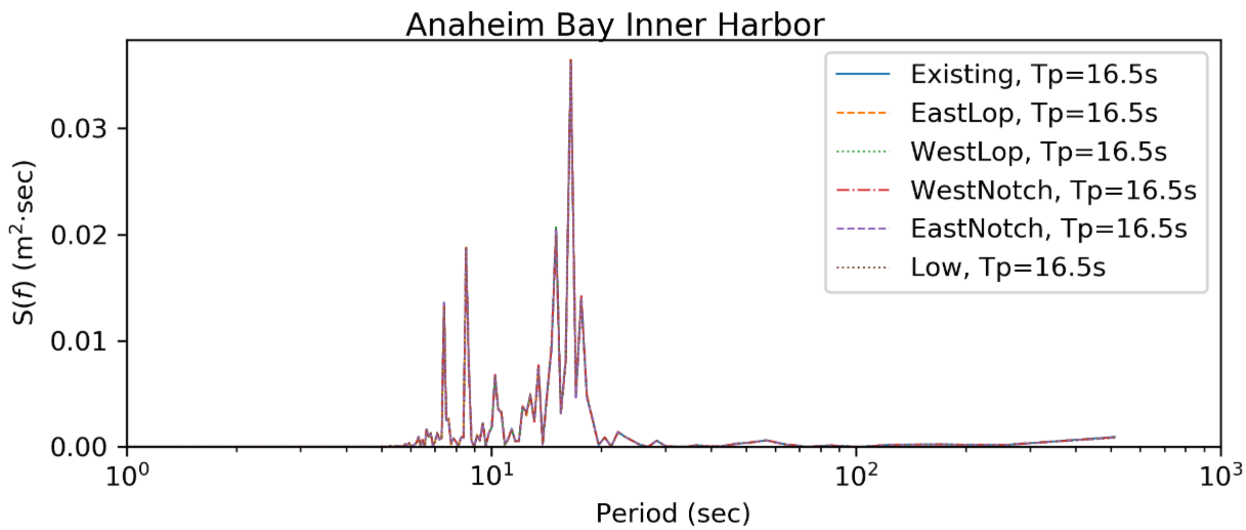


Figure 7-28: Period Distribution of Energy within the Inner Harbor at Anaheim Bay

Pier J South is one of the areas most affected by increases to wave heights. Figure 7-29 to Figure 7-31 presents the energy distribution just outside and inside of the protective jetties at Pier J South as well as near the berth within Pier J South, respectively. Within Pier J, the energy distribution trend remains similar to that of the existing condition, but the magnitude increases due to the breakwater modifications on the western side. Notching or removing a section of the breakwater will not drastically alter the period distribution within Pier J South. Lowering of the entire breakwater will allow for energy to become re-distributed to both higher and lower frequency components. For all changes to the system, the high frequency components can be generally ignored since there would not be a noticeable impact to port operations. The increase of the low frequency component, greater than 60 seconds, will lead to unwanted ship motions within Pier J both during transit and at berth. The western removal configuration has a similar effect on Pier J as the lowering of the entire breakwater.

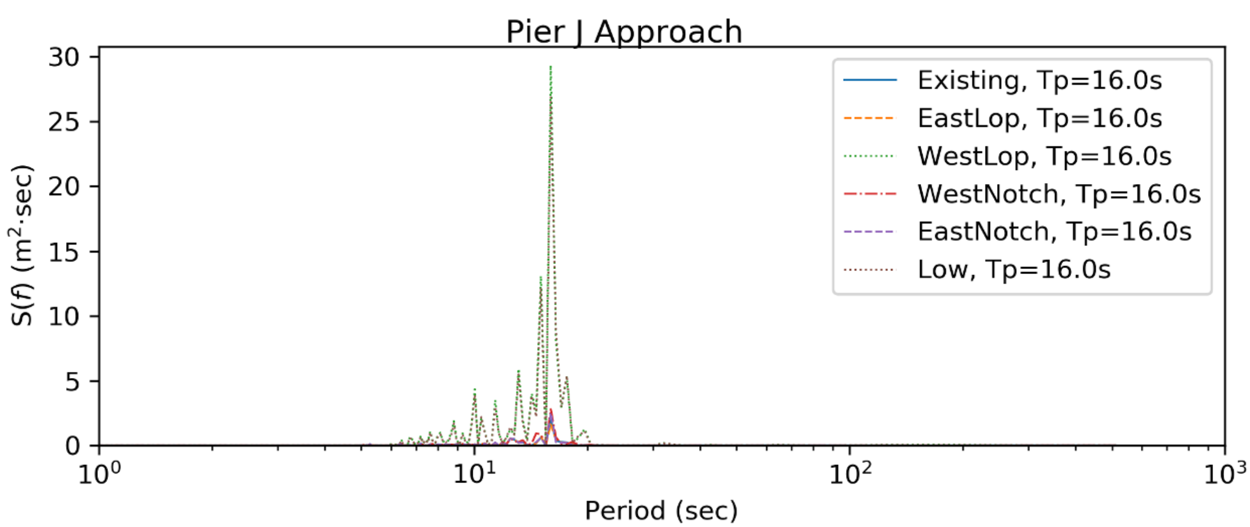


Figure 7-29: Period Distribution of Energy at Pier J South Approach

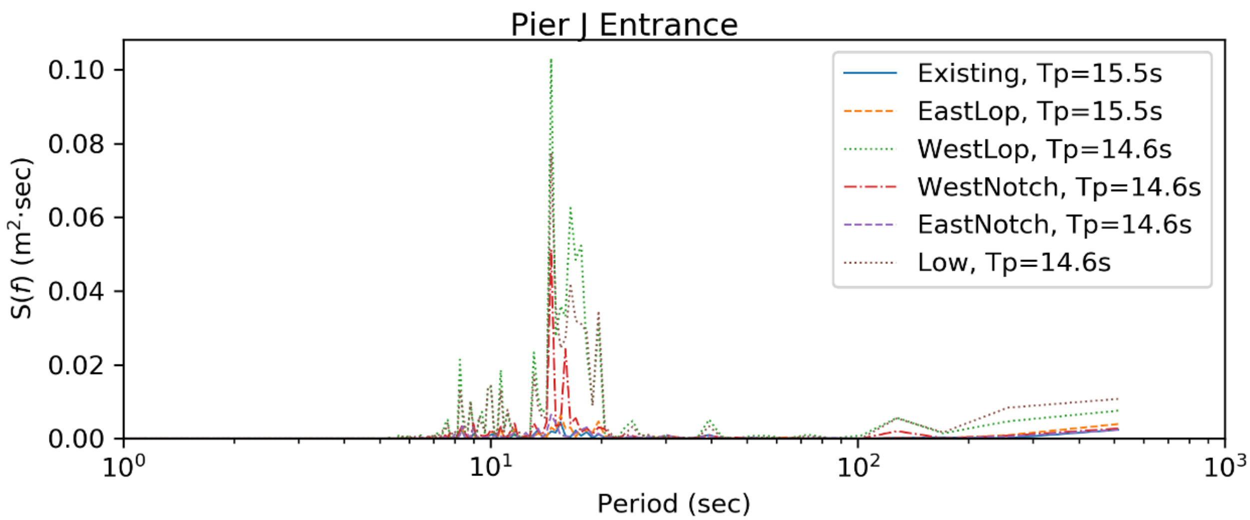


Figure 7-30: Period Distribution of Energy Directly inside the Pier J Jetties

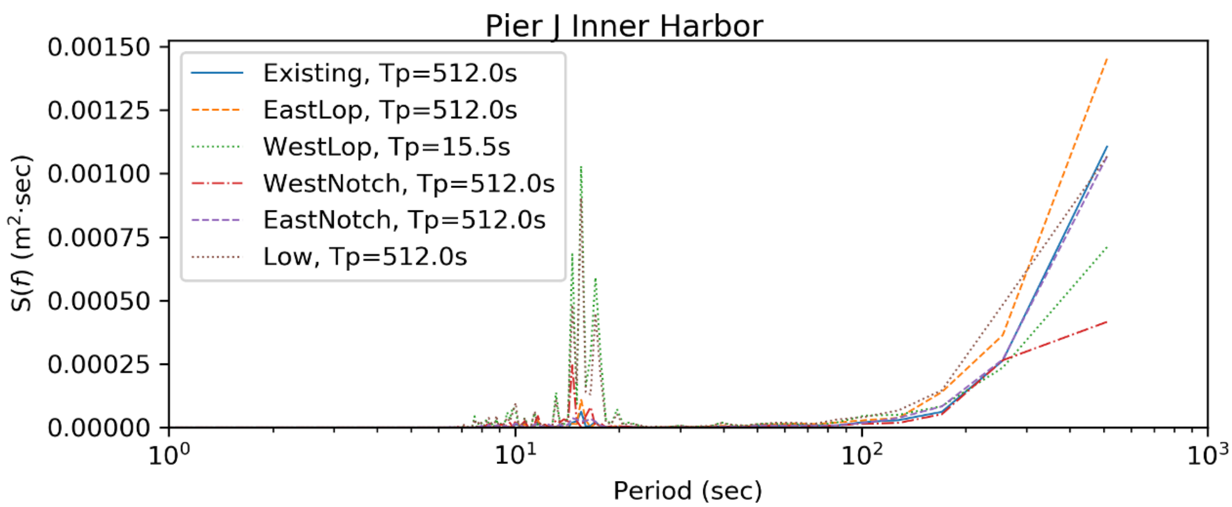


Figure 7-31: Period Distribution of Energy within Pier J South

The wave energy distribution near the Carnival Cruise Lines terminal remains relatively similar throughout the frequency space with some exceptions, as seen in Figure 7-32. Although the distribution is similar, the total energy for each breakwater modification alternatives is significantly different. Western notches tend to shift the period distribution to lower frequencies. Lowering and eastern removal have significant energies in the lower frequency bands (long periods). In all cases, the low frequency energy is greater when compared to the existing conditions near the Carnival Cruise Lines terminal which may lead to increases of ship motions when at berth.

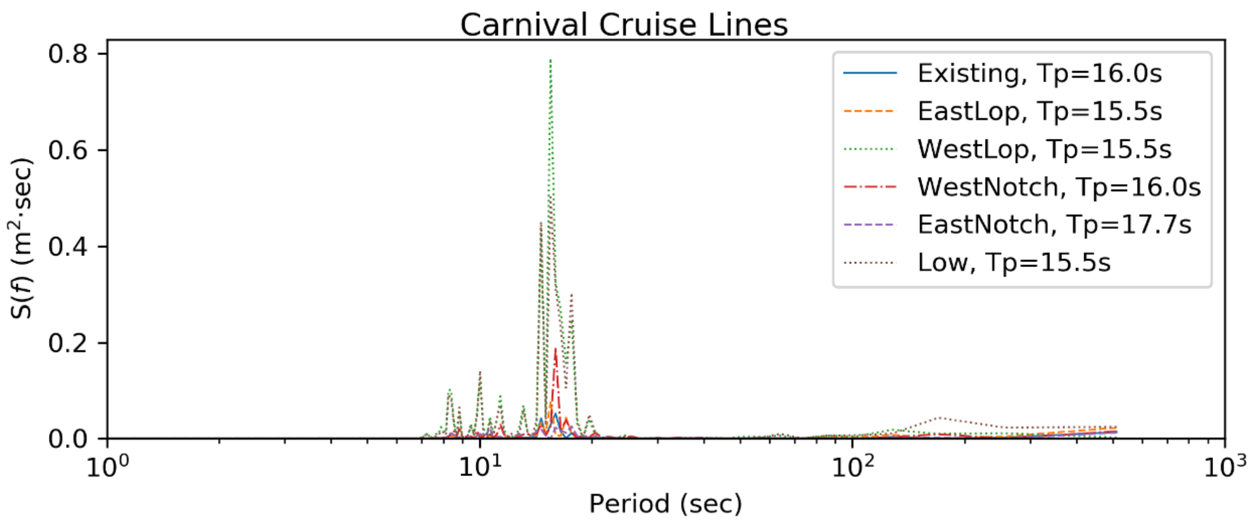


Figure 7-32: Period Distribution of Energy Near the Carnival Cruise Lines Berth

Near Belmont Pier, the breakwater modifications cause a minor shift of energy to other frequency components, shown in Figure 7-33, but remains near the existing conditions distribution. As more of the breakwater is modified, higher frequency components are apparent due to the wave-wave interactions especially in the shallower water depths.

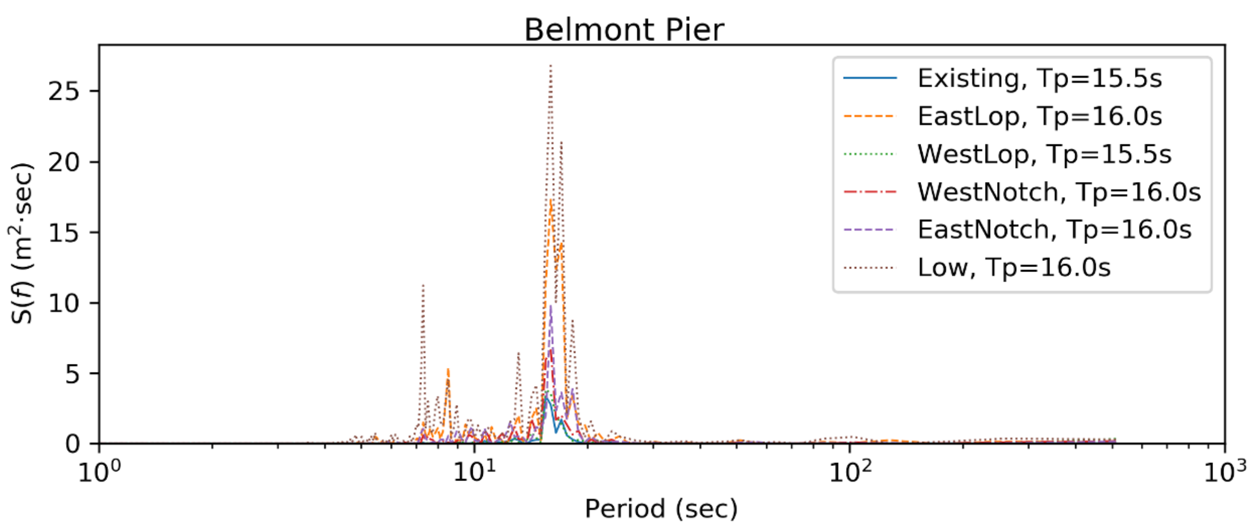


Figure 7-33: Period Distribution of Energy near Belmont Pier

7.2.4 <u>WAVE GENERATED VELOCITIES</u>

Wave generated mean velocities are relatively small within the project area. The change in mean velocity for a 1-yr southern swell event due to breakwater modifications is shown in Figure 7-34, modeled using Bouss-2D. Mean flow directions produced by waves is shown in Figure 7-35 and plotted on top of the wave height. The direction of the nearshore current does not significantly change due to the breakwater modifications; the Peninsula Beach shoreline will remain erosive independent of the breakwater modification.

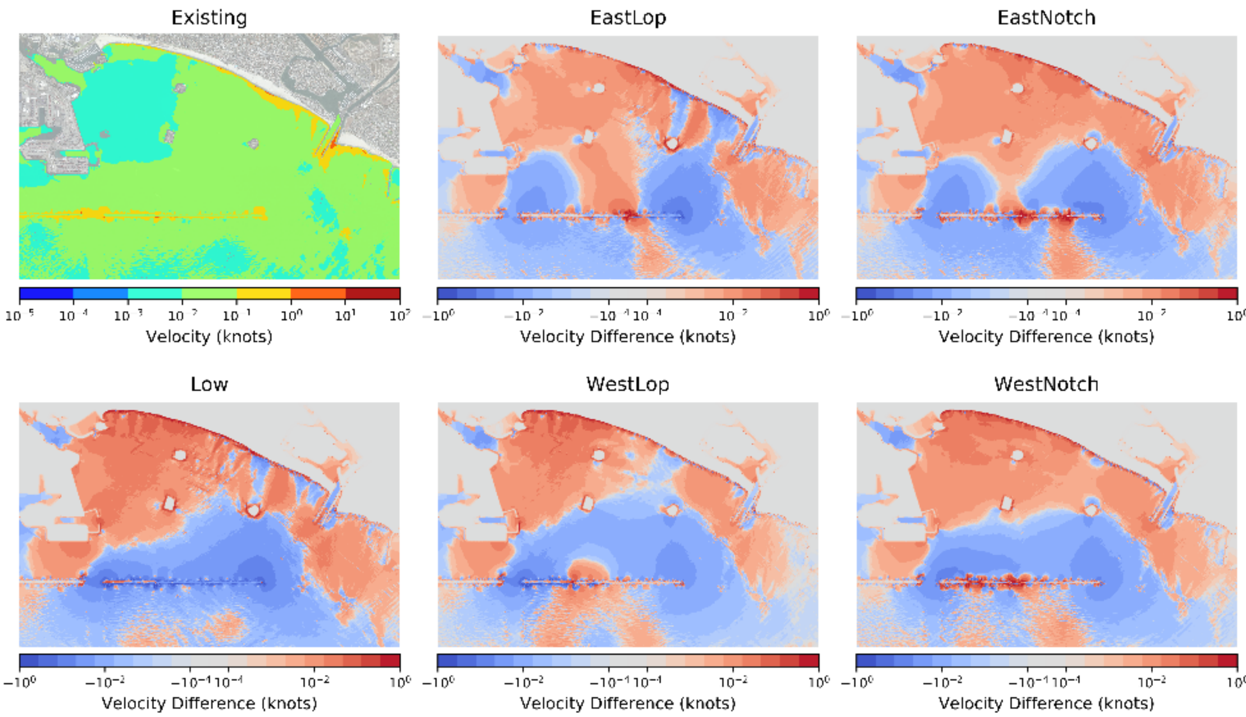


Figure 7-34: Wave Generated Velocities and Percent Differences for the Breakwater Modifications

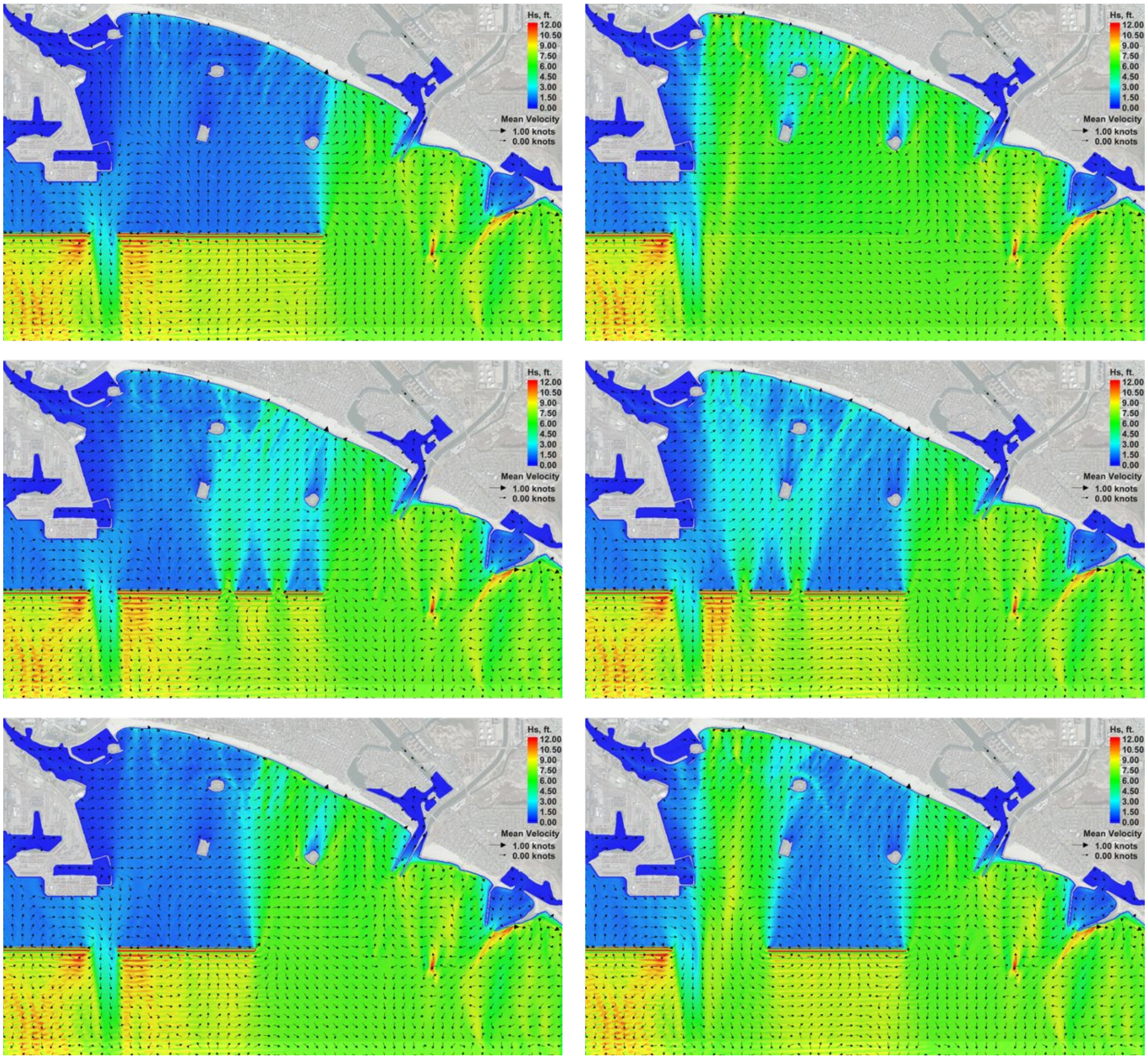


Figure 7-35: Mean Velocities and Wave Heights for a 1-Year Southern Swell Event Corresponding to: Existing Conditions (Top Left), Complete Lowering (Top Right), Eastern Notching (Middle Left), Western Notching (Middle Right), Eastern Removal (Bottom Left), Western Removal (Bottom Right)

7.3 LITTORAL TRANSPORT AND SEDIMENT MOBILITY

7.3.1 LONGSHORE TRANSPORT AND GENCADE

The calibrated GenCade model is used to evaluate the shoreline change resulting from the focused array of alternatives. More analysis is needed during the design phase to determine the exact location and structure height of the nearshore features to both protect infrastructure from runup and overtopping as well as to limit the effects of local increases to erosion of the shoreline. The following shoreline change analysis provides the shoreline trend in response to the proposed structural changes for the focused array of alternatives discussed in the MAIN REPORT. Small changes to structure placement and crest height will influence the shoreline position and potential transport rate. This analysis is preliminary and will be verified using a physical model during the PED phase if such breakwater modifications are included in the selected plan.

7.3.1.1 EXISTING CONDITIONS

To model the existing conditions, the shoreline is assumed to be unchanged from the current position until the start of the simulation at the base year of 2030. This assumption requires that the current backpassing operations remain constant and cross-shore losses are replenished; which can come from dredging events at the Los Angeles River Estuary or Alamitos Bay.

For these GenCade simulations, only the first 10 years will be presented to show the response of the shoreline to the structural modifications. This pattern is assumed to persist over the remainder of the project lifetime. Sea level rise will affect the entire shoreline equally, so the change in water level is excluded for this modeling effort. The shoreline evolution is shown at 3 year intervals in Figure 7-36 corresponding to the existing condition. The shoreline position is the offset distance from the reference line presented in Figure 6-22. Due to the difference in the longshore transport, the shoreline near Peninsula Beach begins to erode immediately. The material is accreted along the beach directly west of the eroded area. If variable backpassing is employed (which is currently implemented), the erosion along Peninsula Beach can be controlled. This shoreline change model is developed to show the difference that the structural modifications have on the shoreline; the exact magnitude may vary from the reported value. Long-term shoreline changes are highly dependent on wave height and directions; any deviation from the historic conditions will modify the transport patterns and rates and will not be accounted for in this analysis.

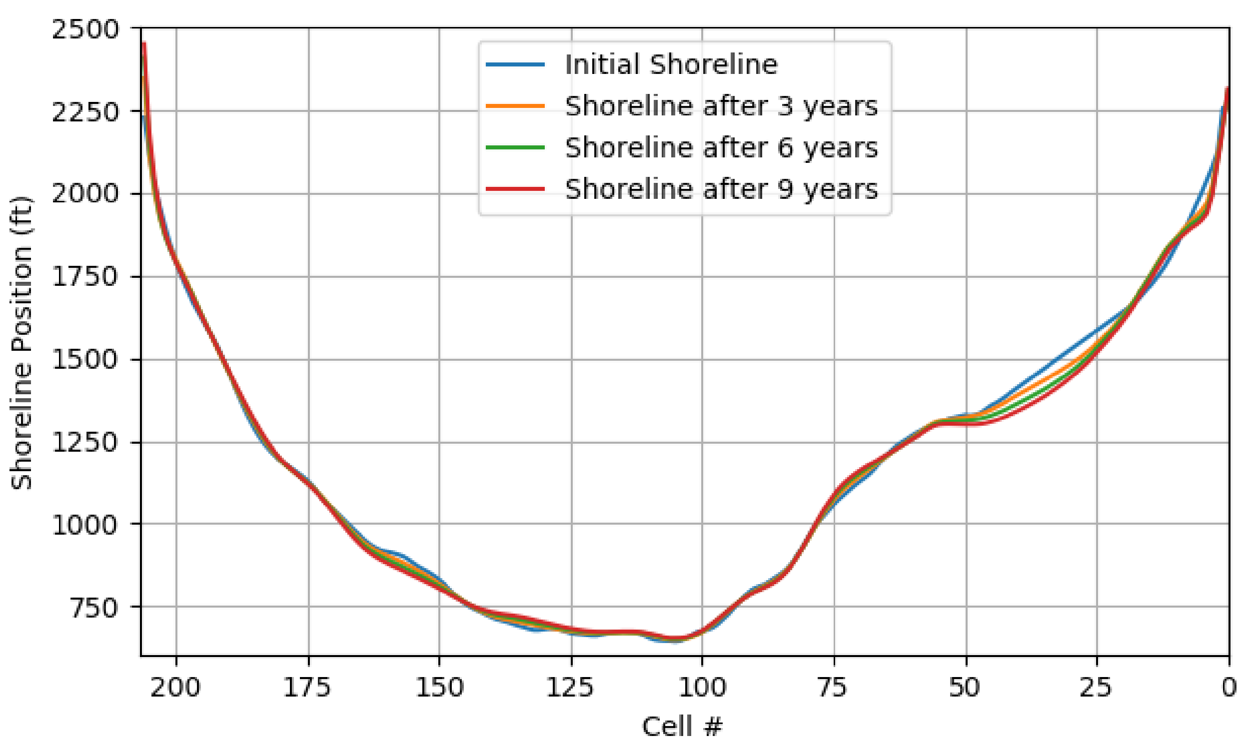


Figure 7-36: Shoreline Position Evolution at 10 Years Corresponding to Existing Conditions

7.3.1.2 NEARSHORE REEFS (ORIGINAL)

The nearshore reefs cause waves to shoal and break, dissipating energy which allow for lower localized sediment transport rates. Since there is no net gain of sediment in the system, the shoreline is eroded in some areas and accreted in others. The net effect is zero shoreline change; although there are some offshore losses. Figure 7-37 shows the difference from the modeled shoreline produced from the existing conditions and with the shoreline produced with the inclusion of the nearshore reefs.

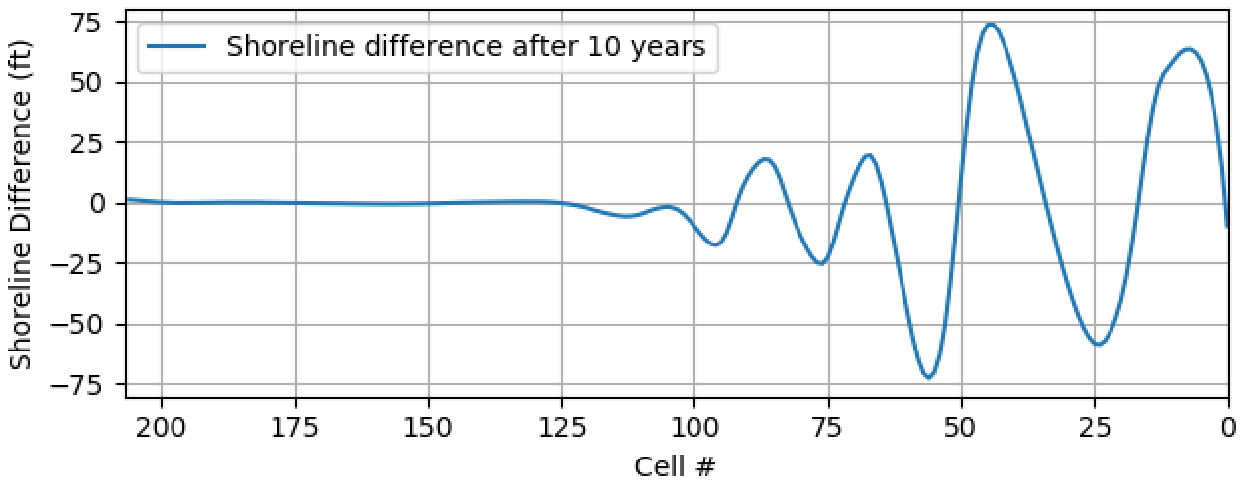


Figure 7-37: Shoreline Comparison of Existing Conditions and Nearshore Reefs after 10 Years

Figure 7-38 shows the shoreline position as compared to the shoreline without any nearshore reefs. The reefs cause a more undulating shoreline as compared with the existing conditions. Sediment is slowed behind the nearshore reefs causing accretion in some areas and erosion in others. For ease of description, the nearest cross street is used as delimiters.

Accretion:

- Alamitos Bay to 64th Pl.
- 57th Pl. to directly between 54th Pl. and 55th Pl.
- Pomona Ave. to Covina Ave.
- West of Granada Ave. to Prospect Ave.

Erosion:

- 64th Pl. to 57th Pl.
- Between 54th Pl. and 55th Pl. to Pomona Ave.
- Covina Ave. to west of Granada Ave
- Prospect Ave. to Redondo Ave. (minor)

Due to the limitation of the shoreline change model in this feasibility level analysis, exact volumes of eroded or accreted sediments is unknown.

Figure 7-38: Shoreline Position with Nearshore Reefs

7.3.1.3 NEARSHORE REEFS - SHIFTED

If the nearshore reefs are shifted to provide increased protection to Peninsula Beach, the transport pattern slightly changes. The difference between the existing conditions calculated shoreline and the shoreline produced by the shifted nearshore reefs is shown in Figure 7-39.

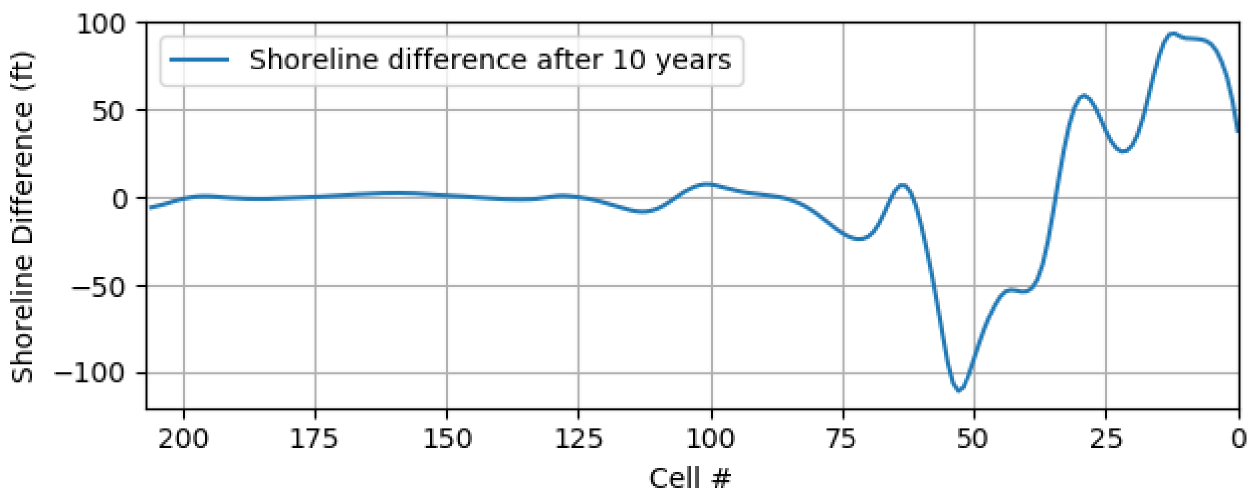


Figure 7-39: Shoreline Comparison from Existing Conditions to the Shifted Nearshore Reefs

Again, the net shoreline change is zero (minus the lateral and offshore losses), but localized effects cause erosion or accretion in some areas, as seen in Figure 7-40, but is has a more limited extent than the previous configuration.

Accretion:

- Alamitos Bay to 57th Pl.

Erosion:

- 57th Pl. to Santa Ana Ave.
- Santa Ana Ave to Park Ave. (minor)

Figure 7-40: Shoreline Position with Shifted Nearshore Reefs

The longshore transport rate for conditions with the nearshore reefs are shown in Figure 7-41. The inclusion of the original nearshore reefs does little to modify the longshore transport rate and direction. There are localized effects directly landward of the reefs, but no substantial large-scale changes. The shifted nearshore reefs allow for less movement of sediment along Peninsula Beach; the rate slows to ~30,000 yd^3/yd. The decrease in transport occurs from the eastern boundary until Belmont Pier, where the rate begins matches the existing conditions more closely.

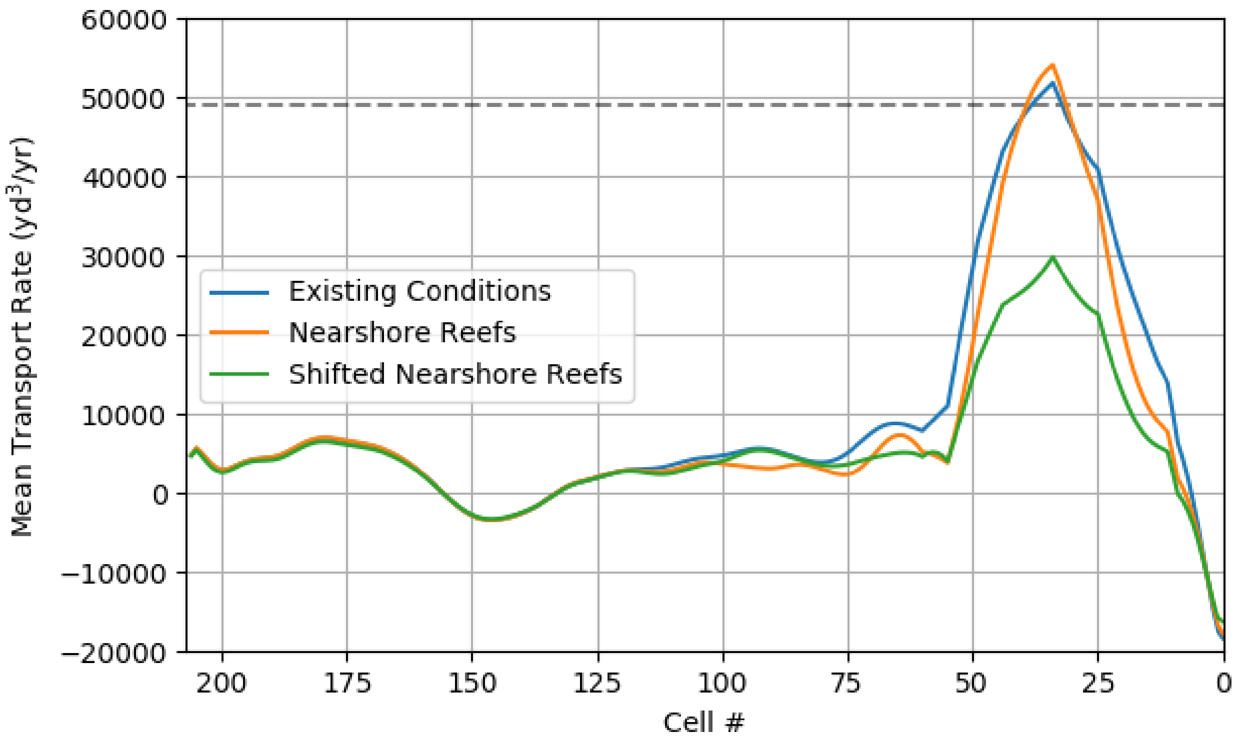


Figure 7-41: Mean Transport Rate Comparison for Nearshore Reefs

7.3.1.4 SANDY ISLAND WITH REEFS

The inclusion of an increasing number of nearshore reefs and an emergent island greatly reduces the incident wave height along the shoreline and allows for wider beach growth. The difference from the existing conditions is shown in Figure 7-42.

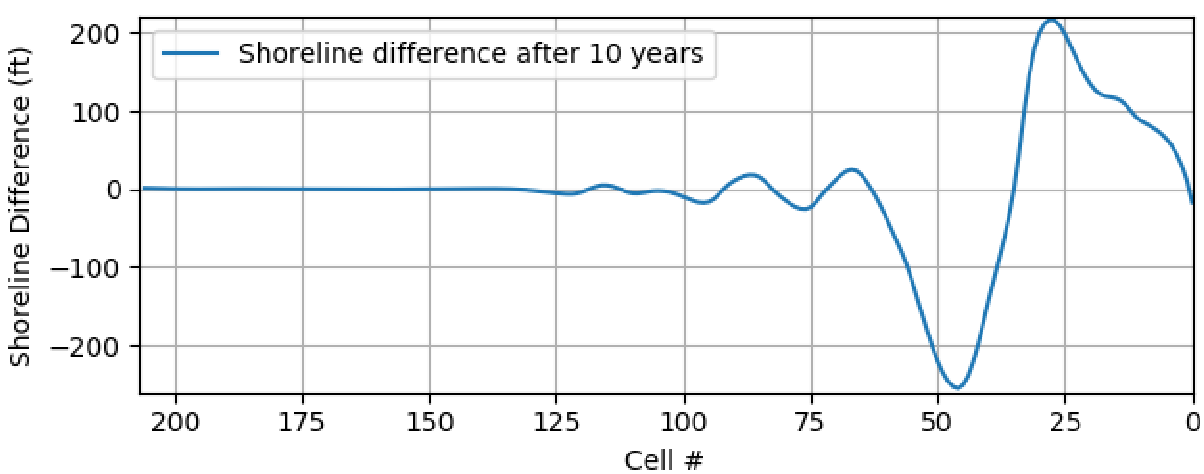


Figure 7-42: Shoreline Comparison from Existing Conditions, Nearshore Reefs and Emergent Island

Away from Peninsula Beach, the shoreline generally follows that of the existing conditions, shown in Figure 7-43. The low crested nearshore reefs do little the change the transport patterns when compared to how the emergent island governs the trend of the shoreline. Since the sediments are slowed, behind the island, an erosional pocket is created directly west due to the interface of the lower and natural sediment transport rates. The localized accretion and erosional effects are:

Accretion:

- Alamitos Bay to Dana Pl.
- Pomona Ave. to Nieto Ave.
- Park Ave. to Ximeno Ave.

Erosion:

- Dana Pl. to Pomona Ave.
- Nieto Ave. to Park Ave.
- Ximeno Ave. to west of Belmont Pier

Figure 7-43: Shoreline Position from with Nearshore Reefs and Emergent Island

The emergent island is a terrific barrier to incoming waves as longshore transport is basically stopped altogether as seen in Figure 7-44. The shoreline west of Belmont Pier remains similar to the existing conditions.

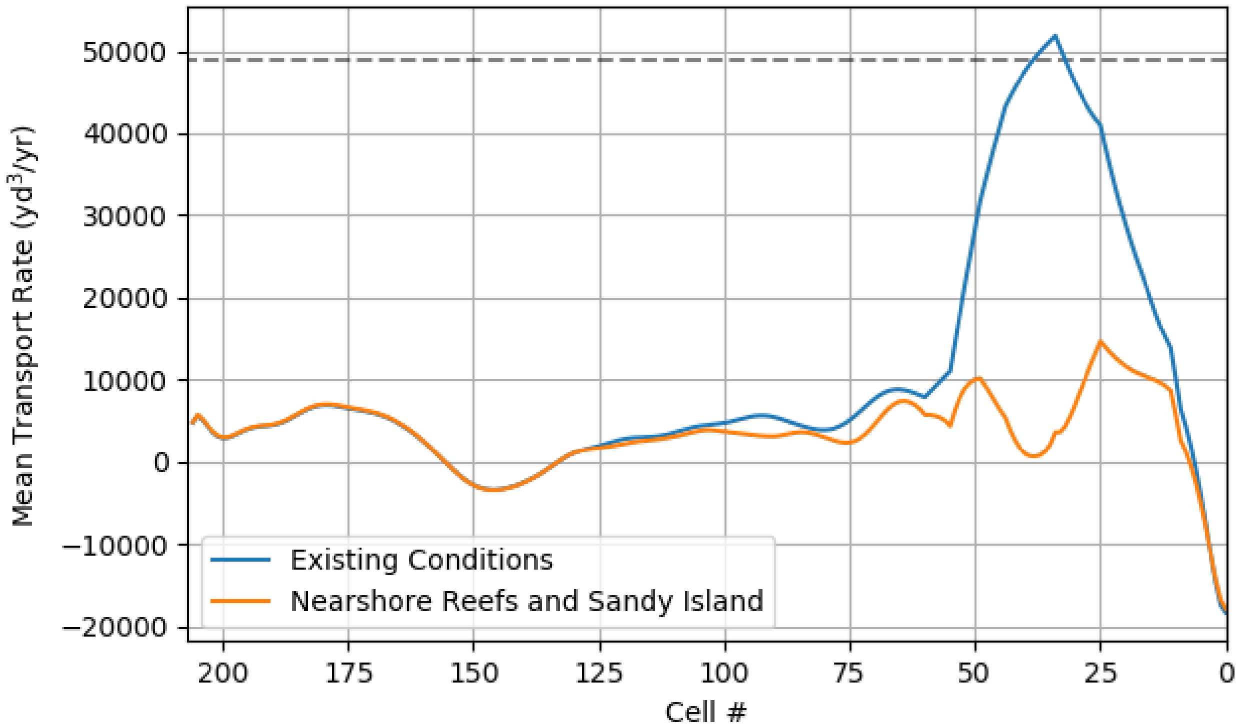


Figure 7-44: Mean Transport Rate for Nearshore Reefs and Emergent Island

7.3.1.5 EASTERN REMOVAL + NEARSHORE REEF (MOD 1) + ADDITIONAL REEF

As the Long Beach Breakwater is modified, the nearshore reefs must be raised to a higher crest elevation (notated as submerged breakwaters) to provide adequate protection to the shoreline as well as the habitats under consideration, such as eelgrass. Figure 7-45 and Figure 7-46 show the difference between the shoreline with existing conditions and removal of the eastern 1/3 of the Long Beach Breakwater. Without any protection, the shoreline erodes locally near Peninsula Beach, but with the increase transport potential, sediments are now distributed alongshore providing for a wider beach west of Belmont Pier. The inclusion of the protective submerged breakwaters decreases the localized erosion near Peninsula Beach but creates an undulating shoreline of erosion and accretion until west of Belmont Pier.

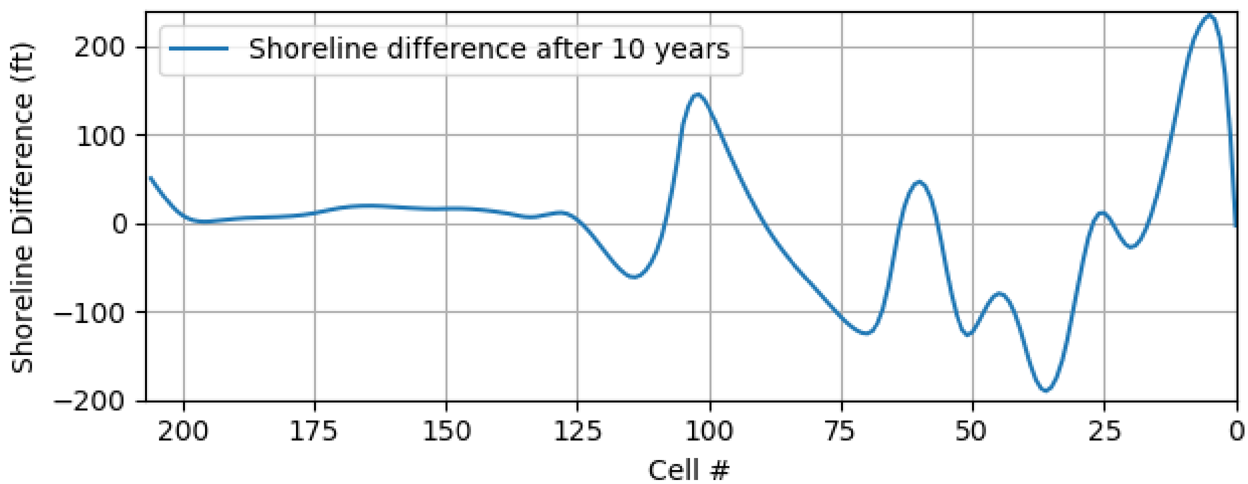


Figure 7-45: Shoreline Difference between Existing Conditions and Removal of Eastern 1/3 of Breakwater

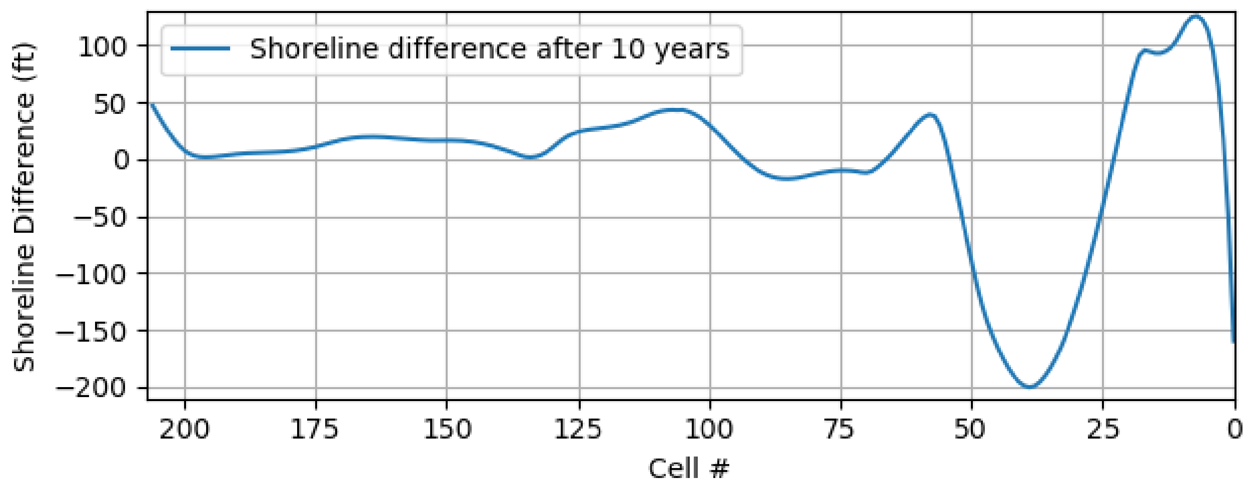


Figure 7-46: Shoreline Difference between Existing Conditions and Removal of Eastern 1/3 of Breakwater without Submerged Breakwaters

The shoreline position is shown in Figure 7-47 for each of the above described simulations. Minor difference between the protected and un-protected shoreline exist but are limited to direct effect produced by the submerged breakwaters. Shoreline change as a result of the breakwater modification must be verified by physical model. With the increase of incident wave heights, both the depth of closure and the berm height will be altered and would change the quantity of sediment available within the littoral cell which is not accounted for in this analysis. This analysis is provided to give an overview of the potential changes and not the exact magnitude. As stated earlier, a physical model will need to be developed in PED to quantify all the impacts of such a breakwater modification.

Accretion:

- Alamitos Bay to 59th Pl.
- 54th Pl. to La Verne Ave.

- Roycroft Ave. to Belmont Pier
- Redondo Ave. to Shoreline Marina

Erosion:

- 59th Pl. to 54th Pl.
- La Verne Ave. to Roycroft Ave.
- Belmont Pier to Redondo Ave.

Figure 7-47: Shoreline Position with Eastern 1/3 Breakwater Removal with and without Protective Submerged Breakwaters

Due to the modification of the breakwater, the longshore transport rate increases overall, increasing the total amount of sediment that is mobilized throughout the shoreline, as seen in Figure 7-48. The inclusion of the submerged breakwaters locally slows the transport rate near Peninsula Beach but produces increases in transport rate outside of this area. Overall, the removal of the eastern 1/3 of the breakwater will locally increase erosion along Peninsula Beach and cause accretion west of Belmont Pier.

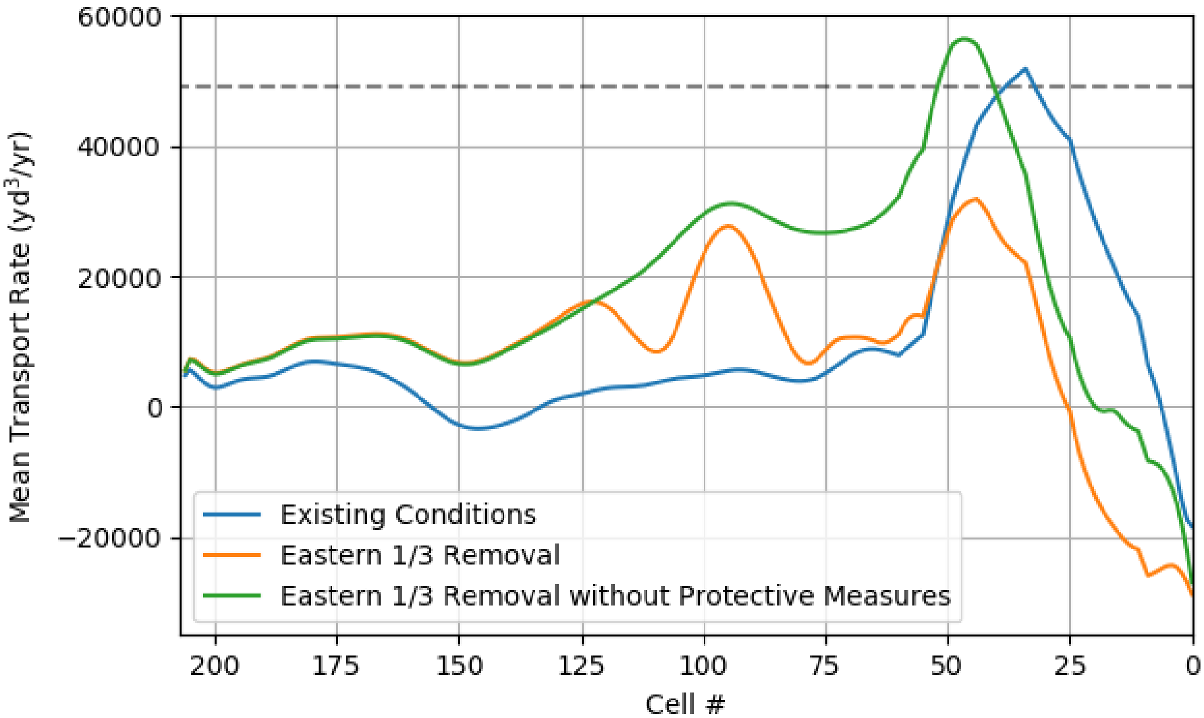


Figure 7-48: Mean Transport Rate for Removal of Eastern 1/3 of the Breakwater

7.3.1.6 WESTERN NOTCHING + NEARSHORE REEF (MOD 1) + PARKING LOT PROTECTION

With the inclusion of two 1000 ft. notches on the western side of the Long Beach Breakwater, sediment transport patterns produce an interesting result as shown in Figure 7-49. Sediments are transported laterally toward Shoreline Marina in the west, creating a fillet along the jetty, and to the east where sediments slow as the waves from wave entering the bay through notches produce a quasi-equilibrium near 54th Pl. when interacting with waves entering the bay from the east of the Long Beach Breakwater. This trend generally holds true with the addition of a protective submerged breakwater near the Junipero Beach parking lot and the previously describe nearshore reefs in the east of the modeled area, as shown in Figure 7-50.

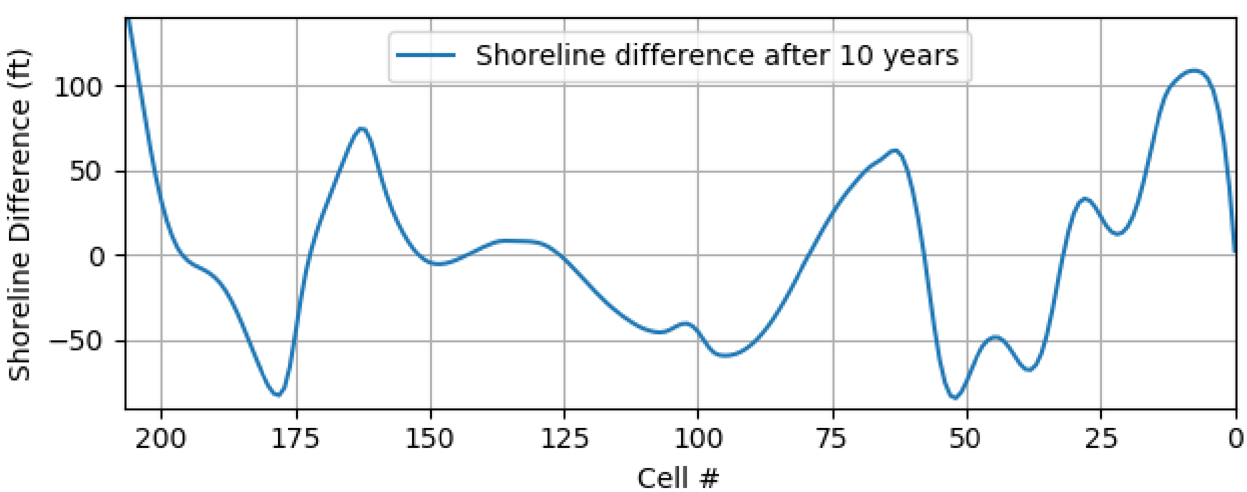


Figure 7-49: Shoreline Difference between Existing Conditions and Western Notching of Breakwater

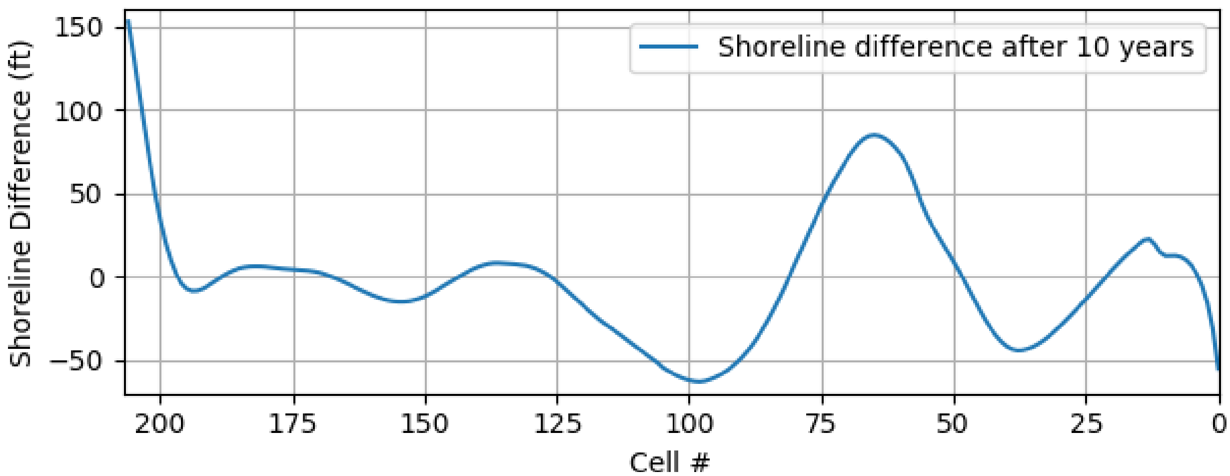


Figure 7-50: Shoreline Difference between Existing Conditions and Western Notching of Breakwater without Submerged Breakwaters or Nearshore Reefs

The inclusion of the nearshore reefs follows a similar pattern, shown in Figure 7-51, without a breakwater modification east of Belmont Pier. To prevent the excessive erosion and increase of wave run-up, the Junipero Beach parking lot must be protected by a submerged breakwater, indicated in Figure 7-52. With the inclusion of this protective measure along with the nearshore reefs the accretion/erosion areas are:

Accretion:

- Alamitos Bay to 58th Pl.
- Bay Shore Ave. to Argonne Ave.
- Redondo Ave. to Cherry Ave.
- Orange Ave. to Shoreline Marina

Erosion:

- 58th Pl. to Bay Shore Ave.
- Argonne Ave. to Redondo Ave.
- Cherry Ave. to Orange Ave.

Figure 7-51: Shoreline Position with Western Notches

Figure 7-52: Shoreline Position with Western Notches, Near Junipero Beach Parking Lot

Due to the notches, the mean transport rate is modified as shown in Figure 7-53. Near Peninsula Beach, the transport rate is generally unaffected by the western notches. The increase in wave energy entering the bay from the western notches causes the net transport to reverse direction east of the pier. This reversal produces an accumulation of sediment where waves from different directions converge. The transport rate increases from existing conditions west of Belmont Pier. The inclusion of the submerged breakwater near Junipero Beach allows the protective beach in front of the parking lot to remain, but increases the transport rate directly west, causing localized erosion to occur.

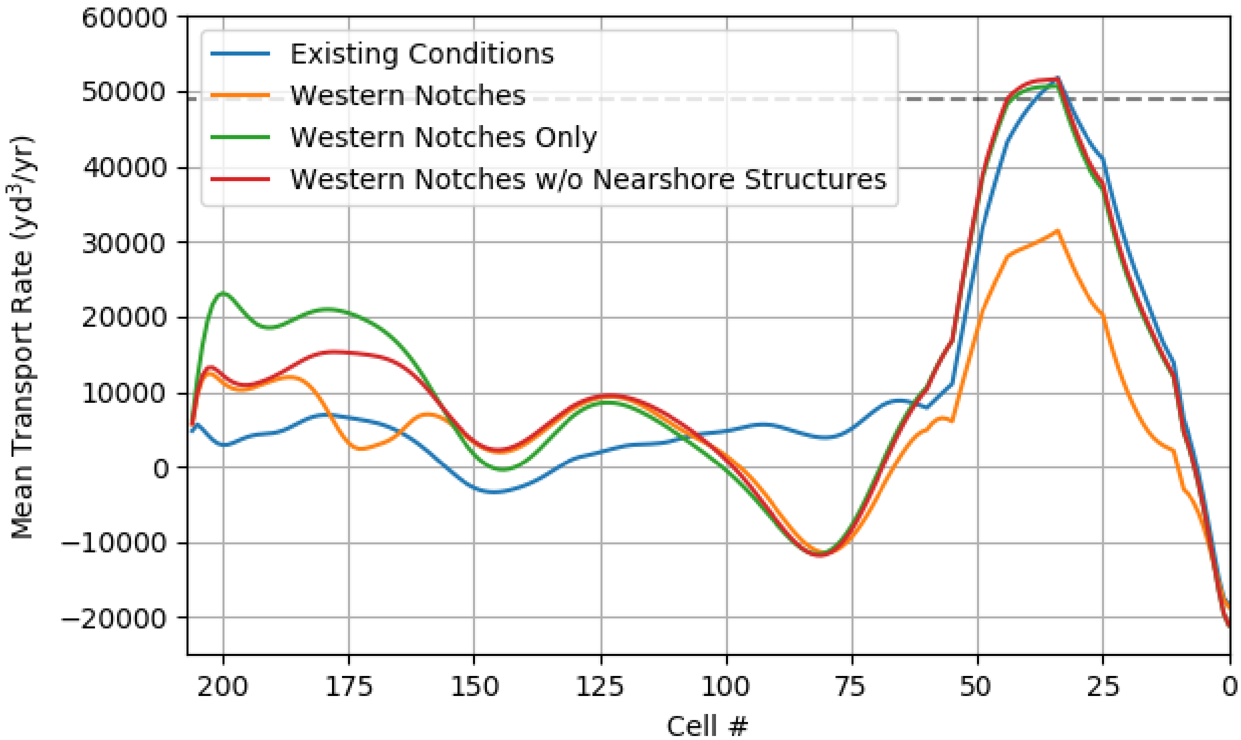


Figure 7-53: Mean Transport Rate for Western Notching of the Breakwater

7.3.1.7 GENCADE CONCLUSIONS

Overall, the net effect of a breakwater modification or inclusion of nearshore reefs is zero; the sediment is redistributed locally. The City of Long Beach's backpassing operations would need to continue, and may have to be modified, to keep a protective shoreline in front of Peninsula Beach.

7.3.2 <u>SEDIMENT MOBILITY</u>

Following the same analysis as described in the SEDIMENT MOBILITY section. Modifications to the breakwater will alter the potential to mobilize sediments that can be carried away by the underlying currents. Only breakwater modifications with corresponding hydrodynamic modeling can be considered since the sediment mobility formulation is based on the contributions from waves and currents.

7.3.2.1 REMOVAL OF EASTERN 1/3

Removal of the eastern 1/3 (4,500 ft.) of the breakwater allows for additional sediment to be mobilized, shown in Figure 7-39, further west into the bay than without the modification. The lateral limits of the increase are linearly dependent on the length of breakwater removed. No sediments are mobilized directly leeward the breakwater as diffractive effects do not play an important role for this physical process.

7.3.2.2 2 X 1000 FT. NOTCHES ON THE EASTERN SIDE

Results produced by two notches on the eastern side of the breakwater have a similar effect to the removal of 1/3 of the structure. Sediments are mobilized locally at the notches, but the increase does not extend landward for more than 2000 ft. The reinforcement of additional wave energy coming through the notches allows for a more western extent of mobilization, but not further in extent than the removal of the eastern 1/3.

7.3.2.3 LOWERING OF ENTIRE STRUCTURE TO -30 FT. MLLW

Lowering of the entire breakwater allows sediment of all sizes to be mobilized throughout the bay. Higher mobility potential is prevalent within the western side of the project area as well as the lee of the energy islands. This condition will cause sediments to be suspended for longer periods, allowing the tidal currents to transport and redistribute the sediments. Bottom sediments will become coarser over time due to the increase of wave energy.

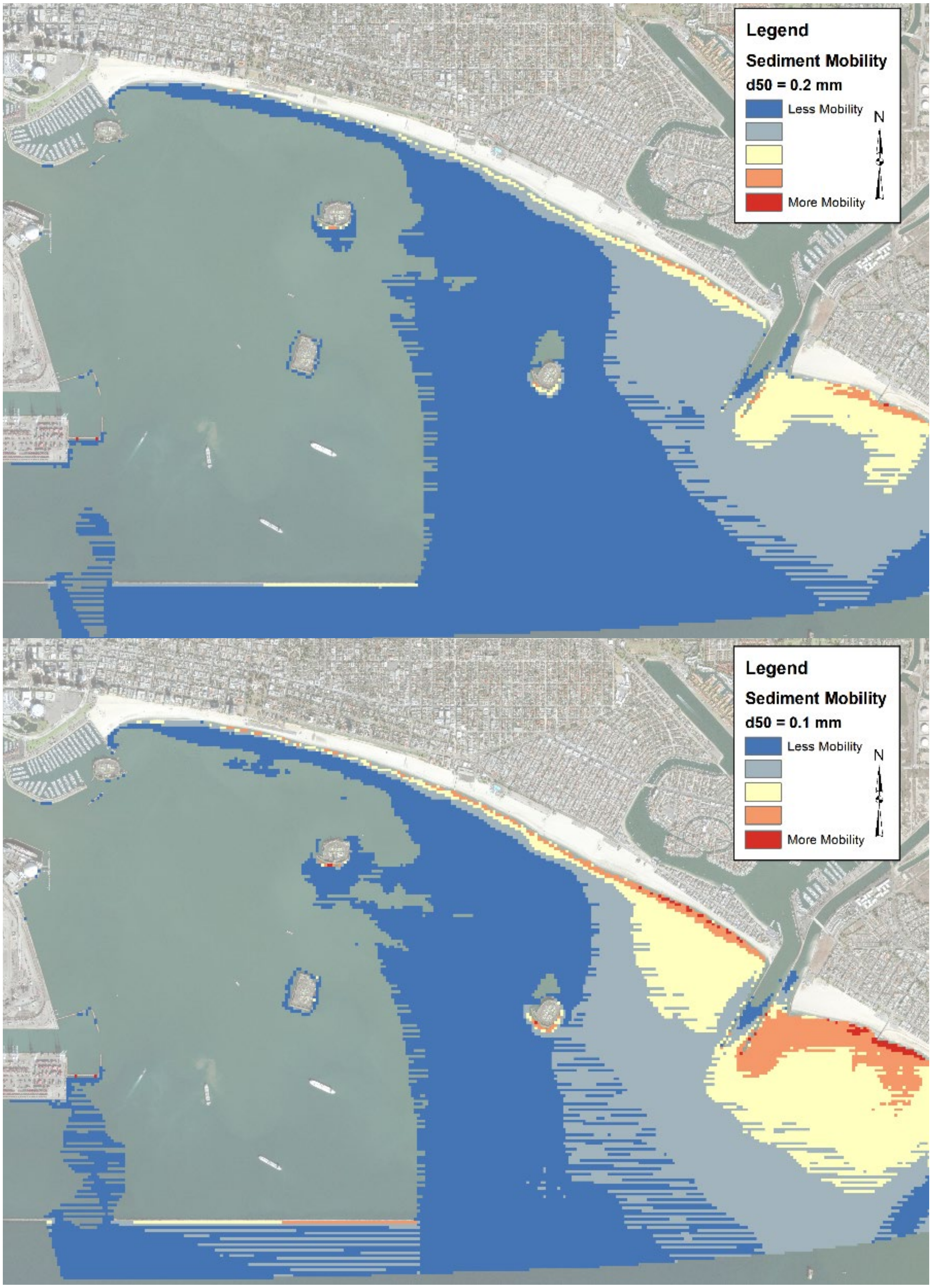


Figure 7-54: Potential Sediment Mobility for Eastern 1/3 Removal of Breakwater with Finer Grain Sediments (Top) and More Coarser Sediments (Bottom)

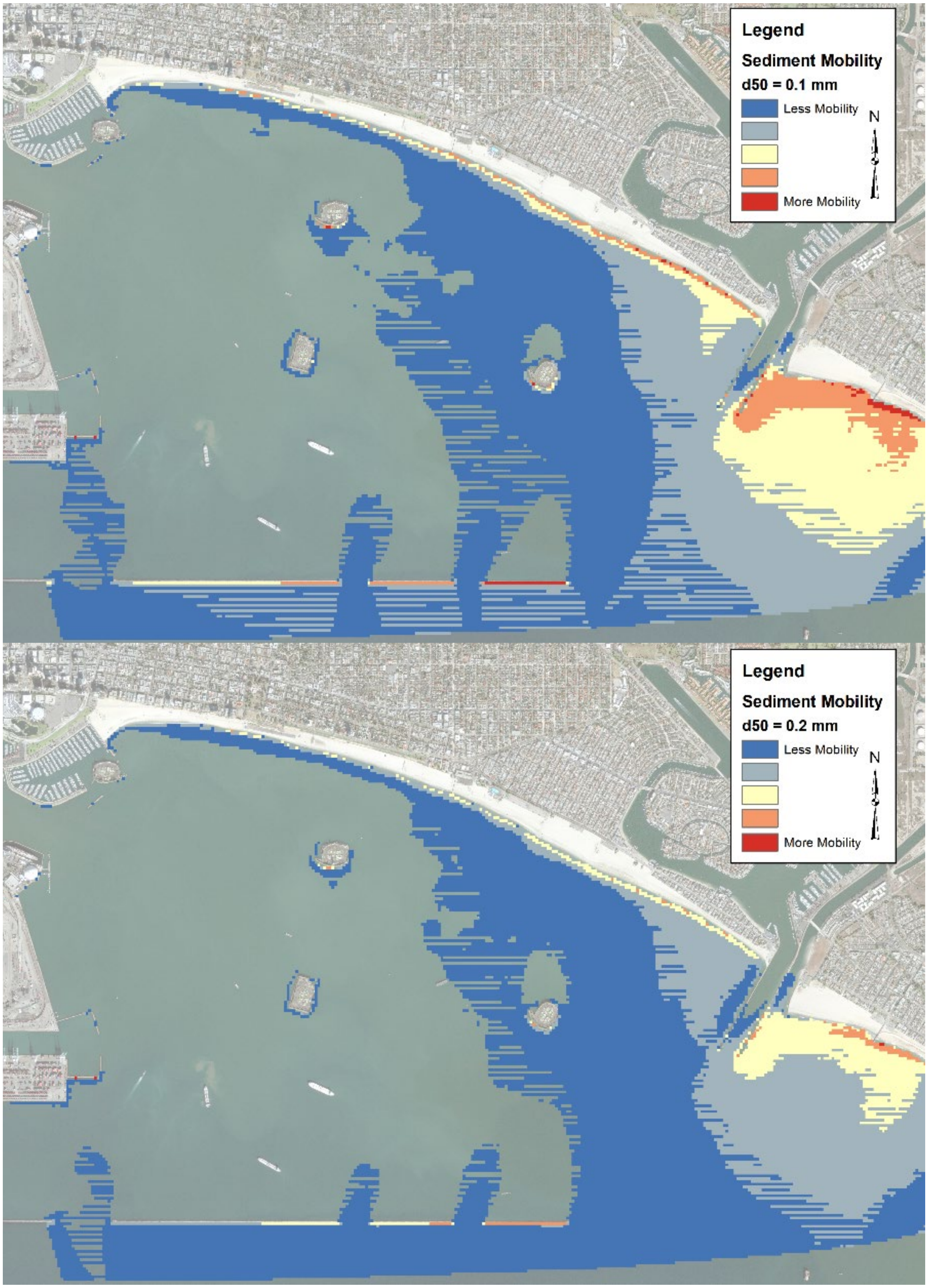


Figure 7-55: Potential Sediment Mobility for Two Notches on Eastern Side of Breakwater with Finer Grain Sediments (Top) and More Coarser Sediments (Bottom)

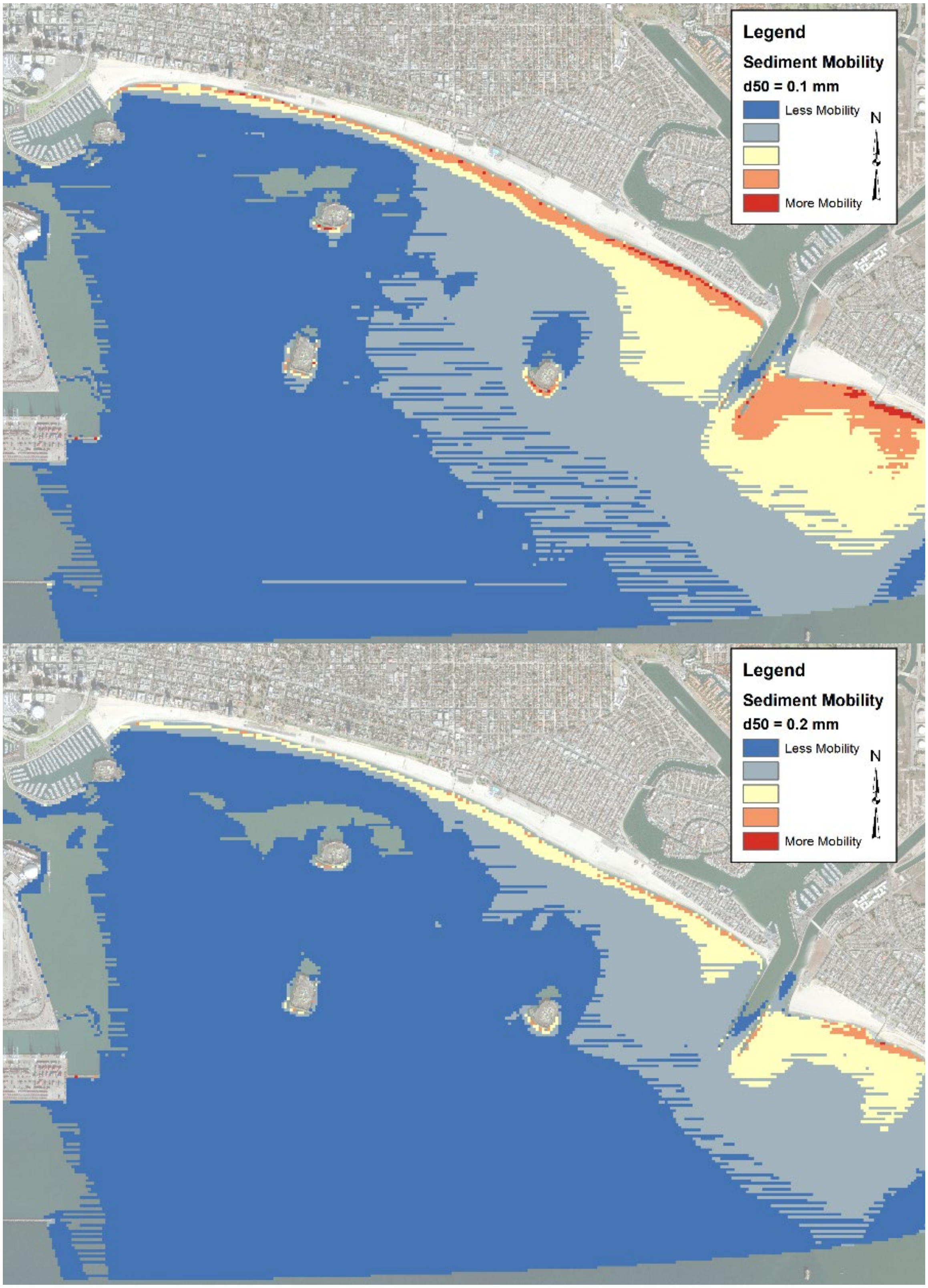


Figure 7-56: Potential Sediment Mobility for Lowering Of Breakwater to -30 Ft. with Finer Grain Sediments (Top) and More Coarser Sediments (Bottom)

7.3.3 CROSS-SHORE TRANSPORT DURING STORM EVENTS

The loss of dry beach width is expected to increase during a storm event if the breakwater is modified. During a storm event, sediments are re-distributed in the cross-shore direction and tend to form a sand bar in the location of the breaking waves. This sediment will naturally return to the shoreline and the beach will return to near the original size within a few months. This cross-shore transport pattern is common to coastal environments; with the Southern California Bight, the shoreline recedes in the high-energy winter months and returns during the calmer summer months. If the breakwater is modified, it is expected that the beach width will diminish during the winter months and seasonal variation of the dry beach would increase, as observed before the Long Beach Breakwater construction. Wave drive erosion has typically been a problem near Peninsula Beach, a timber bulkhead was used to protect existing infrastructure during the period of time that the shoreline was directly exposed to incident waves, seen in Figure 7-57.

Figure 7-57: Waves Impacting the Bare Timber Bulkhead along Peninsula Beach in 1940 without the Protection of the Long Beach Breakwater

7.4 PARTICLE TRACKING AND TRANSPORT

Using the same procedure as described in PARTICLE TRACKING, the representative particle duration is calculated and shown in Table 7-3 for the modeled scenarios. Overall, bay-wide currents are only slightly affected by either breakwater modifications or structural changes. Impermeable structures will locally alter the flow field, but the overall wide-scale currents are not greatly modified. Breakwater modifications Output of the results from two release locations, near the Los Angeles River and Belmont Shores are presented in Figure 7-58 and Figure 7-59, respectively, and show the limited variability in path and time due the structural modifications. The largest change in both the time the particle takes to reach the boundary and the path taken is a result of the structural measure of the Los Angeles River training wall. For a further discussion of the numeric tracers, see APPENDIX A-1. EFDC modeling scenarios 1 and 5 are not shown to better compare the changes due to breakwater modification or training wall. The average representative particle durations were provided for habitat evaluation purposes.

Table 7-3: Representative Particle Duration, in Days, for EFDC Modeled Scenarios

	Layer	Area						Average	Sum	Structural Features
		A	B	C	D	E	F			
Existing Conditions	Bottom	8.73	1.55	1.02	0.55	1.24	1.26	2.39	14.34	N/A
	Surface	0.34	0.99	3.64	0.36	0.86	0.88	1.18	7.07	
	Average	4.53	1.27	2.33	0.45	1.05	1.07	1.78	10.71	
Scenario 2	Bottom	8.73	1.55	0.35	0.66	1.21	1.25	2.29	13.75	2x Eastern Notches
	Surface	0.31	0.32	3.09	0.36	1.08	0.74	0.98	5.89	
	Average	4.52	0.94	1.72	0.51	1.14	0.99	1.64	9.82	
Scenario 3	Bottom	8.73	4.5	0.85	0.55	1.14	1.24	2.84	17.02	Eastern Removal
	Surface	0.3	0.89	3.0	1.38	0.88	0.54	1.16	6.98	
	Average	4.51	2.69	1.92	0.97	1.01	0.89	2.0	12	
Scenario 4	Bottom	8.73	2.68	0.65	0.56	1.4	1.28	2.55	15.29	LAR Training Wall, Emergent Islands
	Surface	0.28	0.93	2.86	0.36	0.54	1.01	0.99	5.97	
	Average	4.5	1.8	1.75	0.46	0.97	1.15	1.77	10.63	
Scenario 6	Bottom	7.46	1.82	2.92	0.52	1.5	1.19	2.57	15.42	Complete Breakwater Lowering
	Surface	0.62	0.88	0.3	0.41	1.08	1.02	0.72	4.32	
	Average	4.04	1.35	1.61	0.47	1.29	1.11	1.65	9.87	

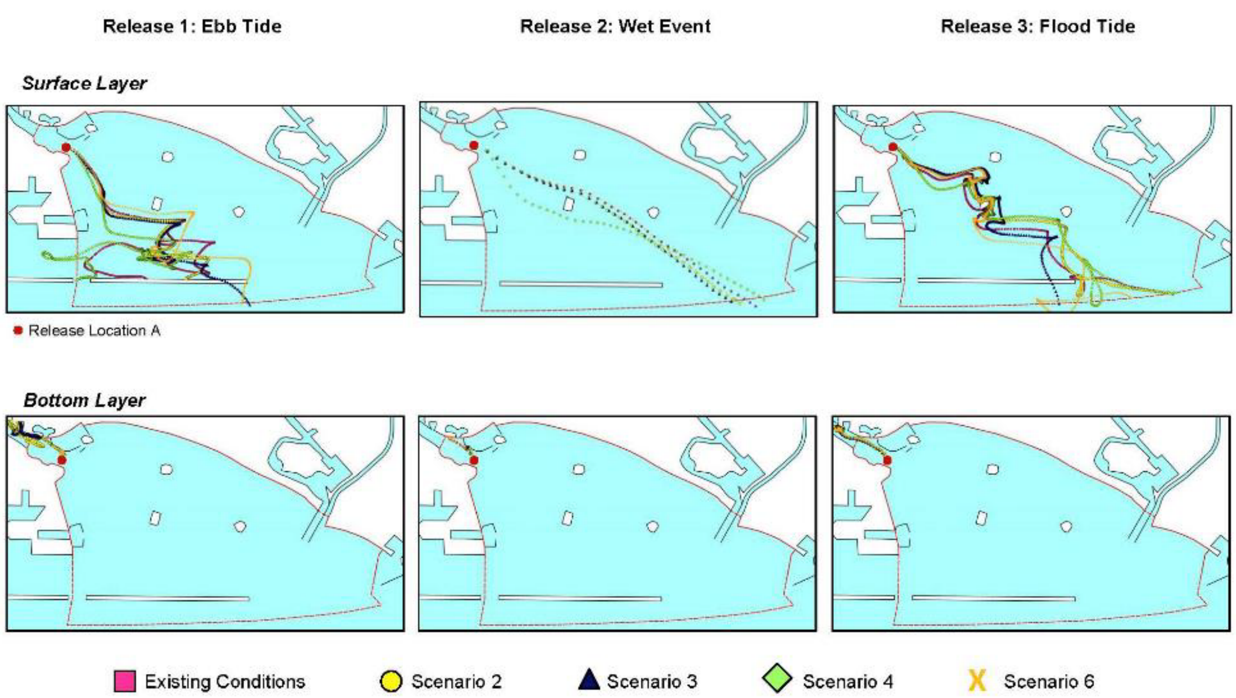


Figure 7-58: Path of Tracers Released near the Los Angeles River

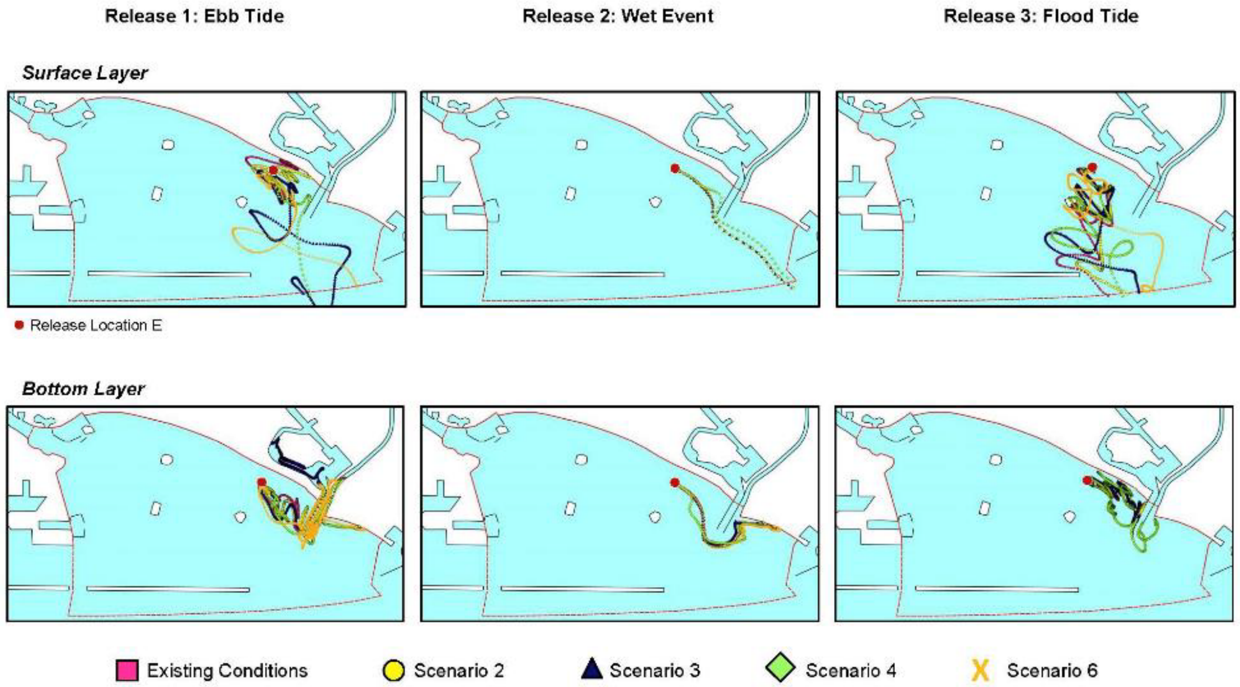


Figure 7-59: Path of Tracers Release near Belmont Shore

Based on the sample particle tracing results, dredging operations performed within the Port Complex or the bay at large will not impact this project or the project features. Flow during most conditions cause transport out of the bay and does not significantly linger within the footprint of the restoration measures.

7.5 WAVE RUNUP

7.5.1 SHORELINE

Runup along the shoreline is calculated using the same analysis as in the previous WAVE RUNUP section. Results are shown in Figure 7-60 to Figure 7-63 for each breakwater modification at each location previously described. For each wave event, the runup along the shoreline is not significantly different than current conditions for most breakwater modifications. Only extensive modifications, such as the complete lowering or removal of a third section, cause the runup elevations to increase substantially. For the small modifications, the increase of energy quickly spreads to redistribute laterally.

Figure 7-60 show the 2% runup elevations near Peninsula Beach. For this location, only the eastern third removal, eastern notching and entire lowering modification produce increases in runup elevations on the order of 1 – 3 feet. The western third removal case produces a slight increase to runup elevation during the less frequent northwest events. Overall, Peninsula Beach will be most susceptible to modifications on the eastern side of the breakwater.

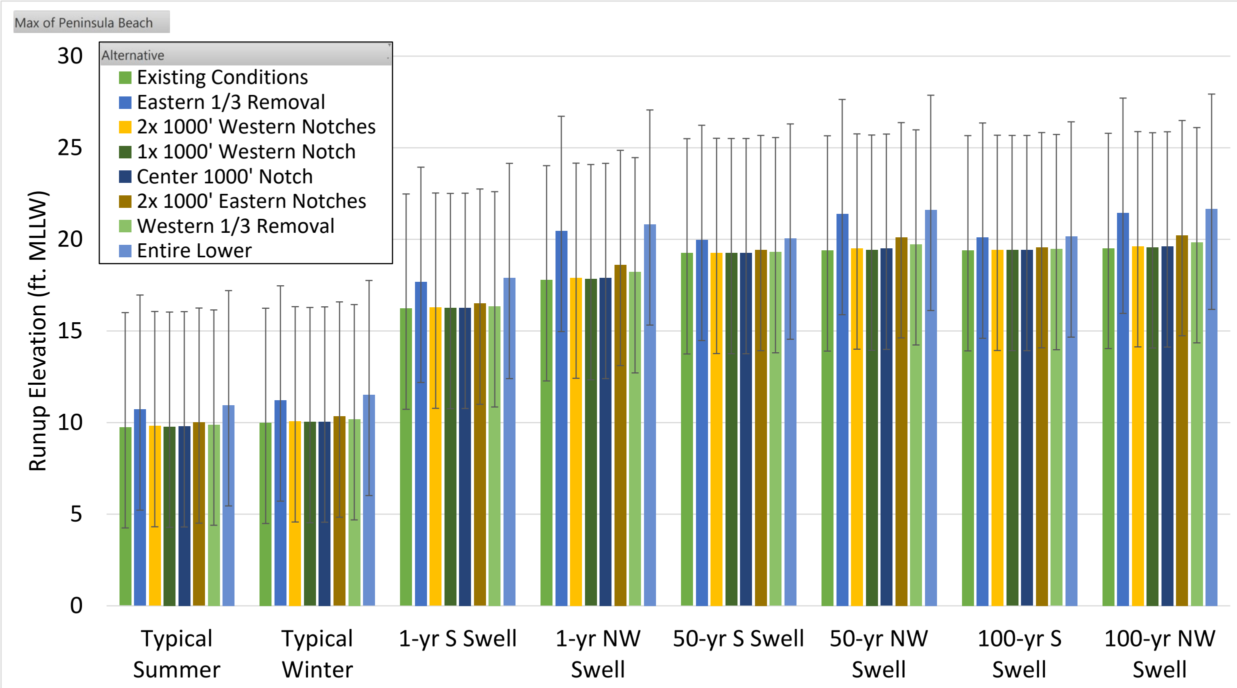


Figure 7-60: 2% Runup Elevation along Peninsula Beach
Bars Show Elevation At MHHW And Error Limits Show Elevation At MLLW (Minimum) And High Sea Level Rise (Maximum) Water Levels

Progressing west from Peninsula Beach, the Belmont Shore, shown in Figure 7-61, currently experiences a lower runup elevation due to the protection provided by the current breakwater. Since this area projects more into center of the project area, western breakwater modifications will begin to have a larger effect on the runup elevations than areas to the east. The small notches in the breakwater do not have a significant effect on altering the runup elevations along the shoreline (i.e. notching); only the large modifications (i.e. lowering or removal) drastically increase the runup.

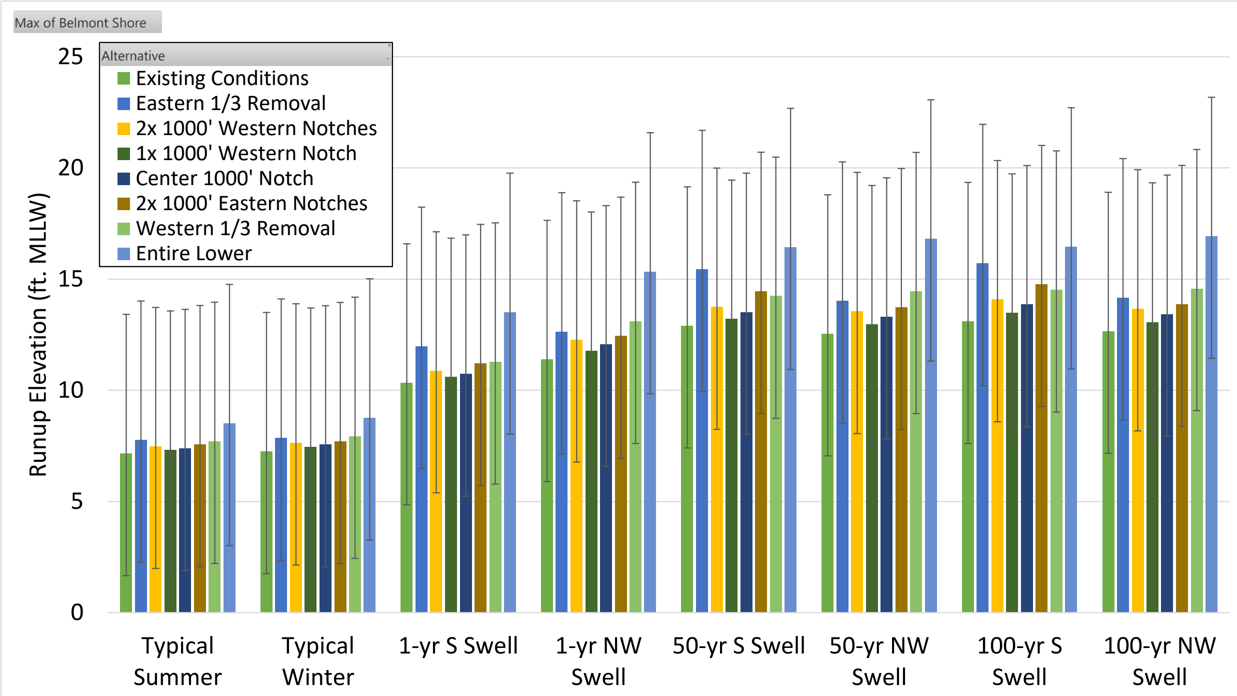


Figure 7-61: 2% Runup Elevation along Belmont Shore

Figure 7-62 shows the runup elevation near Belmont Pier due to the various breakwater modifications. This pattern of increases similar to the previous location along Belmont Shore, but with overall lower runup elevations due to the sheltering of the existing breakwater and energy islands.

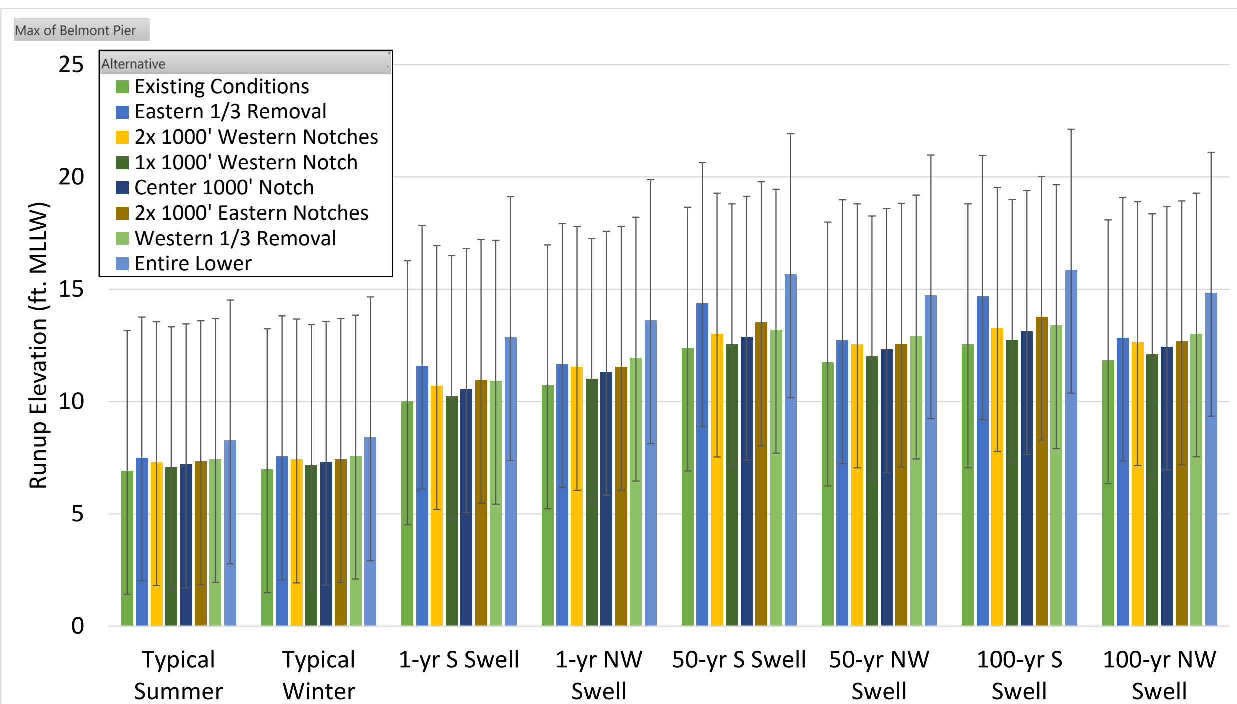


Figure 7-62: 2% Runup Elevation near Belmont Pier

The runup near Junipero Beach is shown in Figure 7-63. This area is in the western side of the project area, so is more susceptible to the western modifications. Besides removal of the western third or lowering the entire breakwater, other modifications do not significantly increase the runup elevations. The western notching increases the runup elevation, but no more than a 1 ft. increase during any swell event.

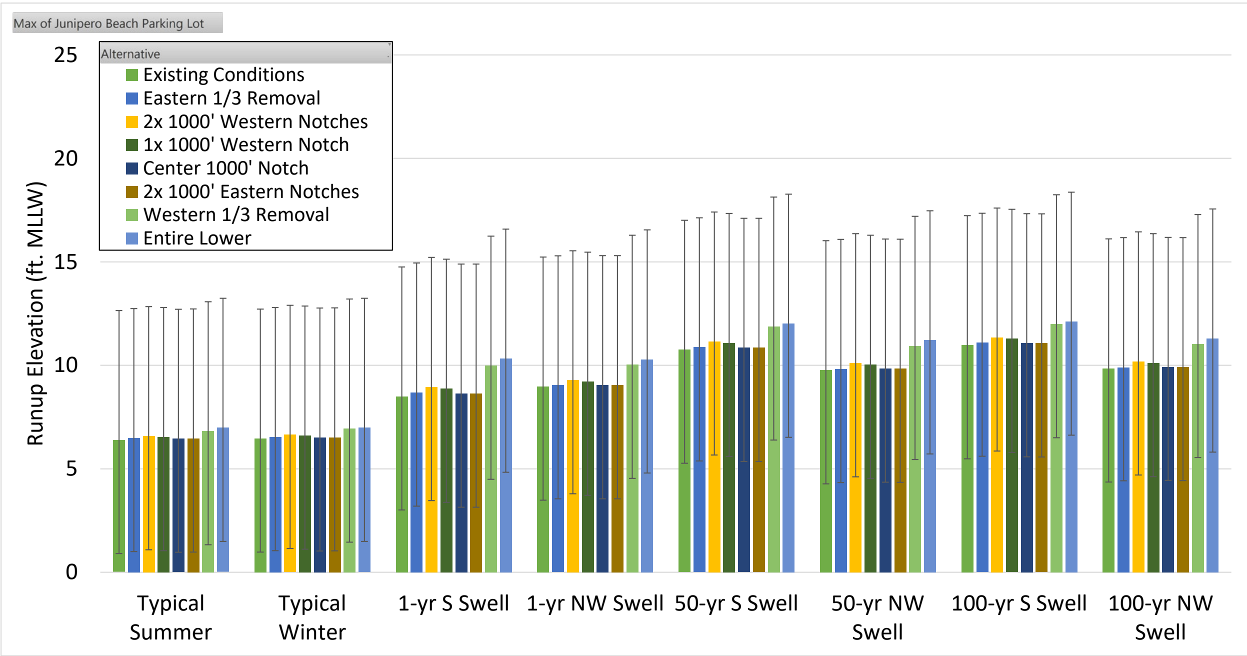


Figure 7-63: 2% Runup Elevation near Junipero Beach Parking Lot

7.5.2 <u>OFFSHORE ENERGY ISLANDS</u>

Unlike at the shoreline, where the waves have time to diffract and spread energy laterally to adjacent areas, the offshore energy islands are in closer to the breakwater, the incident waves do not have time to diffract so most of the energy impacts the islands. Figure 7-64 to Figure 7-67 show the 2% runup elevation at the energy islands.

At Freeman Island, shown in Figure 7-64, is the most offshore island, but is entirely sheltered by the existing breakwater. Only modifications on the western side will have a significant effect on the total runup on this structure. Also, the port complex provides sheltering from large northwest swells; only southern swells produce the extremely large runup elevations that would require protective measures to be built.

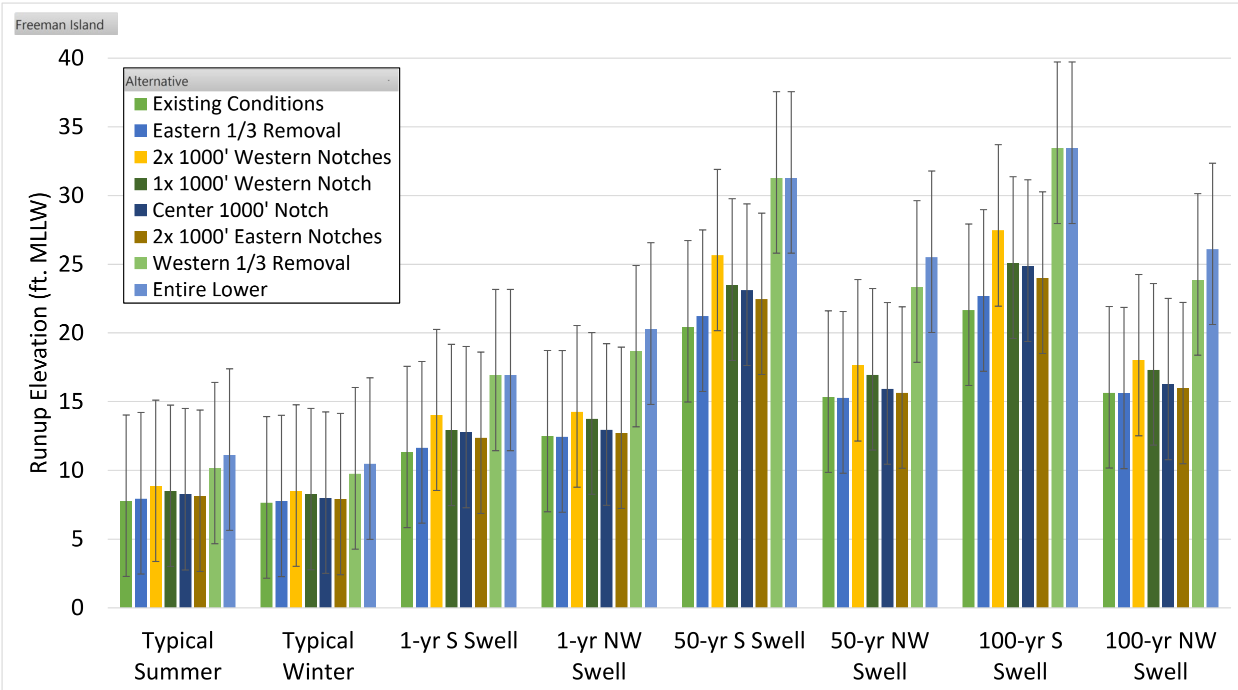


Figure 7-64: 2% Runup Elevation for Freeman Island
Bars show elevation at MHHW and error limits show elevation at MLLW (minimum) and high sea level rise (maximum) water levels.

Chaffee Island is on the east side of the project area and is sheltered less from the port complex. Calculated runup elevations can be seen in Figure 7-65. The largest elevations correspond to the eastern removal or complete lowering of the breakwater. Smaller, but still significant, runup elevations occur with the notching of the eastern side of the breakwater.

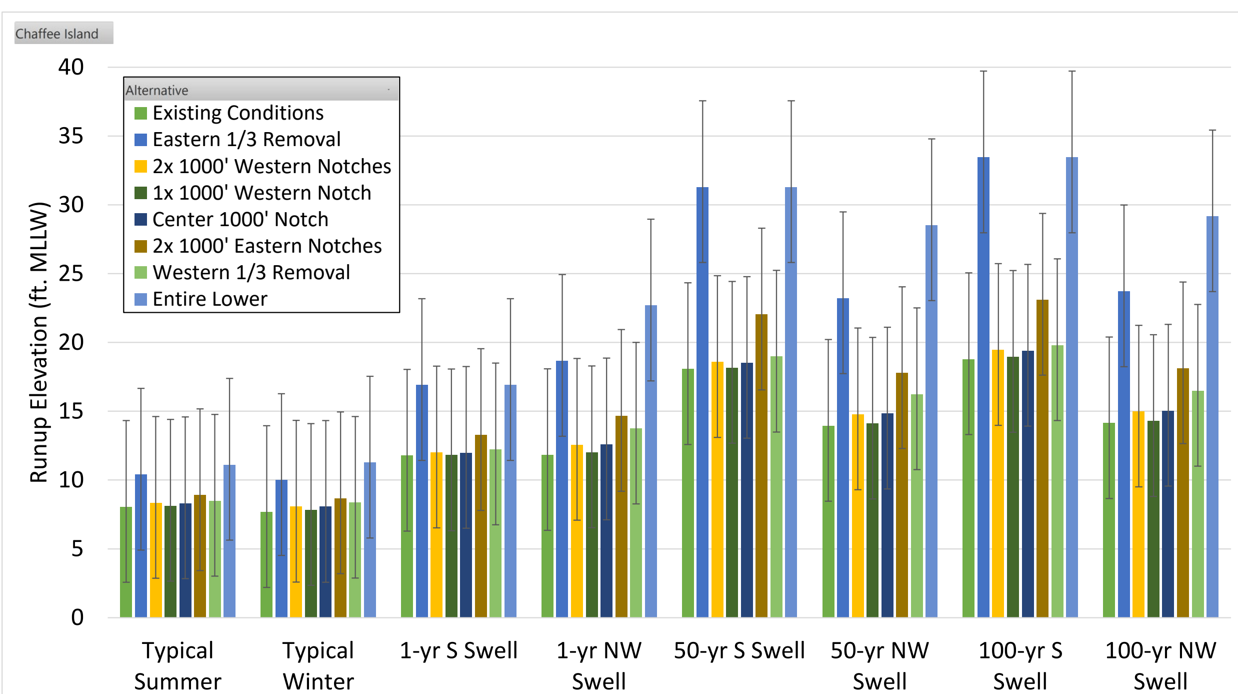


Figure 7-65: 2% Runup Elevation for Chaffee Island

Figure 7-66 shows the 2% runup elevations for White Island. This island is sheltered by Freeman Island, almost directly south, so any southern swell energy is reduced even without the protection afforded by the breakwater. Due to the location of the island, western modifications significantly affect the runup elevation along this island.

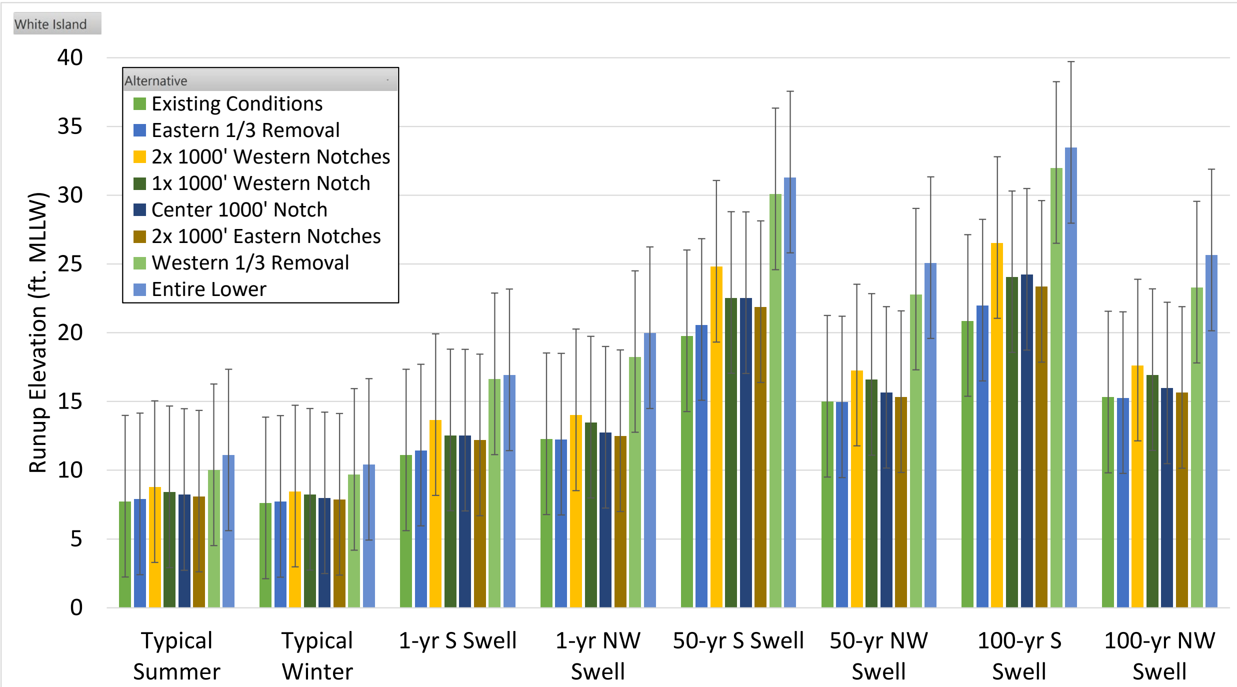


Figure 7-66: 2% Runup Elevations for White Island

Grissom Island is the most sheltered of all the offshore energy islands Runup elevations are shown in Figure 7-67. Only modifications to the western side changes the magnitude of the runup. Southern swells are generally sheltered by various port features and the energy islands regardless of the breakwater configuration.

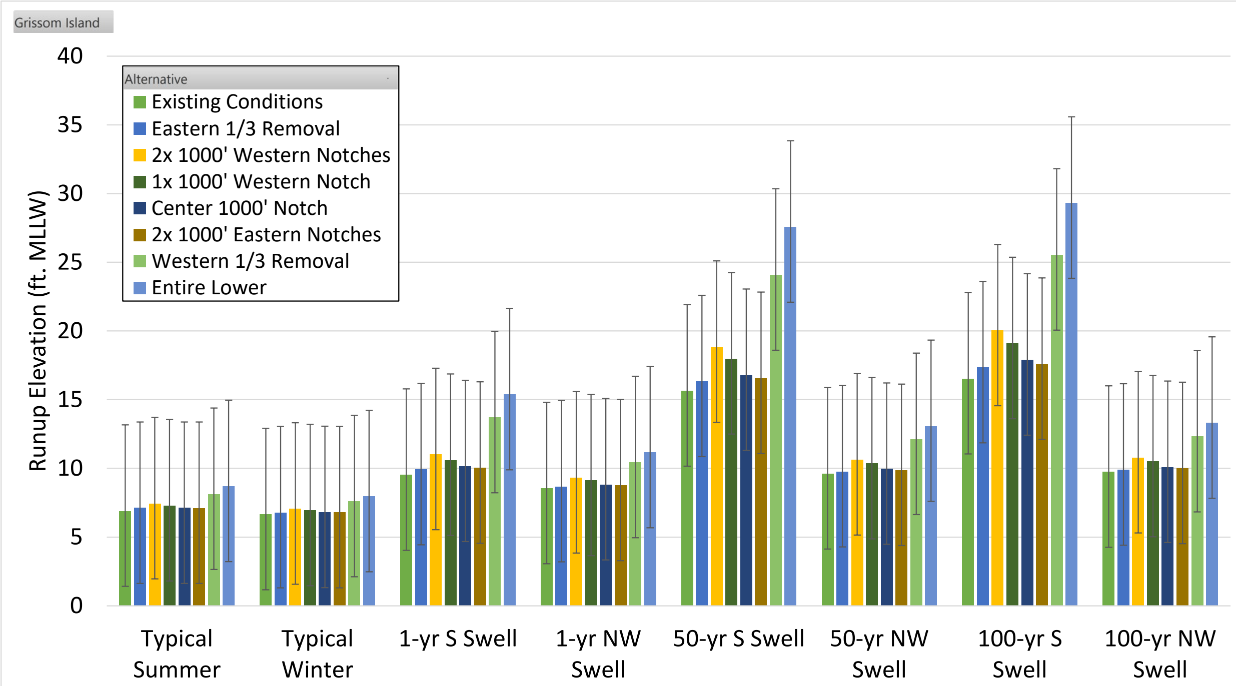


Figure 7-67: 2% Runup Elevations for Grissom Island

7.5.3 PORT COMPLEX FEATURES

For the two port complex features, Pier J South and the Shoreline Marina Detached Breakwater, shown in Figure 7-68 and Figure 7-69, runup elevations increase significantly for the western removal and lowering of the entire breakwater. Once again, these features are sheltered by the port complex, so the large northwest events do not produce the largest runup elevations. Waves coming from the south, basically un-interrupted, would cause runup elevations to be increased by more than 5 feet for the complete lowering and removal of the western third of the breakwater.

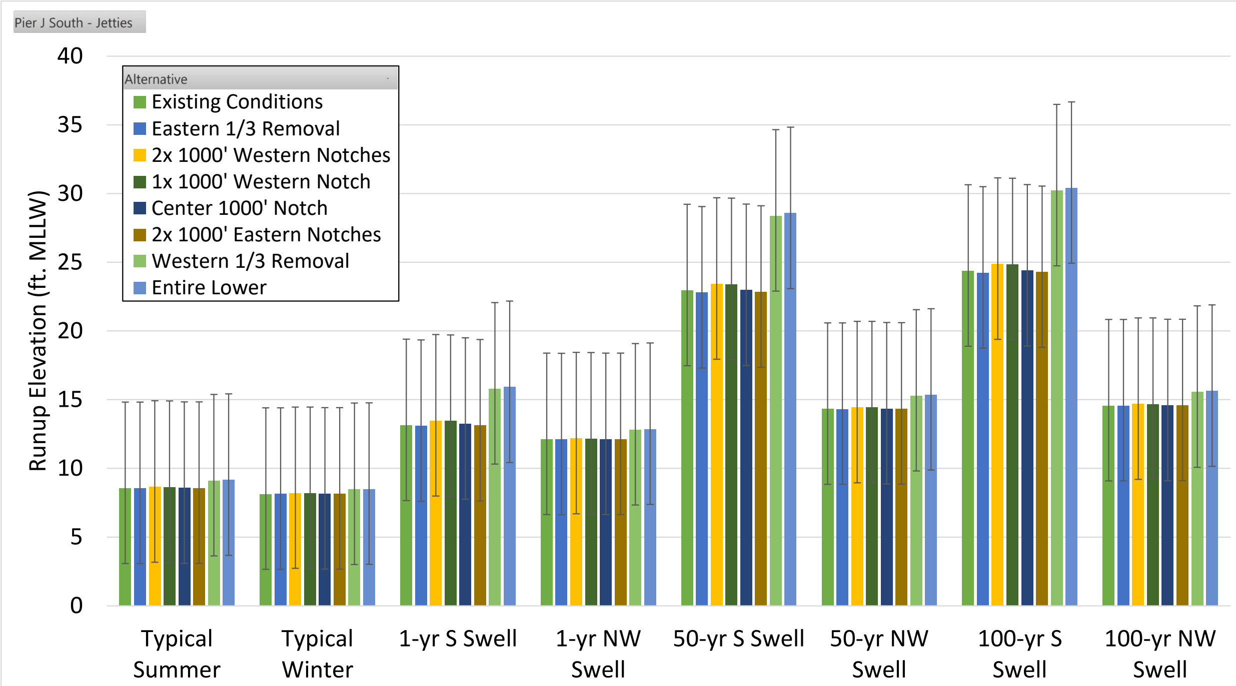


Figure 7-68: 2% Runup Elevations for the Jetties at Pier J South

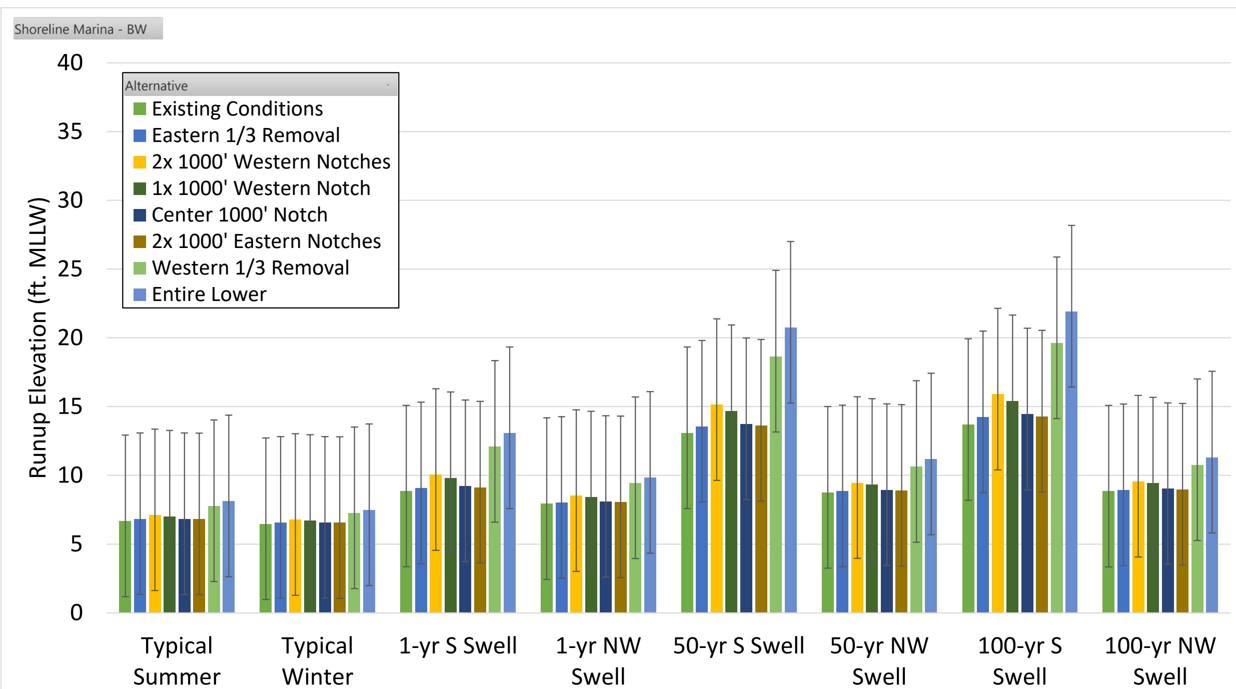


Figure 7-69: 2% Runup Elevations at the Detached Breakwater of Shoreline Marina

8 IMPACTS TO LOCAL OPERATIONS

8.1 EXISTING CONDITIONS

Figure 9-1 shows a heat map of vessel traffic in the proposed project area that has been filtered to show only the areas with many vessel movements. This data was obtained from the Automated Information System (AIS) and contains all recorded movements during 2015. The AIS system is required on all commercial vessels but not all recreational craft. Major vessel traffic is confined to the navigation channels or an anchorage.

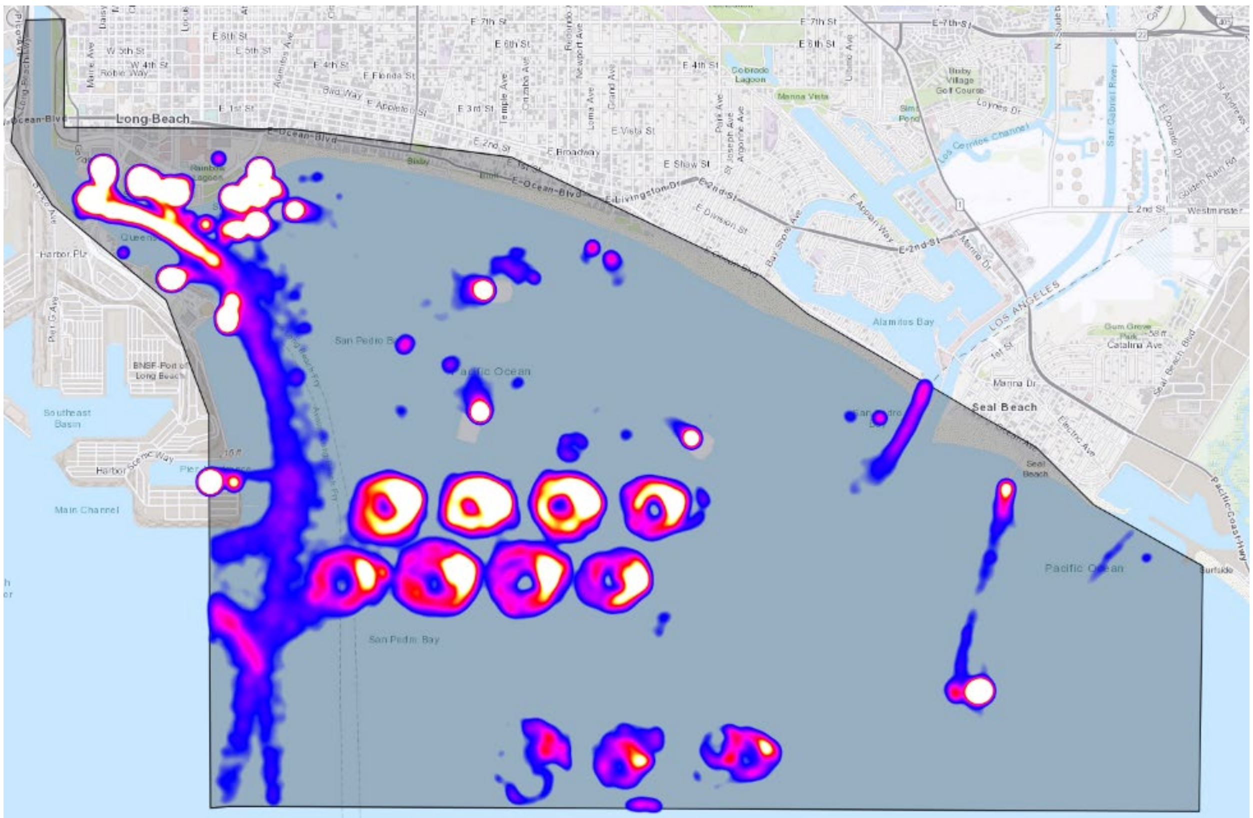


Figure 8-1: Vessel Tracking Heat Map for East San Pedro Bay in 2015

8.1.1 BACKGROUND INFORMATION AND TERMINOLOGY

When considering impact to navigation or port operations, there are a few concepts that are extremely important and will be briefly discussed.

A floating vessel, whether powered or un-powered, has six degrees of motion, as illustrated Figure 8-2.

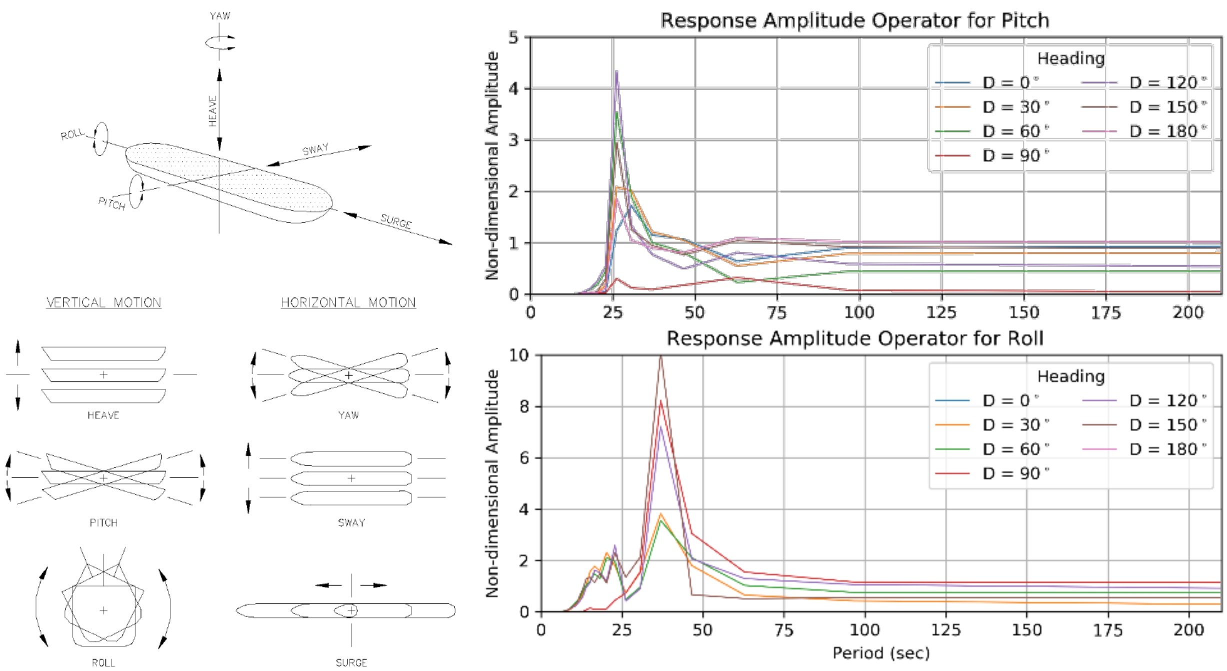


Figure 8-2: Ship Motion Definition and Sample RAO's for a Large Container Vessel

To determine the degree of ship motion, a Response Amplitude Operator (RAO) can be developed using the ship geometry and describes how a vessel responds to a specific wave period. A sample RAO for a large container vessel (as expected to call to Pier J South) traveling at 1 knot is shown in Figure 8-2. The directional convention is in relation to the wave direction and is 0° when transiting in the same direction as the wave propagation and 180° heading directly into the wave. For this type of vessel, waves with a period greater than 20 seconds will have a significant impact on ship motions. Vessel roll caused by typical wind waves can be in excess of double the wave height at periods of 12 – 18 seconds. Other vessels (even of the same type) may have drastically different RAO spectrums and will react differently to the same incident wave condition.

8.1.2 PORT FACILITIES AND OPERATIONS

Based on preliminary feedback from stakeholders within the port complex, the following is assumed to be the present conditions relating to port operations and the impacts from large wave events.

Pier J South:

The terminal is susceptible to southern swell events. As the wave heights increase, the time to unload the vessel at berth greatly increases and damages to the wharf face and safety of workers becomes may occur. Mooring lines will continually break and excessive ship motions will cause loading operations to halt when wave heights are larger than 4 feet, locally. The terminal currently experiences 8 – 12 cases per year with waves over 4 ft. (Ferrigno, 2018).

Carnival Cruise Lines

Like Pier J South, this terminal is also susceptible to southern swells. Excessive waves cause conditions that make it unsafe for berthing operations approximately 2 times per year (Wilkins, 2016).

Energy Islands

Transfer of crew and materials occurs daily between the main port complex and the offshore energy islands. Currently, operations are impacted approximately 5 days per year by waves in excess of 3 ft. (Tougas, 2018).

Pilots, Tugs and Anchorages

The port pilots and tug operators guide vessels into the port complex then to berth or an open anchorage. Along with excessive ship motions that cause safety issues, underkeel clearance (UKC) is also of concern. Most of the container vessels that call to Pier J South only have an UKC of only 3 feet. Since the vessels are incredibly long and wide, increase to wave heights may produce a 2° roll that will increase the draft by 3 ft. causing the vessel to impact the bottom (Jacobson, 2018).

8.1.3 U.S. NAVY

The Navy uses the protection provided by the breakwater for loading ordinance at the D-7 and D-8 anchorages near the eastern side of the Long Beach Breakwater. No information is provided about the number or duration of use, or if/when the site is impacted by large wave events due to operational security measures (10 U.S.C. §130e). According to UFC 4-159-03, safe tolerance for this type of loading/unloading require relative vessel motions to be less than 2 feet which can easily be caused by wave heights no larger than 1 ft.

8.1.4 LONG BEACH SHORELINE BACKPASSING

As described in the NOURISHMENT/OPERATIONAL ACTIVITIES section, the city of Long Beach conducts sediment backpassing to counter the natural westward transport. Approximately 70,000 yd^3/yr is transported to the east to protect the infrastructure near east beach.

8.1.5 RECREATION

Various recreational activities are currently enjoyed within the bay. For further description of recreation impacts, see APPENDIX C, ECONOMICS. This appendix will only focus on the breaking waves and the direct effects on recreation. The breaking wave location and intensity was extracted from the CMS-Wave output and is calculated using the Extended Goda Formula (USACE, 2008).

Figure 8-3 shows this breaking wave dissipation rate but can be thought of as both where the wave breaks and with what intensity. The colors correspond to the amount of energy that is dissipated through the breaking process. Note that the small dissipation terms are removed since the surface rollers or white-capping in deeper water does not play an important role in the surf zone location and width. A more detailed analysis on wave breaking will not be discussed for this study but will be conducted during the PED phase of the project.

For the existing breakwater configuration, breaking waves are generally limited to the eastern side of the project area, near Peninsula Beach and Belmont Shore. During larger events, such as a 50 year southern swell, enough wave energy is transmitted through and over the breakwaters to allow the energy to re-form into waves that break on the shoreline. Figure 8-4 presents the less-filtered results for a typical winter condition. Note the dissipation occurs along the entire shoreline, even in the low wave case. Approximate surf zone sizes range from 5,000 to 20,000 ft. in the longshore direction and from 200 to 400 ft. in the cross-shore direction.

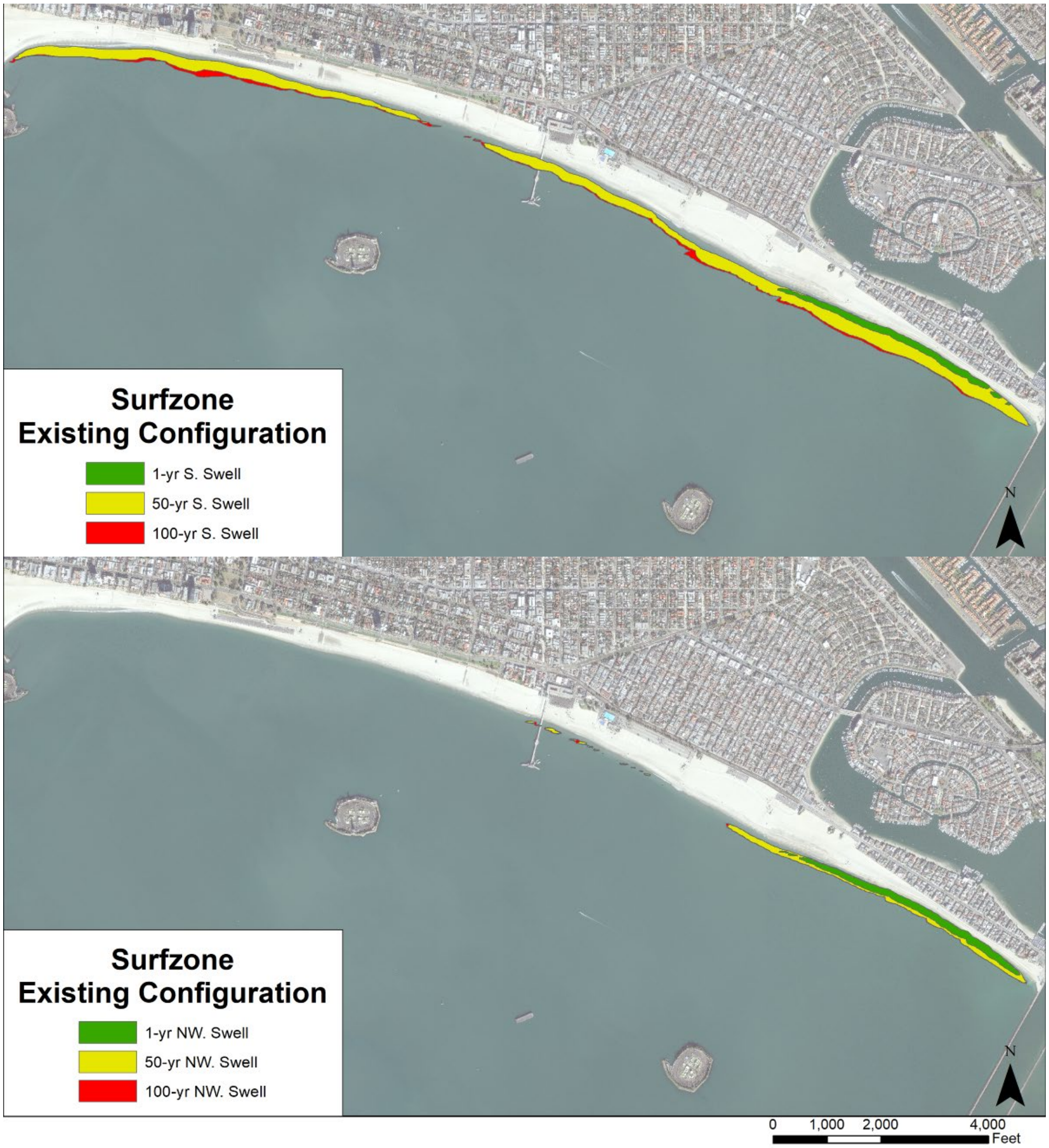


Figure 8-3: Surf Zone Extents Caused by Southern (Top) and Northwestern (Bottom) Storm Events due to Existing Breakwater Configuration

Figure 8-4: Un-Filtered Energy Dissipation for Typical Winter Conditions and Existing Breakwater Configuration

8.2 RESTORATION MEASURES

General restoration measures do not include breakwater modifications. There may be minor impacts regarding these measures, as described below.

8.2.1 PORT FACILITIES AND OPERATIONS

- Eelgrass Beds: These measures are away from commercial navigation. There is no expected impact due to the eelgrass beds.
- Rocky Reefs
    - Nearshore: These measures are away from commercial navigations. There is no expected impact due to the nearshore rocky reefs.
    - Open water: The maximum depth of this features will be -15 ft. MLLW. Commercial traffic to/from the offshore energy islands will need to avoid these locations of high relief. According to the AIS information from 2015, ship traffic rarely transits across the tentative placement locations. Operations should be modified if needed to avoid these measures and should not increase time and cost of operations due to the potential minimal alteration. Impacts from the offshore reefs on the nearby mooring areas is minimal is similar to the existence of the energy islands; a small portion (~10 %) of the wave energy is reflected by the perturbation and may interact with vessels in the mooring areas.
- Kelp Reefs: The extensive kelp canopy may interfere with navigation but will only occur during transits between the project area and Anaheim Bay.
- Oyster Reefs: No impacts to port operations are expected with this measure.
- Sandy Islands: Since this structure resides in depths less than 20 ft., no commercial vessel impacts are expected. Breaking waves on the face of the newly created revetment will limit the

amount of energy that is reflected back to deep water. This must be confirmed during the PED phase of the project.

- Tidal Salt Marshes: The presence of this newly created landform will alter the navigation patterns in the vicinity. Reflections from the vertical structure may interact with transiting and moored vessels; future design work would limit these reflections to the most practical extent possible.

8.2.2 U.S. NAVY

- Eelgrass Beds: No impacts expected.
- Rocky Reefs
    - Nearshore: No impacts expected
    - Offshore: Similar to port operations. Reflected energy from the reefs is not expected to be significant.
- Kelp Reefs: Similar to port operations.
- Oyster Reefs: No impacts expected.
- Sandy Islands: Similar to port operations
- Tidal Salt Marshes: Similar to port operations

8.2.3 LONG BEACH SHORELINE BACKPASSING

The only impact on the city's backpassing operation would be the nearshore reefs. Design during the PED phase, effort would be undertaken to optimize the reef complex with the goal of reducing the longshore transport potential. Any reduction in longshore transport could lead to possible reductions in the cost and duration of the current city-run operations. Based on the preliminary modeling discussed in the LONGSHORE TRANSPORT AND GENCADE section, longshore transports rates have the potential to be reduced by roughly 40 percent of current rate as shown in Figure 7-41. City operations would need to be adjusted to account for the differing sedimentation but would reduce overall costs of implementation.

8.2.4 RECREATION

- Eelgrass Beds: No significant impacts.
- Rocky Reefs:
    - Nearshore: Structures will cause some waves to break decreasing the energy impacting the shoreline. Changes in the wave height near the shoreline due to the nearshore reefs are shown in Figure 7-26.
    - Open water: Recreational boaters may need to avoid areas with depths of 15 ft. MLLW.
- Kelp Reefs: Recreational vessels will need to avoid areas with extensive kelp canopy.
- Oyster Reefs: No impacts expected.
- Sandy Islands: The structure will shelter the shoreline and reduce incident wave heights similarly as shown in Figure 7-26. Recreational vessels and other nearshore recreational activities would need to avoid the sandy island. Human activities would need to be limited to ensure proper environmental outcomes.
- Tidal Salt Marshes: Recreational vessels will need to avoid the salt marshes. The perimeter of the marsh could include a walking/biking path and may also incorporate existing plans for a series of fishing piers in the same area.

8.3 BREAKWATER MODIFICATIONS

8.3.1 PORT FACILITIES AND OPERATIONS

Using the information from the previous section provided by the port users, threshold values of wave height, direction and period are established. Using the provided values as calibration factors and the wave modeling results presented in the GROUPED DATA ENCOMPASSING ENTIRE RECORD section, impacts to port operations can be estimated as shown in Table 8-1. Each event is assumed to last for 12 hours and independent of one another. Threshold criteria were provided by the port users, described in the EXISTING CONDITIONS section. Regardless of the actual wave data, a threshold value was calibrated to the existing conditions provided by the user. The threshold value was then applied to the cases with breakwater modifications, giving an approximate number of downtime events.

Table 8-1: Potential Increase In Downtime Due To Breakwater Modifications

Southern Swell Events (T_p > 12 sec, D_p < 245°)						
Location	**Approx. Number of Events/Year**					
	Existing	**Eastern Removal**	**Western Notching**	**Entire Lowering**	**Eastern Notching**	**Western Removal**
Pier J	10	11	18	40	12	40
C.C. Terminal	3	5	34	46	11	46

All Events (T_p > 12 sec)						
Location	**Approx. Number of Events/Year**					
	Existing	**Eastern Removal**	**Western Notching**	**Entire Lowering**	**Eastern Notching**	**Western Removal**
Freeman Island	5	25	28	402	21	113
Chaffee Island	5	253	46	674	73	109
Total Possible Events/Yr:		730				

Any breakwater modification will impact operations within the port complex with the largest negative effect on the offshore energy islands.

8.3.2 U.S. NAVY

Any modification to the breakwater will allow more waves to enter the currently protected bay and affect conditions alee of the breakwater. Figure 8-5 to Figure 8-9 show the increase in number of days where wave heights exceed the specified value due to the potential breakwater modification. Note that these values represent the increase from the existing conditions without any breakwater modification. The increase of wave events larger than 1 and 2 ft. are summarized in Table 8-2 for a single point between the D-7 and D-8 anchorages.

Table 8-2: Increase in Wave Events Near The Navy Mooring Area

Breakwater Modification	Approx. increase of days when wave heights:	
	Exceed 1 ft.	Exceed 2 ft.
EastLop	131	7
EastNotch	224	26
WestLop	52	2
WestNotch	135	5
Low	334	229

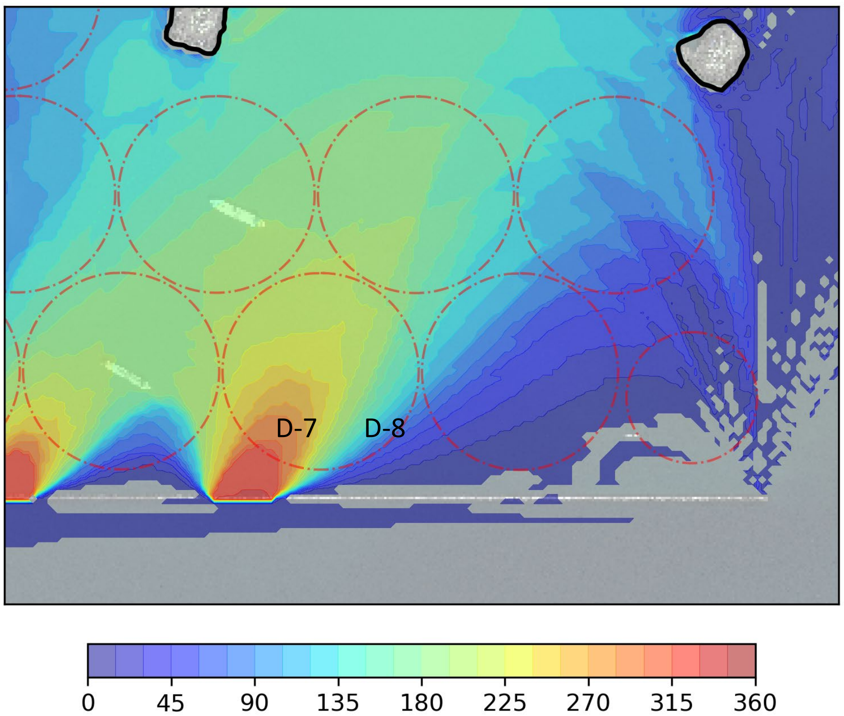


Figure 8-5: Approximate Number of Increased Days with Wave Heights Greater than 1 Ft. Resulting from Western Notching
Current Mooring Areas Are Shown As Red Dash-Dot Line

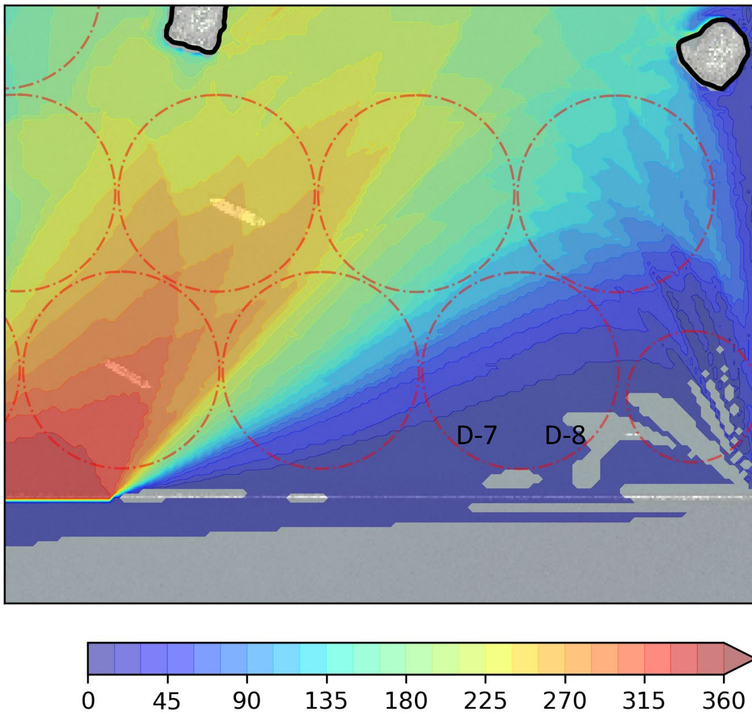


Figure 8-6: Approximate Number of Increased Days with Wave Heights Greater than 1 Ft. Resulting from Western Removal

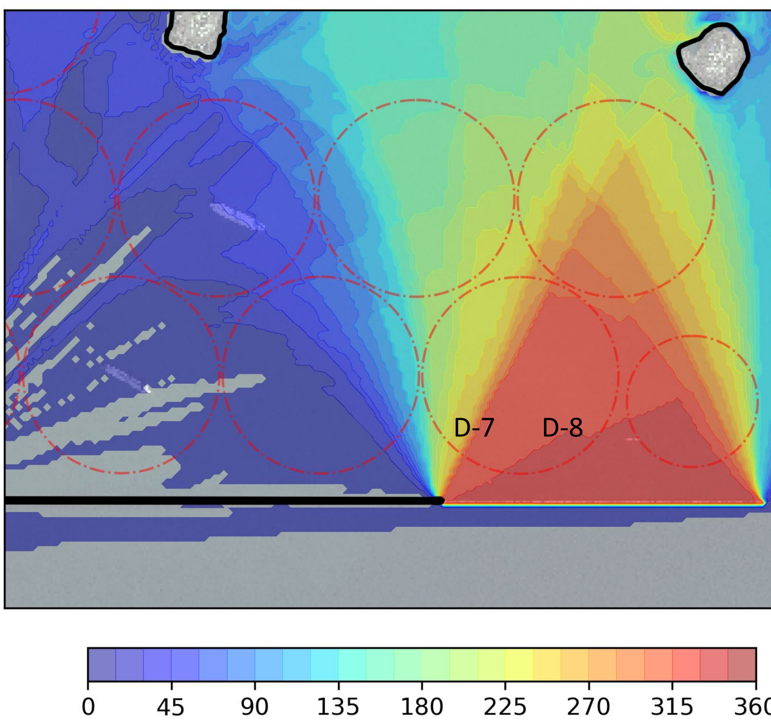


Figure 8-7: Approximate Number of Increased Days with Wave Heights Greater than 1 Ft. Resulting from Eastern Removal

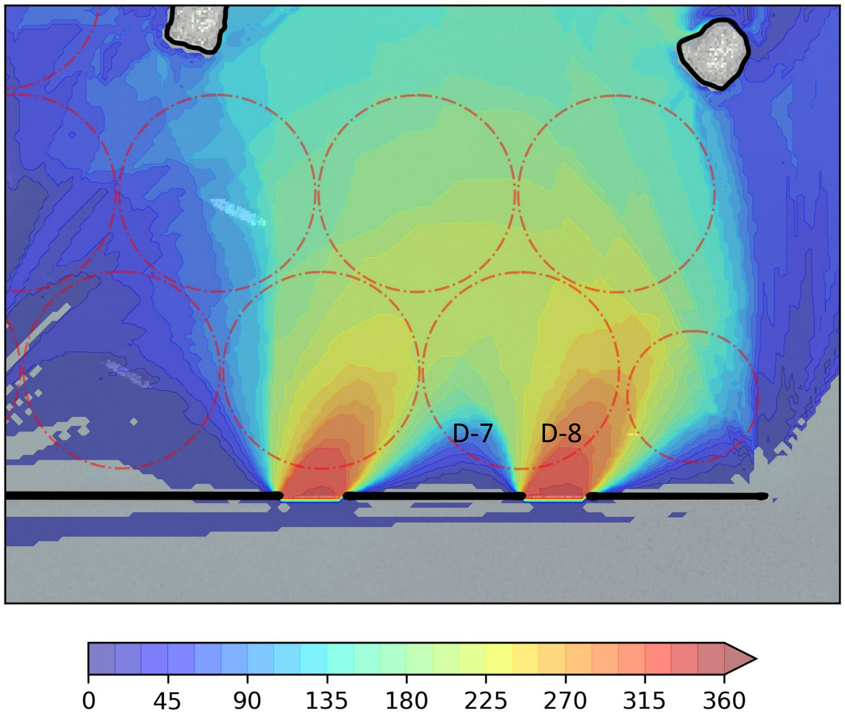


Figure 8-8: Approximate Number of Increased Days with Wave Heights Greater than 1 Ft. Resulting from Eastern Notching

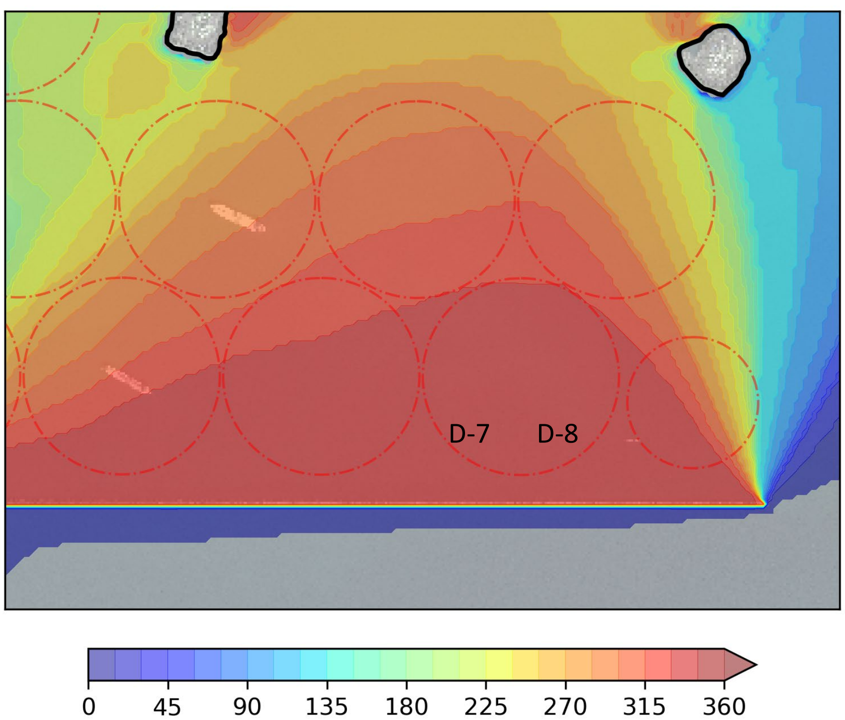


Figure 8-9: Approximate Number of Increased Days with Wave Heights Greater than 1 Ft. Resulting from Complete Lowering

Shifting of the Navy anchorage is possible, but safety concerns and required offset distances from existing infrastructure and people, the shift may be unacceptable for stakeholders. Figure 8-10 shows how the arc would need to be shifted if the eastern removal modification was employed. Movement of the arc would then encompass locations with commercial or recreational activities; these activities would also need to be re-located. For this example, the closure of Queen's Gate, Pier J South approach channel and Freeman Island will be required. Vessels would not be allowed to transit through Queen's Gate effectively closing the Port of Long Beach when the exclusion zone is in effect.

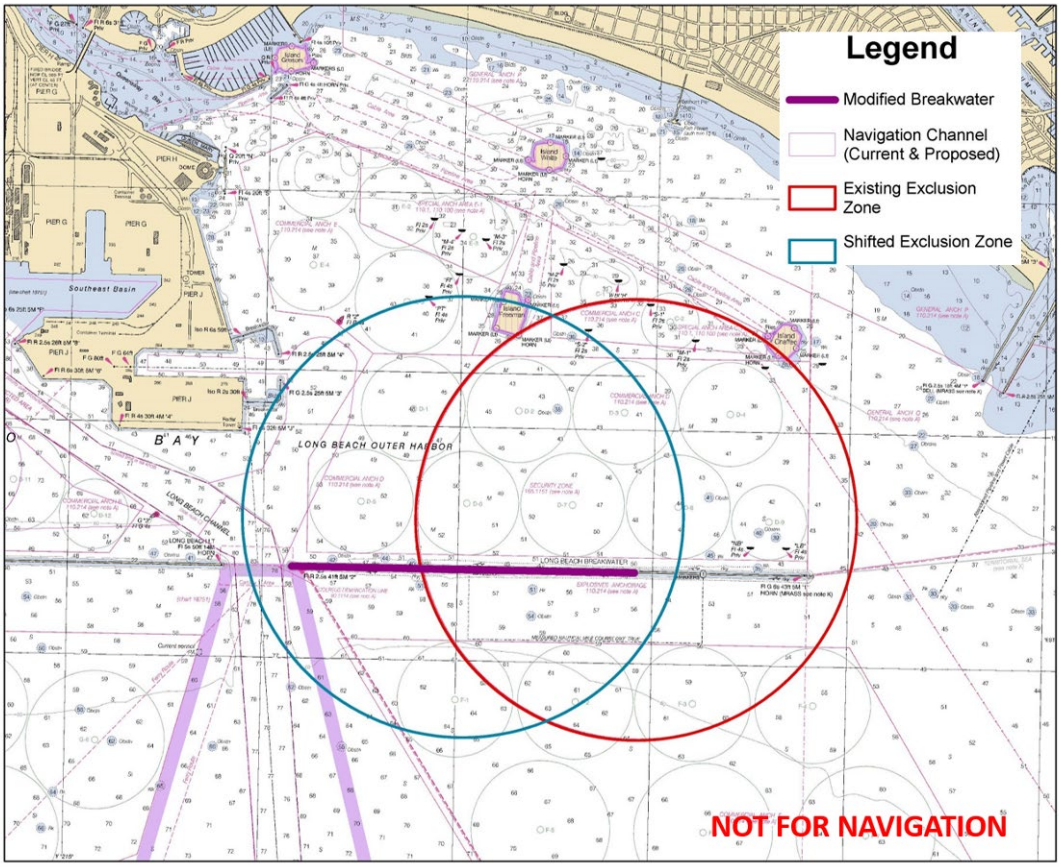


Figure 8-10: Example of Required Exclusion Zone due to Eastern of Breakwater

8.3.2.1 OTHER PORT ANCHORAGES

Besides the Navy's mooring areas inside the bay, other commercial anchorages are present that will also be affected by breakwater modifications. The increase of wave heights within the anchorages will reduce the potential productivity that is provided by the calm waters. The impact is difficult to quantify since the anchorages can still be utilized, only to a less extent. The loss of calm waters would require maintenance work, vessel loading/unloading, fuel transfer, etc. to be postponed until a time where such activities can be performed safely.

8.3.3 LONG BEACH SHORELINE BACKPASSING

If the breakwater is modified, impacts to the city's backpassing operations would be substantial. Since the larger waves impact more of the shoreline, sediments would be transported further west. Even with the increased protection provided by the emergent, nearshore breakwaters, sediments would continue to be eroded from east beach. An increase in both the quantity and the borrow location would lead to additional operational costs and duration; more sediment would need to be backpassed every year and the distance traveled by the equipment would increase.

8.3.4 RECREATION

The anticipated surfzone extent corresponding to the potential breakwater modifications are shown from Figure 8-11 to Figure 8-16. Overall, waves will continue to break along the shoreline. This analysis ignores the bathymetric changes that occur during storm events that may alter the breaking pattern and intensity.

Notching of the breakwater on the western side allows more energy to impact the shoreline towards the west. The small notches in the breakwater only slightly increase the surf zone width but extends the limits to a more westward location during the larger events.

Figure 8-11: Surf Zone Extent for a 1-Year Southern Swell Event

Figure 8-12: Surf Zone Extent for a 1-Year Northwest Swell Event

Figure 8-13: Surf Zone Extent for a 50-Year Southern Swell Event

Figure 8-14: Surf Zone Extent for a 50-Year Northwest Swell Event

Figure 8-15: Surf Zone Extent for a 100-Year Southern Swell Event

Figure 8-16: Surf Zone Extent for a 100-Year Northwest Swell Event

9 PLAN SELECTION

9.1 FINAL ARRAY OF ALTERNATIVES

The final array of alternatives is determined using various screen criteria, model outputs and economic/environmental analysis. All the final alternatives consist of various restoration alternatives that has already been discussed. For more information of these alternatives, see the MAIN REPORT and APPENDIX D. The following sections summarize the affects due to each alternative plan. Final engineering design during the PED phase of the project would include a physical model to properly capture parameters that are difficult to describe with numerical models.

9.1.1 ALTERNATIVE 1 – NO ACTION ALTERNATIVE

The results of the no action alternative would be similar to results described in the DESIGN CONDITIONS and NUMERIC MODELS AND RESULTS chapters.

Sea level rise is expected to affect the project area as a whole. The rise in water levels will increase magnitudes and returns of overtopping events on structures and facilities throughout the bay. The sandy shoreline of Long Beach is expected to erode at a certain magnitude commensurate with sea level rise as discussed in the BRUUN RULE section.

9.1.2 ALTERNATIVE 2 – KELP RESTORATION ALTERNATIVE

Alternative 2, shown on Sheet CE101, contains a series of nearshore rocky reefs with placement beginning near Alamitos Bay progressing to the west. Each reef is approximately 1,000 ft. by 175 ft. in the longshore and cross-shore direction, respectively. Exact reef spacing and size will be determined during the detailed design phase; initial sitting is as indicated in CE101. Crest elevations of the more than 4,500 linear feet of reef structure will decrease from -3 ft. MLLW in the east to -10 ft. MLLW in the west with variable spacing ranging from 500 to greater than 1,000 ft. Eelgrass beds will be situated in the lee of the nearshore reefs. Sand will be imported from the Surfside/Sunset borrow area to allow for the formation of the perched shoreline. Other opportunistic material can be utilized but may require a re-analysis of the environmental considerations. Open water kelp reefs will be placed in locations with water depths greater than 30 ft. MLLW as a single layer of placed stone with a maximum perturbation off the seafloor of no more than 3 ft. Total projected acreages of restoration areas is shown in Table 9-1 and material quantities shown in Table 9-2.

Table 9-1: Alternative 2 Restoration Areas

Final Array Alternative	Total Area (ac)
Alternative 2	162.26
Eelgrass	25.01
Kelp	121.38
Nearshore Reef	15.87

The final design phase for this alternative will take approximately 1 year to complete. A physical model is required and would guide exact placement and orientation of the structures and to ensure no unidentified impacts are excluded that may cause impacts on existing infrastructure and marine operations.

Table 9-2: Approximate Quantities for Alternative 2

Measure	Material Type	Approximate Quantity	Unit	Representative Size
Nearshore Reefs	Armor Stone	137,000	tons	1 - 10 tons
	Filter Stone	55,000	tons	~ 1 ton
	Core Stone	120,000	tons	~ 10 - 1000 lbs
Kelp Reefs	Quarry Stone	132,000	tons	500 lbs
Eelgrass	Sand	100,000	yd^3	0.2 mm

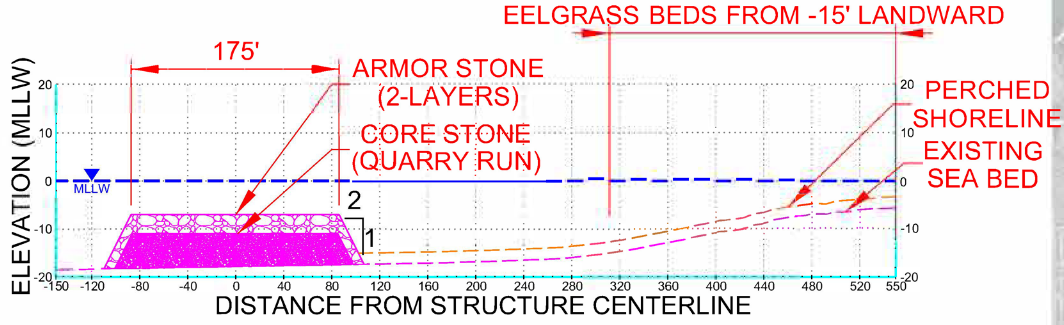
Construction materials will be transported from a production site to project site by barge or by truck. Loaded barges can be stored within the port complex's mooring areas until needed for construction. Materials transported by truck will be offloaded at the staging and storage area within the port and loaded onto an awaiting barge. For typical rubble-mound structures, 1-2 loaded barges are held in stand-by, usually moored close by for easy access. Sand material, if obtained from the Surfside/Sunset borrow area, would be mechanically dredged, placed into a split-haul scow, transported to the project site then deposited by opening the scow. Additional information on dredging can be found in the RECOMMENDED PLAN section. During placement of all restoration measures, project limits will be established by GPS coordinates and marked by buoys in-place before the start of construction. A local notice to mariners will be announced to ensure all affected parties are aware of the increased traffic and potential local changes to navigation patterns. Construction techniques will be similar as those described in the PLAN FORMULATION AND DESCRIPTION OF ALTERNATIVES chapter. Required equipment will include a derrick crane/barge, hydraulic dredge, scows, tug boats, various other crew boats and earthmoving equipment such as a frontloader or dozer.

Due to the large quantity of armor and quarry stone, production to build-up and maintain a stockpile must begin at least 6 months in advance of planned construction and continue until the required quantity of stone has been quarried. It is anticipated that the nearby Pebbly Beach Quarry on Catalina Island will be used for all quarry stone materials and transportation to the project site will be conducted by barge and tug. Navigation obstacles, such as the submerged nearshore reefs, would be marked by a surface penetrating navigation aids that would warn of the submerged reefs. Collaboration with the U.S. Coast Guard is required for the establishment of navigation aids and updates to the applicable navigation charts. The duration for the construction of all the restoration measures would take approximately 30 months to complete. For a further discussion on construction durations, see Appendix B, Cost Engineering. Work is assumed to progress concurrently in each of the two restoration zones.

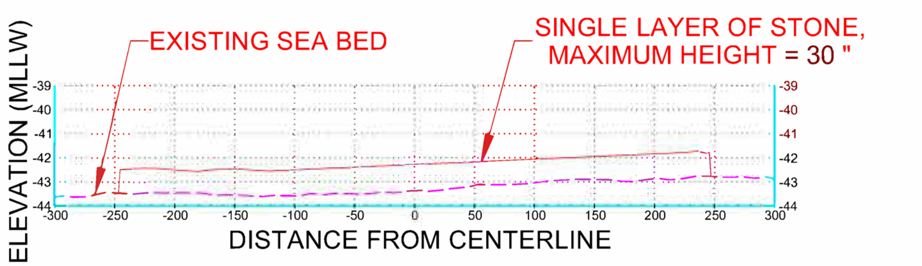
Limited maintenance is expected and only applies to the nearshore rocky reef structures. It is expected that maintenance will be conducted every 10 years or as needed after large storm events to restore the structure to the design parameters. Kelp reefs will not require maintenance; individual stones have the potential to mobilize during the extremely rare events but will not limit the potential to grow kelp. Adaptive management (MAMP) may be required for the eelgrass beds, but long-term maintenance is not expected. This will be accomplished by modifying the crest elevations of the nearshore reef complex to control sedimentation. Additional maintenance by removing sediment covering the eelgrass beds is infeasible since this would cause damage to the organisms and would require additional mitigation. A short-term loss in eelgrass coverage is expected after a large storm event but will return to the design conditions during the long duration of low wave energy.

As a result of the placement of the submerged reefs, wave breaking location and magnitude will be altered. The submerged structures will cause some waves to break and loose energy before impacting the shoreline. This reduction in energy will also reduce the potential quantity of sediment that is transported by the wave generated longshore current. Sediment transport patterns will be modified by the structures, creating a more undulating shoreline than the existing condition, as shown in FIGURE 7-38. Changes to the nearshore wave heights are shown in FIGURE 7-26. Wave reflections from the structures in on the order of 10% of the incident wave height. Although run-up on the shoreline in the lee of the submerged reefs is expected to slightly decrease, the exact magnitude has not been quantified in this analysis. Increasing water surface elevations, caused by sea level change, will cause the submerged reefs to be less effective in breaking the wave energy. The submerged reefs will offer the shoreline more protection from the incident wave energy than is currently in-place. Current back-passing activities are assumed to continue as currently conducted by the City of Long Beach with a minimal reduction in quantity and duration. No impacts are expected to port operations or infrastructure as a result of Alternative 2. Impacts to the Navy is limited to the existence of a new kelp reef between Anaheim Bay and D-7 & D-8 anchorages currently used. During transits between the two locations, the kelp canopy should be avoided. Exact placement locations will be coordinated with the U.S. Navy and other affected stakeholders but will remain in the general area as shown on Sheet CE101 in water depths greater than 30 ft. MLLW. The nearshore reefs and eelgrass beds will affect the location and magnitude of wave breaking. The submerged structures will cause some waves to break and loose energy before impacting the shoreline. Overall there will be no increase in wave run-up or overtopping along the shoreline, offshore energy islands or other port features.

Sea level rise will affect the bay as a whole in a similar manner than as the no-action alternative. Additional effects would cause the nearshore reefs and eelgrass beds to become less productive. Both measures will need to be adapted to the new water surface elevations as future sea level rise is fully realized. Additional stone and sediment is required for these adaptive maintenance measures. The nearshore reefs have a footprint large enough to support the additional stone required to raise the elevation. During the PED phase, the foundation thickness and extent will also be designed to incorporate this addition of stone. Open water and kelp reefs are not sensitive to the additional 2.5 feet (maximum) of potential water.



F1 NEARSHORE ROCKY REEF & EELGRASS (3) CROSS-SECTION
HORIZONTAL SCALE = 1" = 160'
VERTICAL SCALE: 1" = 40'
H: 0 80' 160'
V: 0 20' 40'



D1 OFFSHORE KELP REEF CROSS-SECTION
HORIZONTAL SCALE: 1" = 160'
VERTICAL SCALE: 1" = 8'
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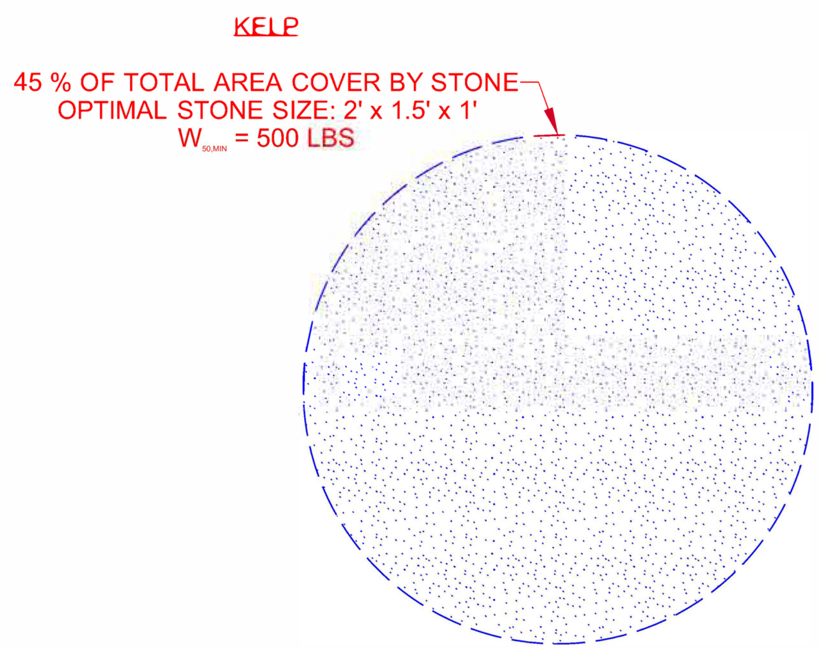
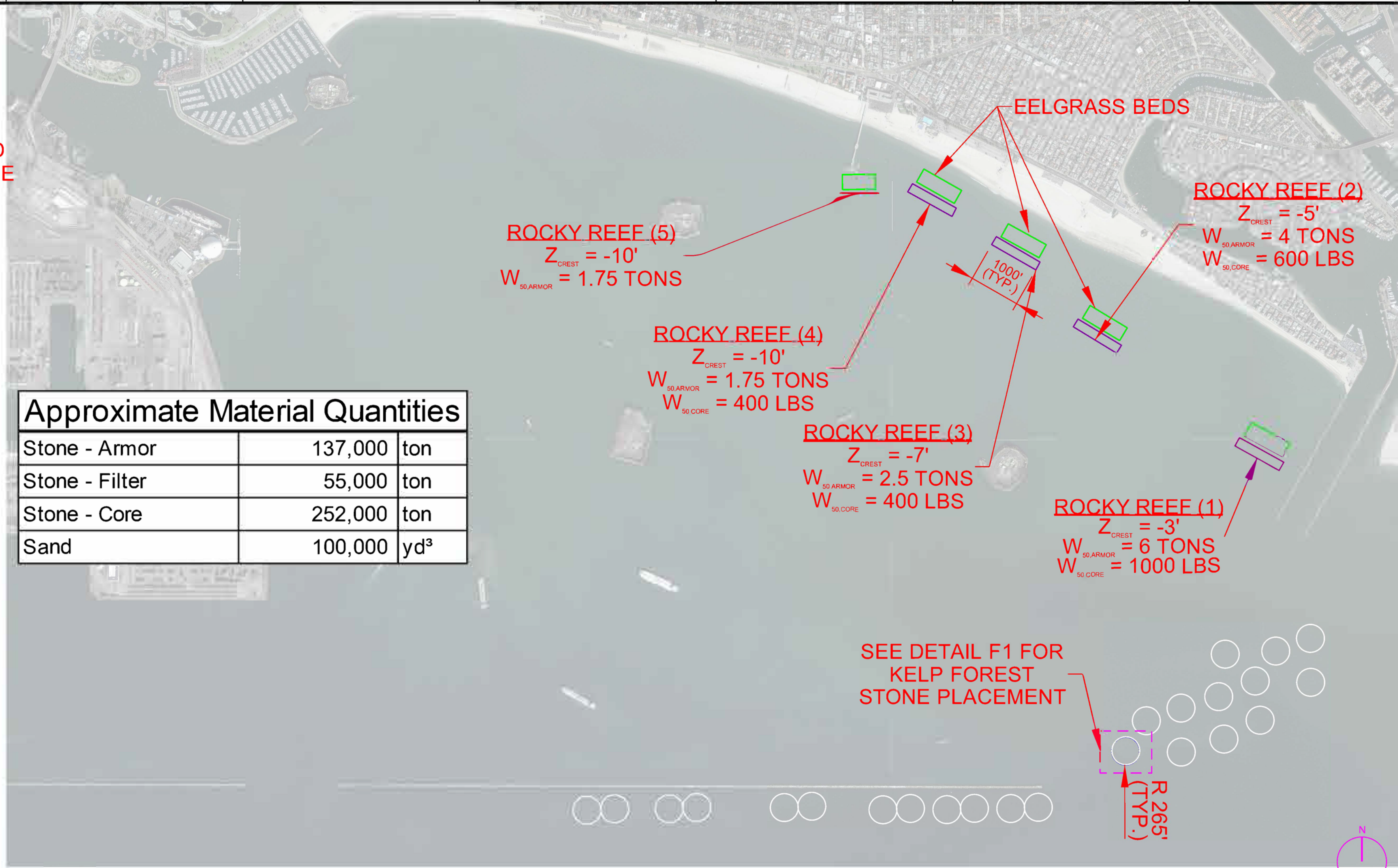
V: 0 4' 8'



A1 STONE PLACEMENT FOR KELP FOREST
SCALE: 1" = 200'
0 100' 200'



Approximate Material Quantities		
Stone - Armor	137,000	ton
Stone - Filter	55,000	ton
Stone - Core	252,000	ton
Sand	100,000	yd³

D5 BEST BUY ALTERNATIVE 2 RESTORATION PLAN VIEW
SCALE: 1" = 3000'
0 1500' 3000'

NEARSHORE ROCKY REEF

1. ELEVATIONS AND STONE WEIGHTS AS SHOWN ON DETAIL D5
2. STONE WEIGHT CALCULATED FROM VAN DER MEER (1991)
3. WAVE TRANSMISSION ASSUMES THE MINIMUM WATER LEVEL WILL NOT OCCUR AT THE TIME OF MAXIMUM WAVE HEIGHT
4. ARMOR LAYER THICKNESS MAY CHANGE DEPENDING ON STONE SIZE AND CREST ELEVATION
5. STONE PROPERTIES:
   S.G. = 2.65 (2.5 MIN.)
   Dmin/max= ±20% OF W50
   SUB-ROUNDED, SUB ANGULAR OR ANGULAR

EELGRASS

1. MAXIMUM TRANSMITTED WAVE HEIGHTS MUST BE LESS THAN 1.5 FT
2. TRANSMISSION FACTOR CALCULATED FROM SEABROOK & HALL (1998)
3. ALLOWABLE TRANSMITTED WAVE HEIGHTS BASED ON 1-YR STORM EVENT

KELP

1. STABILITY CALCULATIONS FROM BLANKET STONE STABILITY UNDER CONSTANT FLOW
2. THE OPTIMAL STONE SIZE CAN BE UTILIZED IN ALL AREAS BASED WAVE CONDITIONS
3. SAFETY FACTOR APPLIED TO THE STONE WEIGHT IS SET AT 3.0 TO ACCOUNT FOR INERTIAL FORCES THAT ARE NOT REPRESENTED IN THE BLANKET STONE STABILITY CALCULATION
4. STONE PROPERTIES:
   S.G. = 2.65 (2.5 MIN)
   ANGULAR STONE REQUIRED
   Dmin/max = 30", 12" (MAXIMUM/MINIMUM IN SINGLE DIMENSION)

GENERAL SHEET NOTES

1. ALL DIMENSION IN U.S. SURVEY FEET UNLESS OTHERWISE INDICATED
2. ALL PLANS/CROSS-SECTIONS/PROFILES ARE BASED ON FEASIBILITY LEVEL ANALYSIS, DETAILED DESIGN MAY ALTER THE PLACEMENT, SIZE, ELEVATION OF THE MEASURES
3. ELEVATIONS FROM A COMBINATION OF NOAA NAVIGATION CHARTS, NCMP LIDAR, AND LOCAL USACE CONDITION SURVEYS AND SHOWN AS FEET FROM THE MEAN LOW LOWER WATER (MLLW) LEVEL

US Army Corps of Engineers®

FEASIBILITY ONLY
NOT FOR CONSTRUCTION PURPOSES

DESIGNED BY: | ISSUE DATE:
DRAWN BY: | SOLICITATION NO.
CHECKED BY: | CONTRACT NO.:
SUBMITTED BY: | FILE NAME
SIZE: 11 X 17 | CAD FILE 328345_CE101_L.dgn

U.S. ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT
LOS ANGELES, CALIFORNIA

LOS ANGELES COUNTY, CALIFORNIA
EAST SAN PEDRO BAY
ECOSYSTEM RESTORATION STUDY

ALTERNATIVE 2
KELP RESTORATION PLAN

SHEET ID
CE101

9.1.3 ALTERNATIVE 4A – REEF RESTORATION ALTERNATIVE

The next final array alternative, shown on Sheet CE102, builds on the previous plan and includes measures in additional restoration zones with an increased project footprint. An additional nearshore reef would add another 1,000 linear feet to the reef series. Like the previous alternative, this reef would allow eelgrass to be planted and establish in the lee of the structure. Offshore kelp reefs remained unchanged from Alternative 2. An open water rocky reef complex will be constructed near Grissom Island. This reef complex consists of individual rubble mound reefs, on the order of 75 to 100 ft. in diameter, that are spaced approximately 100 ft. apart. Crest elevations within the reef complex will range from -30 ft. MLLW to -15 ft. MLLW. Total projected acreages of restoration areas is shown in Table 9-3 and material quantities in Table 9-4.

Table 9-3: Alternative 4A Restoration Areas

Final Array Alternative	Total Area (ac)
BBP4	200.69
Eelgrass	30.27
Kelp	121.38
Nearshore Reef	19.86
Offshore Reef	29.19

The final design will be as described in Alternative 2. A revised design and construction schedule can be found in the RECOMMENDED PLAN section. A physical model will also be required to verify all assumptions and impacts to existing operations.

Table 9-4: Approximate Quantities for Alternative 4A

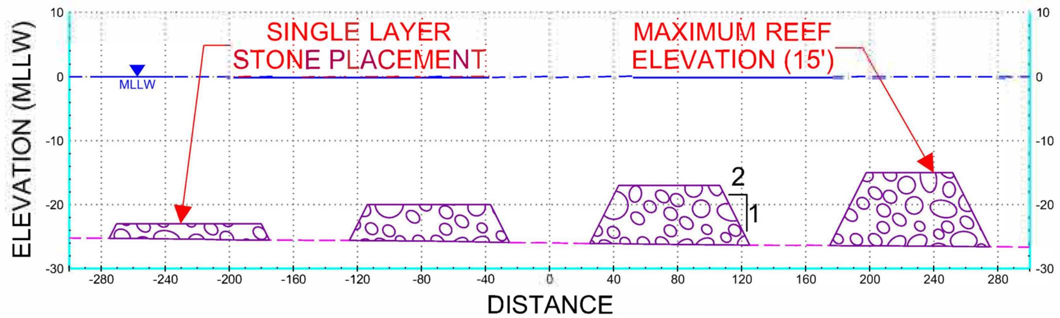
Measure	Material Type	Approximate Quantity	Unit	Representative Size
Open Water Reefs	Armor Stone	183,000	tons	10 tons
Nearshore Reefs	Armor Stone	176,000	tons	1 - 10 tons
	Filter Stone	55,000	tons	~ 1 ton
	Core Stone	134,000	tons	~ 10 - 1000 lbs
Kelp Reefs	Quarry Stone	132,000	tons	500 lbs
Eelgrass	Sand	100,000	yd^3	0.2 mm

Construction is similar to Alternative 2 with the addition of the two open water reef complexes. Open water reef complexes will be constructed in a similar manner to that of a typical rubble mound structure. Stones will be required to be individually placed to achieve the proper size, shape and void spaces to ensure the required habitat. Construction techniques will be the same as described in the PLAN FORMULATION AND DESCRIPTION OF ALTERNATIVES chapter. Equipment is similar to that required for the previous alternative. For a more complete discussion on construction duration, equipment and methods, see the RECOMMENDED PLAN section.

Similar maintenance is expected as in Alternative 2. The additional nearshore reef will slightly increase the magnitude of the required maintenance. The open water reefs should not require any maintenance since they are more deeply submerged and do not experience the same wave forces at these depths than the nearshore reefs and consists of stones larger than design guidance suggest to maintain the specified void spaces for the habitat.

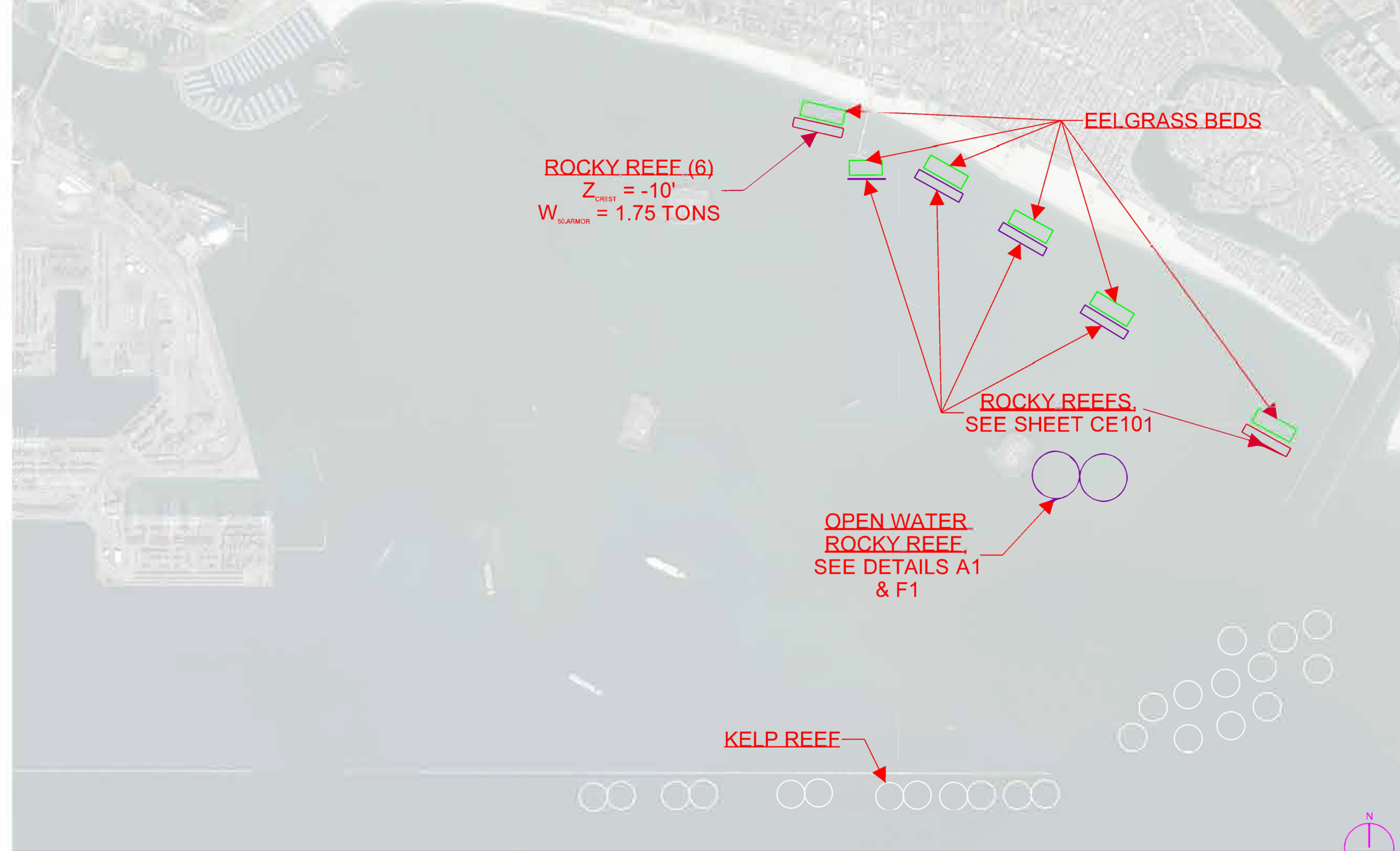
City conducted backpassing operations on the beach would be reduce further in quantity and duration from Alternative 2. The overall impact on port operations would be minimal under this alternative. There would be a small reflection from the open water reef complexes that may increase wave heights near the "D" anchorages within the project area. This reflection will be limited to no more than 15 percent of the incident wave height.

Sea level rise will have similar bay-wide affects as in the two previous alternatives. Specific affects would include additional adaptive management for the new nearshore reefs as well as the need to raise the additional eel grass bed. Considerations and triggers for these adaptations will be developed during the PED phase of the project.



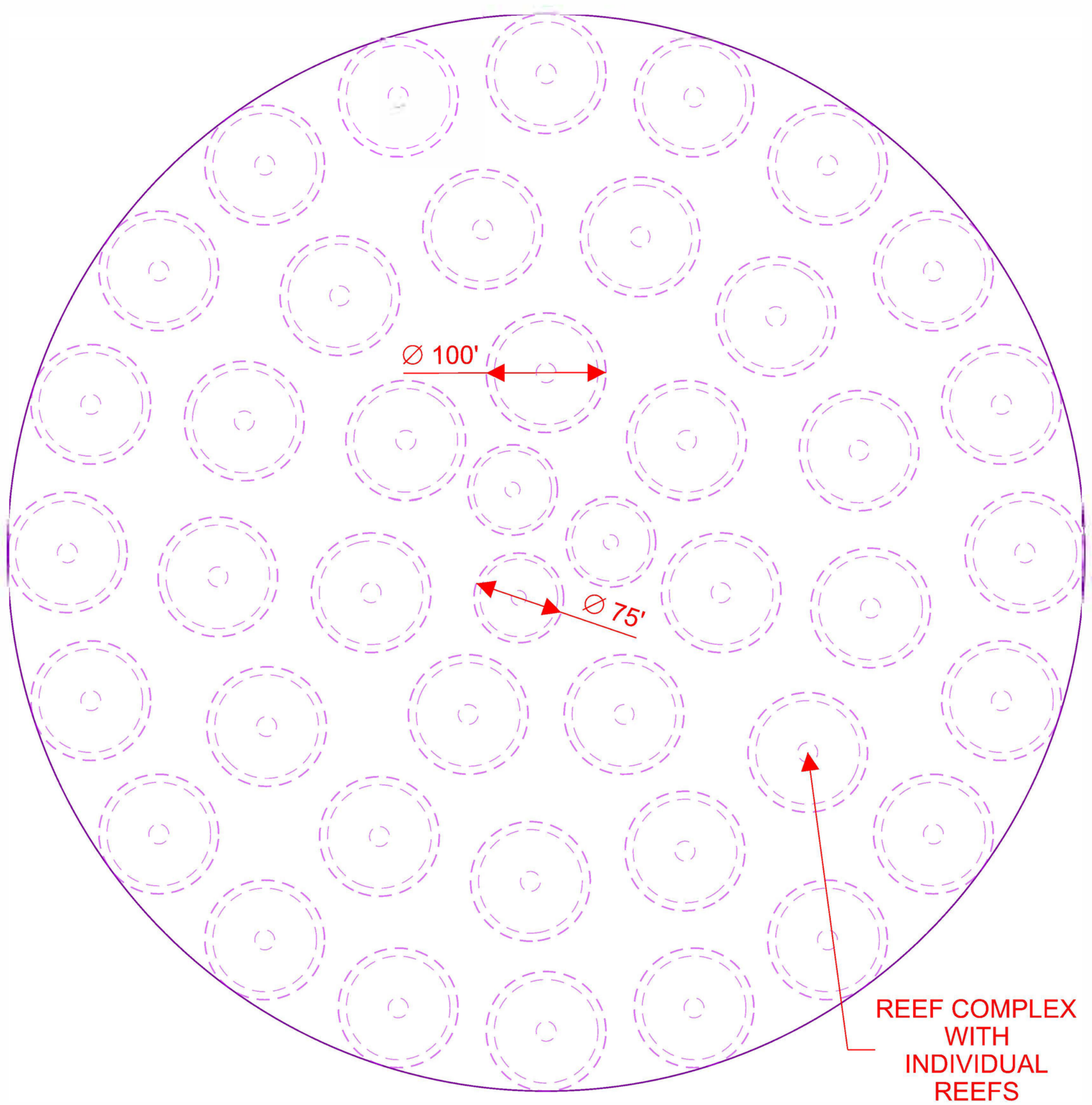
F1 OPEN WATER ROCKY REEF CROSS-SECTION

HORIZONTAL SCALE: 1" = 120'
VERTICAL SCALE: 1" = 30'



D5 ALTERNATIVE 4A RESTORATION PLAN VIEW

SCALE: 1" = 3000'



A1 OPEN WATER ROCKY REEF PLAN VIEW

SCALE: 1" = 160'

OPEN-WATER ROCKY REEF

1. MAXIMUM VOID SPACES REQUIRED FOR PROPER RESTORATION, ASSUME STRUCTURE POROSITY TO BE 0.6
2. STONES WILL BE SPECIALLY PLACE TO ACHIEVE REQUIRED VOID SPACING
3. INDIVIDUAL REEF SPACING IS 20' (MINIMUM) TO 50' (MAXIMUM)
4. DISTRIBUTION OF REEF RELIEF:
   - 3' -> 20%
   - 6' -> 25%
   - 9' -> 35%
   - 12' -> 25%
5. STONE PROPERTIES:
   - ANY ANGULARITY BUT REQUIRE 2H:1V SIDE SLOPE
   - ASSUMED W50 = 2 TONS (D50 = 3.0')
   - S.G. = 2.65 (2.5 MIN)
   - REQUIRE NARROW GRADATION (NEED SINGLE STONE SIZE FOR VOID SPACES)

GENERAL SHEET NOTES

1. ALL DIMENSION IN U.S. SURVEY FEET UNLESS OTHERWISE INDICATED
2. ALL PLANS/CROSS-SECTIONS/PROFILES ARE BASED ON FEASIBILITY LEVEL ANALYSIS, DETAILED DESIGN MAY ALTER THE PLACEMENT, SIZE, ELEVATION OF THE MEASURES
3. ELEVATIONS FROM A COMBINATION OF NOAA NAVIGATION CHARTS, NCMP LIDAR, AND LOCAL USACE CONDITION SURVEYS AND SHOWN AS FEET FROM THE MEAN LOW LOWER WATER (MLLW) LEVEL

Approximate Material Quantities		
Stone - Armor	359,000	ton
Stone - Filter	55,000	ton
Stone - Core	266,000	ton
Sand	100,000	yd^3

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EAST SAN PEDRO BAY
ECOSYSTEM RESTORATION STUDY

ALTERNATIVE 4A
REEF RESTORATION PLAN

SHEET ID
CE102

9.1.4 ALTERNATIVE 8 – SCARCE HABITAT RESTORATION ALTERNATIVE

The last alternative within the final array, shown on Sheet CE103, also builds on the previous final array plans. All features remain as in Alternative 4A but with expanded sizes and a few new restoration measures and zones. The nearshore rocky reefs, eelgrass beds and offshore kelp reefs remain exactly as in Alternative 4A. The open water reef complexes are expanded in size with an additional location near Freeman Island. Three additional restoration measures are added; oyster reef, emergent sandy island and tidal salt marsh. Oyster reefs are to be placed in appropriate water depths. Two reefs will be constructed using shell hash on the existing Alamitos Bay Jetties. Each reef has a length of approximately 500 linear feet. No more than 500 yd^3 of shell hash will be utilized for a base course followed by juvenile seeding of the reefs. The sandy island is placed between two nearshore reefs and allows for eelgrass to thrive in the lee of the structure due to the calm water provided by this near impermeable structure. The constructed location is approximately 1,500 ft. from the nearest landform. The island is armored by stone on the seaward side to protect from the large incident waves but contain a pocket beach on the leeward side that will slope until meeting the natural bathymetry. Two tidal salt marshes are included in this plan to be placed nearest to the Los Angeles River mouth. The surface penetrating features will come to an elevation of 16 ft. (MLLW) and the interior will be filled to achieve the required contouring to be determined in the final design phase. Total projected areas of restoration is shown in Table 9-5 and quantities shown in Table 9-6.

Table 9-5: Best Buy Plan 8 Restoration Areas

Final Array Alternative	**Total Area (ac)**
BBP9	371.86
Eelgrass	52.31
Emergent Island	23.82
Kelp	121.38
Nearshore Reef	19.86
Offshore Reef	102.15
Oyster Reef	0.27
Tidal Salt Marsh	52.07

Final design for Alternative 8 will be the longest of all final array plans and will last approximately 3 years. Detailed design of the vertical caisson structure and perforations will be performed to limit the wave reflection. Two separate physical models would need to be developed under this alternative; the entire project area (similar to the previous Final Array Plans) and the tidal salt marshes individually. A larger scale is needed for the vertical caisson structures due to the turbulent processes that are important for these types of structures that cannot be fully described in the smaller scale model required to capture the entire affected area.

Table 9-6: Approximate Quantities for Alternative 8

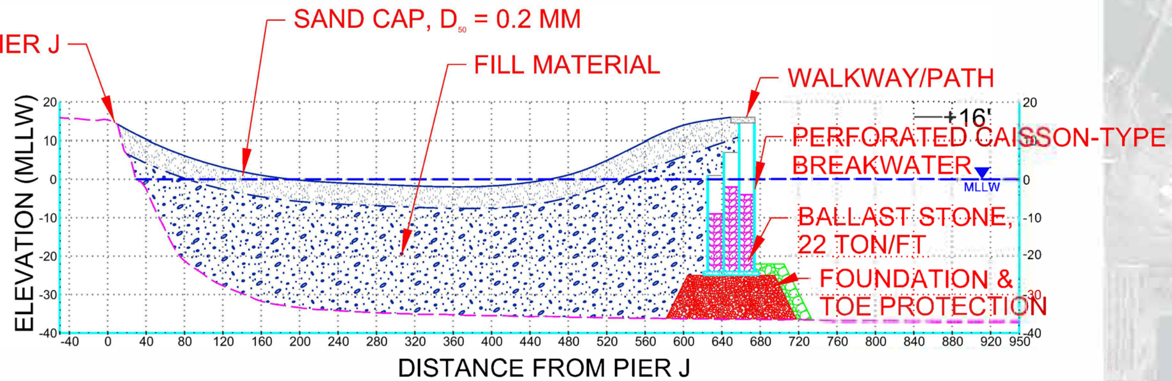
Measure	Material Type	Approximate Quantity	Unit	Representative Size
Sandy Islands	Armor Stone	336,000	tons	11 tons
	Filter Stone	37,000	tons	~ 1 ton
	Fill Material	1,057,000	yd^3	N/A
	Sand	276,000	yd^3	0.2 mm
Coastal Wetlands [LARE / Pier J]	Quarry Stone	10,000 / 24,000	tons	~ 10 - 1000 lbs
	Armor Stone	3,000 / 24,000	tons	1 - 3 tons
	Concrete	5,000 / 43,000	yd^3	N/A
	Fill Material	34,000 / 1,899,000	yd^3	N/A
	Sand	81,000 / 339,000	yd^3	0.2 mm
Open Water Reefs	Armor Stone	1,540,000	tons	10 tons
Nearshore Reefs	Armor Stone	176,000	tons	1 - 10 tons
	Filter Stone	55,000	tons	~ 1 ton
	Core Stone	134,000	tons	~ 10 - 1000 lbs
Kelp Reefs	Quarry Stone	132,000	tons	500 lbs
Eelgrass	Sand	100,000	yd^3	0.2 mm

Construction of this alternative is similar to the previous plans with the addition of the three new habitat types. For oyster reefs, shell hash will be placed onto the locations shown on SHEET CE103 and seeding of the juvenile organisms will follow. Construction techniques and methods for the emergent island and tidal salt marshes can be found in the PLAN FORMULATION AND DESCRIPTION OF ALTERNATIVES chapter. Equipment requirement is similar to that of Alternative 4A with the inclusion of a land-based staging location to pre-cast the caisson structures. Additional tug boats will be required for the transfer of the caissons between the landside plant and project area. The construction duration is determined to be 53 months in total. Construction within each zone should proceed concurrently.

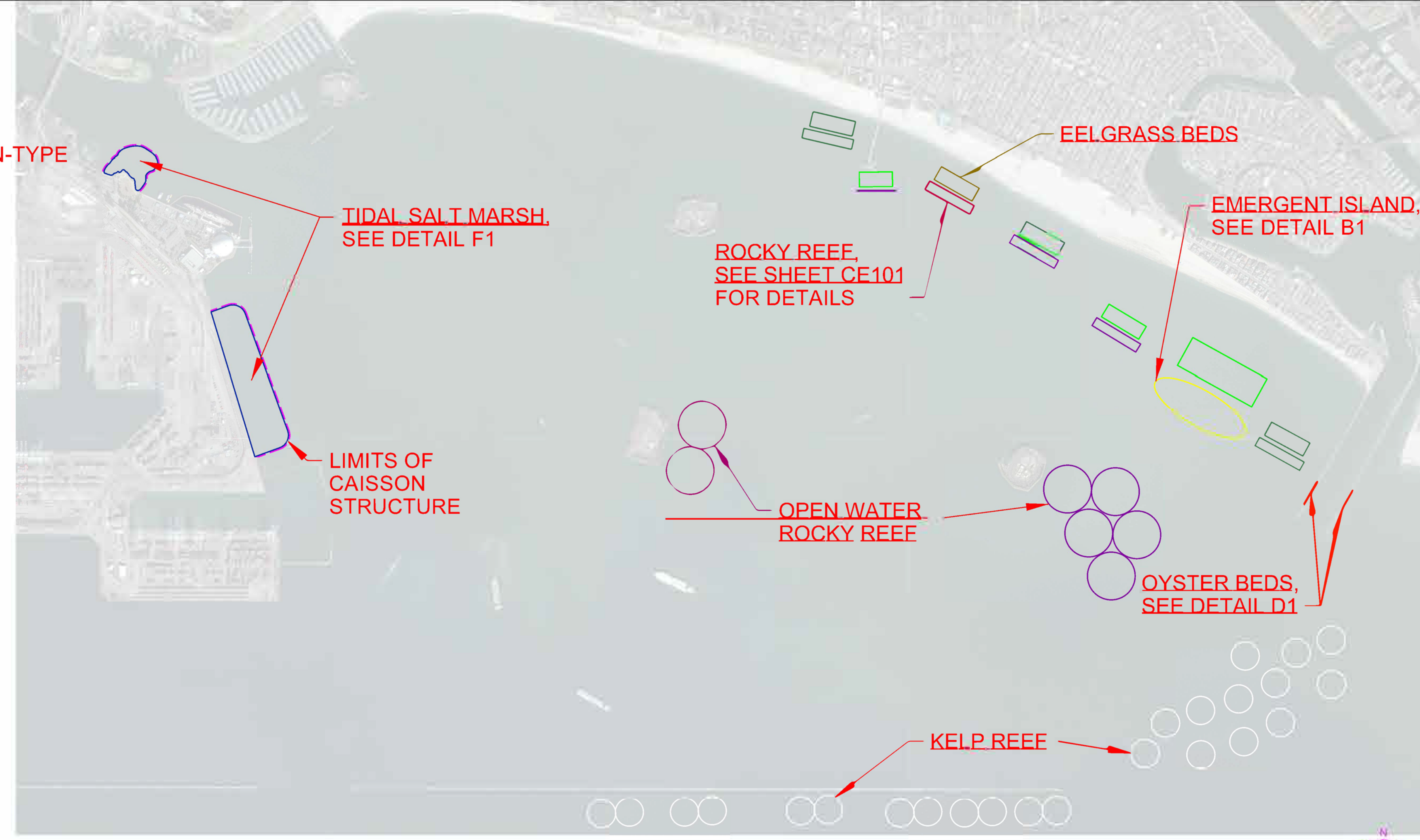
Similar maintenance to Alternative 4 is required. The additional surface penetrating island and salt marsh will require periodic maintenance such as clearing and grubbing unwanted vegetation, re-contouring or dredging of the interior of the tidal salt marsh, grooming of the top layer of sand on the emergent island and adding additional fill material to replenish material lost through natural processes.

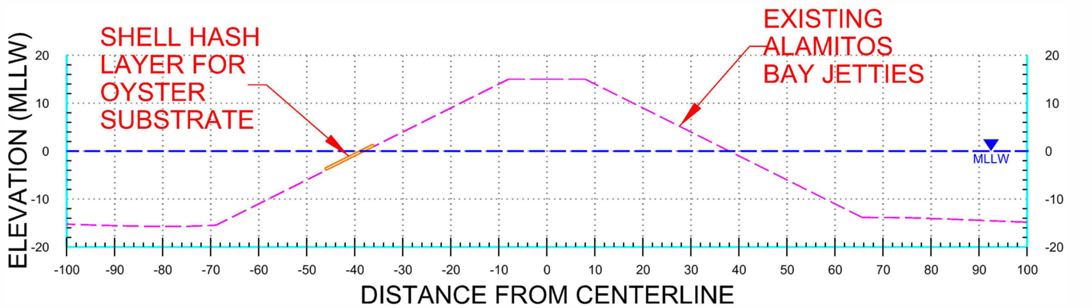
Minor alterations to vessel traffic are expected as a result of the tidal salt marsh near Pier J as vessels transiting the area will need to avoid the new feature. Minor, local wave height increase are expected due to the reflections from the vertical face of the tidal salt marshes but will be limited to the most practicable extent possible during the final design phase of the project. A minor increase to wave height due to reflections from emergent island is expected, impacting the energy islands and port's mooring areas. Further analysis during the final design phase will limit this reflection to the greatest extent possible. The emergent sandy island would shelter the shoreline to a larger extent. Waves would break on the offshore structure thereby limited the transmitted energy and reducing the local run-up.

Potential sea level rise will have a similar effect on the bay as in the previous alternatives. Additionally, adaptive management will be required for the tidal salt marshes and emergent island. Additional sand will be required for both measures to maintain the habitats to the design condition. No proposed measures will increase the susceptibility of the bay to sea level rise.



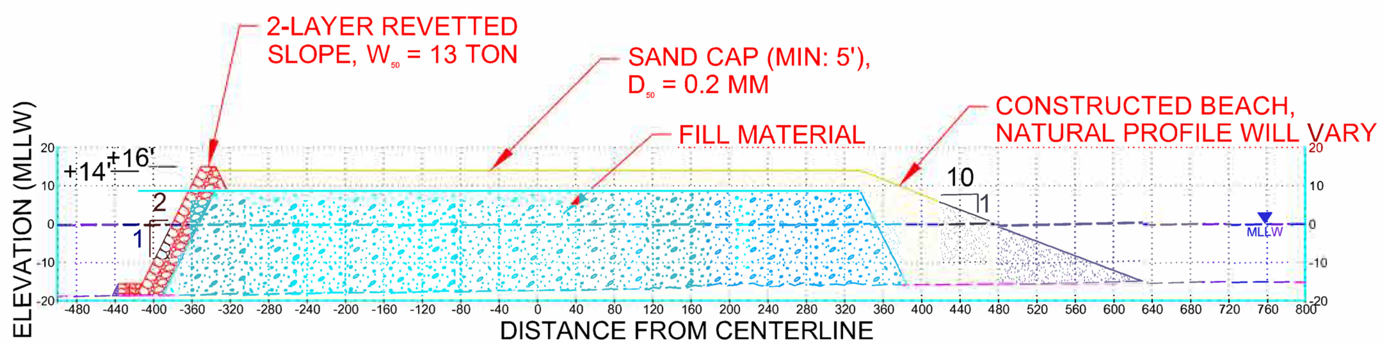
F1 PIER J TIDAL SALT MARSH CROSS-SECTION
HORIZONTAL SCALE: 1" = 200'
VERTICAL SCALE: 1" = 50'
H: 0 200' 400'
V: 0 50' 100'





D1 OYSTER BED PLACEMENT CROSS-SECTION
HORIZONTAL SCALE: 1" = 40'
VERTICAL SCALE: 1" = 40'
H: 0 20' 40'
V: 0 20' 40'

D5 ALTERNATIVE 8 RESTORATION PLAN VIEW
SCALE: 1" = 3000'
0 1500' 3000'



B1 EMERGENT SANDY ISLAND CROSS-SECTION
HORIZONTAL SCALE: 1" = 200'
VERTICAL SCALE: 1" = 50'
H: 0 100' 200'
V: 0 25' 50'

TIDAL SALT MARSH

1. SAND CAP MATERIAL FROM SURFSIDE/SUNSET BORROW AREA
2. DESIGN OF CAISSON PERFORATIONS TO ABSORB WAVE ENERGY TO BE DETERMINED
3. LAYOUT OF INTERIOR CHANNELS AND DETAILED DESIGN TO BE DETERMINED
4. STONE PROPERTIES:
   FOUNDATION:
   QUARRY RUN
   TOE PROTECTION:ANGULAR
   W50 = 2 TONS (MIN: 1 TON, MAX: 3 TON)
   S.G. = 2.65
   BALLAST STONE:
   22 TON/FT REQUIRED
   S.G. = 2.65
   NO SHAPE REQUIREMENT

SANDY ISLAND

1. CLEAN SAND FROM SURFSIDE/SUNSET BORROW AREA
2. STRUCUTRE CONSTRUCTED IN LIFTS WITH CORRESPONDING REVETMENT ARMORING TO LIMIT WAVE ACTION ON FILL MATERIAL
3. STONE PROPERTIES:
   ANGULAR, INTERLOCKING REQUIRED
   W50 = 11 TON (MIN: 9 TON, MAX: 14 TON)
   S.G. = 2.65

Approximate Material Quantities		
Stone - Armor	2,079,000	ton
Stone - Filter	92,000	ton
Stone - Core	300,000	ton
Fill	2,990,000	yd³
Sand	796,000	yd³
Concrete	48,000	yd³

GENERAL SHEET NOTES

1. ALL DIMENSIONS IN U.S. SURVEY FEET
2. ALL PLAN/CROSS-SECTIONS/PROFILES ARE BASED ON FEASIBILITY LEVEL ANALYSIS, DETAILED DESIGN MAY ALTER THE PLACEMENT, SIZE , ELEVATION OF THE MEASURES
3. ELEVATIONS FROM A COMBINATION OF NOAA NAVIGATION CHARTS, NCMP LIDAR AND LOCAL USACE CONDITIONS SURVEYS AND SHOWN AS FEET FROM THE MEAN LOWER LOW WATER (MLLW) LEVEL

OYSTER REEF

1. THIN LAYER OF SHELL HASH REQUIRED FOR OYSTER SUBSTRATE
2. JUVENILE SEEDING REQUIRED

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ECOSYSTEM RESTORATION STUDY

ALTERNATIVE 8
SCARCE HABITAT
RESTORATION PLAN

SHEET ID
CE103

9.2 PLANS NOT CARRIED FORWARD

The following are plans that were analyzed with additional detail but ultimately have been screened out of the final array of alternatives. For further discussion on this topic, see MAIN REPORT. Each of these breakwater alternatives expand on Alternative 2 as the base restoration features with the specific breakwater modification. These alternatives have not been fully analyzed at the same extent as the Final Array of Alternatives.

9.2.1 BREAKWATER PLAN 1 – BREAKWATER WESTERN NOTCHING ALTERNATIVE

The first breakwater plan, shown on Sheet CE104, builds on restoration features from Alternative 2. A spatial shift of the nearshore reefs will assist in the City's annual backpassing operations by limited the incident wave energy on the section of open coastline. This shift will provide more complete coverage of Peninsula Beach which will limit the incident wave energy and potential sediment transport. Additionally, the Long Beach Breakwater is modified by removing two 1,000 ft. sections on the western side. A single submerged breakwater with a similar plan view as the previously described nearshore reefs with a crest elevation of 0 ft. (MLLW) is required fronting the parking lot at Junipero Beach to protect the structure from erosion that is expected to increase as a result of the increase in wave energy allowed by the notches in the breakwater. The offshore energy islands of Freeman and White requires an increased size of armor stone to be wrapped around the existing revetment but will remain at the current crest elevation. Additionally, a ~5 ft. tall seawall would be required on Island Freeman to reduce the potential for overtopping. Additional protection to existing port features is required due to the proximity to the breakwater notches. Increase in armor stone size and crest height is needed for portions of the Pier J Jetties, the revetment along Pier J to the Queen Mary, the detached breakwater protecting Shoreline Marina, and the revetments of Shoreline Marina and Grissom Island. Additional design will be required if this modification goes forward. Total direct restored or modified areas are shown in Table 9-7 and quantities shown in Table 9-8.

Table 9-7: Breakwater Plan 1 Restoration Areas

Alternative	Total Area (ac)
Western Notching Plan	182.19
Eelgrass	30.27
Kelp	121.38
Nearshore Reef	19.76
Breakwater Mod.	10.78

The final design of this alternative is be similar to that of Best Buy Plan 2, but with focus on the breakwater modification void length, the spacing of the submerged reefs and size and crest elevation of the submerged breakwater fronting Junipero Beach. Extensive physical modeling use will be required to determine the exact modification to the breakwater as well as to as to verify the impacts to navigation. This phase is expected to take 36 months to complete and prepare a final design for implementation.

Table 9-8: Approximate Quantities for Breakwater Plan 1

Measure	Material Type	Approximate Quantity	Unit	Representative Size
Nearshore Reefs	Armor Stone	202,000	tons	12 tons
	Filter Stone	55,000	tons	~ 1 ton
	Core Stone	229,000	tons	~ 10 - 1000 lbs
Kelp Reefs	Quarry Stone	132,000	tons	500 lbs
Eelgrass	Sand	600,000	yd^3	0.2 mm
Breakwater Stone Reused[2]	Armor Stone	88,000	tons	12 tons
	Filter Stone	95,000	tons	2 tons
Protective Measures	Armor Stone	315,000	tons	12 tons
	Filter Stone	270,000	tons	~ 1 ton
	Concrete	4,000	yd^3	N/A

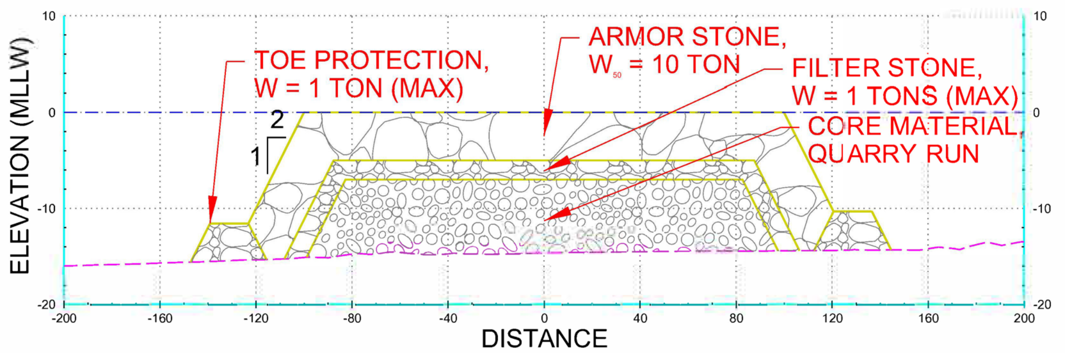
Creation and construction of the restoration measures will be similar to that of Alternative 2. Removal of the two sections of breakwater will be accomplished using a large barge-mounted derrick crane. Stones will be individually picked up by the crane and placed on an adjacent barge for transportation to other areas within the project site and used as protective armor stone for existing infrastructure or creation of new protective measures. Stones should be cleaned and removed of any foreign debris before being placed in different locations. Four enlarged head sections must be created on the newly formed ends of the breakwater which will utilize much of the large "A" armor stone removed from the notches. The seawall on Freeman Island may be cast-in-place after the addition of larger armor stone along the island's revetment. Creation of the nearshore submerged breakwater near Junipero Beach and the emergent breakwater near Belmont Pier would follow the placement techniques of a typical rubble mound structure already described. For a further discussion, see the PLAN FORMULATION AND DESCRIPTION OF ALTERNATIVES chapter. Equipment will be like that of Alternative 2 with an additional barge-mounted crane to assist with the increasing amount of stone placement.

Maintenance of these measures are similar to that of the Alternative 2. The costs to maintain the submerged breakwater near Junipero Beach is expected to be larger than the other nearshore reefs due to the increase of waves caused by the notches within the breakwater.

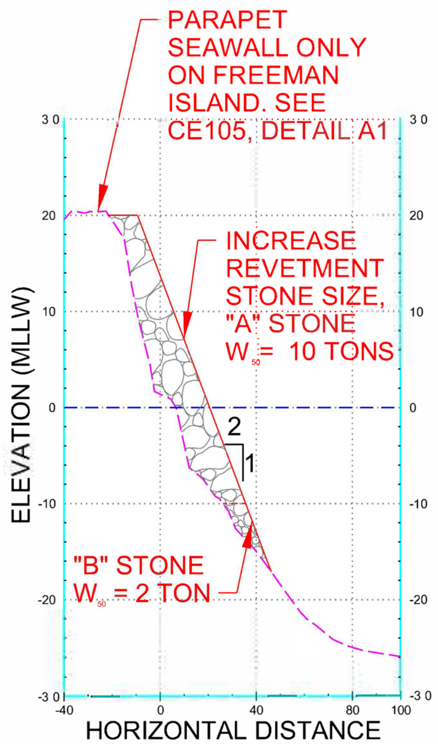
Extensive impacts to port operations is expected as a result of the breakwater modification. Near Pier J South, eight additional large wave events per year is expected that may delay transits or offloading/loading operations. Operations at the Carnival Cruise Line terminal may be limited by more than 30 additional events per year. The increase in wave energy will raise the number of events per year that cause delays to both transiting and loading/offloading operations of the energy islands by from 20

[2] The quantity of reused breakwater stone is subtracted from the final required stone quantity.

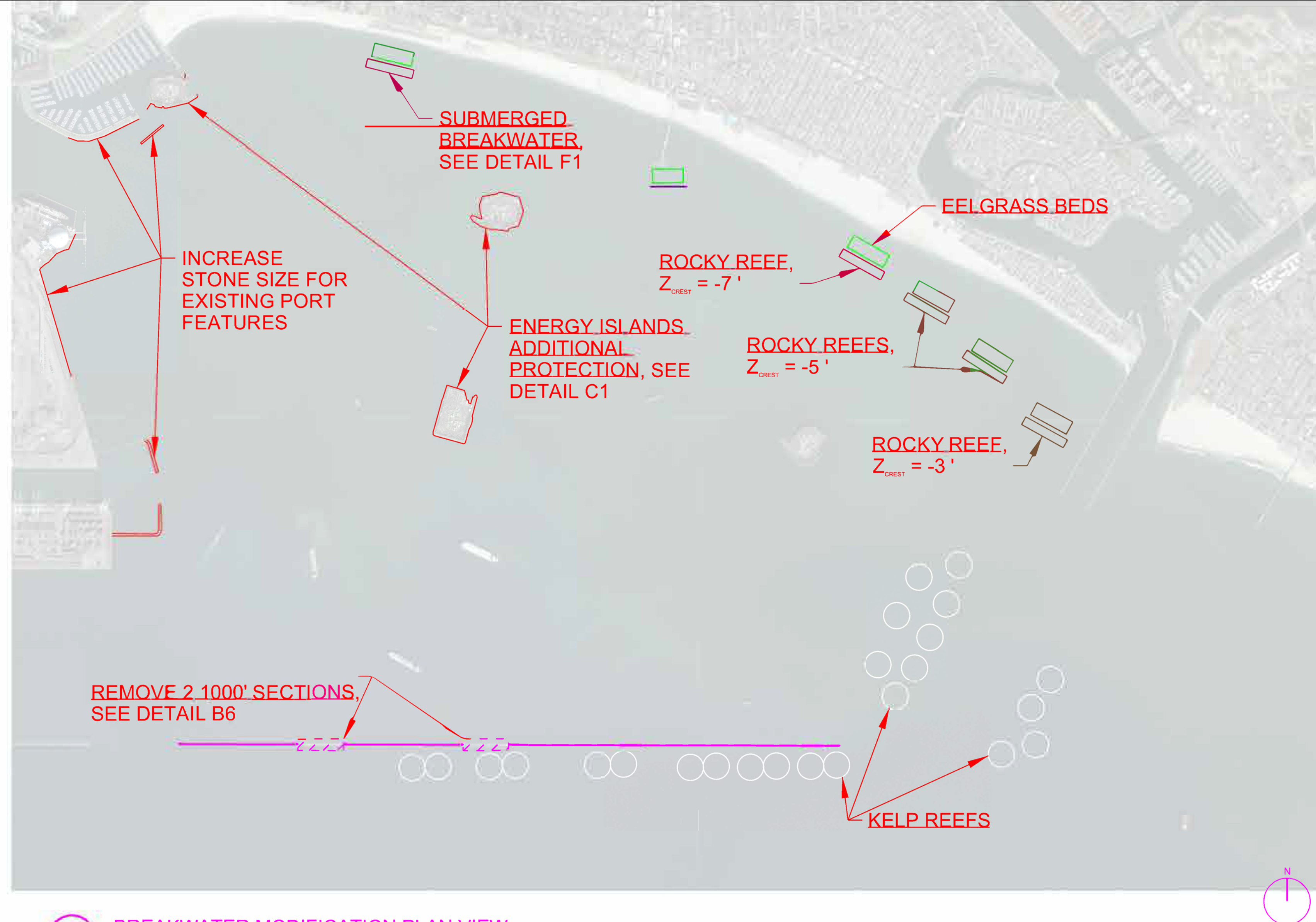
to 40 events. Use of the mooring areas would need to be modified; it is expected that these mooring areas will still be used but the added vessel motion will decrease productively and increase the duration of sheltering to make up for the efficiency loss. The larger waves would increase vessel motions within the breakwaters, especially those that are moored within the "D" Anchorage. The increase in vessel motions may interfere with maintenance activities or loading/offloading operations. The probability that a wave height threshold is exceeded within each mooring area can be seen in Figure 8-5. Due to the breakwater modification, the increase in the number of days with wave heights over 1 ft. and 2 ft. are 135 and 5 days respectively. The average surf zone size increases as much as 50% of the current size during smaller events (1-yr swell) but greatly increases to more than 200% during the large southern swell events.



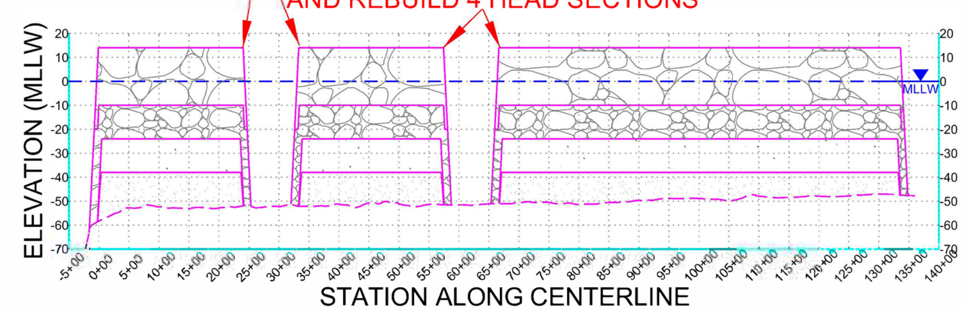
F1 SUBMERGED BREAKWATER
HORIZONTAL SCALE: 1" = 80'
VERTICAL SCALE: 1" = 20'



C1 ENERGY ISLAND PROTECTION
HORIZONTAL SCALE: 1" = 80'
VERTICAL SCALE: 1" = 20'



C5 BREAKWATER MODIFICATION PLAN VIEW
SCALE: 1" = 3000'



B6 BREAKWATER MODIFICATION CROSS-SECTION
HORIZONTAL SCALE: 1" = 3000'
VERTICAL SCALE: 1" = 80'

GENERAL NOTES

1. STONES AND MATERIAL FROM BREAKWATER MODIFICATIONS WILL BE UTILIZED IN PROTECTIVE MEASURES CONSTRUCTION

2. JUNIPERO BEACH PARKING LOT PROTECTION REQUIRED DUE TO INCREASE EROSION RATES CAUSED BY BREAKWATER MODIFICATION

3. STONE PROPERTIES:
   ANGULAR
   W,min/max = ±20% OF W50
   S.G. = 2.65

Approximate Material Quantities		
Stone - Armor	429,000	ton
Stone - Filter	230,000	ton
Stone - Core	361,000	ton
Sand	600,000	yd³
Concrete	4,000	yd³

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ECOSYSTEM RESTORATION STUDY

BREAKWATER ALTERNATIVE 1
WESTERN NOTCHING PLAN

SHEET ID
CE104

9.2.2 BREAKWATER PLAN 2 – BREAKWATER EASTERN REMOVAL ALTERNATIVE

The second breakwater plan, shown on Sheet CE105, also builds on Alternative 2 with some additional modifications. Firstly, the Long Beach Breakwater is modified by completely removing the eastern third of the structure, approximately 4,500 ft. The nearshore reefs and offshore kelp reefs are shifted as in the previous breakwater plan, but protection is no longer needed at Junipero Beach to control the erosive processes. This single submerged breakwater is shifted back to the eastern side the project area and grouped with previously proposed nearshore reefs. To protect the shoreline from the increase of wave energy allowed into the bay by the breakwater modification, the nearshore reefs will need to be bolstered with a higher crest elevation and heavier stone size. These now submerged breakwaters, with a top crest elevation of 0 ft. (MLLW), will cause waves to break and lose energy before impacting the shoreline and in turn will require an increase in the median armor stone size to 10 tons. The single small reef fronting Belmont pier will become a surface penetrating structure with a crest elevation of 10 ft. (MLLW). Stones removed from the breakwater will be used in the construction of these protective measures. Additionally, the offshore energy islands will also experience increase wave heights and must be protected. The required armor stone size need for protection will increase and additional layers of stone will be added to each island. Additionally, a seawall would be required on Freeman, White and Chaffee Islands to prevent overtopping greater than is currently experienced. Additional design will be required if this modification goes forward. Total restored or modified areas are shown in Table 9-9 and quantities shown in Table 9-10.

Table 9-9: Breakwater Plan 2 Restoration Areas

Alternative	Total Area (ac)
Western Notching Plan	195.45
Eelgrass	30.27
Kelp	121.38
Nearshore Reef	19.83
Breakwater Mod.	23.97

The final design process will follow that of the previous breakwater plan but with focus on the eastern end instead of the western notches. The physical model will guide the exact length of the breakwater that is removed as well as the orientation and size of the nearshore submerged breakwaters. Completion of the final design is also expected to take 36 months.

Table 9-10: Approximate Quantities for Breakwater Plan 2

Measure	Material Type	Approximate Quantity	Unit	Representative Size
Nearshore Reefs	Armor Stone	379,000	tons	12 tons
	Filter Stone	55,000	tons	~ 1 ton
	Core Stone	357,000	tons	~ 10 - 1000 lbs
Kelp Reefs	Quarry Stone	132,000	tons	500 lbs
Eelgrass	Sand	600,000	yd^3	0.2 mm
Breakwater Stone Reused[3]	Armor Stone	285,000	tons	12 tons
	Filter Stone	220,000	tons	2 tons
	Core Stone	383,000	tons	~ 100 – 1000 lbs
Protective Measures	Armor Stone	267,000	tons	12 tons
	Filter Stone	314,000	tons	~ 1 ton
	Concrete	4,000	yd^3	N/A

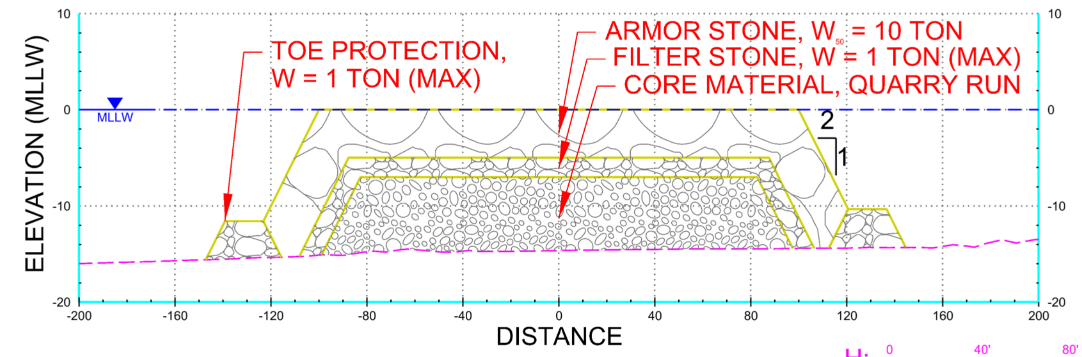
Construction methods would be similar to that of the previous breakwater plan and will required the same equipment. For this alternative, only one head section would need to be created, so more armor stone can be used for protective measures. As with the previous, stones removed from the breakwater will be placed on an awaiting barged to be transported to areas that require additional protection. The construction duration will last approximately 72 months. Construction of restoration measures can occur concurrently, but the protection measures, such as the nearshore submerged breakwaters, should be placed at the same rate to not leave the shoreline unprotected.

Maintenance is similar to that of Alternative 2. Increases in maintenance costs for the series of nearshore submerged breakwaters and single surface penetrating detached breakwater near Belmont Pier is expected. Changes in sediment transport patterns may alter the City's annual backpassing of sediments, but due to the creation of the nearshore breakwaters, the total quantity of sediment available to transport is expected to remain similar as to current conditions. The total quantity of sediments to transport is not expected to change, but the distance between the deposited and the eroded areas will increase.

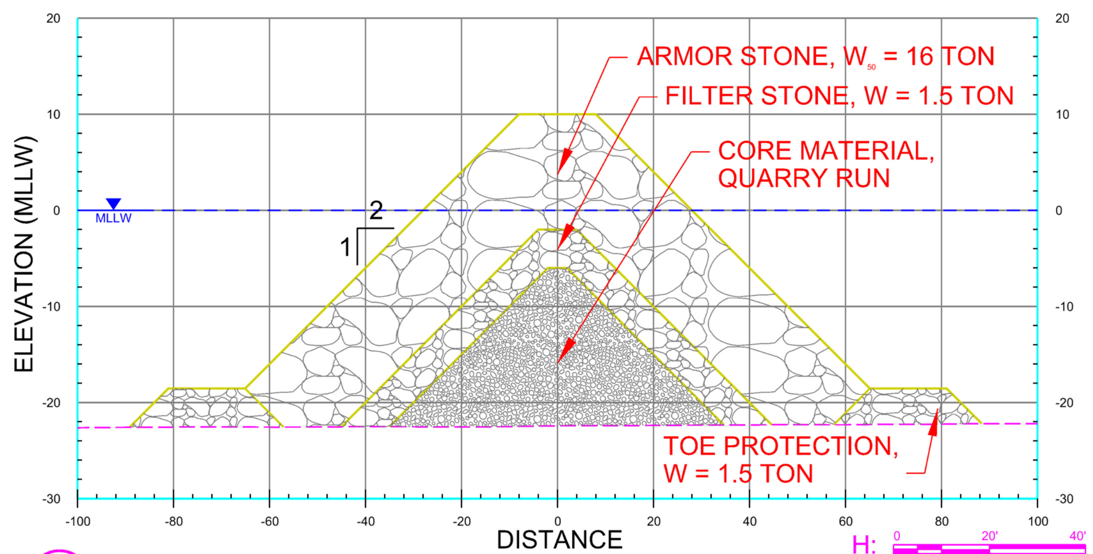
Impacts to the Port of Long Beach will be similar to that of the previous Breakwater Plan but different in magnitude. Near Pier J South, a single additional large wave event per year is expected that may delay transits or offloading/loading operations. Operations at the Carnival Cruise Line terminal may be limited by more than 2 additional events per year. Impacts to the offshore energy islands will cause delays both during transit to/from the islands and loading/offloading of supplies ranging from 20 to 250 events per year. The large range is a result of the un-sheltered wave energy that impacts Chaffee Island while not

[3] The quantity of reused breakwater stone is subtracted from the final required stone quantity.

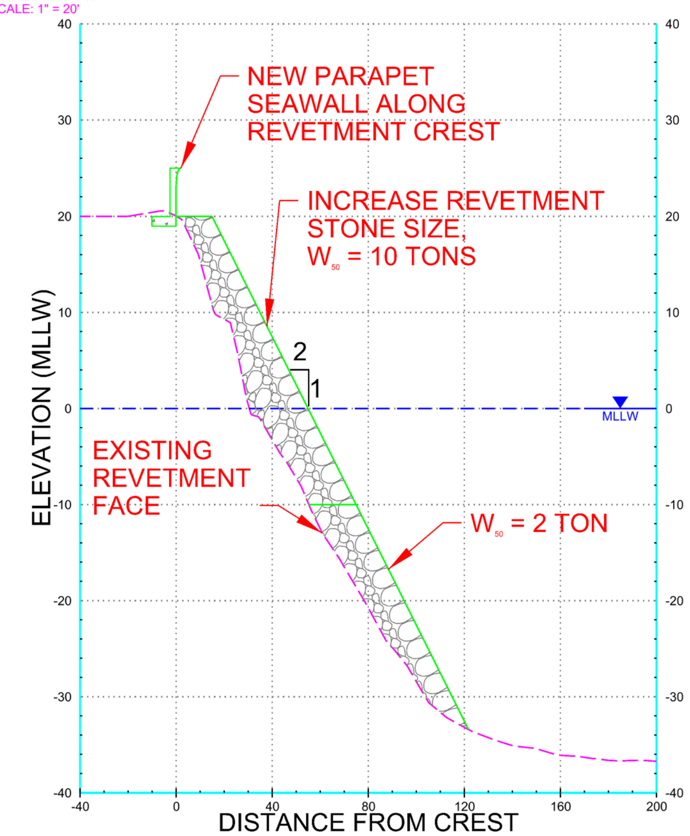
altering the incident wave climate of the other islands. Mooring areas will also be directly affected. Due to the breakwater modification, the increase in the number of days with wave heights over 1 ft. and 2 ft. are 131 and 7 days, respectively. As stated earlier, the U.S. Navy exclusion zone cannot be moved due to Navy offset requirements provided for public safety. The average surf zone size typically doubles due to the breakwater modification in the vicinity of the East Beach but returns to the original size from Belmont Pier to the west.



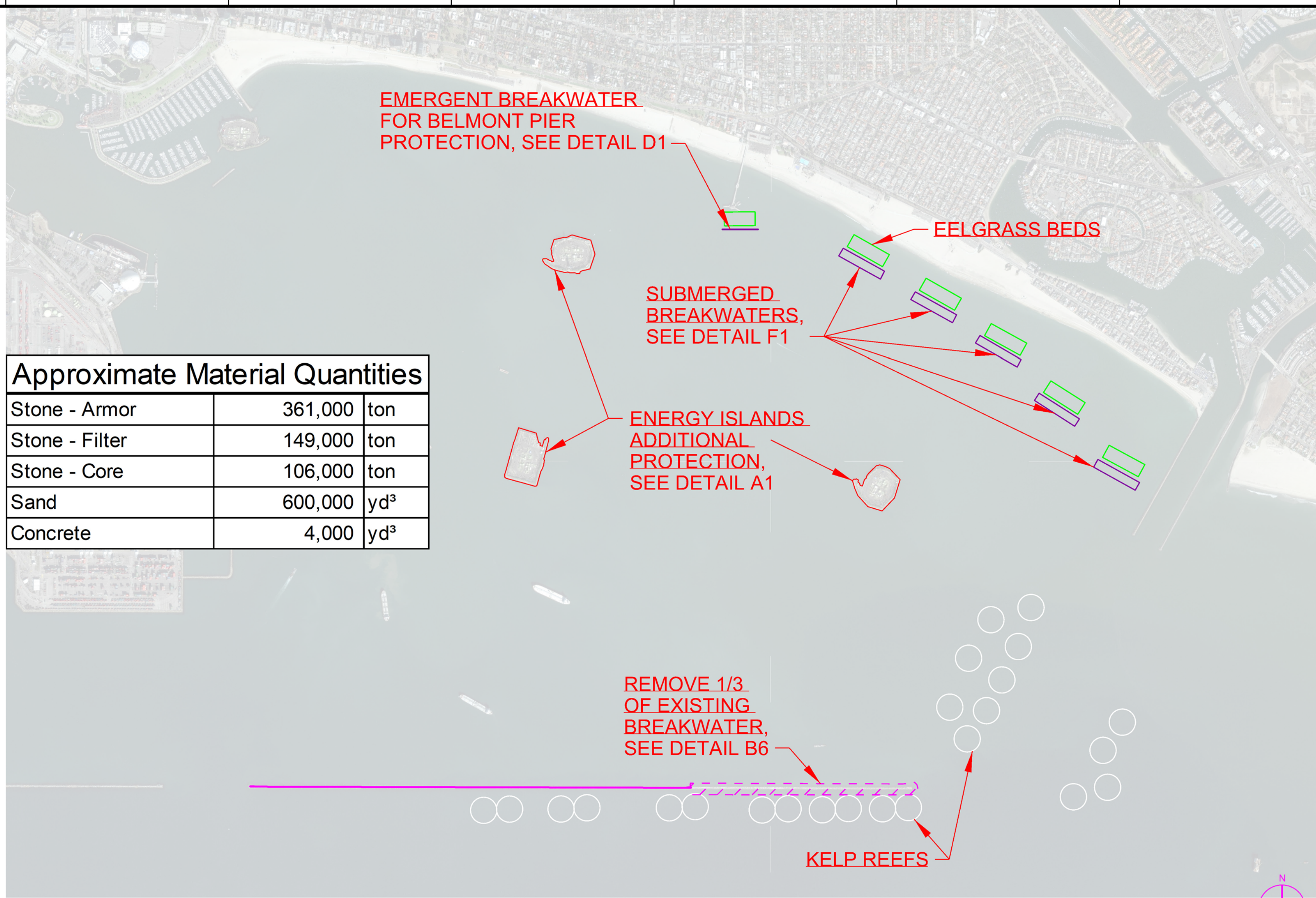
F1 SUBMERGED BREAKWATER CROSS-SECTION
HORIZONTAL SCALE: 1" = 80'
VERTICAL SCALE: 1" = 20'



D1 BELMONT PIER PROTECTION CROSS-SECTION
HORIZONTAL SCALE: 1" = 40'
VERTICAL SCALE: 1" = 20'

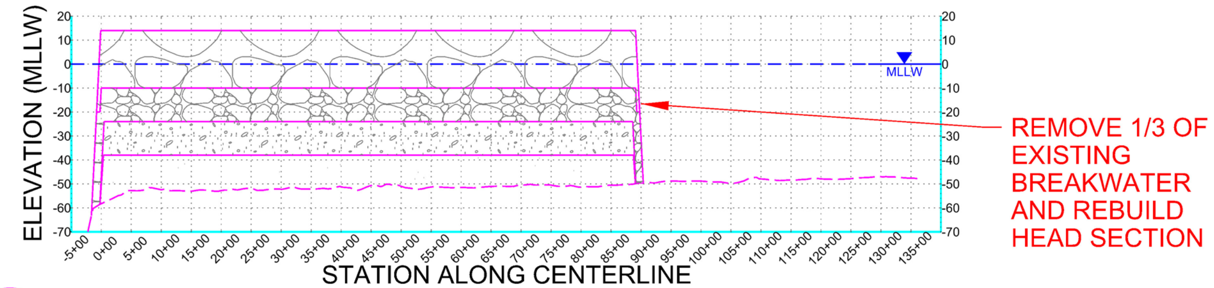


A1 ENERGY ISLANDS PROTECTION CROSS-SECTION
HORIZONTAL SCALE: 1" = 80'
VERTICAL SCALE: 1" = 20'



Approximate Material Quantities		
Stone - Armor	361,000	ton
Stone - Filter	149,000	ton
Stone - Core	106,000	ton
Sand	600,000	yd³
Concrete	4,000	yd³

C5 EASTERN BREAKWATER REMOVAL RESTORATION PLAN VIEW
SCALE: 1" = 3000'



B6 BREAKWATER MODIFICATION CROSS-SECTION
HORIZONTAL SCALE: 1" = 3000'
VERTICAL SCALE: 1" = 80'

GENERAL SHEET NOTES

1. STONES AND MATERIAL FROM BREAKWATER MODIFICATIONS CAN BE UTILIZED IN PROTECTIVE MEASURE CONSTRUCTION
2. SHORELINE PROTECTION REQUIRED DUE TO INCREASE IN WAVES FROM BREAKWATER MODIFICATON
3. STONE PROPERTIES:
   ANGULAR
   W,min/max = ±20% OF W50
   S.G. = 2.65

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LOS ANGELES, CALIFORNIA

LOS ANGELES COUNTY, CALIFORNIA
EAST SAN PEDRO BAY
ECOSYSTEM RESTORATION STUDY

BREAKWATER ALTERNATIVE 2
EASTERN 1/3 REMOVAL PLAN

SHEET ID
CE105

9.3 RECOMMENDED PLAN

The recommended plan is Alternative 4A, Reef Restoration Alternative. Minor adjustments to the feature footprints are required to satisfy regulatory requirements. Sizes and shape are not altered at this stage, only shifting of the complete footprints as shown in Figure 9-1. Further shifting of measures will based on results from the physical model yet to be performed, coordination from local stakeholders and other reasons determined during the PED phase.

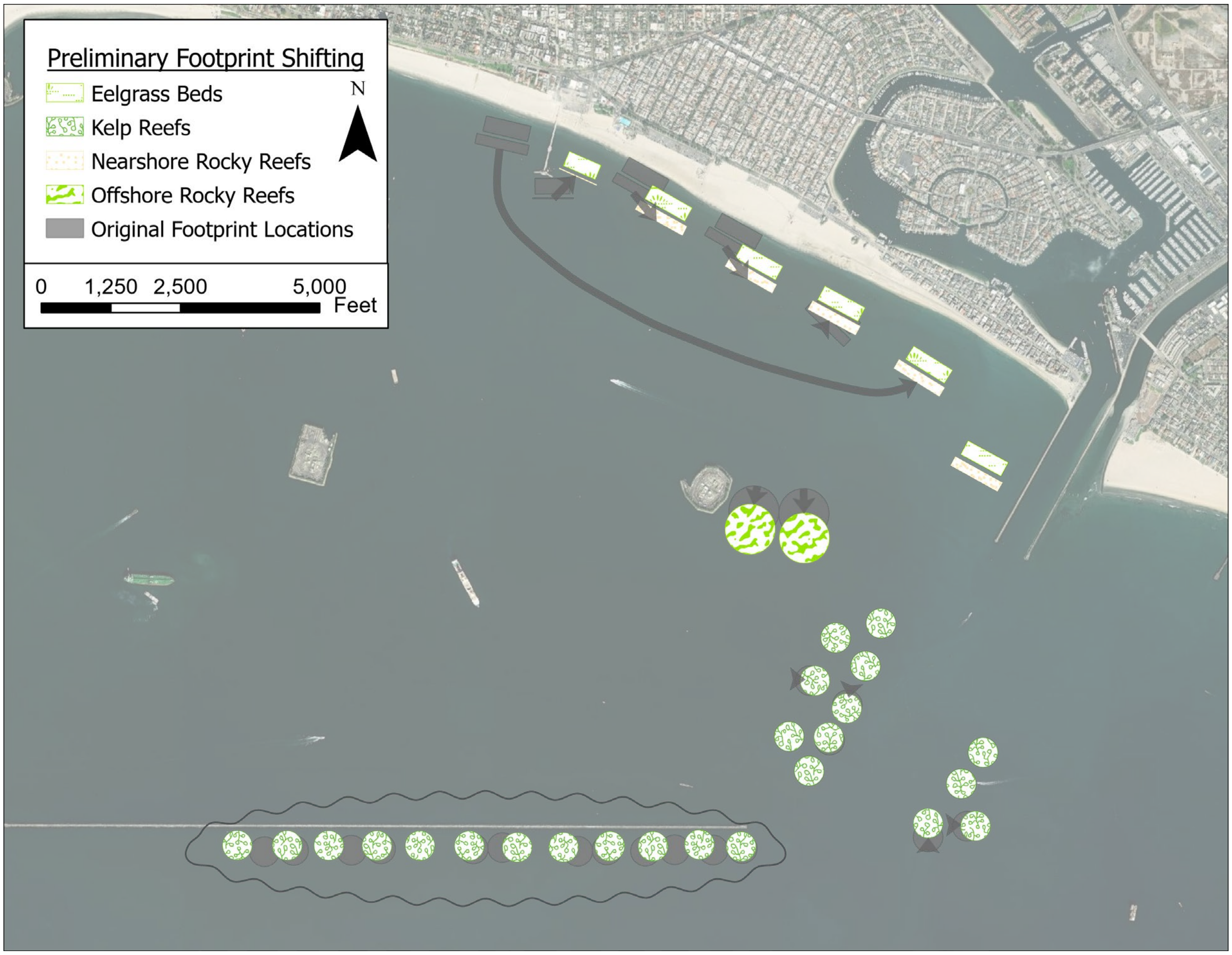

Figure 9-1: Measure Shifting in Response to Environmental and Cultural Resources

9.3.1 CONSTRUCTION SCHEDULE

The previously discussed construction duration and schedule does not account for sufficient stone production throughout major components of the project. When separating individual work elements based on available or obtainable quantities into discrete construction contracts, total construction durations are extended. This is in part due to requirements to award separate construction contracts and avoid weather windows along with ensuring sufficient quantity of material is available when needed. A preliminary schedule for construction is shown in Figure 9-2. This assumes that most construction will not progress through the winter weather window from December to April. Additionally, to ensure no construction impacts relating to the Olympics, which is planned for the summer of 2028, no construction is anticipated between September 2027 and March 2029. This schedule will be refined and altered during the PED phase but is shown to present a sequential method of construction phasing at this stage in the study.

Contracts are separated based on the required stone quantity and estimated production rates at local quarries for the specified stone, approximately 90,000 tons/contract. Stone production will continue during the winter blackout windows to maintain sufficient material through the remaining working months of the year. Separation of the larger contracts will also limit the required stockpile at the quarry, staging areas or other mooring areas (if stone is stored on a barge).

Construction durations are a combination of the anticipated production rate, mobilization and demobilization from the project site, relocation of construction equipment within the site once mobilized, potential weather delays, contract administration and other unexpected delays such as equipment breakdown. Construction durations discussed in APPENDIX B: COST ENGINEERING typically only include material production rates and do not include the other factors stated above. Total construction working days are shown in Table 9-11 and Table 9-12.

Table 9-11: Production Rates and Construction Durations (Contracts 1 - 3)

	(1) Construct Breakwater Kelp Reefs	(2) Construct Nearshore Reefs			(3) Construct Nearshore Reefs		
Material	Stone*	Armor*	Filter*	Core*	Armor*	Filter*	Core*
Estimated Quantity	66,000	88,000	27,500	67,000	88,000	27,500	67,000
Unit	ton	ton	ton	ton	ton	ton	ton
Production Rate	140	25	40	150	25	40	150
Unit	ton/hr	ton/hr	ton/hr	ton/hr	ton/hr	ton/hr	ton/hr
Placement Duration (day)	47.1	352.0	68.8	44.7	352.0	68.8	44.7
Mobilization / Demobilization (day)	45	75			75		
Administration (day)	45	45			45		
Relocations / Delays (day)	20	60			65		
Weather (day)	10	30			40		
Total Construction Days (excluding winter blackout)	168	676			691		

Table 9-12: Production Rates and Construction Durations (Contracts 4 - 7)

	(4) Construct Eelgrass		(5) Construct Open Water Reef	(6) Construct Open Water Reef	(7) Construct Open Water Kelp Reefs
Material	Dredging**	Eelgrass*	Armor*	Armor*	Stone*
Estimated Quantity	100,000	263,800	91,300	91,300	66,000
Unit	yd^3	ft^2	ton	ton	ton
Production Rate	4,000	7,500	25	25	140
Unit	yd^3/day	ft^2/day	ton/hr	ton/hr	ton/hr
Placement Duration (day)	25.0	35.2	365.2	365.2	47.1
Mobilization / Demobilization (day)	55		55	55	45
Administration (day)	30		30	30	30
Relocations / Delays (day)	5		40	40	20
Weather (day)	5		20	20	10
Total Construction Days (excluding winter blackout)	156		511	511	153

		Name	Duration	Start	Finish	Predecessors
1		**Kelp Reefs (Breakwater - x12)**	**686 days**	**10/1/25 8:00 AM**	**8/17/27 5:00 PM**	
2		PED	364 days	10/1/25 8:00 AM	9/29/26 5:00 PM	
3		1. Award	1 day	9/30/26 8:00 AM	9/30/26 5:00 PM	2
4		**Construction**	**321 days**	**10/1/26 8:00 AM**	**8/17/27 5:00 PM**	
5		Submittals	28 days	10/1/26 8:00 AM	10/28/26 5:00 PM	3
6		WinterBlackout	125 days	10/29/26 8:00 AM	3/2/27 5:00 PM	5
7		Construction	168 days	3/3/27 8:00 AM	8/17/27 5:00 PM	6
8		**Olympics**	**472 days?**	**9/1/27 8:00 AM**	**12/15/28 5:00 PM**	
9		Olympics Balckout	472 days?	9/1/27 8:00 AM	12/15/28 5:00 PM	7
10		**Nearshore Reefs (x6)**	**2,703 days**	**10/29/26 8:00 AM**	**3/23/34 5:00 PM**	
11		PED	547 days	10/29/26 8:00 AM	4/27/28 5:00 PM	5
12		2. Award	1 day	12/16/28 8:00 AM	12/16/28 5:00 PM	9;11
13		**Construction - Contract #1 (x3)**	**930 days**	**12/17/28 8:00 AM**	**7/4/31 5:00 PM**	
14		Submittals	44 days	12/17/28 8:00 AM	1/29/29 5:00 PM	12
15		Winter Blackout	30 days	1/30/29 8:00 AM	2/28/29 5:00 PM	14
16		Construction	275 days	3/1/29 8:00 AM	11/30/29 5:00 PM	15
17		Winter Blackout	90 days	12/1/29 8:00 AM	2/28/30 5:00 PM	16
18		Construction	275 days	3/1/30 8:00 AM	11/30/30 5:00 PM	17
19		Winter Blackout	90 days	12/1/30 8:00 AM	2/28/31 5:00 PM	18
20		Construction	126 days	3/1/31 8:00 AM	7/4/31 5:00 PM	19
21		PED	142 days	12/1/29 8:00 AM	4/21/30 5:00 PM	16
22		3. Award	1 day	7/5/31 8:00 AM	7/5/31 5:00 PM	20;21
23		**Construction - Contract #2 (x3)**	**992 days**	**7/6/31 8:00 AM**	**3/23/34 5:00 PM**	
24		Submittals	30 days	7/6/31 8:00 AM	8/4/31 5:00 PM	22
25		Construction	118 days	8/5/31 8:00 AM	11/30/31 5:00 PM	24
26		Winter Blackout	91 days	12/1/31 8:00 AM	2/29/32 5:00 PM	25
27		Construction	275 days	3/1/32 8:00 AM	11/30/32 5:00 PM	26
28		Winter Blackout	90 days	12/1/32 8:00 AM	2/28/33 5:00 PM	27
29		Construction	275 days	3/1/33 8:00 AM	11/30/33 5:00 PM	28
30		Winter Blackout	90 days	12/1/33 8:00 AM	2/28/34 5:00 PM	29
31		Construction	23 days	3/1/34 8:00 AM	3/23/34 5:00 PM	30
32		**Stone Production**	**2,640 days**	**7/5/31 8:00 AM**	**9/25/38 5:00 PM**	
33		Nearshore Reefs	120 days	7/5/31 8:00 AM	11/1/31 5:00 PM	20
34		Openwater Reefs	180 days	3/24/34 8:00 AM	9/19/34 5:00 PM	31
35		Kelp Reefs	1 day	9/25/38 8:00 AM	9/25/38 5:00 PM	59
36		**Eelgrass & Dredging**	**1,411 days**	**4/22/30 8:00 AM**	**3/2/34 5:00 PM**	
37		PED	180 days	4/22/30 8:00 AM	10/18/30 5:00 PM	21
38		4. Award	1 day	7/9/32 8:00 AM	7/9/32 5:00 PM	25;37
39		**Construction**	**183 days**	**9/1/33 8:00 AM**	**3/2/34 5:00 PM**	
40		Submittals	28 days	9/1/33 8:00 AM	9/28/33 5:00 PM	38
41		Construction	155 days	9/29/33 8:00 AM	3/2/34 5:00 PM	40
42		**Open Water Reefs (x2)**	**1,646 days**	**3/24/34 8:00 AM**	**9/24/38 5:00 PM**	
43		PED	366 days	3/24/34 8:00 AM	3/24/35 5:00 PM	31;40
44		5. Award	1 day	3/25/35 8:00 AM	3/25/35 5:00 PM	43
45		**Construction - Contract #1 (x1)**	**720 days**	**3/26/35 8:00 AM**	**3/14/37 5:00 PM**	
46		Submittals	28 days	3/26/35 8:00 AM	4/22/35 5:00 PM	44
47		Construction	192 days	4/23/35 8:00 AM	10/31/35 5:00 PM	46
48		WinterBlackout	91 days	11/1/35 8:00 AM	1/30/36 5:00 PM	47
49		Construction	275 days	1/31/36 8:00 AM	10/31/36 5:00 PM	48
50		WinterBlackout	90 days	11/1/36 8:00 AM	1/29/37 5:00 PM	49
51		Construction	44 days	1/30/37 8:00 AM	3/14/37 5:00 PM	50
52		PED	180 days	4/23/35 8:00 AM	10/19/35 5:00 PM	46
53		6. Award	1 day	11/1/36 8:00 AM	11/1/36 5:00 PM	34;49;52
54		**Construction - Contract #2 (x1)**	**692 days**	**11/2/36 8:00 AM**	**9/24/38 5:00 PM**	
55		Submittals	28 days	11/2/36 8:00 AM	11/29/36 5:00 PM	53
56		Winter Blackout	62 days	11/30/36 8:00 AM	1/30/37 5:00 PM	55
57		Construction	274 days	1/31/37 8:00 AM	10/31/37 5:00 PM	56
58		Winter Blackout	91 days	11/1/37 8:00 AM	1/30/38 5:00 PM	57
59		Construction	237 days	1/31/38 8:00 AM	9/24/38 5:00 PM	58
60		**Kelp Reefs (Open Water - x12)**	**849 days**	**11/30/36 8:00 AM**	**3/28/39 5:00 PM**	
61		PED	160 days	11/30/36 8:00 AM	5/8/37 5:00 PM	55
62		7. Award	1 day	9/26/38 8:00 AM	9/26/38 5:00 PM	35;61
63		**Construction**	**183 days**	**9/27/38 8:00 AM**	**3/28/39 5:00 PM**	
64		Submittals	28 days	9/27/38 8:00 AM	10/24/38 5:00 PM	62
65		Construction	155 days	10/25/38 8:00 AM	3/28/39 5:00 PM	64

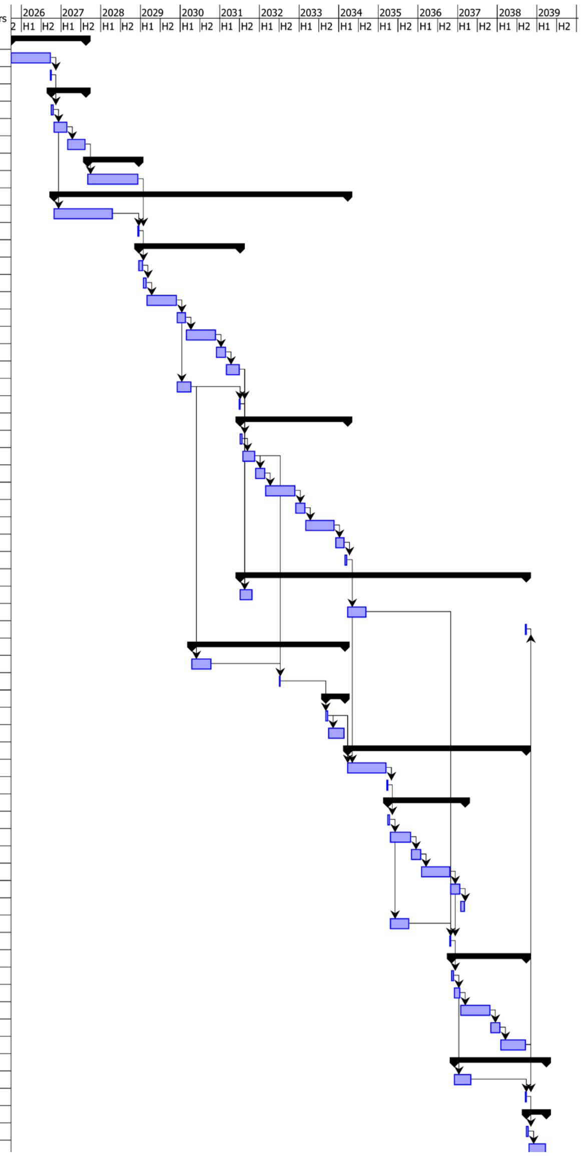

Figure 9-2: Preliminary Design & Construction Schedule

Due to the modification of the construction schedule from the previous discussion in the FINAL ARRAY OF ALTERNATIVES section, durations for all activities are extended. Assuming that the first 30 to 45 days of construction is administrative work and winter days do not account for days worked, the actual number, in calendar days, of all activities are shown in Table 9-13; numbers in parentheses are the contract number presented in the above schedule. The table also indicated the number of transportation trips required for construction. Construction durations are slightly different from Table 9-11 and Table 9-12 due to rounding differences.

Table 9-13: Active Construction Duration and Transportation

Activity	Active Days	Estimated Quantity	Unit	Pebbly Beach Quarry (Catalina)			3M Quarry			Other (Surfside/Sunset)		
				Material loaded directly onto a barge and transported to site			*Material loaded onto truck, transported to storage area, unloaded, re-loaded to barge and transported to site*			*[1] Material dredged, transported and placed on site [2] Eelgrass obtained in 24 batches*		
				Rate (Unit/Trip)	Total Estimated Trips	Approx. Trips/Day	Rate (unit/trip)	Total Estimated Trips	Approx. Trips/Day	Rate (unit/trip)	Total Estimated Trips	Approx. Trips/Day
(1) Construct Breakwater Kelp Reefs	168	66,000	ton	4,000	16.5	0.10	20	3,300	19.64	-	-	-
(2) Construct Nearshore Reefs	676	182,500	ton	3,500	52.1	0.08	20	9,125	13.50	-	-	-
(3) Construct Nearshore Reefs	691	182,500	ton	3,500	52.1	0.08	20	9,125	13.21	-	-	-
(4) Construct Eelgrass	155	100,000	yd^3	-	-	-	-	-	-	2,000	74.00	0.48
(5) Construct Open Water Reef	512	91,300	ton	3,500	26.1	0.05	20	4,565	8.92	-	-	-
(6) Construct Open Water Reef	512	91,300	ton	3,500	26.1	0.05	20	4,565	8.92	-	-	-
(7) Construct Open Water Kelp Reefs	155	66,000	ton	4,000	16.5	0.11	20	3,300	21.29	-	-	-

9.3.2 CONSTRUCTION METHODS

Previously discussed constructions methods will remain as before. Additional information of the derrick crane, rock barge and support vessels has been requested at this stage in the study. Exact methods during construction will vary depending on the Contractor's experience, expertise and equipment, but the below gives a general overview of potential anchoring and vessels placement during construction.

Anchoring of a derrick crane barge and rock barge is conducted with four anchor wires connected to moorings that lie on the seabed. Figure 9-3 shows the typical layout of the anchoring in relation to the barge position. Each mooring is marked by a floating buoy to indicate a submerged feature is present. The buoys are lighted during the nighttime hours and contain a radar reflector to show up on a nearby vessel's interactive charting system.

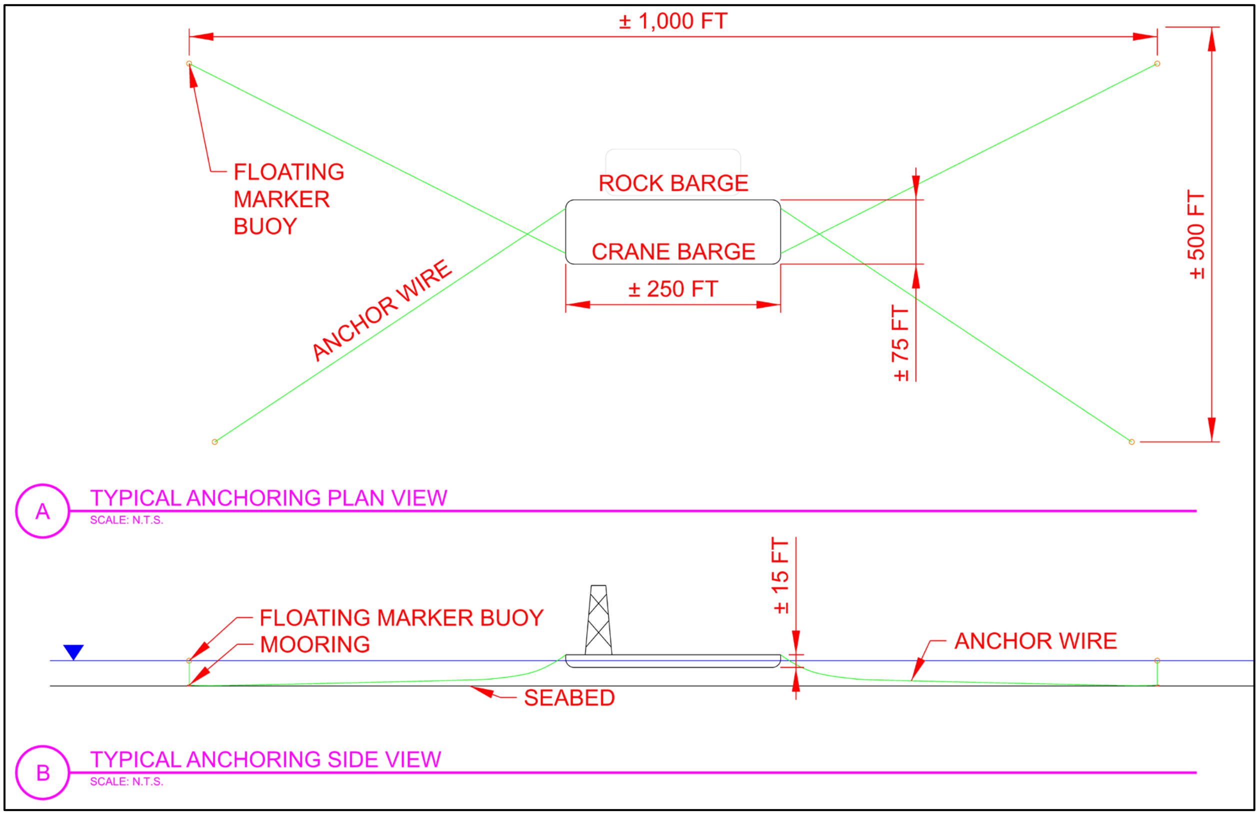

Figure 9-3: Typical Anchoring Plan for Stone Placement

Placement of the moorings is conducted by a support tug. During the initial mobilization, the moorings would be dropped from the tug in the approximate locations, the anchor wires will be pulled taut to limit the motion of the crane and rock barges. Some movement of the moorings is expected on the seabed as the wires are drawn in or as wave and current forces act on the moored system. Once set, the barge is allowed to adjust the position within the moorings by adjusting the lengths of the anchor wires; the wires allow for movement within a box approximately 500,000 ft². Through the construction process, wires can be in tension or in slack as needed to maneuver the barge to the construction footprint. Wires in slack can scrape the seabed as they experience forces due to waves, currents and vessel motion. Supplementary anchors may be required during periods of large swell. When a single anchor spread cannot reach the entire construction footprint, the mooring will be moved. This is accomplished similarly to initial placement although instead of lifting the moorings from the seabed and transporting above water, the moorings are dragged over the seabed by the support tug into the final position. This will create a shallow trench in the seabed sediments which will infill over time by natural processes. Care will be taken to avoid existing habitats and submerged objects during initial and subsequent anchor placement and movements. When a feature of construction is completed, the mooring can be dragged within the reach of the crane barge where they are lifted and placed on the deck. The entire barge setup is relocated to the next feature and the process repeats. On-the-spot adjustments will be made to anchor spread and the barge position depending on the construction footprint, nearby obstructions, type of work and sea state conditions.

Preliminary anchor placements for each habitat feature is shown in Figure 9-4. Since the feature footprints are conceptual at this point and will change as a result of PED, the anchoring plan will adjust accordingly in response. The final anchoring plan will need to be developed in coordination with the construction contractor based on their equipment and operating requirements. Full bottom coverage surveys will be conducted during this PED phase which will help guide this placement plan before construction begins.

The figure indicates the barge placement over each measure during construction and the corresponding anchor points signified by the ends of the anchor wire lines. The top panel, A, presents an overview of the restoration locations with the colored boxes corresponding to the colored panels below. The kelp reefs, shown in panels B and C for the breakwater and open water locations, will require a few anchor moves after the initial placement. When progressing to adjacent reefs, the anchor placement must be coordinated as to not damage previously constructed aspects of the restoration. Similarly, the open water reef complexes will require an initial setting followed by some additional moves per reef. The nearshore reefs should only require the initial setting per structure; the barge has sufficient room to reach the limits of the reef without moving the moorings but may need to be resituated depending on the conditions at the time. If smaller equipment is used than assumed in this report, more anchor moves will be required. Reversely, if larger equipment, then less moves will be required. When working near eelgrass or other debris to avoid, onboard GPS will guide the placement to ensure no impacts on existing habitats or features is realized.

The bottom panels, H and I, show the expected transits of a tug and scow to deposit sediments to bolster the eelgrass beds. Each eelgrass bed will require somewhere between 8 to 16 transits (8 shown). The tug and scow will enter the feature footprint from a single side. Once over the footprint the scow doors will open and deposit the sediment. The rate of deposition can be regulated by the timing and the magnitude of the door opening. Not all material in the scow will be deposited during a single transect, so multiple runs will be required for each full scow. This thin layer placement of material has been successively accomplished in other west coasts locations, most notably, at the mouth of the Columbia River on the boarder of Washington and Oregon. As shown in panel H, a specified buffer from known and historic eelgrass can be established. Scows will be restricted from depositing material within these buffer limits. Exclusion zones shown here and determined during PED can be imported into the transiting vessel’s onboard heads up display with real-time vessel tracking.

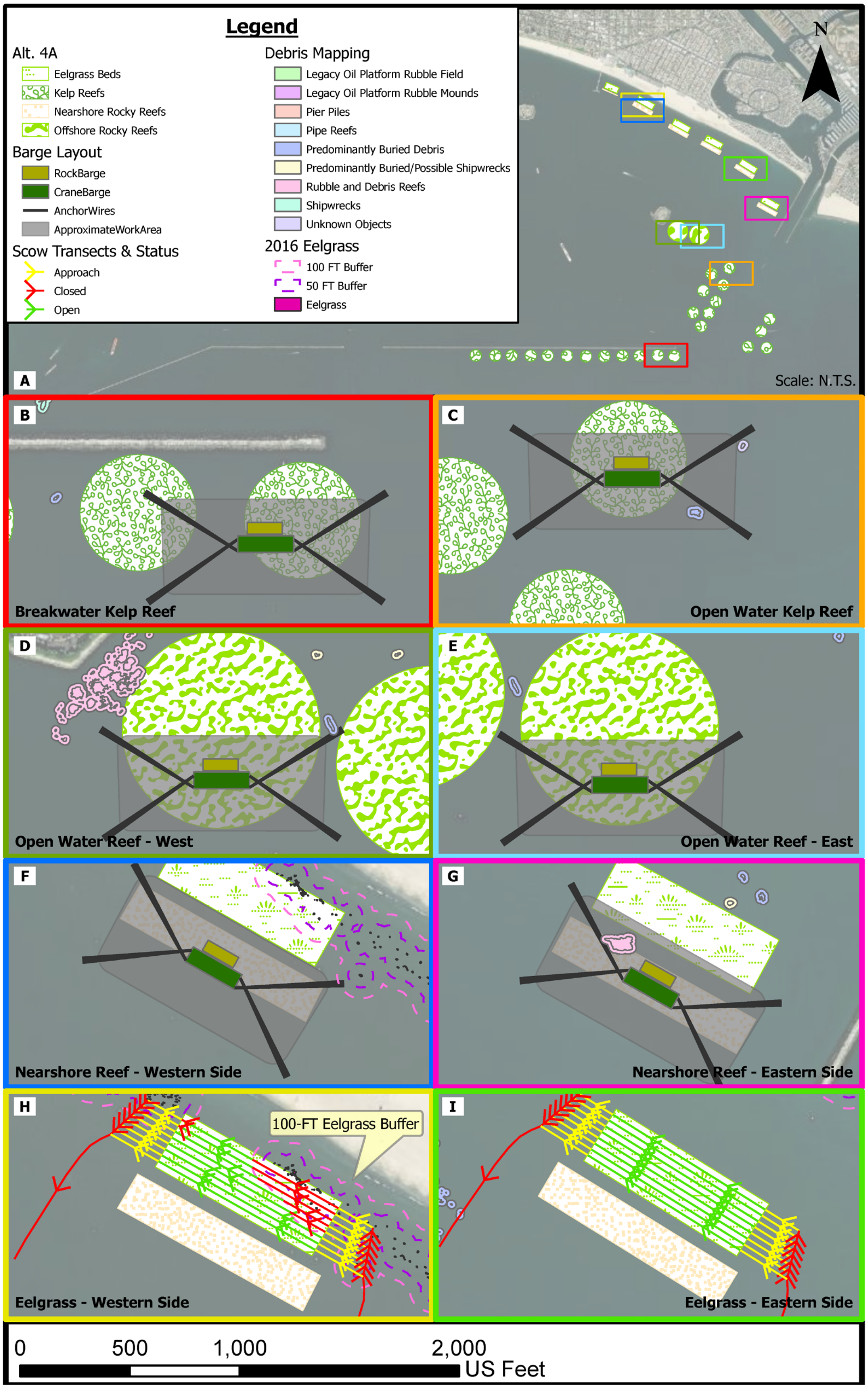

Figure 9-4: Preliminary Anchoring Layouts

9.3.3 DREDGING PLAN

As part of the recommended plan, approximately 100,000 yd³ of sand will be obtained from the nearby Surfside-Sunset borrow area for the federal Surfside-Sunset Coastal Storm Risk Management Project (Surfside-Sunset). This material has been used to nourish the Orange County shoreline for the last 60 years. There is a sufficient quantity of suitable beach material, more than 42,000,000 yd³ determined from sediment sampling and sub-bottom profiling, to supply this project in addition to the ongoing federal Surfside-Sunset project. The sediment was chemically tested as deemed suitable for direct shoreline placement in 2018 but will be repeated to confirm the sediment still meets the requirements as the dredging phase of the project nears. This material lies in water depths from 40 – 60 feet; well below the depth of closure where little sediment is mobilized. Only the top 3 – 4 feet of sediment will be dredged from the borrow area. The borrow area and project site are separated by approximately 4 miles, shown in Figure 9-5. During the PED phase, the borrow area location will be further refined within the bounds shown. It is expected that only 20 acres, out of the more than 1,700 acres shown, will be needed for borrow.

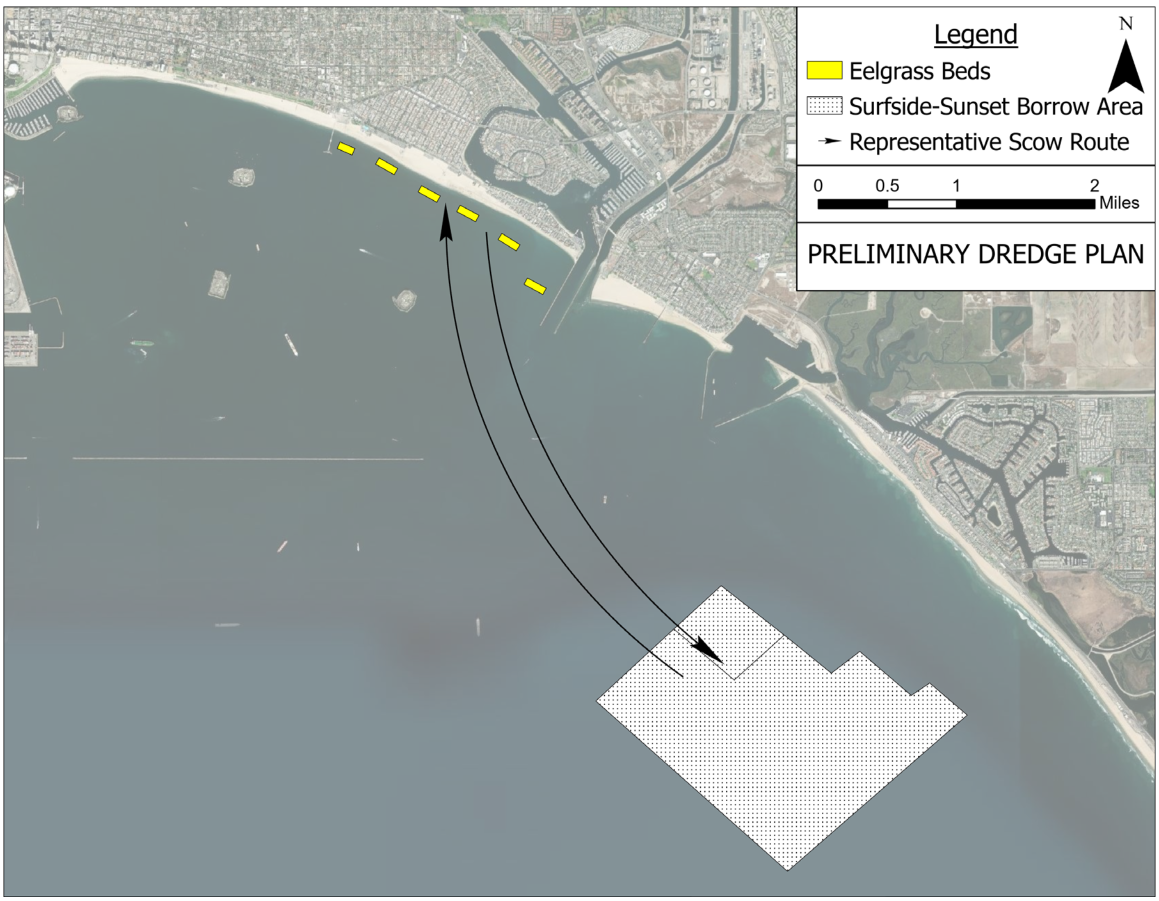

Figure 9-5: Preliminary Dredging Plan

At the borrow area, the mechanical dredge will be held in place by anchors with placement similar to that of the crane and rock barges previously described. The dredger may elect to use spuds, or long piles that can be raised and lowered by the dredge and act as an anchor. These spuds will penetrate the seabed for a few feet and hold the dredge in a fixed position. The spuds only rely on gravity to penetrate the seabed; they will not be mechanically driven. Spuds can only be used in a lower wave energy environment, so anchors will be the preferable method at this borrow area. After the scow is towed to the project site, the haul is opened to deposit material in the specified location. The rate of placement can be controlled by the size and speed of the opening haul. Pilot studies on the west coast indicate thin layers of sediment can be placed using this method without interrupting the natural ecosystem (Moritz et al, 2019).

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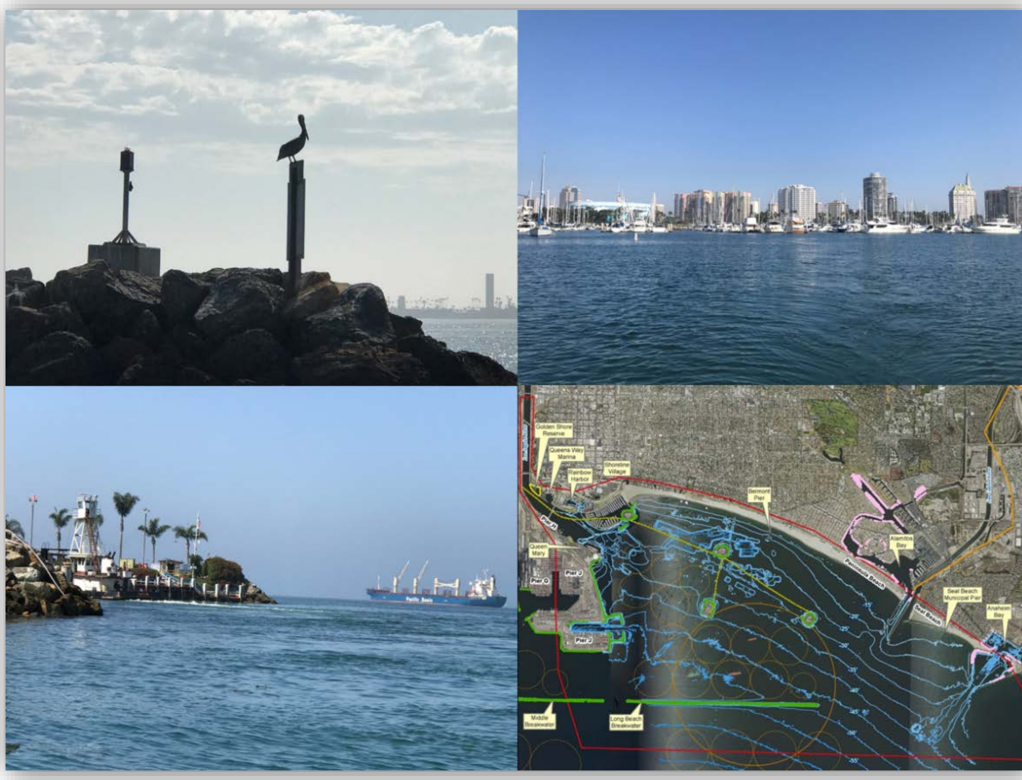
FINAL INTEGRATED FEASIBILITY REPORT AND ENVIRONMENTAL IMPACT STATEMENT / ENVIRONMENTAL IMPACT REPORT (EIS/EIR)

APPENDIX A-1: EFDC MODELING

EAST SAN PEDRO BAY ECOSYSTEM RESTORATION STUDY

Long Beach, California

January 2022



US Army Corps of Engineers ®

HYDRODYNAMIC MODELING SUPPORT FOR THE EAST SAN PEDRO BAY ECOSYSTEM RESTORATION PROJECT

Prepared For:

City of Long Beach
U.S. Army Corps of Engineers, Los Angeles District

Prepared By:

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October 2017

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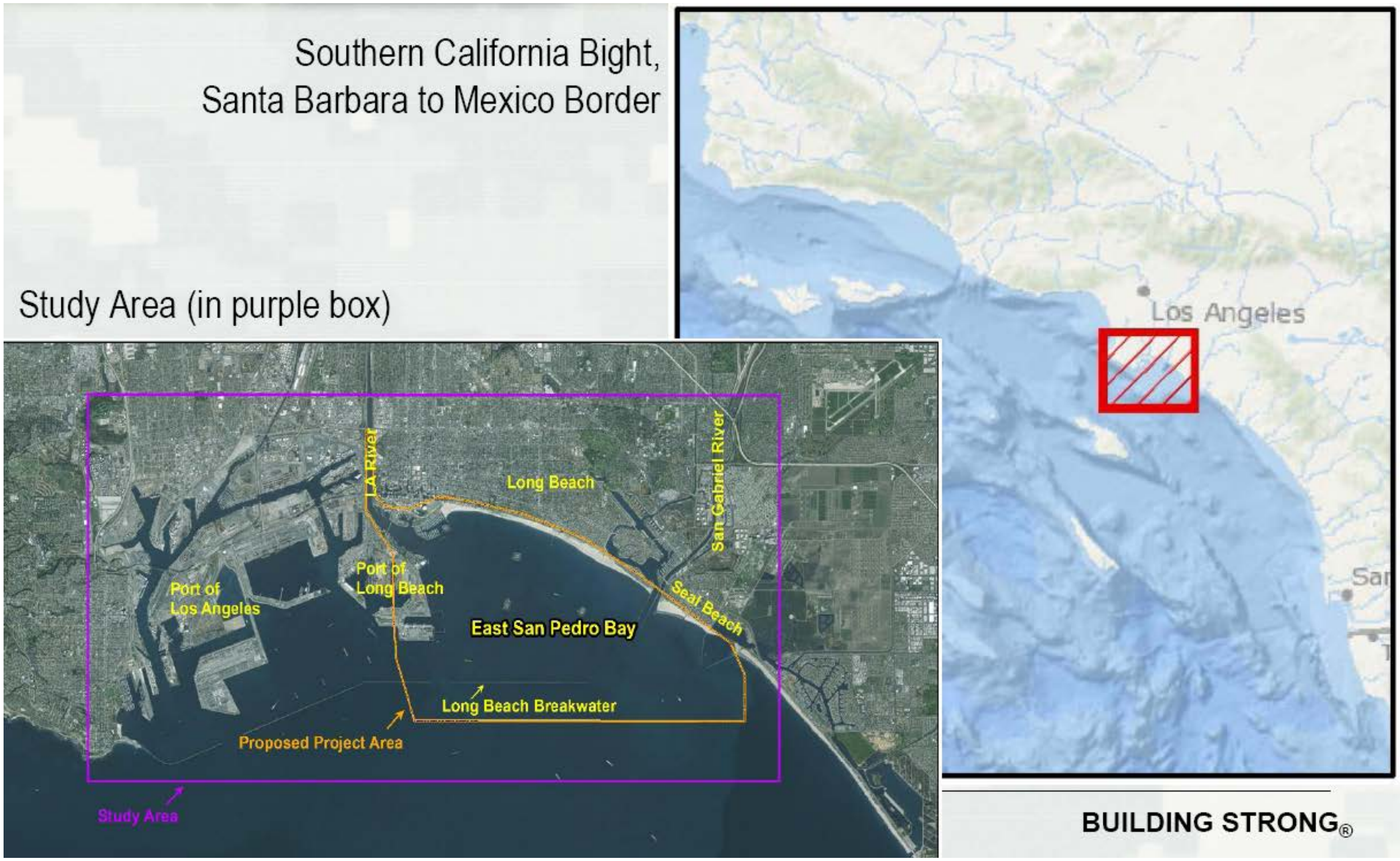
1. INTRODUCTION

The City of Long Beach (City) and U.S. Army Corps Engineers, Los Angeles District (USACE) are conducting the East San Pedro Bay Ecosystem Restoration Project to evaluate opportunities for providing ecosystem restoration, increased recreational opportunities, and other improvements within East San Pedro Bay (ESPB). The ESPB Ecosystem Restoration Project (Project) aims to restore and improve aquatic ecosystem structure and function for increased habitat biodiversity and ecosystem value of the Southern California Bight within ESPB. The project location and proposed restoration area are shown in Figure 1.1. Proposed restoration alternatives will include the restoration of aquatic habitat such as kelp, rocky reef, and coastal wetlands to support diverse resident and migratory species within ESPB (USACE 2016).

Based on restoration opportunities and constraints at ESPB, USACE has screened a wide range of restoration alternatives to select the final six alternatives to be modeled. Details of these six alternatives are provided in Section 2. USACE will conduct habitat evaluations for these restoration alternatives and the existing (baseline) condition utilizing a habitat evaluation model, and will conduct wave modeling using CMS-Wave Model to determine surface wave conditions for ESPB. To support the evaluation of restoration alternatives, a hydrodynamic and water quality model of ESPB is needed to provide information on currents and water quality conditions under existing and alternative conditions. The City has retained Anchor QEA, LLC and Everest International Consultants, Inc. to provide hydrodynamic modeling support for the Project. This document summarizes the data and methodology used in the development of the hydrodynamic model, and the use the developed model to simulate hydrodynamic and water quality conditions in ESPB.

The hydrodynamic model of ESPB was developed based on a previously developed and calibrated model of the Los Angeles and Long Beach Harbor (LA/LB Harbor) and San Pedro Bay. Details of the model development for this study are summarized in Section 3. Results of the hydrodynamic modeling and comparisons of model scenarios are presented in Section 4. A numeric tracer tracking analysis, provided in Section 5, was also conducted using the hydrodynamic modeling. Lastly, a summary of the major findings of the study is provided in Section 6. All figures for each section are included at the end of their respective sections.



Source: USACE 2016

Figure 1.1 East San Pedro Bay Ecosystem Restoration Project Area

2. RESTORATION ALTERNATIVES

Six restoration alternatives were evaluated as part of the hydrodynamic modeling. These restoration alternatives were defined by combinations of aquatic habitat sites and structural features that included:

- Eelgrass beds,
- Intertidal rock/sand shoals or sandy islands,
- Oyster beds,
- Kelp beds,
- Rocky reefs,
- Coastal wetlands,
- Breakwater notching,
- Breakwater lowering, and
- A training wall.

The restoration alternatives were developed to increase habitat biodiversity and improve bay-wide circulation. Existing conditions and six restoration alternatives were modeled based on structural and habitat features that may impact the hydrodynamics in ESPB. Structural features included modifications to the existing breakwater. Aquatic habitats were simulated by modeling bathymetric changes to the shoals and islands.

2.1 Existing Conditions and Model Scenarios

Bathymetry within ESPB, under existing conditions, is shown in Figure 2.1. Bathymetries, and structural and aquatic habitat features for the six restoration alternatives, as provided by USACE (2017a), are illustrated in Figures 2.2 – 2.7.

As shown in Figure 2.2, there will be no structural changes under Scenario 1. This scenario only has aquatic habitats that include four rocky/sand nearshore shoals along the shoreline and wetlands in the Los Angeles River Estuary and near Pier J.

Scenario 2 includes two notches in the existing breakwater to the rocky/sand nearshore shoals and wetlands, as illustrated in Figure 2.3. Located approximately 800 m from the eastern end of the breakwater, these notches each measure about 220 m wide.

For Scenario 3, the eastern third of the breakwater is lowered, while rocky/sand nearshore shoals and wetlands are added, as provided in Figure 2.4.

As shown in Figure 2.5, Scenario 4 adds a training wall along the mouth of the Los Angeles River Estuary to divert flows from the Los Angele River away from the ESPB. Aquatic habitats for Scenario 4 include the wetlands, three rocky/sand nearshore shoals, and two emergent islands that are situated upcoast from Alamitos Bay.

Scenario 5 includes the lowering of the eastern third of the breakwater, as illustrated in Figure 2.6. This scenario is same as Scenario 3, except with the additions of the emergent islands along the shoreline just north of the San Gabriel River mouth.

For Scenario 6, as shown in Figure 2.7, the entire breakwater is lowered. There are no aquatic habitats for this scenario.

The structural features and aquatic habitats for these six model scenarios to be evaluated as part of the hydrodynamic modeling are summarized in Table 2.1.

Table 2.1 Model Scenarios for Hydrodynamic Modeling Study

Model Scenario	Structural Features	Aquatic Habitats
Existing Conditions*	No changes	No changes
Scenario 1	No changes	Rocky/sand nearshore shoals Wetlands
Scenario 2*	Breakwater notching	Rocky/sand nearshore shoals Wetlands
Scenario 3*	Breakwater lowering of eastern third	Rocky/sand nearshore shoals Wetlands
Scenario 4*	Training wall	Rocky/sand nearshore shoals Wetlands Emergent islands
Scenario 5	Breakwater lowering of eastern third	Rocky/sand nearshore shoals Wetlands Emergent islands
Scenario 6*	Lowering entire breakwater	No changes

**Scenario included in particle tracer tracking analysis*

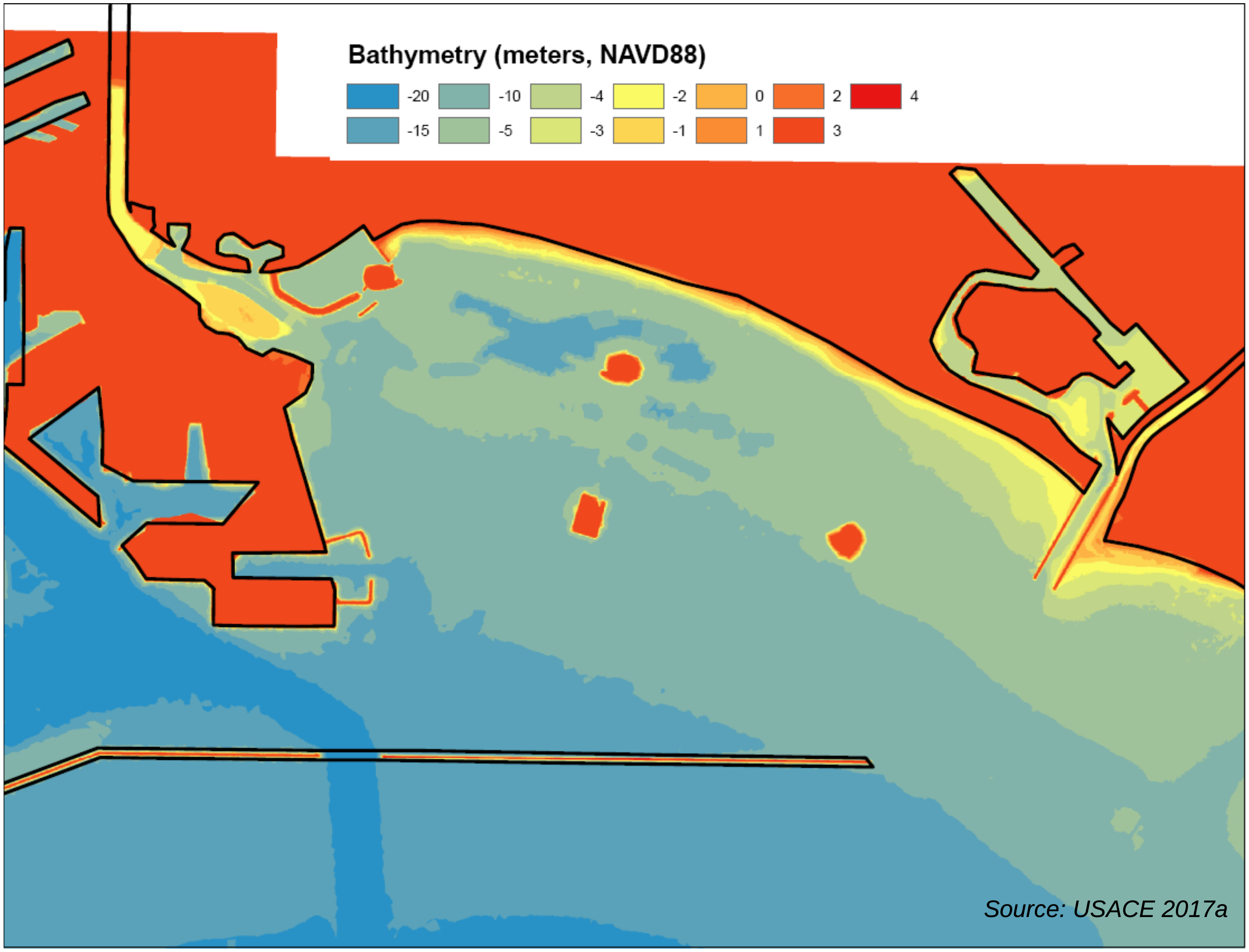


Figure 2.1 East San Pedro Bay Ecosystem Restoration Existing Conditions

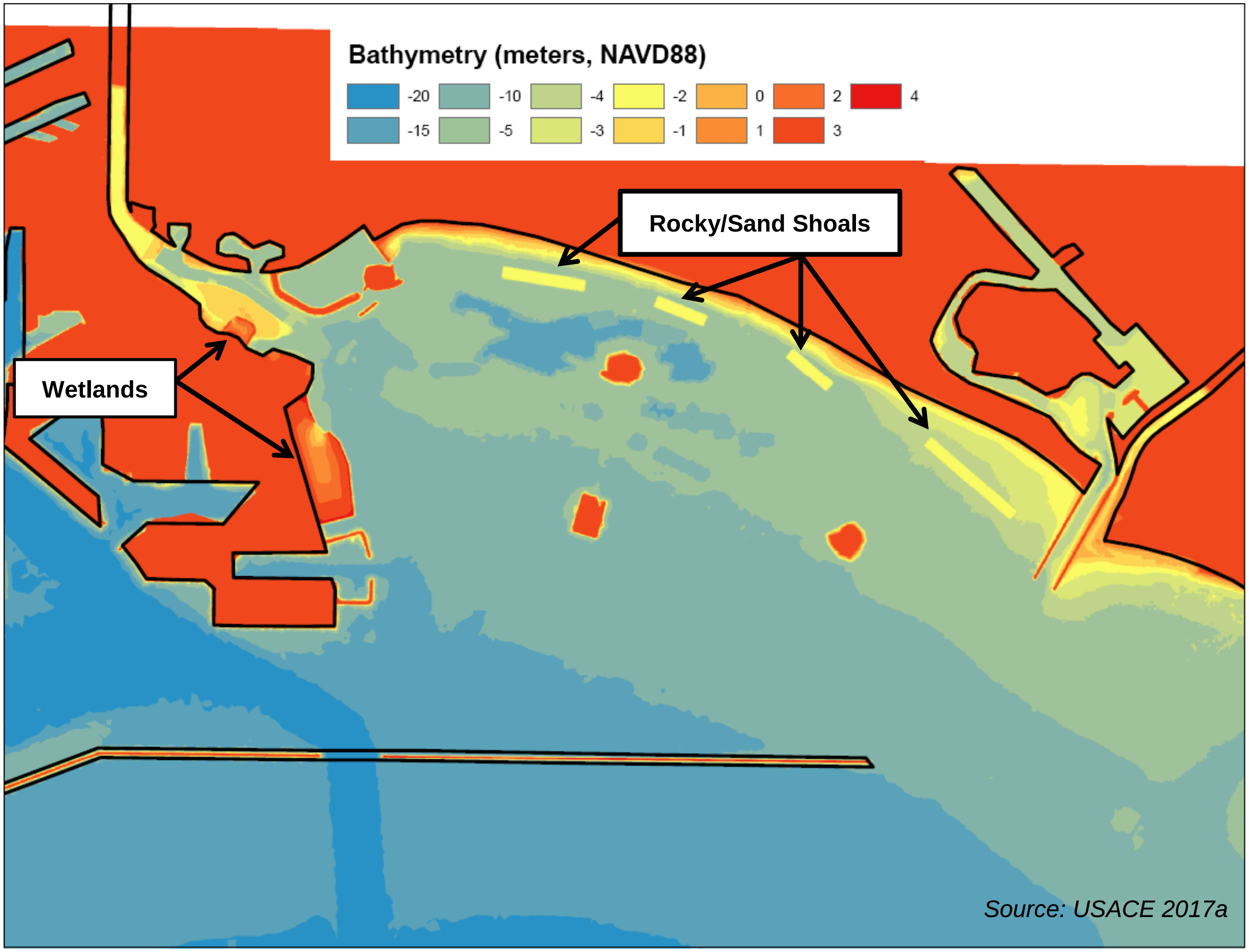


Figure 2.2 East San Pedro Bay Ecosystem Restoration Scenario 1

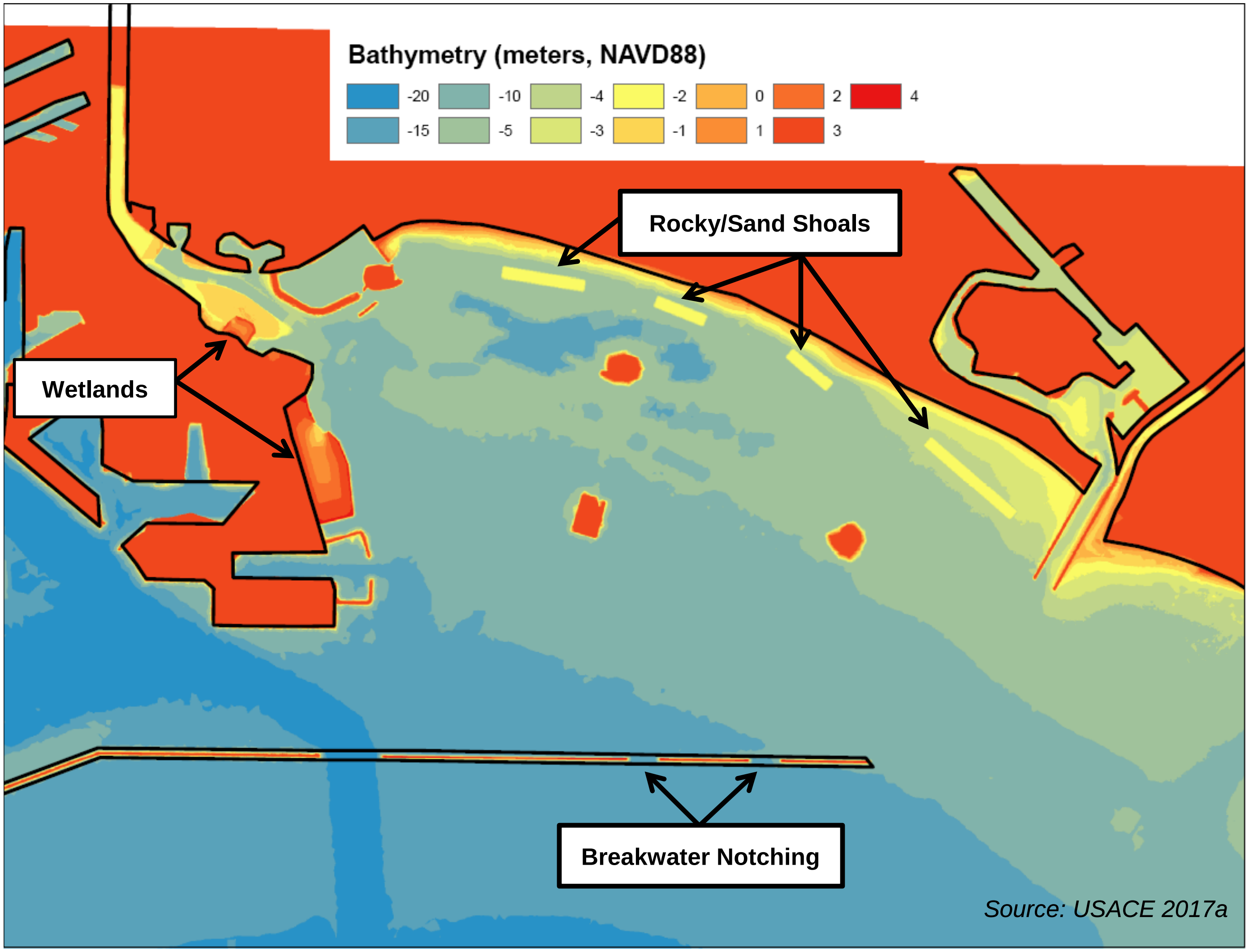


Figure 2.3 East San Pedro Bay Ecosystem Restoration Scenario 2

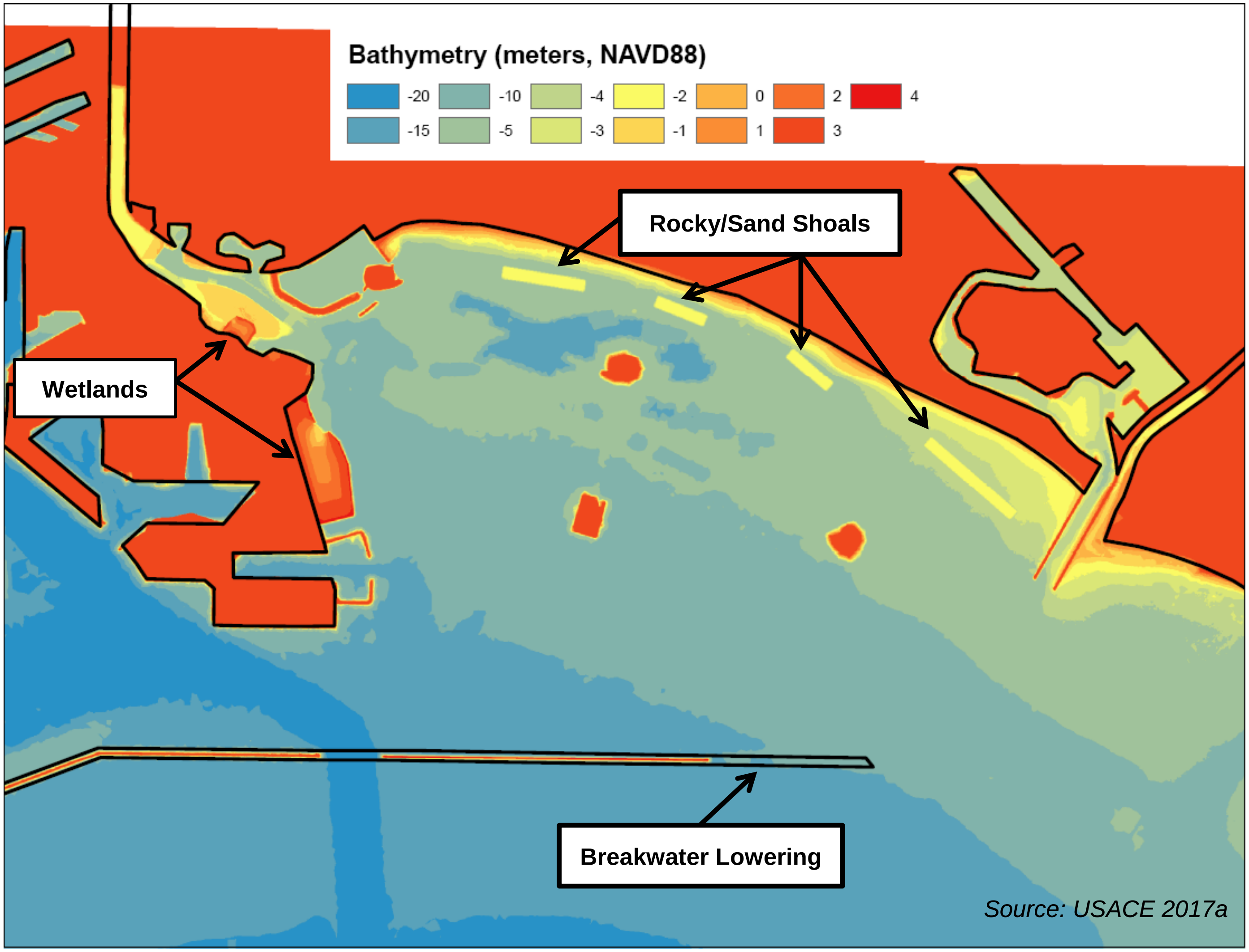


Figure 2.4 East San Pedro Bay Ecosystem Restoration Scenario 3

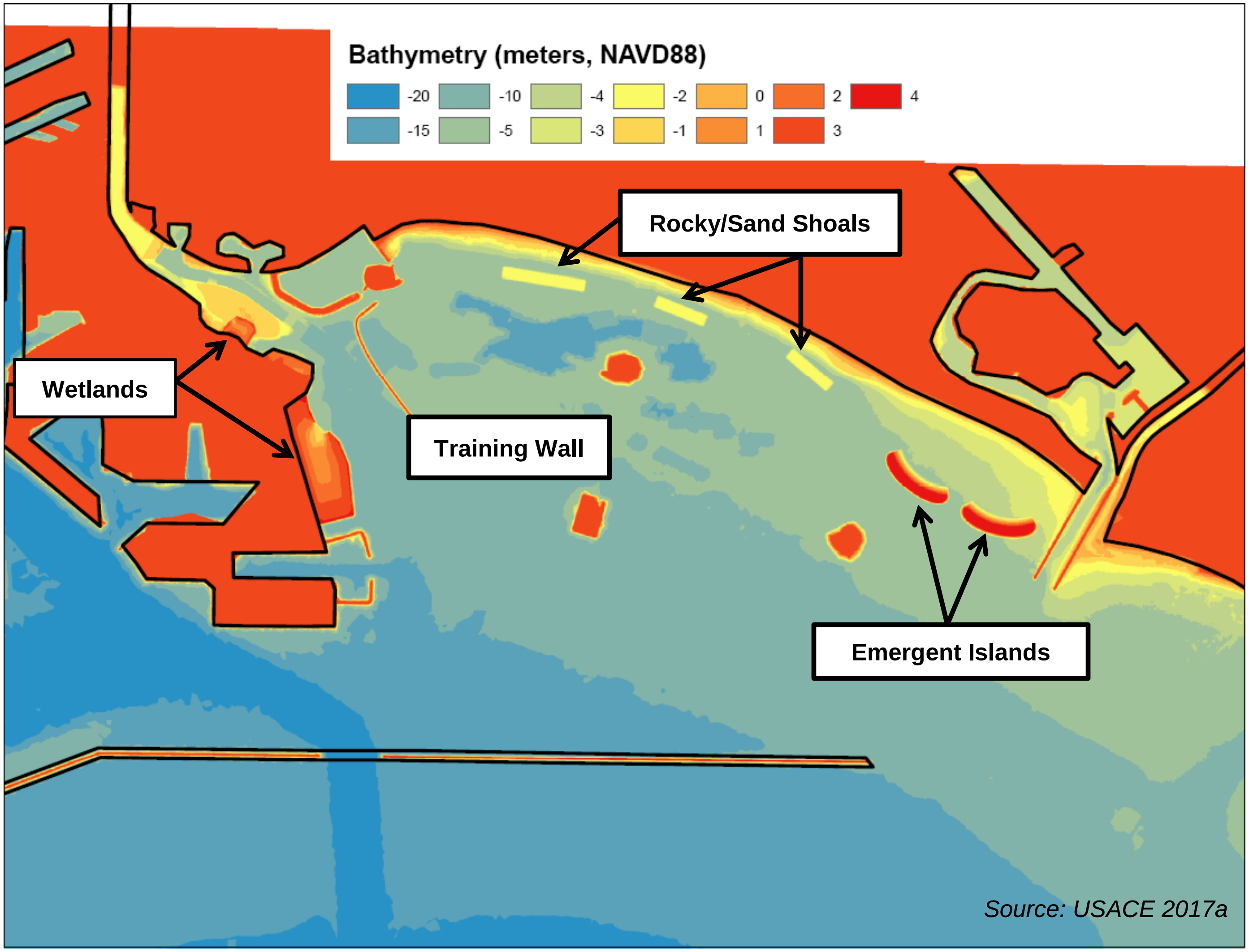


Figure 2.5 East San Pedro Bay Ecosystem Restoration Scenario 4

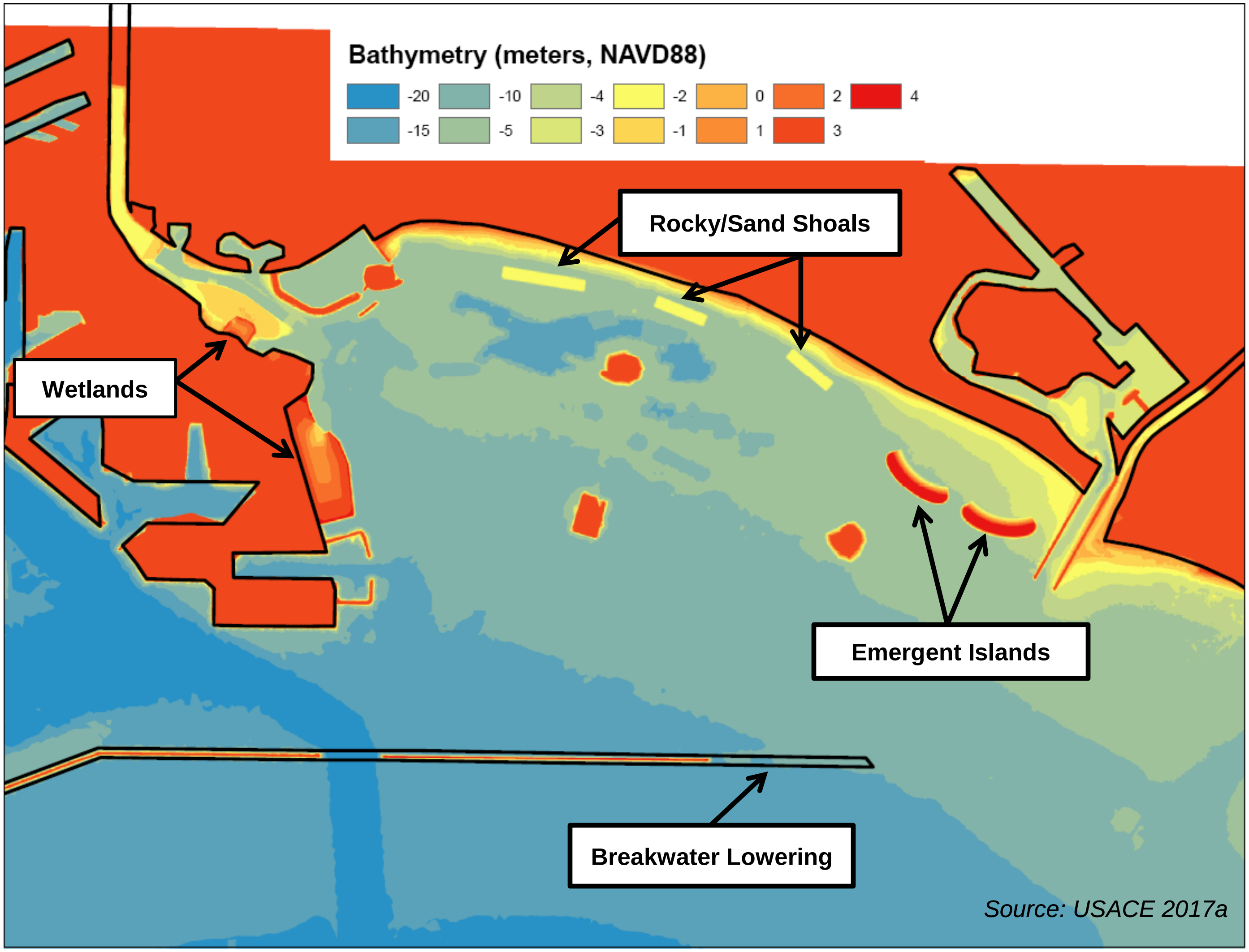


Figure 2.6 East San Pedro Bay Ecosystem Restoration Scenario 5

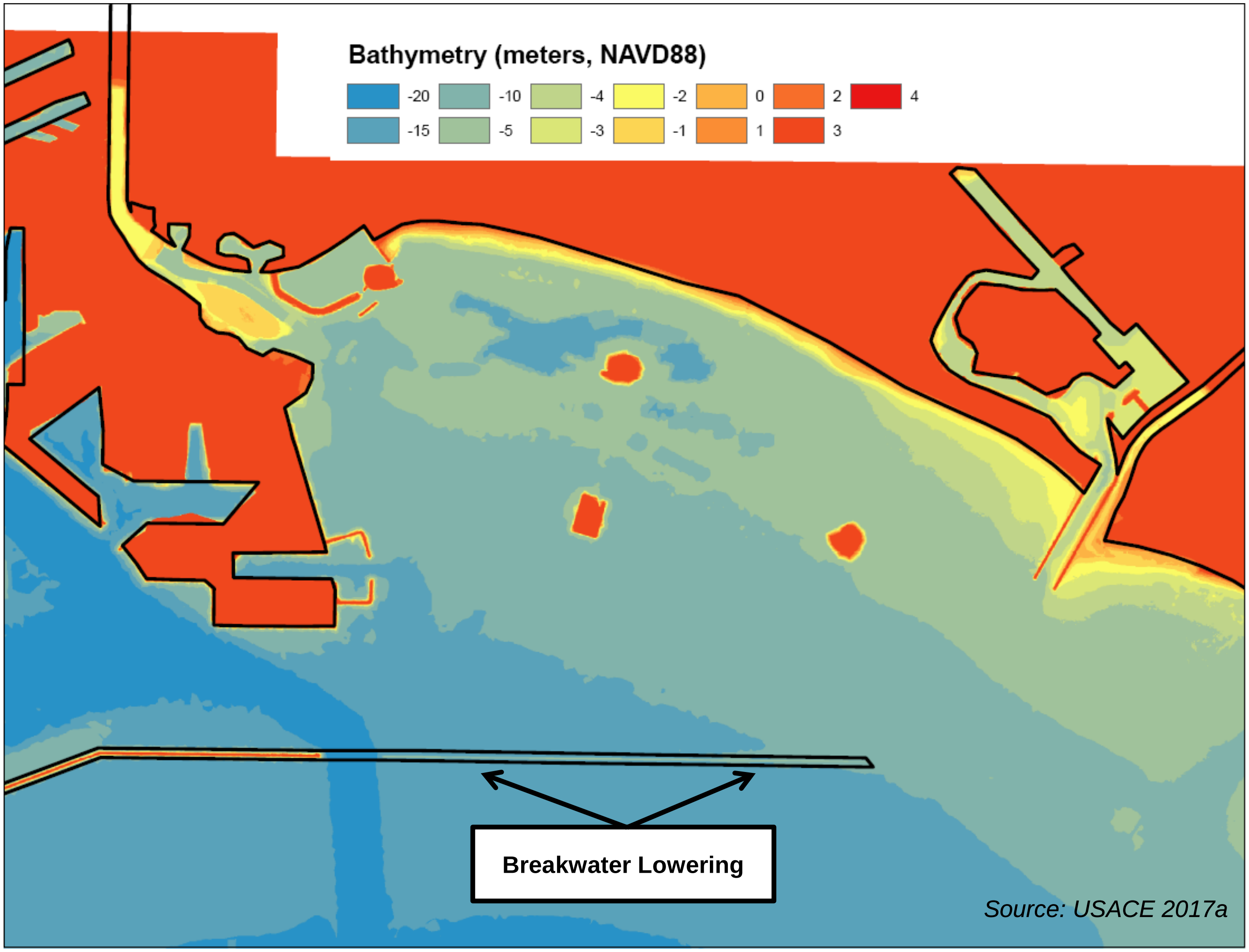


Figure 2.7 East San Pedro Bay Ecosystem Restoration Scenario 6

3. MODEL DEVELOPMENT

3.1 Background

A three-dimensional (3D) hydrodynamic, sediment transport, and water quality model for ESPB (ESPB Model) was developed to provide hydrodynamic and water quality prediction that will be used by USACE for the habitat evaluation model. The ESPB Model was designed to evaluate the proposed restoration alternatives described in Section 2, and to enable data transfers with the USACE wave and habitat evaluation models. ESPB Model simulations were conducted to provide currents, salinity, and total suspended solids (TSS) conditions to be used for habitat evaluation. In addition, the ESPB Model was also used for a numeric tracer tracking study, which was conducted to evaluate potential sediment (and associated contaminants) transport from the Los Angeles and San Gabriel Rivers into ESPB.

The ESPB Model was designed based on the previously developed WRAP Model - a 3D hydrodynamic, sediment transport, and chemical fate model of the LA/LB Harbor and San Pedro Bay. The WRAP Model was developed based on the Environmental Fluid Dynamic Code (EFDC) modeling platform, which is a surface water modeling system developed and distributed by the Environmental Protection Agency (EPA) Center for Exposure Assessment Modeling. EFDC dynamically couples hydrodynamic, sediment, and water quality transport. The WRAP Model has been calibrated to provide accurate simulations of the complex hydrodynamic and transport conditions of the LA/LB Harbor, and is currently being used for TMDL applications in the greater San Pedro Bay (Everest 2017). The development and extensive calibration of the WRAP Model has been conducted with oversight and approval by governmental agencies, technical advisory boards, and peer reviewers.

The WRAP Model development includes methods for estimating model inputs including tides, winds, and storm water flows from the surrounding watersheds, as shown in Figure 3.1. Tide and wind inputs were specified using data from the National Oceanic and Atmospheric Administration (NOAA) National Ocean Service, which are monitored as part of the Physical Oceanographic Real-Time System (PORTS®) for the LA/LB Harbor. Water levels and temperature are monitored at the NOAA LA Outer Harbor tide gage. Tides are mixed and semi-diurnal with two daily highs and two daily lows. Tidal datums based on the latest National Tidal Datum Epoch (NTDE) from 1983 to 2001 are provided in Table 3.1. Wind speed, wind gust, and wind direction are monitored at seven meteorological stations. Spatially and temporally varying wind conditions were utilized, which are important for accurately representing the overall wind circulation pattern. Storm water flows that discharge into the LA/LB Harbor and San Pedro Bay are simulated with approximately 200 model inflows (shown as orange dots in Figure 3.1). Flows were estimated based on flow data from the four major rivers – Los Angeles River (LAR), San Gabriel River (SGR), Coyote Creek, and Dominguez Channel. Other model parameters and inputs were defined based on an extensive compilation of site-specific data and prior model calibrations.

Table 3.1 Tidal Datums for LA/LB Harbor

TIDAL DATUM	ELEVATION (M, MLLW)
Highest Observed Water Level (01/10/2005)	2.414
Mean Higher High Water (MHHW)	1.674
Mean High Water (MHW)	1.449
Mean Sea Level (MSL)	0.861
Mean Low Water (MLW)	0.287
North American Vertical Datum – 1988 (NAVD88)	0.062
Mean Lower Low Water (MLLW)	0.000
Lowest Observed Water Level (12/17/1933)	-0.832

Source: NOAA 2011
Tidal Epoch 1983 – 2001

3.2 Grid Resolution

The ESPB Model, as shown in Figure 3.2, has a similar spatial extent as the WRAP Model, extending across the LA/LB Harbor and ESPB. However, the ESPB Model grid resolution throughout ESPB was refined to accommodate integration of the USACE wave and habitat models. In general, the refined model grid has a grid resolution of approximately 50 meters throughout the ESPB. The finer grids were designed to provide a greater detail along the shoreline, proposed habitat areas, and around the breakwaters where the effects of waves on hydrodynamic conditions occur.

3.3 CMS-Wave Coupling

The ESPB Model incorporated wave effects via a one-way coupling between CMS-Wave and EFDC. USACE developed and conducted wave modeling that provided wave outputs based on the CMS-Wave grid. The CMS-Wave grid was a finer resolution than the ESPB Model, thus the wave conditions were interpolated into the ESPB Model grid. The EFDC source code was modified to enable the coupling of the CMS-Wave and EFDC models.

Originally based on the scope of work, an average summer wave condition and an average winter wave condition were developed by USACE for each of the 14-day summer and winter simulation periods to be incorporated in the hydrodynamic modeling. However, after testing the incorporation of the CMS wave output with the EFDC model, it was determined that the use of one average wave condition for a 14-day simulation produces unrealistic hydrodynamic conditions at ESPB. This is because using one constant wave condition for fourteen days will result in unrealistic wave-induced stresses that will not change throughout the 14-day simulation period; which in turn, would drive unrealistic wave-induced currents along the shoreline, and near the jetties and breakwaters. Hence, additional wave and hydrodynamic modeling was tested to find the optimal wave input requirements for simulate the wave driven currents. After further testing with the use of hourly, 3-hour and 6-hour interval wave conditions, it was determined that the use of 6-hour intervals would be a good compromise for producing similar realistic wave-induced currents to hourly wave data, while saving a lot of processing time that would otherwise be needed for interpolating the hourly wave output into the EFDC model grid. Ultimately, USACE (2017b) provided the summer and winter wave conditions on a 6-hour interval that were used in the hydrodynamic modeling.

3.4 Model Inputs

USACE has provided guidance on the model simulation period to be used for evaluating existing and alternative conditions. The model simulation period will consist of a 14-day summer condition and a 14-day winter condition that can be used to represent watershed inputs to ESPB for a typical year. Precipitation and flow data from the Los Angeles County Department of Public Works (LACDPW) monitoring station for the Los Angeles River (LAR) at Wardlow Road were reviewed to identify a representative year. Based on precipitation records, as shown in top panel of Figure 3.3, the annual precipitation varied from 3.6 to 20.4 inches with an average of 9.1 inches/yr. Thus, Year 2012 – which had an annual precipitation of 8.9 inches – was designated as the typical year from which the summer and winter conditions were selected. The flows and tides for 2012 are shown in the middle and bottom panels of Figure 3.3, respectively. Based on the flow data, the summer condition was selected as the two-week period from July 8 to July 22, 2012 and the winter condition was selected as the two-week period from April 8 to April 22, 2012.

Summer Condition

The 14-day summer simulation period was selected as a two-week period with no precipitation. Tides and flows during the summer condition are provided in Figure 3.4. Tide levels are shown in the top panel with the tidal datums – MHHW, MHW, MLW and MLLW, as indicated by the solid and dashed lines. Tide levels during the model simulation period generally fall within the mean tide conditions. Flows for the LAR and SGR are shown in the lower panel. Perennial dry weather flows throughout the year (i.e., no precipitation) are due urban discharges, such as wastewater reclamation plant effluent and urban land uses. Dry weather flows from the LAR are relatively constant, whereas dry weather flows from the SGR are more variable due to discharges from wastewater reclamation plants.

Winter Condition

The tides and flows for the 14-day winter condition are shown in Figure 3.5. Tide levels, as shown in the top panel, were generally within the mean tide conditions. The selected winter simulation period has two rain events, which are provided in the lower panel. Most of the wet weather flows entering ESPB are from the LAR.

Sediment

The WRAP Model sediment bed properties were based on a compilation of sediment data from multiple studies taken at different times to provide sufficient spatial coverage throughout the LA/LB Harbor and ESPB. Sediment data for ESPB, shown as percent fines, is illustrated in Figure 3.6. Sediment data within ESPB were primarily from the Southern California Bight Regional Monitoring, which indicate sediments on the bay floor are predominantly fines (i.e., clays and silts).

Estimates of sediment loadings from the surrounding watersheds were simulated following the methodology developed for the WRAP Model. The WRAP Model was developed based on five sediment classes: coarse sand, fine sand, coarse silt, fine silt, and clay. The storm water sediment composition was defined as 3% coarse sand, 14% fine sand, 46% coarse silt, 29% fine silts, and 8% clay. Storm water sediment concentrations were specified based on seasonal average concentrations for dry weather and wet weather (Everest 2017).

Water Temperature

Water temperatures in the LA/LB Harbor and East San Pedro Bay are relatively consistent throughout the year. Monthly water temperatures at the NOAA LA Outer Harbor gage are shown in Table 3.2. The water temperatures are monthly averages based on data from 2002 to 2012. Annually, water temperatures generally range from 14.4 to 18.0°C with an average of 16.4°C. Seasonal fluctuations occur with warmer water temperatures during summer months and peak in September. Cooler water temperatures occur in early spring, with the lowest temperatures in March and April. Annual average water temperatures between 2002 and 2012 ranged from 15.3 to 17.8°C.

Water temperatures from the summer and winter simulation periods are provided in Figure 3.7. The average water temperatures during the summer and winter conditions were 16.6 and 13.6 °C, respectively.

Table 3.2 Average Monthly Water Temperatures at NOAA LA Harbor

MONTH	TEMPERATURE (°C)
January	15.0
February	15.2
March	14.7
April	14.4
May	16.0
June	17.5
July	17.9
August	17.6
September	18.0
October	17.8
November	17.0
December	15.7
Average	***16.4***

3.5 Data Transfer

Results from the ESPB Model were provided for transfer into the habitat evaluation model. The data transfer included model cell outputs for ESPB, as shown in Figure 3.8. The area of ESPB for the data transfer is indicated by the red outlined area, which consists of approximately 3,300 cells. For each cell, model outputs provided included velocity vectors, salinity, and TSS for all five vertical water layers. Results were provided at an hourly interval for both the summer and winter simulation periods. The data transfers were provided in GIS format using a geodatabase of the model outputs.

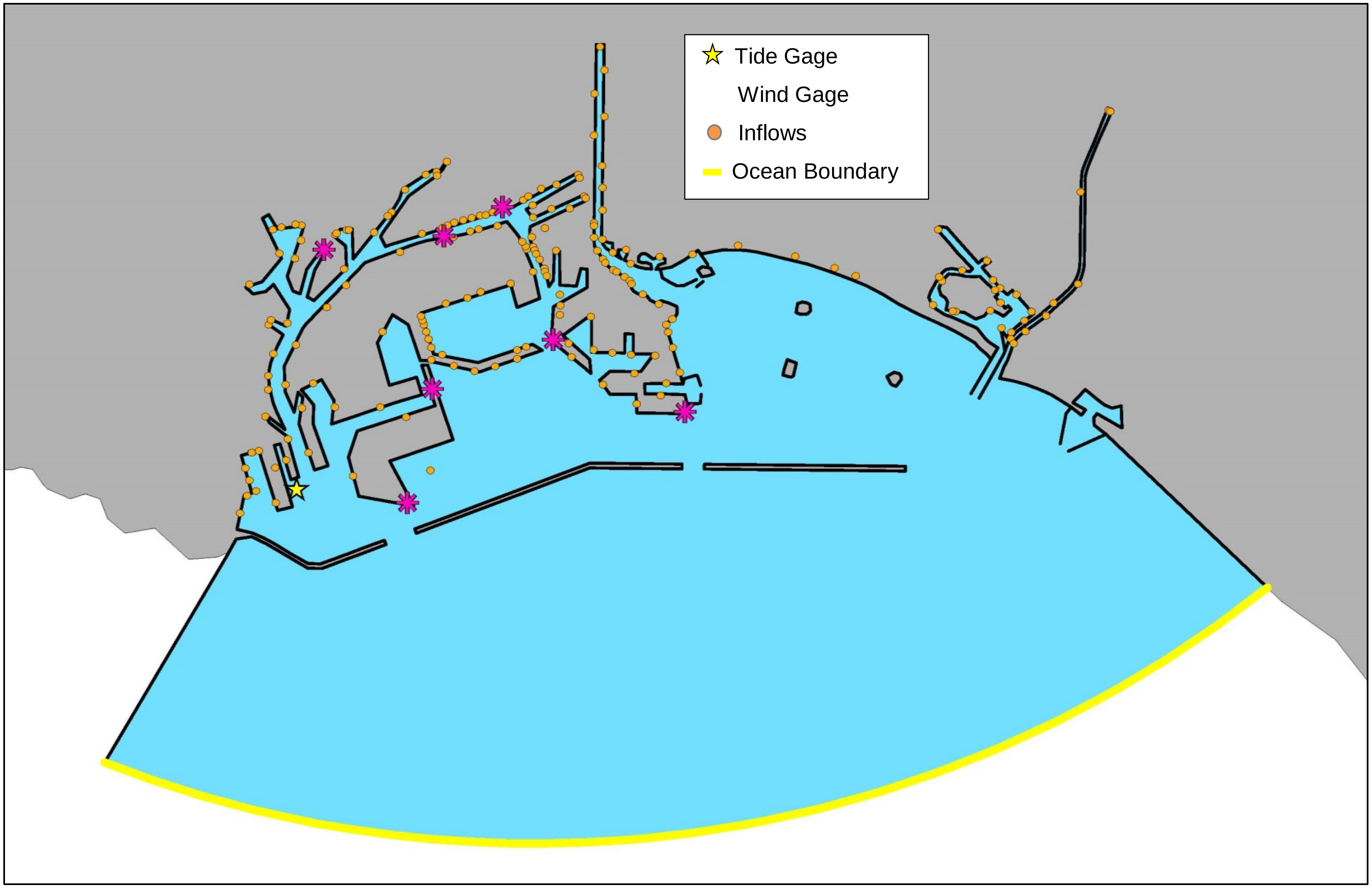


Figure 3.1 WRAP Model Inputs and Boundary Conditions

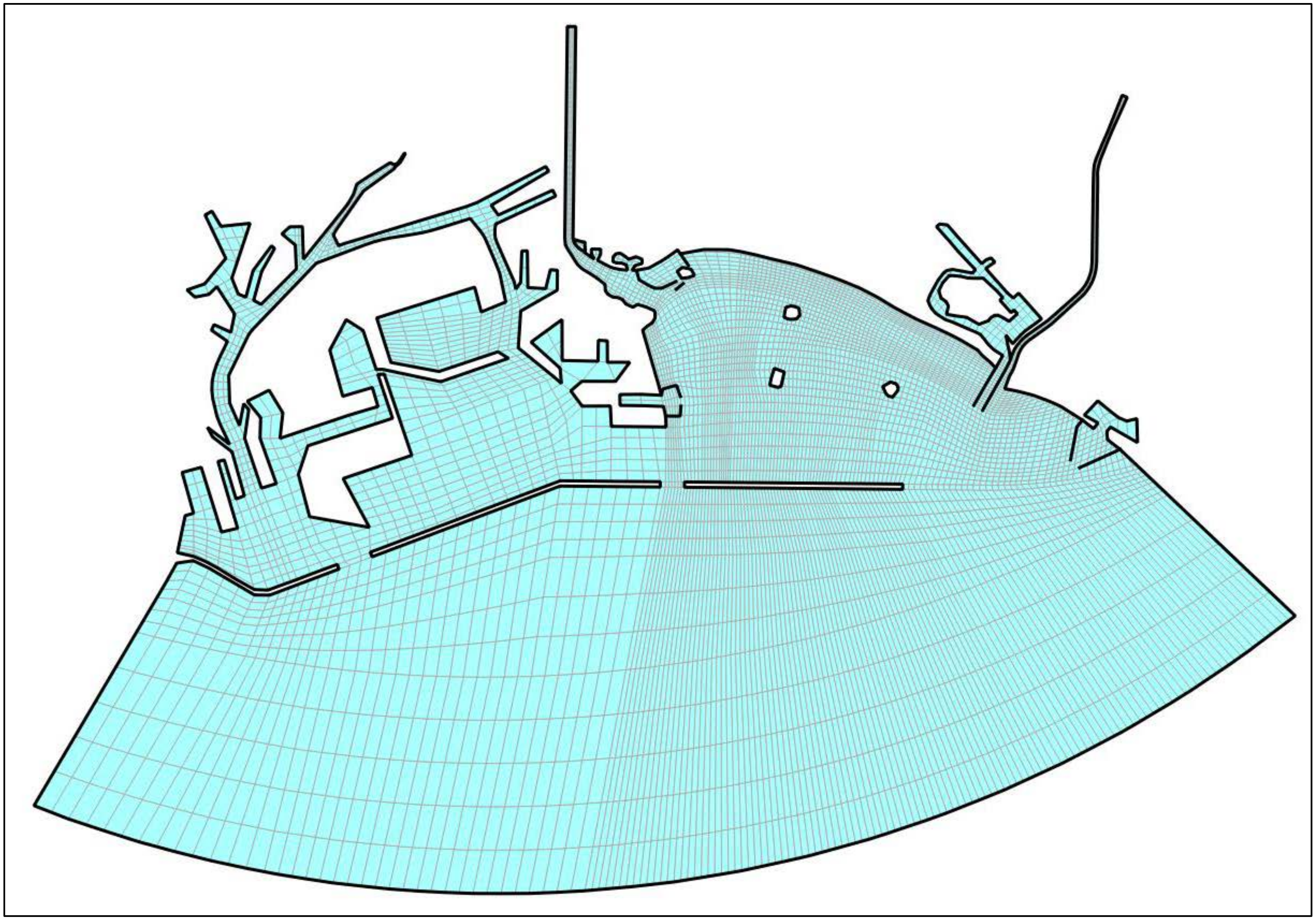


Figure 3.2 East San Pedro Bay Hydrodynamic Model Grid

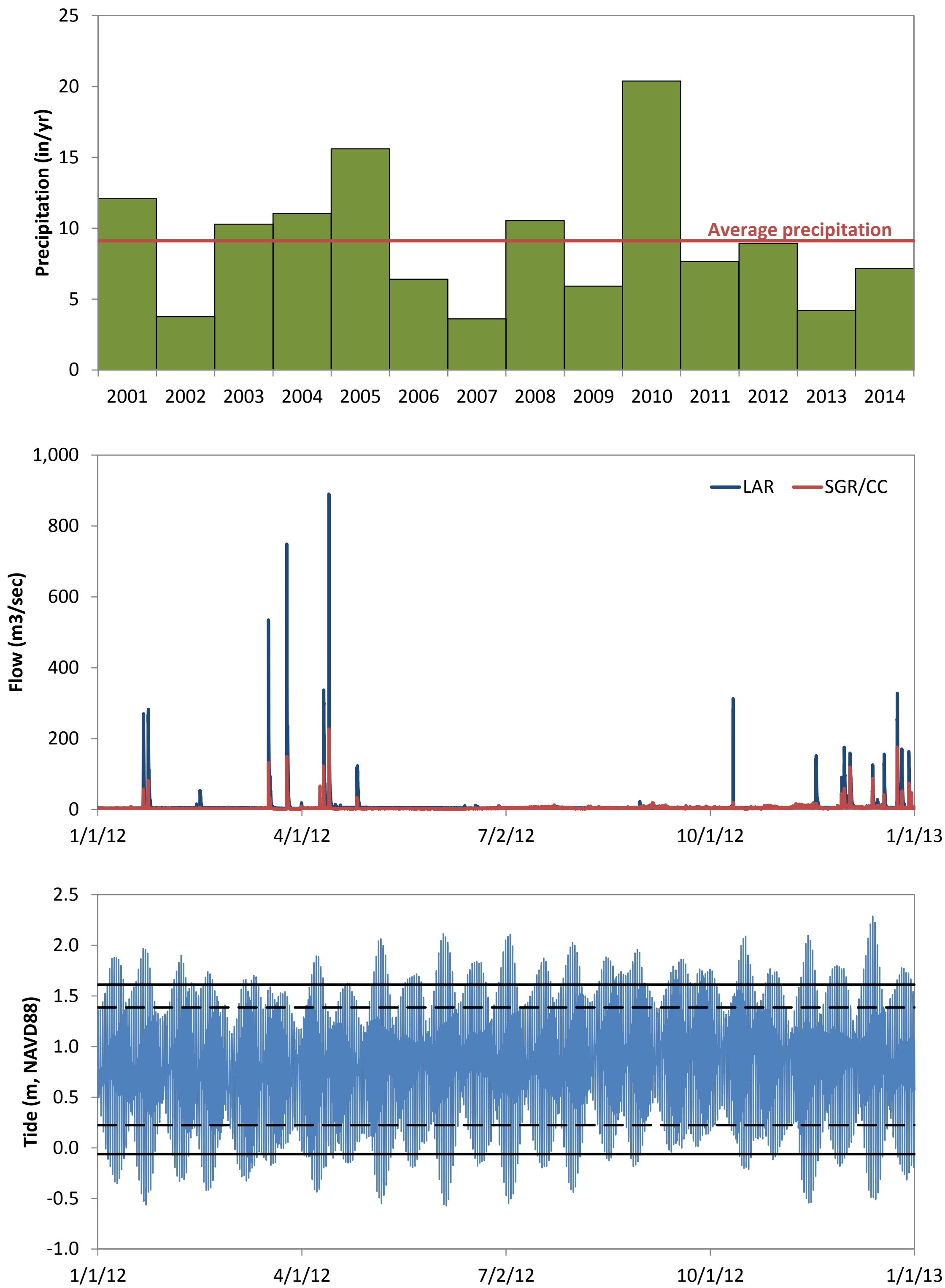
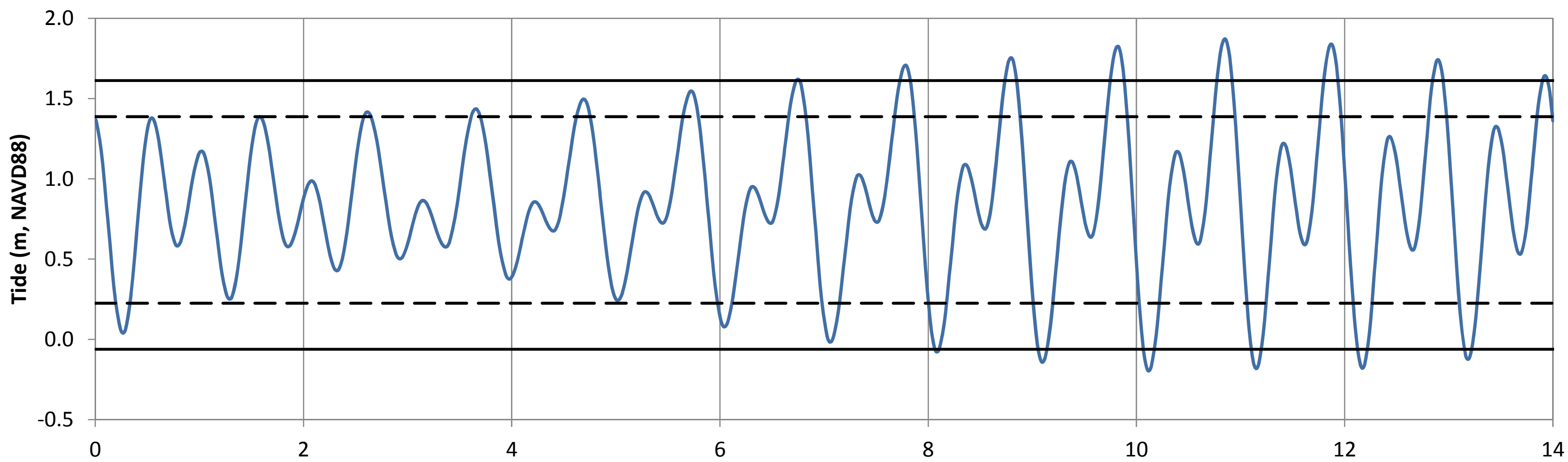


Figure 3.3 Precipitation, Flows, and Tides for 2012



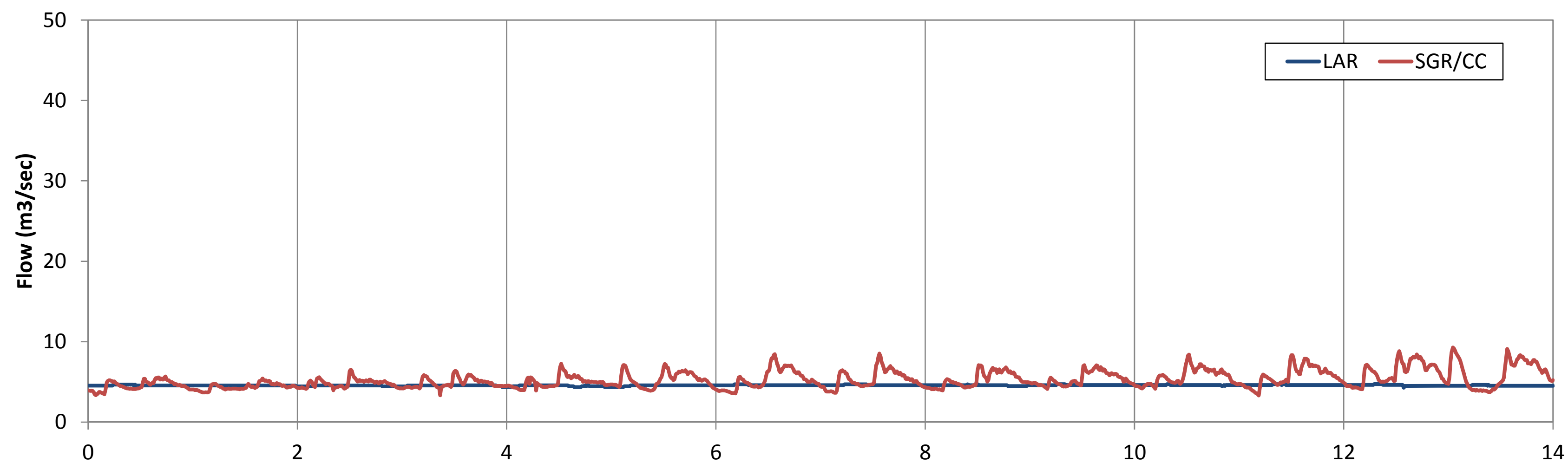


Figure 3.4 Summer Condition Tides and Flows

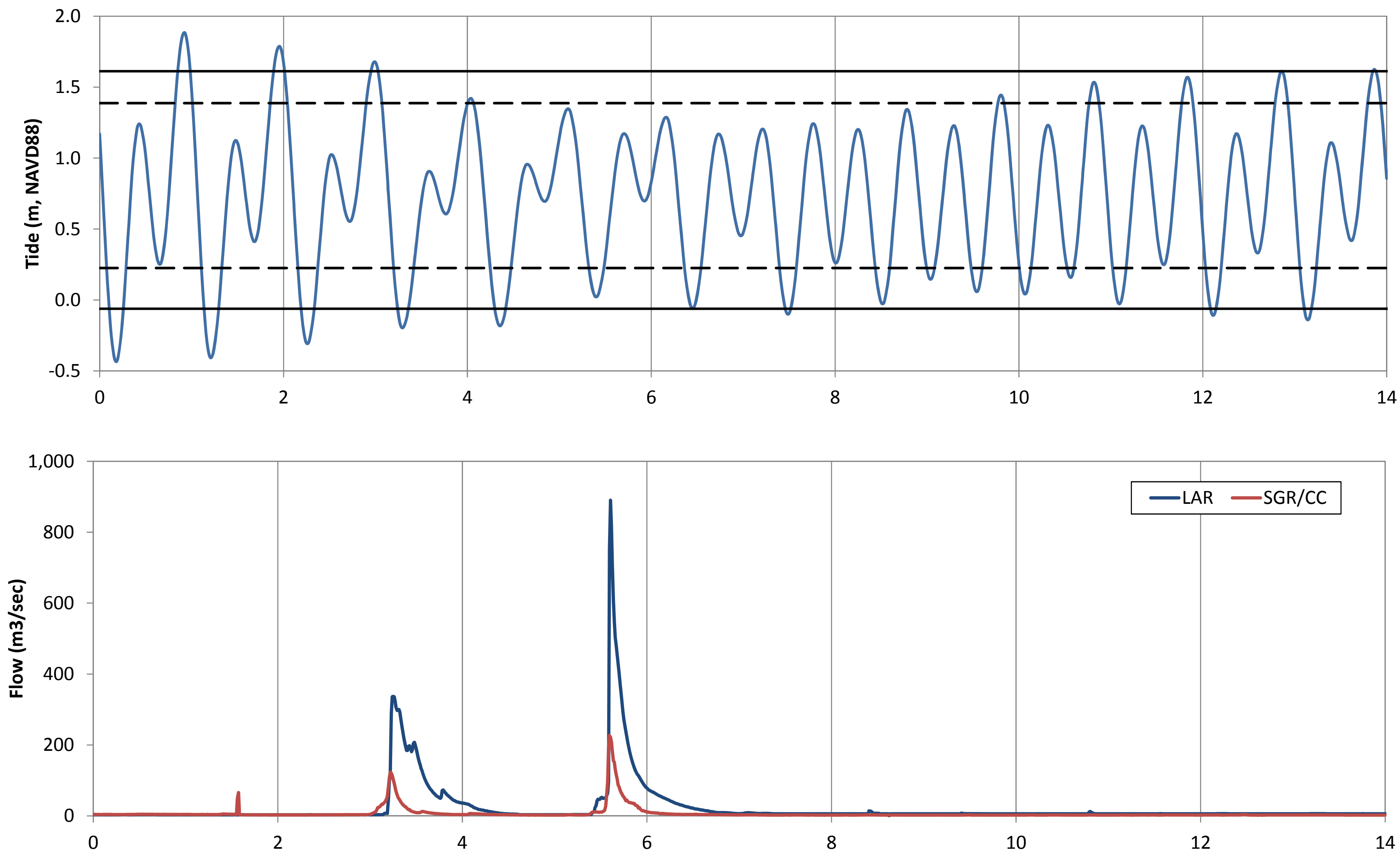
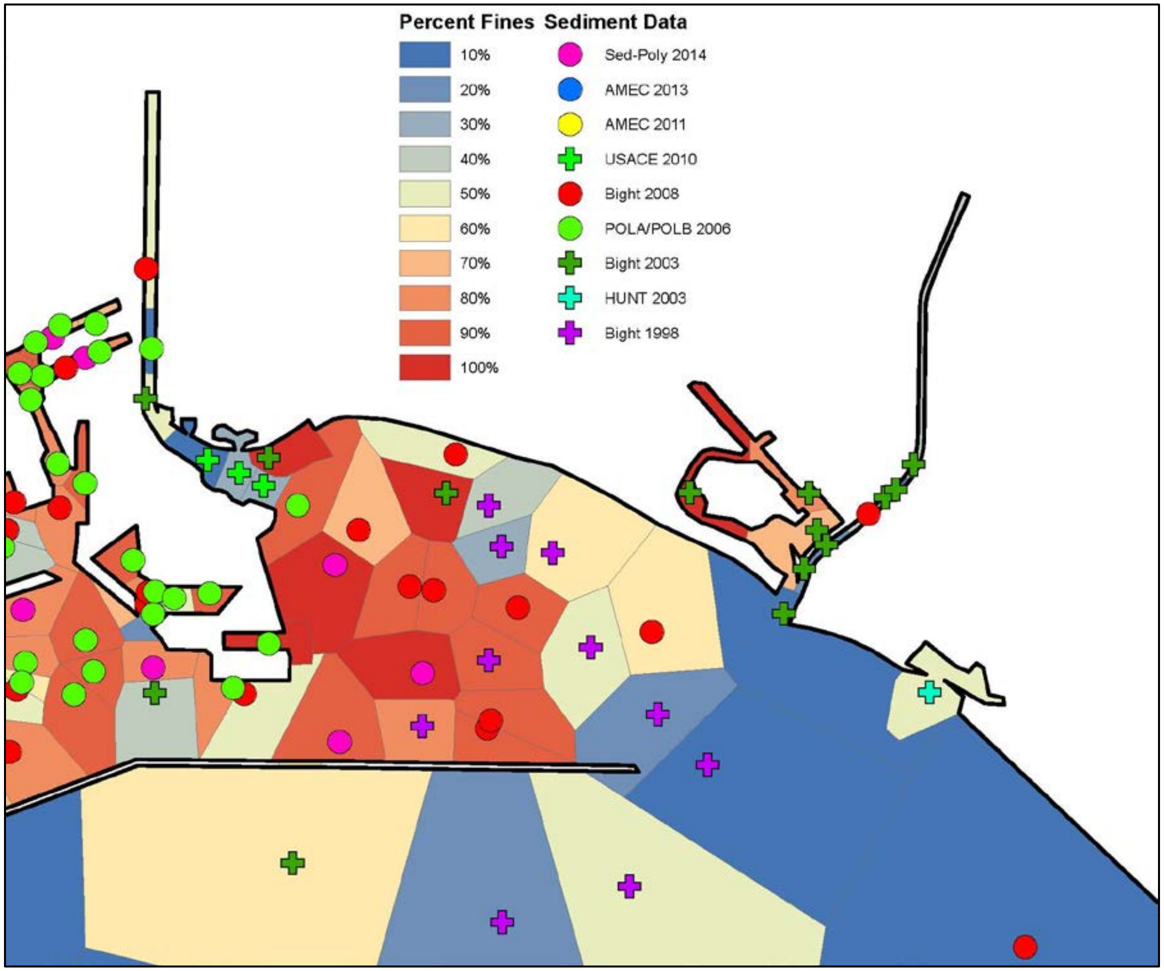


Figure 3.5 Winter Condition Tides and Flows



Source: Everest 2016

Figure 3.6 Sediment Data for East San Pedro Bay

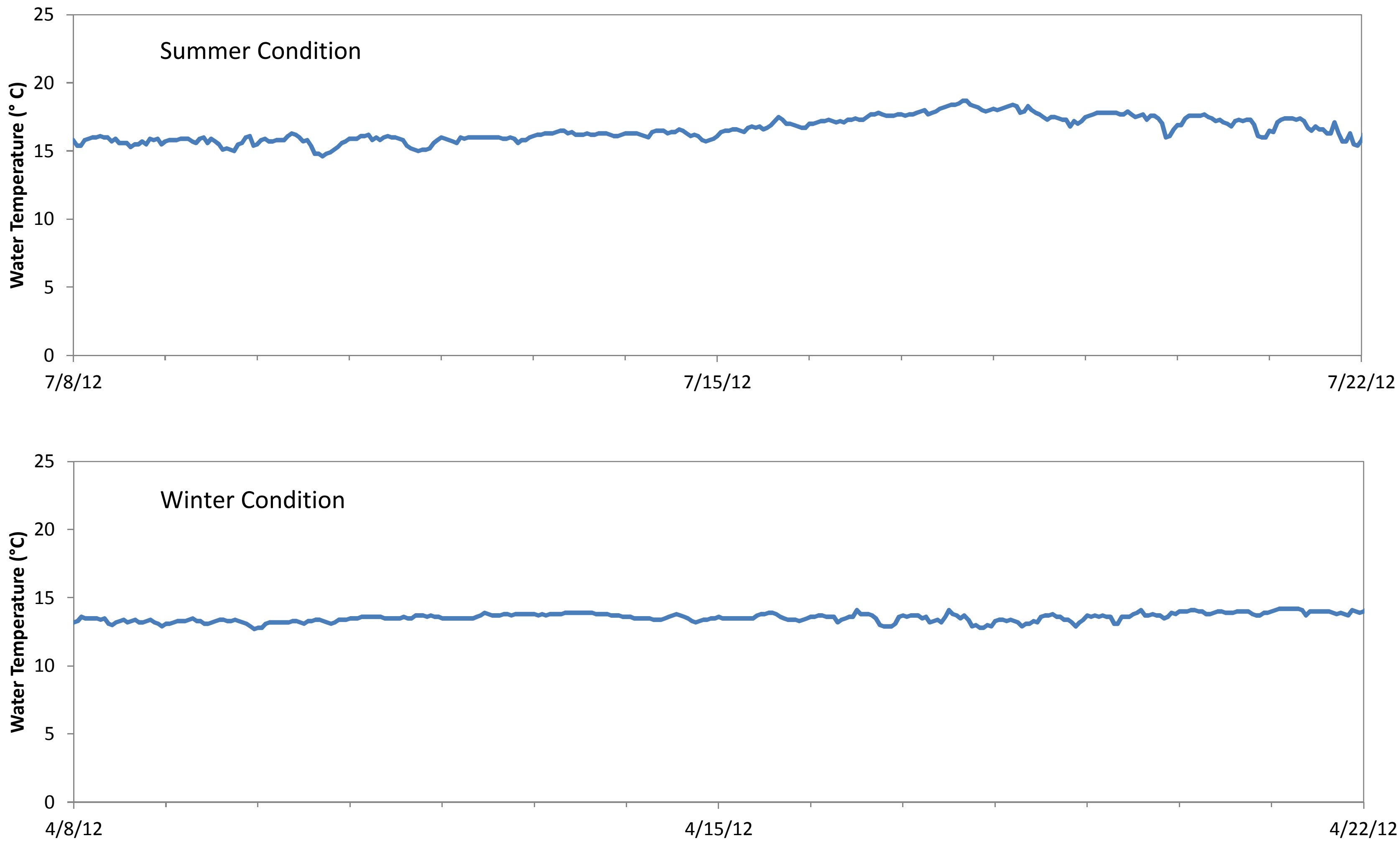


Figure 3.7 Water Temperatures for Summer and Winter Conditions

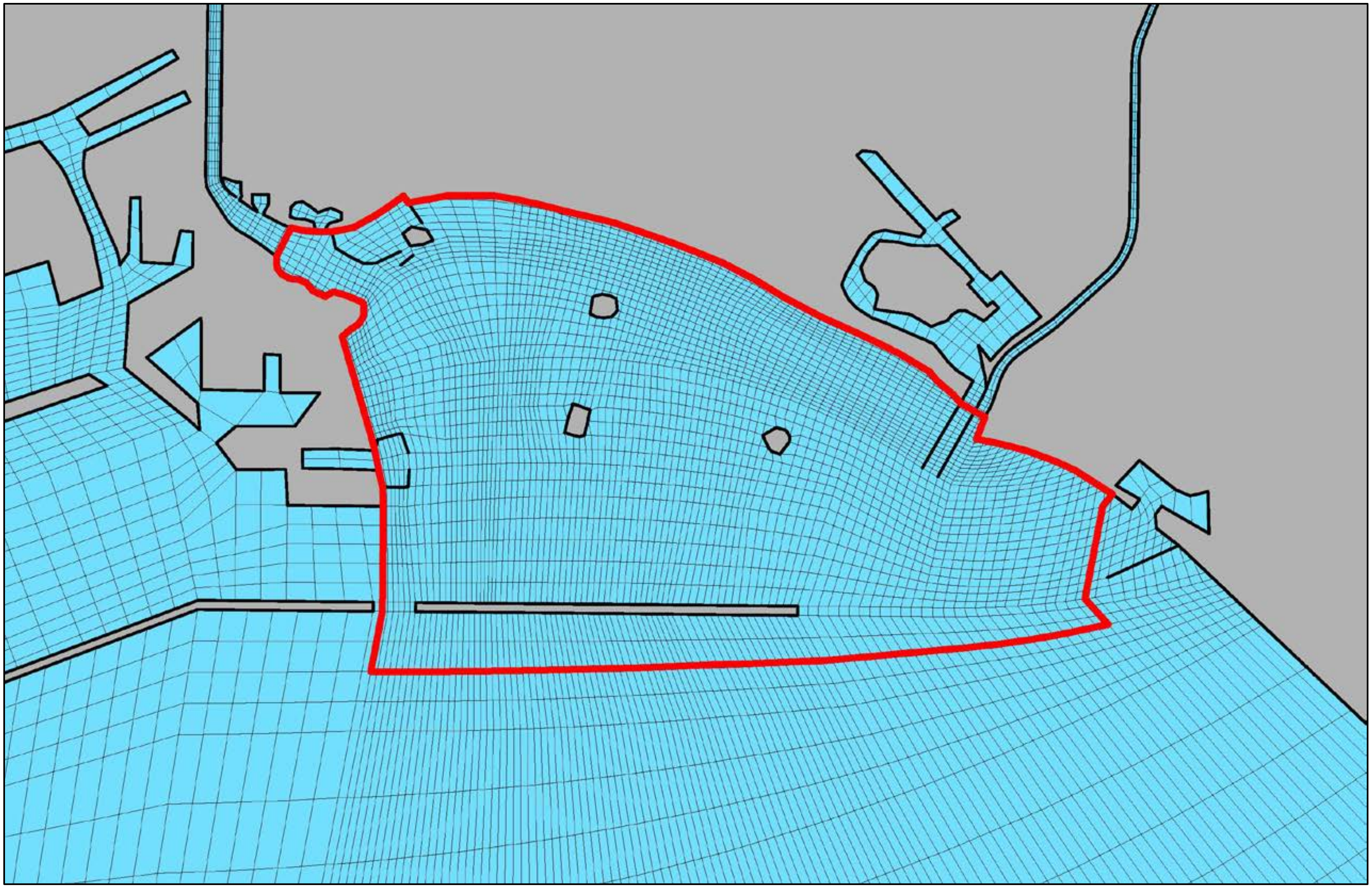


Figure 3.8 East San Pedro Bay Area for Data Transfer

4. HYDRODYNAMIC MODELING RESULTS

The hydrodynamic modeling was conducted to provide hydrodynamic and water quality information under Existing Conditions and the six restoration alternatives. The ESPB Model was used to simulate currents, salinity, and sediment during the summer and winter conditions. Changes from Existing Conditions were used to determine the impact of the restoration alternatives. Model results for Existing Conditions and the restoration alternatives are individually presented, followed by comparisons between the alternatives – using both spatial plots and time series plots. Within these sections, results of the simulated parameters are generally presented starting with velocity, followed by salinity, then sediment.

4.1 Existing (Baseline) Conditions

Snapshots of spatial plots of the model simulated results were provided for the Existing Conditions, to show the spatial variations of velocity, salinity and sediment (TSS) concentration throughout ESPB. These snapshots were taken at a time during flood tide, ebb tide, and at the end of a wet weather event from the 14-day wet weather simulations. The times when the snapshots were taken are shown in Figure 4.1. As shown in the figure, the snapshots for flood tide and ebb tide are taken at times when the tide is near mean tide level, and the flood and ebb velocities are near their peak values.

Velocities

The velocity spatial plots for Existing Conditions are shown in Figure 4.2. In the figure, the surface and bottom layer velocities are shown at peak flood tide, peak ebb tide, and during wet weather. The color scale was selected to illustrate the differences in velocity magnitude. The spatial plots show the differences in velocities at the surface compared with those at the bottom layer.

During peak flood tide conditions, surface velocities are highest between the breakwater and SGR, while the bottom velocities are higher through Queens Gate. During peak ebb tide conditions, velocities are higher compared to velocities during flood tide conditions. At the surface, during peak ebb tide, higher velocities occur throughout the eastern end and along the west end between the LAR and Queens Gate. Bottom velocities, during peak ebb tide, are typically lower than the surface velocities except in a few areas. During wet weather, surface velocities increase throughout the bay, while at the bottom layer, velocities are higher in the western half of the bay closest to the LAR. Additionally, surface velocities are much higher than bottom velocities during wet weather.

Differences between surface and bottom velocity are generally visible during all selected conditions, and serve to illustrate the 3D structure of velocities in ESPB. Overall, there is a strong tidal current at the surface, where the tide goes through Queens Gate. At most locations within ESPB, except at the gaps in the breakwater and at the surface during wet weather, currents are typically small with velocities measuring around 1 cm/s.

Salinity

Salinity levels in ESPB under Existing Conditions are depicted in Figure 4.3. Near the LAR and SGR, there are variations in salinity between the surface and bottom layers. At the peak flood and peak ebb tides, slight decreases in surface salinity occur due to dry weather flows from the LAR and SGR. Overall, during peak flood and peak ebb tides, there is minimal stratification of salinity levels. Salinity levels show a greater decrease during wet weather, with lower salinity levels propagating out from the rivers and into the bay.

Salinity levels show decreases where higher velocities occur due to the freshwater inputs from rivers. Like the velocity results, salinity results also show differences between the surface and bottom concentrations, particularly during wet weather. This illustrates the 3D structure of salinity concentrations in ESPB.

Sediment (TSS) Concentration

Spatial plots of sediment concentrations for Existing Conditions are shown in Figure 4.4. During peak flood and peak ebb tide conditions, surface and bottom sediment concentrations are similar, except along the rivers, where the bottom layer sediment concentrations are higher. Higher surface sediment concentrations occur during wet weather from discharges from the rivers. These elevated sediment concentrations are correlated with the higher velocities and lower salinities that also occur due to freshwater input. The difference in surface and bottom sediment concentrations during wet weather illustrates the 3D structure of sediment concentrations that can occur in ESPB.

4.2 Restoration Alternatives

For each of the restoration alternatives, velocity spatial plots are provided at peak flood tide, peak ebb tide, and during wet weather to show overall velocity patterns. Changes in velocity from Existing Conditions are also shown in spatial difference plots, to highlight the differences for each model scenario. In these difference plots, areas where habitat and structural features are added or changed for the restoration alternatives, are indicated by pink-dashed lines.

Scenario 1

Velocities under Scenario 1 are illustrated in Figure 4.5. In general, these velocities are similar to those under Existing Conditions. The changes in velocity, as shown in Figure 4.6, highlight the changes under Scenario 1. Increases and decreases in velocities occur in the vicinity of habitat features, due to the blocking effects of these features. Adjacent to elevated habitat features, particularly at the four nearshore shoals, plumes of lowered velocities are visible. Decreases in velocities also occur over the wetlands in the LAR Estuary and north of Pier J. Increases in velocities are taken place adjacent to the wetlands. The greatest changes occur during wet weather conditions, which include decreases in velocities along the rocky/sandy nearshore shoals. Only localized changes in velocities near habitat features are expected to occur under Scenario 1. Generally, these changes in velocities are very small, except where elevations of habitat features are significantly shallower than those same areas are under Existing Conditions.

Scenario 2

Scenario 2 features two notches in the breakwater. Velocities under Scenario 2 are shown in Figures 4.7. Spatial plots of the velocities show high velocities at the proposed gaps in the breakwater, particularly during peak flood and peak ebb tide conditions. Changes in velocity for Scenario 2 are provided in Figure 4.8, which indicates more differences with Existing Conditions than does Scenario 1. At peak flood tide, increases in velocities mainly occur in the breakwater gaps, especially at the bottom layer. Decreases in velocities are also observed at the eastern end of the bay between the breakwater and SGR. Similarly, increases in velocities at the breakwater gaps and decreases to the east of the breakwater occur at peak ebb tide. Changes in velocities occur in the vicinity of Queens Gate. During wet weather, changes in velocities are apparent around the breakwater with increases in the breakwater gaps. Similar to Scenario 1, plumes of lowered velocities are sometimes observed at the nearshore shoals, and lowered velocities are typically present at the elevated wetlands. Overall, changes in velocities occur in the vicinity of habitat and structural features with minimal changes in velocities in the center portion of ESPB.

Scenario 3

Velocities with the breakwater lowering under Scenario 3 are provided in Figure 4.9. Under tidal conditions, the highest velocities occur at both ends of the shortened breakwater. During wet weather, high velocities are evident throughout the bay. Changes in velocities due to Scenario 3, and relative to Existing Conditions, are shown in Figure 4.10. In general, Scenario 3 results in increases in velocities where the breakwater was lowered, while decreasing velocities to the east between the breakwater and SGR due to widening of that opening. Under tidal conditions, increases in velocities extend from the breakwater into the bay. Changes in velocities also occur around Queens Gate. Relative to the changes that occur at the bottom layer, the changes at the surface layer are much greater. Overall, Scenario 3 results in velocity changes in the vicinity of habitat and structural features with some changes in the bay. The overall pattern of changes under Scenario 3 has similarities to that under Scenario 2, since both include changes in the breakwater, though Scenario 3 generally shows larger areas of increases and decreases in velocity than Scenario 2.

Scenario 4

Scenario 4 included emergent islands and a training wall along the mouth of the LAR. Velocities and changes in velocities under Scenario 4 are provided in Figures 4.11 and 4.12, respectively. The effects on velocities for Scenario 4 are mainly seen in the vicinity of the emergent islands and training wall. The emergent islands reduce velocities around the islands and increase velocities between the islands and the Alamitos Bay jetties. Near the training wall, changes in velocities occur due to the re-direction of flows from the LAR; thus, changes are more pronounced during wet weather. The training wall re-directs LAR flows in a more southerly direction. Increases in velocities occur at the end of the training wall, between the wetland and training wall. Correspondingly, decreases in velocities occur on the other side of the training wall. Changes are also seen outside of the Long Beach Marina and east end of the breakwater. Under Scenario 4, changes in velocities primarily occur during wet weather, and are noticeable throughout the bay. Due to the addition of emergent islands and the training wall placement, the overall pattern of changes under Scenario 4 differs substantially from those under Alternatives 1 through 3.

Scenario 5

Scenario 5 includes a combination of habitat features from Scenario 4 and structural features from Scenario 3. Velocities for Scenario 5 are shown in Figure 4.13. In general, higher velocities occur near the emergent islands and breakwater modification. Surface velocities are typically higher compared to bottom velocities. Impacts of Scenario 5 are highlighted in Figure 4.14, which shows the changes in velocities from Existing Conditions. During peak flood and peak ebb tide conditions, velocities increase where the breakwater was lowered and decrease between the breakwater and emergent islands. Velocities also decrease at the west end of the breakwater around Queens Gate. During wet weather, increases in velocities also pronounced where the breakwater was lowered. Overall, Scenario 5 results in velocity changes in the vicinity of the emergent islands and breakwater lowering with some changes extending into the bay. As expected, due to the similarities in features, the overall pattern of changes under Scenario 5 shows similarities to those of both Scenario 4 and 3.

Scenario 6

Scenario 6 consisted of lowering the entire eastern portion of the breakwater. Velocities during tidal and wet weather conditions are shown in Figure 4.15. During peak flood tide, higher velocities occur in the vicinity of Queens Gate and are relatively low throughout the bay. Surface velocities are relatively higher in most of the bay with higher velocities near Queens Gate during peak ebb tide, whereas bottom velocities are relatively low throughout the bay. Wet weather surface velocities are high throughout the bay and lower for the bottom layer. Changes in velocity under Scenario 6 are shown in Figure 4.16. In general, velocities increased along the original breakwater and decreased to the east. Changes also occur in the vicinity of Queens

Gate. For Scenario 6, the changes in velocity occur around the lowered breakwater and a portion of the bay. The overall pattern of changes under Scenario 6 shows some basic similarities to that under Scenario 4, since both scenarios include significant sections of breakwater lowering.

4.3 Comparison between Alternatives

This section provides two different types of comparisons between the restoration alternatives. Comparisons are made based on spatial plots, and on time series plots of results from select locations in ESPB. A total of three locations were selected for the times series plots, including one location at each of the following areas – open bay, shoreline, and river mouth. Results at the three selected locations are intended to be representative of conditions within those areas. The spatial plots focus on velocity comparisons, while the time series plots include comparisons of velocity, salinity concentrations, and sediment concentrations.

4.3.1 Spatial Plots

Comparisons of the restoration alternatives are shown for changes to surface velocities from Existing Conditions. To illustrate the differences between model scenarios, comparisons were made for the peak flood tide, peak ebb tide, and wet weather conditions.

Changes in surface velocities from Existing Conditions at peak flood tide are compared for the restoration alternatives in Figure 4.17. Scenarios 1 and 4 have the least impact on velocities during the peak flood tide. Scenario 2 shows increases in velocity at the breakwater gaps. Scenarios 3 and 5 results in similar changes to velocities, mainly near the lowered portion of the breakwater. Scenario 6 results in the greatest changes to peak flood tide velocities, with increases along the lowered breakwater and in the vicinity of Queens Gate, and decreases at the eastern end of ESPB.

Comparisons during the peak ebb tide are provided in Figure 4.18. Changes in velocities are greater during the peak ebb tide compared to the peak flood tide. However, comparisons among the restoration alternatives show similar trends. The least extensive changes occur under Scenarios 1 and 4. More changes result under Scenarios 2, 3, and 5. The most overall changes occur under Scenario 6, which shows changes throughout the bay.

Wet weather comparisons of the changes in surface velocity are shown in Figure 4.19. Scenarios 1 and 2 show the least amount of changes in velocity among the model scenarios. Scenario 3, 5, and 6 have changes near the structural changes to the breakwater. Scenario 4 results in the greatest overall changes in surface velocity due to the re-direction of flows from the LAR. Most changes in surface velocity under Scenario 4 occur near the river mouth, where freshwater input is the greatest.

Overall, habitat features including wetlands, rocky/sand nearshore shoals, and emergent islands have localized effects on velocities. Changes to the breakwater generally increase velocities in the vicinity of the breakwater modifications and decrease velocities to the east of the breakwater. Scenarios with breakwater changes have limited changes to velocities in the bay, except under Scenario 4 during wet weather. Among the six restoration alternatives, Scenario 6 shows the most changes in velocity.

4.3.2 Time Series Plots

Results of the hydrodynamic modeling are presented as time series comparisons at the three locations shown in Figure 4.20. Time series comparisons of velocity, salinity, and sediment concentrations were made at these locations. Location A is adjacent to an offshore island in the bay. Location B is along the shoreline near a proposed rocky/sand shoal. Location C is located where the LAR enters ESPB. Spatial plots during tide and wet weather dominant conditions were also made and provided in Section 4.2.

Currents

Currents at Location A under Existing Conditions are compared with the currents under each of the six model scenarios in Figure 4.21. In the figure, time series of the velocity magnitude are shown for the surface and bottom layers. The upper two panels show the velocity during the summer conditions, while the lower two panels show the velocity during winter conditions. In general, surface velocities are higher than velocities at the bottom layer. As expected, Scenario 6 with the lowering of the eastern portion of the breakwater, shows the greatest difference from Existing Conditions. Overall, however, the model scenario velocity magnitudes are similar to those under Existing Conditions.

Model scenario currents at Location B are compared in Figure 4.22. Velocities at Location B are similar to Existing Conditions during the summer and winter conditions with only minor differences.

Figure 4.23 shows the velocities at Location C. The greatest difference from Existing Conditions occurs for Scenario 4 during both the summer and winter conditions. In general, the velocities are higher under Scenario 4 than Existing Conditions and the other scenarios, particularly for the bottom layer. Scenario 5 shows some elevated velocities as well, particularly at the surface during winter conditions.

Salinity

Salinities for the model scenarios are compared in Figures 4.24 – 4.26 for Locations A – C, respectively. During summer conditions, salinities in the surface and bottom layers are similar at all three locations, but the surface salinity has some fluctuations at Location C, which reflects dry weather flows from the LAR. During winter conditions, depressions in surface salinity are noticeable during wet weather events. Overall, the model scenario salinities are similar to those under Existing Conditions, especially at Locations A and B. However, at Location C, the salinity results differ and are noticeably lower under Scenario 4 during wet weather.

Sediment

Comparisons of sediment concentrations at Location A are shown in Figure 4.27. During summer conditions, sediment concentrations under the restoration alternatives are similar to those under Existing Conditions. Some differences are observed during wet weather, particularly under Scenario 6.

For Location B, sediment concentrations are presented in Figure 4.28. During summer conditions, the results show increases in sediment concentrations, which correspond to peak ebb tides during mean tide conditions. The bottom layer sediment concentrations are greater than the surface, which indicates the effects of bed erosion. Sediment concentrations for Scenarios 1 – 5 are slightly higher than those under Existing Conditions during peak ebb tides. For Scenario 6, sediment concentrations are similar to those under Existing Conditions. Sediment concentrations during winter conditions show a greater difference compared with Existing Conditions, particularly during wet weather. Differences occur under Scenarios 1 – 5, which have habitat features near Location B.

Sediment concentrations at Location C are shown in Figure 4.29. Sediment concentrations among the model scenarios are similar during summer conditions. During wet weather, sediment concentrations are higher at the surface than at the bottom, indicating that the sediment is from the LAR. Higher sediment concentrations occur for Scenarios 1 - 5. Scenario 4 has the highest sediment concentrations due to the training wall, which redirects flows from the LAR.

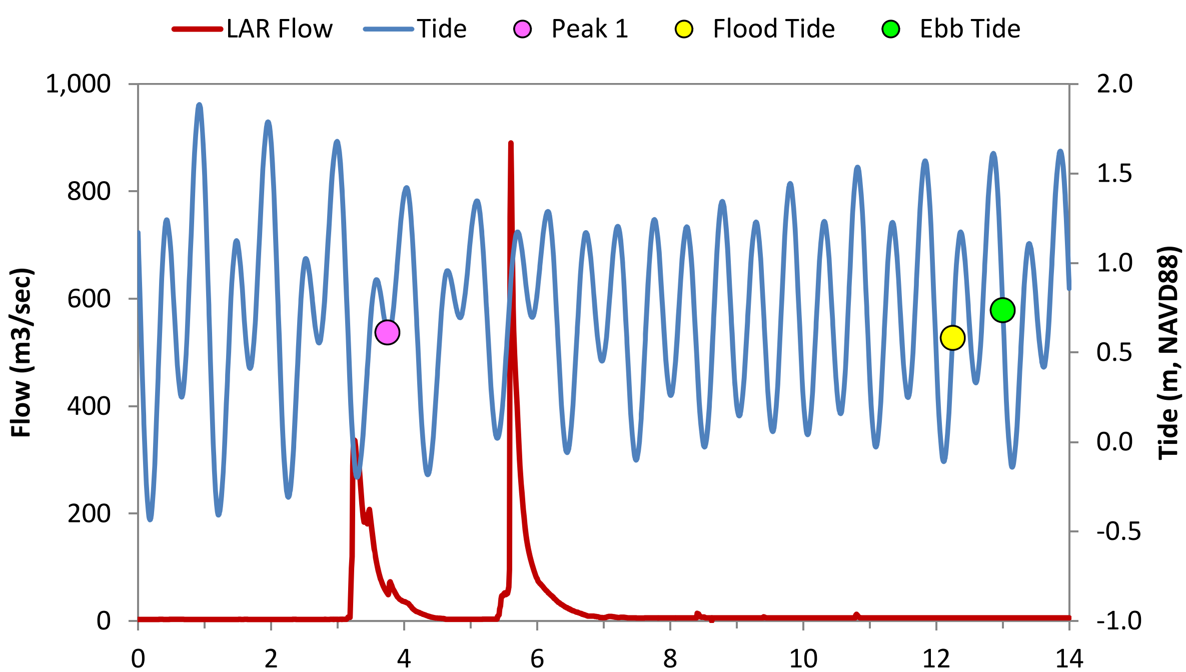


Figure 4.1 Snapshot of Times for Spatial Plots

Peak Flood Tide **Peak Ebb Tide** **Peak 1**

Surface Layer

Bottom Layer

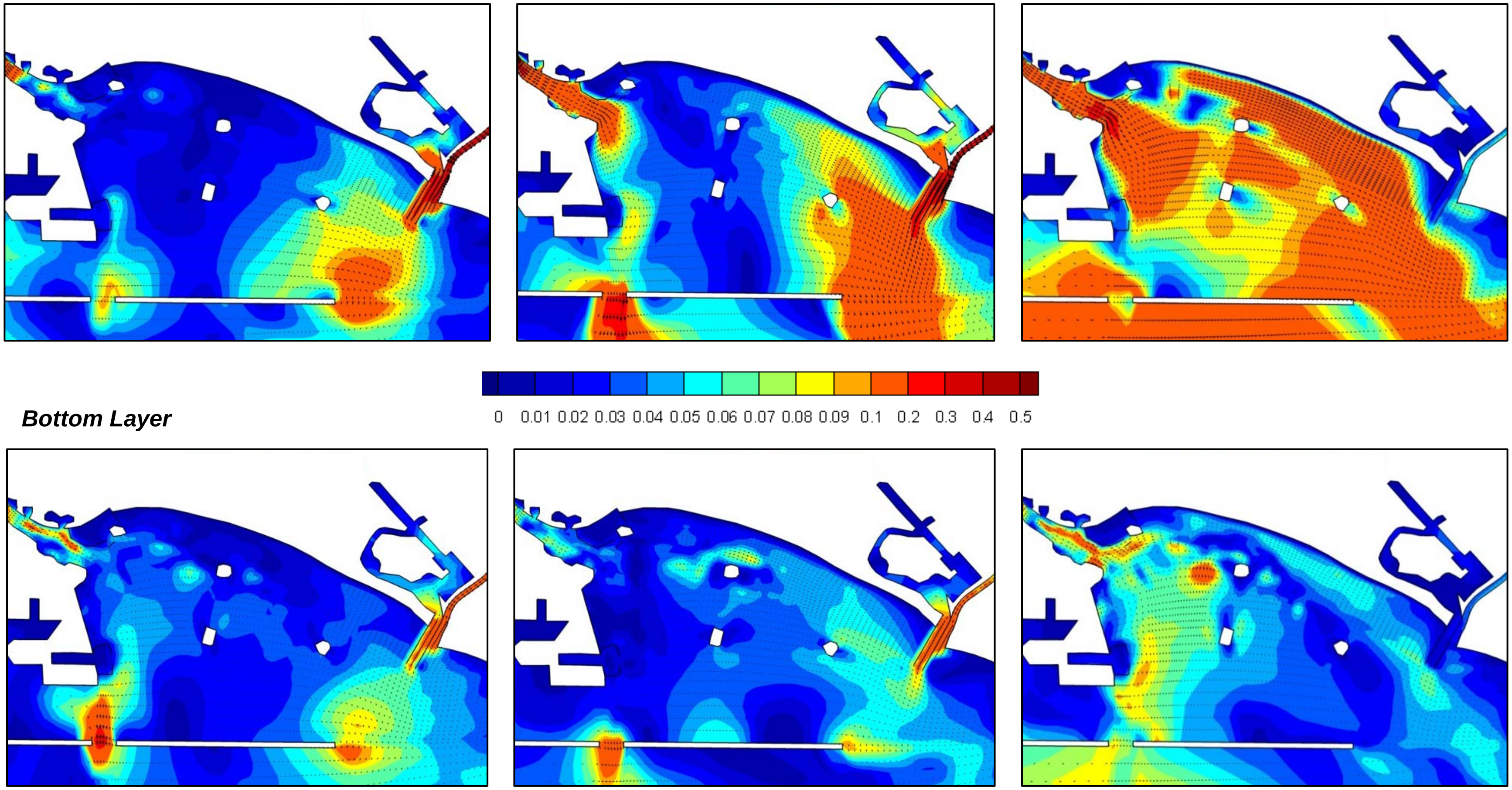


Figure 4.2 Existing Conditions Velocity Spatial Plots

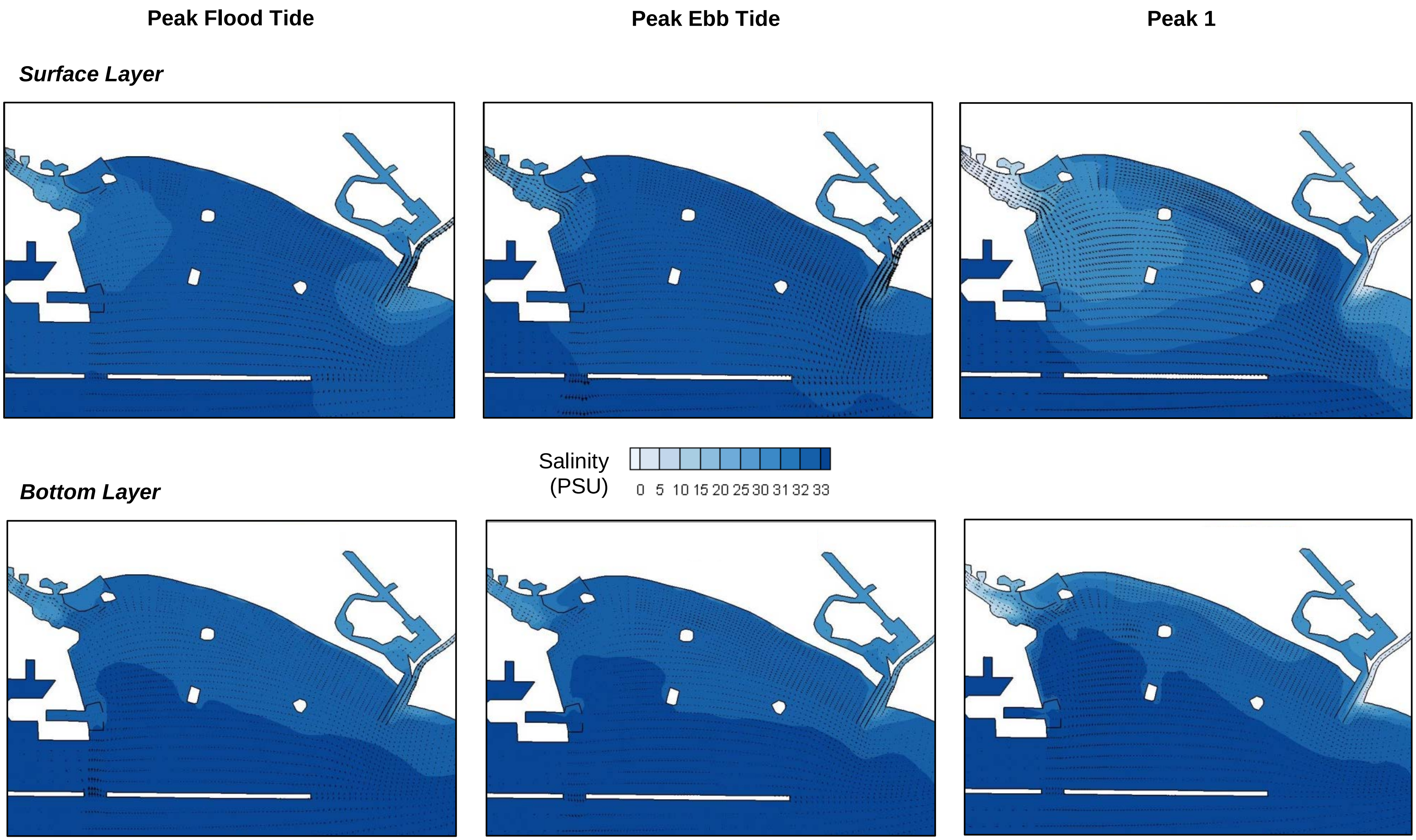


Figure 4.3 Existing Conditions Salinity Spatial Plots

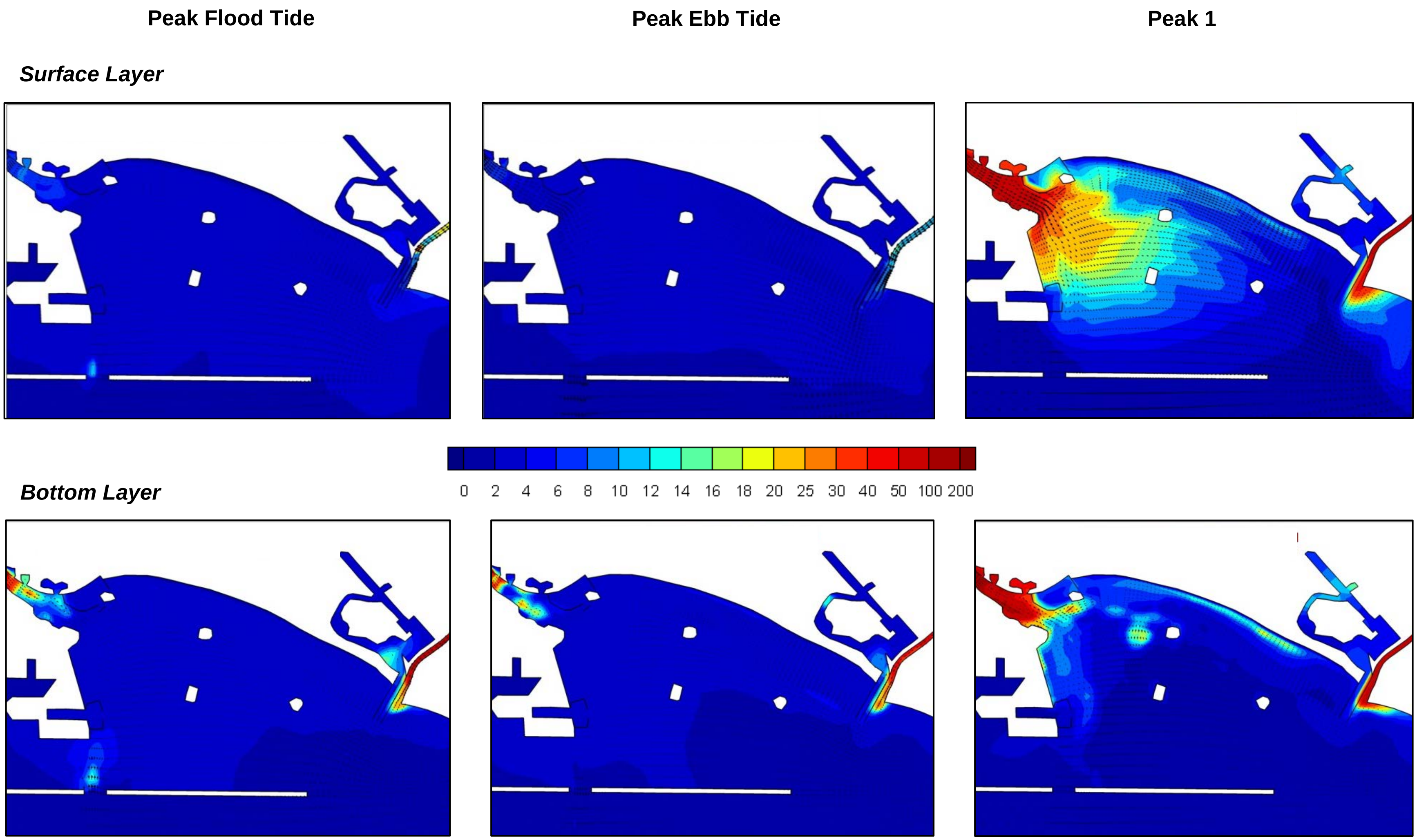


Figure 4.4 Existing Conditions Sediment Spatial Plots

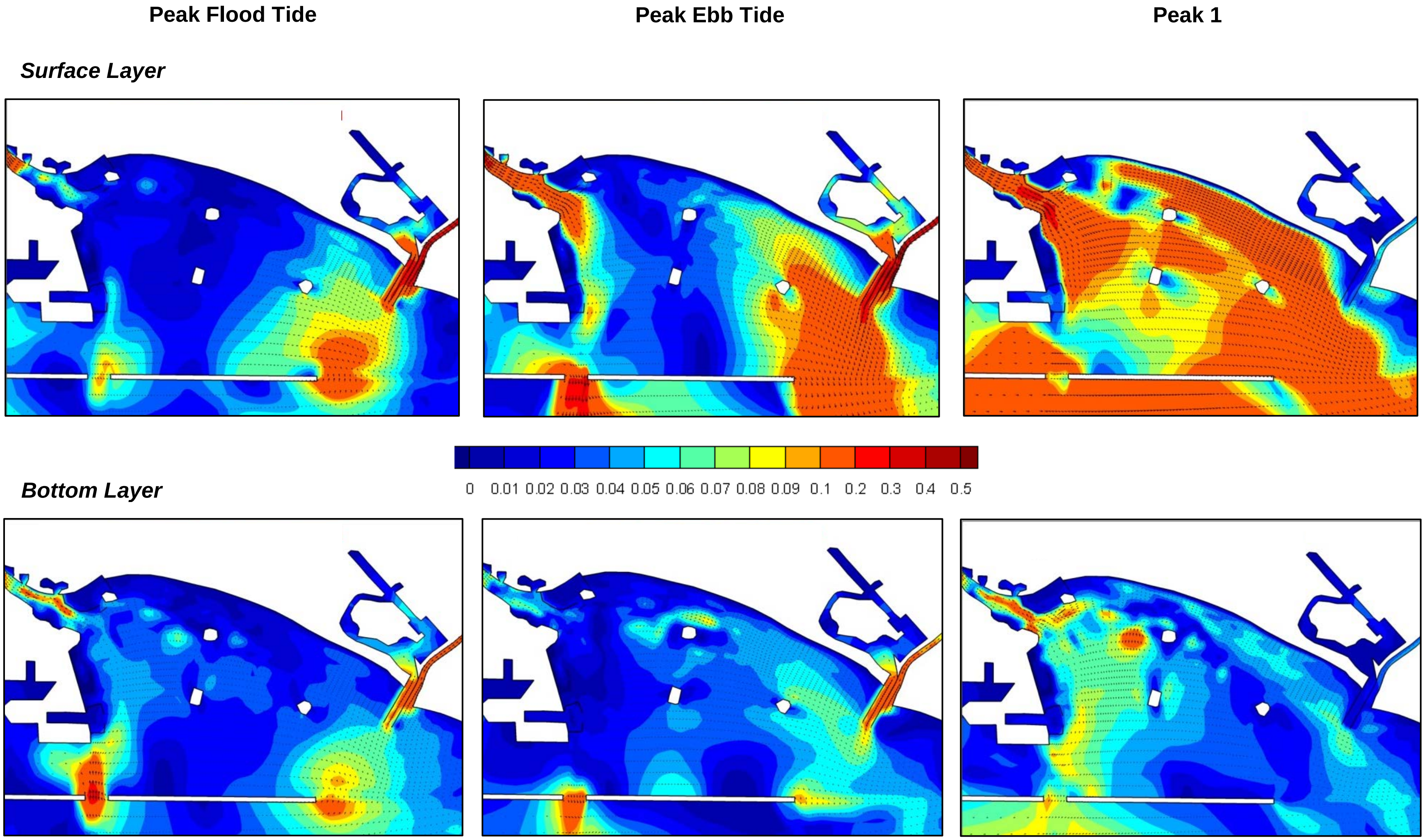


Figure 4.5 Scenario 1 Velocity Spatial Plots

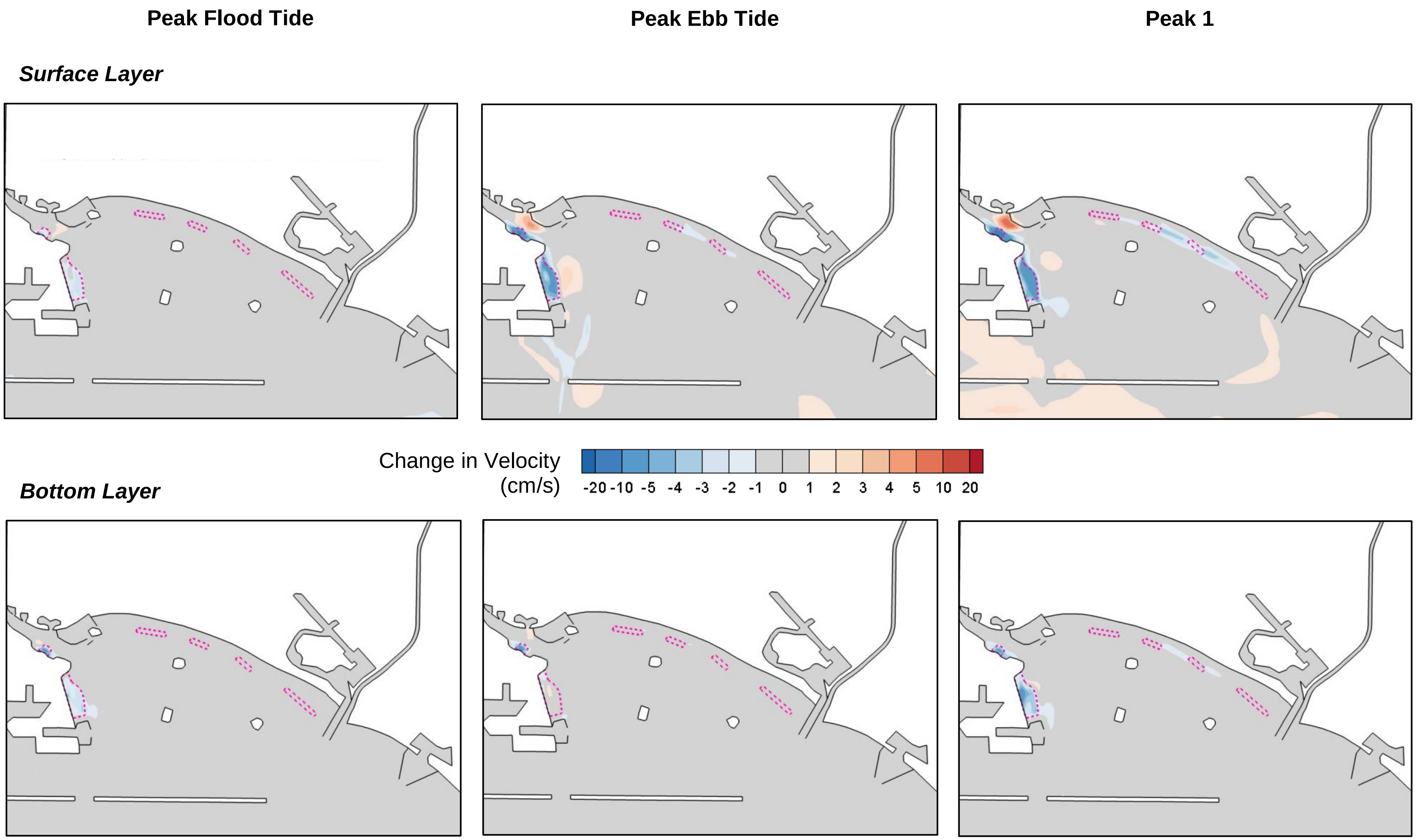


Figure 4.6 Scenario 1 Changes in Velocity Spatial Plots

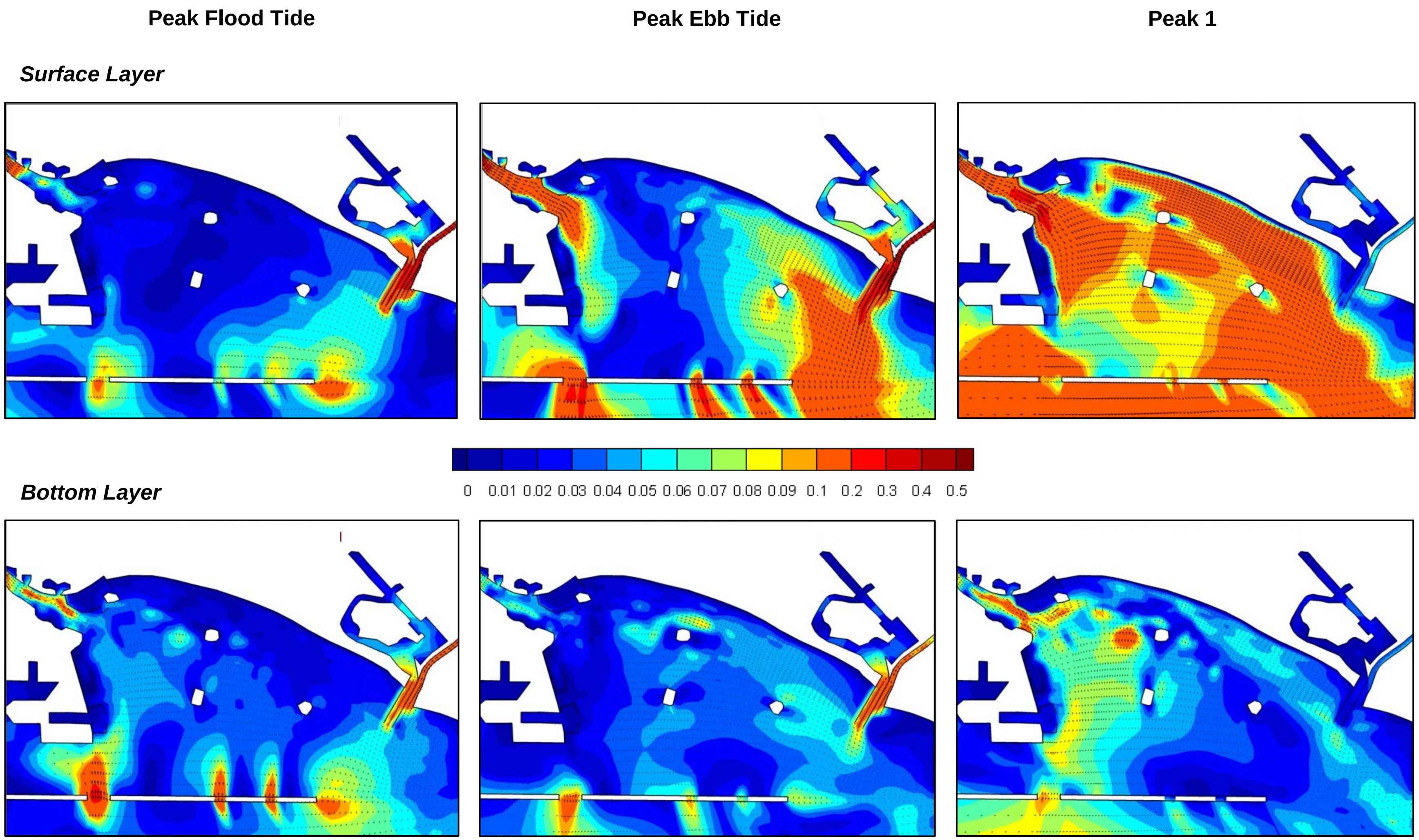


Figure 4.7 Scenario 2 Velocity Spatial Plots

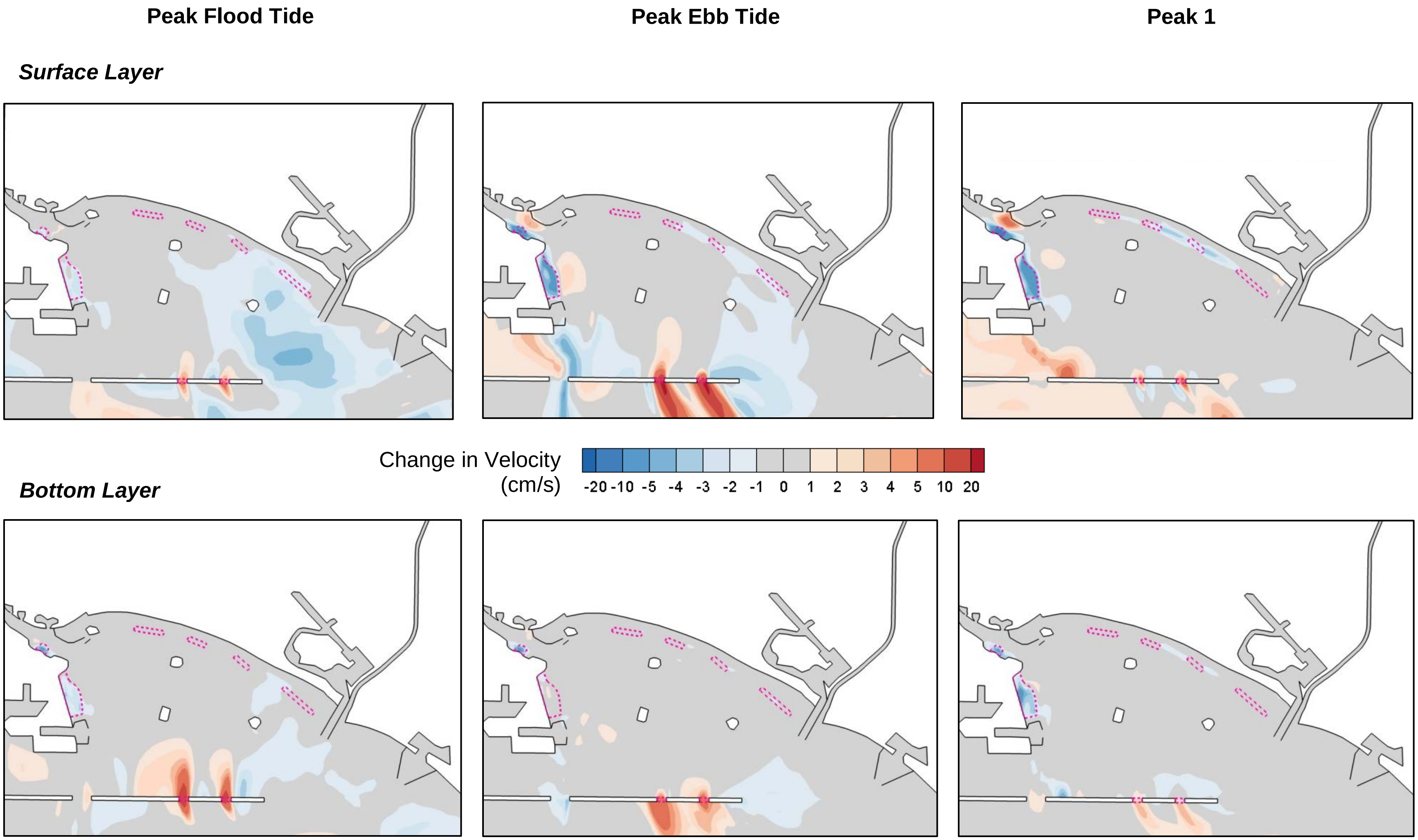


Figure 4.8 Scenario 2 Changes in Velocity Spatial Plots

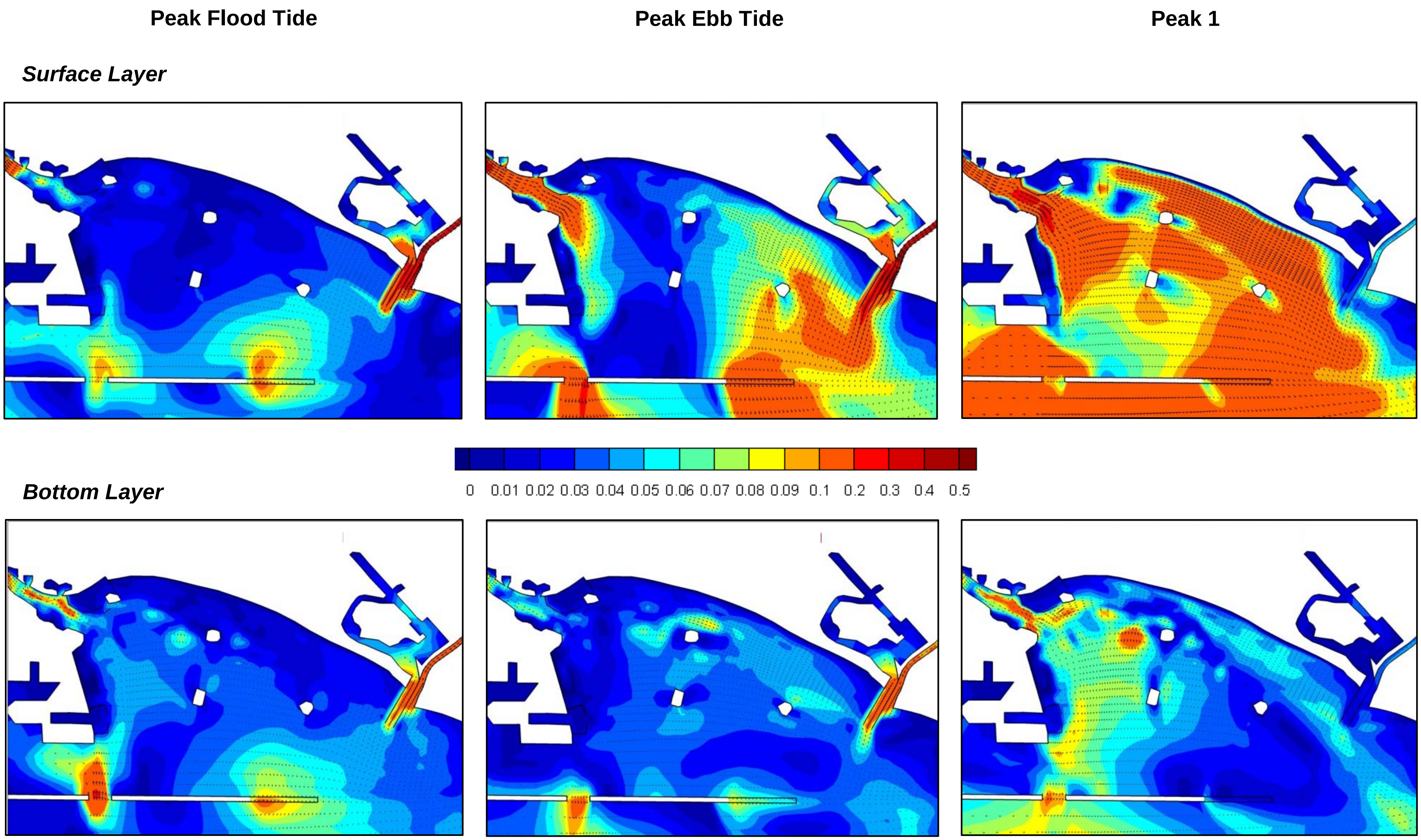


Figure 4.9 Scenario 3 Velocity Spatial Plots

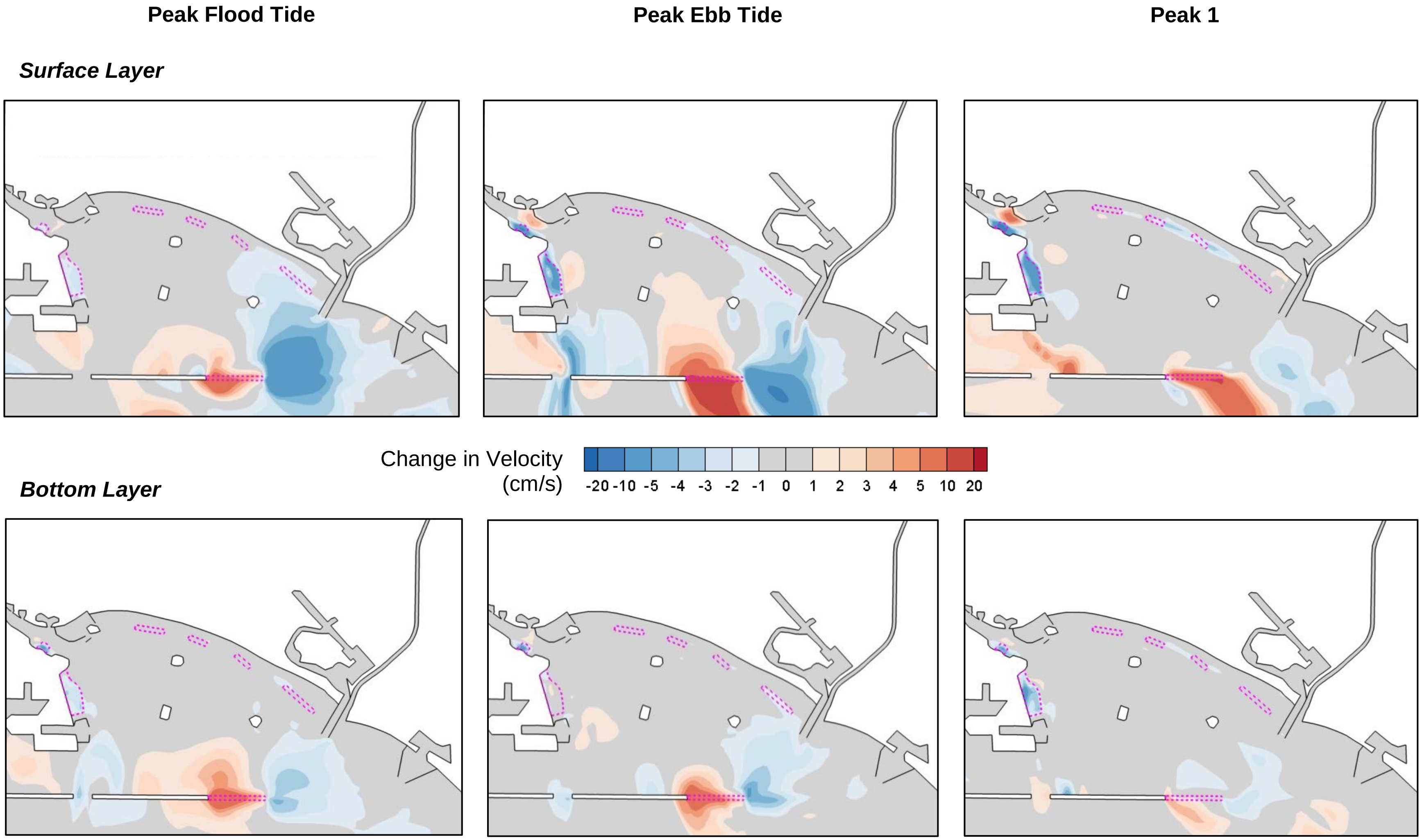


Figure 4.10 Scenario 3 Changes in Velocity Spatial Plots

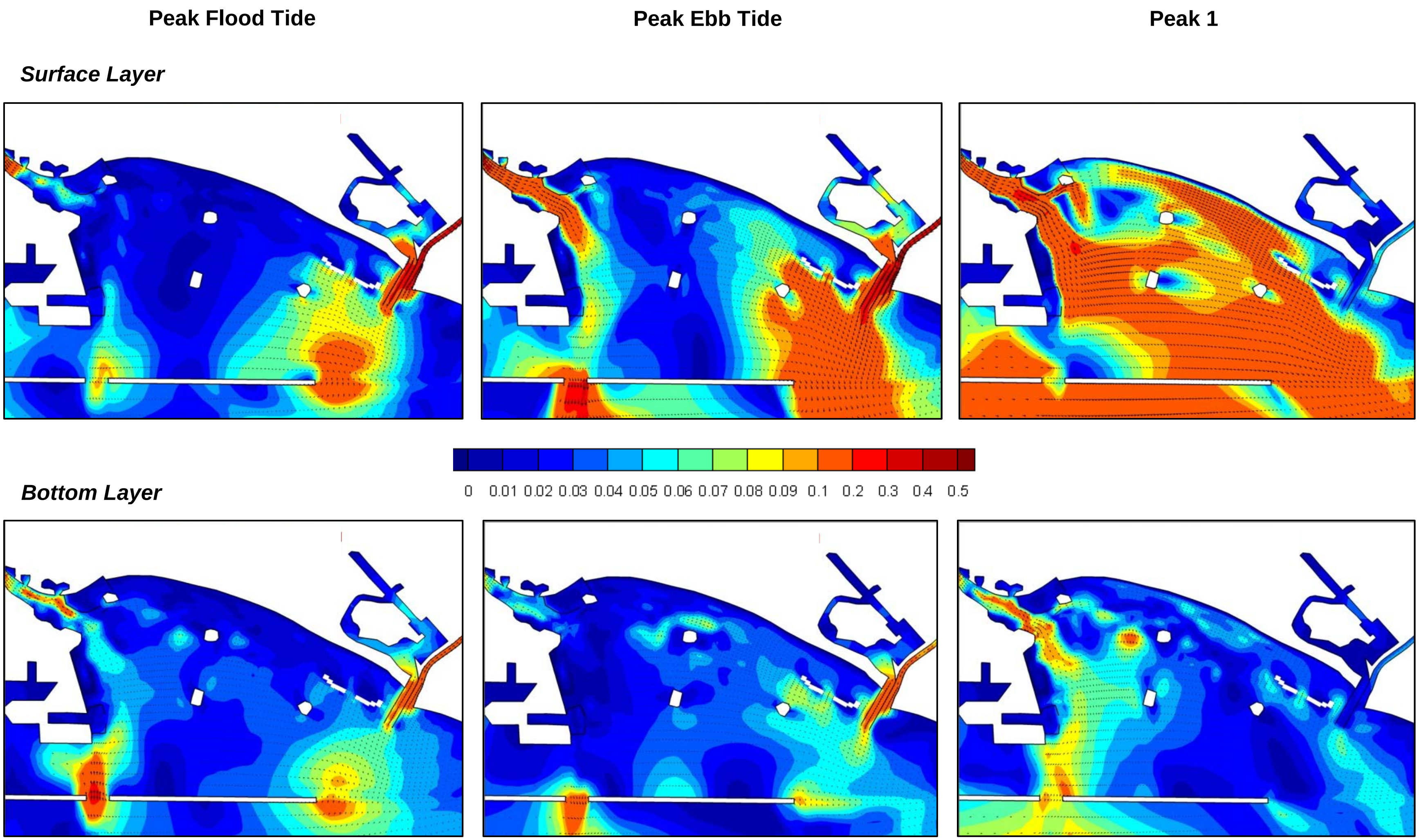


Figure 4.11 Scenario 4 Velocity Spatial Plots

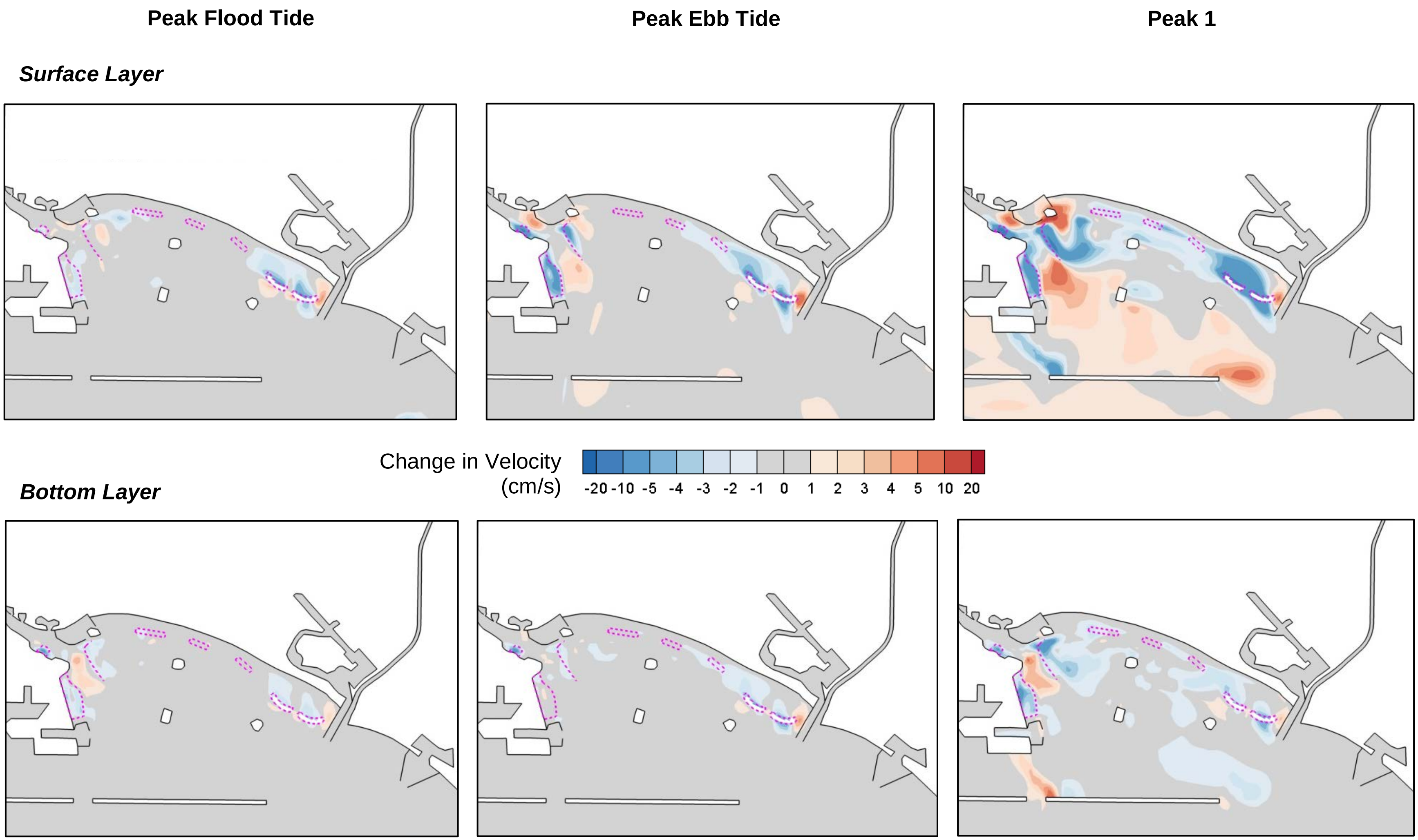


Figure 4.12 Scenario 4 Changes in Velocity Spatial Plots

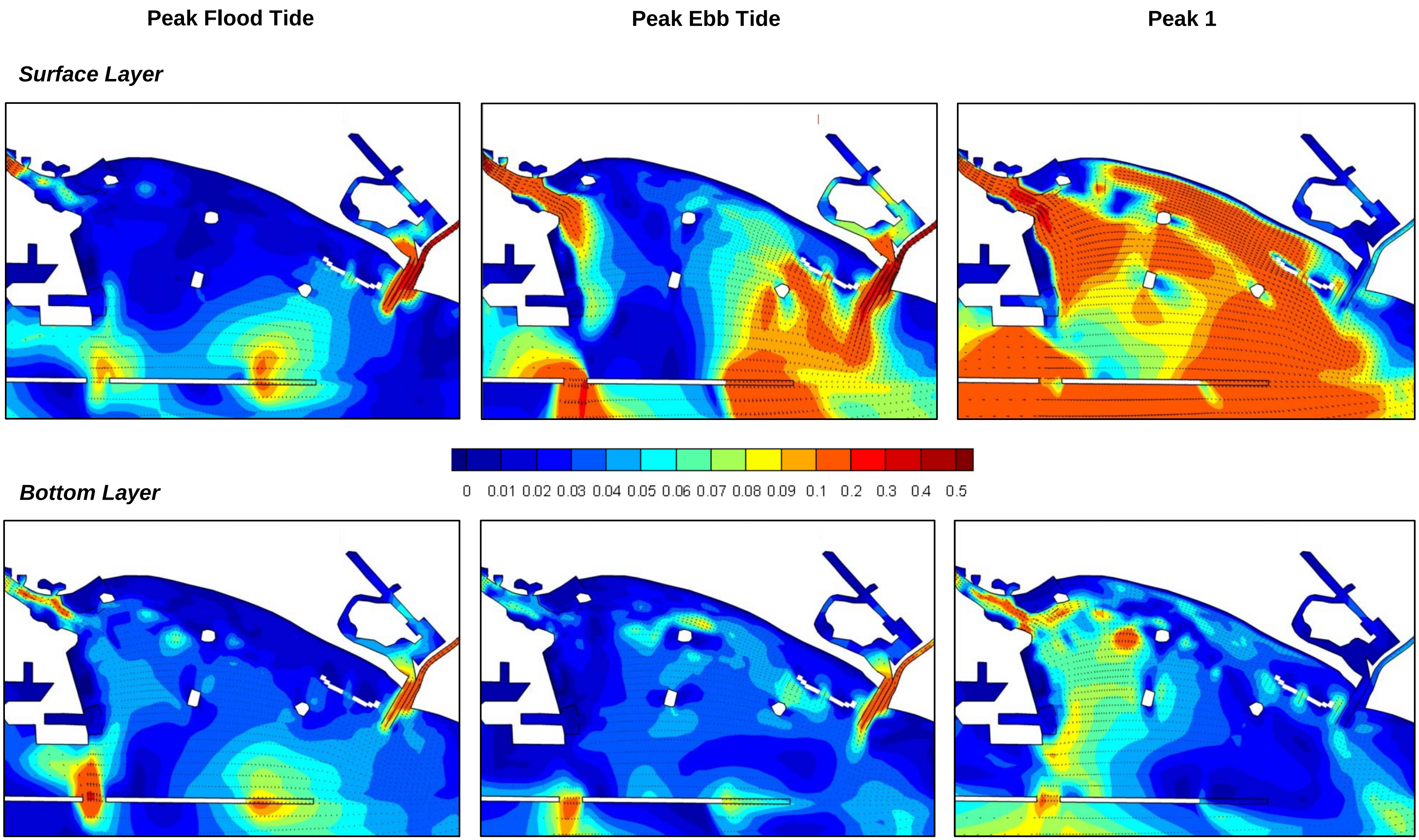


Figure 4.13 Scenario 5 Velocity Spatial Plots

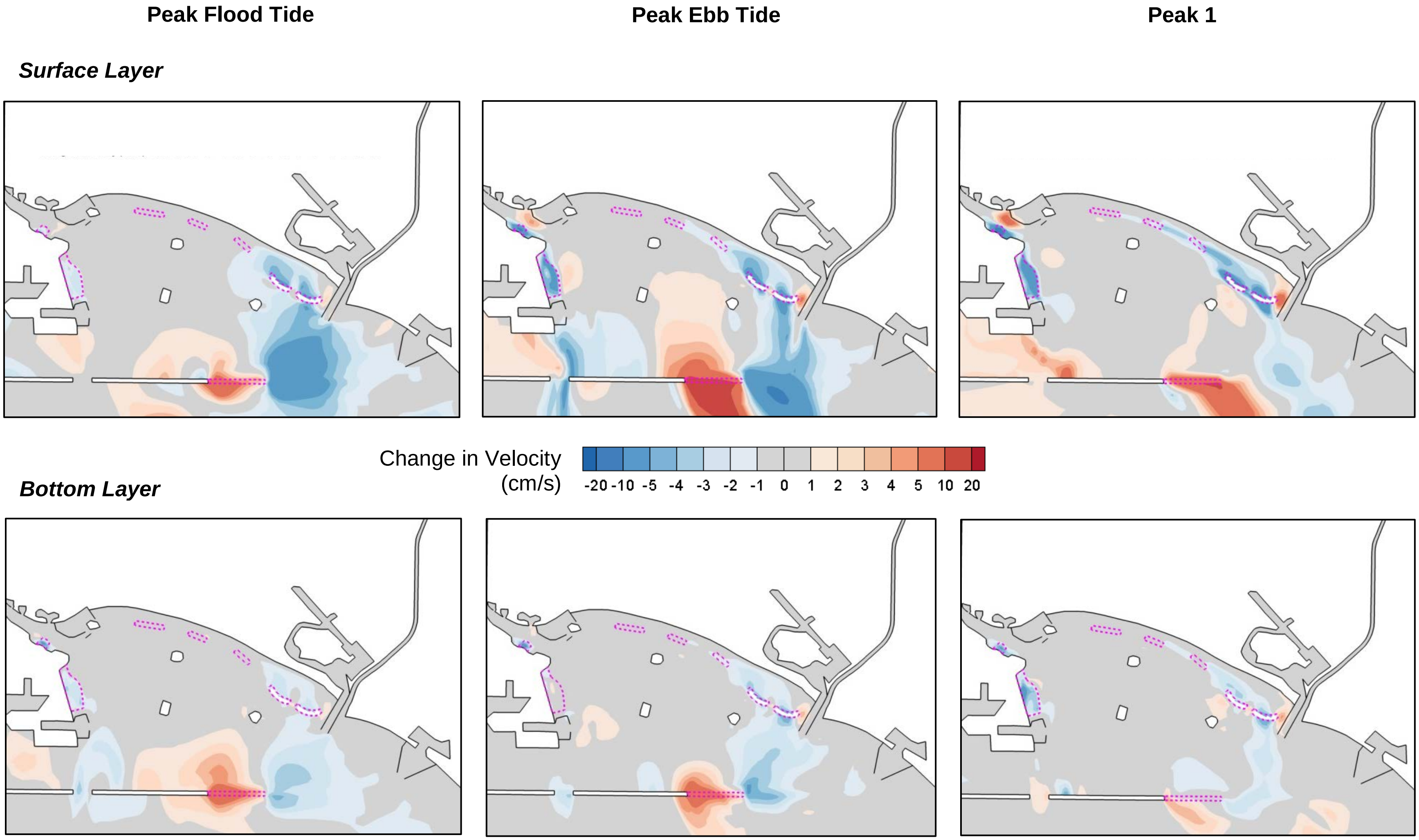


Figure 4.14 Scenario 5 Changes in Velocity Spatial Plots

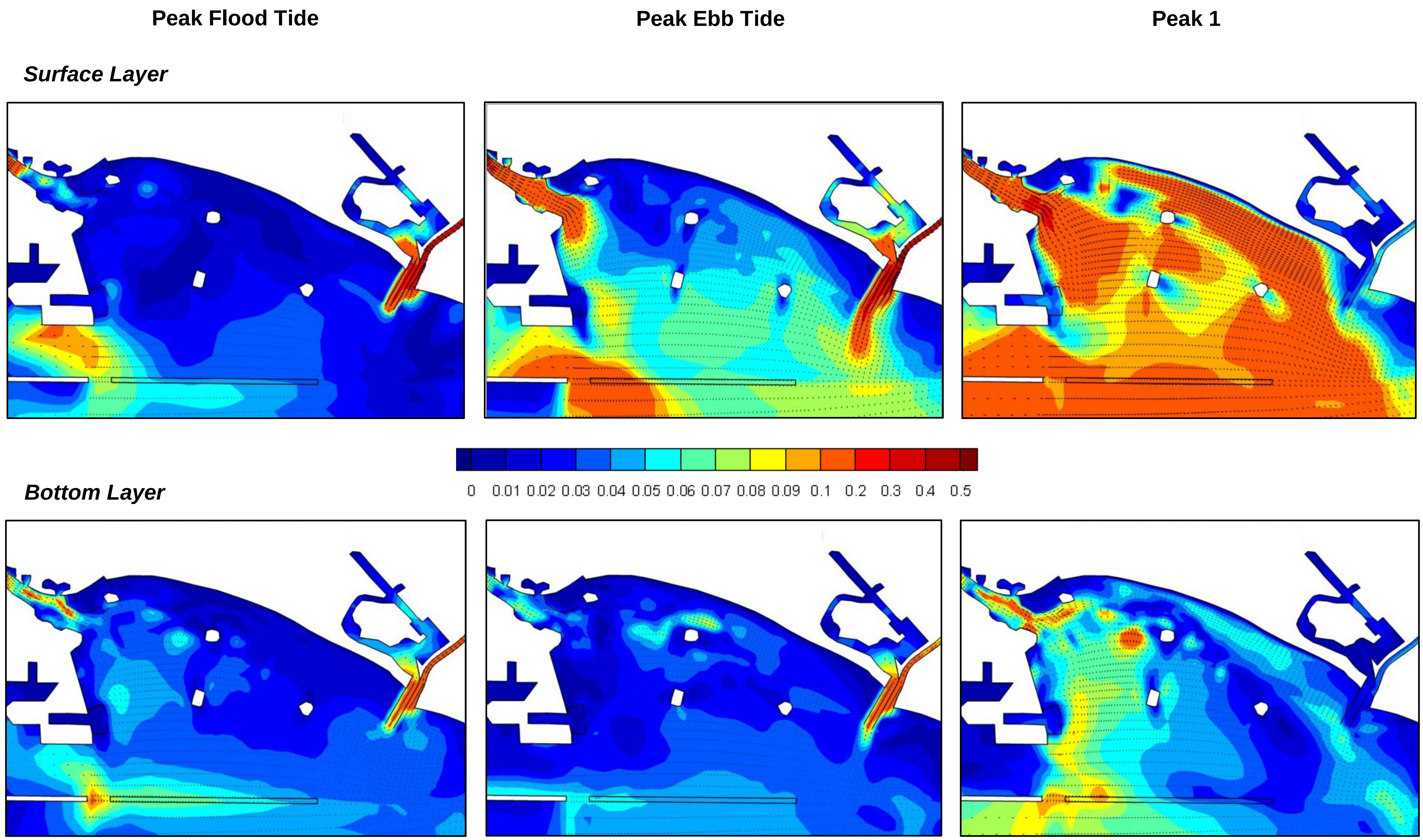


Figure 4.15 Scenario 6 Velocity Spatial Plots

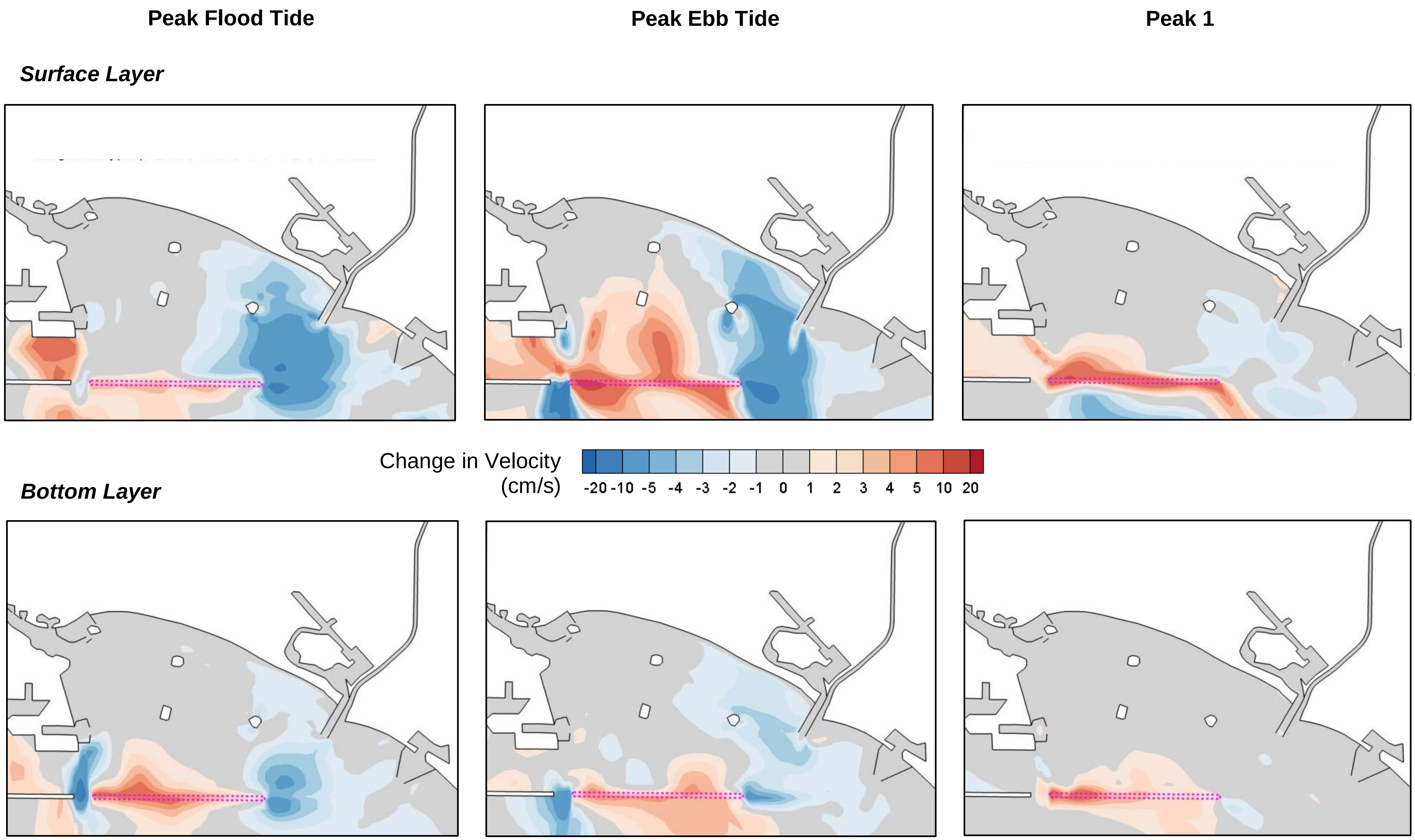


Figure 4.16 Scenario 6 Changes in Velocity Spatial Plots

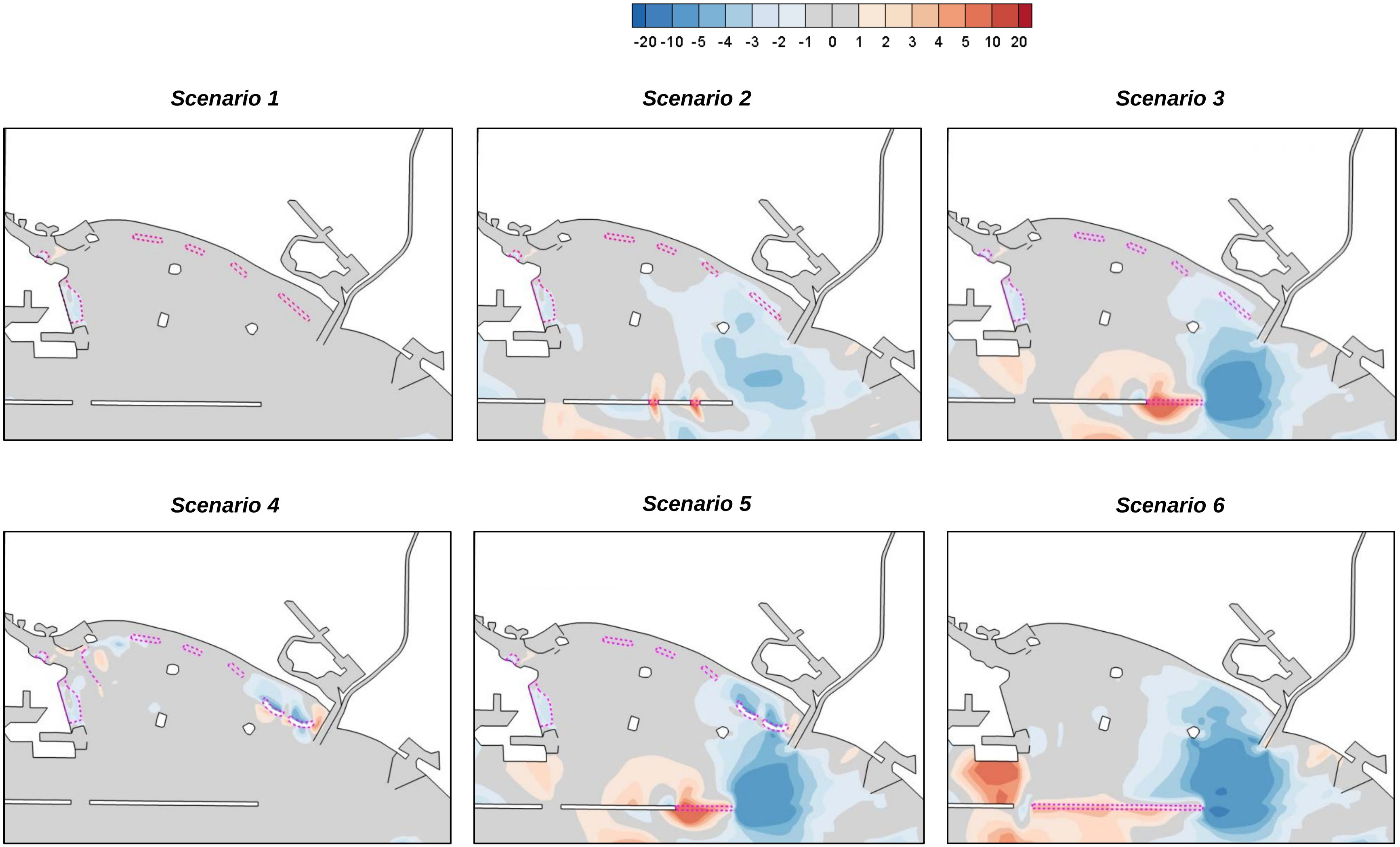


Figure 4.17 Comparison of Changes in Surface Velocity for Peak Flood Tide

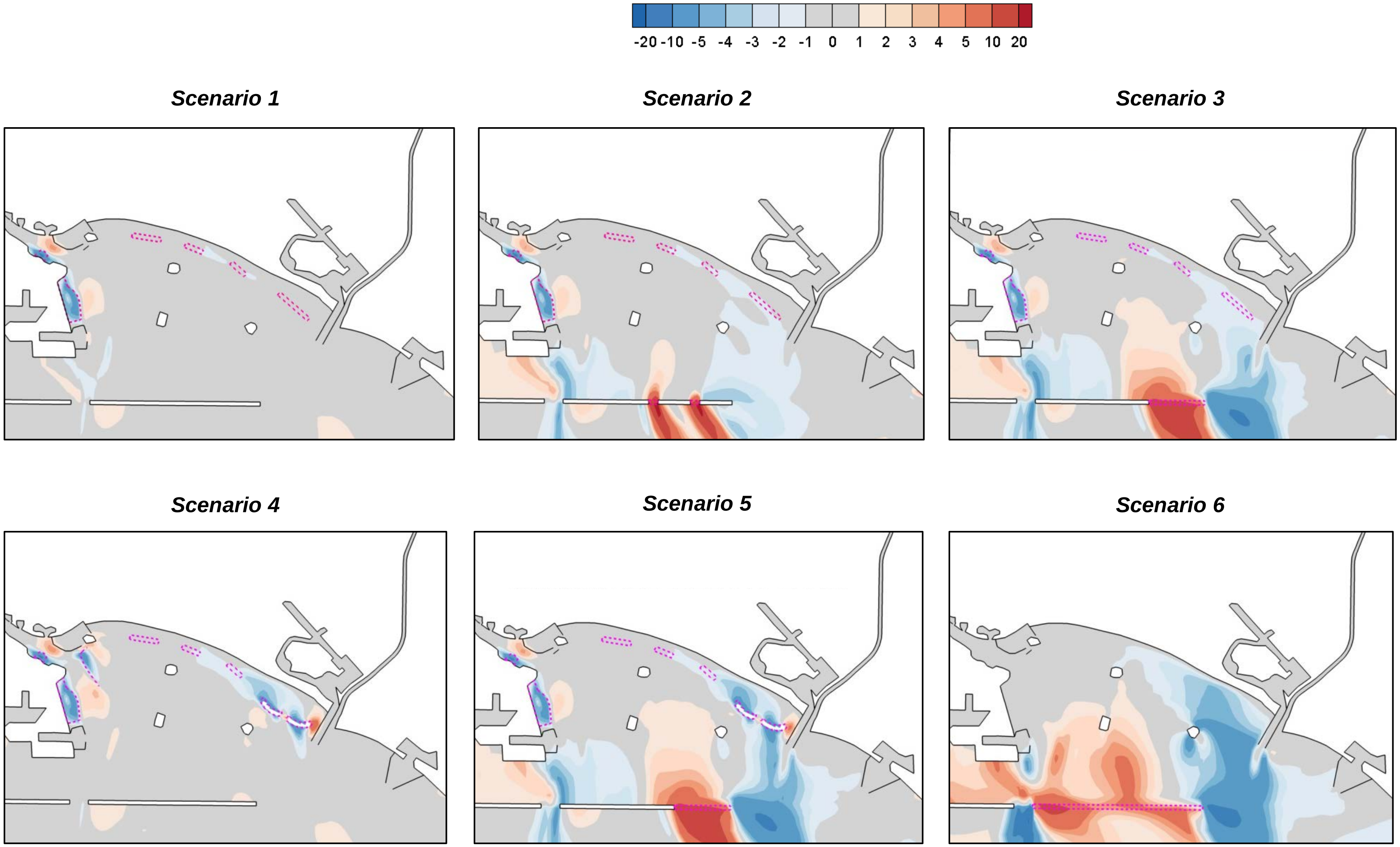


Figure 4.18 Comparison of Changes in Surface Velocity for Peak Ebb Tide

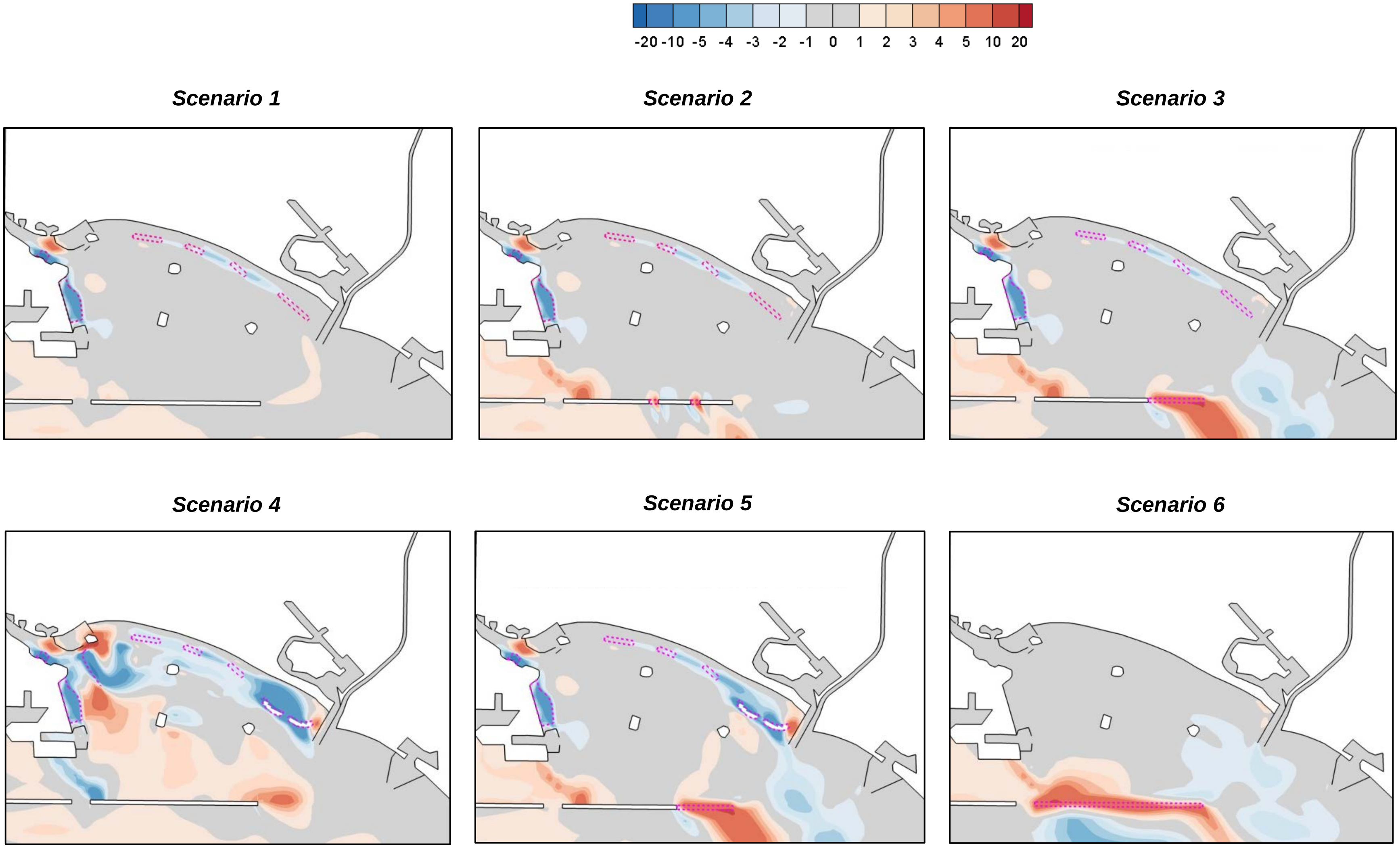


Figure 4.19 Comparison of Changes in Surface Velocity for Wet Weather

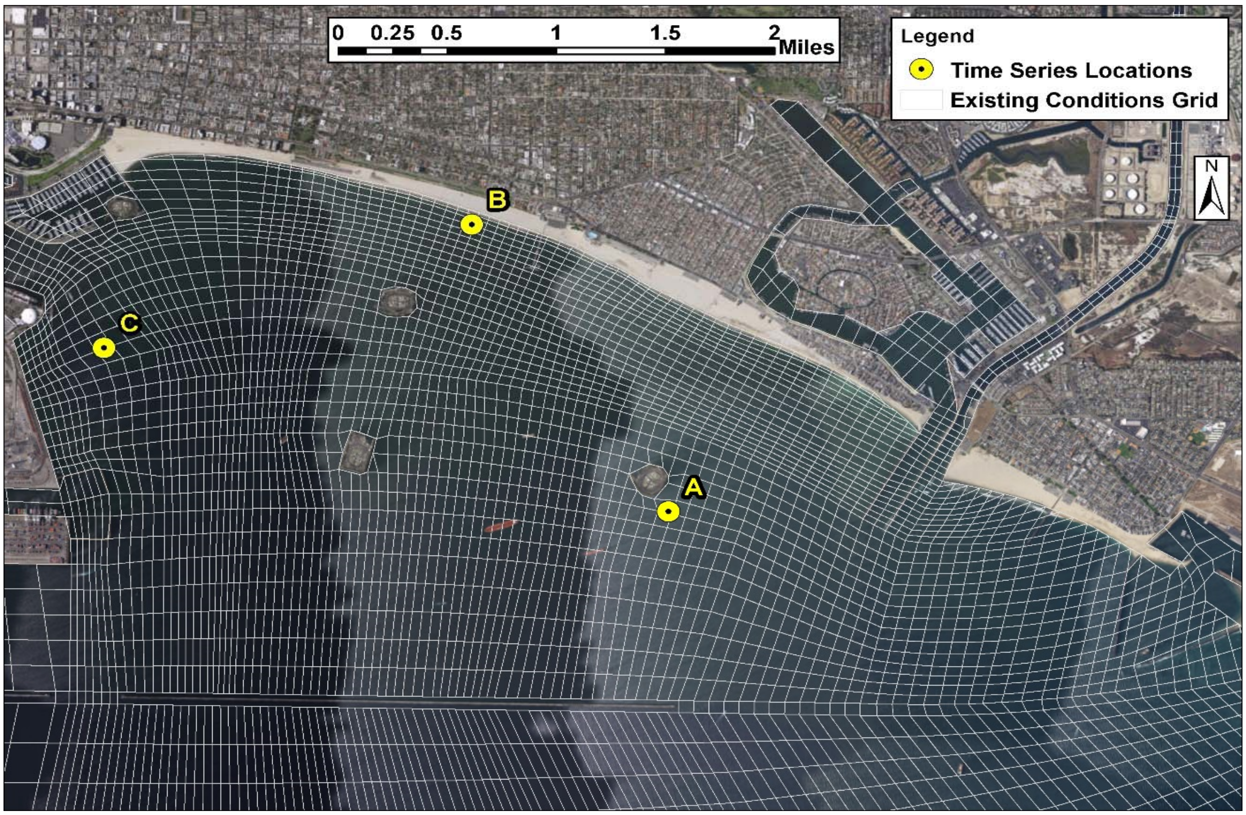
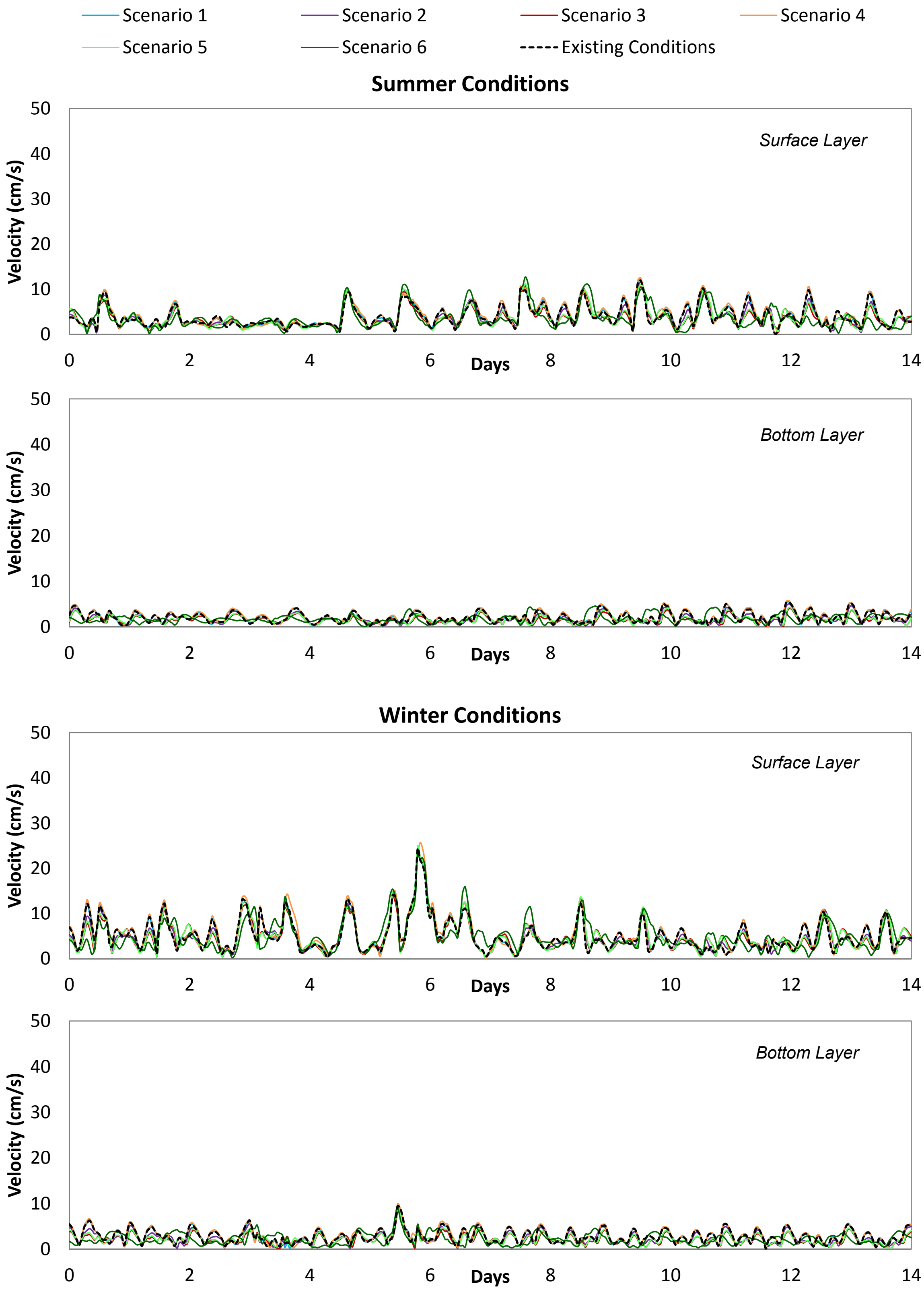
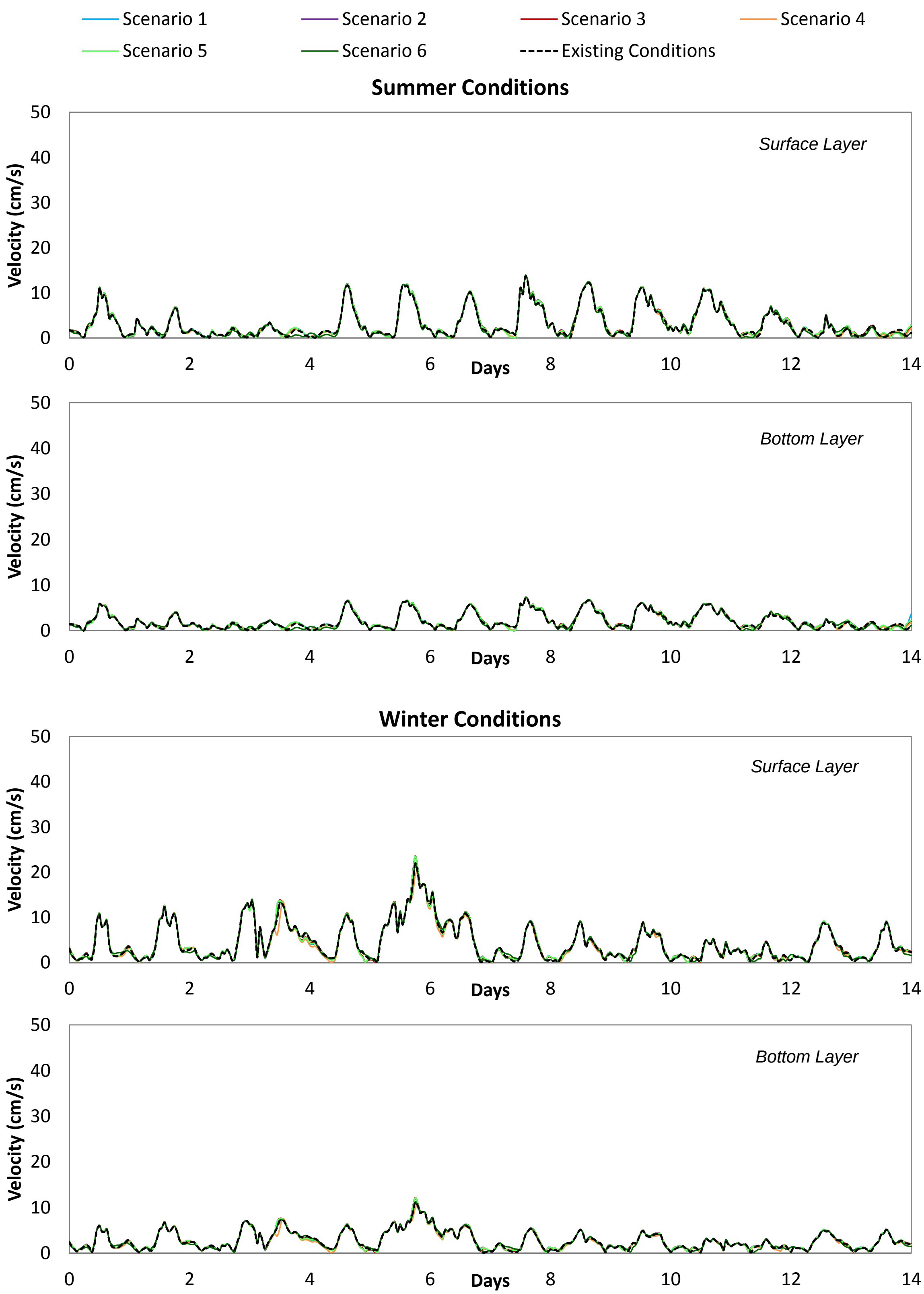


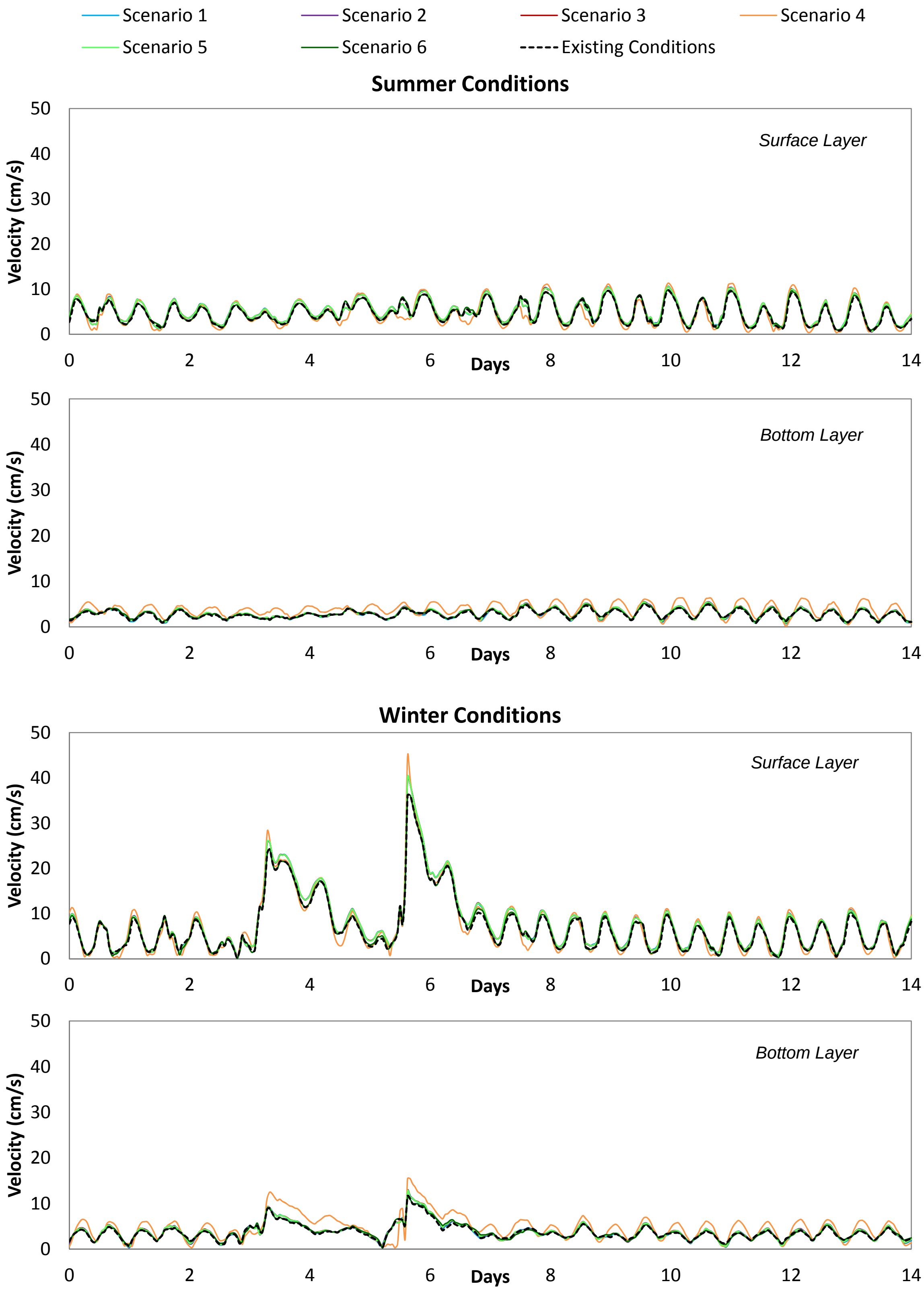
Figure 4.20 Model Output Locations for Time Series Plots



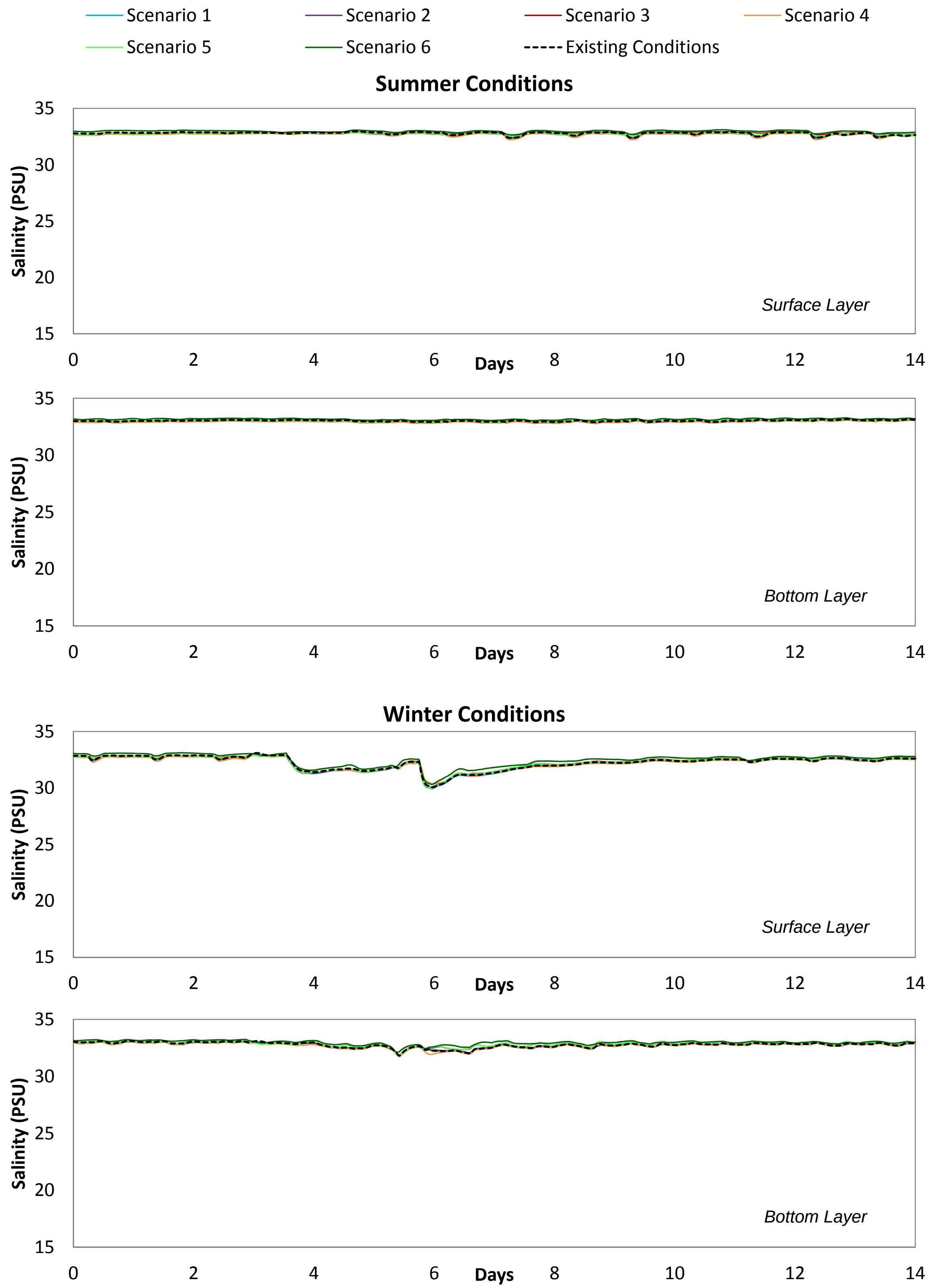
Figue 4.21 Currents at Location A



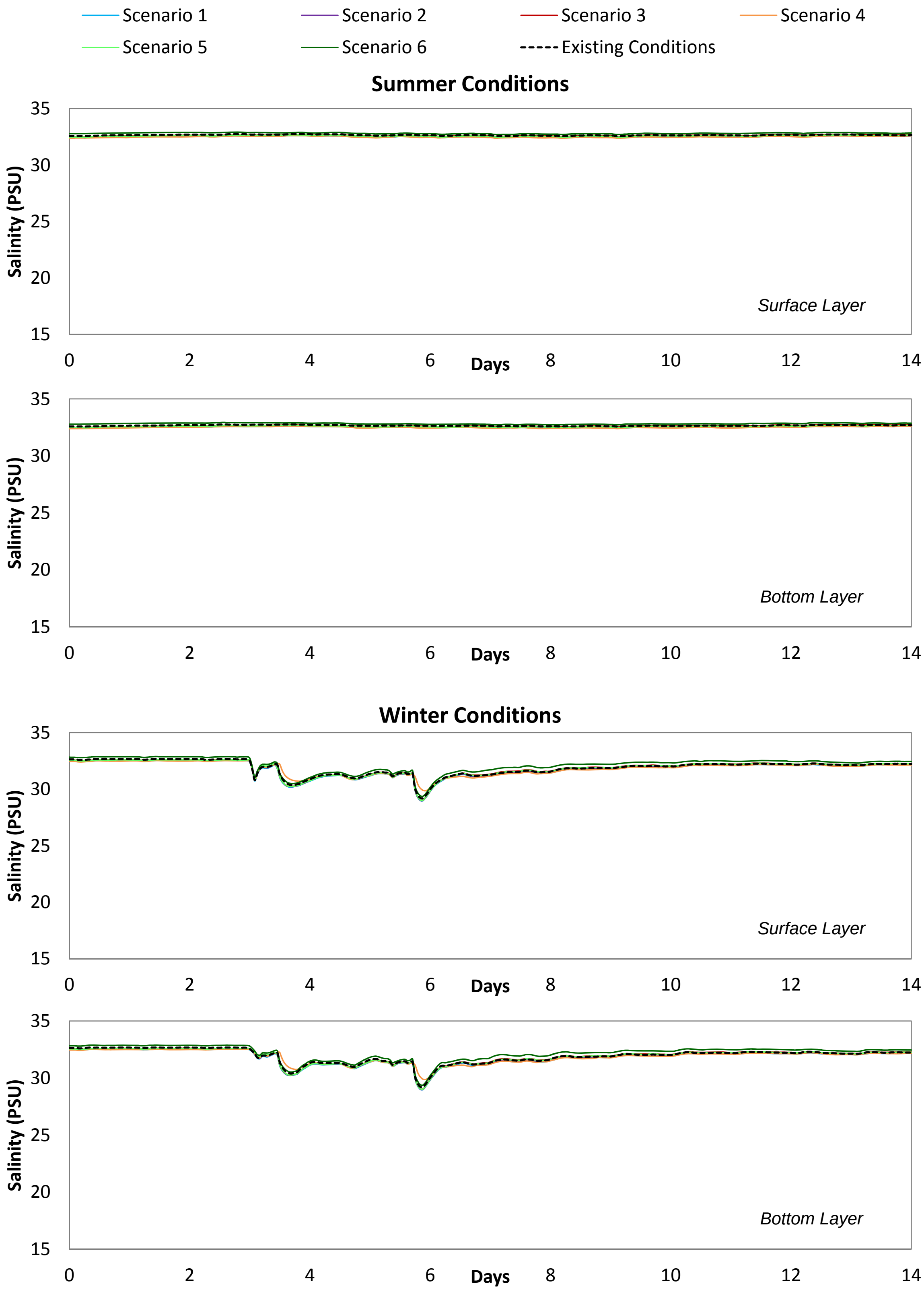
Figue 4.22 Currents at Location B



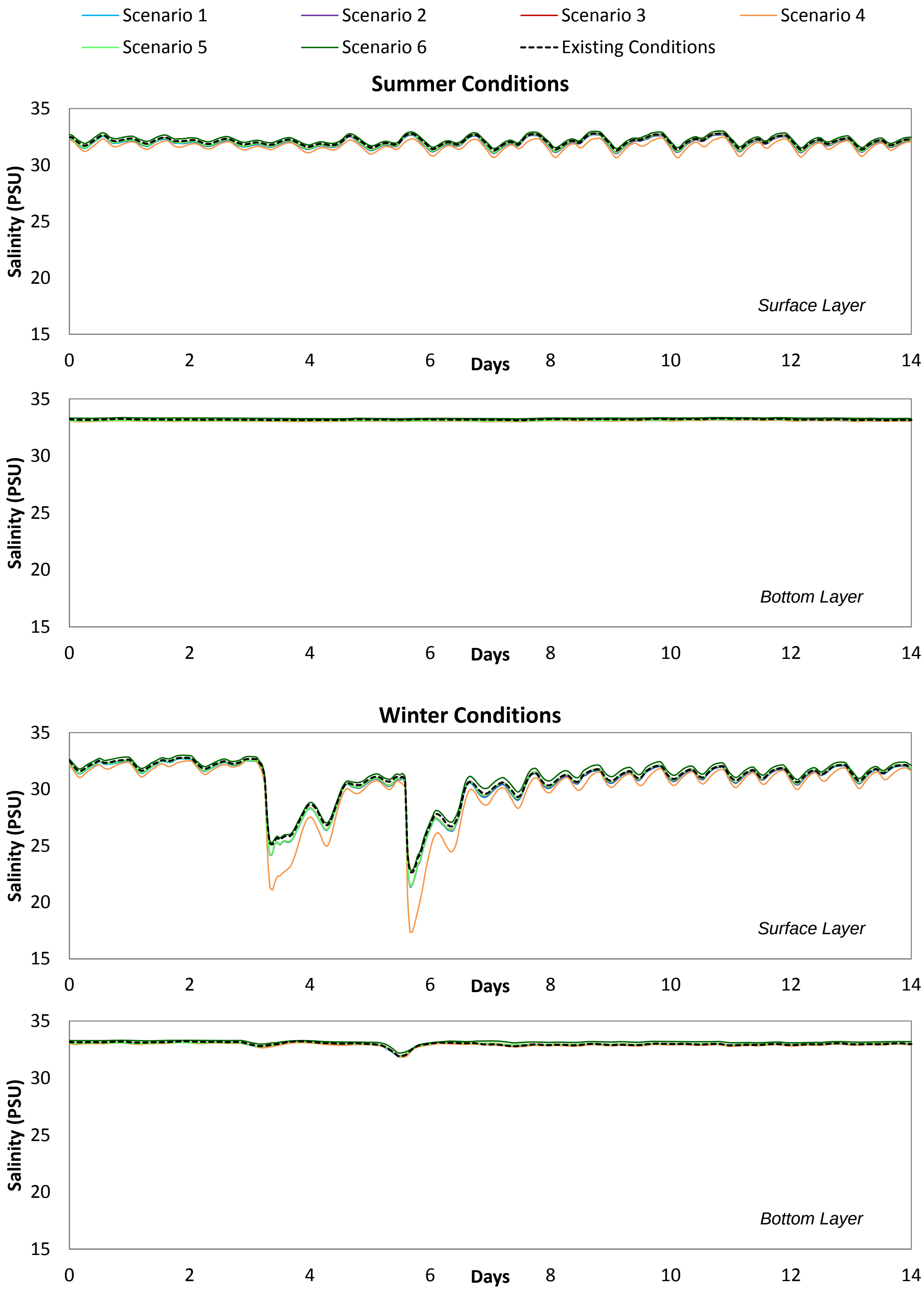
Figue 4.23 Currents at Location C



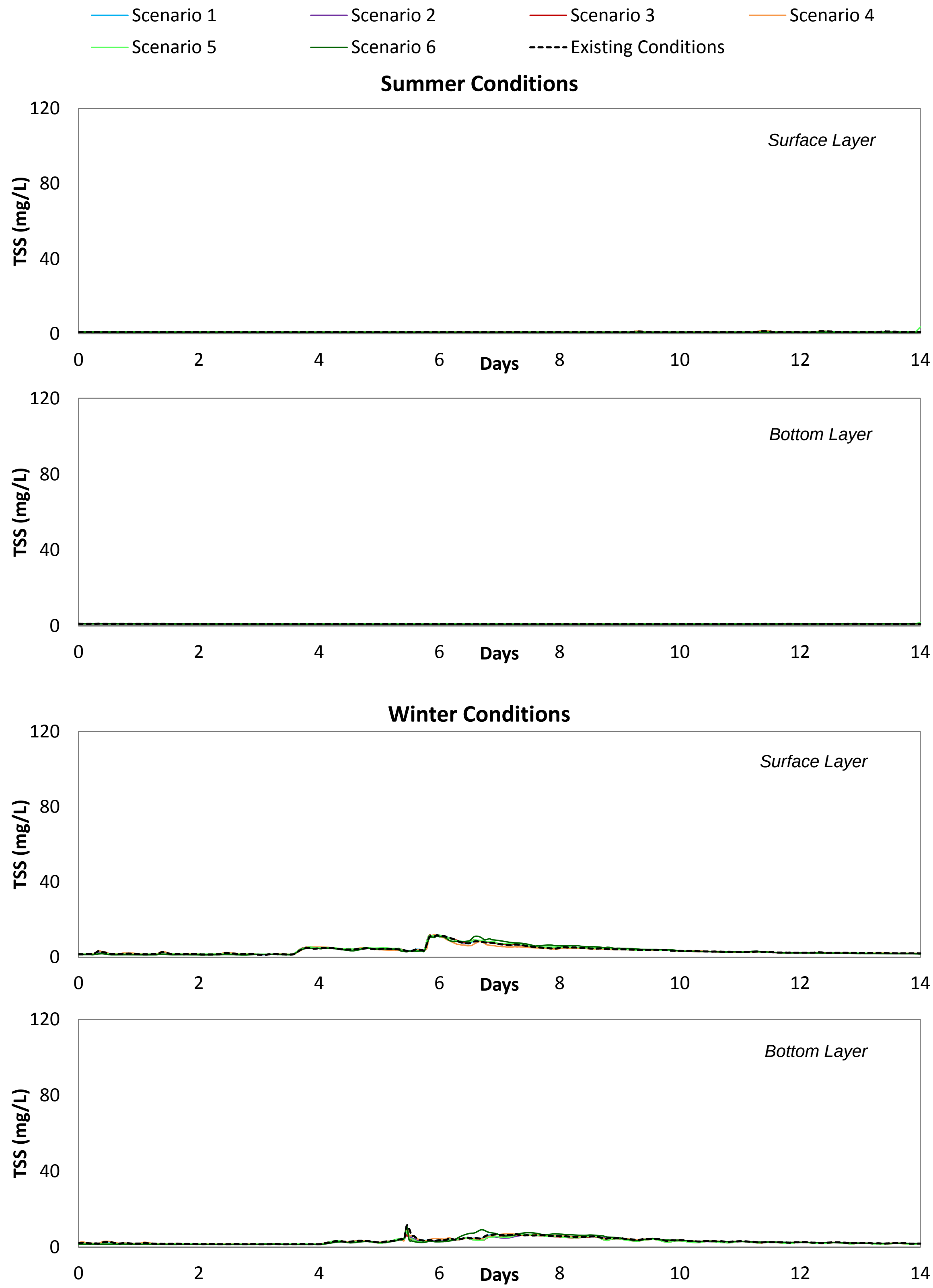
Figue 4.24 Salinity at Location A



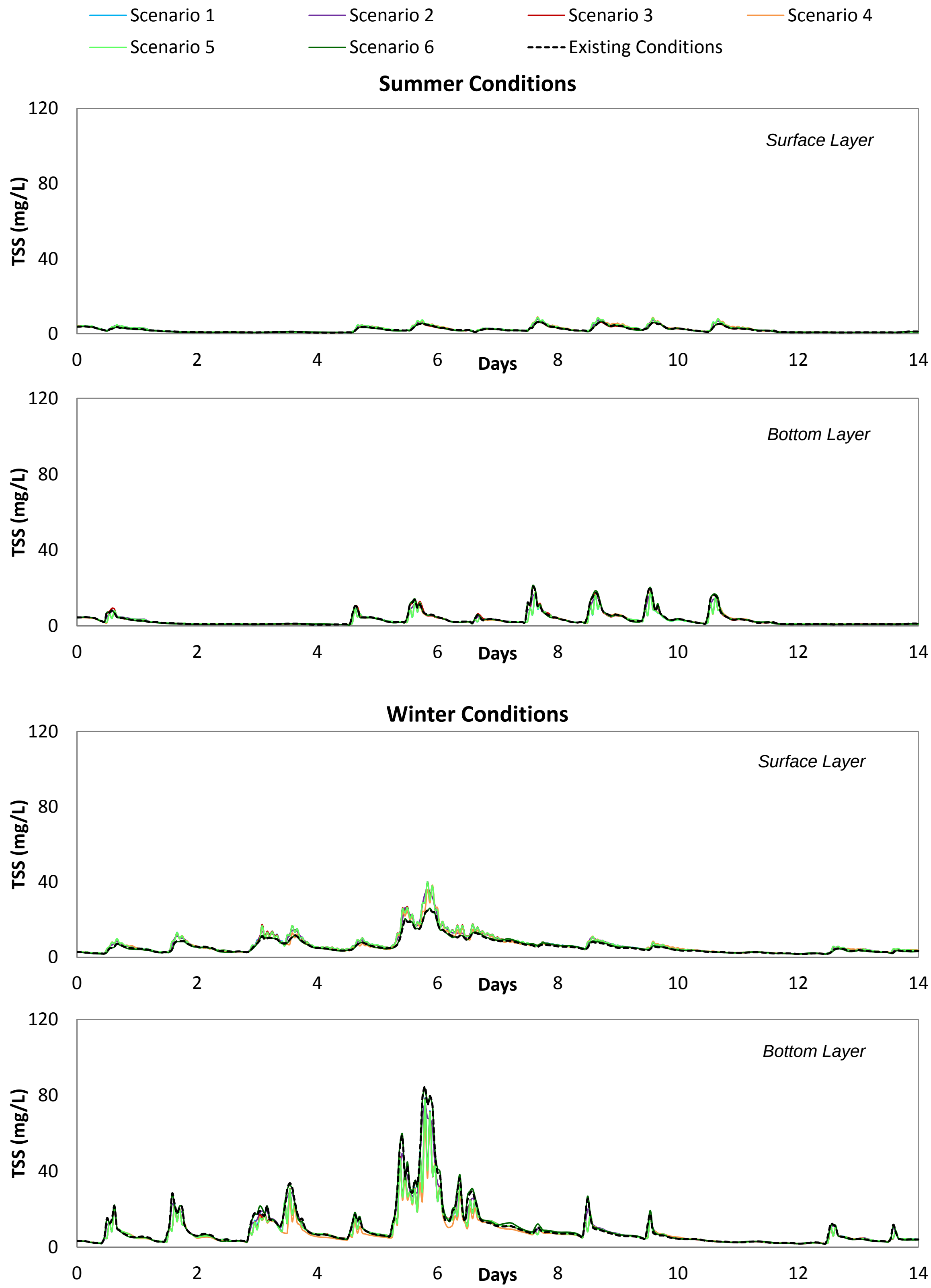
Figue 4.25 Salinity at Location B



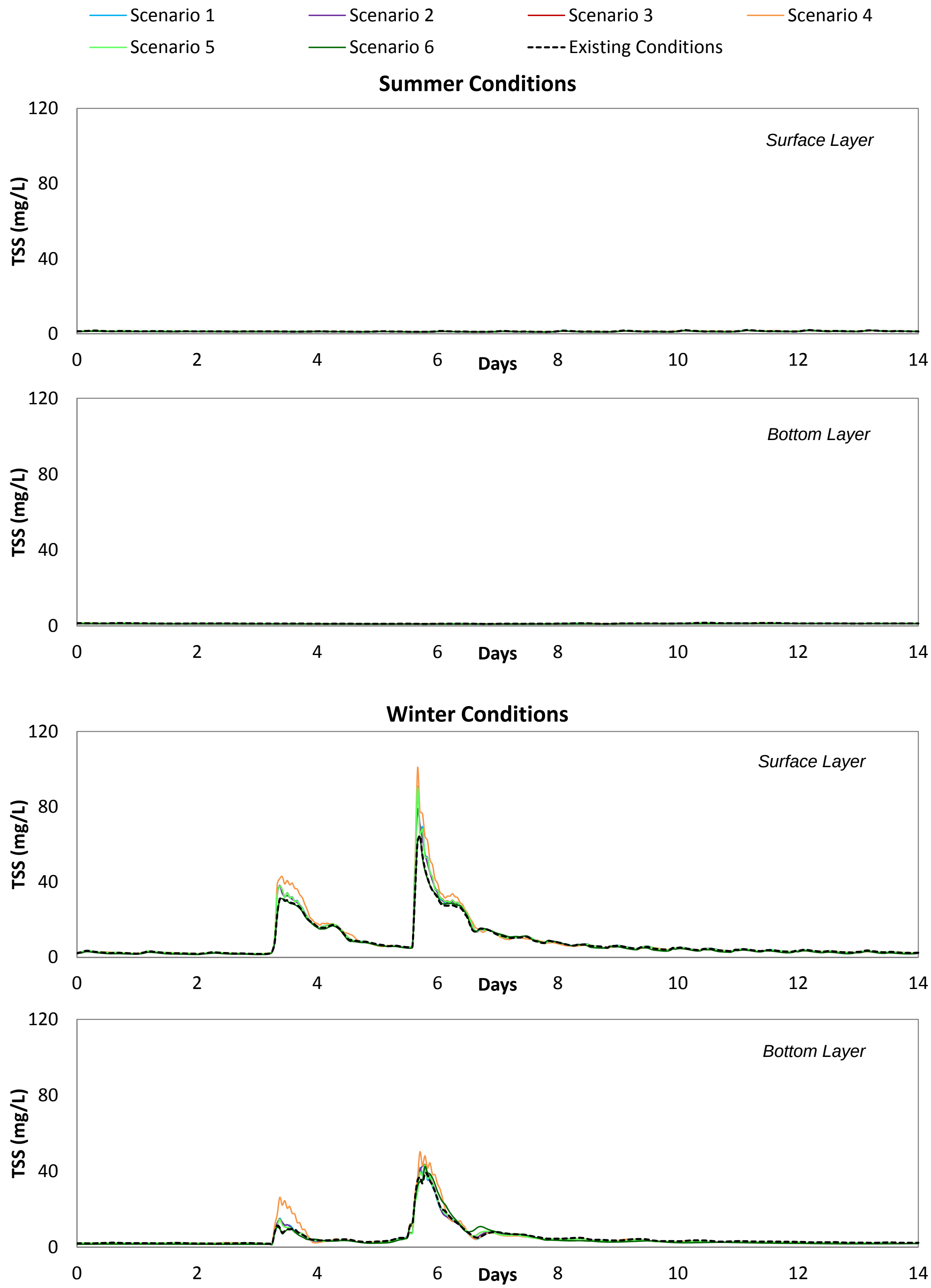
Figue 4.26 Salinity at Location C



Figue 4.27 Sediment at Location A



Figue 4.28 Sediment at Location B



Figue 4.29 Sediment at Location C

5. NUMERIC TRACER TRACKING ANALYSIS

A numeric tracer tracking study was conducted to evaluate changes in bay circulation, and for use as a surrogate to examine potential sediment transport from LAR and SGR into ESPB during rain events. For this study, neutrally buoyant tracer particles were released at various locations in ESPB and near the LAR and SGR estuary areas at different times during the winter conditions, and tracked to determine the transport of the tracer particles once they were released.

The numeric tracer tracking was conducted for Existing Conditions and four of the restoration alternatives with structural features – Scenario 2 (breakwater notching), Scenario 3 (breakwater lowering, partial), Scenario 4 (training wall), and Scenario 6 (breakwater lowering, of entire structure). Scenario 1 was excluded since this restoration alternative only involved habitat features that have localized effects on velocities. Scenario 5 was not included since changes in the hydrodynamics for that alternative are similar to, and already represented by, those under Scenario 3.

5.1 Release Times and Locations

Hydrodynamic modeling of Existing Conditions was utilized to select particle release locations and times depicted in Figure 5.1. Six particle release locations were selected – with two at the river mouths, two near the entrance to ESPB, and two along the shoreline. Particle tracking at the six locations was conducted for three release times under different hydrodynamic conditions. Release 1 corresponds to the start of an ebbing tide, Release 2 represents transport during wet weather conditions, and Release 3 corresponds to the start of a flood tide.

The numeric tracer tracking analysis involved using the ESPB Model hydrodynamic results to compute the transport (i.e., movement) of a particle based on velocity vectors. Hydrodynamic conditions of ESPB show 3D flow characteristics with varying velocity magnitudes and directions through the water column. Hence, the numeric tracer tracking was conducted for surface and bottom water layers. Since the numeric tracer particle was assumed to be neutrally buoyant with no settling, it does not represent actual sediment transport, and instead is used only as a surrogate to provide general movement of sediment particles.

5.2 Particle Tracking Results

The particle tracking results are shown by release location, and particle tracks for all the model scenarios are compared together in Figures 5.2 to 5.7. The particle tracks were trimmed if the particles reached land or physical structures – to maintain realistic results, or when they were transported beyond ESPB. The results for the surface and bottom layer results are provided for the three release times. In the figures, each panel shows the particle tracks for a particular release time and water layer, and different colors are used for each model scenario.

Release Locations A and B – Near River Mouths

Particle tracks for Location A, near the LAR mouth, and Location B, near the SGR mouth, are shown in Figure 5.2 and Figure 5.3, respectively. Overall, it can be seen that the particle tracks of all the model scenarios – represented by different colors – are similar to each other, indicating similar transport and harbor circulation under all the model scenarios.

For releases at Location A, near LAR, the hydrodynamic conditions transport particles at the surface layer into ESPB. During tidal conditions, as shown by Release 1 and 3, particles gradually move back and forth in a southeast direction. During wet weather (Release 2), velocities are higher and particles moved out of the bay in a short amount of time. In contrast, the hydrodynamics of the bottom layer drive particles upstream of the LAR, illustrating the 3D hydrodynamic conditions near the estuary – with the flows of the top and bottom layer moving in different directions.

Transport from the SGR is illustrated by Location B, as shown in Figure 5.3. Under tidal conditions, surface velocities transport particles southward out of the bay. In the bottom layer, where velocities are lower, particles either oscillate along the SGR or are entrained into Alamitos Bay. Under wet weather conditions, surface particles are quickly transported out of the bay, while particles at the bottom layer are transported into Alamitos Bay.

Release Locations C and D – Near Entrances to ESPB

Releases at Location C illustrate transport conditions near Queens Gate. Particle tracks for Location C are provided in Figure 5.4. The particle tracks for the surface and bottom layers show transports in different directions, reflecting the spatial variance of the hydrodynamic conditions. In general, particles released at the surface layer may be transported in and out of the bay via Queens Gate, while particles released at the bottom layer are mainly transported into the bay.

Particle tracks for Location D, just east of the end of the breakwater, are shown in Figure 5.5. In general, the surface currents transport particles out of the bay over a short period of time, while particles released at the bottom layer tend to stay within the bay or become entrained in the tidal flows of Alamitos Bay.

Release Locations E and F – Along Shoreline

Results for particles released at Location E are depicted in Figure 5.6, which shows that the particles are generally transported downcoast and out of the bay. Greater movement is shown for the particles released at the surface layer, compared to those released at the bottom layer. For Release 2, during a rain event, particles are transported out of the bay much faster than Release 1 and Release 2 – reflecting the much faster velocities in the bay during rain events.

Particle tracks for release at Location F are shown in Figure 5.7. In general, the particles follow the tidal currents and move alongshore, downcoast, eastward, and eventually out of the bay. In general, particles released at the surface layer move faster than particles released at the bottom layer. All particles released at the surface layers were transported out of the bay during the 14-day simulation period, while particles released at the bottom remained in the bay.

5.3 Summary

Overall, the numeric tracer tracking results generally reflect the spatial distribution of the hydrodynamic conditions previously presented in Section 4, which showed similar transport conditions among the various model scenarios. In addition, the movement of particles released at the surface layer generally differed from those released at the bottom layer, since – as shown in Section 4 – the velocities at the surface layer are generally very different from the velocities near the harbor bed. Sometimes, the surface currents and near bottom currents moved in opposite directions, as shown by the results of the particle tracking analysis. These results show that, at times, the surface particles traveled southeastward towards the bay entrance, while the bottom particles moved in the opposite direction.

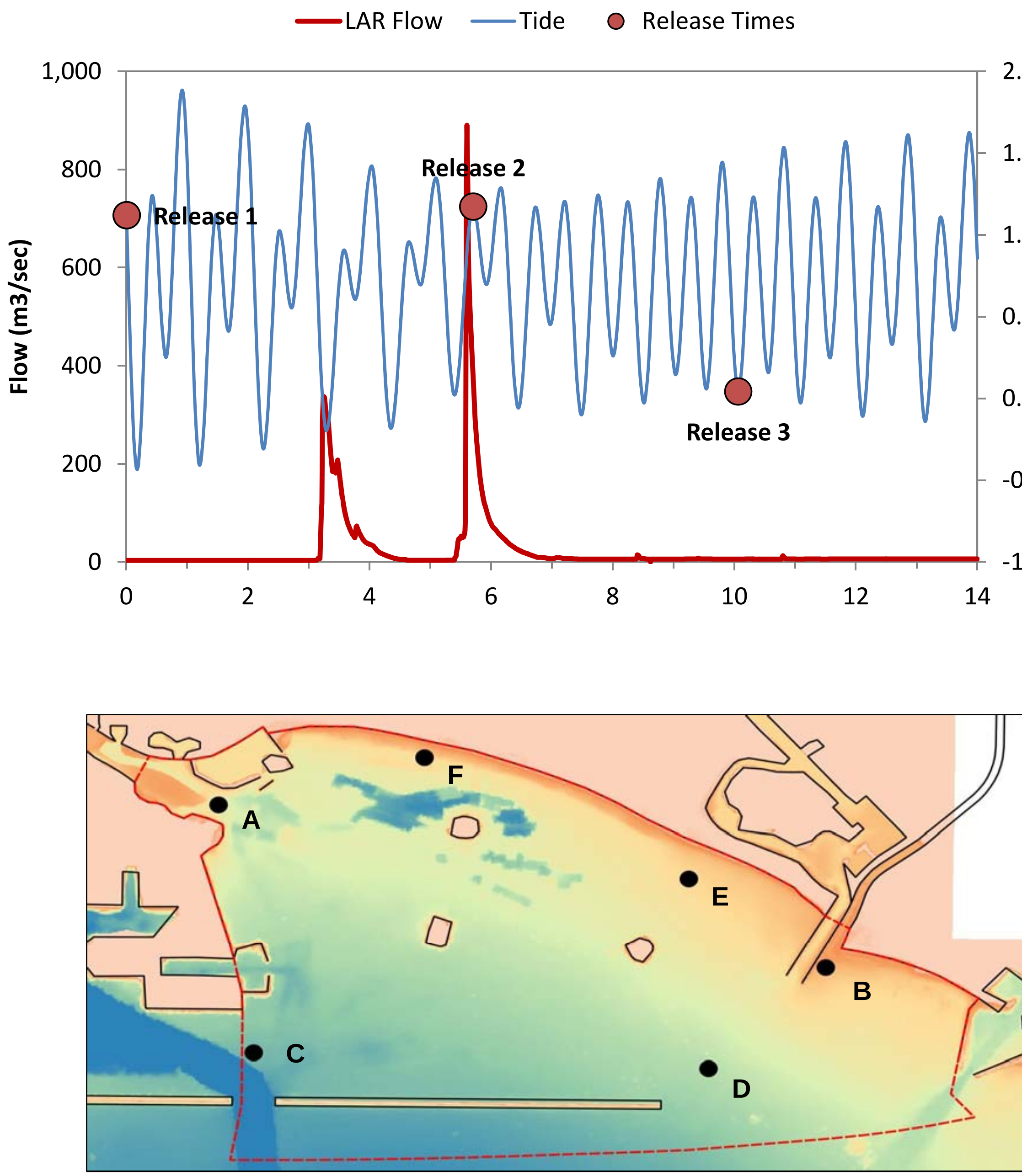


Figure 5.1 Numeric Tracer Tracking Release Times and Locations

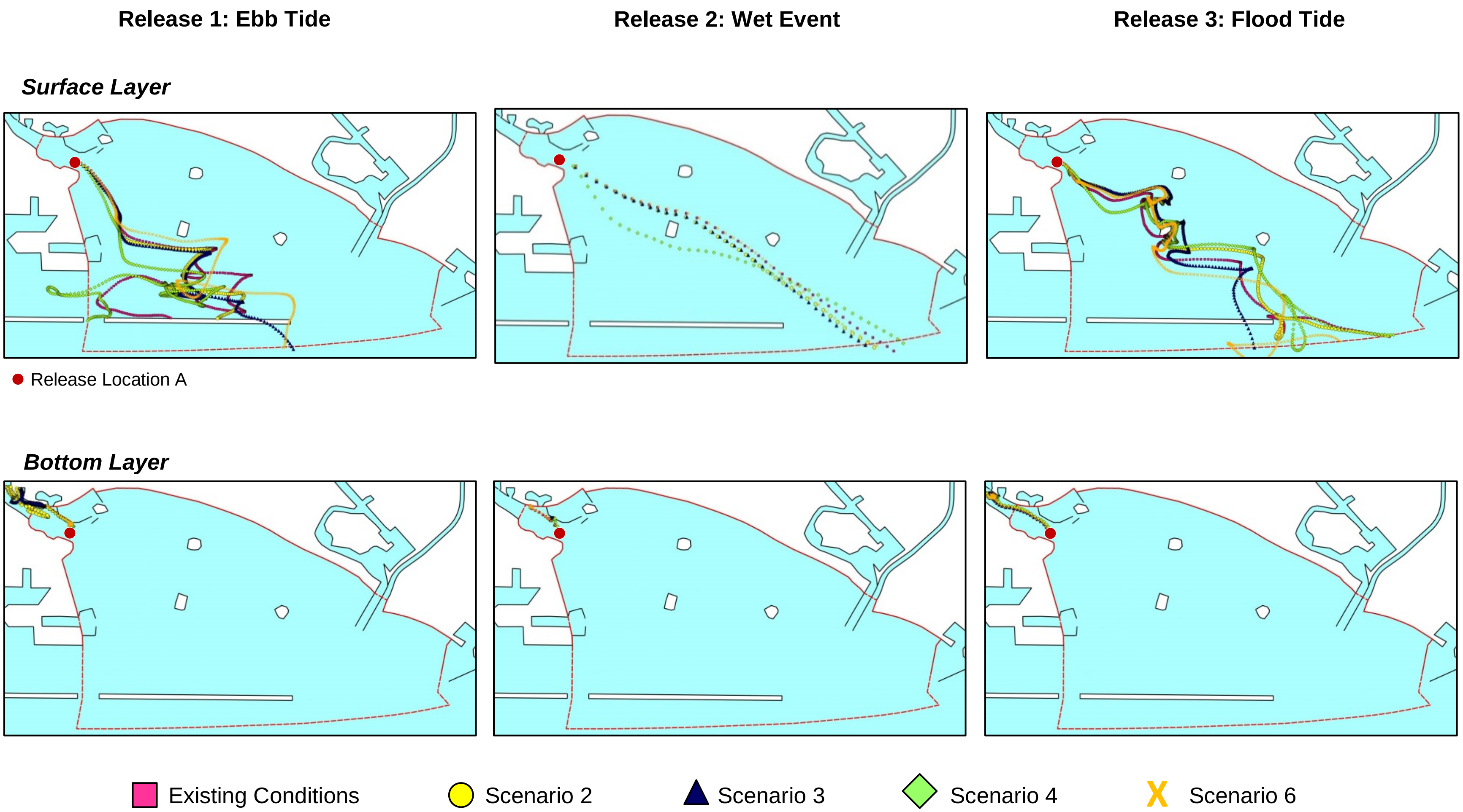


Figure 5.2 Particle Tracks for Location A

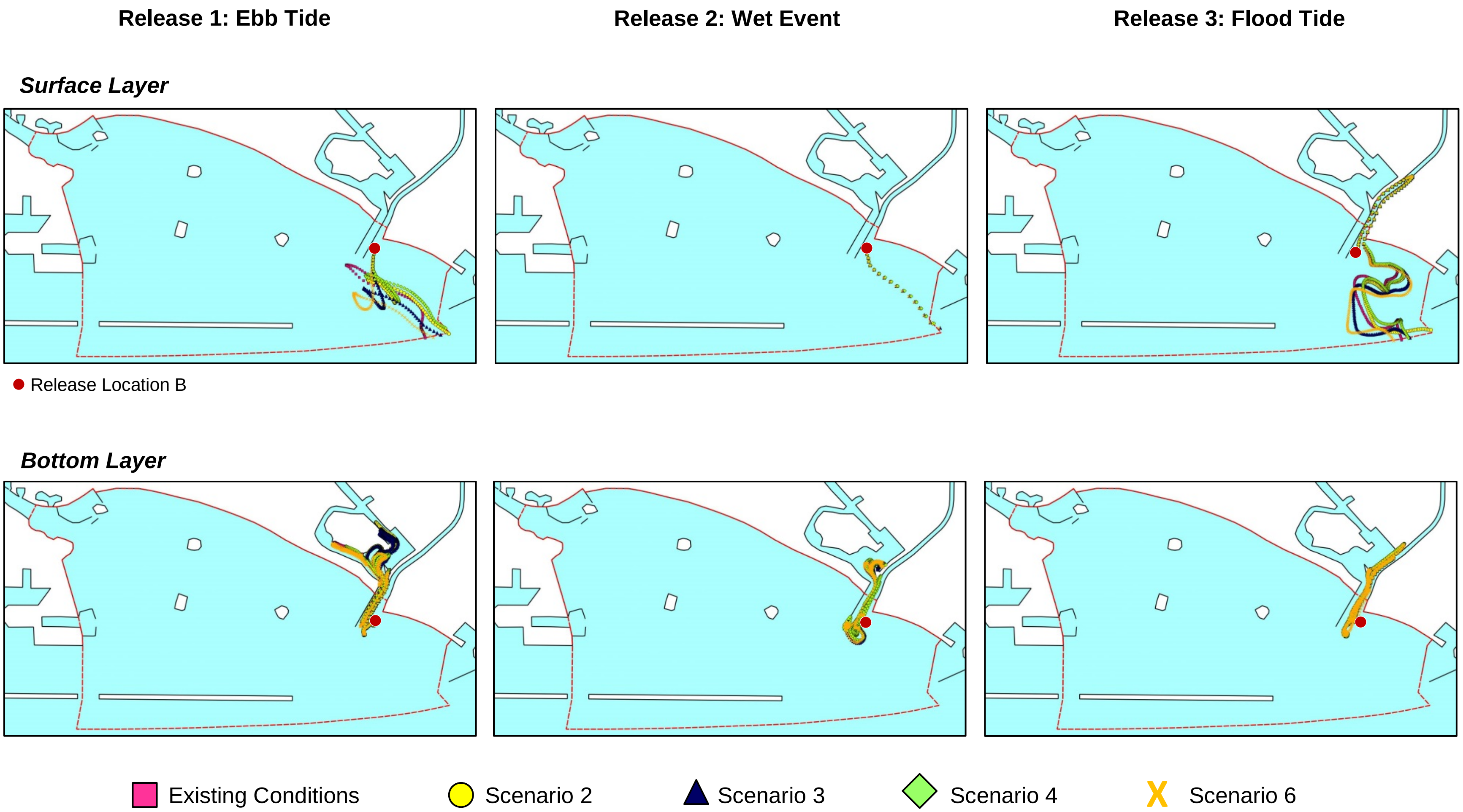


Figure 5.3 Particle Tracks for Location B

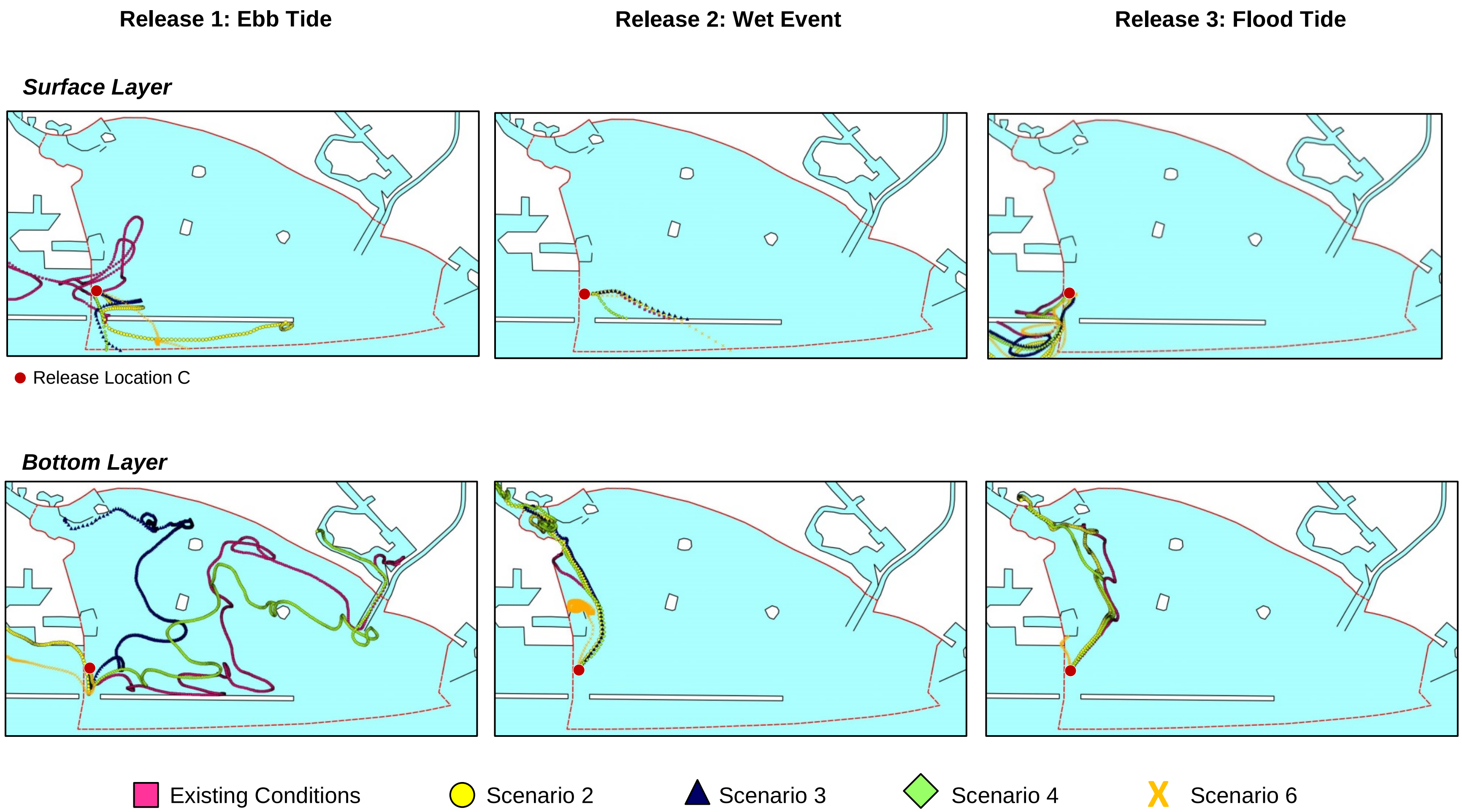


Figure 5.4 Particle Tracks for Location C

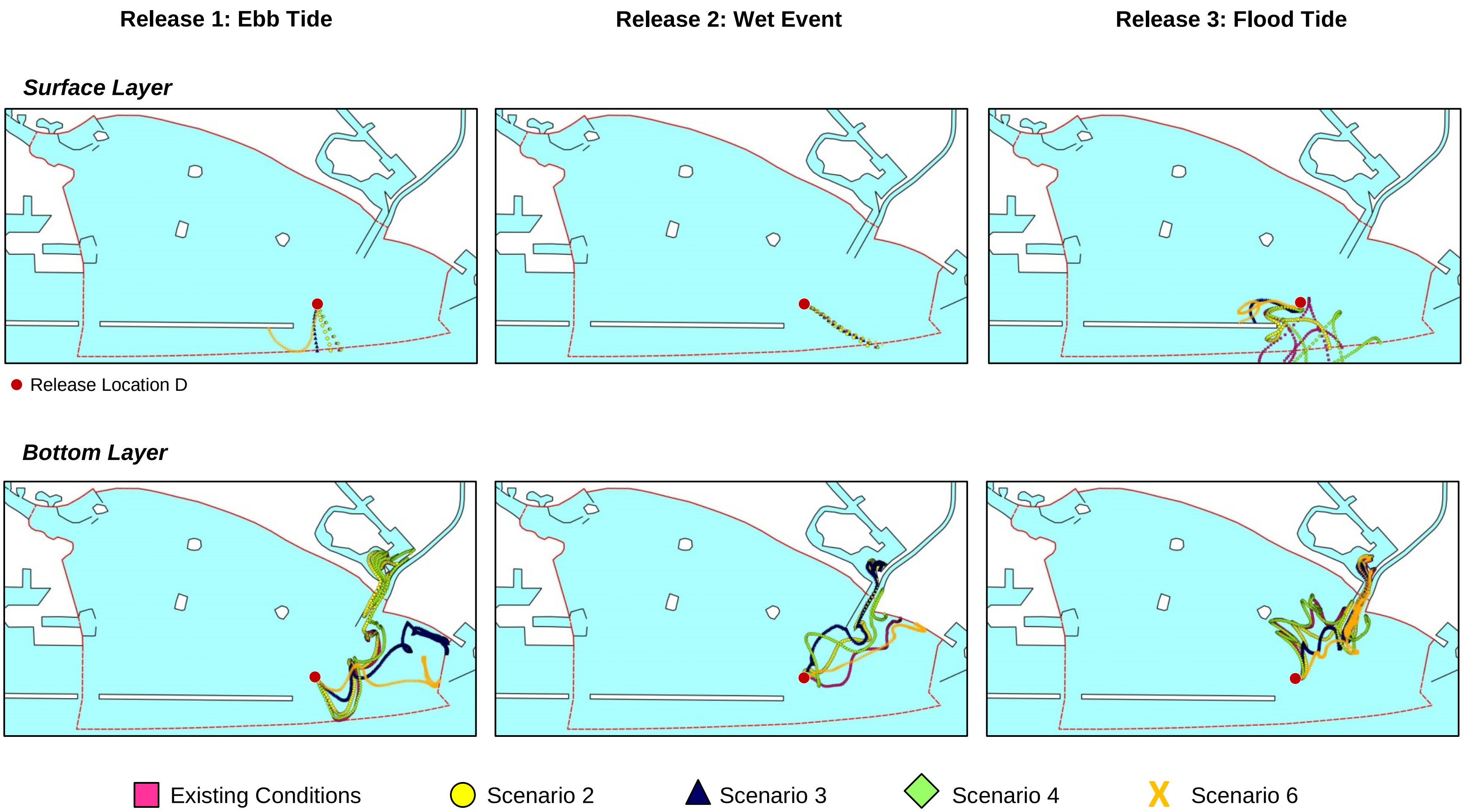


Figure 5.5 Particle Tracks for Location D

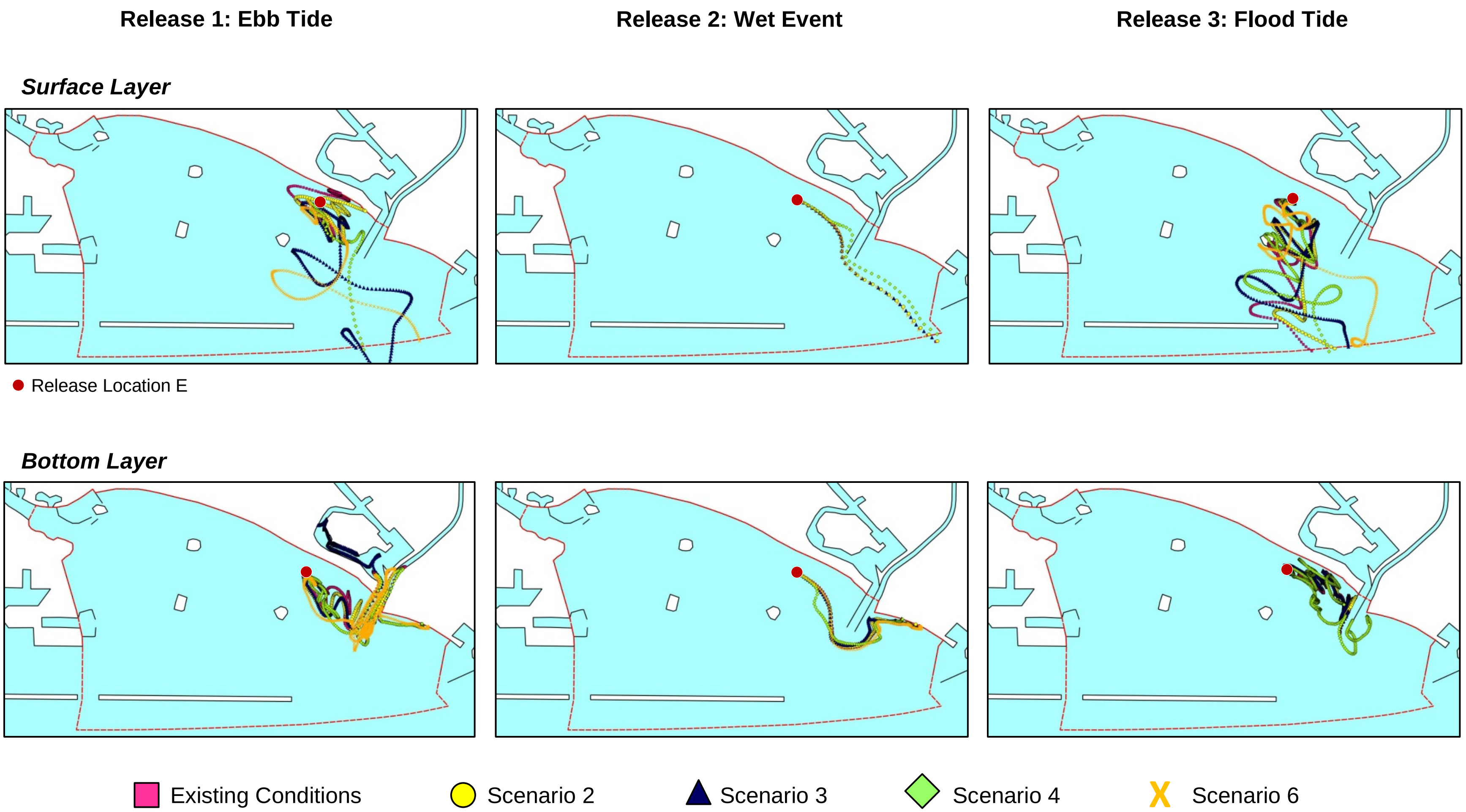


Figure 5.6 Particle Tracks for Location E

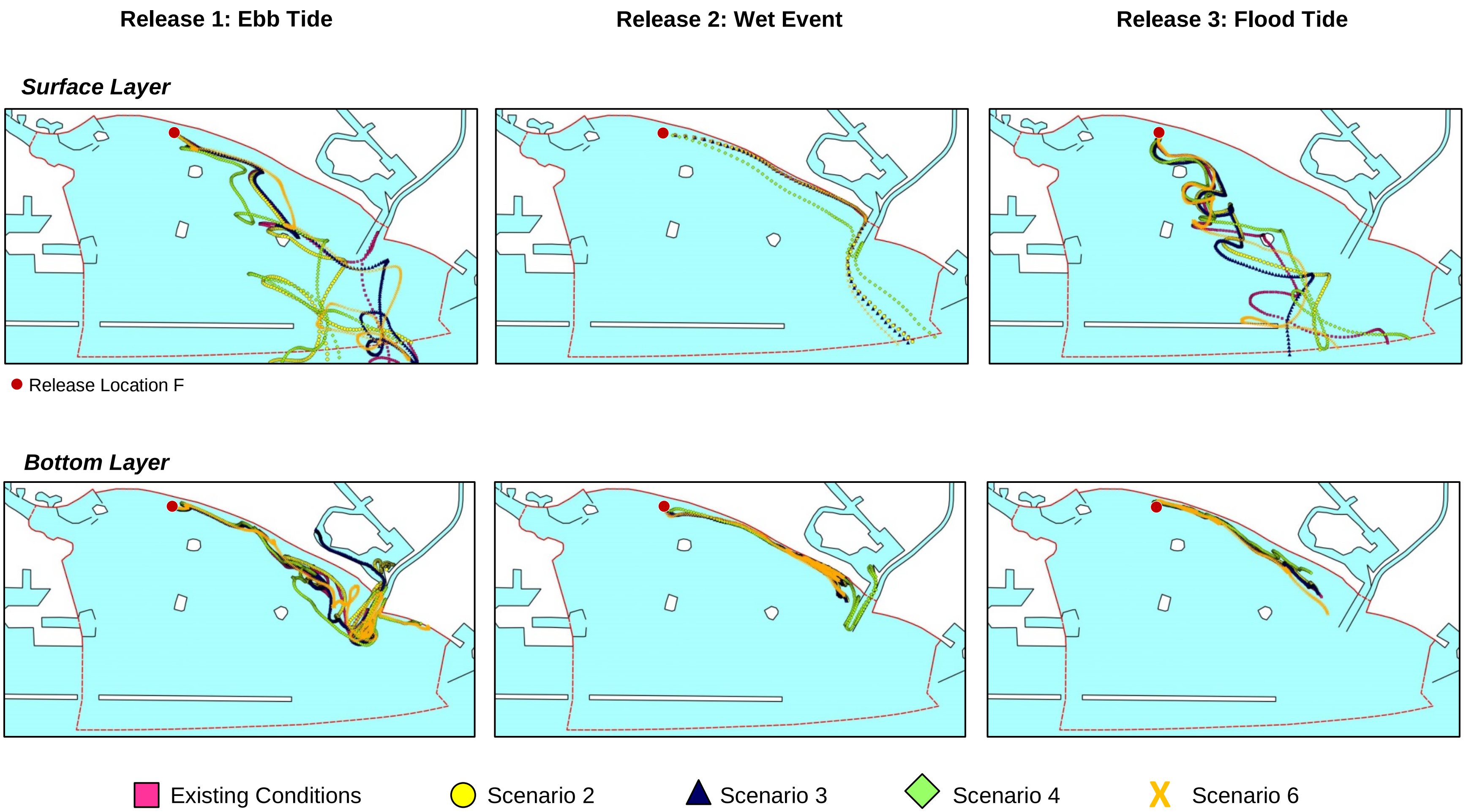


Figure 5.7 Particle Tracks for Location F

6. SUMMARY

A three-dimensional hydrodynamic model was developed for the East San Pedro Bay (ESPB) based on the Water Resources Action Plan (WRAP) Model. The hydrodynamic model accounts for the effect of waves on the hydrodynamics, and was used to provide information on currents and water quality conditions under Existing Conditions and six restoration alternatives. The proposed restoration alternatives include aquatic habitats throughout the bay and structural features aimed at improving circulation. Results of the hydrodynamic model were provided to USACE for use in the habitat evaluation model.

The hydrodynamic modeling results indicate that the proposed restoration alternatives generally have localized effects on the bay hydrodynamics. For the alternatives with wetlands, rocky/sand nearshore shoals, and emergent islands, changes in currents were observed only near the proposed features. Modifications to the breakwater (i.e., notching or lowering) significantly changed flows around the breakwater, but would not significantly change the overall circulation of ESPB. In general, the breakwater modification increased velocities around the breakwater and decreased velocities east of the breakwater. The training wall under Scenario 4 diverted flows from the LAR, resulting in hydrodynamic changes that occurred primarily during wet weather. Scenario 6, which lowered the entire section of breakwater, showed the greatest changes to hydrodynamic conditions during both tidal and wet weather conditions.

7. REFERENCES

Everest 2017. WRAP Model Development in Support of Final Dominguez Channel and Greater Los Angeles and Long Beach Harbor Waters Toxic Pollutants Total Maximum Daily Load Final Draft Report. Prepared for Port of Long Beach and Port Los Angeles. Prepared by Everest International Consultants, Inc. February 2017.

NOAA 2011. https://tidesandcurrents.noaa.gov/datums.html?id=9410660. Station ID 9410660, Los Angeles, CA. Provided by the National Oceanic and Atmospheric Administration (NOAA). Accepted October 7, 2011.

USACE 2016. East San Pedro Bay Ecosystem Restoration Feasibility Study Long Beach, CA Alternatives Milestone. U.S. Army Corps of Engineers, Los Angeles District and City of Long Beach. Presented September 9, 2016.

USACE 2017a. GIS raster files of depth and elevation for Existing Conditions and Scenarios 1 - 6 (ESPB_Bathy.zip). Engineering Division, Coastal Engineering Section Los Angeles District, U.S. Army Corps of Engineers. Provided January 26, 2017.

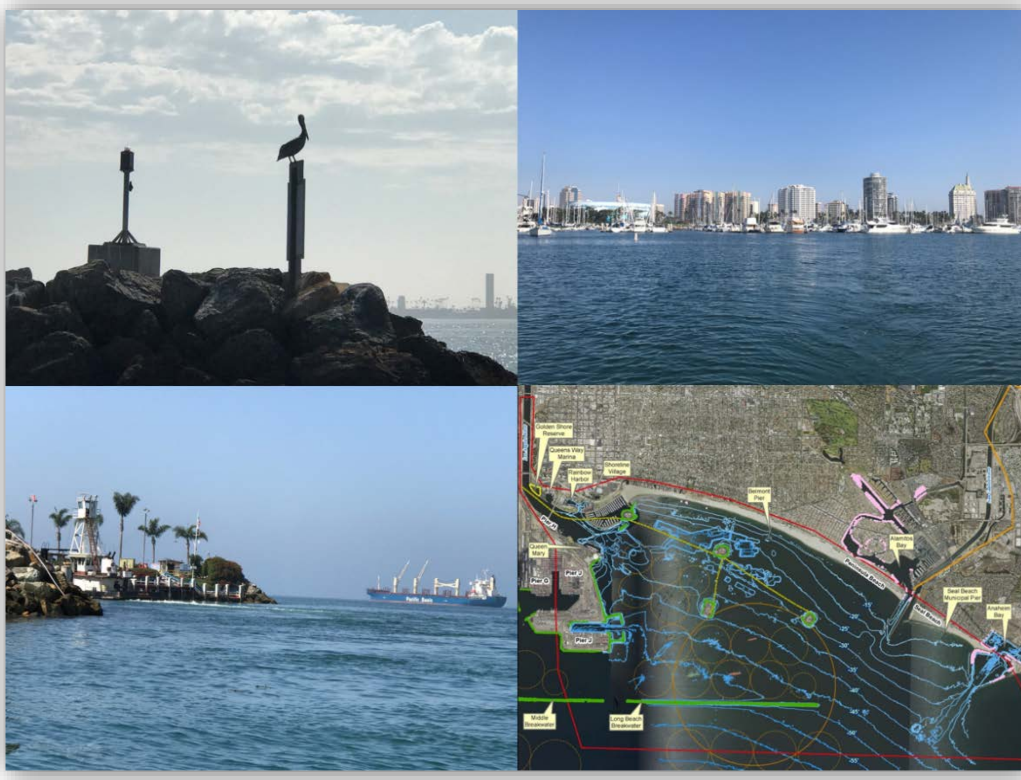
USACE 2017b. CMS-Wave Model outputs for Existing Conditions and Scenarios 1 -6. Engineering Division, Coastal Engineering Section Los Angeles District, U.S. Army Corps of Engineers. Provided March 22, 2017.

FINAL INTEGRATED FEASIBILITY REPORT AND ENVIRONMENTAL IMPACT STATEMENT / ENVIRONMENTAL IMPACT REPORT (EIS/EIR)

APPENDIX A-2: WAVE CONDITIONS

EAST SAN PEDRO BAY
ECOSYSTEM RESTORATION STUDY
Long Beach, California

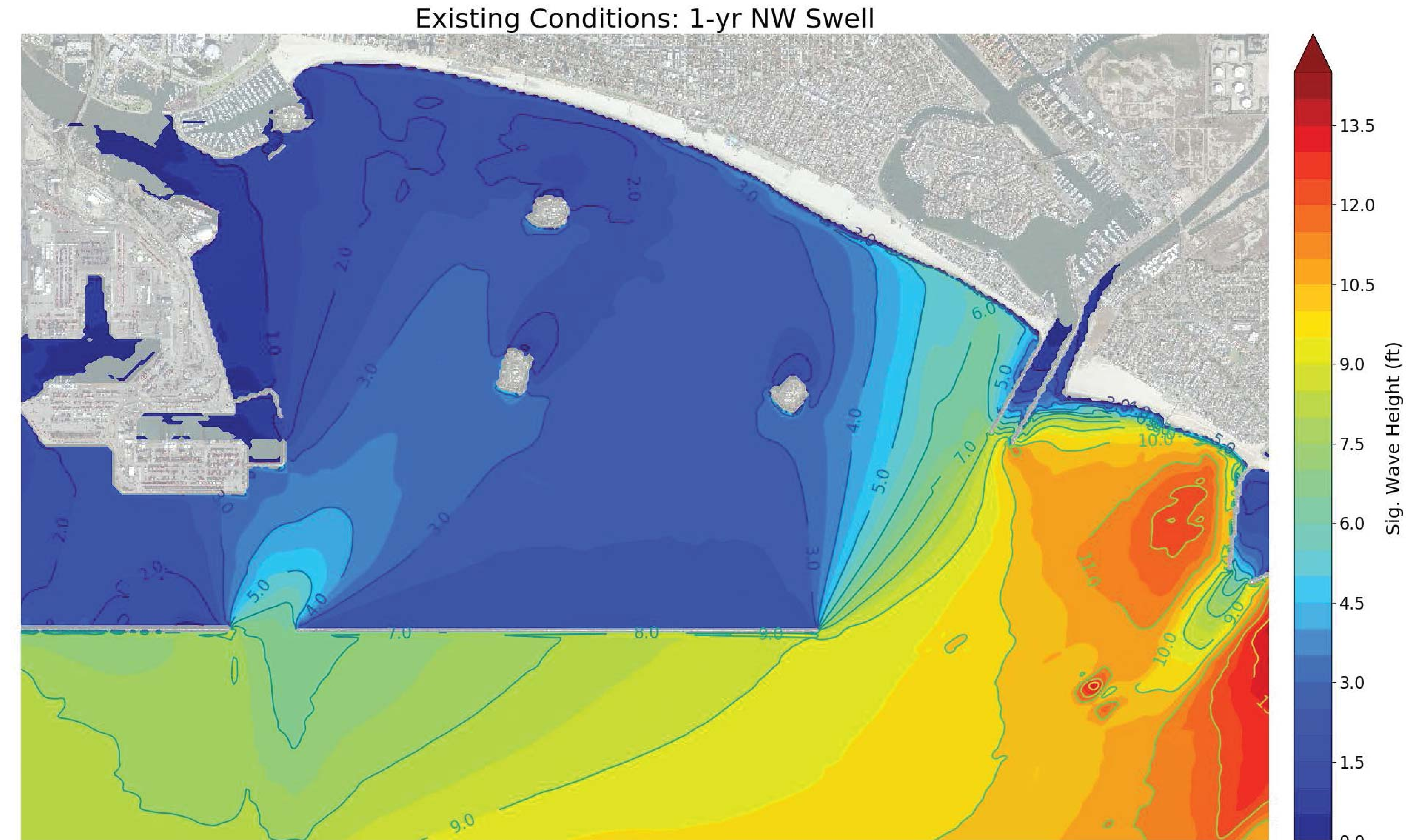
January 2022



US Army Corps of Engineers®

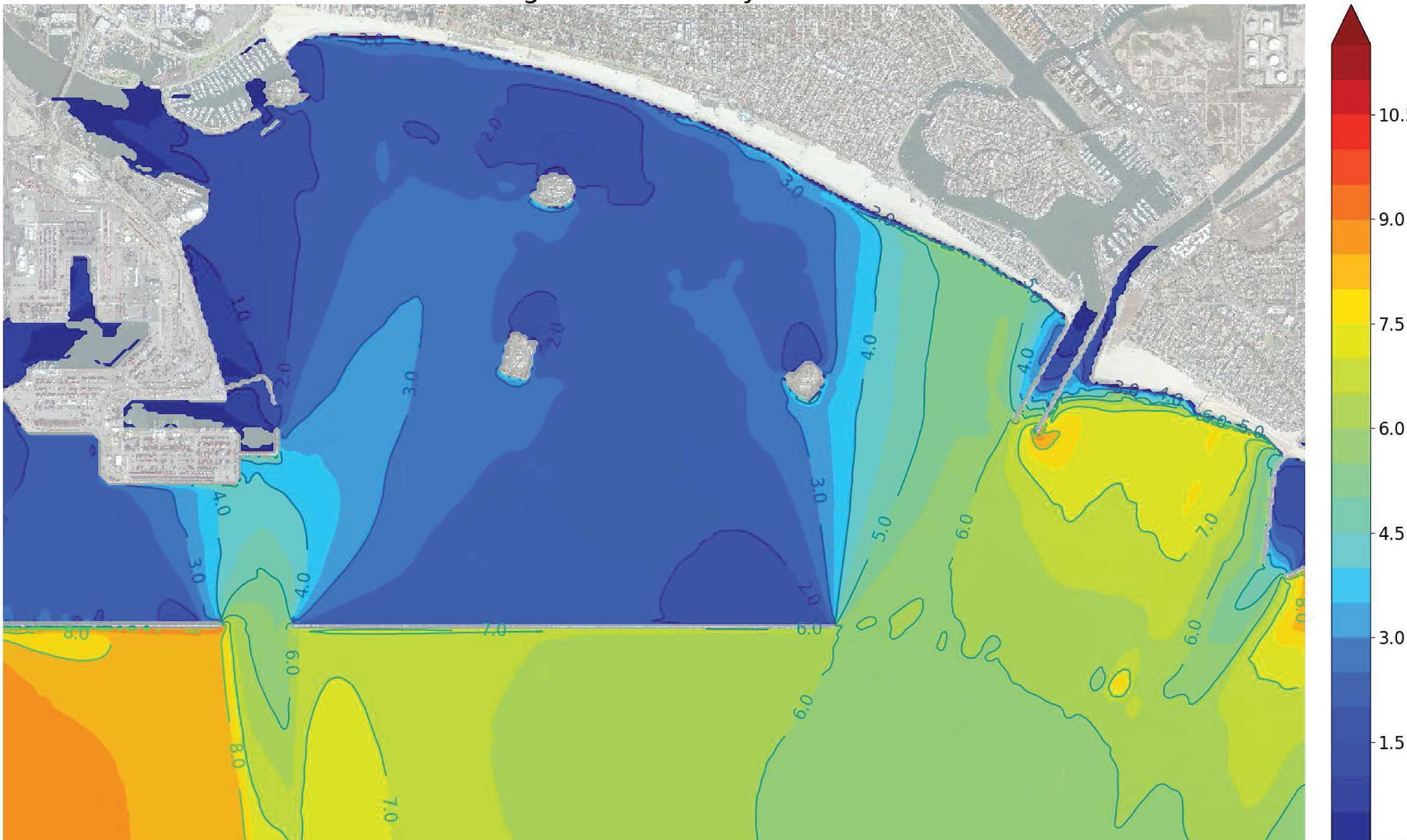
Existing Conditions: Typical Winter

Existing Conditions: Typical Summer

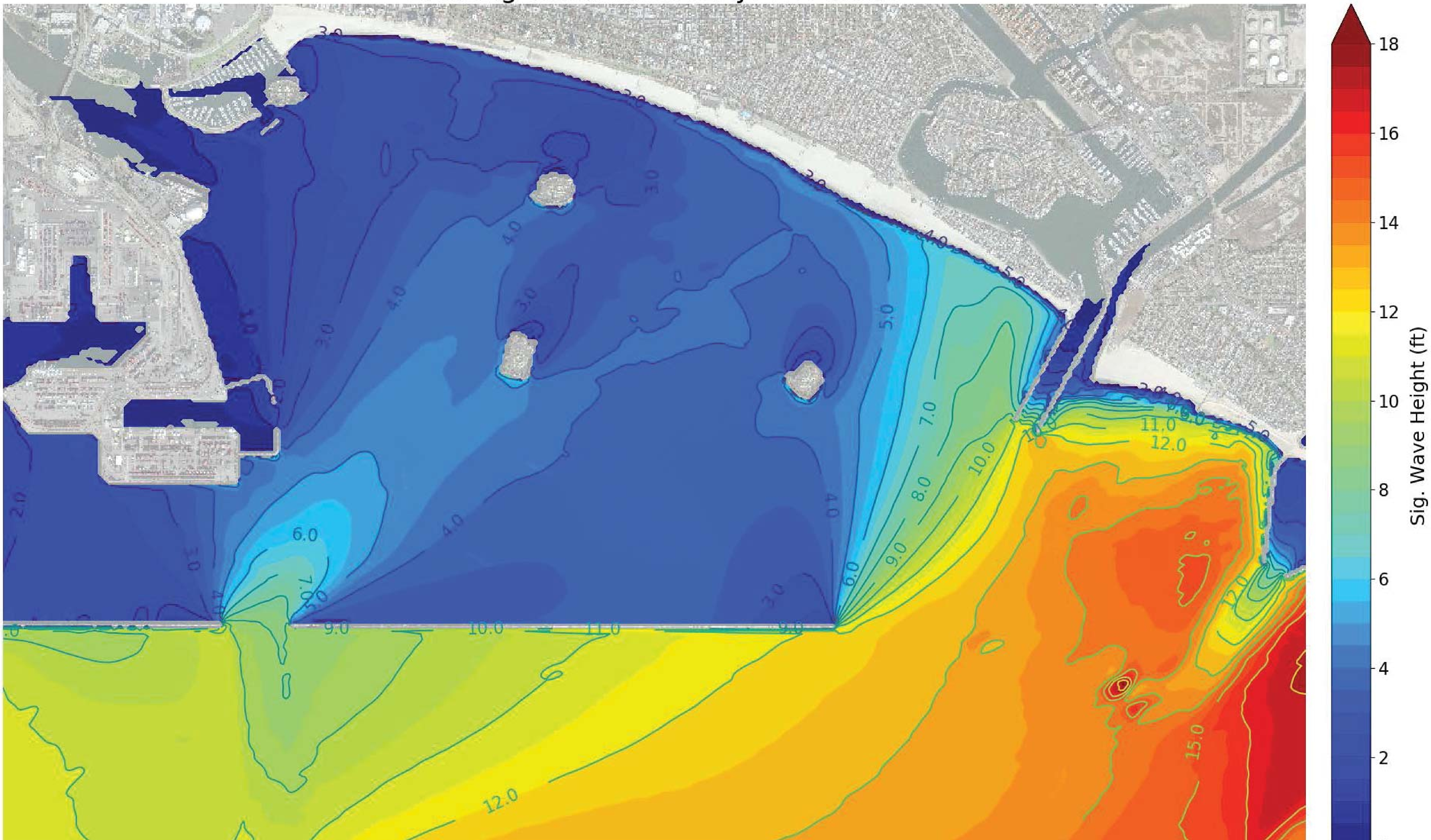


Existing Conditions: 1-yr NW Swell
Sig. Wave Height (ft)
0.0
1.5
3.0
4.5
6.0
7.5
9.0
10.5
12.0
13.5

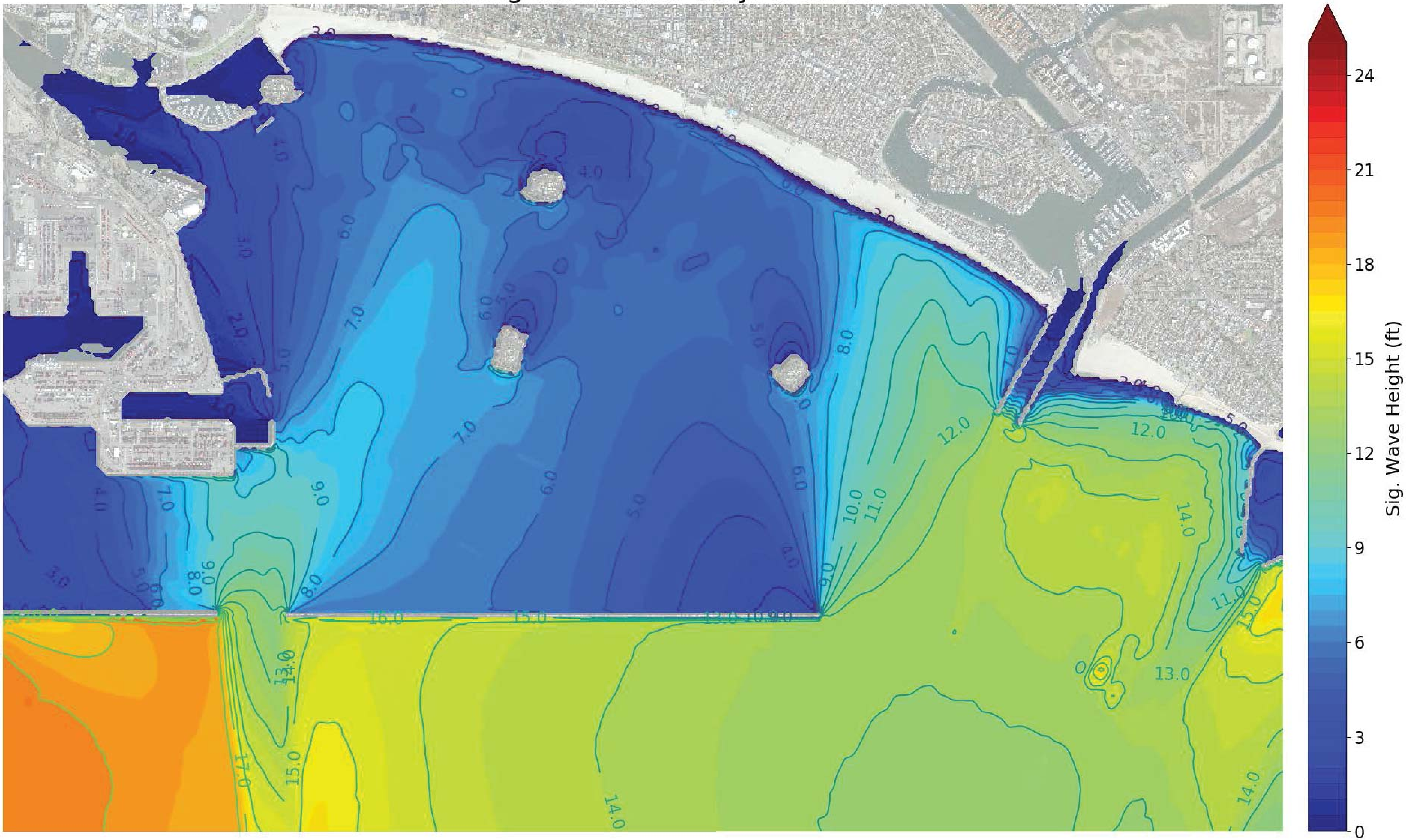
Existing Conditions: 1-yr S Swell

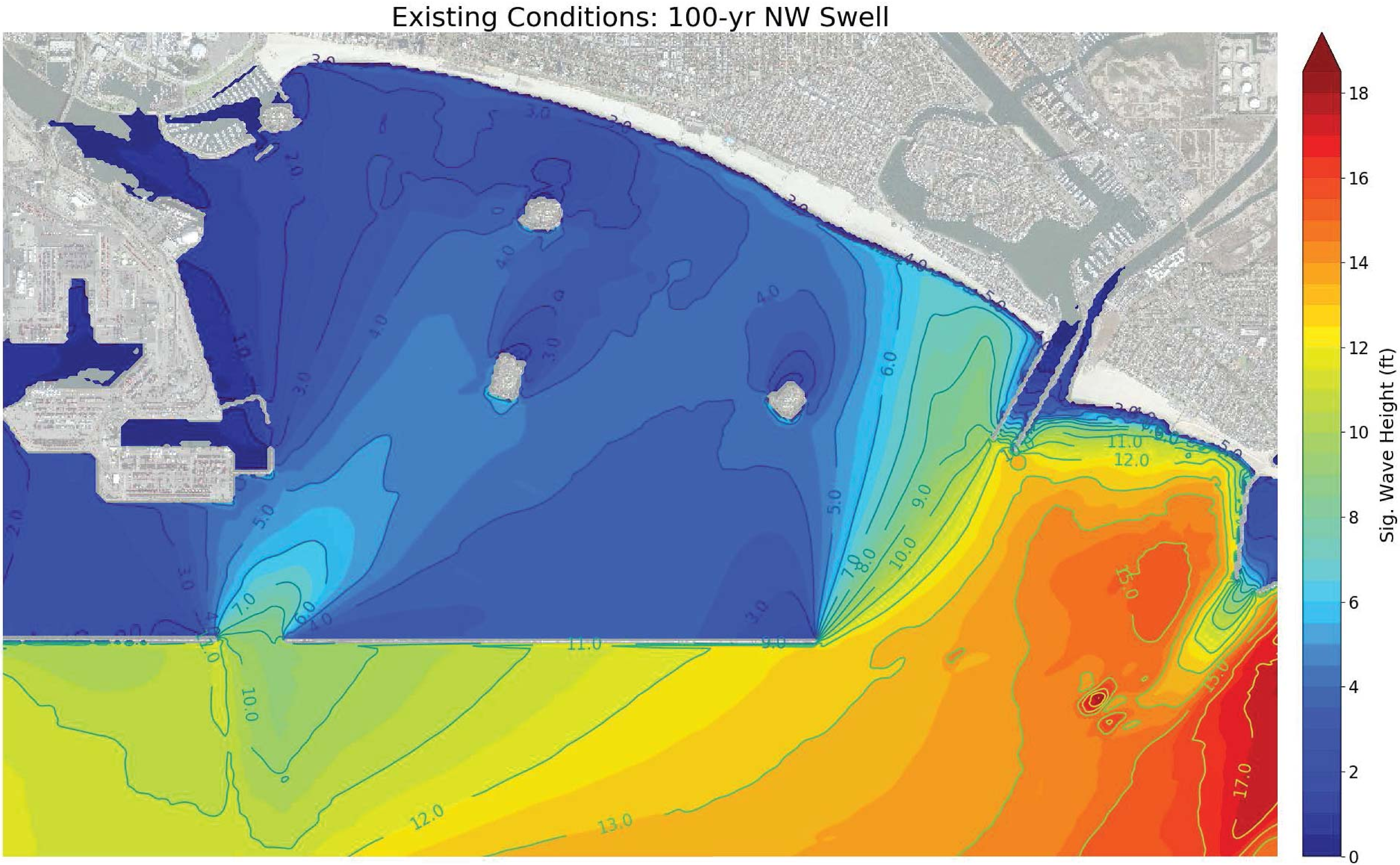


Existing Conditions: 50-yr NW Swell

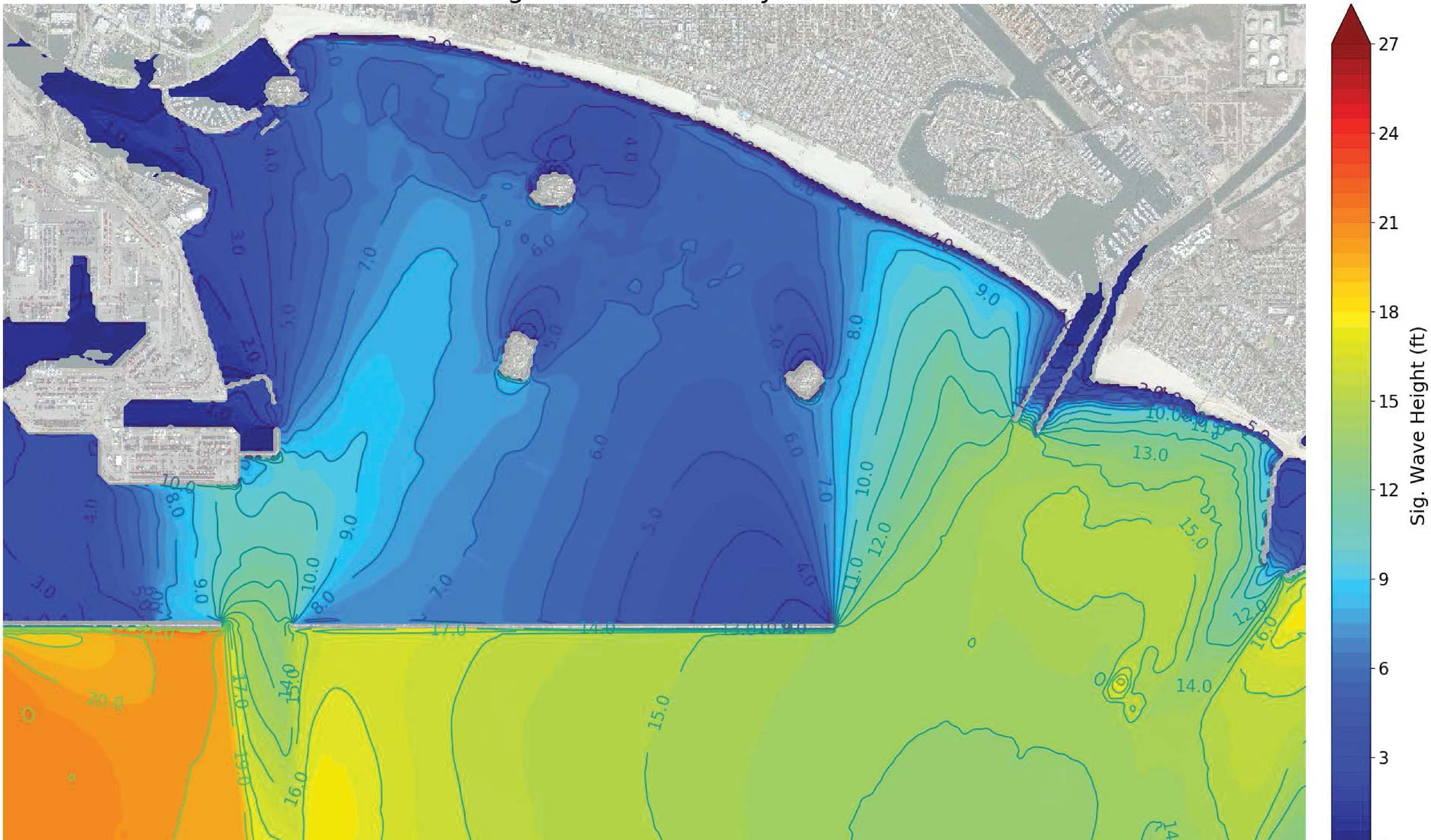


Existing Conditions: 50-yr S Swell

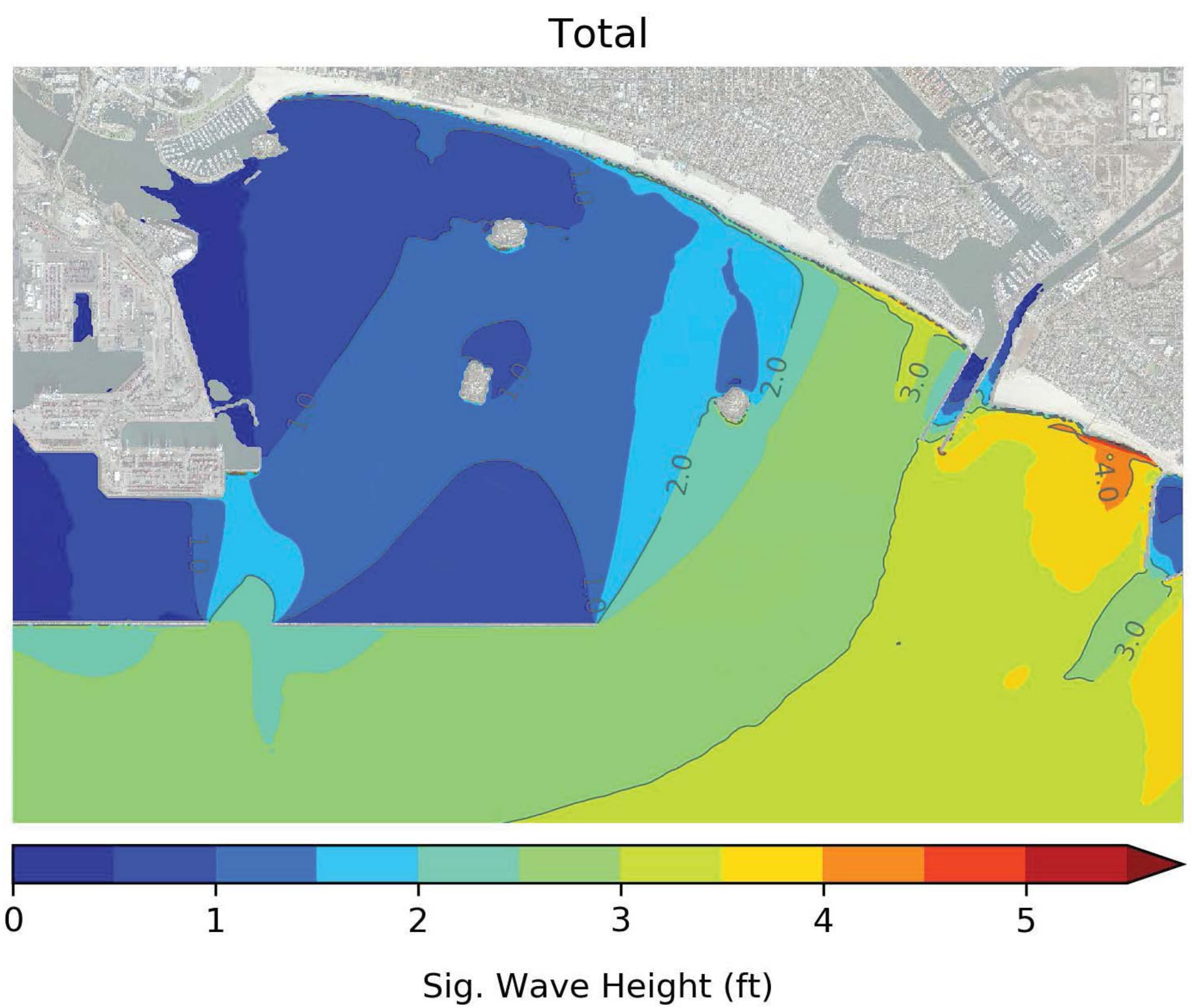


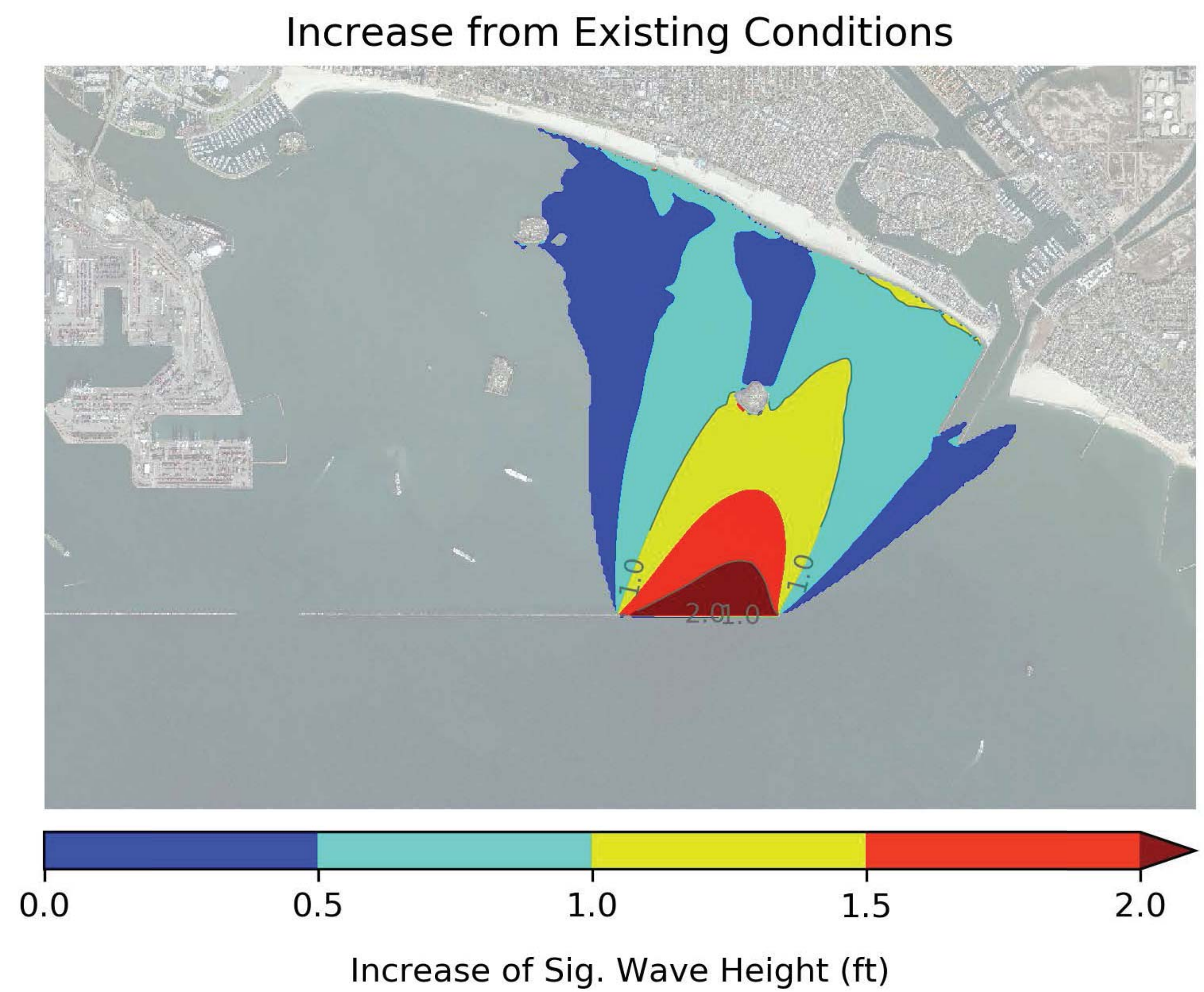

Existing Conditions: 100-yr NW Swell
Sig. Wave Height (ft)
0
2
4
6
8
10
12
14
16
18
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
15.0
17.0

Existing Conditions: 100-yr S Swell

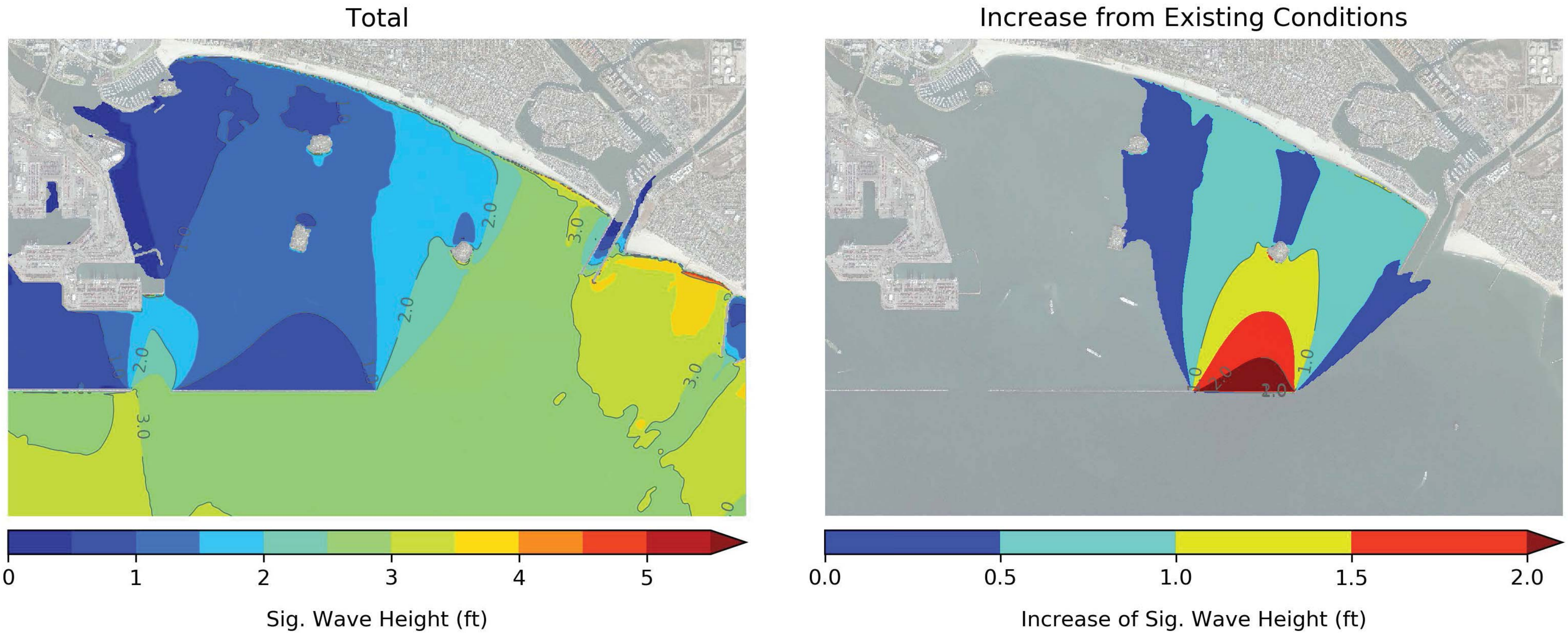


Eastern 1/3 Removal: Typical Winter

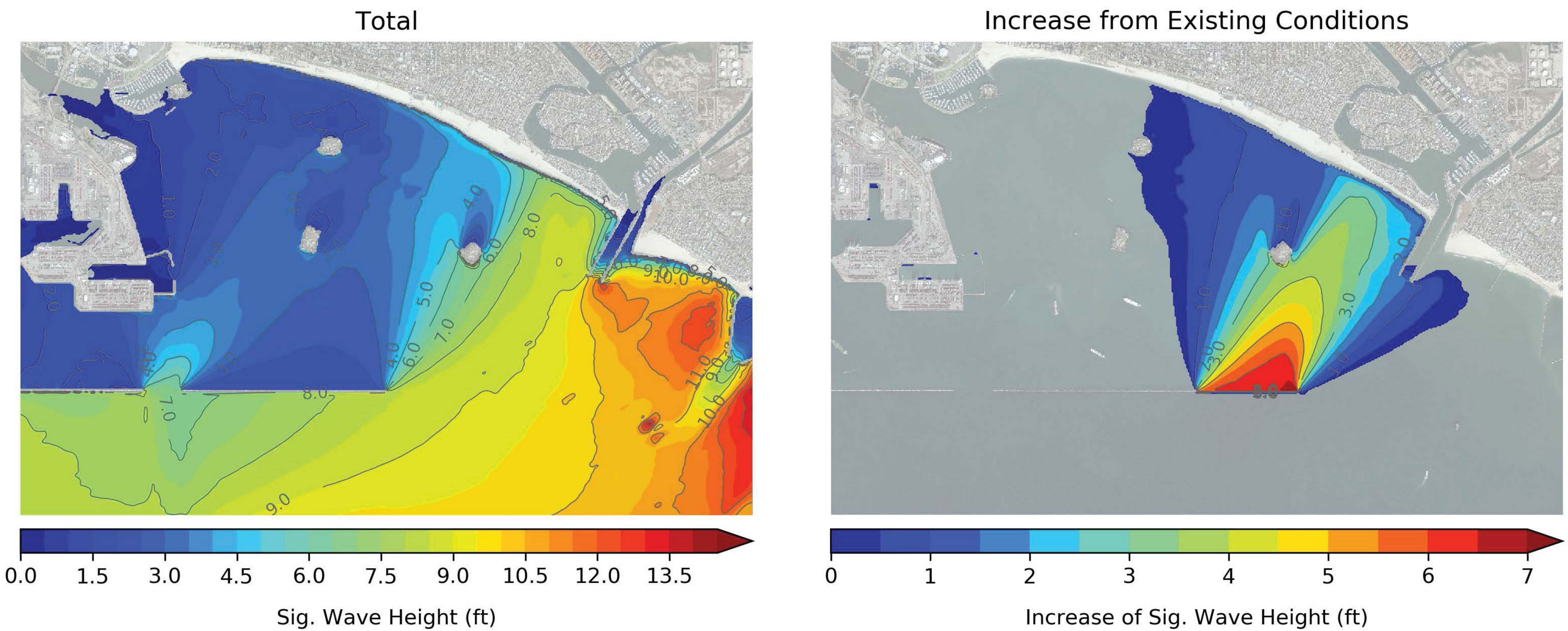




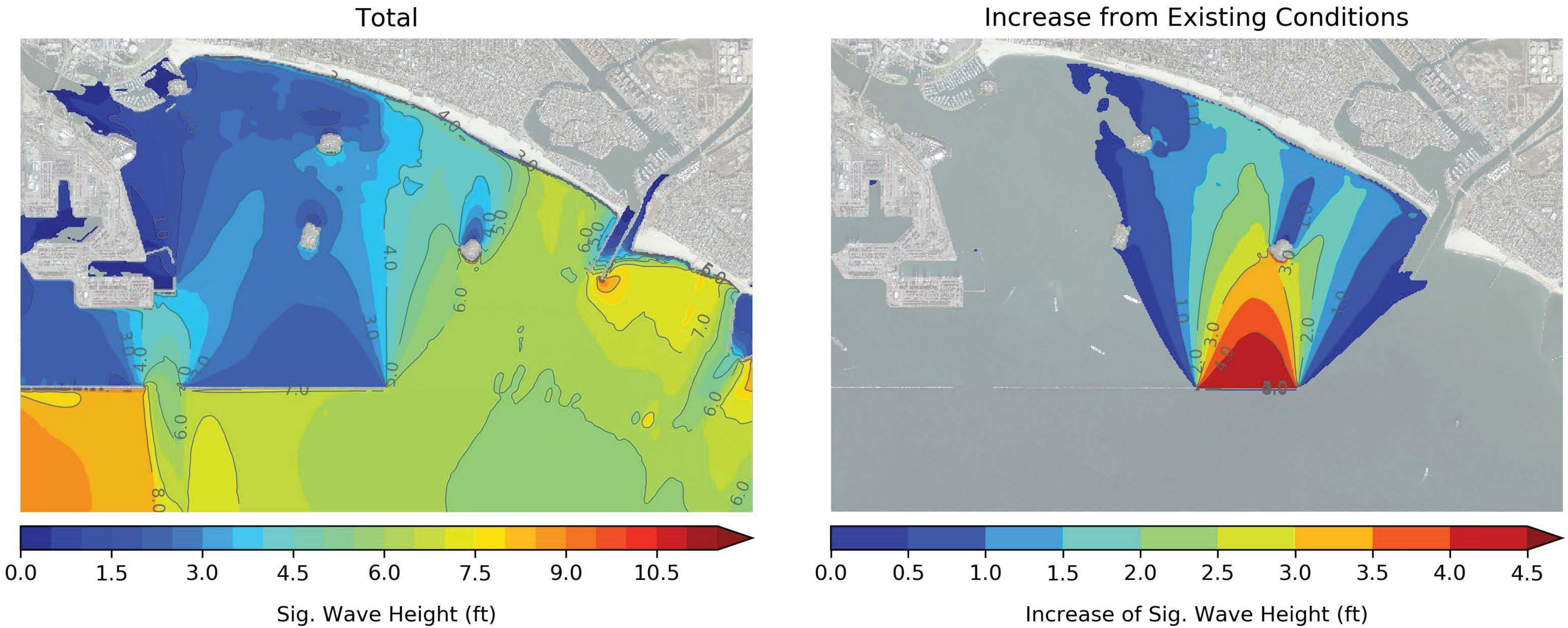
Eastern 1/3 Removal: Typical Summer



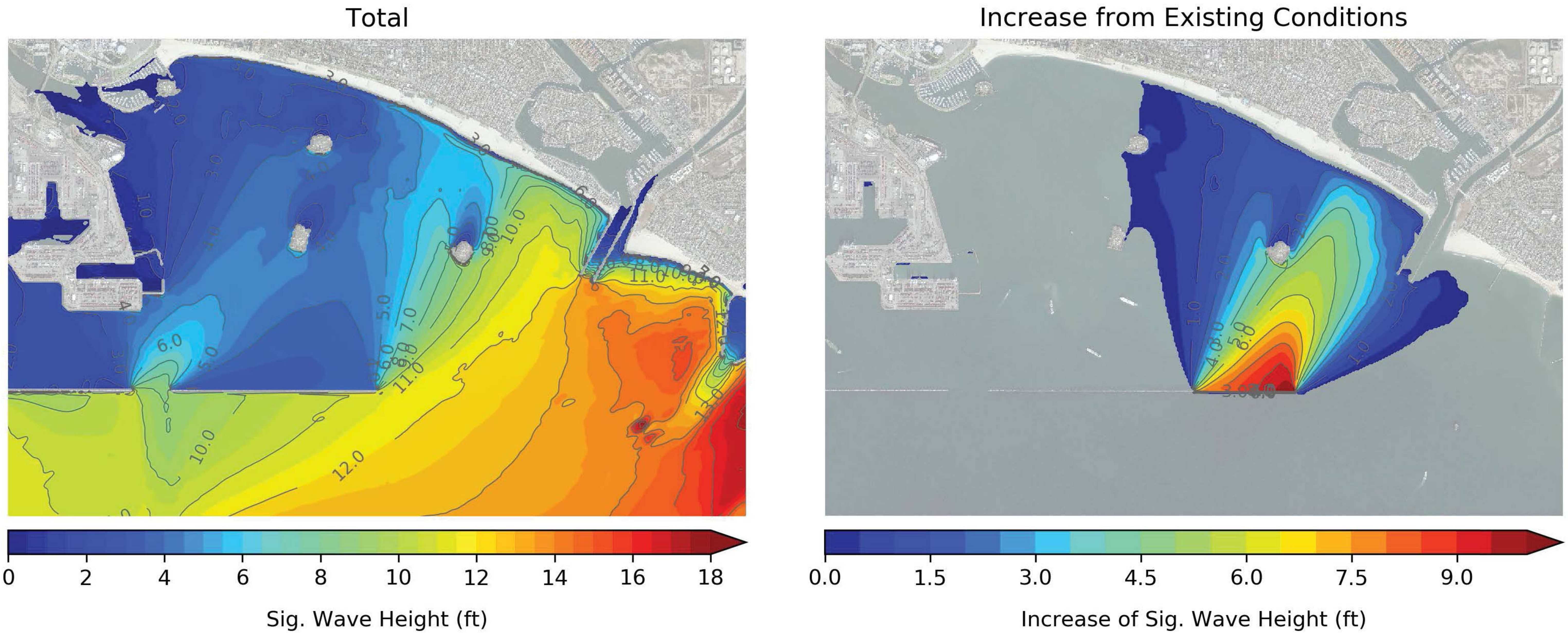
Eastern 1/3 Removal: 1-yr NW Swell



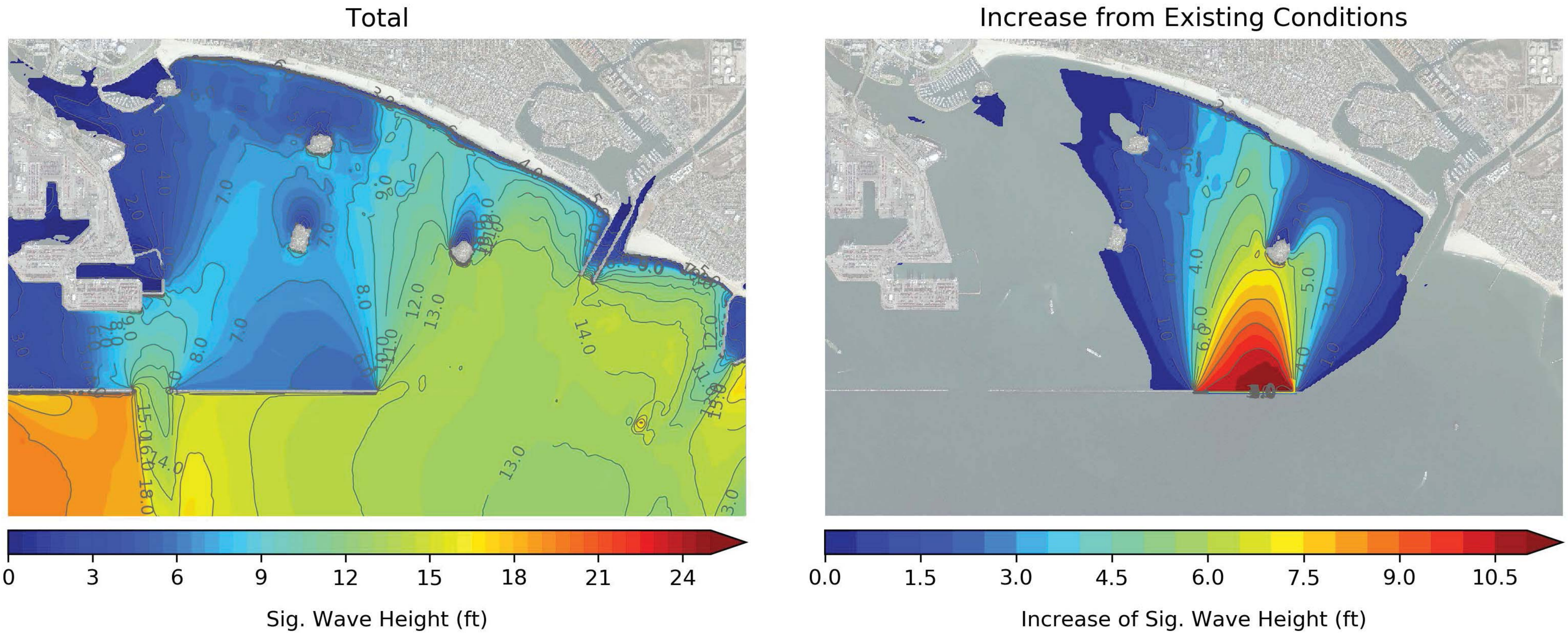
Eastern 1/3 Removal: 1-yr S Swell



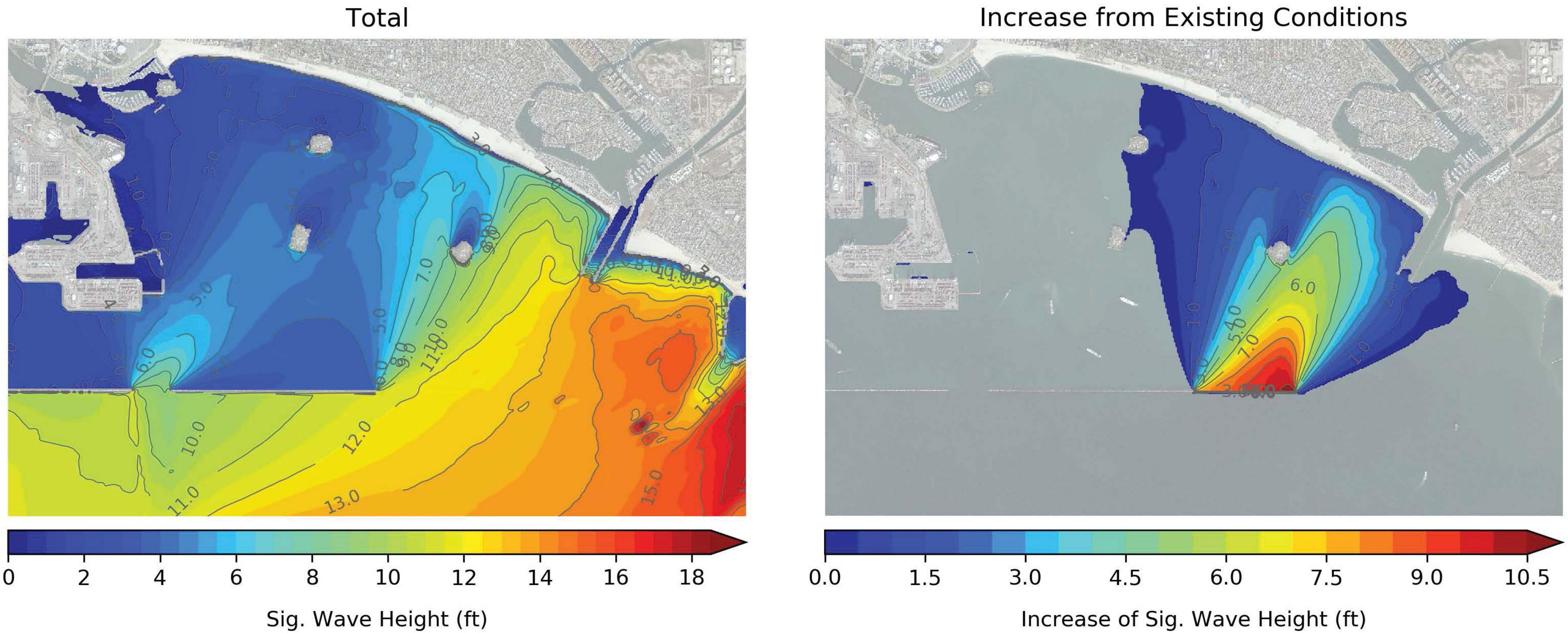
Eastern 1/3 Removal: 50-yr NW Swell

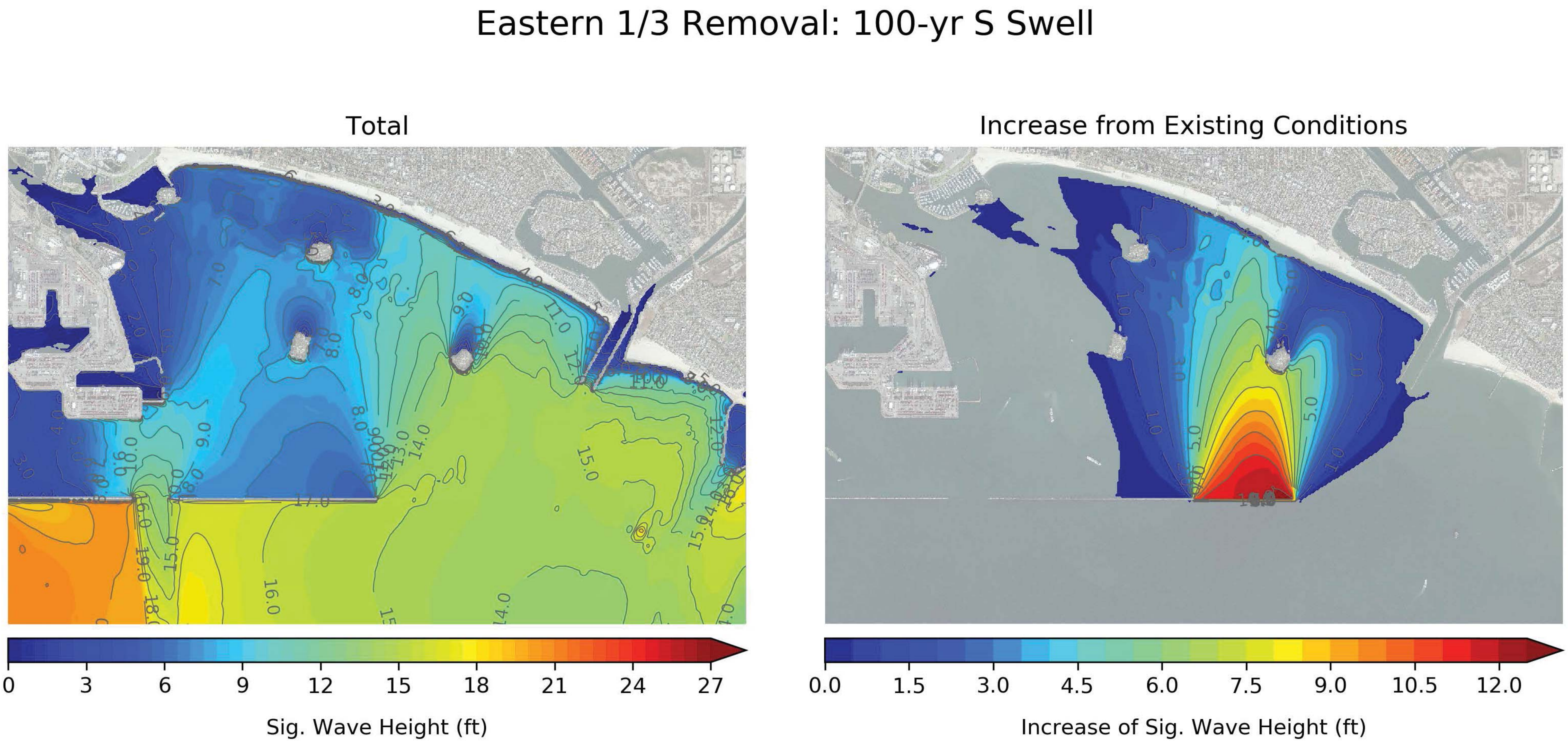


Eastern 1/3 Removal: 50-yr S Swell

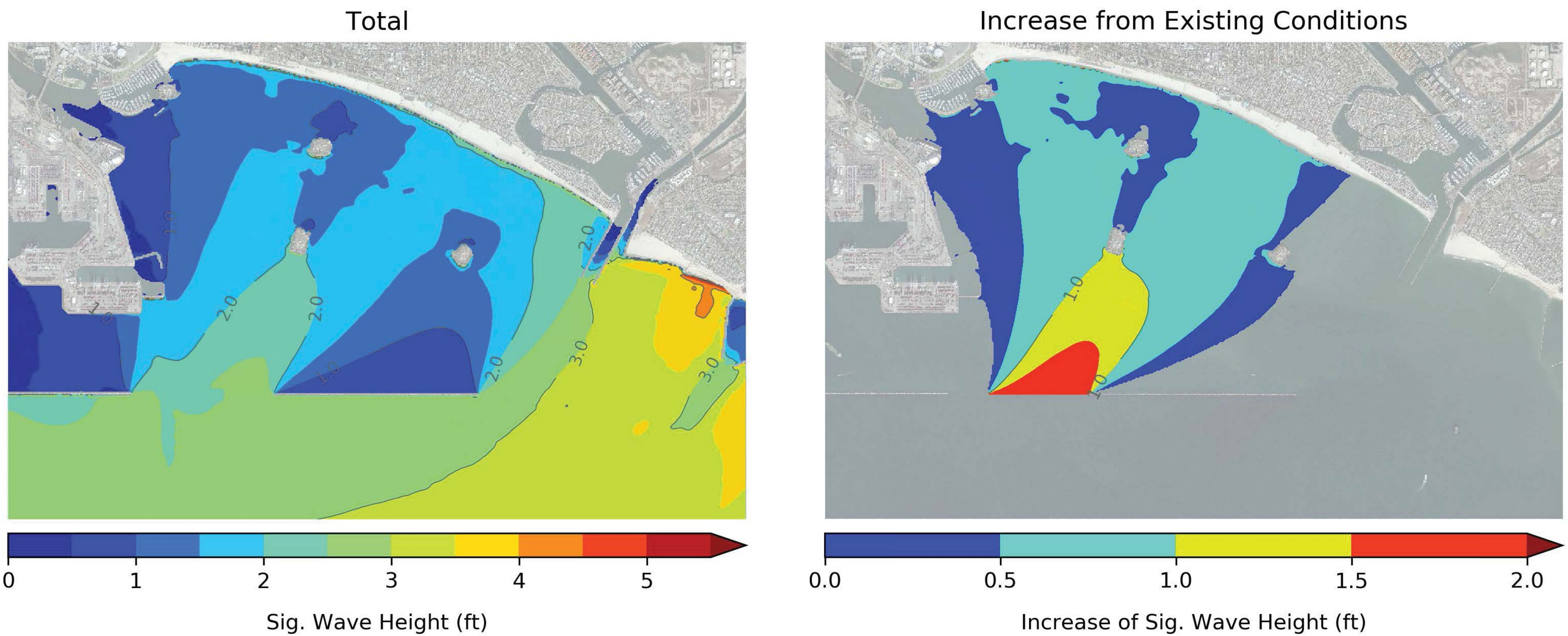


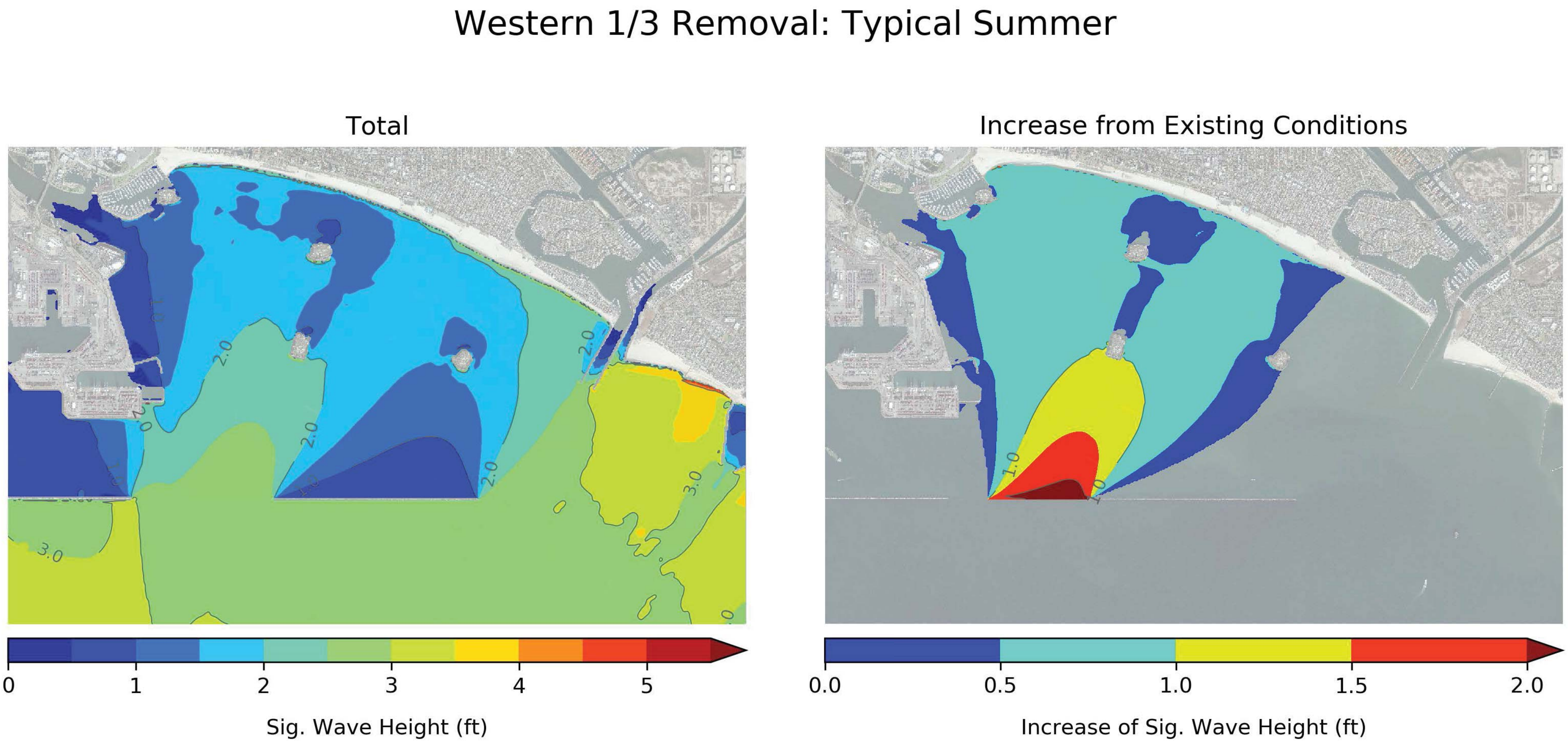
Eastern 1/3 Removal: 100-yr NW Swell



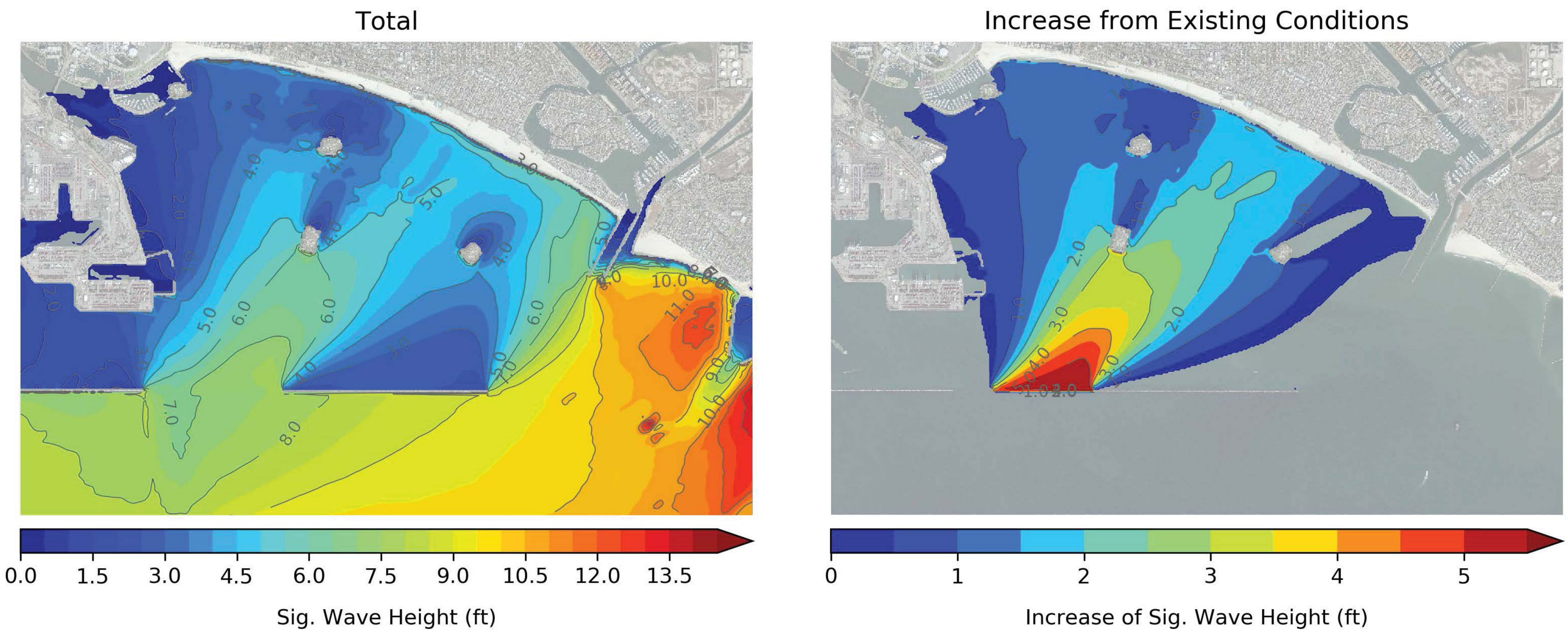

Eastern 1/3 Removal: 100-yr S Swell
Total
Increase from Existing Conditions
0
3
6
9
12
15
18
21
24
27
Sig. Wave Height (ft)
0.0
1.5
3.0
4.5
6.0
7.5
9.0
10.5
12.0
Increase of Sig. Wave Height (ft)

Western 1/3 Removal: Typical Winter

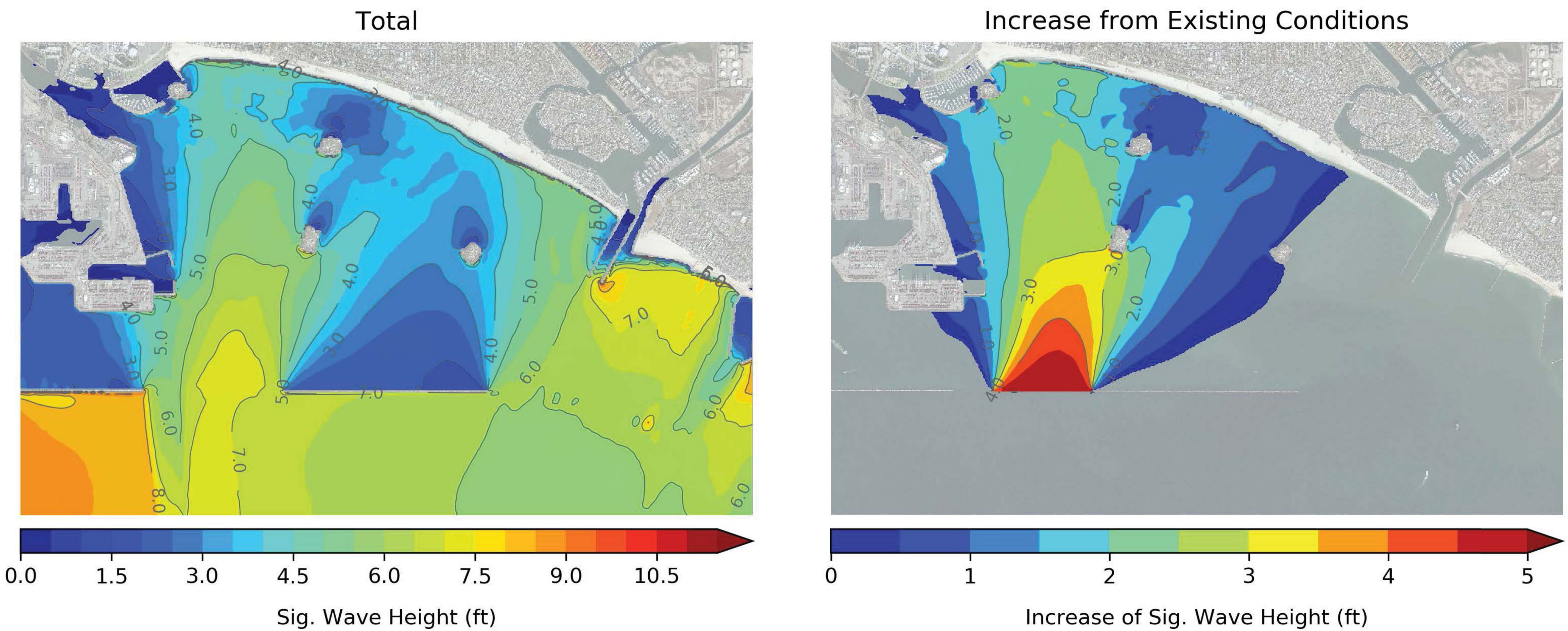



Western 1/3 Removal: Typical Summer
Total
Increase from Existing Conditions
0
1
2
3
4
5
Sig. Wave Height (ft)
0.0
0.5
1.0
1.5
2.0
Increase of Sig. Wave Height (ft)

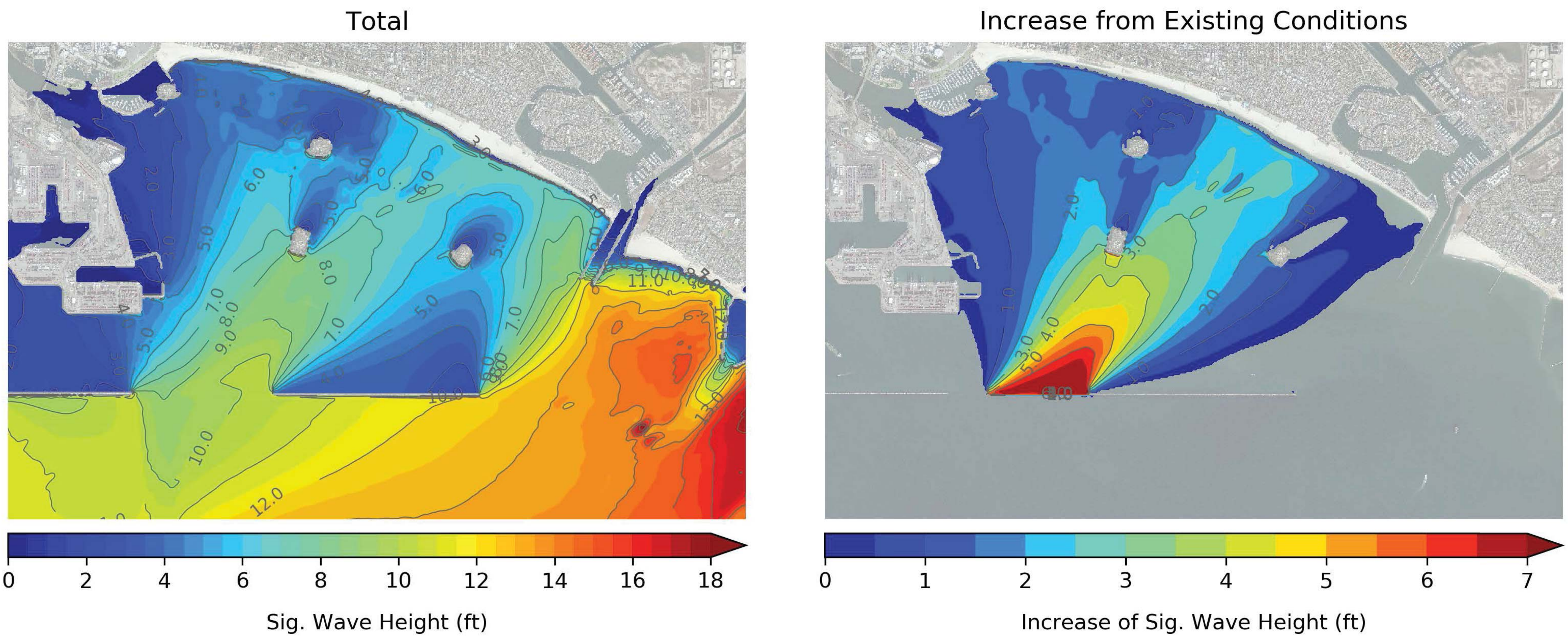
Western 1/3 Removal: 1-yr NW Swell



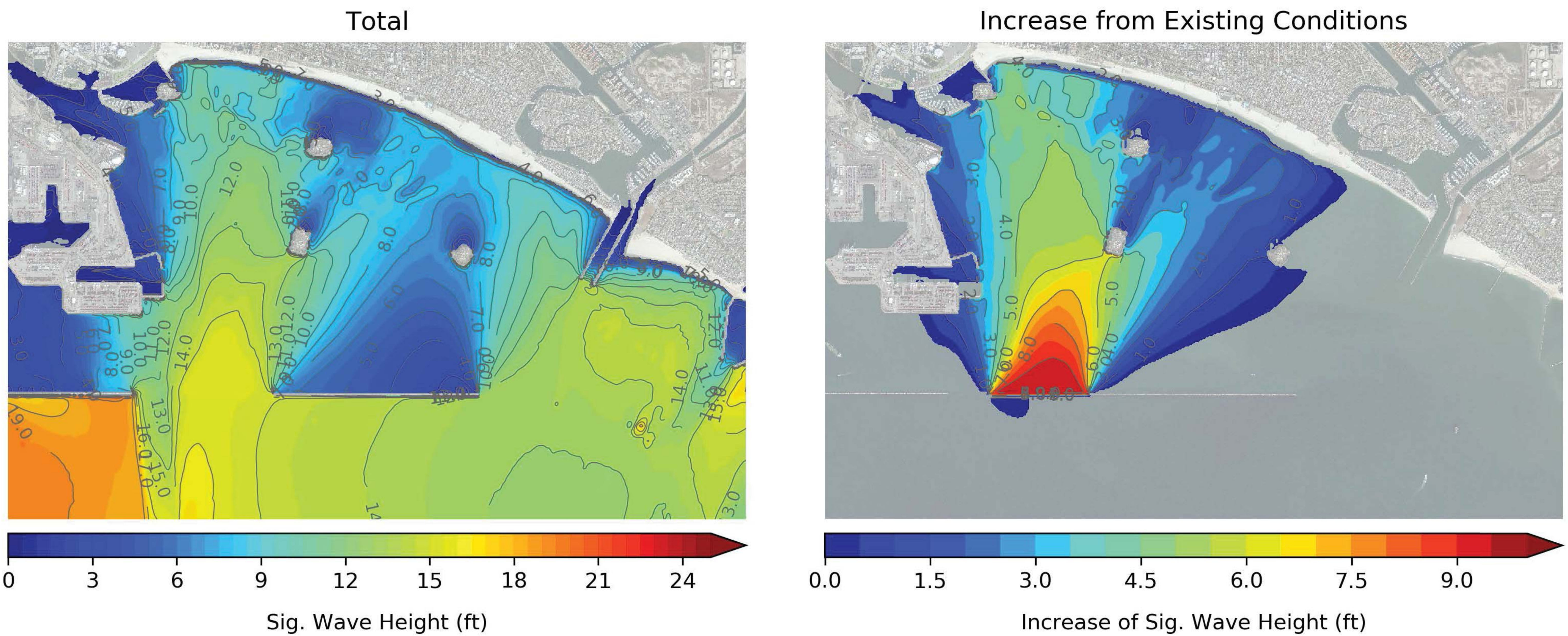
Western 1/3 Removal: 1-yr S Swell



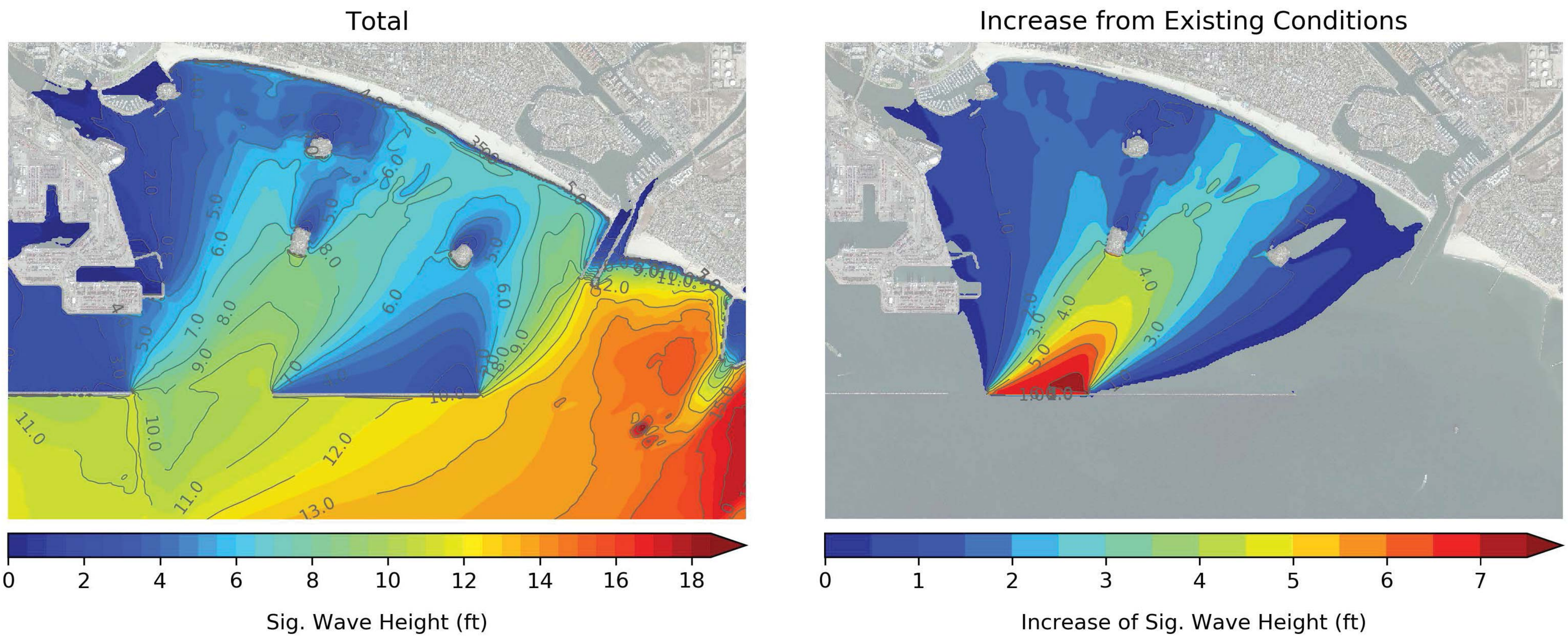
Western 1/3 Removal: 50-yr NW Swell

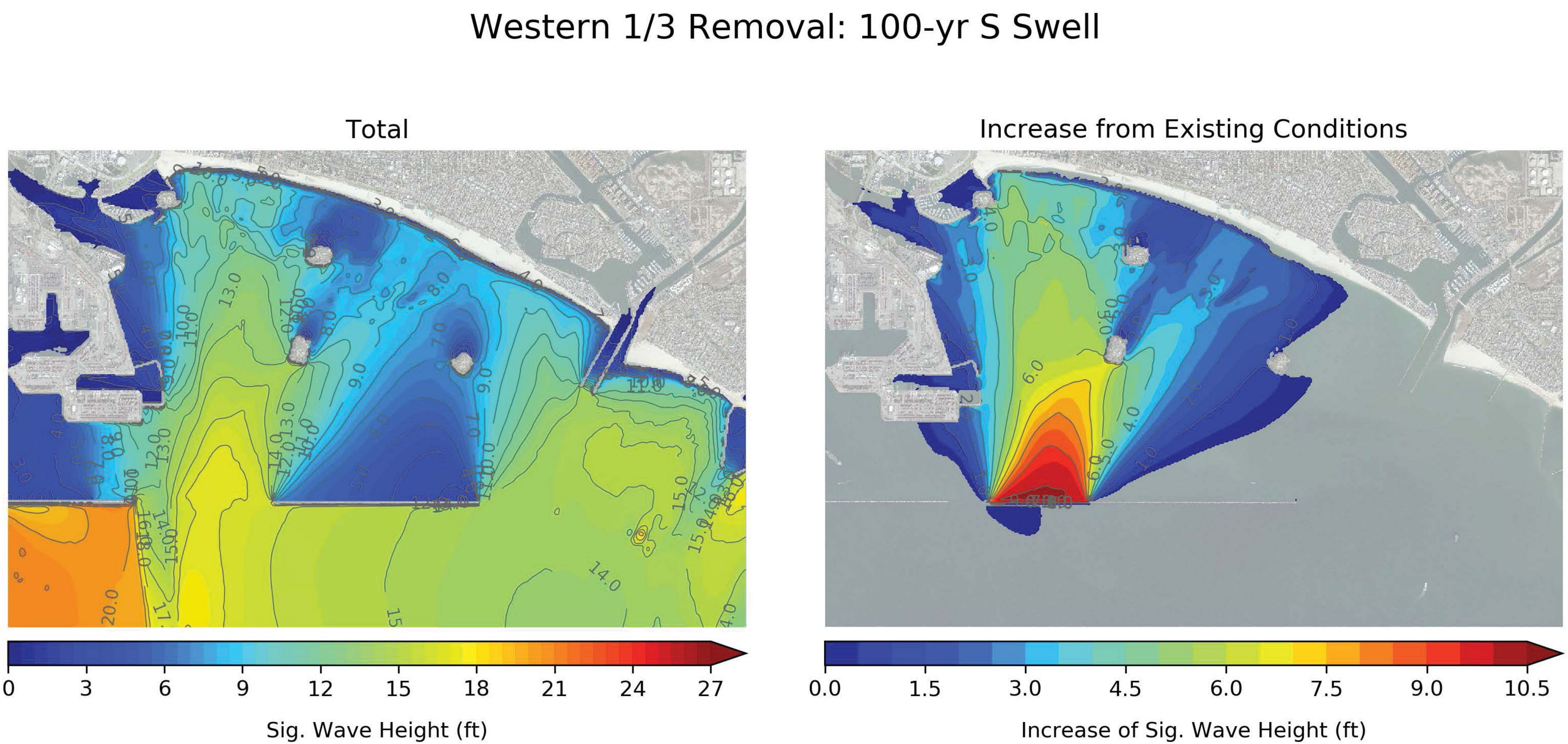


Western 1/3 Removal: 50-yr S Swell

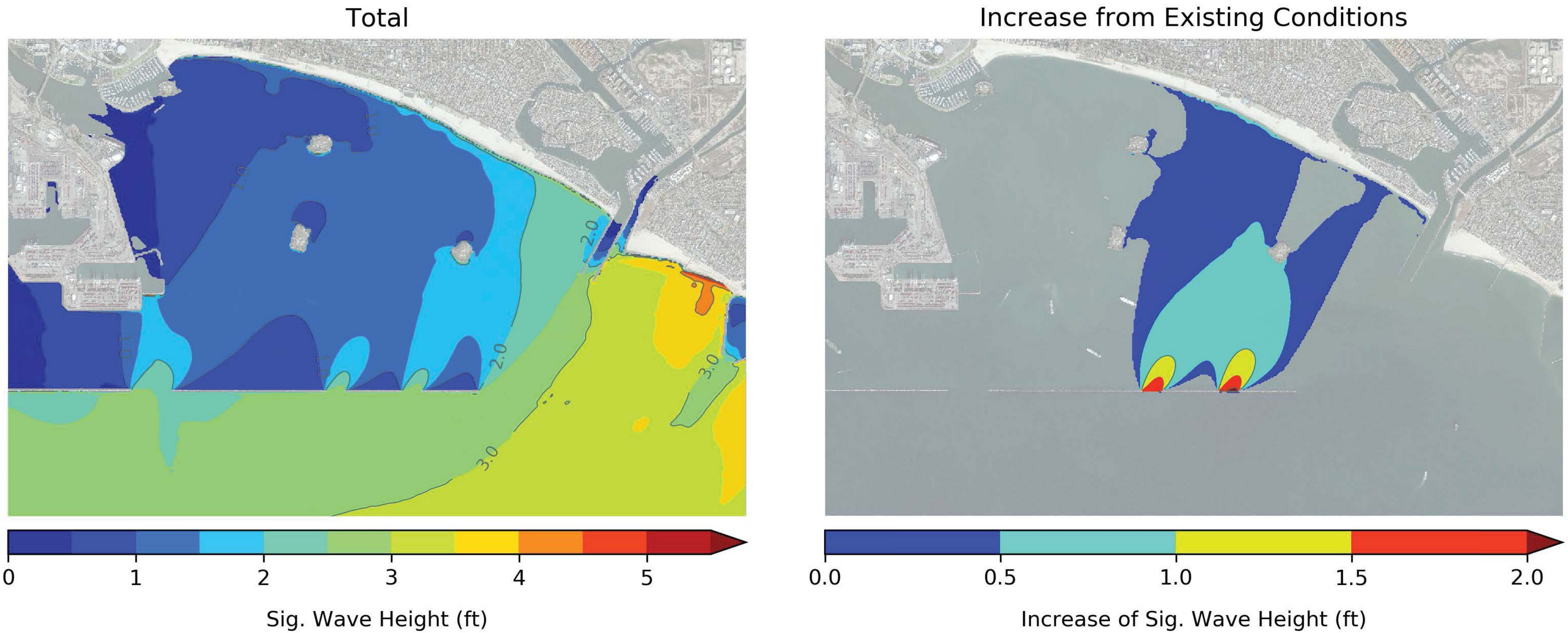


Western 1/3 Removal: 100-yr NW Swell

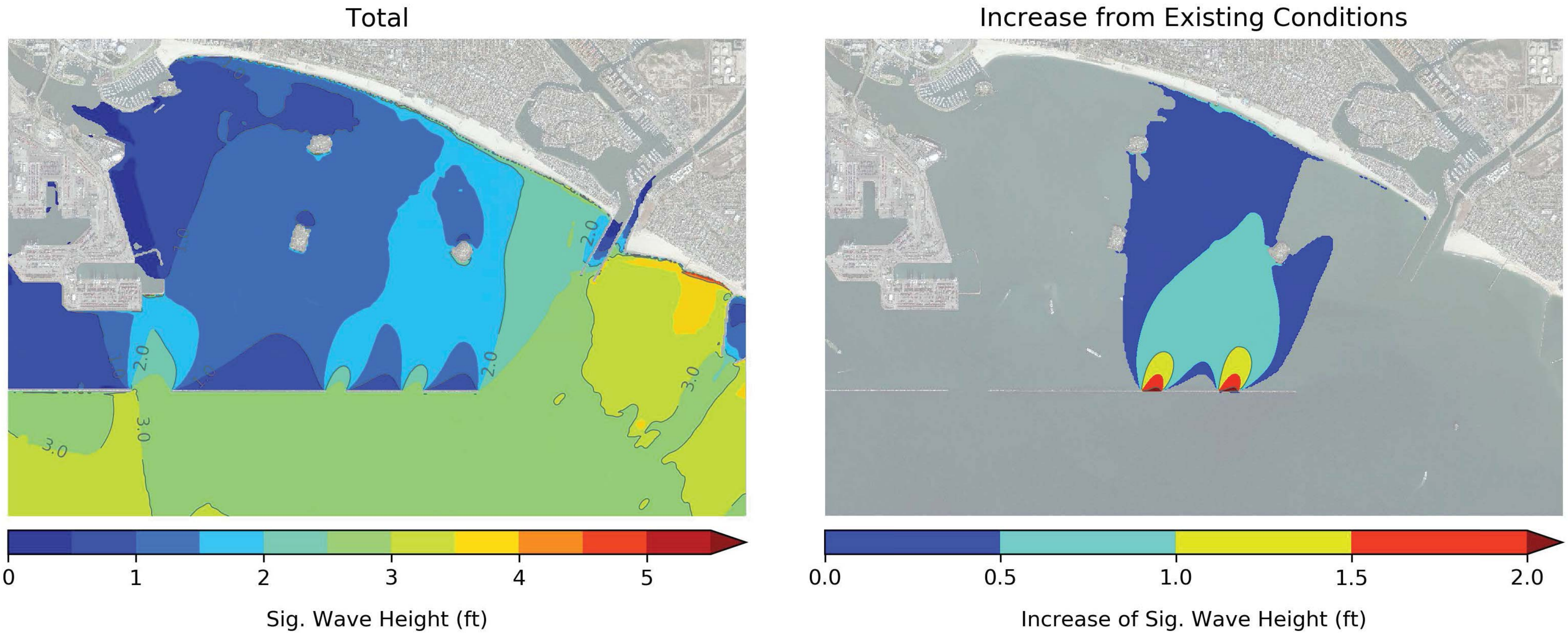



Western 1/3 Removal: 100-yr S Swell
Total
Increase from Existing Conditions
0 3 6 9 12 15 18 21 24 27
Sig. Wave Height (ft)
0.0 1.5 3.0 4.5 6.0 7.5 9.0 10.5
Increase of Sig. Wave Height (ft)

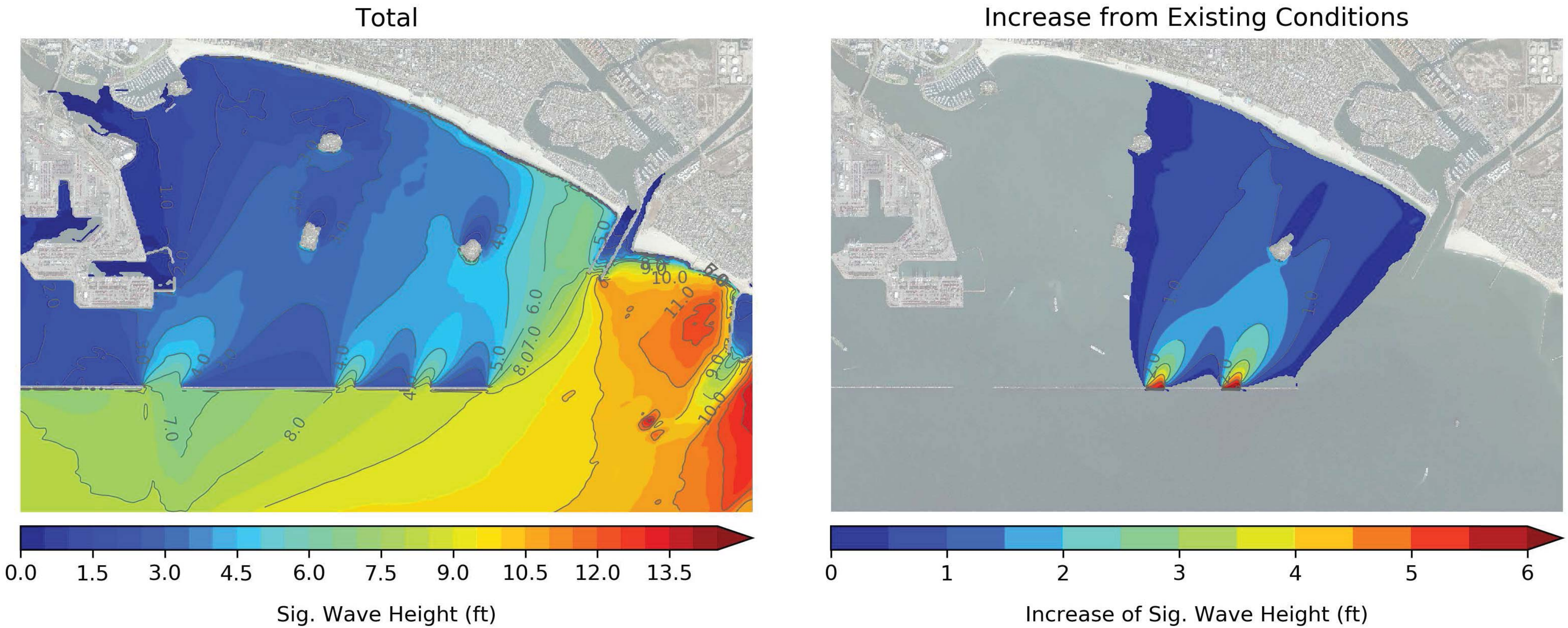
2x 1000' Eastern Notches: Typical Winter



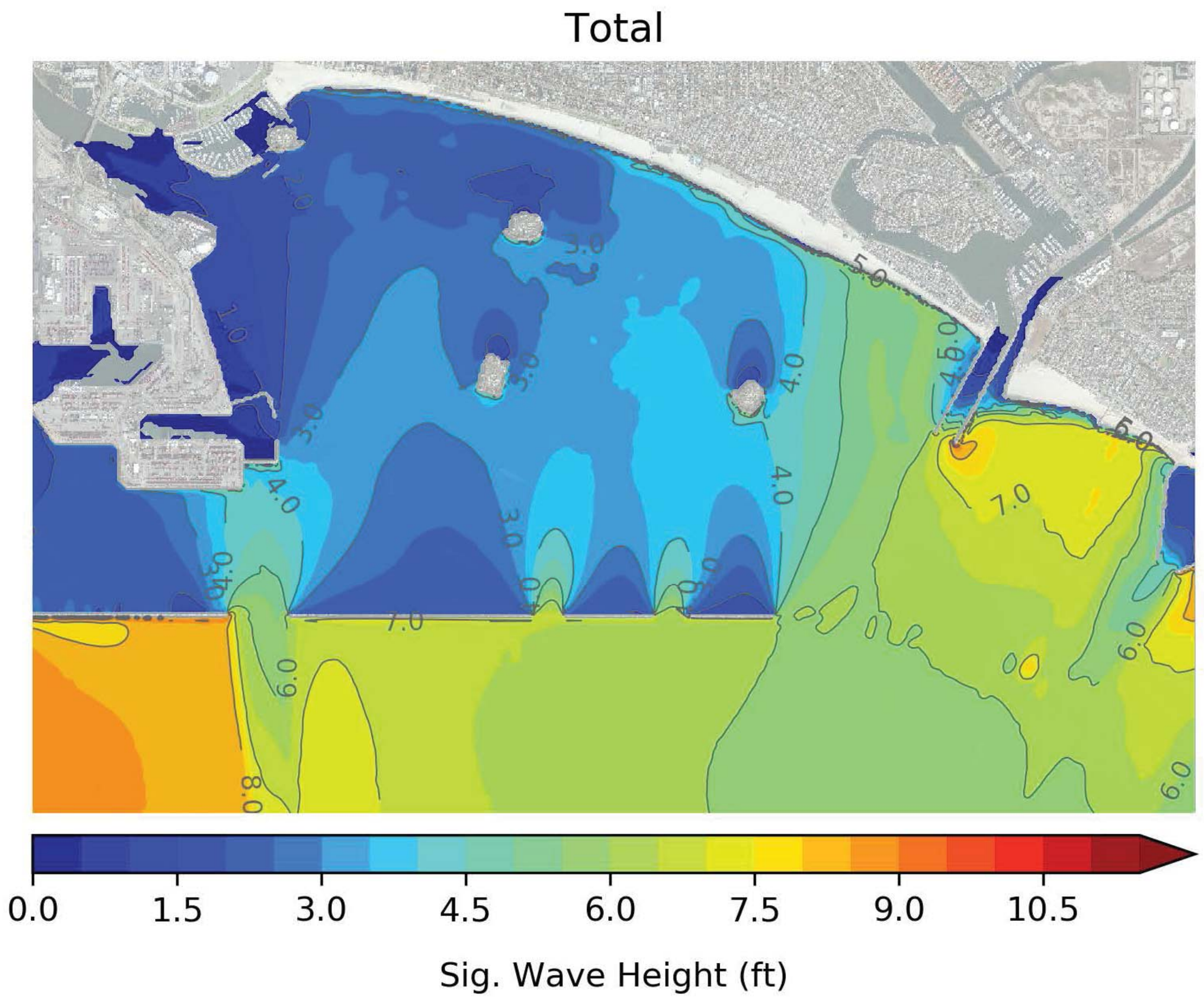
2x 1000' Eastern Notches: Typical Summer

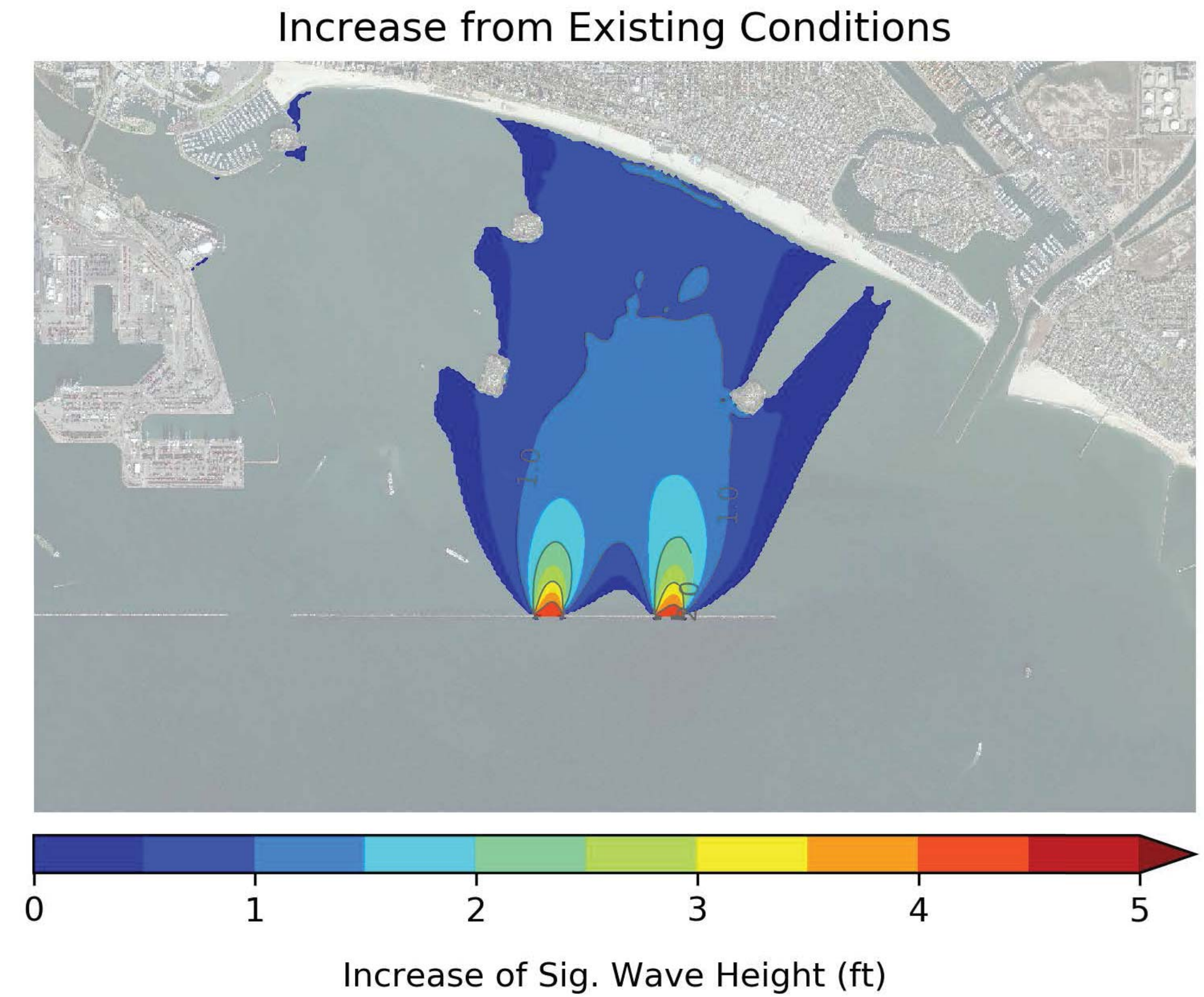


2x 1000' Eastern Notches: 1-yr NW Swell

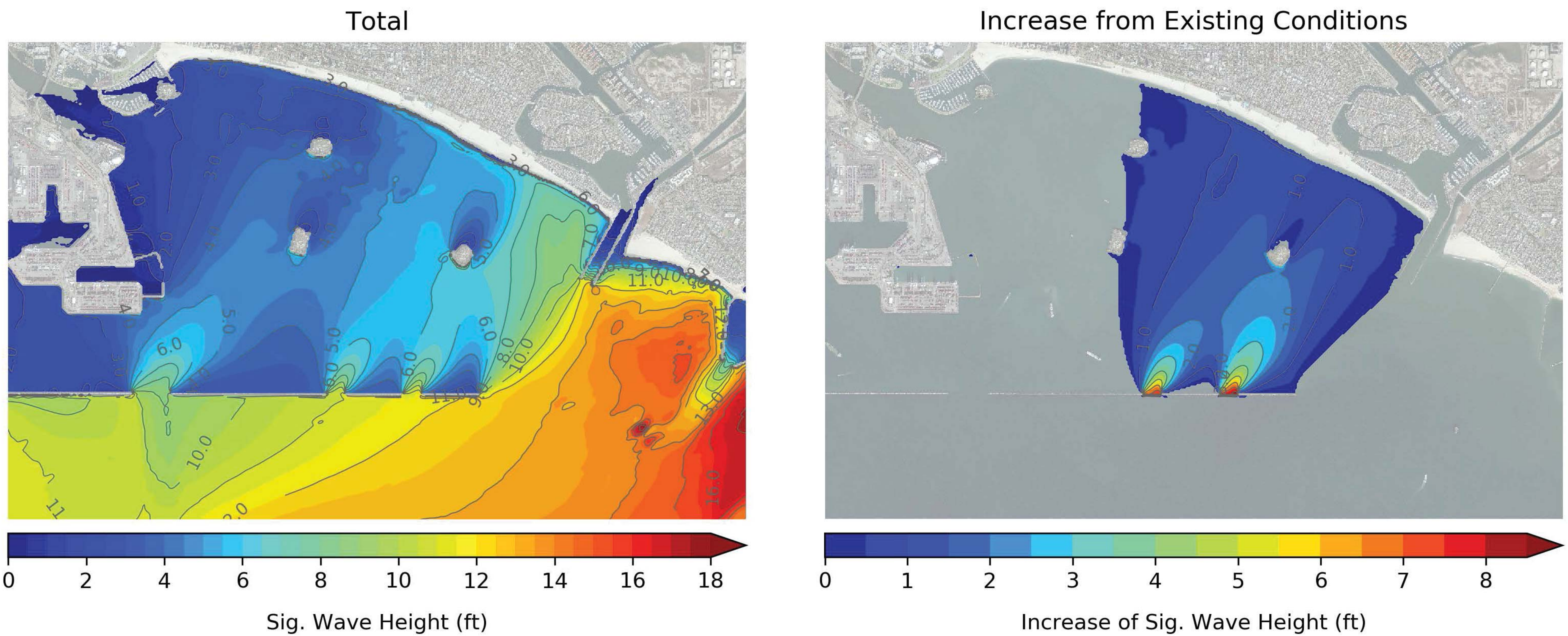


2x 1000' Eastern Notches: 1-yr S Swell

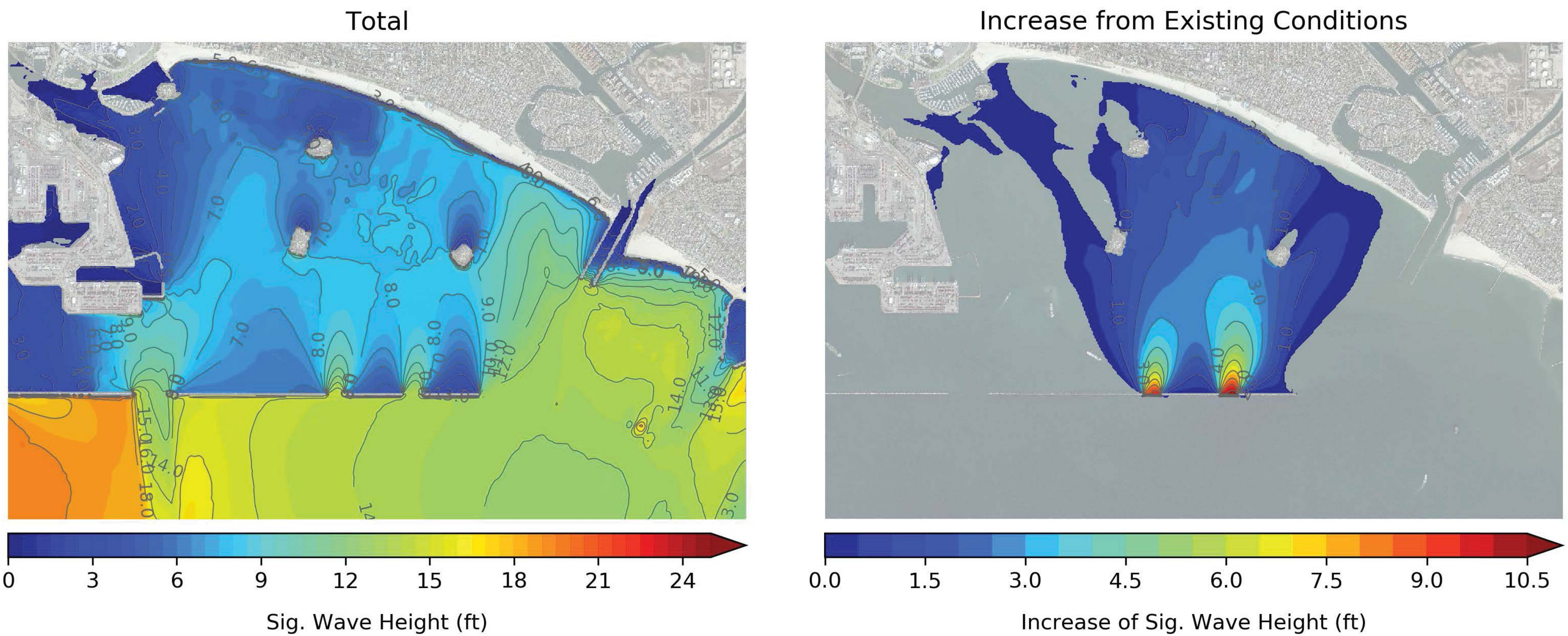




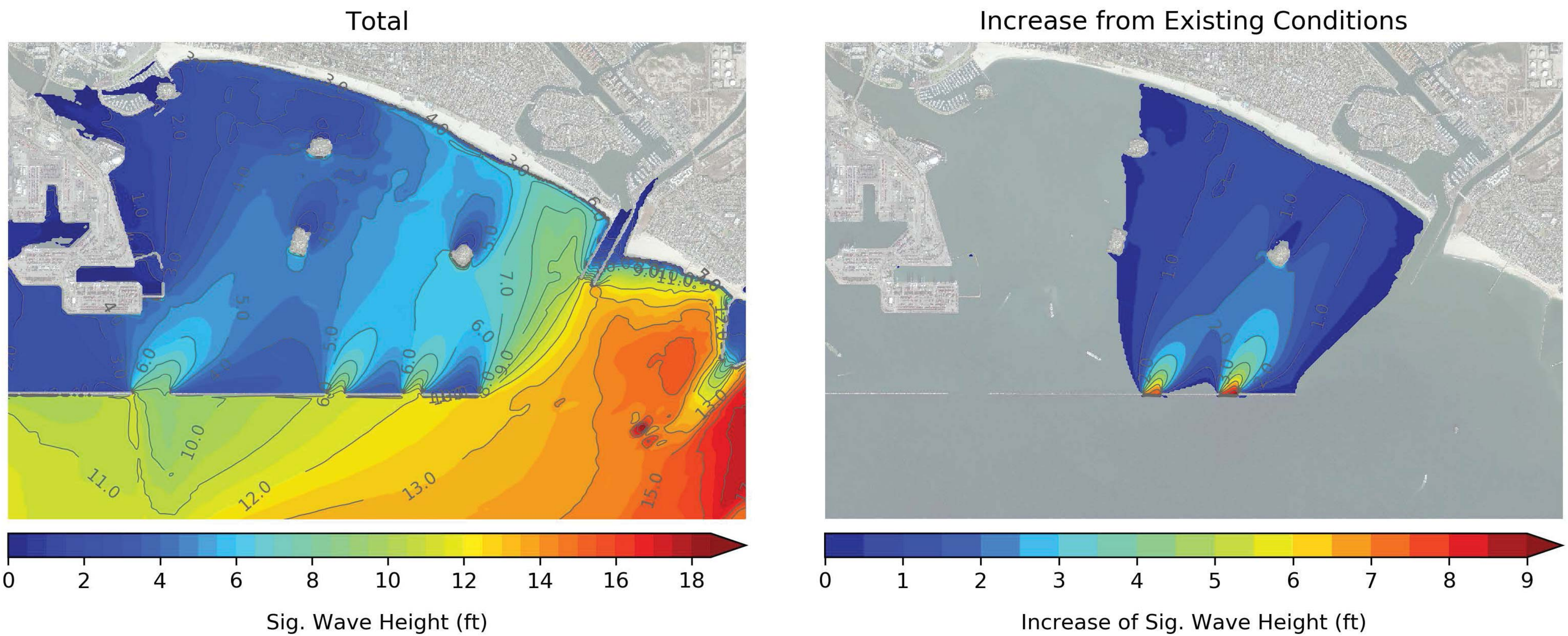
2x 1000' Eastern Notches: 50-yr NW Swell



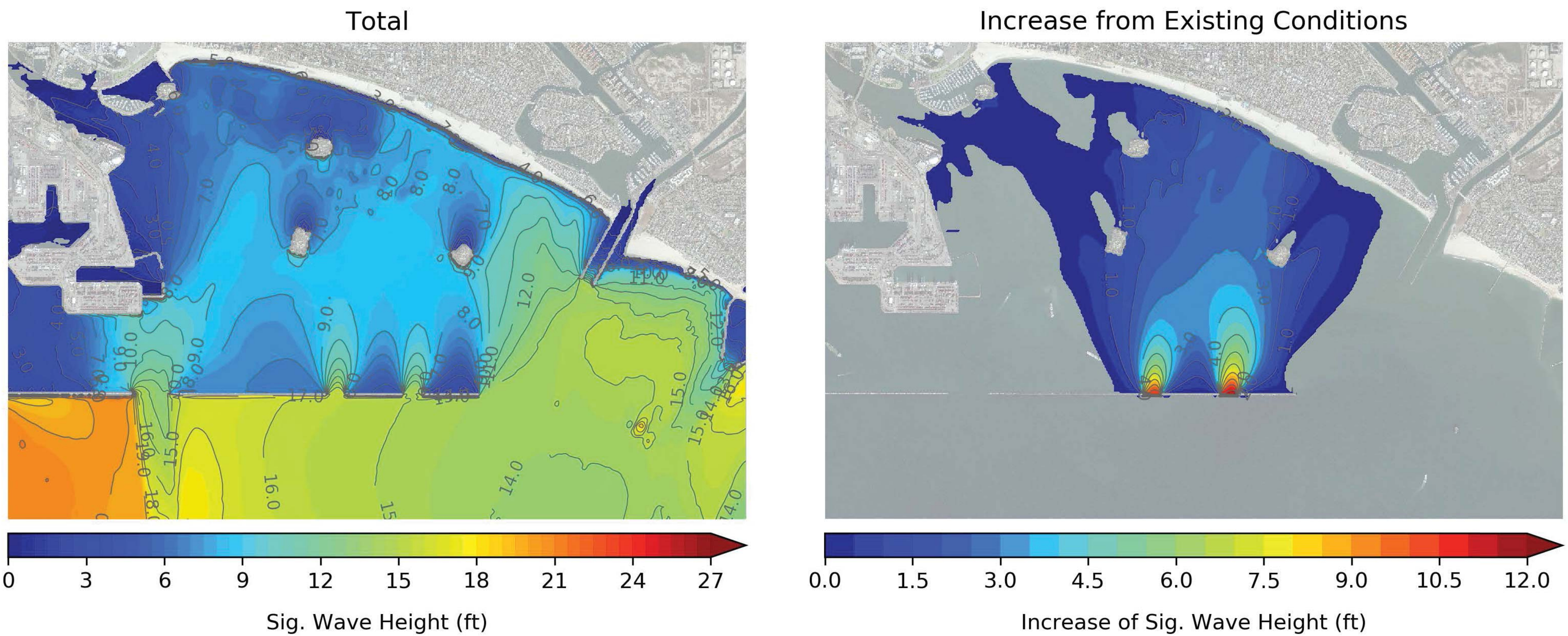
2x 1000' Eastern Notches: 50-yr S Swell



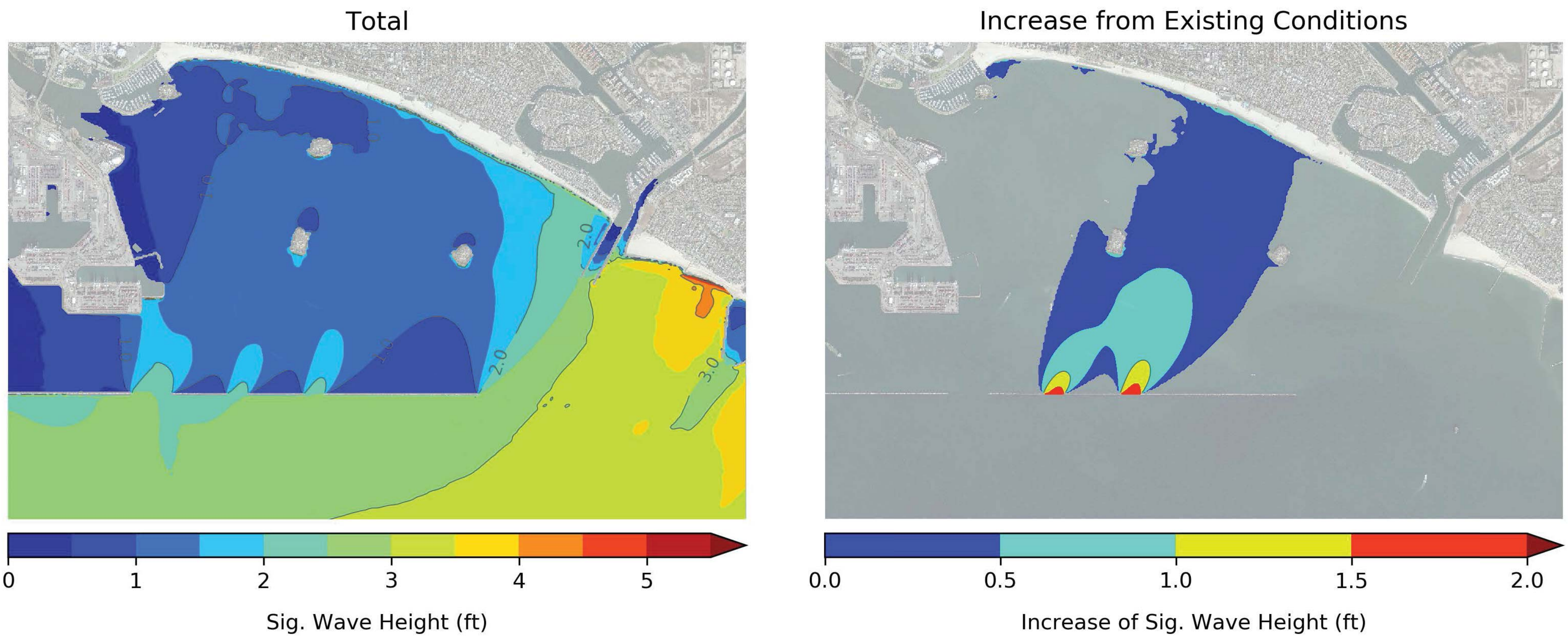
2x 1000' Eastern Notches: 100-yr NW Swell



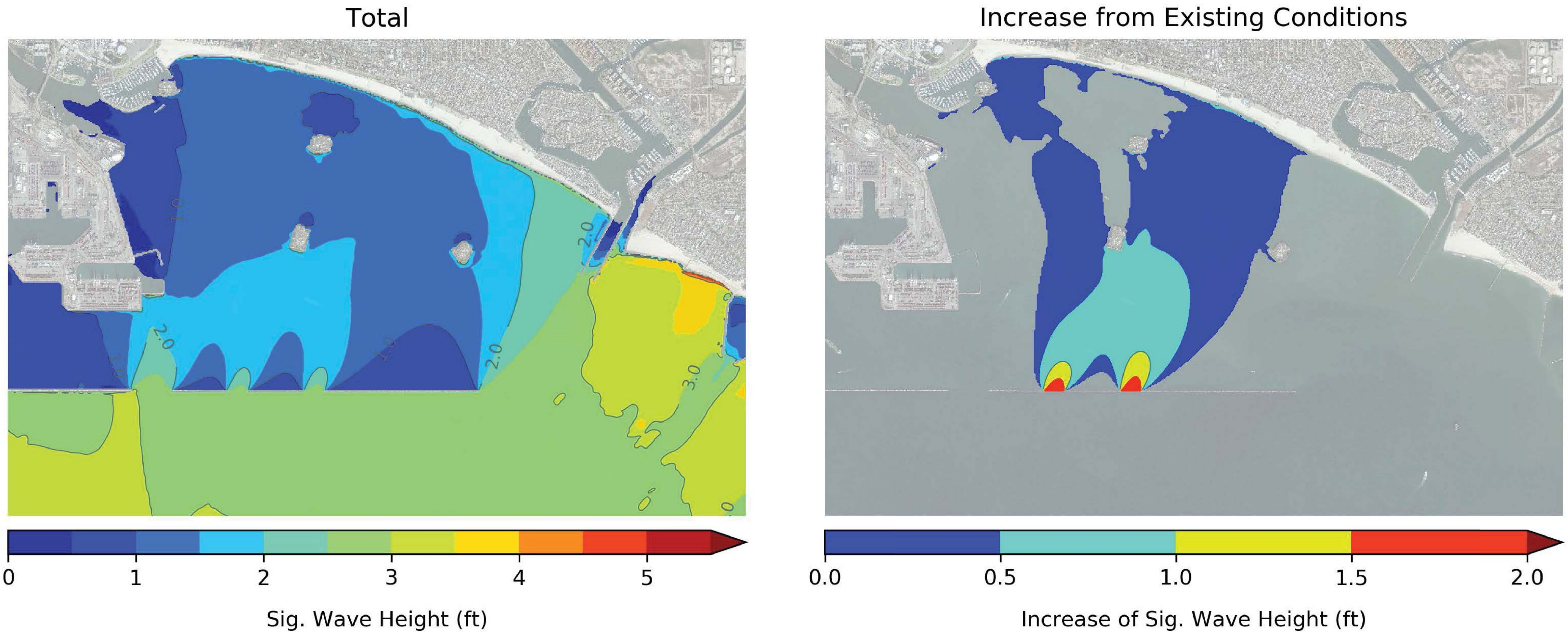
2x 1000' Eastern Notches: 100-yr S Swell



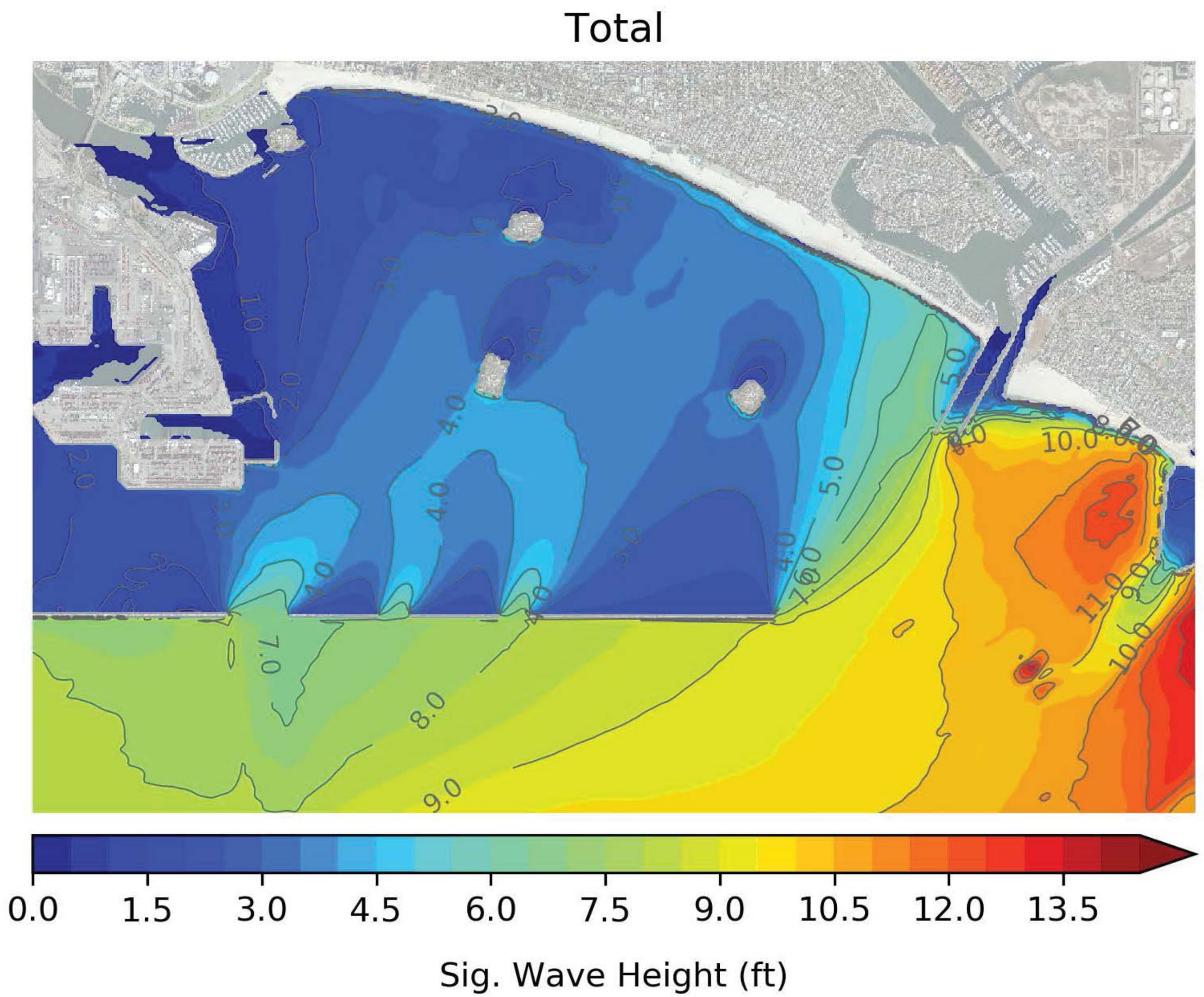
2x 1000' Western Notches: Typical Winter

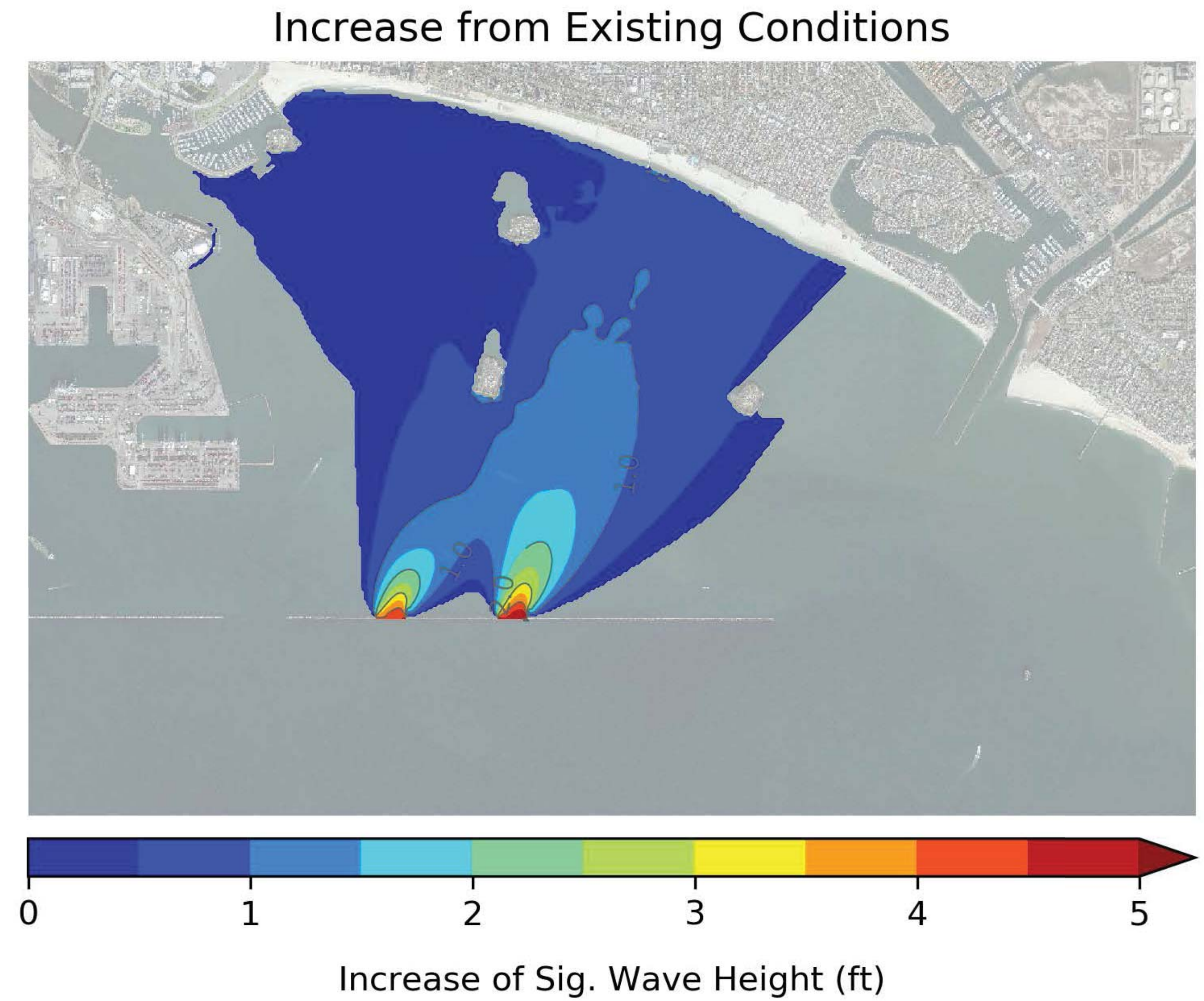


2x 1000' Western Notches: Typical Summer

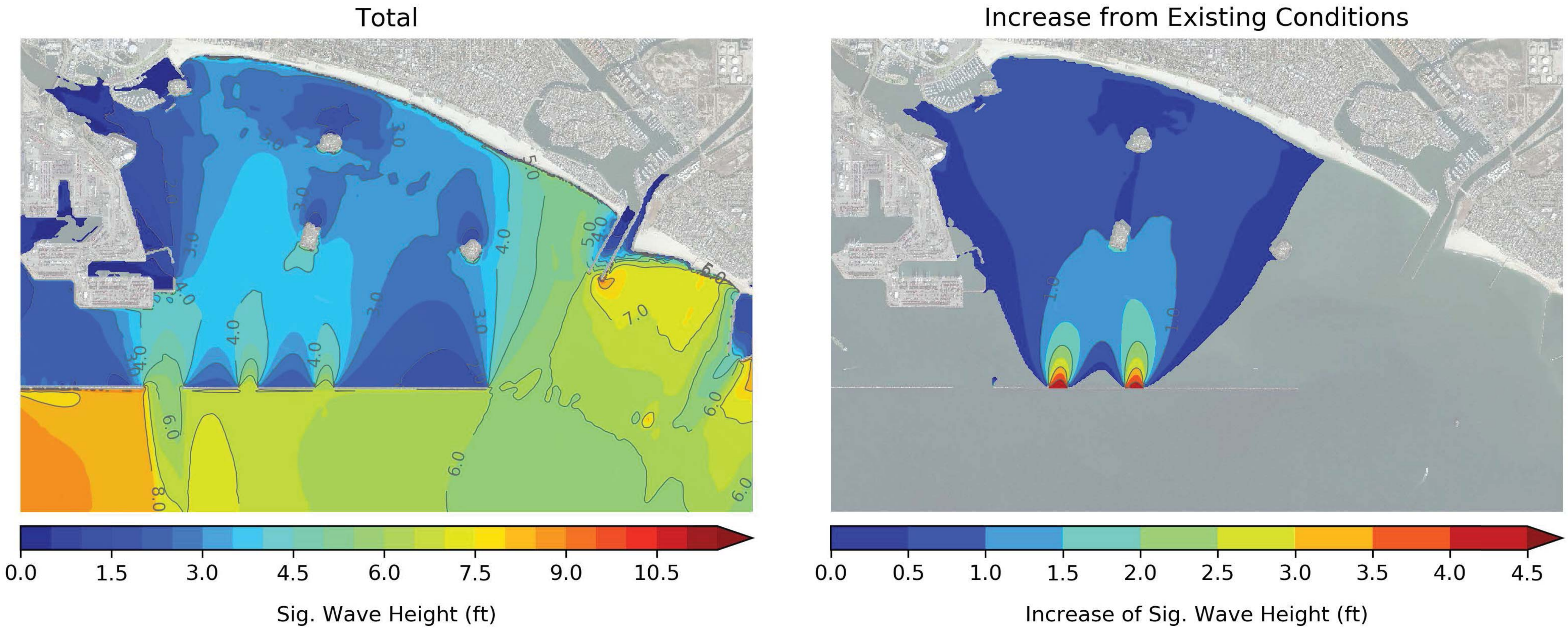


2x 1000' Western Notches: 1-yr NW Swell

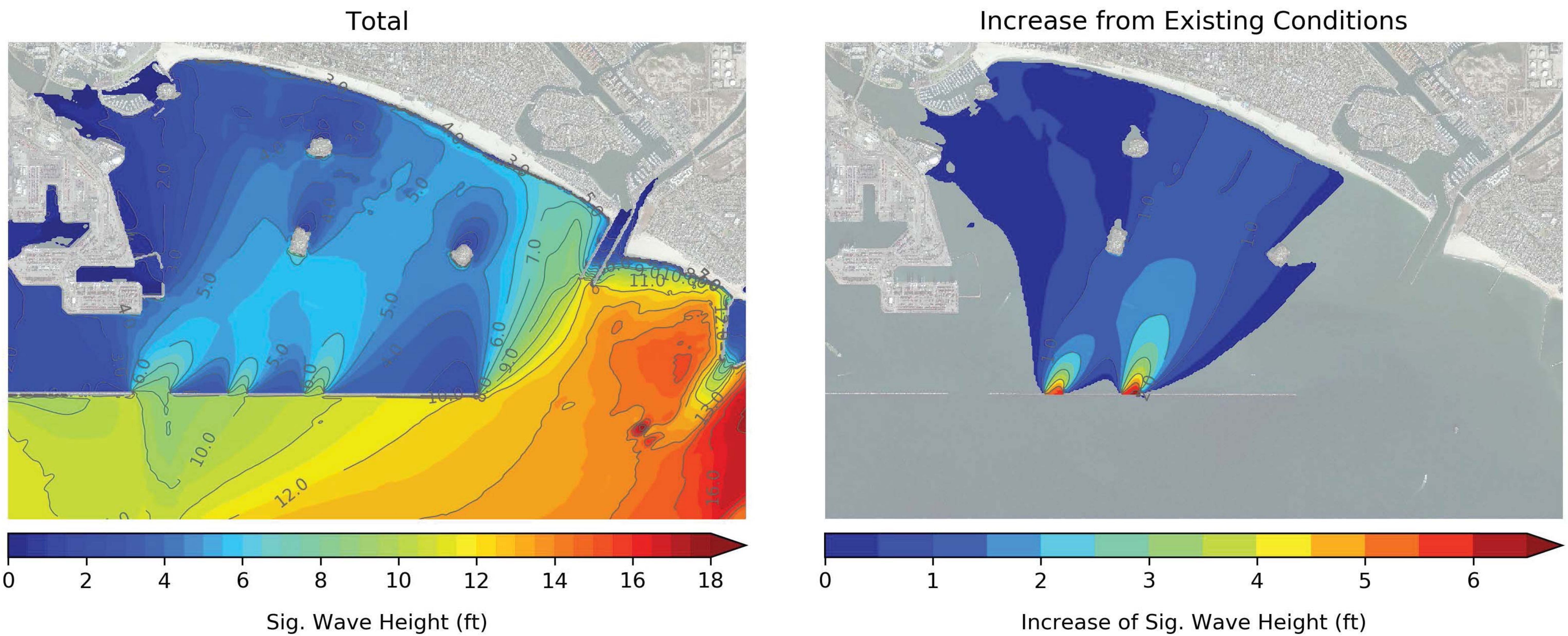




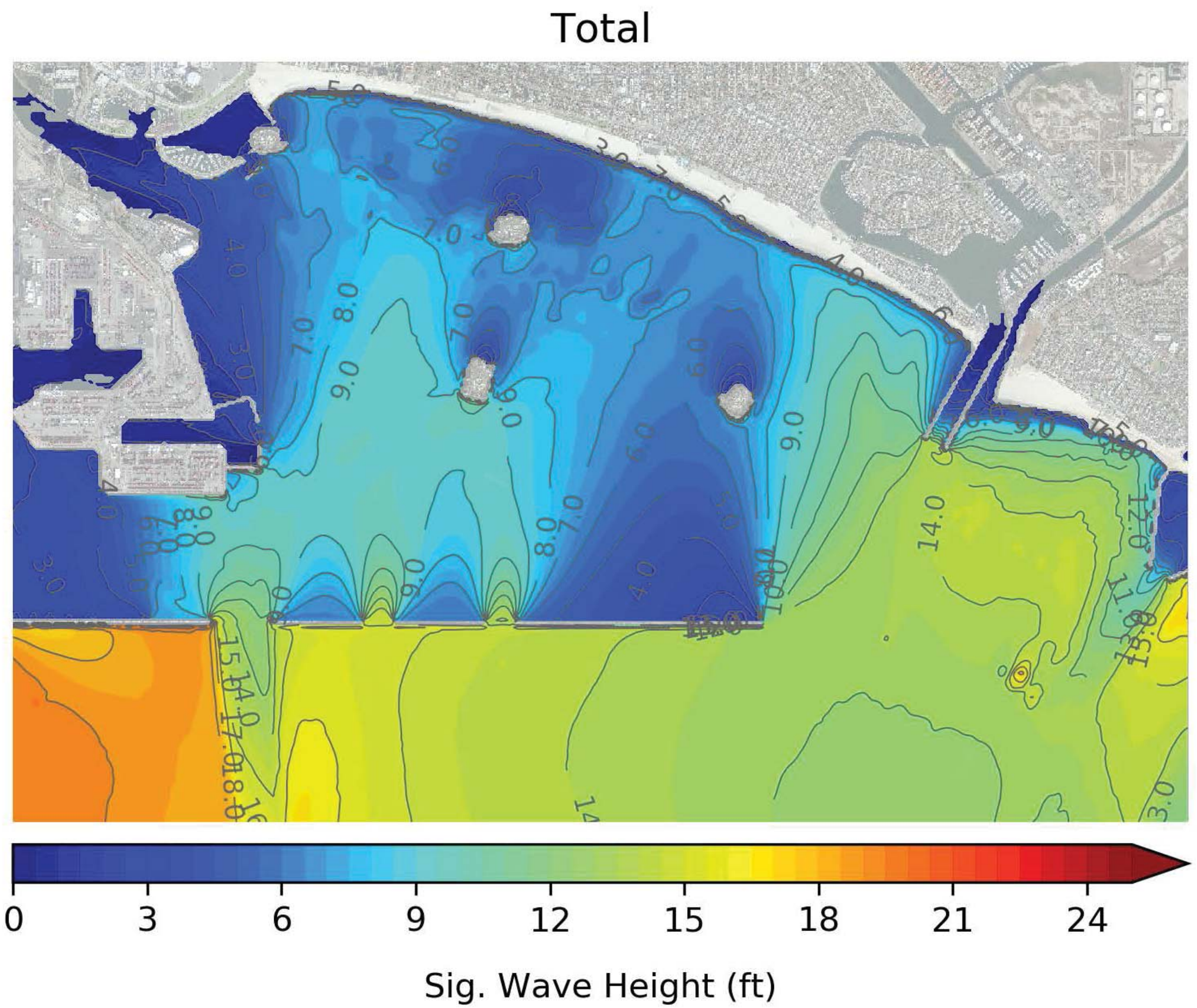
2x 1000' Western Notches: 1-yr S Swell

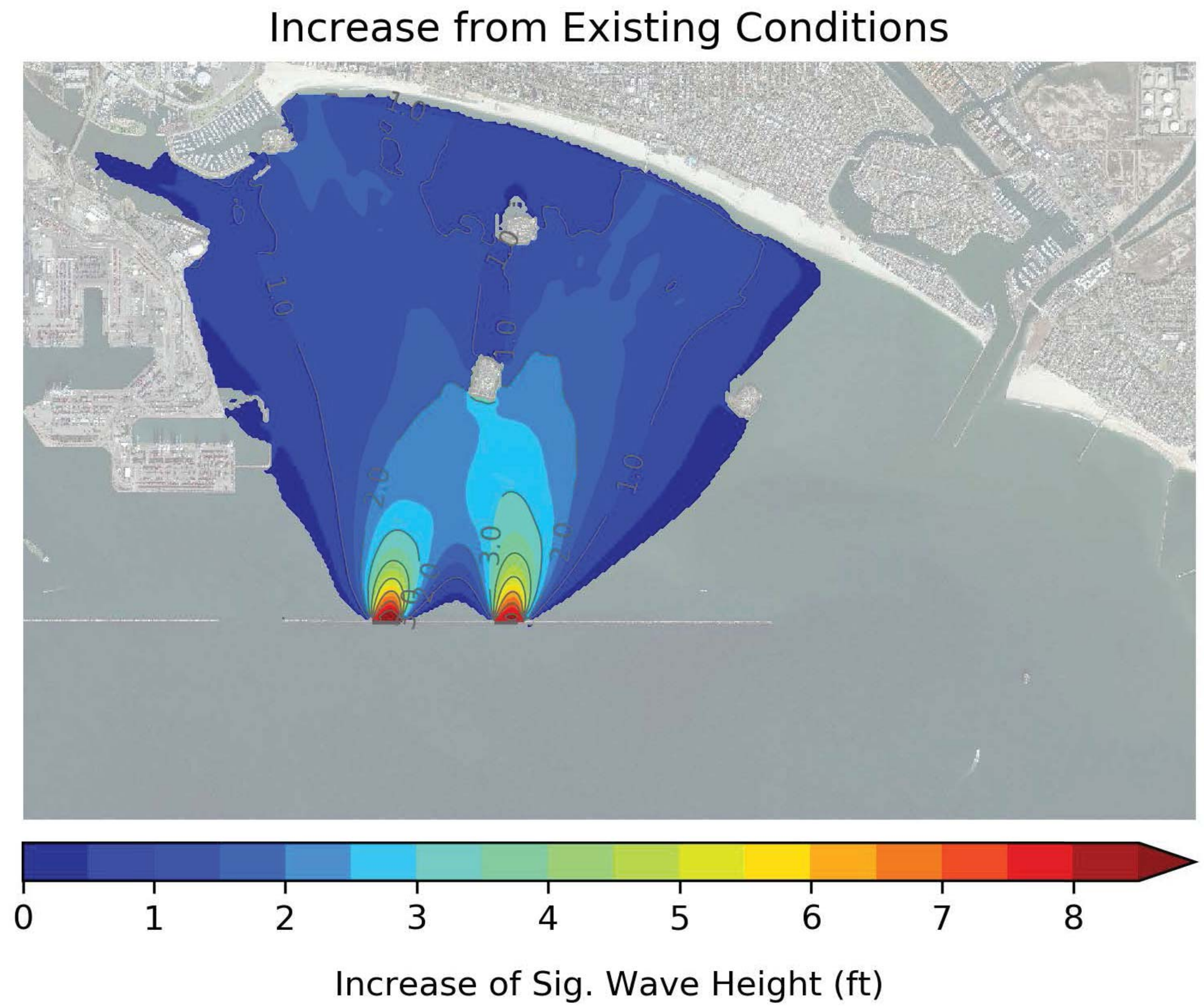


2x 1000' Western Notches: 50-yr NW Swell

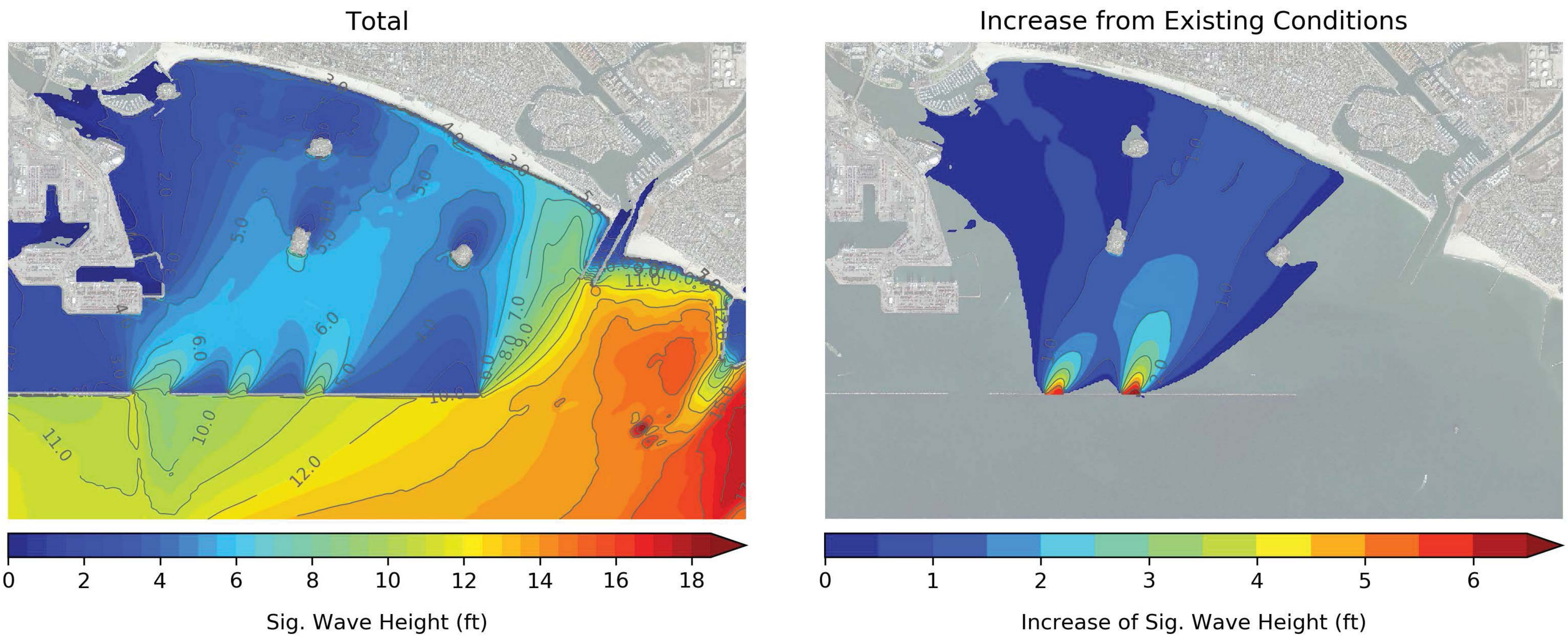


2x 1000' Western Notches: 50-yr S Swell

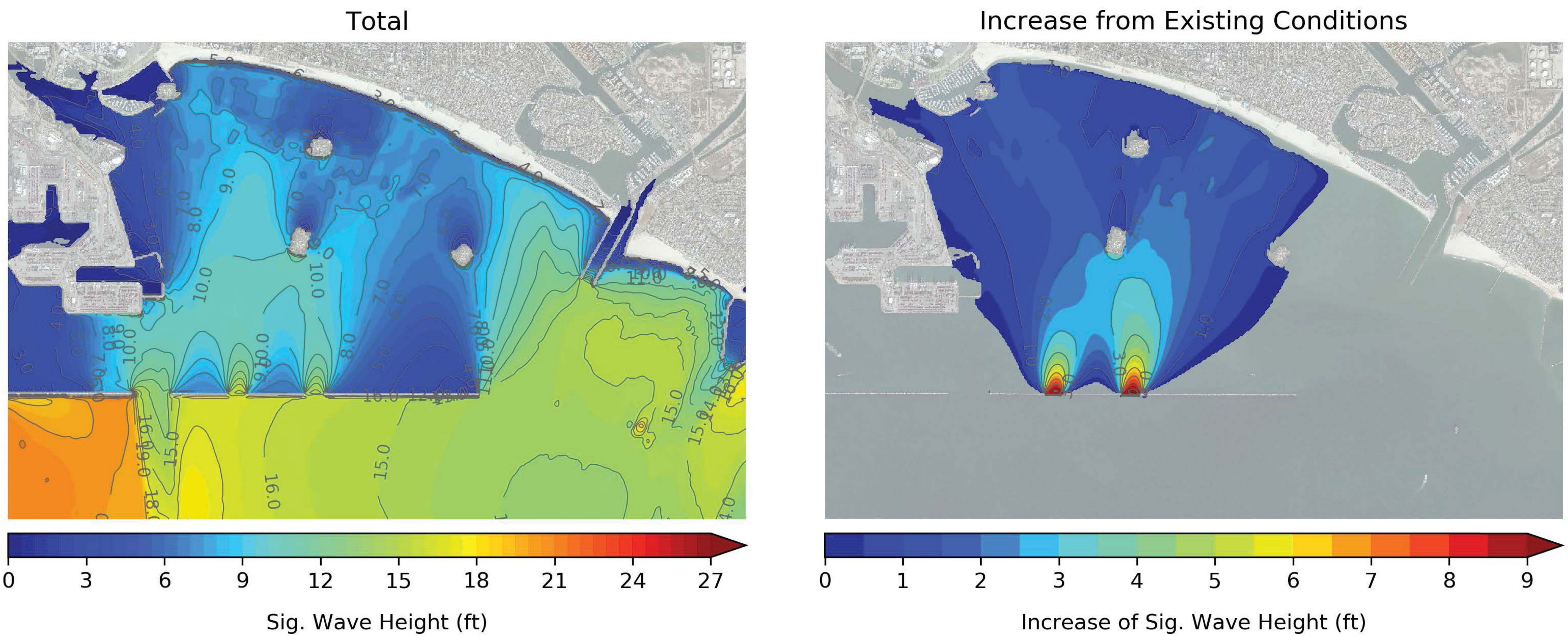




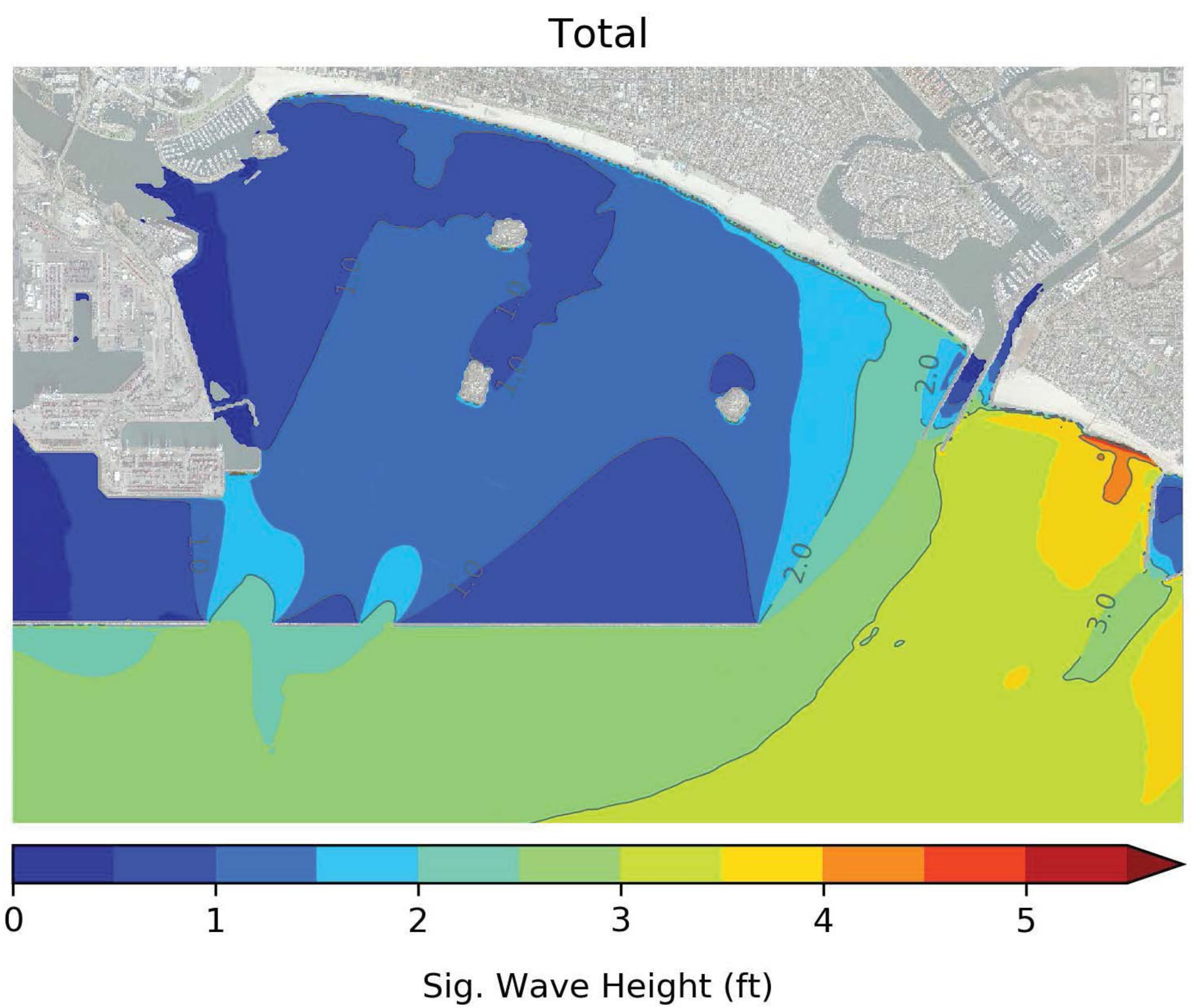
2x 1000' Western Notches: 100-yr NW Swell

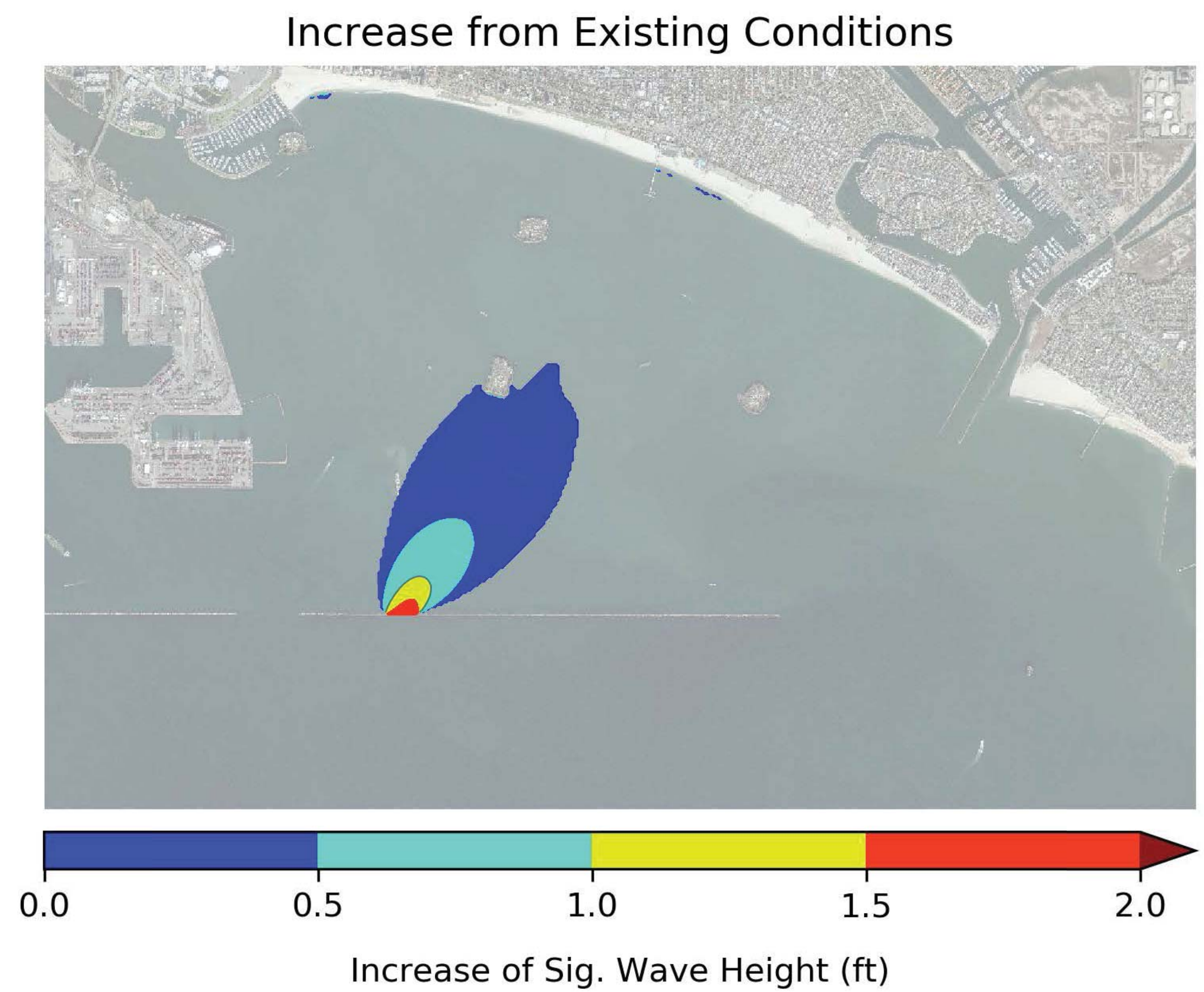


2x 1000' Western Notches: 100-yr S Swell

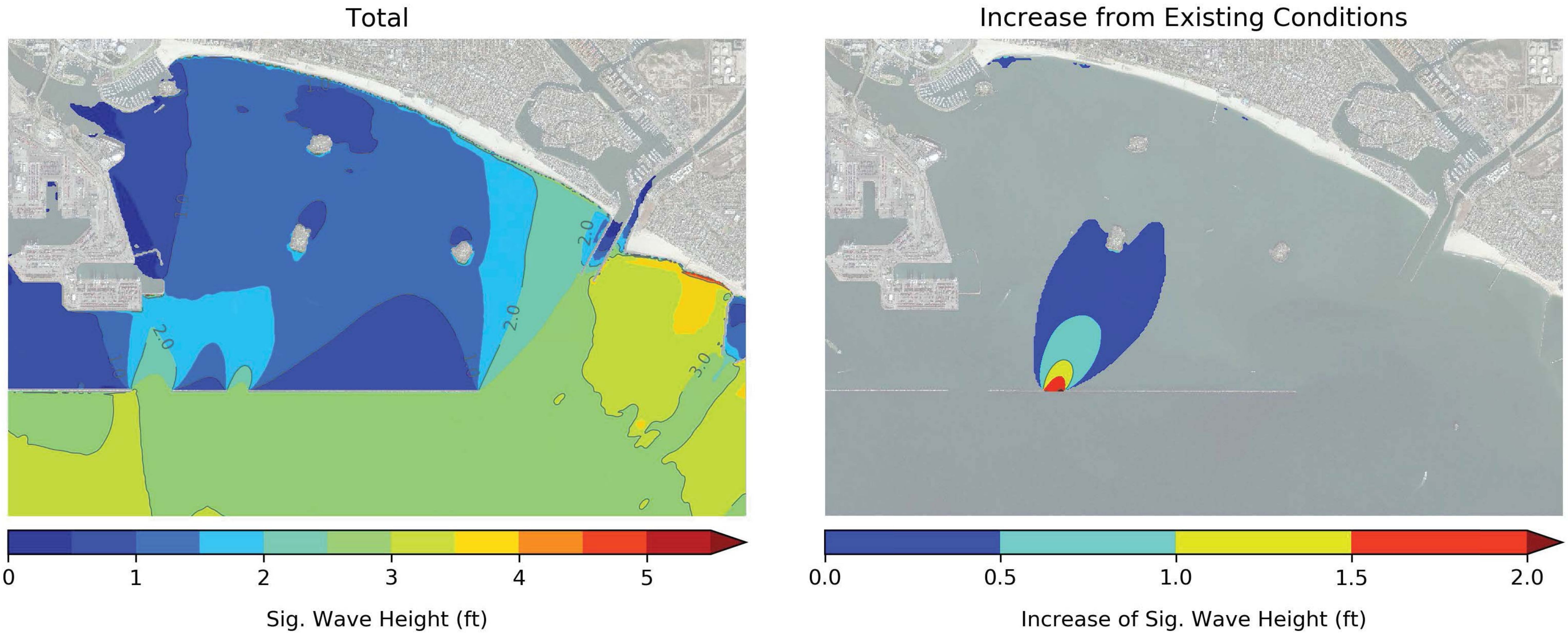


1x 1000' Western Notch: Typical Winter

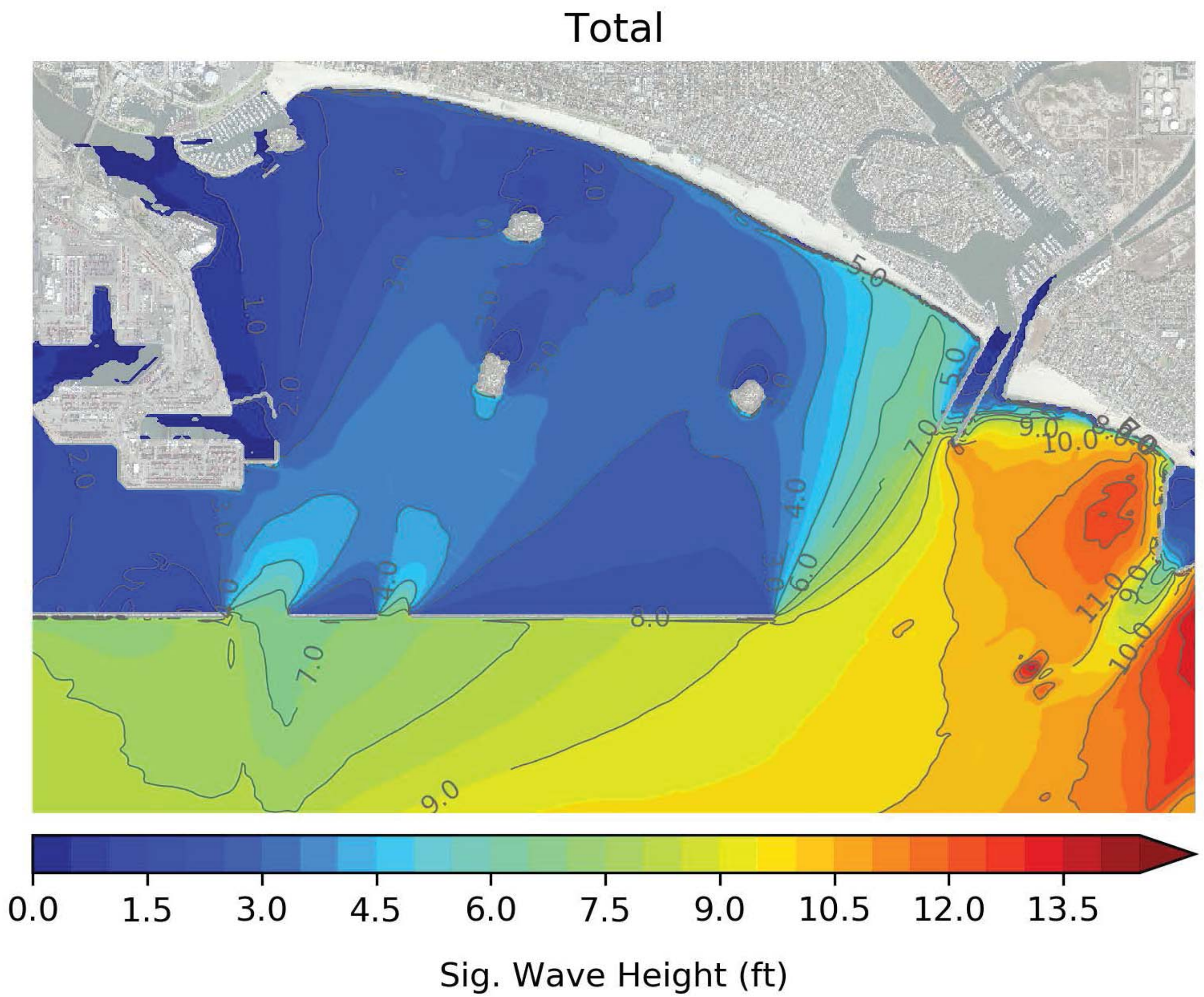


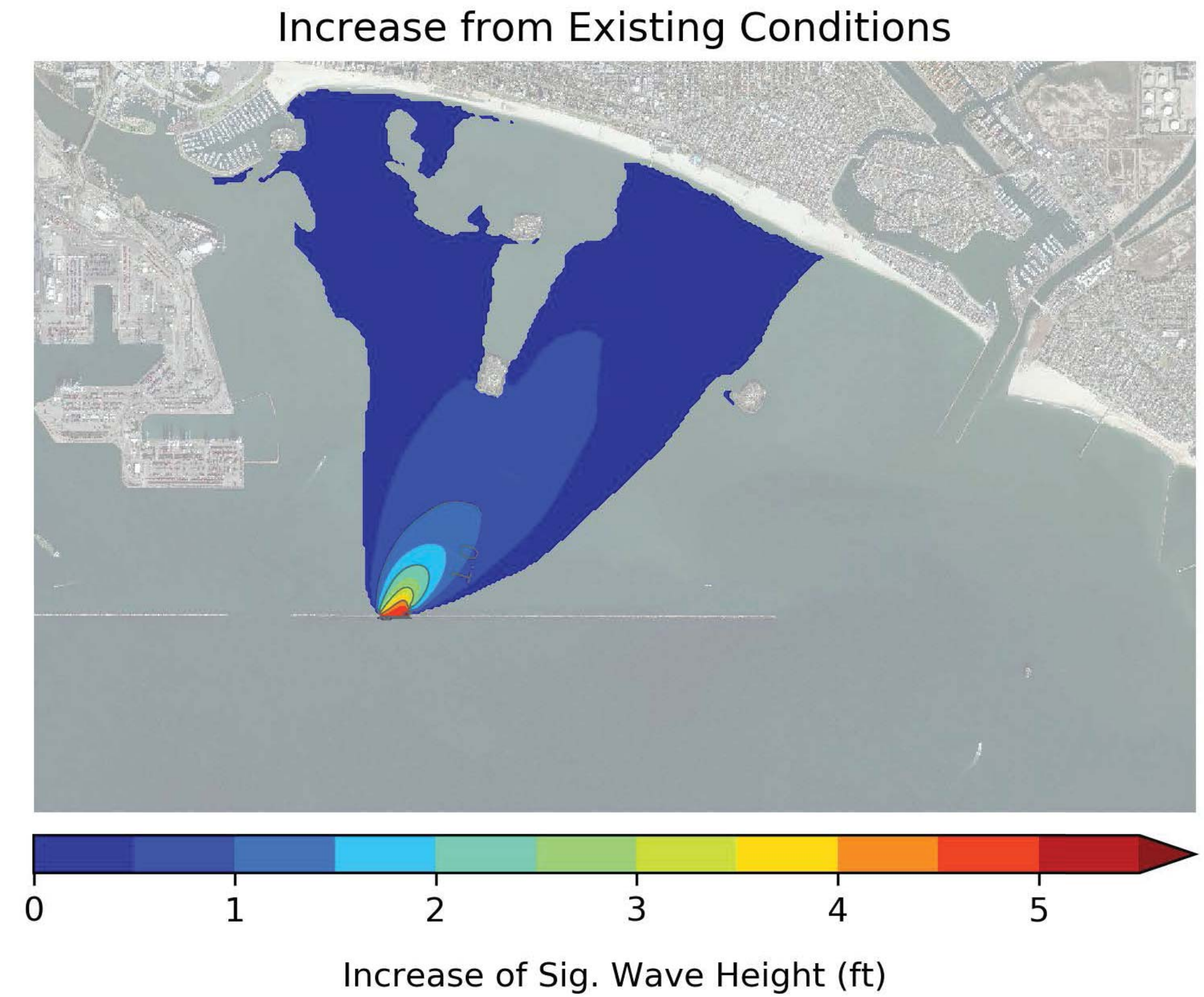


1x 1000' Western Notch: Typical Summer

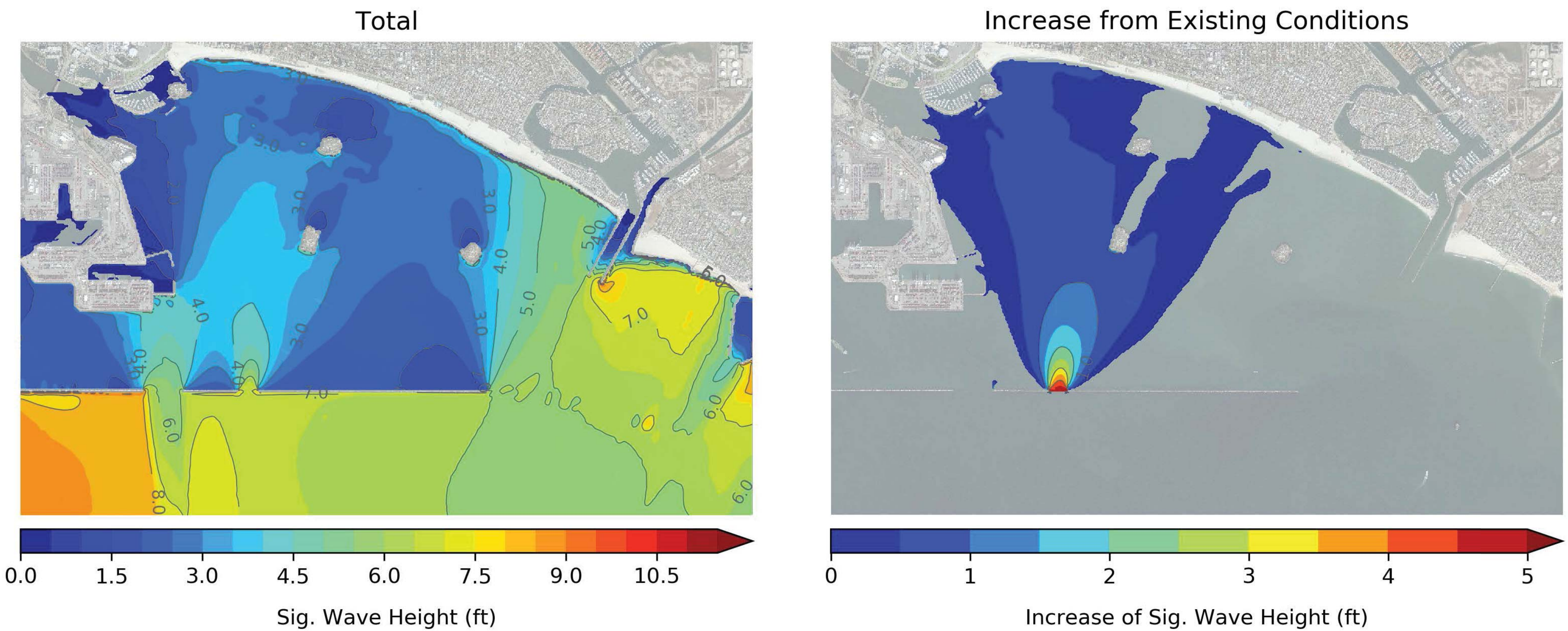


1x 1000' Western Notch: 1-yr NW Swell

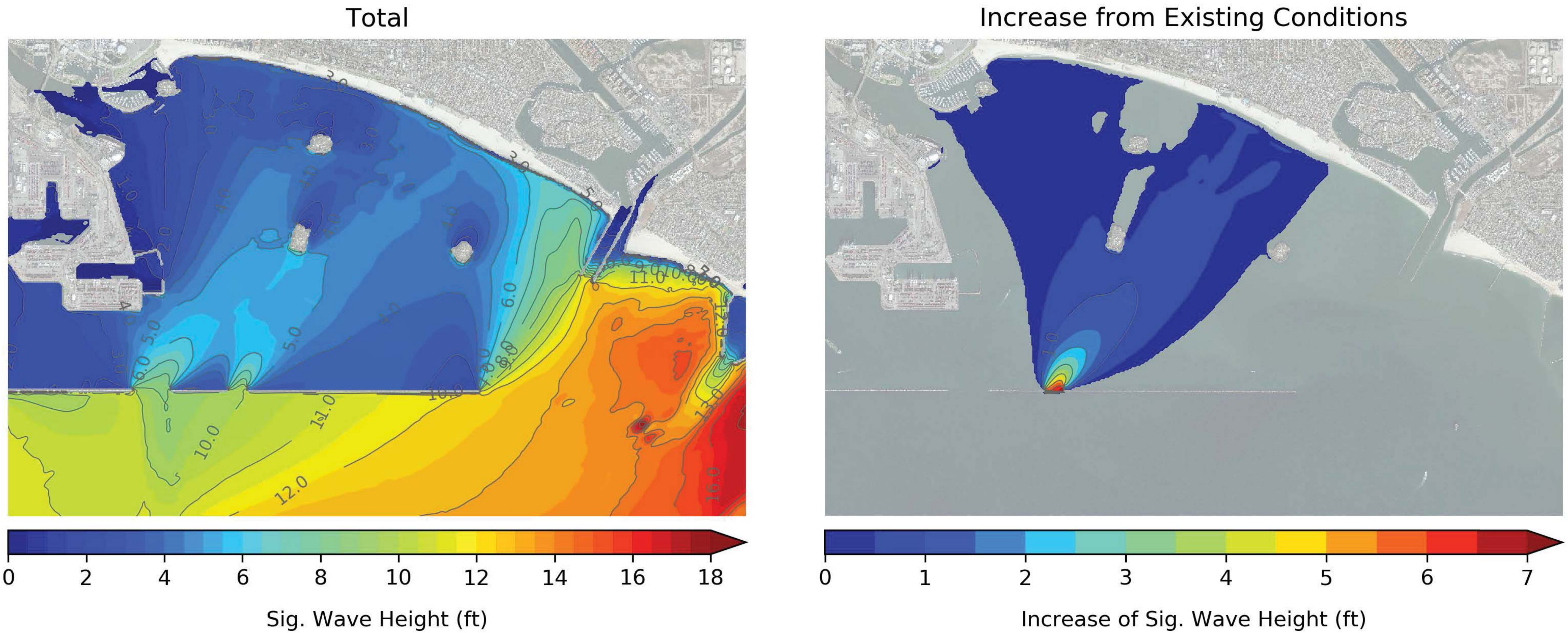


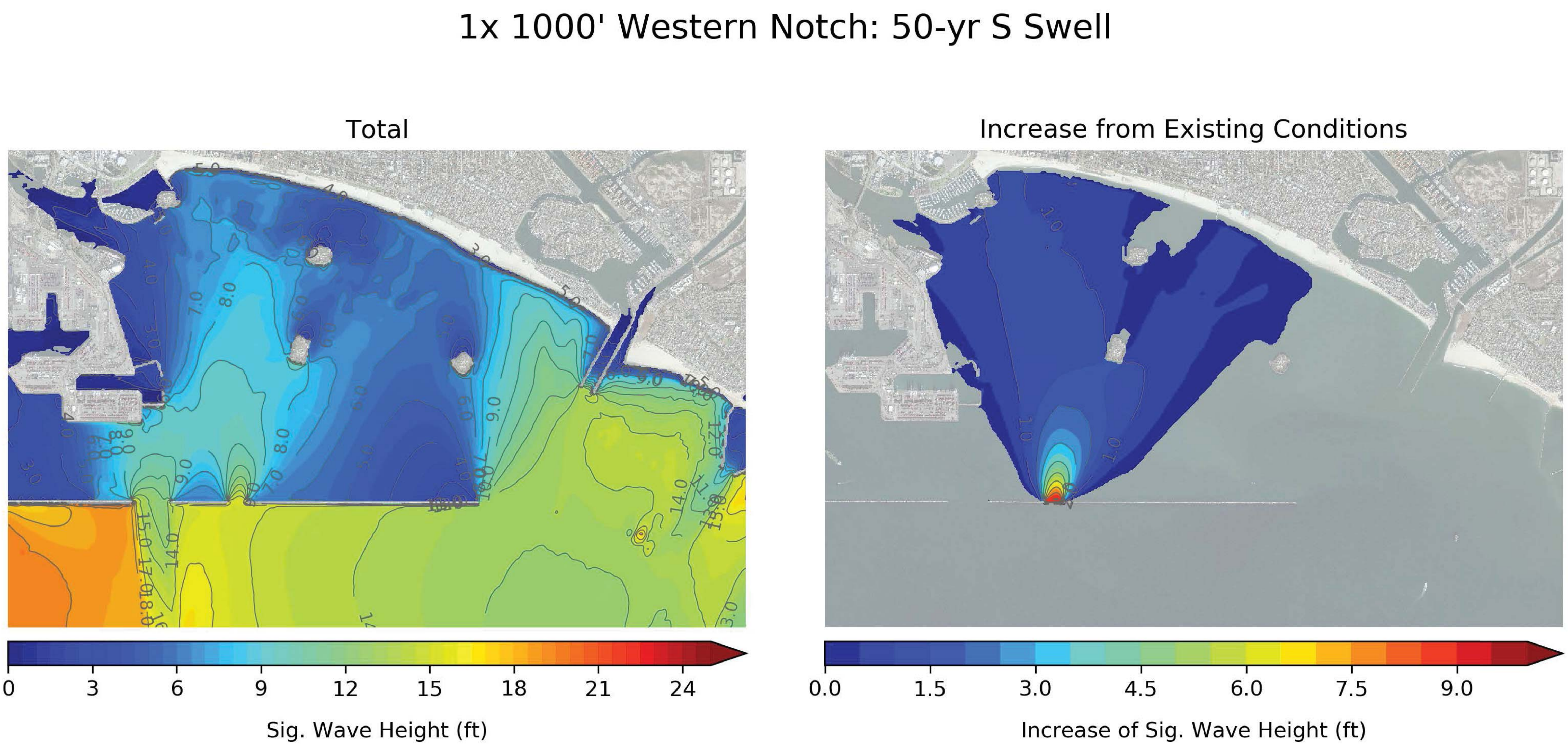


1x 1000' Western Notch: 1-yr S Swell

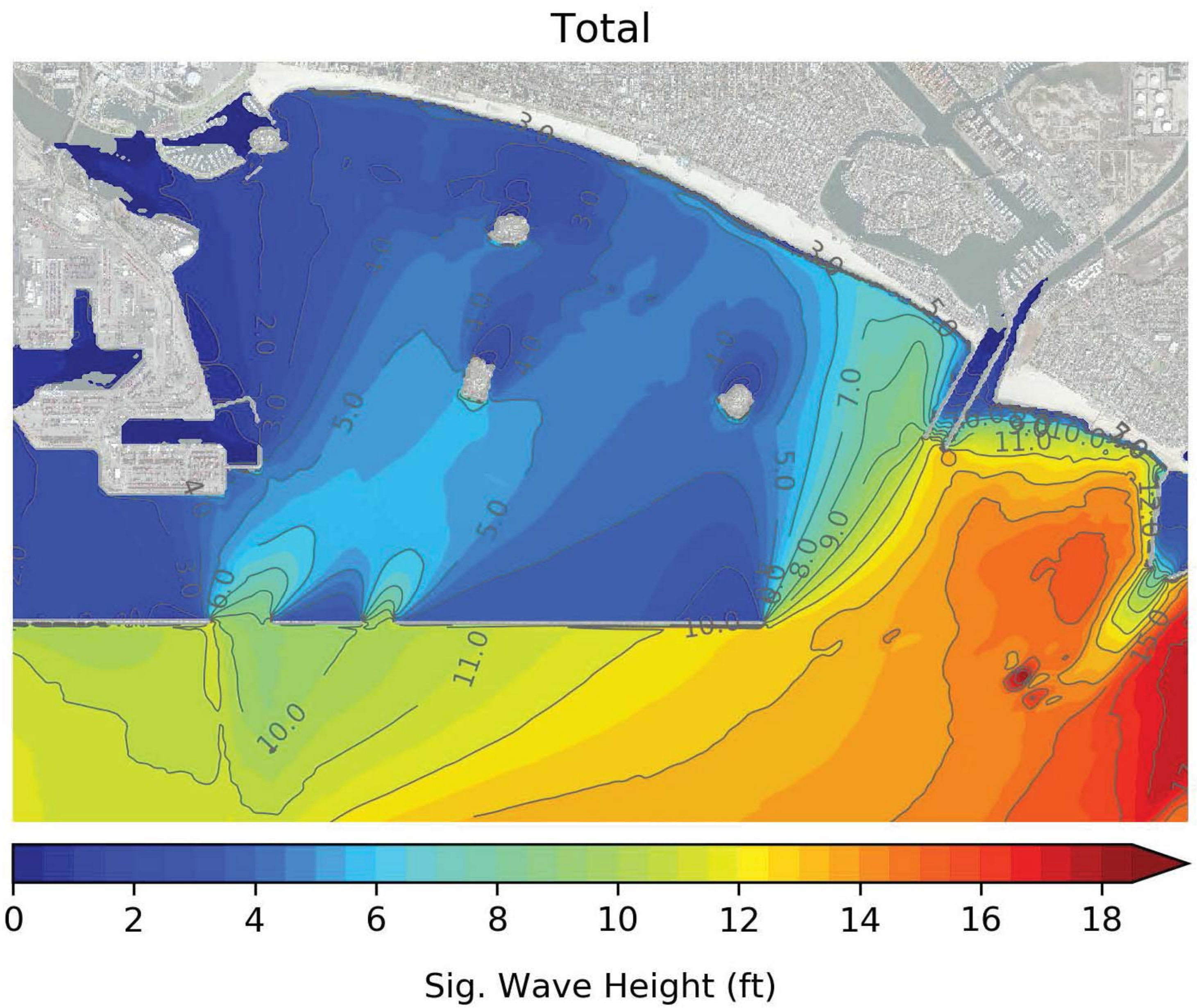


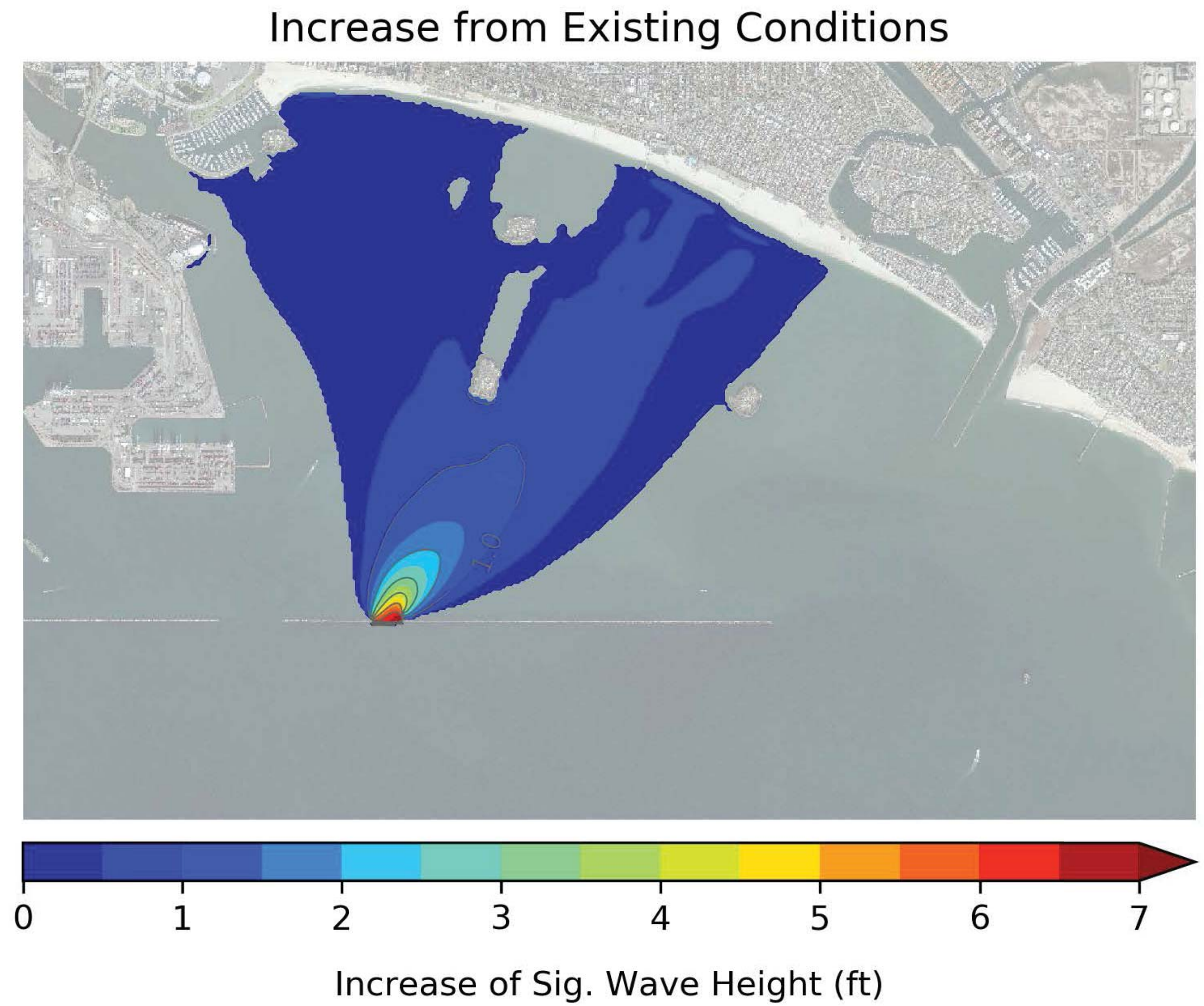
1x 1000' Western Notch: 50-yr NW Swell



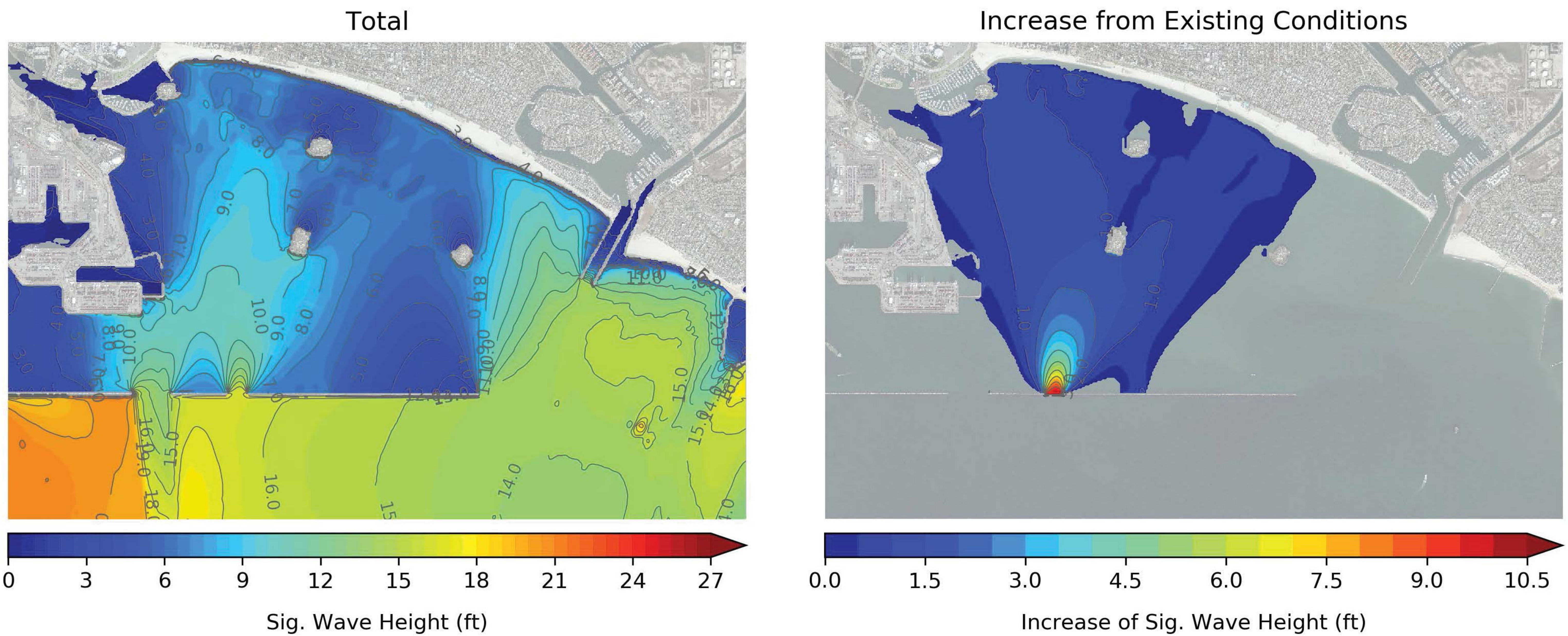

1x 1000' Western Notch: 50-yr S Swell
Total
Increase from Existing Conditions
0
3
6
9
12
15
18
21
24
Sig. Wave Height (ft)
0.0
1.5
3.0
4.5
6.0
7.5
9.0
Increase of Sig. Wave Height (ft)

1x 1000' Western Notch: 100-yr NW Swell

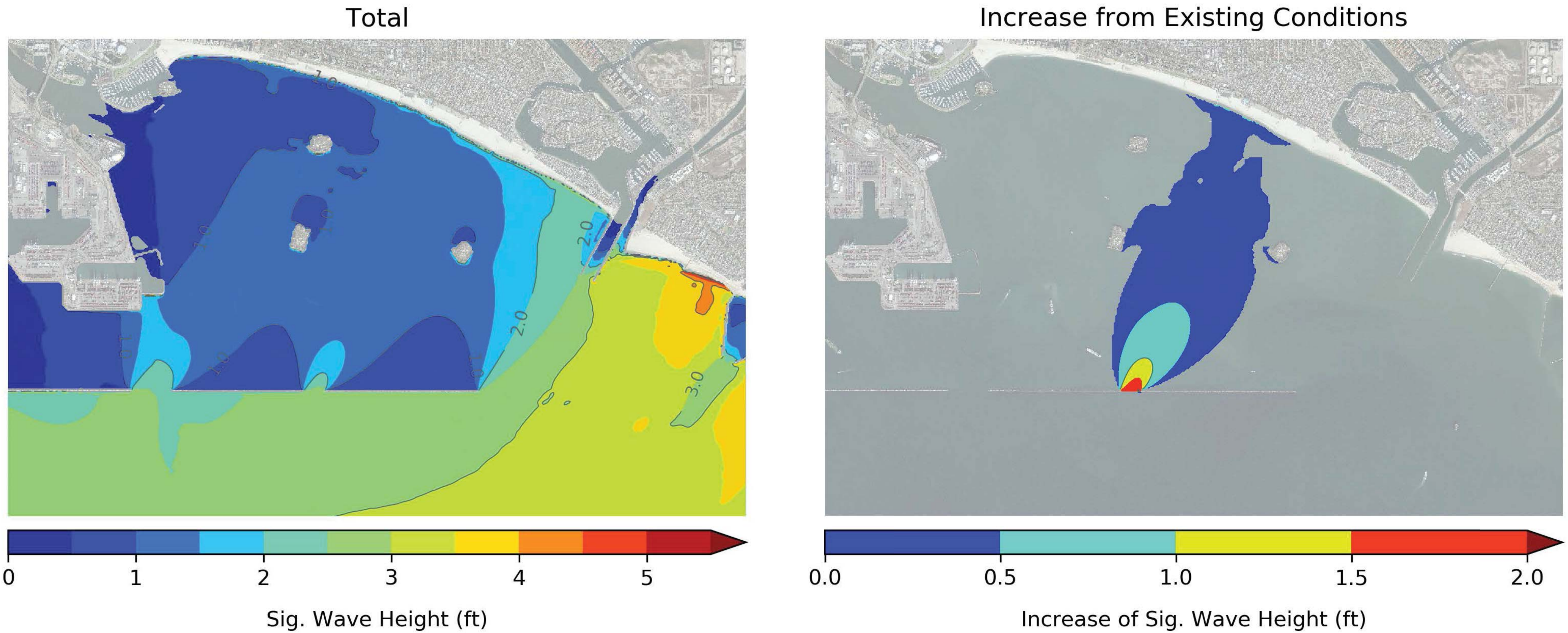




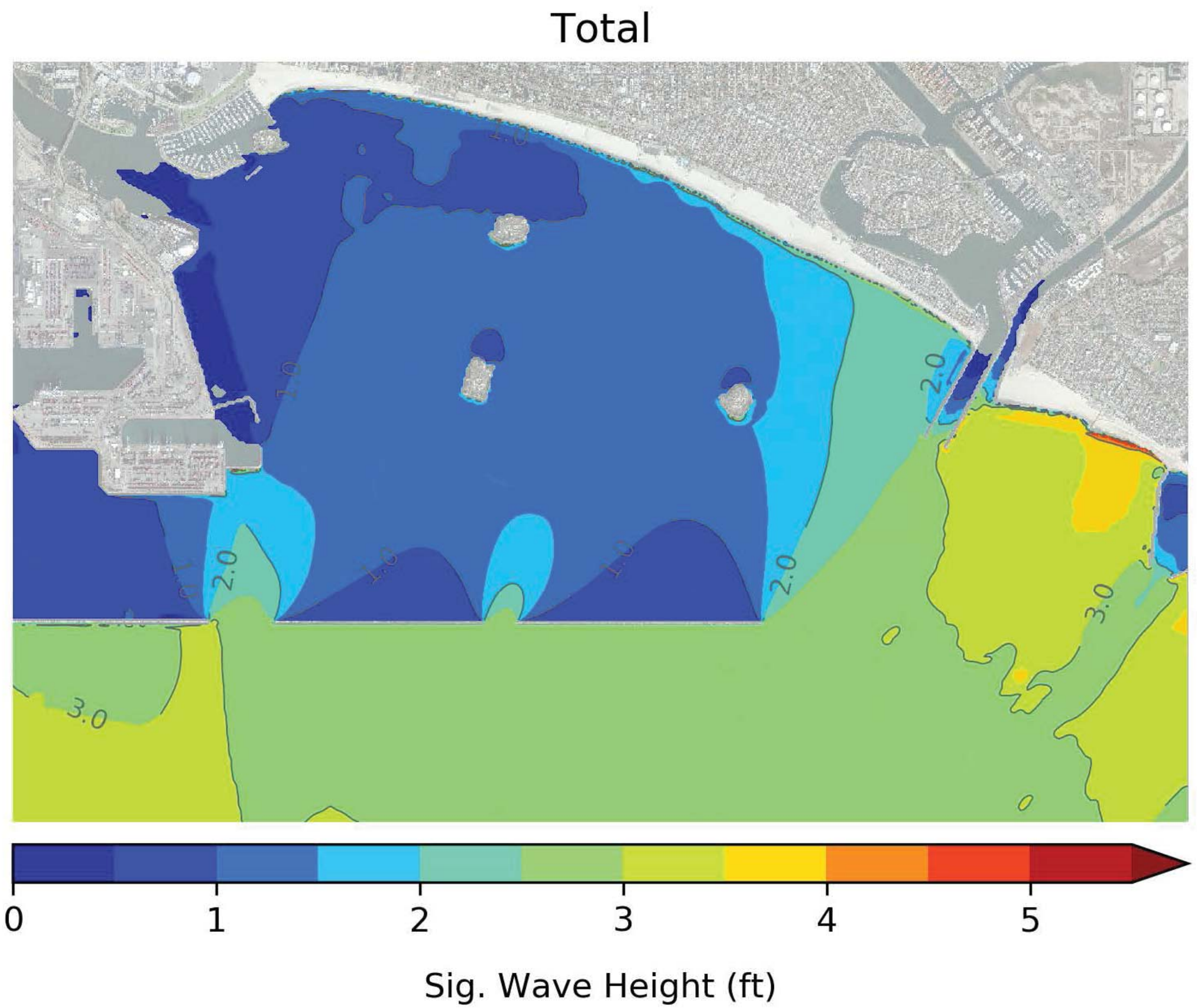
1x 1000' Western Notch: 100-yr S Swell

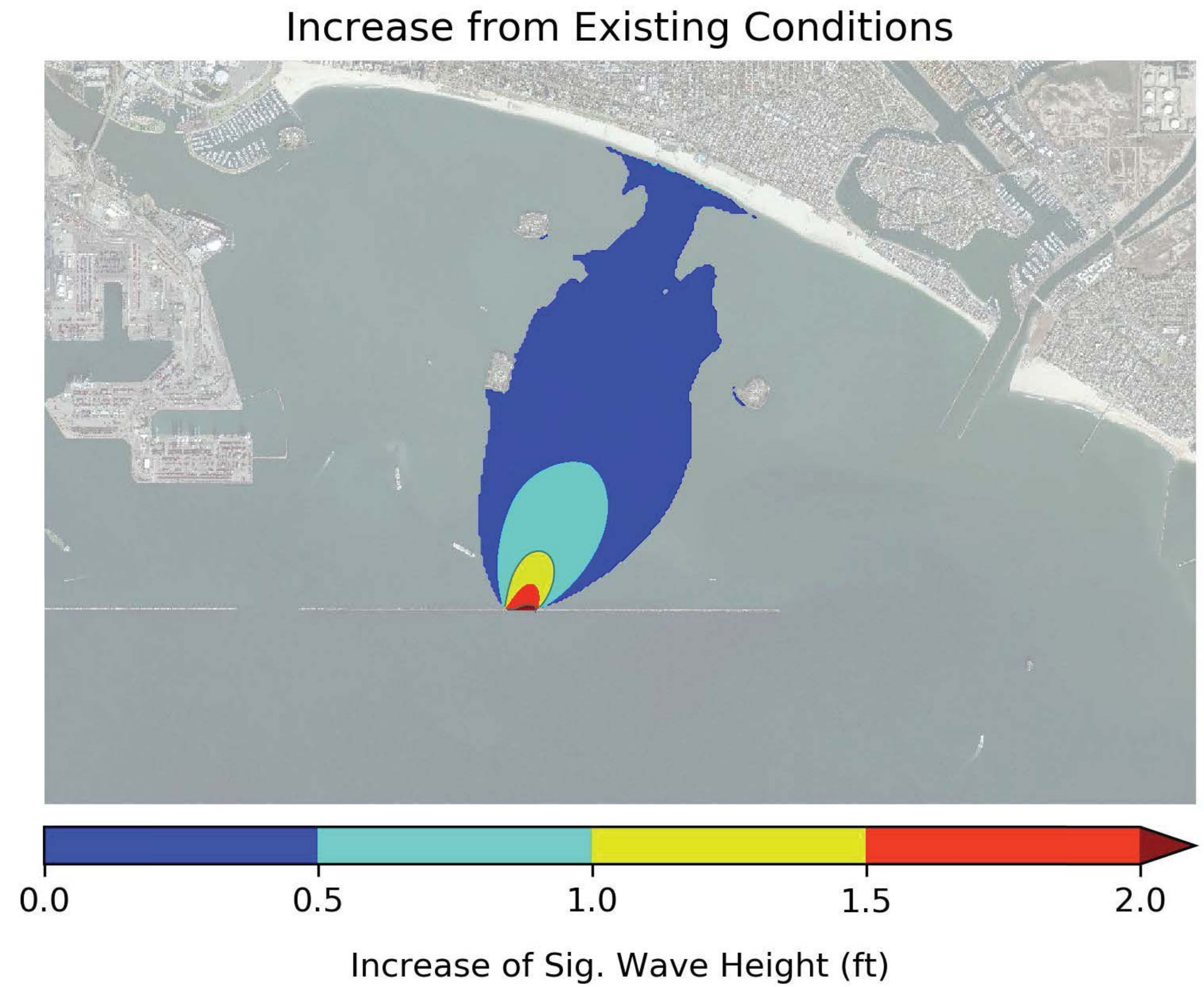


Center 1000' Notch: Typical Winter

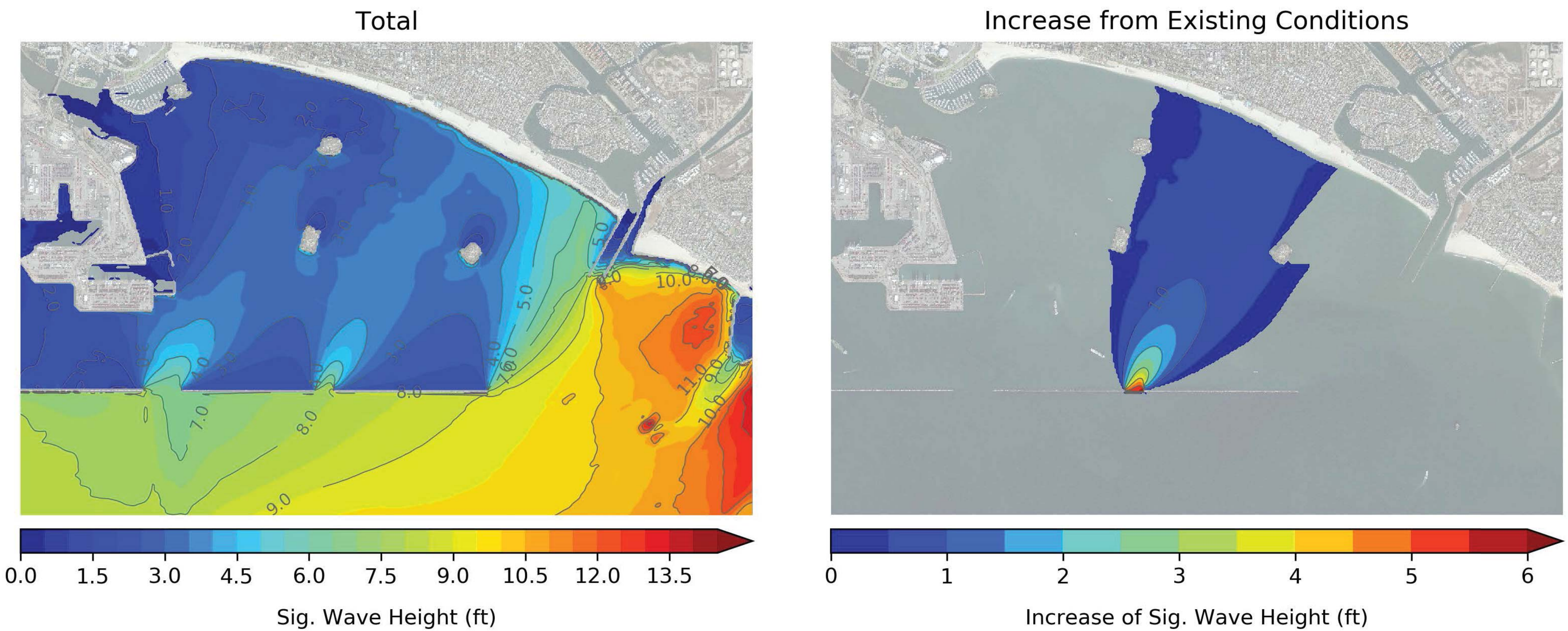


Center 1000' Notch: Typical Summer

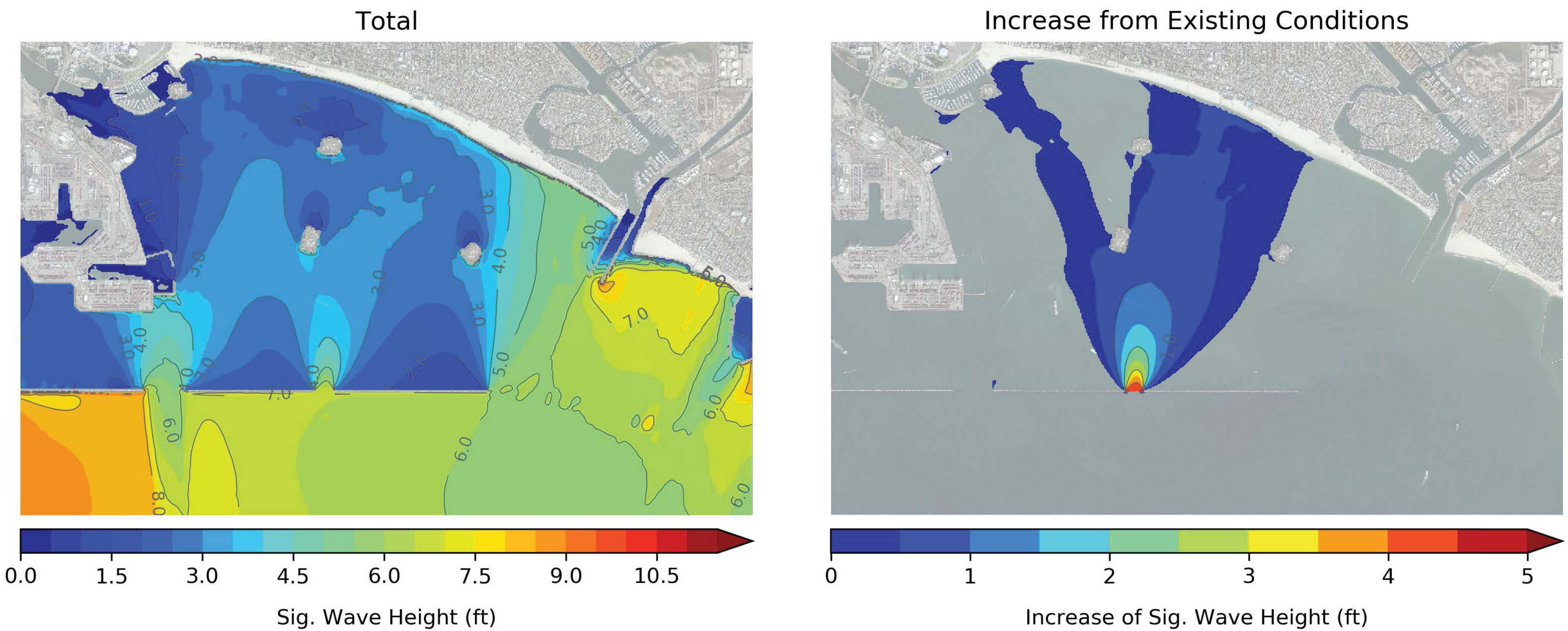




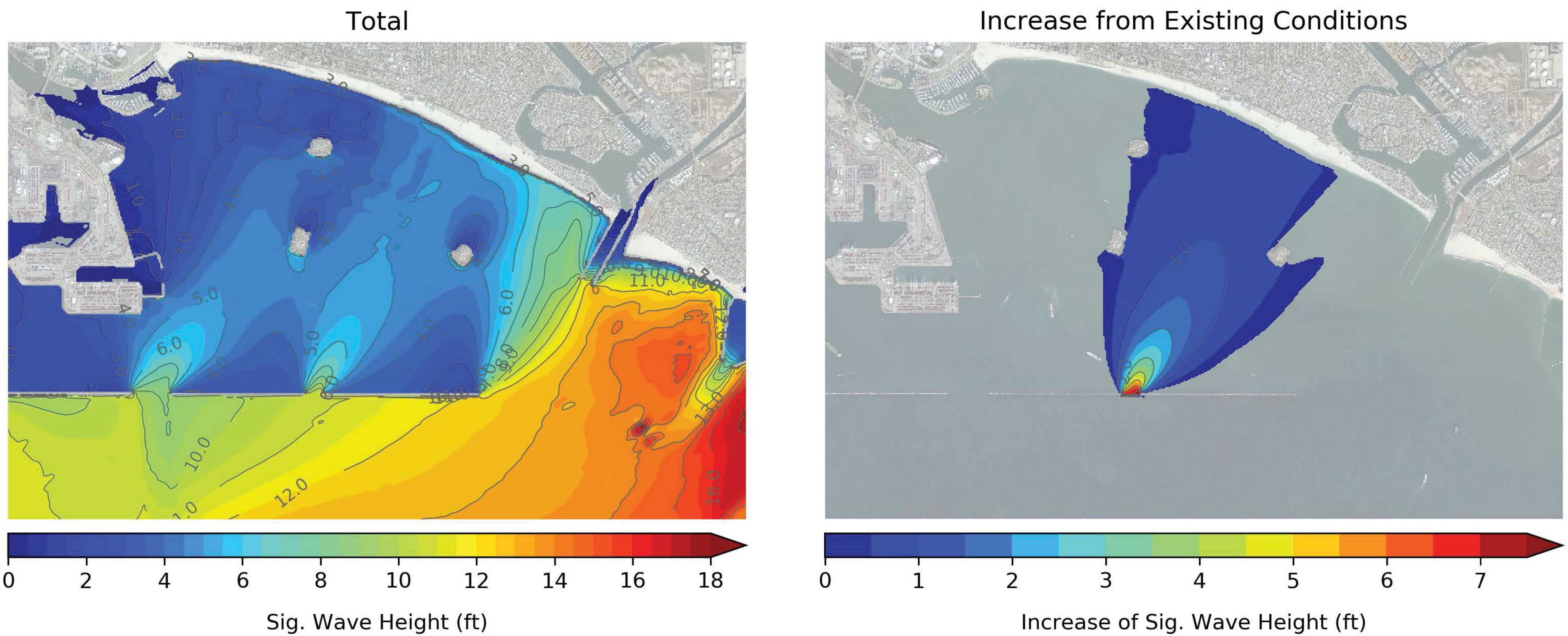
Center 1000' Notch: 1-yr NW Swell



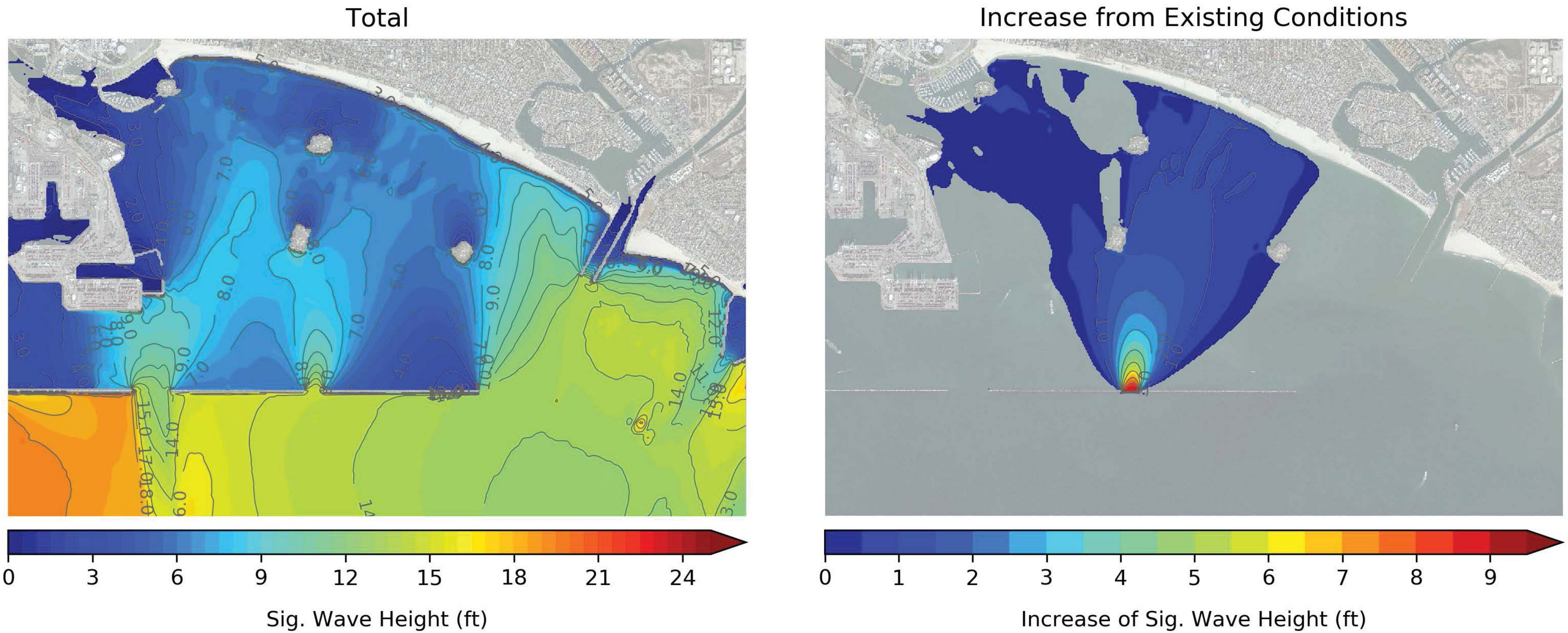
Center 1000' Notch: 1-yr S Swell



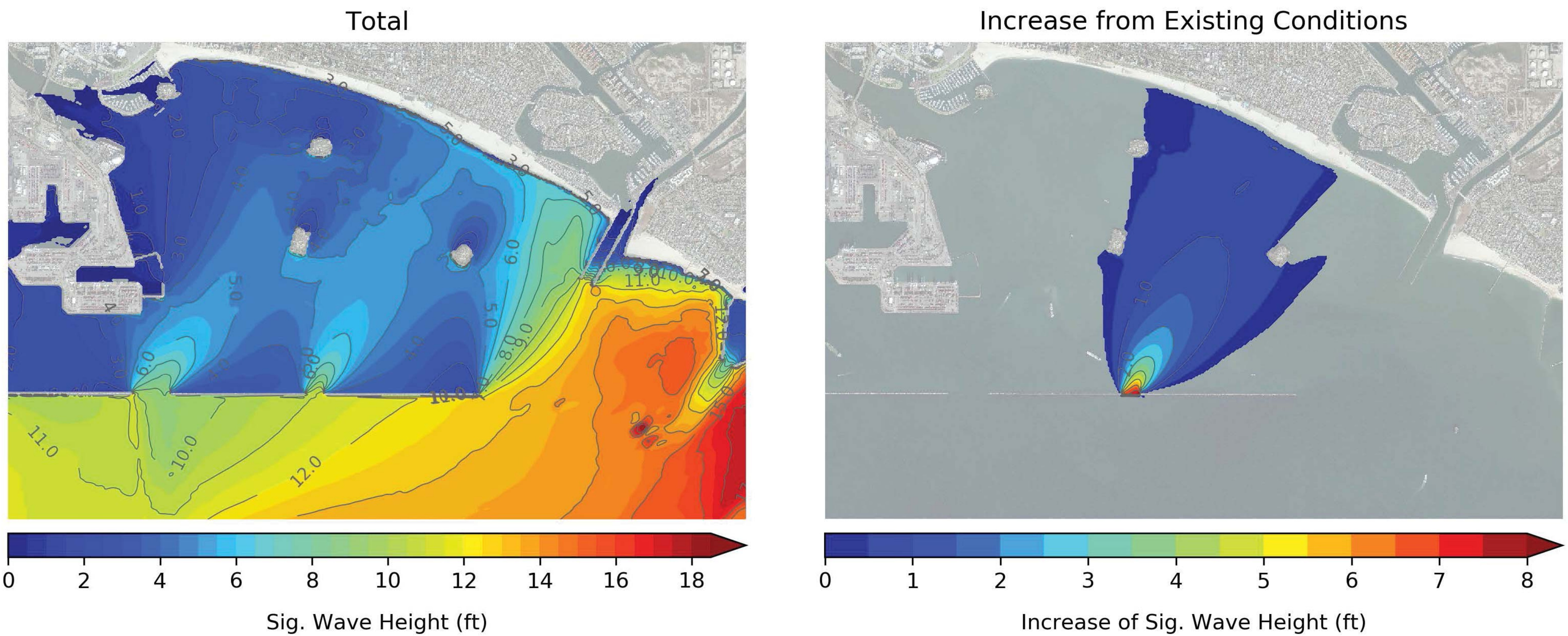
Center 1000' Notch: 50-yr NW Swell



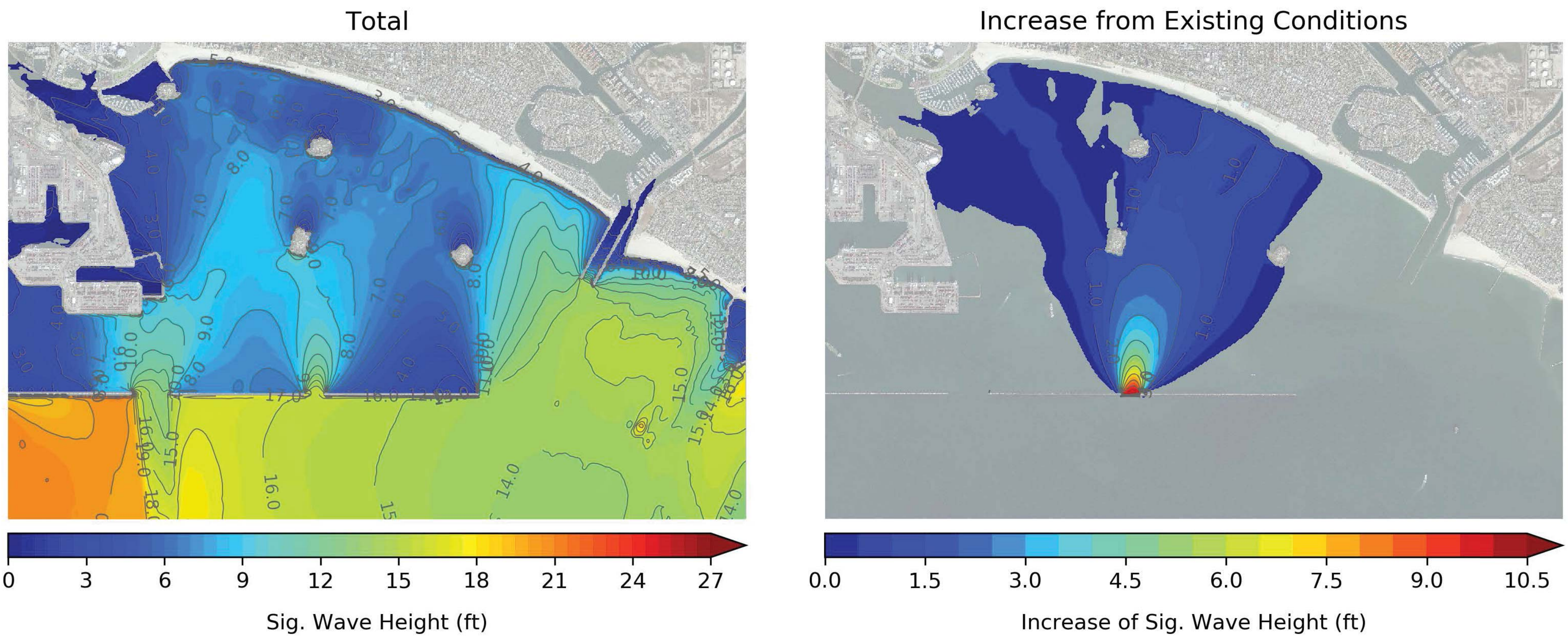
Center 1000' Notch: 50-yr S Swell



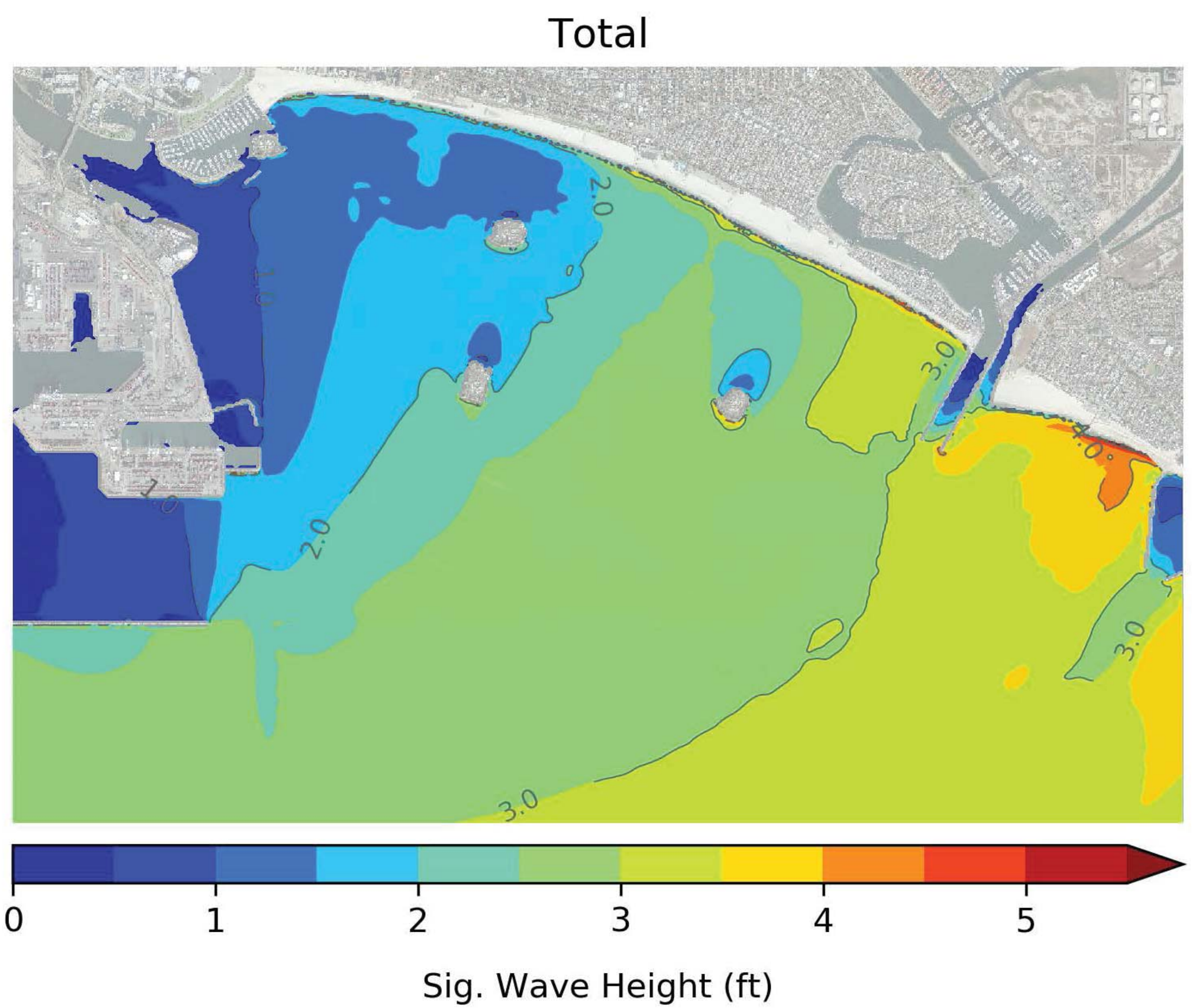
Center 1000' Notch: 100-yr NW Swell

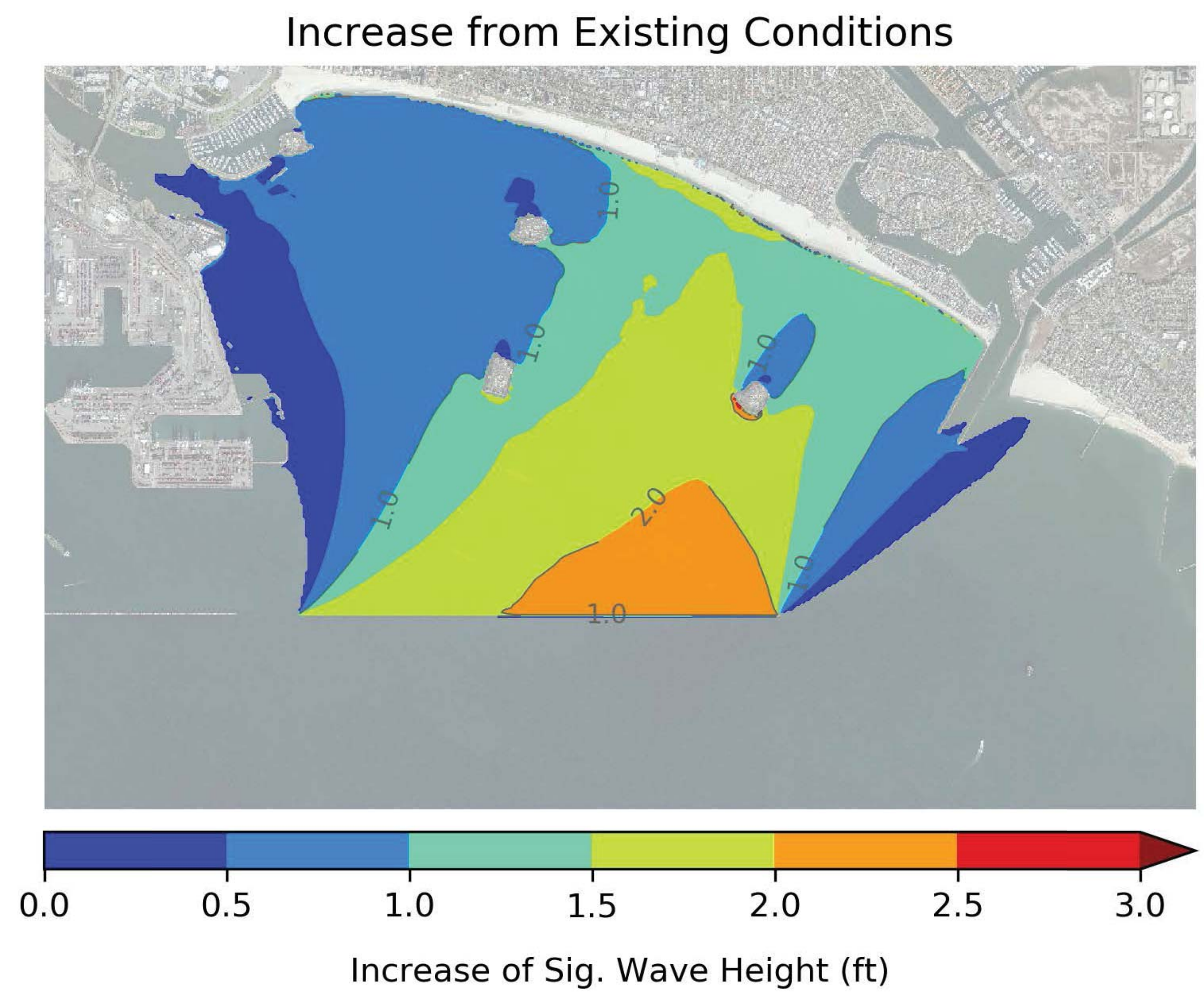


Center 1000' Notch: 100-yr S Swell

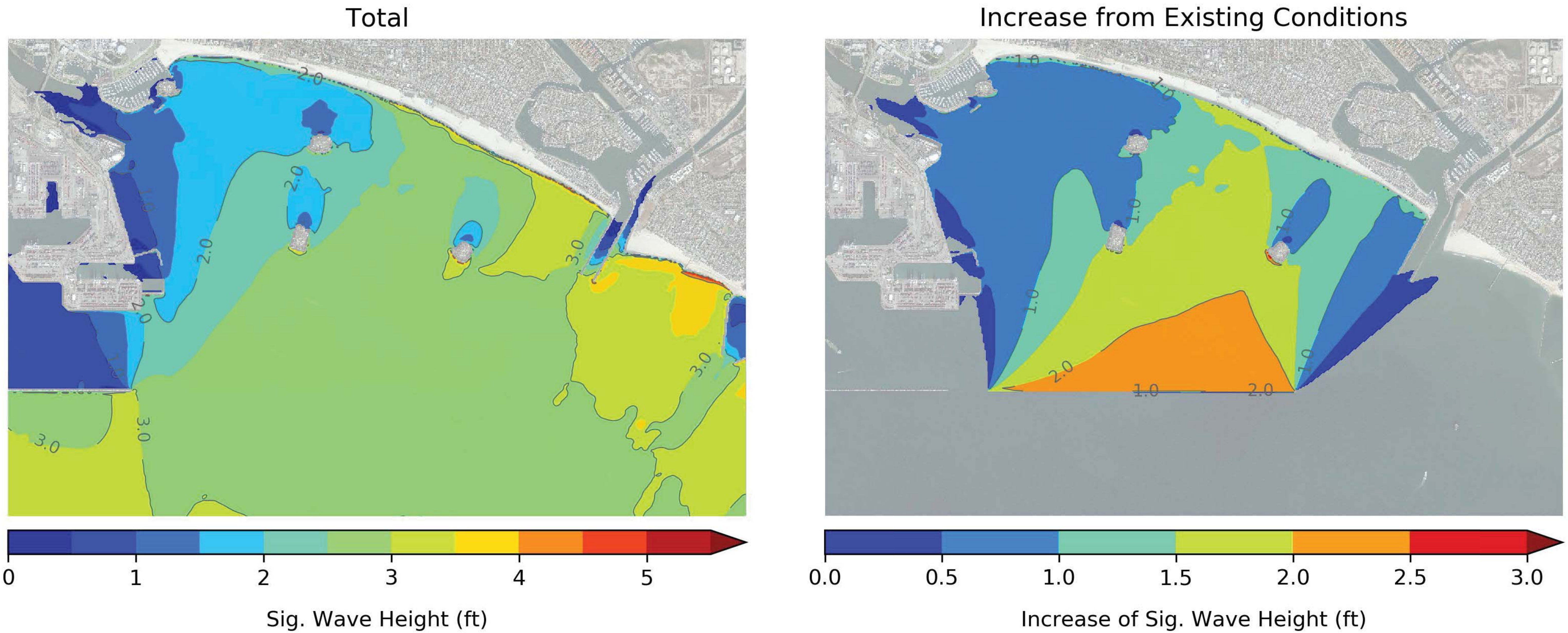


Entire Lower: Typical Winter

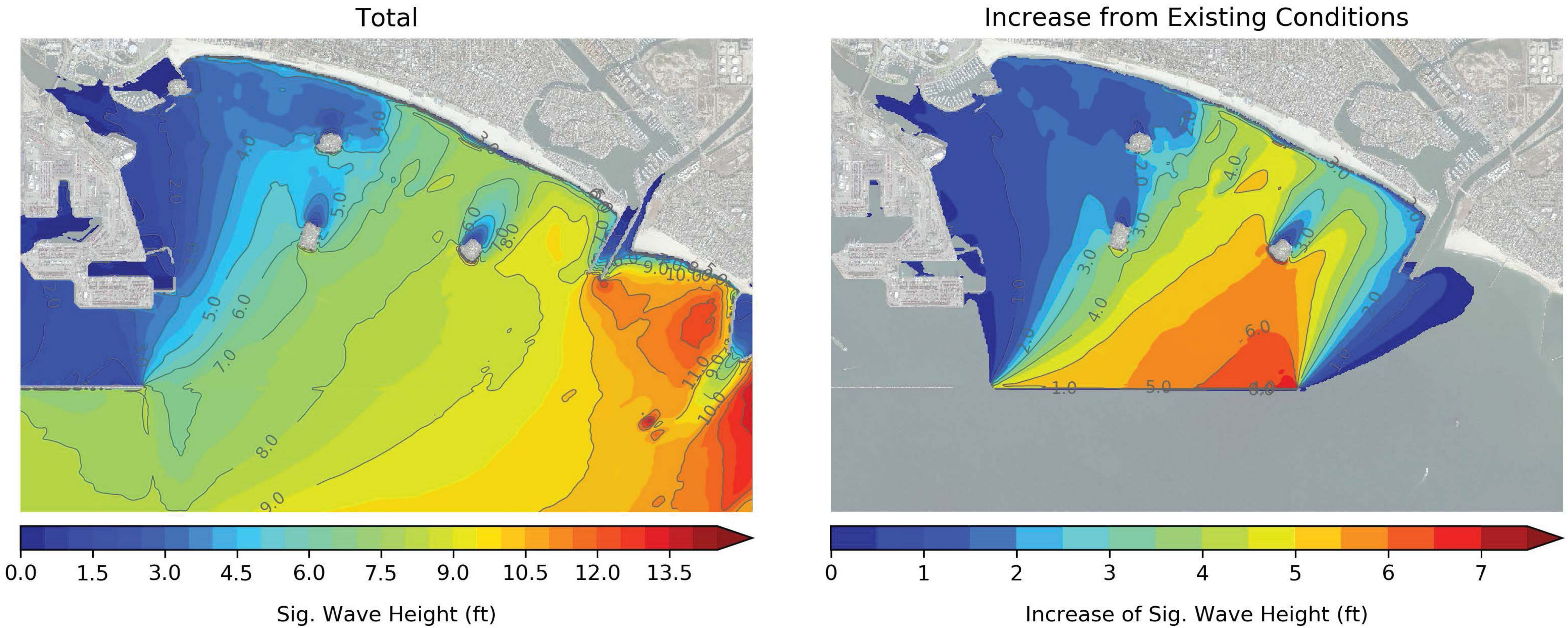




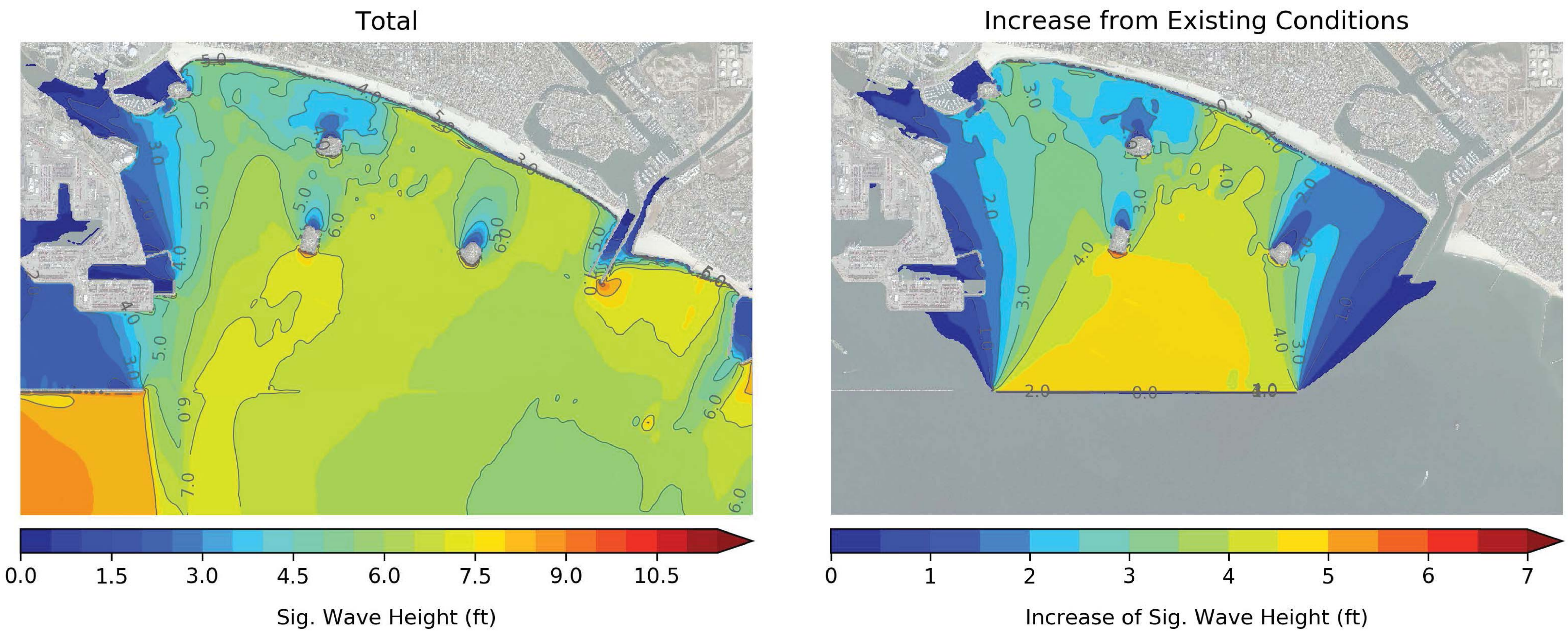
Entire Lower: Typical Summer



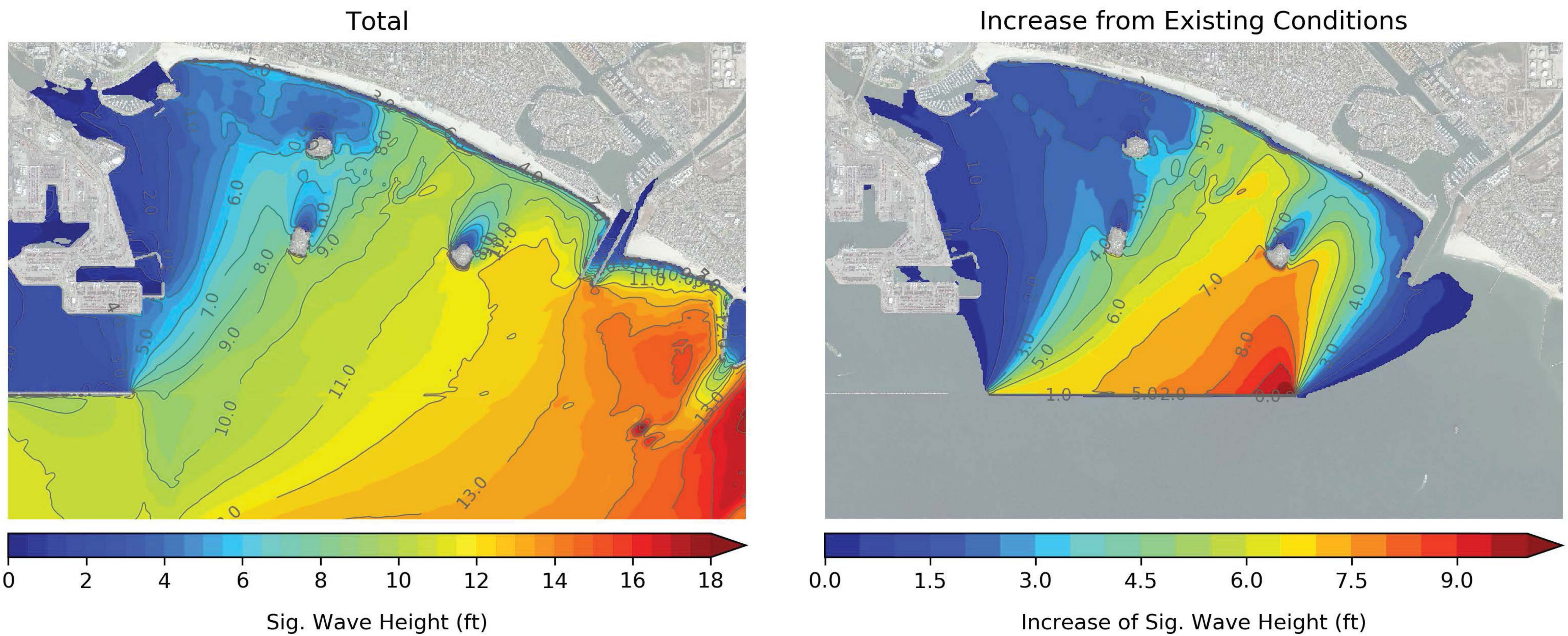
Entire Lower: 1-yr NW Swell



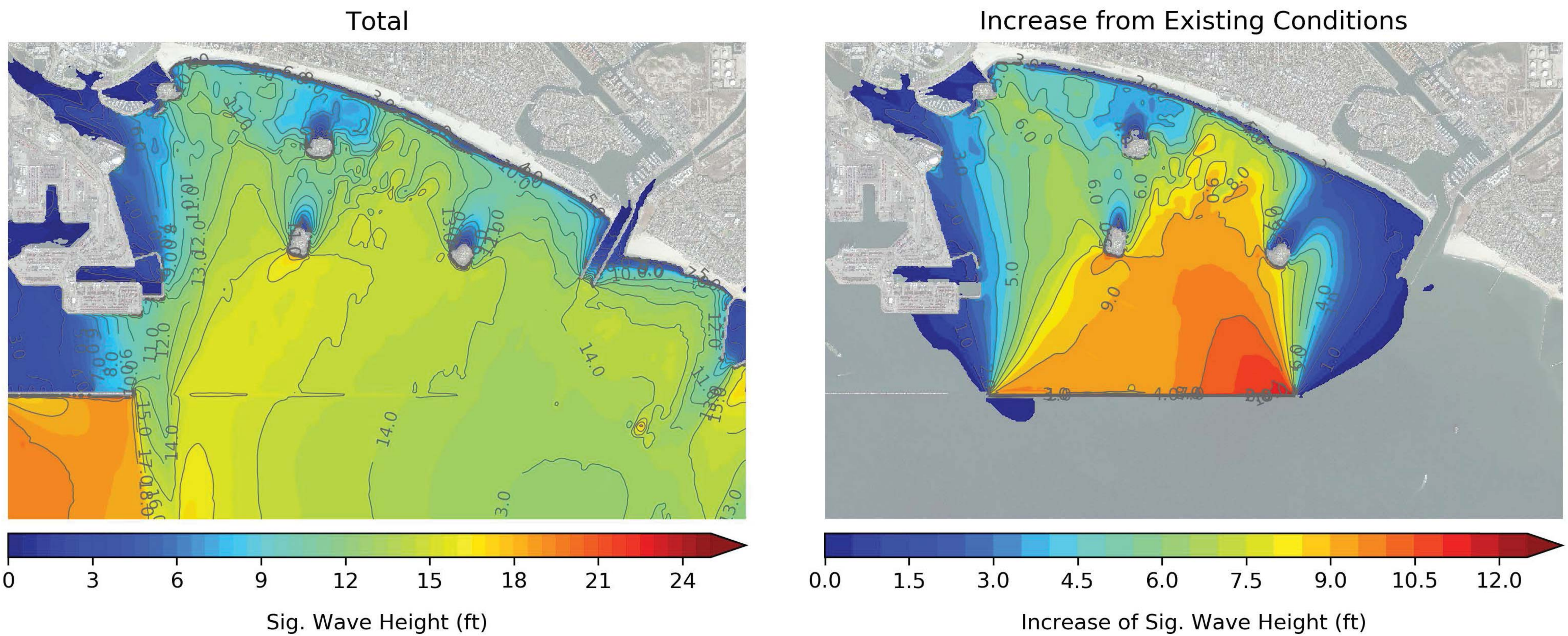
Entire Lower: 1-yr S Swell



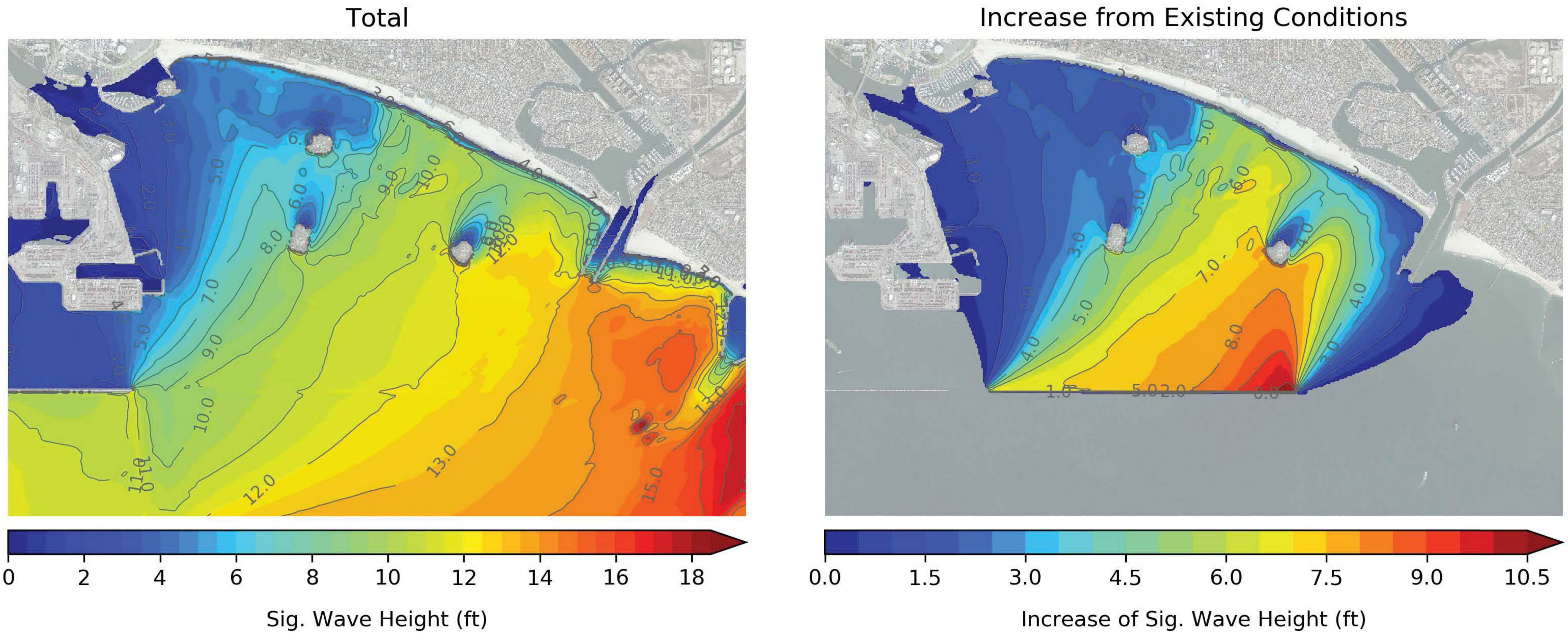
Entire Lower: 50-yr NW Swell



Entire Lower: 50-yr S Swell



Entire Lower: 100-yr NW Swell



Entire Lower: 100-yr S Swell