APPENDIX D: COASTAL STUDY

DRAFT

DECEMBER 2025
CITY OF BERKELEY

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

COASTAL STUDY

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DECEMBER 2025
CITY OF BERKELEY

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

COASTAL STUDY

PROJECT NO.

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Executive summary

This report documents the results of the coastal engineering study conducted for the Berkeley Water Transportation Pier Ferry (BWTPF) Project. The study supports the design and planning of resilient ferry infrastructure in San Francisco Bay, incorporating assessments of wave and wind conditions, extreme water levels, sea level rise (SLR), wave-induced downtime, and sediment transport processes. The analysis aims to ensure long-term performance and adaptability of the terminal under current and future climate conditions.

Basis of study

The environmental setting was characterized through analysis of tidal datums, wind and wave climates, and current patterns in the vicinity of Berkeley Pier. NOAA tide gauge data and outputs from regional hydrodynamic models provided the foundation for defining water levels and currents. Wind data from both long-term regional (1996–2023) and short-term local stations were evaluated, with limitations identified in the representativeness of local wind data for offshore wave generation. Therefore, regional wind records and hindcast datasets were prioritized for driving wave models.

Water level conditions were established based on:

- NOAA tidal datums (1983–2001 epoch)00
- AECOM's 2016 updated extreme tide elevations for the Bay, and
- The 2024 California Sea Level Rise Guidance, using the Intermediate scenario and a 50-year project design life.

The adopted design still water level includes:

- 1% AEP storm tide elevation of 10.3 ft (Berkeley MLLW),
- A sea level rise allowance of 1.6 ft,
- Resulting in a design still water level of 11.9 ft (Berkeley MLLW).

The SLR vulnerability assessment evaluated structural performance under four future scenarios, considering both 2075 and 2100 planning horizons under Intermediate, Intermediate-High and High SLR trajectories.

Wave modeling and diffraction simulations

A two-tiered numerical modeling approach was used to simulate wave propagation and wave disturbance within the terminal:

- MIKE 21 SW was applied to generate a 27-year time series of nearshore wave conditions, incorporating NOAA hindcasts, buoy data, and spatially variable wind forcing.

- MIKE 21 BW, a Boussinesq-type diffraction model, simulated wave behavior within the terminal for three breakwater configurations (250 ft, 300 ft and 400 ft). The model produced wave disturbance coefficients at the berths and plaza for multiple wave directions and periods.

Model results confirm that the inclusion of a breakwater is essential for reducing wave agitation at the ferry berths and shoreline. The comparative analysis of breakwater lengths shows that increasing the structure from 300 to 400 feet yields only marginal improvements in wave disturbance and operational downtime. The 250-ft length alternative was also analyzed, it provides noticeably less sheltering, particularly at the outer berth, resulting in slightly higher agitation levels. The 300-ft breakwater configuration provides sufficient sheltering for both berths, minimizes wave exposure, and achieves nearly the same reduction in downtime as the longer alternative. This supports the selection of the 300-ft breakwater as the recommended option. Design wave heights for this configuration were extracted at critical locations for use in minimum structures elevation evaluations.

Structures elevation design and overtopping assessment

Minimum crest elevations for the breakwater, pier deck, and shoreline plaza were defined using extreme wave conditions, overtopping formulas from EurOtop (2018), and SLR allowances:

- Breakwater: +17.6 ft (Berkeley MLLW), meeting overtopping threshold of 0.1 l/s/m under design conditions.

- Pier Deck: Max green water levels from 14.2 to 15.2 ft (Berkeley MLLW) informed the deck elevation range.

- Plaza: Final elevation set at +16.5 ft (Berkeley MLLW), exceeding the minimum requirement of 15.8 ft to ensure safety under current conditions and future storm events.

A detailed vulnerability assessment confirmed that under all but the most extreme sea level rise condition (High Scenario), the plaza, pier and breakwater maintain functionality and public safety.

Wave-induced downtime assessment

Berth operability was evaluated using the 27-year wave hindcast data. Downtime was defined by significant wave heights exceeding 1.5 ft, consistent with operational criteria

established in consultation with San Francisco Bay Ferry and the City of Berkeley. Key results include:

- The no-breakwater layout experiences seasonal peaks in monthly average downtime, exceeding 2% in summer months.
- The 250 ft, 300 ft and 400 ft breakwaters reduce average monthly and peak monthly downtime to below 0.1% and 2%, respectively.

Sediment transport and maintenance dredging

A coupled wave–current–sediment transport model was developed using MIKE 21 to assess sediment deposition under existing and proposed dredged conditions. The model was calibrated against observed sedimentation in the Berkeley Marina Channel. Key findings include:

- Over a six-year simulation, the average annual sedimentation rate was:
  - 0.20 feet per year in the approach channel
  - 0.09 feet per year in the berth pockets
- Based on these rates, estimated intervals for maintenance dredging are:
  - Channel: every 10 to 15 years, depending on operational depth (-9 or -10 ft MLLW)
  - Berth pockets: every 22 to 33 years

Conclusions

This integrated coastal engineering study confirms that the proposed terminal layout, including a 300-foot breakwater and design elevations for the deck and plaza, meets operational and resilience objectives. The design accounts for future sea level rise, ensures safe public access under storm conditions, and minimizes long-term maintenance requirements. The adopted methodology reflects current coastal engineering best practices and California coastal planning guidance, offering a solid foundation for project implementation.

1 Introduction

The Berkeley Water Transportation Pier Ferry (BWTPF) Project is a major waterfront infrastructure initiative intended to enhance waterborne transit access along the San Francisco Bay. The project (BWTPF) aims to revitalize a significant piece of waterfront infrastructure in Berkeley, California, transforming the existing pier into a modern, dual-purpose ferry terminal and public pier.

The project involves the design and implementation of a new ferry terminal at the foot of Berkeley Pier, including associated breakwaters, floating berths, a trestle, and shoreline plaza features. Given the dynamic coastal environment and long-term exposure to climate change, this project requires a thorough understanding of site-specific coastal conditions.

This report presents the findings of a detailed coastal engineering study conducted to inform the terminal design and evaluate its long-term performance. The study applies best-practice numerical modeling and coastal analysis methods to assess environmental forcing factors, determine minimum structural elevations, and evaluate sediment transport dynamics. It also considers future sea level rise projections, operational reliability, and design resilience in alignment with the latest 2024 California sea level rise guidance. Key components of the study include:

- Characterization of environmental conditions at the project site.
- Long-term spectral wave modeling and high-resolution diffraction modeling to evaluate wave transformation across the project area.
- Development of design still water levels using extreme tide conditions and sea level rise allowances consistent with the 2024 California Sea Level Rise Guidance.
- Calculations of minimum crest and deck elevations for breakwaters, deck, and shoreline infrastructure using wave overtopping and runup criteria.
- Evaluation of ferry berth downtime due to wave and wind conditions based on long-term hindcast data.
- Hydrodynamic and sediment transport modeling to assess the impact of the proposed dredging layout on sediment deposition and maintenance requirements.

The modeling results and vulnerability assessments serve to verify that the proposed terminal layout is resilient under both current and projected future conditions, including sea level rise and extreme weather events. The study supports the selection of a 300-ft breakwater as the recommended configuration, balancing effective wave protection with efficient use of resources. The findings provide a technical basis for design decisions, ensuring the terminal infrastructure will deliver reliable service and remain resilient under current and projected future conditions.

2 Basis of study

This section outlines the environmental parameters that form the basis for the coastal study. These conditions influence the physical processes affecting coastal infrastructure and natural systems. They are important for accurate assessments and modeling. The data and methodologies used to characterize water levels, currents, winds, and waves are presented herein, providing the foundation for analyses of coastal dynamics and design conditions.

2.1 Water levels

2.1.1 Tidal datum

Water levels were retrieved from the Berkeley tidal gauge (ID 9414816), which is the closest station to the project site, and reference information provided by the National Oceanic and Atmospheric Administration (NOAA) benchmark sheet. Table 2-1 presents the water levels recorded at the station which form the basis for our project site. Tidal data is per NOAA averages based on the 1983-2001 tidal epoch.

Note: City of Berkeley MLLW vertical datum differs from NOAA MLLW. This project adopted the City of Berkeley datum and refers to it as “MLLW” throughout this document. The conversion between datums is presented in Appendix A (eTrac, 2024).

Table 2-1 Tidal datum at Berkeley station, CA (ID 9414816)

Datum (NOAA)	Value [ft NAVD88]	Value [ft-Berkeley MLLW]	Description
HAT	7.8	8.4	Highest Astronomical Tide
MHHW	6.2	6.8	Mean Higher-High Water
MHW	5.6	6.2	Mean High Water
MSL	3.4	4.0	Mean Sea Level
MLLW	0.1	0.7	Mean Lower-Low Water
NAVD88	0.0	0.6	North American Vertical Datum of 1988
LAT	-2.1	-1.6	Lowest Astronomical Tide

2.1.2 Extreme water levels

A study done by (AECOM, 2016) - San Francisco Bay Tidal Datums and Extreme Tides Study - updated and expanded the USACE 1984 study of tidal datums and extreme tides for San Francisco Bay. The study modelled and presented detailed extreme tide information for the entirety of the Bay shoreline. The model captures the Bay's hydrodynamic conditions at a scale and resolution that exceeded available tide gauge data.

Table 2-2 reproduces the extreme tide elevation for point ID 519, coinciding with the Berkeley Pier location.

Table 2-2 Extreme tide elevation – ID 519 (AECOM, 2016)

Return Period (years)	Extreme Tide Elevation [ft NAVD88]	Extreme Tide Elevation [ft MLLW]
1	7.4	8.0
10	8.5	9.0
50	9.3	9.9
100	9.7	10.3

The extreme water level considered for this project is 10.3 ft MLLW, with a return period of 100 years (1% Annual Exceedance Probability - AEP).

2.1.3 Sea Level Rise (SLR)

The BWTPF Feasibility Study (GHD, 2023) was based on the 2018 California Sea-Level Rise Guidance, which relied on probabilistic projections informed by the IPCC Fifth Assessment Report (AR5). Since then, scientific advancements, including improved climate models, better ice-sheet dynamics understanding, and refined regional projections, have prompted an update. The 2024 California Sea Level Rise Guidance (OPC, 2024) integrates findings from the IPCC Sixth Assessment Report (AR6) and the Special Report on the Ocean and Cryosphere in a Changing Climate (SROCC), offering deeper insights into drivers of SLR such as thermal expansion, glacier melt, and ocean circulation shifts.

The updated guidance recommends a stepwise process for incorporating SLR into planning, ensuring adaptation efforts are methodical and responsive to new science. For the BWTPF Project, defining SLR allowances is critical for determining minimum structural elevations for the breakwater, pier, and shoreline plaza, ensuring resilience and safety throughout the infrastructure's design life.

The BWTPF Project adopts a 50-year design life (2025–2075) and uses the Intermediate Scenario from the 2024 California Guidance as the baseline for design. This scenario represents a precautionary yet practical estimate, balancing risk tolerance and cost-effectiveness.

Tide gauge selection

Projections from the San Francisco tide gauge indicate higher relative sea level rise compared to Alameda, which supports the decision to use it for setting minimum structural elevations. This conservative approach accounts for regional variability and potential underestimation of localized impacts if a lower-projection gauge were used.

As shown in Table 2-3, the differences between projections from the two gauges can be significant. For the year 2100, San Francisco projections exceed those of Alameda by approximately 40% under the low-end scenario and by about 5% under the high-end scenario. These variations underscore the importance of selecting a tide gauge that supports risk-averse design decisions, particularly for long-term infrastructure.

Table 2-3 Differences for Sea Level Rise allowance (Year 2100) for San Francisco and Alameda tide gauges – relative to year 2000

Sea Level Rise Scenarios for Year 2100, in feet					
Tide Gauge	Low	Int-Low	Intermediate	Int-High	High
San Francisco	1.0	1.6	3.1	4.8	6.5
Alameda	0.6	1.2	2.8	4.4	6.2
Difference	-40%	-25%	-10%	-8%	-5%

The design sea level rise for the project, given a 50-year design life (2025-2075) and Intermediate risk scenario, is 1.6 feet.

By adopting this scenario and tide gauge, the BWTPF Project ensures that critical infrastructure remains functional and safe through its intended lifespan without reliance on future retrofits, while aligning with state and regional climate resilience policies.

A more detailed description of the allowance selection and vulnerability assessment under more extreme scenarios (Intermediate-High and High) is provided in Section 5.5 and Appendix B.

2.1.4 Still water level

The still water level at the project location is shown in Table 2-4. This elevation excludes waves impacts but includes astronomical tides, storm surge and sea level rise allowance for the design life of the project.

Table 2-4 Design still water level

Parameter	Elevation [ft NAVD88]	Elevation [ft-Berkeley MLLW]
Extreme Water Level (1% AEP)	9.7	10.3
Sea Level Rise Allowance	1.6	1.6
Design Still Water Level	11.3	11.9

2.2 Currents

Current speeds in the vicinity of the Berkeley Pier are generally low, with variability influenced by tidal cycles and local wind conditions. While direct measurements in the immediate project area are limited, regional datasets suggest that nearshore current speeds are typically below 1 knot under normal tidal conditions. A detailed assessment of current patterns and speeds, including spatial and temporal variability, is presented in Section 6 and Appendix D, where a hydrodynamic model is used to simulate fields across the study area as one of the inputs for the sediment transport model.

2.3 Wind

2.3.1 Long-term wind data

The San Francisco Bay region is characterized by varied topography and water bodies which cause significant variations in the wind speed and directionality inside and outside the Bay area. Figure 2-1 shows annual wind roses at four measurement stations where data are available from 1997-2023 and indicates that the wind speed and directionality in the approaches to the project site can vary considerably from the offshore region to the inside of San Francisco Bay.

While the offshore region winds reach up to 30 knots and are mainly from WNW-NW direction, wind speeds measured inside the San Francisco Bay are generally 10-15 knots with significant variation in the directionality, depending on the location of measurements.

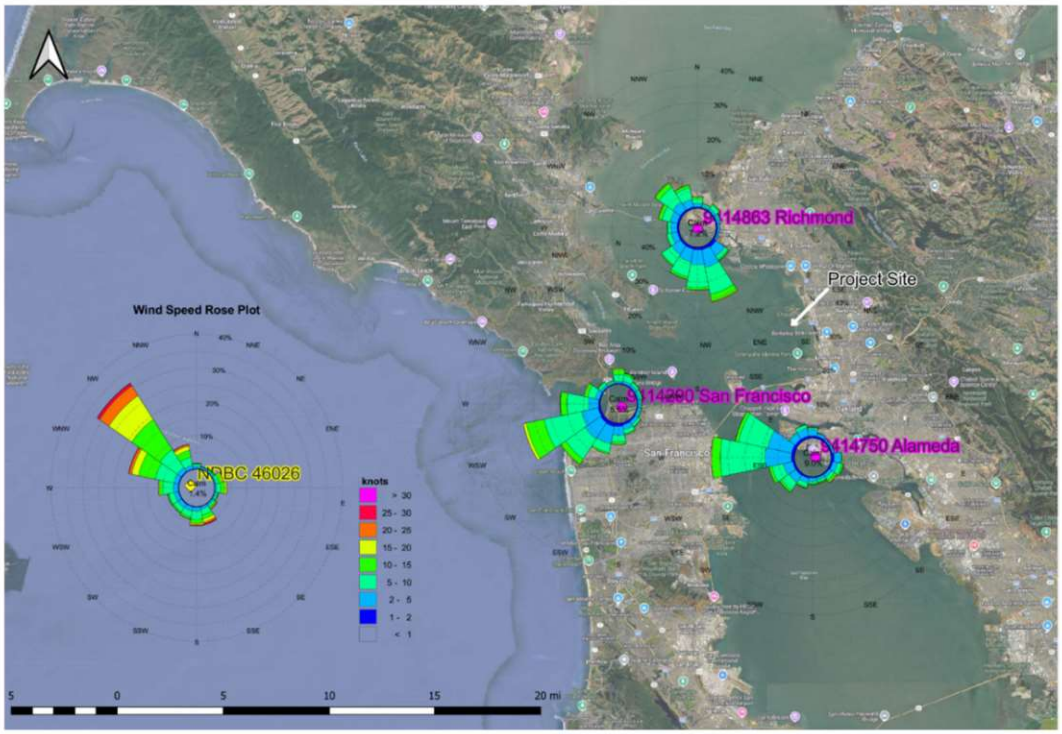


Figure 2-1 Wind roses from measured data in the vicinity of the project site.

Within San Francisco Bay, data from U.S. National Center for Environmental Information (NCEI)/ NOAA stations were analyzed. Station ID 9414290 – San Francisco is located west of the project site and Station ID 9414863 – Richmond is located north. Winds at San Francisco are seen to be predominately from SW-W directions (conforming to the geographical features of the area), whereas at Richmond, the winds are predominantly from S-SSE and from NW. At Station ID 9414750 – Alameda, located south of the project site, winds are from W-WNW directions.

Wind data from stations near the project site, shown in Figure 2-2, were also used to ascertain the variation in directionality near the project site.



Figure 2-2 Wind roses from measured data within the San Francisco Bay near the project site.

2.3.2 Local wind data

Local wind data were available from two stations, as shown in Figure 2-3 and Table 2-5.

At the Berkeley Marina station, winds are distributed across a broad directional range, with dominant flows from the southwest to west sectors. Average wind speeds are generally moderate, mostly within the 3- to 12-knot range, and the wide directional spread indicates significant influence from local shoreline orientation, urban development, and terrain effects. This variability reflects the complex wind environment near the shoreline, where small-scale features can disrupt the broader regional wind pattern. The Berkeley Marina dataset spans less than six years (Jan 2014 to Dec 2019), which is insufficient for establishing robust climatological trends or for use in statistical analyses, especially in support of wave modeling.

The Reef Light station has prevailing winds from the west-southwest to southwest. Compared to Berkeley Marina, Reef Light experiences a greater frequency of stronger winds (particularly those exceeding 12 knots) and exhibits a narrower directional spread for prevailing winds. While this indicates better alignment with regional wind patterns, it is still influenced by its proximity to land and the limited fetch available at the measurement location. This dataset is also limited to about one year (Nov 2018 to Dec 2019), which restricts its reliability for long-term wind characterization and for

estimating extreme wind events. The annual wind rose for the Berkeley Marina (left) and Reef Light (right) stations are shown in Figure 2-3.

Table 2-5 Period of measurements for local wind data

Station	Period of Measurements
Berkeley Marina	Jan-2014 to Oct-2019
Reef Light	Nov-2018 to Dec-2019

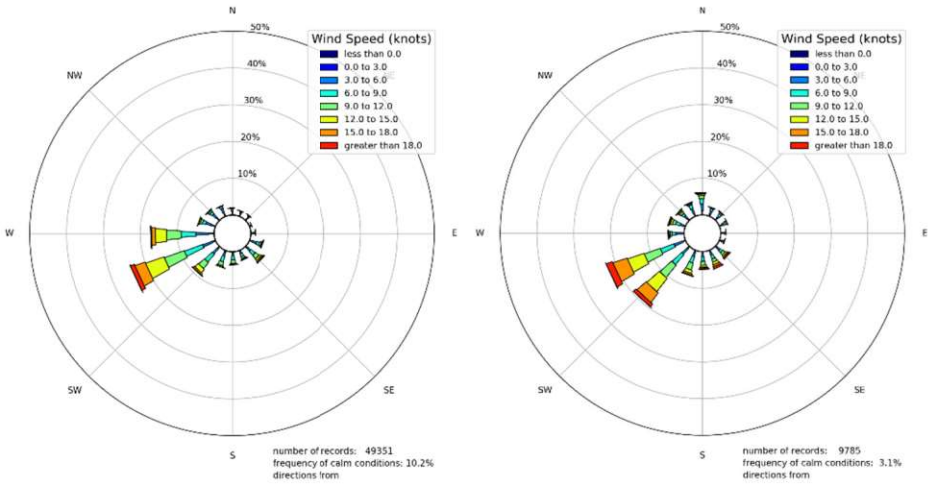


Figure 2-3 Annual wind rose for Berkeley Marina (left) and Reef Light (right).

Despite the differences between these two sites, neither station provides wind data that is fully representative of open-water conditions required for wind wave generation modeling. Both locations are constrained by limited fetch, short measurement durations, and proximity to complex coastal terrain, all of which reduce their suitability for driving wind wave models aimed at capturing large-scale wave dynamics. Their use should therefore be limited to local-scale assessments or for validating near-shore model behavior. In contrast, long-term offshore wind datasets should be prioritized for wave modeling and hindcasting.

2.3.3 Wind seasonality

Winds in the San Francisco Bay region show pronounced seasonal variability in both speed and direction, influenced by large-scale atmospheric circulation patterns and the region's complex topography.

The seasonal wind patterns were observed at long-term monitoring stations (San Francisco, Alameda, and Richmond) and short-term (Berkeley Marina and Reef Light).

Figure 2-4 to Figure 2-8 show the differences in wind roses between the month of June, the month of December, and the full year (annual) wind rose.

During the spring and summer months, the regional wind field is dominated by strong, persistent winds from the west to southwest, particularly at San Francisco and Alameda. These winds might be driven by the formation of inland thermal lows that enhance the onshore pressure along the California coast. Wind speeds during this period frequently exceed 12 knots, and in exposed areas such as San Francisco, can regularly reach or exceed 18 knots. This seasonal pattern is responsible for the most energetic wind forcing observed in the region and contributes substantially to wave generation and transport processes in the Bay and adjacent offshore areas.

Autumn and winter months are characterized by a weakening of the inland thermal gradient, leading to reduced wind speeds and greater directional variability. Across most stations, including local sites such as Berkeley Marina and Reef Light, wind roses show a broader spread during these months, with increased occurrences of southerly, southeasterly, and even northerly winds.

The Richmond station consistently deviates from the dominant west-to-southwest wind patterns observed at other locations. Across all seasons, Richmond displays a bimodal wind regime, with dominant winds from the S–SSE and N–NNW directions. This pattern is likely driven by the orientation of local topographic features and channeling effects within the northern Bay, highlighting the spatial heterogeneity of wind conditions across the region.

These seasonal trends have direct implications for wave modeling, coastal exposure assessments, and the planning of marine operations, and must be accounted for when evaluating wind-driven processes within the Bay and surrounding coastal waters.

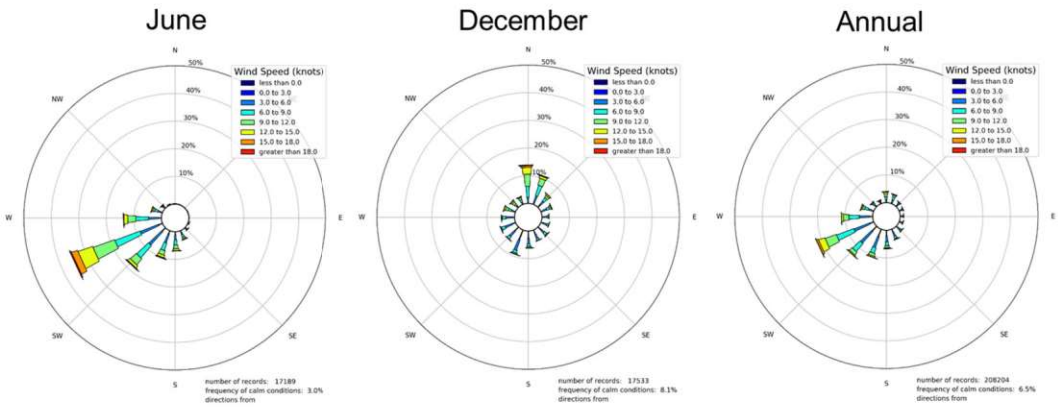


Figure 2-4 San Francisco Station wind roses (1997-2023) for June, December and the full year.

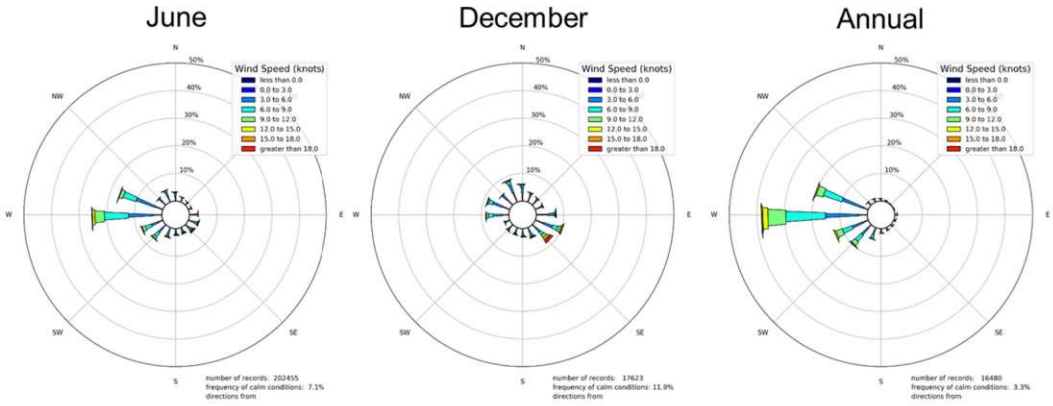


Figure 2-5 Alameda Station wind roses (1997-2023) for June, December and the full year.

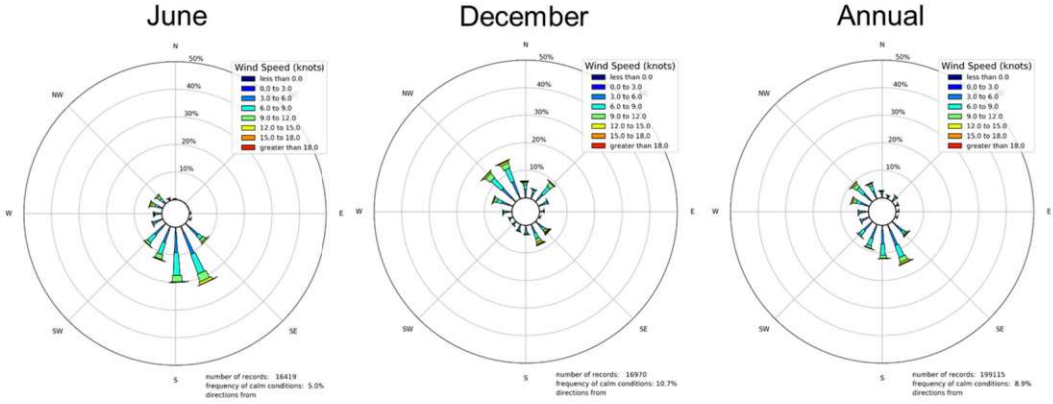


Figure 2-6 Richmond Station wind roses (1997-2023) for June, December and the full year.

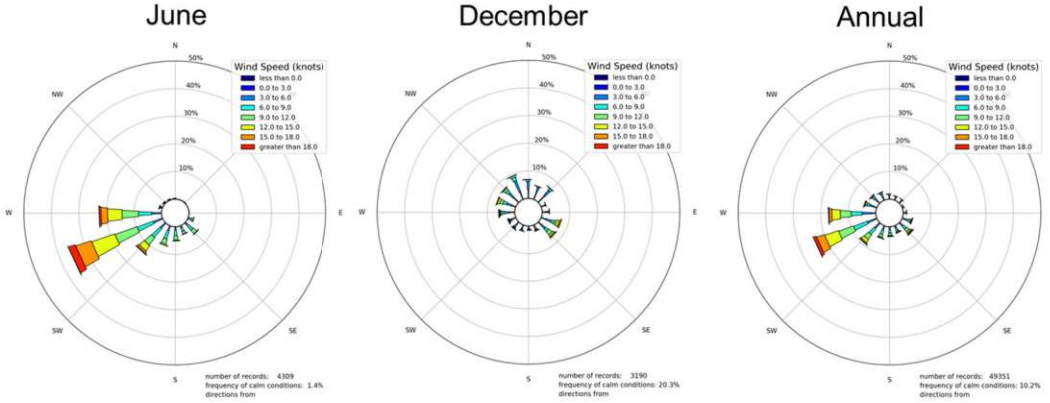


Figure 2-7 Berkeley Marina Station wind roses (2014-2019) for June, December and the full year.

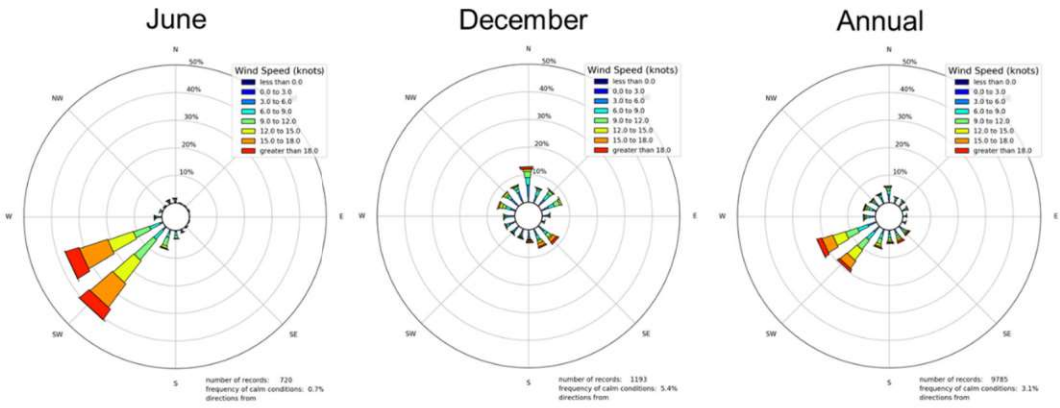


Figure 2-8 *Reef Light Station wind roses (2018-2019) for June, December and the full year.*

2.3.4 Extreme wind analysis

An extreme wind speed analysis was conducted using the 24-year dataset from the Alameda Station - Figure 2-1 and Figure 2-5. This station was selected for extreme analysis since it has the longest time series associated with the highest annual maxima. Weibull and Gumbel distributions were applied to the extreme values. Table 2-6 presents the estimated wind speeds for various return periods, along with the corresponding lower and upper bound values for the 95% confidence interval derived from the Weibull distribution (more conservative than Gumbel). It is important to note that the 100 and 200-year return period values carry lower confidence due to the limited length of the available time series.

Table 2-6 *Extreme wind analysis for Alameda station*

Return Period (years)	Extreme Wind Speed Central (Lower-Upper bounds) (knots)
10	35.0 (30.9 – 38.9)
25	37.9 (32.7 – 43.0)
50	39.8 (34.0 – 45.9)
100	42.0 (35.2 – 48.6)
200	43.7 (36.2 – 51.3)

2.4 Waves

Section 3 presents the methodology used to define the design wave conditions at the site.

3 Wave modeling

To define both operational and extreme wave conditions at the site, a two-tiered numerical modeling approach was adopted including a nearshore spectral wave model followed by a high-resolution diffraction model.

The nearshore wave model was used to simulate wave propagation and local wind-wave generation across the project area. The model accounts for key physical processes including wave growth due to wind forcing, refraction over variable bathymetry, shoaling, depth-induced breaking, and energy dissipation due to bottom friction. The resulting wave parameters at selected points were then used as inputs for the high-resolution diffraction model coefficients.

A diffraction model, based on the Boussinesq-type equations, was used to simulate wave transformations within the sheltered basin. This model accounts for complex phenomena including wave diffraction, reflection, nonlinear interactions, and partial wave blocking caused by structures. It was used to assess wave conditions at the berths, along the trestle, and at the plaza under the influence of the proposed breakwater.

3.1 Data basis

This section summarizes the metocean data sets used for this study, which include various data sets (hindcast, measurements) used for setting up the bathymetry and boundary conditions for numerical modeling.

3.1.1 NOAA WaveWatch III 30-year hindcast phase 2

NOAA WaveWatch III 30-year hindcast, provides wave climatology from 1979 to 2009. The wave model suite consists of global and regional nested grids. The parameters are available at 3-hour intervals. Spatially varying fields of significant wave height, peak wave period, and peak wave direction for total sea at 3-hour intervals were retrieved from the ‘West Coast of USA 4 minute (1/15 degrees)’ regional hindcast database.

3.1.2 NOAA WaveWatch III production hindcast

NOAA WaveWatch III production hindcast provides wave climatology from February 2005 to May 2019. The wave model suite consists of global and regional nested grids. The parameters are available at 3-hour intervals. Spatially varying fields of significant wave height, peak wave period, and peak wave direction for total sea at 3-hour intervals were retrieved from the ‘West Coast of USA 4 minute (1/15 degrees)’ regional hindcast database.

3.1.3 NOAA CO-OPS

NOAA's Center for Operational Oceanographic Products and Services (CO-OPS) is the authoritative source for accurate, reliable, and timely tides, water levels, currents, and other coastal oceanographic and meteorological information. CO-OPS maintains ocean observing infrastructure, including more than 200 permanent water level stations on the U.S. coasts and Great Lakes, an integrated system of real-time sensors concentrated in busy seaports, and temporary meters that collect observations for tidal current predictions.

Table 3-1 shows the stations, the data types and the periods for which data were extracted near the project site. Locations of the CO-OPS stations are shown in Figure 3-1.

Table 3-1 NOAA CO-OPS stations and data description

Station ID	Station Name	Data Type	Period
9414290	San Francisco	Wind	1996-2024
9414750	Alameda	Wind and Hourly Water Levels	1996-2024
9414863	Richmond	Wind	1996-2024

Tidal datums obtained at the three CO-OPS stations are provided in Table 3-2.

Table 3-2 Tidal datums at CO-OPS stations within the study region (in feet-MLLW).

Datum	Station 9414290 San Francisco	Station 9414750 Alameda	Station 9414763 Richmond
MHHW	5.8	6.6	6.0
MHW	5.2	6.0	5.4
MSL	3.1	3.4	3.3
MLW	1.1	1.1	1.1
MLLW	0.0	0.0	0.0
NAVD88	-0.1	0.2	0.0

3.1.4 NOAA NDBC

NOAA's National Data Buoy Center (NDBC) maintains a network of marine observation and monitoring infrastructure which collect and disseminate in-situ, real-time, quality-controlled observations in the marine environment.

Table 3-3 shows the stations, the data types and the periods for which data were extracted. Locations of the NDBC stations are shown in Figure 3-1.

Table 3-3 NOAA NDBC stations and data description

Station ID	Station Name	Data Type	Period
46026	San Francisco Offshore	Wind, Significant Wave Height, Peak Wave Period, and Peak Wave Direction	1982-2024
46237	San Francisco Bar	Significant Wave Height, Peak Wave Period, and Peak Wave Direction	2007-2024

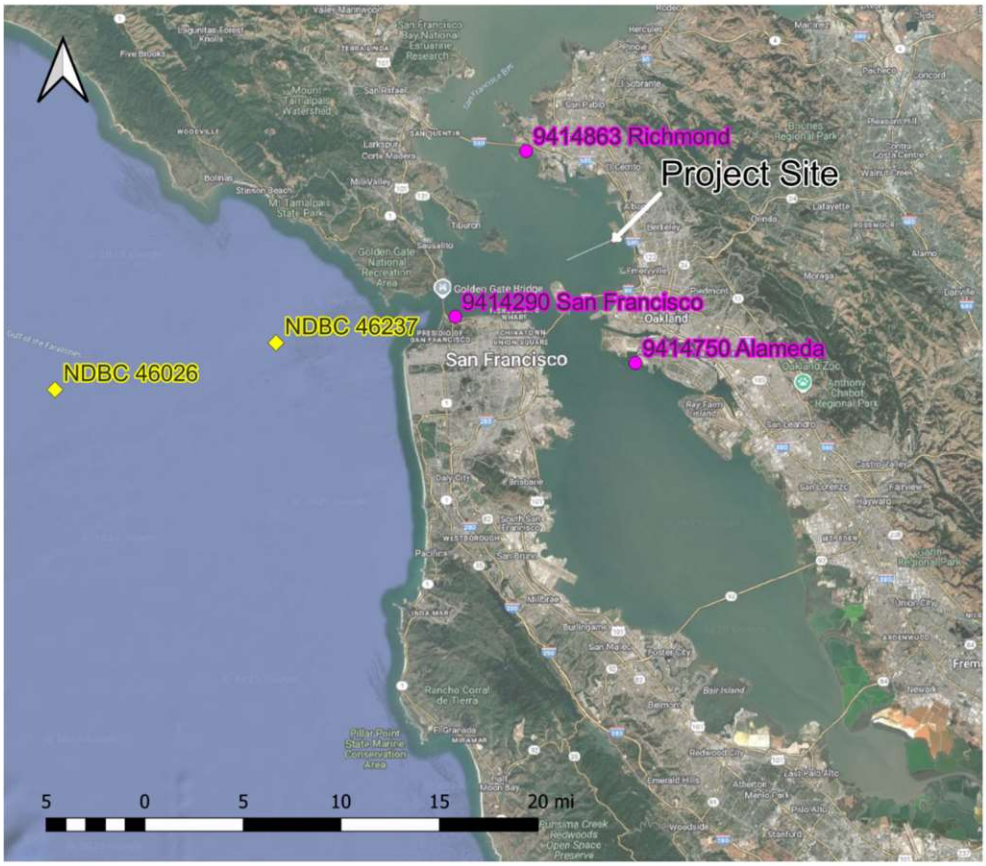


Figure 3-1 NOAA CO-OPS and NDBC wind, water levels, and wave measurement locations.

3.2 Bathymetry

The model bathymetry for the wave transformation study was created based on the USGS DEM for nearshore regions and survey data. Details of applied bathymetry data are given below:

› **USGS DEM** (2024): A digital bathymetric model with 1/3-arc-second (approximately 10-m) resolution, derived from diverse source data. The DEM covers the conterminous United States, Hawaii, Puerto Rico, other territorial

islands, and limited areas of Alaska. See Figure 3-2 for extent of coverage available within the model domain used in this study.

› **Local bathymetric survey** (2024): Bathymetry data for the site (Figure 3-3) was produced by eTrac Engineering Inc, a Woolpert Company for COWI. Survey data was collected on July 24, 25, 27 and 29, 2024 and plotted on August 22, 2024.

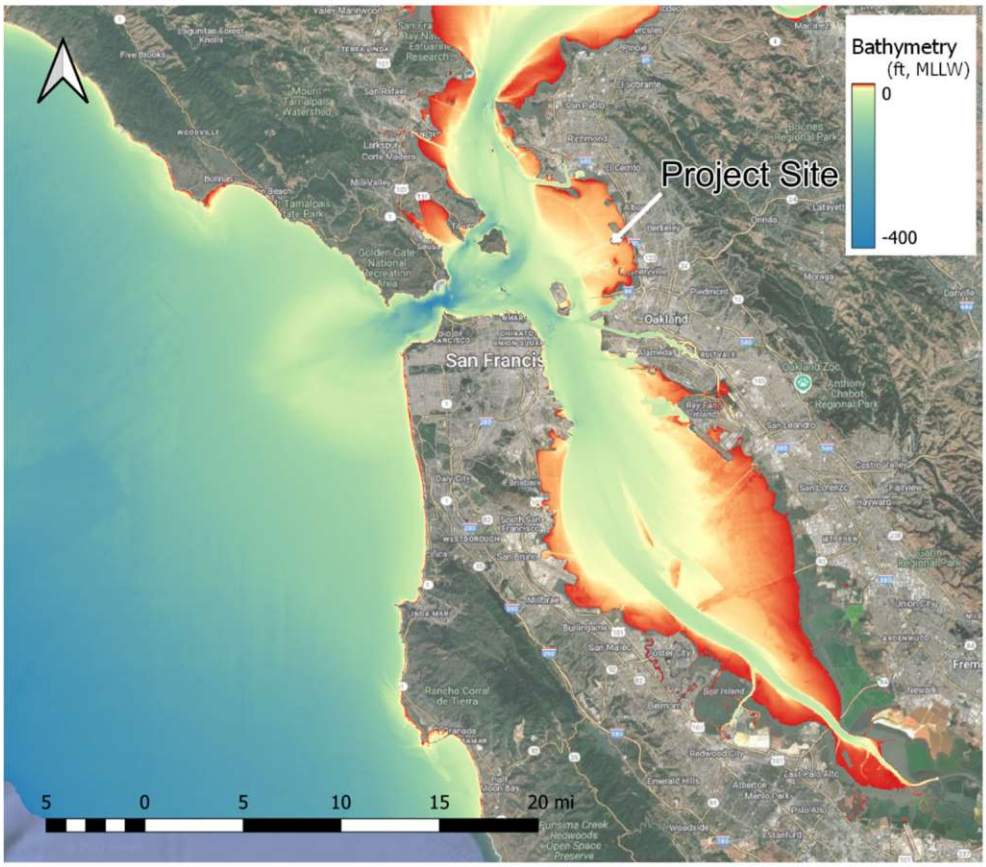


Figure 3-2 USGS DEM.

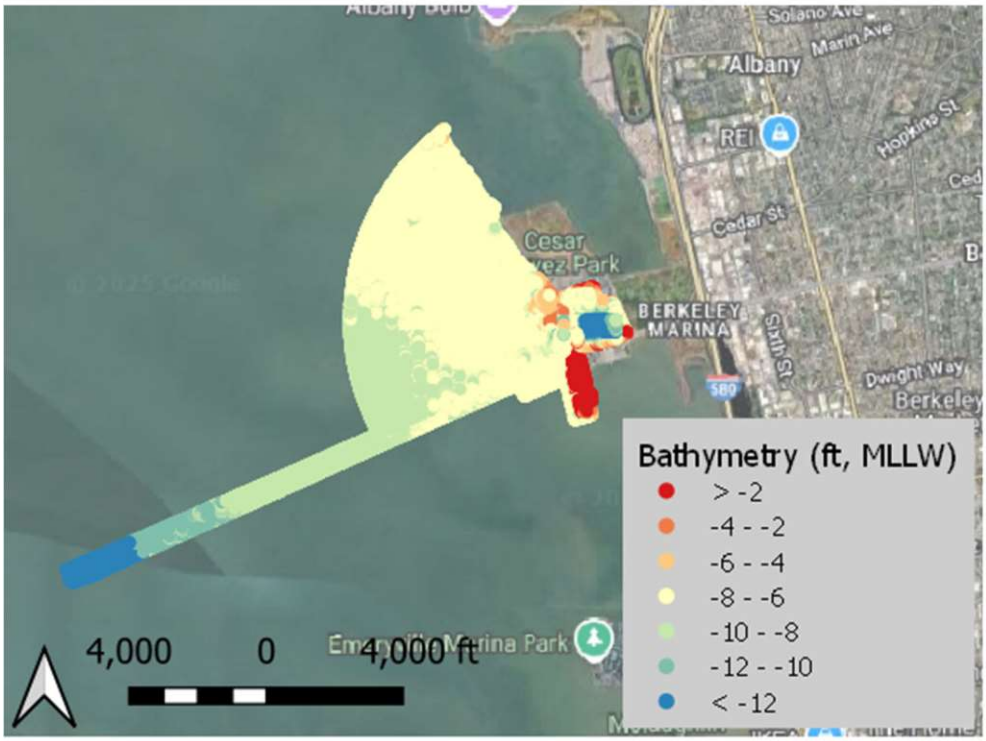


Figure 3-3 Local bathymetric survey from eTrac.

3.3 Nearshore wave modeling

A numerical nearshore wave model was conducted to obtain both operational and extreme wave conditions at locations in front of the breakwater as well as to derive the boundary conditions required as input for the tranquility (diffraction) model.

3.3.1 Model description

Numerical modeling of the wave conditions was performed using the Spectral Wave (SW) module of the comprehensive 2-dimensional MIKE 21 modeling suite from DHI, Denmark. The model is based on a flexible mesh, and hence very fine resolution bathymetries, where necessary, can be incorporated with this model.

MIKE 21 SW is a third-generation time-dependent spectral wind-wave model based on unstructured meshes. It simulates growth, propagation, decay, and transformation of wind generated waves and swells in offshore and coastal areas. The model includes wave growth by action of wind, interaction between waves with different frequencies and dissipation due to white capping. Furthermore, the model includes shallow water effects like refraction and shoaling due to varying depth and dissipation due to depth limited wave breaking and bottom friction.

The applied MIKE 21 SW model requires the following main inputs.

- Bathymetry of the area.
- Wind condition (wind speed and direction, or wind velocity components).
- Bottom friction parameters.
- Wave breaking parameters.
- Water level conditions.

3.3.2 Nearshore wave modeling methodology

The following methodology was adopted to develop a numerical wave model to simulate wave conditions for January 1997-December 2023:

- Bathymetry: Develop a mesh for a numerical wave model using various bathymetry data sets described in Section 3.2.
- Boundary conditions – waves:
  - Wave conditions for the January 1997 – December 2009 period were obtained from NOAA WaveWatch III 30-year Hindcast Phase 2 (Section 3.1.1).
  - Wave conditions for the January 2010 – December 2018 period were obtained from NOAA WaveWatch III Production Hindcast (Section 3.1.2).
  - Wave conditions for the January 2019 – December 2023 period were obtained from NOAA NDBC Buoy 46026 (Section 3.1.4).
- Boundary conditions – winds: Wind speed and direction time series from NOAA NDBC Buoy 46026, CO-OPS Stations 9414290, 9414750, and 9414863 were used to create of a spatial wind field mosaic using the extents shown in Figure 3-4. The mosaic thus created was used as wind forcing. See Section 2.3 for wind data and directionality analysis.
- Initial conditions – water levels: Hourly time series of measured water levels at NOAA CO-OPS Station 9414750 was used to specify water level variation.



Figure 3-4 Polygons used to create spatial wind field mosaic.

3.3.3 Model bathymetry

The model domain and mesh established for the wave transformation study is presented in Figure 3-5. The model domain includes the entire southern and central portions of San Francisco Bay and extends north up to San Pablo Bay (excluding). The offshore boundaries are located near the NDBC Buoy 46026.

The domain is divided into several zones with different triangular mesh resolutions. The spatial resolution of the mesh varies from an average element size of ~1-1.5 km in the region outside San Francisco Bay, transitioning gradually to a minimum of 150 m within the project site. Figure 3-6 shows the model bathymetry.

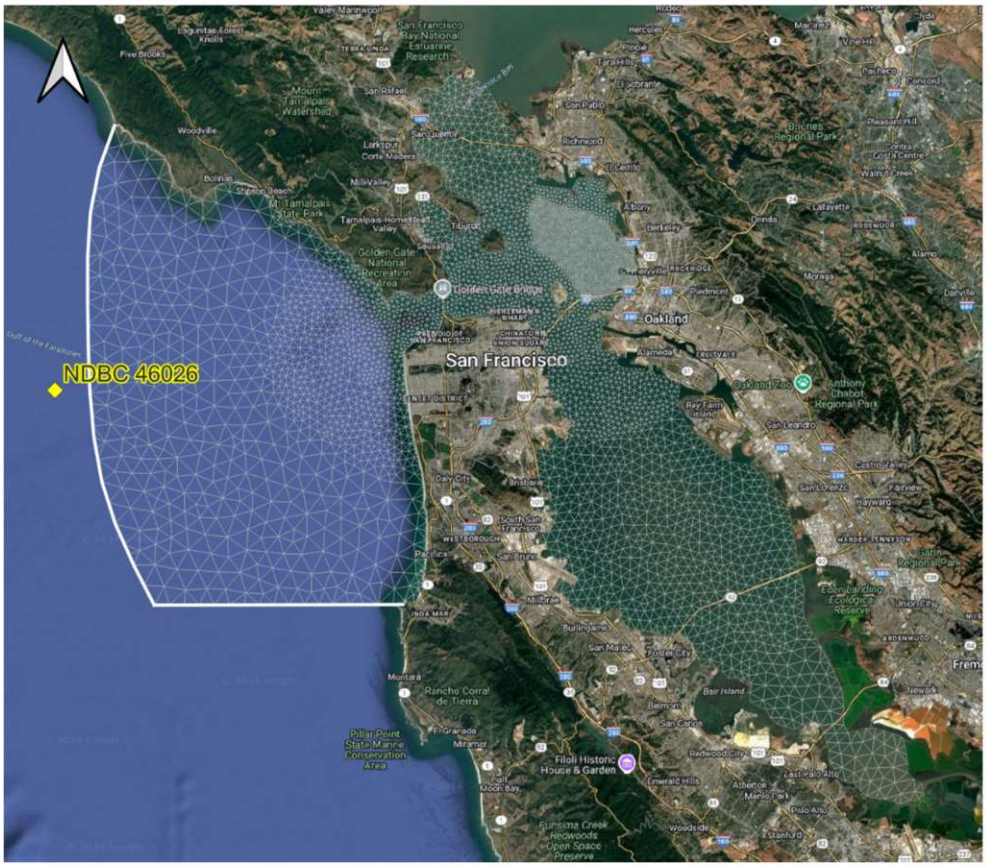


Figure 3-5 *Model domain and mesh.*

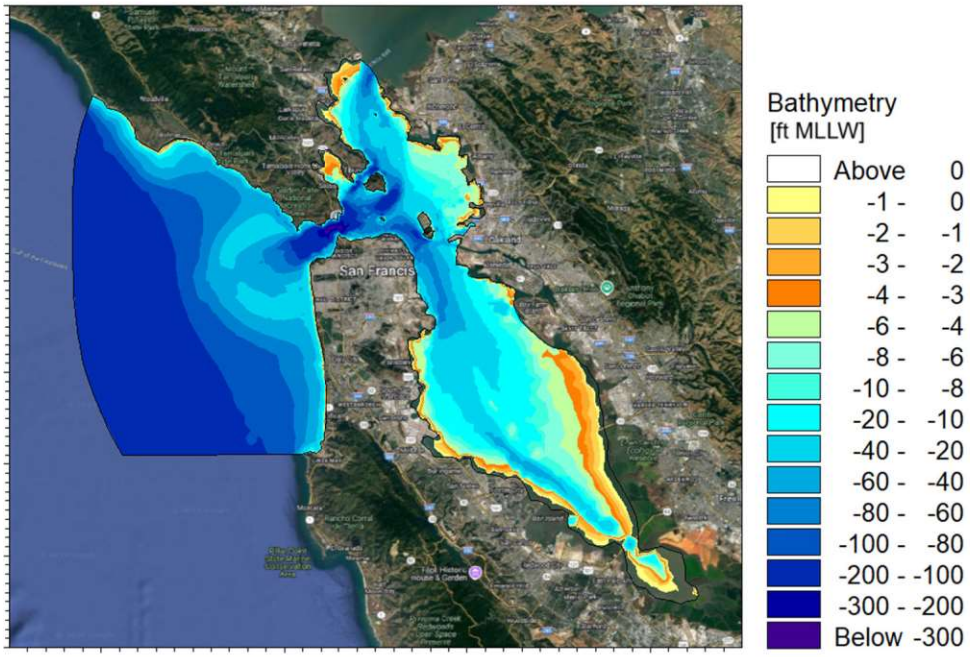


Figure 3-6 Model bathymetry.

3.3.4 Model validation

The numerical wave model was validated using measurements of significant wave height available at NDBC Station 46237. Figure 3-7 shows the quantile-quantile comparison of significant wave height between model results and measurements for a 4-year period between 2020 and 2023. A time series comparison of model results and measurements are shown in Figure 3-8 for a period between Oct 2022 and Jun 2023.

Figure 3-7 and Figure 3-8 indicate that the numerical wave model adequately represents the measured wave conditions at NDBC Station 46237. Significant wave heights generated from the model are on average slightly greater than the real-world observations. This adds a small amount of conservativeness to the model results and analysis.

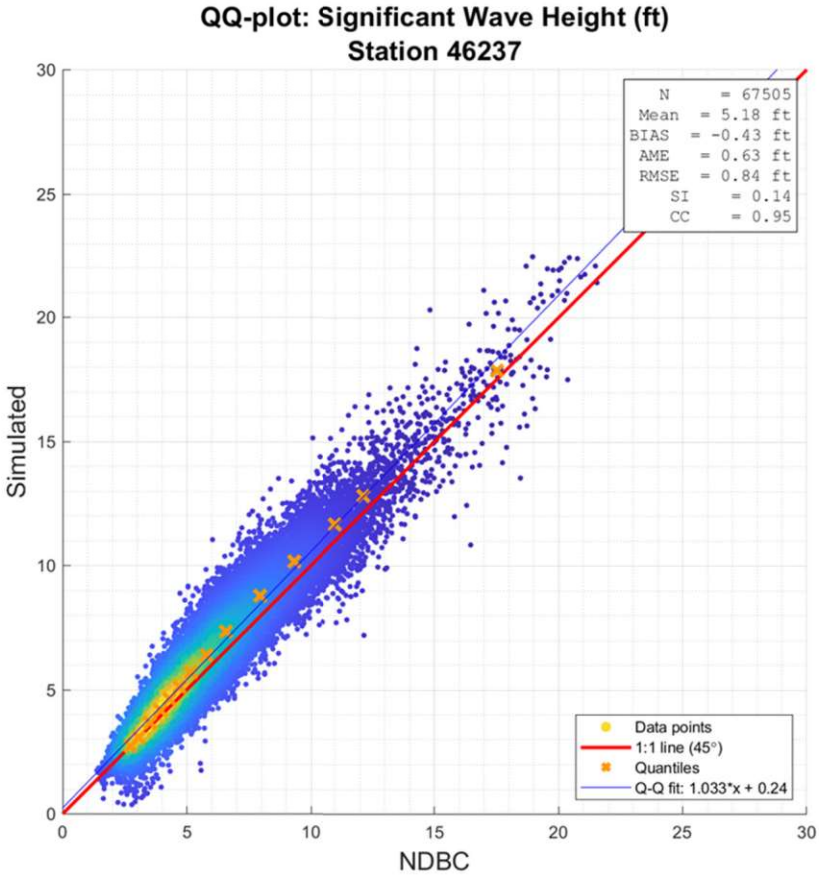


Figure 3-7 *Comparison of significant wave height between model results and measurements at NDBC Buoy 46237 for a 4-year period between 2020 and 2023.*

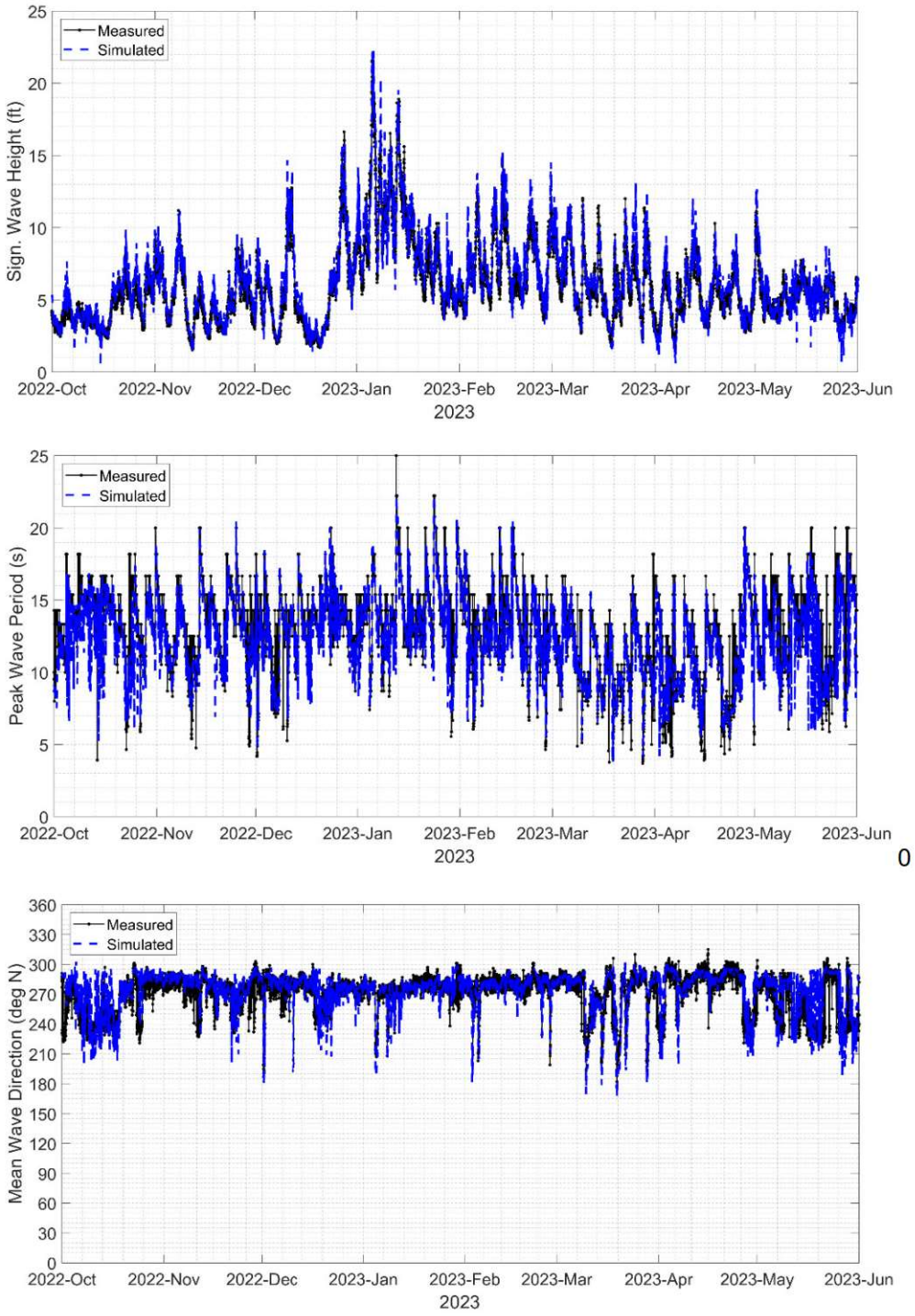


Figure 3-8 *Comparison of significant wave height (top), peak wave period (middle), and mean wave direction (bottom) between model results and measurements at NDBC Buoy 46237 for a period between Oct 2022 and Jun 2023.*

3.3.5 Model parameters

Table 3-4 contains a description of model parameters selected to obtain the comparison shown in the sections above.

Table 3-4 Model Parameters

Parameters		Value
Discretization	Spectral	Logarithmic: 26 frequencies $f_{j=0,1,2,..25} = 0.045*1.1^{j}$
	Directional	16 x 22.5°
Wind	Growth parameter	1.3
	Wave age tuning parameter	0.011
	Air-sea interaction type	Uncoupled
	Background Charnock parameter	0.0185
	Correction of friction velocity type	Simple cap
	Cap value	0.06
Wave Breaking		0.8
Bottom Friction	Nickuradse	0.02 m (0.8 in)
White capping	Cdis	1.33
	DELTA dis	0.5

3.3.6 Model results

The numerical wave model was simulated for a continuous period of 27-years (January 1997-December 2023) and was used to extract wave parameters at locations near the breakwater, see Figure 3-9. Example results from the wave model at BrkWtr-E output location are plotted in Figure 3-10, Figure 3-11 and Figure 3-12.

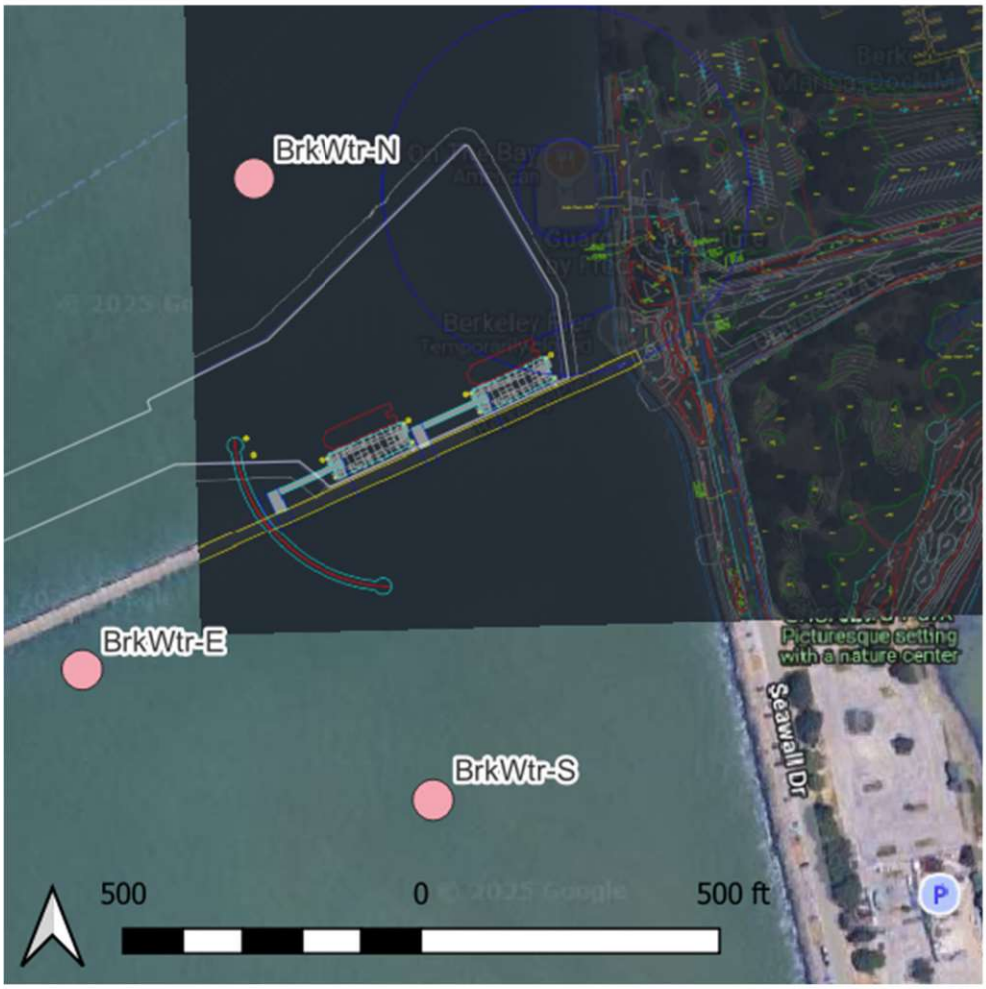


Figure 3-9 *Model output locations.*

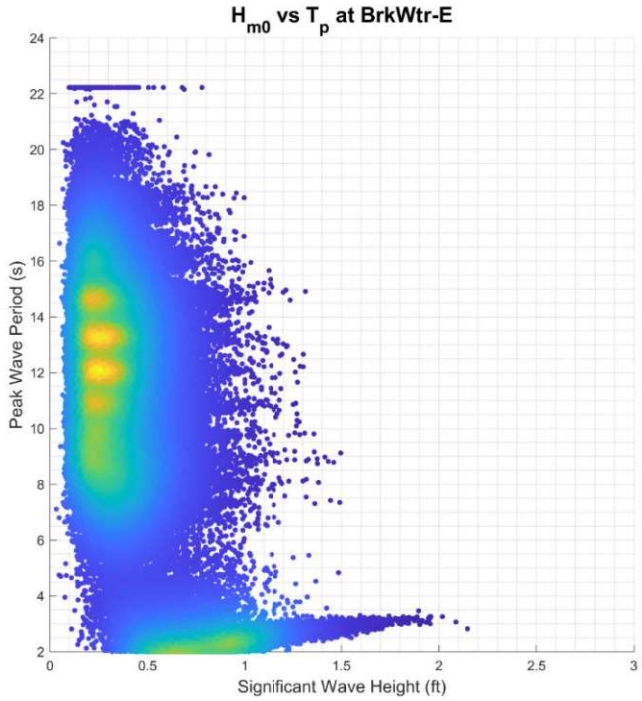


Figure 3-10 *Scatter of significant wave height and peak wave period at BrkWtr-E location.*

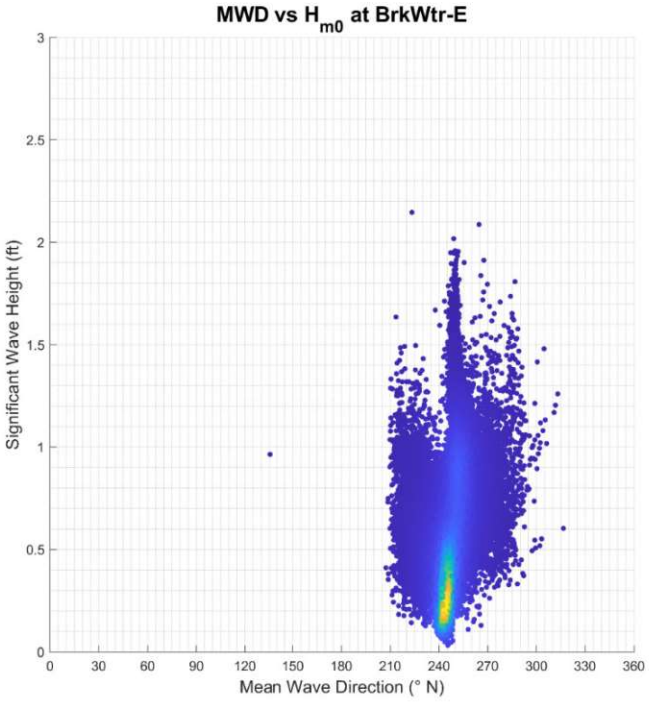


Figure 3-11 *Scatter of significant wave height and mean wave direction at BrkWtr-E location.*

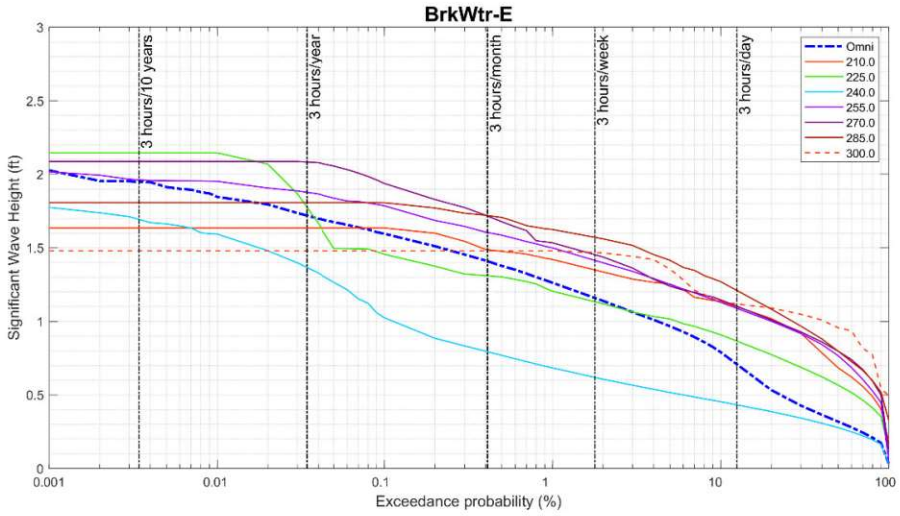


Figure 3-12 Directional exceedance of significant wave height at BrkWtr-E location.

3.3.7 Model sensitivity

The bathymetry in the vicinity of the project site, described in Section 2.5, is generally from -8 to -10 ft MLLW. As these are very shallow depths, the bottom friction model parameter can play a significant role in influencing the wave characteristics at the project site. In the absence of any measured data at the project site, model sensitivity to bottom friction parameter was studied by simulating extreme conditions with a lower bottom friction parameter of 0.005 m (0.2 in).

The extreme conditions were determined by performing a point-over-threshold analysis on wind speed at the three (3) CO-OPS stations near the project site. A similar analysis was also conducted on the offshore wave boundary conditions to obtain a set of extreme events for the sensitivity runs. Table 3-5 summarizes the threshold limits and the resulting number of events.

Table 3-5 Selection criteria for model sensitivity simulations

Quantity	Threshold	Number of Events
Wind speed at CO-OPS 9414290 San Francisco	20 knots	34
Wind speed at CO-OPS 9414750 Alameda	25 knots	36
Wind speed at CO-OPS 9414893 Richmond	22 knots	30
Significant Wave Height at Offshore Boundary	5 m (16.5 ft)	35

Figure 3-13 presents the increase in significant wave height at BrkWtr-E location with a lower bottom friction value and indicates that the median increase is 0.15 ft and maximum increase is 0.3 ft.

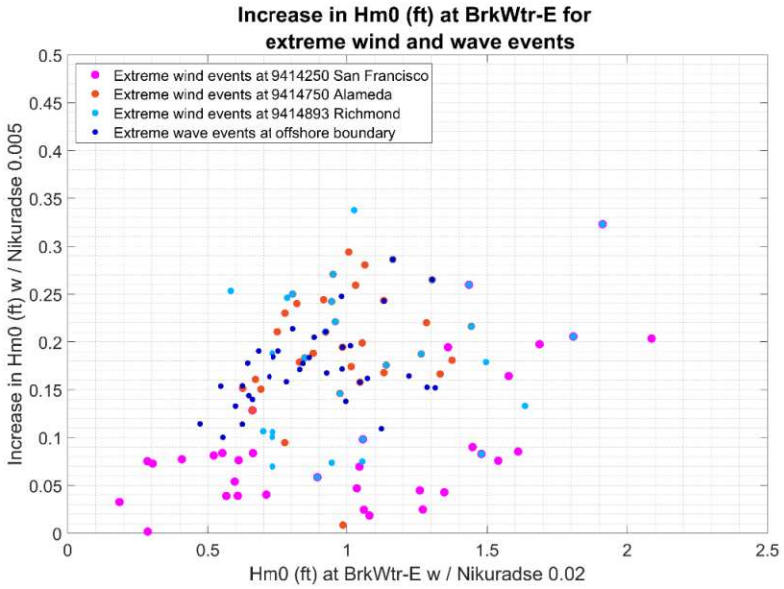


Figure 3-13 *Increase in significant wave height at BrkWtr-E location with a lower bottom friction value for different extreme events.*

In the absence of measured wave data at the project site, it could be considered to increase the extreme values calculated using model results time series by a maximum of 0.3 ft to mitigate any uncertainty due to bottom friction effects.

For the downtime analysis and design (extreme) wave conditions, a factor of 1.1 (10% increase in significant wave height) was applied to address model uncertainties and ensure robust decision-making.

3.3.8 Extreme wave analysis

An extreme wave analysis was conducted using the 24-year dataset from the wave model for an output point at the proposed breakwater location. Weibull and Gumbel distributions were applied to the extreme values. Table 3-6 presents the estimated wave heights for various return periods, along with the corresponding lower and upper bound values for 95% confidence derived from the Gumbel distribution (more conservative than Weibull and better fit).

The significant wave height from the upper bound of the 100-year return period (2.6 ft) was used as design wave condition for the breakwater.

Table 3-6 Extreme wave analysis at the proposed breakwater location

Return Period (years)	Extreme Wave Height Central (Lower-Upper bounds) (ft)
10	2.0 (1.9 – 2.2)
25	2.2 (2.0 – 2.4)
50	2.3 (2.0 – 2.5)
100	2.4 (2.1 – 2.6)
200	2.4 (2.1 – 2.7)

3.4 Wave diffraction modeling

To evaluate wave conditions within the sheltered basin, including at the berths, a diffraction modeling study was carried out to quantify wave disturbance in the lee of the proposed breakwater. Accurate assessment of wave conditions inside terminals is critical for ensuring safe and reliable operations, particularly under moderate to severe sea states.

A Boussinesq-type wave model, MIKE 21 BW, was used to simulate wave transformation processes such as diffraction, partial reflection, and nonlinear wave interactions within the terminal. These models are well-suited for resolving complex wave behavior in confined or semi-enclosed basins, including long-period motions and directional spreading effects. The model was applied to assess wave disturbance coefficients at key locations, including the berths, pier, and plaza. Results from this analysis are presented in the following sections.

3.4.1 Model Description

The numerical model applied for modeling of the wave propagation into the terminal was the Boussinesq Wave Module of DHI's MIKE 21 numerical model (MIKE 21 BW).

MIKE 21 BW is a Boussinesq type non-linear wave model, which simulates in the time domain the properties of propagation of irregular, directional waves into the harbor/terminal considering all important effects like shoaling, depth refraction, diffraction, bottom friction, non-linear wave-wave interaction, frequency and directional dispersion, partial and full reflection, and transmission through porous structures. Wave breaking is not included in the model.

The model requires the following input:

- Basic model parameters describing the extent of the model area, the grid spacing of the computational model grid, the time step and the duration of the simulation.
- A digitized bathymetry
- Incident wave conditions described by flux time series on the boundaries of the model area. Prior to simulation, these time series are generated on basis of specified wave spectra.
- Porosities (reflection and transmission coefficients) to describe the reflection characteristics for all structures and natural obstructions (breakwaters, quay walls, cliffs, beaches etc.) in the model area. The reflection is described by specification of the porosity of the nearest grid points to the reflective object. For impermeable objects, the porosity layer describing the reflection characteristics of a structure is backed by an impermeable land point.
- Description of so-called sponge layers, which are areas absorbing all wave energy propagating into the area (i.e., no reflection). Sponge layers are used to ensure that no unwanted and unnatural reflections occur in the model area (e.g., from the boundaries of the model).

The investigated layouts and the various input parameters applied in the simulations are described in the sections below.

Limitation of MIKE 21 BW

Depending on whether the deep-water correction terms are included in the solution of the Boussinesq-equations, the MIKE 21 BW numerical model is subject to different restrictions, as shown in Table 3-7.

Table 3-7 *Limitations of MIKE 21 BW when excluding and including deep-water correction terms in the computations.*

Deepwater terms *excluded*	Deepwater terms *included*
$\frac{h_{max}}{L_0} \leq 0.22$	$\frac{h_{max}}{L_0} \leq 0.60$
$\frac{L_{min}}{\Delta x} \geq 7$	$\frac{L_{min}}{\Delta x} \geq 7$
$\frac{T_{min}}{\Delta t} \geq 7$	$\frac{T_{min}}{\Delta t} \geq 32$
$\Delta t \leq \frac{\Delta x}{\sqrt{g h_{max}}}$	$\Delta t \leq \frac{\Delta x}{\sqrt{g h_{max}}}$

h_{max}	:	maximum water depth in the model area (m)
h_{min}	:	minimum water depth in the model area (m)
L_0	:	deep-water wavelength (m)
T_{min}	:	minimum wave period (s)
L_{min}	:	minimum wavelength (m) determined from T_{min} and h_{min}
Δx	:	grid spacing (m) in both x- and y-direction
Δt	:	time step (s)
g	:	gravitational acceleration (m/s^2)

Although the ferry terminal is in relatively shallow water, the prevalent waves in the area have very short periods. Hence, the simulations performed in this study were carried out with an h_{max}/L_0-ratio up to 0.50 for the shortest wave period. Therefore, it was necessary to include the deep-water correction terms in the computations.

3.4.2 Layout

The initial layout considered in the present study is shown in Figure 3-14, the breakwater length for the initial runs was 400 ft.

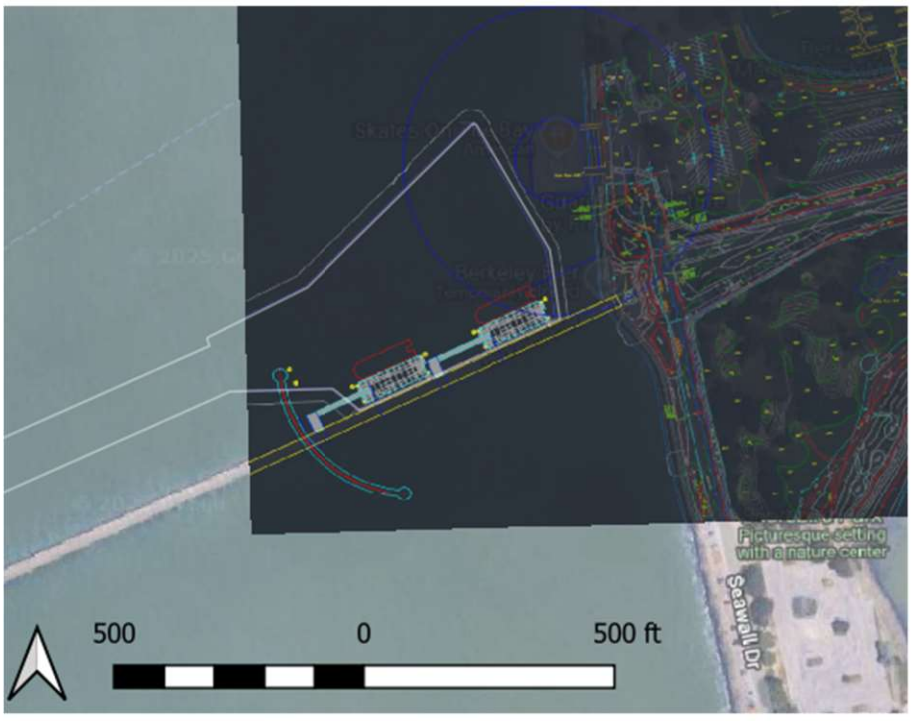


Figure 3-14 Project site showing the terminal layout used on the initial simulation runs.

3.4.3 Simulation matrix

The modeling was performed with an incoming significant wave height of $H_{m0,incoming}$=0.7 ft (0.2 m). A simulation matrix was prepared including four directions and six wave periods as presented in Table 3-8.

Table 3-8 Test matrix used for the wave disturbance simulations at the model boundary

Model Orientation	Wave Direction	Sign. Wave Height (ft)	Peak Wave Period (sec)
0°N	255°N	0.7	3s, 6s, 9s, 12s, 15s, 18s
	270°N		
315°N	210°N		
	225°N		
	240°N		
	300°N		

3.4.4 Model domain and bathymetry

Considering waves scattered in front of the breakwater (refer to Figure 3-11), wave directions at every 15° between 210°N and 270°N, and 300°N are considered for the present study. Since the wave direction varies, two model bathymetries were developed with orientation towards 0°N and 315°N. The purpose of having two orientations was to reduce the required extent of the model (saving computation time) and to make sure that the wave propagation in the model was generally perpendicular to the grid orientation as recommended for improved model performance.

The bathymetry generated with the grid orientation of 0°N is used for the wave directions of 255°N and 270°N, where the oblique mean wave direction relative to the model axes are ±10°-30° on either side. The bathymetry generated with an orientation of 315°N is used for simulating 210°N, 225°N, 240°N and 300°N. See Figure 3-15 and Figure 3-16 for the model domains used. Figure 3-17 shows the bathymetry within the model domains.

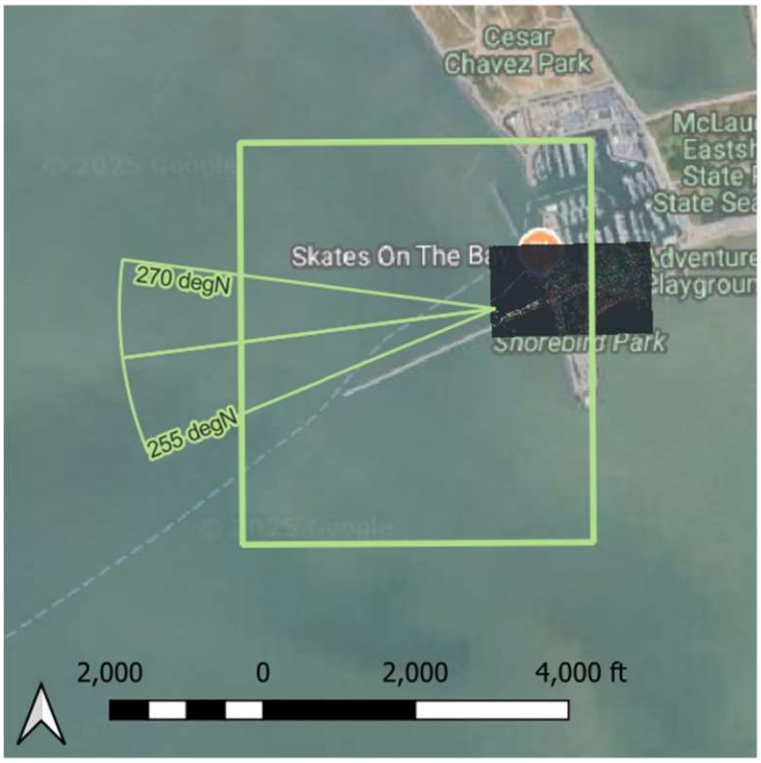


Figure 3-15 *BW model orientation 0°N used for incoming wave directions of 255°N and 270°N.*

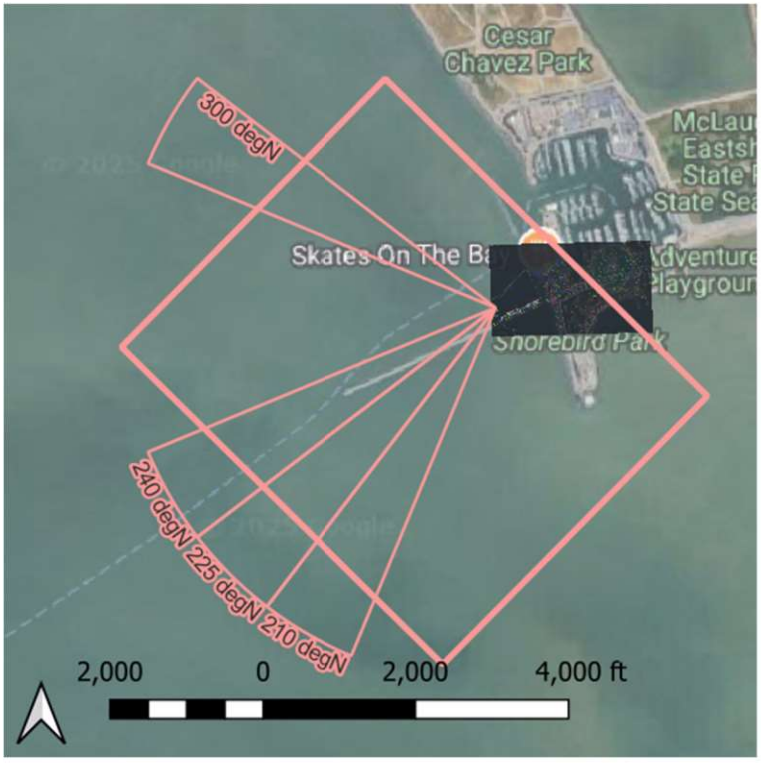


Figure 3-16 *BW model orientation 315°N used for incoming wave directions of 210°N, 225°N, 240°N, and 300°N.*

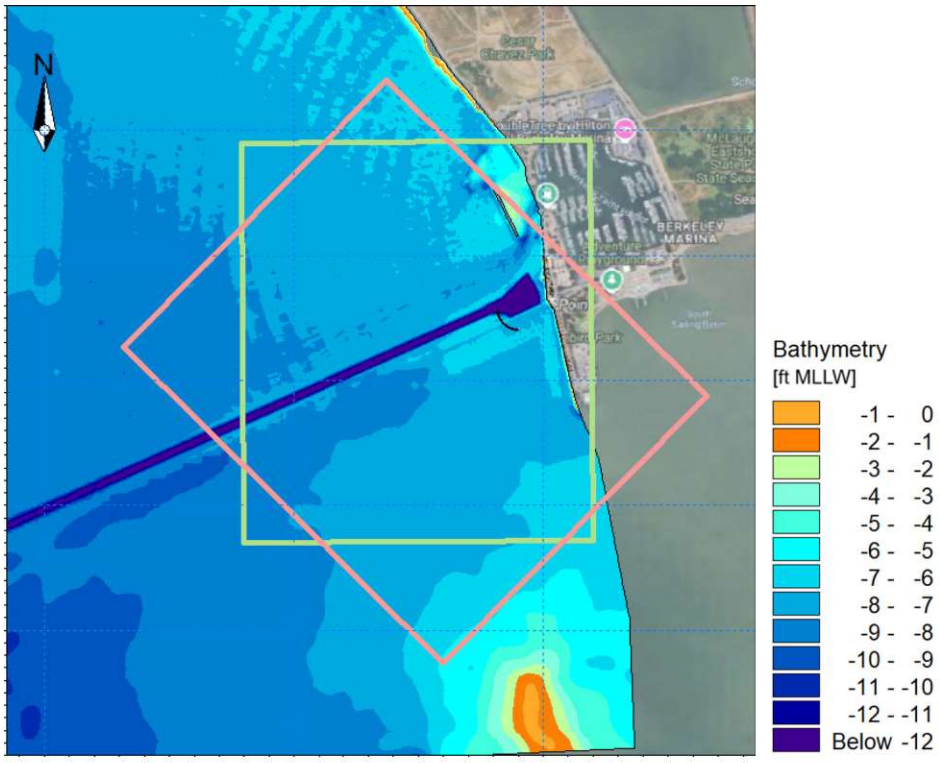


Figure 3-17 Bathymetry (ft MLLW) within the BW model domains.

As an example, the model bathymetry for model orientation of 0°N (see Figure 3-15 for box extent) is presented in Figure 3-18. In the model bathymetry, the horizontal datum is WGS1984/UTM 10N. A grid spacing of 2 m x 2 m was used for the model bathymetry.

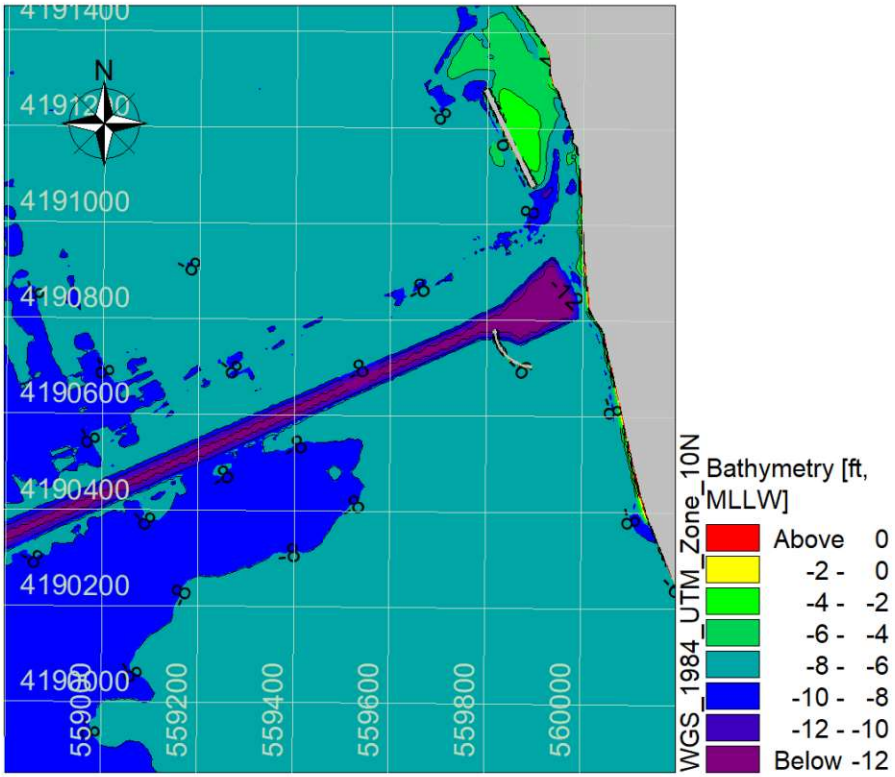


Figure 3-18 Bathymetry (ft MLLW) for model orientation = 0°N

3.4.5 Water level

All numerical simulations were carried out with two water levels: Mean Lower Low Water (MLLW) and Mean Higher High Water (MHHW). The MHHW level used for the diffraction (tranquility) simulations was determined by calculating the average MHHW at the three CO-OPS stations listed in Table 3-2. Thus, MHHW = 6.1 ft MLLW.

3.4.6 Minimum and maximum water depths

Due to the restrictions of the MIKE 21 BW model (see Table 3-7), the maximum water depth was set to h_{max}= 15 ft in all simulations and depths larger than h_{max} were reduced to h_{max}. This simplification of the bathymetry only has a very minor influence on the wave propagation and refraction due to the wavelength (L) to water depth (h_{max}) ratio.

Furthermore, to avoid and minimize model instability due to wave breaking, the minimum water depth was set to h_{min}= 3 ft in all simulations and depths shallower than h_{min} were increased to h_{min}.

3.4.7 Simulation duration and time step

To ensure fully developed wave patterns in the basin and reliable wave height statistics, the duration of each simulation generally corresponded to 30 minutes. The

last 20 minutes of each simulation were used for statistical analysis, i.e. determination of the significant wave height (H_{m0}) throughout the model area. The time step (Δt) applied in the simulations was 0.02 s.

3.4.8 Partial reflection and transmission

The reflective characteristics of various coastal structures in the model domain, such as rubble mounds, vertical walls, etc., is defined by porosity layers. The location of partial reflective and almost fully reflective structures is defined as shown in Figure 3-19. These are specified by the porosity values corresponding to the reflection characteristics of these structures. The assumed reflection characteristics are given in Table 3-9.

Table 3-9 Reflection coefficients specified in the model.

Feature / Structure	Reflection Coefficient
Revetment	0.40
Vertical Wall / Bulkhead	0.95

The porosity values applied in the model are calculated for these reflection coefficients considering actual water depths, wave height and period in front of the structure.

It must be noted that, as shown in Figure 3-19, the breakwater is specified to be highly reflective only at the ends. The middle segment is specified to be less reflective to minimize model instability, given the convex shape of the breakwater, the reflected waves radiate out in all directions causing instabilities near the wave boundaries.

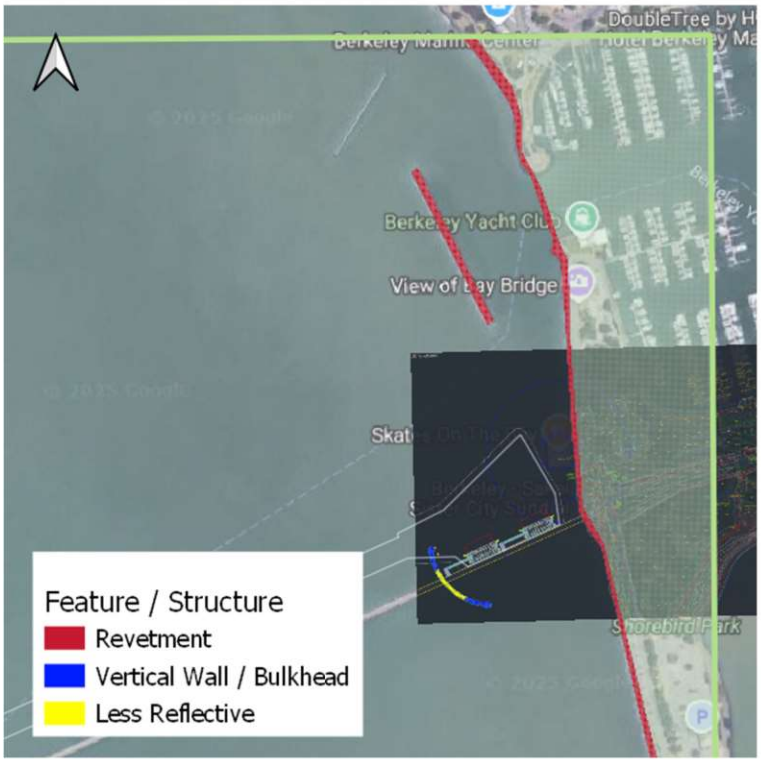


Figure 3-19 Location of almost fully reflecting vertical walls and partially reflecting revetments.

3.4.9 Sponge layer

The waves in the numerical model were generated along lines inside the model boundary, and since no reflection must occur from the boundaries of the numerical model, so-called sponge layers (layers which smoothly absorb all wave energy entering the layers) were introduced. Additionally, wave energy reaching the shoreline and areas within the model domain that do not influence waves at project site are also absorbed using "sponge" layers. This method is generally adopted in wave tranquility simulations to avoid wave reflection which is not useful in describing wave conditions within the terminal. The location of absorbing sponge layers is shown in Figure 3-20.

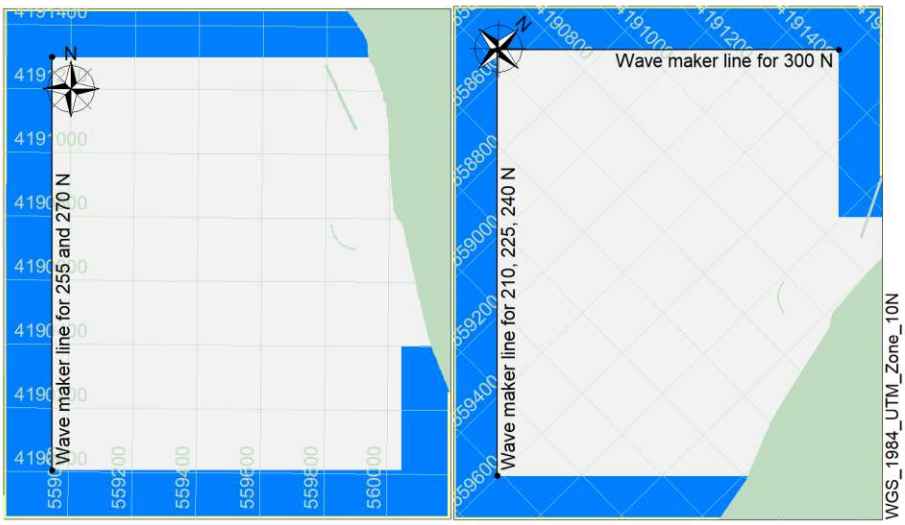


Figure 3-20 *Location of sponge layers (blue area) for wave absorption. Left: Model oriented towards 0°N. Right: Model oriented towards 315°N.*

3.4.10 Model wave generation

Waves are generated in the model domain by internal wave generation lines along the model boundaries. Irregular random waves were generated based on the JONSWAP spectrum for the input values of wave height (H_{m0}) and wave period (T_p). A maximum deviation of 20° from the mean wave direction is chosen for the simulations.

3.4.11 Model results

This section presents the results of the simulations. The purpose of the simulations is to document the wave disturbance at berth locations. The simulation matrix with the offshore wave conditions is presented in Table 3-8. The total duration of the simulations was 30 minutes, of which the last 20 minutes were used for determination of significant wave heights (H_{m0}) throughout the port.

An incident significant wave height of $H_{m0,incoming}$ = 0.2 m (approximately 0.7 ft) was applied in the simulations.

Results of the simulations are presented as wave disturbance coefficients. Schematic representation of the ferry terminal is given in Figure 3-21. The wave disturbance coefficients ($K = H_{m0}/H_{m0,incoming}$) have been calculated at the different berth locations, see Figure 3-21. For detailed analysis of the wave conditions at the two berths shown in Figure 3-21, average wave disturbance coefficients, K, were calculated in areas of 120 ft x 35 ft corresponding to the approximate vessel dimensions and are given in Table 3-10 for MLLW water level and in Table 3-11 for MHHW water level.

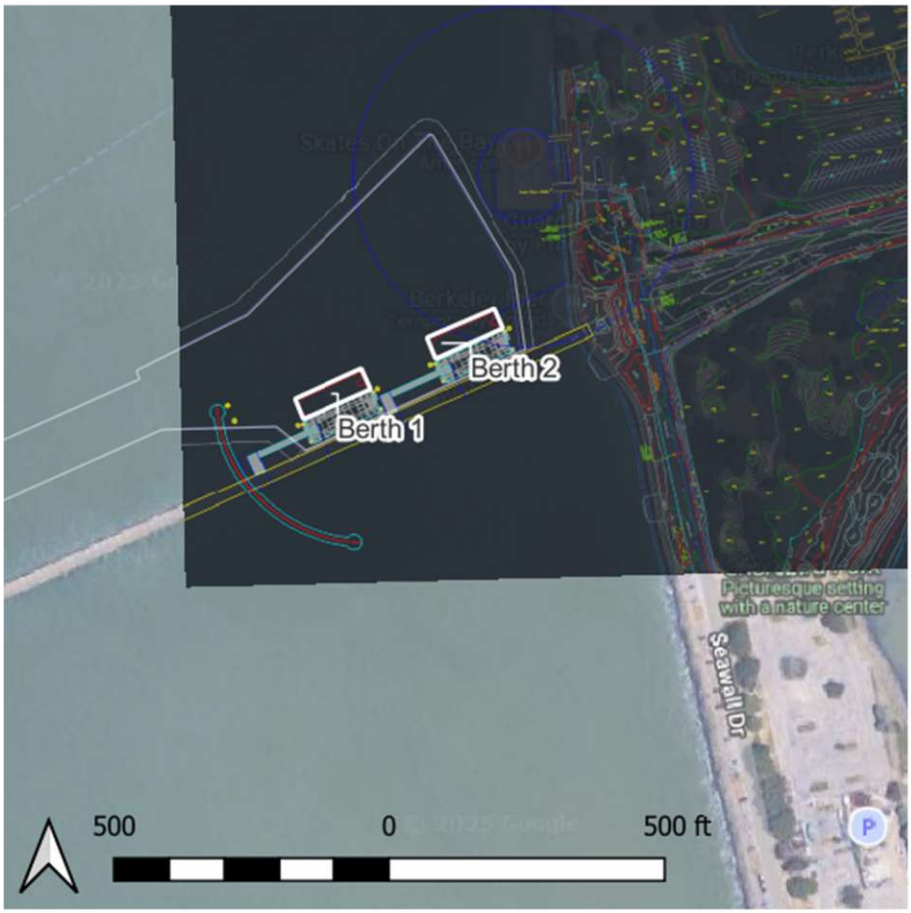


Figure 3-21 Areas within the ferry terminal used for mean wave disturbance calculation.

Table 3-10 Mean wave disturbance coefficients for MLLW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.35	0.51
210	6	0.41	0.46
210	9	0.41	0.42
210	12	0.41	0.43
210	15	0.38	0.41
210	18	0.38	0.40
225	3	0.36	0.36
225	6	0.35	0.35
225	9	0.35	0.35
225	12	0.33	0.33
225	15	0.31	0.31
225	18	0.29	0.34
240	3	0.32	0.25
240	6	0.29	0.26
240	9	0.26	0.26
240	12	0.27	0.25
240	15	0.25	0.26
240	18	0.24	0.25
255	3	0.23	0.31
255	6	0.17	0.18
255	9	0.17	0.19
255	12	0.19	0.20
255	15	0.20	0.22
255	18	0.20	0.22
270	3	0.54	0.78
270	6	0.32	0.51
270	9	0.28	0.46
270	12	0.29	0.45
270	15	0.30	0.44
270	18	0.28	0.41
300	3	1.04	1.05
300	6	1.02	1.14
300	9	1.02	1.13
300	12	0.94	1.09
300	15	0.86	1.00
300	18	0.85	0.97

Table 3-11 Mean wave disturbance coefficients for MHHW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.66	0.64
210	6	0.64	0.60
210	9	0.64	0.57
210	12	0.71	0.60
210	15	0.67	0.58
210	18	0.57	0.54
225	3	0.69	0.57
225	6	0.59	0.52
225	9	0.61	0.49
225	12	0.63	0.50
225	15	0.65	0.49
225	18	0.56	0.50
240	3	0.59	0.53
240	6	0.45	0.41
240	9	0.45	0.40
240	12	0.46	0.36
240	15	0.45	0.32
240	18	0.40	0.33
255	3	0.45	0.59
255	6	0.29	0.33
255	9	0.31	0.29
255	12	0.36	0.39
255	15	0.39	0.42
255	18	0.37	0.41
270	3	0.74	0.91
270	6	0.51	0.81
270	9	0.49	0.74
270	12	0.50	0.72
270	15	0.56	0.81
270	18	0.58	0.75
300	3	1.17	1.13
300	6	1.05	1.10
300	9	1.07	1.13
300	12	0.92	1.10
300	15	0.96	1.16
300	18	0.91	1.10

Wave disturbance coefficient maps for 240°N and 255°N, the two directions with most wave incidence for overall model domain and zoomed view of the terminal are presented in Appendix C.

3.4.12 Alternative breakwater lengths

In addition to model simulations as described above, wave tranquility was also assessed for alternative breakwater lengths of 300-ft and 250-ft. Figure 3-22 shows the layouts for the alternative breakwater lengths.

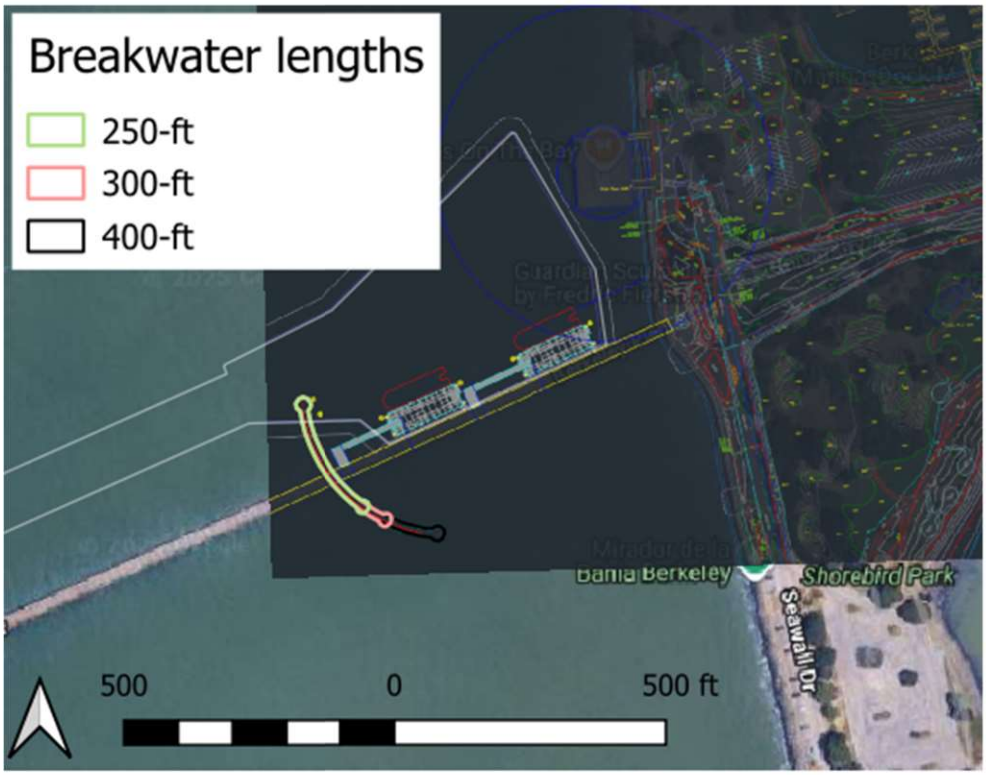


Figure 3-22 Layout for alternative breakwater lengths.

3.4.13 Model results for layout with 300-ft breakwater

Average wave disturbance coefficients, K (corresponding to the total wave energy), were calculated in berth areas and are given in Table 3-12 and Table 3-13 for the layout with 300-ft breakwater.

Table 3-12 Mean wave disturbance coefficients with 300-ft breakwater for MLLW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.42	0.74
210	6	0.46	0.61
210	9	0.46	0.54
210	12	0.45	0.58
210	15	0.42	0.54
210	18	0.43	0.50
225	3	0.40	0.53
225	6	0.39	0.44
225	9	0.39	0.43
225	12	0.40	0.42
225	15	0.37	0.40
225	18	0.35	0.41
240	3	0.31	0.30
240	6	0.30	0.28
240	9	0.29	0.27
240	12	0.28	0.30
240	15	0.25	0.28
240	18	0.24	0.25
255	3	0.23	0.30
255	6	0.18	0.17
255	9	0.20	0.18
255	12	0.21	0.21
255	15	0.20	0.22
255	18	0.21	0.21
270	3	0.54	0.78
270	6	0.33	0.50
270	9	0.28	0.45
270	12	0.28	0.44
270	15	0.30	0.45
270	18	0.29	0.41
300	3	1.04	1.04
300	6	1.02	1.13
300	9	1.00	1.13
300	12	0.92	1.06
300	15	0.85	0.98
300	18	0.87	0.96

Table 3-13 Mean wave disturbance coefficients with 300-ft breakwater for MHHW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.70	0.88
210	6	0.66	0.80
210	9	0.67	0.77
210	12	0.73	0.75
210	15	0.66	0.68
210	18	0.58	0.65
225	3	0.71	0.79
225	6	0.61	0.62
225	9	0.64	0.59
225	12	0.64	0.60
225	15	0.64	0.54
225	18	0.62	0.57
240	3	0.57	0.54
240	6	0.45	0.43
240	9	0.48	0.47
240	12	0.45	0.39
240	15	0.42	0.36
240	18	0.42	0.42
255	3	0.46	0.60
255	6	0.29	0.32
255	9	0.31	0.30
255	12	0.36	0.38
255	15	0.37	0.39
255	18	0.37	0.41
270	3	0.74	0.91
270	6	0.51	0.80
270	9	0.47	0.73
270	12	0.52	0.74
270	15	0.58	0.79
270	18	0.55	0.70
300	3	1.15	1.13
300	6	1.04	1.08
300	9	1.05	1.11
300	12	0.93	1.10
300	15	0.97	1.18
300	18	0.89	1.10

3.4.14 Model Results for Layout with 250-ft Breakwater

Average wave disturbance coefficients, K (corresponding to the total wave energy), were calculated in berth areas and are given in Table 3-14 and Table 3-15 for the layout with 250-ft breakwater.

Table 3-14 Mean wave disturbance coefficients with 250-ft breakwater for MLLW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.46	0.81
210	6	0.49	0.68
210	9	0.46	0.59
210	12	0.47	0.66
210	15	0.46	0.6
210	18	0.46	0.55
225	3	0.43	0.6
225	6	0.41	0.46
225	9	0.43	0.46
225	12	0.41	0.46
225	15	0.4	0.43
225	18	0.38	0.45
240	3	0.31	0.26
240	6	0.3	0.22
240	9	0.3	0.24
240	12	0.28	0.25
240	15	0.25	0.26
240	18	0.25	0.26
255	3	0.23	0.31
255	6	0.18	0.16
255	9	0.2	0.16
255	12	0.2	0.19
255	15	0.2	0.2
255	18	0.22	0.2
270	3	0.54	0.78
270	6	0.32	0.51
270	9	0.28	0.45
270	12	0.29	0.44
270	15	0.32	0.46
270	18	0.3	0.41
300	3	1.04	1.04
300	6	1.02	1.13
300	9	1.02	1.11
300	12	0.92	1.05
300	15	0.87	0.97
300	18	0.87	0.96

Table 3-15 Mean wave disturbance coefficients with 250-ft breakwater for MHHW simulations.

MWD (°N)	Tp (s)	Berth 1	Berth 2
210	3	0.66	0.88
210	6	0.65	0.86
210	9	0.67	0.82
210	12	0.73	0.77
210	15	0.69	0.69
210	18	0.62	0.68
225	3	0.62	0.79
225	6	0.55	0.63
225	9	0.61	0.6
225	12	0.64	0.59
225	15	0.61	0.56
225	18	0.63	0.6
240	3	0.6	0.47
240	6	0.46	0.39
240	9	0.47	0.43
240	12	0.45	0.37
240	15	0.43	0.37
240	18	0.5	0.44
255	3	0.47	0.61
255	6	0.29	0.32
255	9	0.3	0.3
255	12	0.33	0.36
255	15	0.33	0.38
255	18	0.36	0.41
270	3	0.74	0.91
270	6	0.51	0.81
270	9	0.47	0.73
270	12	0.53	0.74
270	15	0.55	0.77
270	18	0.48	0.67
300	3	1.16	1.13
300	6	1.05	1.08
300	9	1.03	1.1
300	12	0.94	1.11
300	15	0.97	1.18
300	18	0.89	1.09

3.4.15 Comparison of breakwater lengths

A review of the wave-disturbance plots in Appendix C and comparisons shown in Figure 3-23 (for the representative case of direction 255° (N), MHHW, and a 6-second period) indicate that shortening the breakwater has minimal influence on conditions at

Berth 1. The berth is sheltered close enough to the breakwater, so the reduction in the breakwater length does not meaningfully change the level of protection it receives.

Berth 2 is more exposed to incoming wave energy. The analyses show a modest increase in wave disturbance at this berth when the breakwater is shortened, a result of the reduced attenuation of waves for this more exposed location. The increase in wave exposure at Berth 2 is relatively small when reducing the breakwater from 400 ft to 300 ft, but becomes slightly more noticeable when the length is further reduced to 250 ft.

Based on these results, the 300 ft configuration provides the best balance between maintaining adequate sheltering and reducing structure length and is considered the optimal breakwater length for these priorities.

These findings, together with the operational downtime assessment, are used to determine the recommended breakwater length. The wave-disturbance results and operational performance will be considered together to identify the most appropriate design length for the breakwater.

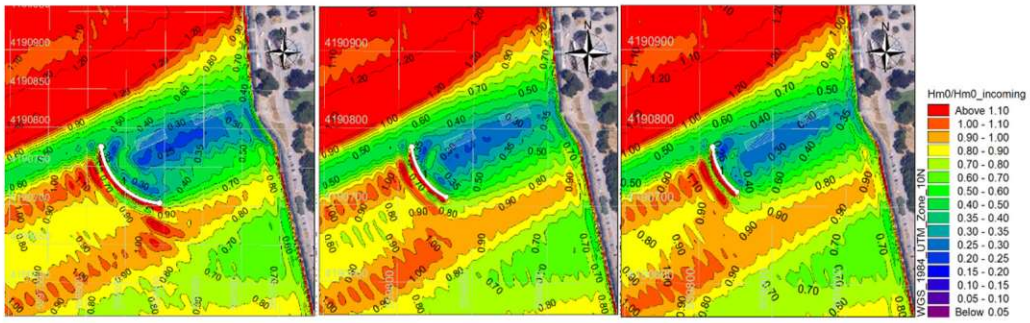


Figure 3-23 Wave disturbance plot (Dir = 255°N at MHHW, Tp = 6 sec) for modeled breakwater lengths

3.4.16 Wave time series in the ferry terminal

To establish the basis for calculating the wave statistics at the ferry terminal berths, see Figure 3-21, the results of the wave disturbance modeling were combined with the long-term wave conditions outside the port at point BrkWtr-E, see Figure 3-9. This was done by establishing transfer functions, describing the transformation of wave heights from point BrkWtr-E outside the port to a given area inside the port (based on the wave disturbance simulations), and then combining the transfer functions with the long-term time series of wave conditions at point BrkWtr-E. This provides a long-term time series of the wave heights at selected locations, for subsequent statistical analysis.

Time-series analyses of the wave conditions for each breakwater length were used to assess operational downtime and berth exposure, informing the final breakwater length, as described in Section 4.

3.4.17 Extreme diffraction wave model

The model was also run for extreme/design condition. The inputs consisted of the 100-year design wave conditions (Hs = 2.6 ft / Tp = 3.2 s) and the design water level (11.9 ft).Table 3-16 presents the results, for the 300-ft breakwater configuration, extracted at the approximate locations A (berth 1), B (between berths), C (berth 2) and D (between berth 2 and shoreline). These results were used to calculate deck and plaza minimum elevations and vulnerability to sea level rise.

Table 3-16 Significant wave heights offshore (input) and on the lee side of the breakwater under design conditions.

Location	H_s (ft)
Offshore (input)	2.6
A (berth 1)	1.8
B (between berths)	2.3
C (berth 2)	2.4
D (between berth 2 and shoreline plaza)	2.4

4 Downtime assessment

The objective of this downtime assessment is to evaluate potential weather-induced downtime frequency at the designated berth locations of the ferry terminal. Weather downtime refers to periods when operational activities at the berths are disrupted due to environmental conditions exceeding safety or operational thresholds. Specifically, this assessment quantifies the number of hours per year during which wave heights or wind speeds surpass these critical limits, rendering the berths temporarily inoperable.

The analysis focuses on understanding how local environmental conditions, including wind and wave climate, impact berth operability. This information will support a comprehensive overview of the site-specific weather constraints, providing a baseline for evaluating potential adjustments in operations or the physical layout of the terminal to minimize disruptions.

4.1 Downtime criteria

The threshold defining the upper limit of safe marine operations is a subjective quantity and it is best determined from local pilots and terminal operators experience.

In 2017, COWI conducted a downtime assessment for Mission Bay Ferry Landing (COWI, 2017) where COWI was provided with a chronological summary of cancelled service due to severe weather in 2016 and 2017. According to these data, the weather conditions that often create service issues for WETA involve winds above 20 knots (10.3 m/s). As part of the assessment, COWI compared the wave agitation at the ferry terminal locations for the specific days of winter in 2016 and 2017 when the operational terminals had to shut down due to bad weather conditions. The comparison reproduced the wave conditions that made the terminals inoperative, making it possible to set up an operational threshold for the Mission Bay Ferry Landing downtime assessment. A significant wave height downtime threshold of 1.5 feet was selected for that assessment.

Given that background, the following criteria have been agreed on with WETA and the City of Berkeley for this study.

- Wave height threshold: 1.5 feet
- Wind speed threshold: 20 knots (1-hour average)

4.2 Downtime due to waves

To assess the impact of wave action on operational reliability at the proposed terminal, a wave downtime analysis was conducted for two berths (B1 and B2) under three different layout configurations:

- No breakwater
- 250-foot breakwater
- 300-foot breakwater
- 400-foot breakwater

The analysis utilizes the downtime threshold of significant wave height greater than 1.5 feet.

Wave conditions were derived from a 27-year hindcast time series, as described in Section 3. This long-term dataset captures the variability of wave energy and supports a robust assessment of operability.

Downtime was evaluated on a monthly basis to investigate seasonal variations in exposure, informed by the wind seasonality analysis presented in Section 2.3.3. This approach provides insights into the expected operational performance of each breakwater alternative throughout the year.

The results quantify the potential improvement in berth downtime with the introduction of a breakwater and highlight the relative effectiveness of the 300-foot and 400-foot options in reducing wave-induced service disruptions.

Figure 4-1 presents the monthly average downtime calculated over a 27-year hindcast wave time series, representing the long-term average frequency with which the significant wave height exceeds the 1.5-foot threshold in each month.

The no-breakwater layout shows a clear seasonal pattern, with notable peaks in downtime during late spring and summer, particularly in June (2.1%) and July (1.6%). This reflects the influence of persistent southwesterly wind conditions during this period, which generates higher wave activity at the site.

In contrast, the breakwater configurations (250, 300 and 400 feet in length) result in reduced downtime across all months. The monthly average downtime for these scenarios remains consistently below 0.1%, demonstrating the impact of a breakwater on wave-induced operational disruptions.

The comparison between the breakwater alternatives indicates no significant difference in operational performance. The reduction in wave agitation and resulting downtime is nearly the same for the different configurations, indicating the additional breakwater length does not yield further meaningful improvement in operational reliability based on the established threshold for monthly averages.

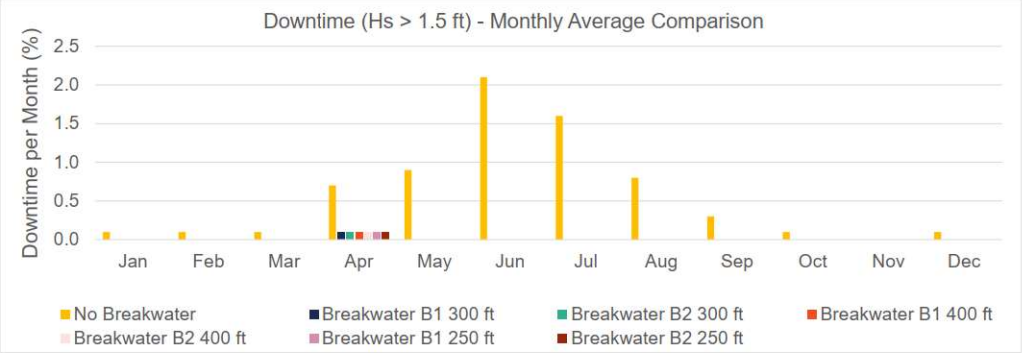


Figure 4-1 Monthly average wave downtime for different breakwater configurations.

In addition to the monthly averages, average monthly maxima were calculated to assess typical peak downtime conditions across the 27-year modeled time series. For each calendar month, the maximum downtime recorded in each individual year was extracted, and the average of those 27 yearly maxima was calculated. This approach provides a more representative estimate of the typical worst-case conditions experienced each month, rather than a single extreme value.

The results, shown in Figure 4-2, indicate that the no-breakwater configuration is subject to higher peak downtimes, averaging over 6% in June, and exceeding 4% in July and August. These values reflect recurring operational challenges during high-energy wave conditions typical of spring and summer months.

In contrast, the 250, 300 and 400-ft breakwater configurations show a significant reduction in average monthly maxima. Downtime remains below 2% for all months, including the more energetic periods of April through August. The difference between the three breakwater lengths downtime is minimal, reinforcing the earlier conclusion that a shorter breakwater is sufficient for mitigating wave-induced downtime to acceptable levels.

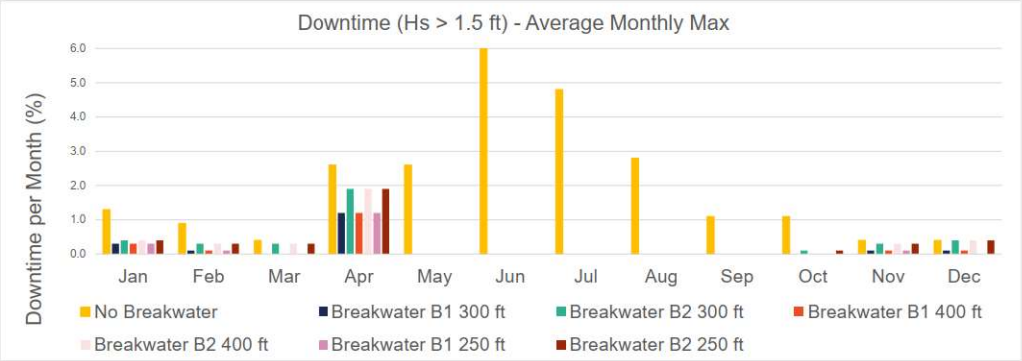


Figure 4-2 Average monthly maxima wave downtime for different breakwater configurations.

When the downtime results are considered alongside the wave-disturbance findings in Section 3.4.15, the 300-ft breakwater length is considered as the optimal option. It provides sufficient sheltering at both berths, avoids the increase in wave exposure

seen with the 250-ft configuration for berth 2, and performs nearly the same as the 400-ft breakwater in terms of downtime. Based on both the wave-disturbance and downtime assessments, the 300-ft breakwater is the recommended length.

5 Coastal structures minimum design elevations

5.1 Wave runup and overtopping

Runup and overtopping assessments were carried out in accordance with the methods and guidelines outlined in EurOtop (EurOtop, 2018), a standard for evaluating wave overtopping behavior on coastal structures. This approach ensures accuracy and consistency in determining the performance and safety of the breakwater under design wave conditions.

Runup was evaluated to estimate the maximum height that waves can reach on the breakwater structure. The calculation considered the incident design wave conditions, structure geometry, and breaker wave type.

The resulting 2% wave runup height was calculated, representing the height exceeded by 2% of waves. Overtopping was assessed to quantify the mean overtopping discharge and maximum overtopping wave volumes under design conditions. The calculations considered:

- Tolerable Overtopping Discharges: Determined based on the intended use of the area behind the breakwater, focusing on public access and recreation. EurOtop (EurOtop, 2018) provides guidelines for acceptable overtopping rates to ensure structural and public safety. It ensures the overtopping does not compromise the integrity of the breakwater, and limits discharge to prevent hazards such as excessive water flow or debris impacting recreational users.

- Influence of Foreshore: factored into account for energy dissipation before waves reach the structure.

For the purposes of design, a conservative overtopping threshold of 0.1 l/s/m (Mean Discharge Limits) was adopted. This value is consistent with guidance (EurOtop, 2018) for areas where impulsive wave action may occur and pedestrian safety is a primary concern. Limiting overtopping to this level ensures that wave impact, splash, and ponding are minimal and do not present hazards to recreational users or damage to infrastructure.

For vulnerability assessments, which explore conditions beyond the design horizon and under more extreme sea level rise scenarios, a more relaxed criterion was considered appropriate. In such cases, overtopping discharges up to 10–20 l/s/m may still be tolerable for pedestrian areas, particularly when wave heights are low ($H < 1.0$ m), the overtopping is non-impulsive, and the area is designed to drain quickly or remain safely accessible during intermittent overtopping events. These thresholds remain within the broader range of public safety criteria outlined in the guidelines (EurOtop, 2018) and provide flexibility for assessing long-term risk and performance under future conditions.

5.2 Breakwater crest elevation

The design of the breakwater crest elevation is critical as it directly influences the structure's ability to provide protection against waves, prevent overtopping, and ensure stability for the area it safeguards. Several factors were considered to arrive at the minimum crest elevation:

- Water Levels: including tidal range, storm surge, and sea level rise (Section 2.1).
- Wave Climate: the energy impacting the structure (Section 3).
- Wave Effects (Runup and Overtopping): to maintain functionality and prevent overtopping damage, the crest elevation must accommodate the wave runup height under design storm conditions. Acceptable overtopping rates for this design were considered based on categorizing the usage of the area behind the structure as recreational.
- Structural Considerations: the type of breakwater (vertical) affects the crest elevation due to wave energy dissipation and runup behavior.

5.2.1 Breakwater runup and overtopping

Table 5-1 summarizes the calculation results. The run up 2% formula applied a partial safety factor of one standard deviation. The minimum breakwater elevation is set at +17.6 ft Berkeley MLLW. This elevation is the run-up 2% elevation (calculated based on the 100-year wave height – Section 3.3.6) plus the design still water elevation needed to guarantee acceptable and safe overtopping discharge rates (<0.1 l/m/s).

Table 5-1 Wave effects and minimum breakwater crest elevation

Wave Run-up 2%	4.5 ft
Average Overtopping Discharge*	0.09 l/m/s
Minimum Breakwater Elevation	17 ft NAVD88 / 17.6 ft Berkeley MLLW

*Acceptable discharge at El. +17.56ft Berkeley MLLW

5.3 Deck elevation

The wave diffraction model developed for the project included a model run with the design still water level (11.9 ft MLLW) and the 100-year design incident wave height (2.6 ft). The incident waves vary along the trestle as the breakwater reduces the shelter towards the shoreline. Along the trestle, the significant wave heights at the deck for design conditions vary from 1.8 to 2.4 ft. The results for maximum green water elevation are presented in Table 5-2.

The maximum green water elevation is the highest vertical level reached by the crest of incoming waves during design conditions.

Table 5-2 Maximum green water elevation

Parameter	Elevation (ft MLLW Berkeley)
Design Still Water Level	11.9
100-year Max Crest Height	2.3-3.3
Max. Green Water Level	14.2-15.2

5.4 Plaza elevation

The minimum design elevation of the shoreline plaza was defined to ensure long-term public safety and avoid flooding under projected sea level rise and design wave conditions. The elevation was established based on a 50-year design life, with consideration for both hydrodynamic performance and public use requirements.

The following parameters were considered in determining the minimum recommended elevation:

- Design still water level, including sea level rise projections for the Intermediate Scenario - 2075 planning horizon.
- Wave transformation results for the shoreline located in the partially sheltered zone behind the breakwater. The considered wave height at the plaza, based on diffraction modeling and partial sheltering by the 300 ft-long breakwater, which reduces incident wave energy at the shoreline. The resulting design significant wave height at the plaza was estimated at 2.4 ft, and this value was used in calculating runup and overtopping under design storm conditions.
- Current shoreline revetment slope: assumed to be maintained at 1:3, extending approximately+11.5 ft to 12.5 ft, providing moderate energy dissipation and limits runup extent under typical storm conditions. Above the revetment, a seawall vertical structure was considered (containing the plaza fill), up to +16.5 ft (design elevation). The slope and rock material of the revetment were factored into the estimate of maximum wave runup using standard formulations.
- Public use considerations: the plaza is intended for frequent public access. The elevation was selected to ensure safe use without requiring closure due to minor flooding events.

Based on these parameters, a minimum elevation of +15.8 ft MLLW was defined to achieve overtopping rates close to zero under design conditions. This elevation was

deemed sufficient to avoid surface flooding and ensure safe pedestrian access, with negligible risk of discomfort or hazard during routine storm events.

However, to align with overall site grading, accessibility, drainage, and architectural integration, the final minimum design elevation (close to the plaza) was set at +16.5 ft MLLW and the maximum, +17.6 ft MLLW. This provides a generous safety buffer and ensures the plaza remains dry and fully operational not only under design conditions but also under more conservative sea level rise scenarios, as confirmed in the vulnerability assessments presented in the following section.

5.5 SLR vulnerability assessment

The breakwater, deck, and plaza elevation were established based on the minimum recommendations outlined in Sections 5.2, 5.3, 5.4, guided by the 2075 planning horizon and the Intermediate Scenario from the 2024 California Sea Level Rise Guidance. Table 5-3 summarizes these minimum elevations alongside the final design elevations selected by the project team to meet broader functional and operational requirements.

Table 5-3 Minimum and design elevations

Elevation (ft Berkeley MLLW)	Breakwater	Deck	Plaza
Minimum	17.6	14.2-15.2	15.8
Design	17.6	16.5-17.6	16.5

To evaluate the long-term resilience of these design elevations, a Sea Level Rise Vulnerability Assessment was conducted. The vulnerability assessment for the BWTPF Project evaluates the performance of key marine structures (breakwater, ferry pier deck, and shoreline plaza) under a range of future sea level rise scenarios extending beyond the 50-year design horizon. These scenarios include Intermediate, Intermediate-High, and High projections for both 2075 and 2100, as defined by the 2024 California Sea Level Rise Guidance. The assessment considered the effects of reduced freeboard, overtopping discharge, inundation risk, and public safety thresholds under increasingly severe conditions.

Results confirm that the infrastructure maintains full functionality and safety under the design condition, which corresponds to the Intermediate Scenario for 2075, and remains resilient under Intermediate-High conditions through 2100, with only minor overtopping and splash impacts. Under the High Scenario for 2100, overtopping and inundation exceed conservative thresholds, requiring operational restrictions and adaptive measures such as improved drainage and controlled public access. Despite these challenges, all components retain structural integrity, and the breakwater

continues to provide significant coastal protection even under extreme conditions. Appendix B presents the detailed assessment.

6 Hydrodynamic and sediment transport

The sediment transport study for the Berkeley Water Transportation Pier Ferry Project provides a detailed assessment of sedimentation dynamics and maintenance needs for the proposed ferry terminal and is presented in Appendix D. The analysis was conducted using a combination of hydrodynamic and sedimentation modeling, calibrated with historical and recent survey data from the Berkeley Marina Channel. The modeling focused on predicting sediment deposition rates within the approach channel and berth pockets, both of which are critical for ensuring navigational reliability and planning future dredging operations.

The results of the six-year simulation indicate that sedimentation rates are highest in the first year following dredging, particularly in the channel, and then gradually stabilize. Over the modeled period, the average annual sedimentation rate across the full dredged area was 0.15 feet, with the channel experiencing a higher average rate of 0.20 feet per year and the berth pockets a lower rate of 0.09 feet per year.

Based on these modeled sedimentation rates, the study estimates the frequency of maintenance dredging required to maintain operational depths in both the channel and berth pockets from the initial dredge elevation -12 ft. Two operational depths were considered: –10 and –9 ft. The estimated intervals until maintenance dredging is needed are presented in Table 6-1

Table 6-1 Estimated dredging intervals (years) for channel and berth pockets

Operational Depth (ft Berkeley MLLW)	Est. Years until Maintenance Dredging	
	Channel	Berth Pocket
-10	10	22
-9	15	33

Despite the robust modeling approach, the study acknowledges limitations. The sedimentation rates used for calibration are based on survey data from a narrow segment of the Berkeley Marina Channel, and there is a lack of site-specific suspended sediment concentration measurements, which introduces some uncertainty into the predictions. Additionally, the results are most reliable within the project footprint, and extrapolation beyond these areas is not recommended. The study also notes that the modeled breakwater length was later reduced from 400 feet to 300 feet, but this change is not expected to significantly affect sedimentation patterns in the navigation channel or berth areas.

In conclusion, the sediment transport analysis provides a reasonable and locally validated estimate of sedimentation rates and maintenance dredging needs for the

BWTPF project. The results support long-term planning and highlight the importance of ongoing monitoring and data collection to refine future predictions and maintenance strategies.

7 Conclusion

This study presents a comprehensive technical assessment confirming the feasibility and resilience of the Berkeley Water Transportation Pier Ferry Project. The integration of long-term environmental datasets, regional hindcast records, and advanced numerical modeling has established robust design parameters for water levels, wave conditions, wind, and currents specific to the Berkeley Marina area. These parameters have informed a series of simulations and analyses that characterize how wave energy, sediment transport, and operational constraints will interact with the proposed ferry terminal structures under a range of present and future conditions.

Wave modeling results demonstrate that the inclusion of a breakwater is essential for reducing wave agitation at the ferry berths and shoreline. The comparative analysis of breakwater lengths shows that increasing the structure from 300 to 400 feet yields only marginal improvements in wave disturbance and operational downtime. The 300-ft breakwater configuration provides sufficient sheltering for both berths, minimizes wave exposure, and achieves nearly the same reduction in downtime as the longer breakwater configuration.

Design elevations for the breakwater crest, terminal deck, and shoreline plaza were determined using overtopping thresholds and future sea level rise projections. The methodology follows EurOtop (EurOtop, 2018) criteria and incorporates a 1% annual exceedance probability storm tide event combined with sea level rise allowances consistent with the 2024 California Sea Level Rise Guidance. The recommended elevations ensure structural safety and accessibility under both current conditions and future scenarios extending to 2100. The vulnerability assessment confirms that these elevations are adequate for most of the evaluated sea level rise trajectories, with limited exceedances occurring under only the most extreme conditions.

The study also evaluated the operational reliability of ferry service by quantifying wave-induced berth downtime across 27 years of hindcast data. Without a breakwater, significant downtime occurs, particularly in summer months. The addition of either breakwater configuration results in a near-zero average monthly downtime and ensures that service reliability remains above typical thresholds throughout the year. The minimal difference in performance between the 300-ft and 400-ft breakwater options provides a clear argument in favor of the more efficient design.

Sediment transport modeling provides locally validated estimates of sedimentation rates and maintenance dredging needs for the approach channel and berth pockets. The six-year simulation indicates that sedimentation rates are highest in the first year following dredging and then stabilize, with average annual rates of 0.20 feet in the channel and 0.09 feet in the berth pockets. Based on these rates, maintenance dredging is expected to be required every 10 to 15 years for the channel and every 22 to 33 years for the berth pockets, depending on the operational depth maintained. These intervals support long-term planning and operational reliability, though ongoing

monitoring and data collection are recommended to refine future predictions and maintenance strategies.

Overall, this study provides a sound technical foundation for advancing the BWTPF project into permitting, detailed design, and construction. The recommended design is robust, cost-effective, and aligned with regional transportation and climate adaptation objectives. The methods applied reflect best practices in coastal engineering and support the long-term resilience and reliability of the ferry terminal infrastructure under both current and projected future coastal conditions.

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Appendix A – eTrac Memo, vertical datum conversion

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Memo

Title: Vertical Shift Analysis Between NOAA MLLW and City of Berkeley MLLW

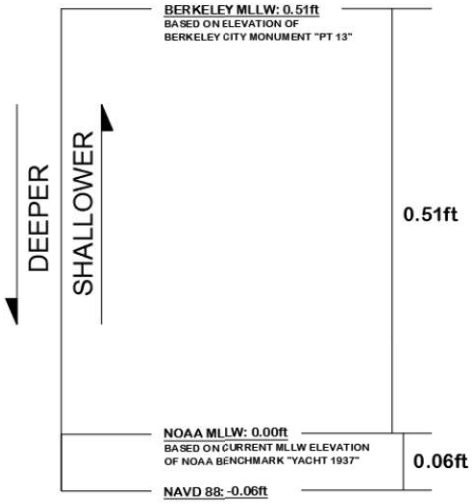
Project: Berkeley Water Transportation Pier Ferry (BWTPH)

Dredging operations at the Berkeley Marina are based on the current MLLW elevation at NOAA benchmark “YACHT 1937”. The MLLW elevation of "YACHT 1937" is documented at 10.99ft. Using an eTrac GNSS base station, eTrac observed an elevation on this benchmark of 10.93ft NAVD88. This would result in a vertical shift of the 0.06ft shallower to bring the NAVD88 elevation to the MLLW elevation.

For the Berkeley Water Transportation Pier Ferry Project, the provided vertical control uses the Berkeley City Monument “Pt 13” with an MLLW elevation of 19.96ft. Using the same eTrac GNSS base station, eTrac observed an elevation at this benchmark of 19.39ft NAVD88. This would result in a vertical shift of 0.57ft shallower to bring the NAVD88 elevation to the MLLW elevation.

Based on the eTrac observations at “YACHT 1937” and “PT 13”, the difference between the two NAVD88-to-MLLW shifts is 0.51ft. To adjust data based on “YACHT 1937” MLLW to bring it to “PT 13” MLLW, the data would be vertically adjusted shallower by 0.51ft.

Vertical Shift Analysis Between NAVD88, NOAA MLLW, and City of Berkeley MLLW



If you have any questions, please contact:

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Appendix B Sea Level Rise risk assessment

DECEMBER 2025

BERKELEY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SEA LEVEL RISE RISK ASSESSMENT

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DECEMBER 2025

BERKELEY WATER PIER FERRY (BWTPF) PROJECT

SEA LEVEL RISE RISK ASSESSMENT

PROJECT NO.

A272767

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
Final	12/12/2025	Sea Level Rise Risk Assessment	GAML	BRLW	JMC/ JRVS

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Attachment A - Regional Shoreline Adaptation Plan Maps for Project Vicinity

EXECUTIVE SUMMARY

This executive summary describes the sea level rise scenarios, design inputs, and risk assessment results for the BWTPF project along the Berkeley waterfront. It focuses on functional performance of the breakwater, ferry pier deck, and shoreline plaza through the 50-year design life (to 2075) and under end-of-century conditions (2100).

Highlights

- Design life and scenario: intermediate SLR scenario with allowance of 1.6 ft by year 2075 (project life).
- Still water and total water levels: Design SWL = 11.9 ft MLLW (Extreme tide 10.3 ft + SLR 1.6 ft). Structure minimum elevations were set using TWL (SWL + wave runup/green water).
- Performance through 2100: Under intermediate scenario (SLR 3.1 ft), all elements remain functional; overtopping increases but stays within tolerable ranges for managed public areas.
- High scenario stress test: Under high scenario (SLR 6.5 ft by 2100), plaza and deck are inundated; breakwater experiences overtopping far above pedestrian safety thresholds, requiring access restrictions and adaptive measures.

Inputs and Scenarios

- Nearest tide gauge used: San Francisco; Alameda was evaluated for comparison
- Vertical datum: Berkeley MLLW (MLLW = NAVD88 + 0.57 ft)
- Extreme Water Level: 100-yr (1% AEP) = 10.3 ft MLLW
- Design SWL: 11.9 ft MLLW (10.3 ft MLLW + 1.6 ft SLR)
- Design SLR allowance: 1.6 ft (Intermediate scenario, 2075)
- SLR scenarios considered for vulnerability assessment: Intermediate, Intermediate-High, High
- SLR projections (selected years) 2075: 1.6/2.6/3.6 (Int/Int-High/High); 2100: 3.1/4.8/6.5 (Int/Int-High/High)
- Structures assessed: Breakwater, Fixed Pier Deck, Shoreline Plaza

Results Summary

Number	Scenario	Year	SLR (ft)	SWL (ft MLLW)	Breakwater Status	Deck Status	Plaza Status
0 (Design)	Intermediate	2075	1.6	11.9	OK	OK	OK
1	Intermediate	2100	3.1	13.4	OK	OK	OK
2	Intermediate-High	2075	2.6	12.9	OK	OK	OK
3	Intermediate-High	2100	4.8	15.1	OT	OT	OT
4	High	2075	3.6	13.9	OT	OT	OT
5	High	2100	6.5	16.8	OT	Inundation	Inundation

OK: Element is above the design water elevation and will not be subjected to flooding during the scenario considered (overtopping < 1 l/m/s)

OT: Element will be subjected to overtopping during the scenario considered

1 Introduction

This report presents a comprehensive Sea Level Rise (SLR) Risk Assessment for the Berkeley Water Transportation Pier Ferry (BWTPF) Project, a transformative infrastructure initiative located along the Berkeley waterfront in California. The project intends to revitalize the existing Berkeley Pier by converting it into a modern, dual-purpose facility that functions both as a public ferry terminal and a recreational pier. This redevelopment is a critical component of the city's broader vision to enhance regional mobility, improve climate resilience, and expand public access to the shoreline. Figure 1-1 shows the location of the BWTPF Project within the regional shoreline context.

As part of the permitting process, the San Francisco Bay Conservation and Development Commission (BCDC) requires that all shoreline projects evaluate flood risk using the 100-year storm event, incorporating the best available projections of future sea level rise. In alignment with this requirement, the BWTPF SLR Risk Assessment applies the latest guidance from the 2024 State of California Sea-Level Rise Guidance by the Ocean Protection Council (OPC, 2024) and the BCDC Climate Change Policy Guidance (BCDC, 2025b). These frameworks emphasize a precautionary, science-based approach to infrastructure planning in the face of accelerating climate change.

This assessment evaluates the vulnerability of key project components, including the breakwater, fixed pier, floating structures, gangway, and shoreline plaza, under a range of sea level rise scenarios and planning horizons. It also identifies design elevations and performance thresholds that ensure the infrastructure remains functional, safe, and adaptable through the end of design life and beyond. The analysis incorporates both still water and total water level projections, considers overtopping and inundation risks, and provides recommendations for long-term resilience and adaptive capacity.

Given the project's 50-year design life, the assessment applies the Intermediate Sea Level Rise Scenario as the baseline for establishing design elevations and evaluating long-term performance. Additional analyses under the Intermediate-High and High scenarios for both 2075 and 2100 were completed to understand how the project may perform under more extreme future conditions. These evaluations provide a forward-looking basis for determining the elevations and adaptive capacity needed to ensure that the breakwater, pier, floating structures, and shoreline plaza continue to function safely and reliably through the design horizon and into later-century conditions.

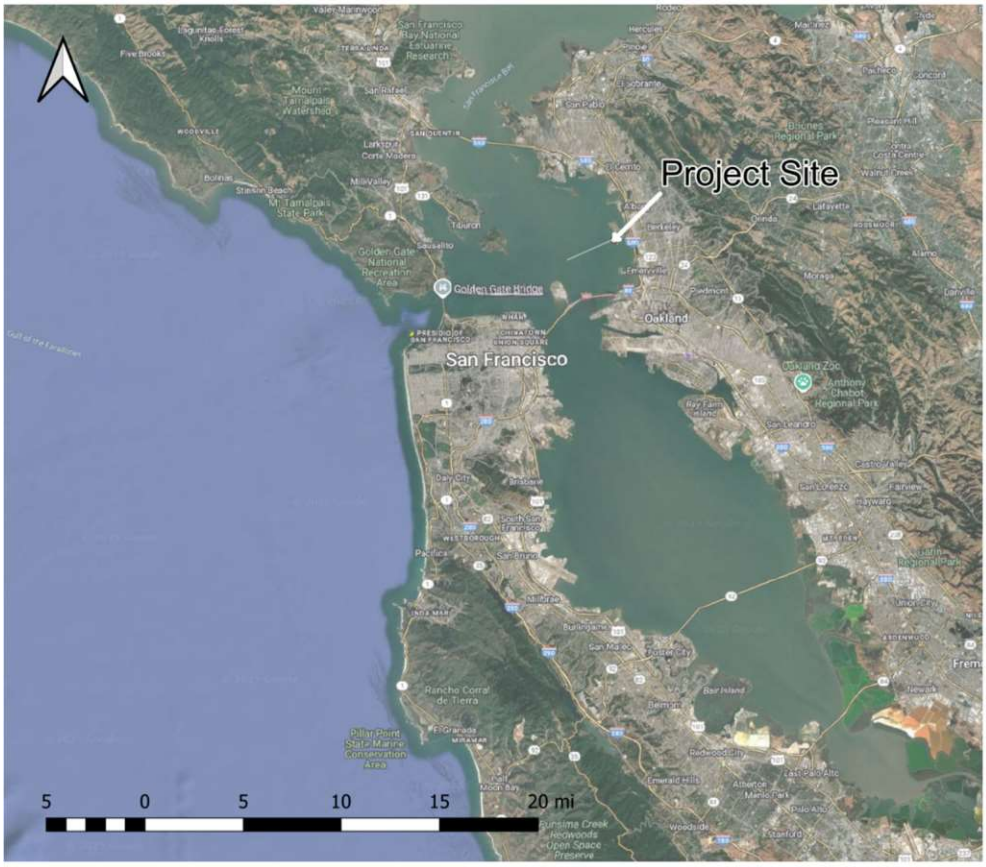


Figure 1-1: Project site

2 Current policies

The development of this assessment was guided by recommendations outlined in a set of planning and policy documents that collectively inform current approaches to sea level rise adaptation in the region.

- State of California Sea Level Rise Guidance, 2024 Science & Policy Update (OPC, 2024)
- BCDC Regional Shoreline Adaptation Plan, December 2024 (BCDC, 2024)
- BCDC San Francisco Bay Plan, March 2025 (BCDC, 2025a)
- BCDC San Francisco Bay Plan Climate change Policy Guidance, May 2025 (BCDC, 2025b)

The approach to defining sea level rise allowances for projects in California has evolved over recent years, reflecting ongoing advancements in scientific understanding and policy development. Recognizing the accelerating pace of climate change and its impacts on coastal regions, the state has continuously refined its approach to ensure that planning efforts are scientifically robust, precautionary, and adaptable to evolving climate projections.

The probabilistic approach used in the feasibility study was informed by data from the Intergovernmental Panel on Climate Change Fifth Assessment Report (IPCC, 2014), which evaluated sea level rise projections based on various Representative Concentration Pathways (RCPs). These pathways represented different greenhouse gas emissions scenarios and their corresponding impacts on global climate systems, including sea level changes.

There have been advancements in sea level rise science since then, including refinements in climate models, improved ice-sheet dynamics understanding, and better regional projections. These scientific advancements compelled a reassessment and update of the guidance. The 2024 California Sea Level Rise Guidance (OPC, 2024) integrates these updates, drawing from the most recent IPCC's Sixth Assessment Report (AR6) - (IPCC M.-D. V., 2021) and the Special Report on the Ocean and Cryosphere in a Changing Climate (SROCC) - (IPCC H.-O. P.-D., 2019). These reports provide deeper and updated insights into the primary drivers of sea level rise, such as the thermal expansion of seawater, the melting of glaciers and ice sheets, and shifts in ocean circulation patterns.

Building on this scientific foundation, the San Francisco Bay Conservation and Development Commission (BCDC) incorporated the state-defined projections into its

climate adaptation policies and planning frameworks. These projections serve as the baseline for evaluating shoreline vulnerability and guiding long-term adaptation strategies throughout the Bay Area. BCDC's updated Climate Change Policy Guidance (2025), the Regional Shoreline Adaptation Plan (2024), and the San Francisco Bay Plan (2025) all reference these projections to ensure consistency in how sea level rise is addressed in permitting, planning, and regional coordination.

By adopting the state's projections, BCDC has established a unified approach that supports coherent adaptation planning across jurisdictions.

2.1 Scientific advancements in the California 2024 guidance

One of the most notable advancements in the 2024 guidance is the shift from an exclusive reliance on probabilistic projections to a more comprehensive approach that incorporates a range of emissions pathways. This shift acknowledges the uncertainties associated with ice-sheet dynamics and tail-risk events, such as the potential collapse of the West Antarctic Ice Sheet, which could contribute to more extreme sea level rise scenarios. By spanning multiple emissions scenarios, the 2024 guidance enables decision-makers to better anticipate a range of potential outcomes and implement precautionary adaptation measures.

Another major update is the inclusion of localized assessments of vertical land motion, which were not as comprehensively addressed in the 2018 guidance. Vertical land motion, which includes both uplift and subsidence, can significantly affect relative sea level changes at specific locations. The 2024 guidance incorporates detailed localized projections for each of the 14 NOAA coastal tide gauges in California, offering site-specific information for planners. This improvement is particularly relevant for regions like the San Francisco Bay Area, where additional tide gauges in Alameda and Port Chicago provide refined local data.

The guidance also incorporates the latest methodologies for high-resolution coastal modeling, integrating new observational data from satellite altimetry, GPS, and advanced coastal monitoring programs. These data sources enhance the accuracy of sea level rise projections and improve the capacity to model storm surge, tidal influences, and wave dynamics in conjunction with rising sea levels. Additionally, the guidance accounts for compound flood risks, which consider the interaction of sea level rise with extreme precipitation events and riverine flooding.

2.2 Sea Level Rise planning framework

The BCDC Climate Change Policy Guidance recommends a stepwise process for incorporating sea level rise scenarios into planning and decision-making. This framework ensures that adaptation efforts are methodical, iterative, and responsive to new scientific findings. The steps include:

1 Identifying the Nearest Tide Gauge (Section 4)

2 Evaluating the Planning or Project Time Horizon (Section 5)

3 Choosing Multiple Sea Level Scenarios for Vulnerability Assessment and Evaluate Project Risk Tolerance (Section 6)

4 Conducting Risk Assessment (Section 7 and 8.2)

5 Exploring Adaptation Options (Section 8.3)

6 Preparing Adaptative Management Plan

For the Berkeley Water Transportation Pier Ferry Project, the definition of sea level rise allowance is critical in determining the minimum structural elevations required for the various components of the project. This includes ensuring that the elevations of the breakwater, pier, and shoreline plaza meet the anticipated future sea level rise conditions. The purpose of incorporating sea level rise allowances is to build in resilience to potential future conditions, ensuring the longevity and safety of the infrastructure over its designed lifespan.

To define these allowances, this study will follow a structured approach outlined in the 2024 California Sea Level Rise Guidance and the recent 2025 San Francisco Bay Plan Climate Change policies. By following this methodology, the project aims to minimize future risks while maintaining operational integrity based on the latest scientific knowledge.

3 Definitions and project water elevations

3.1 Water elevation definitions

The following definitions, related to water elevations and components of flood hazards, are used in the subsequent sections.

- SWEL: Still-Water Elevation is the water elevation that results from the superposition of astronomical tide, storm surge, and tsunami (if applicable) over an observation period. It does not include the effects of wave action (dynamic effect) and therefore constitutes a static water surface elevation that remains constant for minutes to hours. If the SWEL exceeds a certain shoreline elevation, the area behind it will experience extreme flooding.

- TWL: Total Water Level represents the maximum elevation the water reaches as the result of superimposing the static still-water elevation (SWEL) with the dynamic effects such as wind-waves, swell, and related wave runup. The TWL is reached for a few seconds at a time, but it is repeated during a storm event. If TWL exceeds a certain shoreline or structure elevation, the area behind it will be subject to overtopping, but the magnitude of flooding will vary.

The distinction between SWEL and TWL is important to note, particularly along coastal areas, because a shoreline exceeded by the SWEL constitutes an inundation or large-scale flooding scenario whereas embankments exceeded by TWL constitute an overtopping scenario that may only cause short-term nuisance flooding.

3.2 Project tidal elevations

3.2.1 Vertical datum

This project adopted the City of Berkeley vertical datum and refers to it as “MLLW” throughout this document. The Berkeley MLLW datum is 0.57 ft shallower than the NAVD88 vertical datum (+0.57’ NAVD88). The referenced elevations are given in feet-Berkeley MLLW (MLLW), unless noted otherwise.

3.2.2 Still Water Elevation (SWEL)

The San Francisco Bay Tidal Datums and Extreme Tides Study (AECOM, 2016) updated the tidal datums and extreme tides for the San Francisco Bay. The study modeled and presented detailed extreme tide information for the entirety of the Bay

shoreline. The model captures the Bay's hydrodynamic at a scale and resolution that exceeded available tide gauge data.

Table 3-1 reproduces the extreme tide elevation for point ID 519, coinciding with the Berkeley Pier. The provided elevations exclude waves impacts but include astronomical tides and storm surge.

The extreme water level for current conditions is 10.3 ft MLLW, with a return period of 100 years (1% Annual Exceedance Probability - AEP).

Table 3-1: Extreme Tide Elevations at Point ID 519 (AECOM, 2016)

Return Period (years)	Extreme Tide Elevation [ft-NAVD88]	Extreme Tide Elevation [ft-MLLW]
1	7.4	8.0
10	8.5	9.0
50	9.3	9.9
100	9.7	10.3

The design still water level at the project location is shown in Table 3-2. This elevation excludes waves impacts but includes astronomical tides, storm surges and sea level rise allowance for the design life of the project.

Table 3-2: Design Still Water Level

Parameter	Elevation [ft-NAVD88]	Elevation [ft- MLLW]
Extreme Water Level (1% AEP)	9.7	10.3
Sea Level Rise Allowance	1.6	1.6
Design Still Water Level	11.3	11.9

3.2.3 Total Water Level (TWL)

The TWL is evaluated for each structure, determining the minimum structure elevation used to evaluate the sea level rise risk in Section 8.2. The TWL was determined using combined extreme conditions (1% AEP storm surge and 1% AEP wave heights) adding a conservative margin to support the protection strategy and define minimum structure elevations.

3.2.4 Sea Level Rise projection

For the BWTPF Project, the projected sea level rise proposed by the California Natural Resources Agency and the Ocean Protection Council on “State of California Sea-Level Rise Guidance: 2024 Update” (OPC, 2024) was used.

Steps 1 through 4 of the California 2024 guidance were followed to define the design sea level rise allowance. The project does not consider a phased approach for adaptation, so steps 5 and 6 were not considered. These steps are described from Section 4 to 8 of this document.

4 Identifying the nearest tide gauge

The San Francisco and Alameda tide gauges are the nearest monitoring stations to the BWTFP site. Although the Alameda gauge is geographically closer, the San Francisco gauge was selected for use in this study as a more conservative basis for sea level rise projections. This choice is aligned with precautionary planning principles, ensuring that infrastructure design is robust under higher-end risk scenarios.

As shown in Table 4-1, the differences between projections from the two gauges can be significant. For the year 2100, San Francisco projections exceed those of Alameda by approximately 40% under the low-end scenario and by about 5% under the high-end scenario. These variations underscore the importance of selecting a tide gauge that supports risk-averse design decisions, particularly for long-term infrastructure. See Section 6 for definitions of sea level rise scenarios referenced in Table 4-1.

Table 4-1: Differences for Sea Level Rise Allowance (Year 2100) for San Francisco and Alameda Tide Gauges – relative to year 2000

Sea Level Rise Scenarios for Year 2100, in feet					
Tide Gauge	Low	Int-Low	Intermediate	Int-High	High
San Francisco	1.0	1.6	3.1	4.8	6.5
Alameda	0.6	1.2	2.8	4.4	6.2
Difference	-40%	-25%	-10%	-8%	-5%

5 Evaluating project time horizon

For the BWTPF, a design life of 50 years has been established, placing its planning horizon around the year 2075. According to the 2024 California Sea Level Rise Guidance, the time horizon is a critical determinant in selecting appropriate sea level rise values, as it directly influences design assumptions and risk tolerance. Given the absence of a phased adaptation strategy in this project, it is essential to incorporate a sea level rise projection that accounts for the full 50-year operational lifespan from the outset. This ensures that all structural elements are adequately elevated and resilient to anticipated sea level conditions throughout their service life, without reliance on future upgrades or retrofits. In addition to designing for 2075 conditions, the project will also include a vulnerability assessment for the year 2100 to evaluate longer-term exposure and inform future planning considerations, especially considering evolving climate scenarios.

This dual-focus strategy aligns with best practices in climate resilience by balancing present-day investment with future risk awareness.

6 Sea Level Rise scenarios and risk tolerance

California's 2024 Sea Level Rise Guidance outlines five distinct sea level rise scenarios: Low, Intermediate-Low, Intermediate, Intermediate-High and High. Each scenario is defined and labeled according to the target value of global mean sea level (GMSL) rise in 2100:

- Low (0.3 m or 1.0 ft by 2100)
- Intermediate-Low (0.5 m or 1.6 ft by 2100)
- Intermediate (1.0 m or 3.3 ft by 2100)
- Intermediate-High (1.5 m or 4.9 ft by 2100)
- High (2.0 m or 6.6 ft by 2100)

These scenarios are based on updated global climate projections, including findings from the IPCC Sixth Assessment Report, and reflect a range of possible future conditions driven by varying levels of greenhouse gas emissions, rates of ice sheet melt, and global policy action. Each scenario provides a different trajectory for sea level rise, with the Low and Intermediate-Low Scenarios reflecting highly optimistic futures requiring accelerated decarbonization and large-scale carbon removal technologies. In contrast, the High and Extreme Scenarios are considered more precautionary, particularly in the context of high warming levels and low-confidence but potentially high-impact ice sheet loss mechanisms.

The selection of a scenario for planning or design purposes depends on multiple factors, including the project's design life, the criticality of the asset, the level of acceptable risk, and the socio-economic and environmental consequences of over- or under-estimating future conditions. For most applications with medium to high risk aversion, the guidance recommends evaluating at least the Intermediate and Intermediate-High Scenarios. The High Scenario may be included when the consequences of failure are severe or when a worst-case analysis is needed. Conversely, the Low and Intermediate-Low Scenarios are generally not recommended for planning or design, as they are unlikely to be achieved under current global climate trajectories without transformative change.

For the BWTPF Project, the Intermediate Scenario from the 2024 California Sea Level Rise Guidance was selected to establish sea level rise design values. This scenario provides a reasonable and practical estimate for projected sea level rise by mid-to-late century, aligning with the project's 50-year design life.

California Sea-Level Rise Guidance (OPC, 2024) and San Francisco Bay Climate Change Policy (BCDC, 2025b) states that the Intermediate Scenario is recommended for sea level rise projections up to and including 2050. It also states that the Intermediate-High Scenario may be appropriate for multiuse paths and public access that are part of a transportation network or provide services that the public relies on. While the Intermediate Scenario is used for defining minimum elevations and structural thresholds, a broader risk assessment will also be conducted using the Intermediate-High and High Scenarios to account for more extreme, yet plausible, sea level rise outcomes and ensure long-term resilience of the project components.

The median value of sea level rise scenarios provided by the 2024 California Guideline (OPC, 2024) incorporate the local estimate of vertical land motion. Across all locations, the long-term rate of past vertical land motion is assumed to persist into the future, and future relative sea level rise is adjusted accordingly. This average rate of vertical land motion is near zero (0.1mm or 0.003ft uplift per year) at the project site.

7 Conducting risk assessments

For each marine structure included in the BWTPF Project (specifically the deck, breakwater, and plaza) a recommended minimum elevation will be determined based on a design water level that incorporates sea level rise projections from the Intermediate Scenario and a 50-year design life. This recommended elevation will serve as a baseline to ensure the structures are appropriately adapted to anticipated future conditions.

The final design elevations established by the project team, are informed by the minimum recommended values and consider additional factors such as constructability, aesthetics, operational requirements, and stakeholder input.

The structure-specific sea level rise risk assessment will evaluate potential risks under the Intermediate-High and High scenarios and extend the analysis beyond the 50-year design horizon, through the year 2100. Table 7-1 shows the sea level rise allowance used in design and vulnerability assessment for the marine structures.

Table 7-1 Risk Assessment Conditions

Condition	Scenario	Year	SLR Allowance (ft)	Differences from Design (ft)
Design	Intermediate	2075	1.6	0.0
Assessment 1	Intermediate	2100	3.1	1.5
Assessment 2	Intermediate- High	2075	2.6	1.0
Assessment 3	Intermediate- High	2100	4.8	3.2
Assessment 4	High	2075	3.6	2.0
Assessment 5	High	2100	6.5	4.9

8 Project design features and analysis

The project involves the design and implementation of a new ferry terminal at the foot of the existing Berkeley Pier, consisting of 1) Breakwater, 2) Fixed Pier, 3) Floats with Guide Piles, 4) Gangway, and 5) Shoreline Plaza, see Figure 8-1. Among these elements, the float, guide piles, and gangway were excluded from the sea level rise risk analysis. The rationale for their exclusion is detailed in the following points.

- Float and Guide Piles: the float is able to move vertically along the piles with the tides. The float is not vulnerable to flooding as it is able to rise and lower with tides and the top of the guide piles is set at an elevation above the TWL for all years considered.

- Gangway: the gangway connects the float and the fixed pier. The gangway is an aluminum structure hinged at the fixed pier end at elevation 17.6 ft and has rollers at the float end allowing it to rise and lower with the float. The lowest elevation of the gangway under flooding is the same as the fixed pier and has the same expected exposure to flooding, that will be estimated in the following sections.

A summary of the sea level rise evaluation for the remaining project components is presented in the subsequent sections

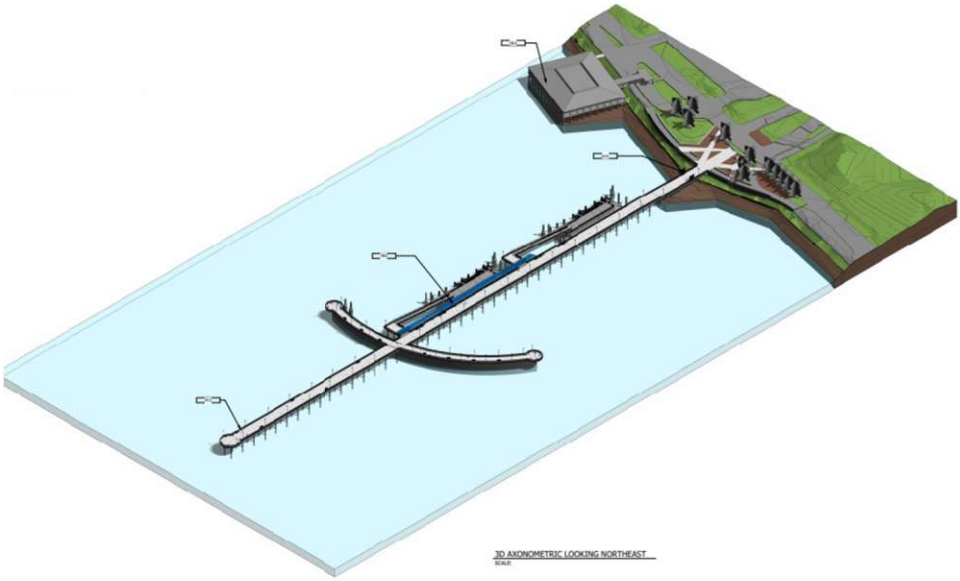


Figure 8-1: Berkeley Pier – Axonometric View (30% Submittal)

8.1 Sea Level Rise projections

The sea level rise projections are provided in Table 8-1. These sea level rise scenarios and projections for San Francisco are also adopted by FEMA/NOAA which coincide with the California 2024 Guidance (OPC, 2024). Both recommendations follow the same methodology from the most recent IPCC's Sixth Assessment Report (AR6) and the Special Report on the Ocean and Cryosphere in a Changing Climate.

The design sea level rise allowance for the project, based on a 50-year design life (2025–2075) and the Intermediate risk scenario, is 1.6 feet. Risk assessments were conducted for the scenarios highlighted in the table, specifically for the end of the project life in 2075 and the end-of-century condition in 2100. While the guidance from BCDC (BCDC, 2025b) recommends including a mid-century (2050) analysis, this study incorporates the end-of-design-life scenario, which is considered more conservative than the 2050 projection.

Table 8-1: Scenarios and Projection FEMA/NOAA and California Coastal Commission Sea Level Rise Policy Guidance 2024 Update and Design Interpolation for 2075

	Sea Level Rise Scenarios and Projection [feet]			
Year	Intermediate-Low	Intermediate	Intermediate-High	High
2050 (Mid-Century)	0.6	0.8	1.0	1.3
2070	1.0	1.4	2.2	2.9
2075* (Project Life)	1.1	1.6	2.6	3.5
2080	1.2	1.8	3.0	4.1
2100 (End of Century)	1.6	3.1	4.8	6.5

*Interpolation

8.2 Sea Level Rise risk assessment

The breakwater, deck, and plaza elevations were established based on the minimum recommendations from the 2075 planning horizon and the Intermediate Scenario from the 2024 California Sea Level Rise Guidance. This minimum elevation is the Total Water Level and includes the wave impacts on each structure added to the Still Design Water Level (11.9 ft MLLW).

- Breakwater: the minimum elevation includes an allowance for runup and acceptable overtopping on a vertical wall.

- Deck: green water elevation was included based on the modelled incident design wave along the deck. It is important to note that the deck is designed to withstand the wave loads without damage, to even higher than the design wave loads.

- Plaza: the wave impact (runup and acceptable overtopping) considering the design wave height at the toe of the revetment (partially protected by the breakwater) was added to the still water level.

Table 8-2 summarizes these minimum elevations for the design life (50 years) of the project, considering the Intermediate SLR scenario. The final design elevations are also included in the table. These final elevations were selected by the project team to meet broader functional and operational requirements. (COWI, 2025) provides details of the calculations for each minimum elevation structure including the overtopping volumes.

Figure 8-2 presents the typical breakwater cross-section, highlighting the crest elevation set at 17.6 ft MLLW. Figure 8-3 shows the corresponding pier section, where the elevation gradually decreases from 17.6 ft MLLW down to 16.5 ft MLLW at the connection with the plaza.

Table 8-2 Minimum and design elevations

Elevation (ft Berkeley MLLW)	Breakwater	Deck	Plaza
Minimum	17.6	15.2	15.8
Design	17.6	16.5-17.6*	16.5

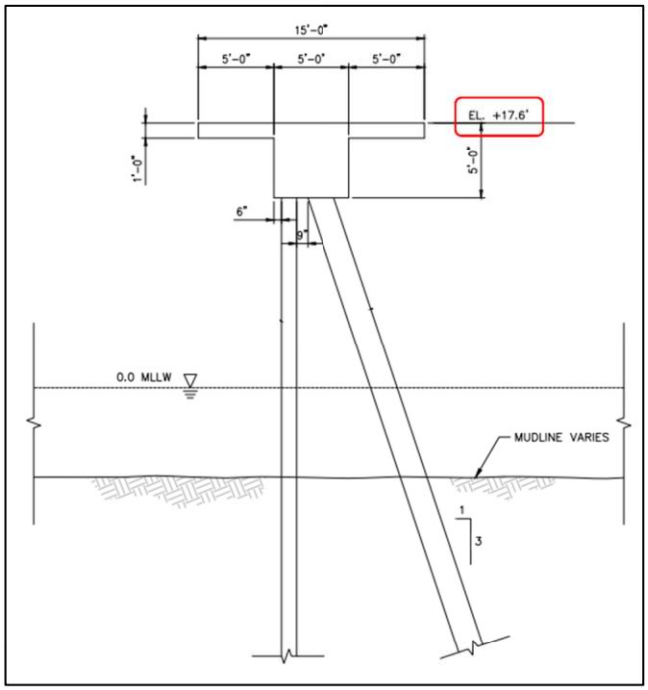


Figure 8-2: Breakwater section

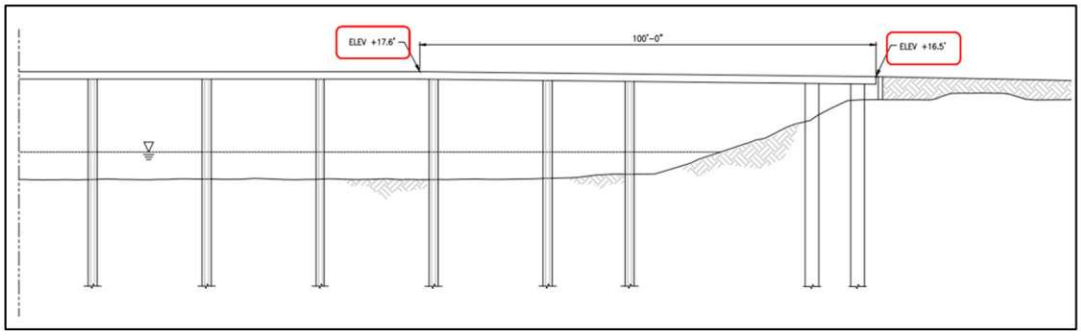


Figure 8-3: Pier and Plaza Elevation

To evaluate the long-term resilience of these design elevations, a Sea Level Rise Risk Assessment was conducted. The assessment considered the impact of future conditions beyond the 50-year design life and more conservative sea level rise scenarios. These conditions include projections for the years 2075 and 2100, under Intermediate, Intermediate-High and High scenarios.

The mean overtopping discharge rate is a key indicator of safety and hydraulic performance, particularly for coastal infrastructure adjacent to publicly accessible areas. It provides a practical measure of the volume of water expected to pass over the crest of a structure during storm events and is used to assess whether overtopping

remains within acceptable thresholds for pedestrian safety, user comfort, and structural serviceability.

In this assessment, the mean overtopping discharge rate will be evaluated for each sea level rise assessment scenario to determine whether design criteria are exceeded, to identify potential consequences for public use and structural integrity, and to support a risk-based understanding of when and where adaptive measures may be necessary. This approach ensures a consistent, safety-focused evaluation of long-term performance under evolving sea level and wave conditions.

The design threshold for overtopping volume, followed a conservative approach, by establishing mean overtopping volume to be less than 0.1 l/s/m. For this assessment, the criteria is stretched to the recommended range of 10-20 l/s/m for people at seawall and wave height 3.2 ft (EurOtop, 2018).

The following sections summarize the anticipated performance of the breakwater, deck, and plaza under each assessed sea level rise scenario. The analysis focuses on the ability of each component to continue functioning safely and effectively in the face of rising water levels and associated elevated wave impacts.

All the vertical elevations in this report are referenced to datum Berkeley MLLW.

8.2.1 Assessment 1: intermediate scenario – year 2100

The SLR allowance for year 2100 in an intermediate scenario (median) is 3.1 ft. When applied to the current 1% AEP extreme water level of 10.3 ft, this results in a projected design still water level of 13.4 ft (see Table 8-3). At this elevated water level, key shoreline and marine infrastructure elements, including the breakwater, ferry pier deck, and shoreline plaza, were reassessed to evaluate their functional and safety performance. The following subsections summarize the expected impacts of this sea level rise on each feature, focusing on crest freeboard reduction, wave overtopping behavior, wave slamming potential, and exposure to flooding or drainage challenges.

Table 8-3 Design Still Water Level for Assessment 1 – Intermediate Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	3.1 ft
Design Still Water Level	13.4 ft-MLLW

Breakwater: Under Assessment 1, the projected sea level rise of 3.1 ft increases the resulting mean overtopping discharge to approximately 1.0 l/s/m.

This value is near the upper threshold of 1.0 l/s/m, commonly referenced in (EurOtop, 2018) for safe pedestrian conditions in areas where wave action is non-impulsive and wave heights are moderate. Although this value exceeds the original design criterion of 0.1 l/s/m, selected as a conservative threshold for public safety behind a vertical wall, the condition remains within the broader acceptable range of 10–20 l/s/m identified in (EurOtop, 2018) for low wave conditions (H<3.3 ft) and well-managed pedestrian environments.

The breakwater remains functionally and operationally viable under this scenario, with overtopping discharges that exceed the original conservative design target but still fall within acceptable tolerances for safe public use.

Deck: Under Assessment 1, the projected sea level rise increases to 3.1 ft, which is 1.5 ft higher than the design SLR allowance used to establish deck elevation. This results in a revised still water level of approximately 13.4 ft MLLW, compared to the original 11.9 ft design level.

The combined water level (maximum green water elevation) under Assessment 1 is expected to increase to approximately from 15.7 to 16.7 ft, assuming the same wave height remains applicable.

The design deck elevation of 16.5 to 17.5 ft MLLW might be exceeded at some sections of the deck (closer to shore) during the design storm event. The structural integrity and operational function of the platform are therefore expected to be preserved through 2100 under the Intermediate Scenario, since platform components are designed to withstand these wave loads.

Plaza: Under Assessment 1 (Intermediate Scenario – Year 2100), the average overtopping discharge at the plaza is approximately 0.3 l/s/m, based on the design elevation of 16.5 ft MLLW. While this value exceed the conservative threshold of 0.1 l/s/m typically used for pedestrian areas, it remains below the broader upper limit of 10–20 l/s/m identified in (EurOtop, 2018) as tolerable for public areas where wave conditions are non-impulsive and the site includes adequate drainage and splash management.

At this discharge level, minor surface wetting and intermittent splash may occur during storm events or extreme tides, but public safety is not expected to be compromised, and the area would remain generally accessible and functional.

The plaza continues to perform within acceptable limits under Assessment 1 conditions. This result indicates that the structure is resilient through 2100 under the Intermediate Scenario, and no immediate adaptation measures are required beyond ongoing monitoring and operational readiness.

8.2.2 Assessment 2: intermediate-high scenario – year 2075

This assessment considers a projected sea level rise of 2.6 ft by the year 2075, consistent with the Intermediate-High Scenario from the 2024 California Sea Level Rise Guidance. This value represents a 1.0 ft increase above the design sea level rise allowance and reflects a more conservative planning trajectory that accounts for accelerated ice melt and higher-end global emissions outcomes.

The resulting increase in still water levels affects the performance of all coastal infrastructure elements, particularly in terms of reduced freeboard, increased overtopping, and greater exposure to flooding impacts. While the structures were not originally designed for this higher scenario, their vulnerability under these conditions provides important insight into operational limits and informs management considerations.

The following items describe the expected performance of the breakwater, ferry pier deck, and shoreline plaza under this scenario, including overtopping behavior, wave impact risks, and implications for public access and infrastructure resilience.

Table 8-4 Design Still Water Level for Assessment 2 – Intermediate Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	2.6 ft
Design Still Water Level	12.9 ft-MLLW

Breakwater: Under Assessment 2 (Intermediate-High Scenario – Year 2075), the projected sea level rise allowance increases to 2.6 ft, resulting in a still water level of 12.9 ft MLLW, approximately 1.0 ft above the design condition. This reduces the freeboard at the breakwater crest increasing the calculated mean overtopping discharge to approximately 0.5 l/s/m.

While this discharge exceeds the conservative design threshold of 0.1 l/s/m, it remains well within the tolerable range of 10–20 l/s/m for public areas under non-impulsive wave conditions, as described in the guidelines (EurOtop, 2018). Given the vertical structure and the wave climate characterized by design wave height of 2.6 ft, overtopping at this level is not expected to pose a safety risk to pedestrians.

In conclusion, the breakwater remains functional and safe under this scenario, with overtopping discharge levels that, while exceeding design targets, are still considered tolerable and manageable for the intended public use behind the structure.

Deck: Under Assessment 2, the projected sea level rise allowance increases to 2.6 ft, which represents a 1.0 ft rise above the design condition. This elevates the still water level to approximately 12.9 ft, compared to the original design level of 11.9 ft. The green water elevation could reach 16.2 ft.

Compared to the minimum deck design elevation of 16.5 ft, this results in a residual clearance of 0.3 ft, which remains within a safe operational range. The platform still provides sufficient clearance to maintain operational safety and performance under most storm conditions.

Assessment 2 conditions result in a reduction in freeboard compared to the design case but do not compromise the deck's performance.

Plaza: A sea level rise allowance of 2.6 ft is considered, 1.0 ft above the project's design scenario. The shoreline plaza, requiring a minimum elevation of 15.8 ft based on design calculations, was designed to an elevation of 16.5 ft, providing an additional margin of safety to support long-term usability, drainage efficiency, and integration with adjacent site elements.

Under this assessment condition, the calculated mean overtopping discharge is close to zero, indicating that the existing crest elevation and shoreline protection features continue to effectively prevent wave overtopping, even with the higher still water level. This suggests that no flooding or splash impact is expected during typical storm or tidal events through 2075, and the plaza can remain fully functional and safe for public access under these conditions.

This performance reinforces the effectiveness of the elevated design elevation and the current revetment slope in managing both sea level rise and storm wave activity. It also reflects the conservative design approach taken during planning, which successfully accommodates more extreme projections without the need for immediate adaptation.

8.2.3 Assessment 3: intermediate-high scenario – Year 2100

This assessment corresponds to the Intermediate-High Scenario for the year 2100, with a projected sea level rise of 4.8 ft (Table 8-5). This value is 3.2 ft higher than the design allowance and reflects a high-end trajectory that includes faster-than-expected ice sheet loss and limited success in global emissions reductions.

The significant increase in water levels reduces the available freeboard for marine structures and changes wave interaction behavior during extreme tide and storm conditions. This assessment tests the long-term resilience of the breakwater, deck, and shoreline plaza under higher exposure.

Table 8-5 Design Still Water Level for Assessment 3 – Intermediate-High Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	4.8 ft
Design Still Water Level	15.1 ft-MLLW

Breakwater: With a projected still water level of 15.1 ft MLLW, the vertical breakwater's crest freeboard is reduced, limiting its capacity to contain wave energy. Under this condition, the mean overtopping discharge increases to approximately 10 l/s/m.

While this discharge surpasses the original design criteria, it still falls within the range of 10–20 l/s/m identified in the guidelines (EurOtop, 2018) as conditionally acceptable for pedestrian areas, particularly when wave conditions are non-impulsive, as is the case for this site (wave heights below 3.3 ft). (EurOtop, 2018) notes that at these discharge levels, public safety can still be maintained if the space behind the structure is designed or managed to handle occasional overtopping, such as splash-tolerant paving, passive drainage, and limited occupancy during adverse conditions.

Despite the exceedance of the initial design target, the breakwater continues to provide valuable coastal protection and risk reduction. The structure maintains its wave energy dissipation capacity, prevents direct storm wave attack, and significantly reduces the frequency and severity of flooding compared to an unprotected condition.

Deck: The projected sea level rise reaches 4.8 ft, representing a 3.2 ft increase over the design SLR allowance. This results in a still water level of approximately 15.1 ft, still under the design deck elevation range of 16.5-17.6 ft.

The green water elevation exceeds the deck elevation. However, the deck can resist wave loads without damage, including loads exceeding the design wave conditions, as wave loading was not a governing factor in its design.

Plaza: Under Assessment 3 conditions, the projected sea level rise reaches 4.8 ft, increasing the still water level and wave impact potential. At the design elevation of 16.5 ft MLLW, the calculated mean overtopping discharge at the plaza is less than 15 l/s/m.

This value exceeds the original conservative design target of near-zero overtopping but remains within the broader range of 10–20 l/s/m identified by (EurOtop, 2018) as conditionally tolerable for public areas when wave conditions are non-impulsive and infrastructure is designed to manage occasional surface flooding. Given the sheltered

wave height, overtopping at this level would likely result in intermittent splashing, surface wetting, and temporary accumulation of water, particularly during high tides or storm events.

While this may reduce the comfort of usability of the plaza during extreme conditions, it does not pose a critical hazard, and the underlying structure (including the revetment, seawall, and surface grading) will remain intact. The surface drainage system, if properly maintained, would be expected to accommodate these volumes without compromising the long-term functionality of the space.

8.2.4 Assessment 4: high scenario – year 2075

This assessment corresponds to the High Scenario for the year 2075, with a projected sea level rise of 3.6 ft (see Table 8-6). This value is 2.0 ft higher than the SLR design allowance.

Table 8-6 Design Still Water Level for Assessment 4 – High Scenario, Year 2075

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	3.6 ft
Design Still Water Level	13.9 ft-MLLW

Breakwater: With a projected still water level of 13.9 ft MLLW, the vertical breakwater's crest freeboard is reduced, also reducing its capacity to contain wave energy. Under this condition, the mean overtopping discharge increased to approximately 3.0 l/s/m.

While this discharge surpasses the original design criteria, it still falls under the range of 10–20 l/s/m identified in the guidelines (EurOtop, 2018) as conditionally acceptable for pedestrian areas, particularly when wave conditions are non-impulsive, as is the case for this site (wave heights below 3.3 ft). EurOtop (EurOtop, 2018) notes that at these discharge levels, public safety can still be maintained if the space behind the structure is designed or managed to handle occasional overtopping, such as splash-tolerant paving, passive drainage, and limited occupancy during adverse conditions.

Despite the reduction in crest freeboard, the vertical breakwater remains structurally sound and fully capable of withstanding the projected still water level of 13.9 ft MLLW. While overtopping volumes are minimal and not expected to compromise the integrity of the breakwater, it is advisable to implement precautionary measures, particularly during high tide or storm conditions. Limiting pedestrian access to the breakwater,

especially in areas with restricted visibility or no clear line of sight to the water, would enhance public safety without necessitating structural modifications.

Deck: Under Assessment 4, the projected sea level rise allowance increases to 3.6 ft, which represents a 2.0 ft rise above the design condition. This elevates the still water level to approximately 13.9 ft, still under the design deck elevation range of 16.5-17.6 ft MLLW.

Although the calculated green water elevation exceeds the deck height, the deck can withstand the associated wave forces, even for conditions more severe than those used for design. Wave loading did not control the deck design, and no damage is expected under these scenarios

Plaza: The shoreline plaza, originally defined with a minimum elevation of 15.8 ft, was constructed to a design elevation of 16.5 ft, providing an additional margin of safety to support long-term usability, drainage efficiency, and integration with adjacent site elements.

Under this assessment condition, the calculated mean overtopping discharge is slightly higher than the design conditions, but still under the design threshold of 1.0 l/m/s, indicating that the existing crest elevation and shoreline protection features continue to effectively prevent wave overtopping, even with the higher still water level. This suggests that no flooding or splash impact is expected during typical storm or tidal events through 2075 in this SLR scenario, and the plaza can remain fully functional and safe for public access under these conditions.

This performance reinforces the effectiveness of the elevated design elevation and the current revetment slope in managing both sea level rise and storm wave activity. It also reflects the conservative design approach taken during planning, which successfully accommodates more extreme projections without the need for immediate adaptation.

8.2.5 Assessment 5: high scenario – year 2100

This assessment corresponds to the High Scenario for the year 2100, with a projected sea level rise of 6.5 ft (see Table 8-7). This value is 4.9 ft higher than the design allowance and reflects the long-term implications of accelerated climate change. Such a significant increase in sea level would substantially reduce the vertical clearance of coastal infrastructure and challenge the assumptions made during initial design phases.

The significant increase in water levels reduces the available freeboard for marine structures and changes wave interaction behavior, particularly during extreme tide and storm conditions. This assessment tests the long-term resilience of the breakwater,

deck, and shoreline plaza under high-exposure, high-impact scenarios. The results identify critical performance thresholds under climate extremes. This assessment includes a high emission scenario beyond the design life of the project. One consequence of this extreme rise in water level is the increase in the design wave height. However, for this assessment, the original design wave height was retained, as the results already indicate elevated levels of overtopping and flooding of the designed structures.

Table 8-7 Design Still Water Level for Assessment 5 – High Scenario, Year 2100

Parameter	Elevation
Extreme Water Level (1% AEP)	10.3 ft-MLLW
Sea Level Rise 2100	6.5 ft
Design Still Water Level	16.8 ft-MLLW

Breakwater: With a projected still water level of 16.8 ft MLLW, the vertical breakwater's crest freeboard is reduced, limiting its capacity to contain wave energy. Under this condition, the mean overtopping discharge increases to over 75.0 l/s/m.

This discharge surpasses the original design criteria and falls beyond the range of 10–20 l/s/m identified in the guidelines (EurOtop, 2018) as conditionally acceptable for pedestrian areas. EurOtop (EurOtop, 2018) notes that at these discharge levels, public safety cannot be maintained. The area behind the breakwater should not be used by pedestrians under these storm conditions.

Despite this exceedance, the breakwater continues to provide valuable coastal protection. The structure maintains its wave energy dissipation capacity, prevents direct storm wave attack, and significantly reduces the frequency and severity of flooding compared to an unprotected condition.

Past the breakwater design life, particularly during high tides and storm events, overtopping at this magnitude will cause temporary splash flooding, surface water accumulation, and no accessibility in the recreational area behind the breakwater. However, with adaptations, such as elevated hardscape edges, improved drainage, and controlled public access during extreme conditions, the space can remain safe and functional for some of its intended use.

Deck: The projected sea level rise reaches 6.5 ft, representing a 4.9 ft increase over the design SLR allowance. This results in a still water level of approximately 16.8 ft, already exceeding the minimum design deck elevation of 16.5 ft.

Assessment 5 clearly indicates that the deck will experience unacceptable flooding conditions for safe public use. However, it is important to note that the deck was designed with robust structural integrity and is fully capable of withstanding periodic inundation without sustaining structural damage. Public access should be restricted during these extreme conditions, especially in areas lacking clear visibility or escape routes.

Plaza: Under Assessment 5 conditions, the projected sea level rise of 6.5 ft results in a still water level of approximately 16.8 ft MLLW, which exceeds the Plaza's design elevation of 16.5 ft MLLW. This elevation leads to inundation of the Plaza during high tides and storm events, increasing the potential for wave impact and crest erosion. The combination of elevated water levels and wave action may destabilize the crest and erode the land behind it, compromising the long-term integrity of the surrounding landscape and infrastructure.

While the Plaza may retain some structural resilience, it is not safe for pedestrian use under these conditions. The risk of splash flooding, slippery surfaces, and unpredictable wave overtopping would present a hazard to public safety.

8.2.6 Assessment summary

This section provides a consolidated overview of the performance of key project components (Breakwater, Ferry Pier Deck, and Shoreline Plaza) under six sea level rise scenarios spanning from the project design condition (Intermediate Scenario, Year 2075) to the High Scenario for Year 2100. Table 8-8 shows a summary of the SLR conditions considered in this assessment.

Table 8-8 Summary of SLR Risk Assessment Conditions

Assessment	Scenario	Year	SLR Allowance (ft)	SWL (ft MLLW)
0 - Design	Intermediate	2075	1.6	11.9
1	Intermediate	2100	3.1	13.4
2	Intermediate-High	2075	2.6	12.9
3	Intermediate-High	2100	4.8	15.1
4	High	2075	3.6	13.9
5	High	2100	6.5	16.8

The assessments demonstrate that the infrastructure has been thoughtfully designed to withstand a wide range of future conditions, with most components maintaining functionality and safety well beyond their original design thresholds.

- Breakwater: Performs reliably under all intermediate-high scenarios and continues to offer substantial coastal protection even under high sea level rise conditions. Under the High Scenario for Year 2100, overtopping volumes exceed safe limits for pedestrian areas, requiring restricted access and adaptive design measures.

- Ferry Pier Deck: Maintains structural integrity across all scenarios. Even under elevated water levels (including inundation), the deck remains resilient, with only the most extreme conditions requiring access restrictions.

- Shoreline Plaza: Remains functional and safe under all but the most extreme scenario. Inundation and erosion risks emerge under the High Scenario for Year 2100.

Table 8-9 summarizes the flooding potential and operational status of each component under each scenario. Overall, the results affirm the robustness of the design and its capacity to accommodate evolving climate conditions with minimal disruption.

Table 8-9: Summary of Flooding Potential of Berkeley Pier During Design Conditions (elevations in ft MLLW)

Assessment Number	**0 - Design**	**1**	**2**	**3**	**4**	**5**
Scenario	Intermediate	Intermediate	Intermediate-High	Intermediate-High	High	High
Sea Level Rise Allowance (ft)	1.6	3.1	2.6	4.8	3.6	6.5
Year	2075	2100	2075	2100	2075	2100
SWL Elevation (ft MLLW)	11.9	13.4	12.9	15.1	13.9	16.8
Breakwater (EL +17.6 ft)	**OK**	**OK**	**OK**	**OT**	**OT**	**OT**
Pier (EL +16.5 to +17.6 ft)	**OK**	**OK**	**OK**	**OT**	**OT**	**I**
Shoreline Plaza (EL +16.5 ft)	**OK**	**OK**	**OK**	**OT**	**OK**	**I**

Notes:

OK	Element is above the Design Water elevation and will not be subjected to flooding during the scenario considered (overtopping < 1 L/m/s)
OT	Element will be subjected to overtopping during the scenario considered
I	Element will be subjected to inundation during the scenario considered

8.3 Resiliency and adaptability

The BWTPF Project was designed with emphasis on resiliency and adaptability to ensure long-term performance under evolving SLR conditions. The marine infrastructure components (breakwater, ferry pier deck, and shoreline plaza) were evaluated across a range of SLR scenarios extending through the year 2100, including Intermediate, Intermediate-High, and High projections from the 2024 California Sea Level Rise Guidance.

The project's design elevations were selected to meet the minimum requirements for the 2075 planning horizon but also to provide additional freeboard and structural robustness to accommodate more extreme future conditions. As demonstrated in Section 8.2, the breakwater, deck, and plaza maintain acceptable performance under all scenarios up to the Intermediate-High projection for 2100 and continue to offer partial functionality even under the High Scenario for 2100.The resiliency features include:

- Elevated design thresholds that exceed minimum recommendations, providing buffer capacity against overtopping and inundation.
- Material and structural specifications that support durability under splash, wave impact, and intermittent flooding.
- Passive drainage and splash-tolerant surfaces that mitigate the effects of overtopping in pedestrian areas.

Adaptability for future conditions

While the project does not incorporate a phased adaptation strategy, its design allows for future interventions to extend functionality and safety. These include:

- Operational adaptations, such as temporary access restrictions during extreme tide or storm events.
- Infrastructure enhancements, including elevated hardscape edges, reinforced drainage systems, and splash barriers.
- Monitoring and maintenance protocols to track performance and implement responsive measures as conditions evolve.

The breakwater, for example, remains structurally sound even under overtopping rates exceeding safe pedestrian thresholds in the High Scenario for 2100. Adaptive management, such as limiting access and improving drainage, can preserve its protective function without requiring full reconstruction.

Similarly, the ferry pier deck and shoreline plaza retain structural integrity under inundation conditions, allowing for managed use and potential retrofits to maintain public safety and usability.

Alignment with regional guidance

The resiliency and adaptability strategy aligns with the principles outlined in the BCDC Climate Change Policy Guidance (BCDC, 2025b), the Regional Shoreline Adaptation Plan (BCDC, 2024), and the OPC's 2024 Sea Level Rise Guidance (OPC, 2024). These frameworks emphasize precautionary planning, flexible design, and iterative adaptation to support long-term infrastructure viability in the face of climate uncertainty.

8.4 Project vicinity

Attachment A provides the Regional Shoreline Adaptation Plan (RSAP) Maps for the area of the Berkeley Water Transportation Pier Ferry Project. The following information can be determined from the maps:

- Intermediate Scenario Map: Surface flooding is shown within the shoreline plaza area of the project. As noted in earlier sections of this report, this area will be elevated to +16.5 ft MLLW. The maps also indicate inundation in portions of the adjacent land and at the Berkeley Marina.

- Intermediate-High Scenario Map: Flooding is identified along the shoreline immediately south of the project site and north of the peninsula, near the Berkeley Marina.

- High Scenario: Similar to the Intermediate-High Scenario, surface flooding is observed in the same shoreline section south of the project and north of the peninsula, at the Berkeley Marina.

The RSAP is a regional planning framework developed to address sea level rise across the San Francisco Bay. Under Senate Bill 272 (SB 272), local governments in the Bay Area are required to develop Subregional Shoreline Adaptation Plans. The RSAP Atlas is a supporting tool that helps jurisdictions meet these requirements by providing region-wide data on coastal flood hazards, community vulnerability, existing conditions, strategic regional priorities, and suitability for nature-based adaptation.

The maps referenced in this report were sourced directly from the RSAP Atlas tool. It is recommended that users exercise caution when interpreting these maps, particularly because the Intermediate-High and High scenarios show very similar flooding extents in the project vicinity.

9 Conclusions

The Berkeley Water Transportation Pier Ferry Project has been evaluated for its resilience and adaptability to future sea level rise (SLR) conditions, using the latest guidance from the State of California and regional policy frameworks. The assessment considered a range of scenarios, including Intermediate, Intermediate-High, and High projections, to understand how the breakwater, ferry pier deck, and shoreline plaza would perform through the project's 50-year design life (through 2075) and beyond (up to the year 2100).

Under the baseline (design) Intermediate scenario, which incorporates a sea level rise allowance of 1.6 feet for the year 2075, all major infrastructure components are expected to remain fully functional and safe. The breakwater, deck, and plaza are designed with conservative elevation thresholds, ensuring that they are positioned above the projected still water level and can withstand the combined effects of tides, storm surges, and wave action. Even as sea level rise projections increase for the end-of-century condition (SLR of 3.1 feet by 2100 under the Intermediate scenario), the structures continue to perform reliably. Overtopping rates rise modestly but remain within tolerable limits for managed public spaces, and the deck and plaza maintain their operational integrity.

For the Intermediate-High scenario, with SLR allowances reaching 2.6 feet by 2075 and 4.8 feet by 2100, the breakwater and plaza experience increased overtopping, but these rates are still manageable with appropriate drainage and operational protocols. The deck retains sufficient clearance and structural strength, even as freeboard is reduced.

In the High scenario, representing the most severe projections with SLR allowances of 3.6 feet by 2075 and 6.5 feet by 2100, the infrastructure faces challenges. The breakwater is subjected to overtopping volumes that exceed pedestrian safety thresholds, and both the deck and plaza are at risk of inundation during extreme tide and storm events. Under these conditions, public access would need to be restricted, and adaptive measures such as enhanced drainage, splash-resistant surfaces, and revised operational protocols would be required to maintain safety and usability.

The BWTPF Project's design reflects a precautionary approach, with elevated thresholds and materials that provide buffer capacity against overtopping and flooding. The assessment confirms that the breakwater continues to offer coastal protection, the deck maintains its structural integrity, and the plaza remains usable with minor adaptive measures, even under the most extreme scenarios considered. These conclusions highlight the project's alignment with best practices in adaptive management and its readiness to meet the challenges of sea level rise and climate uncertainty through the end of the century.

10 References

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Attachment A - Regional Shoreline Adaptation Plan Maps for Project Vicinity

rsap_atlas_map



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0 0.05 0.1 0.2 mi

0 0.07 0.15 0.3 km

Cities & Census Designated Places (Analysis) (CALFire 2022)

3.1 ft (2100 Intermediate) SLR Scenario (BCDC 2024)

Existing Bay

Surface Flooding (e.g. Tidal Inundation, Storm Surge, Emergent Groundwater)

Shallow Groundwater (0-9 ft below ground)

BCDC, USGS, ART, Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community

rsap_atlas_map



12/15/2025, 1:23:44 PM

1:8,862

0 0.05 0.1 0.2 mi

0 0.07 0.15 0.3 km

Cities & Census Designated Places (Analysis) (CALFire 2022)

4.9 ft (2100 Intermediate-High) SLR Scenario (BCDC 2024)

Existing Bay

Surface Flooding (e.g. Tidal Inundation, Storm Surge, Emergent Groundwater)

Shallow Groundwater (0-9 ft below ground)

BCDC, USGS, ART, Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community

rsap_atlas_map



12/15/2025, 1:27:25 PM

1:8,862

0 0.05 0.1 0.2 mi

0 0.07 0.15 0.3 km

Cities & Census Designated Places (Analysis) (CALFire 2022)

6.6 ft (2100 High) SLR Scenario (BCDC 2024)

Existing Bay

Surface Flooding (e.g. Tidal Inundation, Storm Surge, Emergent Groundwater)

Shallow Groundwater (0-9 ft below ground)

BCDC, USGS, ART, Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community

Appendix C – Wave disturbance maps

Overall wave disturbance maps – 400 ft breakwater length

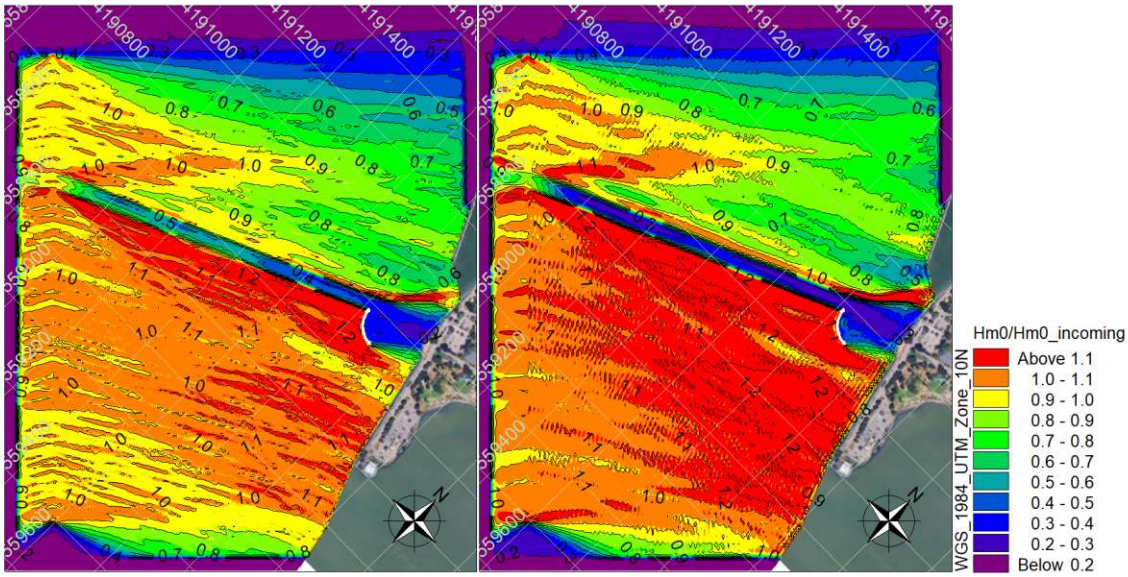


Figure C- 1 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

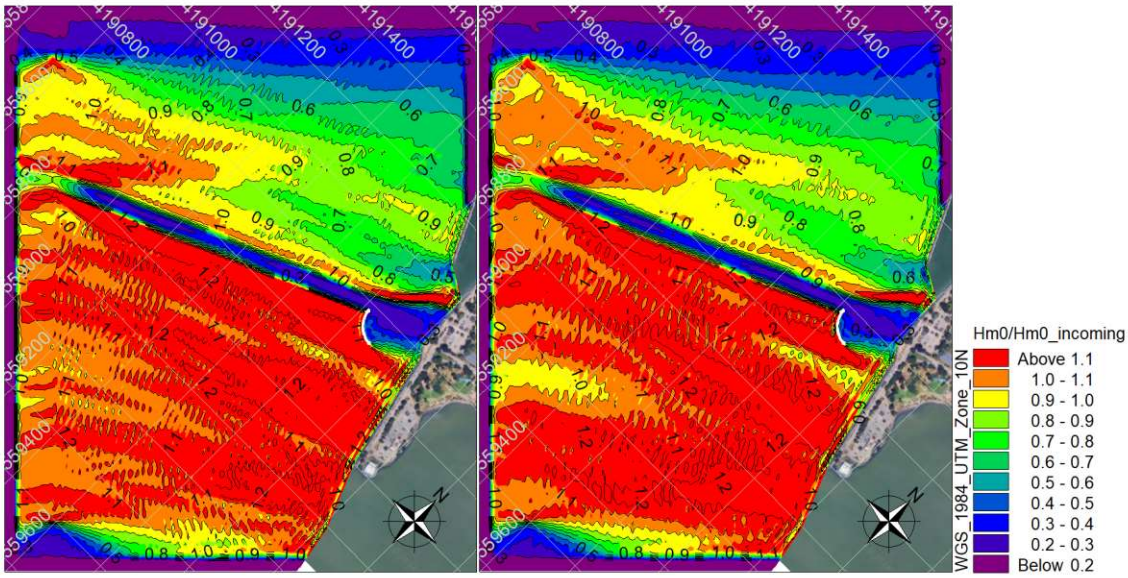


Figure C- 2 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 se, 400 ft breakwater length.*

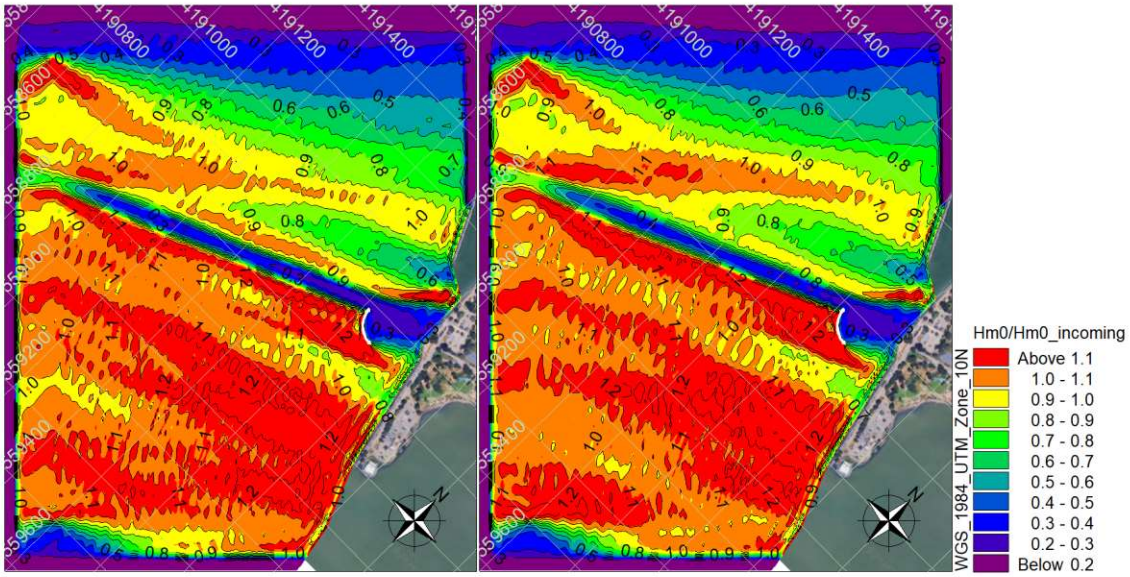


Figure C- 3 *Wave disturbance plot for wave direction = 240°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.

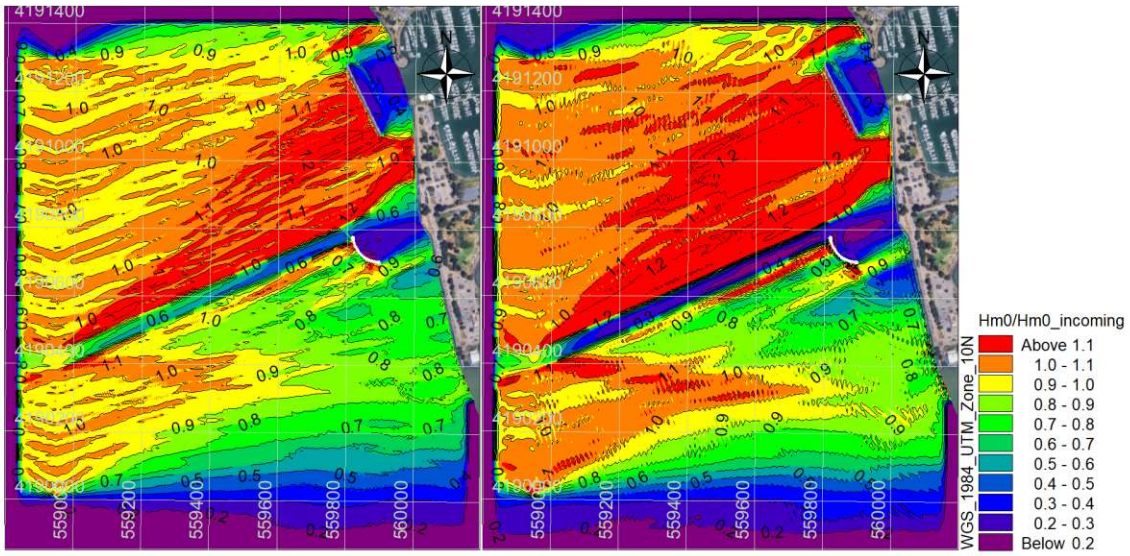


Figure C- 4 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.

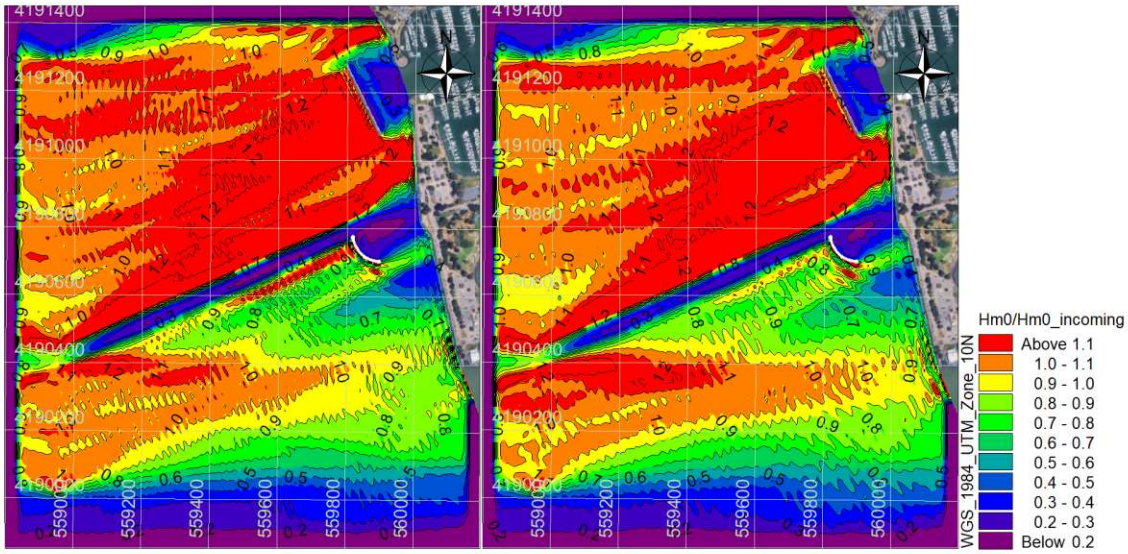


Figure C- 5 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

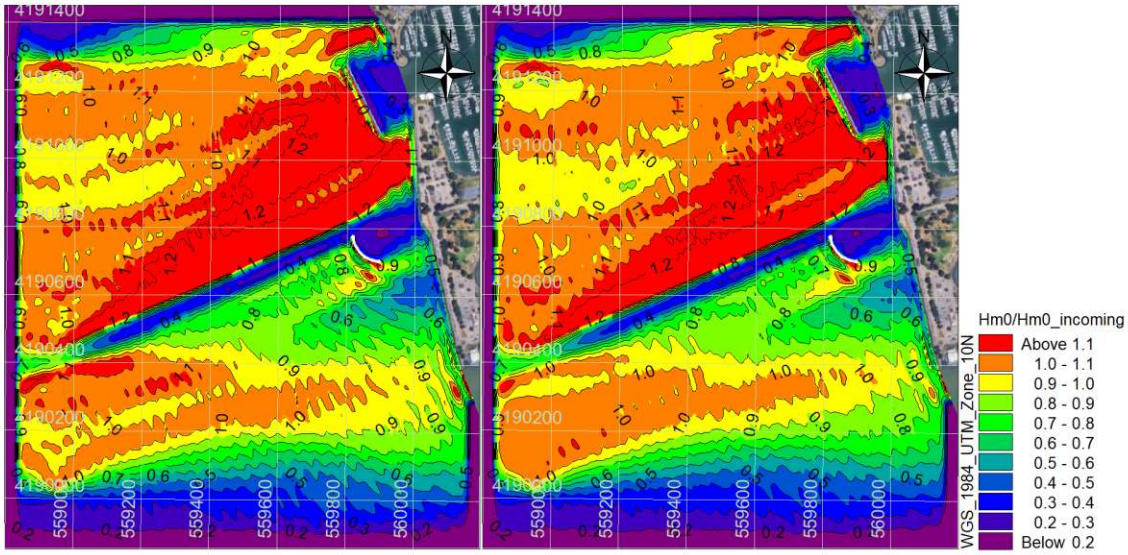


Figure C- 6 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.

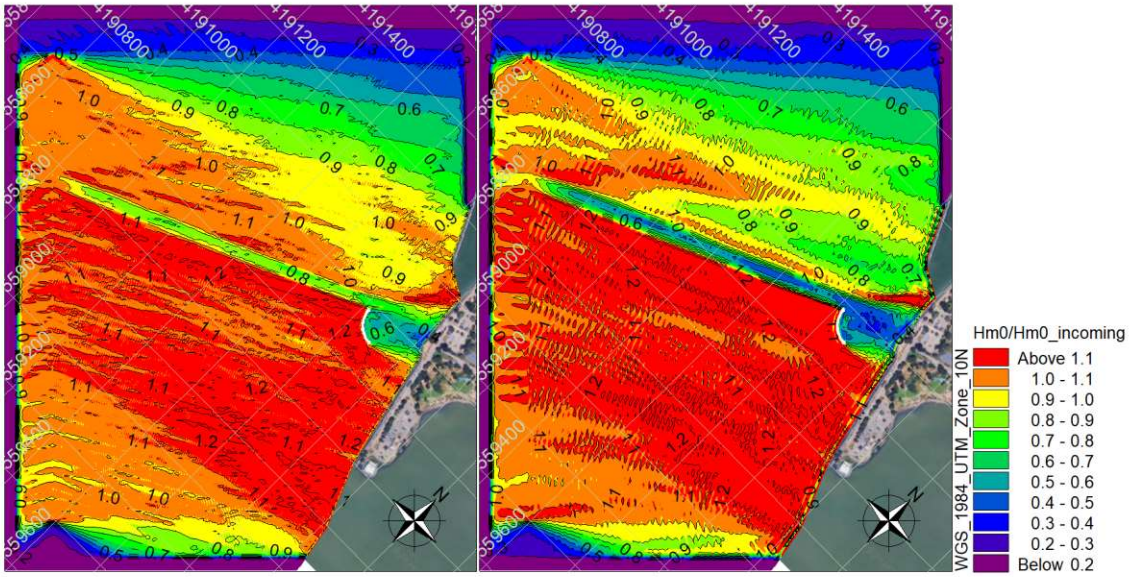


Figure C- 7 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

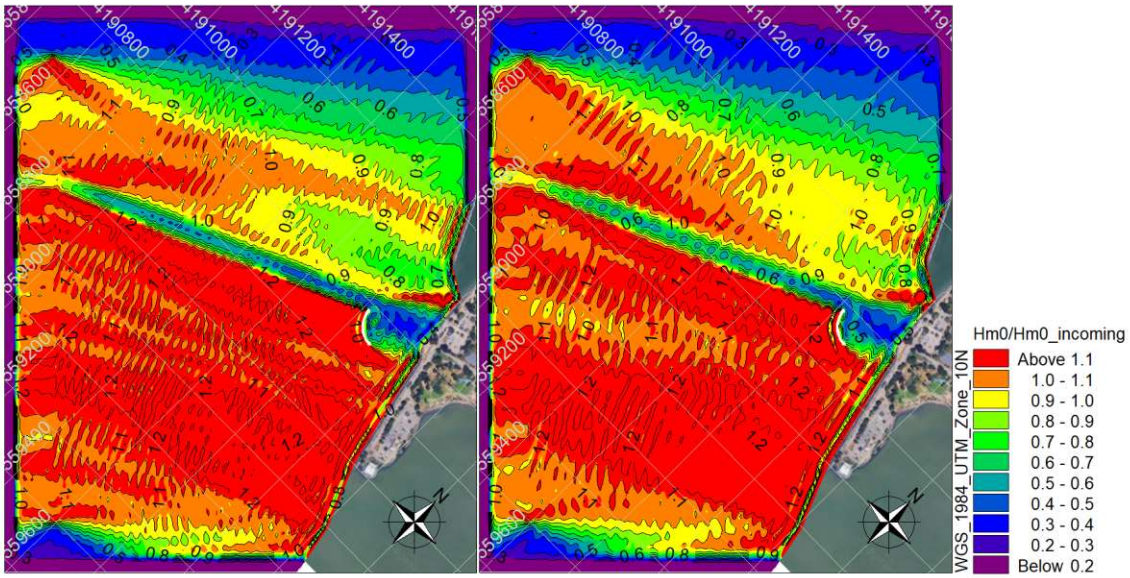


Figure C- 8 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.*

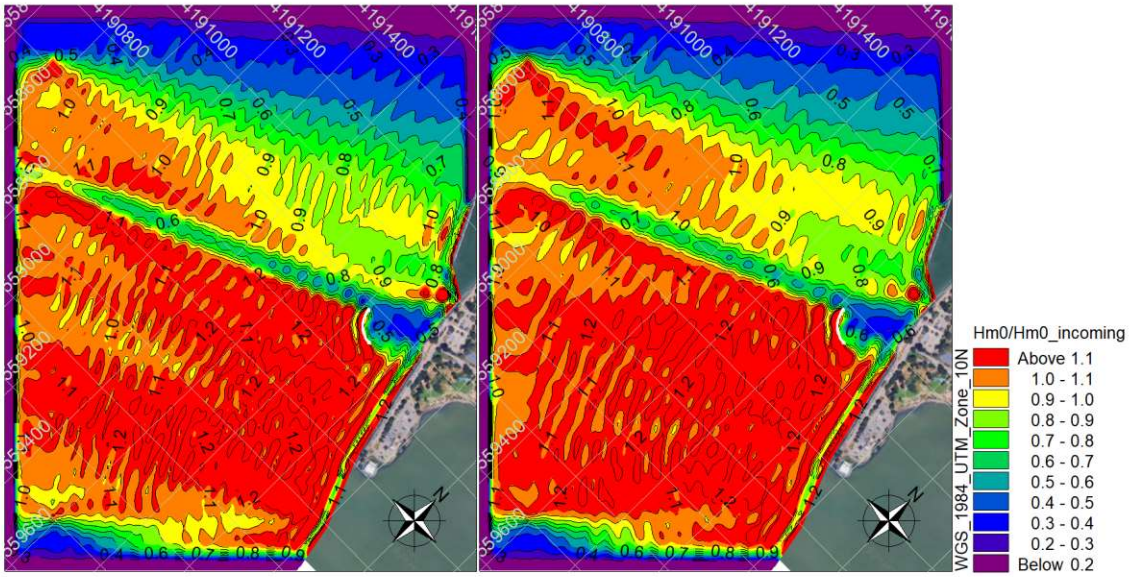


Figure C- 9 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

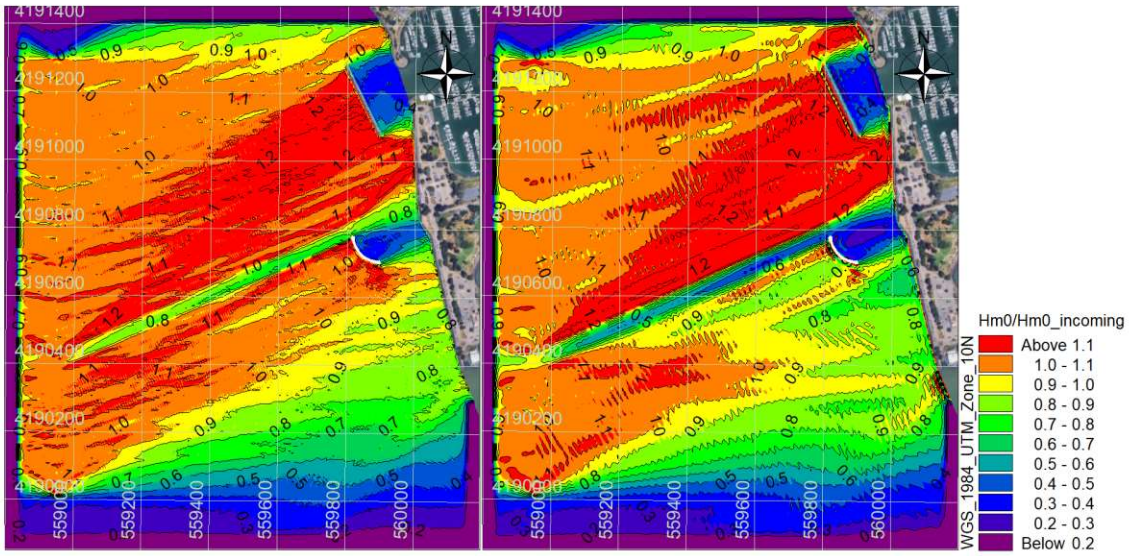


Figure C- 10 *Wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

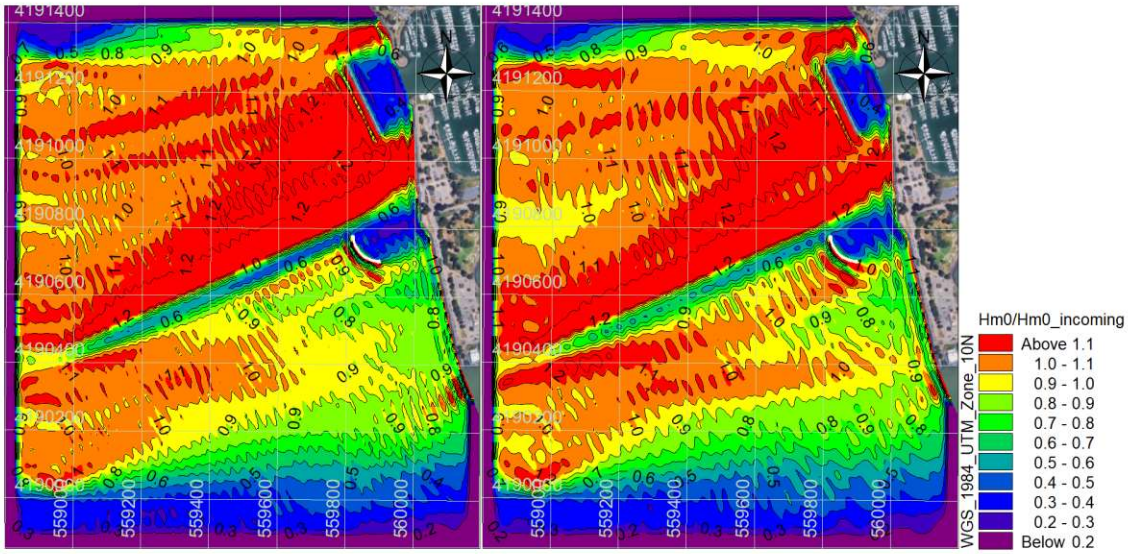


Figure C- 11 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

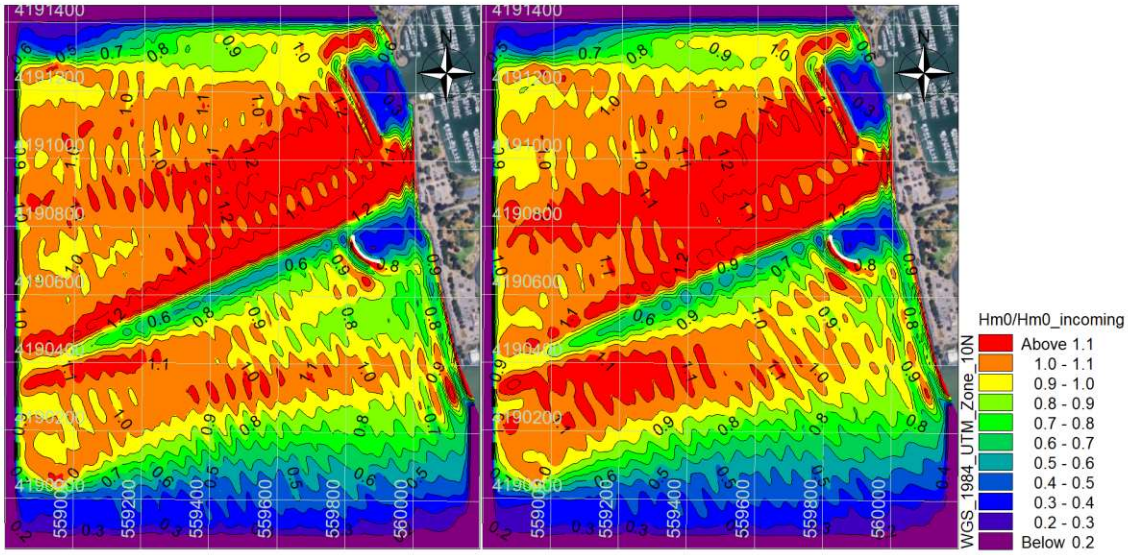


Figure C- 12 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.

Ferry terminal wave disturbance maps – 400 ft breakwater length

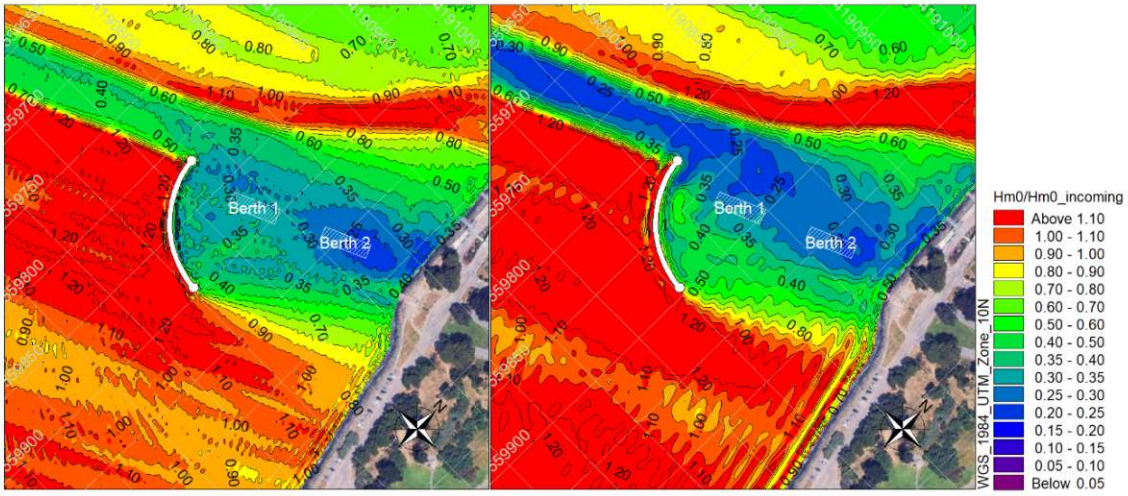


Figure C- 13 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

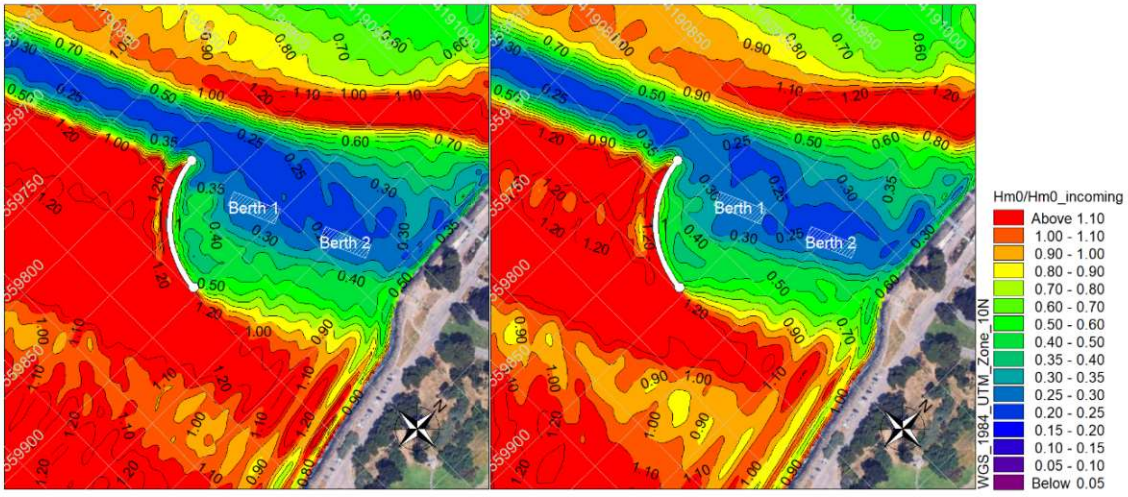


Figure C- 14 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.*

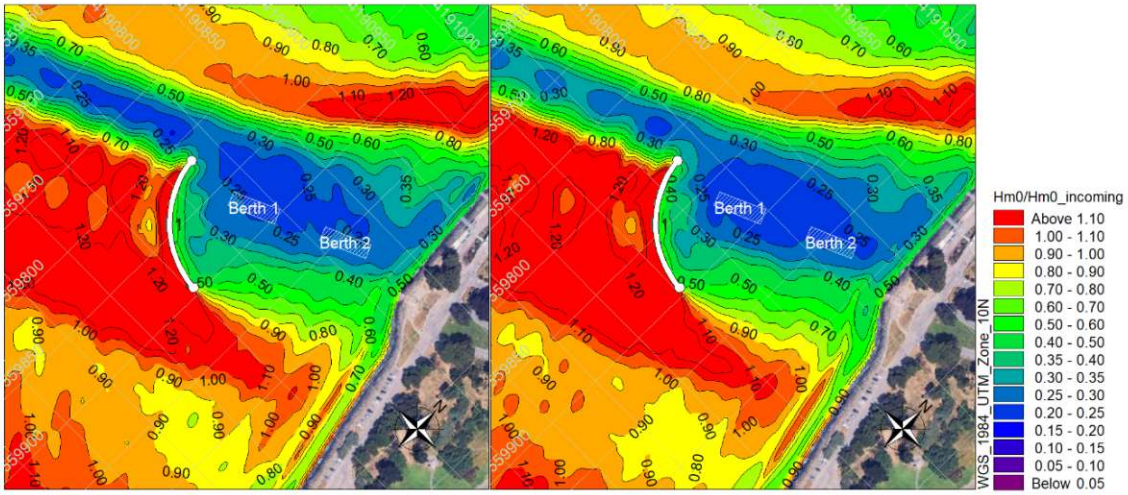


Figure C- 15 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

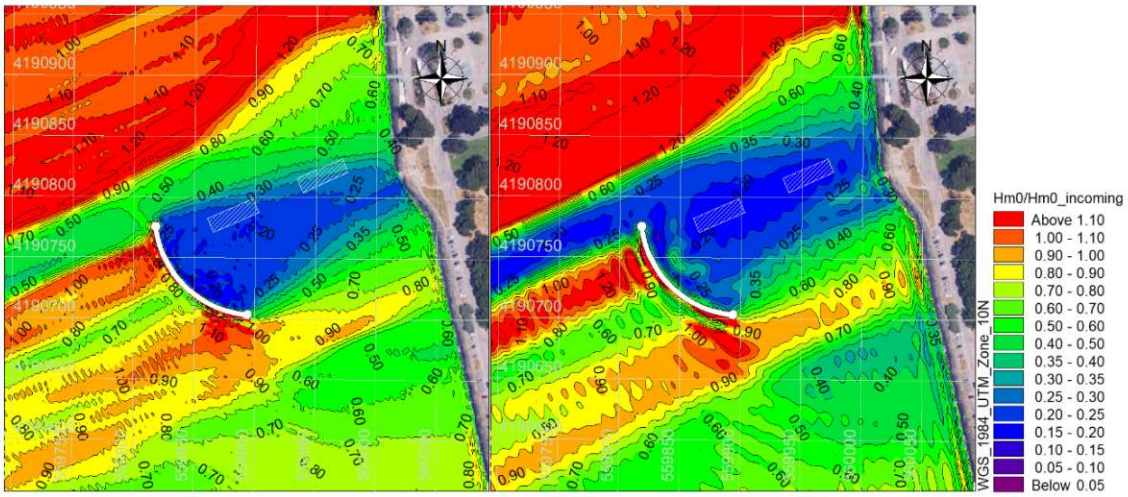


Figure C- 16 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

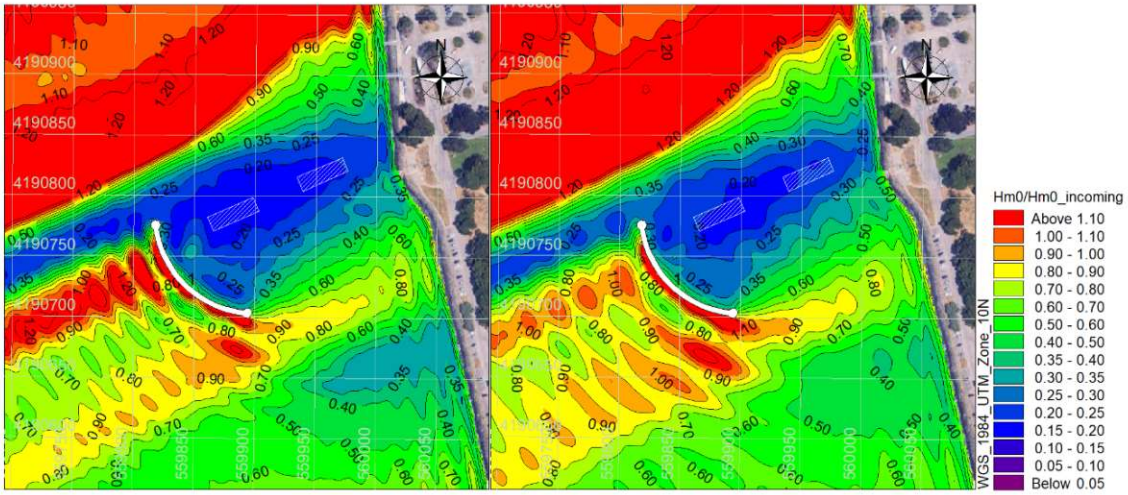


Figure C- 17 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.*

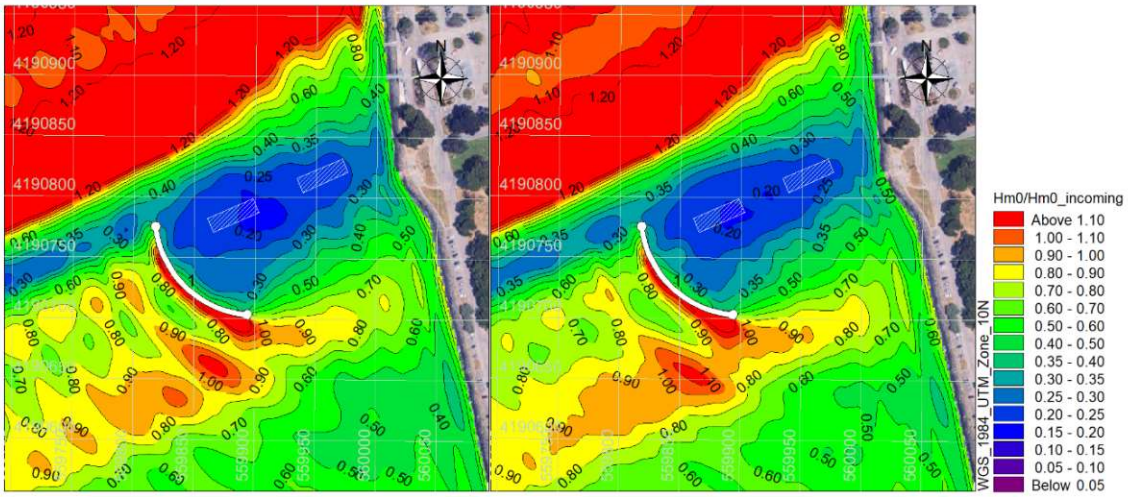


Figure C- 18 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

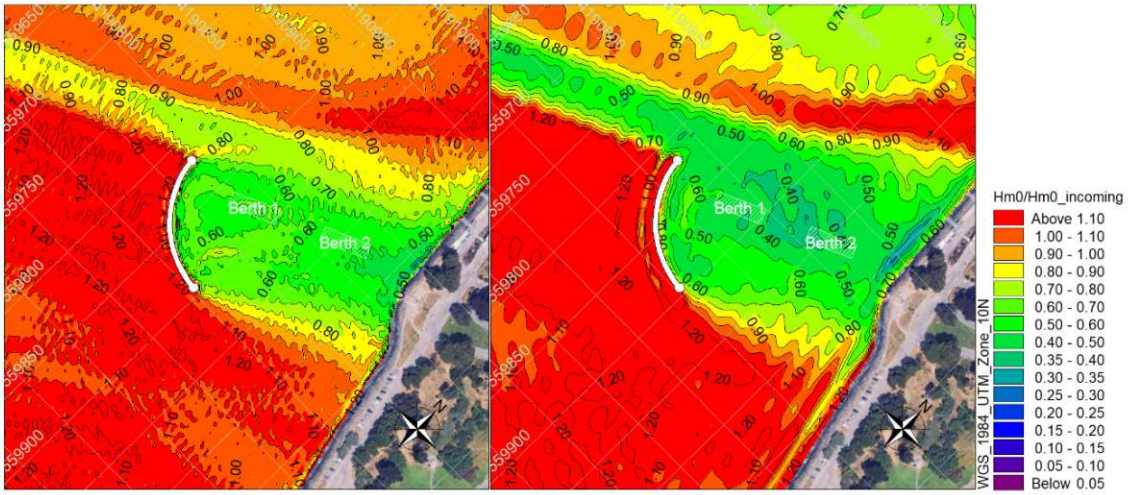


Figure C- 19 Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.

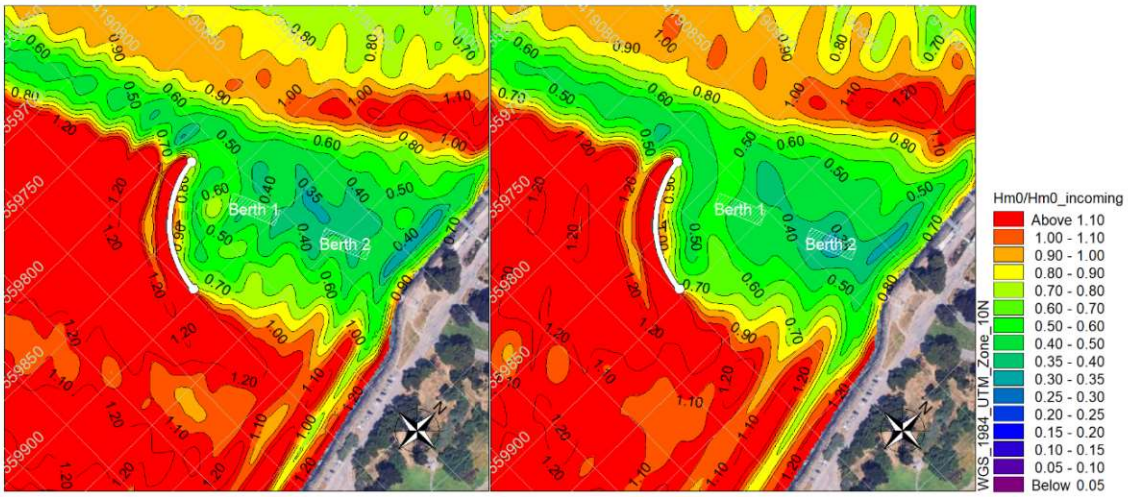


Figure C- 20 Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

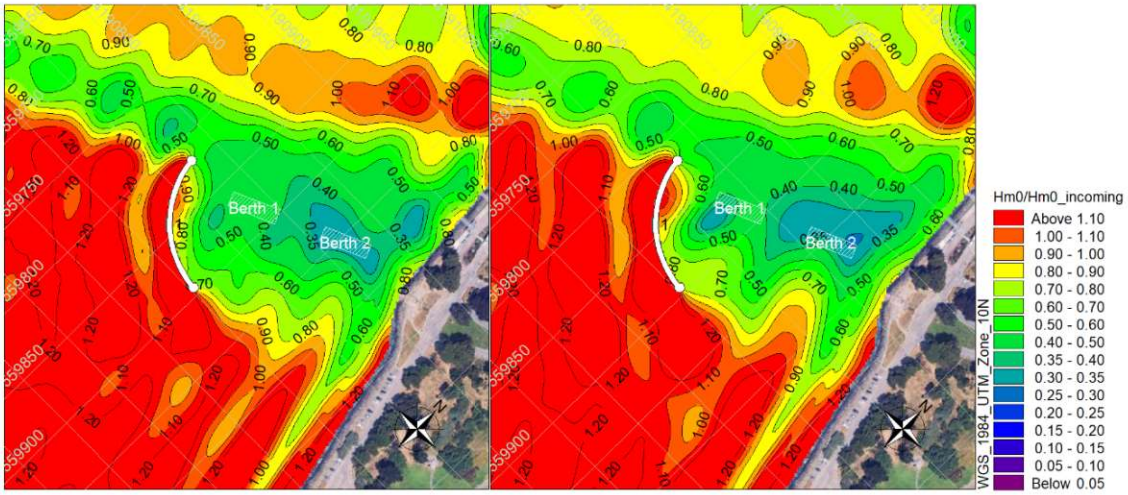


Figure C- 21 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.*

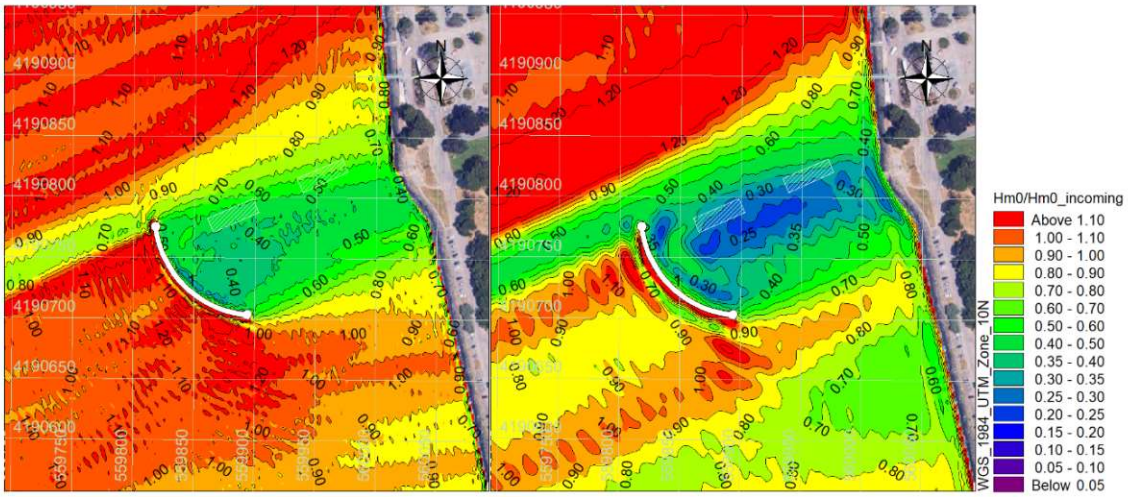


Figure C- 22 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 400 ft breakwater length.*

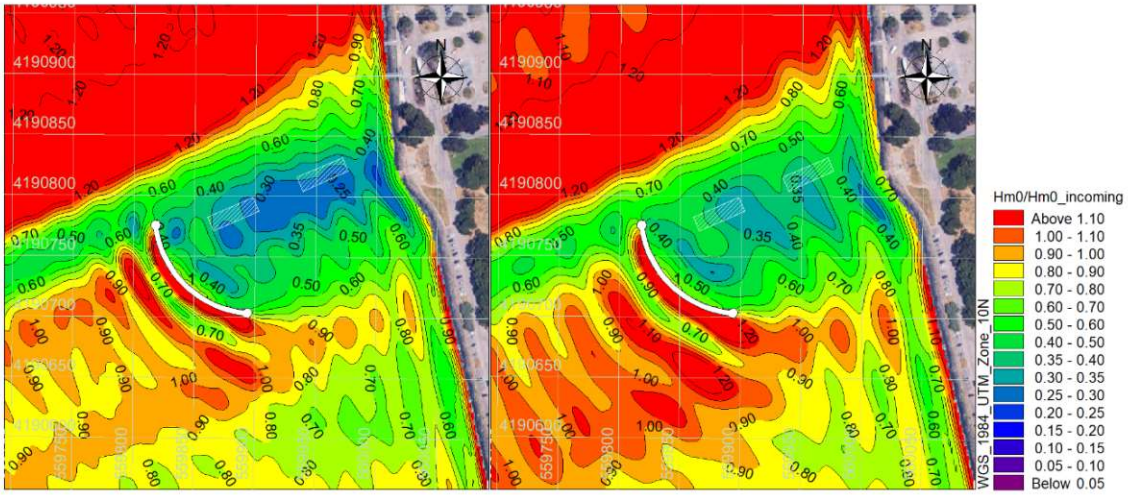


Figure C- 23 Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 400 ft breakwater length.

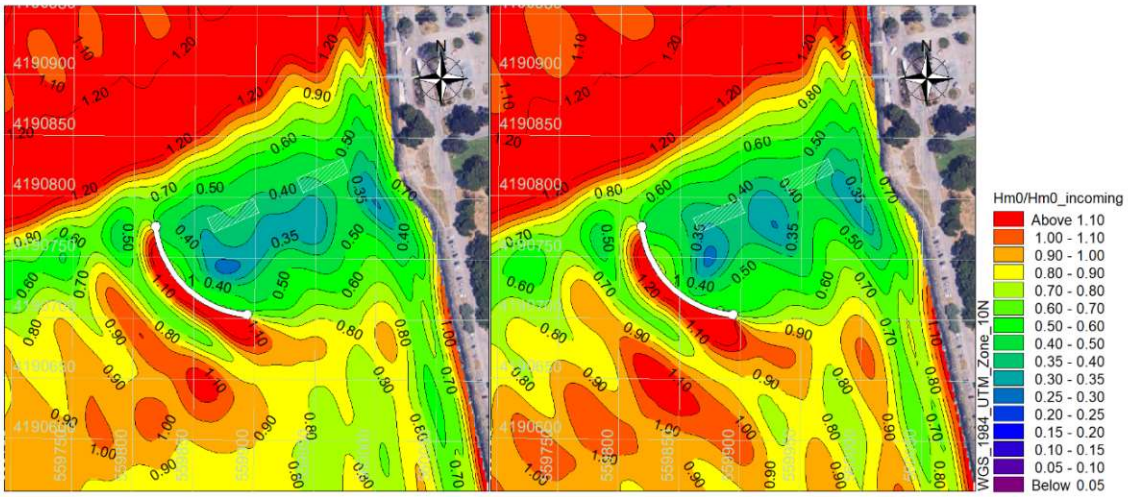


Figure C- 24 loseup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 400 ft breakwater length.

Overall wave disturbance maps – 300 ft breakwater length

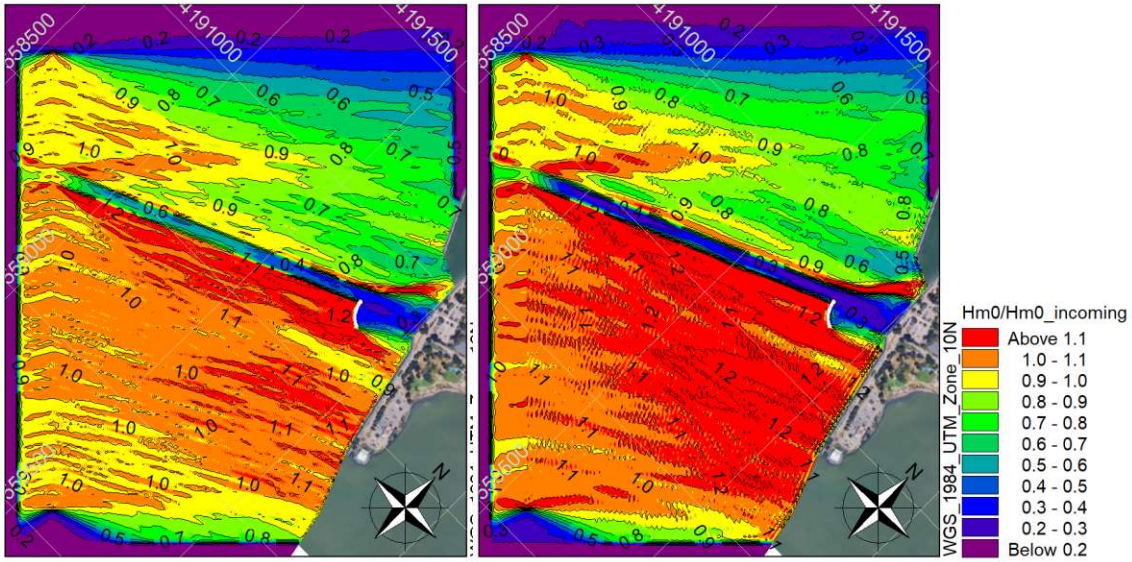


Figure C- 25 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

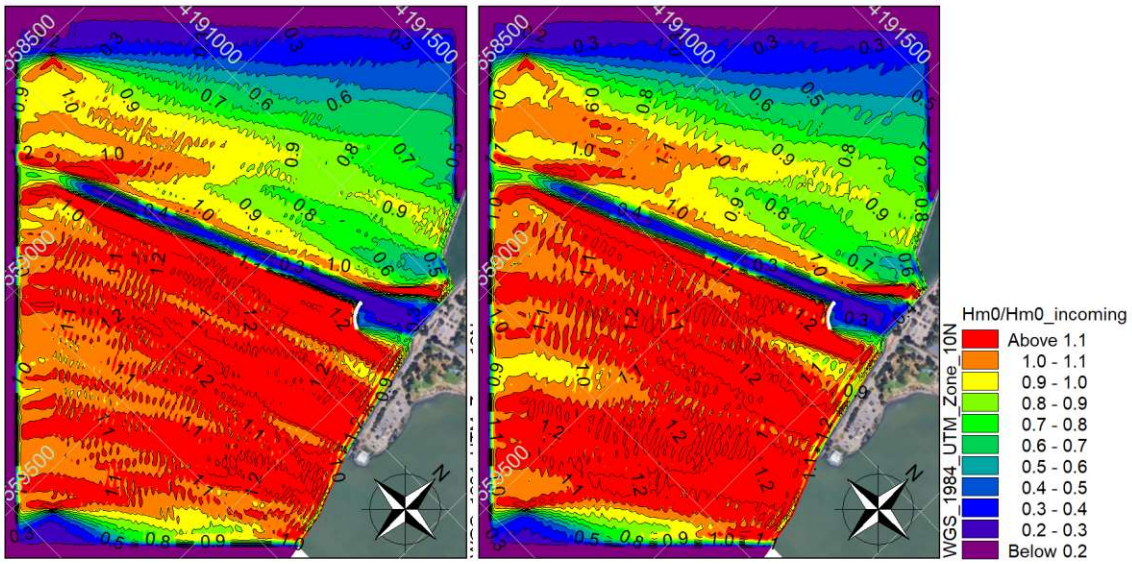


Figure C- 26 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

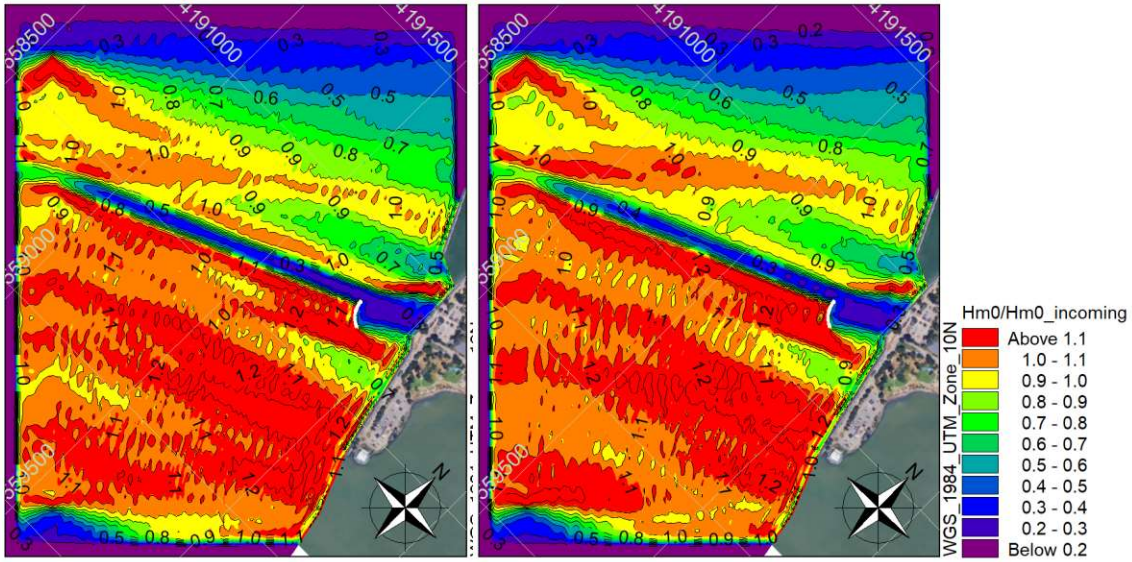


Figure C- 27 *Wave disturbance plot for wave direction = 240°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length

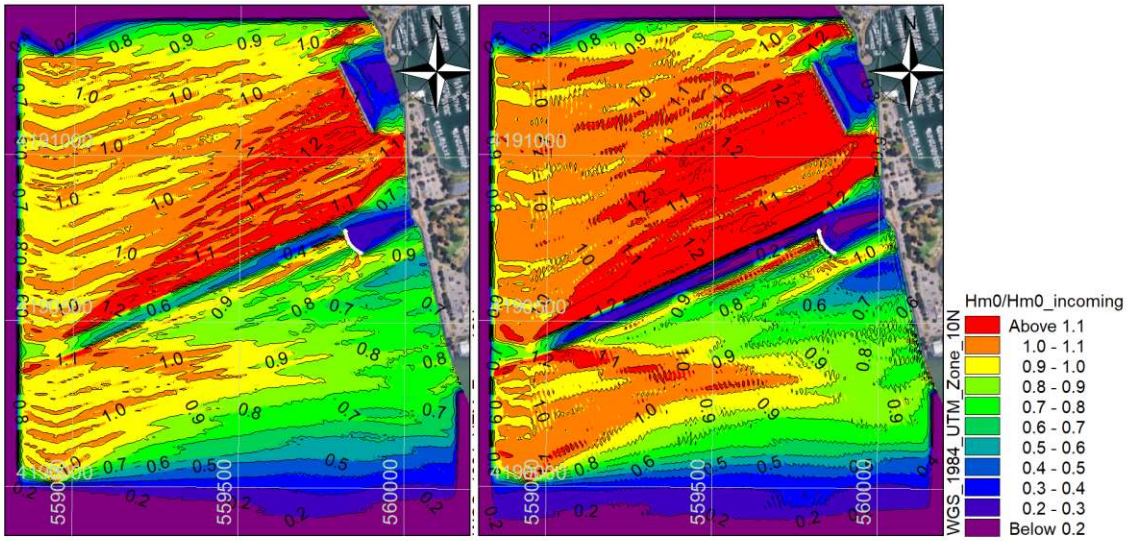


Figure C- 28 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length

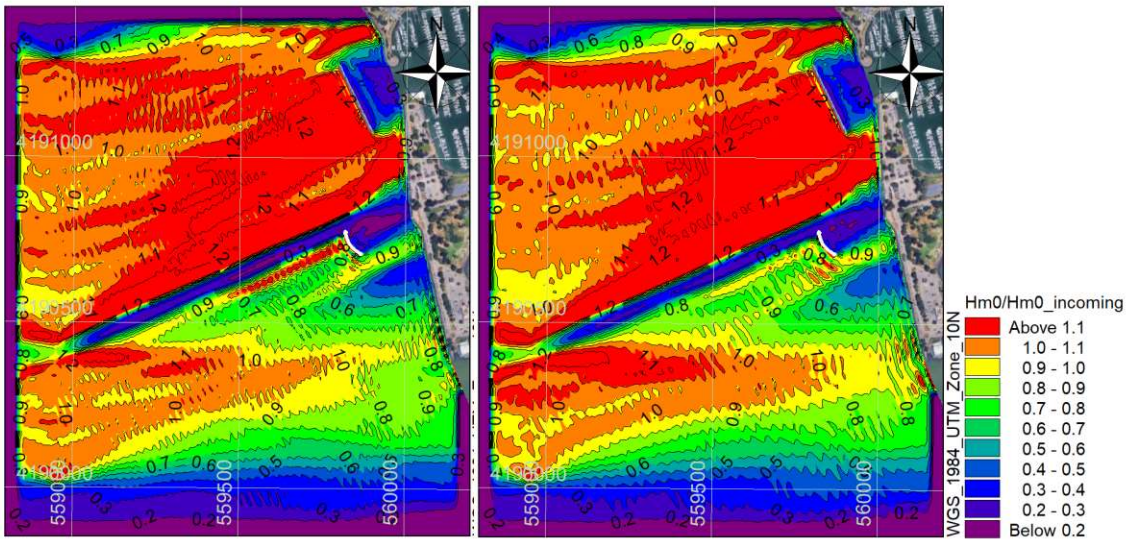


Figure C- 29 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length

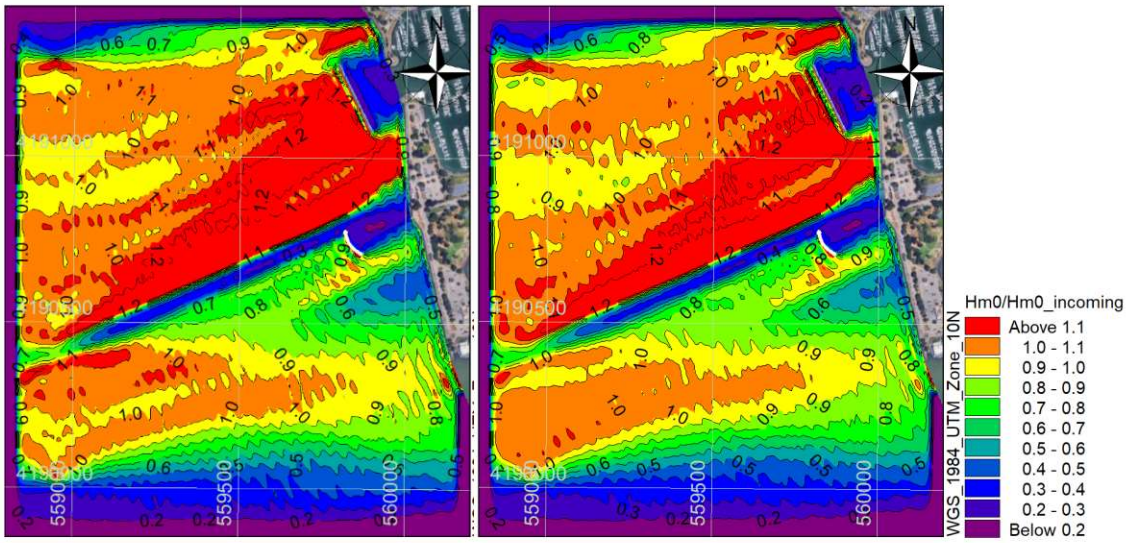


Figure C- 30 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length

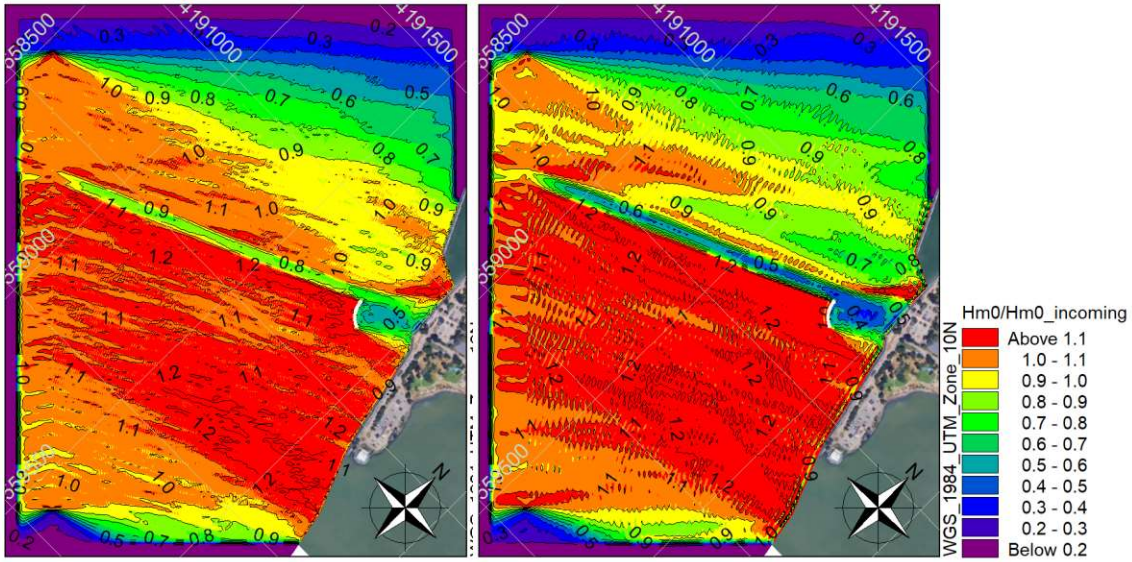


Figure C- 31 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length

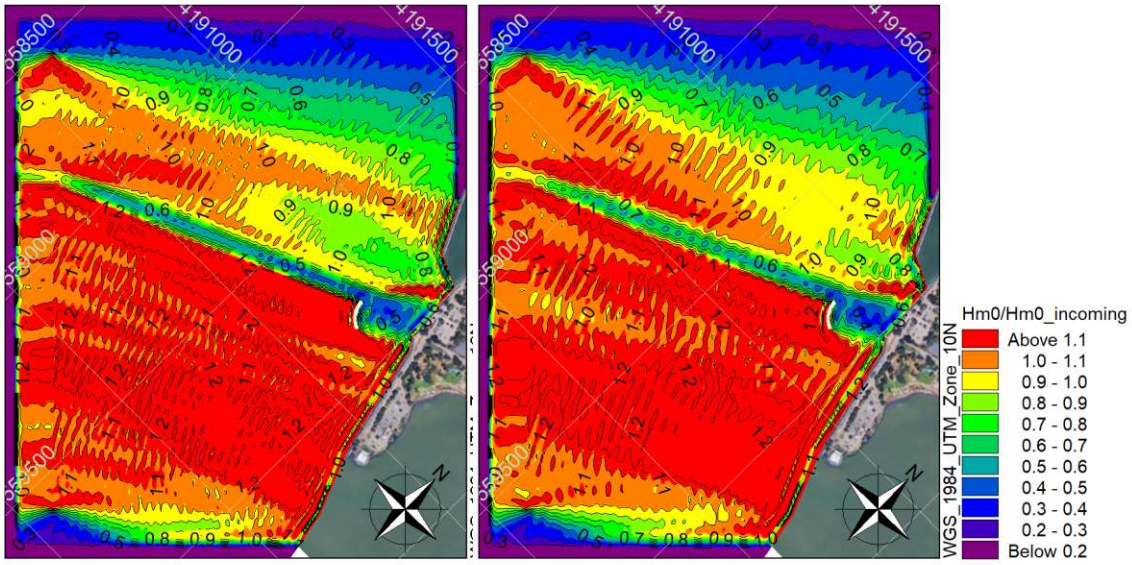


Figure C- 32 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length

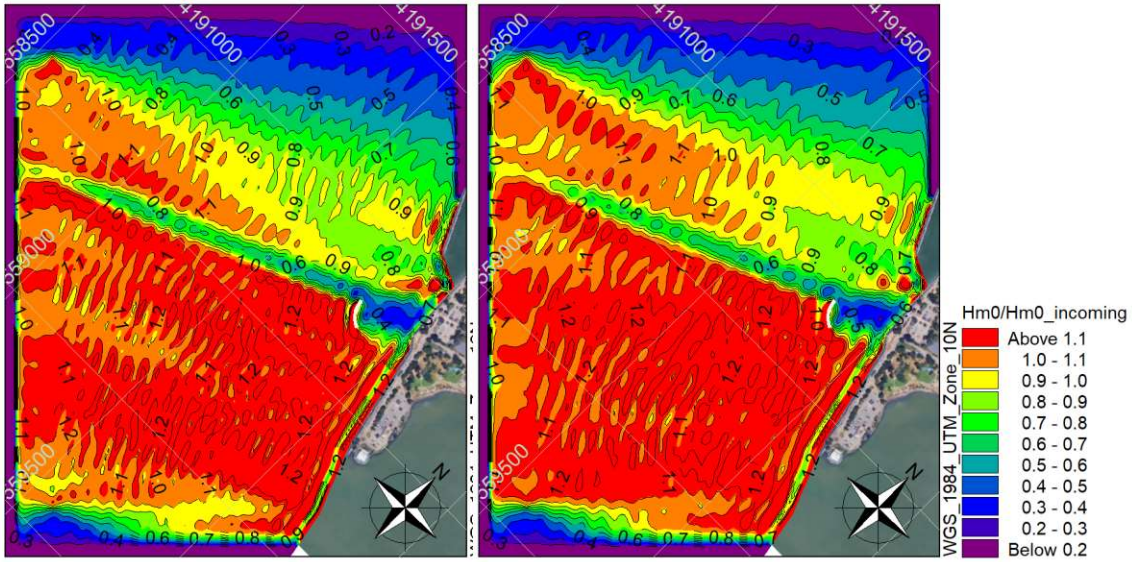


Figure C- 33 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length

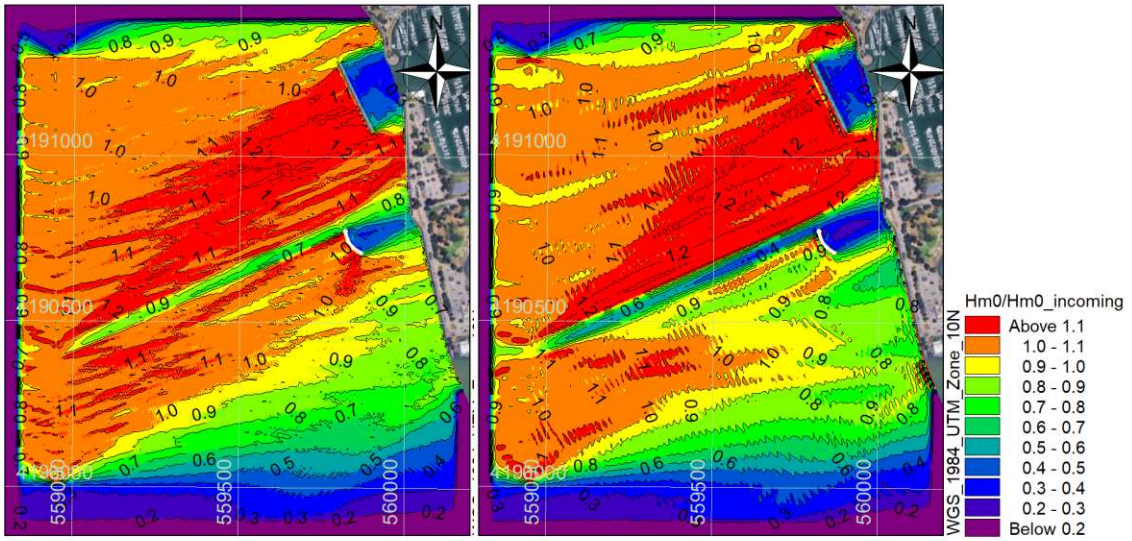


Figure C- 34 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length

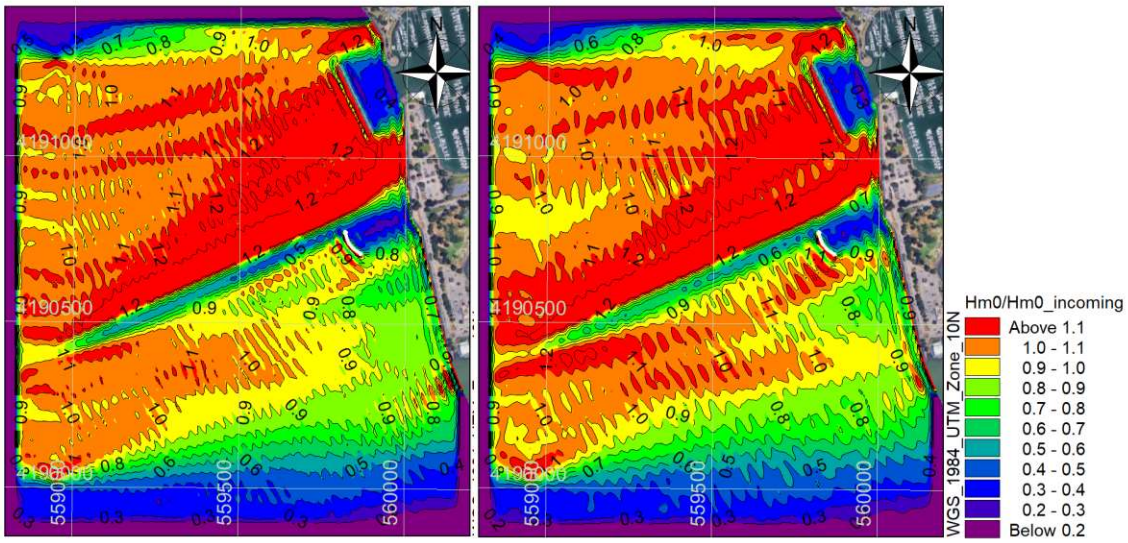


Figure C- 35 *Wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

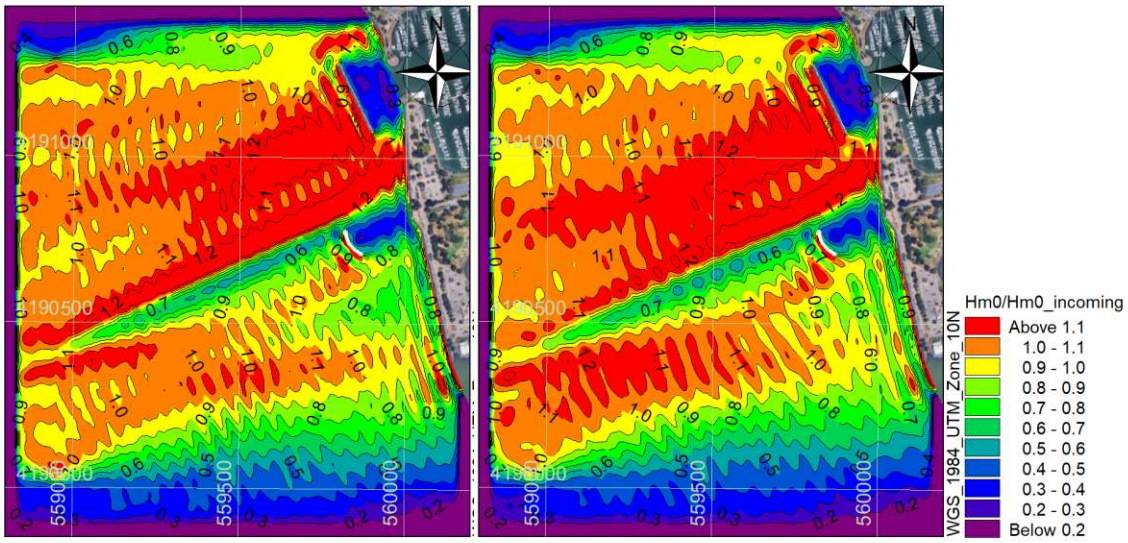


Figure C- 36 *Wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

Ferry terminal wave disturbance maps – 300 ft breakwater length

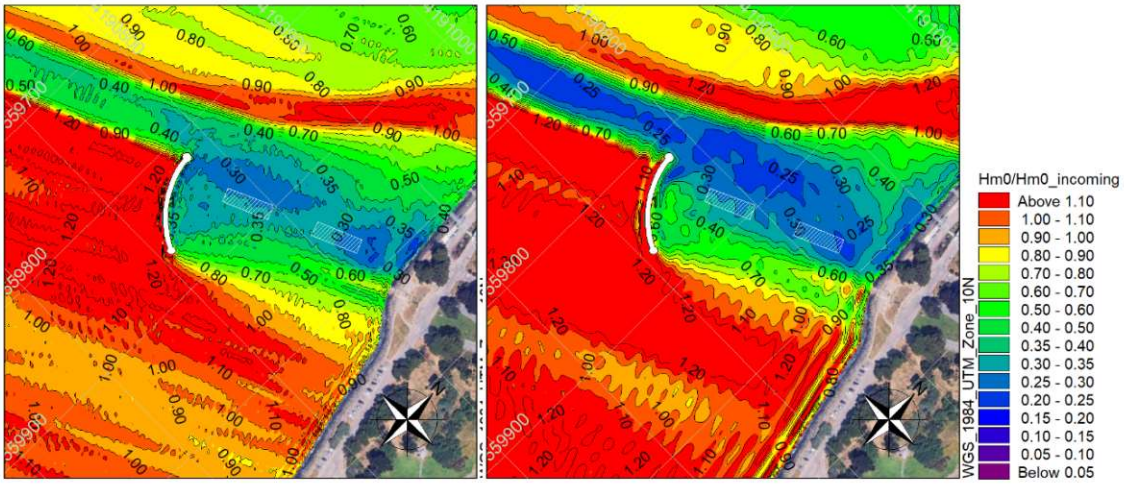


Figure C- 37 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

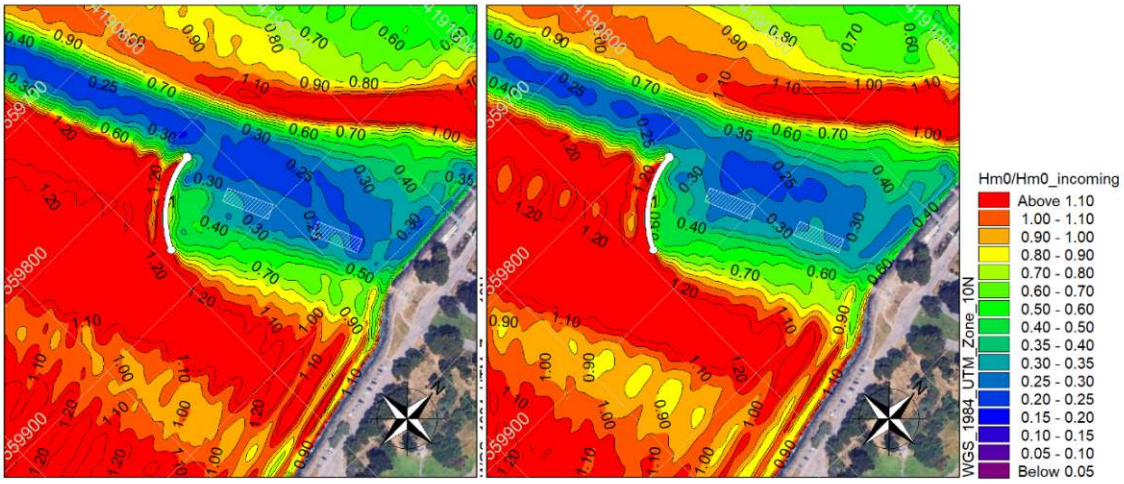


Figure C- 38 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

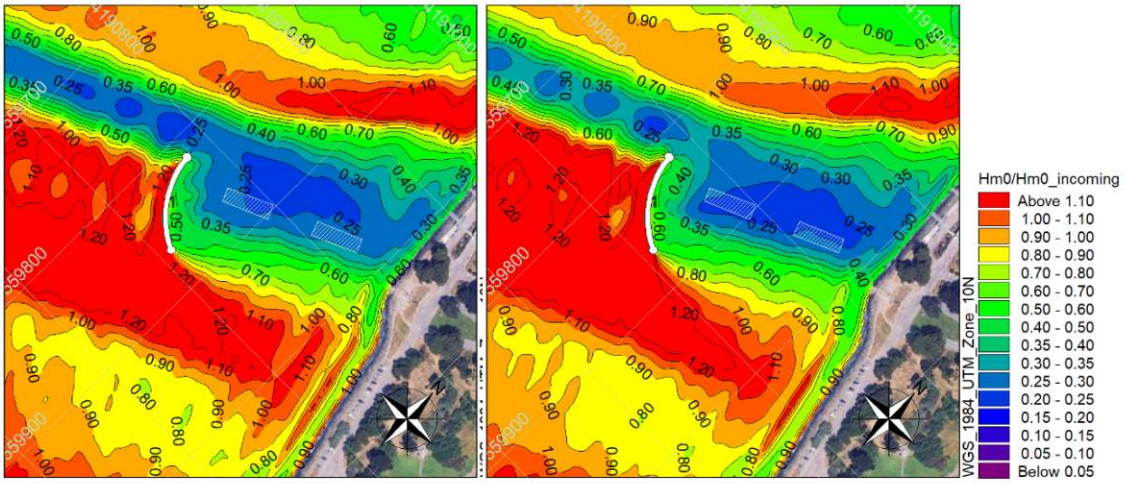


Figure C- 39 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

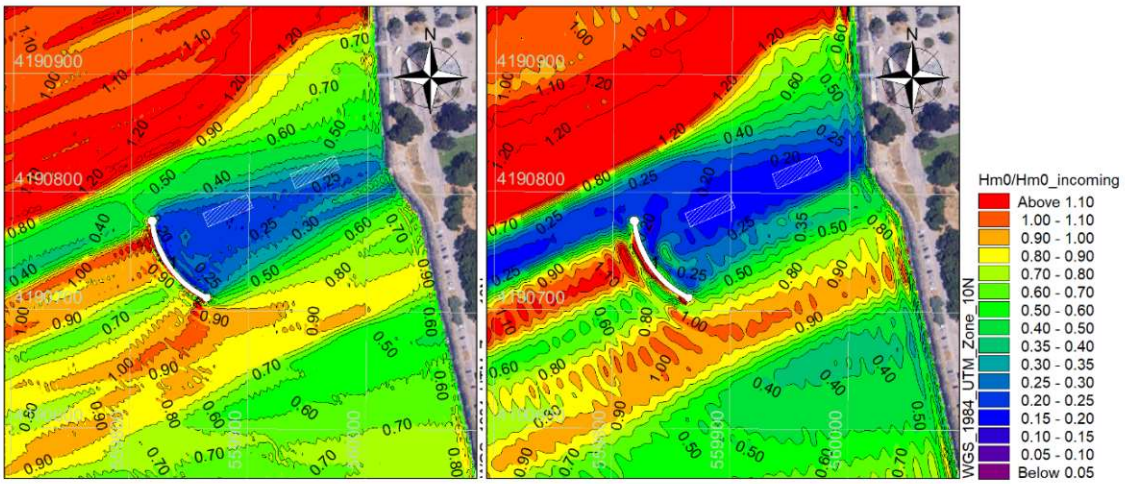


Figure C- 40 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

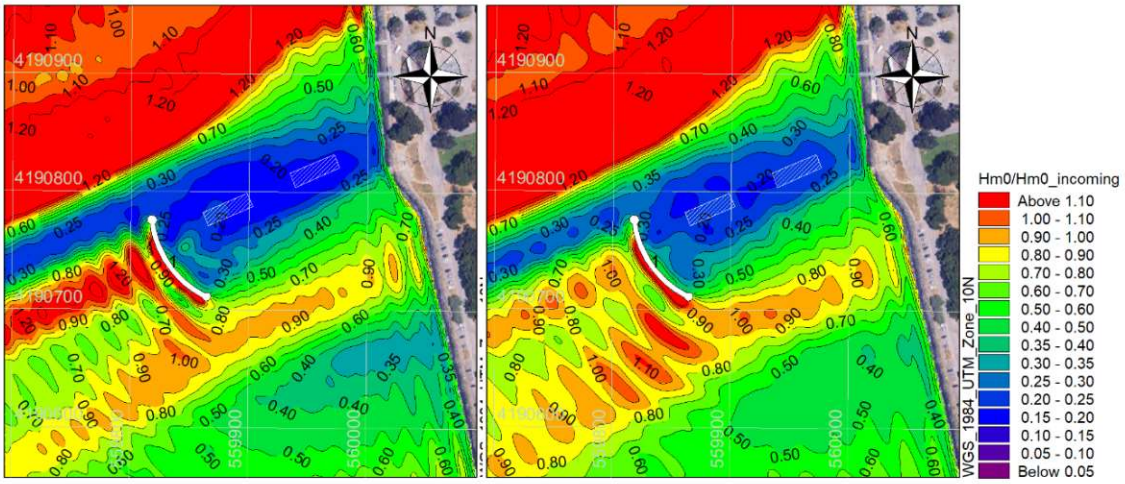


Figure C- 41 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

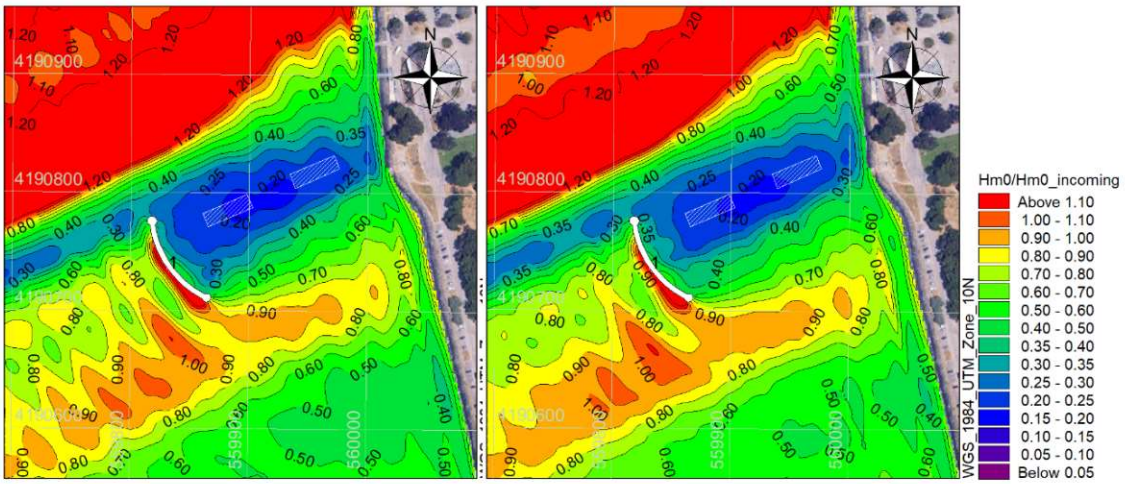


Figure C- 42 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

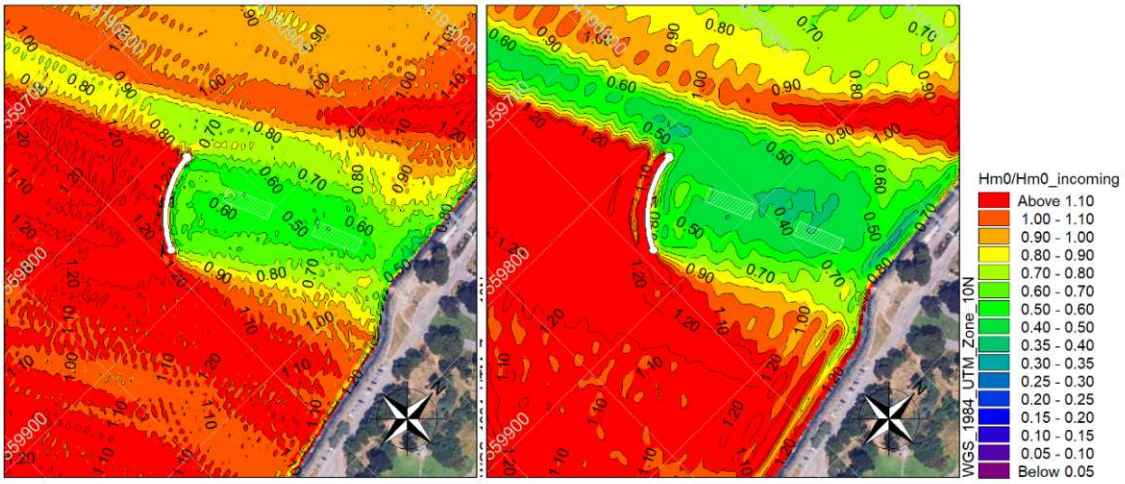


Figure C- 43 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

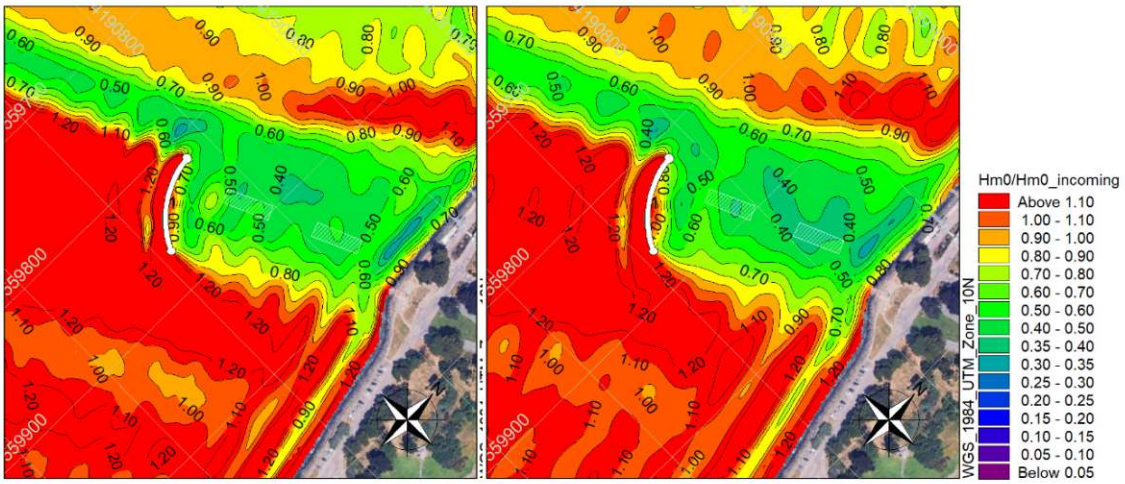


Figure C- 44 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

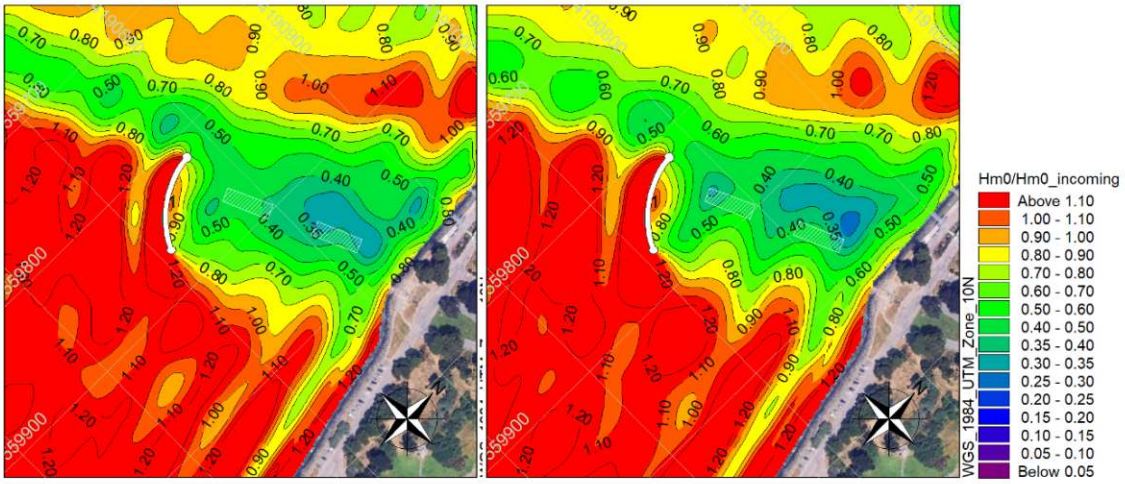


Figure C- 45 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

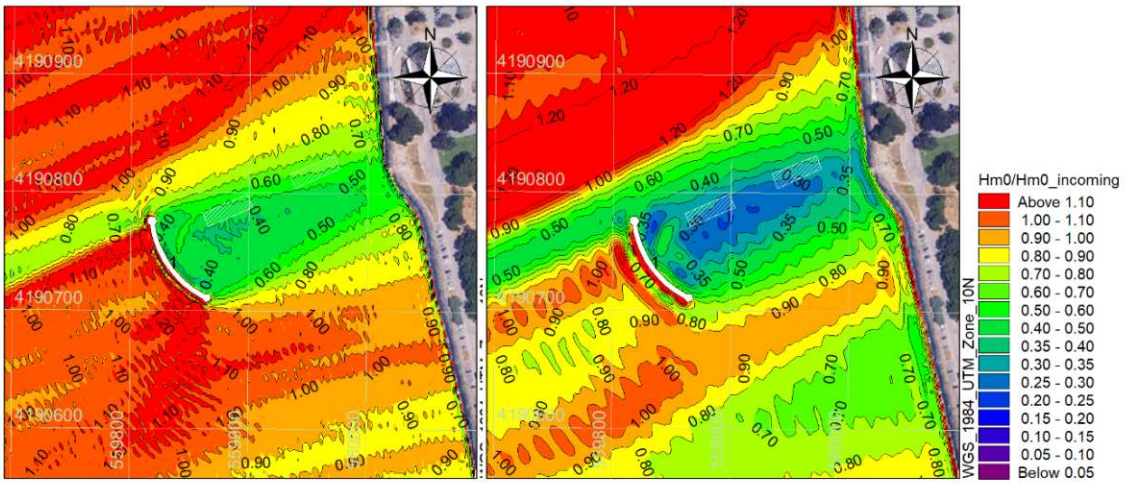


Figure C- 46 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 300 ft breakwater length*

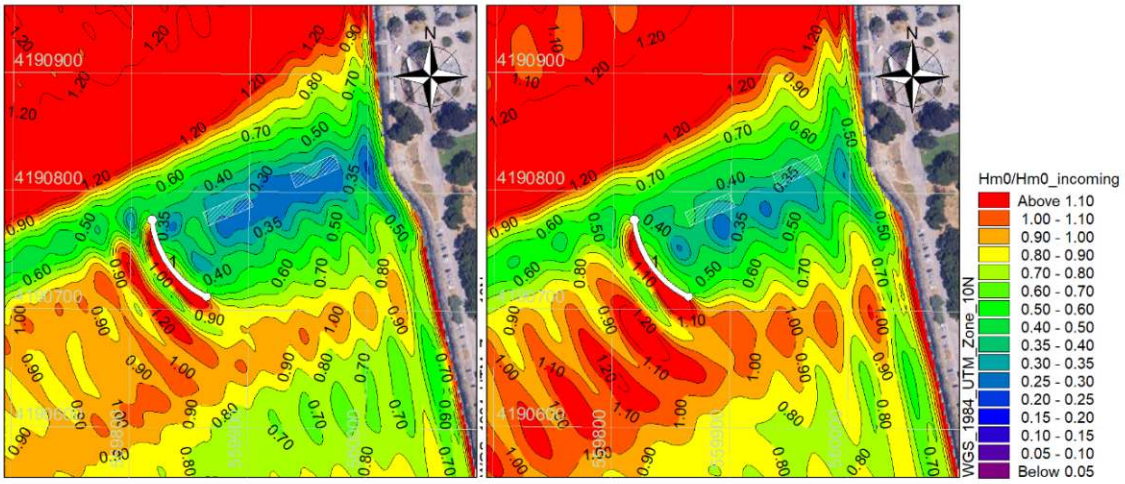


Figure C- 47 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 300 ft breakwater length*

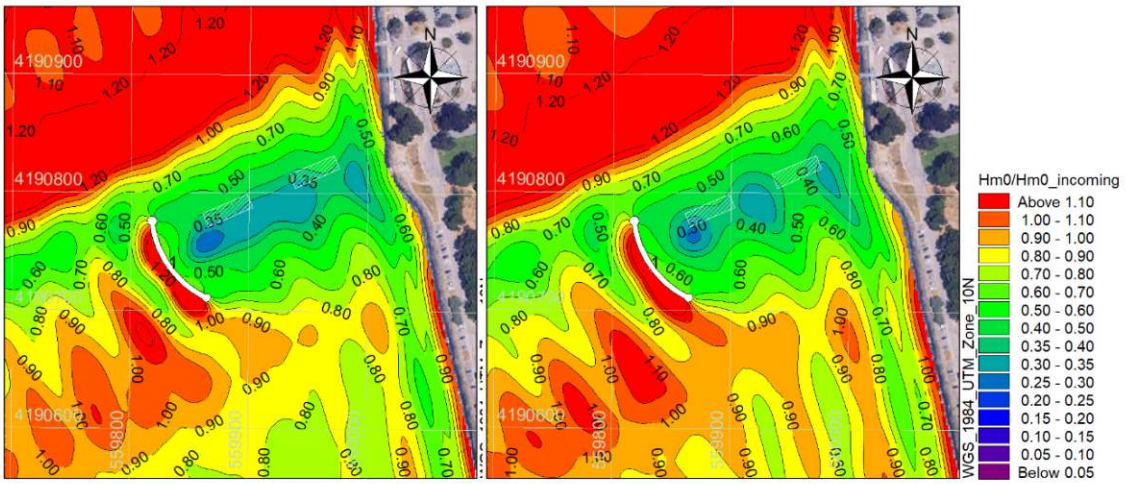


Figure C- 48 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 300 ft breakwater length*

Overall wave disturbance maps – 250 ft breakwater length

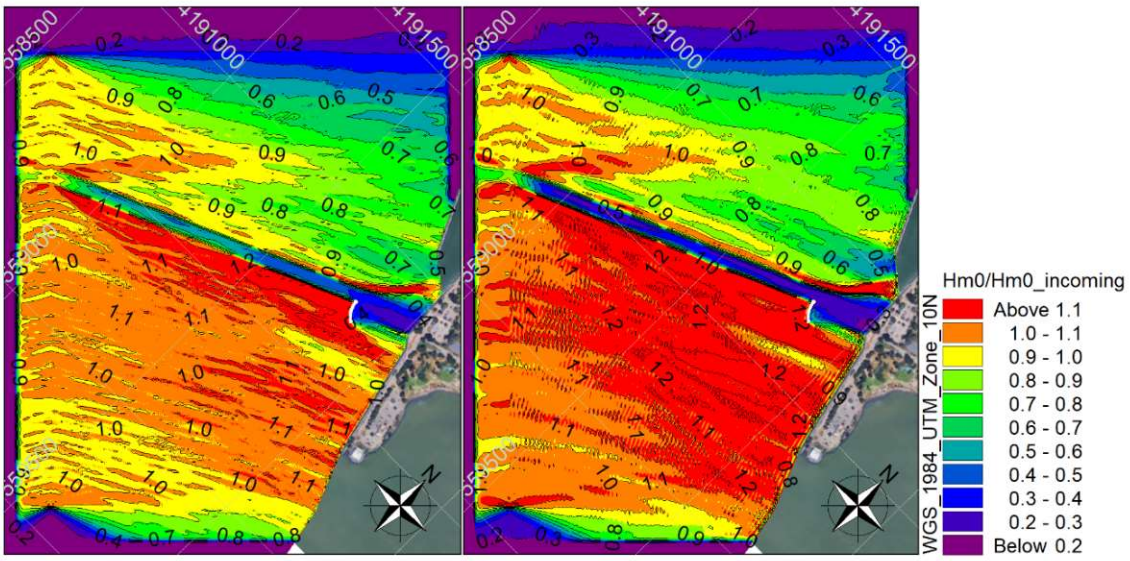


Figure C- 49 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

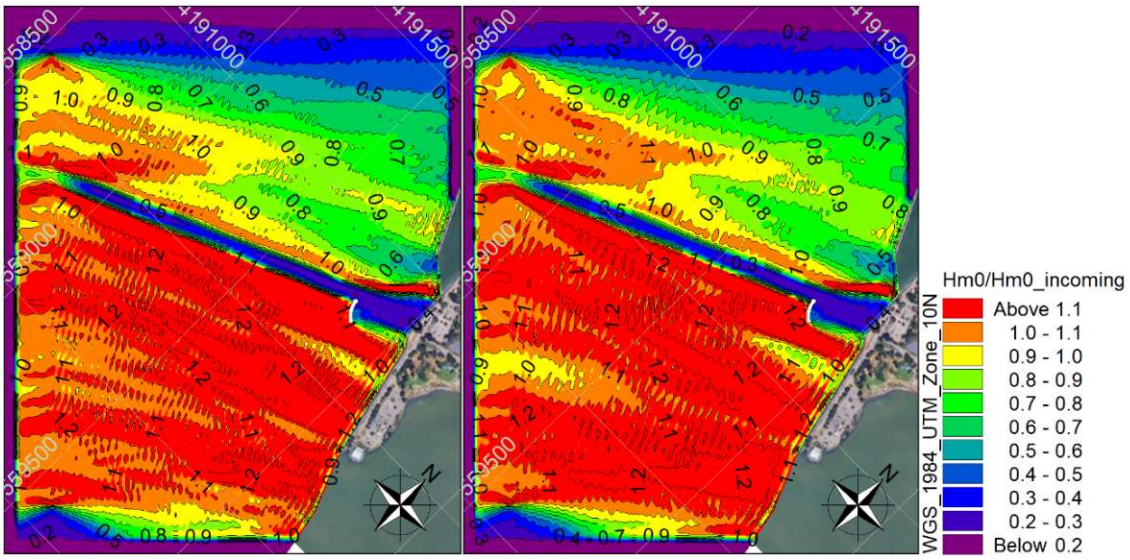


Figure C- 50 *Wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

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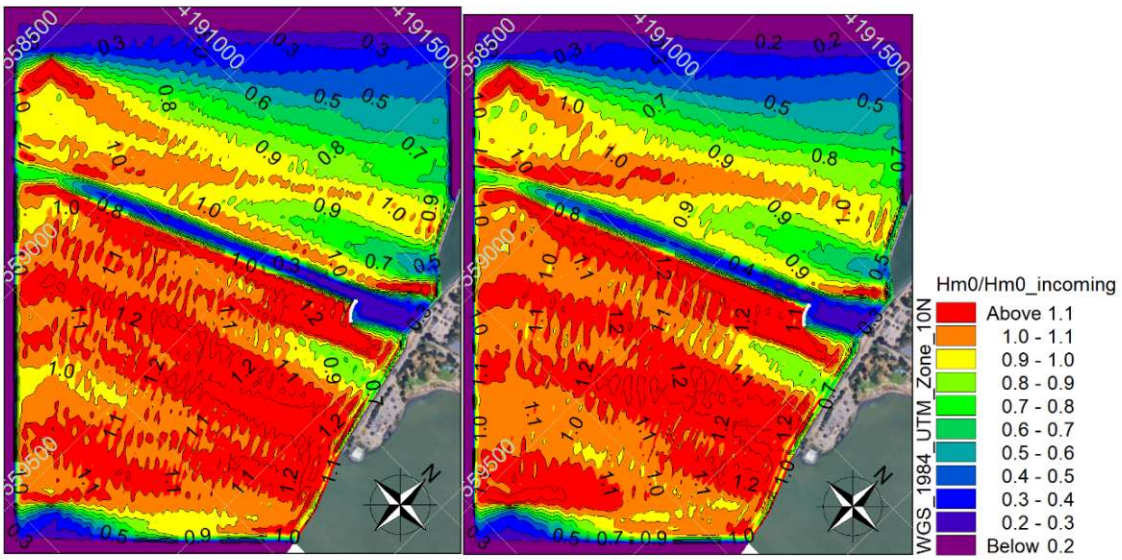


Figure C- 51 *Wave disturbance plot for wave direction = 240°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

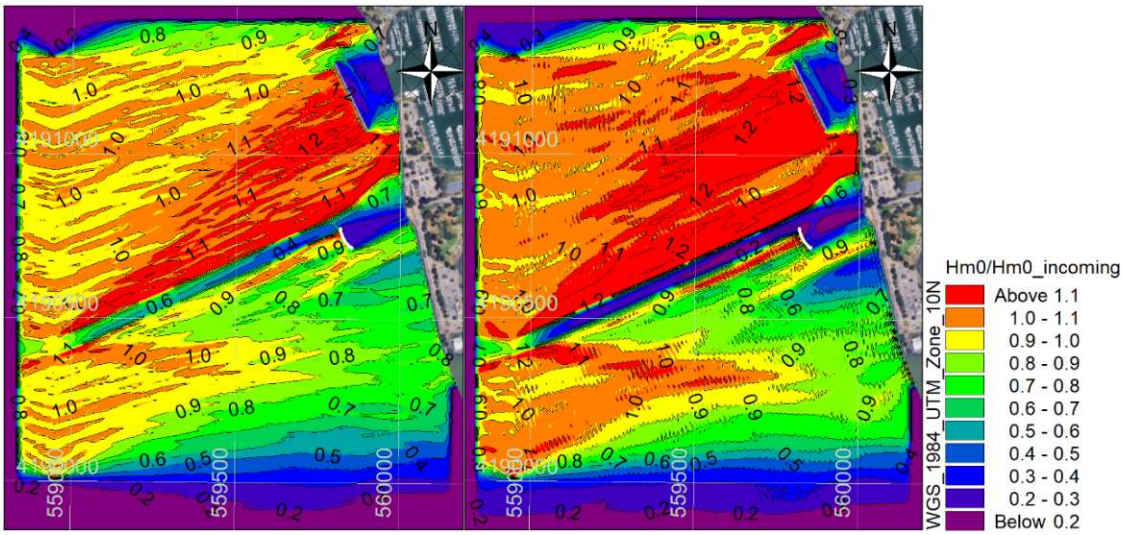


Figure C- 52 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length

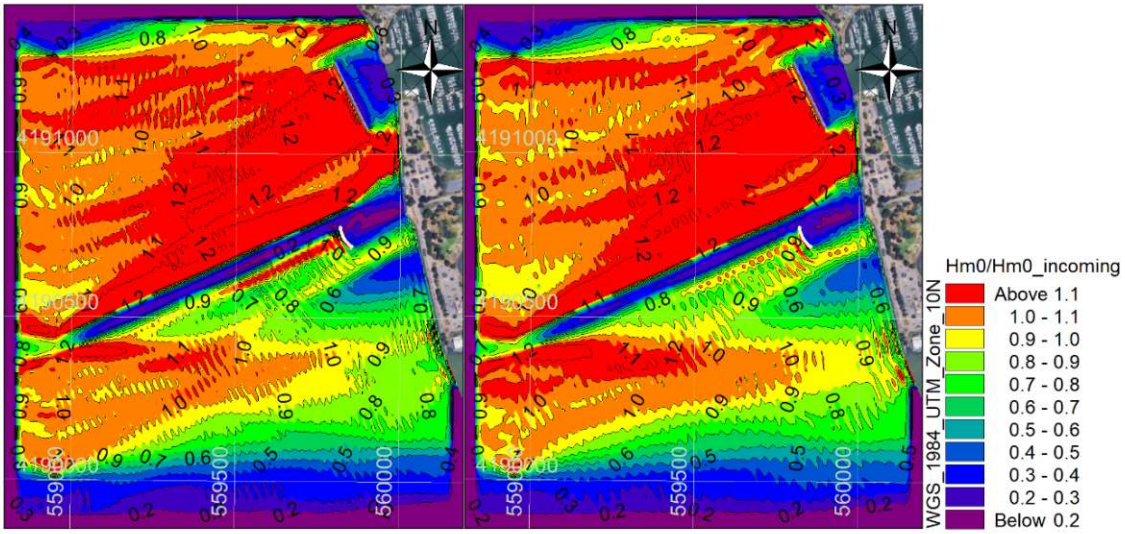


Figure C- 53 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length

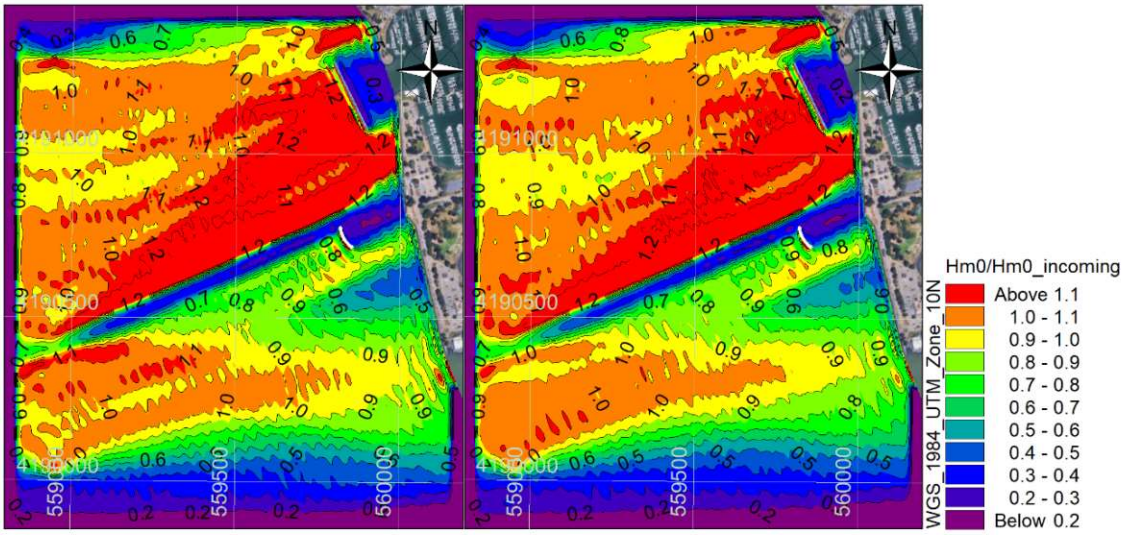


Figure C- 54 *Wave disturbance plot for wave direction = 255°N at MLLW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

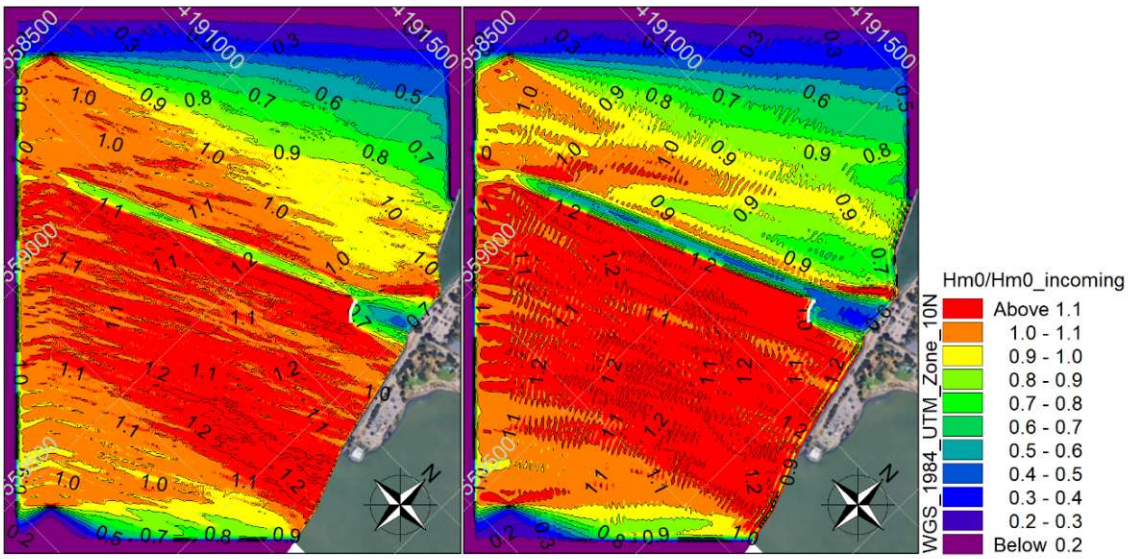


Figure C- 55 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

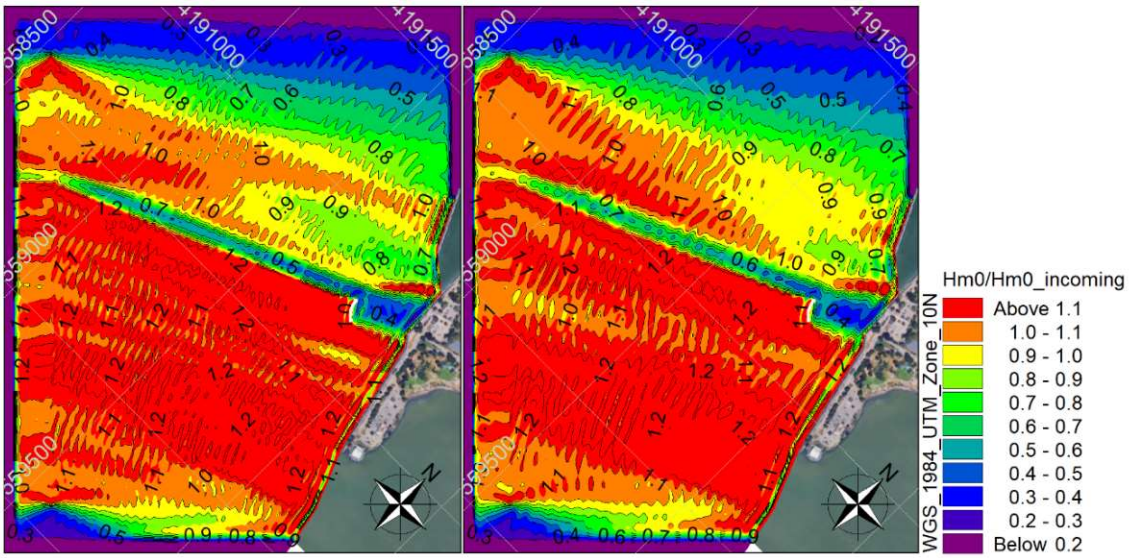


Figure C- 56 *Wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

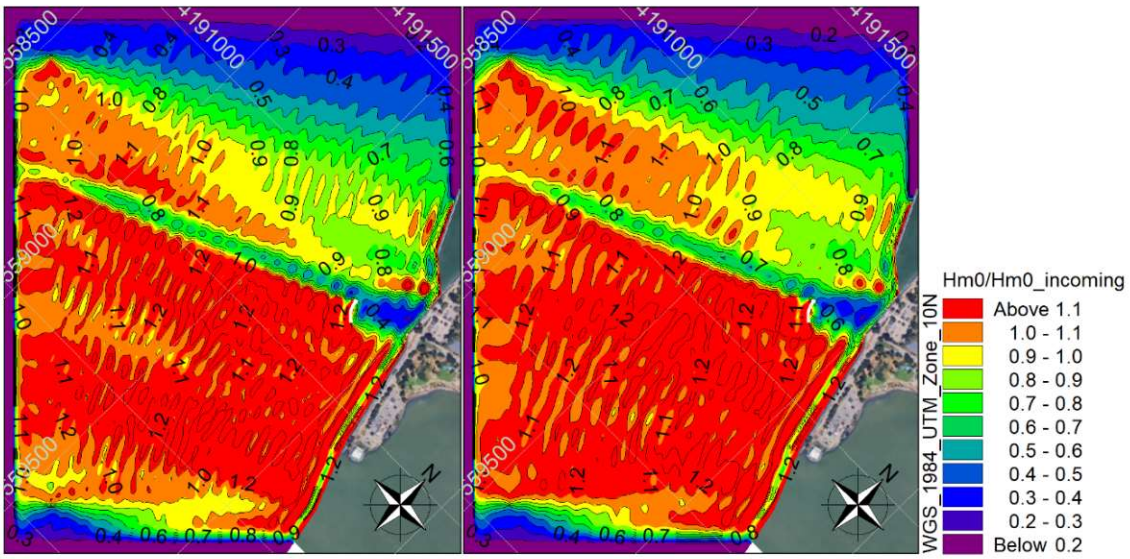


Figure C- 57 *Wave disturbance plot for wave direction = 240°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

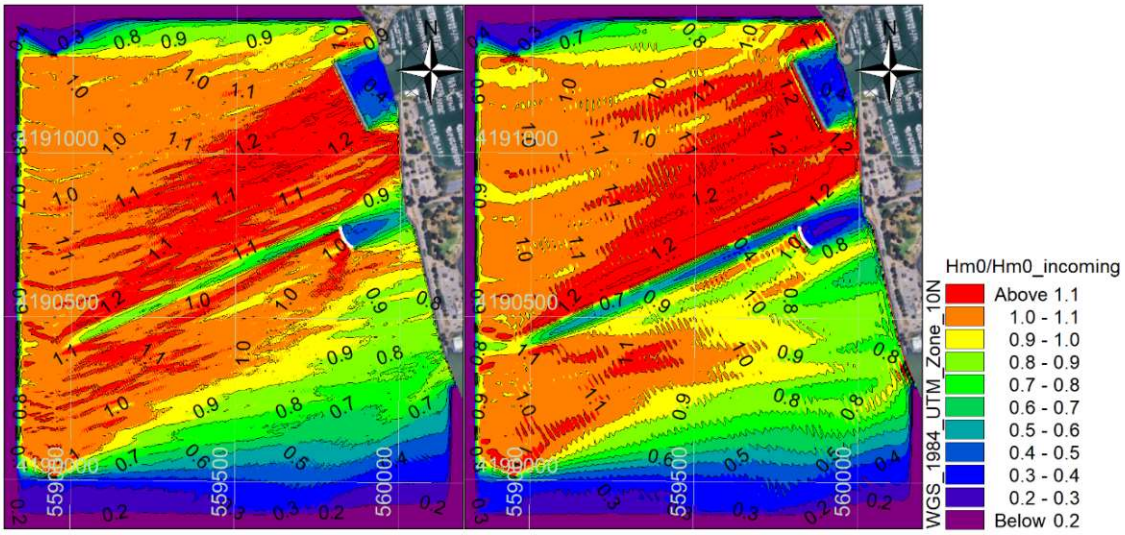


Figure C- 58 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length

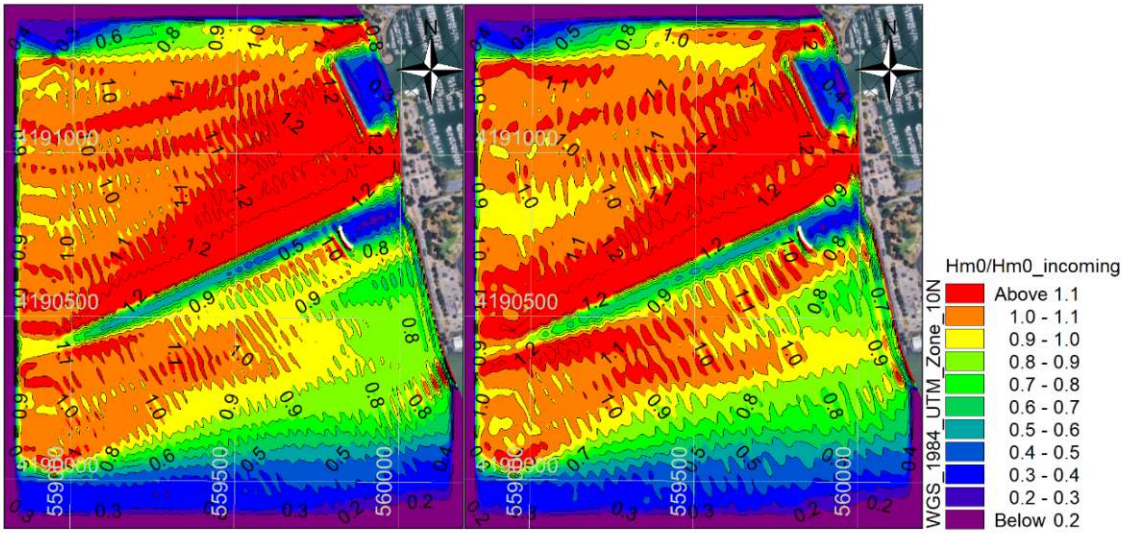


Figure C- 59 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length

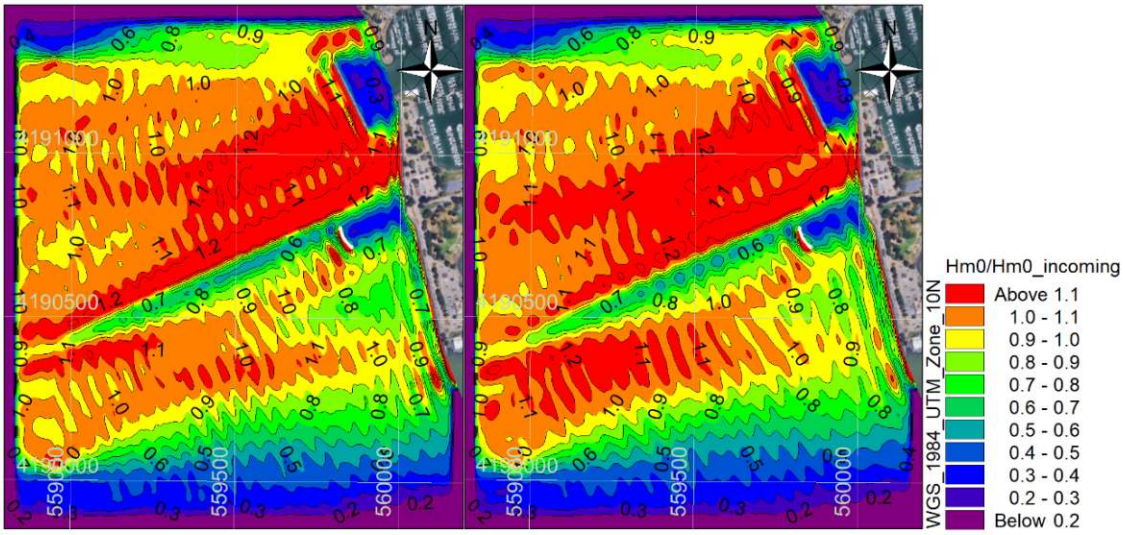


Figure C- 60 *Wave disturbance plot for wave direction = 255°N at MHHW.*
Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length

Ferry terminal wave disturbance maps – 250 ft breakwater length

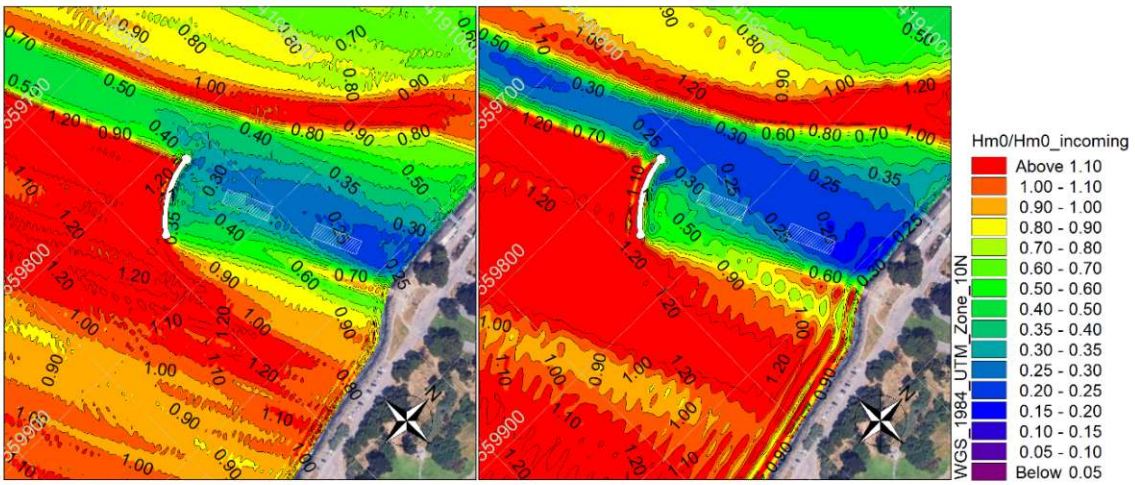


Figure C- 61 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

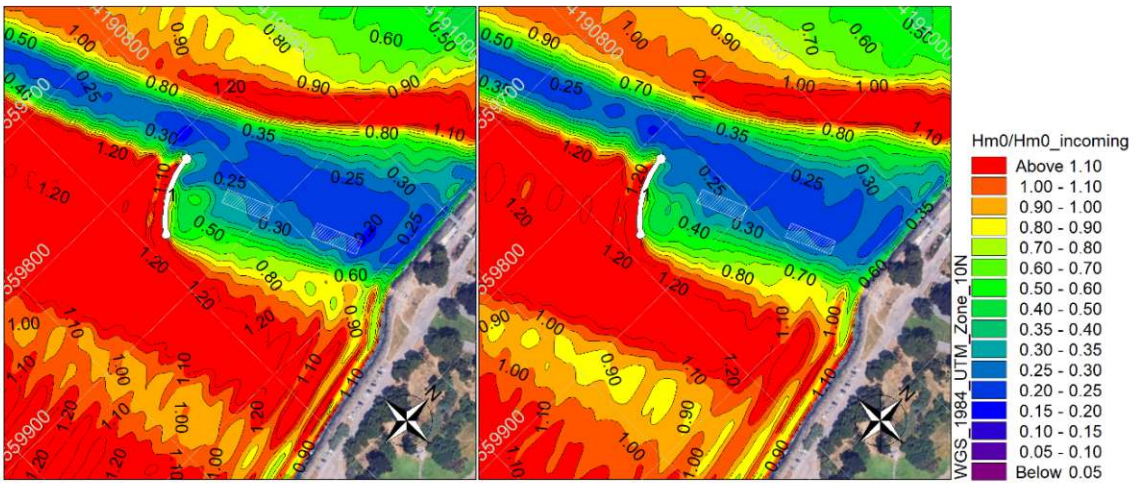


Figure C- 62 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

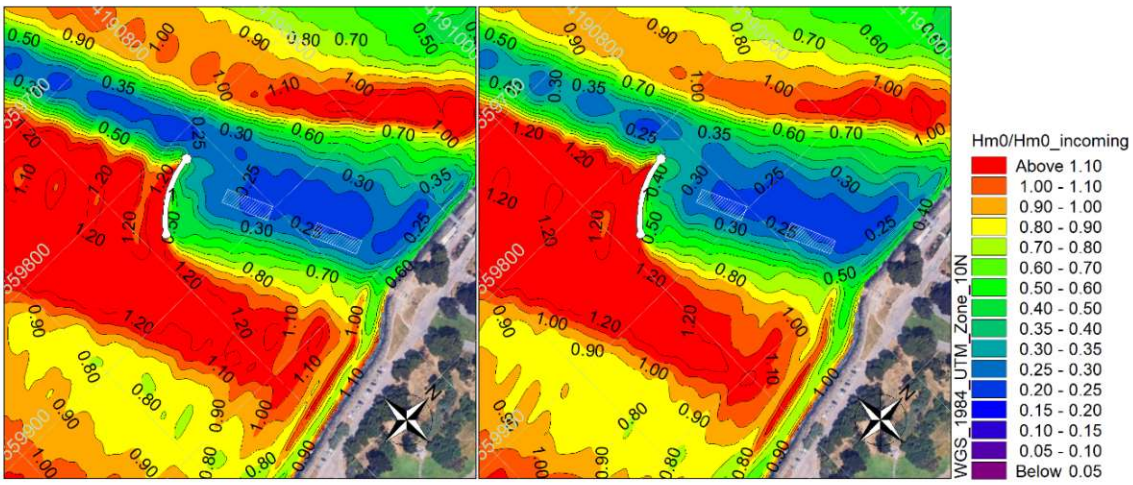


Figure C- 63 *Closeup of wave disturbance plot for wave direction = 240°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

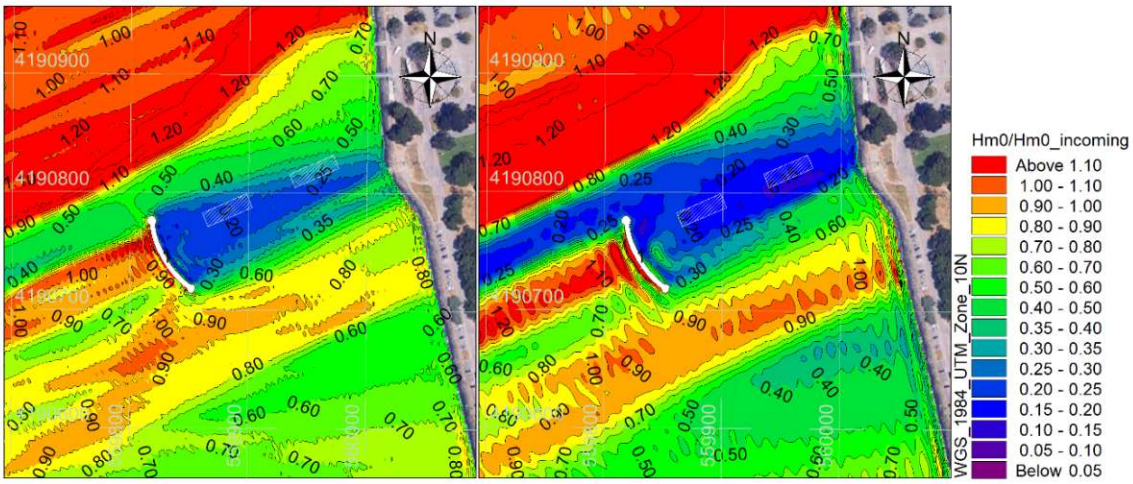


Figure C- 64 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

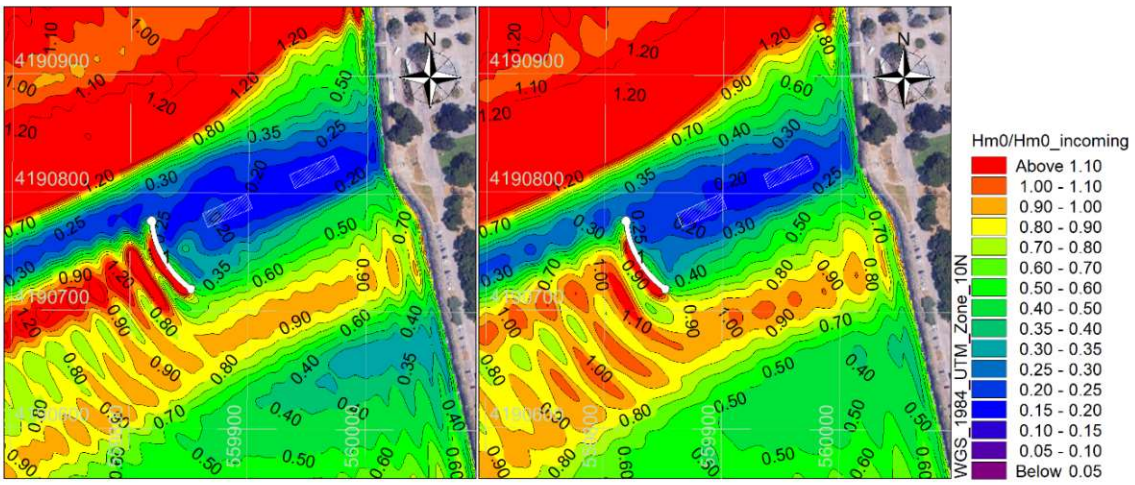


Figure C- 65 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

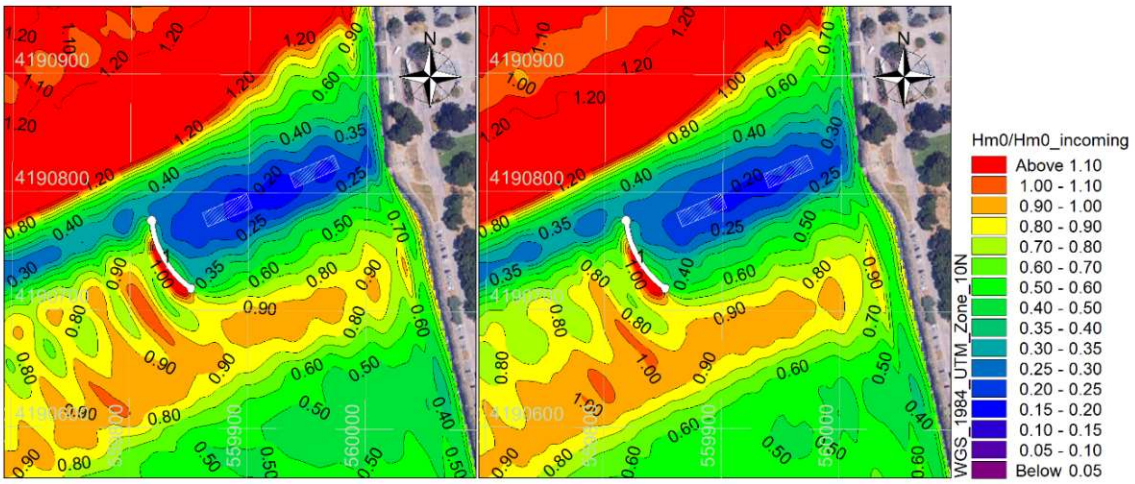


Figure C- 66 *Closeup of wave disturbance plot for wave direction = 255°N at MLLW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

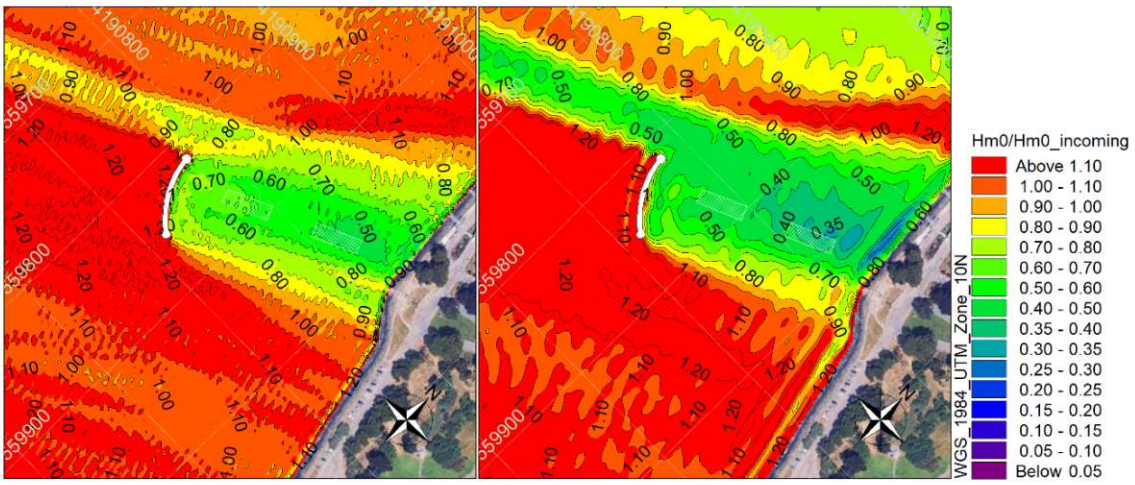


Figure C- 67 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

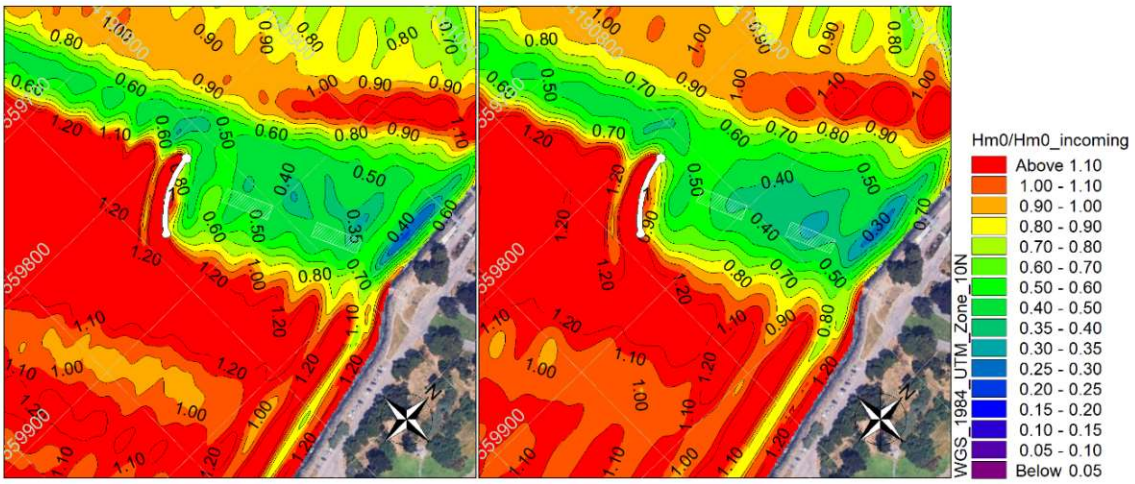


Figure C- 68 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

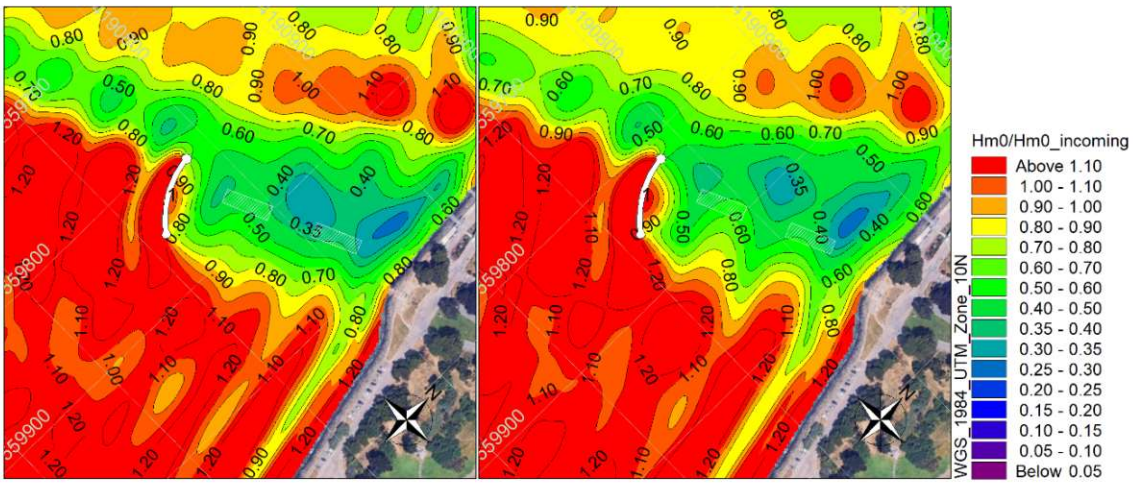


Figure C- 69 *Closeup of wave disturbance plot for wave direction = 240°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

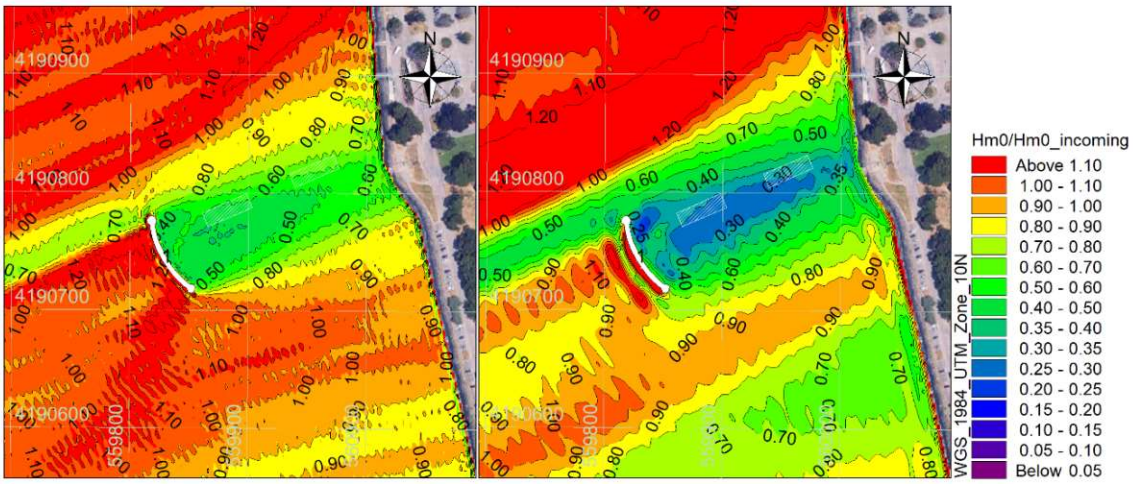


Figure C- 70 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 3 sec and Right: Tp = 6 sec, 250 ft breakwater length*

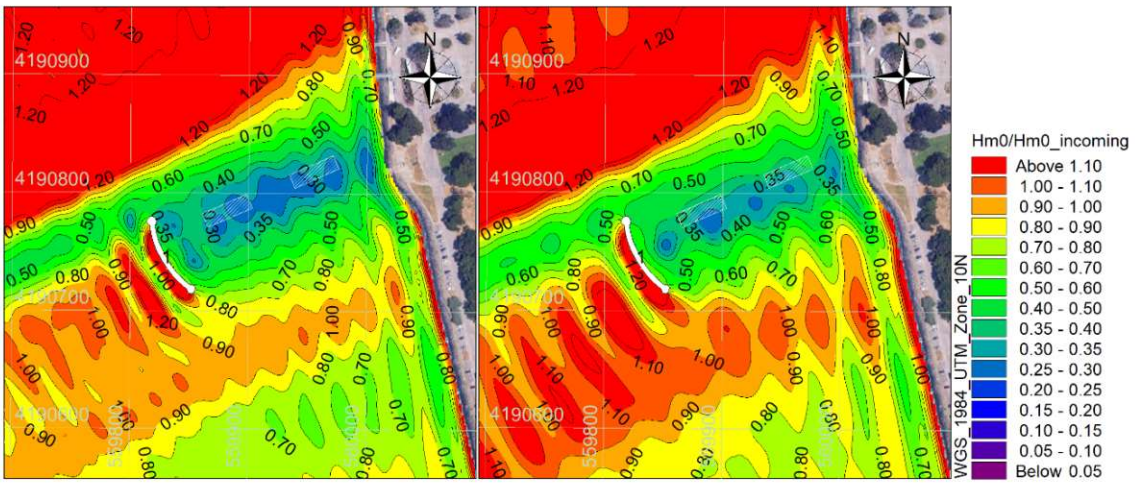


Figure C- 71 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 9 sec and Right: Tp = 12 sec, 250 ft breakwater length*

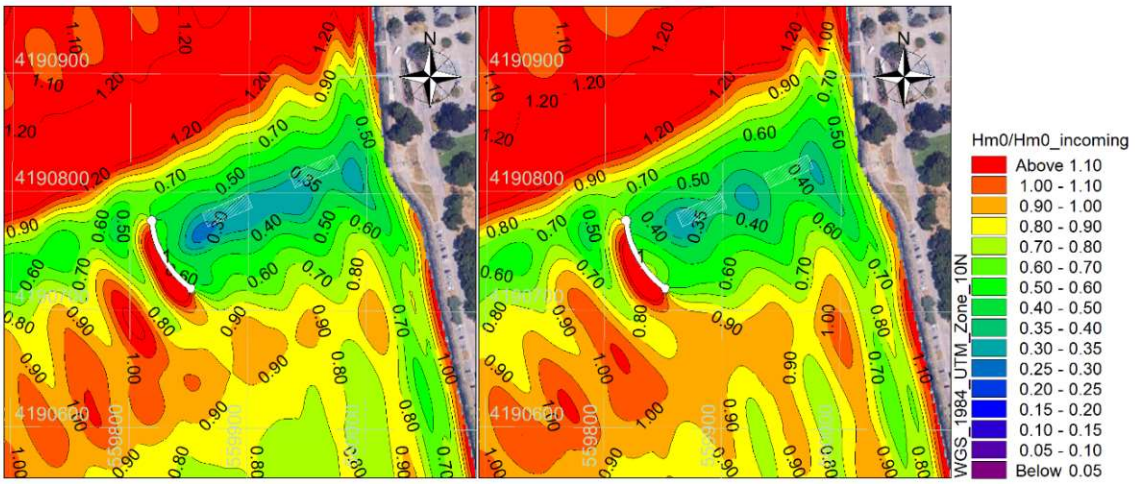


Figure C- 72 *Closeup of wave disturbance plot for wave direction = 255°N at MHHW. Left: Tp = 15 sec and Right: Tp = 18 sec, 250 ft breakwater length*

Appendix D - Sediment transport study

DECEMBER 2025
CITY OF BERKELEY

BERKELY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SEDIMENT TRANSPORT STUDY

COWI

DECEMBER 2025
CITY OF BERKELEY

BERKELY WATER TRANSPORTATION PIER FERRY (BWTPF) PROJECT

SEDIMENT TRANSPORT STUDY

ADDRESS COWI North America
555 12th Street
Suite 1700
Oakland, CA 94607

PROJECT NO.

A272767

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
Final	12 December 2025	Final	ARBV/GAML	GAML/BRLW	JRVS/JMC

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1 Introduction

The Berkeley Water Transportation Pier Ferry (BWTPF) Project is a major waterfront infrastructure initiative intended to enhance waterborne transit access along the San Francisco Bay (Figure 1-1). The project aims to revitalize a significant piece of waterfront infrastructure in Berkeley, California, transforming the existing pier into a modern, dual-purpose ferry terminal and public pier.

The project involves the design and implementation of a new ferry terminal at the foot of Berkeley Pier, including associated breakwaters, floating berths, a trestle, and shoreline plaza features. The concept layout is shown in Figure 1-2.

The sediment transport analysis was carried out for a 400 ft breakwater configuration, before design was optimized to a 300 ft breakwater length. The 100 ft reduction does not meaningfully change sedimentation patterns in the navigation channel. Modeled sedimentation within the berth and turning basin remains minimal, indicating that the results are also representative for the 300 ft breakwater layout and provide a reasonable approximation of sediment transport behaviour given the model and data limitations.

As part of the design process, COWI carried out hydrodynamic and sedimentation modeling to assess currents, wave-driven transport, and long-term sediment deposition within the dredged areas. The study indicates that sedimentation at the site is generally low, consistent with historical USACE surveys in the Berkeley Marina Channel, where long-term rates average approximately 0.07 ft per year.

For the BWTPF dredged areas, the six-year simulations show average infilling of approximately 0.20 ft per year in the approach channel and 0.09 ft per year within the berth pockets. Based on the modeled rates, maintenance dredging in the channel is expected approximately every 10 to 15 years depending on the selected operational depth (–10 ft or –9 ft MLLW), while the more sheltered berth pockets would require dredging roughly every 22 to 33 years. These findings align with post-dredge survey data collected in 2024–2025 and provide confidence in the localized predictions within the project footprint, while underscoring the importance of continued monitoring and future refinement of site-specific sediment inputs. This document summarizes the hydrodynamic and sedimentation modeling methodology, results, and implications for long-term maintenance planning for the BWTPF project.



Figure 1-1 BWTPF Location

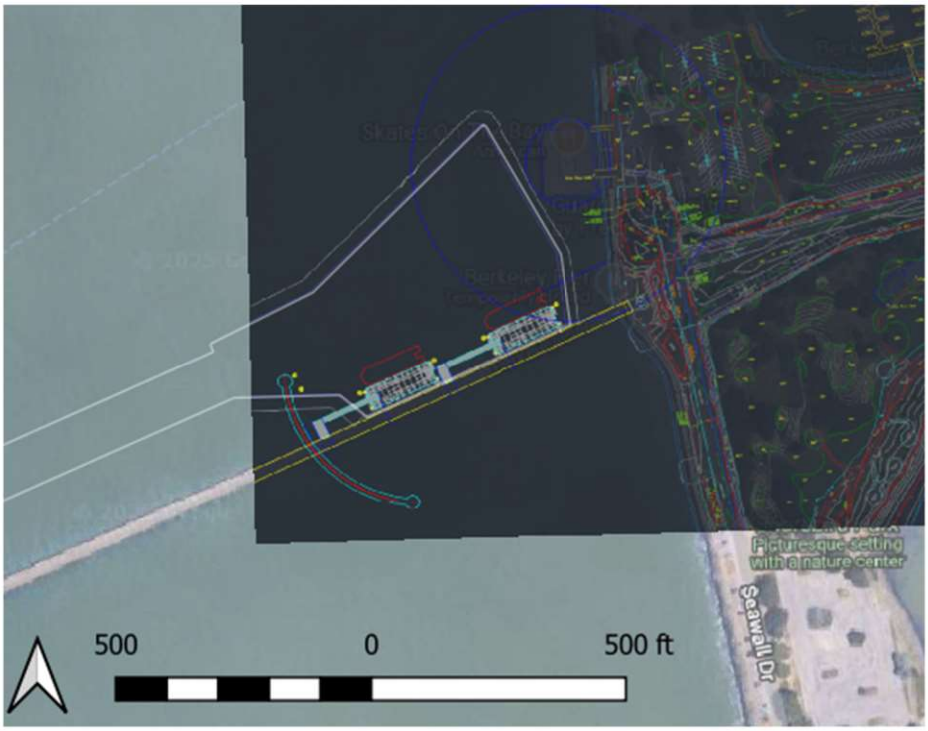


Figure 1-2 BWTPF Concept Layout

2 Data basis

This chapter summarizes the metocean data sets used for this study, which include various data sets (hindcast, measurements) used for setting up the bathymetry and boundary conditions for the numerical modeling.

2.1 NOAA CO-OPS

NOAA's Center for Operational Oceanographic Products and Services (CO-OPS) is the authoritative source for accurate, reliable, and timely tides, water levels, currents, and other coastal oceanographic and meteorological information. CO-OPS maintains ocean observing infrastructure, including more than 200 permanent water level stations on the U.S. coasts and Great Lakes, an integrated system of real-time sensors concentrated in busy seaports, and temporary meters that collect observations for tidal current predictions.

Table 2-1 shows the stations, the data types and the periods for which data were extracted. Location of the CO-OPS stations are shown in Figure 2-1.

Table 2-1 NOAA CO-OPS Stations and Data Description

Station ID	Station Name	Data Type	Period
9414290	San Francisco	Wind and Hourly Water Levels	1996-2024
9414750	Alameda	Wind and Hourly Water Levels	1996-2024
9414863	Richmond	Wind and Hourly Water Levels	1996-2024
s08010	Southampton Shoal Channel LB6	Currents	Aug 2023
s09010	Oakland Outer Harbor LB3	Currents	Aug 2023

Tidal datums obtained at the three CO-OPS stations are provided in Table 2-2.

Table 2-2 Tidal Datums at CO-OPS stations within the study area (ft MLLW).

Datum	Station 9414290 San Francisco	Station 9414750 Alameda	Station 9414763 Richmond
MHHW	5.8	6.6	6.0
MHW	5.2	6.0	5.4
MSL	3.1	3.4	3.3
MLW	1.1	1.1	1.1
MLLW	0.0	0.0	0.0
NAVD88	-0.1	0.2	0.0

2.2 NOAA NDBC

NOAA's National Data Buoy Center (NDBC) maintains a network of marine observation and monitoring infrastructure which collect and disseminate in-situ, real-time, quality-controlled observations in the marine environment.

Table 2-3 shows the stations, the data types and the periods for which data were extracted. Location of the NDBC station is shown in Figure 2-1.

Table 2-3 NOAA NDBC Station and Data Description

Station ID	Station Name	Data Type	Period
46026	San Francisco Offshore	Wind	1982-2024

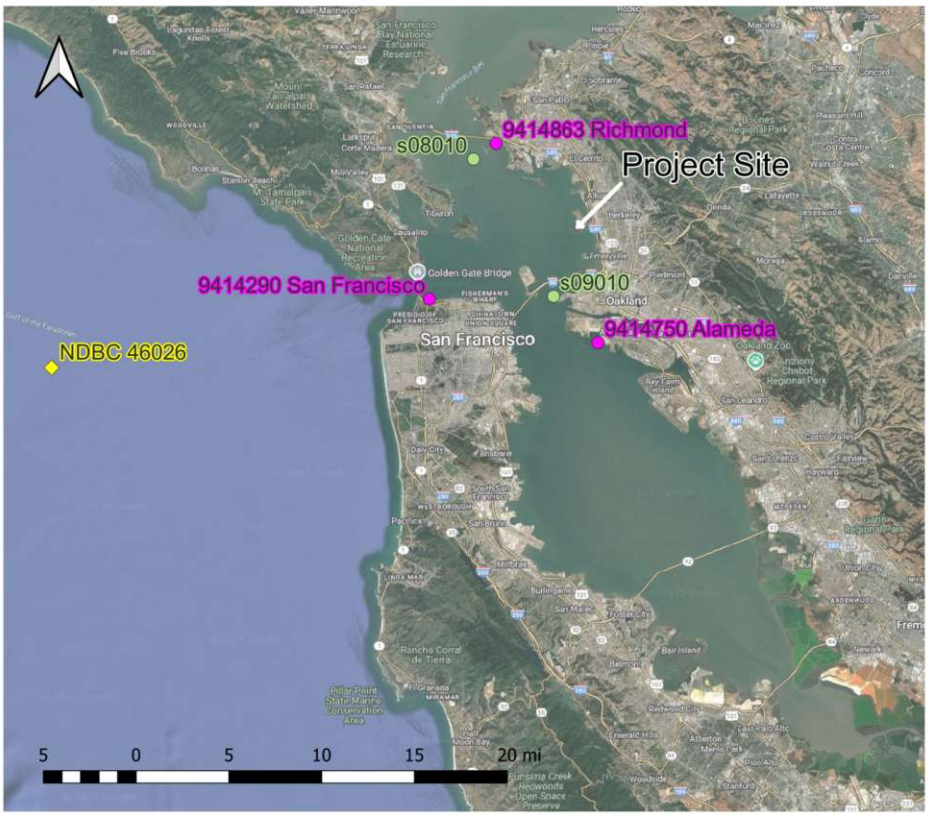


Figure 2-1 Wind, water levels, and current measurement locations.

2.3 Bathymetry

The model bathymetry for the sedimentation study was created based on the United States Geological Survey (USGS) Digital Elevation Model (DEM) for nearshore regions and survey data. Details of applied bathymetry data are given below:

- **USGS DEM** (2024): A digital bathymetric model with 1/3 arc-second (approximately 10 m) resolution, derived from diverse source data. The DEM covers the conterminous United States, Hawaii, Puerto Rico, other territorial islands, and limited areas of Alaska. See Figure 2-2 for extent of coverage available within the model domain used in this study.

- **Local Bathymetric Survey** (2024): Bathymetry data for the site (Figure 2-3) was produced by eTrac Engineering Inc, a Woolpert Company for COWI. Survey data was collected on July 24, 25, 27 and 29, 2024 and plotted on August 22, 2024.

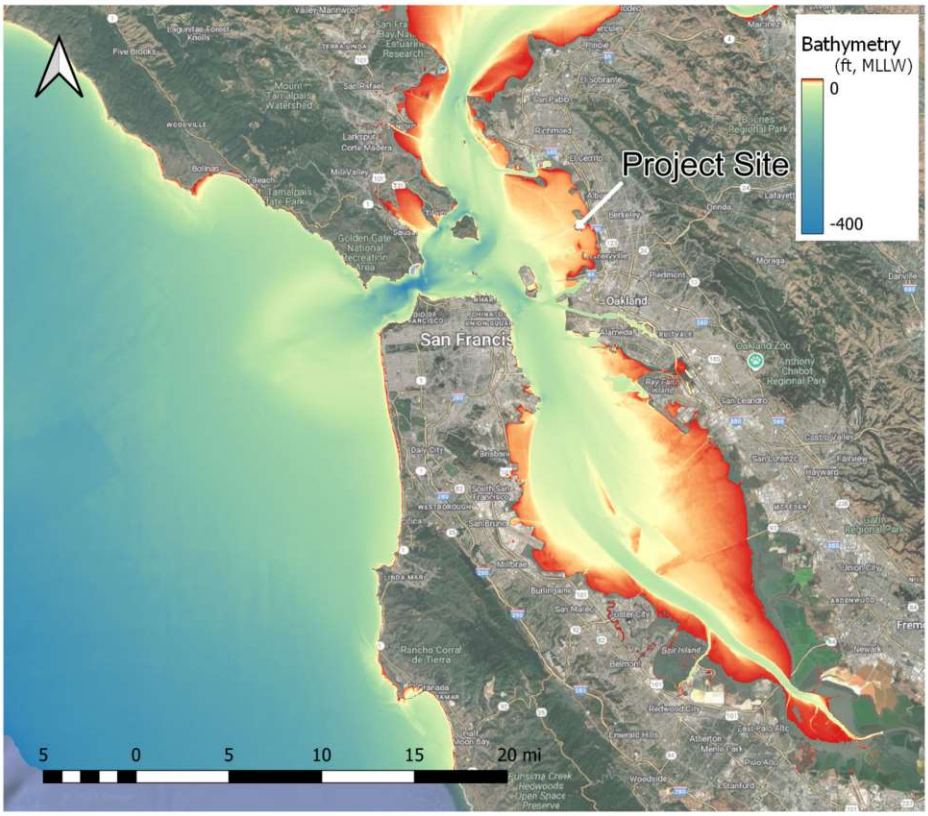


Figure 2-2 USGS DEM.

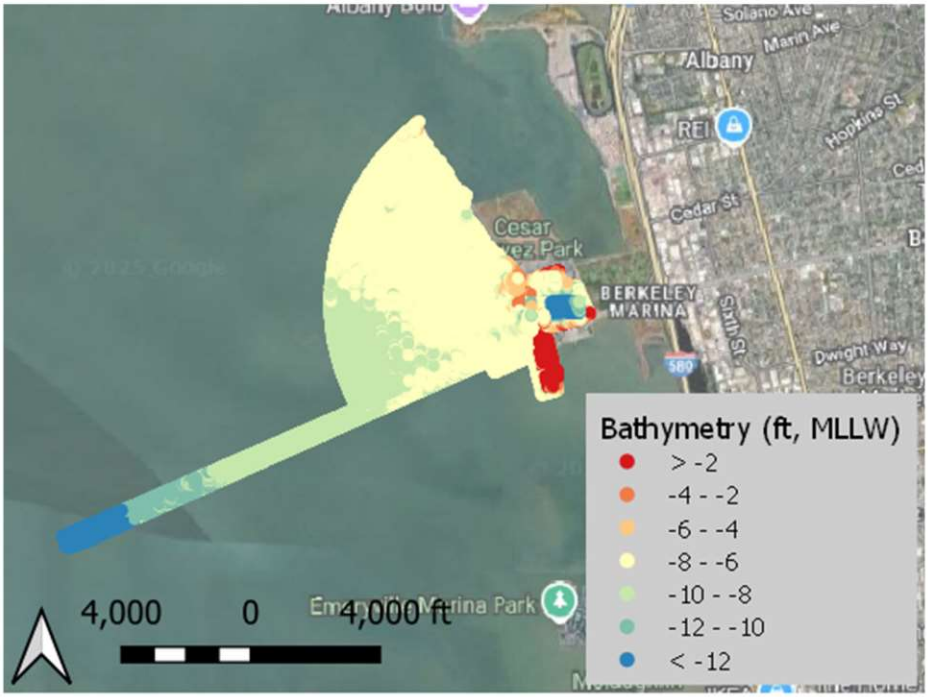


Figure 2-3 Bathymetric data from eTrac survey.

2.4 Seabed properties

The geotechnical report dated December 16, 2024, prepared by ENGEO Incorporated (Ref (ENGEO Incorporated, 2024)) contains soil boring samples at locations around the breakwater and berthing area. The seabed type in this area is generally of sandy silt type and the D50 is given to be 0.2 mm.

2.5 Suspended sediment concentration

A review of available literature such as USGS reports with suspended sediment concentration data in San Francisco Bay (Ref (Buchanan & Ganju, 2004) and Ref (Buchanan & Morgan, 2014)) show that the mean and median suspended sediment concentration in the Central San Francisco Bay region varies between 17-20 mg/L.

2.6 USACE hydrographic surveys for Berkely Marina Channel[1]

United States Army Corps of Engineers (USACE) hydrographic conditions surveys were retrieved to calculate the prevalent sedimentation rates in the project site. The latest survey data available is shown in Figure 2-4 to indicate the extent of data available. The sedimentation rates in the channel area indicated as black hatch in Figure 2-4 was calculated using survey data over four 3-year periods between 2011 and 2023 and are tabulated in

Table 2-4.

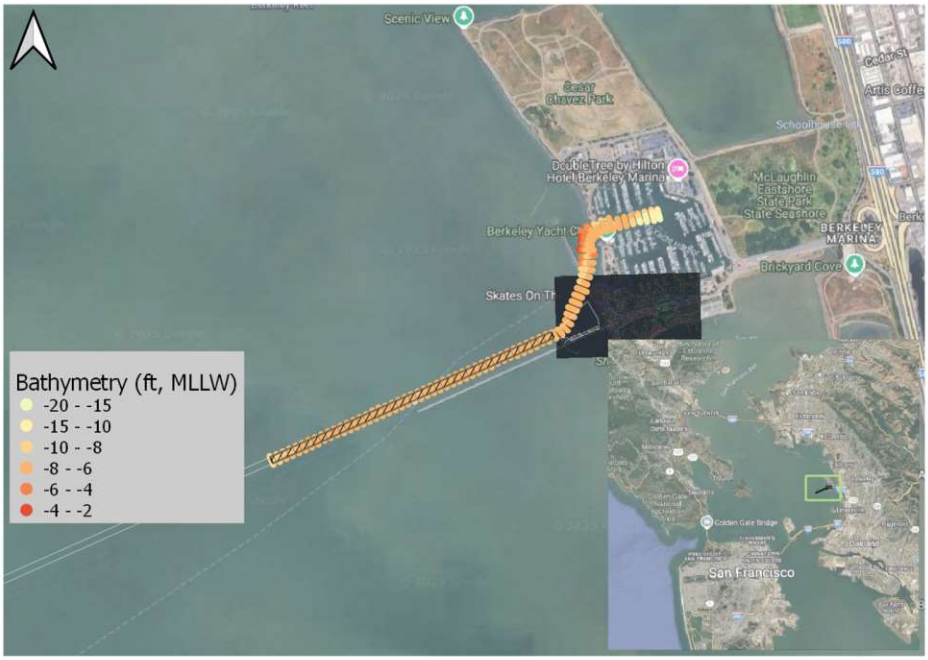


Figure 2-4 USACE hydrographic condition survey for Berkeley Marina Channel dated May 2024.

Table 2-4 Mean Sedimentation over different 3-year periods in Berkeley Marina Channel

From	To	Mean Sedimentation (ft)

[1] USACE Berkeley Marina Federal Navigation Channel (hereafter referred to as the Berkeley Marina Channel)

July 2011	Sep 2014	0.284
Sep 2014	Nov 2017	-1.267
Nov 2017	Apr 2020	0.147
Apr 2020	Dec 2023	0.044

Table 2-4 shows that the sedimentation was < 0.05 ft in the 2020-2023 period, indicating practically no sedimentation. The 2014-2017 period showed erosion to the channel, possibly related to dredging or issues with the survey/datum. These periods were excluded from the average used for calibration.

The two remaining periods with sedimentation have values of 0.147 ft. and 0.284 ft. Therefore, an average of 0.21 ft over 3 years, i.e., 0.07 ft/year is considered as the prevalent sedimentation rate for this channel segment.

3 Wind analysis

San Francisco Bay region is characterized by varied topography and water bodies which cause significant variations in the wind speed and directionality within and outside the Bay area. Figure 3-1 shows annual wind roses at the four measurement stations where data is available from 1997-2023 and indicates that the wind speed and directionality in the approaches to the project site indeed vary from the offshore region to the inside of San Francisco Bay.

While the offshore region winds are up to 30 knots and mainly from WNW-NW direction, wind speeds measured inside the San Francisco Bay are generally 10-15 knots with significant variation in the directionality.

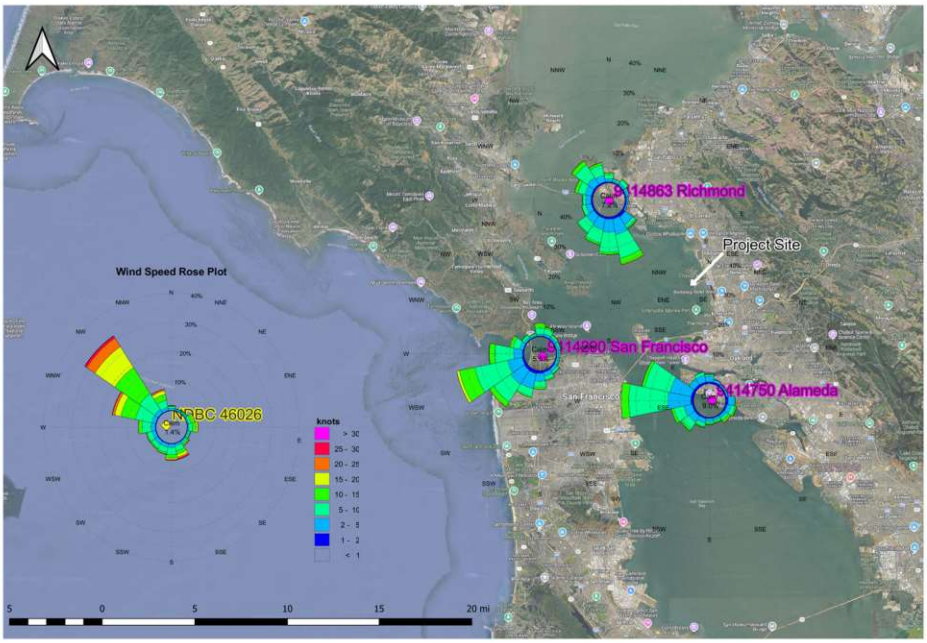


Figure 3-1 Wind roses from measured data in the vicinity of the project site.

Within San Francisco Bay, CO-OPS 9414290 is located to the east of the project site and CO-OPS 9414863 is located north. Winds at CO-OPS 9414290 are seen to be from SW-W directions (conforming to the geographical features of the area), whereas at CO-OPS 9414863, the winds are predominantly from S-SSE and from NW. At CO-OPS 9414750, which is located south of the project site, winds are from W-WNW directions.

Wind data from U.S. National Center for Environmental Information (NCEI) stations near the project site, shown in Figure 3-2, were also used to ascertain the variation in directionality near the project site.

Since wind conditions are one of the main inputs to a numerical sedimentation model, it is important to specify a spatially varying wind field in this scenario.



Figure 3-2 *Wind roses from measured data within the San Francisco Bay near the project site.*

4 Hydrodynamic modeling

A hydrodynamic modeling study was carried out by setting up a regional model to derive flow conditions at the site. The regional model was forced using astronomical tidal constituents for surface elevation and velocities from OSUTide database (Ref (Egbert & Erofeeva, 2002)) at its offshore boundaries.

4.1 Model description

The hydrodynamic model was developed using the MIKE 21 HDFM (Hydrodynamic Flexible Mesh) module of the comprehensive 2-dimensional MIKE 21 modeling package from DHI, Denmark. MIKE 21 HDFM is a modeling system for 2D free-surface flows (Ref (DHI, 2023)). It can be applied to a wide range of hydraulic and related phenomena. This includes modeling of tidal hydraulics, wind and wave generated currents, storm surges, and flood waves.

The HDFM module is the basic module of the system and is used in the simulation of hydraulics and related phenomena in lakes, estuaries, bays, coastal areas, and seas where flexibility inherited in the unstructured meshes can be utilized.

The applied MIKE 21 HDFM model requires the following main input for flow simulations:

- Bathymetry of the area.
- Hydrographic boundary conditions (water levels, flow velocity components or fluxes).
- Wind and air pressure of the area.
- Eddy viscosity and bed resistance (Manning number).

4.2 Modeling Methodology

The purpose of the hydrodynamic study is to derive flow conditions close to the project site for a 3-year period.

The following approach is adopted for the regional hydrodynamic study.

- A regional hydrodynamic model was established using the bathymetry data sets described in Section 2.3.
- Derivation of tidal boundary conditions from OSUTide database in the form of flow velocities and surface elevations.
- Wind speed and direction time series from NOAA NDBC Buoy 46026, CO-OPS Stations 9414290, 9414750, and 9414863 were used to create of a spatial wind field mosaic using the extents shown in Figure 4-1. The mosaic thus created was used as wind forcing. See Section 3 for wind data and directionality analysis.
- The regional hydrodynamic model was validated with water level and current speed measurements from NOAA CO-OPS at multiple locations within the model domain.

› The validated regional hydrodynamic model was used to generate boundary conditions for the local model.



Figure 4-1 Polygons used to create spatial wind field mosaic.

4.3 Model bathymetry

The model domain and mesh established for the regional hydrodynamic model is presented in Figure 4-2. The model domain includes the entire southern and central portions of San Francisco Bay and extends in the north up to San Pablo Bay (up to Benicia Bridge). The offshore boundaries are located near the NDBC Buoy 46026.

The domain is divided into several zones with different triangular mesh resolutions. The spatial resolution of the mesh varies from an average element size of ~500 m in the region outside San Francisco Bay, transitioning gradually to a minimum of 100 m in the Central San Francisco Bay where the site is located.

Figure 4-3 shows the model bathymetry.

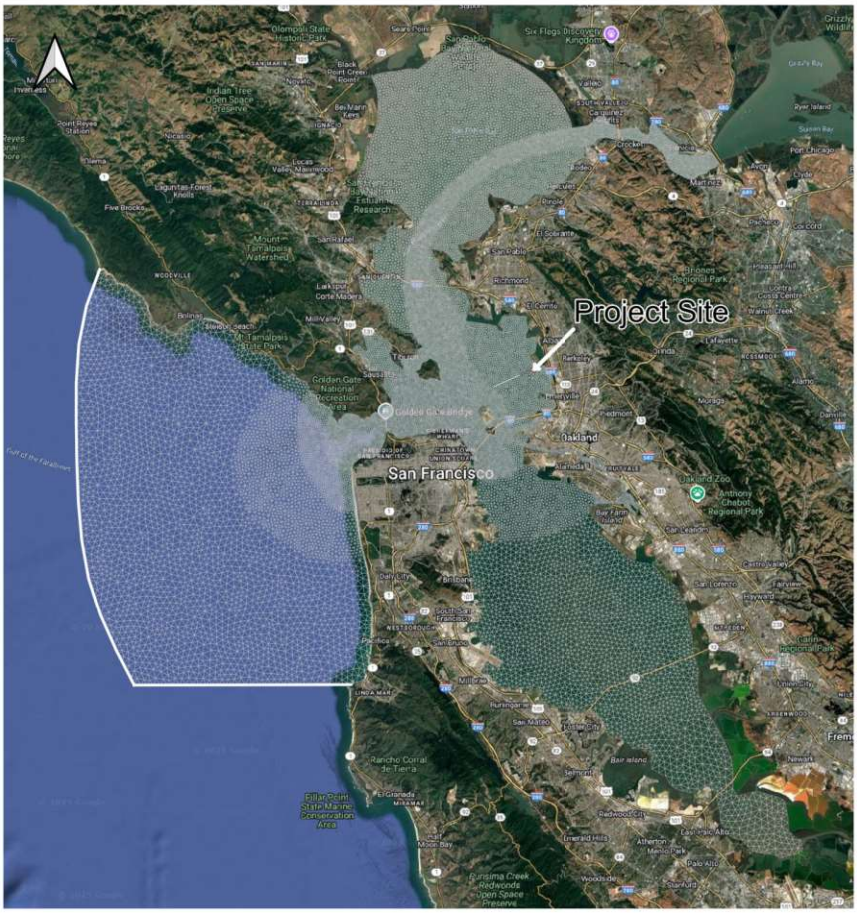


Figure 4-2 Model domain and mesh.

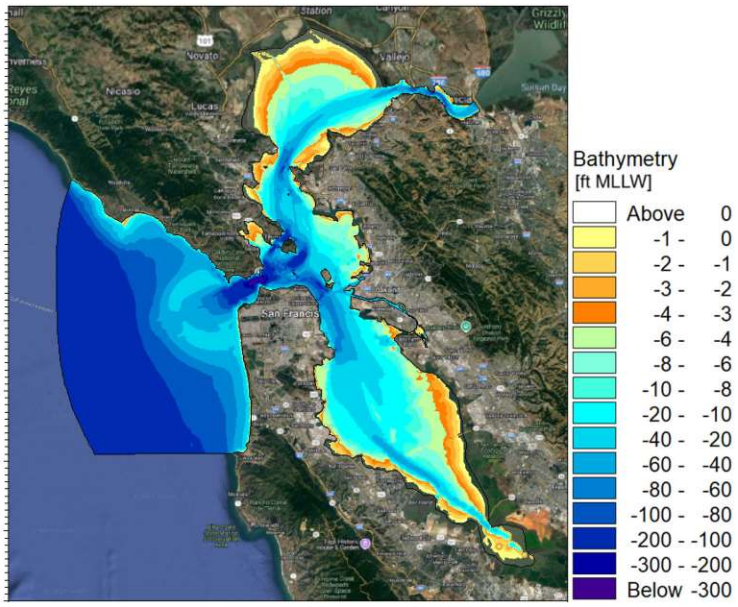


Figure 4-3 Model bathymetry.

4.4 Model validation

The regional hydrodynamic model was evaluated using measurements of water levels available at NOAA CO-OPS Stations 9414750 (Alameda), 9414290 (San Francisco), and 9414863 (Richmond). The model performance was also compared against measured current speeds at NOAA CO-OPS Stations s080010 and s090010. Figure 2-1 shows the locations of water levels and current measurements.

Figure 4-4 shows the comparison of water levels between model results and measurements during August 2023.

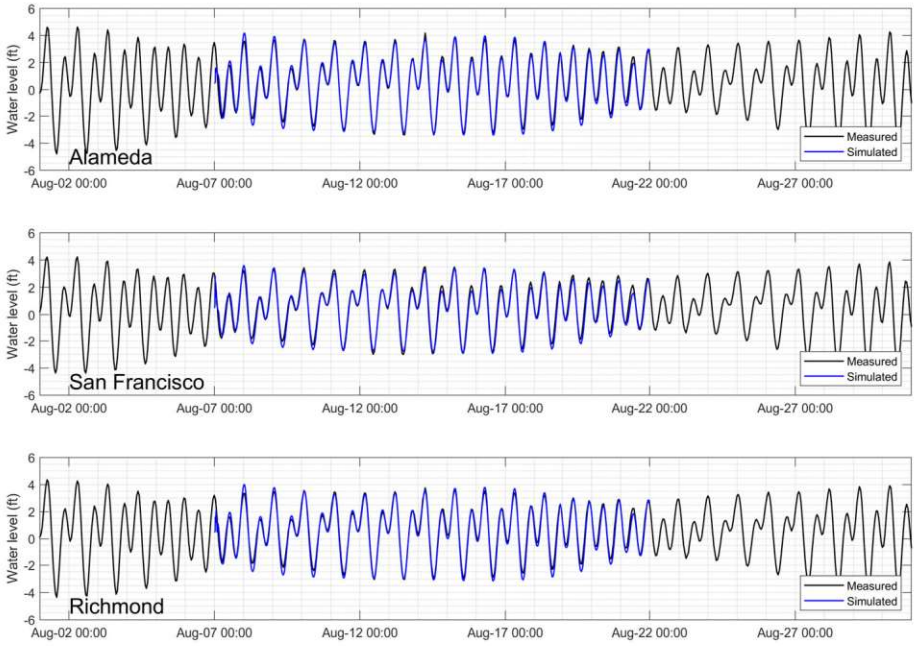


Figure 4-4 Comparison of water levels between model results and measurements at NOAA CO-OPS stations during August 2023.

A time series comparison of current speeds between model results and measurements are shown in Figure 4-5 during August 2023.

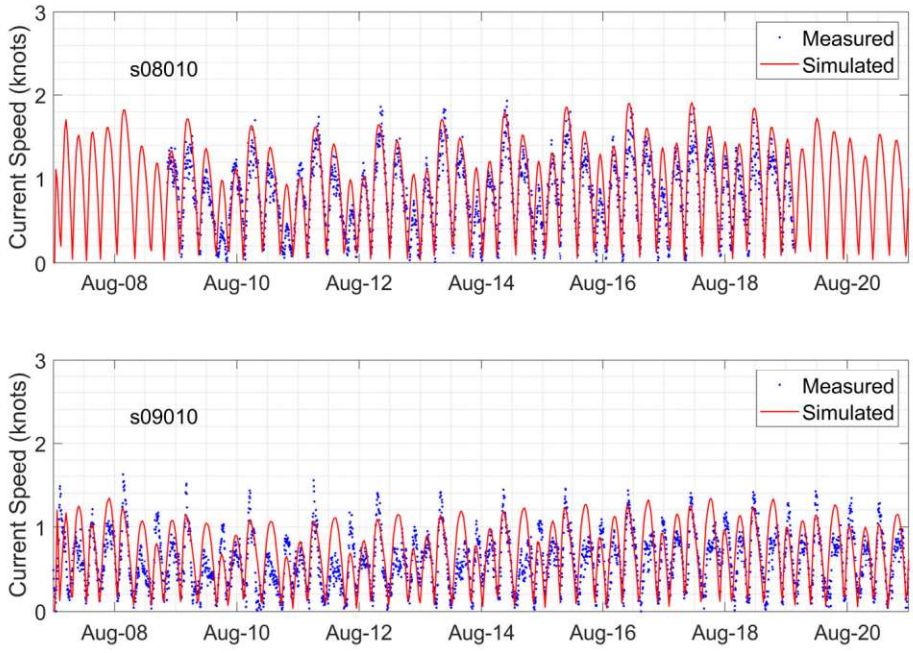


Figure 4-5 *Comparison of current speeds between model results and measurements at NOAA CO-OPS stations during August 2023.*

Figure 4-4 and Figure 4-5 indicate that the regional hydrodynamic model is adequately representing the flow conditions within Central San Francisco Bay.

4.5 Model parameters

Table 4-1 contains a description of model parameters selected to obtain the comparison shown in the sections above.

Table 4-1 *Model Parameters*

Parameters		Value
Flood and Dry	Drying depth	0.005 m (0.20 in)
	Wetting depth	0.1 m (4 in)
Eddy Viscosity	Smagorinsky formulation	0.28
Bed Resistance	Manning number	55
Wind Friction	Constant	0.001255

4.6 Model results

The regional hydrodynamic model was simulated for a continuous period of 3-years, i.e., 2020-2023 and was used to extract flow parameters at local model boundaries during sedimentation model setup and development.

Figure 4-6 and Figure 4-7 show flow pattern in the project site during typical spring flood and ebb phase of the tide. The current speeds at the project site are between 0.2 – 0.4 knots during peak flood and ebb tides and are generally less than 1 knot in the extended vicinity.

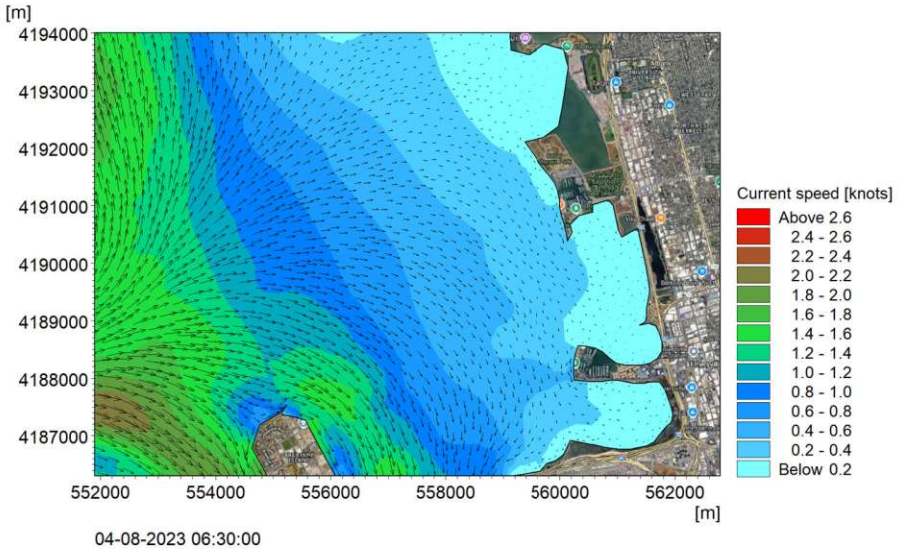


Figure 4-6 Flow field near the project site during a typical peak flood phase of a spring tide.

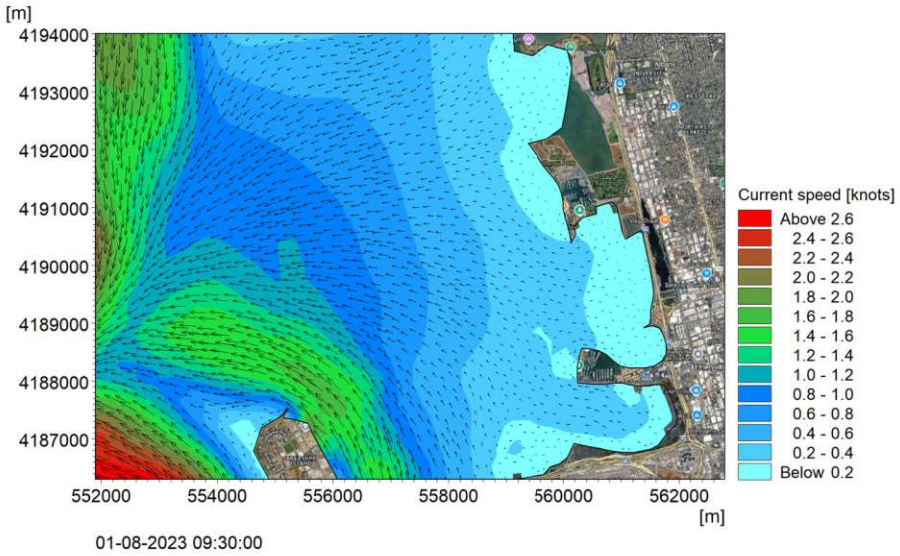


Figure 4-7 Flow field near the project site during a typical peak ebb phase of a spring tide.

5 Sedimentation model

The sedimentation assessment was carried out using the MIKE 21 coupled model that combines HD (hydrodynamics), SW (spectral wave), and MT (mud transport) models and describes the bottom evolution of the seabed for combinations of currents, water levels, and waves. The simulation was carried out for a period of 6 years to assess the sediment deposited at the project site with deepened bathymetry incorporated.

5.1 Modeling methodology

The following approach was adopted for the sedimentation study.

- A coupled model was established for the assessment of flow, wave, and sediment transport condition.
- Sediment grain size information for the sedimentation study was established based on the analysis of seabed sampling provided in Ref (ENGEO Incorporated, 2024). An average grain diameter of d_{50}=0.2 mm was assumed.
- Analysis of USACE hydrographic surveys available for Berkeley Marina Channel (ref Section 2.6) showed that the average sedimentation for a 3-year period was 0.21 ft. Hence, a 3-year calibration period was selected as the model duration to determine model parameters.
- Wave boundary conditions for the Coupled model was established from the Wave Transformation Study, ref. (COWI, 2023).
- Flow boundary conditions for the Coupled model were established using the validated regional model (Section 4).
- The coupled model was simulated for a 6-year period to determine sedimentation rates within the project site.

5.2 Modeled scenarios

Two scenarios were modeled:

Scenario 1: Existing seabed

Scenario 2: 400-ft breakwater with approach channel and berth pocket dredged to -12 ft MLLW.

5.3 Model bathymetry

A local model was established for the sedimentation study. The local model domain and mesh is presented in Figure 5-1 and is defined for extent of ~5 kms west and north-south of the project site. The model mesh is comprised of triangular elements, whose side, i.e., mesh resolution, varies from 100 m at the offshore boundaries to 10 m in the approach channel and close to the project site.



Figure 5-1 Model domain and mesh used for sedimentation study.

The model bathymetry used for the existing seabed scenario is given in Figure 5-2 and the model bathymetry for 400-ft breakwater scenario is given in Figure 5-3.

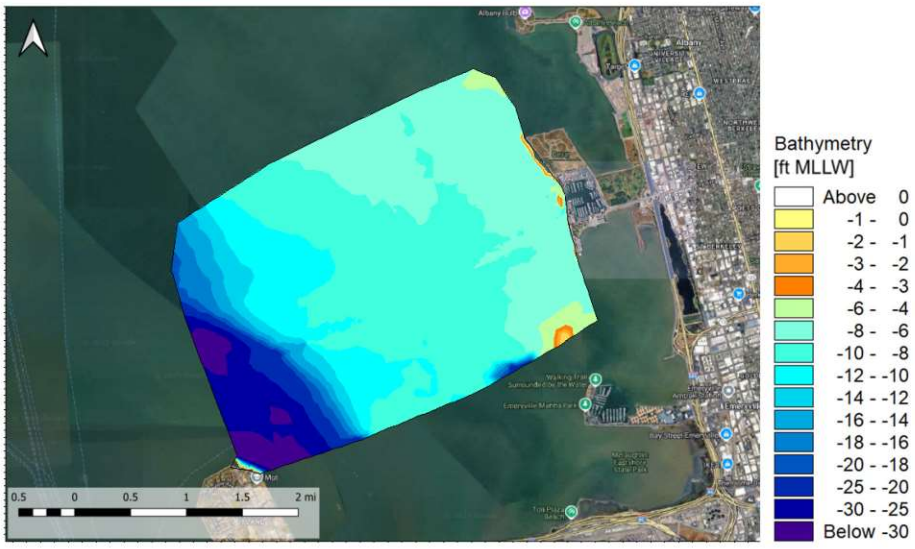


Figure 5-2 Model bathymetry used for existing seabed scenario.

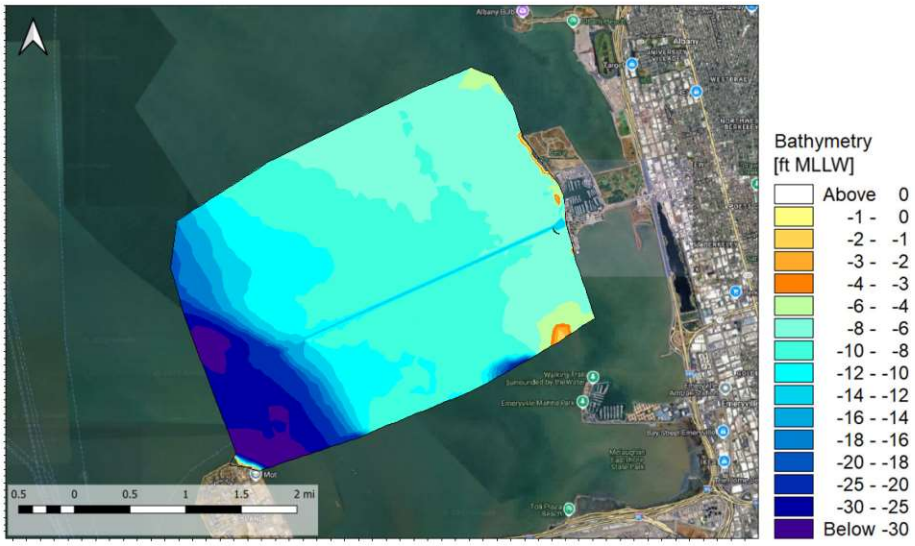


Figure 5-3 *Model bathymetry used for 400-ft breakwater scenario with approach channel and berth pocket dredged to -12 ft MLLW.*

5.4 Model results

The coupled model was simulated for the two scenarios described in Section 5.2. The model parameters were adjusted such that the sedimentation in the existing Berkely Marina Channel was 0.2 ft for a 3-year period. Subsequently, the same parameters were applied to simulate the scenario with 400-ft breakwater and deepened approach channel and berth pockets.

Both scenarios were simulated for a 6-year period to obtain net deposition patterns and sedimentation assessment within different areas.

Figure 5-4 and Figure 5-5 illustrate the net deposition patterns for the two scenarios simulated after 6 years of simulation. Figure 5-4 presents the results for the existing scenario. Similarly, Figure 5-5 shows the estimated sedimentation pattern for the scenario with 400-ft breakwater and deepened approach channel and berth pockets.

Table 5-1 provides the net sediment deposition estimated within the different areas for the two scenarios simulated.

Figure 5-6 shows the extent of the areas provided in Table 5-1.



Figure 5-4 *Net deposition map for existing seabed scenario using results from 6 years of simulation. Berths and dredge envelope shown for illustration purpose only.*



Figure 5-5 *Net deposition map for 400-ft breakwater scenario using results from 6 years of simulation with approach channel and berth pocket dredged to -12 ft MLLW.*

Table 5-1 Summary of mean sedimentation rates using results from 6 years of simulation.

Area	Mean sedimentation in existing seabed scenario (ft/yr)	Mean sedimentation with 400-ft breakwater, approach channel and berths dredged to -12 ft MLLW scenario (ft/yr)
Berkeley Marina Channel	0.05	0.10
Berth 1	0.07	0.04
Berth 2	0.04	0.08
Berth Pocket	0.05	0.09
Channel to -12 ft MLLW	0.1	0.20

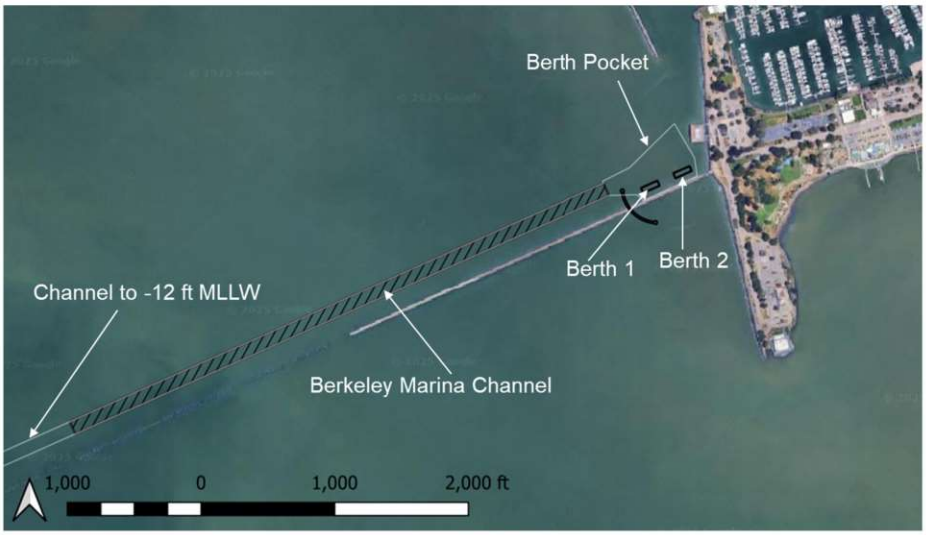


Figure 5-6 Areas for net deposition rates calculation.

Table 5-2 presents the simulated sedimentation rates in each year of the BWTPF project over the six-year modeled period. The table shows results for the scenario after dredging within the overall dredged areas (berth and channel) and separated berth pocket and channel. The results show that overall sedimentation after dredging across the full dredged area averages 0.15 ft/yr, with the highest accumulation occurring in the first year (0.32 ft). Within the berth pockets, sedimentation is relatively low and stable, averaging 0.09 ft, while the channel areas (excluding berths) experience higher and more variable deposition, with an average of 0.20 ft and a peak of 0.48 ft in the first year.

These results indicate that the channel is more prone to sediment accumulation than the berth pockets, and that most of the infilling occurs during the early years following dredging, after which deposition rates decline and stabilize. The relatively low deposition within the berth pockets reflects the sheltering effect of the breakwater, which reduces wave-driven sediment transport into these more protected areas and helps limit infilling compared to the more exposed channel.

Table 5-2 Yearly net sediment deposition per dredged area

Year	Overall sedimentation (ft) – full dredged area	Sedimentation in berth pocket (ft)	Sedimentation in channel (ft) Not including berth
1	0.32	0.10	0.48
2	0.18	0.10	0.24
3	0.12	0.09	0.14
4	0.11	0.09	0.12
5	0.10	0.08	0.11
6	0.09	0.08	0.11
Annual Average	0.15	0.09	0.20

5.5 Maintenance dredging

Table 5-3 provides an estimate of the maintenance dredging frequency required for the BWTPF project, using the average annual sedimentation rates obtained from the six-year simulations. The results are based on an average of 0.20 ft/year of infilling in the channel and 0.09 ft/year in the berth pockets. Two potential operational depths (–10 ft and –9 ft) were assessed to determine the approximate time until dredging would be needed, recognizing that the current dredged depth is –12 ft.

The analysis indicates that the channel would require dredging approximately every 10 years if maintained at –10 ft, extending to 15 years at –9 ft. In comparison, the berth pockets accumulate sediment at a slower rate due to the sheltering provided by the breakwater, leading to longer dredging intervals of 22 and 33 years for the same operational depths.

Overall, the findings suggest that dredging frequency is strongly dependent on the selected operational depth. A deeper target elevation would ensure improved navigational reliability but comes with shorter dredging intervals, particularly in the channel. Operating at a shallower depth reduces the dredging frequency significantly but may limit navigational access for higher ferry drafts. Balancing these trade-offs is critical for optimizing long-term maintenance costs while maintaining the required depth for safe operations.

Table 5-3 Estimated dredging intervals (years) for channel and berth pockets

Operational Depth (ft)	Est. Years until Maintenance Dredging	
	Channel	Berth Pocket
-10	10	22
-9	15	33

5.6 Post-dredging sedimentation assessment at Berkeley Marina

Dredging operations at the Berkeley Marina were completed in late 2024. A post-dredge bathymetric survey was subsequently conducted in December 2024 to establish baseline conditions. A follow-up survey was carried out in June 2025, enabling a comparative analysis of the two datasets to evaluate sedimentation rates during the initial six-month period following dredging.

Although the target dredging depth was -8 ft MLLW, the spatial proximity of the dredged area to the BWTTP channel allows for a qualitative assessment of sediment dynamics in relation to the channel's influence. Figure 5-7 highlights, within the red rectangle, the specific region where the two consecutive surveys were analysed. This comparative analysis provides insight into early-stage infill patterns and sediment redistribution processes post-dredging.



Figure 5-7 Berkeley Marina dredged area (surveys 2024 and 2025)

Figure 5-8 shows a differential analysis of bathymetric data collected over a six-month interval, comparing the December 2024 post-dredge survey with the June 2025 follow-up. To increase interpretability, the colour scale was selectively muted for regions exhibiting minimal sedimentation or erosion, improving visual contrast in areas of significant morphological change. The results indicate a mean sediment accumulation of approximately 0.3 ft within the delineated area of interest. This observed change in seabed elevation provides a preliminary estimate of the sedimentation rate during the initial post-dredging period.

A comparative assessment of sedimentation rates between the Berkeley Marina and the BWTPF dredge channel reveals consistent patterns in post-dredging infill dynamics. The BWTPF channel, unaffected by breakwater influences and thus directly comparable to the dredged section at Berkeley Marina, exhibited an average modeled sedimentation rate of 0.48 ft during the first year following dredging. It is well established that sedimentation rates are typically higher in the initial months post-dredging, gradually decreasing over time as the system approaches equilibrium. In this context, the observed mean sediment accumulation of 0.3 ft within the Berkeley Marina's area of interest during

the first six months aligns closely with model predictions for the BWTPF channel. This correspondence supports the validity of the sedimentation model and underscores the similar depositional processes governing both sites during the early post-dredging phase.

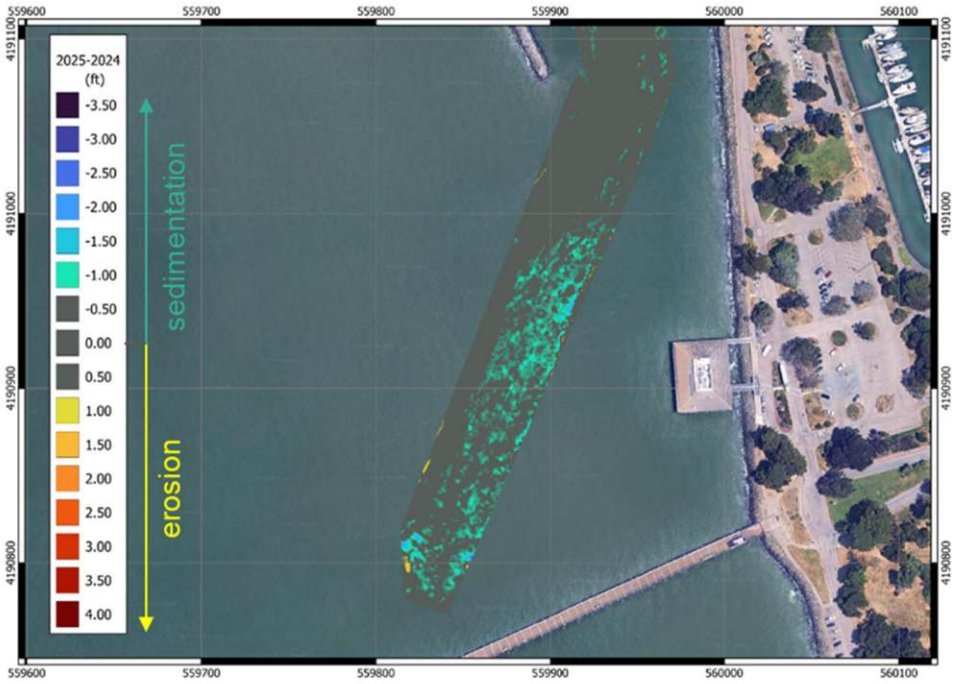


Figure 5-8 Bathymetric change over six months post dredge

5.7 Conclusion

The model results presented in this report are recommended to be used only within the project footprint, i.e., the areas where bathymetry changed or deepened, due to reasons as discussed in this section.

The survey data used to derive existing mean sedimentation rates near the project site are only available along the narrow segment called “Berkely Marina Channel” (see Figure 5-6). Hence any model sedimentation away from the channel has no survey data available for comparison.

Moreover, no site-specific suspended sediment concentration data, i.e., the sediment dispersed in water column, is available. This parameter is one of the main inputs, which determines the amount of sediment available or susceptible to settling in the seabed. Due to unavailability of site-specific data, a value derived from available literature, such as USGS reports with suspended sediment concentration data in San Francisco Bay (Ref (Buchanan & Ganju, 2004) and Ref (Buchanan & Morgan, 2014)), was used. However, suspended sediment concentration could be much lower in the project site due to very low sedimentation rates experienced in the existing Berkeley Marina Channel (0.07 ft / year average, see Section 2.6).

It should also be noted that the sedimentation experienced in the existing Berkeley Marina Channel, although generally low, is varying between 0.015 and 0.09 ft / year, as shown in Section 2.6. USGS report on sedimentation in San Francisco Bay (Ref (Fregoso, Foxgiver, & Jaffe, 2024)) also indicates that some segments along the channel deepening footprint have experienced sedimentation rates of

0.03-0.1 ft/year historically. Although this range encompasses the sedimentation rate selected to validate the model in this study, the USGS report also shows that there are segments where sedimentation is almost zero. In this study, a value slightly higher than median, i.e., 0.06-0.07 ft/year was considered mainly because very low or negligible sedimentation rates (as seen in USGS report) are hardly achievable in a numerical modeling exercise for such large area.

Comparison with the 2024/2025 Berkeley Marina Channel survey further supports this conclusion, as the measured sedimentation rates fall within the same order of magnitude as the modeled results for the first-year post-dredge. This alignment between recent survey data and the model increases confidence in the localized predictions within the project footprint, while also emphasizing the need for continued monitoring to refine site-specific assumptions. It is therefore recommended that the sedimentation study and methodology be revisited as more field data, particularly additional bathymetric surveys and suspended sediment concentration measurements, become available.

The sediment transport analysis was conducted in parallel with the breakwater length optimization study, based on a layout incorporating a 400 ft breakwater. A subsequent reduction of the breakwater length by approximately 100 ft is not anticipated to cause any significant variation in sediment transport rates within the navigation channel. In addition, the simulated sedimentation rates at the berth and turning basin are already minimal. Therefore, the overall results remain representative for the 300 ft breakwater configuration and provide a reasonable approximation of sediment transport patterns, considering the inherent limitations of the model and input data.

6 References

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