Coastal Inlets Research Program

Kikiaola Light Draft Harbor Monitoring Plan

Part 2: Numerical Wave Modeling for Evaluation of Structural Alternatives

Zeki Demirbilek, Lihwa Lin, Okey G. Nwogu, William C. Butler, Kent K. Hathaway, and Thomas D. Smith

March 2015

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Kikiaola Light Draft Harbor Monitoring Plan
Part 2: Numerical Wave Modeling for Evaluation of Structural Alternatives

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Final report
Approved for public release; distribution is unlimited.

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Under Coastal Inlets Research Program
Abstract

This report documents field data collection and monitoring, and numerical wave modeling for evaluation of structural alternatives for modification of Kikiaola Light Draft Harbor (KLDH), located on the southwest coast of Kauai Island, Hawaii. The entrance and interior of KLDH is largely affected by waves transforming over a wide shallow reef. Unfavorable navigation conditions for boats using this harbor are caused by high-energy northwest swells, southern swells, and storms. The harbor entrance is flanked by two breakwaters that shelter the harbor interior from waves, current, and sedimentation. A six-month field data collection study provided wave data for calibration of numerical models. Surveys of bathymetry and structures were conducted to obtain data for numerical modeling.

The hydrographic survey data collected was used in the numerical modeling study to evaluate eight structural alternatives. CMS-Wave, a spectral wave model, was used to transform deepwater waves measured by buoys to the project site. BOUSS-2D (B2D), a Boussinesq-type wave model, was used to investigate alternatives representing structural modifications inside and outside of the harbor. These modifications were breakwaters or spurs. Benefits and consequences of modifications were evaluated for improving navigation safety and utilization of the existing harbor. Potential impacts of each alternative on navigation in the entrance channel, access channel, and harbor basin were examined.

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Preface

The Kikiaola Light Draft Harbor (KLDH) study was performed by the Coastal and Hydraulics Laboratory (CHL) of the USACE Engineer Research and Development Center (ERDC) at the request of the U.S. Army Engineer Honolulu District (POH).

Dr. Zeki Demirbilek of the Harbors, Entrances, and Structures Branch (HNH) of the Coastal Hydraulics Laboratory (CHL) Navigation Division, and Dr. Lihwa Lin of the Coastal Engineering Branch (HNC) of the CHL Navigation Division conducted the study and wrote this report, with contributions by William Butler of the Coastal Field Data Collection and Analysis Branch (HNF) of the CHL navigation Division, and Kent Hathaway of the Coastal Observation and Analysis Branch (HFA) in the CHL Flood and Storm Protection Division. The Coastal Inlets Research Program (CIRP) provided partial support for this study, a research and development program in the Navigation Business Line administered by the U. S. Army Corps of Engineers (USACE) Headquarters under the direction of W. Jeff Lillicrop, Technical Director, and Charles E. Wiggins, Associate Technical Director. Dr. Julie Rosati was Program Manager of CIRP during the period of study. Thomas D. Smith of the Honolulu District provided input and oversight for the study.

During this study period, the technical work was conducted under the general administrative supervision of Donald Ward and Jackie Pettway, the Acting Chief of Branch and the CHL Navigation Division, respectively. Richard Styles and Edmond Russo were the Acting Deputy Directors, and Jose Sanchez was Director of CHL.

At the time of publication of this report, COL Jeffrey R. Eckstein, EN, was Commander and Executive Director of ERDC, and Dr. Jeffery P. Holland was ERDC Director.
## Unit Conversion Factors

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1 Introduction

1.1 Background
This report describes Part 2 of a two-part study conducted for Kikiaola Light Draft Harbor (KLDH), located on Kauai Island, Hawaii. Part 1 was a field data collection study, which acquired detailed bathymetric surveys of the reef, harbor entrance, and harbor interior, along with tripod-LiDAR (Light Detection and Ranging) surveys of the structures. The field study also collected wave and current measurements at five gauge locations: two gauges outside and three gauges inside the harbor. Part 2 is the numerical modeling study that develops wave estimates inside and outside of the harbor starting at the 60 ft (18 m) depth contour offshore. Field data collection and numerical wave modeling have provided critical information for assessing existing and proposed modifications to the harbor.

The harbor entrance is flanked by two breakwaters and a large reef system that extends to about 60 ft (18 m) depth before transitioning to much deeper water, where offshore depths drop sharply to an extremely deep ocean canyon. Consequently, incident wind waves are not significantly affected by the bathymetry until they encounter the reef system. Water depth over rough and rugged reef bottom decreases approaching the harbor and adjacent shorelines, and this causes waves to break. Breaking waves generate currents that mobilize and transport sediments towards and into the harbor. The focus of numerical modeling was the assessment of the efficacy of proposed alternatives for minimizing penetration of wave energy into the harbor. The impacts of waves on navigation were examined using the Boussinesq and CMS-Wave models. Details of numerical modeling study, data requirements, tasks, results, and major findings are provided in this report. The purpose of this chapter is to present an overview of the study.

1.2 Objective
The objective of this investigation was to analyze characteristics of waves in relation to improvement of navigation at the entrance and in the harbor. An analysis of hydrodynamics and sediment transport were not in the scope of this study. The effects of waves on navigation at Kikiaola Harbor were investigated using field data and numerical modeling.
1.3 Study area

The study area is Kikiaola Harbor, which was originally constructed by the State of Hawaii in 1959. It is a small boat harbor located along the southwest coast of the island of Kauai, between the towns of Kekaha and Waimea (Figure 1). Figure 2 is a sketch of the harbor configuration prior to federal modifications in 2009.

Federal project modifications were authorized under Section 101 of the River and Harbor Act of 1968 to improve navigation and utilization of the Kikiaola Light Draft Harbor (KLDH). The project was completed in 2009 at a cost of $18,771,500. The shaded features modified by the federal project are shown in Figure 3. These general navigation improvements were designed to eliminate dangerous breaking wave conditions within the entrance channel and allow for the safe passage of vessels entering the basin. The non-federal sponsor, State of Hawaii Department of Land and Natural Resources (DLNR), continues to work towards fulfilling its obligation to construct items of non-federal cooperation such as the sand-bypass system and west breakwater root repair. These features are integral to the overall function of the modified harbor project.

Figure 1. Study area location map for Kikiaola Harbor, HI.
Figure 2. The original harbor configuration (1959-2008).

Figure 3. Schematic of the federal project modifications (shaded areas).

The schematic of modifications made in the 2009 federal project are depicted in Figures 3 and 4. These included a dredged 700 ft (214 m) long
entrance channel varying in width from 105 to 205 ft (32 to 62 m) to a depth of 11 ft (3.4 m), a dredged 320 ft (98 m) long access channel varying in width from 70 to 105 ft (21 to 32 m) to a depth of 7 ft (2 m), removal of an existing 150 ft (46 m) outer east breakwater stub (Figure 2), raising the crest elevation and flattening the seaward slope of approximately 764 ft (233 m) of the existing east breakwater, removing and reconstructing the 71 ft (22 m) long inner east breakwater, and modifying 245 ft (75 m) of the seaward portion of the existing west breakwater.

Figure 4. Areas dredged in the federal project (in blue, green, yellow, and purple).

1.4 Problem statement

Numerical modeling studies conducted with HARBD (Thompson et al. 1998) and utilized for design of the KLDH federal general navigation features (GNFs) indicated that the recommended improvements would result in reduced wave action within the harbor entrance channel and basin. Following construction of the federally recommended GNFs in 2009, harbor users have expressed concern that navigability and berthing conditions at the harbor have since deteriorated.
Navigation Business Line operation and maintenance funds were provided in the FY12 Honolulu District Work Plan for monitoring at KLDH. The monitoring plan was designed to augment existing data with a field-data collection program in support of state-of-the-art numerical modeling of the existing harbor configuration and alternative structural modifications designed to improve navigation conditions within the harbor entrance channel and basin. The sketches of proposed alternatives are shown in Figure 5 through Figure 12. Field-data collection and monitoring were conducted in 2012 and 2013, multi- and single-beam hydrographic surveys were acquired, and wave data were collected at five locations over a period of four months (Oct 2012-Jan 2013). Subsequently, Boussinesq 2-D (BOUSS-2D), a Boussinesq numerical wave model, was used with the survey data for grid development and the wave-gauge data were used for model calibration and verification. Comparison of results from the initial numerical modeling effort with HARBD and the new BOUSS-2D investigations, as well as evaluation of potential benefits associated with eight proposed alternatives, are presented below.

1.5 Description of Alternatives

For completeness, the harbor configuration prior to implementation of the federal improvements in 2009 is provided in Figure 2. The proposed configurations of eight alternatives were evaluated and compared to the present harbor to determine extent of wave-energy reduction in the channel and harbor. These alternatives were numbered as 1, 2, 3, 3a, 4, 5, 6, and 7, and abbreviated as “Alt-xx”, where “xx” is the ID of an alternative. For consistency, the existing harbor, serving as the baseline in the evaluation of alternatives, was designated as “Alt-0”. A brief description of all configurations follows.

1.5.1 Existing Harbor

The existing harbor (Figure 4) is the federally improved configuration in 2009 that included the removal of an exterior 150 ft (46 m) spur, as well as shortening of the interior spur from the east breakwater (Figure 2). The modifications in 2009 also included deepening of the entrance and access channels, and parts of the interior harbor. These are shown in Figure 4. Figure 5 depicts the BOUSS-2D model grid coverage of the present harbor, which hereafter will also be referred to as “Alt-0,” since it serves as the baseline for evaluating the proposed eight alternatives. The approximate
boundaries of deepened areas have been color-coded in Figure 4 and marked by brown lines in Figure 5.

Figure 5. BOUSS-2D grid of existing harbor (Alt-0).

1.5.2 Alternative 1 (Sheet Pile Wall)

Figure 6 is a sketch of Alternative 1 (Alt-1), which consists of a 300 ft long sheet pile wall located between the access channel and wood dock (Figure 3). Currently, waves penetrating into the harbor entrance propagate unobstructed towards the wood dock and concrete ramp. The Alt-1 structure is intended to shelter the dock from incident wave energy impacting that section of the inner harbor. The purpose of the sheet-pile structure is to reflect incident-wave energy towards the mooring area and to the east breakwater, where it would dissipate along the structure’s toe berm. With the implementation of Alt-1, the wave threshold for on-off loading of boats at the dock could be significantly reduced, if not eliminated completely. The reduction in the amount of wave energy reaching the dock for the existing condition (Alt-0) and the Alt-1 configurations are quantified later in
Chapters 3 and 4 of this report. Alt-1 also has the potential to improve launch and landing operating conditions at the boat ramp.

Figure 6. Configuration of Alt-1.

The implementation of Alt-1 might have two potential negative consequences. There could be an increase in wave action in the eastward end of the access channel, and less maneuvering space remaining at the dock and in the mooring areas. The sheet-pile wall will reflect nearly 100 percent of waves towards the access channel, mooring area, and east breakwater. When reflected and incident waves interact in the interior harbor, and depending on the characteristics of superposition of incident and reflected
waves, wave heights in the access channel could increase or decrease. As boats stage just off of the dock during periods of peak harbor use while waiting for access to load/unload passengers and goods, Alt-1 would decrease the boats’ available maneuvering space at the dock and adjacent areas.

1.5.3 Alternative 2 (West Breakwater Spur)

Alt-2 (Figure 7) includes a 250 ft spur added to the west breakwater to increase its length to reduce the amount of wave energy getting into the harbor through the entrance. According to harbor users, following the removal of the east breakwater outer and inner spur in 2009, more wave energy is able to penetrate into the harbor because the width of the entrance has been increased. The premise of Alt-2 is to extend the length of west breakwater with a short spur that follows the channel’s western edge. This extension would shelter the harbor basin by intercepting incident wave energy from the west (W) and southwest (SW). Therefore, Alt-2 should reduce the amount of wave energy entering the harbor from W and SW directions by obstructing harbor entrance. To construct the proposed extension at the tip of west breakwater, the crest of the structure may need to be temporarily lowered to allow construction access, which was the case during the federal improvements to the west breakwater.
1.5.4 Alternative 3 (West and East Breakwater Spurs)

Alternative 3 (Figure 8) includes two exterior spurs that follow the channel edges. The west breakwater spur in Alt-2 is retained and a 200 ft long short spur would be added to the east breakwater along the eastern limits of the entrance channel. Wave directions that impact navigability within the harbor are generally out of the south through the southwest directions. Consequently, the addition of east breakwater spur in the proposed alignment is intended to reduce primarily the shoaling in the entrance channel,
and may also be beneficial as a wave attenuator. Potential wave attenuation benefits of the dual exterior spurs will be quantified in Chapters 3 and 4 through comparison of the Alt-0, Alt-2, and Alt-3 numerical modeling results. The access for the east breakwater spur construction would be provided over the landward toe berm of the east breakwater to avoid having to lower the crest of the structure.

Figure 8. Configuration of Alt-3.
1.5.5 Alternative 3a (West and East Breakwater Spurs)

Alt-3a essentially extends the dual spurs used in the Alt-3 by lengthening each spur (Figure 9). The lengths of west and east breakwater spurs have been increased to 460 ft and 425 ft, respectively, to further minimize the amount of wave energy and sediment getting into the entrance channel. The extended spurs reach beyond the 11 ft depth contour to provide additional sheltering from incident-wave energy and to intercept and divert sediments from the channel.

Figure 9. Configuration of Alt-3a.
1.5.6  Alternative 4 (West and East Breakwater Spurs)

Alternative 4 (Figure 10), similar to Alt-3, has dual parallel spurs, each 200 ft long that are oriented straight southward (i.e., spurs are not aligned to the channel edges). The west breakwater spur would shelter the harbor from wave energy from the southwest. The east breakwater spur would have the dual purpose of providing wave sheltering from the southeast waves and reduced shoaling in the entrance channel, but extends into half of the existing entrance channel footprint. The entrance channel would have to be realigned since decreased width of the entrance channel would pose a safety hazard to vessels entering and departing the harbor.

Figure 10. Configuration of Alt-4.
1.5.7 Alternative 5 (Return to Previous Harbor Configuration)

Based on discussions with harbor users, reinstating the harbor back to its configuration prior to implementation of the federal modifications would be preferable to the current harbor. Under Alt-5, the exterior east breakwater spur would be reconstructed along the same alignment that was in the pre-federal modifications configuration. The spur would be approximately 165 ft in length and extend into the existing federal channel (Figure 11). Once implemented, it is likely that the sediment transport that limited entrance channel depths and negatively impacted the pre-federal modifications configuration would be reestablished at the harbor.

Figure 11. Configuration of Alt-5.
1.5.8 **Alternative 6 (Extension of the East Breakwater Inner Spur)**

Federal modifications to KLDH resulted in shortening of the east breakwater inner spur. Harbor users indicate that this has created a wider access channel area that allows more wave energy to enter the harbor basin. Alt-6 would result in the extension of the inner spur by approximately 50 ft (Figure 12) to potentially improve navigability within the basin, at the dock, and at the boat ramp. This alternative would also require a slight realignment of the federal-access channel to the north to compensate for the overlap of the spur extension and the footprint of the existing federal-access channel.

*Figure 12. Configuration of Alt-6.*
1.5.9 Alternative 7 (Detached Offshore Breakwater)

Alt-7 consists of a detached offshore breakwater (approximately 600 feet in length) designed to shelter the existing entrance channel from incident wave energy. The breakwater would be aligned east to west, similar to the existing east breakwater (Figure 13). It would be located a short distance offshore of the seaward limit of the entrance channel. Not only would the offshore breakwater provide significant sheltering from incident-wave energy, it would also result in deposition of sediment seaward of the east breakwater and updrift of the existing entrance channel. The offshore breakwater would be constructed to an elevation that would afford outgoing vessel captains a view of incident-wave conditions prior to transit across the shallow reef to deeper water.

Figure 13. Configuration of Alt-7.
1.6 Study plan

1.6.1 Purpose

This study was initiated to investigate post-construction harbor performance, quantify wave conditions at and adjacent to the harbor, and evaluate the merits of proposed alternatives for improving navigation conditions inside the harbor. The tasks completed in support of field data collection and numerical modeling were: (1) conduct hydrographic survey of the harbor interior, entrance, breakwaters, reefs, and vicinity to 40 ft depth contour to provide bathymetric data for numerical modeling; (2) collect water levels, wave and current data at five gauge locations for numerical modeling; (3) evaluate proposed alternatives for improving conditions in the harbor; (4) compare previously conducted HARBD numerical modeling results with Boussinesq wave modeling; and (4) perform Boussinesq wave modeling to investigate time-and spatially-varying nature of waves approaching the entrance and interior of KLDH. The issues investigated in the course of the study include: a) the relationship between offshore wave conditions and harbor utilization; b) quantification of waves at the entrance and interior of harbor by field measurements and numerical modeling; and c) evaluation of proposed alternatives to improve the present harbor configuration. The operational conditions of the harbor after the federal-improvement project in 2009 are investigated by using state-of-the-art numerical modeling tools, which are also used to evaluate alternatives that can improve navigability and usage of the harbor.

1.6.2 Motivation

KLDH is located within the coastal ocean wave environment and is affected by interaction of locally generated sea, swell, and distant storms. After completion of federally funded modifications of the breakwaters, channels, and interior of harbor, feedback from the harbor users indicated that the harbor is not functioning as predicted by the numerical modeling used in the feasibility phase of this project. Complaints from the harbor users include increased wave energy within the harbor basin, shoaling at the boat ramp, and increased erosion of the shoreline west of the harbor.

This report provides details of a comprehensive assessment study for understanding how waves affect navigation at KLDH. The nature of waves reaching the entrance of KLDH is strongly affected by the coastal reef present outside the harbor, and also by the characteristics of the shoreline on
both sides of harbor. The alternatives POH identified for general improvement of navigation in the harbor are evaluated in this study by considering these effects. The pros and cons of proposed structural modifications to the exterior and interior of harbor for improving navigation in the entrance and access channels and within the harbor are described. The field data collected and numerical wave-modeling findings provided in this report are useful to future works at KLDH for potential modification of breakwaters, development of improved navigation aids, channel re-alignment or widening and deepening, and analysis of sediment-transport pathways.

1.6.3 Approach

Waves are the major forcing and primary concern at KLDH. Increased wave energy along the dock and boat ramp appears to be the main issue to users of the harbor. Excessive wave energy may also be causing more sediment shoaling in the entrance channel and interior harbor areas. Users of KLDH have reported higher waves inside the harbor along the northeast (NE) and east (E) side of interior basin, following implementation of the federal improvements in 2009. Possible causes of these problems are investigated in this wave-modeling study by evaluating different alternatives that can eliminate or reduce these problems.

An important characteristic of KLDH is a wide area of reef that covers the east, west, and south sides of the harbor (Figure 14). The reef extends offshore to approximately the 60 ft (18 m) depth contour; the cross-shore width of the reef varies east to west, with average width greater than 1,500 ft (457 m). The reef extends approximately 1 mile (1.6 km) to the east and 0.5 mile (0.8 km) to the west of the entrance channel. It covers nearly the entire south side of the harbor, serving as a natural protector to the harbor from incident waves. As waves reach the vicinity of the harbor and propagate over the reef, they break and generate strong currents. The wave-and reef-interaction processes affect the wave energy, navigation, resulting currents, and sedimentation occurring at KLDH. These complex effects interact to negatively impact navigation at KLDH.
1.6.4 Details of study plan

The purposes and motivation of the present study require a comprehensive analysis of the bathymetric change, waves, currents, and their interaction that affect navigation at KLDH. Resolution of wave problems in KLDH requires a systematic investigation of the roles of entrance configurations with different structural modifications. Eight alternatives were proposed by POH in relation to existing harbor configuration for investigation of the relative merits of each alternative. Field data-collection was necessary to quantify the existing navigation difficulties experienced inside and outside the harbor, and for evaluating potential usefulness of each alternative.

The existence of a hard bottom reef outside the KLDH affected the type of alternatives considered. For example, the reef system may limit the amount of dredging to reduce channel shoaling rates, channel configurations most favorable for safe navigation, as well as alignment of entrance and access channels. The Boussinesq wave modeling was used to identify pros and cons of each alternative and examine potential impact of modifications on key areas of interest. Changes that can occur inside the harbor due to modifications being introduced include adverse effects such as increased wave energy and shoaling/sedimentation in the channels and harbor basin. The Coastal Modeling System wave-transformation model
CMS-Wave was used to transform offshore wave information to nearshore to provide input waves to the Boussinesq model.

The wave-modeling study started by developing estimates of incident-wave conditions using the most recent data from offshore buoys and hindcast to check the previous modeling results used to design the federal modifications to the harbor. These updated wave conditions were used in the evaluation of the merits of each alternative relative to the existing harbor configuration. Findings from this study enable POH to identify alternatives with the greatest potential to improve navigation and increase utilization of the harbor. Implementation of the modeling approach and associated tasks are described later in this section. CHL and POH worked closely on defining wave modeling tasks, and assembled a comprehensive set of bathymetric data, shoreline and harbor boundary characteristics, model grids, boundary conditions, input conditions, output stations, and types of engineering estimates desired by the POH project team.

Eight structural modification alternatives were proposed and evaluated for incident waves from three directions (SE, S, and SW) with two water levels (low and high water). The incident waves used in the evaluation of alternatives had a spectral peak period of 16 sec, significant wave height of 2.4 m, and water levels of 0 m and 0.6 m (referenced to MLLW). For consistency with the previous numerical modeling study (Thompson et al., 1998), CMS-Wave and Boussinesq model simulations were performed with the same set of selected incident wave conditions, water levels, and model output locations.

There is an important difference between this modeling study and the previous numerical modeling study by Thompson et al. (1998). The wave-model grids in the present study were extended to the offshore edge of the reef, which was not possible to do in the earlier study because of HARBD model computational requirements. Model waves in the present study were propagated over reef into the entrance and access channels and the inner harbor. This was necessary in order to account for the effects of complex bathymetry and structures present in the study area. Important wave processes such as wave shoaling, refraction, diffraction, reflection, and wave breaking were represented in simulations. The fully nonlinear capabilities of BOUSS-2D (B2D) model results showed areas that are affected by different wave processes. This information is necessary to objec-
tively evaluate proposed project alternatives for solving potential problems. A summary of study tasks follows.

### 1.6.5 Tasks

A summary description of numerical modeling study tasks follows.

**Task 1. Grids and boundary conditions.** Figures 1-15 and 1-16 show the B2D model grids for the pre-federal project and existing harbor (post-federal project), respectively. Aside from dredging of the harbor, key differences between the two harbor configurations are the structural modifications to the east and west breakwaters, which changed the harbor entrance.

![Figure 15. BOUSS-2D model grid for pre-federal project.](image)
For all eight alternatives investigated, incident waves were generated near the southern grid boundary by a wavemaker (shown by a straight E-W green line in deep water section in Figures 1-15 and 1-16). The north boundary of the grid defined the land side and extended southward to include the west and east breakwaters. These define the outside boundaries of harbor, including a short interior groin that reduces wave transmission into the harbor basin.

POH provided topographic survey data that included the breakwaters and land surrounding KLDH’s interior, entrance, and east and west shorelines. These data files were combined with bathymetry survey from the field monitoring and data-collection phase of the study. The combined data were used to generate the CMS-Wave and B2D model grids.

The miscellaneous photographs and videos from field surveying provided useful guidance about the reflectivity of structures and land boundaries of interest in the B2D wave modeling domain, and areas where reflection may be important. The extents of modeling grid domain were determined by ensuring that the locations where waves start to refract, shoal, and break, along with the location of the offshore edge of the reef were adequately represented. Preliminary B2D test simulations were performed to ensure that the model was setup and working properly. The model pa-
Parameters were determined for each alternative’s production runs based upon the calibration with field data described in Chapter 4. Incident waves were generated based on measured and wave hindcast data including significant wave height, peak period, and mean wave direction.

**Task 2. Test runs/model calibration.** A set of test runs were made for each alternative and discussed with the POH project delivery team (PDT). Model results were interpreted based on the experience of modelers, and model parameters were adjusted as necessary. Model estimates were calibrated against collected wave data as soon as this data became available.

**Task 3. Progress meetings.** Periodic discussions were held with POH before the start of production runs on information pertinent to model setup, assumptions, model limitations, planned model runs, and results of the test runs; appropriate adjustments were made.

**Task 4. Production runs.** Due to the relatively large modeling domain size and a very large number of simulations required, production runs were performed concurrently on multiple desktop machines and supercomputers.

**Task 5. Post processing and analysis.** Model results were post-processed and presented in tables, figures, snapshots, and animation files. Model results were output at the same stations (save points) selected by POH which were used in the HARBD numerical model study (called basins). Appropriate two-dimensional (2D) and three-dimensional (3D) spatial and temporal output of B2D model results were extracted for post-analysis of model solutions in time- and-frequency domain. See Chapter 4 for additional information.

**Task 6. Draft report and review.** Study results were documented in a draft letter report. Comments from the review were incorporated into this final ERDC Technical Report (TR). Hard copies and digital format of the final report will be provided to POH together with all other derivative products generated from the numerical modeling study.
2 Data

2.1 Bathymetry and coastline data

Coastline digital data for this study were extracted from the National Geophysical Data Center (NGDC, http://ngdc.noaa.gov), and a geo-referenced image file downloaded from Google Earth 5.0 (http://earth.google.com).

The bathymetry data used in the present study came from various sources to cover the harbor, land, nearshore, and offshore area. The harbor area including the interior, exterior, jetties, breakwaters, and near fields is based on surveys conducted by CHL in 2012. The details of hydrographic field surveys, survey equipment, and operations are presented in Appendix A. The nearshore and coastal zones are based on SHOALS (LiDAR) data obtained in 2000 by the USACE Joint Airborne Lidar Bathymetry Technical Center of Expertise. The land-elevation data were downloaded from USGS Geographical Digital Elevation models (DEM, http://edc2.usgs.gov/geodata/index.php). The offshore bathymetry is based on the GEOphysical DAta System (GEODAS), developed and managed by the NGDC (http://www.ngdc.noaa.gov/mgg/bathymetry/relief.html). All datasets were converted to the local mean lower low water datum.

2.2 Water levels

Water-level data near the study site are available from two NOAA Coastal Stations: Port Allen (1611347) in Hanapepe Bay from 1989 to 1997, and Nawiliwili (1611400) from 1954 to present time. Table 1 lists measured mean and diurnal tidal ranges at these two stations. The Hawaii area has a mixed semidiurnal tidal cycle with two high and two low tides of different sizes every day. Figure 17 shows the time history of measured water levels at Port Allen and Nawiliwili gages in 1996.

<table>
<thead>
<tr>
<th>Station</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Mean Tidal Range</th>
<th>Diurnal Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>1611347</td>
<td>21° 54.2’ N</td>
<td>159° 35.5’ W</td>
<td>0.38</td>
<td>0.56</td>
</tr>
<tr>
<td>1611400</td>
<td>21° 57.2’ N</td>
<td>159° 21.3’ W</td>
<td>0.37</td>
<td>0.56</td>
</tr>
</tbody>
</table>
2.3 Wind and wave data

Coastal wind data are available at Port Allen station (1611347) from 1989 to 1997 and at Nawiliwili Station (1611347) from 2009 to the present time. Offshore wind data are available from four National Data Buoy Center (NDBC) Buoys 51001, 51002, 51003, and 51101. Buoys 51001 and 51101 are located approximately 170 and 190 nm, respectively, northwest of Kauai. Buoys 51002 and 51003 are located approximately 200 nm southwest of Hawaii and Oahu Islands, respectively. These NDBC buoys, and two Coastal Data Information Program (CDIP) Buoys 106 and 165, also collect continuous wave information. CDIP Buys 106 and 165 are located near the northwest and southwest coastline of Oahu, respectively. Table 2 lists buoy locations and information. Figure 18 shows the location map of NDBC and CDIP buoys. NDBC 51001, 51002, and 51003 are non-directional wave buoys while NDBC 51101 and the two CDIP buoys are directional wave buoys.
Figure 17. Measured water levels at Port Allen and Nawiliwili for 1996.

Table 2. List of NOAA and CDIP Buoy Information.

<table>
<thead>
<tr>
<th>Station</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Nominal Depth (m)</th>
<th>Data Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>NDBC 51101*</td>
<td>24° 19.3’ N</td>
<td>162° 3.5’ W</td>
<td>4791</td>
<td>2009-2013</td>
</tr>
<tr>
<td>NDBC 51001</td>
<td>23° 26.7’ N</td>
<td>162° 16.7’ W</td>
<td>3430</td>
<td>1981-2009</td>
</tr>
<tr>
<td>NDBC 51002</td>
<td>17° 5.65’ N</td>
<td>157° 48.4’ W</td>
<td>5002</td>
<td>1984-2013</td>
</tr>
<tr>
<td>NDBC</td>
<td>19° 1.1’ N</td>
<td>160° 34.9’ W</td>
<td>4919</td>
<td>1984-2013</td>
</tr>
<tr>
<td>Station</td>
<td>Latitude</td>
<td>Longitude</td>
<td>Nominal Depth (m)</td>
<td>Data Period</td>
</tr>
<tr>
<td>---------</td>
<td>--------------</td>
<td>--------------</td>
<td>-------------------</td>
<td>-------------</td>
</tr>
<tr>
<td>51003</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CDIP 106*</td>
<td>21° 40.3’ N</td>
<td>158° 7.03’ W</td>
<td>200</td>
<td>2001-2013</td>
</tr>
<tr>
<td>CDIP 165*</td>
<td>21° 16.8’ N</td>
<td>158° 7.4’ W</td>
<td>302</td>
<td>2010-2013</td>
</tr>
</tbody>
</table>

* directional wave data available

**Figure 18. Location of NDBC and CDIP buoys.**

The Hawaii region is dominated by trade winds most of the year. Major storms primarily occur during the winter season, generated by cold fronts or low pressure systems. True tropical storms or hurricanes may occur but they are rare. Figure 19 shows the time history of measured wind and waves at NDBC 51002, 51003, and 51101 in 2012.

Figure 20 shows the time history of wave measurements from CDIP 106 and 165 for 2012. Both CDIP buoys show smaller wave heights in the nearshore region than the NDBC buoys in the offshore area as a result of wave energy dissipation over the coastal hard bottom. Waves observed at CDIP 106 (north shore) generally have larger height and smaller wave pe-
riods than waves at CDIP 165 (southwest shore). The difference in wave height at the two CDIP buoys is greater in the winter but lesser in the summer. On the contrary, the difference in wave period at the two CDIP buoys is greater in the summer but lesser in the winter. This indicates the seasonal nature of wave characteristics in the Hawaiian Islands.

Throughout the year the islands receive swell, peaking in the primary winter season (October to March) with powerful northwest swell. The seasonal cycle of north Pacific swells reaches a peak in winter with a daily average significant wave height around 4 m. Southerly and easterly swells dominate during the summer months (May to September). Southern swell is usually generated from storms south of the equator near Australia, New Zealand, and as far as the Southern Ocean. Southern swells reach Hawaii with the significant wave height of 2.5 to 3 m and peak period of 14 to 22 sec. Consistent trade winds, predominantly from the northeast, generate the trade wind sea with average wave heights of 2 m and peak periods of 9 sec. Figure 21 shows wind roses at NOAA Coastal Station 1611400 (Nawiliwili) and three NDBC buoys 51002, 51003, and 51101 for 2012. These wind roses indicate the heavy dominance of trade winds in the Hawaii region. Figure 22 shows wave roses at NDBC 51101, and two CDIP Buoys 106 and 165, for 2012. These wave roses show offshore waves are dominated by northwest and easterly waves while nearshore waves are influenced by offshore wave conditions as well as wave refraction and dissipation over the coastal reef bottom.
Figure 19. Time history of measured wind and waves at NDBC Buoys 51001, 51002, 51003, and 51101 for 2012.
Wave data in the nearshore and in the interior of Kikiaola Harbor for the present study were collected by the CHL as part of the field data collection effort, which was conducted between 12 October 2012 and 27 February 2013. In this field-gauging study, two AWAC (Acoustic Wave And Current)-type gauges were deployed at stations K5 and K3, as shown in Figure 23. Three Aquadopp (Acoustic Doppler current profiler or current meter, http://www.nortekusa.com) instruments were deployed at Stations K1, K2, and K4. Figure 23 shows the location map for five stations (or gauges) K1, K2, K3, K4, and K5. Table 3 lists the coordinates of gauge locations and data collection information. Stations K1 and K4 were inside the harbor; K2 was placed slightly to the north of the access channel; K3 and K5 were located outside the harbor.

Gauge K2 was unable to be recovered during initial instrument retrieval, but was recovered later in 2013, after the numerical modeling study was completed. Apparently, the gauge was buried under the sand and mud, stopped recording data shortly after it was deployed, and the stored signal
is extremely noisy. Data analysis by the field monitoring team is still in progress, and attempts to analyze data have so far not been successful. No useful data could be obtained from Station K2 for numerical models to use, and consequently, K2 will not be considered in the remainder of this report. More details of nearshore data collection and analysis are provided in Appendix B and in Part 1 of this study (ERDC/CHL TR-14-8).

Figure 21. Wind roses at NOAA Station 161140 and NDBC Buoys 51002, 51003, and 51101 for 2012.
Figure 22. Wave roses at the NDBC 51101 and CDIP 106 and 165 for 2012.
Figure 23. Nearshore and harbor wave data collection stations.

Table 3. List of field gauges K1 to K5.

<table>
<thead>
<tr>
<th>Gauge</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Nominal Depth (m)</th>
<th>Data Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1 (ADCP)</td>
<td>21° 57.56’ N</td>
<td>159° 41.53’ W</td>
<td>0.9</td>
<td>12 October 2012 – 4 January 2013</td>
</tr>
<tr>
<td>K2 (ADCP)</td>
<td>21° 57.52’ N</td>
<td>159° 41.59’ W</td>
<td>2.7</td>
<td></td>
</tr>
<tr>
<td>K3 (AWAC)</td>
<td>21° 57.41’ N</td>
<td>159° 41.57’ W</td>
<td>5.2</td>
<td>12 October 2012 – 27 February 2013</td>
</tr>
<tr>
<td>K4 (ADCP)</td>
<td>21° 57.53’ N</td>
<td>159° 41.54’ W</td>
<td>1.8</td>
<td>12 October 2012 – 4 January 2013</td>
</tr>
<tr>
<td>K5 (AWAC)</td>
<td>21° 57.14’ N</td>
<td>159° 41.49’ W</td>
<td>9.6</td>
<td>12 October 2012 – 27 February 2013</td>
</tr>
</tbody>
</table>
Figure 24 shows time series of measured wave height, peak period, and spectral peak mean direction at K1, K2, K3, K4, and K5 from 12 October 2012 to 27 February 2013. Outside the harbor at K3 and K5, wave heights are in the range of 0.5 to 1.7 m during the wave data collection period. The majority of waves during the data collection period approached the harbor from the south direction. Wave periods are in the range of 8 to 21 sec. Inside the harbor at K1 and K4, wave heights are much smaller, with the maximum wave height less than 0.3 m. The wave period data show more long waves (12 to 25 sec) inside the harbor. The wave direction data show a wide range, likely a result of combined diffraction and reflection of small, long waves inside the harbor. The average wave directions at K1 and K4 correspond to southeast and southwest, respectively.

Figure 25 shows the wave roses at K3 and K5 for the period of 12 October 2012 to 27 February 2013. Waves travelling from K5 (offshore edge of reef) to K3 (nearshore reef) during the data collection period changed wave direction from SSW to SSE due to refraction and dissipating wave energy over the hard bottom.

In addition to the wind and wave measurements, there are wave hindcast databases from the USACE Wave Information Studies (WIS) and NOAA’s numerical model WaveWatch III (WW3) for the Hawaii region. WIS data are available for the period of 1980 to 2011. Figure 26 shows the location map for WIS Stations covering the Kauai and Niihau area. Figures 27 and 28 show the wind and wave roses, respectively, at WIS Stations 82500, 82559, and 82562 for 2011. Table 4 lists the WIS station location information and types of data available at these stations. The WIS wind roses show clearly the dominance of trade winds around the Kauai and Niihau Islands. The wave roses, on the other hand, are the result of combined northeast, southern, and trade wind waves, as well as the sheltering effect of islands.
Figure 24. Wave data collected at gauges K1, K2, K3, K4, and K5 from 12 October 2012 to 27 February 2013.
Figure 25. Wave roses at gauges K3 and K5 for the period of 12 October 2012 to 27 February 2013.
Figure 26. WIS Stations in the Kauai and Niihau area.
Figure 27. Wind roses at WIS Stations 82559, 82562, and 82500 for 2011.
Figure 28. Wave roses at WIS Stations 82559, 82562, and 82500 for 2011.

Table 4. List of WIS Stations 82559, 82562, and 82565 information.

<table>
<thead>
<tr>
<th>WIS Station</th>
<th>Latitude</th>
<th>Longitude</th>
<th>Nominal Depth (m)</th>
<th>Data Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>82559</td>
<td>21° 30' N</td>
<td>159° 45' W</td>
<td>3420</td>
<td>1980-2011</td>
</tr>
<tr>
<td>82562</td>
<td>21° 39' N</td>
<td>160° 30' W</td>
<td>820</td>
<td>1980-2011</td>
</tr>
<tr>
<td>82500</td>
<td>22° 24’ N</td>
<td>160° 03’ W</td>
<td>4512</td>
<td>1980-2011</td>
</tr>
</tbody>
</table>

2.4 Current data

The AWAC instruments used for wave measurements at Kikiaola Harbor also collected current data. The accuracy of velocity magnitude and direction measurements is within ±0.5 cm/sec and ±5° relative to true north.
Figure 29 shows the time series of depth-averaged currents at AWAC Station K1, K2, K3, K4, and K5 during the data collection period from 12 October 2012 to 27 February 2013. The majority of currents outside the harbor at K3 and K5 are westward and stronger than currents inside the harbor at K1 and K4 locations. The current outside the harbor is mainly driven by the strong easterly wind along the coastline. The measured currents at all four AWAC stations are overall small, less than 40 cm/sec, except for the data collected at K3 in February 2013, likely caused by an instrument error that occurred in this period. The current direction at K4 mainly follows the tidal flow in and out of the inner basin. The current at K1 is the net northwest flow in the back of the inner basin. Figure 30 shows the corresponding current roses for the depth-averaged current at AWAC Station K2, K3, K4, and K5.

Figure 29. Time series of current magnitude and direction at gauges K1, K3, K4, and K5 for the data collection period of 12 October 2012 to 27 February 2013.
Figure 30. Current roses at gauges K3, K4, and K5 for the data collection period of 12 October 2012 to 27 February 2013.
CMS-Wave Modeling

Wave modeling for KLDH was conducted using two types of numerical models: BOUSS-2D (B2D) and CMS-Wave. B2D is a Boussinesq type two-dimensional (2D) wave model (Demirbilek et al. 2007b; Demirbilek et al. 2005a, 2005b; Nwogu and Demirbilek 2001). It is used in this study to investigate alternatives representing different proposed structure changes to the inside and outside of the harbor. Because B2D is a fully nonlinear time-domain model able to represent linear and nonlinear nearshore wave processes, it is a computationally resource-demanding model. B2D is used in the present study over a small area covering details of the harbor, structures, and its immediate vicinity, including reefs and shorelines. The areas covered in the B2D modeling domain consist of a nearshore reef fronting the harbor entrance that extends to approximately 60 ft (18 m) depth offshore, rocky shorelines on both sides of the harbor, entrance and access channels, and structures surrounding the perimeter of harbor.

Because large domain modeling around KLDH was not possible with BOUSS-2D for a wide range of wave conditions, it was necessary to augment BOUSS-2D modeling with a spectral wave model capable of providing estimates of waves over much larger domains and for a large number of wave conditions. The CMS-Wave was selected because it is a steady-state 2D spectral wave model (Lin and Demirbilek 2012; Lin et al. 2011; Demirbilek and Rosati 2011; Lin et al. 2008; Demirbilek et al. 2008) for simulating wave processes with ambient currents at navigation channels, coastal inlets, and harbors. Because of complementary features of B2D and CMS-Wave, these models are frequently used in tandem in similar coastal studies.

In the present study, CMS-Wave was used to transform offshore wave information provided by deep water coastal buoys or the wave hindcast databases to the project site at the seaward boundary of BOUSS-2D. CMS-Wave was also used to check the reliability of field gauges deployed to obtain nearshore wave data for BOUSS-2D modeling. CMS-Wave is part of an integrated Coastal Modeling System (CMS) developed at CHL for coastal inlets and regional sediment modeling project applications. A brief description of CMS is presented in Appendix A.
CMS-Wave can be used in half-plane or full-plane mode for wave transformation in deep or shallow water. The half-plane mode is the default because CMS-Wave can run more efficiently in this mode as waves are transformed primarily from the seaward boundary toward shore. The model is based on the wave-action balance equation that includes wind-wave generation and growth, wave propagation, refraction, shoaling, diffraction, reflection, breaking, and dissipation. The computational efficiency of the model and recent improvements to capabilities of the model allow the simulation of large domains and a large number of wave conditions in coastal engineering applications. The nested-grid capability of the model is used here to ensure necessary grid resolution for representing fine details of the harbor geometry.

CMS-Wave model was used in this study to develop incident wave-input conditions for B2D model. Several improvements to CMS-Wave were necessary to provide these inputs as well as to address the project’s other needs. Additional R&D developments to enhance the model’s predictive capabilities were funded by the CIRP. The advances included a) simulations for tropical storms and non-storm waves in full-plane mode with parent-child grid capability of CMS-Wave; b) modeling of a dual-peaked wave condition (combined seas and swells from different directions); c) developing two-dimensional wave spectra at the offshore boundary of B2D; d) validation of model with the NOAA buoys and CDIP gauges near the project site; e) development of pre- and post-processing analysis codes for improving model setup; f) providing wave parameters (height, period, direction) and wave spectra as B2D input conditions; and g) developing a number of Fortran and Matlab utilities to facilitate coupling of two wave models.

This chapter describes the CMS-Wave modeling and Chapter 4 provides details of BOUSS-2D (or B2D for short) modeling.

3.1 Model domain

Two domains were used in CMS-Wave modeling to address special needs of the KLDH project: (1) a large regional coastal domain, and (2) a relatively small local domain covering the harbor and vicinity. The regional domain, a large rectangular area approximately 4.3 km x 8.3 km, covers the continental shelf along the southeast coast of Kauai Island. The small domain, approximately 1.2 km x 2.6 km, covers the local coastal area that extends seaward from the harbor to field gauge K5 located on the offshore
edge of reef near the 10 m depth contour. The regional domain is primarily used in the transformation of offshore wave information to the nearshore to provide the wave boundary condition for B2D. The smaller local domain is used in the verification of CMS-Wave based on the data collected from field gauges. Figure 31 shows the CMS-Wave regional and local model grid domains.

Figure 31. CMS-Wave regional coastal and local harbor model domain.

3.2 Model verification

The CMS-Wave verification was performed for the regional and local grids (Figure 31). For the regional grid, the incident wave condition was based on directional spectra measured from CDIP 165. Wave model results were compared to data at Gauges K1, K3, K4, and K5. For the local grid, the wave data obtained from Gauge K5 served as the incident wave condition. CMS-Wave was run for 13 October 2012 to 28 February 2013. Water levels measured from NOAA Station Nawiliwili (1611400) were included in the wave model simulation.

For October-December 2012, the calculated and measured waves at K1, K3, K4, and K5 locations are shown in Figures 32 to 35. Figures 36 and 37 show the calculated and measured waves at K3 and K5 locations for Janu-
ary-February 2013. Three statistical parameters are used in the evaluation of model results: (1) the mean of the absolute error of model and measured wave height; (2) the mean of the absolute error of model and measured wave period; and (3) the mean of the absolute difference of model and measured-wave direction. For the regional grid, Table 5 shows the statistics for model and measured wave comparison (October 2012-February 2013) at K1, K3, K4, and K5. Table 6 shows the statistics at K1, K3, and K4 for the local grid. The model-data comparison at K3 indicated that calculated wave heights agree better with data in the local grid than the regional grid. This is because wave spectra measured at K5 were used as incident wave conditions for the local grid, while the regional grid used the wave data obtained from the nearest coastal buoy CDIP165. Consequently, model results agree better with data at K1 and K4 in the local grid than the regional grid. To obtain realistic estimates of waves inside the harbor, these results show that it is necessary to provide accurate incident waves as input boundary conditions to both regional and local grids.

Figure 32. Model-data comparison at K1 (October-December 2012).
Figure 33. Model-data comparison at K3 (October-December 2012).
Figure 34. Model-data comparison at K4 (October-December 2012).
Figure 35. Model-data comparison at K5 (October-December 2012).
Figure 36. Model-data comparison at K3 (January-February 2013).
Figure 37. Model-data comparison at K5 (January-February 2013).

Table 5. Statistics of model-data comparison for the regional grid.

<table>
<thead>
<tr>
<th>Location</th>
<th>Mean of absolute relative wave height error (%)</th>
<th>Mean of absolute relative wave period error (%)</th>
<th>Mean of absolute direction error (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1</td>
<td>37.7</td>
<td>37.0</td>
<td>114</td>
</tr>
<tr>
<td>K3</td>
<td>30.6</td>
<td>30.4</td>
<td>14</td>
</tr>
<tr>
<td>K4</td>
<td>40.8</td>
<td>31.0</td>
<td>5</td>
</tr>
<tr>
<td>K5</td>
<td>31.0</td>
<td>24.2</td>
<td>11</td>
</tr>
</tbody>
</table>

Table 6. Statistics of model-data comparison for the local grid.

<table>
<thead>
<tr>
<th>Location</th>
<th>Mean of absolute relative wave height error (%)</th>
<th>Mean of absolute relative wave period error (%)</th>
<th>Mean of absolute direction error (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>K1</td>
<td>19.0</td>
<td>32.8</td>
<td>114</td>
</tr>
<tr>
<td>K3</td>
<td>9.3</td>
<td>19.6</td>
<td>4</td>
</tr>
<tr>
<td>K4</td>
<td>20.9</td>
<td>24.6</td>
<td>5</td>
</tr>
</tbody>
</table>
3.3 Simulations of average wave conditions

The wave amplification factors, defined as wave height normalized by incident wave height (Thompson et al. 1998), were calculated at selected locations for a range of average wave conditions simulated using CMS-Wave. Table 7 presents the incident wave periods (four peak wave periods, $T_p = 8, 12, 16, 20$ sec) and approach directions (three mean wave directions, $\theta_m = 160, 180, 200$ deg, meteorological, coming from) used in the present study. The water level was selected as 0.305 m (1 ft) above the MLLW.

The model output was saved at 15 locations used by Thompson et al. (1998) along the navigation channel from the harbor entrance to the dock at the back of the harbor basin (Figure 38). The coordinates of those 15 output locations are provided in Table 8. The CMS-Wave output at these locations was averaged over periods of 8-20 sec and compared to the results from the previous study (Thompson et al. 1998), which used a numerical wave model called HARBD. The HARBD model was a steady-state hybrid element model for linear wave response in harbors of varying size and depth (Chen, 1986; Chen and Houston, 1987; Lillycrop and Thompson, 1996). The HARBD results from the previous study (Thompson et al. 1998) were based on the harbor configuration and bathymetry in 1996, and used a much smaller model domain (Figure 39) as compared to the CMS-Wave model domain (Figure 31) used in the present study. The water depth offshore at the seaward boundary of HARBD was approximately 4 m (13 ft).

The harbor configuration in 1996 included an exterior spur which extended seaward from the east breakwater alongside the entrance channel to prevent sediment transport into the harbor. The spur also provided additional protection to the harbor entrance from large wave attack. The bottom friction was set to zero in the previous study (Thompson et al. 1998).

<table>
<thead>
<tr>
<th>Table 7. Summary of incident wave conditions simulated with CMS-Wave.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave Period, $T_p$ (sec)</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>8, 12, 16, 20</td>
</tr>
</tbody>
</table>
Figure 38. Fifteen save locations for CMS-Wave model output.
Figure 39. HARBD model domain, save locations, and harbor configuration in 1998.

Table 8. Field gauge coordinates in Geographic and Hawaii State Plane Zone 4.

<table>
<thead>
<tr>
<th>NAME</th>
<th>E</th>
<th>N</th>
<th>LAT</th>
<th>LONG</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>480152.964</td>
<td>13958.9256</td>
<td>21.95928647</td>
<td>-159.6921381</td>
</tr>
<tr>
<td>2</td>
<td>480039.9472</td>
<td>13884.27094</td>
<td>21.95861098</td>
<td>-159.6932313</td>
</tr>
<tr>
<td>3</td>
<td>480082.8478</td>
<td>13694.00868</td>
<td>21.95689319</td>
<td>-159.6928137</td>
</tr>
<tr>
<td>4</td>
<td>480129.7565</td>
<td>13906.07023</td>
<td>21.95880887</td>
<td>-159.6923621</td>
</tr>
<tr>
<td>5</td>
<td>480214.3629</td>
<td>13191.39348</td>
<td>21.95235554</td>
<td>-159.6915343</td>
</tr>
</tbody>
</table>

The wave-amplification factors calculated by HARBD model at 15 output locations (Figure 39) are shown in Table 9 and Figure 40. In principle, the CMS-Wave results could be extracted at these output stations, but a comparison of the two models is not provided because the fundamentals of two models are different (i.e., different modeling domains, simulations conducted for a single frequency monochromatic wave in HARBD vs. multiple frequencies using a spectrum in CMS-Wave, and differences between the
two models’ representation of wave breaking, reflection and diffraction, etc). For completeness and future reference, HARBD results are shown in Table 9, including mean and standard deviation of wave amplification factors at 15 output locations for three mean wave directions.

Table 9. Wave-amplification factors from HARBD model (Thompson et al. 1998).

<table>
<thead>
<tr>
<th>Station</th>
<th>Incident Wave Angle (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>160</td>
</tr>
<tr>
<td>1</td>
<td>0.08</td>
</tr>
<tr>
<td>2</td>
<td>0.09</td>
</tr>
<tr>
<td>3</td>
<td>0.10</td>
</tr>
<tr>
<td>4</td>
<td>0.06</td>
</tr>
<tr>
<td>5</td>
<td>0.07</td>
</tr>
<tr>
<td>6</td>
<td>0.11</td>
</tr>
<tr>
<td>7</td>
<td>0.15</td>
</tr>
<tr>
<td>8</td>
<td>0.06</td>
</tr>
<tr>
<td>9</td>
<td>0.11</td>
</tr>
<tr>
<td>10</td>
<td>0.15</td>
</tr>
<tr>
<td>11</td>
<td>0.30</td>
</tr>
<tr>
<td>12</td>
<td>0.35</td>
</tr>
<tr>
<td>13</td>
<td>0.47</td>
</tr>
<tr>
<td>14</td>
<td>0.71</td>
</tr>
<tr>
<td>15</td>
<td>1.15</td>
</tr>
<tr>
<td>Mean</td>
<td>0.26</td>
</tr>
<tr>
<td>S.D.</td>
<td>0.31</td>
</tr>
</tbody>
</table>
3.4 Simulations of storm waves

Based on the WIS hindcast data of 1980-2011, one representative month was selected for high, median, and low storm waves at KLDH. This selection was determined based on the range of all wave conditions from WIS hindcast during summer months (June-August) and winter months (December-February). Data at the WIS Station 82560 (21.5 N, 160.05 W) were used for summer storm-wave month selection, and WIS 82563 (21.95 N, 160.5 W) was used for winter storm-month selection. Table 10 presents the list of selected months for the representative high, median, and low storm wave conditions. The representative high-and median-storm wave months were simulated.

Comparison of the WIS hindcast and NDBC buoy data for February 1981, June 1986, and June 1992 is shown in Figures 41 to 43. In general, WIS calculated wave periods agree with offshore buoy data. The hindcast wave heights at WIS 82560 and 82563 are smaller than offshore buoy measurements because both WIS stations situated close to Kauai are affected by sheltering from islands present in the area.
Table 10. Representative summer and winter storm-wave months.

<table>
<thead>
<tr>
<th>Season (WIS Station)</th>
<th>Wave Height Range</th>
<th>Year/Month</th>
<th>Monthly Mean Wave Height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Summer (WIS 82560)</td>
<td>High</td>
<td>1986/June*</td>
<td>1.69</td>
</tr>
<tr>
<td></td>
<td>Median</td>
<td>1992/June*</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>Low</td>
<td>2001/June</td>
<td>1.31</td>
</tr>
<tr>
<td>Winter (WIS 82563)</td>
<td>High</td>
<td>1983/February*</td>
<td>4.43</td>
</tr>
<tr>
<td></td>
<td>Median</td>
<td>1981/February*</td>
<td>3.50</td>
</tr>
<tr>
<td></td>
<td>Low</td>
<td>2000/January</td>
<td>2.51</td>
</tr>
</tbody>
</table>

* CMS-Wave simulations were conducted for high and median storm wave months.
Figure 41. Time series of NDBC 51001 and WIS 82563 data for February 1981.
Figure 42. Time series of data for June 1986 at NDBC buoys 51001, 51002, 51003, and WIS Station 85260.
CMS-Wave simulations were conducted for high- and median-storm wave months (Table 10) using the regional model domain grid shown in Figure 31. The WIS 82560 and 82563 hindcast waves were used as input to CMS-Wave. The model results including significant wave height, peak period, and mean wave direction were saved at a 1 hr interval for the entire model
domain and also at the 130 output locations shown in Figures 44 and 45. Points Pt 1 to Pt 130 are located in the harbor and surrounding areas. The first 15 output locations (Pts 1 to 15) are the same as those used in the comparison of local CMS-Wave grid and HARBD domain (Figures 38 and 39). Pt 35 is at the location of CHL Gauge K5, and Pts 1, 19, and 30 are at the locations of Gauges K1, K4, and K3, respectively.

Figure 46 shows three transects (T1, T2, and T3), where T1 connects output locations Pts 16 to 35, while T2 connects Pts 36 to 103, and T3 connects Pts 104 to 130. Figure 47 shows a snapshot of calculated storm wave field on 26 February 1981 at 15:00 GMT. Figure 48 shows a snapshot of calculated storm wave field on 10 February 1983 at 18:00 GMT. Figure 49 shows a snapshot of storm wave field on 17 June 1986 at 00:00 GMT. Figure 50 shows a snapshot of storm wave field on 14 June 1992 at 15:00 GMT.

Figure 51 shows the comparison of storm wave heights at model output location Pts 1 to 15 for four large storm-wave conditions. The time stamp corresponding to the storm-wave condition is an 8-digit number YYMMDDHH where YY, MM, DD, and HH are the year, month, date, and hour, respectively. The USACE criteria for berthing areas in shallow-draft small boat harbors is that significant wave height should not exceed 1 ft (0.305 m) more than 10% of the time. In the channels, significant wave height should not exceed 2 ft (0.61 m) more than 10% of the time. These thresholds (acceptability criteria for small harbor) are shown in the figure as solid black lines.

Figure 52 shows the comparison of storm waves along Transect T1 (Pts 16 to 35). Results indicate storm waves approaching the harbor break over the reef and rocky bottom outside the harbor. For large storm waves entering the harbor (landward of Pt 28), the wave height does not exceed the threshold due to depth-limited breaking (Figures 51 and 52).

Figures 53 and 54 show the comparison of storm waves along Transects T2 (Pts 36 to 103), and T3 (Pts 104 to 130). The wave-height distribution along T3 (between Pts 115 and 120) shows large storm waves converge towards the harbor by wave refraction and shoaling over the shallower reef outside the harbor entrance (Figure 45).
Figure 44. Model-output locations within and in the immediate vicinity of harbor area.

Figure 45. Model-output locations around the harbor.
Figure 46. Transects T1, T2, and T3 location map.

Figure 47. Calculated storm-wave field on 26 February 1981 at 15:00 GMT.
Figure 48. Calculated storm-wave field on 10 February 1983 at 18:00 GMT.

Figure 49. Calculated storm-wave field on 17 June 1986 at 00:00 GMT.
Figure 50. Calculated storm-wave field on 14 June 1992 at 15:00 GMT.

Figure 51. Comparison of storm waves at model output locations (Pts 1 to 15).
Figure 52. Comparison of storm waves along T1 (Pts 16 to 35).

Figure 53. Comparison of storm waves along T2 (Pts 36 to 103).
Figure 54. Comparison of storm waves along T3 (Pts 104 to 130).
4 Boussinesq Wave Modeling

As noted earlier in this report, two numerical wave models, CMS-Wave and BOUSS-2D, were used in this study. CMS-Wave was applied to a large modeling domain that included offshore deep water areas up to the shorelines. The computational efficiency of CMS-Wave allowed the simulation of the entire wave conditions for weeks, months, and years. BOUSS-2D (B2D), on the other hand, is a computationally demanding model because it is capable of modeling linear and nonlinear nearshore wave processes accurately. Consequently, B2D is appropriate for smaller domains and a limited number of wave conditions. This model is capable of handling short- and long-period waves by solving time-domain shallow-water nonlinear wave processes using advanced computational methods (Nwogu and Demirbilek 2008, 2006, 2004, 2001). These constraints limit the number of simulations possible, requiring a combination of spectral and Boussinesq type wave models to address a broad range of wave modeling needs in the present study (Nwogu 2006, 2007).

The wave modeling conducted for Kikialoa Light Draft Harbor (KLDH) included both offshore and nearshore areas. A wide, fringing reef exists along adjacent shorelines and the entrance channel which lead into the interior harbor. The use of B2D and CMS-Wave models was necessary to properly represent different wave processes in the KLDH area. Field data were used to calibrate and verify models. Details of CMS-Wave modeling were provided in Chapter 3. This chapter provides details of wave propagation with B2D model conducted for two water levels (MLLW and MHHW) for eight alternatives with different configurations. Wave diffraction, reflection, refraction, shoaling, breaking, and nonlinear wave-wave and wave-current interactions were represented in B2D simulations. The model provided estimates of wave-related parameters of interest (height, period, direction) affecting navigation in KLDH, including wave-induced currents (circulation), and infragravity wave effects (Nwogu and Demirbilek 2010; Nwogu 1994, 1996, 2000).

Three wave-related parameters of primary interest used for characterizing ocean waves are:

- significant wave height, \( H_s \) (m)
• spectral peak period, $T_p$ (sec)

• mean wave direction, $\bar{\theta}$ (deg)

As described in the Coastal Engineering Manual, CEM (Demirbilek and Vincent 2010) these parameters are widely used in coastal engineering practice. A steady-progressive wave train can be defined by the mean water depth $h$, the wave crest-to-trough height $H$, and the wavelength $L$, which is related to wave period. Shallow-water waves can be characterized by three dimensionless quantities, $H/h$, $L/h$, and $H/L$. In most cases, the wave period is known rather than the wave length. In a prototype environment, waves in coastal applications travel on a finite current, where the wave speed and hence the measured wave period, depend on the current (i.e., waves travel faster with the current than against it). The parameters $H/L$ and $H/h$, which measure the nonlinearity of waves, are used in Boussinesq wave theory to define nonlinear features of waves. Wave nonlinearity generally increases with the wave height as depth decreases, leading to an increase in the wave asymmetry (Nwogu 2006, 2007). BOUSS-2D model is designed to represent these and other nonlinear shallow-water wave processes and wave characteristics which strongly influence wave estimates in the KLDH area (Nwogu 2009, 1993a, 1993b; El Asmar and Nwogu 2006).

4.1 Numerical model grids

The same grid domain was used in evaluation of alternatives as was used for the existing federally authorized general navigation features for Kikiaola Light Draft Harbor. The model grids for each alternative were developed based on B2D requirements. For consistency of extracting and comparing model results at selected output points, all model grids for alternatives used the same spatial extent and output stations. Figures 56 through 63 show model bathymetry grids for the existing harbor after federal modifications, considered here as Alt-0, and eight alternatives. These model grids were generated from the sketches shown in Figure 6 through Figure 13, and the bathymetric data assembled from different sources as described in Chapter 2 and Chapter 3.
Figure 55 shows approximate boundaries of the channels and basin. The depth-contour range has been set to -3 m to 3 m MLLW to highlight the areas of interest. The entrance-channel depth was 11 ft MMLW (3.35 m), and it transitioned to 7 ft MLLW (2.13 m) in the access channel and parts of the harbor basin. Figure 56 shows the Alt-1 configuration with a sheet pile wall, approximately 300 ft (91.4 m) long, extending eastward from the south end of the existing inner groin.
Figure 56. Alt-1 grid with an interior sheet pile structure.
In Alt-2 (Figure 57), a 245 ft (74.5 m) southward oriented spur was added to the south end of west breakwater. The average water depth was approximately 7 ft (2.13 m) in the area where the spur was situated.
In Alt-3, two spurs were added to the existing harbor entrance breakwaters. Each spur was approximately 250 ft (76 m) long, and positioned along the channel edges where water depth varies (Figure 58).
Alt-3a is similar to Alt-3, with the west spur 460 ft (140 m) long, and east spur 425 ft (130 m) long. These south-southeast extending spurs (Figure 59) were positioned along the edges of channel at changing water depth.
Figure 60. Alt-4 grid with two straight and short exterior spurs attached to breakwaters.

In Alt-4 (Figure 60), both the west and east spurs were 200 ft (65 m) long. The channel was between these straight spurs extending southward.
In Alt-5 (Figure 61), the previous east spur that was removed during the federal modification study of the harbor was restored. The spur oriented in southwest direction was 165 ft (55 m) long, reaching the channel centerline. The boat traffic was shifted to the west side of channel.
In Alt-6 (Figure 62), the interior spur of east breakwater shortened in the federal modification study was restored and extended northward by approximately 50 ft (16 m). The interior spur protrudes slightly into the access channel.
In Alt-7 (Figure 63), a detached breakwater was placed on the offshore side of entrance channel. This 600 ft (200 m) long structure protects the existing entrance channel from incident waves coming from SW and S. In this configuration, the boat traffic takes place around both ends of the detached breakwater. Table 11 summarizes the main features of nine configurations.

**Table 11. Summary of nine harbor configurations investigated.**

<table>
<thead>
<tr>
<th>ID</th>
<th>Configuration</th>
<th>Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alt-0</td>
<td>Existing Harbor</td>
<td>Harbor geometry after federal modifications</td>
</tr>
<tr>
<td>Alt-1</td>
<td>Sheet Pile Wall</td>
<td>Added to inner breakwater tip, extends eastward, between the access channel and boat dock</td>
</tr>
<tr>
<td>Alt-2</td>
<td>West Breakwater Spur</td>
<td>Extends the length of west breakwater by adding a short spur that follows the channel’s western edge</td>
</tr>
</tbody>
</table>
### Configuration and Features

<table>
<thead>
<tr>
<th>ID</th>
<th>Configuration</th>
<th>Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alt-3</td>
<td>Short East and West Breakwater Spurs</td>
<td>Two exterior spurs which follow the channel edges</td>
</tr>
<tr>
<td>Alt-3a</td>
<td>Long East and West Breakwater Spurs</td>
<td>Extends dual spurs of Alt-3 (longer spurs)</td>
</tr>
<tr>
<td>Alt-4</td>
<td>Two Straight Spurs Pointing Southward</td>
<td>Dual parallel spurs oriented straight southward</td>
</tr>
<tr>
<td>Alt-5</td>
<td>East Breakwater Spur Oriented SW</td>
<td>Restores east breakwater spur removed in Federal modification of the harbor</td>
</tr>
<tr>
<td>Alt-6</td>
<td>East Breakwater Inner Spur</td>
<td>Extends the inner spur</td>
</tr>
<tr>
<td>Alt-7</td>
<td>Detached Offshore Breakwater</td>
<td>East-West oriented breakwater that intercept waves from S and SW</td>
</tr>
</tbody>
</table>

### 4.2 Model wave input data

As described in Chapter 3, incident-wave conditions were based on directional wave data from the CDIP Buoy 165 (Figure 18) and CHL field Gauge K5 (Figure 23). Using CMS-Wave, CDIP 165 wave data were transformed to the seaward boundary of the B2D grid, and the transformed wave conditions were compared to measured data at the deep water Gauge 5 (K5) obtained from the field-wave gauging study. This transformation was done in part to check and verify the accuracy of field-wave gauges. See Chapter 3 for additional information.

The maximum wave condition used in the federal modification study (Thompson et al. 1998) was also used in B2D for evaluation of alternatives. Significant wave height and peak period for this condition were $H_s=2.4$ m and $T_p=16$ sec, respectively. This wave condition was simulated for three incident wave directions (160, 180, and 200 degrees), and two water levels (0.0 m and 0.6 m, referenced to MLLW).

The field gauging data described in Appendix C were used in model calibration. Results from model calibration tests will be described later in this chapter.

### 4.3 B2D model domain

Nine model bathymetry grids covering the same domain were used for the existing harbor and eight alternatives selected. The near harbor sections of these grids are depicted in Figure 55 through Figure 63. The extent of en-
tire modeling domain is depicted in Figure 64. The nine grids were different at the harbor entrance and inner harbor where infrastructure modifications were introduced. The origin of these nine grids was located at the lower left corner, and the extent of model domain for nine grids was the same (1.9 km by 2.1 km). The offshore boundary of model grids was placed beyond field Gauge 5 (K5) at the 20 m (65.6 ft) depth contour. The grids were oriented in N-S direction. The locations of five field gauges are marked on Figure 26.

The straight green line with an arrow shown in Figure 64 represents the interior wavemaker used in the nine B2D model grids. The wavemaker was located at the 20 m (65.6 ft) constant water depth region of model domain, approximately 250 m shoreward of the offshore boundary. Depending on the characteristics of incident waves specified for the three incident wave directions, sensitivity tests were conducted for input parameters affecting the computational stability of model and solutions. The effects of Chezy and Smagorinsky coefficients and damping layers on model results were investigated. For oblique waves (E and W side grid boundaries), these tests provided guidance for adjustment to lateral damping layers. Absorbing lateral damping layers with 10 m width were used along parts of the east and west boundaries for oblique waves coming from SE and SW direction.
Model calibration with field data was performed next. These simulations indicated harbor interior shorelines with less than 10% wave reflection, and therefore subsequently considered as absorbing-type land boundaries. For consistency, the same damping layers were applied to the harbor interior boundaries for nine model grids. Model calibration results were sensitive to damping width and coefficient selected because of long-period infragravity waves (IG) developing within the confined harbor area. In this application, for wave periods less than 20 sec and damping width of 5 m to 25 m, the damping coefficients of 0.0, 0.05 and 0.1 were used. See Section 4.5 for additional information about model calibration.

B2D model simulations provide temporal and spatial solution files of the mean water level (i.e., wave setup), mean wave direction, significant wave height, mean velocity (current), and other optional outputs which include water surface elevation and time series of water surface, pressure, etc., at
Probe locations. For the nine grids, B2D model results were saved over the entire grid domain, as well as at 20 selected output locations (called probes). The B2D model interface in the Surface-water Modeling System (SMS) was used to view, extract, and post-process model results (i.e., wave parameters such as wave height, period, direction, water level, and wave-induced current) in the computational domain. Additional analyses of model time-series solution files saved at the specified probes were performed both in time-domain and frequency-domain to obtain wave parameters and statistical estimates for this study.

4.4 Details of wave simulations

Wave conditions

Wave conditions used in the federal modification study (Thompson et al. 1998) were also used in the B2D simulations. A wave-climate representative of the seasonal mean-wave conditions was considered in that study, where the range of short- and long-wave conditions incident to Kikiaola Harbor was based on the Wave Information Study (WIS) hindcast. A representative range of wave periods and directions affecting the harbor was modeled. The shortest wave period (6 sec) represented a local sea condition and the longest wave period (22 sec), a long swell condition. Three incident-wave directions relative to true north (164, 184, and 204 degrees, waves coming from SE, S, and SW, respectively) were chosen as the most likely approach directions to the harbor entrance (an adequate representation of the sea states obtained from the WIS hindcast). The incident waves from the SE to SW sector included all waves coming straight into the harbor from south (180 deg), and 20 deg oblique waves either side of the entrance from the southeast (160 deg) and southwest (200 deg) directions. These three incident wave directions are used in the present study.

The previous study results indicated harbor response was relatively insensitive to water level and incident long wave direction approaching the harbor entrance from deep water. The significant wave height and peak period of the largest incident waves, from southwest, south, and southeast directions, were \( H_s = 2.4 \, \text{m} \) and \( T_p = 16 \, \text{sec} \), respectively. This most severe wave condition was used in the present study B2D simulations with two water levels, Mean Lower Low Water (MLLW = 0 \, \text{m}) and Mean Higher High Water (MHHW = 0.6 \, \text{m}). These combinations of waves, water levels,
and directions were used in B2D to evaluate eight alternatives and the existing harbor (i.e., nine harbor configurations).

**Output Locations**

B2D results were saved over the entire grid domain, including special locations of interest. The same 15 output stations, called basins in the previous study (see Figure 65), are used in the present study (Figure 66). Not only do these two figures show locations of the basins, but also two different harbor geometries. Wave estimates at the basins are used in the present study to compare eight alternatives to the existing harbor, which is the post-modification harbor geometry. However, the closest in-water point is used to extract B2D results in the present study if a save location was over a structure in any alternative. The parameters extracted from model solution files were averaged over the neighboring points surrounding the output location. This averaging method was also used in the previous study.

For clarity, since the configuration of the previous harbor was different from the existing harbor, the location and numbering of 15 basins used in the federal modification study are shown separately in Figure 65. For the existing harbor grid (i.e., after federal modification), these same output stations are provided in Figure 66, which are also used in the model grids for the eight alternatives investigated.
Figure 65. Basins (output stations) used in previous modeling study.

Figure 66. Output stations used in the present modeling study.
4.5 Model calibration and validation

A number of simulations were conducted to calibrate the B2D using the CMS-Wave calculated wave conditions offshore of Gauge K5 and measured data at K5. In these simulations, day-to-day operational wave conditions at KLDH were considered first, followed by one extreme wave condition that was recorded during the measurement period. Findings from these model calibration simulations are described next.

The first set of calibration tests were made with four selected CMS-Wave calculated wave parameters seaward of Gauge K5. These were typical wave conditions (i.e., waves neither too small nor too large) in October and November 2012: 12 Oct 202 at 2200 GMT (Hs=1.45 m, Tp=20 sec, theta =186 deg), 14 Oct 2012 at 0300 GMT (Hs=0.98 m, Tp=14.3 sec, theta =192 deg), 14 Nov 2012 at 1500 GMT (Hs=1.04 m, Tp=14.3 sec, theta =188 deg), and 27 Nov 2012 at 1200 GMT (Hs=0.57 m, Tp=12.5 sec, theta =184 deg). The default values of computational parameters for B2D (Demirkilek et al. 2005a and 2005b) were used in these validation tests. The wave parameters determined from CMS-Wave were used in B2D simulations with the assumed values of directional wave spreading and wave spectrum peakedness parameters. Because these two assumed parameters control the distribution of wave energy in the wave spectrum over the frequency range, these can strongly influence the model-data comparison. A second type of model validation was conducted using the measured time series data from Gauge K5 that will be described subsequently.

Tables 12 and 13 present the comparison of model and data for two wave conditions of 2200 GMT, 12 Oct 2012, and 0300 GMT, 14 Oct 2012. The measured data from Gauge K5 were analyzed and used to compare B2D model predictions to data. It should be noted that the B2D wavemaker was located seaward of Gauge K5 and wave conditions were simulated as unidirectional sea states. Test results from these two simulations indicated wave parameters (height, period, and direction) were changing before waves reached the location of K5 on the reef face. This required a rescaling (adjustment) of the wavemaker inputs and calibrated parameters to ensure model predictions are close to data at the K5 location. B2D was then driven with the adjusted inputs and parameters starting from the wavemaker using the best test values of wave spreading and peakedness parameters. The model results were extracted at the locations of gauges K1, K3, K4, and K5 and the integrated wave parameters (height, period, and direction) were compared to data (e.g., validation).
Percent difference and bias statistics for model-data comparison were calculated and provided in Tables 12 and 13. Agreement between model and data is better for two outside gauges (K3 and K5) situated on the reef, while larger differences occur at two inner gauges (K1 and K4). The high noise level in data at two inner gauges (high noise-to-signal ratio) combined with strong effects of wave reflection and diffraction from the boundaries and structures contributed to poor model-data agreement at these gauges. The wave heights obtained from analysis of K1 and K4 gauge data were consistently low, wave periods were high, and wave directions changed erratically in the vicinity of interior gauges. For these reasons, larger differences occur between model and data at these gauges.

Generally, the model-calculated wave heights compare better, while the comparison for wave periods and directions is mixed. The calculated wave periods were lower than data. The largest difference between model and data was in wave directions. Overall, model results followed the field-data trend correctly. Wave height decreased from offshore to shore. Wave periods inside the harbor generally increased. Wave directions at two outside gauges were largely from south, and wave directions at interior gauges were rapidly changing. A fine-tuning of the model was not considered because such tuning for these particular wave conditions might not work for other wave conditions. A different type of model calibration for a severe wave condition is described later to demonstrate the model can be calibrated for evolution of the wave spectra.

As Tables 12 and 13 indicate, model results agree with data at the two outside gauges (K3 and K5). For the two inner gauges (K1 and K4), there are noticeable differences between the model and data. Although the model could be tuned to improve these results, this was not considered for two reasons. First, given that different types of instruments were used in the exterior and interior harbor, the observed difference might be related to types of gauges used (e.g., the Aquadopp gauges were used inside the harbor and AWAC directional gauges were deployed outside). The appendices of this report describe how different gauge types collected data, including differences in their sampling rate, burst duration, and rest periods, data binning applied to pressure, surface elevation, and velocities, etc. Most importantly, wave directions were not measured by gauges K1 and K4; these were estimated from the post-analysis of data. These main features of instruments used in the interior of harbor affected the characteristics of
data collected inside the harbor, causing larger discrepancies between model and data at gauges K1 and K4.

Table 12. Model validation with data on 12 Oct 2012.

<table>
<thead>
<tr>
<th>12Oct2012 2200 GMT</th>
<th>Data</th>
<th>B2D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hs</td>
<td>Tp</td>
</tr>
<tr>
<td>K5</td>
<td>1.45</td>
<td>20.0</td>
</tr>
<tr>
<td>K3</td>
<td>1.05</td>
<td>20.0</td>
</tr>
<tr>
<td>K4</td>
<td>0.20</td>
<td>20.0</td>
</tr>
<tr>
<td>K1</td>
<td>0.20</td>
<td>20.0</td>
</tr>
</tbody>
</table>

Values in parentheses are percent difference (*) and bias (**) between model and data

Table 13. Model validation with data on 14 Oct 2012

<table>
<thead>
<tr>
<th>14Oct2012 0300 GMT</th>
<th>Data</th>
<th>B2D</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hs</td>
<td>Tp</td>
</tr>
<tr>
<td>K5</td>
<td>0.98</td>
<td>14.3</td>
</tr>
<tr>
<td>K3</td>
<td>0.92</td>
<td>14.3</td>
</tr>
<tr>
<td>K4</td>
<td>0.16</td>
<td>16.7</td>
</tr>
<tr>
<td>K1</td>
<td>0.13</td>
<td>50</td>
</tr>
</tbody>
</table>

Values in parentheses are percent difference (*) and bias (**) between model and data

The method used in the two calibration tests from October was applied to two randomly selected wave conditions from the month of November 2012. The two wave conditions selected were 14 Nov 2012 at 1500 GMT (Hs=1.04 m, Tp=14.3 sec, theta =188 deg), and 27 Nov 2012 at 1200 GMT (Hs=0.57 m, Tp=12.5 sec, theta =184 deg). Table 14 and Table 15 provide comparison of model and data for these simulations. Overall, results are similar to October 2012 results.
Table 14. Model validation with data on 14 Nov 2012.

<table>
<thead>
<tr>
<th>14Nov2012</th>
<th>Data</th>
<th>B2D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1500 GMT</td>
<td>H_s</td>
<td>T_p</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K5</td>
<td>1.04</td>
<td>14.3</td>
</tr>
<tr>
<td>K3</td>
<td>0.89</td>
<td>14.3</td>
</tr>
<tr>
<td>K4</td>
<td>0.13</td>
<td>7.7</td>
</tr>
<tr>
<td>K1</td>
<td>0.18</td>
<td>14.3</td>
</tr>
</tbody>
</table>

Values in parentheses are percent difference (*) and bias (**) between model and data.

Table 15. Model validation with data on 27 Nov 2012.

<table>
<thead>
<tr>
<th>27Nov2012</th>
<th>Data</th>
<th>B2D</th>
</tr>
</thead>
<tbody>
<tr>
<td>1200 GMT</td>
<td>H_s</td>
<td>T_p</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K5</td>
<td>0.57</td>
<td>12.5</td>
</tr>
<tr>
<td>K3</td>
<td>0.51</td>
<td>12.5</td>
</tr>
<tr>
<td>K4</td>
<td>0.12</td>
<td>12.5</td>
</tr>
<tr>
<td>K1</td>
<td>0.13</td>
<td>12.5</td>
</tr>
</tbody>
</table>

Values in parentheses are percent difference (*) and bias (**) between model and data.

Following the above four calibration tests, one more detailed calibration was performed using one of the largest measured wave conditions. Table 16 shows the list of the six largest events reported at gauge K5 during field-gauging deployment. These events include incident waves coming from SE, S, and SW directions. The wave heights for these large wave events were generally similar, varying from 1.3 m to 1.6 m. The wave periods for these events were quite different, and ranged from 6 sec to 20 sec. Because the characteristics of large incident waves were different from those of daily wave conditions, it was necessary to investigate model-data comparison for one of these large wave events to properly calibrate the model at this site.
Table 16. Six largest wave events recorded during field measurements.

<table>
<thead>
<tr>
<th>Event Date &amp; Time</th>
<th>$H_s$ (m)</th>
<th>$T_p$ (sec)</th>
<th>Theta (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12 Oct 2012, 2200 GMT</td>
<td>1.45</td>
<td>20.0</td>
<td>186</td>
</tr>
<tr>
<td>23 Oct 2012, 2100 GMT</td>
<td>1.51</td>
<td>16.7</td>
<td>184</td>
</tr>
<tr>
<td>14 Nov 2012, 0500 GMT</td>
<td>1.27</td>
<td>14.3</td>
<td>192</td>
</tr>
<tr>
<td>7 Dec 2012, 1800 GMT</td>
<td>1.36</td>
<td>8.3</td>
<td>172</td>
</tr>
<tr>
<td>27 Jan 2013, 0900 GMT</td>
<td>1.57</td>
<td>6.7</td>
<td>168</td>
</tr>
<tr>
<td>27 Jan 2013, 1300 GMT</td>
<td>1.54</td>
<td>8.3</td>
<td>182</td>
</tr>
</tbody>
</table>

For a severe wave condition, the first wave condition on 12 Oct 2012 was chosen to investigate the capabilities of the B2D for transformation of wave spectra (evolution of the spectral shape) occurring while waves move over the reef and into the interior of harbor. This required running the B2D with the real field data (i.e., measured time series and spectrum) and calibrating the bottom friction and turbulence coefficients. So, in this second type of model validation, only the two calibrated coefficients would affect model and data agreement. In this case, model-data comparison is presented in the form of a spectral shape change, showing the variation of wave-energy density as a function of wave frequencies at different water depths for different values of the two calibration coefficients. Results are discussed next.

B2D model simulations for the first event in Table 16 were performed as unidirectional and multi-directional sea states. Wave parameters (height, period, and direction) obtained from analyses of field data were used in the unidirectional simulations. For multi-directional simulations, the model was driven with the real measured time series of surface elevation and velocities. The directional spread was introduced either by using the measured time series of horizontal velocities ($u,v$), or by a user-specified directional spread (Nwogu and Demirbilek 2001; Demirbilek et al. 2009, 2008, 2007a, 2007b, 2007c). Model results for this event are described next, including effects of the bottom friction (Chezy) and turbulence (Smagorinsky) coefficients on model calibration and comparison of the transformed wave spectra at four gauge locations (K5, K3, K4, and K1).

The overall agreement between the model and data improved, and the differences between measured and model calculated wave spectra at gauges
K3, K4, and K1 were reduced significantly. Comparison of wave spectra obtained by different input types and using different model parameters at gauges K5, K3, K4, and K1 are shown in Figure 67 through Figure 74. The comparison of wave spectra with Chezy coefficients 25 and 30, and from different wavemaker inputs for Smagorinsky coefficient of 0.2 and 0.5 is shown in Figures 67 and 68. Results for gauge K3 are shown in Figures 69 and 70, for K4 in Figures 71 and 72, and for K1 in Figures 73 and 74. For each gauge, the first figure shows the best agreement between model and data using user-specified directional spread for Smagorinsky and Chezy coefficients of 0.5 and 30, respectively. The second figure shows the comparison of measured and calculated spectra for three different combinations of inputs and coefficients (i.e., sensitivity tests). These tests showed B2D captured the evolution of wave spectra and re-distribution of wave energy over wave frequencies as waves transformed from deep water into shallow depths and inside the harbor complex.

The harbor is also subject to impacts of long period (14-20 sec) NW swell refracting into the harbor area. No field data are available to validate the model for NW swell, however it is assumed to be covered by the SW wave cases tested.
Figure 67. Comparison of measured and B2D model calculated wave spectra at K5 with Chezy=25.

Gauge #5

Figure 68. Comparison of measured and B2D model calculated wave spectra at K5 by three methods with Chezy=30.

Gauge #5
Figure 69. Comparison of measured and B2D model calculated wave spectra at K3 with Chezy=25.

Gauge #3

Figure 70. Comparison of measured and B2D model calculated wave spectra at K3 by three methods with Chezy=30.

Gauge #3
Figure 71. Comparison of measured and B2D model calculated wave spectra at K4 with Chezy=25

Figure 72. Comparison of measured and B2D model calculated wave spectra at K4 by three methods with Chezy=30.
Figure 73. Comparison of measured and B2D model calculated wave spectra at K1 with Chezy=25.

Gauge #1

Figure 74. Comparison of measured and B2D model calculated wave spectra at K1 by three methods with Chezy=3.

Gauge #1
4.6 Comparison of Alternatives

The B2D model provides estimates of wave height, period, direction, wave-induced current, and time series data. One or more of these outputs, or other engineering parameters derived by post-processing model results, could be used in evaluation of improvements to harbor utilization and for checking navigation safety. Engineering parameters selected from B2D model when checked against operational criteria will determine pros and cons of a proposed alternative. For consistency with the previous federal modification study, the wave height was selected in the present study as the primary parameter to compare the alternatives. The reduction of wave energy was the ultimate goal for improving conditions in existing harbor, which was used as the baseline in evaluation of alternatives investigated.

B2D simulations were performed for nine harbor configurations (i.e., existing harbor plus eight alternatives). Model simulations for each alternative considered three directions and two water levels, for a total of 54 simulations. Model results for each alternative are presented as two-dimensional (2D) color contours of wave height, providing the big picture of wave fields showing low and high wave energy areas. B2D model results are provided for three incident wave directions from southeast (SE=160 deg), south (S=180 deg), and southwest (SW=200 deg). Results are provided for two water levels, MLLW=0 m, and MHHW=0.6 m.

The calculated wave heights at the basins, or output stations used in previous study, are used to compare alternatives. B2D model results are presented in two different types of plots: all nine configurations presented together in a graph, and results presented for a group of five alternatives to show additional details. To improve presentation of model results, plots for two sub-groups are provided. The first group consists of Alt-0, Alt-1, Alt-2, Alt-3, and Alt-3a, and the second group Alt-0, Alt-4, Alt-5, Alt-6, and Alt-7. The existing harbor (Alt-0), the baseline for comparing the alternatives, is in both groups. A third type of graphical plot provides model results at the ten most interior basins (i.e., basins 1 through 10). The fourth plot provides model results at the five entrance channel basins (i.e., basins 11-15).

Following these different graphical presentations, calculated wave-height statistics at 15 basins are provided in tabular format for each incident-wave direction. These tables summarize the maximum, minimum, and av-
verage values of wave height at two water levels for each alternative. A description of the results for each incident wave direction follows.

4.6.1 Southeast (SE) Waves

For the selected incident wave condition \((H_s = 2.4 \text{ m}, T_p = 16 \text{ sec}, \theta = 160 \text{ deg})\) and MLLW=0.0 m, calculated wave heights extracted at 15 basins for nine configurations are shown in Figures 75, 76, and 77. The results for all alternatives are provided in Figure 75. Figures 76 and 77 show results for the two groups. The plots for the first group (Alt-0, Alt-1, Alt-2, Alt-3, Alt-3a) and second group (Alt-0, Alt-4, Alt-5, Alt-6, Alt-7) provide more details about the variation of waves useful to compare the alternatives. Wave- height estimates inside the harbor and the extent of wave reduction achieved inside the harbor for each alternative required separating the interior and exterior basins. Wave heights at ten interior basins (1 through 10) for MLLW=0.0 m are shown in Figure 78. Wave heights at five exterior basins (11 through 15) are shown in Figure 79. The corresponding results for MHHW=0.6 m are provided in Figures 80-84.
Figure 75. Comparison of alternatives (SE waves, MLLW).
Figure 76. Comparison of first group alternatives (SE waves, MLLW).
Figure 77. Comparison of second group alternatives (SE waves, MLLW).
Figure 78. Comparison of alternatives at 10 interior basins (SE waves, MLLW).

Figure 79. Comparison of alternatives at five exterior basins (SE waves, MLLW).
Figure 80. Comparison of alternatives (SE waves, MHHW).
Figure 81. Comparison of first group alternatives (SE waves, MHHW).
Figure 82. Comparison of second group alternatives (SE waves, MHHW)
Figure 83. Comparison of alternatives at 10 interior basins (SE waves, MHHW).

Wave height at 10 stations

Wave height at exterior stations 11 - 15

Figure 84. Comparison of alternatives at five exterior basins (SE waves, MHHW).
4.6.2 South (S) waves

Results for incident waves from south at MLLW are shown in Figure 85 through Figure 89. Calculated wave heights for all alternatives at the 15 basins are shown in Figure 85. Results for two groups are presented in Figures 86 and 87. Calculated wave heights for all alternatives at the 10 interior and five exterior basins are shown in Figures 88 and 89, respectively. Model results for MHHW are depicted in Figure 90 through Figure 94.

Figure 85. Comparison of alternatives (S waves, MLLW).
Figure 86. Comparison of first group alternatives (S waves, MLLW).
Figure 87. Comparison of second group alternatives (S waves, MLLW).
Figure 88. Comparison of alternatives at 10 interior basins (S waves, MLLW).

Figure 89. Comparison of alternatives at five exterior basins (S waves, MLLW).
Figure 90. Comparison of alternatives (S waves, MHHW).
Figure 91. Comparison of first group alternatives (S waves, MHHW).
Figure 92. Comparison of second group alternatives (S waves, MHHW).
4.6.3 Southwest (SW) Waves

Calculated wave heights for MLLW=0 m at 15 basins are provided in Figure 95 for all alternatives. Figures 96 and 97 show detailed results for the two groups. Calculated wave heights for all alternatives at the 10 interior and five exterior basins are provided in Figures 98 and 99, respectively.
The corresponding B2D model results for the MHHW=0.6 m are presented in Figure 100 through Figure 104.

**Figure 95.** Comparison of alternatives (SW waves, MLLW).
Figure 96. Comparison of first group alternatives (SW waves, MLLW).
Figure 97. Comparison of second group alternatives (SW waves, MLLW).
Figure 98. Comparison of alternatives at 10 interior basins (SW waves, MLLW).

Figure 99. Comparison of alternatives at five exterior basins (SW waves, MLLW).
Figure 100. Comparison of alternatives (SW waves, MHHW).
Figure 101. Comparison of first group alternatives (SW waves, MHHW).
Figure 102. Comparison of second group alternatives (SW waves, MHHW).
Figure 103. Comparison of alternatives at 10 interior basins (SW waves, MHHW).

Figure 104. Comparison of alternatives at five exterior basins (SW waves, MHHW).
4.7 Discussion of results

Based on results shown in Figures 75 through 104 for SE, S, and SW incident waves and two water levels, the following observations are made:

1. For all eight alternatives investigated, the maximum change in wave height at 15 output stations between the two water levels was less than 10%. This agrees with the conclusion of previous study that water level effect on waves at the KLDH complex is negligible.

2. Wave height estimates in the inner basins (Stations 1 through 10) were comparatively smaller than calculated wave heights at other basins located in the entrance and access channels. In general, a lesser reduction in wave height was attained for the majority of alternatives (Alt-7 being the exception) with infrastructure modifications to exterior of harbor. These had a minimal effect on waves reaching the inner harbor area.

3. Relative to the existing harbor (Alt-0), the sheet pile wall (Alt-1) produced the largest reduction in the wave height at inner basin, including at the boat ramp/dock area (Basins 1 through 4). Alt-7, Alt-5, and Alt-6 achieved the next best reduction in wave height at the inner basins.

4. Relative to the four alternatives with the largest wave height reduction (Alt-1, Alt-7, Alt-5, Alt-6) at the interior harbor, other alternatives (Alt-2, Alt-3, Alt-3a, Alt-4) produced comparatively a smaller wave height reduction, and hence were the bottom four ranked alternatives.

5. The calculated wave heights for all alternatives in the boat ramp/dock area (Stations 1 through 4) were amongst the smallest of the 15 output locations. Because wave heights for every alternative were small (and similar in magnitude) in this interior area of the harbor, the efficacy of each alternative had to consider the wave-height reduction attained at other interior output stations.

The above general observations hold true for incident waves coming from SE, S, and SW directions. For these three incident wave directions, Figures 75 through 104 show a very similar trend in wave-height variation at the 15 output stations. For each alternative, the largest difference in wave height occurred at the inner basins with changing incident wave direction. Wave-height statistics for three directions are summarized in Tables 17, 18 and 19. These statistics at the 15 output stations are based on data used in Figures 75 through 104 for three incident wave directions (S, SW, and SE) and two water levels (MLLW and MHHW).
Table 17. Summary of SE wave-height statistics at 15 output stations for Alt-0 to Alt-7.

<table>
<thead>
<tr>
<th>Alts</th>
<th>Max Hs</th>
<th>Min Hs</th>
<th>Ave Hs</th>
<th>Ave Hs (interior)</th>
<th>Ave Hs (exterior)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MLLW</td>
<td>MHHW</td>
<td>MLLW</td>
<td>MHHW</td>
<td>MLLW</td>
</tr>
<tr>
<td>0</td>
<td>1.77</td>
<td>2.01</td>
<td>0.58</td>
<td>0.62</td>
<td>0.82</td>
</tr>
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<td></td>
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<td>0.91</td>
<td>0.61</td>
<td>0.65</td>
</tr>
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</tr>
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<td>1.23</td>
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</tr>
<tr>
<td>2</td>
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<td>0.60</td>
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Table 18. Summary of S wave-height statistics at 15 output stations for Alt-0 to Alt-7.

<table>
<thead>
<tr>
<th>Alts</th>
<th>Max Hs</th>
<th>Min Hs</th>
<th>Ave Hs</th>
<th>Ave Hs (interior)</th>
<th>Ave Hs (exterior)</th>
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<tbody>
<tr>
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Table 19. Summary of SW wave-height statistics at 15 output stations for Alt-0 to Alt-7.

<table>
<thead>
<tr>
<th>SW Wave</th>
<th>Alts</th>
<th>Max Hs (MLLW)</th>
<th>Min Hs (MLLW)</th>
<th>Ave Hs (MLLW)</th>
<th>Ave Hs (interior) MHHW</th>
<th>Ave Hs (exterior) MHHW</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
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</tr>
<tr>
<td></td>
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<td>1.82</td>
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</tr>
<tr>
<td></td>
<td>2</td>
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<td>2.16</td>
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</tr>
<tr>
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<td>0.67</td>
<td>0.64</td>
<td>0.92</td>
</tr>
<tr>
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<td>0.00</td>
<td>0.72</td>
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<td>1.10</td>
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<td>0.66</td>
</tr>
</tbody>
</table>

The wave-height statistics in Tables 17, 18, and 19 indicate that incident waves from SW and S directions have a greater impact on the harbor. This occurs because the present harbor entrance has a greater exposure to incident waves coming from SW and S directions.

The calculated minimum, maximum, and average wave heights at the 15 output stations help to determine wave-height variation occurring between the harbor entrance and boat ramp/dock area in inner harbor. The average wave heights in the interior and exterior output stations provide estimates of wave attenuation between the entrance and interior harbor. The wave-height statistics indicated about 50% reduction in wave height from the entrance to harbor interior. For all alternatives, calculated wave heights at the dock area (output stations 1-4) for three wave directions and two water levels simulated were amongst the lowest of the 15 output locations. Wave heights at the exterior output stations 11 through 15 are higher than wave heights at the interior output stations 1 through 10. For incident waves from SE, S, and SW, the wave heights for Alt-1, Alt-7, and Alt-5 at the exterior stations are on average 2.5, 2.1, and 1.8 times greater than wave heights at the interior stations, respectively.
4.7.1 Ranking of Alternatives

The alternatives have been ranked based on calculated average wave-height statistics at the ten most interior output stations in Table 20 and Figure 105. Table 21 and Figure 106 show the ranking using average height statistics at the five exterior output stations. Table 22 and Figure 107 show the ranking based on average wave-height statistics at the all 15 output stations. Tables 20 through 22 show the relative ranking of alternatives for three incident wave directions. These are displayed graphically in Figure 105 to Figure 107. Table 20 and Figure 105 show the top three ranked alternatives are Alt-1, Alt-7, and Alt-5, which have the smallest average wave height at the ten interior basins.

Based on the five exterior basins, Table 21 and Figure 106 show that the top-ranked three alternatives are Alt-7, Alt-5, and Alt-3a for SE and S directions, and for SW incident wave direction Alt-7, Alt-3a, and Alt-5 top the list.

Using all 15 basins, Table 22 and Figure 107 show that top-ranking three alternatives for SE and S directions are Alt-7, Alt-5, and Alt-1, while Alt-7, Alt-3a, and Alt-5 emerge as the top three for the SW incident-wave direction.

The same rankings were obtained for two water levels (MLLW and MHHW), and wave heights for MHHW were generally greater than those for MLLW. Wave-height differences for two water levels were much less for the interior basins than those for the exterior basins.

The USACE has a maximum wave height of 2 ft threshold criterion at the entrance channels for small boat shallow-draft harbors. These results show that only Alt-7 comes close to meeting this criterion (2.1 ft based on the five exterior basins and 1.87 ft for all 15 basins). The alternatives investigated did not meet the USACE’s 1 ft threshold limit in the interior of shallow-draft small boat harbors.
Table 20. Ranking of alternatives using model results at 10 interior output stations.

<table>
<thead>
<tr>
<th></th>
<th>SE waves</th>
<th>S waves</th>
<th>SW waves</th>
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</thead>
<tbody>
<tr>
<td>Alt-1</td>
<td>Alt-1</td>
<td>Alt-1</td>
<td></td>
</tr>
<tr>
<td>Alt-7</td>
<td>Alt-7</td>
<td>Alt-7</td>
<td></td>
</tr>
<tr>
<td>Alt-5</td>
<td>Alt-5</td>
<td>Alt-5</td>
<td></td>
</tr>
<tr>
<td>Alt-6</td>
<td>Alt-6</td>
<td>Alt-6</td>
<td></td>
</tr>
<tr>
<td>Alt-0</td>
<td>Alt-2</td>
<td>Alt-3a</td>
<td></td>
</tr>
<tr>
<td>Alt-2</td>
<td>Alt-0</td>
<td>Alt-4</td>
<td></td>
</tr>
<tr>
<td>Alt-3a</td>
<td>Alt-3a</td>
<td>Alt-2</td>
<td></td>
</tr>
<tr>
<td>Alt-4</td>
<td>Alt-4</td>
<td>Alt-0</td>
<td></td>
</tr>
<tr>
<td>Alt-3</td>
<td>Alt-3</td>
<td>Alt-3</td>
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</tbody>
</table>

Figure 105. Ranking of alternatives based on model results at 10 interior output stations.
Table 21. Ranking of alternatives based on model results at five exterior output stations.

<table>
<thead>
<tr>
<th>SE waves</th>
<th>S waves</th>
<th>SW waves</th>
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</thead>
<tbody>
<tr>
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<td>Alt-5</td>
<td>Alt-5</td>
<td>Alt-3a</td>
</tr>
<tr>
<td>Alt-3a</td>
<td>Alt-3a</td>
<td>Alt-5</td>
</tr>
<tr>
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<td>Alt-1</td>
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<td>Alt-0</td>
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<td>Alt-3</td>
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</tr>
<tr>
<td>Alt-3</td>
<td>Alt-3</td>
<td>Alt-1</td>
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</table>

Figure 106. Ranking of alternatives based on model results at five exterior output stations.
Table 22. Ranking of alternatives using model results at 15 output stations.

<table>
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<tr>
<th>SE waves</th>
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<th>SW waves</th>
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</thead>
<tbody>
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<td>Alt-3a</td>
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<td>Alt-1</td>
<td>Alt-1</td>
<td>Alt-5</td>
</tr>
<tr>
<td>Alt-3a</td>
<td>Alt-3a</td>
<td>Alt-6</td>
</tr>
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<td>Alt-2</td>
</tr>
<tr>
<td>Alt-0</td>
<td>Alt-0</td>
<td>Alt-4</td>
</tr>
<tr>
<td>Alt-2</td>
<td>Alt-2</td>
<td>Alt-0</td>
</tr>
<tr>
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<tr>
<td>Alt-3</td>
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</tr>
</tbody>
</table>

Figure 107. Ranking of alternatives based on model results at 15 output stations.

The ranking of alternatives in Table 20 through Table 22 was verified for a different incident-wave condition with a smaller wave height and shorter wave period (Hs = 1 m and Tp = 12 sec). This condition represented normal operational (i.e., non-storm) sea states. Although results for this condition did not change the top-ranking alternatives in Tables 20 to 22, they
revealed that wave heights in shallow water depths were not linearly proportional to the incident-wave height. This finding is important as it would suggest that except for small amplitude non-breaking long period swells, wave propagation over the complex reef system into the harbor cannot be treated as a linear problem.

Simulations were conducted with incident waves of 1 m height and greater moving over the shallow reef toward the entrance; strong wave breaking was evident in B2D model simulations. Consequently, the wave-amplification factor, defined as the ratio of local wave height to incident-wave height, may not be used to accurately extrapolate one specific value of incident-wave height and period to other values of incident-wave heights or periods. Also, this finding indicates that because strong wave shoaling, refraction, and breaking occur over the reef system, the offshore boundary of the numerical model grid should be located at the water depth where these wave processes have not yet started to occur. Both of these requirements were taken into consideration in setting up the B2D grids to evaluate the alternatives.

4.7.2 Comparison to previous study results

A linear wave model called HARBD was used in the previous federal modifications study. At the time, HARBD was the state-of-the-art model for harbor applications. The model grid boundary was placed at the entrance in shallow depth (approximately at 10 ft depth contour), and thus the modeling domain covered the harbor entrance and interior. The model’s open boundary was situated right at the entrance close to structures. The incident waves at the open boundary of HARBD at such shallow depth are expected to be strongly affected by the exterior reef system, as well as by wave reflection from the shorelines and structures. The depths at the entrance and access channel and interior of the pre-project harbor were comparatively shallower than the post-project (existing harbor). The entrance of the pre-project harbor configuration was different than the entrance of the existing harbor entrance. The former had exterior and interior spurs attached to the east breakwater, which were removed from the existing harbor.

There are other differences between the two model setups. For example, the grid for B2D extends to 60 ft depth contour and covers a much larger domain, including the entire reef system, and nearshore areas to the east and west of harbor. Monochromatic waves were used in HARBD to con-
struct an approximate wave spectrum. Incident waves in B2D were simulated as multi-directional or uni-directional random seas. Additionally, there are other differences between these models (i.e., governing equations are different, modeling of wave physics are different, including wave breaking, diffraction, reflection, etc.).

Due to the above noted differences between these two model grids and their theoretical and numerical formulations, a one-to-one direct comparison between these two different classes of wave models is not possible. In the following section, B2D model results will be extracted at the same output locations (called basins in HARBD). Because HARBD results were reported as the amplification factors (e.g., calculated-wave height divided by the incident-wave height), B2D results are also provided as amplification factors. It is noted that the concept of amplification factor is not applicable to nonlinear models and calculated nonlinear responses (e.g., wave heights) by such models. Consequently, results provided in Table 23 are for information, and should not be used to compare models. For the reasons described above, a comparison of two models would not be meaningful.

Listed in Table 23 are the wave heights calculated by B2D and HARBD models for SE, S, and SW waves at the 15 output locations at MLLW=0 m. Wave heights for HARBD are calculated by multiplying the amplification factor with the incident-wave height (H_s=2.4 m). Results from two models are different for incident waves from 160 deg, 180 deg, and 200 deg. HARBD predictions are higher than B2D at the five exterior basins (11 through 15). This is because the incident-wave height of 2.4 m is applied to the B2D boundary at the 60 ft depth, while the same height was used at the boundary of HARBD grid at about 10 ft depth. B2D results indicate waves are modified by the reef, and wave height reduces to 1.82 m, 1.72 m, and 1.9 m respectively for SE, S, and SW incident waves. Consequently, HARBD estimates of wave shoaling are higher at the entrance channel at the five exterior basins 11 to 15 for waves incident from three directions.

Overall, B2D produced comparatively higher wave amplification factors inside the harbor for incident waves from SW, S and directions (e.g., 160 deg, SSE, and 180 deg, S). Wave-height reduction of more than 50% occurs at the inner harbor basins (1 to 10). Calculated wave height is less than 15% of the incident wave height in the back of the inner harbor (e.g., basins 1 to 4 near the dock).
The results provided in Table 23 can be expressed in terms of the wave-amplification factor, which is defined as the local wave height at any computational point divided by the incident-wave height. This definition can be used only for linear models, where calculated response is not dependent on the incident-wave parameters. This is not true for nonlinear models where the calculated response varies with incident-wave parameter, not just with the incident-wave height but also with the wave period and direction, as well as nearshore wave processes which affect the calculated response. These include wave breaking, dissipation, diffraction and reflection, etc., which are treated differently in two wave models. Because HARBD results were provided in the past study as the wave-amplification factors, the normalized wave heights are shown in Table 24 for readers’ information. Overall, B2D calculated amplification factors for the interior basins (1 through 10) are greater than HARBD amplification factors. For the exterior basins (11 through 15), HARBD amplification factors are greater.

**Table 23. Wave heights calculated by HARBD and B2D models for incident-wave condition (H_s=2.4 m, T_p=16 sec at MLLW= 0 m).**

<table>
<thead>
<tr>
<th>Location</th>
<th>Incident Wave Direction, θ_m (deg, meteorological)</th>
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<th>180</th>
<th>200</th>
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<td></td>
<td>HARBD</td>
<td>B2D</td>
<td>HARBD</td>
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<tr>
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<td>0.19</td>
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<td>0.29</td>
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<tr>
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Table 24. Wave-amplification factors calculated by HARBD and B2D models for incident-wave condition ($H_s=2.4$ m, $T_p=16$ sec at MLLW= 0 m).

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<td>HARBD</td>
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</table>

Figure 108 shows comparison of the predicted wave heights near the dock along an NW-SE oriented transect in front of the dock. Shown are the pre-project and post-project results which were obtained with B2D model by using the corresponding geometries and bathymetries of both harbor configurations. The pre-project bathymetry harbor structures were extracted from the HARBD grid. These were augmented with the bathymetry outside harbor, covering the reef and shorelines, which were common to both geometries. The intent of this modeling was to provide wave estimates as close to the dock as possible and to show the difference in the wave energy...
between the pre- and post-projects near the dock. Results plotted along a line indicate that comparatively much greater wave energy reaches the dock area after the removal of exterior spur and shortening of the interior spur which used to be connected to the east breakwater.

**Figure 108.** Comparison of predicted wave height for pre-and post-modification projects along a transect near the dock.
5 Summary and Conclusions

Until the implementation of federal modifications at Kikiaola Light Draft Harbor, the entrance had a 150 ft (46 m) external spur connected to the east breakwater (see Figure 2). This external spur was oriented in the southwest-northeast direction, protruding halfway into what is now the existing entrance channel. After the removal of this spur as part of federal modifications, boat owners have reported comparatively higher waves at the boat ramp and dock, and increased waves in the access and entrance channels. This increase in wave energy inside the harbor, according to boat owners, negatively impacts on and off loading at the boat ramp and dock.

This study was developed to address these user concerns by evaluating several infrastructure modifications proposed to improve wave conditions in the interior and exterior of Kikiaola Harbor. Estimate of waves at KLDH are developed using the latest bathymetric surveys, numerical models, and field-monitoring data. The effects of proposed structural alternatives on waves are evaluated and quantified at the entrance, access channels, and inner harbor.

CMS-Wave model was used in this study to develop incident-wave input conditions for B2D model. Several improvements to CMS-Wave were necessary to provide these inputs as well as to address the project’s other needs. Additional R&D developments to enhance model’s predictive capabilities were funded by the CIRP. The advances included a) simulations for tropical storms and non-storm waves in full-plane mode with parent-child grid capability of CMS-Wave; b) modeling of a dual-peaked wave condition (combined seas and swells from different directions); c) developing two-dimensional wave spectra at the offshore boundary of B2D; d) validation of model with the NOAA buoys and CDIP gauges near the project site; e) development of pre- and post-processing analysis codes for improving model setup; f) providing wave parameters (height, period, direction) and wave spectra as B2D input conditions; and g) developing a number of Fortran and Matlab utilities to facilitate coupling of two wave models.
Due to the previously noted differences between HARBD (used in the previous federal modifications study) and B2D model grids and their theoretical and numerical formulations, a one-to-one direct comparison between these two different classes of wave models is not possible. However, in order to make a general comparison between the present study and federal modifications study results, estimates of wave heights by B2D model for Alt-5 were extracted at the basin locations used in the HARBD modeling. The wave heights calculated inside the harbor at the interior basins by B2D are larger than wave heights calculated by HARBD. At the exterior basins, the opposite occurs, and wave heights calculated by HARBD are greater than those from B2D. The largest difference between the two model-wave height predictions is for incident waves from SE direction.

In addition, pre-project and post-project results were obtained with the B2D model by using the corresponding geometries and bathymetries of both harbor configurations. The intent of this modeling was to provide wave estimates as close to the dock as possible and to show the difference in the wave energy between the pre- and post-projects at the dock. Results indicate that comparatively, much greater wave energy reaches the dock area after the removal of exterior spur and shortening of the interior spur which used to be connected to the east breakwater. The results of the study are summarized below, though they should not be extrapolated beyond the scope of the present study parameters and harbor configurations investigated. Future decisions based on these results for other alternatives should be verified and approved by POH.

The alternatives have been ranked based on calculated average wave-height statistics at the a) ten most interior output stations; b) five exterior output stations; and c) all 15 output stations. Alt-1, Alt-7, and Alt-5 are the top-three ranked alternatives with the smallest average wave height at the ten interior basins. Based on the five exterior basins, the top-ranked three alternatives are Alt-7, Alt-5, and Alt-3a for SE and S directions, and for SW incident-wave direction Alt-7, Alt-3a, and Alt-5 top the list. Using all 15 basins, the top-ranking three alternatives for SE and S directions are Alt-7, Alt-5, and Alt-1, and while Alt-7, Alt-3a, and Alt-5 emerge as top three for the SW incident wave direction. Generally, incident waves coming from SW and S penetrated into the interior of harbor, most affecting the inner harbor area. The top-four ranked alternatives offering the highest reduction of wave energy inside the harbor have potential as a long-term solution to improving the conditions inside the existing harbor.
Aside from the extent of wave-height reduction, there are other factors that must be taken into consideration in future investigation. For example, Alt-1 (sheetpile structure near the dock) offers the largest wave-height reduction in interior harbor that includes the boat dock. However, implementation of this alternative has some challenges and consequences. Three concerns which are worthy to mention are a potential increase in wave action at the eastward end of the access channel, less maneuvering space in the boat dock, and reduced mooring area. These concerns may arise because Alt-1, a sheet pile structure, can be expected to reflect nearly 100% of wave energy towards the access channel, mooring area, and east breakwater. Depending on characteristics of superposition of incident and reflected waves, wave height in the access channel could increase or decrease. Boats presently stage off the dock during periods of peak harbor use, while other boats are waiting for access to the dock to load-unload passengers and goods. Alt-1 would reduce the maneuvering area available to boats at the dock, and in the areas adjacent to it, including the waiting area to the south. In addition, this alternative does not result in any wave reduction within the entrance channel.

Alt-7 (offshore breakwater) was a consistently high-ranked alternative for wave reduction both in the entrance channel and at the interior harbor. This alternative could have implications to channel shoaling, navigation, and operations if implemented. It would also involve significantly increasing the footprint of the project area, which would require enhanced environmental review and consideration. Finally, this alternative would be costly to construct given its size and the need to build it from an ocean-based platform such as a barge.

Alt-5 (Return Exterior East Breakwater Spur to Previous Configuration) also ranked near the top for reduction both in the entrance channel and at the interior harbor. This alternative would protrude into the existing entrance channel, making it virtually unusable. The navigation channel would need to be realigned to allow for safe transit of vessels in/out of the harbor in order to make this a viable option.

Finally, Alt-3a (West and East Breakwater Spurs) performed well for wave reduction in the entrance channel. Placing spurs on both the east and west breakwaters addresses incoming waves from both the SE and SW directions, as well as the south to some degree. This alternative, however, did not rank high in reduction of waves at the dock. This alternative would
also increase the project footprint significantly, triggering both environmental and economic implications.

In conclusion, seven alternative structural modifications were considered in order to address user concerns about wave conditions at Kikiaola Light Draft Harbor. Each of the alternatives that most efficiently address the current problems will require further consideration in order to fully evaluate the trade-offs between the improvements and the implications that would result from their implementation.
References


Nwogu, O. and Z. Demirbilek. 2010. Infra-gravity wave motions and runup over shallow fringing reefs, ASCE Journal of Waterway, Port, Coastal, and Ocean Engineering (http://dx.doi.org/10.1061/(ASCE)WW.1943-5460.0000050)


Appendix A: Hydrographic Field Survey

Surveys

In 2012, ERDC conducted multibeam and single beam hydrographic surveys and a Mean High Water survey at KLDH. Field operations of both bathymetric and topographic data at KLDH and the surrounding beaches were performed on 16 July 2012 through 22 July 2012. These operations utilized two vessels, a multibeam echo sounder (MBES), and single beam echo sounder. Collection began inside the harbor and continued outside. Full-coverage multibeam of the channel was completed. Survey lines were referenced with respect to the 2006 survey to the extent possible. All data were processed and compiled into a final digital format for delivery to the POH. The horizontal datum for these surveys was North American Datum of 1983 (NAD83) HARN 1993, Hawaii State Plane Zone 4. The surveys were performed using Differential Global Positioning System (DGPS) and Real Time Kinematic (RTK) equipment. The vertical datum for these surveys was Mean Lower Low Water (MLLW) at the 1983-2001 tidal epoch referenced to a Local Tidal Datum. The horizontal and vertical unit of measure used was U.S. Survey Feet. Details follow.

Survey equipment

A 26 ft SeaArk vessel with twin 150 HP outboard engines was used for multibeam collection. A 17 ft RIB survey vessel was used for the single beam data collection. The integration of positioning equipment and specialized computer software permitted real-time navigation capabilities. The SeaArk survey vessel was navigated and positioned by use of an Applanix POSMV 320, a high-performance GPS/IMU with radio beacon DGPS and RTK capability with centimeter accuracy. A Trimble R8 with Trimble Access was used to navigate and collect the singlebeam data.

A GeoAcoustics GeoSwath Plus 250kHz system was used to collect bathymetric data in shallow (less than 100 ft) water, which offers efficient simultaneous swath bathymetry and side scan seabed mapping with accuracies exceeding the present standards for hydrographic surveys. The applied phase measuring bathymetric sonar technology provides data coverage of up to 12 times the water depth, giving unsurpassed survey efficiency in shallow-water environments.
The Sonarmite echo sounder and Trimble R8 with Trimble Access Software with digital recording capabilities, and a 200 kHz 10 deg transducer mounted on and over the stern mount were used for the single-beam data collection. For heave compensating, an Applanix POSMV 320 measured the heave, pitch, and roll of the survey vessel for the correction of the heave. For speed of sound velocity calibration, an Odom Digibar Pro was used to measure the speed of sound in the water column for calibration of the echo sounder. The single beam conventional bar check was a round disk lowered below the transducer at 5 ft (1.5 m) intervals to check the calibration of the echo sounder. This was also accomplished with the collecting RTK data on the bottom in the harbor to verify the soundings and elevations.

Horizontal position data and the echo sounder data were sent to an onboard computer via serial and network ports. The data were processed by HYPACK. The HYPACK program provided navigation, data collection, editing, and plotting capabilities. For auto-gauge reading, an In-Situ, Inc LevelTroll 700 Data Recorder, with a 15 psi vented transducer, was set to take 60 readings a minute, averaging the 60 readings and recording one reading every 10 minutes. This enabled the survey to catch the rise-and-fall of the tide at accuracies of one-tenth of a foot.

**Horizontal and vertical datum**

The North American Datum of 1983 (NAD83) HARN 1993, Hawaii State Plane Zone 4 was the horizontal datum for these surveys was. The surveys were performed with Differential Global Positioning System (DGPS) and Real Time Kinematic (RTK) equipment, and the differential correction signal broadcast from the Continuous Operating Reference Stations (CORS) was used to provide DGPS positions. RTK corrections were received from a base station occupied at Gauge 3 (called here KIKI3). The vertical datum for these surveys was Mean Lower Low Water (MLLW) at the 1983-2001 tidal epoch referenced to a Local Tidal Datum. Staff gauges and an In-Situ, Inc LevelTroll 700 Data Recorder were used to provide backup water surface elevations for the RTK surveys. All staff gauges and the data recorder were set and/or calibrated from monuments provided by POH. All monument elevations were verified by a second monument. The horizontal and vertical unit of measure used was U.S. Survey Feet.
Survey operations

Prior to the start of the hydrographic survey, coordinates for each line to be surveyed were pre-computed and used to create line files in HYPACK. These line files were then uploaded to the survey vessel computer and hand-held data collector, and were used to navigate the survey vessel along each line. Horizontal position checks were made at the start of each survey area. Echo sounder calibration checks were made at the start and end of each workday.

A Hydrographic Survey Log was prepared for each day’s survey operation. This log was used to record the progress of the survey. Items recorded in this log included: calibration check data; line numbers run; time of survey of each line; staff gage readings; computer data file identification; and a description of conditions that may affect the progress or quality of the survey.

After the survey was completed, the data files were checked and edited. The editing process was performed by use of HYPACK and Trimble Business Center software. The purpose of the editing process is to remove extraneous and erroneous data typical of hydrographic surveys such as spikes in the horizontal position and depth-measuring systems. After editing, the survey data points were thinned to the appropriate scale. After editing and thinning, ASCII text files were created. Figure A-1 shows the hydrographic surveys of the entrance and harbor.

Calibration procedures

The GPS receiver and the DGPS Beacon receiver (horizontal positioning system) were checked at the start of each survey area. These checks were made by placing the GPS antenna over a control station provided by POH. The coordinate values being displayed were then compared to the known coordinate values for the control station. The echo sounder was calibrated at the start and end of each workday by the use of a speed of sound probe (Odom DigiBar). An Applanix POSMV 320 was used for roll, pitch, yaw, and heave compensation. It ran an internal program to adjust mounting angles and to log raw observables at 100 Hz for post-processing.

Tide boards and the data recorder were set from monuments and checked to a second monument both provided by POH. Tide boards were set in the survey area and removed at the completion of the survey, and the data re-
corder was set next to each tide gage. The data recorder measured one sec
water surface elevation for one min and averaged for that one min in the
10 min interval. The tide board readings were used to monitor the data re-
corder.

Figure A-1. Sample bathymetric survey (in ft) of harbor entrance and interior.
Appendix B: Field Data Collection

Wave and current data collection

The purpose of the field-data collection was to deploy, collect, retrieve, and process wave and current gauges to identify the wave energy and along shore sediment transport into Kikiaola Harbor. Data were obtained to support numerical modeling to aid in evaluation of perceived problems from a breakwater groin removal and to obtain site information to be used in a monitoring plan for the harbor. Five gauges, namely K1 to K5, were deployed for data collection as shown in Figure B-1. Table B-1 presents gauge coordinates in geographical coordinates (latitude/longitude) and state plane (HI Zone 4).

The gauge K2 could not be found during the gauge retrieval in Feb 2013, and consequently, it was considered lost. This gauge was later recovered by the Navy diving team. At that time, the numerical modeling study was completed and this report had already been written. Because no data from gauge K2 were available for analysis and numerical modeling study, this gauge is not included in the model-data comparison shown in this report. The field-data report may provide additional information about the gauge K2, and any data recorded by the gauge that could be recovered.

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Data collection of both wave and current data of inner and outer channel areas required multiple field operations that started with the deployment on 9 October 2012, and ended with the retrieval on 1 March 2013. These operations utilized a rented vessel and a USACE-owned vessel. Deployment began inside the harbor and continued outside until all five gauges were installed. Retrieval began inside the harbor, continued outside the following day, and ended with the search for Gauge K2 inside the harbor. All data were processed and compiled into a final digital format for delivery to the ERDC modelers and POH.

**Deployment operations**

Prior to the start of the deployment, coordinates for each gauge to be deployed were pre-located by collaboration between ERDC and POH. These coordinates were used to create a target file in HYPACK, which was uploaded to the survey vessel computer and used to navigate the survey ves-
sel to each location. A dive log was prepared for each day’s survey operation. This log was used to record the progress of the survey, and included the following information: time of day; location; time below surface; starting/ending air pressure in dive tanks; bottom time; and a description of conditions that may affect the progress or quality of the dive. After the deployment was completed, the log files from the Nortek deployment software were compiled and entered into a Google Earth *.kmz file along with pictures of instruments.

**Calibration procedures**

The compass calibration was used to adjust for magnetic materials that may be present in deployment frame. As a consequence, calibration of the compass was applied after the AWAC has been mounted in its frame. The AWAC was assembled and put in the frame with the battery canister, and the extra weight needed to create a stable frame. It was then connected to the computer and assigned deployment variables. The compass calibration option was started. The assembly was then rotated 360 degrees horizontally (AWAC + frame, including battery canister and extra weight). Once completed, the values were automatically updated in the unit. The same procedure was used for the Aquadopp’s.

**Wave and current gage equipment**

In principle, the single-point current meters used in this study (AWAC and Aquadopp) are equipped with acoustic transducers that transmit short bursts of sound into the water. The sound propagates along narrow acoustic beams and is scattered by small particles or zooplankton suspended in the water. The echo that reaches the transducers is analyzed for change in frequency (Doppler shift) and water velocity is calculated along each of the acoustic beams. By combining the velocities with the exact beam geometry, either 2D (2 beams) or 3D (3 beams) velocity is calculated and recorded on the internal recorder or reported online by data transmission. In addition to velocity measurements, the current meters may contain compass and tilt sensors making it possible to measure the orientation of the instruments. The AWAC is a current profiler and a wave directional system in one unit. It can measure the current speed and direction in 1 m thick layers from the bottom to the surface. Waves of all varieties are measurable; this includes long waves, storm waves, short wind waves, or transient waves generated by local ship traffic.
Wave and current recorders at K2, K3, and K5 were AWACs. The AWAC (Acoustic Wave And Current; http://www.nortek-as.com) recorder, manufactured by Nortek AS of Norway, uses acoustic technology to measure the waves and currents. The instrument is mounted on the sea bed and uses two separate acoustic systems. The first is an inverted echo sounder which uses the travel time of a narrow-beam acoustic pulse returning from the surface to monitor the surface elevation. The second acoustic system uses acoustic Doppler techniques to measure the water velocity at different levels in the water column, including that due to the wave motion. The instrument, a plastic cylinder 250 mm high and 250 mm in diameter, has three acoustic transducers mounted on the top at 120° spacing and angled at 20° from the vertical. These transducers send out acoustic pulses which are reflected from particles and minute bubbles in the water column. By using the travel time of the reflected sound and determining the shift in frequency due to the Doppler effect, the current speed and direction can be determined in a series of depth layers through the water column. Combining the current measurements with the surface elevation (AST – Acoustic Surface Track) the directional wave spectrum can be determined. The instrument also has a pressure sensor which can be used if there is not a clear return from the surface, for example due to aeration caused by wave breaking. A flux-gate compass and tilt sensors recorded the instrument orientation and provided a reference frame.

The AWAC recorder was mounted on a stainless steel frame which fitted on stainless pipe driven into the sea bed. The accuracy of the velocity measurements was quoted by the manufacturer as “1% of the measured value ±0.5 cm/sec.” The acoustic surface track has a resolution of 1 mm. The pressure sensor has an accuracy of 0.125 m and a resolution of 0.0025 m. The accuracy of the instrument compass is quoted as 2° with a resolution of 0.1°. A residual magnetism in the mounting frames or batteries will lead to deviations from magnetic north in the compass readings. Therefore, at each deployment, a compass calibration was done utilizing the Nortek software to calibrate the instrument orientation.

The 2 MHz Aquadopp wave and current recorders were used for field measurements at K1, K2, and K4. The Aquadopp 2Mhz recorder, manufactured by Nortek AS of Norway, uses acoustic technology to measure the waves and currents. This acoustic instrument is mounted on the sea bed. The first is an inverted echo sounder which uses the travel time of a narrow-beam acoustic pulse returning from the surface to monitor the surface
The system uses acoustic Doppler techniques to measure the water velocity at different levels in the water column, including that due to the wave motion. The Aquadopp Profiler provides quality data using the industry-standard autocovariance method to assure accurate and unbiased data, and it retains the classical advantages of acoustic Doppler systems including insensitivity to biofouling and no moving or protruding parts.

The Aquadopp Profiler is a tool for shallow water measurements on time scales larger than 1 sec. It can give speed and direction in 128 different layers of the water column. The system electronics integrate Doppler velocity with temperature, pressure, tilt, and compass sensors, and has built-in solid state recorder and batteries.

A 26 ft SeaArk vessel with twin 150 HP outboard engines was used for deployment. A rented 45 ft RIB was utilized for retrieval. For navigation and positioning system, the integration of positioning equipment and specialized computer software permitted real-time navigation capabilities. The SeaArk survey vessel was navigated and positioned by use of an Applanix POSMV 320, a high-performance GPS/IMU with radio beacon DGPS capability to achieve sub-meter accuracy. Computer data processing called HYPACK program provided navigation, target data collection, editing, and plotting capabilities.

**Wave and current data analysis**

Nortek AWACs have four beams: one vertical and three slanted at 25°. The vertical beam is an AST signal that samples at 4 Hz, twice that of the oblique beams, and provides measurements of surface displacement. A spectral reconstruction technique was developed that uses a combination of wave analysis methods. The lower frequency end of the spectrum uses iterative maximum likelihood method (IMLE) processing of the orbital currents computed from the three oblique beams and the AST (SUV method: S=surface track, UV= horizontal orthogonal current), which is treated as a point measurement. The upper end of the spectrum uses IMLE processing with the AST and individual radial-velocity beams as a spatial array (array method). If the AST signal fails quality checks the pressure (P) signal is substituted for the AST (PUV method). Waves breaking over the gauge can induce bubbles into the water column that interfere with the AST, flagging a QC failure.
Nortek Aquadop sensors have a pressure sensor, three oblique beams, and collect PUV data. IMLE processing of the PUV data was used for the directional wave spectra estimates.

Data processing details were as follows. The record length was 2304 for 2400 point records, and 1024 for 1024 point records. Sample rate was 2 Hz (1 Hz for k3). Ensemble size was 512 points (eight ensembles) for 2400 point records, and 256 points (five ensembles) for 1024 point records. Wave spectra were computed using 50% overlap (Welch Method). Frequency resolution was 0.01 Hz, and high frequency cutoff was the lesser of 0.5 Hz or when spectrum PRF was less than 0.01.

For each gauge, processed data were stored in the Plots, TS, Cur, and Spec directories. Plots had cur and spec sub-directories. TS (Time Series) had “ASCII” and “Misc” sub-directories, with “ASCII” sub-folder containing data for 2012 (Oct, Nov, Dec) and 2013 (Jan, Feb) in the respective subfolders. The subdirectory “Misc” had extra information on roll, pitch, azimuth, water temperature. The directories “Cur” and “Spec” include MATLAB files for current profiles and directional wave spectra. Figures B-2 and B-3 show sample plots of one- and two-dimensional wave spectra, respectively, at K5.

Figure B-2. Sample 1-D spectrum of AWAC Gauge K5 on 10 Dec 2012 at 17:35 UTC.
Figure B-3. Sample 2-D spectrum of AWAC Gauge K5 on 10 Dec 2012 at 17:35 UTC.
Appendix C: Description of CMS

The CMS-Wave module of the Coastal Modeling System (CMS) was used for the numerical modeling estimates of waves at KLDH. A brief description of the CMS is provided here for completeness.

As shown in Figure C-1, the CMS is an integrated suite of numerical models for waves, flows, sediment transport, and morphology change in coastal areas. This modeling system includes representation of relevant nearshore processes for practical applications of navigation channel performance, and sediment management at coastal inlets and adjacent beaches. The development and enhancement of CMS capabilities continues to evolve as a research and engineering tool for desktop computers.

CMS uses the Surface-water Modeling System (SMS) interface for grid generation and model setup, as well as plotting and post-processing. The Verification and Validation (V&V) Report 1 (Demirbilek and Rosati 2011) and Report 2 (Lin et al. 2011) have detailed information about the CMS-Wave features, and evaluation of model’s performance skills in a variety of applications. The Report 3 and Report 4 by Sanchez et al. (2011a and 2011b) describe coupling of wave-flow models, hydrodynamic and sediment transport, and morphology change aspects of CMS-Flow. The performance of the CMS for a number of applications is summarized in Report 1 and details are described in the three companion V&V Reports 2, 3, and 4.
The CMS-Wave, a spectral wave model, is used in this study given the large extent of modeling domain over which wave estimates were required. Details of the wind-wave modeling are described in Chapter 3 of this report. The main wave processes included in the CMS-Wave are wind-wave generation and growth, diffraction, reflection, dissipation due to bottom friction, white-capping and breaking, wave-current interaction, wave runup, wave setup, and wave transmission through structures. The height and direction of waves approaching the KLDH entrance channel change due to wave shoaling, refraction, diffraction, reflection, and breaking. Waves propagating through the entrance interact with bathymetry, surrounding land features, currents, and coastal structures. These changes to waves affect waves propagating into the access channel and interior of harbor and bed shear stresses and sediment mobility in these areas.

CMS-Wave model solves the steady-state wave-action balance equation on a non-uniform Cartesian grid to simulate steady-state spectral transformation of directional random waves at and around the KLDH. CMS-Wave is designed to simulate wave processes with ambient currents in navigation channels, coastal inlets, and harbors. The model can be used either in half-plane or full-plane mode for spectral-wave transformation.
The half-plane mode is default because in this mode CMS-Wave can run more efficiently as waves are transformed primarily from the seaward boundary toward shore. See Lin et al. (2011 and 2008) for features of the model and step-by-step instructions with examples for application of CMS-Wave to a variety of coastal inlets, ports, structures, and other navigation problems. Publications listed in the V&V reports and this report provide additional information about the CMS-Wave and its engineering applications. Additional information about CMS-Wave is available from the CIRP website: http://cirp.usace.army.mil/wiki/CMS-Wave

The flow model was not used in this study; brief information is provided. The CMS-Flow is a two-dimensional shallow-water wave model that can be used for hydrodynamic modeling (calculation of water level and current). Both the explicit and implicit versions of flow (circulation) model are available to provide estimates of water level and current given the tides, winds, and river flows as boundary conditions. CMS-Flow calculates hydrodynamic (depth-averaged circulation), sediment transport and morphology change, and salinity due to tides, winds, and waves.

The hydrodynamic model solves the conservative form of the shallow water equations that includes terms for the Coriolis force, wind stress, wave stress, bottom stress, vegetation flow drag, bottom friction, wave roller, and turbulent diffusion. Governing equations are solved using the finite volume method on a non-uniform Cartesian grid. Finite-volume methods are a class of discretization schemes, and this formulation is implemented in finite-difference for solving the governing equations of coastal wave, flow, and sediment transport models. See the V&V Reports 3 & 4 by Sanchez et al. (2011a and 2011b) for the preparation of flow model at coastal inlet applications. Additional information about CMS-Flow is available from the CIRP website: http://cirp.usace.army.mil/wiki/CMS-Flow

Although hydrodynamic, sediment transport, and morphology change modeling were not considered in this study, we note for future reference that there are three sediment transport models available in CMS-Flow: a sediment mass balance model, an equilibrium advection-diffusion model, and a non-equilibrium advection-diffusion model. Depth-averaged salinity transport is simulated with the standard advection-diffusion model, and includes evaporation and precipitation. The V&V Report 1, Report 3, and Report 4 describe the integrated wave-flow-sediment transport and
morphology change aspects of CMS-Flow. The performance of CMS-Flow is described for a number of applications in the V&V reports.
Appendix D: Description of BOUSS-2D

The Boussinesq wave model BOUSS-2D (or B2D) is an advanced modeling approach for nonlinear wave propagation nearshore (Nwogu and Demirbilek 2001). This technology was developed and implemented in the Surface-water Modeling System (SM) in 1990s through early 2000, and has since been used by Districts for navigation channels, inlets, harbors, coastal structures, moored vessels, floating breakwaters, and wave runup and overtopping on revetments, shorelines, and levees. Recent publications describe different applications of B2D model (Nwogu and Demirbilek 2010; Demirbilek et al. 2009, 2008, 2007a, 2007b, 2007c; 2005a, 2005b). Additional information about B2D is available from these and other related publications in the References section of this report.

Types of problems for B2D Application

The list below shows types of wave problems which can be simulated using Boussinesq wave models:

- Harbor/port/marina problems: harbor resonance, harbor and marina infrastructure modifications
- Generation of wave sub- and super-harmonics
- Wave dissipation over porous media
- Wave reflection and diffraction from structures, shorelines, and variable surfaces
- Wave-wave interactions in shallow-water
- Channel deepening/widening/realignment
- Wave-structure interactions: levees, flood walls, barriers, revetments, seawalls, groins, and breakwaters design and repair (coastal and inland)

  o Wave runup/overtopping

  o Structure loading (wave forces)

  o Structure freeboard requirements

  o Frictional dissipation (i.e., waves on vegetated surfaces)
Wave interaction with array of structure types

Embankment stability

Wave interaction with complex geometries of levees, navigation channels, and canals, ports/harbors, etc.

- Inundation mapping – overland propagation and runup
- Bore propagation through rivers and canals
- Transient waves (tsunamis, sneaker waves)
- Vessel-generated waves and ship wakes

- Vessel generated waves and effect on shorelines
- Vessel generated bed velocities and shear stresses
- Vessel interactions with other vessels and with locks and dams

A few example applications are shown at the end of Appendix D.

Background

B2D model was used for numerical modeling of wave estimates at KLDH. The POH study plan in Chapter 1 described the purpose of numerical modeling tasks, while the implementation details of the wave modeling tasks were provided in Chapter 4. Only a brief description of the B2D features is provided here for completeness, because details of model theory, numerics, and examples are available from the references listed.

Boussinesq models are essentially shallow-water models with extra dispersive and nonlinear terms. They excel under conditions of nonlinearity (large and/or long waves in shallow depths). Processes modeled well by Boussinesq models include nearshore wind-wave propagation, harbor resonance, nonlinear shoaling, runup and inundation, nearshore circulation, and tsunami. Because Navier-Stokes models are not practical for field-scale problems, Boussinesq models presently are the computational tools for calculating runup and overtopping of vertical or near-vertical walls or impulsive forces on structures. Boussinesq models can propagate vessel-generated waves if a source term is added for generation (i.e., moving pressure source or internal boundary). Boussinesq models are much bet-
ter at this than shallow-water models because they include both short- and long-waves, whereas SWWEs can only represent the long-wave component of the vessel-induced disturbances.

The B2D computes changes to waves caused by shoaling and refraction over variable bathymetry, reflection and diffraction from shorelines and structures, and nonlinear wave-current and wave-wave interactions. The internal Boussinesq equations defining the B2D do not contain adjustable parameters. Potential errors are introduced in numerical discretization of mathematical equations, imperfect boundary conditions, and physical processes that contain process-specific parameters, such as wave turbulence, dissipation, bottom friction, and boundary reflectivity. The B2D needs field data because it can simulate processes that cannot be properly scaled in physical models, and consequently, these B2D model parameters are best calibrated with field data since they may not be estimated well by physical models (i.e., laboratory experiments) due to scaling effects. In the absence of field data, physical model data (if available) could be used in B2D for validation and calibration of boundary conditions, material parameters, and numerical algorithms. Generally, errors in the nearshore wave estimates come from two sources: input to the model and the model itself, including errors in the incident-wave conditions, bathymetry, and boundary specifications. The largest errors are associated with the specification of incident-wave parameters and simplification of wave breaking, dissipation processes, and contamination from model boundaries.

The B2D provides spatially and temporally varying wave, current, and water-level parameter estimates for engineering problems. Estimates include significant wave height, peak period and direction, wave spectrum, time-series of surface elevation, velocity and pressure, and wave-induced circulation. B2D model interface is operational in the SMS for grid generation and visualization of model results. The custom-built SMS interface of B2D allows users to setup and run the model in an intuitive manner with built-in safeguards (Demirbilek et al. 2005a and 2005b). The B2D can be run on PCs, workstations, and super-computers.

The B2D consists of a set of comprehensive numerical modeling systems based on a time-domain solution of Boussinesq-type equations for simulating waves (wind-waves and vessel-generated waves) and their propagation in coastal regions, harbors, and waterways. The B2D represents most of wave phenomena of interest in the nearshore zone for
navigation projects, inlets, harbors, levees, structures, reefs, wetlands, ship-wakes, wave-ship-bank interactions, and wave-current-structure interactions. The B2D-based engineering analysis systems may be used in navigation infrastructure design with a risk-based probabilistic design approach to evaluate life-cycle cost of alternatives, operation, and maintenance of coupled systems in deciding the benefit or negative consequences of structures in projects. The B2D has capability of replacing considerably more expensive physical models with flexibility and generality for extension to sediment transport and morphology change, channel infilling, and water-quality issues. The Corps O&M budget for dredging navigation channels and expansion of ports/harbor economic capacity will continue to increase with calls for deepening and widening of channels and harbors to accommodate future fleets having larger vessels and drafts, and larger and faster boats. Vessel-to-vessel and vessel-to-bank interactions and risk of accidents will also increase with these demands. Aging and natural deterioration of navigation structures increases vessel transit and maneuvering risks along the high-traffic shipping routes, channels, and ports.

Numerical models that solve Boussinesq-type water wave evolution equations are commonly used to investigate surface-wave propagation and transformation in coastal regions. Most of the models use finite difference schemes to discretize the equations over uniformly spaced rectangular grids (Nwogu and Demirbilek 2001). The popularity of finite difference schemes is largely based on their simplicity and ease of implementation. However, the use of structured grids can severely restrict the potential application of such models to complex boundary problems such as coastal flooding over complex topography, wave propagation in curved channels, wave interaction with coastal structures of arbitrary shape, and wave agitation in harbors of arbitrary shape. Because unstructured grids provide users the flexibility of modeling complex geometries, and the grid resolution can be refined where needed such as near structures or in shallow regions, it was therefore highly desirable to develop an unstructured-grid version of the finite-difference B2D model used in civil and military works. The development of an unstructured grid finite-volume version of B2D has been completed. This new model is being tested on super-computers and its interface in SMS is under development.

The B2D is designed to simulate wave processes with ambient currents at coastal inlets and in navigation channels. The model can be used for

http://www.xmswiki.com/xms/SMS:BOUSS-2D

In this study, the coupled B2D model was used for wave modeling nearshore to evaluate merits of eight proposed alternatives to improve conditions inside the existing harbor. Details of B2D modeling are described in Chapter 4 of this report.

Example applications

The images in Figures D-1 through D-10 show some recent examples of B2D model applications. See References for other types of applications.

**Figure D-1: BOUSS-2D calculated wave-induced current field for Pillar Point Harbor, CA.**
Figure D-2: Calculated wave fields by (a) BOUSS-2D, and (b) CMS-Wave at Point Judith Harbor, RI for incident wave from SSE.

Figure D-3. Wave propagation inside a bay.
Figure D-4. Wave field around a detached breakwater.

Figure D-5. Waves, wave-induced current, and circulation near a reflective jetty of an inlet.
Figure D-6. Wave-induced current field developed between two groins placed on a beach.

Figure D-7. Multiple ships moving (in transit) in a harbor.
Figure D-8. BOUSS-2D domain for the Oyster Pt, CA entrance and east marina.

Figure D-9. BOUSS-2D grid for changes to entrance of Diversey Harbor, MI.
Figure D-10. BOUSS-2D runup/overtopping toolbox in SMS for a fringing reef application.
**Kikiaola Light Draft Harbor Monitoring Plan**  
**Part 2: Numerical Wave Modeling for Evaluation of Structural Alternatives**

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**Availability Statement:** Approved for public release; distribution is unlimited.

**Abstract:**  
This report documents field data collection and monitoring, and numerical wave modeling for evaluation of structural alternatives for modification of Kikiaola Light Draft Harbor (KLDH), located on the southwest coast of Kauai Island, Hawaii. The entrance and interior of KLDH is largely affected by waves transforming over a wide shallow reef. Unfavorable navigation conditions for boats using this harbor are caused by high-energy northwest swells, southern swells, and storms. The harbor entrance is flanked by two breakwaters that shelter the harbor interior from waves, current, and sedimentation. A six-month field data collection study provided wave data for calibration of numerical models. Surveys of bathymetry and structures were conducted to obtain data for numerical modeling. The hydrographic survey data collected was used in the numerical modeling study to evaluate eight structural alternatives. CMS-Wave, a spectral wave model, was used to transform deepwater waves measured by buoys to the project site. BOUSS-2D (B2D), a Boussinesq-type wave model, was used to investigate alternatives representing structural modifications inside and outside of the harbor. These modifications were breakwaters or spurs. Benefits and consequences of modifications were evaluated for improving navigation safety and utilization of the existing harbor. Potential impacts of each alternative on navigation in the entrance channel, access channel, and harbor basin were examined.

**Subject Terms:**  
Numerical modeling, evaluation, hydrodynamics, wave reduction, Kikiaola Harbor, HI