HDR | ONE COMPANY *Many Solutions*℠

Memo

To:	Carl Seibe, PE		
From:	Ronny McPherson, PE	Project:	Kodiak Airport Shore Protection
CC:	Dan Heilman, PE Philip Blackmar		
Date:	March 17, 2014	Job No:	187899

RE: Extension of Kodiak Airport Runways into St. Paul Harbor

This technical memorandum documents the 70% preliminary design of shoreline protection for two runway extensions at Kodiak Airport. The runways currently protrude into St. Paul Harbor and are protected by large armor stone. Due to the restrictive terrain surrounding the runways, the extensions will be at the eastern end of each runway, meaning the runways will be extended further into the harbor. This approach will require upgrades to and/or replacement of the existing shoreline protection to mitigate damage from erosion as well as reduce wave overtopping which can lead to unsafe conditions for airport activity. In addition, as the ends of the runways are moved closer to deeper water, larger hydrodynamic forces can occur which may require more stable protection (such as through placement of larger armor stone) than the current shoreline protection.

Project Location

Kodiak Airport sits along the northwest side of Chiniak Bay on Kodiak Island, Alaska approximately 4 miles southwest of the city of Kodiak and 250 miles southwest of Anchorage. Kodiak Airport will be expanding Runways 25 and 36 into St. Paul Harbor. Figure 1 shows a general location map of the project site. Figure 2 shows the location of the runway extensions.

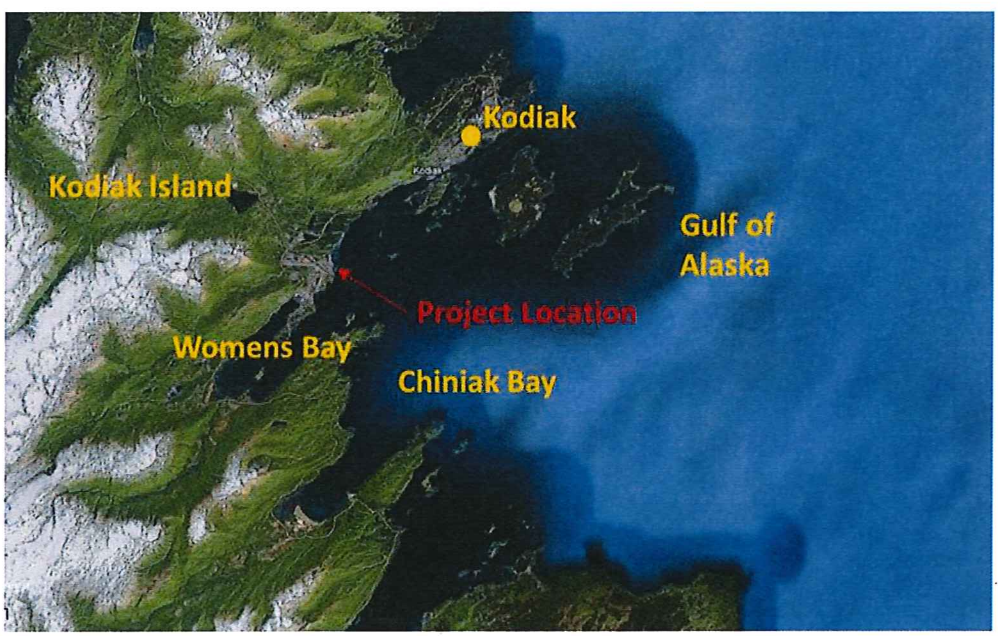


Figure 1. Project location map.

HDR Engineering, Inc.
Texas Registered Engineering Firm F-754

555 N. Carancahua
Suite 1600
Corpus Christi, Texas 78401-0850

Phone (361) 696-3300
Fax (361) 696-3385
www.hdrinc.com

Figure 2. Locations of proposed runway extensions.

Metocean Conditions

The following section discusses the meteorological and oceanographic (metocean) conditions near the project site including bathymetry, water levels/tides, wind speeds, and wave conditions.

Bathymetry

The water in Chiniak Bay ranges from 75 feet to more than 500 feet deep. Based on National Oceanic and Atmospheric Administration (NOAA) navigational maps, the water near the airport is much shallower, ranging from 1 to 5 feet along the existing shoreline protection at low tide and 15 to 20 feet at a distance of approximately 1,000 feet from the shoreline at low tide.

Winds

NOAA operates a tide and weather gage, "Kodiak Island Station 9457292," approximately 1.5 miles southwest of the project location at the North Dock of the U.S. Coast Guard Station (Figure 3). The Kodiak Island station is in a sheltered area of Womens Bay surrounded by mountains to the south and west and a small peninsula of land to the east. The majority of the wind comes from the SW and the NE, effectively blowing nearly parallel to the shoreline and not directly onshore. The average wind speeds range from 2 to 8 m/s (4 to 18 mph) (Figure 4).

ASCE 7-10 was examined for Ultimate Wind Loads as 3-second gusts for events ranging from 5 to 500 years. Table 1 summarizes the results at the project location (ASCE 2010).

Offshore wind data were acquired through the USACE Wave Information Studies (WIS) for stations 81008 and 81009. The WIS stations are in the open ocean with no obstructions. Hindcast wind and wave data are available from 1981 to 2004 for both stations. Winds from the northwest are predominant. The average wind speeds range from 5 to 15 m/s (11 to 34 mph) (Figure 5 and Figure 6).

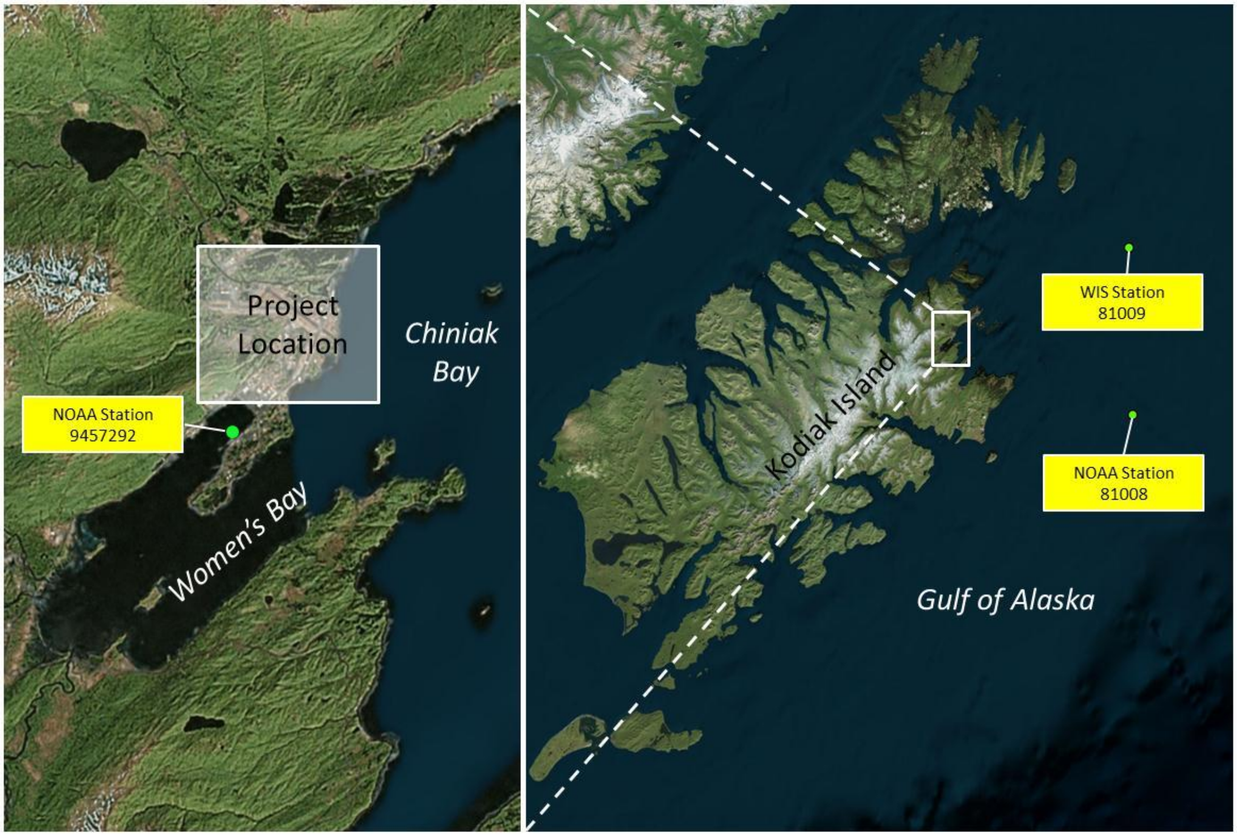


Figure 3. USACE WIS and NOAA Buoy Locations.

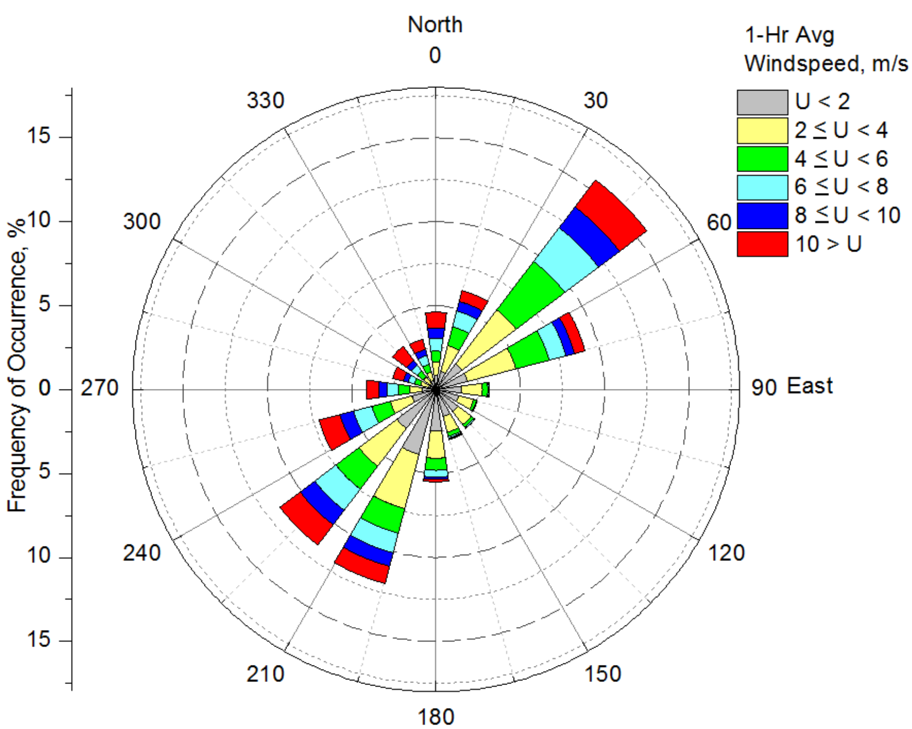


Figure 4. Kodiak Island NOAA Station: Wind Rose 2010-2012 (NOAA 2013).

Table 1. Wind Speeds (mph) from ASCE 7-10 (ASCE 2010).						
Duration	Return Period					
	5-Year	10-Year	25-Year	50-Year	100-Year	500-Year
3-Second	105	113	122	130	138	153
5-Minute	75.9	81.6	88.1	93.9	99.7	110.5
10-Minute	73	78.6	84.9	90.4	96	106.4
20-Minute	71.2	76.6	82.7	88.1	93.6	103.7

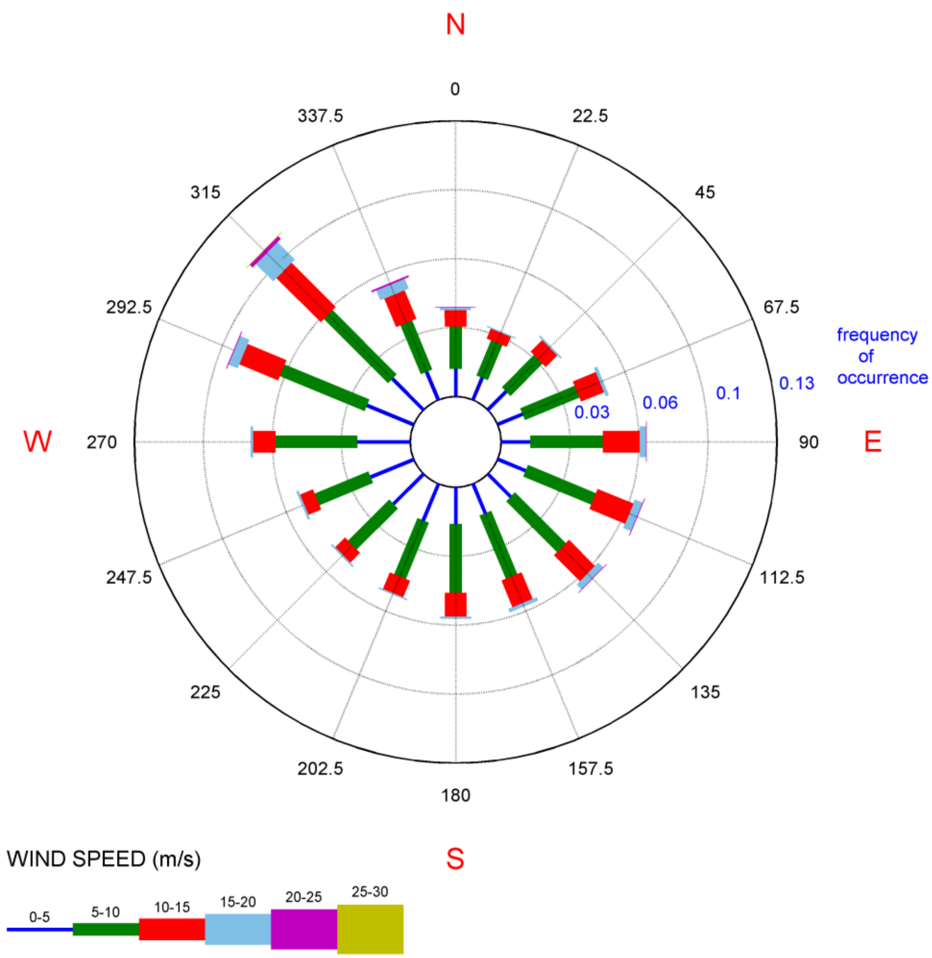


Figure 5. USACE WIS Station 81008: Wind Rose 1980-2011 (Tracy 2004).

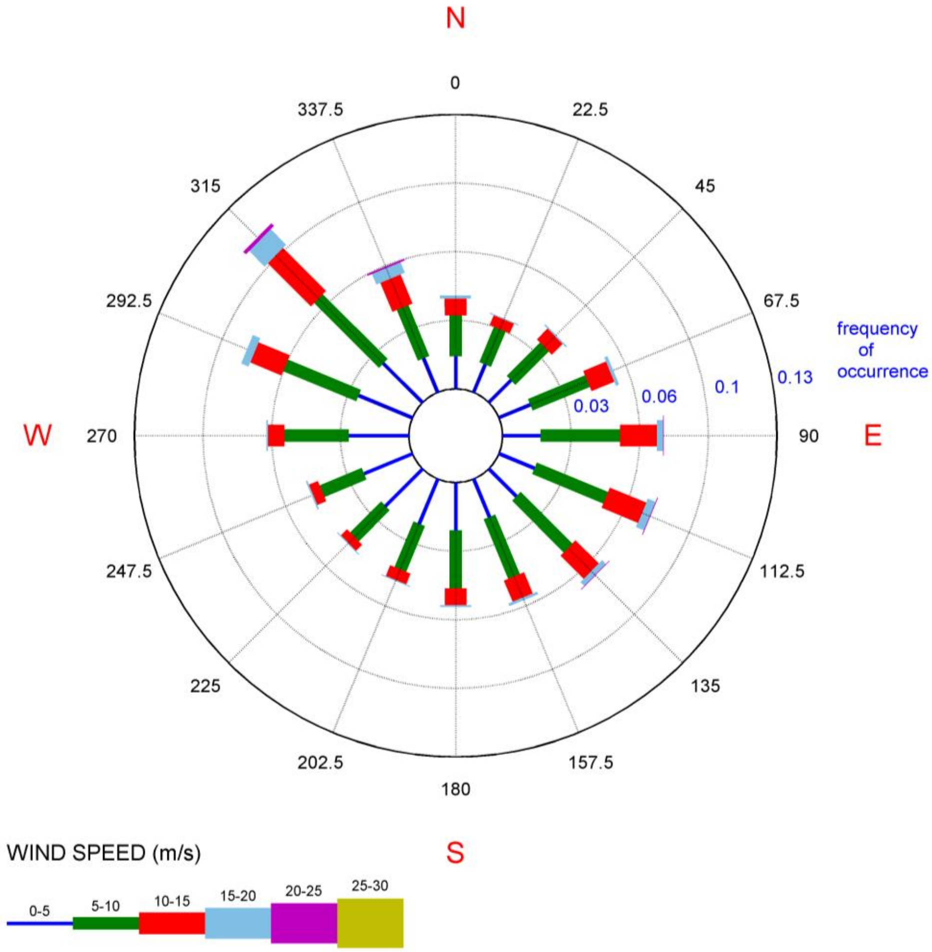


Figure 6. USACE WIS Station 81009: Wind Rose 1981-2004 (Tracy 2004).

Water Levels and Tides

Tidal datum relationships for the project area are reported for the Kodiak Island NOAA gauge. These are provided in Table 2. Mean Sea Level (MSL) is +5.25 ft NAVD[1] and the average tide range is 6.75 ft. Data collection occurred from 2010 to 2012. The Highest Astronomical Tide (HAT) is also provided which represents the highest predicted astronomical tide (meteorological effects not included) over at 19 year metonic cycle.

Table 2. Tidal Datums at the Kodiak Island NOAA Station (NOAA 2013)

Datum	ft, MLLW	ft, NAVD	Description
MHHW	8.77	9.53	Mean Higher High Water
MHW	7.87	8.63	Mean High Water
MSL	4.49	5.25	Mean Sea Level
MLW	1.12	1.88	Mean Low Water
MLLW	0.00	0.76	Mean Lower Low Water
HAT	11.47	12.23	Highest Astronomical Tide

[1] NAVD is an acronym for the North American Vertical Datum of 1988.

Extreme water levels can be inferred from statistics provided by NOAA. Figure 7 and Figure 8 show annual water level exceedance above MHHW and below MLLW, respectively. Based on Figure 7, the 100 year water level (or 1% chance of exceedance in a single year) is approximately +1.3 m (+4.3 ft) MHHW, which equates to +13.0 ft MLLW or +13.8 ft NAVD. The HAT, which is approximately +0.8 m (+2.7 ft) MHHW, has a return period of approximately 2 years (or approximately a 50% chance of occurring in a single year). This means that while on the basis of purely astronomical tides the HAT only occurs once every 19 years, in reality the HAT is met or exceeded more frequently due to meteorological factors such as wind.

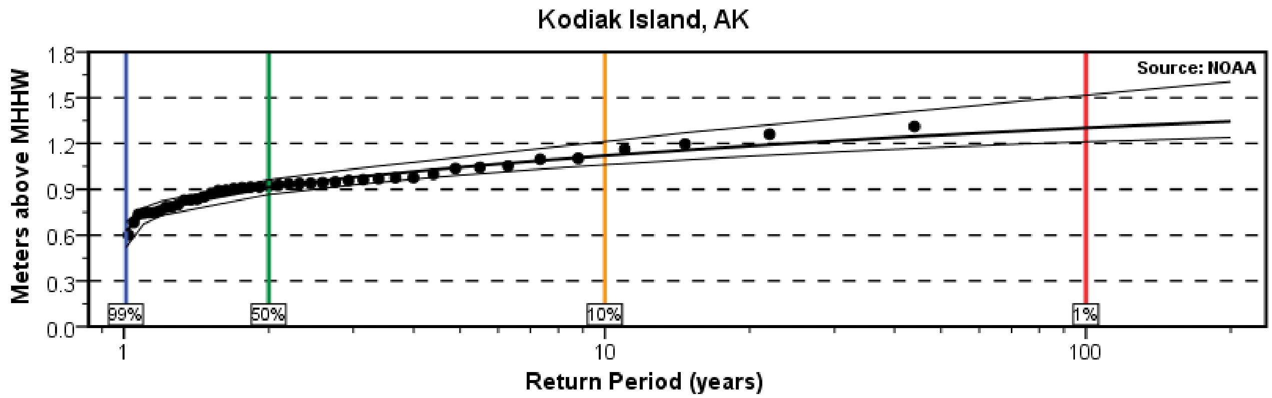


Figure 7. Kodiak Island NOAA Station: Annual water level exceedance above MHHW (NOAA 2013).

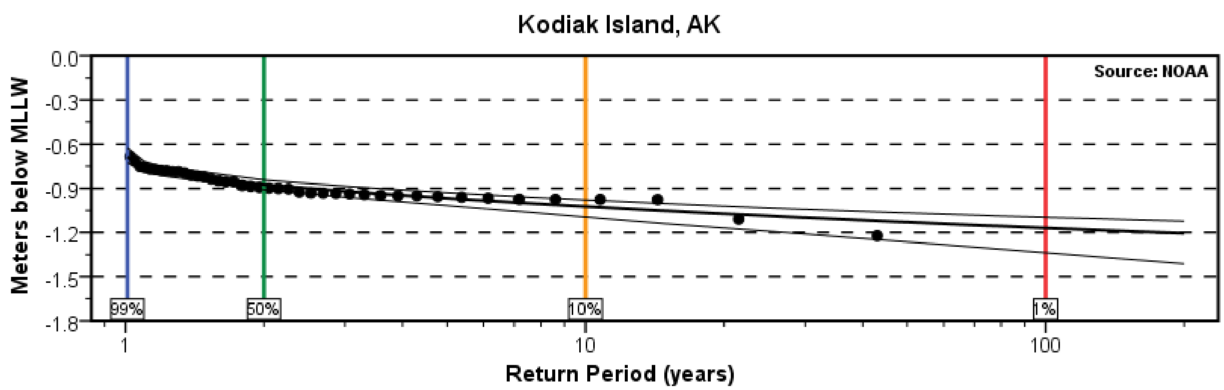


Figure 8. Kodiak Island NOAA Station: Annual water level exceedance below MLLW (NOAA 2013).

Wave Conditions

WIS Stations 81008 and 81009 provide offshore wave statistics. Based on the wave roses shown in Figure 9 and Figure 10, a majority of the waves come from the south and southeast with average waves heights of 1 to 3 m (3 to 10 ft). Extreme wave heights for the each WIS Station are provided in Figure 11 and Figure 12. Based on these figures, the 100 year wave event (or wave height having a 1% chance of exceedance in a single year) is approximately 10 m (33 ft). When reviewing historical storms, waves with heights of this magnitude generally have a peak period of approximately 15 seconds. An offshore wave height of 33 ft and peak period of 15 seconds was therefore applied to force the spectral wave model to determine wave heights near the project area (discussed in the numerical wave modeling section of this report).

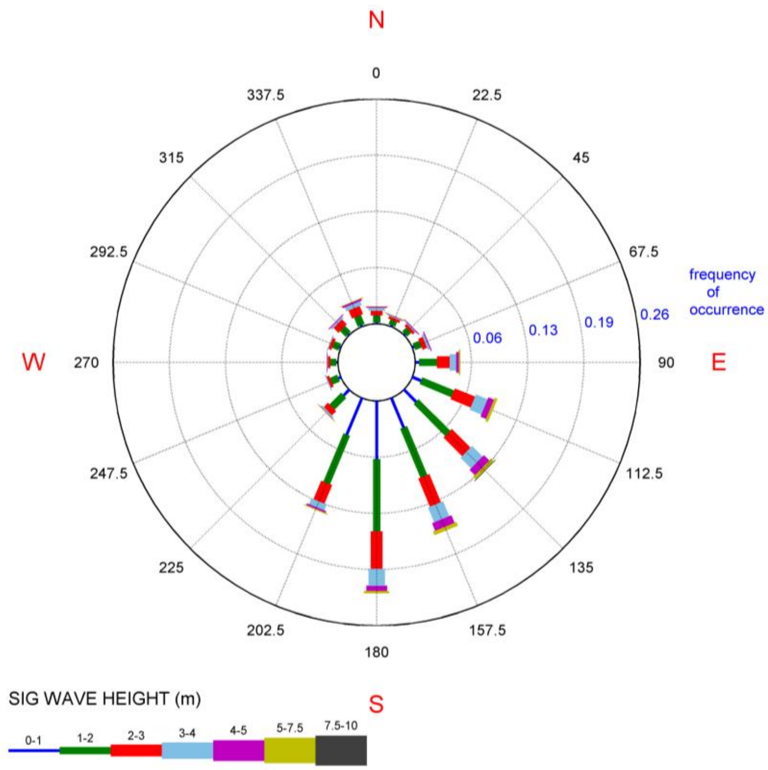


Figure 9. USACE WIS Station 81008: Wave Rose 1980-2011 (Tracy 2004).

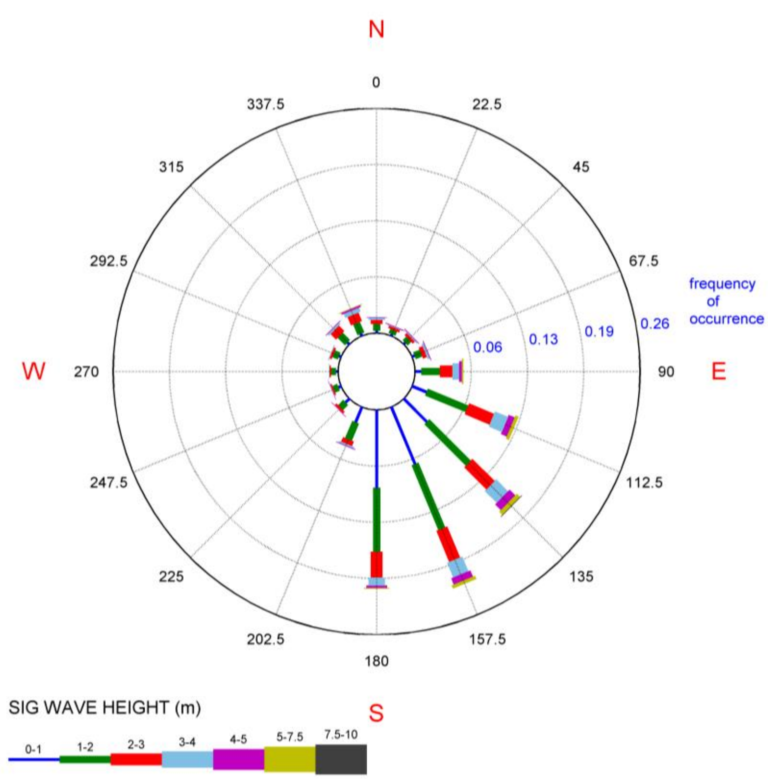
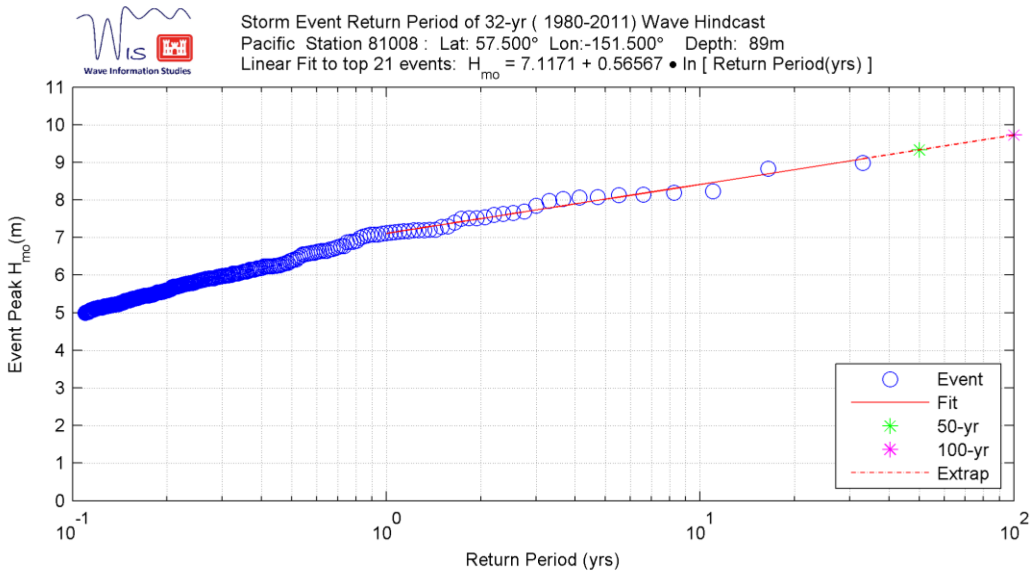


Figure 10. USACE WIS Station 81009: Wave Rose 1980-2011 (Tracy 2004).



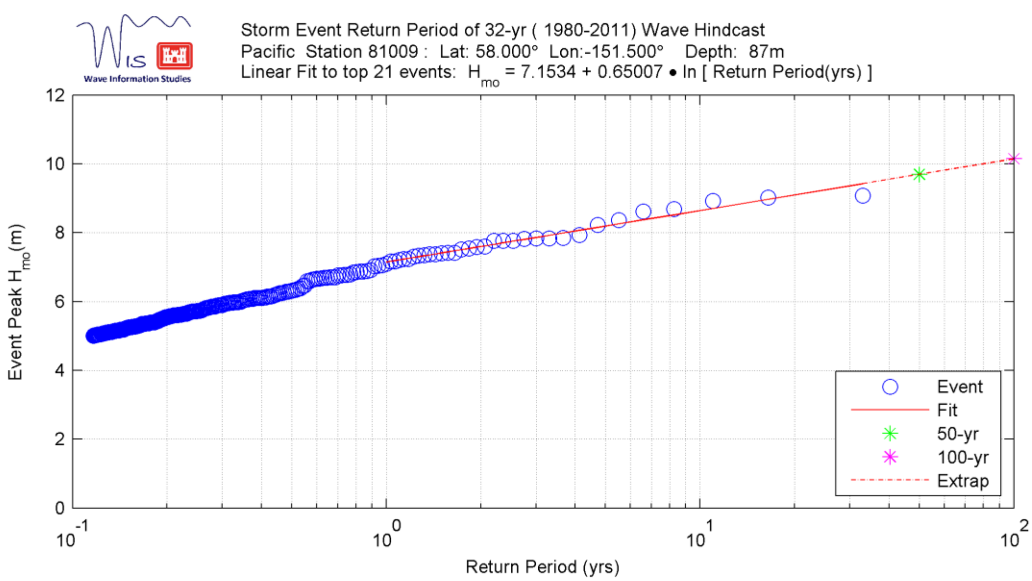
Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	1988/04/16 01:00	8.98	14.73	138.0	6	2002/11/23 12:00	8.13	13.38	134.0
2	1983/01/21 12:00	8.83	14.06	128.0	7	1980/01/21 00:00	8.08	12.90	148.0
3	1997/11/09 09:00	8.23	14.19	125.0	8	2004/01/27 12:00	8.06	13.24	95.0
4	1991/09/23 17:00	8.19	16.01	188.0	9	1984/11/01 03:00	8.03	13.21	117.0
5	1988/03/12 07:00	8.14	12.97	134.0	10	2000/12/20 05:00	7.98	14.56	146.0

An event is defined as any period when H_{mo} > 5.00m θ_{mean} is direction that waves are arriving from

ERDC US Army Engineer Research & Development Center   ST81008_v01

Figure 11. USACE WIS Station 81008: Extreme Wave Heights 1981-2004 (Tracy 2004).



Top 10 events based on Peak H_{mo}

Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}	Event	Date/Time(UTC)	H_{mo}	T_p	θ_{mean}
1	1988/04/16 03:00	9.08	14.91	139.0	6	1988/03/12 07:00	8.37	12.99	130.0
2	1983/01/21 13:00	9.02	14.25	128.0	7	1980/01/21 00:00	8.22	12.92	144.0
3	1997/11/09 09:00	8.92	14.25	121.0	8	2000/12/20 07:00	7.93	14.67	144.0
4	2002/11/23 13:00	8.69	13.61	132.0	9	1980/10/22 06:00	7.85	12.92	130.0
5	2004/01/27 12:00	8.62	13.39	92.0	10	2007/11/26 01:00	7.83	13.26	121.0

An event is defined as any period when H_{mo} > 5.00m θ_{mean} is direction that waves are arriving from

ERDC US Army Engineer Research & Development Center   ST81009_v01

Figure 12. USACE WIS Station 81009: Extreme Wave Heights 1981-2004 (Tracy 2004).

Preliminary Numerical Wave Modeling

This section briefly discusses the wave numerical modeling performed to support preliminary design of the shoreline protection.

Model Description

A nearshore wave model was created using the MIKE 21 Spectral Wave FM (MIKE 21 SW) software developed by the Danish Hydraulic Institute (DHI). This software was developed for the creation of spectral wind-wave models based on a flexible (unstructured) mesh. Models created with MIKE 21 SW can simulate wave growth, decay, and transformation of wind-generated waves and swell in offshore and coastal areas (DHI, 2008). MIKE 21 SW was applied to analyze wave propagation and transformation within the Gulf of Alaska and Chiniak Bay for the shoreline protection design.

Model Domain

The wave model includes all of the Women's Bay, Chiniak Bay, and the Alaskan Gulf up to the USACE WIS gauge (virtual buoy) locations. The flexible mesh contains 22,127 elements ranging in size from approximately 130 ft^2 to 21,300,000 ft^2. Figure 13 and Figure 14 show the flexible mesh of the entire domain and nearshore project area (enlarged), respectively. Bathymetry applied to develop the mesh was gathered from NOAA National Geophysical Data Center (NGDC) digital survey archive as well as NOAA Navigation Chart 16596 "Womens Bay."

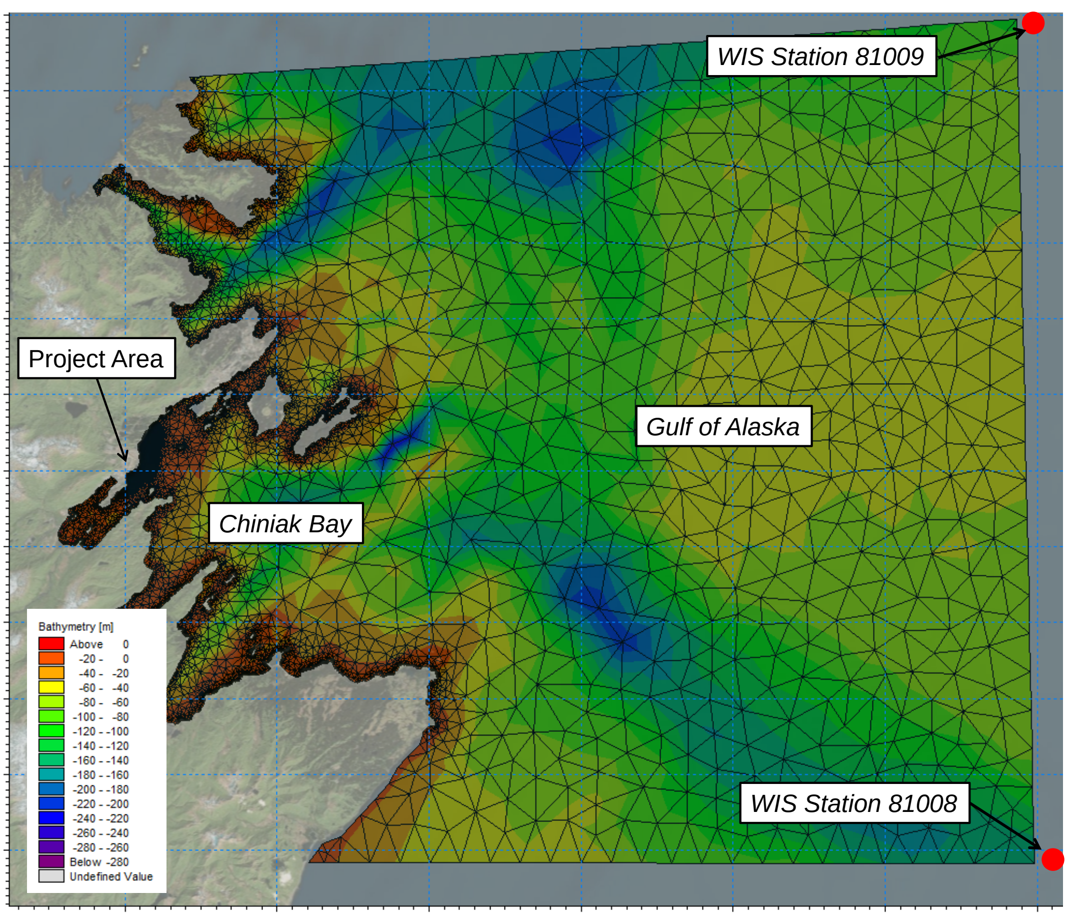


Figure 13. Spectral wave model flexible mesh (full domain).

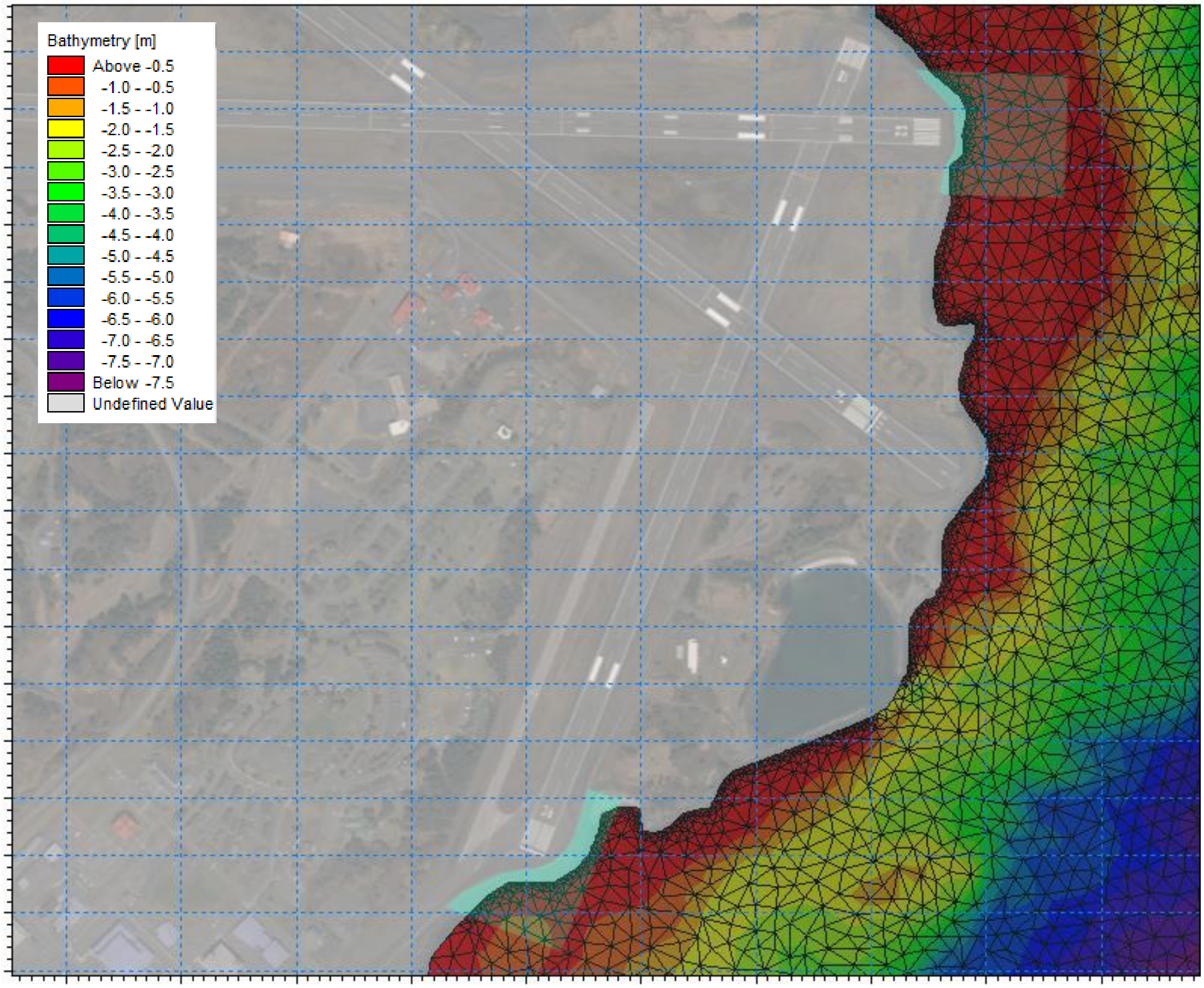


Figure 14. Spectral wave model flexible mesh (near project area).

Boundary Conditions

Water level and offshore wave direction were the only model parameters varied in the model runs. Table 3 provides a summary of the boundary conditions applied within the model. In total, 182 model runs were performed to assess the wave conditions associated with varying water levels and offshore wave directions.

Table 3. Summary of model boundary conditions			
Parameter	Min	Max	Increment
Water Surface Elevation	+0.2 m (+0.7 ft) NAVD	+12.0 m (+39.4 ft) NAVD	0.3 m (1.0 ft)
Wave Direction	15 Deg	150 Deg	15° (decreased to 5° near controlling direction)
Offshore Wave Height	--	11.0 m (36.1 ft)	--
Offshore Peak Wave Period	--	15 s	--

Model Details

MIKE 21 SW is a phase resolving model. The model is robust and allows the user to control many of the model parameters and solution techniques. The model for the Kodiak Runway Shoreline Protection was setup with a directionally decoupled parametric spectral formulation for a full 360^o rose. This model setting parameterizes the wave action conservation equation by treating the first two moments of the spectrum as dependent variables. A quasi-stationary time formulation was used in this model to produce a steady state solution at each time step, providing results for a developed sea for each of the combinations of the variables considered. Diffraction and wave breaking were included in the model. Wave breaking is included by a depth limited, gamma factor, which was set at 0.8. Bottom friction was considered negligible for the waves approaching Kodiak Airport because of the deep water depths and therefore the bottom friction formulation was not included in this model.

Results

Wave conditions were assessed near the project area, specifically near the anticipated toe of the shoreline protection. Data were extracted from 8 locations around the anticipated Runway 25 extension footprint, and 5 locations around the anticipated Runway 36 extension footprint. Figure 15 shows the data extraction locations graphically. Figure 16 shows an example (in plan view) of model output for calculated spectral significant wave height over the model domain and Figure 17 and Figure 18 show the controlling condition model output around Runway 25 and Runway 36, respectively. To illustrate the impact of increasing water levels on wave height, Figure 19 shows the spectral significant wave height model output around Runway 25 for three water surface elevations.

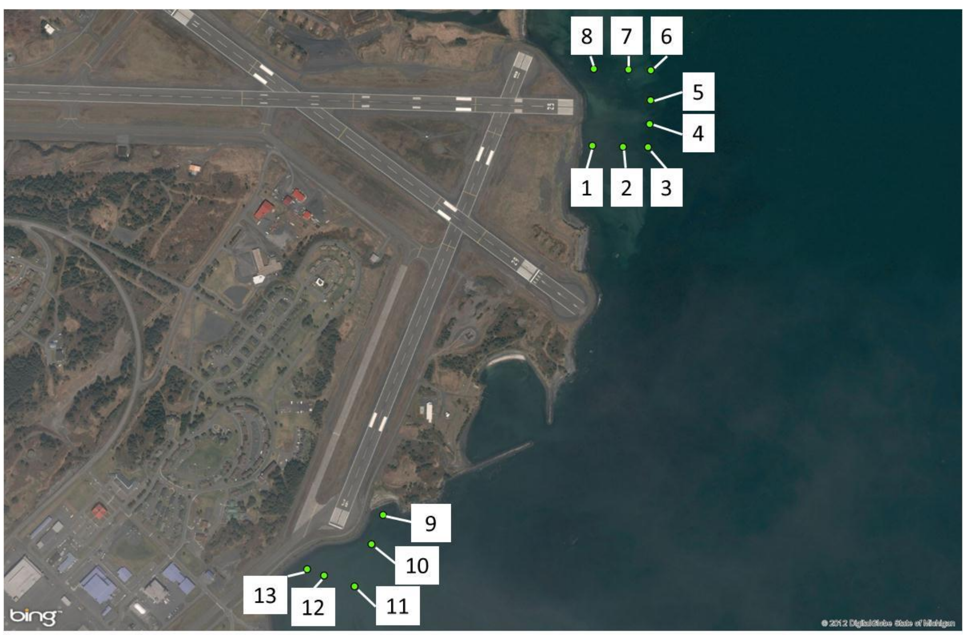


Figure 15. Data extraction locations around runway extension footprints.

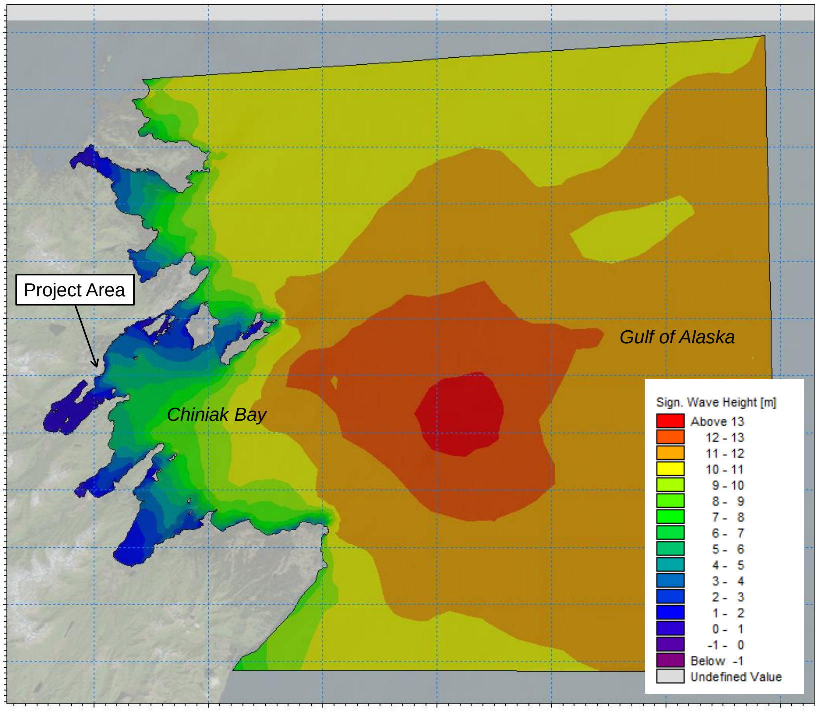


Figure 16. Example wave model output. WSE = +11.8 ft MLLW, Wave Dir. = 75 deg

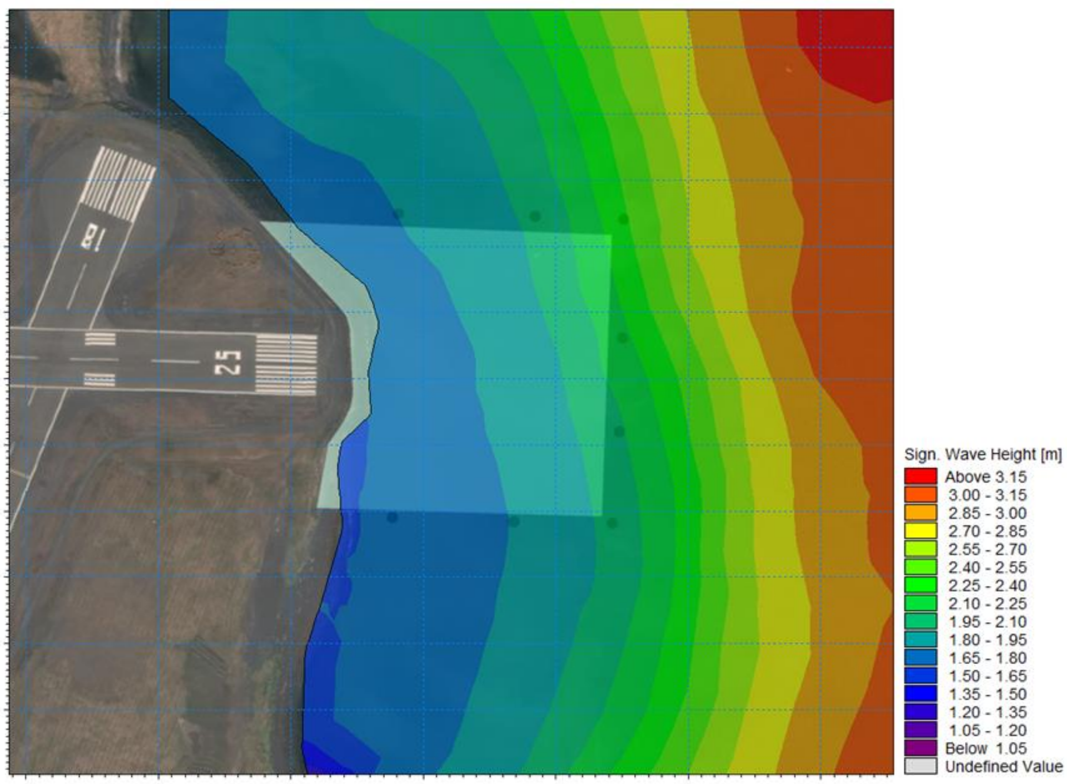


Figure 17. Model output at Runway 25 for controlling condition (WSE = +11.8 ft MLLW, Wave Dir. = 40 deg).

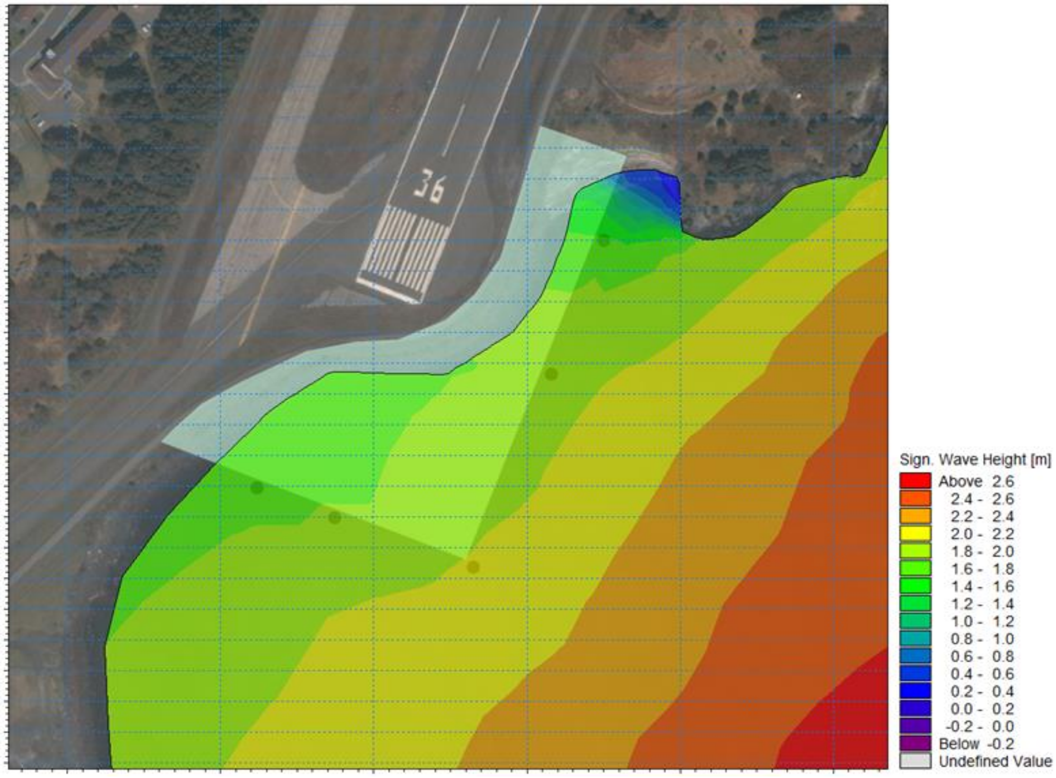


Figure 18. Model output at Runway 36 for controlling condition (WSE = +11.8ft MLLW, Wave Dir. = 40 deg).

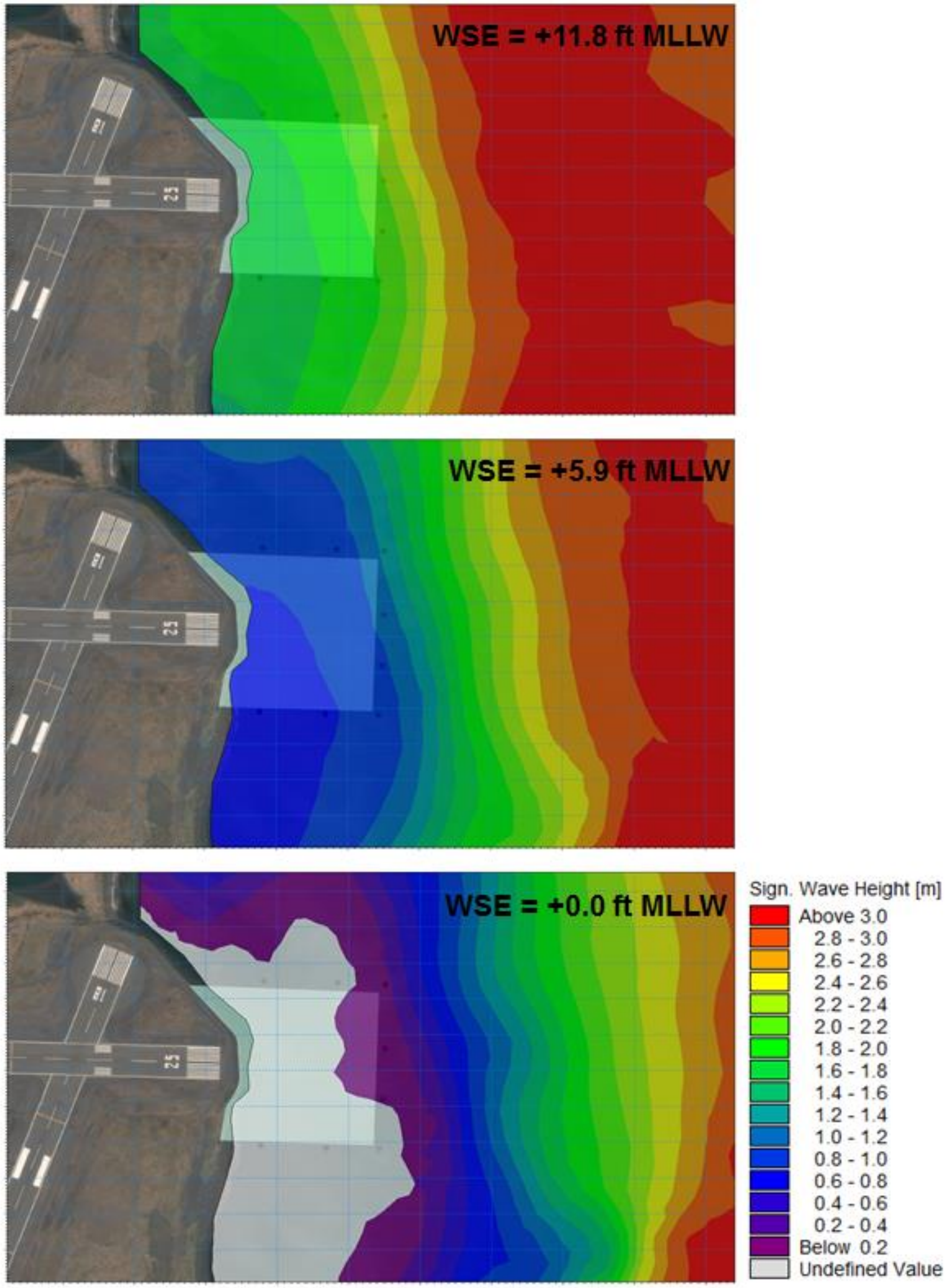


Figure 19. Model output at Runway 25 for controlling wave direction (40 deg) and varying water surface elevation (WSE).

Table 4 shows the maximum wave conditions at the various data extraction locations for the highest water level. Table 5 provides the design wave heights for both Runway 25 and Runway 36 at varying water levels.

Table 4. Wave conditions at controlling wave direction during the maximum water level, +3.8 m (+12.5 ft) NAVD.				
Location	Wave Height, m (ft)	Peak Period, s	Wave Dir. (Controlling)	Water Depth, m (ft)
1 – Runway 25	1.8 (5.9)	9.5	15°	3.8 (12.5)
2 – Runway 25	1.9 (6.2)	10.1	15°	3.8 (12.5)
3 – Runway 25	2.0 (6.6)	10.8	15°	3.8 (12.5)
4 – Runway 25	2.1 (6.9)	10.1	30°	3.8 (12.5)
5 – Runway 25	2.3 (7.5)	10.5	35°	3.9 (12.8)
6 – Runway 25	2.5 (8.2)	10.9	40°	4.0 (13.1)
7 – Runway 25	2.0 (6.6)	9.9	30°	3.8 (12.5)
8 – Runway 25	1.9 (6.2)	10.0	15°	3.8 (12.5)
9 – Runway 36	1.6 (5.2)	9.6	150°	3.8 (12.5)
10 – Runway 36	1.9 (6.2)	10.4	150°	4.0 (13.1)
11 – Runway 36	2.0 (6.6)	11.2	40°	4.2 (13.8)
12 – Runway 36	1.8 (5.9)	11.8	30°	4.5 (14.8)
13 – Runway 36	1.8 (5.9)	11.7	30°	4.0 (13.1)

Table 5. Summary of Design Wave Conditions		
	Wave Height (H_{mo})and Peak Period (T_p)	
Water Surface Elevation, NAVD	Runway 25 Wave Dir. 40° Location 6	Runway 36 Wave Dir. 40° Location 11
+0.2 m (+0.8 ft)	0.2 m (0.6 ft) 2.2 s	0.2 m (0.6 ft) 2.4 s
+0.5 m (+1.8 ft)	0.4 m (1.2 ft) 3.0 s	0.3 m (1.1 ft) 3.2 s
+0.8 m (+2.8 ft)	0.6 m (1.8 ft) 3.8 s	0.5 m (1.6 ft) 4.0 s
+1.1 m (+3.8 ft)	0.8 m (2.5 ft) 4.5 s	0.6 m (2.1 ft) 4.8 s
+1.4 m (+4.8 ft)	1.0 m (3.1 ft) 5.2 s	0.8 m (2.6 ft) 5.6 s
+1.7 m (+5.7 ft)	1.1 m (3.8 ft) 5.9 s	1.0 m (3.1 ft) 6.3 s
+2.0 m (+6.7 ft)	1.3 m (4.4 ft) 6.6 s	1.1 m (3.6 ft) 7.1 s
+2.3 m (+7.7 ft)	1.5 m (5.1 ft) 7.3 s	1.3 m (4.2 ft) 7.8 s
+2.6 m (+8.7 ft)	1.7 m (5.7 ft) 8.1 s	1.4 m (4.7 ft) 8.5 s
+2.9 m (+9.7 ft)	1.9 m (6.4 ft) 8.8 s	1.6 m (5.2 ft) 9.2 s
+3.2 m (+10.6 ft)	2.1 m (7.0 ft) 9.5 s	1.7 m (5.7 ft) 9.9 s
+3.5 m (+11.6 ft)	2.3 m (7.6 ft) 10.3 s	1.9 m (6.2 ft) 10.6 s
+3.8 m (+12.6 ft)	2.5 m (8.1 ft) 10.9 s	2.0 m (6.7 ft) 11.2 s

Preliminary Design

The following section describes preliminary design of shoreline protection for Runway 25 and Runway 36. Only revetment-type shoreline protection methods were given consideration for this assessment. An alternatives analysis was not included as part of the current scope.

Design Criteria

The design criteria applied to develop the preliminary design is as follows:

- Design wave height and period transformed from a 100 year (1% chance of exceedance in single year) offshore wave event.
- Design water level range from +0.8 ft to +12.6 ft NAVD (upper limit equals HAT).
- Revetment will be designed for no or minimal expected damage from 100 year design storm event.
- To extent practicable, limit wave overtopping to a level equivalent to "no damage to grass sea-dikes" based on USACE guidance for Critical Values of Average Overtopping Discharges (USACE 2011).
- Offshore limit of shoreline protection construction corridor limited to the footprint designated in the Environmental Assessment.

Review of Previous Similar Projects

<u>Unalaska Airport Shoreline Protection</u> – The Unalaska Airport was experiencing significant overtopping by approximately 12 ft waves. The shoreline protection originally protecting the runway was a stone revetment, which failed. Eight-ton Core-loc™ concrete armor units were placed to repair the revetment. These units were selected for their lower profile and improved wave runup dissipation capabilities over stone (Smith and Carter 2011).

<u>Sitka Airport Extension Shoreline Protection</u> – Similar to the present project, the runway at Sitka was recently extended. A stone revetment was constructed as the shoreline protection. Water depths at the offshore limit of the revetment were over 100 ft deep[2].

<u>Kodiak Island Breakwater</u> – The breakwaters at Kodiak were bid out with two options, an armor stone and concrete armor units section[2]. The breakwaters were built using armor stone. This method of bidding (with alternate armor sections) is considered to improve competitive bidding when compared to specifying only one section. As design of the shoreline protection for Runway 25 and Runway 36 progresses, additional review of the Kodiak Island Breakwater project should be performed to assess any lessons learned.

Armor Stability Analyses

Armor stability analyses were performed at multiple locations around Runway Extension 25 and 36 to determine the controlling design locations for each extension. It was found that the northeastern corner of the proposed Runway Extension 25 (Location 6) and the southeastern corner of the proposed Runway Extension 36 (Location 11) have the potential to experience the largest waves during storms.

---

[2] Personal communications with Harvey Smith and Ruth Carter, Coastal Engineers with the Alaska Department of Transportation and Public Facilities.

Armor size was calculated at the various data extraction locations around the runway extensions through application of two standard methodologies:

(1) Van der Meer equation

Plunging Waves ($\xi m < \xi cr$):

$$\frac{H_s}{\Delta D_{n50}} = c_{pl} P^{0.18} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \xi_m^{-0.5}$$

Surging Waves ($\xi m \geq \xi cr$):

$$\frac{H_s}{\Delta D_{n50}} = c_s P^{-0.13} \left(\frac{S_d}{\sqrt{N}}\right)^{0.2} \sqrt{cot\alpha}\,\xi_m^P$$

Where H_s is significant wave height ($H_{1/3}$)

(2) Hudson equation

$$W_{50} = \frac{\rho_r g H^3}{K_D \Delta^3 cot\alpha}$$

Where H is $H_{1/10}$

Both equations were applied with reductions to account for overtopping (CIRIA 2007). Calculations for stone stability provided estimates of required median stone diameter (D_{n50}) and weight (W_{50}) as a function of wave height and period, which were calculated as a function of water level and 100-year wave height as described earlier in the Preliminary Numerical Modeling section of this report. Table 6 provides median stone weight results for the data extraction locations for both methodologies at the highest water level analyzed. Table 7 provides median stone weight results at Location 6 (controlling location) for both methodologies at all water levels analyzed.

Recommended sizes for alternative Core-loc™ armor units are also presented. Table 8 provides the armor unit weight following USACE guidance (USACE 1997) at Location 6, the controlling location. The USACE guidance recommends the Hudson formula with a stability coefficient, K_d, of 16 for trunk sections and 13 for head sections. This results in a significantly smaller armor unit (by weight) compared to stone.

<table>
<tr><th colspan="4">Table 6. Summary of Armor Stone Stability Results.</th></tr>
<tr><th colspan="2"></th><th>W₅₀, lb
Van der Meer</th><th>W₅₀, lb
Hudson</th></tr>
<tr><td rowspan="8">Runway Extension 25</td><td>Location 1</td><td>3,090</td><td>4,060</td></tr>
<tr><td>Location 2</td><td>3,480</td><td>4,830</td></tr>
<tr><td>Location 3</td><td>3,860</td><td>5,680</td></tr>
<tr><td>Location 4</td><td>5,100</td><td>6,550</td></tr>
<tr><td>Location 5</td><td>6,560</td><td>8,710</td></tr>
<tr><td>Location 6</td><td>9,040</td><td>11,400</td></tr>
<tr><td>Location 7</td><td>1,630</td><td>5,930</td></tr>
<tr><td>Location 8</td><td>3,450</td><td>4,760</td></tr>
<tr><td rowspan="5">Runway Extension 36</td><td>Location 9</td><td>1,920</td><td>2,810</td></tr>
<tr><td>Location 10</td><td>3,700</td><td>5,260</td></tr>
<tr><td>Location 11</td><td>3,920</td><td>6,020</td></tr>
<tr><td>Location 12</td><td>2,320</td><td>4,230</td></tr>
<tr><td>Location 13</td><td>2,220</td><td>4,230</td></tr>
<tr><td colspan="4">Structure Slope, S: 1.5H:1V
Damage Coefficient, S_D: 2 (VDM)
K_D: 2.2 (Hudson)
Permeability, P: 0.5 (VDM)
*Note: more variations of these parameters were analyzed. Parameters shown represent those corresponding to recommended shoreline protection cross-section design.</td></tr>
</table>

Table 7. Summary of Armor Stone Stability Results at Location 6.

Water Surface Elevation, ft (NAVD)	W_{50}, lb Van der Meer	W_{50}, lb Hudson
+0.8	4	5
+1.8	30	40
+2.8	120	130
+3.7	300	320
+4.7	620	650
+5.7	1,150	1,150
+6.7	1,960	1,870
+7.7	2,870	2,830
+8.7	3,920	4,060
+9.6	5,100	5,570
+10.6	6,390	7,340
+11.6	7,720	9,300
+12.6	9,040	11,400

Table 8. Summary of Core-Loc™ Stability Results at Location 6.	
Water Surface Elevation, ft (NAVD)	W_{50}, lb Hudson
+0.8	1
+1.8	10
+2.8	30
+3.7	80
+4.7	160
+5.7	290
+6.7	470
+7.7	710
+8.7	1,010
+9.6	1,390
+10.6	1,830
+11.6	2,320
+12.6	2,840

Wave Overtopping Analysis

Rates of wave overtopping for both proposed runway extensions were analyzed at the various data extraction locations. Methodology provided by both EurOtop (2007) and Van der Meer & Janseen (1999) as outlined in the CEM were applied to calculate overtopping rates. Results using Eurotop (2007) methodology for the proposed Runway 25 Extension and Runway 36 Extension are provided in Figure 20, which also provides a comparison to published overtopping thresholds (USACE 2011, CIRIA 2007). Most of the overtopping rates calculated for the Runway 25 Extension were below the "Start of damage to grass sea-dikes." Portions of the extension that are at lower elevations located nearest to the existing shoreline experience higher overtopping rates which encroach into "Start of damage to grass sea-dikes."

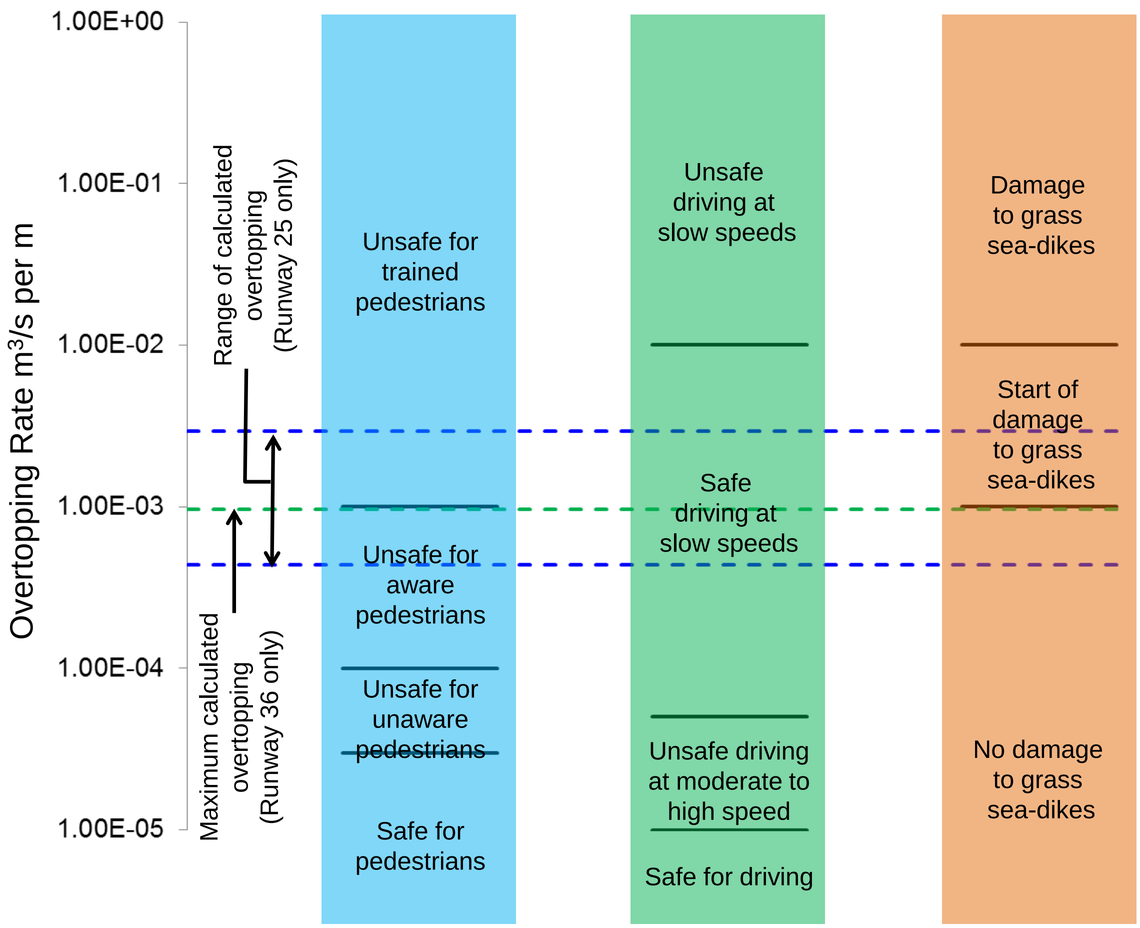


Figure 20. Calculated overtopping rates at Runway 25 with published allowable overtopping rates (USACE 2011, Ciria 2007).

Preliminary Cross-Sections

Two preliminary cross-sections were developed for both Runway 25 and Runway 36 which include a conventional stone revetment and a revetment utilizing Core-loc™ concrete armor units. If the only viable armor stone sources require long travel distances, concrete armor units may be a more cost-effective alternative. Runway 25 is at a much lower elevation than Runway 36, making it more susceptible to wave overtopping (refer to Figure 21). All proposed revetment sections remain within the footprint of the original Environmental Assessment, in many cases requiring the slopes to be relatively steep (i.e. 1.5H to 1V). In locations along Runway 25 where the water is shallower, there is more room to utilize milder slopes. Using a milder slope (such as 2:1) in shallower areas can be given additional consideration during final design, if desired.

The proposed sections for the Runway 25 revetment are provided in Figure 21 and Figure 22. Proposed sections for Runway 36 are provided in Figure 23 and Figure 24. Design notes detailing specific features are provided below the figures.

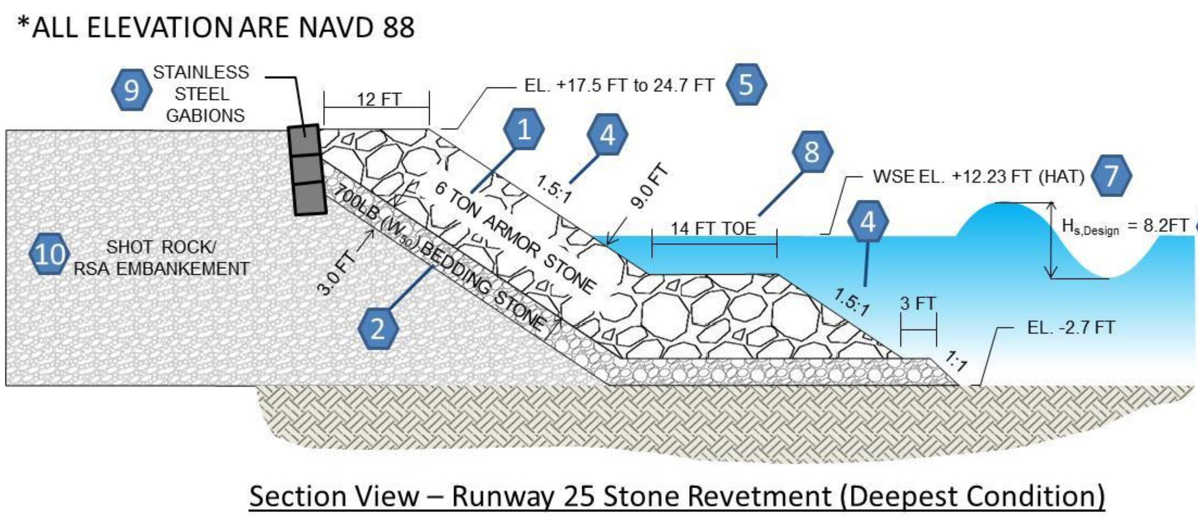


Figure 21. Runway 25 Preliminary Design Cross-Section at Deepest Condition (Stone Revetment Option)

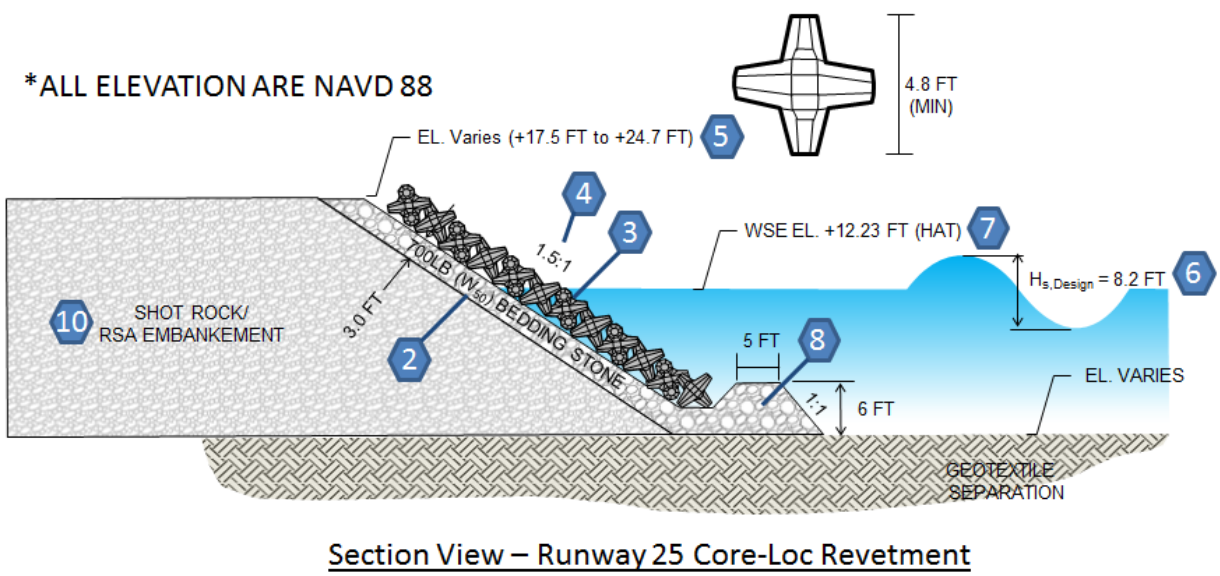
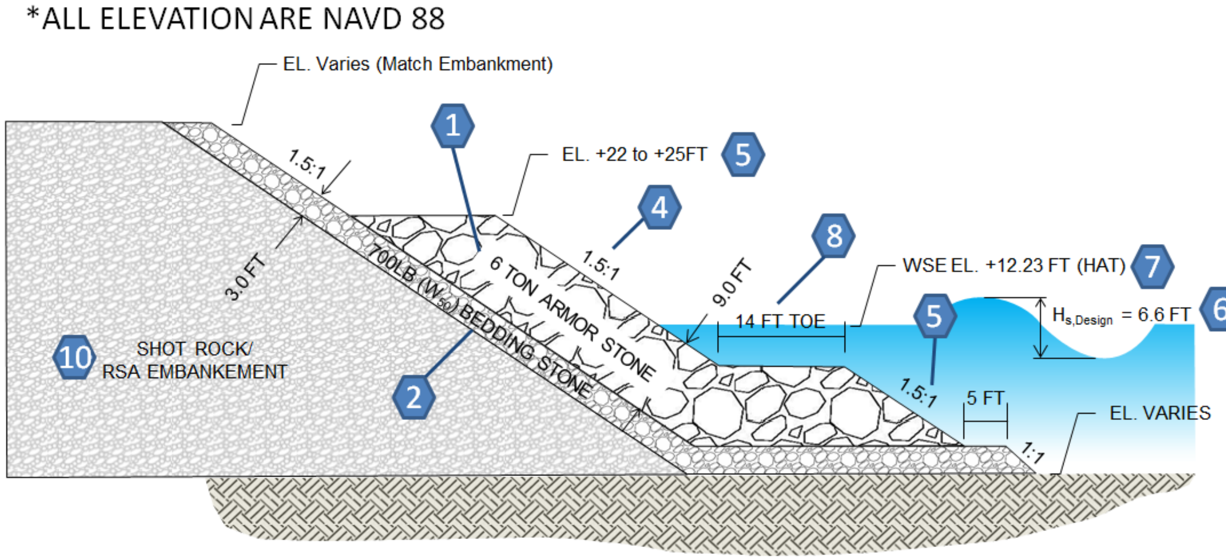
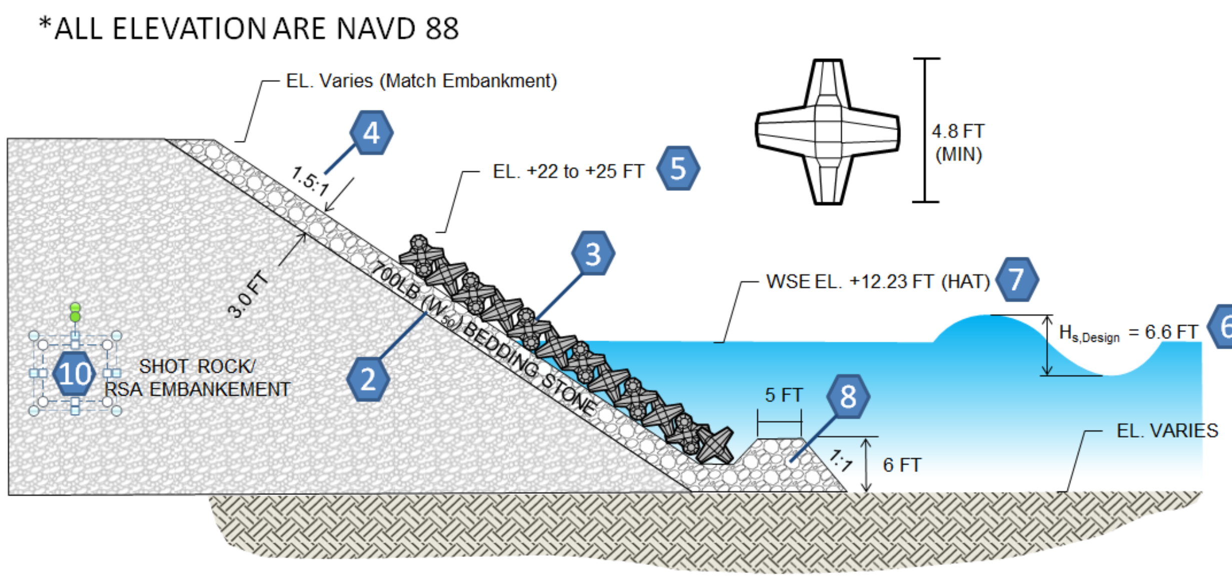


Figure 22. Runway 25 Preliminary Design Cross-Section (Core-Loc Option).



Section View – Runway 36 Stone Revetment

Figure 23. Runway 36 Preliminary Design Section (Stone Revetment Option).



Section View – Runway 36 Core-Loc Revetment

Figure 24. Runway 36 Preliminary Design Cross-Section (Core-loc™ Option).

1 *Armor Stone Size (Stone Revetment)* – Armor stone was sized using stone stability methodologies of Hudson and Van der Meer. Based on a comparison of both methods, a median armor stone weight of approximately 6 tons appears appropriate for the design wave conditions for a 1.5H:1V structure slope. This size is based on the controlling condition which occurs at Runway 25 on the northwest corner. If desired, portions of Runway 25 and all of Runway 36 could utilize smaller stone and still be stable for the design wave conditions. Varying armor stone size with location (and degree of wave exposure) should be analyzed in further detail during final design, especially with respect to cost. Although

reducing material generally reduces cost, requiring a contractor to acquire multiple armor stone gradations increases complexity and may increase overall construction costs.

2 *Bedding Stone Size (Stone Revetment)* – Bedding stone size was calculated based on the size of the armor stone. For both a 6 ton armor stone, a 700 lb median stone weight bedding stone is recommended.

3 *Concrete Armor Unit Size (Core-loc™ Revetment)* – As an alternative to armor stone, Core-loc™ concrete armor units were considered. Core-loc™ are recommended to be placed in a single layer. Hudson methodology was used to calculate stability and select a standard Core-loc™ size. Based on these calculations (see Table 8), the smallest standard Core-Loc unit of 1.85 tons is recommended based on stability. This size unit has an end to end length (commonly denoted as "c") of approximately 4.8 ft. Larger units could also be applied if desired for an added level of conservatism or if constructability is improved in turn improving construction costs.

4 *Revetment Slopes* – For the stone revetment sections, a 1.5H:1V slope is shown to keep within the original Environmental Assessment footprint. Along sections of Runway 25 with shallower water, a 2H:1V slope is possible and can be given additional consideration during final design, if desired. Areas with steeper slopes require the armor stone to be very large. Decreasing the slope significantly decreases the required stone weight, but would require more material; however, specifying smaller stone may increase the number of available stone sources, potentially reducing costs. An advantage of Core-loc™ sections is that they are typically designed at steeper angles such as 1.5:1 or 2:1, allowing a smaller horizontal footprint. Even with the steep 1.5H:1V slope, the Core-Loc armor units can be relatively small.

5 *Armor Crest Elevations* – The armor stone crest elevation of +25 ft NAVD for Runway 36 was chosen to reduce the overtopping to a level that would reduce the risk of damage to the bedding layer at elevations above the armor stone. Some areas on the edge of Runway 36 are below +25 ft NAVD. At these locations, the armor stone is to match the existing grade. Overtopping calculations presented in Figure 20 include these varying crest elevations. The crest elevation at Runway 25 was limited to match the elevation of the embankment.

6 *Design Wave Height* – Design wave heights are shown for the controlling conditions at each Runway Extension (See Table 4). The height of the wave is drawn to scale with the revetment section. The length of the wave is not to scale.

7 *Design Water Level* – All water levels from MLLW to HAT were analyzed for stone stability. The highest water level analyzed (HAT) was determined to be the controlling water surface elevation. The HAT is shown in all sections with the design wave height.

8 *Revetment Toe Width* – All stone revetment sections are recommended to include a toe. An above ground toe is utilized to avoid excavation of the existing grade. The toe helps reduce erosion as well as provide stability for the remaining revetment. Core-loc™ sections have a "key" to help the Core-loc™ maintain placement integrity.

9 *Stainless Steel Gabions* – In an effort to fit the stone revetment structure within the Environmental Assessment footprint, the armor stone has been translated landward and is adjacent to the end of the runway extension. To separate the RSA embankment from the armor stone, stainless steel gabions are recommended.

10 *RSA Embankment* – Overtopping and overall stability of all sections is improved with a higher permeable structure. It is recommended the RSA embankment be constructed with a free draining material such as shot rock.

Summary and Future Recommendations

Preliminary design of shoreline protection for extensions of two runways at Kodiak Airport has been completed. Due to the mountainous terrain, the runways are proposed to be extended further into St. Paul Harbor rather than being extended landward. These extensions into the harbor are expected to be exposed to large hydrodynamic forces from waves traveling from the Gulf of Alaska, requiring shoreline protection to reduce erosional damage. Readily available metocean data were gathered for the project location and used to determine offshore design wave conditions and water levels. A numerical spectral wave model was used to transform the offshore waves into the nearshore environment for use in stability and overtopping calculations to support revetment design of the two runways extensions. Each runway has a traditional stone revetment option and a Core-loc^{TM} concrete armor unit option. The preliminary design considerations primarily focused on developing sections which would resist the 100 year wave event and minimize overtopping.

Future tasks for design of the shoreline protection surrounding the extension of the Kodiak Airport runways should include:

1. Further research and investigation into the designs and lessons learned of similar shoreline protection projects in Alaska including the Unalaska Airport revetment, the Sitka Airport extension, and the Kodiak Island breakwaters.
2. Investigate the feasibility of both the armor stone and Core-loc^{TM} revetments constructability and compare probable construction costs.
3. Further development of revetment sections to explore potential reductions in material quantities and maintain acceptable stability and overtopping.
4. Perform geotechnical analysis of revetment sections.

References

American Society of Civil Engineers. 2010. “Minimum Design Loads for Buildings and Other Structures.” Standard SEI/ASCE 7-10, Reston, VA.

CIRIA; CUR; CETMEF. 2007. The Rock Manual. The use of rock in hydraulic engineering. C683, CIRIA, London.

DHI. 2008. MIKE 21 Flow Model FM Hydrodynamic Module and Spectral Wave Module User Guides, DHI Water and Environment, Denmark.

NOAA. 2013. Center for Operational Oceanographic Products and Services (CO-OPS) Webpage, <http://tidesandcurrents.noaa.gov/>.

Smith and Carter. 2011. “Over Twenty-Five Years of Applied Coastal Engineering in Alaska.” Coastal Engineering Practices 2011 ASCE Conference Proceedings.

Tracy, B.A. 2004. “Wave Information Studies: Hindcast Wave Data for U.S. Coasts.” Vicksburg, MS: U.S. Army Engineer Research and Development Center, http://frf.usace.army.mil/ cgi-bin /wis/atl/atl_main.html.

USACE 1997. “Core-Loc Concrete Armor Units: Technical Guidelines.” Miscellaneous Paper CHL-97-6.

USACE. 2011. “Coastal Engineering Manual.” EM-1110-2-1100 (Part VI). Table VI-5-6.