Coastal Report – Scammon Bay Airport Improvements Feasibility Study

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Prepared by:

HDR

STATE OF ALASKA
49TH
DONALD L. MCPHERSON
No. CE 13591
PROFESSIONAL ENGINEER
12-19-2022

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Acronyms and Abbreviations

AEP	Annual Exceedance Probability
ASOS	Automated Surface Observing System
DEM	Digital Elevation Model
DHI	Danish Hydraulic Institute
DMVA	Department of Military and Veterans' Affairs
DOT&PF	State of Alaska Department of Transportation and Public Facilities
FEMA	Federal Emergency Management Agency
H&H	Hydrology and Hydraulic
HD FM	Hydrodynamic Flexible Mesh
ft	feet
IfSAR	Interferometric Synthetic Aperture Radar
in	inches
LiDAR	Light Detection and Ranging
m	meter
mm	millimeter
NAVD88	North American Vertical Datum of 1988
NOAA	National Oceanic and Atmospheric Administration
RSLR	relative sea-level rise
s	second
SCM	Scammon Bay State Airport (International Air Transport Association's airport code)
SW	Spectral Wave
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

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1. Introduction

1.1 Project Overview

This Coastal Report is prepared for the State of Alaska Department of Transportation and Public Facilities (DOT&PF) Central Region as part of a larger feasibility study to assess improvements to the airport at Scammon Bay (project).

The project is at the Scammon Bay State Airport (SCM), which is a state-owned, public use airport. The airport consists of one runway and one seaplane landing area. The DOT&PF proposes various airport improvements to enhance safety, improve infrastructure, and bring the airport to Federal Aviation Administration standards. These improvements consist primarily of repairing elements that have been damaged by flooding or have otherwise deteriorated, including:

- Increasing the elevation of the runway, taxiway, apron, and access road
- Shifting the runway away from the Kun River
- Replacing the culvert under the runway
- Placing erosion protection adjacent to the Kun River and airport embankments
- Making various building and aviation-specific additions and replacements
- Obtaining additional right-of-way

1.2 Scope of Coastal Analyses

The project involves providing coastal engineering and hydrology and hydraulic (H&H) recommendations to guide a larger feasibility study regarding the various airport improvements to better protect SCM from flooding and scour. Recommended improvements to the airport specific to coastal engineering are detailed within this report. Details on H&H analysis to support this project are provided under separate cover (HDR, 2022).

The coastal analyses for this project include a review of readily available background information, site visit performed in May 2021, storm surge analysis, and wind wave analysis. Details of these analyses are discussed herein.

1.3 Organization of Report

This report is organized as follows:

- Section 2 discusses existing general conditions.
- Section 3 discusses data used in coastal analysis.
- Section 4 discusses the design criteria.
- Section 5 discusses the coastal analysis.

- Section 6 presents the coastal engineering design recommendations.
- Section 7 presents the summary.
- Section 8 presents the references cited.

All elevations provided are based on the North American Vertical Datum of 1988 (NAVD88) unless otherwise specified.

2. General Conditions

2.1 General Physical Characteristics

The project site is located in the community of Scammon Bay in the Kusilvak Census Area, in Western Alaska. Scammon Bay has a population of 594 (U.S. Census Bureau 2020) and covers 299 acres (see Figure 1). The airport is located at the northeast edge of the community. The Scammon Bay community sits at the meeting point of the base of the Askimuk mountain range and flat, intertidal wetlands. Wetlands, ponds, and connecting streams dominate the area to the north and east.

The airport sits on the south bank of the Kun River, a perennial stream with a bankfull width of approximately 900 feet. Several unnamed tributaries of the Kun River are located near the community, one of which flows underneath the runway through a singular culvert. Tidal influence is evident in the tributary by the nearly vertical stream banks that are 2–3 feet in depth. The tributary's confluence is located approximately 2 miles from the mouth of the Kun River.

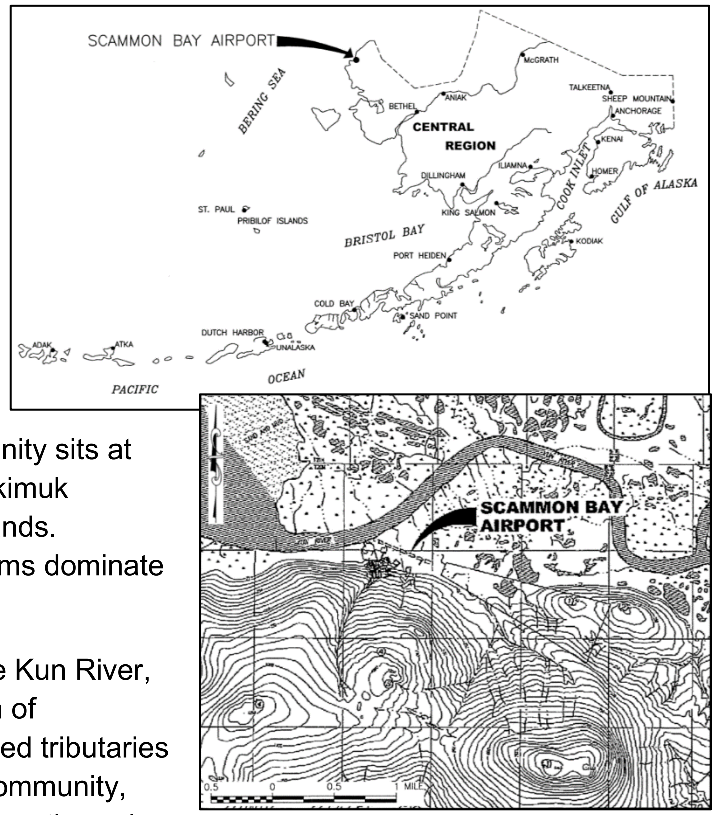

Figure 1: Location and Vicinity Map

Recreated from DOT&PF 2004 and 2013

2.1.1 Runway 10/28 and Seaplane Landing Area 4W/22W

The airport consists of one Type A, gravel runway designated as 10/28, and one seaplane landing area designated as 4W/22W. The runway is located at the northeast edge of the community, sits at an elevation between +10 and +17.5 feet NAVD88, and runs northwest to southeast at a +0.19 percent slope. It is encompassed by intertidal wetlands with the unnamed perennial stream that runs through a culvert under the runway from south to north. One access road connects the runway to the community. The seaplane landing area is located at the northwest edge of the community.

2.1.2 Runway Culvert

The existing structure is a 48-inch-diameter, 405-foot-long, smooth interior wall, corrugated, high-density polyethylene culvert that runs under the runway. It was installed with a 0.1 percent slope, with an inlet invert elevation of +4.0 feet NAVD88 and an outlet invert elevation of +3.6 feet NAVD88. Additional information on the condition of the existing culvert can be found in the accompanying *Hydrology and Hydraulics Report* (HDR, 2022).

3. Data Used for Coastal Analysis

3.1 Metocean Data

Meteorological and oceanic (metocean) data were gathered from readily available sources. For data not available at Scammon Bay, data from the nearest reasonable location were used. The following provides details on metocean data used for the coastal analysis.

3.1.1 Water Level

Tidal datum information from the National Oceanic and Atmospheric Administration (NOAA) is available for the Kun River near Scammon Bay (Station 9467124) and is shown in Table 1. This information comes from a historical short-term tide station that collected water level data from July 24, 2020, to October 22, 2020 (approximately 3 months).

Table 1: Kun River Tidal Datums (NOAA Station ID: 9467124)

Datum	Elevation (feet from MLLW)	Elevation (feet, based on NAVD88)
Mean Higher High Water (MHHW)	6.47	6.77
Mean High Water (MHW)	5.70	6.00
Mean Tide Level (MTL)	3.29	3.59
Mean Sea Level (MSL)	3.20	3.50
Mean Low Water (MLW)	0.88	1.18
Mean Lower Low Water (MLLW)	0	0.30
NAVD88	-0.30	0

Source: NOAA 2021c
Notes: NAVD88 = North American Vertical Datum of 1988.

Long-term water level data for Scammon Bay are not available; thus, review/prediction of relative sea-level rise (RSLR) over time is not possible near the project site. The nearest location to Scammon Bay with a long-term water level dataset is Nome, Alaska, approximately 180 miles to the north. The Nome tide station has measured RSLR at a rate of 0.15 inch per year with a confidence interval of +/- 0.11 inch per year (3.89 millimeter [mm]/year with a 95 percent confidence interval of +/- 2.88 mm/year). Figure 2 shows the long-term trend plot developed by NOAA (NOAA 2021d). Assuming a similar RSLR at Scammon Bay, the increase in sea level over a 50-year period would be 0.64 feet.

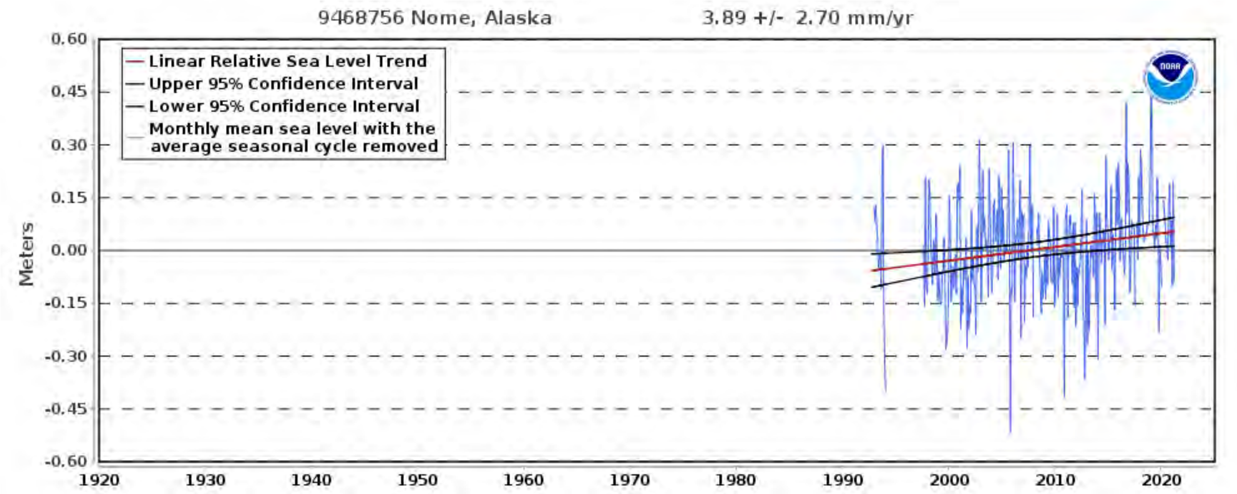

Figure 2: Relative Sea-Level Rise at Nome, AK

Source: NOAA 2021d

Statistical storm surge water level predictions in western Alaska were developed by the U.S. Army Corps of Engineers (USACE) in *Storm-Induced Water Level Prediction Study for the Western Coast of Alaska* (USACE 2009). The study provides statistical storm surge water levels at 17 locations in Western Alaska. The two nearest locations to Scammon Bay for statistical storm surge elevations are Agcklarok, Alaska, and Hooper Bay, Alaska, approximately 50 miles northeast and 30 miles southwest of Scammon Bay, respectively (Figure 3). Storm surge predictions for Agcklarok and Hooper Bay are shown in Table 2.

Table 2: Probabilistic Storm Surge Elevations for Agcklarok, AK, and Hooper Bay, AK

Return Period (years)	Agcklarok Surge Level (feet)	Hooper Bay Surge Level (feet)
5	4.8	6.5
10	6.7	8.1
15	7.4	8.4
20	7.8	8.6
25	8.3	8.8
50	10.1	10.0
100	12.1	11.5

Note: Storm surge elevations are reported independent of tidal influence.
Source:USACE 2009

Figure 3: Statistical Storm Surge Data Source Locations

3.1.2 Wind

Historic wind direction and speed information starting in 2010 at the project site is available via the Scammon Bay Automated Surface Observing System (ASOS). ASOS wind observations are reported as 2-minute averages. These durations were converted to 1-hour averaged wind speeds for wind-generated wave simulations (see Section 5.2). An extreme value analysis using these data was performed to determine statistical wind speeds and associated wind directions at Scammon Bay. An example of the statistical wind speed that includes data for "all directions" is shown on Figure 4. The wind direction data were binned to the nearest 10 degrees. The 1-hour wind speed duration was chosen based on the large fetch that would occur during a flooding event in which the surrounding flats are considered open water.

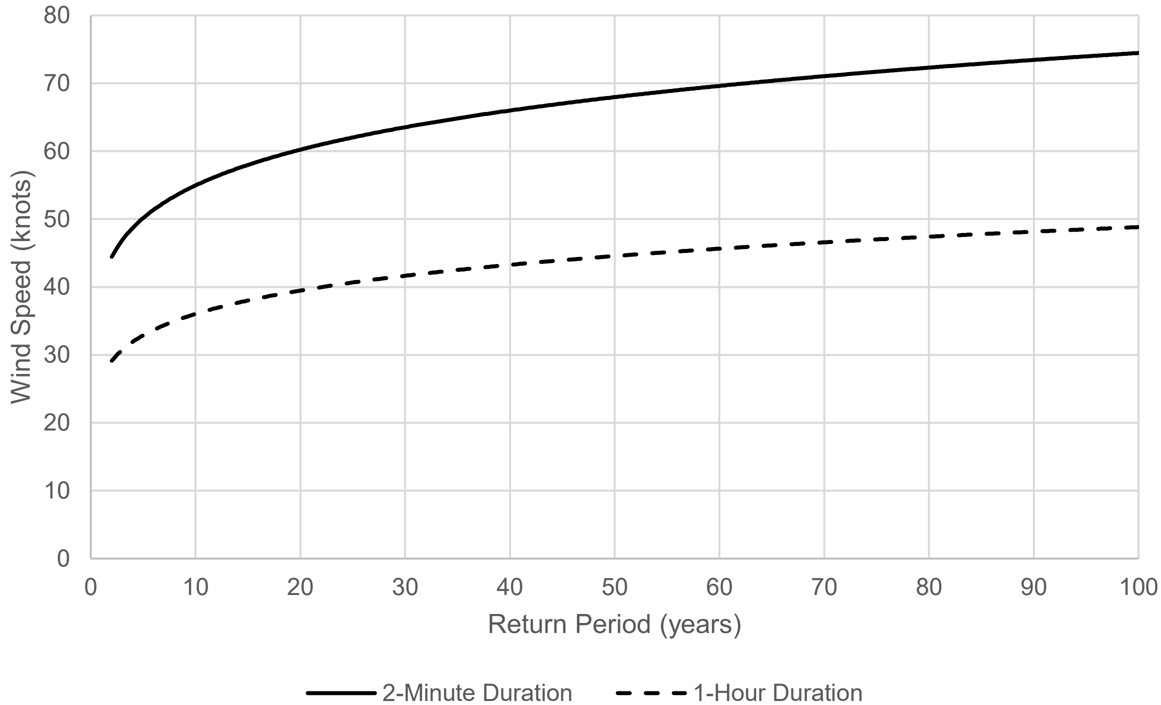

Figure 4: Statistical Wind Speeds for All Directions at Scammon Bay

3.1.3 Waves

Wave data are not available for Scammon Bay during a flooding event. Therefore, wave conditions were determined using MIKE 21 Spectral Wave (SW) software, a two-dimensional depth-averaged spectral wave numerical model. The model simulates wind-generated wave conditions at the project site. Additional wave information based on model results is presented in Section 5.2.

3.1.4 Sea Ice

Historic data from the University of Alaska Fairbanks indicate that Scammon Bay (coastal water body) typically contains at least 80 percent sea ice in January, February, and March, and contains variable levels of sea ice during all other months excluding August, September, and October (UAF 2021).

3.2 Elevation Data

3.2.1 Topography

The topographic data in the form of Digital Elevation Models (DEMs) were obtained using a combination of readily available Interferometric Synthetic Aperture Radar (IfSAR) data (USGS 2019) and Light Detection and Ranging (LiDAR) data (State of Alaska Division of Geological & Geophysical Surveys 2021).

3.2.2 Bathymetry

Bathymetric information in the offshore area to the west of Scammon Bay was gathered from NOAA National Geodetic Data Center datasets (NOAA 2021a) and NOAA Navigation Chart

16240 (NOAA 2021b). The chart reported depths in feet below mean lower low water. These data were then converted to NAVD88 using the relationship provided in Table 1.

Readily available bathymetric data for the Kun River or its tributaries were not found. Therefore, elevation data for the Kun River and three of its unnamed tributaries were estimated using a combination of channel width, estimated bankfull discharge, bathymetric maps of the Yukon River for comparison, and engineering judgement.

4. Design Criteria

Design criteria for coastal recommendations utilize a 50-year return period (2 percent annual exceedance probability [AEP]) for water level (for both concurrent and non-coastal conditions) and 100-year return period (1 percent AEP) for wind-generated waves. Design life duration is assumed to be 50 years.

5. Coastal Analysis

A coastal analysis was performed that consisted primarily of developing a storm surge numerical model and a wave numerical model. The purpose of these models was to better understand potential storm surges and wave conditions that affect the design of runway elevation and erosion mitigation.

5.1 Storm Surge Analysis

A storm surge analysis was performed to approximate potential water surface elevations and current speed/direction at the SCM due to an extreme flood event. The analysis was performed using the MIKE 21 Hydrodynamic Flexible Mesh (HD FM) numerical model. The model was developed to simulate a 50-year (2 percent AEP) and 100-year (1 percent AEP) representative storm surge events.

5.1.1 Storm Surge Model Description

MIKE 21 HD FM, developed by the Danish Hydraulic Institute (DHI), is software used for developing two-dimensional hydrodynamic models based on a flexible (unstructured) mesh. Models developed with MIKE 21 HD FM simulate water level variations and flows in coastal areas, estuaries, and floodplains (DHI 2017a). The flexible mesh module allows for higher-resolution elements at locations requiring better resolution of the hydrodynamics (e.g., near the project site and nearby flow paths).

5.1.2 Model Domain and Mesh

The model domain for the MIKE 21 HD FM storm surge simulations includes offshore, upland (which contains the project site), and backland areas. The offshore area applies coastal surge elevations that subsequently flow through the entire model domain. The backlands area is intended to provide added area/volume for surge inundation to flow to avoid unrealistic boundary effects impacting the project site (i.e., acts as a hydraulic storage area).

The mesh contains 50,524 elements and 27,385 nodes. The backlands area has a relaxed mesh resolution to improve model computation efficiency. The offshore and uplands areas have finer resolution with elements decreasing in size along flow paths and near the project site. Bed resistance information in the form of Manning's M values (reciprocal of Manning's n) were applied to the domain. A Manning's M value of 32 meter$^{1/3}$/second was assigned to the offshore area and a Manning's M value of 20 meter$^{1/3}$/second was assigned to the upland and backland areas.

Primary sources of elevation data used to create the mesh are summarized in Table 3.

Table 3: Elevation Data Summary

Data Source	Source Datum and Units	Model Location
IfSAR, Y-K Delta 2016 LiDAR Scammon 2015 elevation data	Horizontal: UTM Zone 3, meters Vertical: NAVD88, meters	Project area
Alaska Yukon Delta Base Order 2018 D18 Digital Elevation Model	Horizontal: Alaska Albers, meters Vertical: NAVD88, meters	All other upland and backland areas
NOAA Navigation Chart 16240	Horizontal: WGS 1984, degrees Vertical: Depth at MLLW, feet	Offshore area

Note: The horizontal and vertical datums used for the project are UTM Zone 3, Meters and NAVD88, Meters respectively. Source datum/units were converted to these project datums. IfSAR = Interferometric Synthetic Aperture Radar, Y-K Delta = Yukon-Kuskokwim Delta, LiDAR = Light Detection and Ranging, UTM = Universal Transverse Mercator, NAVD88 = North American Vertical Datum of 1988, NOAA = National Oceanic and Atmospheric Administration, WGS = World Geodetic System, MLLW = mean lower low water.

Figure 5 provides a view of the entire model domain. The colors represent bathymetry/topography elevations. Figure 6 and Figure 7 provide enlarged views of the mesh showing the finer resolution for the project site and flow paths.

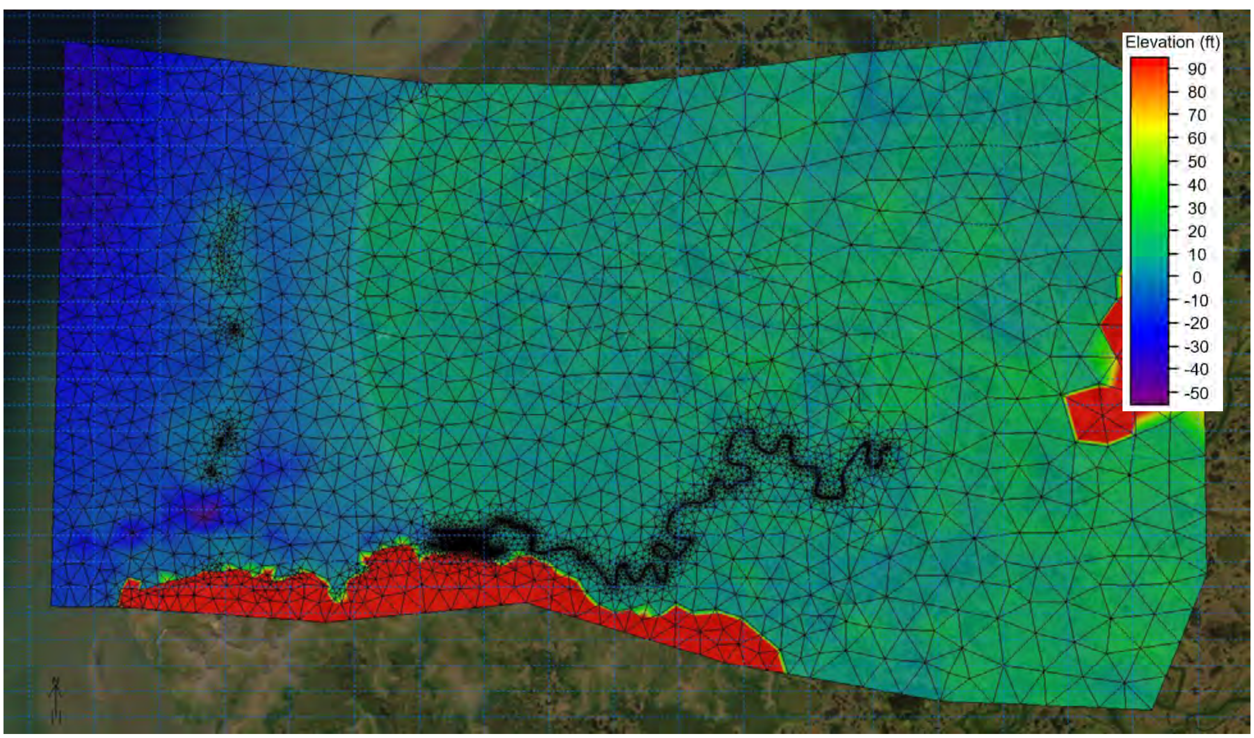

Figure 5: MIKE 21 HD FM Storm Surge Model Mesh - Full Domain

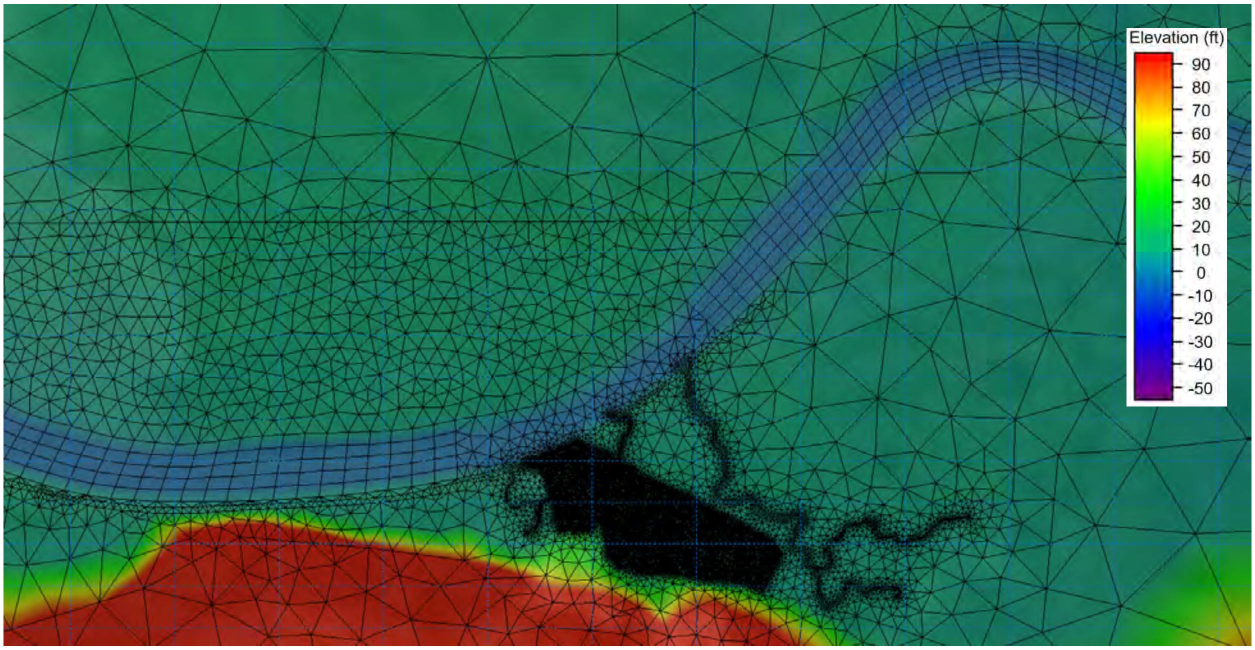

Figure 6: MIKE 21 HD FM Storm Surge Model Mesh - Enlarged View Showing the Kun River and Scammon Bay

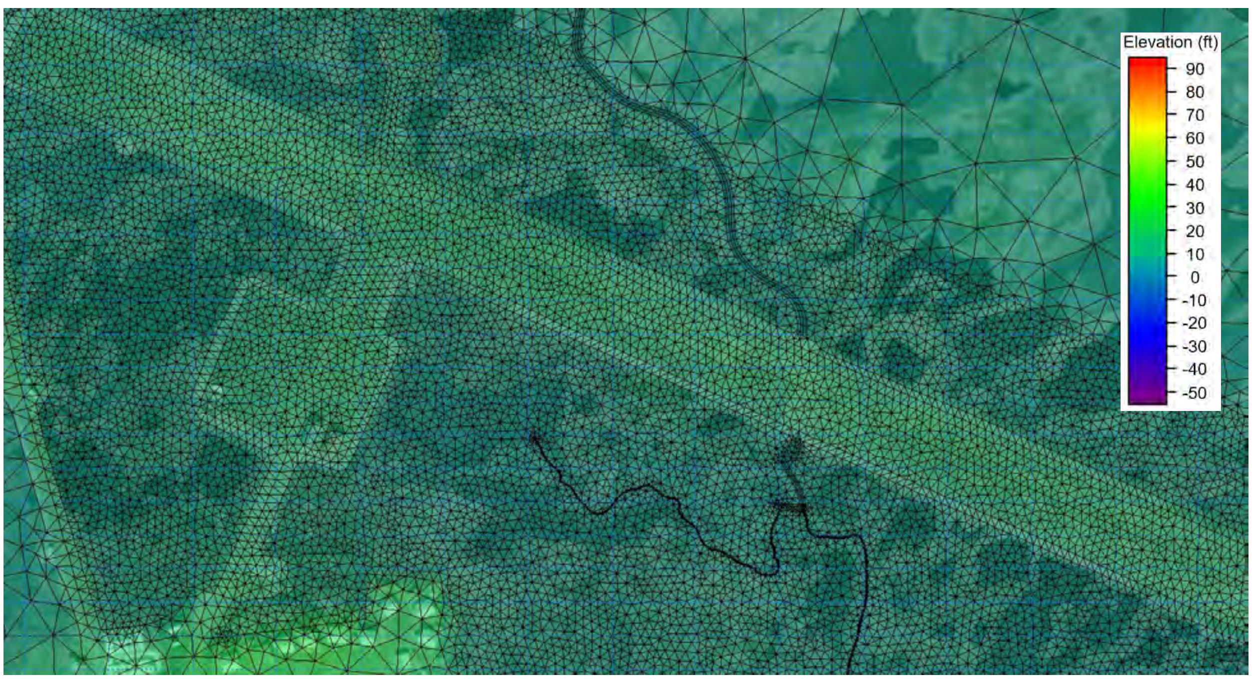

Figure 7: MIKE 21 HD FM Storm Surge Model Mesh - Enlarged View Showing Project Area

5.1.3 Storm Surge Model Boundary Conditions

The storm surge model was forced using both a storm surge hydrograph that was applied to the offshore boundary as well as a flow rate for the Kun River applied upstream of the runway. The storm surge hydrograph combined typical tides, anticipated RSLR, and a statistical storm surge in which the peak surge occurs at a high tide.

Statistical Storm Surge Development: Historic storm surge events identified by USACE (2009) in Nome were evaluated for shape, duration, and season of occurrence. Several events identified by USACE took place during the month of February. These storm surges were not included in the analysis, as sea ice is understood to dampen the effects of coastal storm surges (Barnhart et al. 2014).

All surges were analyzed independent of tidal influence. A representative storm surge unit hydrographic was developed that combined the fast rise of a storm surge observed with the fastest fall (receding water level) of a storm surge observed and maintained a peak level duration of a typical storm surge for Western Alaska. The intent of combining the fastest storm surge rise and fall was to simulate the higher end of current speeds near the runway during a flood event both as the storm surge enters and as it recedes. The unit storm surge hydrograph was scaled using the USACE (2009) 50- and 100-year storm surge heights for the Agcklarok location (see Section 3.1.1). Surge heights from the Agcklarok location were applied in lieu of the Hopper Bay location, as they were found to be more conservative.

Typical Tides: Typical tide data were gathered from NOAA Station 9467124 Kun River (NOAA 2021e). Based on review of Western Alaska storm surge occurrences as well as local anecdotal data, the fall season (September, October, and November) was found to be the most likely time of year for storm surge occurrence. Thus, the typical tide used for the boundary condition utilizes the NOAA tidal predictions during this period. Figure 8 shows the predicted tides for the fall 2021 at the Kun River NOAA station. The highest seasonal tide during this period was identified and used in the storm surge hydrograph.

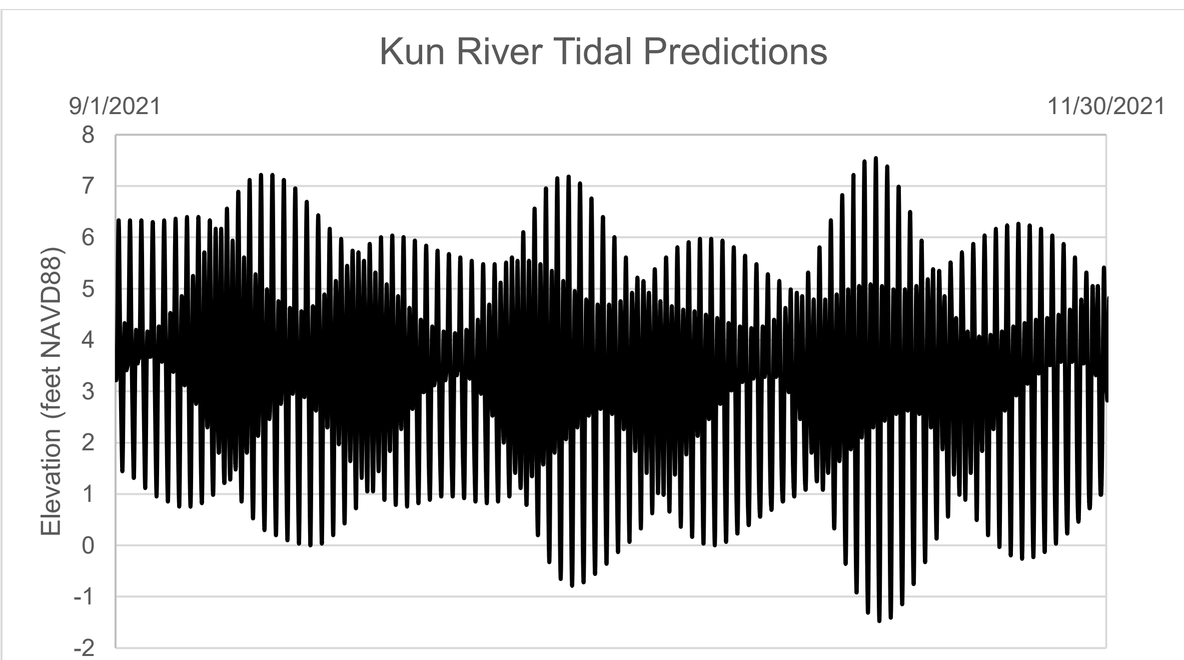

Figure 8: Kun River Tidal Prediction - Fall 2021

Combined Storm Surge Hydrograph: The representative storm surge was superimposed over the tidal predictions such that the peak of the surge coincided with the largest predicted tide.

The surge was set to begin following 2 days of normal tide to allow the model to ramp up and establish typical hydrodynamic conditions prior to the introduction of a storm surge. To account for RSLR, the storm surge hydrograph was increased by 0.64 foot representing potential sea level rise increase over a 50-year project duration. Figure 9 shows the design 100-year return period coastal surge with the typical tide and isolated surge components. All water level information was applied to the domain's western boundary in the offshore area (see Section 5.1.2).

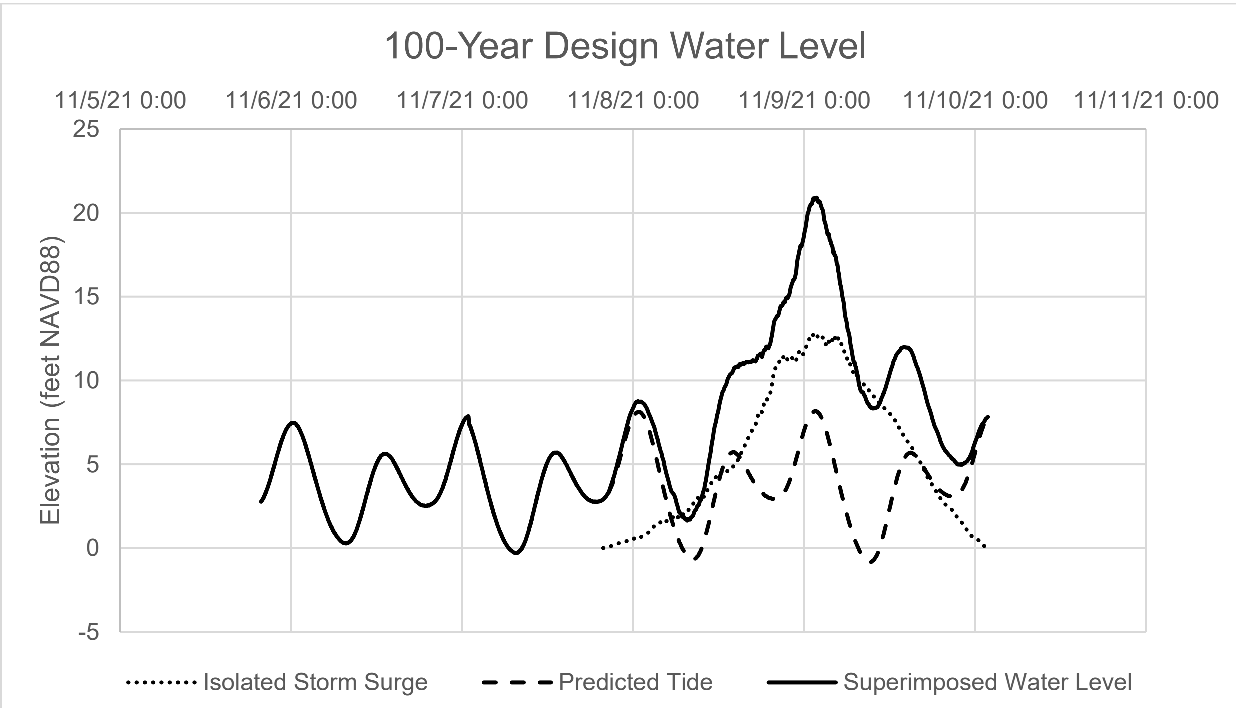

Figure 9: 100-year Return Period Storm Surge Hydrograph with Predicted Tide and Isolated Representative Surge Components

Kun River: Flow from the Kun River was included along the model boundary approximately 45 miles upstream of the runway terminal. The storm surge models assumed that the Kun River was flowing at base flow (40 percent bankfull flow). A sensitivity check comparing flood elevations at SCM during the 2 percent AEP storm surge with base flow with the concurrent 2 percent AEP storm surge and 2 percent AEP extreme runoff event as well as the 2 percent AEP runoff event with no storm surge, was performed. Design discharges for the Kun River are in Appendix B of the in the accompanying *Hydrology and Hydraulics Report* (HDR, 2022). Results of this sensitivity check showed that the additional discharge from the Kun River had a minimal effect on modeling results. The concurrent surge/riverine flood event raised water surface elevations by 0.003 feet at its peak, and the riverine event with no storm surge yielded flood elevations that did not reach the runway in most locations.

5.1.4 Storm Surge Model Limitations and Assumptions

The model developed for the coastal surge assessment at Scammon Bay is intended to be a simplistic approximation of surge inundation due to a 50-year and 100-year return period storm surge at SCM. Thus, the following limitations and assumptions should be noted:

- The model is not calibrated. Field hydrodynamic data required for calibration have not been collected. DHI recommended defaults are used for model parameters. This approach is expected to provide conservative peak water levels.
- The culvert that runs beneath the Scammon Bay Airport Runway was defined by its characteristics detailed in the 2013 Scammon Bay Airport Flood Permanent Repairs Department of Military and Veterans' Affairs (DMVA)/FEMA project plans, which are assumed to be representative of the existing culvert.
- Given that little information on bed resistance is available in the domain, bed resistance values used were assumed constant in each area and were determined by ocular estimation. This is unlikely to be the case in nature, but it provides more realistic results than neglecting roughness entirely.
- RSLR information was obtained from Nome and is assumed to be representative of the RSLR at Scammon Bay.
- Storm surge elevations were obtained from Agcklarok and are assumed to be representative of storm surge elevations at Scammon Bay.
- The shape of storm surge events was obtained from Nome and is assumed to be representative of the shape of storm surge events at Scammon Bay.
- Only one representative surge was used to determine inundation. A sensitivity analysis using different surge slopes was not performed. The surge hydrograph used was assumed to be conservative and is expected to provide higher-end values of current speed.
- Peak surge was aligned to occur simultaneously with a high tide event with the intent to represent a conservative surge elevation. A sensitivity analysis of storm surge effects at different tidal phases was not performed.
- The Kun River was assumed to be flowing at base flow (40 percent bankfull flow). Flow from other streams in the model domain were excluded and were assumed to have minimal impact of results.

5.1.5 Storm Surge Model Simulations

Two model simulations were performed: a 50-year return period (2 percent AEP) storm surge event and a 100-year return period (1 percent AEP) storm surge event. The storm surge input used for the 100-year return period model is shown on Figure 9. The storm surge input for the 50-year return period event is the same, with the peak surge elevation adjusted to match the 50-year maximum surge height provided in Table 2. The model simulations ran for 1,020 timesteps, with each timestep representing 6 minutes. The total simulation time for both models was approximately 4 days (102 hours).

5.1.6 Storm Surge Model Results

Storm surge model results were reviewed for surge inundation and potential impacts near the SCM runway, taxiway, and access road. The storm surge models resulted in a near-complete inundation of the runway and taxiway from both the 50-year and 100-year events. Maximum water surface elevation and current speeds are summarized in Table 4. The higher current speeds in the model are associated with breaching of the roadway as this area is flooded. Assuming that the improved runway is above the surge elevation, this rate of current speed is not anticipated. The fastest current speed observed not associated with a breach (current traveling around the runway/wind cone areas) is also provided since this is anticipated to be more representative of storm surge current speeds under the Future With Project condition. Figure 10 shows maximum predicted water surface elevations for the 50-year storm surge event.

Table 4: Storm Surge Model Water Elevations Results, Current Results, and Reference Elevations

Location	Elevation (feet NAVD88)
Reference Elevations	
Runway Centerline – Southeast End	+17.4 feet
Runway Centerline – At Culvert	+12.7 feet
Runway Centerline – At Taxiway	+13.5 feet
Center of Taxiway	+13.1 feet
50-Year Max Water Surface Offshore	+18.9 feet
100-Year Max Water Surface Offshore	+20.9 feet
Model Results Elevations	
50-Year Max Water Surface Elevation Near SCM	+16.1 feet
100-Year Max Water Surface Elevation Near SCM	+18.4 feet
Model Results Current	
Maximum Current Speed (breaching roadway)	7.5 feet/second
Maximum Current Speed, West Runway Terminal	4.1 feet/second
Maximum Current Speed, East Runway Terminal	2.2 feet/second
Maximum Current Speed, Culverts (either side)	2.2 feet/second

Note: NAVD88 = North American Vertical Datum of 1988

Figure 10: Peak Water Surface Elevation Results for the 50-Year Storm Surge Event

5.2 Wave Analysis

A wave analysis was conducted to determine potential wave conditions at the Scammon Bay Airport that coincide with a flooding event. MIKE 21 SW numerical model software was used to simulate wave conditions at the project site.

5.2.1 Wave Model Description

The MIKE 21 SW numerical model was used to assess wind-generated wave height and period at the project site. MIKE 21 SW, developed by DHI, is software used for developing two-dimensional spectral wave models based on a flexible (unstructured) mesh. Models developed with MIKE 21 SW simulate wind-generated waves and swell (DHI 2017b). The flexible mesh module allows for higher resolution at areas of interest (e.g., near the runway embankment) while relaxing the resolution away from the project site to increase computation efficiency.

5.2.2 Model Domain and Mesh

The model domain for the MIKE 21 SW simulations includes an approximately 30-mile fetch centered at SCM in all directions that are not obstructed by the Askimuk Mountains. The mesh contains 29,873 elements and 15,229 nodes. Mesh elements increase in size as radial distance from SCM increases. Mesh elements along the runway embankment have a fine resolution allowing for multiple (approximately three) elements per wave length. Features with potential to influence wave conditions, such as nearby roads and detention ponds, were also defined with increased resolution.

Primary sources of elevation data used to create the mesh are the same as those for the MIKE 21 HD FM and are summarized in Table 3.

Figure 11 provides a view of the entire model domain. The colors represent different elevations. Figure 12 and Figure 13 provide enlarged views of the mesh showing the finer resolution for the project site.

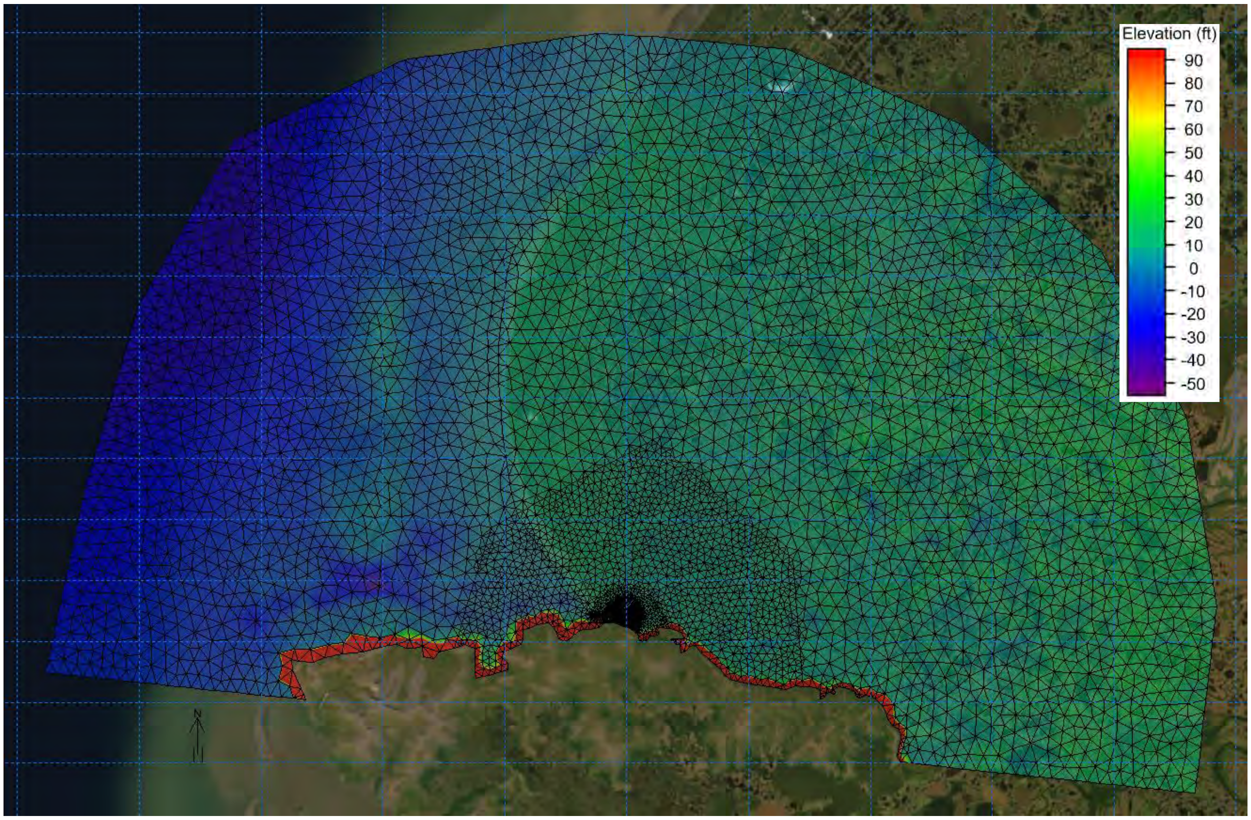

Figure 11: MIKE 21 SW Wave Model Mesh - Full Domain

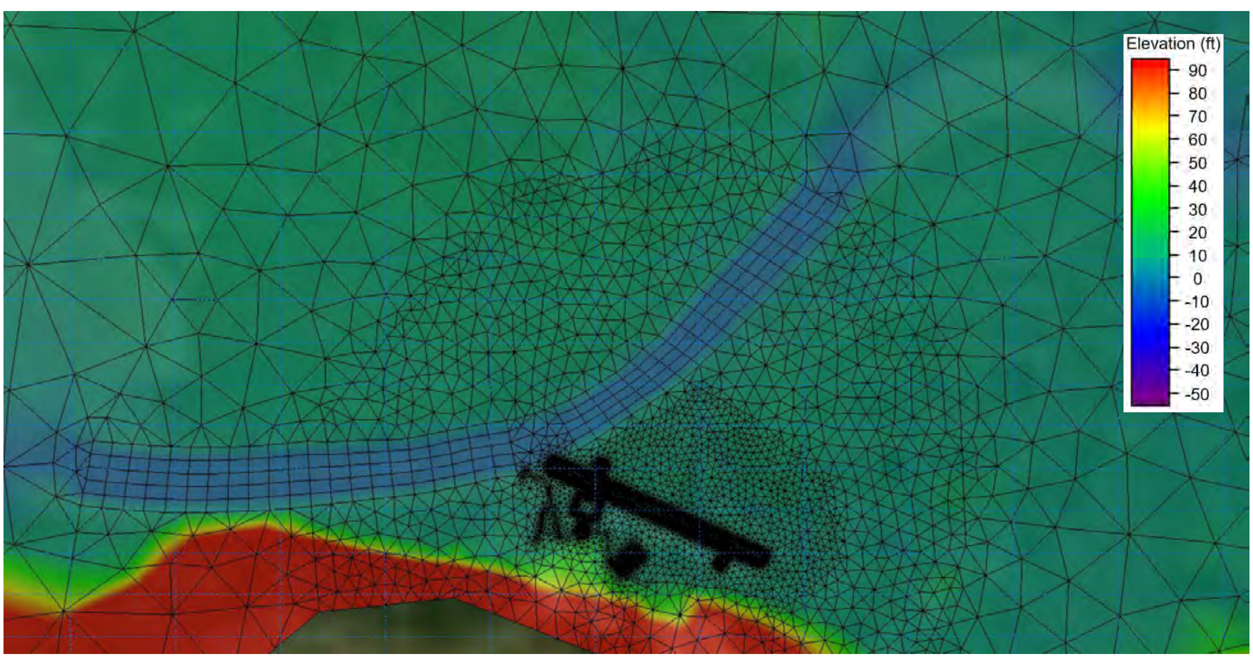

Figure 12: MIKE 21 SW Wave Model Mesh - Enlarged View Showing the Kun River and Scammon Bay

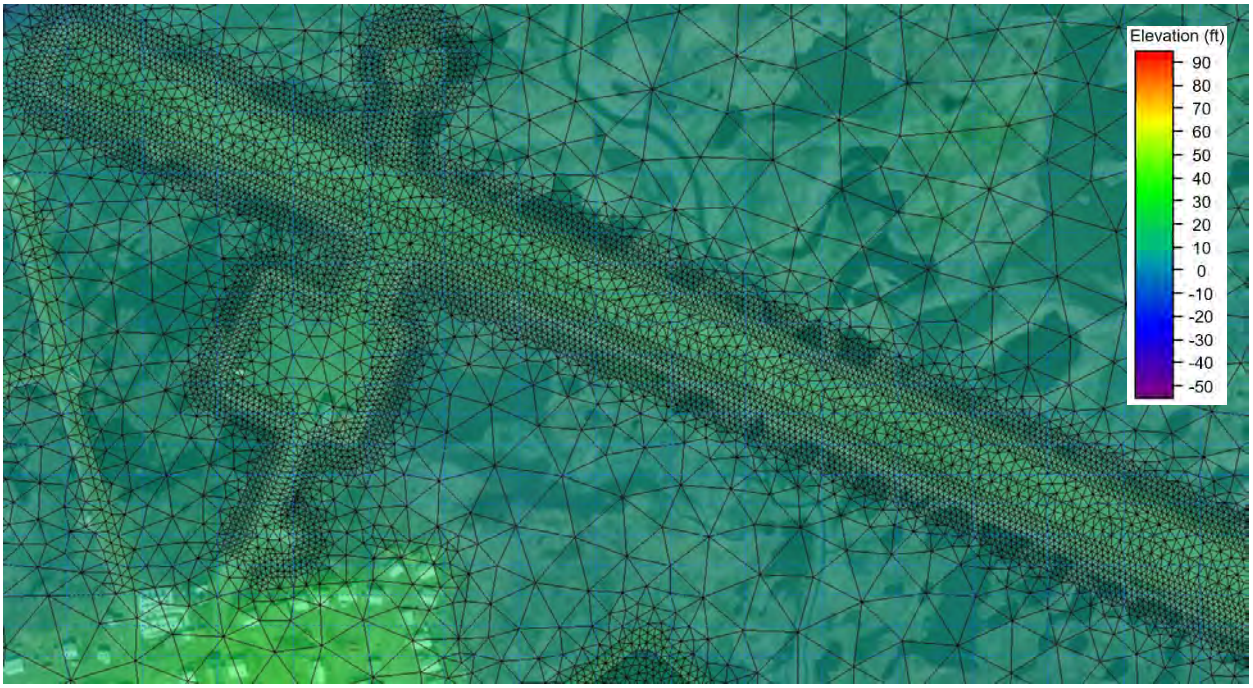

Figure 13: MIKE 21 SW Wave Model Mesh - Enlarged View Showing Refined Mesh Around Runway Embankment

5.2.3 Wave Model Boundary Conditions

Primary model inputs included water level, wind speed, and wind direction. Wave (beyond those generated by wind) and current boundary conditions were not included in the model.

Water Level: The 100-year return period water level was determined by using the maximum water level reached during the Storm Surge Analysis numerical modeling (Section 5.1). This water level was applied constantly throughout the domain for all simulations (i.e., no tidal action), as this provides a more conservative approach for simulating wave conditions.

Wind: Wind events were identified from the Scammon Bay ASOS dataset from 2010 to 2021 (temporal extent of data). These events were sorted into 16 intercardinal directions on a 22.5° interval. An extreme value analysis was performed for “All Directions” (shown on Figure 4) and for each intercardinal direction. The 100-year return period wind speeds for each direction are shown in Table 5.

Table 5: 100-year Wind Events by Direction

Direction	Direction (degrees)	Speed (knots)
North	0	30.7
North by Northeast	22.5	26.4
Northeast	45	23.2
East by Northeast	67.5	28.9
East	90	31.2
East by Southeast	112.5	33.5
Southeast	135	33.3
South by Southeast	157.5	33.3
South	180	43.9
South by Southwest	202.5	33.4
Southwest	225	34.0
West by Southwest	247.5	28.0
West	270	34.7
West by Northwest	292.5	33.4
Northwest	315	29.0
North by Northwest	337.5	24.8

5.2.4 Wave Model Limitations and Assumptions

Limitations and assumptions for the MIKE 21 SW wave model are as follows:

- The model is not calibrated. However, nomographs for wind-generated waves provided in the USACE Shore Protection Manual (USACE 1984) were reviewed for similar water depths and fetches and were found to have good agreement with the wave height and period results.
- Bed resistance was not included in this model. Although this is not a situation that can occur in nature, it provides a conservative approach to wave height estimation.
- Wind events from 2010 to 2021 are assumed to be a representative sample for the statistical analyses.
- Waves in Scammon Bay were assumed to be wind-generated waves only (i.e., swell from the ocean was not included). Swell is assumed to dissipate energy well before reaching the runway during a surge event due to their long wave periods and influence of the shallow water depths.

5.2.5 Wave Model Simulations

Sixteen model simulations (one for each intercardinal direction) varying the wind speed and wind direction were performed. The water level for each simulation was held constant for each simulation, achieving a steady-state wave condition as opposed to continually varying the water level as a tidal cycle. The constant water level was set as the maximum water level during 100-year storm surge model near the runway.

5.2.6 Wave Model Results

Wave model results were extracted at 108 locations around the SCM runway, taxiway, and access road. The largest spectral significant wave heights and associated periods were identified for each extraction location from the 16 model simulations (Figure 14). These results were then used to determine stone stability and overtopping rates at multiple locations along the perimeter of the runway/taxiway/access road.

Figure 14: Spectral Significant Wave Height Results at Scammon Bay Airport

6. Coastal Engineering Design Recommendations

6.1 Airport Surface Elevations

Airport surface elevation recommendations consider storm surge, RSLR, and wave overtopping. Recommendations are provided for both a 50-year (2 percent AEP) and 100-year (1 percent AEP) storm surge event. The RSLR component assumes a 50-year project life duration.

The criteria for determining a recommended runway elevation use critical overtopping discharge rates for revetment seawalls. Table 6 provides the critical discharge guidance from CIRIA (2007). To reduce maintenance and repair due to overtopping, setting the runway elevation to achieve an overtopping discharge at “No Damage” is recommended.

Table 6: Critical Overtopping Discharge for Revetment Seawalls

Description	Q Mean Overtopping Discharge (m^3/s per m)
No Damage	$q < 0.05$
Damage if promenade not paved	$0.05 < q < 0.2$
Damage even if promenade paved	$q > 0.2$

Source: CIRIA 2007
Note: m^3/s per m = cubic meters per second per meter.

Overtopping discharge was calculated at multiple locations (at the same locations shown in Figure 14) around the perimeter of the airport features (runway, taxiway, access road) using the 50- or 100-year return period scenarios assuming side slope of 4H:1V with an armor stone embankment. The elevation was varied until the maximums of all of the locations reviewed were at or below the critical overtopping discharge threshold. Table 7 provides the recommended Airport Surface Elevations and associated overtopping discharges.

Table 7: Recommended Airport Surface Elevations and Associated Overtopping Discharges

Return Period	Recommended Airport Surface Elevation	Overtopping Discharge (m^3/s per m)
50-Year (2% AEP)	**+18.5 feet NAVD88**	0.02 Avg; 0.05 Max
100-Year (1% AEP)	+20.5 feet NAVD88	0.01 Avg; 0.04 Max

Note: m^3/s per m = cubic meters per second per meter; AEP = Annual Exceedance Probability; NAVD88 = North American Vertical Datum.

Airport usability due to storm surge is associated with the probability of occurrence of an event that exceeds the critical overtopping rate of "no damage" over the project life duration. In a storm surge event where the overtopping exceeds this value, it is expected that conditions will exist that do not allow safe use of the runway, such as flooding, damage to the runway or runway safety area, or debris thrown up onto the runway. Unless significant damage is sustained, the duration in which the runway would be unusable would be on the order of a few days to a week. This is based on observations of Western Alaska storm surge hydrographs in which storm surge events will often reach a maximum surge level and sustain that level for 1 to 3 days before receding. It is then assumed that some form of cleanup and minor grading is required to return the runway to a usable condition. Probability of occurrence for the 50- and 100-year storm surge events over varying project life durations is provided in Table 8.

Table 8: Storm Surge Probability of Occurring at Least One Time over the Project Life Duration

Project Life Duration (years)	50-Year Storm Surge (2% AEP)	100-Year Storm Surge (1% AEP)
25	39.7%	22.2%
30	45.4%	26.0%
50	**63.5%**	**39.5%**
75	77.9%	52.9%
100	86.3%	63.4%

Note: AEP = Annual Exceedance Probability.

6.2 Runway Relocation

Historical georeferenced aerial imagery from 1948 and 1977 was gathered for the project area. Riverbank positions were delineated based on the apparent water-land interface. When overlaying these riverbank positions with a recent (2020) aerial image, it can be inferred that the Kun River is migrating towards the runway, albeit slowly. Riverbank retreat near the runway terminal from 1948 to 2020 ranges from 115 to 190 feet, which equates to 1.6 to 2.6 feet per year. Similarly, riverbank retreat from 1977 to 2020 near the terminal ranges from 55 to 100 feet, which equates to 1.3 to 2.3 feet per year.

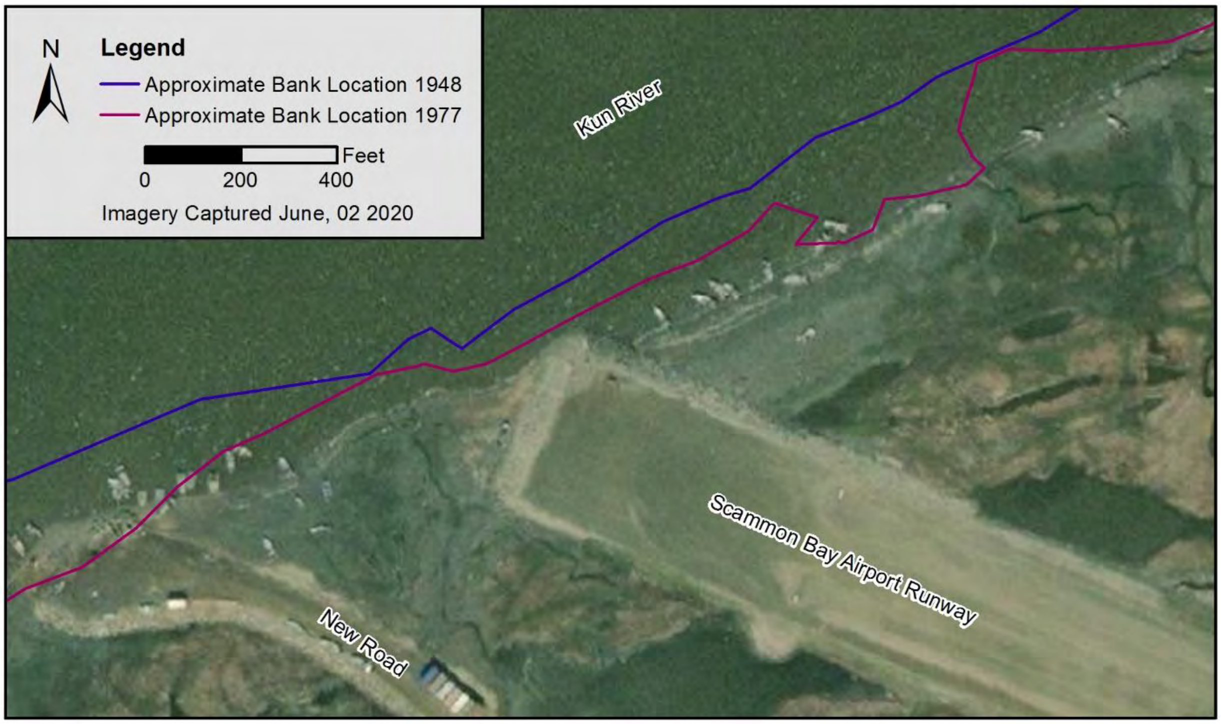

Figure 15: Historical Riverbank Position Superimposed over Recent (2020) Aerial Imagery

Assuming a conservative migration rate of 3 feet per year, the runway would need to be relocated 150 feet from the current riverbank location for a 50-year project life duration. Thus, considering the slightly oblique alignment of the runway and protrusion of the runway terminal beyond the existing riverbank position, the runway would need to shift approximately 340 feet along its current alignment. Figure 16 shows the proposed shifted runway graphically in comparison to the existing runway location. When the runway is shifted, it does not appear that any significant flow paths will be displaced. The distance from the runway terminal to the edge of the wetlands (area where terrain elevation abruptly increases) is shortened from approximately 550 feet to 500 feet with the proposed shift.

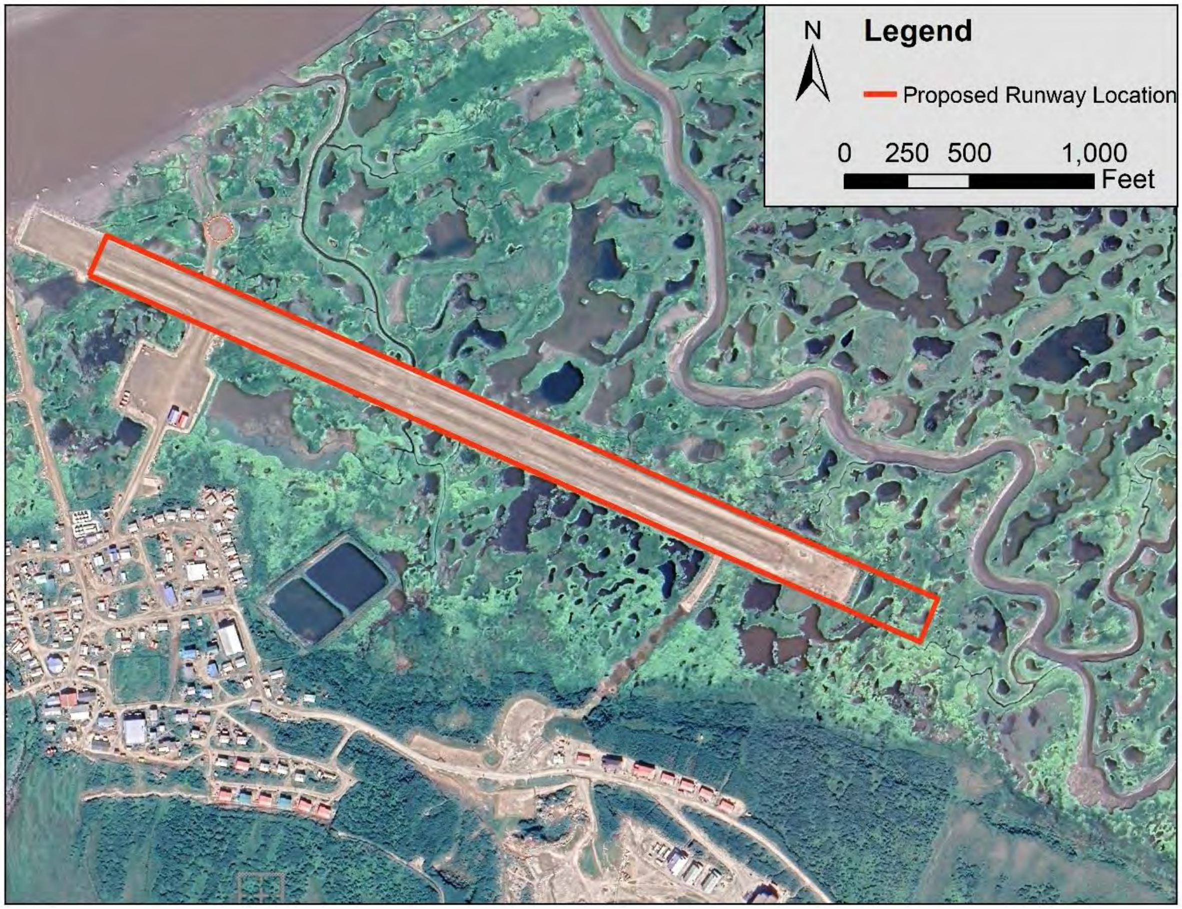

Figure 16: Proposed Runway Relocation

6.3 Erosion Protection

Erosion protection is recommended along the perimeter of the runway, taxiway, and access road to mitigate damage to the embankment due to waves and to reduce wave overtopping, which can damage the surface of the airport features. A traditional buried-toe armor rock revetment, a suitable long-term option for erosion protection that requires minimal maintenance, was the initial method assessed for shoreline protection. This method was assessed using the approximate 4H:1V existing side slopes of the runway. Subsequently, alternative erosion protection methods were assessed to evaluate more cost-effective solutions. These methods included an armor rock revetment with an above-ground toe at various slopes as well as a marine mattress.

6.3.1 Buried-Toe Armor Rock Revetment Method

If a traditional buried-toe armor rock revetment is used, two revetment sections are recommended for different areas of the project area. Each revetment section is a two-layer revetment consisting of a primary amor stone and filter stone material with an underlaying geotextile filter fabric. An embankment slope of 4H:1V was selected based on the proposed repair design in the 2013 Scammon Bay Airport Flood Permanent Repairs DMVA/FEMA project drawings. Armor rock revetments can be constructed at steeper slopes (generally as steep as

2H:1V); however, the size of primary armor stone material and subsequently the layer thickness/volume of stone increases as a result.

6.3.2 Primary Armor Stone (Buried-Toe Armor Rock Revetment Method)

A stone stability analysis was performed to assess primary armor stone size needed for potential waves and currents during a flood event. From this analysis, it was found that wave conditions were the controlling factor. Ice was not considered for armor stone size for the following reasons:

1. The structure will generally be above the tidal level at which ice plucking is not a concern.
2. The runway terminal is a significant distance away from the Kun River, and it is not expected that ice breakup in the river will affect the stability of the revetment.
3. Storm surges generally occur during fall, when sea ice is not present in Scammon Bay.

Stone stability using both the van der Meer and Hudson methodologies was calculated at multiple locations around the runway, taxiway, and access road. From these calculations, it was determined that the maximum required median primary armor stone weight for the van der Meer and Hudson methodologies is 300 lbs. and 400 lbs., respectively. Required median stone size varied along the perimeter runway, taxiway, and access road, with the larger stone calculated at the western runway terminal, primary wind cone, and western embankment of the taxiway and access road. Calculated median stone weight around the perimeter of the runway, taxiway, and access road is shown visually on Figure 17.

Due to the short-period waves anticipated, a riprap-type gradation (wide/uniform gradation) is recommended in lieu of a coastal armor-type gradation (narrow gradation). The riprap-type gradation is generally easier to produce and thus should have a reduced cost compared to a coastal armor-type gradation. The recommended gradation is provided in tabular form in Table 9 and shown graphically in Figure 18. This gradation is the same as ASTM 6092 R-700 with the exception that it is “percent lighter by count” and not “percent lighter by weight.” Also, this gradation is very similar to a DOT&PF Class III gradation.

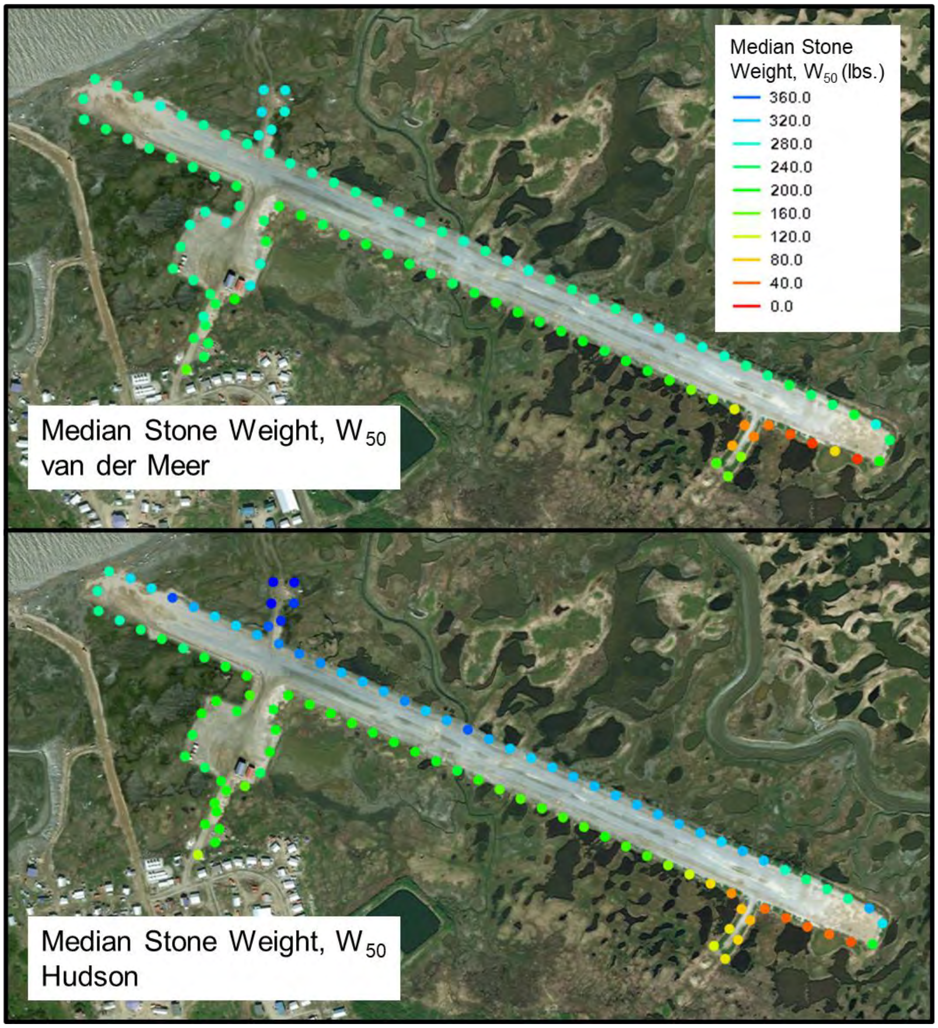

Figure 17: Median Armor Stone Weight using the van der Meer (upper image) and Hudson (lower image) Methodologies

Table 9: Recommended Primary Armor Stone Gradation (PA-700)

Stone Weight, lbs.	Percent Lighter by Count
1,500	100
700	50–100
300	15–50
60	0–15

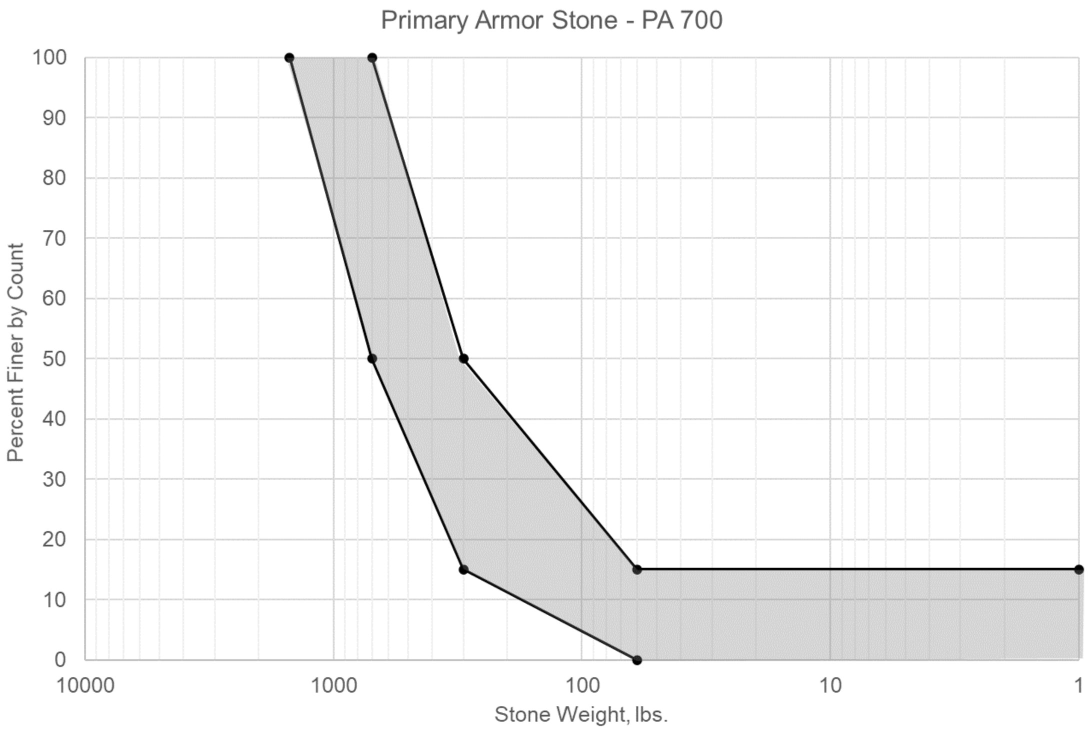

Figure 18: Recommended Primary Armor Stone Gradation (PA-700)

6.3.3 Filter Stone (Buried-Toe Armor Rock Revetment Method)

Filter stone is recommended to be placed under the primary armor stone to provide distribution of the armor stone weight against the underlaying geotextile filter fabric and improved interlocking with the armor stone layer. The filter stone size follows guidance for the USACE *Shore Protection Manual* (USACE 1984) and EM 1110-2-1614 (USACE 1995). The upper bound of the filter stone was selected to match the lower bound of the primary armor stone to increase yield of the processed quarry stone. The recommended gradation for the filter stone is provided in tabular form in Table 10 and shown graphically in Figure 19.

Table 10: Recommended Filter Stone Gradation (F-30)

Stone Weight, lbs.	Percent Lighter by Count
60	100
30	0–50
5	0–15

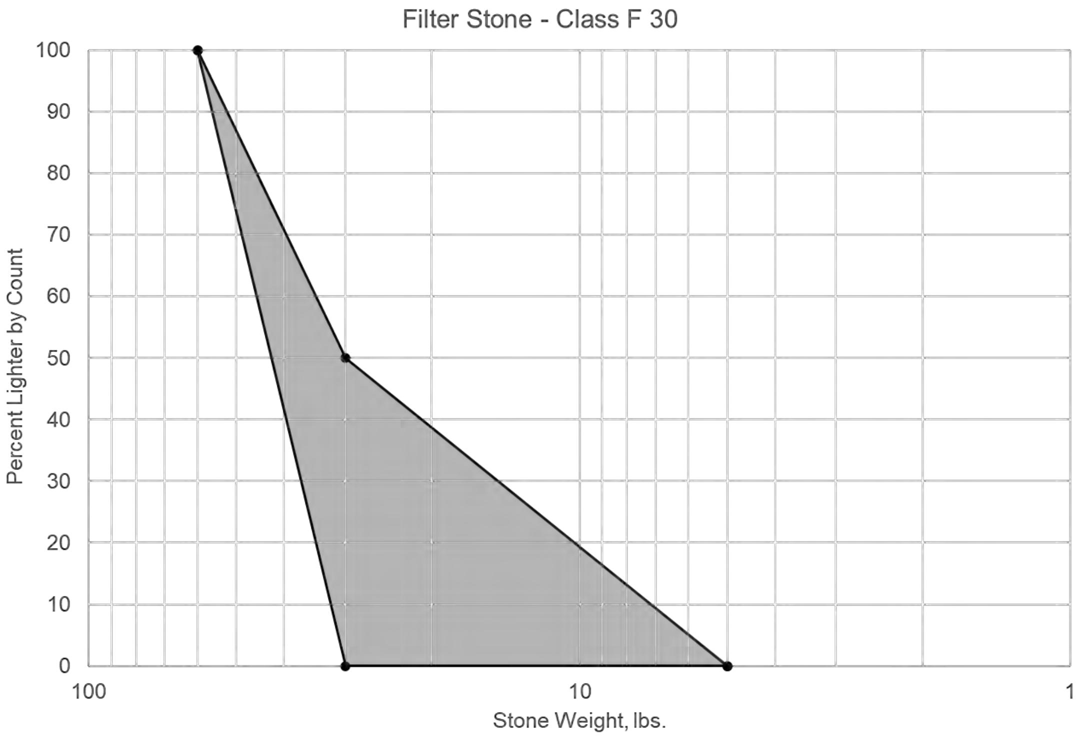

Figure 19: Recommended Filter Stone Gradation (F-30)

6.3.4 Revetment Typical Sections (Buried-Toe Armor Rock Revetment Method)

Two revetment sections are recommended that vary in the design of structure toe. The Erosion Protection – Type I revetment includes a more substantial buried toe that is recommended for areas along the runway, taxiway, and access road with moderate to extreme scour potential. The Erosion Protection – Type II revetment uses a simple entrenched toe with *in-situ* backfill. This section is recommended in areas along the runway, taxiway, and access road with low scour potential. Each toe design follows guidance from EM 1110-2-1614 and should be buried 4 feet below the existing grade to prevent scour. Scour depths were assumed to be equivalent to 1.0-1.5 times the significant wave height.

The revetment typical sections for erosion protection are provided in Figure 20 and Figure 21. Both sections utilize the same primary armor and filter material. The Erosion Protection – Type II areas are expected to have less wave energy and thus could utilize a smaller Primary Armor stone (W_{50} of approximately 200 lbs.). Requiring two primary armor stone and consequentially two filter stone material types is expected to complicate the construction logistics, which may offset any gains from using a smaller material. Given this unknown, a potential procurement strategy to solicit a lower cost is to provide an optional Erosion Protection – Type II with a smaller section utilizing a smaller primary armor stone and filter stone material. Minimum armor and filter stone layer thicknesses (3') are specified to be two times the median stone diameter (D_{50}).

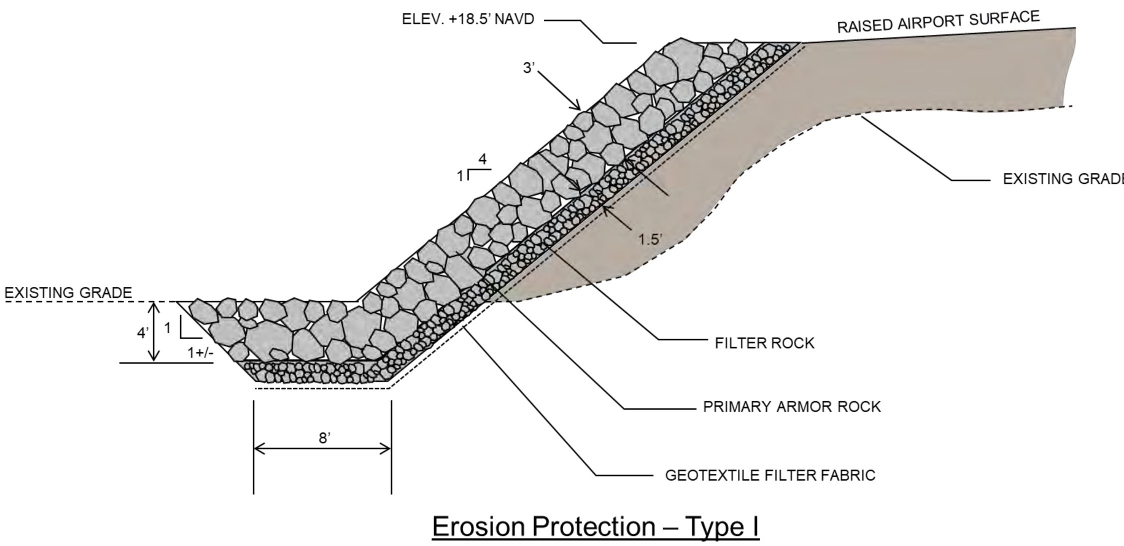

Figure 20: Erosion Protection - Type I Recommended Typical Section

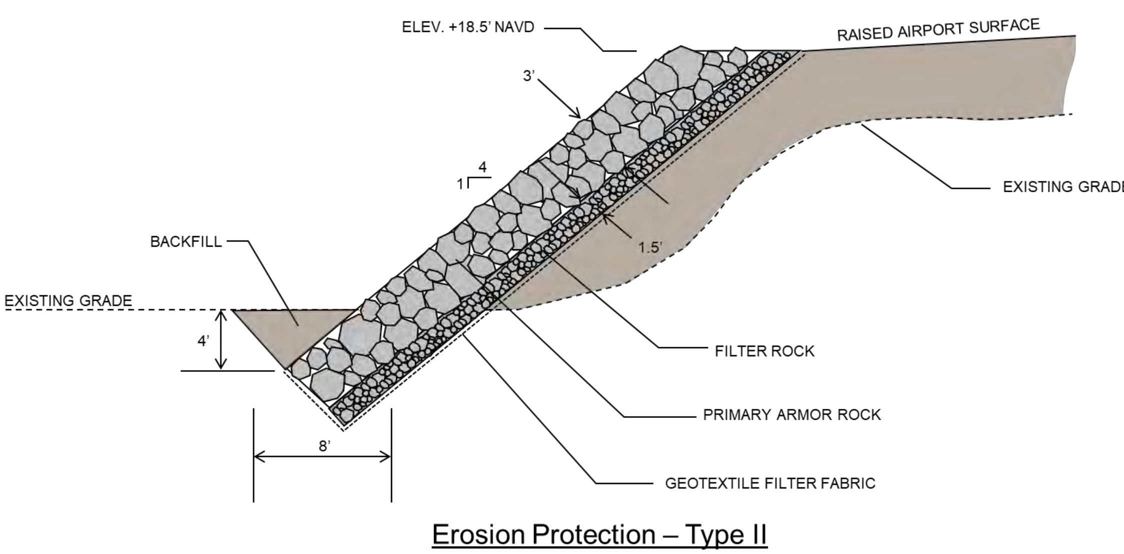

Figure 21: Erosion Protection - Type II Recommended Typical Section

6.3.5 Armor Rock Revetment with an Above-Ground Toe

An armor rock revetment with an above-ground toe reduces the excavation and, when constructed at steeper angle than 4H:1V, requires less material thus reducing the initial construction cost.

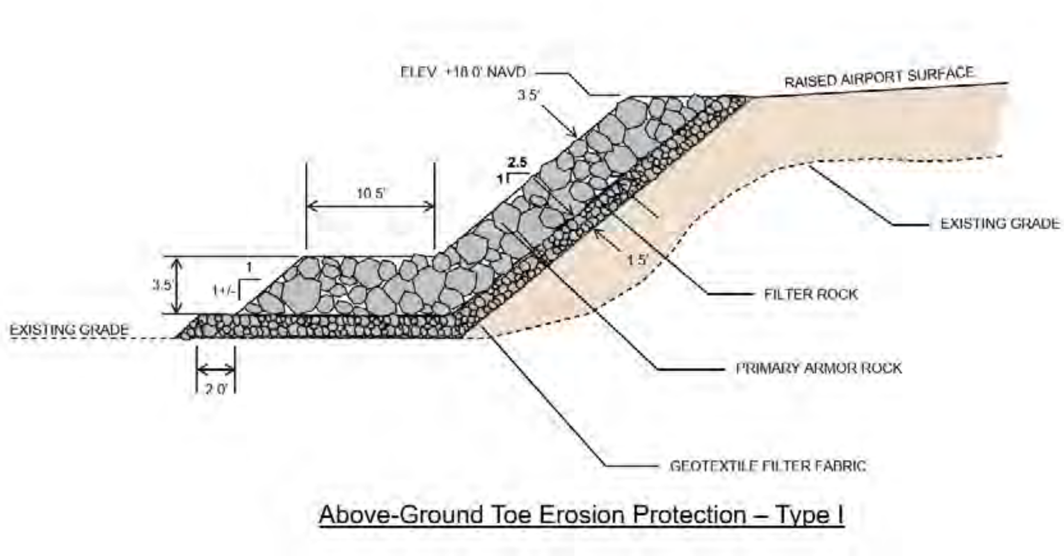

Figure 22: Example of an Above-Ground Toe Erosion Typical Section (Type I 2.5H:1V Concept Shown)

Several configurations of revetments with an above-ground toe were assessed to quantitatively compare construction cost to the initial buried-toe revetment with a 4H:1V slope. These include the following:

- 2.5H:1V & 2H:1V Concept – This concept uses three typical sections covering the entire airport perimeter. In areas with the largest waves, the revetment uses 2.5H:1V slope. In areas with moderate wave action, the revetment uses a 2H:1V slope. In areas with minimal wave action, the revetment uses filter rock material as the primary protection.
- 2.5H:1V Concept – This concept uses three typical sections. In areas with the largest waves as well as moderate waves, a 2.5H:1V slope is used, however, the armor rock size is different creating two different sections. A third section using only filter rock is used in areas with minimal wave action.
- 2H:1V Concept - This concept uses three typical sections. In areas with the largest waves as well as moderate waves, a 2H:1V slope is used, however, the armor rock size is different creating two different sections. A third section using only filter rock is used in areas with minimal wave action.
- 1.5H:1V Concept - This concept uses three typical sections. In areas with the largest waves as well as moderate waves, a 1.5H:1V slope is used, however, the armor rock size is different creating two different sections. A third section using only filter rock is used in areas with minimal wave action.

A summary of these different concepts is provided in Table 11 which also includes a conceptual cost difference from the buried-toe revetment.

Table 11: Summary Comparison of Armor Rock Revetments

Revetment Concept	Type I Armor W_{50}	Type II Armor W_{50}	Revetment Cost Contribution
Buried-Toe 4H:1V Concept	300 lbs.	300 lbs.	$67.7M
Above-Ground Toe 2.5H:1V and 2H:1V Concept	370 lbs.	380 lbs.	$30.1M
Above-Ground Toe 2.5H:1V Concept	520 lbs.	380 lbs.	$30.9M
Above-Ground Toe 2H:1V Concept	520 lbs.	380 lbs.	$31.9M
Above-Ground Toe 1.5H:1V Concept	790 lbs.	590 lbs.	$33.3M

Notes:
1. Revetment cost contribution includes in-place costs of primary armor stone, filter stone, geotextile fabric, and any excavation or fill required.
2. Primary armor stone unit price used is $240 per ton
3. Filter stone unit price used is $200 per ton
4. Geotextile filter fabric unit price used is $10 per square yard
5. Excavation unit price used is $25 per cubic yard
6. Backfill unit price used is $25 per cubic yard
7. A contingency of 30% was used in the cost contribution calculation

6.3.6 Marine Mattress

A potential drawback from using a traditional armor rock revetment, especially in remote locations without suitable local armor material, is the capital cost to construct the project. A marine mattress can be used in environments with low to moderate wave conditions and are advantageous in that they can utilize much smaller, less expensive rock. In other words, the ability to produce high quality large armor stone is not a requirement. A marine mattress is made of geotextile grid in the shape of a 'mattress' that contains small rock. Mattresses are laid in a single layer. Mattress thickness come in a variety of sizes (6", 9", 12", 18", and 24"). Mattresses are generally about 20 to 30 feet long (35 feet max) and 5 feet wide. The mattress can be filled in place or fabricated offsite and placed on a prepared foundation using specialty spreader bars. Figure 23 provides a typical schematic of a marine mattress. An example of marine mattress used as erosion protection is shown in Figure 24.

Figure 23. Typical schematic of a marine mattress (Photo source: Tensar.com)

Figure 24. Example of a marine mattress used for erosion protection (Photo source: tensar.com)

It is anticipated that a 12" mattress placed at 1.5H:1V slope can handle the wave conditions at Scammon Bay. The east side of the runway would require excavation to achieve the 1.5H:1V slope or, alternatively, the marine mattress could be placed directly on grade with minor excavation. These concepts are shown schematically in Figure 25 and Figure 26. The west and mid runway, which have a lower existing grade, would require fill to achieve the 1.5H:1V slope. This concept is shown schematically in Figure 27. For comparison, the marine mattress cost component as shown would be $11M (roughly a third less expensive than any of the armor rock options).

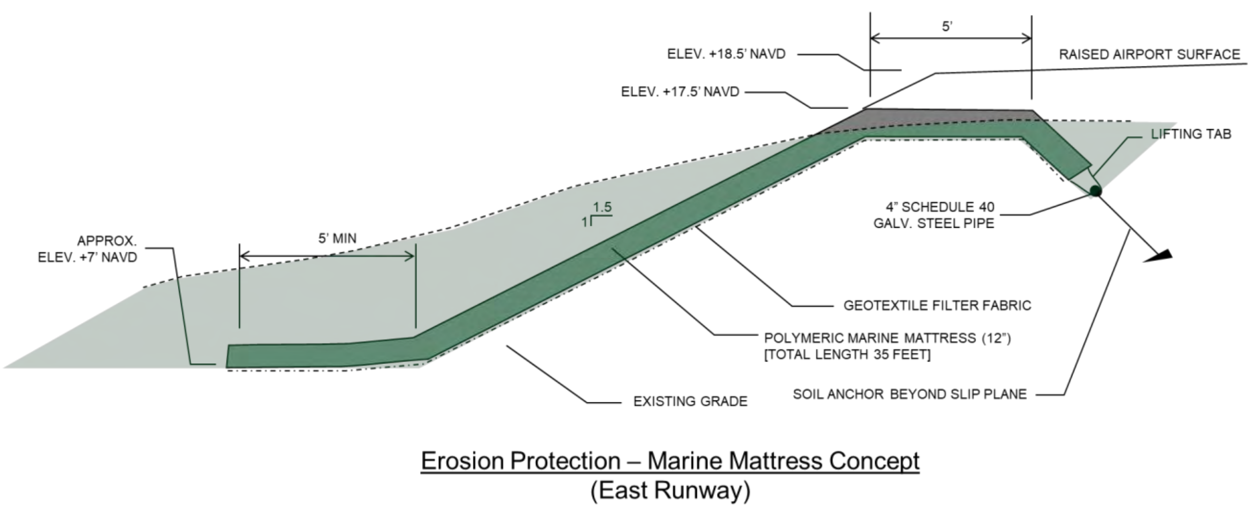

Figure 25. Marine mattress schematic for the east side of the runway (1.5H:1V slope)

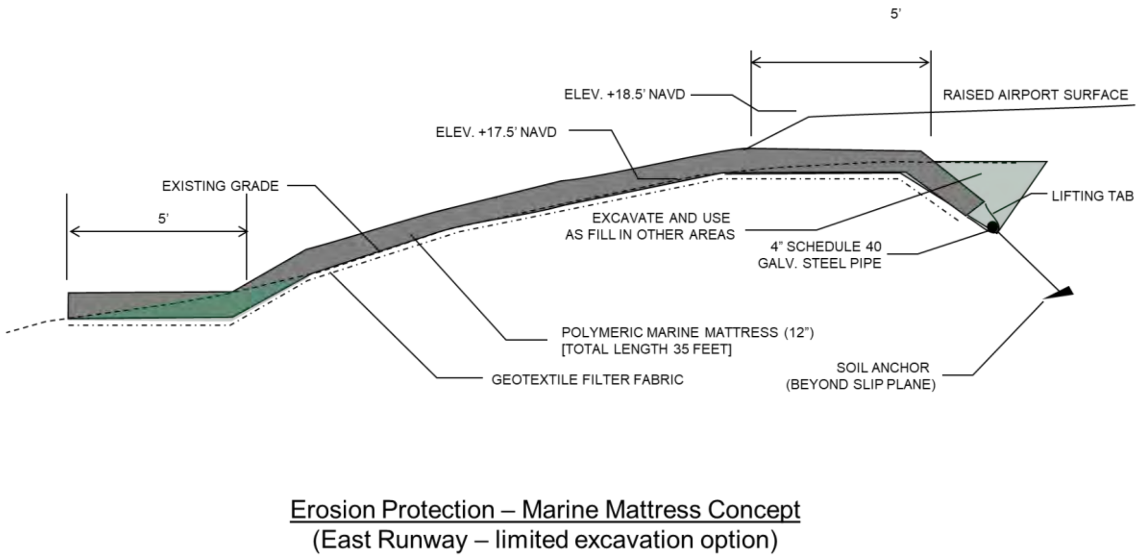

Figure 26. Marine mattress schematic for the east side of the runway (use existing slope)

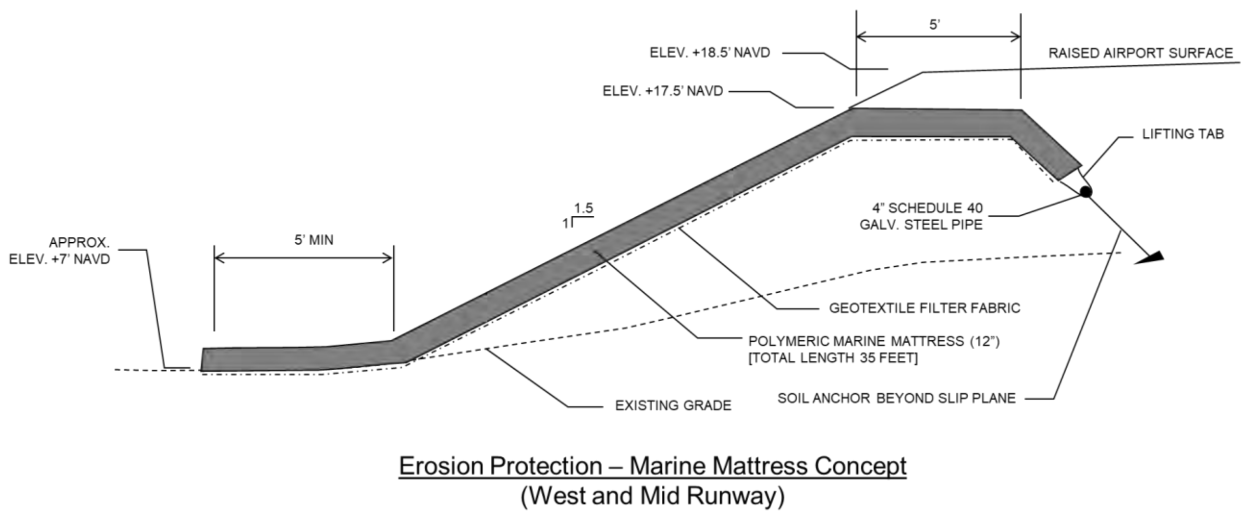

Figure 27. Marine mattress schematic for the west side and mid runway

6.3.7 Other Alternatives Not Assessed

Other alternatives not assessed in detail that may warrant some future consideration include:

- Articulating Concrete Block Mats
- Gabions
- Sacrificial Rock Material/Berm Revetments

Articulating block mats include multiple concrete pieces that interlock via geometry (i.e., puzzle piece), strung together using cable, chain, or rope, or combination of the interlocking geometry and cabling. They can be used in light to moderate wave environments and have a minimal profile compared to armor rock revetments. Similar to a marine mattress, they require a prepared subgrade.

Gabions are like marine mattresses in that they are units that contain small rock, however, are smaller and block-like in geometry. They can be placed more vertically, such as along a riverbank, or at a prescribed slope. There are various gabion materials including zinc and galvanized steel (not suitable for a coastal environment), stainless steel, and HDPE/plastic.

Sacrificial rock material/berm revetments are like the revetments presented in this section, but instead utilize smaller stone and have a larger cross-section, expecting to have movement within the structure during large events. Material is either simply lost (sacrificial) or a berm feature is redistributed by the storm developing a more stable 'S' shape. These structures generally require more material than a traditional armor rock revetment (larger cross section) but may be a benefit economically by using smaller material if the unit price of the rock is significantly cheaper than the equivalent larger material needed for a traditional revetment.

7. Summary

This document presents a preliminary coastal analysis and recommendations pertaining to coastal engineering components as part of a larger feasibility study for improvements to the Scammon Bay Airport. Readily available metocean and elevation data were gathered to develop a coastal storm surge model and spectral wave model to determine potential water levels, current speeds, and wave conditions at the runway, taxiway, and access road. Based on this analysis, airport surface elevation for a 50-year return period storm surge (2 percent AEP) is +18.5 feet NAVD. A 340-foot shift in the runway location along its current alignment away from the Kun River is also recommended based on historical migration rates of the riverbank near the runway terminal. To mitigate against erosion, multiple revetment sections were developed and compared using conceptual costs for protection of the runway, taxiway, and access road perimeters.

The following are key recommendations regarding the feasibility of improving the Scammon Bay Airport:

1. To reduce potential for flood inundation, damage from current flow due to breaching, and damage from flooding and wave overtopping, it is recommended to increase the elevation of the Airport Surfaces. For a 2 percent AEP, an elevation of +18.5 feet NAVD88 is recommended.
2. Relocating the runway along its current alignment at 340 feet is recommended for a project life duration of 50 years.
3. Erosion protection (armor rock revetment or marine mattress) is recommended around the perimeter of the runway, taxiway, and access road is recommended to mitigate potential erosion and scour due to waves and currents during a flood event.

4. In areas expected to sustain larger wave condition, a section with a toe designed for moderate to severe scour is recommended.
5. Different sections that utilize smaller typical sections should be considered in areas of the airport perimeter that experience smaller wave action.
6. Erosion protection utilizing marine mattresses (or other alternatives to armor rock revetment) should be given consideration, given the infrequent and moderate wave conditions expected to reduce overall construction cost.

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