# SR 105 MP 20.05 Graveyard Spit: Hydraulic Design Report


**Julie Heilman PE, State Hydraulic Engineer**
**WSDOT Headquarters Hydraulics Office**

**Garrett Jackson, Hydrologist**
**WSDOT Headquarters Hydrology Program**

**Dave Michalson, Engineer**
**USACE Seattle District**

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# 1 Introduction

WSDOT and the Department of Ecology, with the U.S. Army Corps of Engineers as a design partner, formed a team to address and arrest coastal erosion along the north shore of Willapa Bay. State Route 105 (SR105), the North Cove estuary, and the Shoalwater Tribe Reservation are all threatened by transgression of the shoreline . This project is located along SR105 and in the vicinity of MP 20.05 (Figure 1-1). SR105 runs along this shore and has been extensively modified and strengthened by various shoreline protection schemes over the last 70 years. This project will protect an approximately 1500-foot section of roadway, the community of Tokeland, and a tidal estuary through the use of a hybrid dynamic revetment/dune restoration structure.

Dynamic revetments are relatively new, but the technique has been used successfully in the Pacific Northwest and elsewhere. The strategy is to emulate natural cobble beaches, which are resilient to wave action, absorbing wave volume and energy. The current project will be the most extensive such structure built to date. This is a different approach than the typical “hard” shoreline armoring, such as with concrete or rock bulkheads, seawalls, or levees.

WSDOT provided the hydraulic design and team leadership, permitting, and real estate services. Ecology provided permit input, technical input to hydraulic design, public involvement, and stakeholder outreach. The Corps of Engineers provided design input through hydrodynamic modeling and participation on the hydraulic design subcommittee.

This project will be technologically advanced through the use of a two-dimensional hydrodynamic model developed by the US Army Corps of Engineers and innovative design techniques researched and developed by the WSDOT Hydraulics Section.


**Figure 1-1. Project Vicinity Map**

# 2 Site Assessment

## 2.1 Environmental setting

This section describes the coastal geology, geomorphology, and tidal and wave conditions that affect the shoreline in the project area. Extensive background materials exist from previous projects at the site and nearby. The discussion below therefore draws heavily upon the Corps of Engineers' study of long-term alternatives for coastal protection (2018), the United States Geologic Survey coastal erosion study (USGS, 2004), and analyses related to highway protection (WSDOT 1997; WSDOT 2015).

## 2.2 Geology and Soils

Willapa Bay has been an embayment in Washington's coastline since at least the early Pleistocene (Li and Komar, 1992). When sea level was lower, the Willapa Bay was an incised valley. Accretion of sand onto Cape Shoalwater and the Long Beach Peninsula did not begin until the rise in sea level at the beginning of the Holocene, about 12,000 years ago. Sediment from the Columbia River was the source for much of this sand.

More recently, a succession of parallel beach ridges developed on the north side of the bay (Figure 2-1). The oldest is Kindred Island (now a peninsula due to diking and drainage modifications). Kindred Island is the most landward of the ridges, yet lies at an elevation of less than +13 feet MLLW. It dates back to about 1100 years ago. The Tokeland peninsula rises up to 15 feet above mean lower low water (MLLW) and dates back to about 300 years ago (Morton, et. Al, 2007). These two ridges coincide with ages of subduction zone earthquakes, suggesting that they formed after earthquake-induced subsidence (Morton et. Al, 2007). The Empire/Graveyard spit, and Cape Shoalwater, are the youngest of the beach ridges, and related to various controls on the Willapa Bay entrance channel (USGS, 2004). A cross-section developed by the USGS in the Graveyard Spit area (Figure 2-2) shows that the area underlain at depth by Pleistocene littoral sediments, with coarser channel fill overlying that. Above the channel fill is mud, peat, and sand.

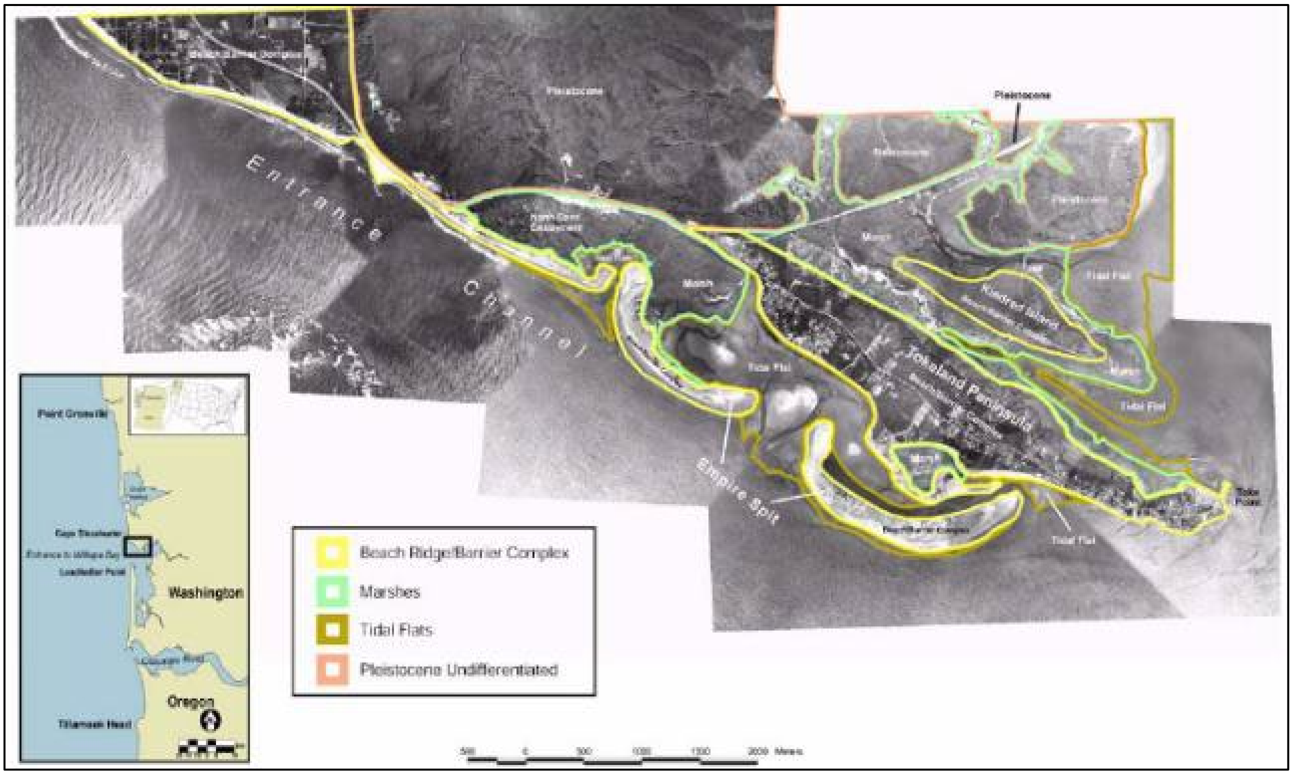


**Figure 2-1. Coastal Geology of the Project Area. Note the parallel active and older beach ridges (Morton, et. al, 2002).**

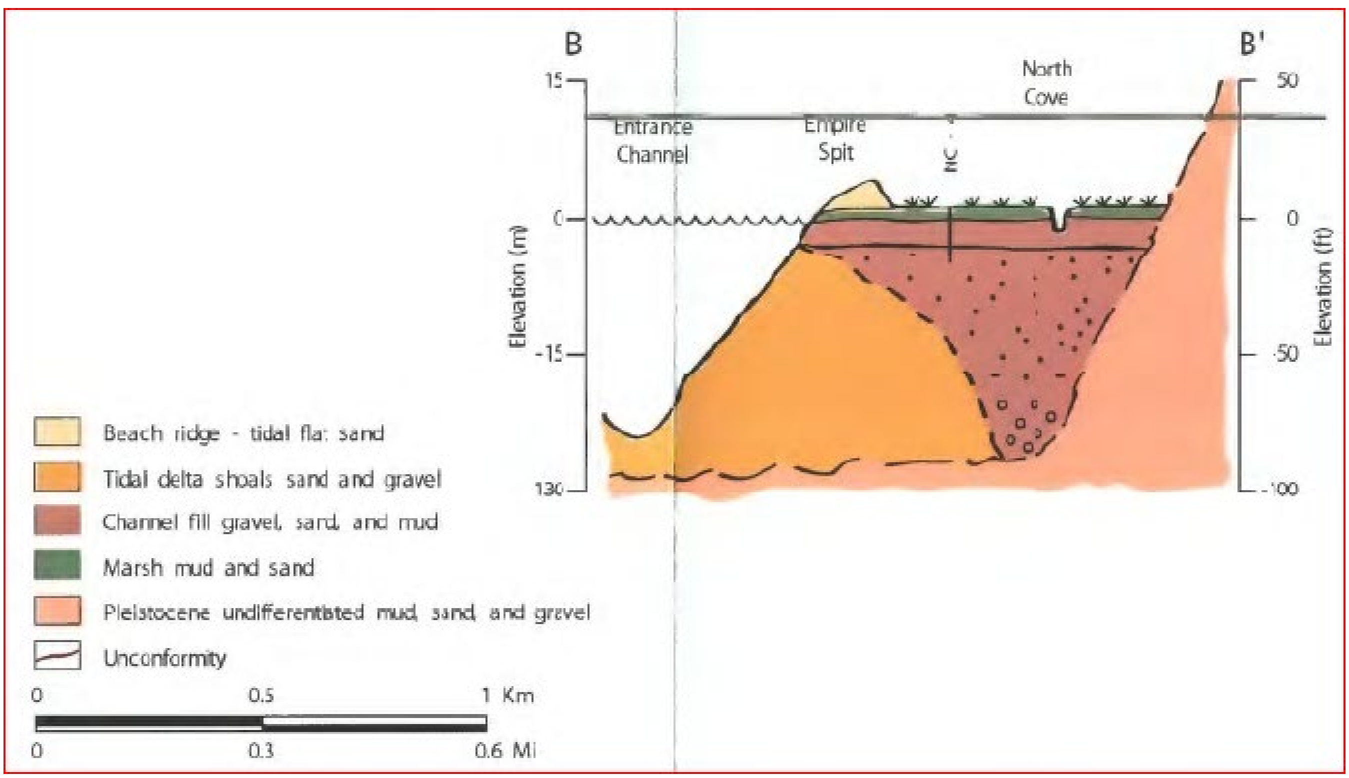


**Figure 2-2. Geologic cross-section of Graveyard Spit (USGS, 2002)**

## 2.3 Morphology of the Willapa Bay Entrance Channel

Erosion at the project site is strongly related to dynamic changes in the location and configuration of the Willapa Bay entrance channel. The entrance channel is bounded to the north by a marine terrace underlain by semi-indurated sedimentary deposits, and to the south by the sand spit that defines the Long Beach Peninsula. A series of shoals extend off the tip of the Long Beach Peninsula, and are periodically crossed by smaller channels.

The entrance channel is a highly dynamic feature that shifts in response to changes in sand deposition patterns and currents. Prior to the late 1800s the channel was located to the south and wrapped around the tip of Cape Shoalwater (Figure 2-3). Between the late 1800s and the 1960s the channel shifted steadily northward as it eroded into the cape. By 1967 Cape Shoalwater was virtually gone and the north side of the channel was running along the base of the marine terrace near the current alignment of SR 105. Graveyard Spit migrated to the northeast and merged with the interior spit that forms the North Cove embayment. The spit is now a fragmented landform that extends southeast from the SR 105 embankment south of the jetty. It is anchored and aligned by erosion resistant terrace deposits (U.S. Army Corps of Engineers, 2009a). However, these terraces, which are held together by sod that were part of the quiescent estuary, are being steadily eroded landward (Figure 2-4).

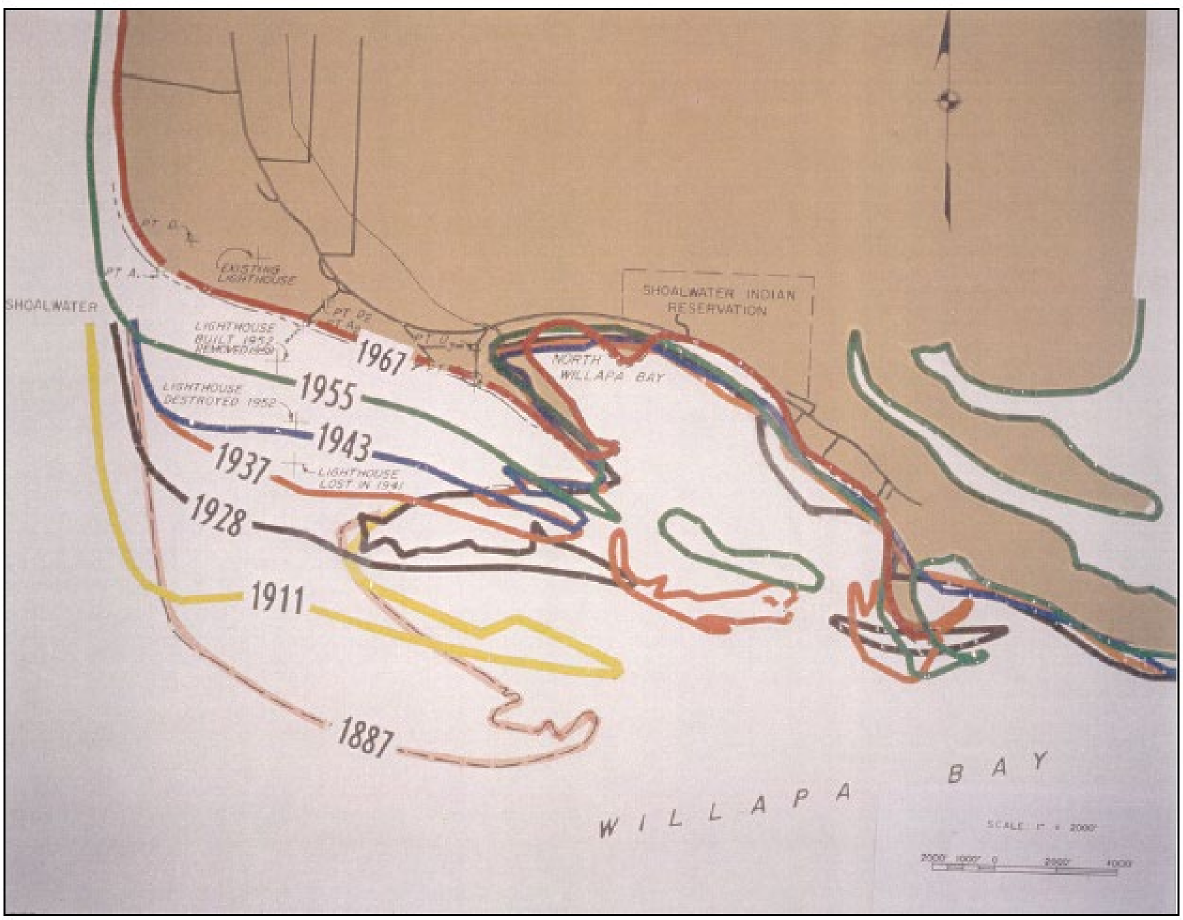


**Figure 2-3. Historical erosion of Cape Shoalwater and northward migration of the entrance channel.**

**Figure 2-4. Remnant estuary marsh sod forming a terrace.**

A series of barrier islands often referred to as Empire Spit extend to the southeast from the tip of Graveyard spit in front of the Tokeland Peninsula. Dunes on these spits have diminished in recent years, in response to reduced sand supply from eroding beaches to the northwest. Breaches in 1995 and 2003 divided Empire Spit into three narrow islands (U.S. Army Corps of Engineers, 2009b). Graveyard and Empire Spits protect portions of SR 105 from wave action, so the loss of these features has increased risks to the highway.

Historic bathymetric data indicate the northward migration of the entrance channel is slowing as it encounters the erosion-resistant terrace deposits, and there has even been some southward shifting of the channel thalweg in recent years (U.S. Army Corps of Engineers, 2009a). The alignment and extent of the erosion-resistant substrate is not fully understood, but the U.S. Army Corps has speculated that Empire Spit will continue to pivot to the northeast. Separately, the Department of Ecology (2018) developed shoreline change predictions based on historic erosion rates to estimate future shoreline change on the northern shoreline of Willapa Bay (Talebi et al 2017). This model projects the shoreline recession to reach SR-105 between 2020 and 2030 in the North Cove area. These estimates do not take into account local hard points, natural or man-made, which may inhibit transgression of the shoreline. For more discussion, see Section 7 of Appendix A.

The Corps of Engineers has congressional authorization to maintain a navigation channel to Willapa Bay that is 500 feet wide and 26 feet deep at Mean Lower Low Water. However, routine dredging ceased in the 1970s due to difficulties in maintaining a channel in the shifting sands at the mouth of the bay (U.S. Army Corps of Engineers, 2009b). Some limited dredging was performed in 2013 along Empire Spit to provide sand for the Corps' dune restoration and coastal protection project. Sand was also pumped from the channel for the construction of the SR 105 jetty in 1998.

Since the construction of the SR-105 dike and groin by WSDOT, and the exposure of more erosion resistant geology on the shoreline, migration of the Willapa Bay entrance channel as slowed. A fault in the area which indicates the exposure of basement geology may present a geological control in this area (McCrory et al . 2002; Morton et al. 2007). However, ongoing rapid retreat of the shoreline at Graveyard Spit indicates entrance channel migration is no longer the dominant factor affecting shoreline retreat. The dike and groin may be limiting the southeasterly, summertime transport of sand to Graveyard and Empire spits.

## 2.4 Currents and Sand Transport

High tide ranges create large exchanges of water into and out of Willapa Bay, with recorded current velocities as high as 10 feet per second (fps) (Lesser, 2009). Figure 2-5 shows peak ebb spring tide currents simulated by the U.S. Army Corps of Engineers (2009a). In most locations the depth-averaged peak ebb tide current is on the order of 3.9 fps, with a high of about 5.6 fps near the tip of the entrance spit. Peak flood tide currents are weaker at about 2.6 fps. Pacific International Engineering (1997) measured point ebb velocities as high as 7.25 fps and depth-averaged velocities as high as 5.11 fps in the project area. Tidal currents in Willapa Bay are strong.

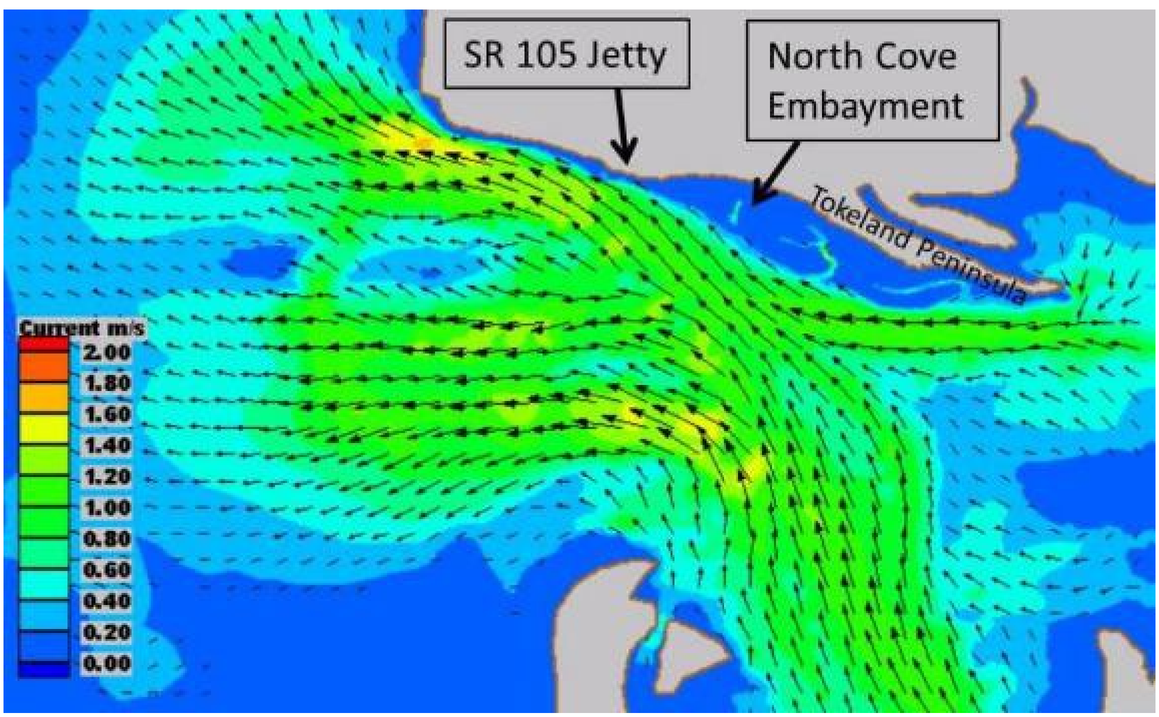


**Figure 2-5. Peak ebb spring tide currents, 2002 channel configuration (U.S. COE, 2009a).**

Lesser (2009) developed detailed models of currents and sand transport in the entrance channel. Beaches and sand spits in the project area are fed by northward longshore transport from the Columbia River. The direction of longshore transport changes from predominantly northerly in the winter to southerly in the summer. Sediments are predominantly well-sorted sand.

Sand transport along the northern margin of the entrance channel is predominantly towards Willapa Bay, but the rate of transport into the bay decreases markedly inland of the SR 105 jetty and along Empire Spit (Lesser, 2009). Model results show asymmetrical sand transport and the formation of a sand sink in the scour hole near the tip of the SR 105 jetty. This asymmetry in sand transport and the loss of sediment supply from the erosion of Cape Shoalwater have resulted in minimal transport of sand to the shallow littoral zone adjacent to Graveyard and Empire Spit. The only remaining supply of sand is the small wave-driven transport along the north side of the main channel.

## 2.5 Wave and Current Climate

The U.S. Army Corps of Engineers (2009a) describes wave conditions along Graveyard Spit for the design of the Shoalwater Bay Shoreline Erosion, Flood and Coastal Storm Damage Reduction project. The wave climate offshore of Willapa Bay is severe, with measured wave heights greater than 23 feet. The wave climate becomes most severe during La Nina and weak El Nino cycles that increase the frequency of large storms tracking from the south-southwest (Northern Economics, Inc., 2005). These typically occur every 3-5 years.

The largest offshore waves approach the project area from the southwest during winter storms, and are substantially attenuated by the shoals at the mouth of Willapa Bay. The attenuated wave heights range between 1.0 and 3.3 feet along Graveyard Spit. Local waves generated in Willapa Bay by winds from the south increase the total potential wave height along Graveyard Spit to a range of about 4.9 to 6.6 feet. Estimated wave heights for the record March 1999 storm used as a design event by the Corps of Engineers peaked at 5.2 feet along the spit. Waves also approach the site from the northwest during the summer but are generally much smaller (USGS, 2004).

Graveyard and Empire spits further attenuate these waves before they reach the shore of the North Cove embayment and Tokeland Peninsula. Recent loss of these dunes therefore increases erosion and inundation risks for low areas of SR 105 that run along the shore of the North Cove embayment. Although the Corps of Engineer's dune restoration project substantially mitigates wave heights along the Tokeland Peninsula, it does not extend north far enough to diminish wave erosion or heights directed at SR105. The Corps' project ends at the unnamed estuary channel (though some old maps call this "Cannery Slough").

As part of the current Graveyard Spit project, the Corps of Engineers completed a coastal engineering analysis, which describes in some detail the tides, winds, and wave climate for the area. It is added to this document as Appendix A, and portions of sections 2 and 4 draw heavily from that document.

The wind rose for Toke Point is shown in Figure 2-6. This diagram shows the strongest winds are from the south. Winds from the north are common, though they are lighter.

Wind stress over the North Pacific can produce ocean wave 30 feet high, and a 'setup' of the mean water level of 1 to 5 feet. The significant wave height (generated during the 50-year recurrence interval storm) was calculated at 37.6 feet (Appendix A). However, a Wiebull distribution of significant wave height indicates that even frequently occurring storms have wave heights of 25 to 30 feet.

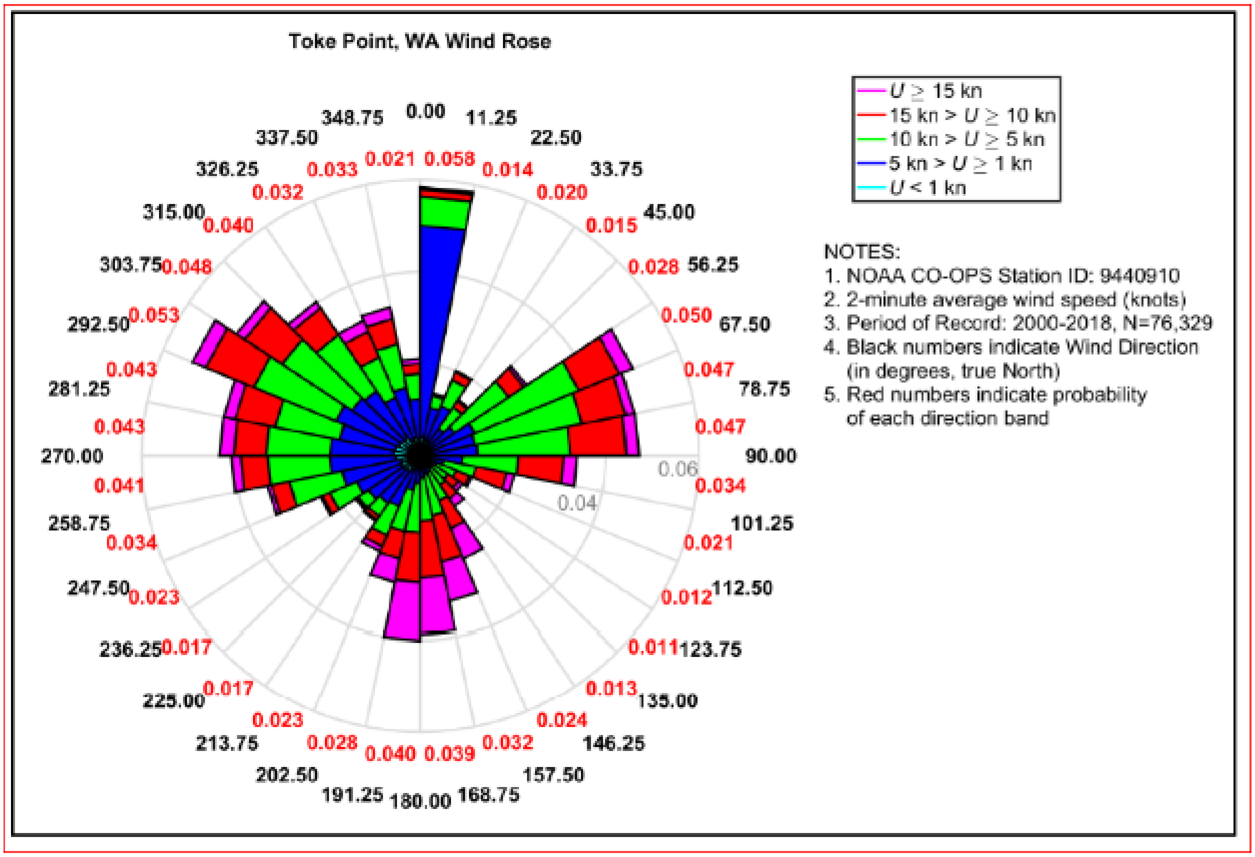


**Figure 2-6. Wind Rose for Toke Point.**

## 2.6 Coastal Flood Elevations

Table 1 summarizes tidal benchmarks for the nearest tide gage at Toke Point (NOAA, 2013). This tide gage is sheltered from ocean waves, so these levels do not reflect the effects of wave setup and runup on exposed ocean beaches. The highest recorded still water tide level was 13.6 feet NAVD88 on November 14, 1981. Recent extreme tide levels include 12.8 feet NAVD on March 7, 1999, and 13.25 feet NAVD on February 4, 2006.

The Flood Insurance Study for Pacific County estimates a 100-year flood level of 14.4 feet NAVD (10.8 feet NGVD) on the west side of Tokeland Peninsula based on analysis of historical tide elevations and wave setup (FEMA, 1985). This elevation assumes the peninsula is sheltered from direct wave action by offshore dunes and does not include wave runup. The USGS (2004) projects flood levels will increase to 19.5 feet NAVD (20.3 feet MLLW) during large storms if the dunes diminish and no longer protect the Tokeland Peninsula from wave runup. The Pacific County Flood Insurance Study estimates a 100-year elevation of 27.9 feet NAVD (24.3 feet NGVD) in exposed segments of the coast north of Cape Shoalwater where waves are not attenuated by the inlet channel shoals.

**Table 1. Tidal benchmarks for the project area (Toke Point, NOAA Station 9440910).**

| Datum | Value (ft, NAVD88) | Description |
|---|---|---|
| MHHW | 8.10 | Mean Higher-High Water |
| MHW | 7.36 | Mean High Water |
| MTL | 3.96 | Mean Tide Level |
| MSL | 3.96 | Mean Sea Level |
| DTL | 3.64 | Mean Diurnal Tide Level |
| MLW | 0.55 | Mean Low Water |
| MLLW | -0.82 | Mean Lower-Low Water |
| NAVD88 | 0.00 | North American Vertical Datum of 1988 |
| HAT | 13.59 | Highest Astronomical Tide |

Climate change will result in an increase in sea level and wave height. These effects are discussed in section 8.0.

## 2.7 Coastline stabilization projects

In this section we look at efforts to combat coastal erosion in the project region, as they may affect and shape the efforts at Graveyard Spit.

### 2.7.1 *Regional projects*

Over the last 50 years, WSDOT and others have implemented a number of measures either responding to coastal retreat or attempting to prevent it. The techniques have varied in scale and complexity, from retreat (highway relocation in the 1970s), to rip rap armoring, to construction of a 1600-foot sea groin/dike in 1998. Pacific Drainage District 1 employed a variety of methods to slow coastal erosion and maintain drainage capacity at Drainage Ditch 1 (also known as Seastrand Creek). WSDOT completed a hybrid berm composed of rounded cobble and large woody material in 2022 at Seastrand Creek. Figure 2-7 shows the projects in the area.

The sea groin and the smaller man-made rock peninsula further west, effectively cut off sediment that moves down the coast in the summer, forcing it into the main channel of Willapa Bay where it can be transported out of the area.   This may cause erosion in the Graveyard Spit and Empire Spit areas, as there is no replenishment of sand following winter storms.

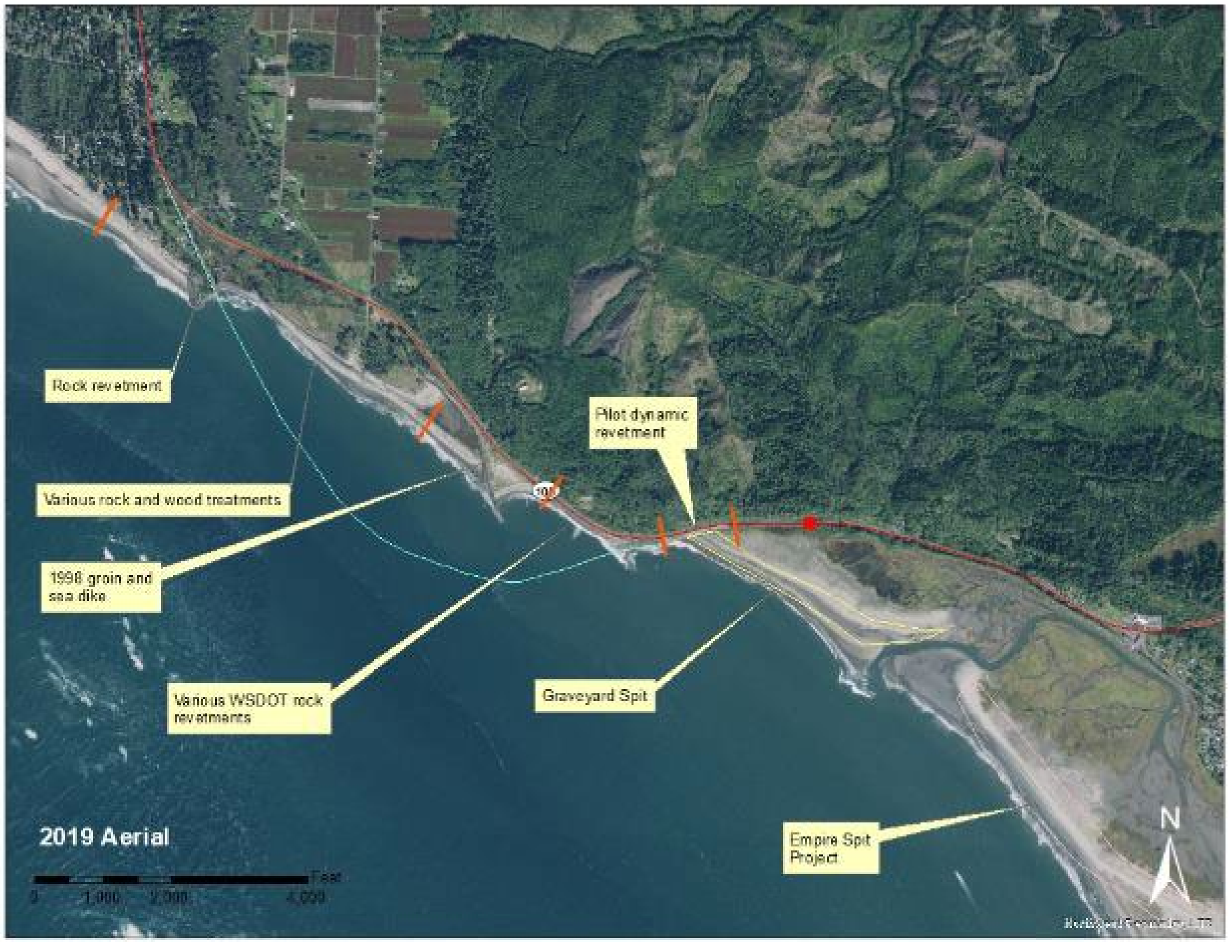


**Figure 2-7. Coastal protection projects in the project area.**

## 2.7.2 *Adjacent projects*

### *2.7.2.1 Dune Restoration on Empire Spit*

In the Tokeland area, coastal protection measures have been implemented at the site of "Empire Spit", a transient sand spit offshore from the community of Tokeland and the Shoalwater Indian Reservation. This project is considered "dune restoration" (Corps of Engineers 2009a). A 12,500-foot linear sand dune was designed across the shallow spit platform. There is no as-built available, but the design plan is shown in Figure 2-8. This project was selected out of a number of different approaches due to its relatively light environmental impact, compared to such things as rock groins and seawalls.

The initial construction used 600,000 cubic yards of sand to protect the Shoalwater Reservation and adjacent shoreline, which is on average 1600 feet landward of the dune. The sand was dredged from a location nearby and adjacent to the main entrance channel of Willapa Bay.

The design elevation of the dune crest was at 25 feet MLLW. This height was intended to prevent inundation from wave run up on the Tokeland Peninsula, and specifically at the Shoalwater Tribe

Reservation. The design elevation was based on 21.6 feet MLLW, which is the sum of the still water level (tide, storm surge, setup) and dynamic water level (wave run-up). This amounts to the water elevation with a 1% chance of occurring in any given year (Corps of Engineers, 2009c). The Corps then added another 3.4 feet, stating “A barrier dune crest elevation of +25 feet MLLW will eliminate the threat of water levels from overtopping the barrier dune and the risk of flooding and erosion on the Shoalwater Reservation shoreline.” Later it is also explained that to achieve the lowest “life cycle cost”, “the initial dune dimensions maximize the volume of sand that is placed within the available plan area.” Although not explicit, we interpret this as meaning the dune is higher and/or wider than is necessary to prevent the 1% recurrence interval high water level.

This project includes $80,000,000 for maintenance over the course of the project life, 40 years. Based on the 2000-2002 erosion rates, the Corps estimated the annual loss of sand from the dune (above +6 feet MLLW) at about 50,000 cubic yards per year. The maintenance planned for the dune was scheduled for once every 5 years, consisting of dredging 250,000 cubic yards from the designated borrow area near the channel (See Figure 2-8). The dune restoration lies on a platform of sand that is part of the Willapa Bay shoals, though likely much older than the surficial deposits of Empire Spit (also referred to as Graveyard Spit). The bathymetry of this location shows that the main entrance channel is 80 feet deep, and its centerline is located 4000-5000 feet from the spit. In addition, there is a shallow terrace offshore that extends 3000-3500 feet from the dune restoration to the north side of the main entrance channel.

Although the planned length was 12,500 feet, a site visit in February of 2014 indicated that the constructed length was significantly shorter - approximately 2000 feet shorter on the south end and a thousand feet shorter on the north end. Also, it was noted that significant portions of the north and south ends of the constructed dune have been eroded by waves. Figure 2-9 shows selected views of the erosion. Erosion has truncated the dune further, and eroded the terrace underneath the dune. In addition, the crest of the dune is lowering in place through wind erosion. Based on the fluvial and aerial erosion, the replenishment rate needed to maintain the constructed configuration of the dune may be greater than the Corps of Engineer’s estimate for replenishment.

To address the wind erosion, wind fences were installed in 2018. These appear at least partially effective (see Figure 2-9). In addition, erosion of the toe of the dune was addressed in 2019 by placing small amounts of angular rocks, about 1-8” in diameter, as a form of dynamic revetment. This did not prevent additional erosion, as seen in photos taken in October of 2020 (Figure 2-9). We speculate that the rock was not effective due to insufficient thickness and porosity. We estimate that the thickness of the material places was only 2 feet or so. Also, since the material is angular, it does not have as much porosity to absorb wave runup as rounded cobbles would. In addition, there is a wide range of sizes of angular rock, indicating less effective energy absorption, and greater energy transference than a natural cobble beach.

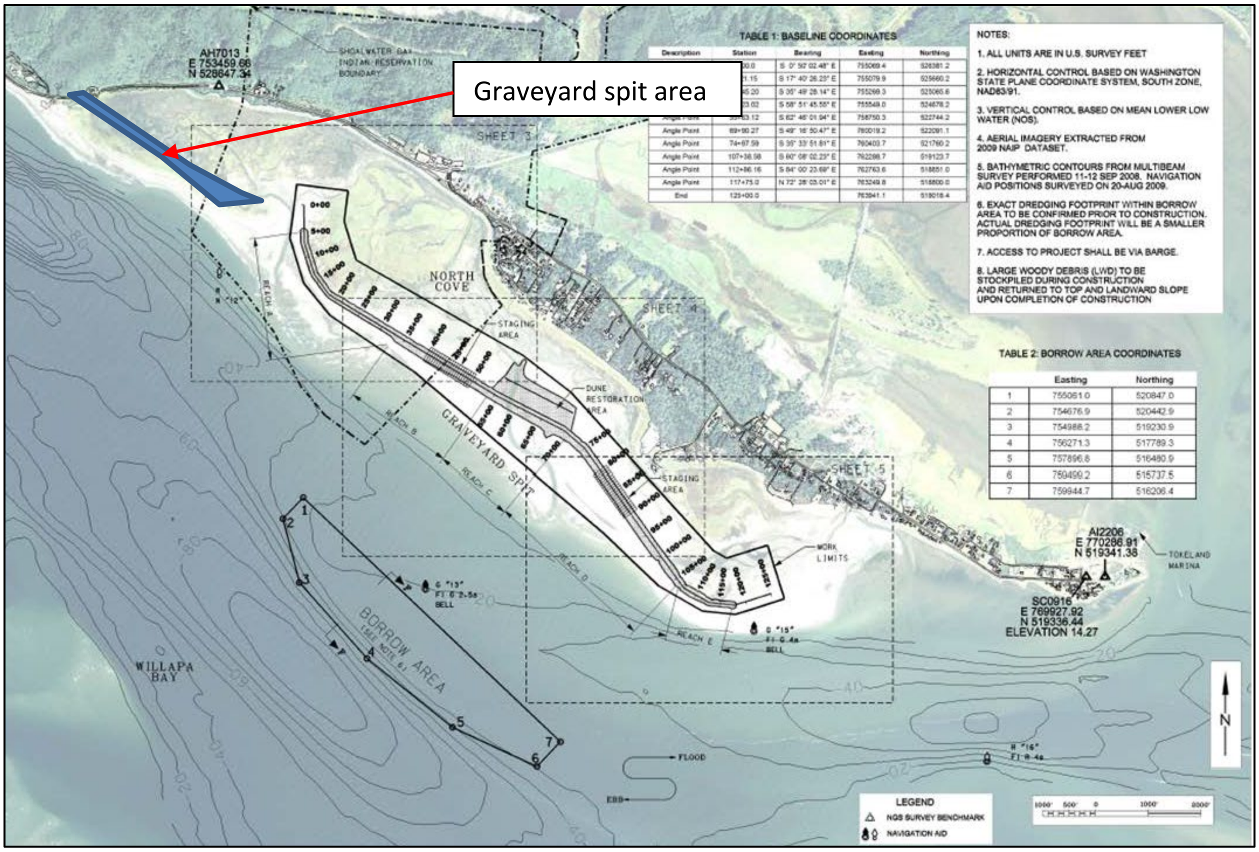


**Figure 2-8. Dune Restoration project location and bathymetry, Tokeland Peninsula and Shoalwater Indian Reservation**

(US Army Corps of Engineers, 2013).

**Figure 2-9. Aeolian and wave erosion of angular rock and dune sand, Empire Spit Dune restoration project. Above: looking directly at dune. Below: looking north along toe of dune.**

Additional repairs of these areas are currently under design. The Corps is planning two major stabilization components – use of dredged sand from Willapa Bay to re-establish dune height and width, and use of angular cobble in the form of a dynamic revetment on the waterward side of the dune. The announcement for the supplemental environmental assessment (USACE, 2022) states that the dynamic revetment would be composed of either angular or rounded cobble. Approximately 900,000 cubic yards of sand would be added to the north end of the existing dune as shown in Figure 2-10. Figure 2-11 shows the typical cross-section. A haul road would be developed (see Appendix D) on Graveyard Spit, along with a channel spanning structure across Cannery Slough, to provide construction access.


**Figure 2-10. Proposed repair of Empire Dune Restoration Project. (USACE, 2022)**

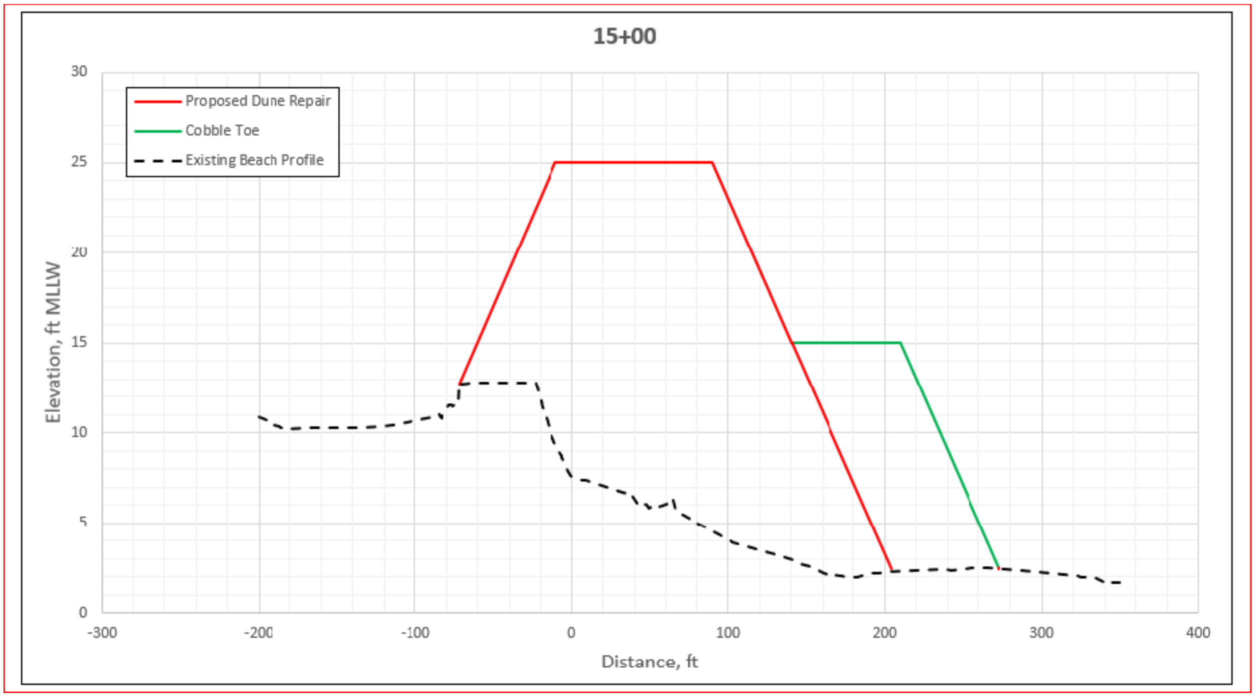


**Figure 2-11., Proposed cross-section of the repair in planning for the Empire Spit Repair. (USACE, 2022)**

### *2.7.2.2 WSDOT Pilot Dynamic Revetment*

In response to continued erosion advancing toward SR105, WSDOT in 2017 implemented a series of measures to protect the highway. This included two main treatment areas – west of the current project, approximately MP 20.15 to 20.36, and an area spanning the west end of the current project area, MP 19.58 to 19.96. About 1200 feet of "porous debris berm" was constructed on the eastern end, to prevent driftwood from covering the highway during extreme tides. A dynamic revetment was built on the western end, joining with the existing Class C angular rock revetment and larger rock. About 260 feet of this structure's eastern end was built with a 25 foot top width, while the western 520 feet was much narrower, at 12 feet (see Figure 2-10). This difference was due to the effort minimize changes to permits that had already been acquired prior to the decision to use a dynamic revetment. Notably, the pilot project was limited in height, width, and mass.

The pilot dynamic revetment has eroded during several winter storms, which was anticipated. During the design it was recommended that a stockpile be established for replenishment when such erosion offered over time. Figure 2-11 shows a portion of the revetment in 2021. The stockpile that was established for replenishment was used during the first episode of erosion in January 2018, however it has not be actively replenished as the original design recommended. Angular rock has been used in several instances to patch portions of the dynamic revetment that have deformed. In each occurrence where angular rock has been used, additional erosion off the end of it has occurred. However, the pilot project has shown that there is transport of gravel to the southeast and that sediment tends to form an arc that parallels the shoreline.

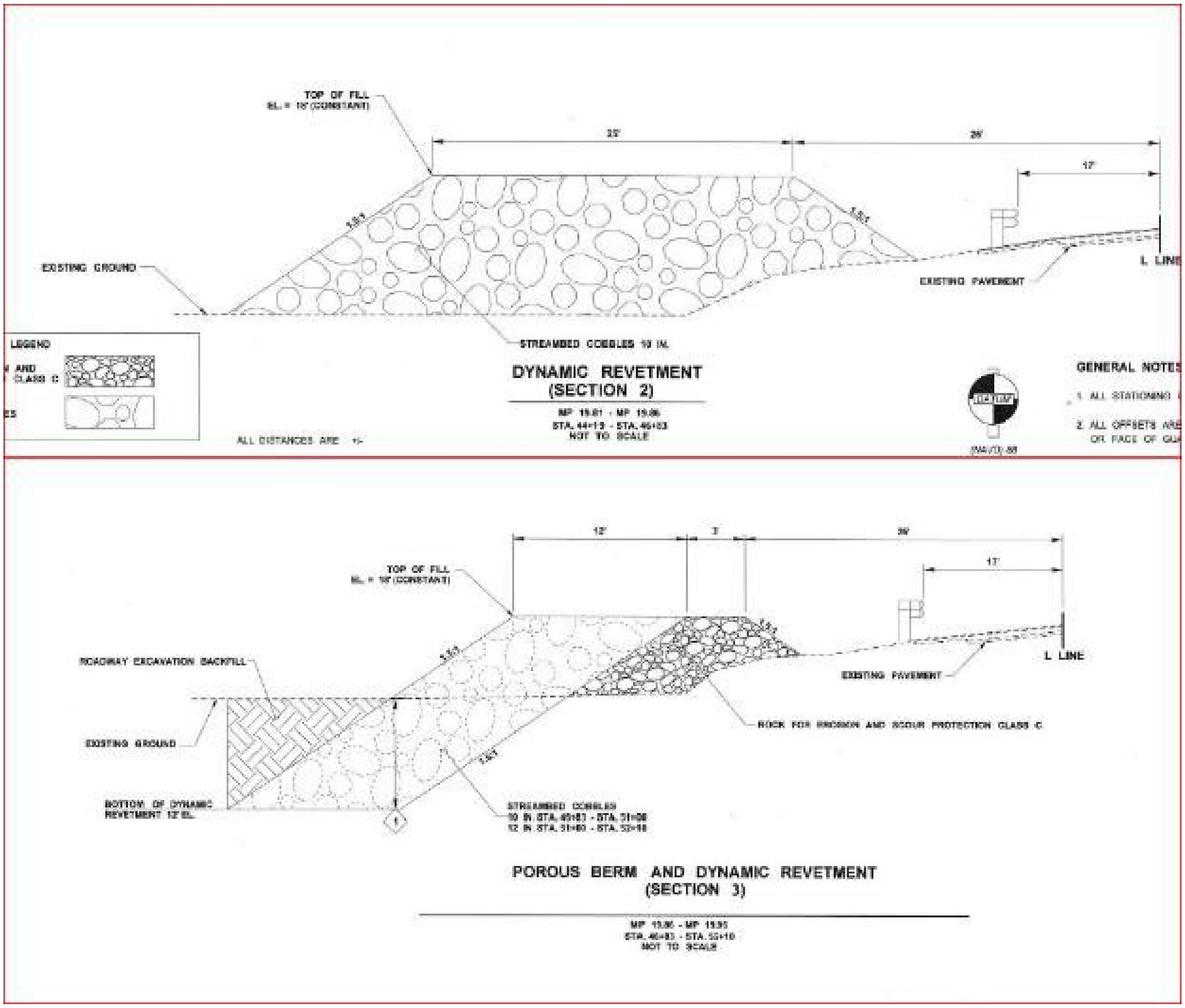


**Figure 2-12. Cross-sections of pilot dynamic revetment as built in 2017.**

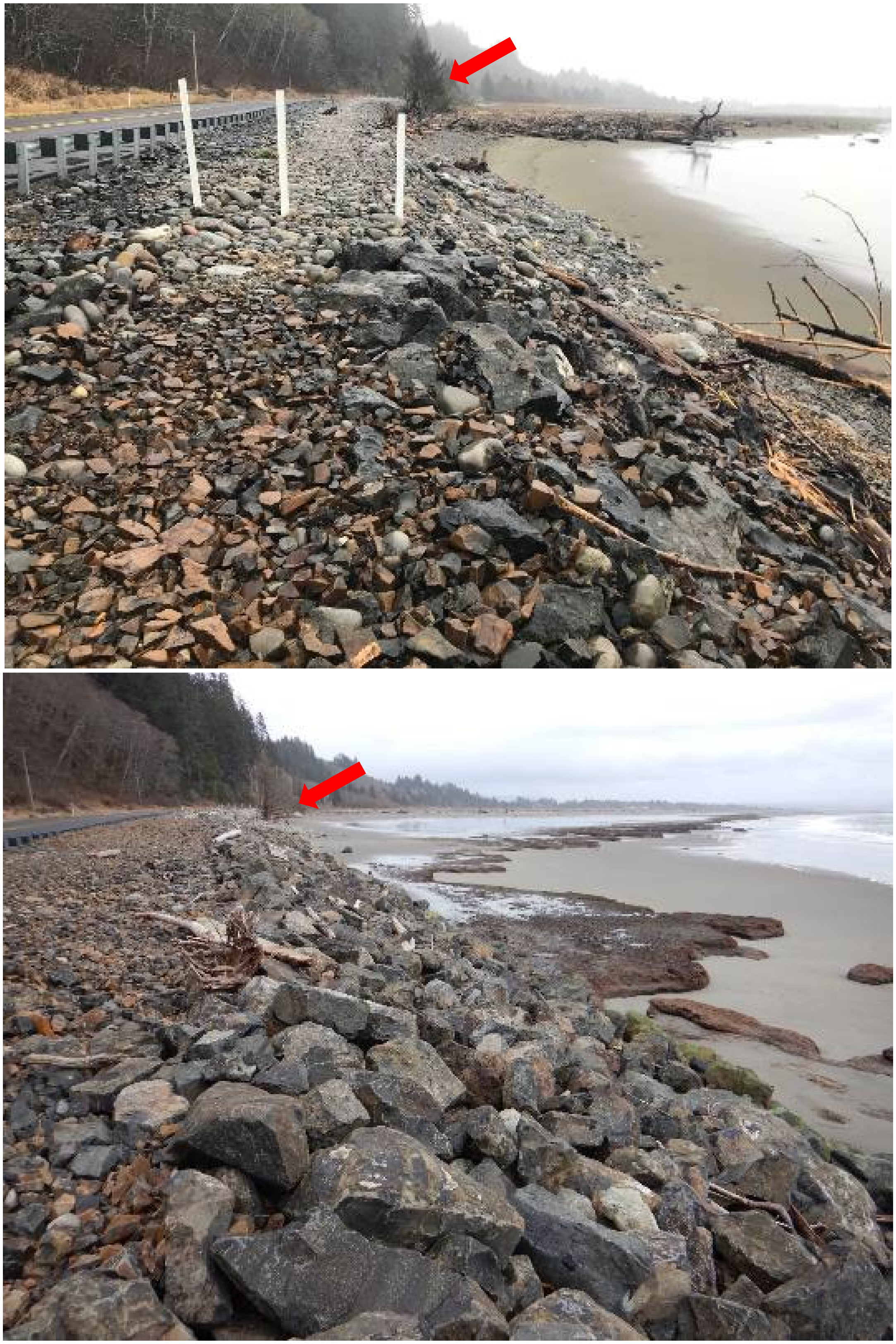

**Figure 2-13. Eastern end of pilot dynamic revetment; above, February 2019; below, February 2021. Note red arrow indicating the same tree in both photos. Additional rock in foreground was placed in December 2020.**

# 3 Reference Beach Selection

The proposed design is based on a natural composite beach. In looking for an analog, we explored the coasts in the region for similar shoreline aspect, wave environment, and for undeveloped beaches with natural shorelines. The beach at Kalaloch, in Olympic National Park (see Figure 3-1, 3-2) was chosen. Beach profiles were surveyed to understand the inflection points in slope, how they related to the materials found, the overall shape of the beach, and heights of the beach formations.

Profiles of several spots along the beach are shown in Figure 3-3. The average slope of the cobble surface was 18%, or 5.5:1. The average elevation of the cobble/sand transition was about 8.5 feet (NAVD88).

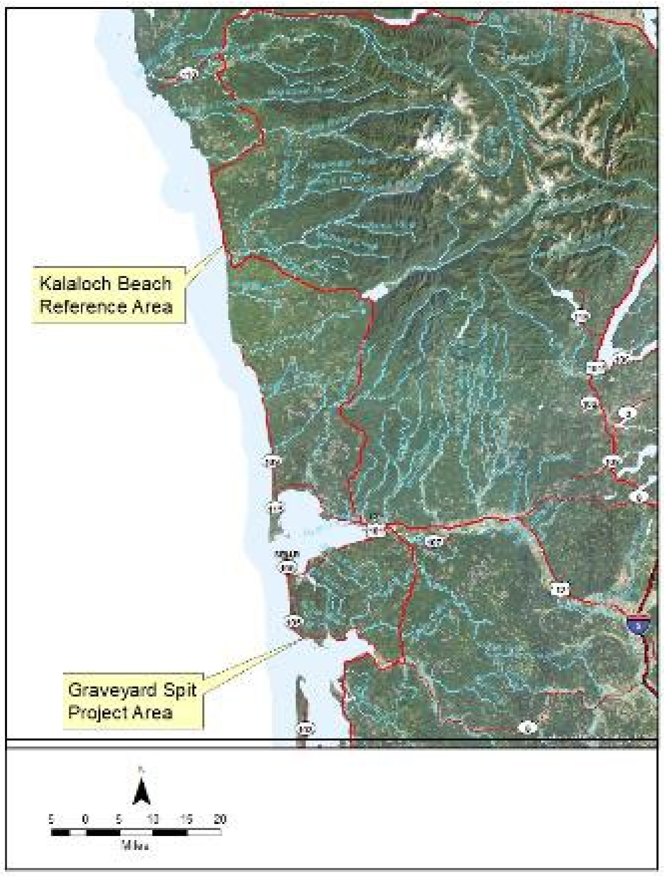


**Figure 3-1: Location of Kalaloch Beach**

**Figure 3-2: Photo of a portion of the reference reach, Kalaloch Beach**

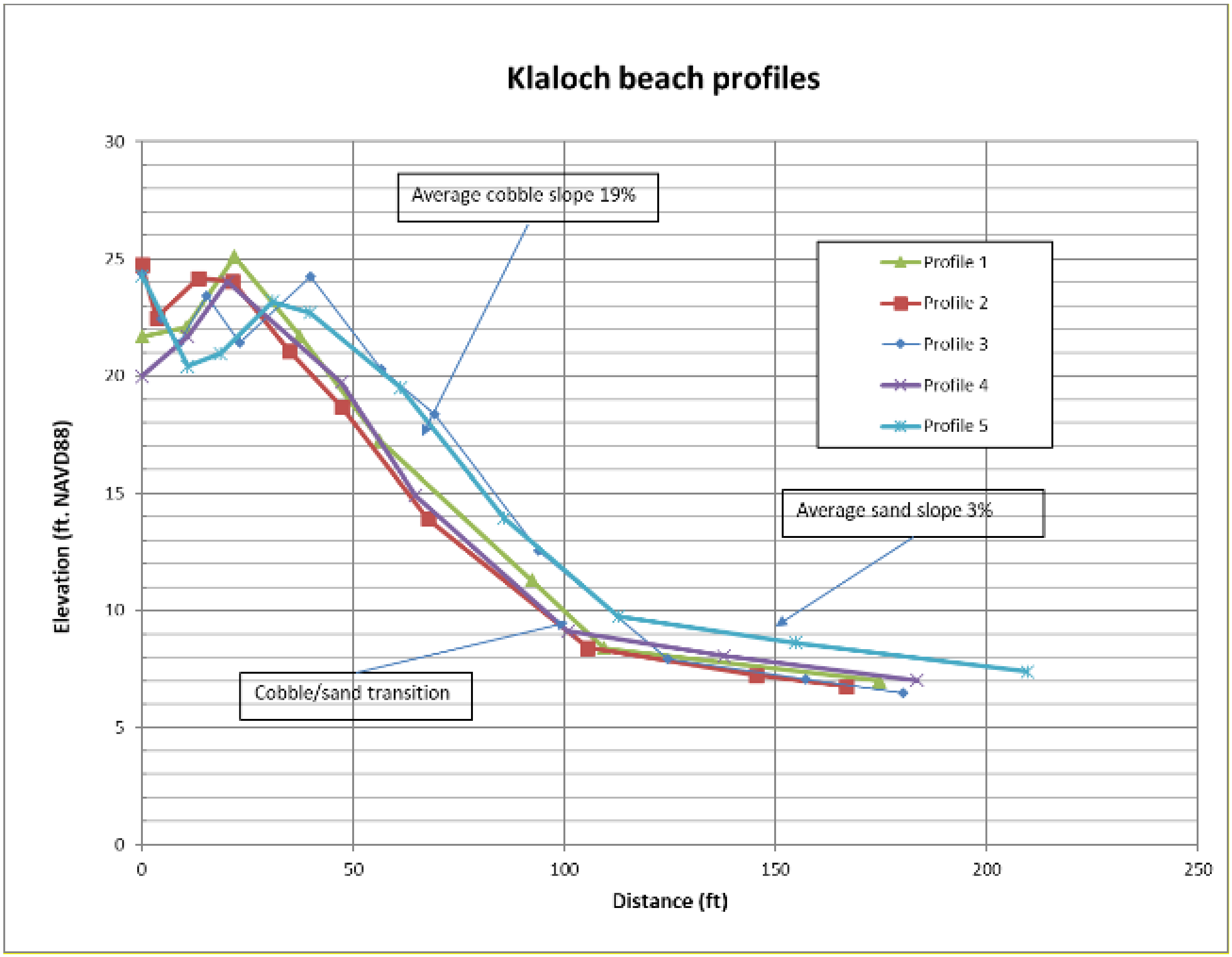


**Figure 3-3: Beach profiles at Kalaloch Beach, with key features noted.**

# 4 Hydraulic Analysis and Design

## 4.1 Model Development

A two-dimensional hydrodynamic model was used to simulate the nearshore wave conditions at Graveyard Spit. The Coastal Modeling System (CMS) -FLOW and -WAVE modules were utilized to analyze the wave and current patterns in Willapa Bay over varying wave heights, tidal elevations, and river discharges. Wave-current interaction is represented through communication between modules. Bed elevation is updated at each time step to incorporate effects of bed morphology on wave and currents through time.

CMS-Wave is based on the wave-action balance equation (Lin et al. 2008). It is a two-dimensional spectral wave model formulated from a parabolic approximation equation with energy dissipation and diffraction terms to simulate a steady-state spectral transformation of directional random waves coexisting with currents in the coastal zone (Mase et al. 2005). Wave refraction, shoaling, diffraction, reflection, breaking, and dissipation are represented in the model. The model operates on a coastal half-plane for waves propagating only from the seaward boundary toward shore. CMS-FLOW solves the two-dimensional, depth-integrated continuity and momentum equations by applying a finite-volume method (Militello et al. 2004). These equations are solved numerically using an implicit finite differencing method.

The model is forced with a water surface elevation at the offshore boundary generated from published NOAA CO-OPS water level time series at Toke Point Station 9440910 and wave heights measured at the Coastal Data Information Program Buoy (CDIP) 036 located 4.5 nautical miles offshore at a depth of 135 feet. The CMS model uses a variable rectangular grid to represent the topography and bathymetry.

### 4.1.1 *Topographic and Bathymetric Data*

The existing CMS grid from USACE (2018) was used for the baseline dataset (Figure 4-1). Newer data at the Willapa Bar and North Entrance Channel was incorporated to reflect the most recent conditions in Willapa Bay. The following data sources were used:

- 2021 USACE Annual Bathymetric Condition Survey

  These hydrosurveys include single beam transects of the Bar and Entrance Channel and was conducted by the USACE, Seattle District's Shoalhunter survey vessel in May 2021.

- 2021 WSDOE Combined Topographic and Bathymetric Survey

  This survey included a combination of backpack GPS and multibeam surveys in the nearshore region. This survey was performed by WSDOE as part of the monitoring program for the USACE North Cove Continuing Authorities Program Section 103 project in February 2021.

- 2019 Shoals

  NAIP 2019 aerial photography was utilized to digitize the horizontal positions of the surface piercing shoals. An elevation representing mean higher high water was assigned to these shoal areas.

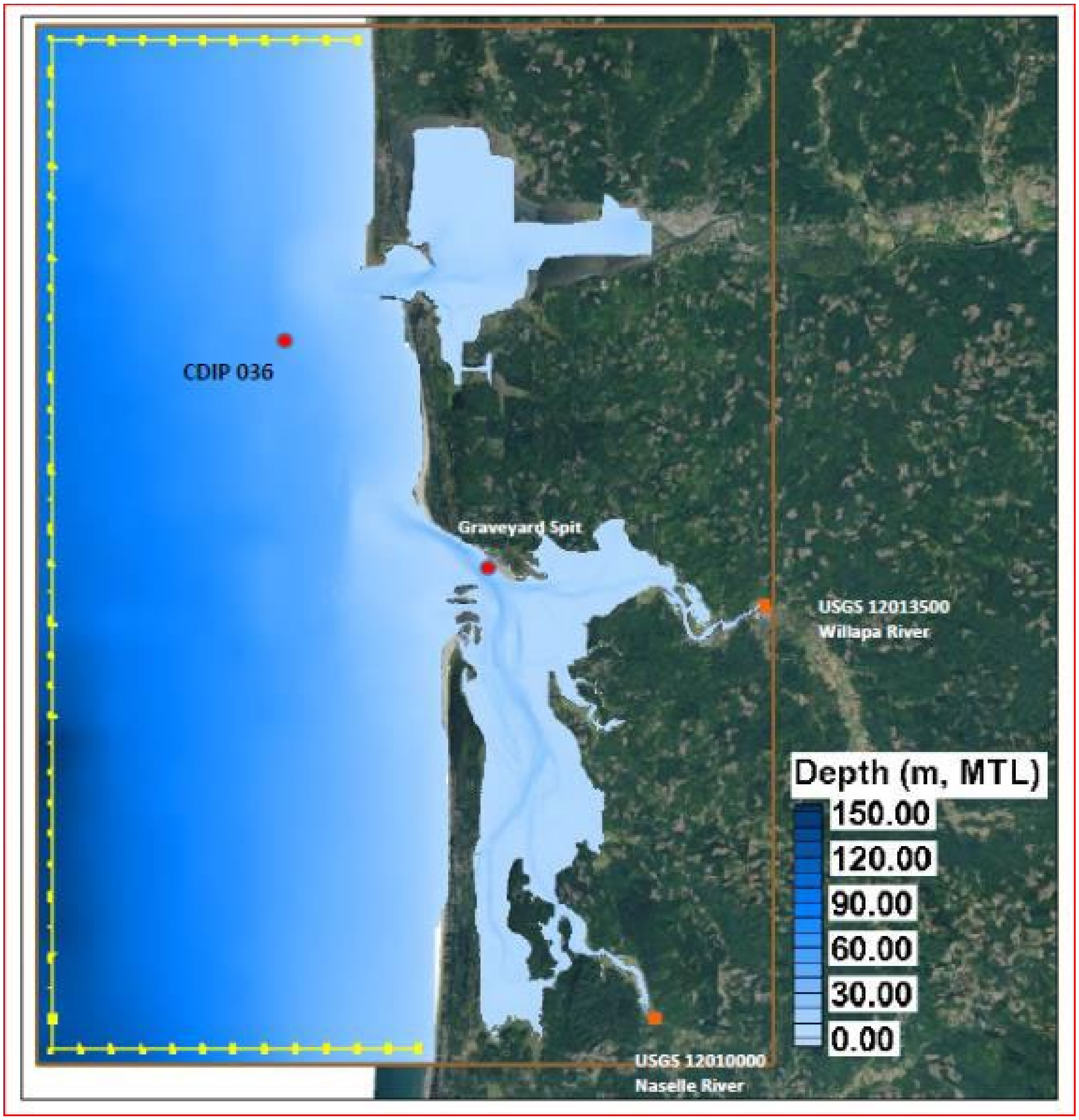


**Figure 4-1. Bathymetry, boundary conditions, and observations points used in CMS.**

### 4.1.2 *Model Setup and Verification*

The CMS model has been previously calibrated and validated for Willapa Bay (USACE 2018). Updates to the bathymetry and topography were incorporated into the new CMS grid for this project. Five storm events were utilized to verify the model performance with measured data at the CDIP buoy and NOAA tide gage. These included the March 3, 1999; February 3, 2006; December 3, 2007; December 10, 2015; and November 15-18, 2020 storm events. These are all classified as extreme storm events and are used to compute wave statistics immediately offshore from Graveyard Spit for design of the dynamic revetment. The time series comparisons of measured versus modeled wave height were compared at the CDIP 036 buoy and at point 10 located at the 10-meter contour offshore from Graveyard Spit (Figure 4-2).

Historic wave data from the 1972-2020 CDIP buoy data set were binned by wave height, period, and direction to develop a statistical distribution of annual incident wave conditions at the project site. This resulted in 1,290 combinations of incident wave conditions (Hs = 0.25 to 11.75 m; Tp = 5 to 21 s; and Dp = 180° to 350°). These were used to develop a lookup table that was then sampled to generate a synthetic time series at the 14 nearshore wave stations located at the 10 m contour from North Cove to Tokeland (Figure 4-2).

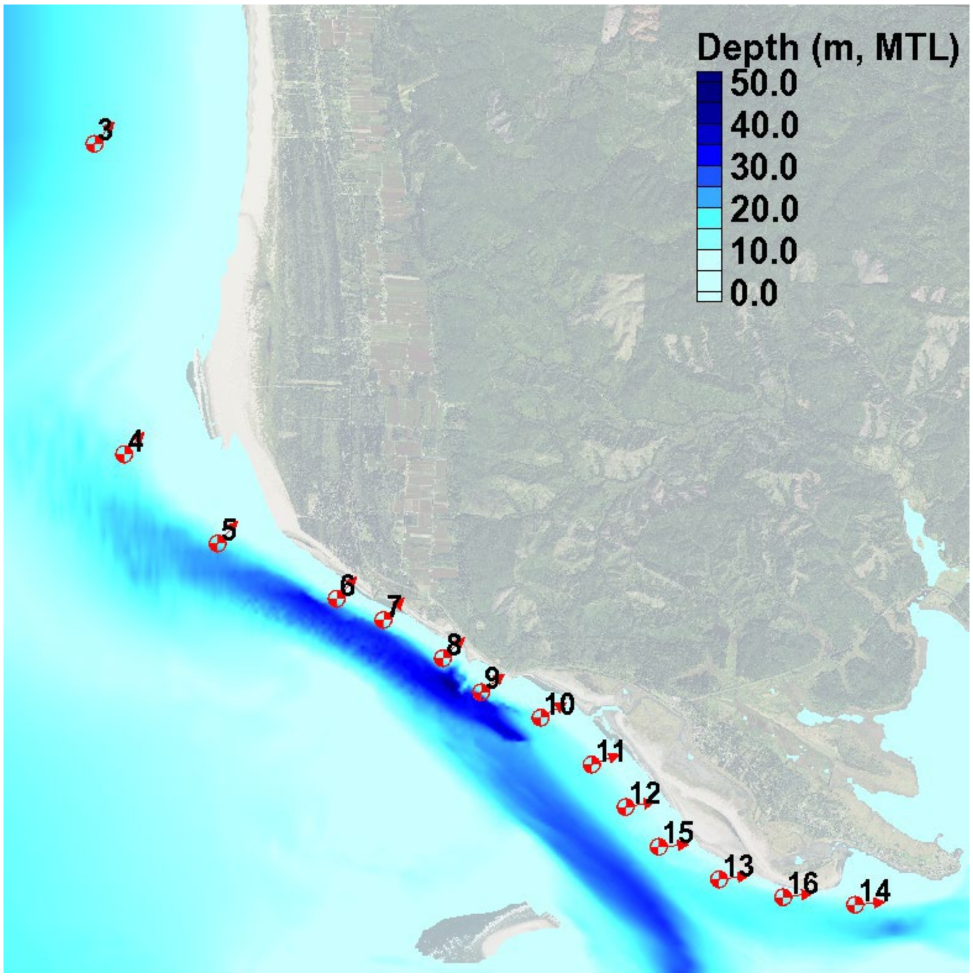


**Figure 4-2. Nearshore wave stations in CMS**

## 4.2 Existing Conditions/Without Project Model Results

### 4.2.1 *Wave Runup*

The CMS modeled wave heights compare well to the measured time series at the CDIP 036 for the December 2007, December 2015, and November 2020 storm events (see Appendix A). The nearshore wave height at Graveyard Spit is also compared relative to the measured water level at the Toke Point tide gage. Nearshore wave heights are depth limited and modulated by water level. Thus, the peak wave heights will always occur at high tide. The peak wave heights computed at Graveyard Spit range from 1.87 to 2.35 m. An example wave height model is shown in Figure 4-3, from pt. 10 at Graveyard Spit, for the November 2020 storm. Additional model runs are shown in Appendix A.

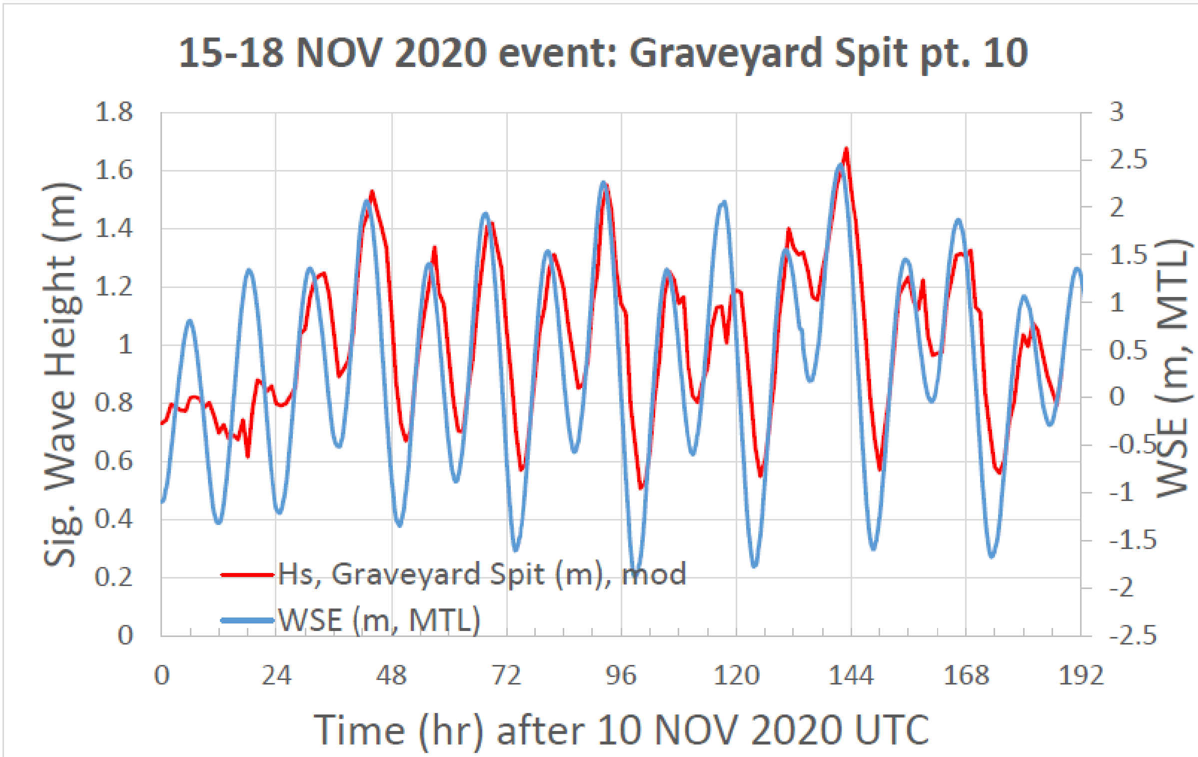


**Figure 4-3: Existing conditions wave prediction, pt. 10.**

### 4.2.2 *Shoreline Change GENCADE modeling*

The GENCADE shoreline change model predicts future erosion on Graveyard and Empire Spit. GENCADE is forced using nearshore wave statistics computed at multiple stations alongshore ranging from north of Warrenton-Cannery Road to the terminus of Empire Spit.

GENCADE simulates shoreline change produced by spatial and temporal differences in longshore sand transport (Frey et al, 2012). The model assumes that the beach profile moves parallel to itself, i.e., that it translates shoreward or seaward without changing shape. Thus, one contour line can be used to describe change in the beach plan shape and volume as the beach erodes or accretes. A second assumption is that sand gets transported alongshore between two well-defined limiting elevations on the profile. The shoreward limit is located at the top of the active berm, and the seaward limit is located where no significant depth changes occur, called the depth of profile closure. Restriction of profile movement between these two limits provides the simplest way to specify the perimeter of a beach cross-sectional area by which changes in volume, leading to shoreline change, can be computed.

An 8-year duration , 2013 to 2021, was simulated to calibrate and validate the model (Figure 4-4). The model domain is represented by 236 cells that are 50 m in size. Hourly wave data from the CDIP buoy 036 was used to generate annual wave statistics and utilized to develop a randomly generated synthetic time series of the incident wave conditions simulated in the model. Sediment sources and sinks were specified in the model using the sediment bypass function. This function allows for input of a net sediment accretion or erosion rate (volume/time) over a section of shoreline. These features were important to represent the supply of sand to the North Spit shoal over time. Additionally, the sediment bypass feature was used to represent offshore loss of sand near hard structures such as the SR-105 groin and where direct wave impact on the dune is observed (i.e., on the north end of Empire Spit). Model parameters used in the GENCADE model were adjusted within the recommended values and model performance was verified using available aerial imagery over the simulation time (Table 4, Appendix A).

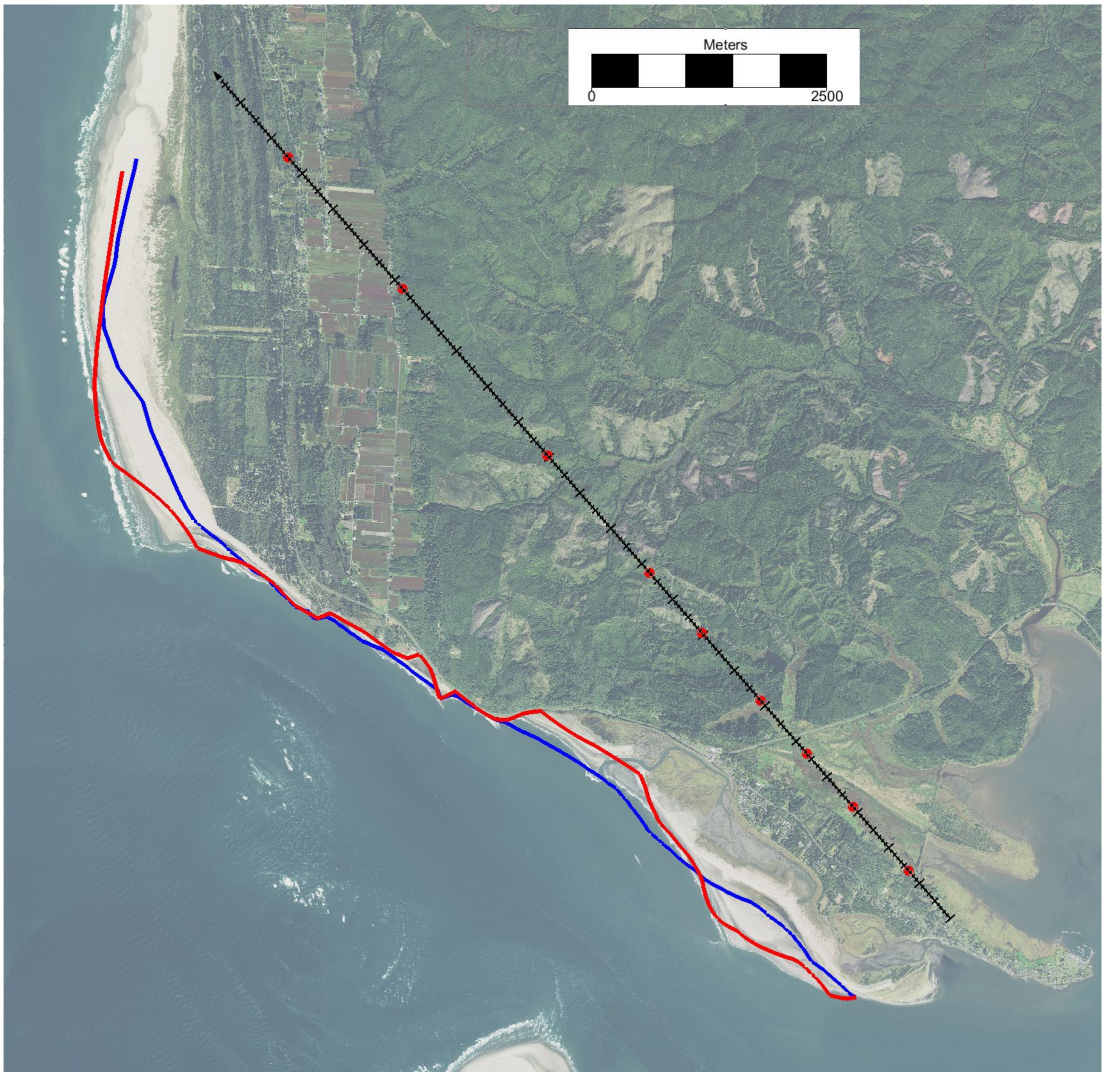


**Figure 4-4: GENCADE shoreline change model validation and verification from 2013 (blue) to 2019 (red). Longshore wave stations forcing model are denoted by red circles**

GENCADE was used to forecast to year 2031 (Figure 4-5). The model predicts continued retreat of the shoreline until it reaches SR105. Some of the material from Graveyard Spit is transported alongshore toward Empire Spit, but most of the sediment is expected to be entrained in the swash zone and transported offshore from the spit or onto the roadway. Additionally, the northern terminus of Empire Spit is predicted to migrate landward toward the Tokeland Peninsula. The model suggests that the northern terminus will rotate clockwise toward the shoreline fronting SR105. The previously established tidal inlet into the embayment will likely be maintained between the Empire Spit terminus and the Tokeland Peninsula.

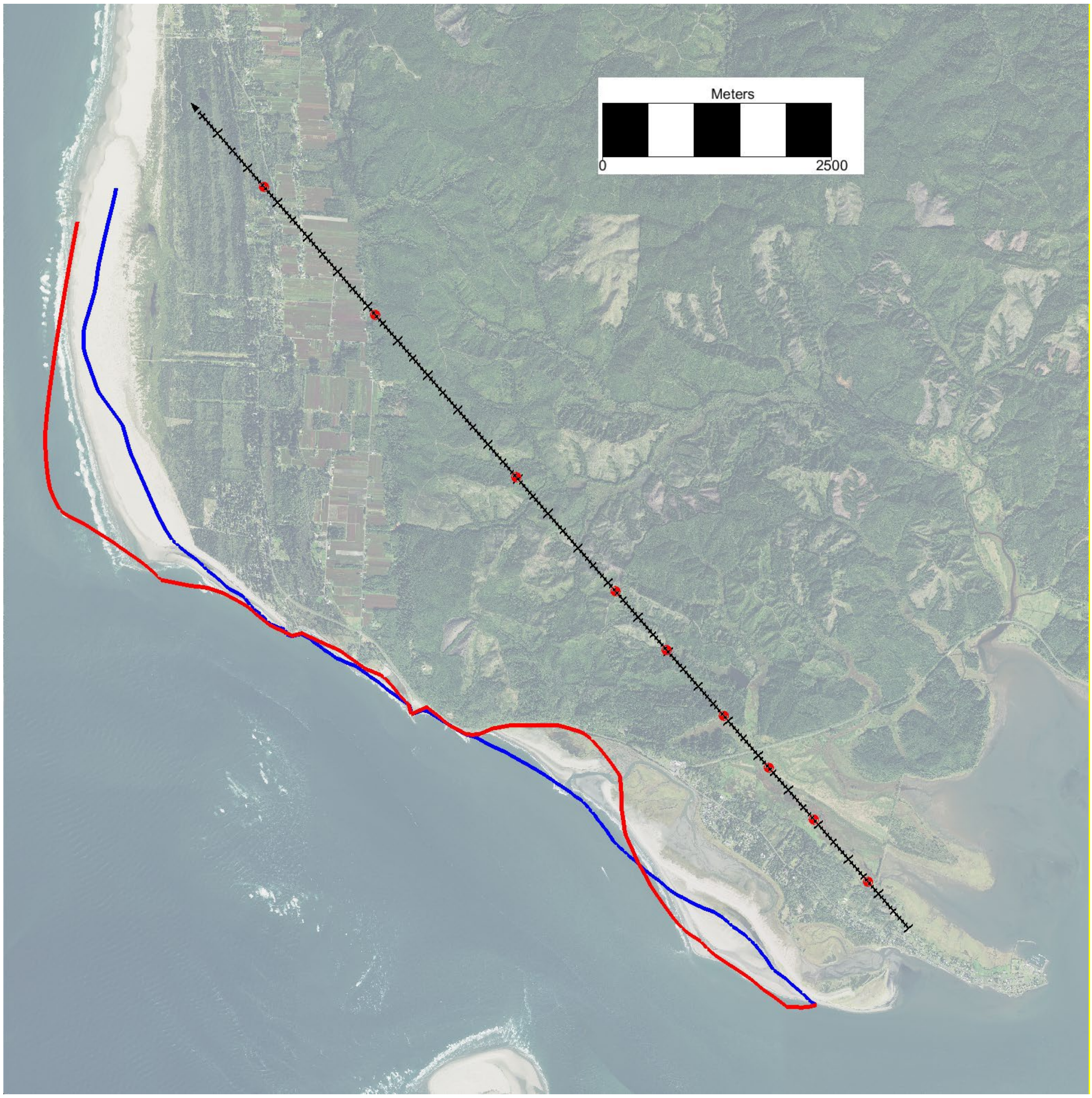

**Figure 4-5: GENCADE model forecast shoreline change without project, 2019 to 2031.**

### 4.2.3 *Forecast without project dune morphology*

A numerical beach profile evolution model, XBeach (Roelvink et al. 2009), was used to simulate wave runup on beaches and dunes during storm events. XBeach is a non-hydrostatic model which is forced by wave frequency spectrum to simulate a time series of waves. The NOAA Tide gage at Toke Point is used to force the storm water level hydrograph. XBeach was originally developed for sand beaches, which generally assumes the beach and dune face are impermeable, therefore wave runup is not significantly impacted by water infiltration in the swash zone. Simulations were performed using a median grain size of $d_{50}$ = 0.2 mm and survey data collected by WSDOT in February 2021 to compute wave runup. XBeach was run for the same storm events produced in the CMS model. The dune crest height measured in the initial condition is 3.02 m above MTL. XBeach results indicate for all 5 storm scenarios run, the dune was overtopped, and the dune crest was pushed landward via overwash (Table 5, Appendix A). This analysis confirms that dune overwash is anticipated to occur on almost an annual basis and Graveyard Spit is highly vulnerable to future degradation if no action is taken to protect the spit.

Beach and dune evolution was also analyzed. The December 3, 2007 event was simulated to determine how the dune crest height would be affected from continuous overwash (Figure 4-6). The results indicate the dune crest would be flattened from 3.0 to 2.5 m MTL and migrate 60 m landward following the event. Most of the sediment eroded from the dune is predicted to move down slope and offshore. Only a thin layer of sand is predicted to deposit on the leeward side of the dune. Given the longshore currents in the area, most of this sediment is expected to be lost, leaving limited sediment available to migrate up the beach profile in the calmer summer months when dune recovery can occur from aeolian transport.

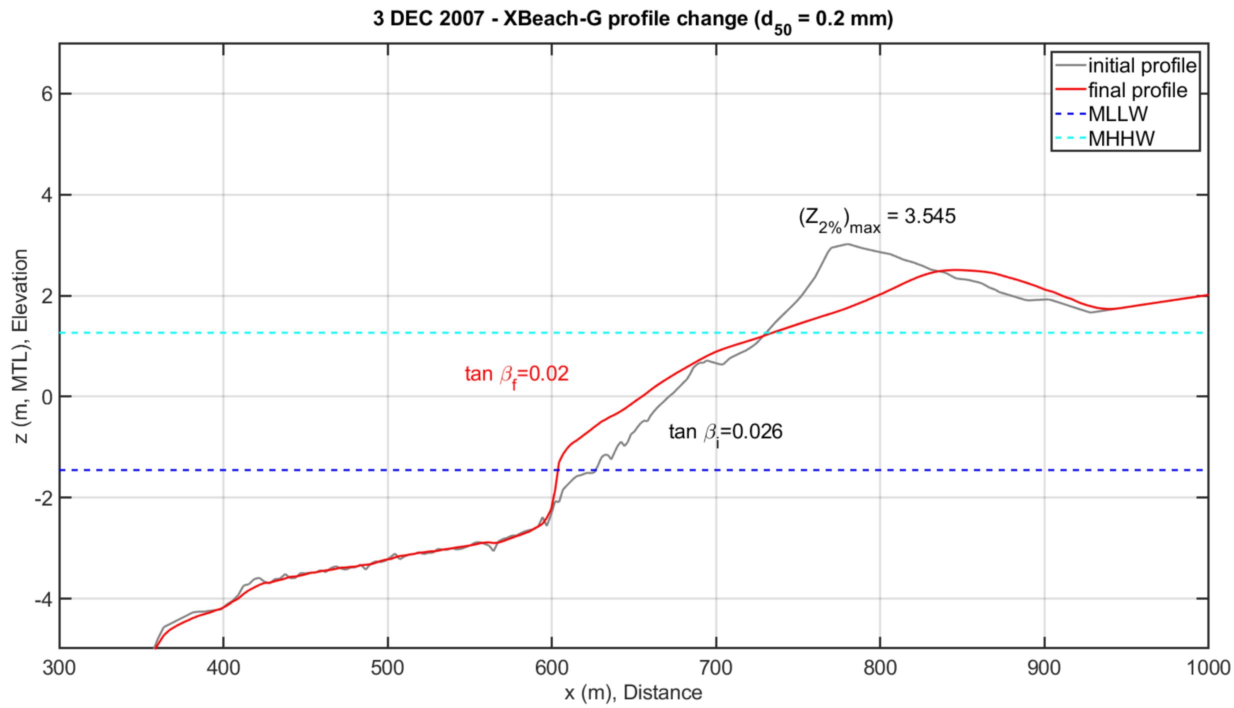


**Figure 4-6. XBeach computed beach and dune profile change after successive storm events for current conditions at Graveyard Spit.**

# 5 Dynamic Revetment Design

## 5.1 Design Methodology

The strategy for protecting the remaining portions of North Cove and the highway is inherently experimental, though it is informed by empirical knowledge of nearby similar designs, and by the results of hydrodynamic modeling. With this two-pronged strategy we seek to minimize uncertainty in project performance – namely, deformation and attrition rates. However, it is recognized that emulating natural beaches – which deform and erode over time – there will be a need for adaptive management. This is considered part of the design and discussed in chapter 8.

The following sections discuss the components of the dynamic revetment structure and the methods and assumptions used to refine them.

## 5.2 Shoreline Design

### 5.2.1 *Shoreline Alignment*

The project limits are informed by constraining features and project objectives. The larger geographic boundaries are SR105 to the northwest, and the tidal channel to the southeast (see Figure 1-1).

The tidal channel (labelled on some maps as Cannery Slough) must remain open, even though king tides can still affect the estuary. With the project in place and the Corps' dune project functioning, waves and the resultant increase in water elevation on top of high tides would be significantly reduced. Blocking the tidal estuary channel would require development of alternative drainage to allow runoff to exit the area. More importantly, the channel must remain open because a major objective of the current project is to restore the estuary's functionality. Tidal flux is necessary to maintain biological and geomorphological functions of the estuary.

Within the project footprint, the general orientation is informed by alignment with the Willapa entrance channel, and historic alignment of spits, which reflect the manifestation of the complex tidal exchange and longshore sediment transport. The orientation is predominately in a northwest to southeast direction, and aligns with the Empire Spit project.

The waterward (west) boundary of the structure is determined by the designed toe elevation (see section 5.2.2) and the setback for estimated coastal erosion between the end of design and start of construction. The project is being designed without secured funding, and therefore the design team anticipates shoreline retreat between design and construction, given the rapid rate of coastal retreat at the project site.

Coastline retreat over the past 15 years was calculated based on repeat aerial photography analysis. The average rate of retreat between 2003 and 2019 was calculated at 75 feet/year. We chose a gap of 2 years between funding and start of construction, considering timing of permitting and administrative work. Therefore, the alignment was set back 150 feet from the 2020 surveyed 7.0' NAVD88 elevation. This alignment is shown in Figure 5-1.

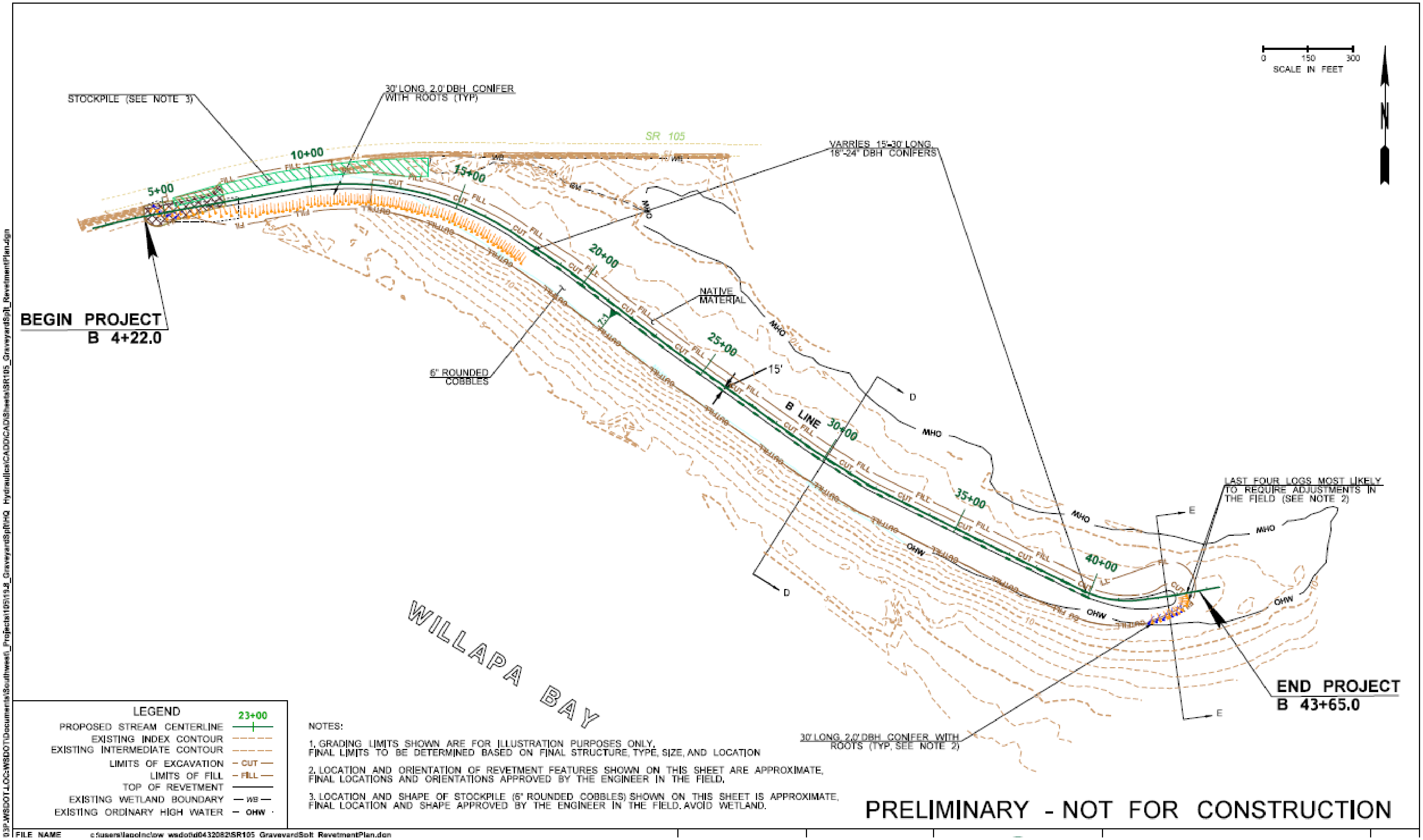


**Figure 5-1. Dynamic Revetment and Dune Alignment.**

### 5.2.2 *Shoreline Cross-section and Elevation*

To provide reliable long-term protection for the highway and estuary, without frequent maintenance, the dynamic revetment must maintain a stable profile without rapidly losing material to longshore and cross-shore currents. This means that the material to be used must be sized to be compliant with wave action and beach profile change while resisting displacement.

The major geometric components of dynamic revetment design are:

- Toe elevation
- Critical mass
- Crest elevation

These components significantly influence the cross-section.

First, the toe elevation must be determined. We used the reference beach at Kalaloch. We determined the average cobble/sand transition point elevation to be 8.5 ft NAVD88. However, considering scour, and not knowing the thickness of the sand layer at the reference beach, we would consider this elevation too high for the toe elevation of the dynamic revetment. The mean higher high water elevation at Toke Point is 8.04, and the mean high water elevation is 7.36. We chose an elevation of 7.0 ftNAVD88, slightly below the mean high water elevation. At this level, the dynamic revetment will be engaged with all of the higher energy wave conditions. Note that this is 2 feet lower than the toe elevation of the MCR dynamic revetment. This is partly to add bulk to the dynamic revetment cross-section, as the crest top width will be 15 feet. The added bulk contributes to a large amount of pore space in the structure, to absorb wave volume and reduce wave run-up. The 7 foot toe elevation also account for scour that may occur lower in the beach profile during later winter storms, when the summertime lens of sand at the lower end of the beach profile may be thinned, and in recognition of a limited sand supply due to the factors discussed in section 2.0.

Next, the critical mass is calculated (Ward and Ahrens, 1992). Ward and Ahrens (1992) conducted wave tank experiments to understand the factors governing deformation and stabilization of cobble beach profiles. To estimate critical mass (in volume/unit length of shoreline), it is necessary to estimate three fundamental dimensions of the dynamic revetment: berm crest height (hc); berm crest length (lc); and erosion length (le). A definition sketch of terms is included in Figure 5-2.

Critical Mass Calculations

$$H_c/H_{mo} = 0.270*(H_{mo}/L_p)^{\wedge}(-0.645)$$

$$L_c/H_{mo} = 0.677*(H_{mo}/L_p)^{\wedge}(-0.521)$$

$$L_e/Ds = \exp(2.24*(H_{mo}/L_p)^{\wedge}(0.143))$$

Where: $h_c$ = berm crest height
$l_c$ = berm crest length
$l_e$ = erosion length
Lp = Wavelength at toe of structure
Hmo = incident zero-moment wave height
Ds = toe water depth
Tp = Peak wave period associated with peak energy density

Hmo was assumed to be equal to significant wave height, 1.5 meters (Michalson, personal communication 2021). Tp was determined to be about 17 seconds (COE, 2018), assuming the peak energy density is during winter storms – typical for this region. We computed Lp iteratively, using the dispersion equation:

$$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$$

Where T = Tp (peak wave period), g = gravitational acceleration (9.81 m/sec2), and d = Ds (toe water depth). To resolve the inclusion of L with the equation, we used Ekhart approximation:

$$L \approx \frac{gT^2}{2\pi} \sqrt{\tanh\left(\frac{4\pi^2}{T^2} \frac{d}{g}\right)}$$

We used the solver function in Xcel to minimize the error between the approximation and the dispersion equation (see Appendix A).

With the wavelength solved, we calculated Hc, Lc, and Le. From there, critical mass is calculated using a scale (As):

As = (Ds + Hc)*(Le + Lc)

The minimum critical mass is expressed as 2/3 of As (Ward and Ahrens, 1992):

At = 0.67*As

However, As is a better choice for design, given the uncertainty is estimating critical mass.

We then developed several scenarios in which we varied the key parameter of toe water depth – the elevation at the toe of the structure. We found that this elevation greatly affects critical mass.

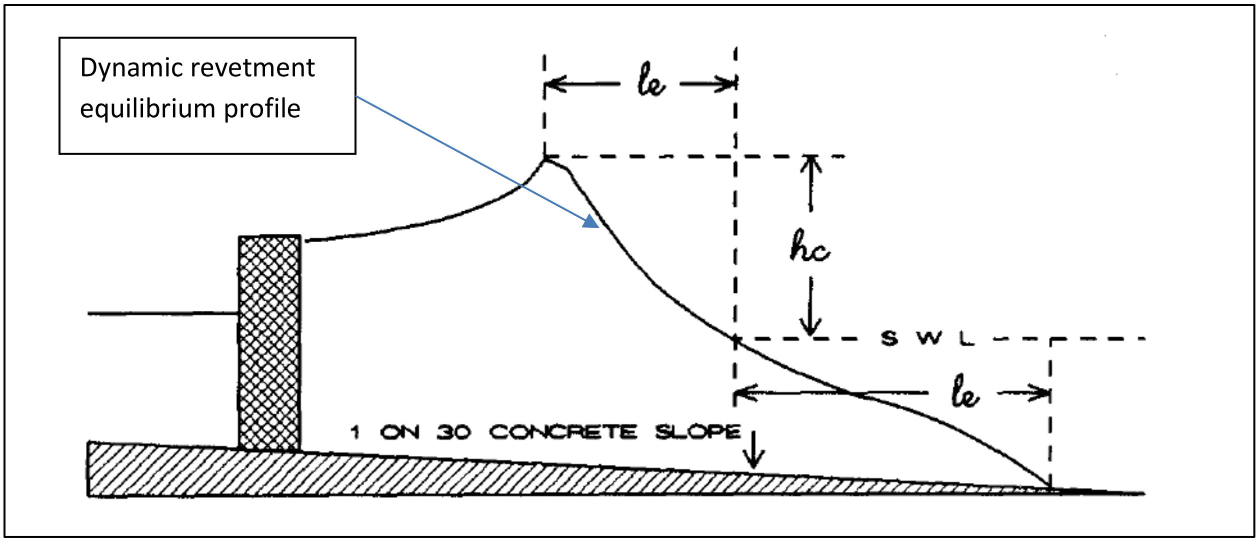


**Figure 5-2: Definition sketch (Ahrens, 1990).**

This is sensitive to toe elevation, and since we want to place only the material necessary for dynamically stable profile, we use the Ahrens equation to run scenarios to achieve optimum volume and placement. Critical mass is expressed in volume per unit length of shoreline (m3/m).

**Table 2. Variation of critical mass with toe water depth**

| Water Level Datum | Total Water level (m, MTL) | total water level (ft, MTL) | Critical Mass (m3/m) | Total Water Level (ft, NAVD88) | Critical Mass (cy/ft) |
|---|---|---|---|---|---|
| MHHW | 1.26 | 4.1328 | 21.34 | 8.09 | 8.5 |
| HAT | 2.03 | 6.6584 | 58.73 | 10.62 | 23.4 |
| 100% AEP | 2.26 | 7.4128 | 70.58 | 11.37 | 28.1 |
| 50% AEP | 2.4 | 7.872 | 77.98 | 11.83 | 31.1 |
| 20% AEP | 2.56 | 8.3968 | 86.63 | 12.36 | 34.5 |
| 10% AEP | 2.66 | 8.7248 | 92.13 | 12.68 | 36.7 |
| 2% AEP | 2.89 | 9.4792 | 105.07 | 13.44 | 41.9 |

As can be seen in the above table, the critical mass increases with increasing toe water level, which relates back to the effect of depth on wave height. The choice of toe water depth will affect critical mass needed for an equilibrium slope. We expect and plan for a maintenance cycle of 5-10 years, and therefore an appropriate return interval for toe water level would be 10-20% AEP. To reduce more frequent maintenance episodes, we have chosen a critical mass on the high end of that corresponding range, at 36 cy/ft, which is close to the 10% AEP critical mass of 36.7 cy/ft. The selected critical mass helps to reduce excavation and disturbance of the project area, while keeping the maintenance cycle at a reasonable frequency. Notably, the Corps of Engineers used a different technique to design dynamic revetment dimension at the MCR South Jetty. The design report (COE, 2013) in fact does not provide an analysis of profile adjustment, only that there is sufficient volume in the design to allow for reshaping by waves.

### 5.2.3 *Shoreline sections and materials*

The alignment is divided into four sections, based on the specific needs and materials, at these locations: the eastern terminus; the main section; the western transition; and the western terminus. The sections are shown in Figure 5-3.

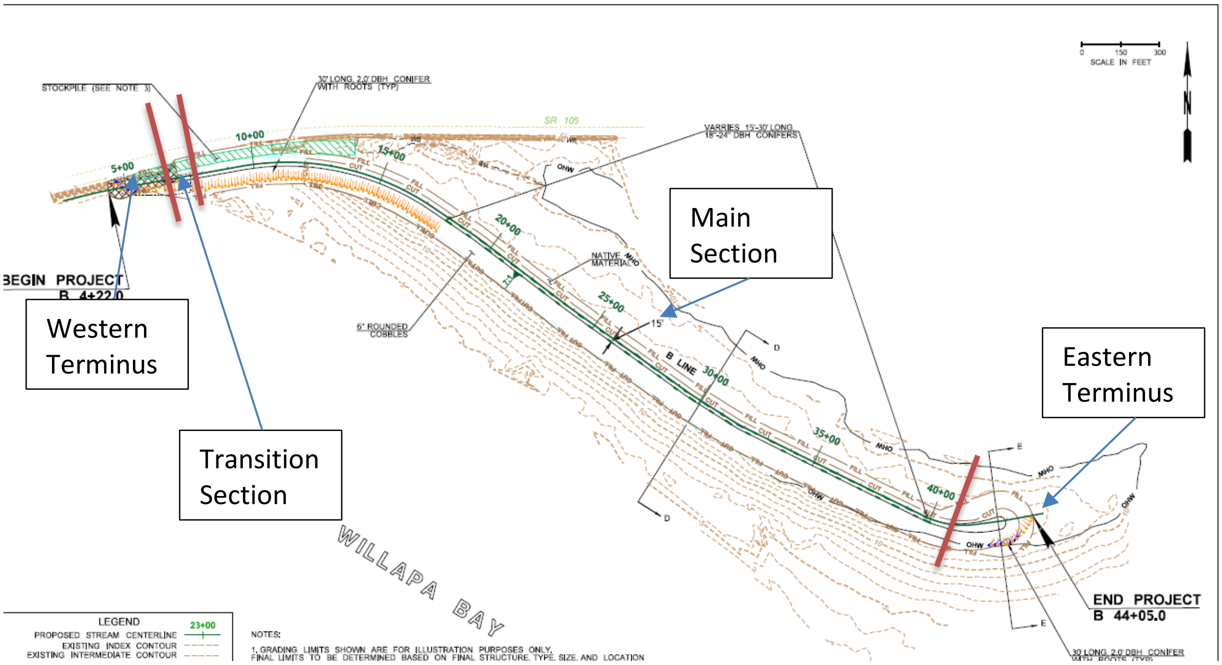


**Figure 5-3. Dynamic revetment sections. Red lines indicate the break between sections.**

We examined nearby similar, completed projects along the Pacific Coast of North America, as well as the reference beach. The backing in this case is usually a cliff composed of bedrock or weakly indurated alluvium.

At the project site, there is no backing for most of the length of the structure alignment. Therefore, an "artificial dune" is needed. An example project with an artificial dune is located at Cape Lookout State Park, Oregon. In this location, a dynamic revetment was constructed in 1999, and a backing of an artificial dune consisting of sand-filled super sacks was included (Figure 5-3).

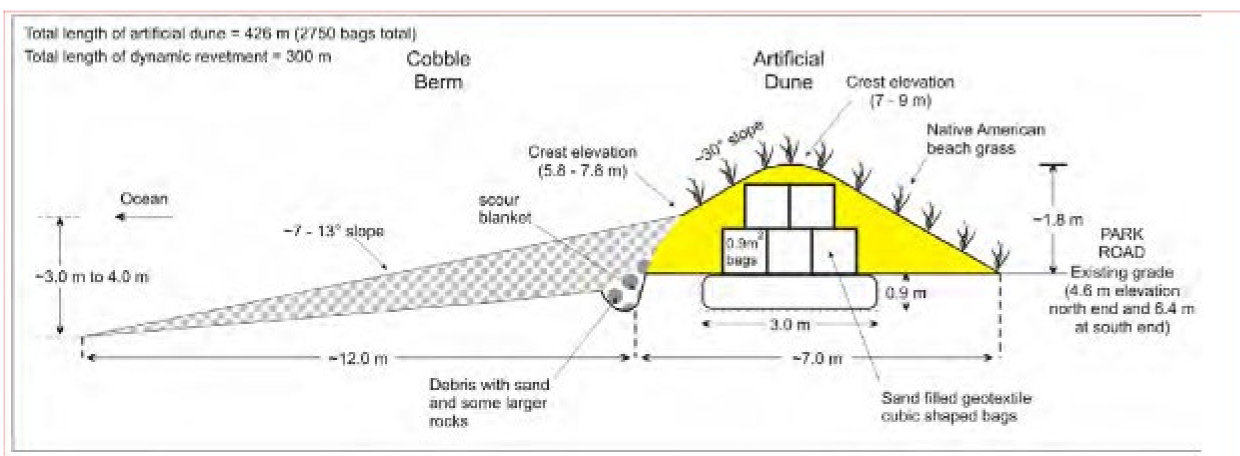


**Figure 5-4. Dynamic revetment and artificial dune at Cape Lookout State Park, Oregon.**

For the current project site, the sand-filled "super sacks" were first considered. It was viewed as potentially economical if sand could be dredged from Willapa Bay, as was done on the Empire Spit project. However, the feasibility of creating a new dredging zone or even using the Corps of Engineers' permitted dredge area was ruled out due to the likely regulatory and logistical complications involved. Because the position of the dynamic revetment structure was selected to account for ongoing shoreline retreat, construction will involve excavation. This will create borrow material, mostly sand with some much and sod. Additionally, one for of the goals of the project is to restore part of the backshore estuary. This would necessitate removal of the thicker portion of the sand lens deposited on the estuary surface near the project alignment. The design team realized that with these two excavation components, enough native material could be retained on side to provide the backing to the dynamic revetment. This also saves on hauling and disposal of borrow material, and provides a planting substrate for dune grasses. Much of the length of the dynamic revetment structure includes a portion of native material used this way.

The following paragraphs describe in more detail the four sections of the dynamic revetment structure: the eastern terminus, the “main” section, the western terminus, and the transition section.

In the eastern terminus section (Figure 5-5), the dynamic revetment is lobe-shaped, to accommodate varying current directions as high tides drain the estuary. There is also large woody material present in the design, to stabilize the terminus, since the structure thins and does not meet the critical mass calculated for the main section of the revetment.

The main section is about 3300 feet long, from station 7+00 to 40+00. The main section has a consistent plan view and cross-section (Figure 5-1, 5-4). The dynamic revetment crest will be 21.5 feet high in elevation (NAVD88) and will be 15 feet wide. The toe elevation will be at 7.0 feet NAVD88 based on critical mass calculation (see Section 5.3), and with a 7:1 slope to the crest. The core layer and the surface layer will have the same slope. The slope of the excavated surface will be flat as shown in the cross-section. On the landward side of the structure, the borrow material will be placed with a 2.5:1 slope at the toe, decreasing to 3:1, and then 4:1 as shown. Because this design assumes a portion of overwashed sand can be reused, hand augering should be conducted just prior to construction to determine the thickness. The ability to efficiently dig up this material will probably be limited to those areas of greater than 2 feet of thickness. Sand excavated to place the toe of the dynamic revetment is also expected to be reused to construct the backshore dune. Depending on the amount of shoreline recession between design and construction, there may be less of this sand available than at the current time.

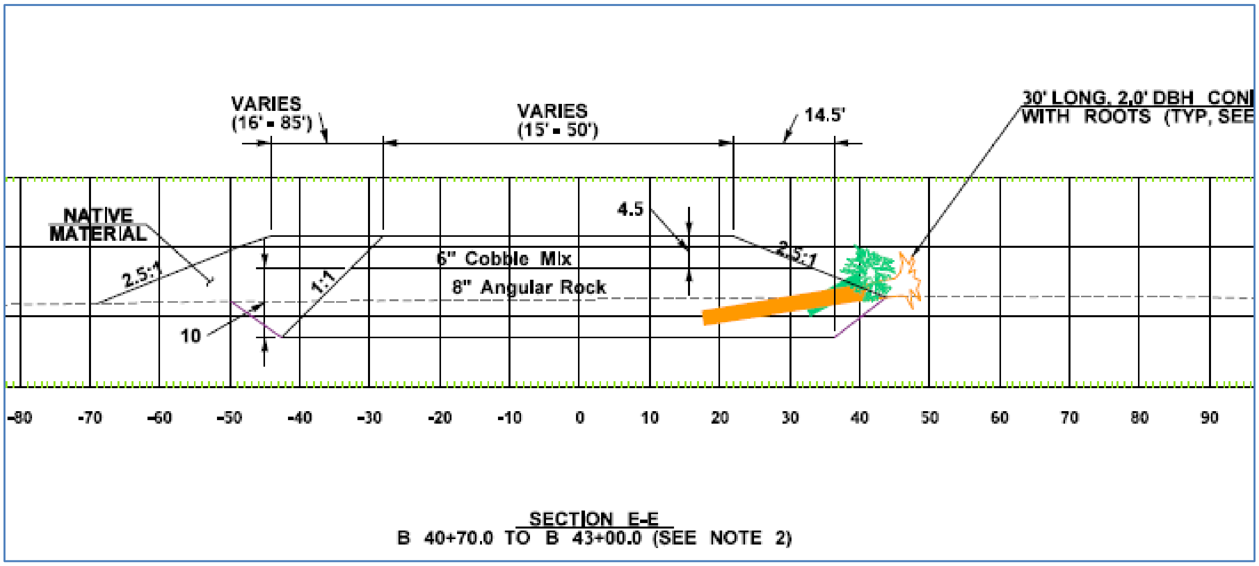


**Figure 5-5. Eastern Terminus cross-section.**

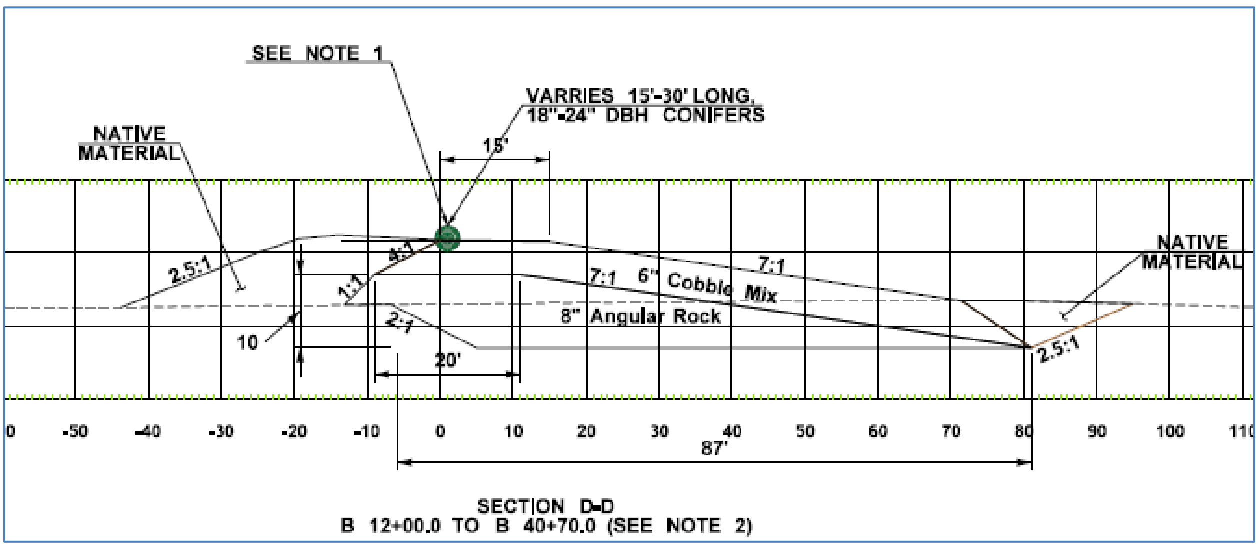


**Figure 5-6. Main section cross-section D-D.**

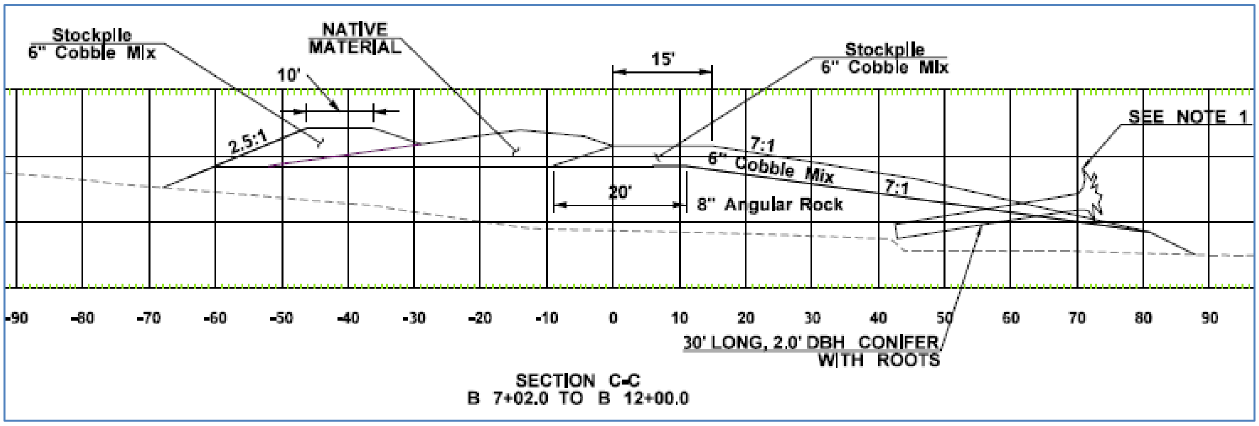


**Figure 5-7. Main section cross-section C-C.**

The placed borrow material will have a top elevation somewhat higher than the dynamic revetment, at about 23.5 feet NAVD88. The crest of the dynamic revetment will have LWM with a mix of diameters and lengths as described in section 5.5. Compared to the critical mass calculation, the main section has 36 yd3/lineal foot of shoreline.

At the western end, where the structure ties into the existing highway embankment, the wave energy is intense. Wave refraction on the existing rock revetment on SR105 produces incident waves. To provide a stable interface to existing terrain, this section is composed of a base layer of large angular rock (see section 5.3), with large woody material placed to break up incident waves (see Figure 5-8) that form on the highway revetment (Figure 5-9). The LWM is oriented to provide a roughness in the water column at most elevations above 8 feet NAVD88, up to about 17 feet.

**Figure 5-8. Location of western transition, during February 28, 2022 storm.**

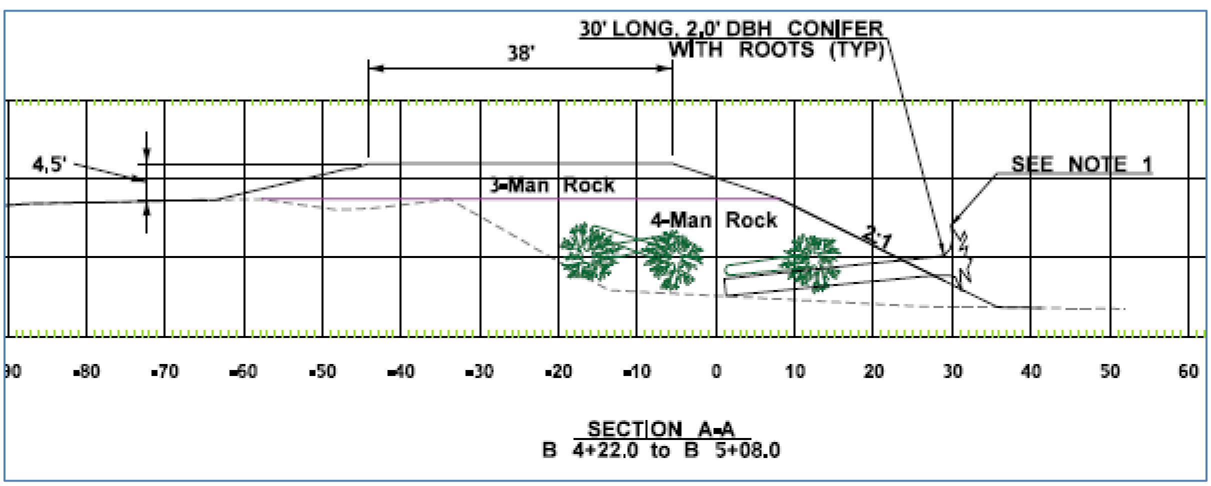


**Figure 5-9. Western terminus cross-section.**

Between the western terminus and the main section is a transition section (Figure 5-10). The materials in this section include a base of four man rock up to 15 feet NAVD88, with another 3 feet of three-man rock, and capped with 3 feet of angular rock, 8" minus. LWM is included in the transition section. This section is slightly more deformable than the western terminus, and meant to reduce transference of wave energy from the relatively state western terminus to the main section. The transition section also includes part of the stockpile, which will be placed such that additional cobbles can become mobilized and be a source for additional cobble downdrift and onto the main section of the dynamic revetment.

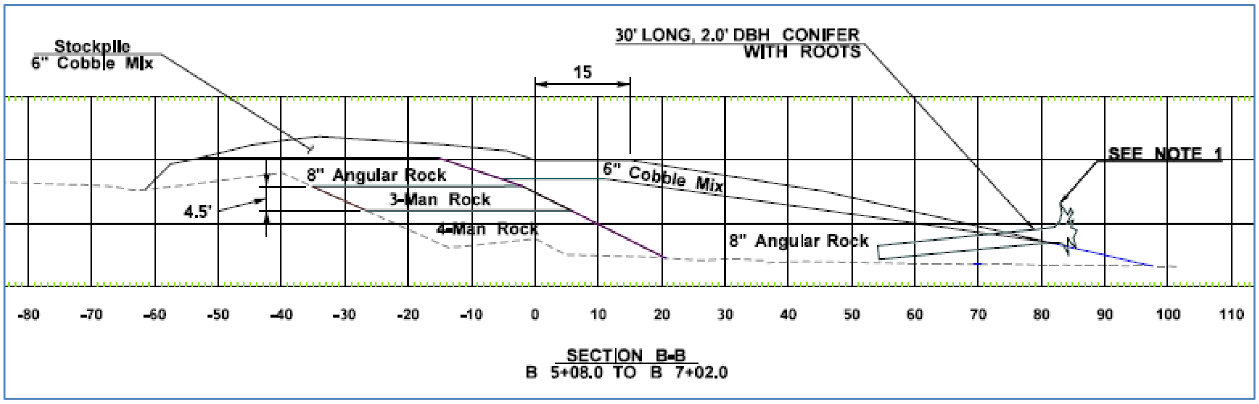


**Figure 5-10. Transition section cross-section.**

## 5.3 Dynamic Revetment Materials

The materials used are a crucial part of performance of a deformable, energy-absorbing structure. The position in the landscape, the wave climate, and project objectives factored in materials design. Also position along the shore was a major consideration in selecting materials. The four sections outlined above each have their own set of materials, though they share some of the same individual types of materials.

### 5.3.1 *Angular cobbles*

Because of the difficulty procuring rounded cobbles – the nearest source of material is about 60 miles away – the core of the dynamic revetment structure will consist of angular cobble-sized rock from a nearby quarry. The angular cobble will be no greater than 8" diameter pieces. The cobble will meet WSDOT Spec 9-03.11(2) 8" Cobbles. Examples of angular cobble in the immediate vicinity of the project indicate this size and type of rock becomes somewhat rounded within a few years of exposure to ocean waves (George Kaminsky, Dept. of Ecology, pers. comm. 2021). WSDOT has also observed some rounding of angular cobble along SR105 (Figure 511).

### 5.3.2 *Rounded Cobbles*

The dynamic revetment surface layer will consist of rounded cobbles, meant to simulate the reference beach at Kalaloch. A layer 4.5 feet thick, appropriate for the estimated significant wave height of 1.5 meters, will be placed on the core layer. The rounded cobbles will have a median grain size of 6 inches, with a gradation equivalent to WSDOT Specification 9-03.11(2) Streamed Cobbles. .

**Figure 5-11  Partial rounding of angular cobble placed as part of rock revetment, just west of Graveyard Spit.**

The anticipated attrition rate is based on empirical data from existing dynamic revetments in Oregon and from modeling results, completed for this project.  At the Columbia River South Jetty, the dynamic revetment completed in 2018 (Figure 5-12) lost about 3800 yd3 of cobble in 5 years (DOGAMI, 2018). This is about 7.5% volume loss, for an average annual volume loss of 1.5%.  Most of the loss was on the northern end of the structure, where it connects with the south jetty.  The location is similar to the western terminus of this project.  Additional design background on particle attrition is presented in section 8.2.

**Figure 5-12. Dynamic Revetment at the South Jetty, Columbia River, in 2018.**

### 5.3.3 *Dune restoration materials*

The landward side of the dynamic revetment will be composed of borrow material, salvaged from the excavation of the toe of the structure on both the landward and waterward side. The borrow material, which will be almost entirely sand, will be placed adjacent to the dynamic revetment, and would be extend slightly higher, to 23.5 ft. This side of the dynamic revetment structure will be revegetated with appropriate plant species. Sand overwash areas beyond the toe of the structure are expected to revegetate natural from adjacent seed sources. This area will be occasionally inundated by high tides (though not waves). The native material will be placed and compacted lightly.

### 5.3.4 *Rock revetment sizing*

In anticipation of wave run up and refraction, the western terminus will be constructed with angular four-man rock. The design is similar to that of the revetment west of the project area. However, there will be large woody material embedded in the rock as discussed in section 5.5.

FHWA recommends using Hudson's equation to size rock armor in high energy coastal environments (FHWA, 2020). This equation (see Figure 5-13) uses significant wave height and revetment slope as key variables. Also, input is buoyant density of the rock used.

$$\frac{H_s}{\Delta D_{n50}} = \frac{(K_D \cot\theta)^{1/3}}{1.27}$$

where:

- $H_s$ is the design significant wave height at the toe of the structure (m)
- $\Delta$ is the dimensionless relative buoyant density of rock, i.e. $(\rho_r / \rho_w - 1)$ = around 1.58 for granite in sea water
- $\rho_r$ and $\rho_w$ are the densities of rock and (sea)water (-)
- $D_{n50}$ is the nominal median diameter of armor blocks = $(W_{50}/\rho_r)^{1/3}$ (m)
- $K_D$ is a dimensionless stability coefficient, deduced from laboratory experiments for different kinds of armor blocks and for very small dama
  - $K_D$ = around 3 for natural quarry rock
  - $K_D$ = around 10 for artificial interlocking concrete blocks

**Figure 5-13. Hudson's Equation**

Using a significant wave height of 1.5 meters, and a 2:1 slope, and assuming basalt as the rock type yields a median rock size of about 2 feet. However, the incident wave angle at the junction of the western terminus with the adjacent rock revetment is expected to accentuate significant wave height. Therefore, a significant wave height of 3 meters was assumed, yielding median armor size of 3.8 feet. Four-man rock (size range 36-48") was therefore selected for the western terminus.

The transition section will include an upper layer of somewhat smaller three-man rock (28"-36"). This size and layer were selected to provide a transition in size distribution from the coarser rock of the western terminus. The wave energy at this height on the structure will be somewhat less than at the base.

The Corps of Engineers is planning on repairs in 2023 of the Empire Spit dune (Shoalwater Bay Project Project). There may be the option of using some of the material in the haul road planned for construction. More information is presented in Appendix D.

## 5.4 With Project Conditions Modeling

### 5.4.1 *XBeach-G modeling*

The XBeach-G model was developed as an extension of XBeach to incorporate groundwater infiltration through permeable media such as cobble beaches (McCall et al. 2012). XBeach-G is a non-hydrostatic form of the XBeach model developed jointly between Deltares and Plymouth University in the UK (McCall et al. 2015; Deltares 2018). Improvements to include the effects of ground water infiltration through porous gravel and cobble beaches has been incorporated (Figure 5-14).

Model boundary conditions used to force the model include a wave and water level time series. This data is obtained from the CMS model output at the Graveyard Spit observation point 10. Other model parameters include grain size information (d50) and hydraulic conductivity (kx) which determines the rate of groundwater infiltration through the cobble layer.

Calibration of XBeach-G was performed by adjusting the friction factor (fs ) and phase lag (ϕ ) defined by Nielsen (2002). Multiple simulations were performed for combinations of ϕ= 0 to 60° and fs = 0.0125 to 0.025. The resultant foreshore slope of the dynamic revetment was compared to the measured slopes found at the North Cove pilot project located to the northwest of the Graveyard Spit project. It was determined that ϕ = 25° and fs = 0.0125 produced the best match for the equilibrium foreshore slope measured at the pilot project. In general, the lower phase lag resulted in more sediment movement downslope and larger phase lag resulted in more erosion near the toe of the structure with steeper foreshore slopes.

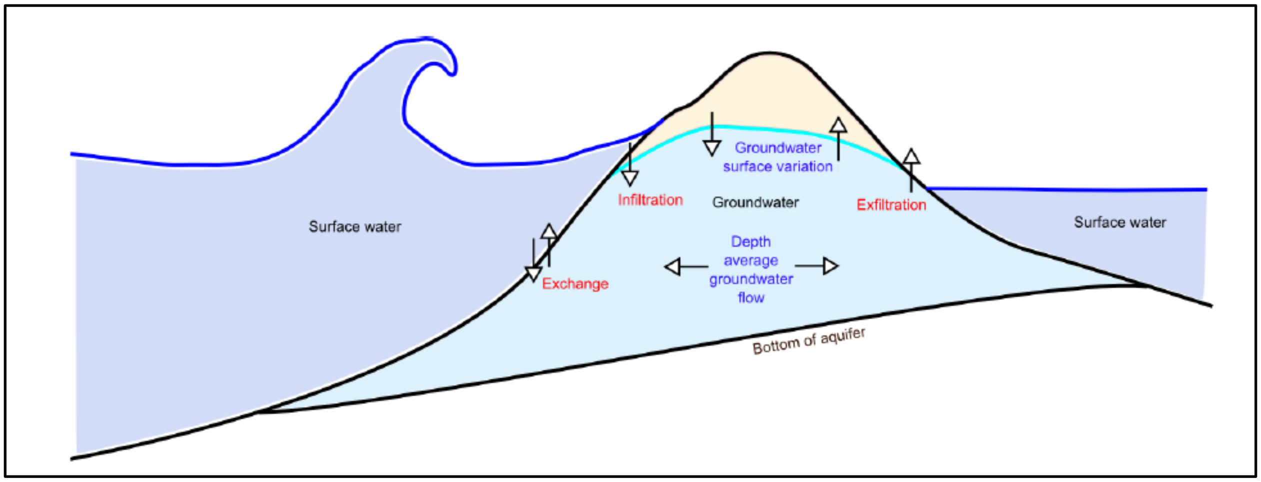


**Figure 5-14. Description of coastal swash zone processes on a permeable gravel/cobble beach**

### 5.4.2 *Input parameters*

Beach profile morphology was evaluated for two cobble gradations for the dynamic revetment (See Appendix A). Default parameters were maintained whenever possible. Grid sizes range from 10 m in the offshore region to 1 m in the nearshore region. The criterion of wavelength to cell size was increased to 30 and the Courant-Friedrichs-Lewy (CFL) condition was set at 0.7. Infiltration through the cobble media was assumed to be 0.1 m/s in all simulations.

The mobility analysis discussed earlier suggests that the gradation of the dynamic revetment cobbles should be specified in the range of d50 = 0.1 to 0.2 m (4 to 8 in.). Similarly, the initial design cross-section of the dynamic revetment proposed is based on the critical mass computation following Ward and Ahrens (1992). This results in a dynamic revetment with a toe elevation of 7 feet NAV88 (0.9 m MTL), a crest elevation of 21.5 feet NAVD88 (5.3 m MTL), a crest width of 15 feet, and an initial foreshore slope of tan β = 1/7 = 0.14.

The performance of the structure with and without the backshore dune was also evaluated in XBeach-G. The backshore sand dune, as discussed in section 5.2.2, would be constructed from to a height of 23.5 feet NAVD88 (5.6 MTL).

### 5.4.3 *Wave run-up computations*

As with existing conditions, five historical storm scenarios were simulated for a dynamic revetment where 1) d50 = 0.1 m and 2) d50 = 0.2 m. Whether or not the structure is overtopped is dependent on both the total water level and offshore wave conditions. An annual exceedance probability (AEP) of the joint occurrence of extreme water levels and wave height was developed to understand the probability of such events. To simplify the analysis, independence between the two parameters is assumed. Thus, the joint probability is computed as the product of the TWL and Hs AEPs (see Appendix A, Table 3).

In the scenario without a backing dune, XBeach-G results indicate that the dynamic revetment prevents overwash of Graveyard Spit for the storm events up to a joint TWL-Hs 2% AEP. However, the dynamic revetment would be overtopped during events equivalent to the February 4, 2006 and December 3, 2007 storm events. Overwash of the dynamic revetment would result in landward migration of the structure (Figure 5-3). In addition, the height would lower by about 3 feet, making it susceptible to future storms of similar or greater magnitude. Storm events more frequent than the joint TWL-Hs 2% AEP would not overtop the dynamic revetment, as the wave runup is computed to achieve a maximum of 4.8 m MTL.

In the scenario with a backing dune, wave overwash is prevented for 4 of 5 storm events evaluated. The maximum wave runup was computed at Z2% = 6.0 m MTL (23.6 ft NAVD88) during the February 4, 2006 storm event. The backshore dune height is 23.5 ft NAVD88, indicating the dune could be overtopped. However, this is an extremely rare event (recurrence interval of 2000 years) and the cost of increasing the dune height to prevent wave overtopping is not warranted.

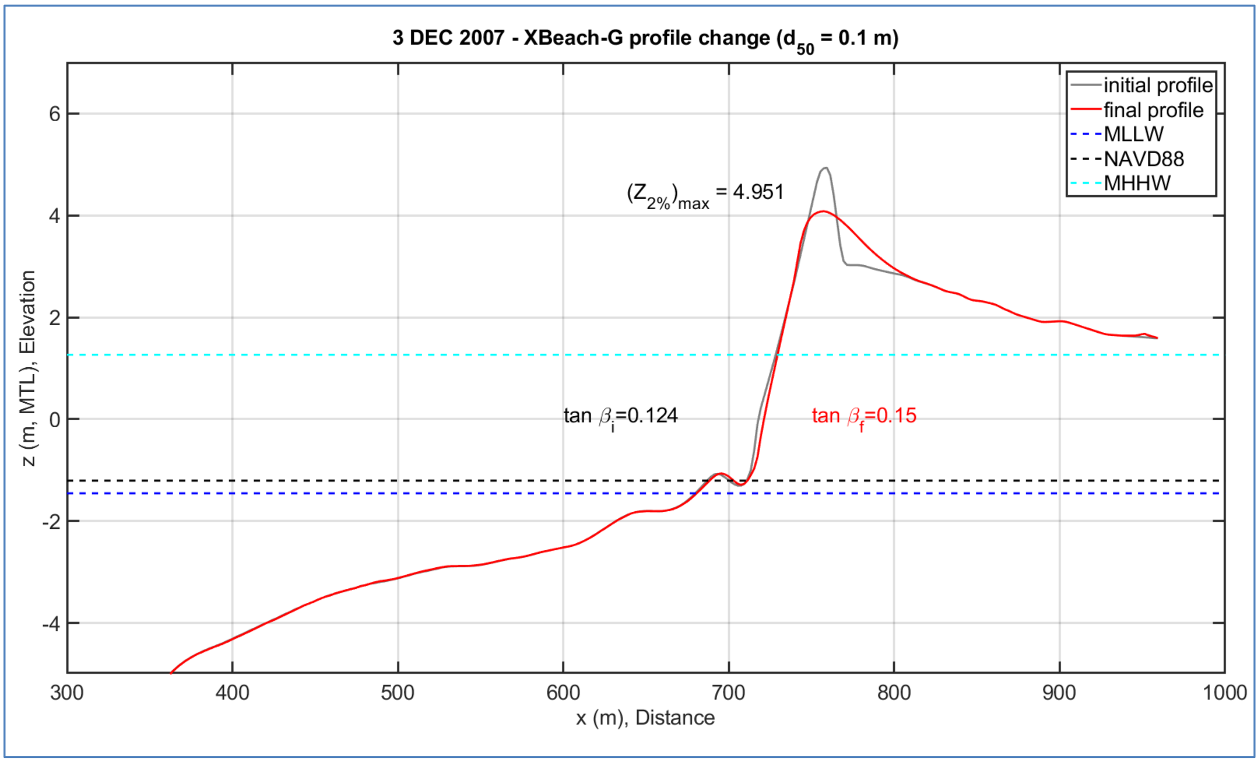


**Figure 5-15. XBeach-G profile change during December 2007 storm events for a dynamic revetment composed of cobbles with median grain size, d50 = 0.1 m. Final foreshore slope tan β = 0.15.**

## 5.5 Habitat Features

### 5.5.1 *Design Concept*

The dynamic revetment project has several elements providing habitat functions, though is some cases the features have dual purposes. The dune restoration component, on the landward side of the structure, will provide habitat typical of the uppermost intertidal zone and associated uplands, similar to nearby areas such as Leadbetter State Park, and Empire Spit. This portion will be planted with appropriate vegetation (this is described in permit documents and plan sheets).

The crest of the structure will include variously spaced logs of native tree species, without rootwads. These will be a range of diameters, from 12" to 24", and a range of lengths, 20 feet to 30 feet. These logs are primarily for upland habitat. However, they will function as wind breaks to some extent, and will be roughness element to the crest of the structure that would mitigate overwash, should that occur in the future. The primary habitat features are incidental to structure features. Large wood with and without rootwads is included in the design. The objective of LWM with rootwads is to distribute wave energy by breaking up incoming waves. The placement of the LWM at the western terminus is designed to interact with waves at various elevations and orientations (Figure 5-16). Additional wood is expected to rack on the placed LWM. The eastern terminus will also have an array of LWM, with dimensions the same as the western terminus (Figure 5-17).

Over time the roots of the rootwad will degrade, but the mass of the rootwad itself can last for decades, as indicated by nearby LWM placements done in the early 2000s.

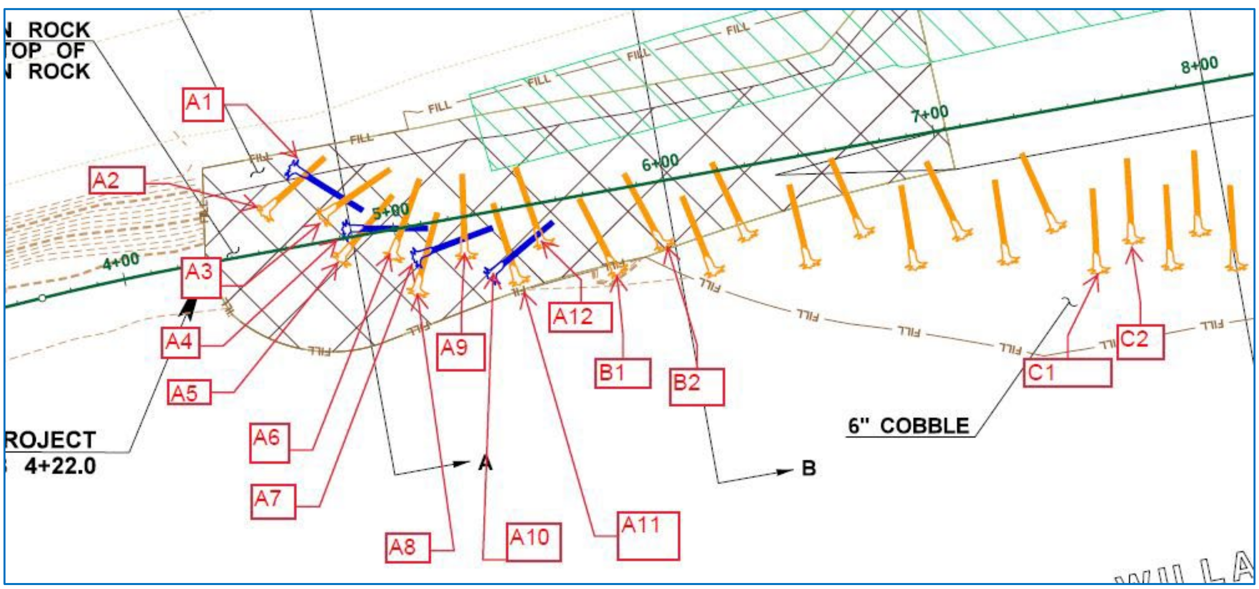


Figure 5-16: Western Terminus LWM.

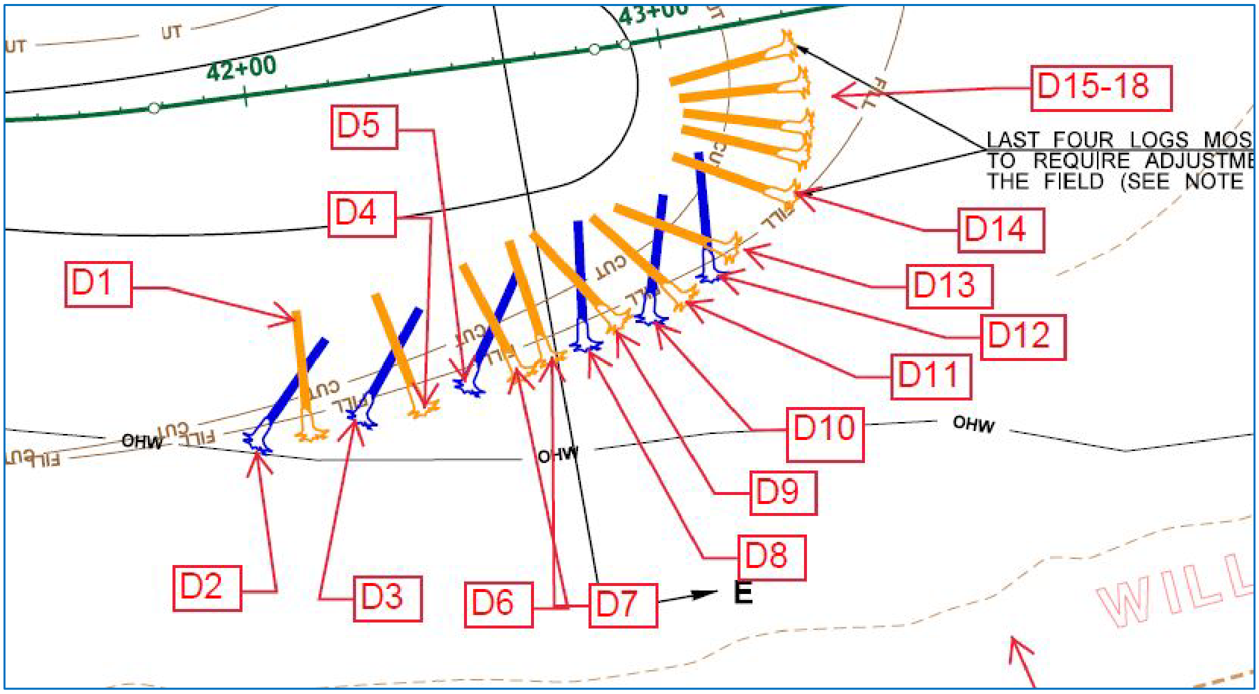


**Figure 5-17. LWM layout at eastern terminus.**

### 5.5.2 *Stability Analysis*

The log stability calculator developed by Rafferty (2016) was used for designing stable LWM for the project. Since the calculator is intended for use along rivers, several assumptions had to be made to adapt it for use in the coastal environment. The maximum water depth was assumed to be the highest astronomical tide. Average water velocity was based on the hydrodynamic modeling for the November 2007 storm event. Since bankfull width is not applicable, 100 feet was assumed. The results of the stability calculations are shown in Table 6.

Table 3: Summary of log stability calculations

| Log (Id Number) | Diameter (inches) | Length (feet) | Buoyancy Fs | Horizontal Fs | Moment Fs | Required Ballast (lbs) |
|---|---|---|---|---|---|---|
| A1 | 24 | 30 | 3.9 | 498 | 10.9 | |
| A2 | 24 | 30 | 6.1 | 707 | 18.1 | |
| A3 | 24 | 30 | 5.2 | 601 | 15.6 | |
| A4 | 24 | 30 | 5.1 | 648 | 19.4 | |
| A5 | 24 | 30 | 8.5 | 1034 | 31.4 | |
| A6 | 24 | 30 | 12.4 | 2280 | 46.9 | |
| A7 | 24 | 30 | 4.5 | 518 | 13.6 | |
| A8 | 24 | 30 | 10.0 | 1984 | 28.1 | |
| A9 | 24 | 30 | 10.6 | 2317 | 30.8 | |
| A10 | 24 | 30 | 5.5 | 623 | 14.8 | |
| A11 | 24 | 30 | 11.5 | 3399 | 33.5 | |
| A12 | 24 | 30 | 11.4 | 6108 | 33.4 | |
| B1* | 24 | 30 | 12.7 | 6434 | 27.8 | |
| B2* | 24 | 30 | 12.1 | 6162 | 26.6 | |
| C1* | 24 | 30 | 11.6 | 4942 | 25.8 | |
| C2* | 24 | 30 | 11.8 | 5818 | 26.9 | |
| D1 | 24 | 30 | 9.4 | 3471 | 20.5 | |
| D2 | 24 | 30 | 8.3 | 648 | 23.3 | |
| D3 | 24 | 30 | 2.7 | 194 | 5.6 | |
| D4 | 24 | 30 | 6.3 | 2731 | 13.8 | |
| D5 | 24 | 30 | 2.8 | 207 | 7.61 | |
| D6 | 24 | 30 | 4.2 | 1216 | 8.4 | |
| D7 | 24 | 30 | 4.2 | 1216 | 8.4 | |
| D8 | 24 | 30 | 3.6 | 392 | 7.2 | |
| D9 | 24 | 30 | 2.0 | 204 | 3.0 | |
| D10 | 24 | 30 | 3.4 | 360 | 6.7 | |
| D11 | 24 | 30 | 1.7 | 169 | 2.6 | |
| D12 | 24 | 30 | 2.8 | 260 | 5.2 | |
| D13 | 24 | 30 | 1.5 | 133 | 2.1 | |
| D14 | 24 | 30 | 4.2 | 1216 | 8.4 | |
| D15-18* | 24 | 30 | 2.2 | 348 | 3.8 | |
| | | | | | | |
| | | | | | | |

(1) Assumes boulders with submerged specific gravity of 1.65

(2) Negative value indicates anchor and overburden moments exceed buoyant moments

(3) *Indicates typical stability for a series of identical logs

# 6 Floodplain Changes

The project area is located entirely in FEMA Flood Zone VE. The project is not expected to result in significant changes to base flood elevation since it dominated by the Pacific Ocean, and there is almost limitless area for flow expansion. According to officials with Pacific County (Zane Johnson, pers. comm. 2021), a conditional letter of map revision (CLMR) will not be needed, nor is a no-rise analysis needed.

# 7 Climate Resilience

WSDOT recognizes climate resilience as a component of the integrity of its structures and approaches design through a risk-based assessment beyond the design criteria. For coastal structures, the largest risk to the structures will come from increases in wave heights and/or sea level rise.

## 7.1 Climate Resilience Tools

Sea level change is an uncertainty, potentially increasing the frequency of extreme water levels. Planning guidance in USACE Engineering Regulation (ER), USACE ER 1100-2-8162 (USACE 2013), incorporates projections by the Intergovernmental Panel on Climate Change and National Research Council. Since predictions of future SLC have uncertainty, the risks associated with three scenarios were analyzed (see Appendix A). These scenarios are “low”, “intermediate”, and “high” and correspond to different rates of sea level rise. Recently, global (eustatic) sea level rise rate has been approximately 1.7 millimeters (mm) per year.

Sea level rise varies geographically as it is the difference between the global rise (1.7 mm/year according to IPCC 2007) and local vertical land movement (VLM), which can be caused by a variety of forces. In the case of the Willapa Bay estuary, there is a pattern of rapid subsidence followed by uplift, with a long-term trend of sea transgression (Petersen and Vanderburgh, 2018). This is due to the ongoing subduction of the Juan de Fuca plate under the North American plate. Thus, local sea level rise is expected to be greatly affected by the underlying tectonic forces.

Long periods of tide levels help estimate the recent amount of sea level rise, which is the sum of eustatic rise and local vertical movement. USACE (2013a) recommends that a National Oceanic and Atmospheric Administration (NOAA) water level station should be used with a period of record of at least 40 years. The mean historic sea level change observed Toke Point since 1972 is approximately 0.4 mm/year. By 2100, the predicted sea level rise at the project ranges from 0.6 to 4.9 feet (Figure 7-1).

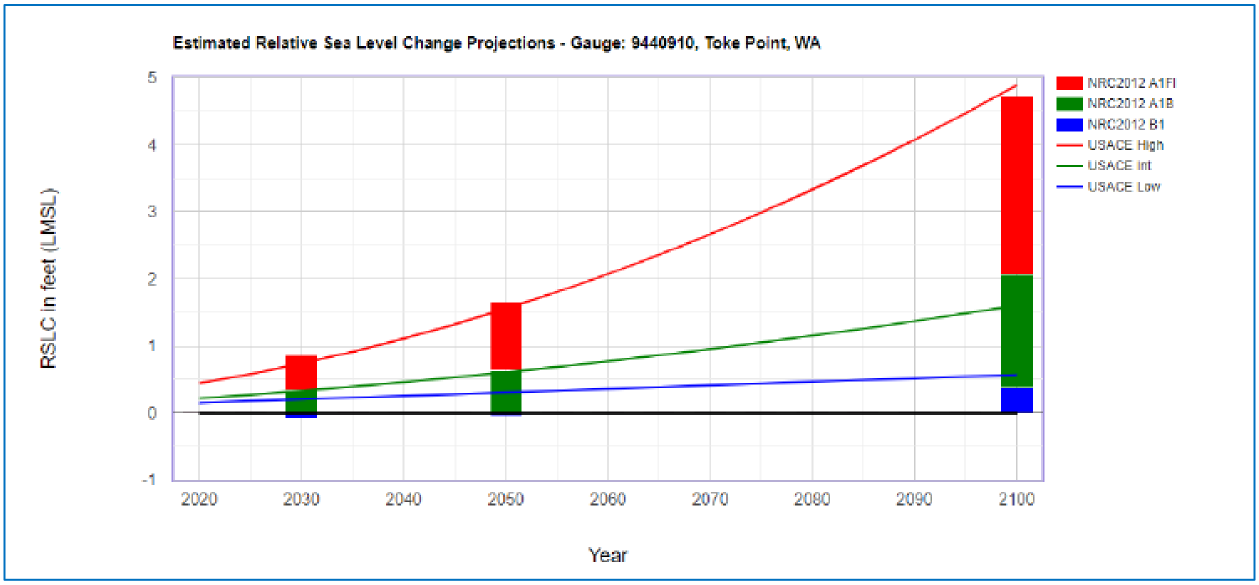


**Figure 7-1. Range in predicted sea level rise at Toke Point.**

## 7.2 Hydrology

Increases in sea level could result in increased toe water depth, which would affect significant wave height. Under the moderate SLR scenario, by 2060, local sea level will be 2.5 feet, and thus the toe water depth would increase by 2.5 feet. However, the basis of the selection of the dynamic revetment is the ability to self-adjust, as shown schematically in Figure 7-2. We expect that the toe depth would move inland with sea level rise, and the crest of the dynamic revetment would be deposited higher. Under the high scenario however, a rise of 6.9 feet, the rate sea level rise could exceed the dynamic revetment's ability to adjust. Additional dune material or cobble could be needed beyond the planned maintenance amount.

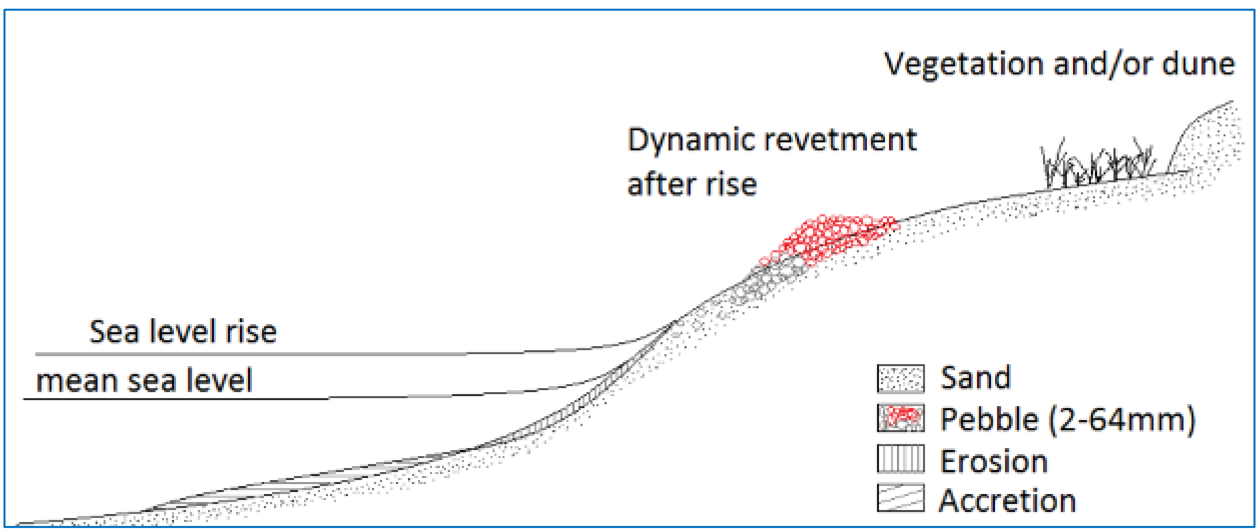


**Figure 7-2. Schematic of dynamic equilibrium after sea level rise (after Bayle, et. al, 2016).**

## 7.3 Climate Resilience Summary

We expect that the dynamic revetment can self-adjust to achieve an equilibrium profile under a moderate scenario of sea level rise. Additionally, the project is designed with maintenance as an integral component. The Adaptive Management Team (see Section 8.0) would be able to add or move material as needed to maintain critical mass and height.

# 8 Adaptive Management Plan

## 8.1 Background

The basis of this project's design includes the understanding that coastal protection measures always require maintenance. Furthermore, use of a dynamic revetment inherently requires maintenance, due to particle attrition, long shore, and cross-shore transport. Although this design uses the best available science, the urgency of coastal retreat in this area requires timely action, which eliminates empirical studies of the site. We do have some information from the pilot project, but the record is only a few years in duration.

Therefore, a flexible, adaptive approach to maintenance is needed to accommodate uncertainty in structure response, and uncertainty in external inputs. The premise of adaptive management is to using monitoring, feedback structures, and anticipated courses of action to respond to changing conditions.

This portion of the document focuses on the adaptive management plan with regard to the dynamic revetment component of the overall structure, including the eastern and western termini. The minimum required life cycle for this project feature is 40 years; expectations are that an effective life cycle of 60 years may be realized if the project is adequately maintained. We expect that prior to 60 years, stakeholders will evaluate this project within the context of the adjacent shoreline projects and decide to revamp or reconfigure the structure as needed, or pursue alternative strategies.

## 8.2 Sediment transport and maintenance cycles

It is anticipated that periodic nourishment of the dynamic revetment on Graveyard Spit will be required. The following analysis from the Corps of Engineers (Appendix A) provides guidance for the frequency of maintenance.

Due to the obliqueness of incident waves, the longshore transport of cobble is expected to move to the southeast over time. However, shoreline change models such as GENCADE are not applicable for coarser grain sizes, therefore empirical relationships specifically are often utilized to estimate longshore transport rates on cobble beaches (van Wellen et al. 2000; Kamphius 1991; van Rijn 2014). The Van Rijn (2014) equation has been validated to several field sites in the UK, Netherlands, and the USA. The relationship relates longshore transport rate by the following:

$$Q_{t,mass} = 0.00018 K_{tide} \rho_s g^{0.5} (\tan\beta)^{0.4} (d_{50})^{-0.6} \left(H_{s,br}\right)^{3.1} \sin(2\theta_{br})$$

where:

Qt,mass = total longshore sediment transport (kg/s)

Hs,br = significant wave height at the breaker line (m)

θbr = wave angle at the breaker line

Ktide = percentage of time the cobble is exposed to the swash zone

Since the original relationship is developed for beaches exposed to the swash zone over the entire tide cycle, the relationship will overestimate the amount of longshore transport for a dynamic revetment. The cobble will be located in the upper intertidal region and native sand will be located below the structure creating a composite beach. To compensate for the probability of time the dynamic revetment structure is exposed to the swash zone, a parameter is used based on the percentage of time that the tide is greater than mean tide level: $K_{tide} = p(\eta(t) \geq \eta_{MTL}) = 0.5$.

The transport rate in mass per second is converted to volume per year

$$Q_{t,vol} = \frac{Q_{t,mass}}{\rho_s} * 86{,}400\ (\frac{sec}{day}) * 365.25\ (\frac{day}{year})$$

Using a 25-year time series developed in the GENCADE simulations, the mean annual transport rate is computed for Graveyard Spit. Employing the van Rijn (2014) equation results in a mean annual longshore transport rate of Qt,vol = 1168.4 m3/year with a standard deviation of 42 m3/year. Converting this into weight is approximately Qt,weight = 2,100 tons/year. This compares to the MRC south jetty dynamic revetment loss of 760 tons/year. The MRC structure has only been in place for 8 years. However, the monitoring data suggest that the longshore transport rate predicted by the Van Rijn equation may be high.

Nevertheless, this estimate will be used for planning purposes, and monitoring (see below) may indicate less attrition. For planning, approximately 6300 tons of cobble every 3 years will need to be replenished to maintain the critical mass on the updrift (western) reaches of the dynamic revetment.

Monitoring will be used to adjust the quantities and frequency of replenishment, if needed. The following sections describe the monitoring plan.

## 8.3 Adaptive Management Team (ADT)

A multi-agency team, the Adaptive Management Team, will guide the management of the dynamic revetment and restored dune. The ADT should be composed of representatives from WSDOT, Ecology, COE, WDFW, and the Shoalwater Tribe (at a minimum). The monitoring team (presumed to be led by Ecology), will produce reports to help ADR with management decisions (i.e., replenishment of the structure).

## 8.4 Monitoring

The objective of monitoring is to provide timely feedback on the condition of the dynamic revetment, and to inform the Adaptive Management Team as to the need for action.

Monitoring will be conducted annually, at the end of the typical storm season, April 1, and also after significant storms. A significant storm is defined as wave heights exceeding the 5 year storm wave heights.

### 8.4.1 *Monitoring Strategy*

The monitoring team will conduct topographic surveys each year, prior to June 1. In addition, supplemental monitoring will occur when storm events when wind gusts reach 40 miles per hour or greater during tides above 8-feet MLLW.

Tidal predictions and observations are found here:

https://tidesandcurrents.noaa.gov/stationhome.html?id=9440910

Wind observations are found here:

https://www.wrh.noaa.gov/mesowest/timeseries.php?sid=KAST&num=168&wfo=pqr

### 8.4.2 *Methods*

Annual monitoring will be conducted using the methods and equipment (or equivalent) used in the Department of Ecology coastal monitoring program. The project area falls within an area already monitored by Ecology, the Columbia River Littoral Cell Beach Monitoring Program. Adjacent to the project area, Ecology has been specifically monitoring the Corps of Engineers' project, the Empire Spit Dune Restoration Project (Weiner, et.al, 2017). This effort includes the use of GPS, using handheld GPS units and a base station. The GPS units will need to be mounted on an all-terrain vehicle (ATV) to cover large areas, as well as using backpack mount to cover areas that are inaccessible to ATVs. Transects will be established based on the lines shown in Figure 8-1. The transects are spaced every 75 meters along most of structure length, with some closer spacing at the eastern and western ends.

Unlike the survey of Empire Spit, the current effort will not include boat-based bathymetry survey. All monitoring work will take place at such times as the beach profile down to MLLW can be obtained. An as built survey will provide the year zero digital terrain model for use in comparison in the monitoring reports.

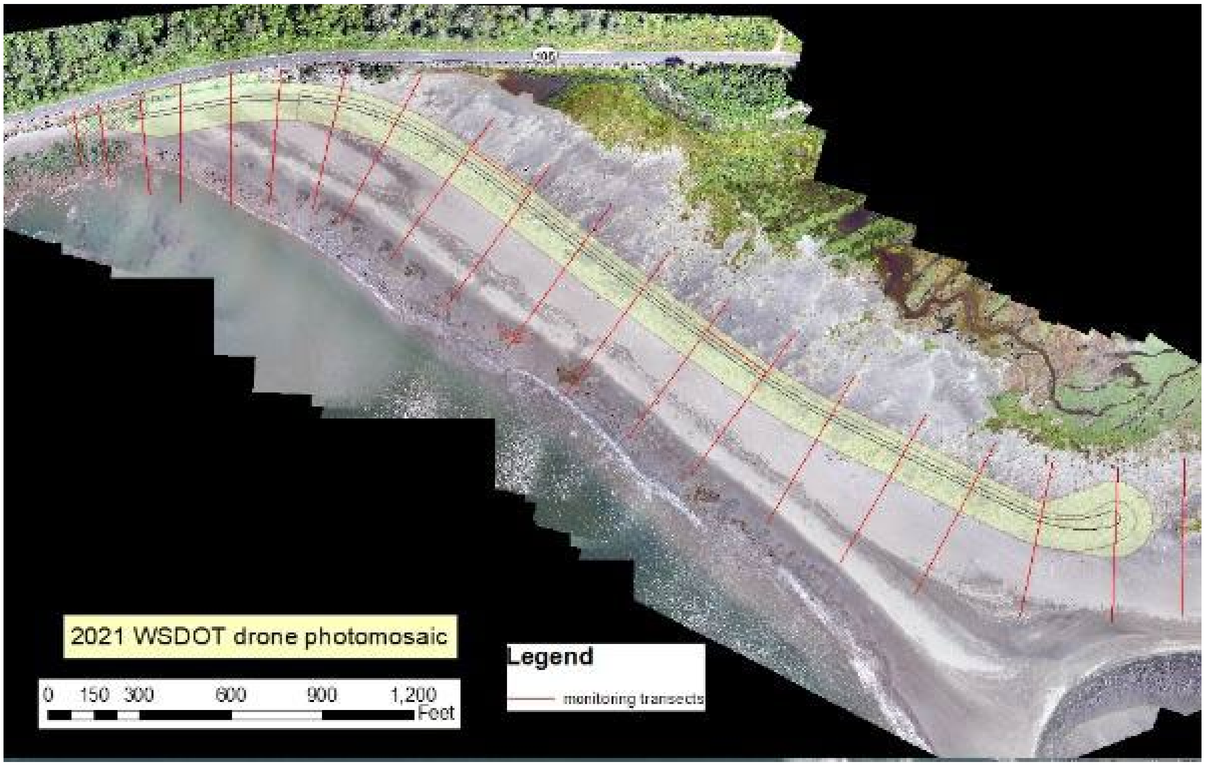


**Figure 8-1. Example of monitoring transect array**

### 8.4.3 *Reporting*

A monitoring report will be produced each year and distributed to the ADT prior to October 1. The report will contain the data collected and include change maps that show the project area topographic change from the prior year, and total change to date.

### 8.4.4 *Meetings*

Before October 31 of each year, the Adaptive Management Team (composed of, at a minimum, representatives from WSDOT, Ecology, COE, WDFW, and the Shoalwater Tribe), will meet in person or virtually to discuss monitoring results. The AMT will determine what actions to be taken, if any, based on decision threshold outline in the section below.

### 8.4.5 *Decision tree/thresholds*

Using the results of annual monitoring, the ADT will initiate replenishment based on the following:

- when the crest width is reduced to less than 10 feet for a continuous length of more than 200 lineal feet, OR
- the critical mass of the structure is reduced to less than 25 yd3/lineal foot for more than 200 lineal feet.

At this point, the revetment cross-section loses its ability to protect the restored dune and backshore from events greater than the 10-year event.

We expect that the western terminus, transition section, and western parts of the main section of the structure will experience the most deformation.   It will likely be the case that cobble moves from the western end of the structure toward the southeast.  This material could be recovered and replaced on the area that it came from.   The designed project includes an on-site stockpile (described in section 8.5), which will be composed of either rounded (preferred) or angular cobble.  The initial material will be 6" rounded cobble.

## 8.5 Stockpiling

An on-site stockpile containing 8000 cubic yards of cobble will be created as shown in project plans.  It will be adjacent to SR105 and placed on top of the western terminus.   The placement of this material allows the stockpile to function like a feeder bluff or be drawn from directly for placement elsewhere along the dynamic revetment.  The volume of material in the stockpile must be maintained periodically. When material is displaced or used from the stockpile, the stockpile will be "topped up" to maintain the volume.   This could be done every 5 years, concurrent with placement of material on the dynamic revetment.  It may need to be topped off sooner than 5 years if it is depleted below 1000 cubic yards.

## 8.6 Revisions

The ADT can revise this adaptive management plan as conditions warrant, in accordance with any applicable permits.

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# Appendices

Appendix A – USACE Coastal Engineering Analysis for a Dynamic Revetment on Graveyard Spit

Appendix B –Plan Sheets, Profile, Details

Appendix C – Large Woody Material Calculations

Appendix D - Option to Include Shoalwater Bay Protection Project Haul Road