US Army Corps of Engineers®
Seattle District

Washington State Department of Transportation

Willapa Bay – Washington State Department of Transportation SR-105

Feasibility of long-term shoreline stabilization alternatives between North Cove and Tokeland, WA

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Contents

1. Introduction

Willapa Bay is a large estuary on the southwest Washington coast located 28 miles north of the Mouth of the Columbia River and 17 miles south of the entrance to Grays Harbor (Figure 1). Its tidal prism at more than 10 billion cubic feet is one of the largest of all inlets on the coast of the continental United States (Jarrett 1976). The large tidal prism is a result of the broad bay area and relatively large tidal range.

Strong tidal currents and energetic waves at the Willapa Bay Entrance collectively act to transport millions of cubic yards of sediment on this predominantly sandy coast. Willapa Bay is a natural inlet unlike neighboring estuaries Grays Harbor and the mouth of the Columbia River, which are protected by jetties that are miles long. Construction of jetties improves navigation by stabilizing the position of the channel, focusing the tidal flow to scour sediment from the channel, and protecting vessels from waves as they transit through the surf zone. Willapa Bay maintains a dynamically stable channel cross section across the Outer Bar of -25 to -30 feet MLLW.

The U.S. Army Corps of Engineers (USACE) has had a navigation mission in Willapa Bay since the early 1900s. The existing project was first adopted in 1916 and last modified through authorization in 1954. The authorization provides for a channel over the bar of the mouth of Willapa Bay to be 26 feet deep, measured to mean lower low water (MLLW), and at least 500 feet wide. Historically the main route for deep-draft navigation was through the main channel thalweg exiting the harbor across a shallow sand bar. The channel across the outer bar shifts from north to south in decadal scale time scales (USACE 1971; USACE 2009). Historically Operations and Maintenance dredging followed the natural thalweg position and deepened these depths to the authorized -26 feet MLLW.

Dredging of the deep-draft channel of Willapa Bay was discontinued by the Seattle District in 1976 because of inadequate economic benefits. Dredging for shallow draft navigation continues in Willapa Bay for local facilities in Tokeland (Toke Point), Bay Center, and Nahcotta. Since 1976, no operations and maintenance dredging has been performed in the Federal navigation channel leading up from the Outer Bar to Port facilities located at Raymond, Washington.

2. Purpose and Scope

The Washington State Department of Transportation (WSDOT) requested assistance from USACE through the Planning Assistance to States Program to analyze potential alternatives to mitigate flood and erosion risk to Washington State Route 105 (SR-105) between the City of North Cove and Tokeland, WA. In recent years, the risk of flooding and erosion has significantly increased due to the loss of the protective dune on Cape Shoalwater. Additionally the shoreline north of a previously constructed groin and dike structure has continued to recede at a rapid rate. The goal of the study is to investigate the technical feasibility of maintaining the roadway in its present configuration over the next 30 years. This study performs the following tasks:

- Reevaluate the Geomorphic Cycle analysis described in USACE (2009) to quantify recent changes near the Willapa Bay North Entrance channel.
- Quantify recent shoreline changes and investigate the shoreface geology
- Update the existing regional wave and circulation model to compute nearshore wave heights, water levels, and currents in Willapa Bay.
- Construct a wave-driven shoreline change model to investigate the performance of various shore protection alternatives over the 30 year life cycle
- Develop conceptual level design alternatives and cost estimates.

3. Historic Harbor Morphology and Prior Engineering Activities

Beach erosion at Cape Shoalwater has been a chronic problem since the turn of the 20th century. The 1880 navigation charts show the entrance to be only 3 miles wide. Between 1887 and 1971, Cape Shoalwater receded 11,700 feet northward. By 1971, shoreline erosion had destroyed 3,000 acres of public and private lands including over 30 homes, businesses, a grange hall, a public schoolhouse, a Coast Guard Station, and twice forced the relocation of the Coast Guard Lighthouse. Washington State Route 105 (SR-105) was relocated landward shortly after 1970 and the prior alignment was lost to erosion by 1978 (USACE 1970; 1978). Since 1984, a total value of lost property was estimated at 20.3 million (WA Sea Grant 2017).

Annual erosion rates ranged from 0 to 250 feet per year. Periods of no erosion were attributed to calmer winter storm years when a continuous outer bar would develop from the Middle Entrance to the North Beach Spit (Figure 2). Higher shoreline erosion rates were correlated to strong winter storm years where the channel thalweg would breach (or sever) the outer bar. It was speculated that these breaching events would cut off southerly transport of sand from the North Beach Spit, which supplied the sediment to maintain a dynamically stable Outer Bar. This change in equilibrium would then allow ocean swells to transport sediments from the severed outer bar into the middle entrance and constrict the North Entrance Channel along Cape Shoalwater. This constriction in channel width then increased the tidal currents and caused acceleration of the shoreline recession along Cape Shoalwater (USACE 1971; Terich and Levenseller 1986).

By 1995, the shoreline had reached SR-105 at the toe of the coastal bluff face, and the road could no longer be moved landward. In 1998, WSDOT constructed a coastal revetment, groin, and dike near the North Entrance Channel at the entrance to Willapa Bay to protect SR-105. The groin structure extended 1600 feet from the upland to the intertidal region and then transitioned into a dike, which extended another 500 feet offshore perpendicular with the North Entrance Channel. In order to compensate for impounded sediments updrift of the groin, 350,000 cubic yards (CY) of sand for dredged from an offshore borrow site and placed downdrift (Southeast) of the groin structure. (WSDOT 2001)

In 2013, the USACE constructed a 12,500 ft. dune restoration on Cape Shoalwater to mitigate flood damage risk the Shoalwater Tribe Reservation. Approximately 710,000 CY of dredged material from an offshore borrow site was used to construct the dune.

In Fall 2017, WSDOT repaired a failing section of SR105 located downdrift of groin and dike and extended the coastal revetment another approximately 1 mile, which also included a test section for a dynamic revetment, or cobble beach.

4. Future shoreline erosion

The Washington State Department of Ecology (WDoE) has developed an empirical model based on historic shoreline positions to estimate future shoreline change on the northern shoreline of Willapa Bay (Talebi et al 2017). The model is based on the inflection point (i.e. intersection) of pre and post shoreline position lines collected from orthorectified aerial photographs (Figure 3). Historically through time, shoreline recession has been observed south of the inflection point and shoreline progradation has been observed northwest of the inflection point. Over time, this inflection point has moved in a northward direction. The shoreline prediction model extrapolates the location of this inflection point into the future over the next 50 years and is used to estimate the future shoreline position (Figure 4). This model projects the shoreline recession to reach SR-105 between 2020 and 2030.

Figure 1: Willapa Bay and vicinity

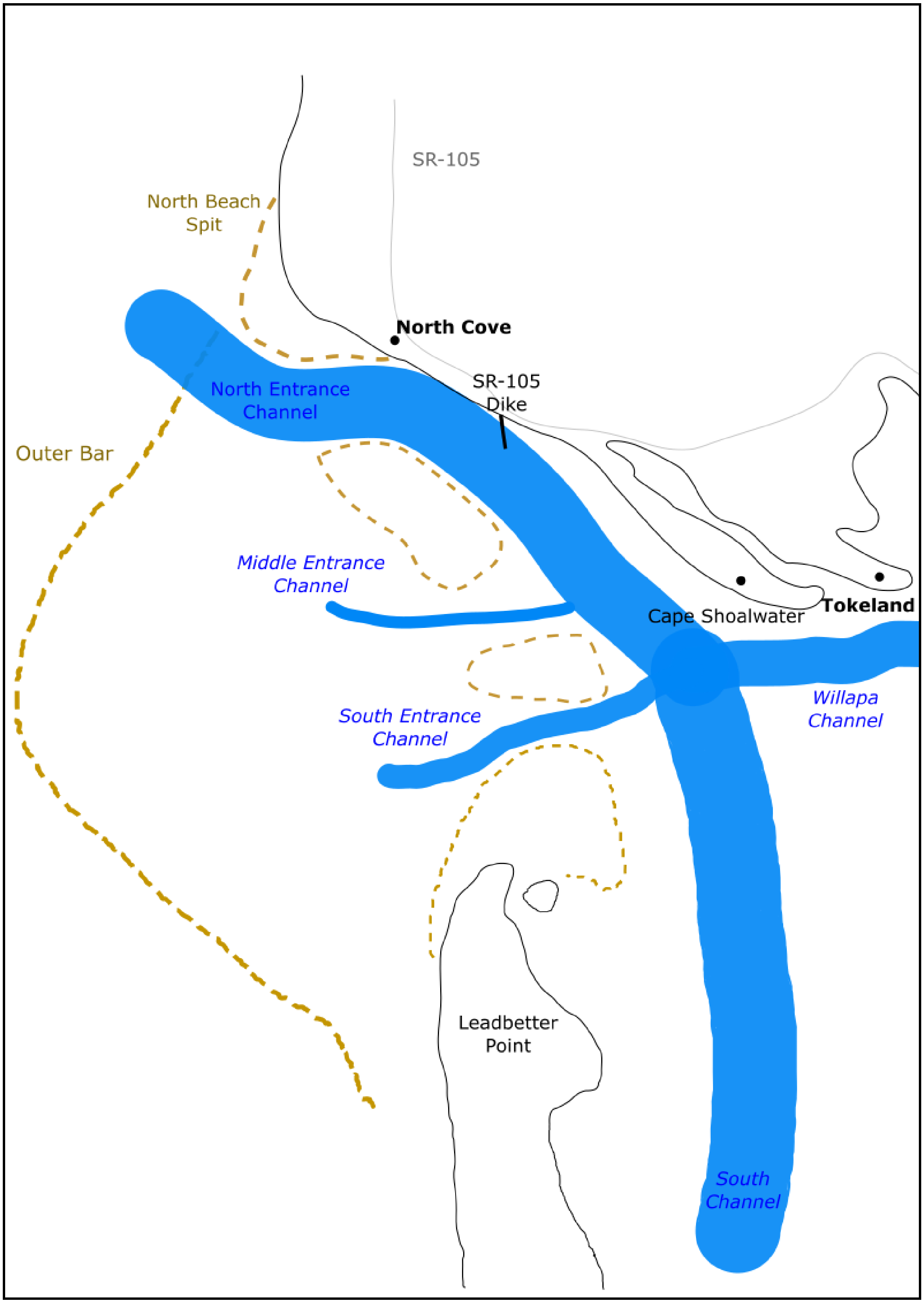

Figure 2: Willapa Bay Entrance features (c. 2017)

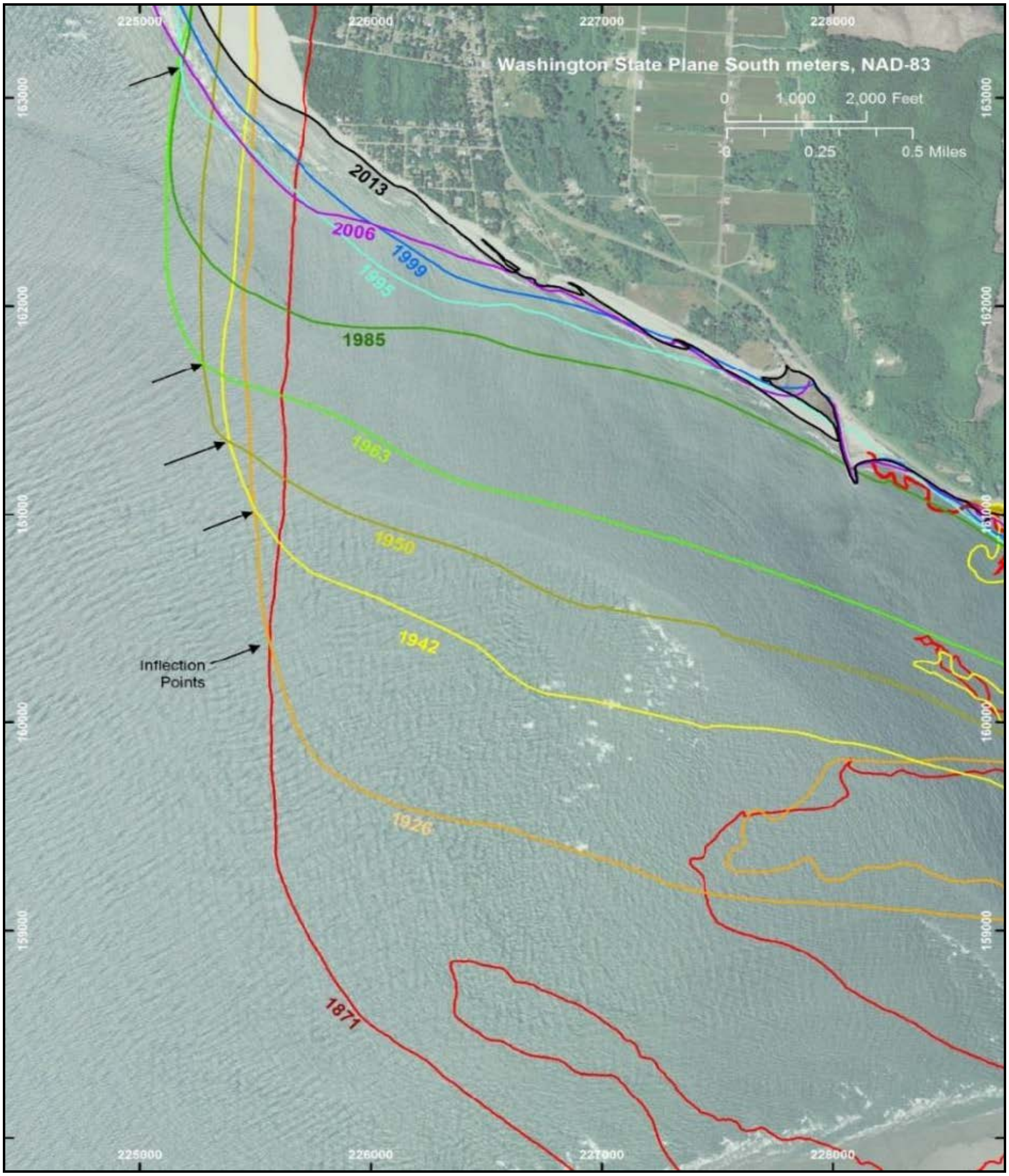

Figure 3: Cape Shoalwater historic shoreline positions from Talebi et al (2017).

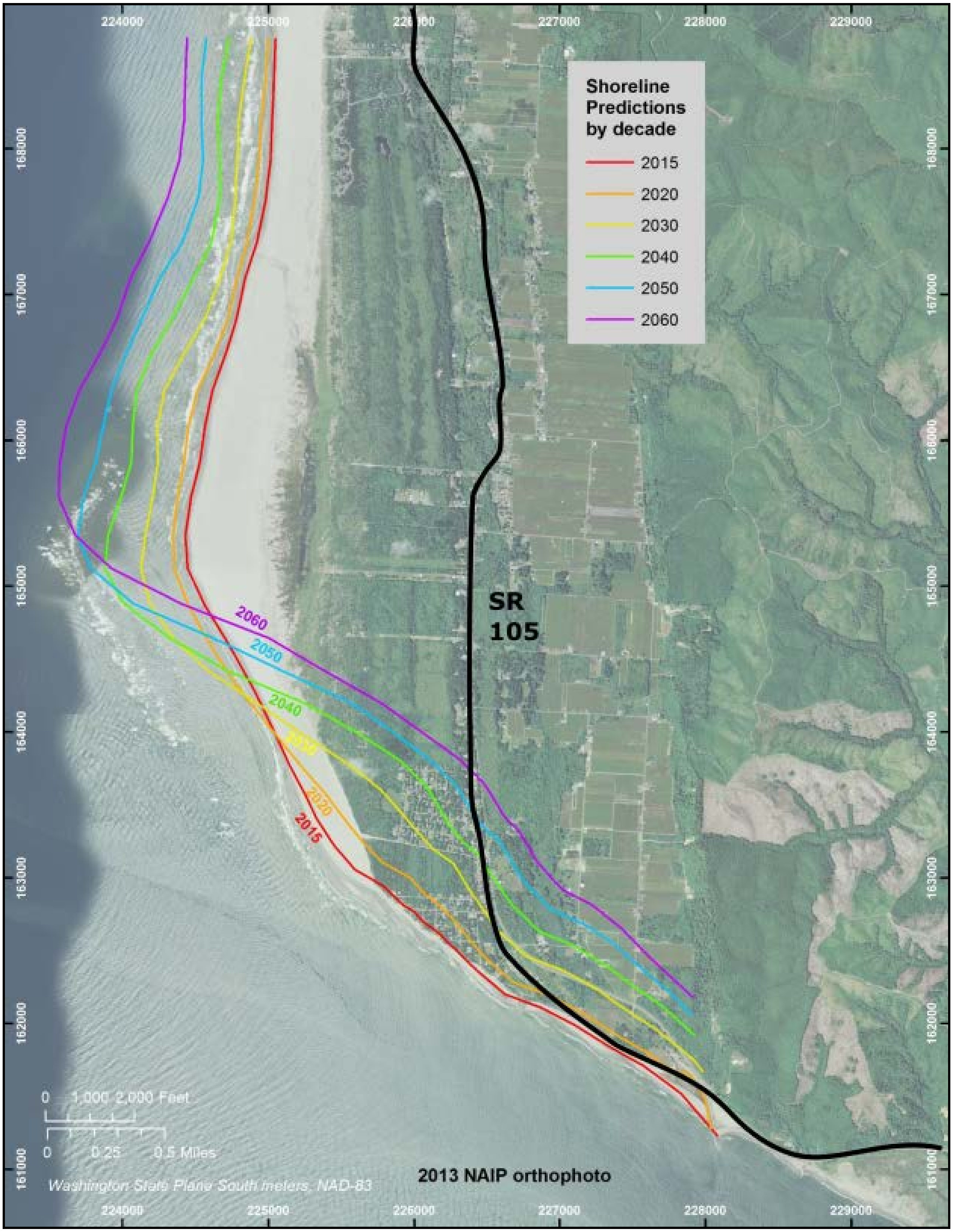

Figure 4: Cape Shoalwater future predicted shoreline positions from Talebi et al (2017).

5. Climatology

3.1 Tides and extreme water levels

Tides in Willapa Bay have the diurnal inequality typical of the U.S. West Coast (Mixed Semidiurnal). The tidal range, as measured by the National Ocean Service (NOS) of the National Oceanic and Atmospheric Administration, is approximately 6.8 feet. Hourly water levels have been collected by NOS at the Toke Point boat basin in Tokeland, WA since 1972. Observed water levels are primarily a function of astronomical tide influences. However, anomalies from the predicted astronomical tide occur due to factors including changes in atmospheric pressure, wind setup, wave setup, and river discharge (Figure 5). Published tidal data for Toke Point, WA are listed in Table 1. An extreme value distribution is fitted to the measured total water level (TWL) from extreme water level data measured between November 1972 to January 2018 using a Weibull distribution (Figure 6).

Table 1: Tidal data at Toke Point, WA - Willapa Bay, NOS Station 9440190

Datum	**Water Level (ft.)**	**Date**
Highest Observed Water Level	14.41	11/14/1981
Highest Astronomical Tide (HAT)	11.44	12/12/1985
Mean Higher-High Water (MHHW)	8.92	
Mean High Water (MHW)	8.18	
Mean Tide Level (MTL)	4.78	
Mean Low Water (MLW)	1.37	
North American Vertical Datum (NAVD)	0.82	
Mean Lower Low Water (MLLW)	0.00	
Lowest Astronomical Tide (LAT)	-2.99	6/23/1986
Lowest Observed Water Level	-3.81	12/19/1983

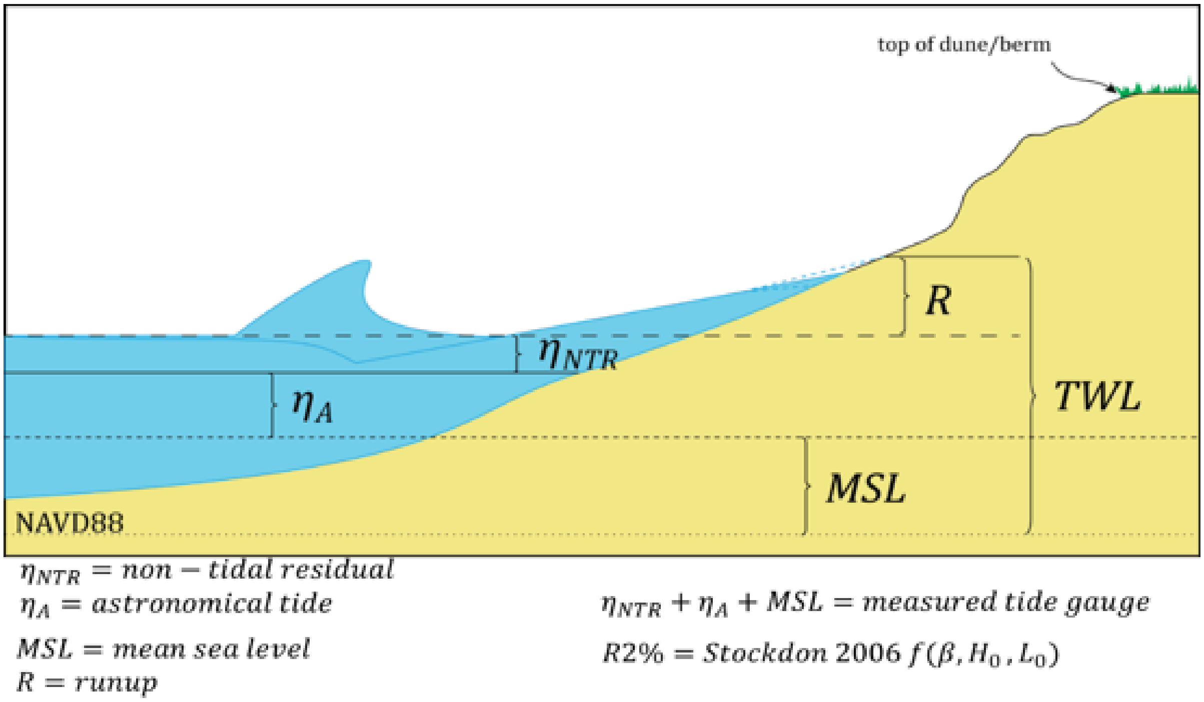

Figure 5: Components of Total Water Level (from Serafin and Ruggiero 2014)

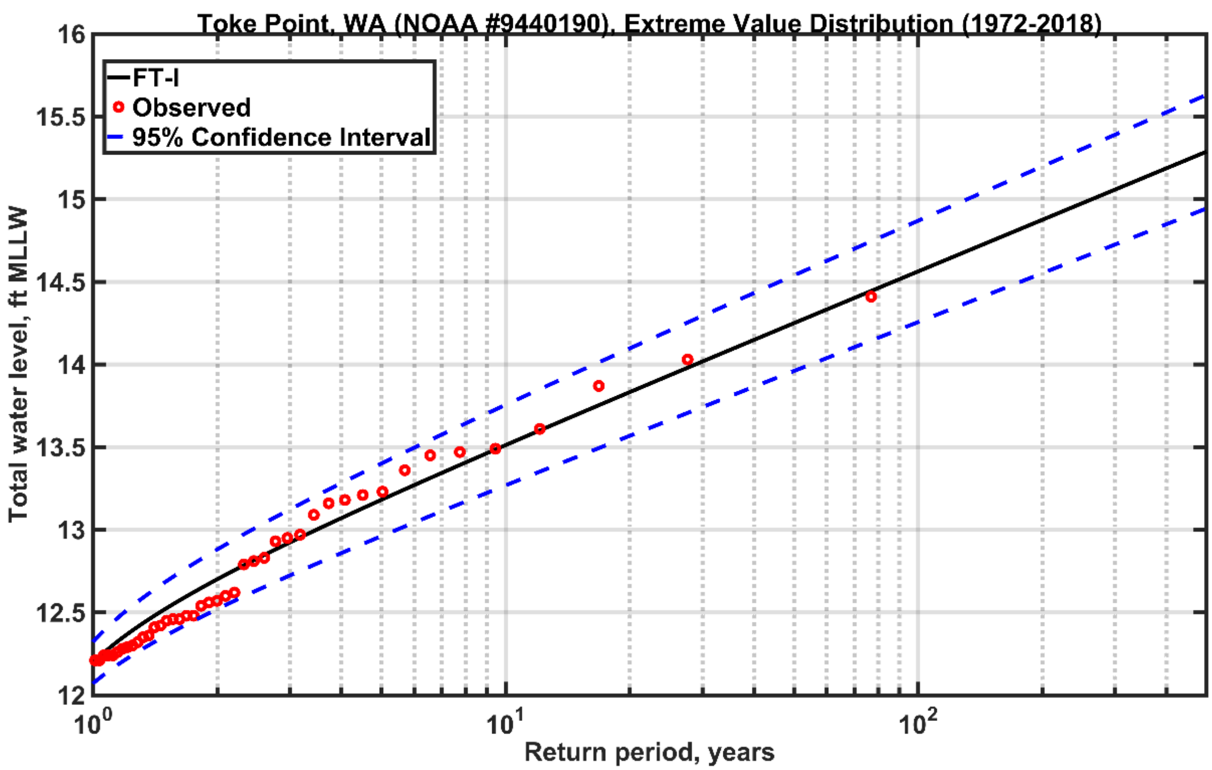

Figure 6: Extreme distribution of Total Water Level at Toke Point Tide Station (NOS 9440910), 1% annual exceedance (100 year Return period) is 14.6 feet MLLW.

3.2 Winds

The seasonal cycle of winds over the northeast Pacific Ocean is largely determined by the circulation about the North Pacific high pressure area and the Aleutian low pressure area which drives the jet stream over the North Pacific. During the summer months, the high reaches its greatest development. In July the center of highest pressure is located near latitude 35° N., longitude 150° W. During this period, the Aleutian low is almost nonexistent. This pressure distribution causes predominantly northwest and north winds over the coastal and near offshore areas of Oregon and Washington. The high weakens with the approach of the winter season and by November is usually little more than a weak belt of high pressure lying between the Aleutian low and the equatorial belt of low pressure. These traveling depressions moving eastward cause considerable day-to-day variation in pressure, particularly in the area north of latitude 40° N. At Tokeland, WA the prevailing wind direction is out of the northwest (Figure 7). The strongest winds originate from the southerly directions and have recorded 2-minute average wind speeds exceeding 50 knots.

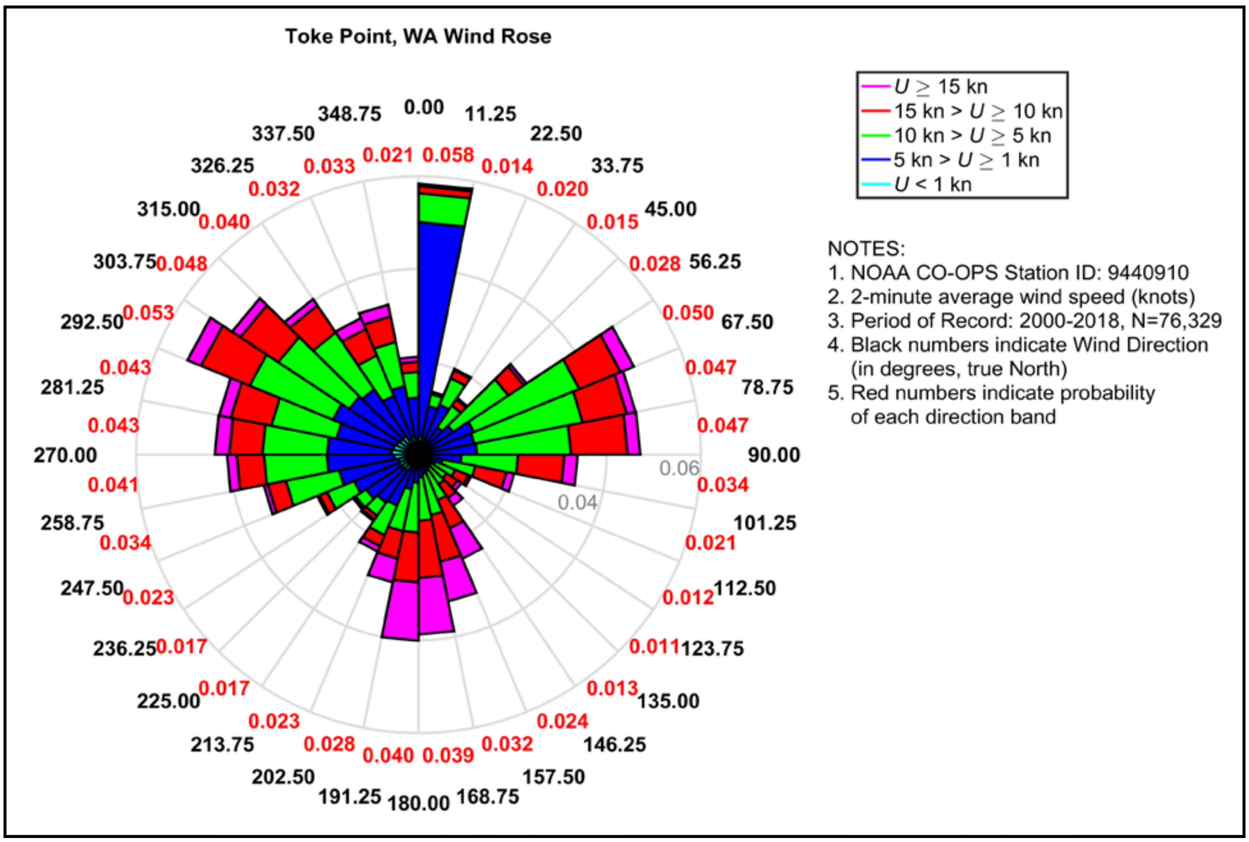

Figure 6: Wind Rose for Toke Point, WA based on 2-minute average wind speed from 2000 to 2018

3.3 Waves

The Coastal Data Information Program (CDIP) buoy 036 offshore of Grays Harbor has collected wave data since 1981. The average annual significant wave height measured at the buoy is 7 feet (2.1 meters) with a period of 11 seconds. The predominant wave direction varies seasonally (Figure 8). The prevailing waves in the milder summer months are from the northwest. While large storms generated in the winter months have a southwesterly directionality. Weather fronts associated with maritime cyclonic storms in the

Northeast Pacific can extend over the ocean for 1,000 miles and cover a latitude difference of 25 degrees. When these maritime low-pressure systems make land fall on the coast they produce hurricane-like conditions. Sustained wind speeds can be greater than 40 knots for fetches greater than 125 miles. The resulting wind stress can produce ocean waves greater than 30 feet high and a "setup" of the mean water level of 1 to 5 feet, depending on storm evolution. An extreme wave height distribution is fitted using the peak over thresholds method (Goda 2000). The 100-year recurrence interval (or 1% annual exceedance probability) of significant wave height is 40.2 feet (Figure 9).

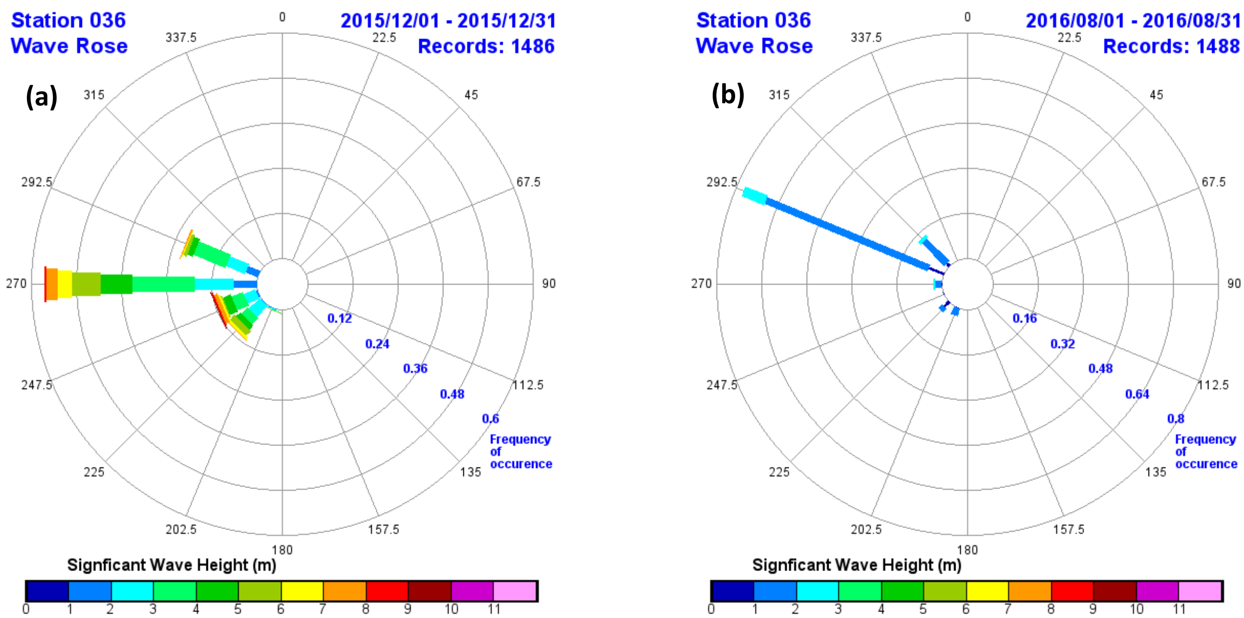

Figure 8: Wave rose for (a) December 2015 and (b) August 2016

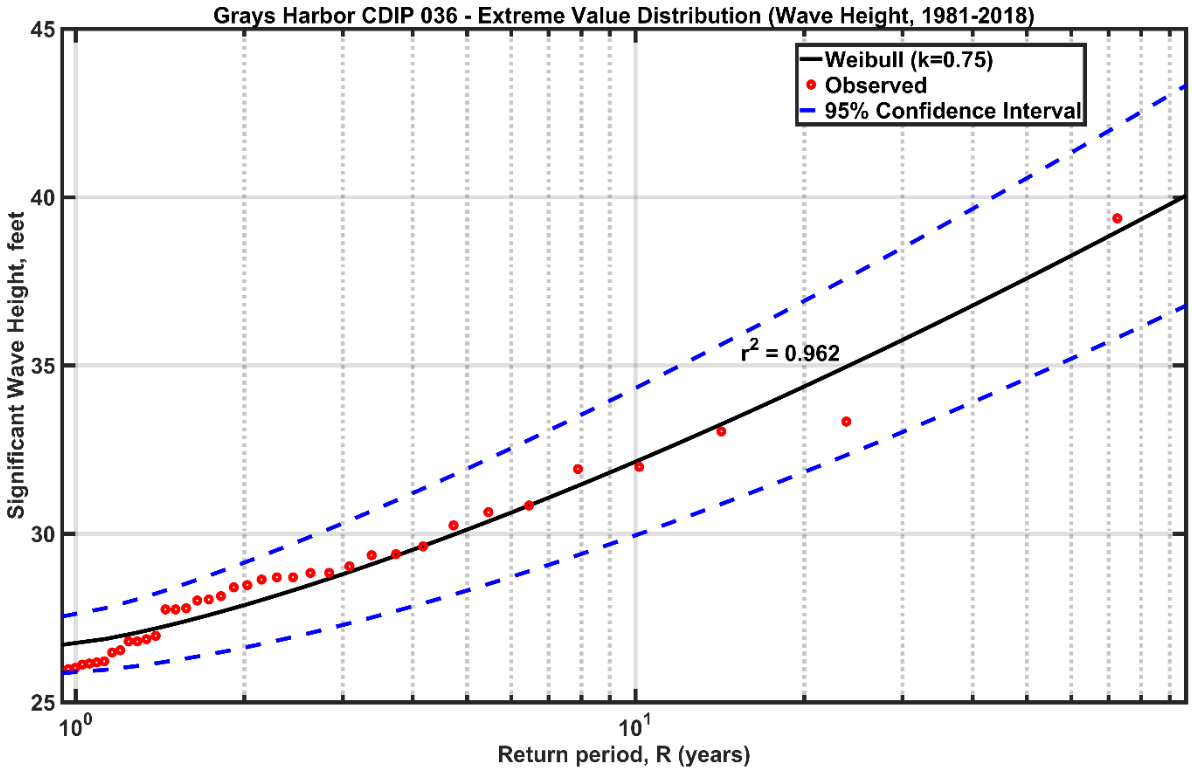

Figure 9: Nearshore extreme value distribution for significant wave height, 1% annual exceedance (100 year Return period) is 40.2 feet MLLW.

3.4 Currents

Tidal currents dominate the circulation in Willapa Bay. In general, the strongest currents follow the channel thalweg[1]. Figure 10 and 11 display velocity magnitude (color contours) and direction (vectors) during peak ebb and peak flood conditions. Willapa Bay has a large asymmetry created by the Long Beach Peninsula. Much of the tidal prism is located in the southern portion of the bay. As a result, tidal flows have a north-south orientation near the North Entrance Channel. This orientation forces much of the flow directly at Cape Shoalwater near the city of North Cove, WA. During peak ebb tides, the current velocities exceed 2 meters per second (m/s), or 4 knots (Figure 10). Currents maintain a similar north-south orientation during peak flood tides, however the magnitude of the currents are not as strong, approximately 1.5 m/s, or 3 knots (Figure 11). Local reduction in velocities downstream and upstream of the dike and acceleration of current velocities over the SR-105 dike are shown in Figure 10(b) and 11(b) respectively.

[1] Thalweg. The line defining the lowest points along the length of a river bed or valley

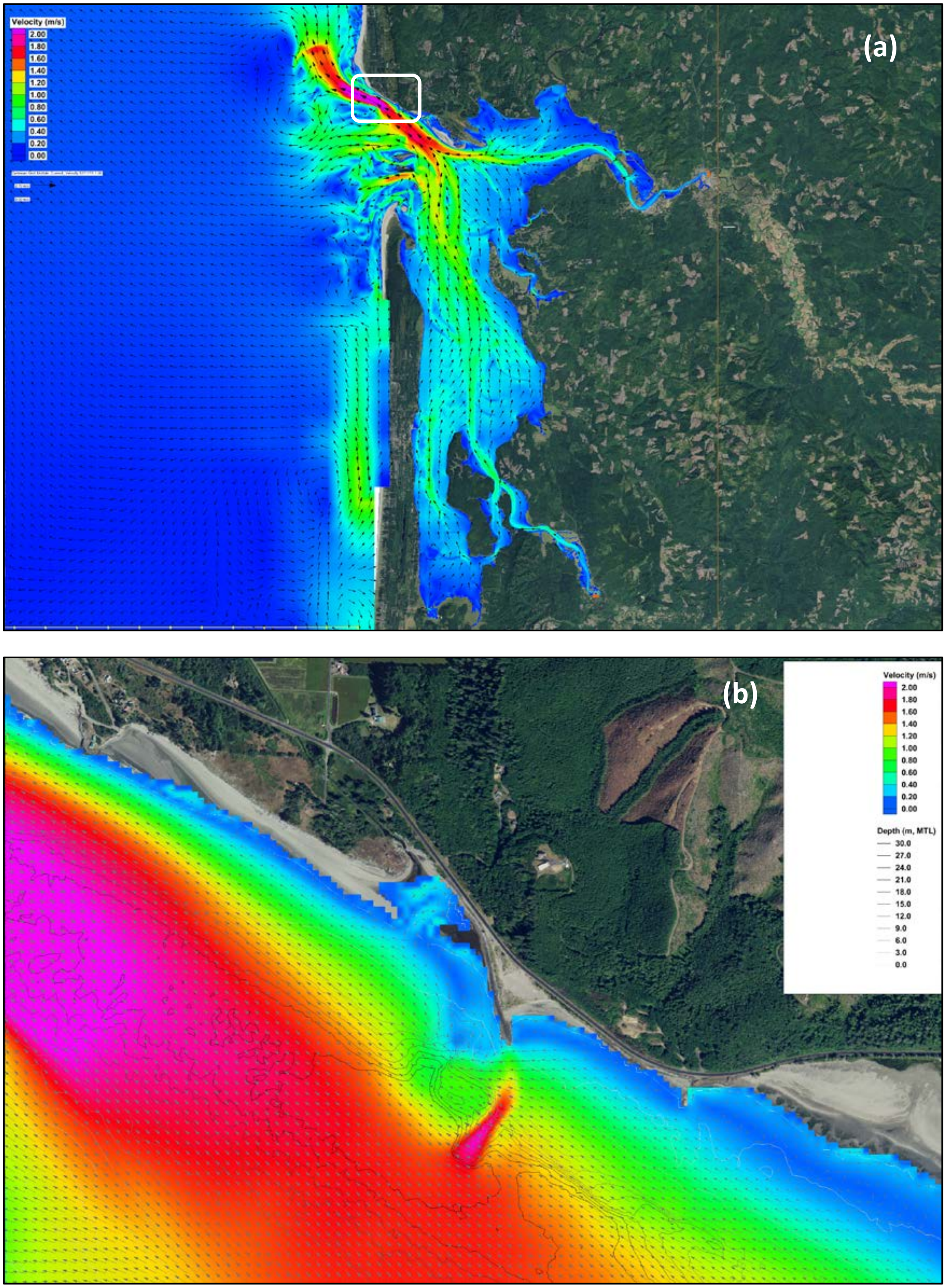

Figure 10: Modeled Currents during <u>*ebb*</u> *tide (a) Willapa Bay (b) near the SR-105 groin/dike structure*

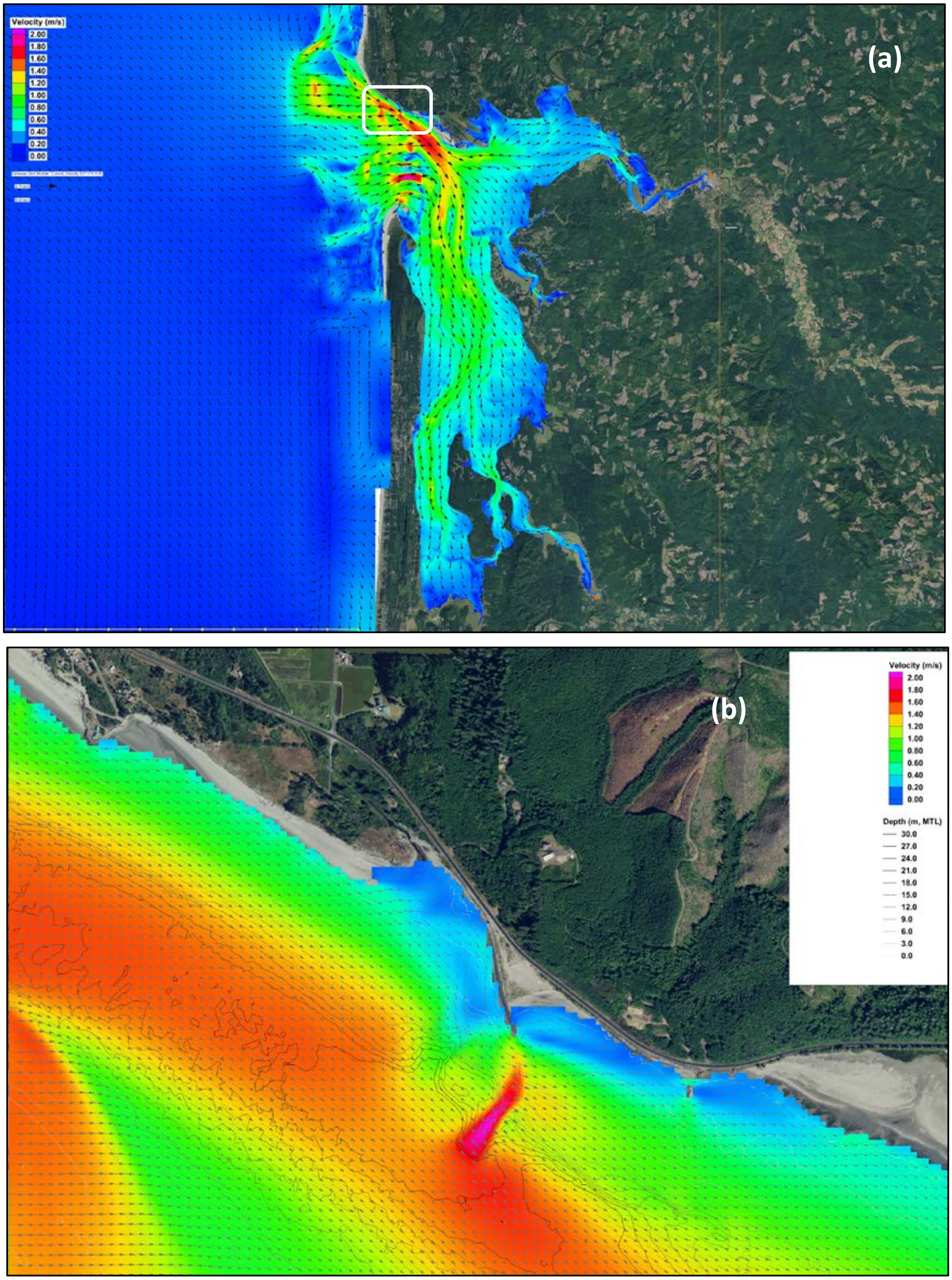

Figure 11: Modeled Currents during <u>flood</u> tide (a) Willapa Bay (b) near the SR-105 groin/dike structure

3.5 Sea level change

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 new information, including projections by the Intergovernmental Panel on Climate Change and National Research Council. Since predictions of future SLC have uncertainty, the risks associated with three SLC scenarios are analyzed. These scenarios are termed low, intermediate, and high and correspond to different rates of global sea level acceleration. Historically, this global (eustatic) sea level rise rate has been approximately 1.7 millimeters (mm) per year.

Locally, SLC varies geographically as it is the difference between the global SLC (1.7 mm/year according to IPCC 2007) and local vertical land movement (VLM). The accuracy of local mean sea level rates is a function of the period of record of the water level time series. ER 1100-2-8162 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 historic sea level change observed Toke Point since 1972 is approximately 0.9 mm/year (Figure 12). In 50 years, the predicted sea level rise at the project ranges from 0.4 to 2.5 feet (Figure 13).

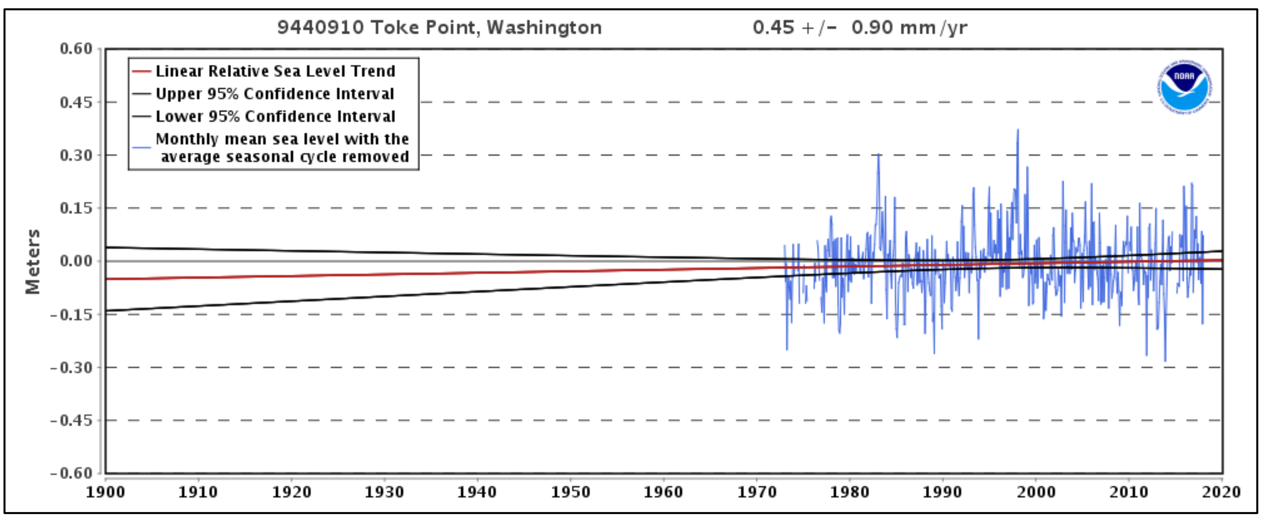

Figure 12: Historic Mean Sea Level Change at Toke Point Tide Station (NOS 9440910)

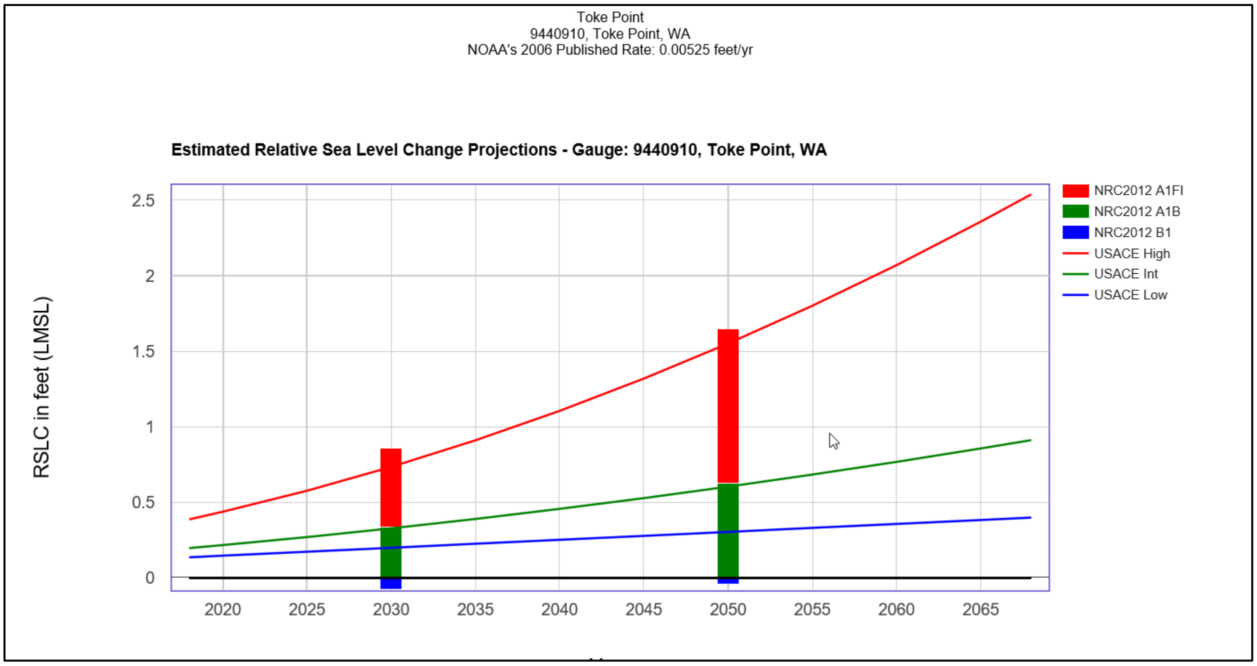

Figure 13: Predicted Mean Sea Level Change at Toke Point Tide Station (NOS 9440910) from USACE (2013) and NRC (2012).

3.6 Recent storm events

The Winter of 2015/2016 was the strongest El Niño year in the El Nino-Southern Oscillation (ENSO) cycle since 1997/1998, and has generally caused greater coastal erosion than observed during the 1997/1998 El Niño (Barnard et al. 2017). Events in December 2015, March 2016, and October 2016 significantly eroded the shoreline of North Cove and Cape Shoalwater resulting in overwash and deposition of sand in the tidal embayment and SR-105.

The December 10, 2015 storm was a strong low pressure event which developed in the North Pacific Ocean (Figure 14). The peak water level measured at the Toke Point, NOS 9440910 tidal gage was +13.2 feet MLLW, or the 11th highest water level on record since 1970, or a 5-year event. The measured storm surge for this event was 3.4 feet. The storm was also coincident with monthly spring tide sequence.

The March 10, 2016 storm was generated by a low pressure event which developed in the North Pacific Ocean. The storm brought measured winds greater than 70 mph on the coast and observed barometric low pressure 976 mb approximately 300 nautical miles west of Aberdeen (NDBC 46005). The maximum hourly significant wave height observed at Cape Elizabeth NDBC buoy 46041 was 27.0 feet. The peak water level measured at the Toke Point, NOS 9440910 tidal gage was +13.5 feet MLLW, or the 7th highest water level on record since 1970, or a 10-year event. The measured storm surge for this event was 4 feet. The storm was also coincident with monthly spring tide sequence.

The October 15, 2016 storm was generated from remnants of Typhoon Songda which at its peak produced 1-minute sustained hurricane force winds over 150 miles per hour (mph) and barometric low pressure of 925 millibars (mb). The typhoon weakened as it traveled across the Pacific and then began to rapidly deepen into an extratropical cyclone offshore of the Pacific Northwest. The storm tracked approximately 60 miles offshore the Olympic Peninsula and produced low barometric pressure of 968 mb and peak winds of 78 mph at Tatoosh Island CMAN Station TTIW1. The maximum hourly significant wave height observed at Cape Elizabeth NDBC buoy 46041 was 28.3 feet. The peak water level measured at the Toke Point, NOS 9440910 tidal gage was +12.3 feet MLLW, or a 2-year event. The measured storm surge for this event was 3 feet. The storm was also coincident with the monthly spring tide sequence. While this storm did not produce the highest total water levels on record, its storm surge hydrograph extended over a long duration. The storm surge exceeded 0.4 meters (1.3 feet) for 43 hours, or approximately 4 high tide cycles (Figure 15). Storm surges of this duration allow for more erosion to the foredune and dune as they allow direct wave attack for longer periods of time.

Based on the recorded water levels and wave heights presented above, the December 10 2015, March 10, 2016, and October 15, 2016 extratropical cyclones were extraordinary based on their measured offshore wave height, direction, storm surge, and coincidence with large spring tide events. The probability of a 2-yr event in a year is 0.5 (50% annual exceedance). The probability of a 5-yr event in a year is 0.2 (20% annual exceedance). The probability of having a 10-year event in a year is 0.1 (10% annual exceedance). The probability of having a 2-year, 5-year, 10-year event in a given year is (0.5)*(0.2)*(0.1) = 0.01 (100-yr event or 1% annual exceedance).

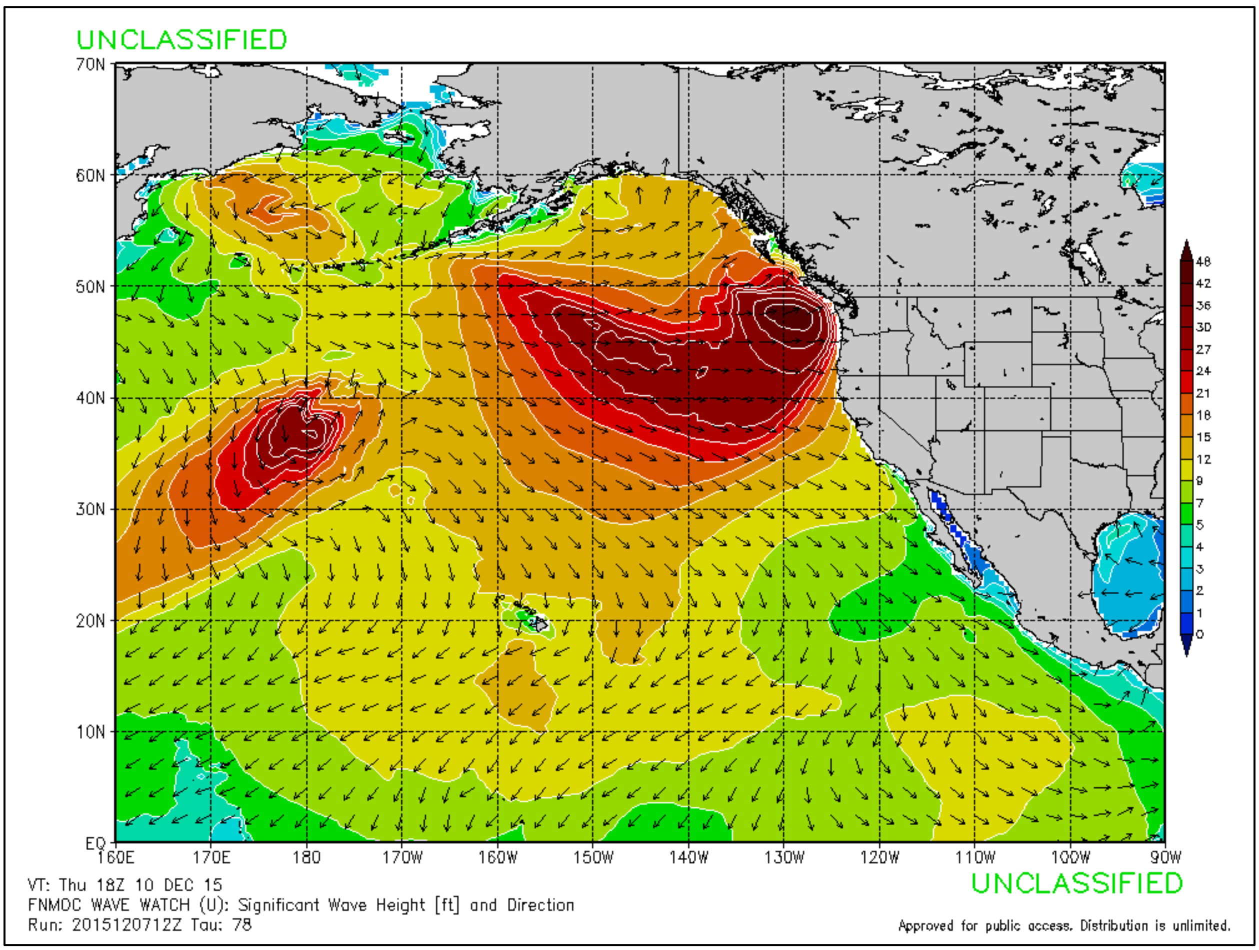

Figure 14. 10 December 2015 WaveWatch III regional forecast

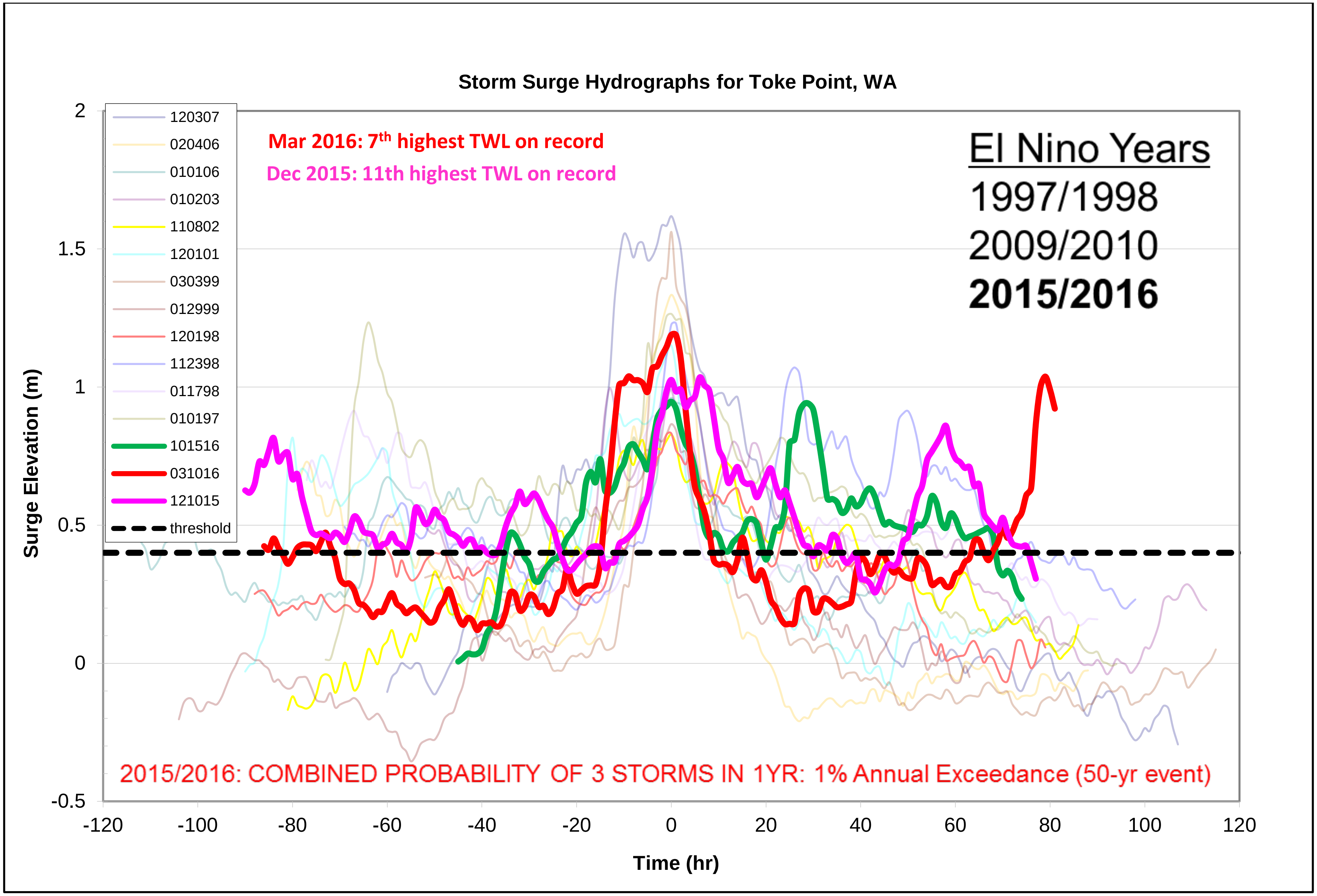

Figure 15: Top 15 non-tidal residual (storm surge) hydrographs at NOS 9440910 since 1996; 2015/2016 El Nino shown in bold

6. Shoreface Geology[2]

7.1 Regional Littoral Cell

Willapa Bay and the greater Columbia River littoral cell have been the subject of extensive research over the last 20+ years. The northern shoreline of Willapa Bay lies within the Grayland Plains Sub-Cell of the Columbia River littoral cell (Figure 17) and, over the Holocene, has largely functioned as a small sediment sink (e.g. Gelfenbaum et al., 1999; Gelfenbaum and Kaminsky, 2010). Regional shorelines have experienced severe shoreline retreat caused by sudden 1-2 m drops in land elevation along the coast associated with large subduction-zone earthquakes that occur about every 500 years (e.g. Gelfenbaum and Kaminsky, 1999; Peterson and Reinhardt 1995). More recently, the sediment supply delivered to the littoral cell from the Columbia River has been reduced due to: (1) the construction of dams within the river itself; (2) dredging of sediment from the channel and subsequent placement offshore, out of the littoral cell; and (3) the construction of jetties which altered sediment exchange across ebb-tidal deltas and adjacent coastlines (e.g. Sherwood et al., 1990; Gelfenbaum et al., 1999; Gelfenbaum and Kaminsky, 2010; Kaminsky et al., 2010). Other potential impacts on shoreline stability include changes in storm tracks and frequency during El Niño events (e.g. Kaminsky et al., 1998; Ruggierio et al., 2005), as well as by variable tectonic activity.

7.2 Geological Framework

Pleistocene Sequence. The Quaternary sediments of the Cape Shoalwater region overlies an extensive Pleistocene sequence comprised of partially indurated fluvial, estuarine, intertidal, and subtidal deposits. These more erosion-resistant deposits form a distinctive topographic terrace along the Cape Shoalwater shoreline. In 1983, Clifton provided a detailed comparison between modern coastal environments and the Pleistocene deposits in order to distinguish different depositional environments, and thus units, in the Pleistocene formation. Briefly, Clifton (1983) identified 5 major units in the Pleistocene formation, and mapped their relative position near Ramsey Point, Willapa Bay (Figure 18). The oldest sequence (~190 kya), Unit I, is a dominantly intertidal sequence of laminated, bluish fine sand and mud, overlain by a thick, bioturbated bluish mud. An erosional contact separates Unit I from the overlying, subtidal Unit II, composed primarily of muddy and/or cross-bedded sand. Throughout Unit I and II, small runoff channels were noted. These small features (e.g. 10’s of cm to meters in extent) lack classic tidal or fluvial stratigraphy (e.g. interbedded sands, muds, and/or gravels), and Clifton (1983) postulated that these were potentially formed by ephemeral tidal flows or upland runoff (see Figure 19 (A) and (B) for an example). Unit III is comprised of well defined, though spatially variable, extensive channel deposits composed primarily of mud with abundant wood. These channels cut into both Units I and II, and are also noted between the laminated sediments and the bioturbated mud within Unit I. It should be noted that in these three Units, gravel deposits are thin and discontinuous, if present at all. Overlying Unit III are the younger (~100 kya) sediments of Unit IV, a laterally continuous, shallowing-upward, stillstand substrate of large-scale sand and mud strata, plus additional layers of cross-bedded gravels, sands, and mud - exemplified in the exposure shown in Figure 19 (C)-(E). Unit IV is distinctive in that it preserves spatially variable, thick lags of disarticulated shells, pebbles, wood fragments, and abundant shells including *Ostrea lurida* in growth position. Finally, Unit V is comprised predominantly of

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[2] Excerpts from Wadman et al (2017).

estuarine-to-fluvial channel fill, dominated by laminated mud and silty-fine sand with little or no bioturbation, which cuts through all the other units. Unit V's channel axes, where exposed at Ramsey's Point, often contain abundant logs and large wood fragments oriented parallel to the channel axis, with thick, discontinuous sequences of pebbles and/or coarse sand.

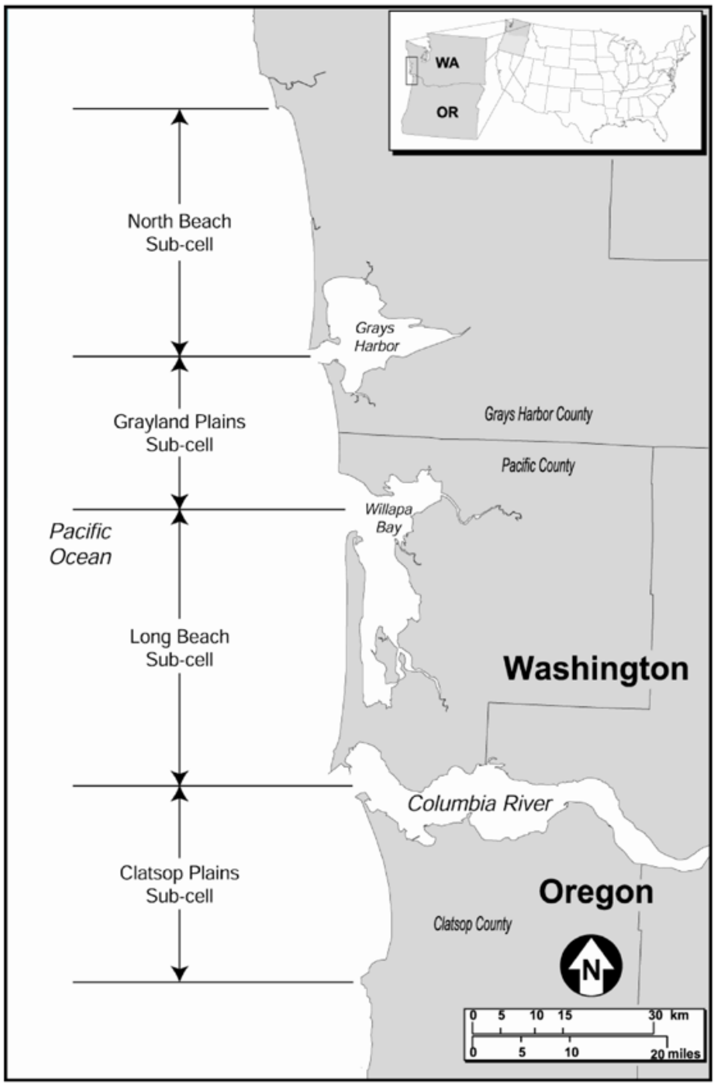


Figure 17. Columbia River Littoral Cell (from USGS 2007)

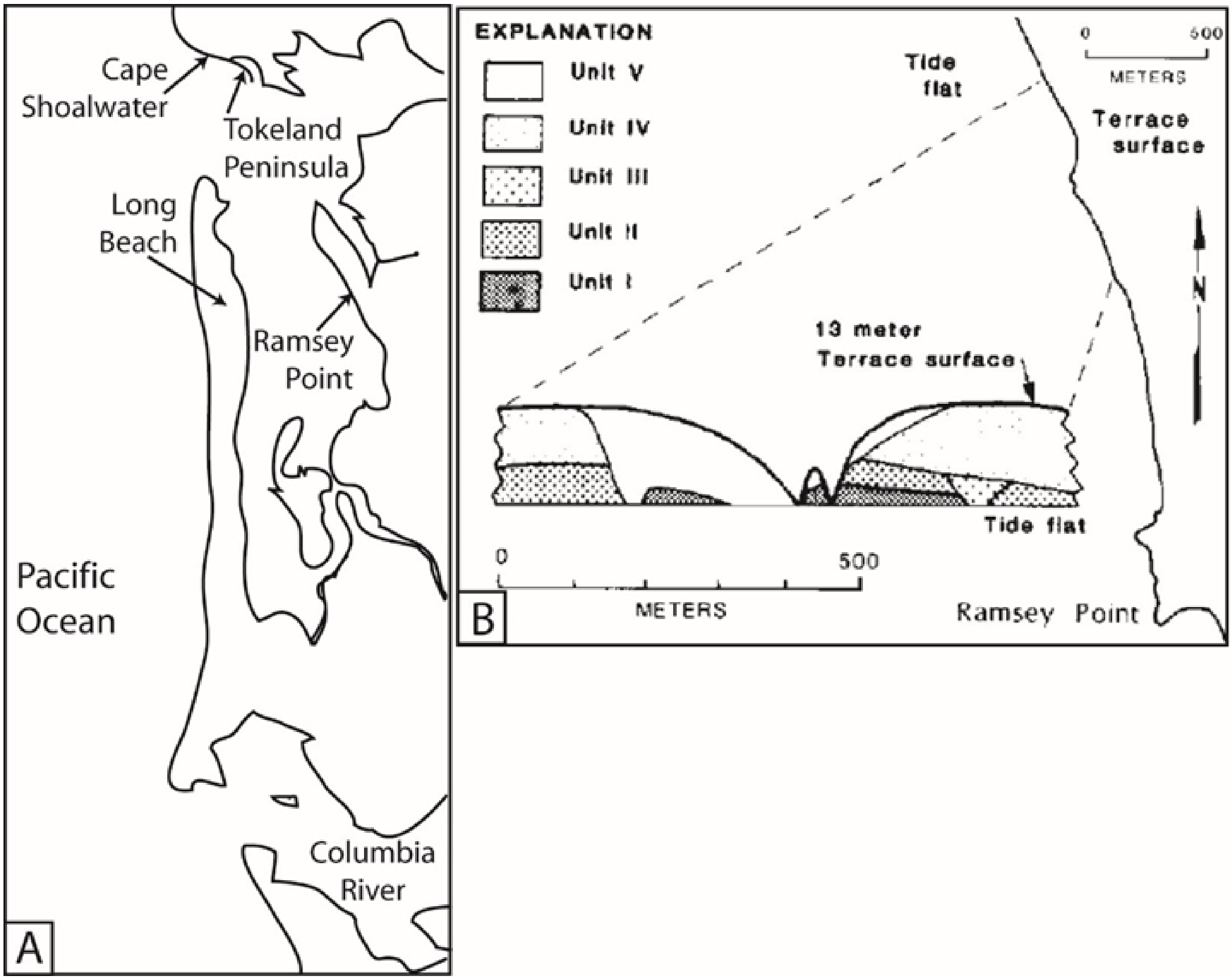


Figure 18. Stratigraphy of Pleistocene Units as exposed along Ramsey Point, Willapa Bay (modified from Clifton 1983)

Local Well Logs. Discerning the extent of the Pleistocene sequence potentially exposed in the shoreface along Cape Shoalwater is challenging without an extensive mapping effort. However, a general sense of the sequence's extent can be derived via existing core logs from numerous wells drilled for both municipal and domestic use near Cape Shoalwater since the 1970's. The Shoalwater Indian Tribe sought to have the water quality of the primary regional aquifer quantified in the early 2000's. As part of this effort, borehole logs of 21 water wells were compiled by Lane and Ebbert (2001), along with extensive water quality testing. An additional 2 water well logs were further compiled by Goldberg (2005) during a more expanded study of the aquifer. All of these well logs included lat/lon location data of the well locations. It should be noted that some of these wells were drilled for the aquifer studies; others predated the effort, occasionally by decades. The locations of these wells are plotted on Figure 20. In addition to these wells, borehole logs for 17 wells not included in either the Lane and Ebbert (2001) or Goldberg (2005) studies were provided by the Shoalwater Tribe (Steven Spencer, *pers. comm.*). Identifying the exact locations of these wells (named as WWR_# in Figure 20) proved challenging as the logs noted, at most, a township, range, and section. Some of the wells had a quadrant identified, or a description of the property, others did not. Thus, the locations as plotted in Figure 20 might be in error.

Despite the location uncertainty, the well logs still provide an insight into the general stratigraphy of the region, and they are included in this interpretation.

Unfortunately, none of the well logs noted the surface elevation of the well relative to a common datum, limiting their use in constructing vertically-referenced fence diagrams of stratigraphy. Despite this limitation, the data still yield insight into the relative distribution of the potentially erosion-resistant, partially-indurated Pleistocene sequence in this region. To aid interpretation, the upper Units IV and V of the Pleistocene sequence have been grouped in this report as the “Upper Member”. Overall, this includes indurated, massively bedded sands and sandy clays, cross-bedded gravels, sands, and mud, and extensive lag deposits (up to 3-5 feet thick) of pebbles to gravel, with abundant shell and wood, plus laminated channel deposits. Well logs that contained this geology are noted by red circles in Figure 20. Units I, II, and III have been grouped into the “Lower Member”, and include white- to blue-gray laminated sands and mud, blueish bioturbated mud, and minor muddy channel sequences. Lag deposits are minor to absent in this sequence, and wells with this sequence are plotted as blue circles on Figure 20. If a well log indicated both Upper and Lower members, it was plotted as a red circle on Figure 20. In addition, it should also be noted that, of the 40 wells mapped in this effort, at least 5 wells were previously drilled in regions of the Cape that have subsequently been lost to shoreline erosion. Those logs are still relevant, however, as they provide insight into the type of sediment potentially exposed in regions of the shoreface.

From Figure 20, it is clear that the most of the wells near the terrace and along the coastal edge of Cape Shoalwater contain at least some of the Upper Member of Pleistocene units. However, exceptions do occur: (1) well 02C01, which was drilled in a lake and thus might be located lower than the base of the Upper Sequence; and (2) wells WWR_1 & WWR_4, now located well offshore of the central shoreline at Cape Shoalwater, all of which appeared to have only Lower Member stratigraphy. Despite these exceptions, overall these data suggest that the indurated Upper Member is still present along some sections of the Cape Shoalwater shoreline, and potentially acting to slow coastal erosion, and thus shoreline retreat, in this region, as some researchers have suggested (e.g., Gelfenbaum and Kaminsky, 1999; Morton et al., 2007; USACE 2009). Wells with only the lower sequence preserved are located primarily in the lower Tokeland Peninsula, as well as in some Cape Shoalwater wells which have been lost to erosion since installation. This overall variability is supported both by the spatial variability in the Pleistocene sequences mapped by Clifton (1983), and anecdotally by the recollections of Ray Williams, one of the primary well drillers in this region during the mid- to late 1900’s. In particular, the distribution of the sequences suggests that the eastern end of Tokeland Peninsula might be comprised of less erosion-resistant sediment than the rest of the region, highlighting the importance of mapping these different geologic units in a comprehensive and quantitative manner in order to identify the most erosion-susceptible substrates near the shoreline.

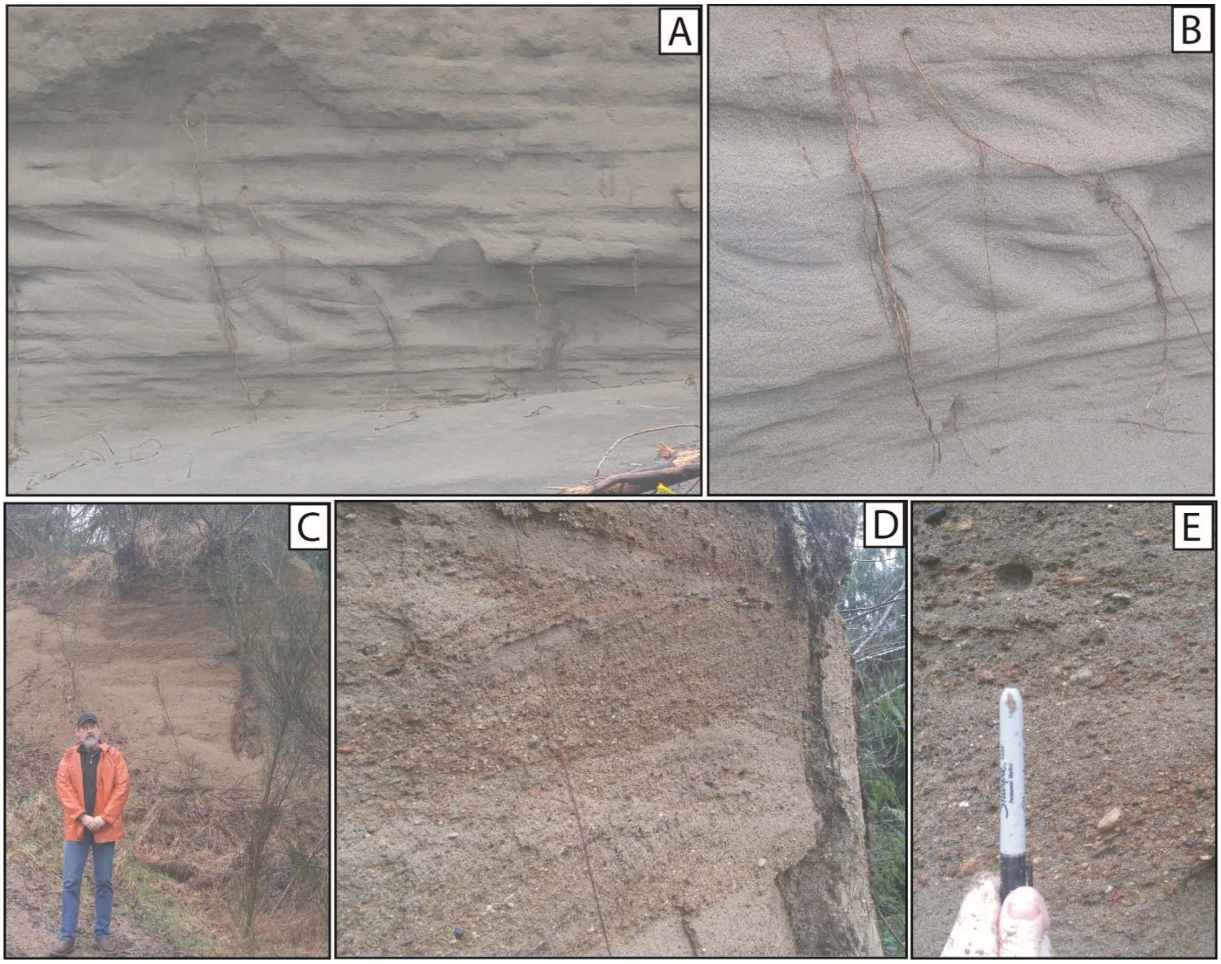


Figure 19. Geology of the Pleistocene sequence comprising the terraces inland of Cape Shoalwater. (A) Runoff channels exposed in Unit I or II along the shoreline of Cape Shoalwater. (B) Close-up of runoff channels. (C) Exposed upper Pleistocene sequence (likely Unit IV) in the terrace inland of Cape Shoalwater. (D) Example of cross-bedding preserved in Unit IV. (E) Close-up of part of a pebble-gravel lag preserved in Unit IV.

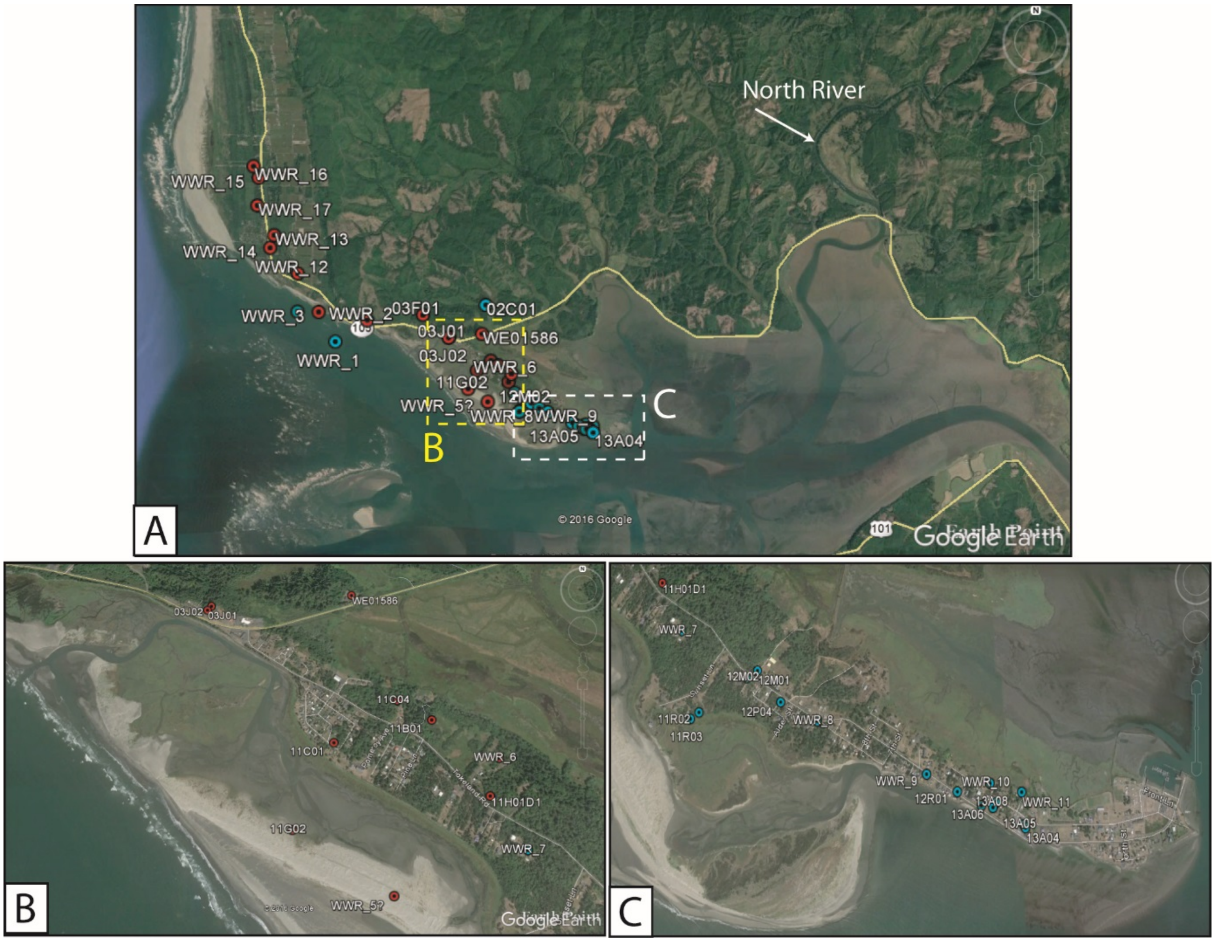


Figure 20. Location of wells used in this study. WW_# indicates an unpublished well at the time this report was written. Red wells indicate the presence of the Upper Member of the Pleistocene sequence; blue wells indicate the presences of only the Lower Member of the sequence. Outcropping bedrock along the North River is noted by the white arrow. (A) All wells used in this study. (B) Close-up of wells from the upper region of the Tokeland Peninsula. (C) Close-up of wells from the lower region of the Tokeland Peninsula.

Basement Geology. Given the possibility that the Pleistocene sequences have been partially or completely eroded in some regions, it is worth examining the potential that basement rock might be cropping out in the nearshore, impacting local erosion rates. The basement rock in this region is the Crescent Formation: a lower- to mid-Eocene sequence of basalt flows and basaltic breccias (Huntting et al., 1961; Beikman et al., 1967). This formation is overlain by the Eocene-Oligocene marine sedimentary rocks of the Lincoln Creek Formation, which are predominantly composed of tuffaceous siltstone and fine-grained sandstone (Beikman et al., 1967). It should be noted that the closest subaerial exposure of the Lincoln Creek Formation to Cape Shoalwater is upstream on the North River, approximately 8 miles west of Tokeland (WSDNR 2010). Above this formation an extensive erosional gap exists, as overlying units are either Tertiary sedimentary rocks, the Pleistocene sequence previously described, or Quaternary sediments (e.g. Beikman et al., 1967; WSDNR, 2010). The erosional and/or non-depositional hiatus between the lower rocks and the overlying Pleistocene or Quaternary sediments is likely due both to fluctuating sea level eroding sedimentary sequences, as well as regional tectonic uplift. The possibility that outcrops of basement rock are being exposed in the channel (Figure 21), thereby resulting in the apparent recent stabilization, cannot be discounted due to: (1) the lack of extensive geologic mapping of the shoreline of this region; (2), the overall variability in thickness and extent of the overlying Pleistocene sequence; and (3) the significant erosional hiatus between the Pleistocene sequence and the underlying basement rock (Clifton, 1983). If the migration of the channel has eroded the overlying Pleistocene sequence completely, the result would be that the position of the channel thalweg would be stabilized by exposed basement rock outcropping along the channel edge, thereby resulting in the observed reduction in migration rates, and possibly explaining the reduction in the rate of retreat of the adjacent shoreline.

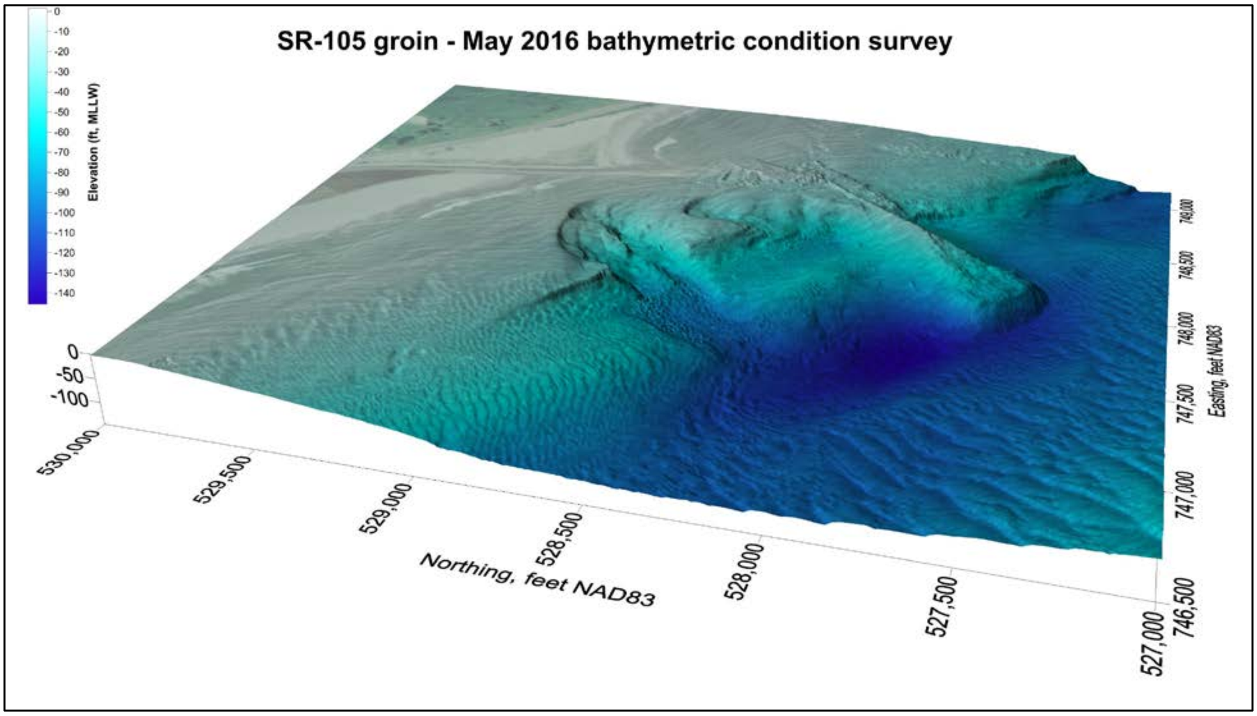


Figure 21. Deep trench adjacent to SR-105 groin/dike. Basement geology may be exposed in this area.

7. Recent Harbor morphology

7.1. Bar and Entrance

The USACE collects annual condition surveys of the Bar and Entrance Channels into WIllapa Bay. The volume of sediment moving through pre-defined polygons over a decadal period is calculated using a sediment budget analysis. The polygons are defined by areas with unique sediment transport patterns. The descriptions of each polygon are as follows:

- ***Bar North***: Includes the Outer Bar into Willapa Bay where the North Entrance channel daylights into the Pacific Ocean.
- ***Bar Middle***: Includes the outer bar into Willapa Bay where the Middle Entrance channel daylights into the Pacific Ocean.

- ***Middle Channel***: Includes the Middle Entrance channel and shallow water shoals between the outer bar and the Entrance Channel
- ***Groin:*** Includes the section of the North Entrance channel between the armored residential home (Tamarack Street) and the landward connection of Cape Shoalwater.
- ***Entrance***: Includes the Entrance Channel parallel to Cape Shoalwater.
- ***Shoals***: Includes the shallow region of shifting shoals on the western side of the Entrance Channel between the Middle Entrance Channel and the North Entrance Channel.
- ***East***: Includes the Willapa Channel thalweg from the distal end of Cape Shoalwater to the confluence of the North River.
- ***South***: Includes the portion of South Bay (Nahcotta Channel) from the South Entrance Channel to Bay Center (Goose Point).

The annual surveys from 2006 and 2016 were used in the analysis. The surveys consist of 1000-foot spaced transects perpendicular to the channel (i.e. cross-channel). These surveys had good overlapping coverage and represented a long enough time period to capture long-term morphology and sediment transport trends over multiple summer and winter cycles, including the El Nino years of 2009/2010 and 2015/2016. The surveys were conducted by the USACE *Shoalhunter* survey vessel using a Ross 835B Singlebeam 3.5 degree, 200 kHz transducer. Real Time Kinematic (RTK) correction was utilized for positioning and vertical elevation corrections.

Golden Software Surfer version 11, software package was used to produce a digital elevation model (DEM) for each data set. The DEM was developed using Delaunay triangulation. A 10-foot x 10-foot gridded DEM was created for each survey. Long triangles were deleted from each DEM to avoid erroneous interpolation errors.

The volume of sand eroded and deposited with each sediment budget polygon is computed by subtracting the 2016 and 2006 DEMs (Table 2). The sum of the erosion and deposition within the entire polygon is the net volume. The *Bar North* and *Bar Middle* polygons experience the most significant net erosion at 28.8 million cubic yards (MCY) and 40.6 MCY, respectively over the 11 year period. The *Middle Channel* and *South* polygons are regions of net sediment deposition, with 8.0 MCY and 12.3 MCY of deposition respectively. Although the *Entrance* has deepened considerably from 2006 to 2016 (over 50 feet in areas), there is a net deposition within this polygon as the shoals near the *Middle Channel* continue to migrate eastward toward the *Entrance.* This analysis confirms the qualitative analysis in Section 3 (USACE 1971; Terich and Levenseller 1986), which describes the process of onshore shoal migration toward the Entrance, resulting in a scouring of the North Entrance Channel and northward migration toward the North Cove shoreline. Major uncertainties with this analysis are the amount of material located in the *Shoals* polygon and the amount of beach progradation north of North Cove (i.e. North Beach Spit – Figure 1). Based on the trend observed in the *Middle Channel* polygon, it is speculated the *Shoals* polygon is also a net depositional area. Without including the *Shoals* polygon and the North Beach Spit progradation in the analysis, the sediment budget results in a net erosion of 52.1 MCY of material, or 4.7 MCY per year from the region.

Table 2. Sediment budget volumes (2006 to 2016)

Polygons	Deposition (CY)	Erosion (CY)	Net (CY)
Bar North	33,723,104	(62,544,130)	(28,821,026)
Bar Middle	33,737	(40,609,737)	(40,576,000)
Middle Channel	8,248,956	(186,956)	8,062,000
Shoals [3]	-	-	-
Groin	2,259,035	(6,860,428)	(4,601,393)
Entrance	34,347,064	(31,714,711)	2,632,353
South	52,646,675	(40,377,003)	12,269,672
East	2,256,307	(3,327,882)	(1,071,575)
North Beach Spit	-	-	-
SUM	**133,514,878**	**(185,620,848)**	**(52,105,970)**

[3] Shoals polygon is not captured in the annual condition surveys due to shallow water

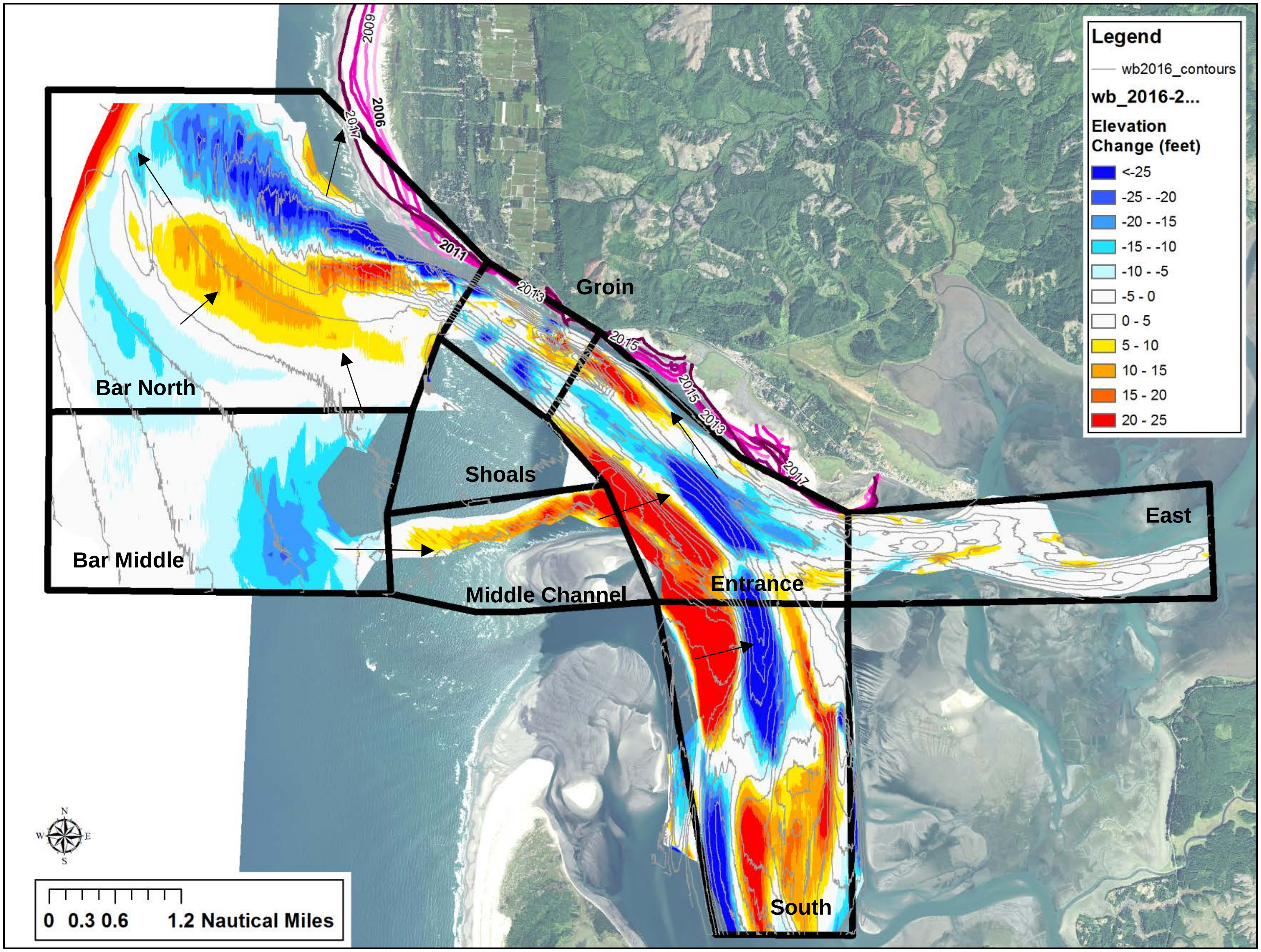


Figure 21. Bathymetric Elevation Change at the mouth of Willapa Bay from 2006 to 2016. (Hot holors denote acretion and cool colors erosion); arrows indicate direction of sediment transport

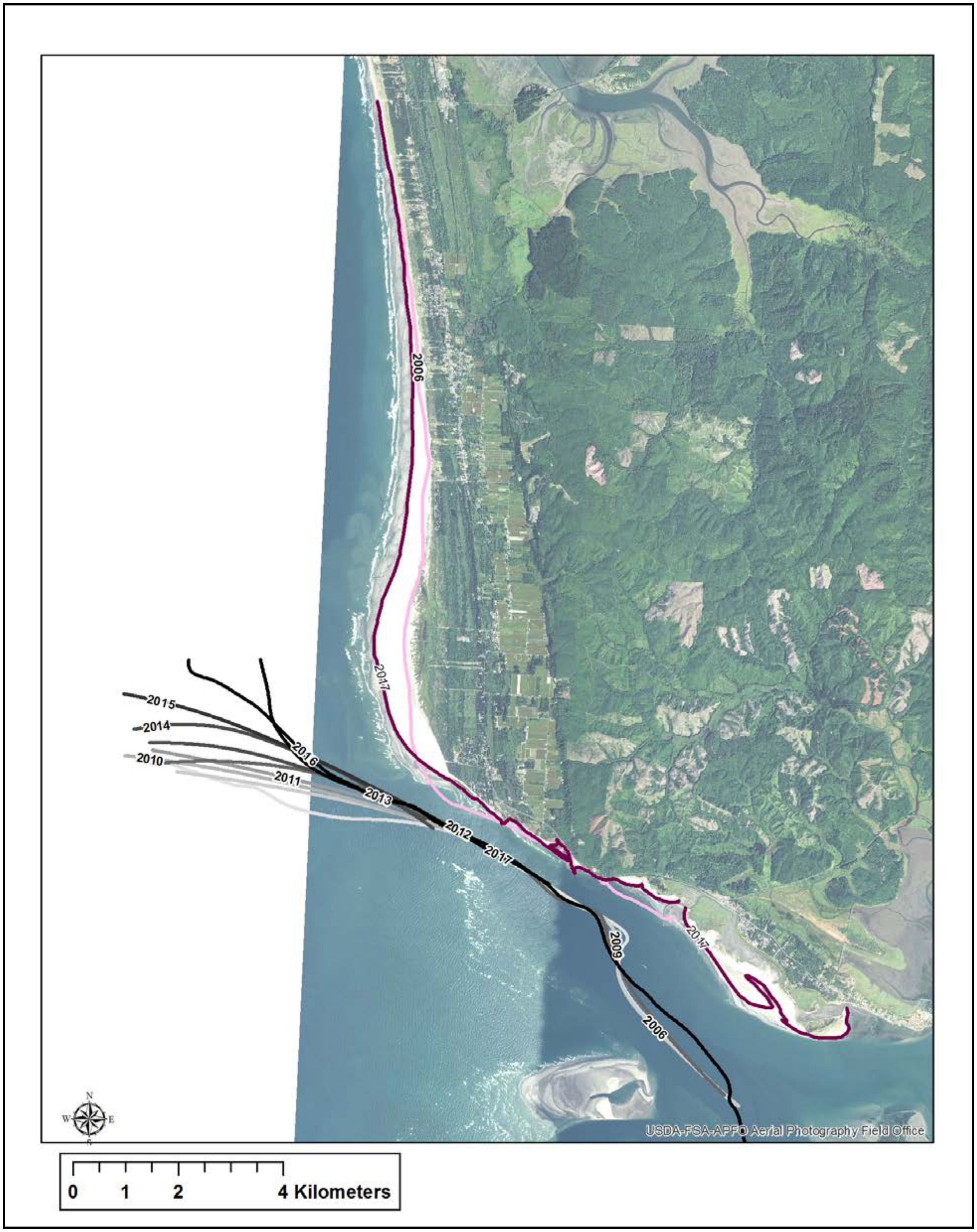


Figure 22. North Beach Spit /Grayland Plains shoreline and North Entrance Channel thalweg position from *2006 to 2017.*

7.2. North Beach Spit (Grayland Plains littoral sub cell)

The North Beach Spit (Grayland Plains littoral sub-cell) has been prograding north of the inflection point on the shoreline as shown in Figure 4. In attempt to quantify the volume of material accreted between the inflection point and the northern extent of shoreline progradation an analysis of georectified National Agricultural Imagery Program (NAIP) photographs was performed in ESRI ArcMap. Photographs from 2006 and 2017 were utilized for the time period of comparison (Figure 22).

The inflection point between the 2006 and 2017 shorelines is located at Northing = 163,336 m, Easting = 225,112 m Washington State Plane South, NAD83. Since elevation data are not available between these years in this location, an approach similar to a shoreline change one-line model is utilized to obtain volume estimates between the two measured shorelines. This involves assuming an equilibrium beach profile that translates landward or seaward normal to the shoreline over time. This equilibrium shoreline shape is defined by two parameters that define the berm height (Db) and the depth of closure (Dc). These define the upper and lower points of the profile where cross-shore sediment transport is negligible (Figure 23).[4]

The berm height can be estimated from yearly monitoring surveys collected by WDoE's CMAP program. Beach profile GELF located in North Cove, WA was observed to have a berm height (D_B) of approximately 6 m above mean sea level (Figure 24). Since, the WDoE monitoring surveys do not extend offshore, the depth of closure is estimated using the inner depth of closure outlined in Birkemeier (1985). Applying this methodology the average D_c is calculated to be 15 m (50 ft.) utilizing the USACE wave information study hindcast dataset from 1981-2011 (Brutsché et al. 2016).

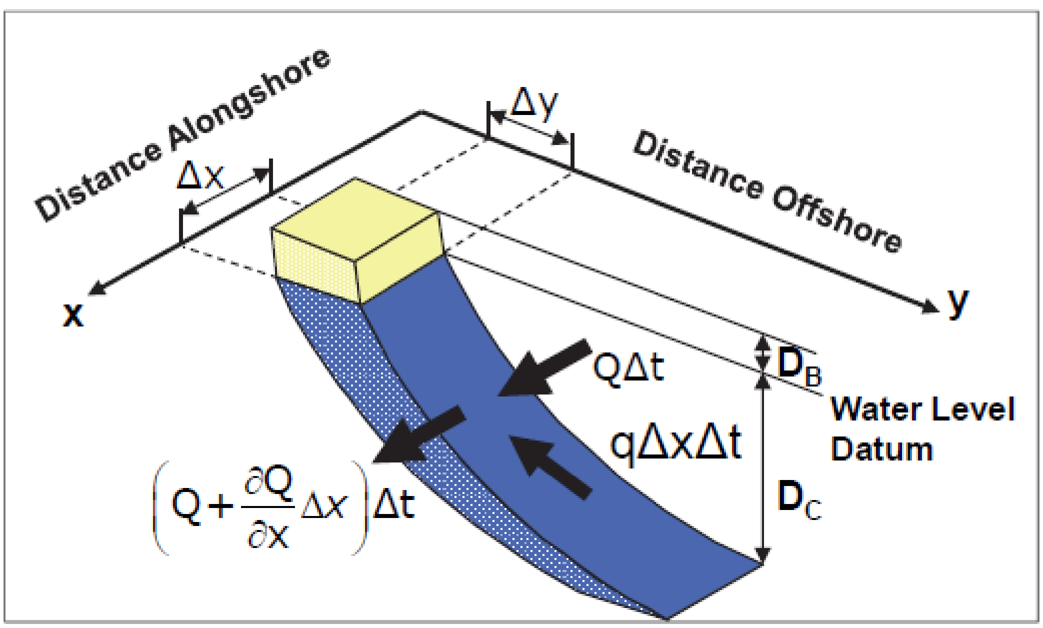


Figure 23. Schematic showing how volumes are computed between shoreline profiles

[4] See Section 10 for more information on one-line models.

The width of shoreline translation Δy is computed at each Δx = 200 m interval (Figure 25). The net volume is computed using the following:

$$volume = (D_B + D_C) \cdot \Delta x \cdot \Delta y \quad (1)$$

The net volume computed between 2006 and 2017 is +56.4 Mm^3 (+73.7 MCY). This quantity includes the balance of sediment transported out of Willapa Bay (Table 2) and material bypassing the harbor originating from the Mouth of the Columbia River.

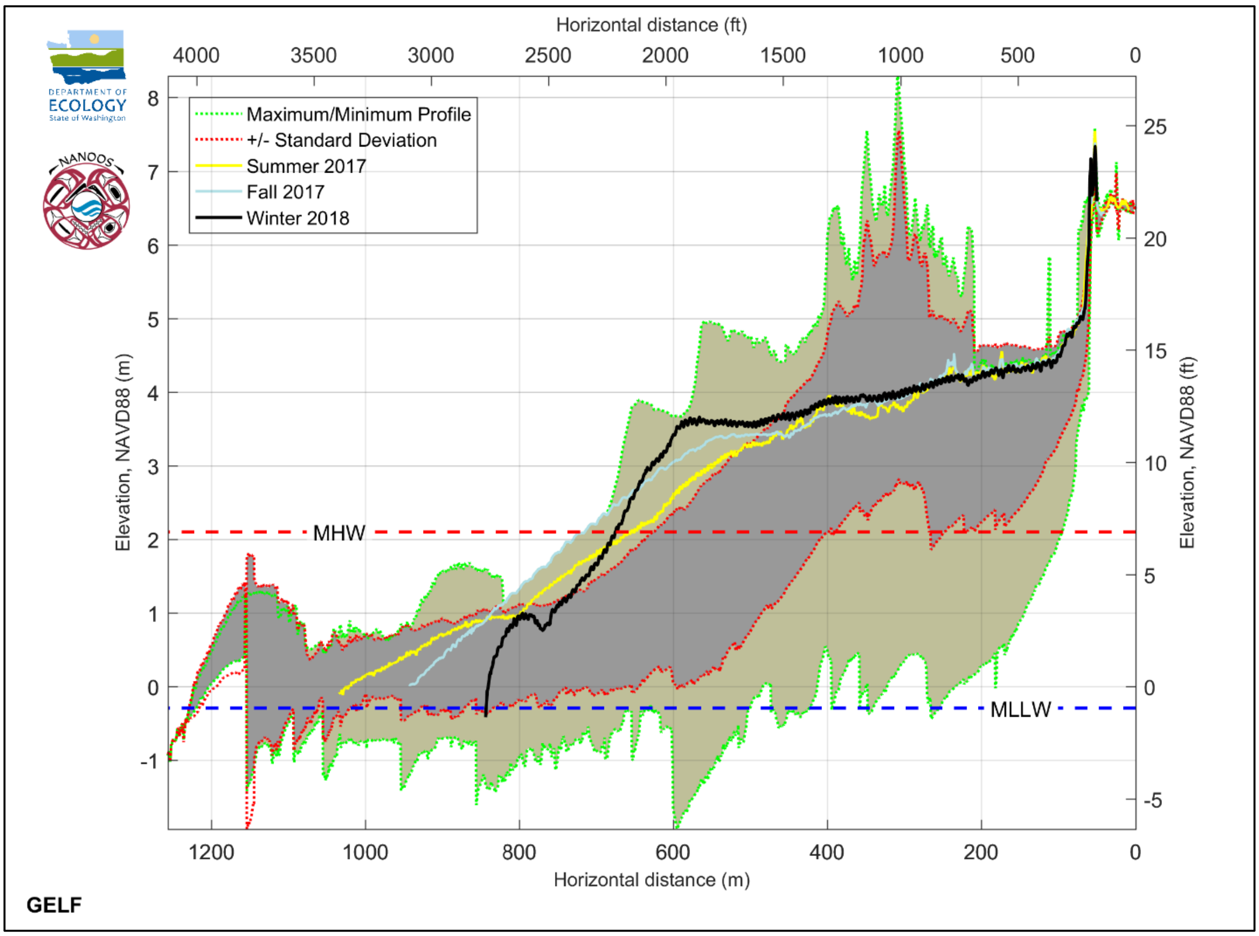


Figure 24. Beach profile GELF monitored by WDoE located at North Cove, WA

7.3. Cape Shoalwater

Another littoral sub-cell called Cape Shoalwater is located southeast of the SR-105 groin and dike. Historically Cape Shoalwater extended into the Willapa Bay and provided shelter for the low-lying topography of Tokeland Peninsula. As the Cape was eroded away, wave driven longshore sediment transport carried a portion of sediment southeast and formed an elongated barrier spit and barrier island called Graveyard Spit and Empire Spit respectively. These features continued to elongate over time until they were eventually breached by storm waves beginning in the early 1990s. In 1998, WSDOT constructed the SR-105 groin and dike which further limited the amount of sediment delivered during summer months. Thus, due to the North Entrance Channel migration and shoreline armoring, the historic sediment source has been significantly reduced.

Following the breach events, the tidal embayment separating the barrier island and Tokeland Peninsula continued to fill with sediments from storm waves. The Shoalwater Tribal Reservation located on the low-lying backshore area requested assistance from the USACE shortly thereafter to investigate the feasibility of protecting their land from future coastal flooding and damages. A dune restoration project was authorized in 2000 and constructed in 2013. The recommended plan included routine beach nourishments at 5-year intervals to maintain the designed level of protection (USACE 2009). The project completed its first nourishment cycle in September 2018.

As part of the initial project construction, a detailed monitoring program was conducted to quantify the sediment transport over time. Graveyard Spit and Empire Spit were discretized in 14 separate sediment budget polygons according to elevation and longshore boundaries altering longshore sediment transport such as structures, landforms, or inlets (Table 3; Figure 26). The erosion rates on Graveyard Spit (3a, 3b) were the largest, while large accretion (deposition) was observed in the subtidal and intertidal regions of Empire Spit (6a, 6b). The significant erosion of Graveyard Spit is evident of wave overwash, lack of sediment feeding the region, and localized wave focusing in lee of the exposed rock outcrop near the land connection point on the shoreline (Figure 27). The analysis indicates the Cape Shoalwater sub-cell has net erosion of 0.3 MCY per year.

Table 3. Cape Shoalwater Sediment budget volumes (Sep 2014 to Sep 2016)

Polygon	Erosion (CY)	Deposition (CY)	Net (CY)
1a	(111,728)	26,446	(85,283)
1b	(57,240)	46,075	(11,165)
2a	(25,923)	34,848	8,925
2b	(18,315)	3,186	(15,129)
3a	(549,723)	89,688	(460,036)
3b	(272,558)	4,733	(267,825)
4	(124,254)	51,437	(72,818)
5	(261,210)	105,617	(155,593)
6a	(286,610)	406,652	120,042
6b	(515,126)	790,972	275,846
7a	(45,016)	17,058	(27,958)
7b	(29,496)	52,687	23,190
7c	(42,082)	85,214	43,132
8	(21,288)	14,825	(6,462)
sum	**(2,360,571)**	**1,729,437**	**(631,134)**

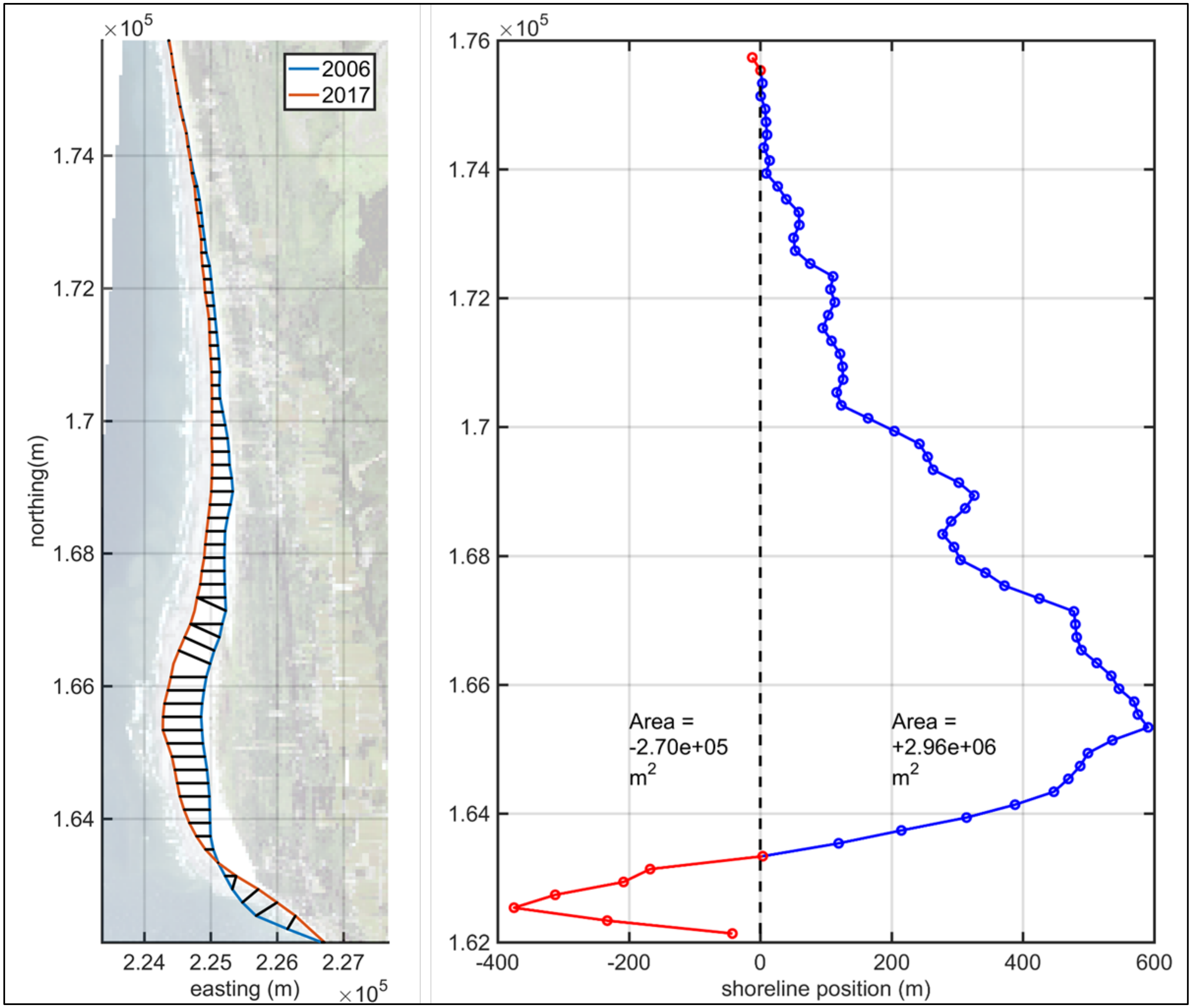


Figure 25. North Beach Spit / Grayland Plains shoreline evolution from 2006 to 2017.

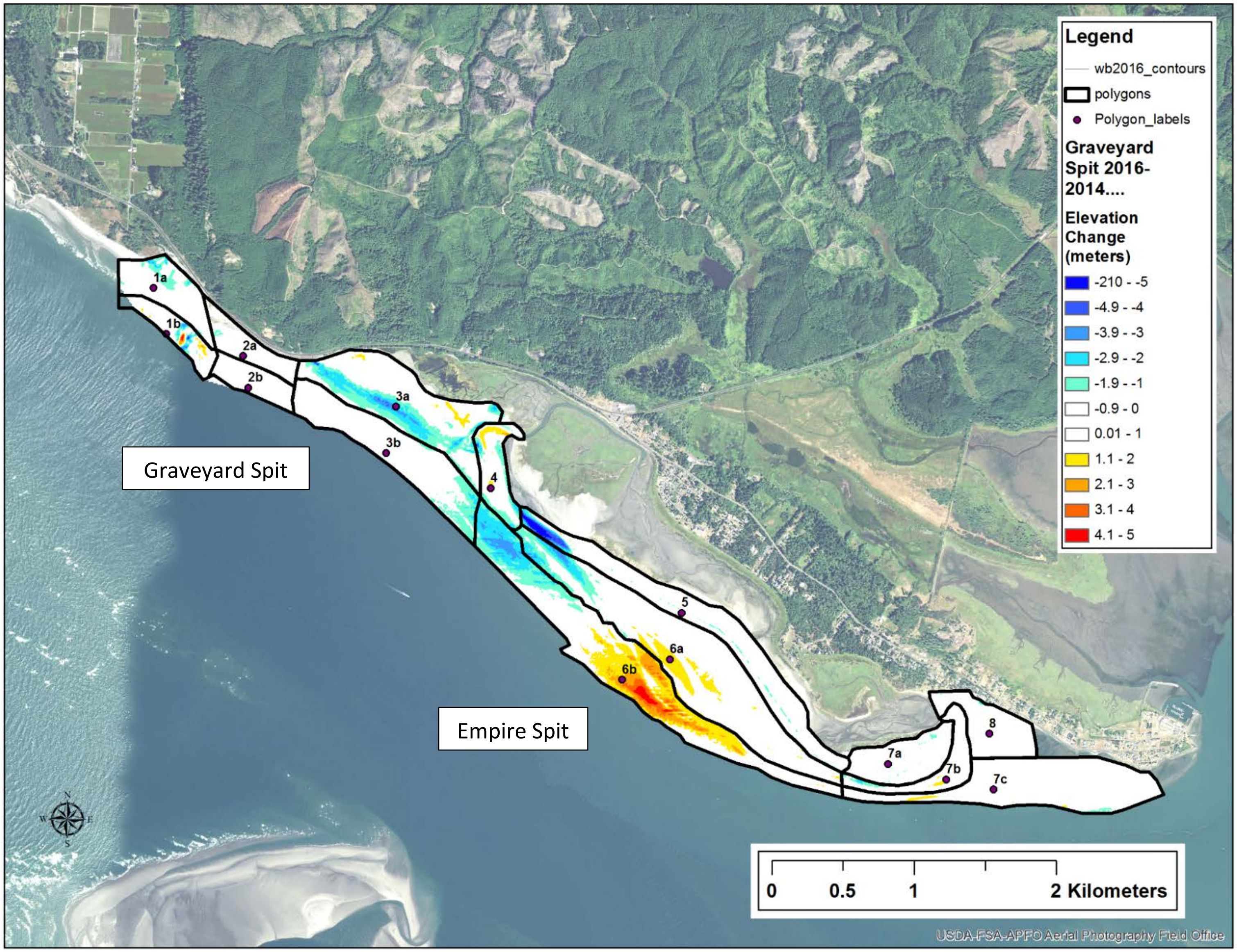


Figure 26. Cape Shoalwater – Beach and dune morphology of Graveyard / Empire Spits from Sep 2014 to Sep 2016.

Figure 27. Cape Shoalwater – Land connection of Graveyard Spit (note shoreline protection and rock outcroppings)

8. Conceptual Model

In order to develop management strategies it is necessary to develop a conceptual model of Willapa Bay. The model will accurately describe the primary processes affecting geomorphology. The previous sections provide the foundation for the conceptual model. The main drivers of sediment transport include (1) the North Entrance channel migration (2) Wave driven longshore transport (3) Storm driven dune erosion and barrier island/spit overwash. Factors which influence these processes include anthropogenic structures such as groins, dikes, revetments, and artificial beach and dunes.

Reach 1. City of North Cove - shoreline north of rock protection near Tamarack Street.

This reach is primarily influenced by migration of the North Entrance Channel. The northward migration of the channel erodes the subtidal region of the shoreface and material is deposited to the shoreline north of the Bar. Analysis of the North Channel migration from 2006 to 2017 shows the North Channel thalweg has shifted shoreward approximately 1 mile near at the Outer Bar contour.

Reach 2. Cranberry drainage ditch outfall - Shoreline between Tamarack Street and SR-105 groin and dike.

This reach is a relatively stable pocket beach bounded by two hard points on the updrift and downdrift boundaries. Analysis of the North Channel thalweg migration indicates little nearshore change since 2006. The primary mechanism influencing sediment transport in this reach are swash zone wave dynamics. Net longshore sediment transport is directed to the southeast as indicative of the elongated spit that forms during the summer month's seaward of the drainage ditch outfall (Figure 28 b). Sediment is impeded from moving further downdrift by the SR-105 groin and dike. Sediment is rather impounded during the summer months and then exported offshore along the groin into the North Entrance channel during more energetic winter months.

The dune elevation of the backshore is low in this reach, therefore the backshore is exposed to wave overtopping during winter storm events. This has resulted in bank erosion and dune incising over the past decade. Additionally a local erosion hotspot has developed to the south of the residential shoreline armoring near Tamarack Street. This flanking is similar to the pattern observed downdrift of a groin.

Reach 3. Cape Shoalwater – Barrier spits southeast of SR-105 groin and Dike

This reach is composed of a series of barrier spits driven primarily by swash zone driven longshore transport. Historically longshore sediment transport from the North Beach Spit region supplied a significant volume of sand to this region in summer months when northwesterly wave incidence is predominant. Wind blown sand formed protective dunes situated on these spits. However, the historic rates of sediment began to change as the North Entrance Channel migration shifted shoreward. Additionally engineered structures to protect SR-105 further reduced the historic sediment load feeding the spits. Over time, the updrift regions of Graveyard spit and Empire Spit became sediment deprived and the protective dunes were incised and breached during storm events. Following the loss of the dune, the spits began to migrate landward toward Tokeland Peninsula as waves overwashed sediments into the tidal embayment. This process was halted on Empire Spit through the restoration of an artificial dune as part of the USACE Shoalwater coastal storm damage reduction project. However, these processes are still active on Graveyard Spit.

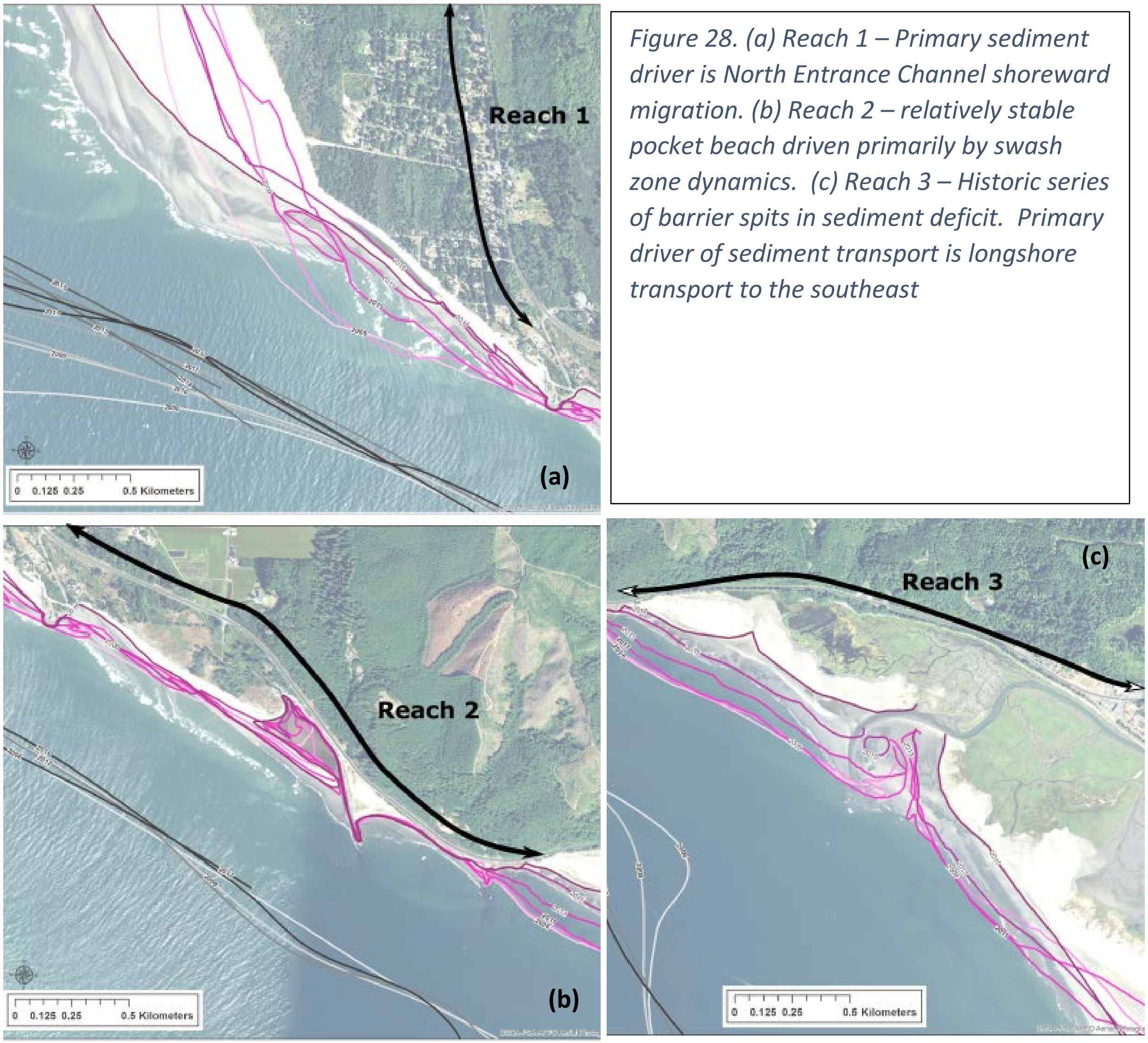


Figure 28. (a) Reach 1 – Primary sediment driver is North Entrance Channel shoreward migration. (b) Reach 2 – relatively stable pocket beach driven primarily by swash zone dynamics. (c) Reach 3 – Historic series of barrier spits in sediment deficit. Primary driver of sediment transport is longshore transport to the southeast

9. Hydrodynamic Modeling

A two-dimensional hydrodynamic model with sediment transport capability was utilized to simulate the baseline and future alternative scenarios. The USACE developed 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 and bed elevation is updated at each time step to incorporate effects of bed morphology on wave and currents through time.

The spectral wave model CMS-Wave is based on the wave-action balance equation (Lin et al. 2008). The processes of wave refraction, shoaling, diffraction, reflection, breaking, and dissipation are represented in the model. It is a two-dimensional spectral wave model formulated from a parabolic approximation equation (Mase et al. 2005) with energy dissipation and diffraction terms to simulate a steady-state spectral transformation of directional random waves co-existing with currents in the coastal zone. 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.

Non-equilibrium combined bedload and suspended load is used to compute sediment transport in CMS using the Soulsby (1997) formula. The non-equilibrium sediment transport algorithm simulates non-cohesive, single size sediment transport and bed change using a Finite Volume method and includes advection, diffusion, hiding and exposure, and avalanching. Non-erodible cells are used to specify the structures in the model domain, including the SR-105 groin/dike, rock outcroppings, and residential shoreline protection.

4.1 Model Description and Setup

The model is forced with a water surface elevation at the offshore boundary generated from published NOAA tidal constituents and wave heights measured at the Coastal Data Information Program Buoy (CDIP) 036 (Figure 29). The CMS model uses a variable rectangular grid to represent the topography and bathymetry. This allows for coarser resolution offshore in deep water and finer resolution near structures and other important land forms. A total number of 507,522 cells with a cell size ranging from 20 m to 2.5 km was utilized.

4.2 Bathymetry & Topography

Various data sources were used to develop a continuous Digital Elevation Model (DEM) of Willapa Bay for the CMS model (Figure 30). The existing DEM from USACE (2009) was utilized as the baseline dataset. However, newer data particularly at the Bar and North Entrance Channel were incorporated in order to reflect the most recent conditions in Willapa Bay. The following data sources were utilized:

- NOAA Astoria, OR 1/3 arc second DEM

This is compilation of a number of sources including historic NOAA surveys from 1851 to 2014 and USACE/USGS LiDAR from 2011. The mosaic was compiled and processed in ArcGIS using error reduction techniques (Love et al. 2012)

- 1998-2002 USACE Bathymetric Condition Survey

These surveys include single beam transects of the South Bay thalweg and was conducted by the USACE, Seattle District's *Shoalhunter* Survey vessel. Surveys were conducted in July-September 1998 and March-June 2002

- 2014 JALBTCX National Coastal Mapping Program Topographic and Bathymetric LiDAR

This survey included combined topographic and bathymetric blue/green LiDAR of the Willapa Bay Entrance channel including the shoals at the mouth of the bay. This survey was conducted in July-August 2014 by the Joint Airborne LiDAR Bathymetry Technical Center of Expertise (JALBTCX).

- 2016 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 April 2016 and August-September 2016. An additional high resolution multibeam survey of the North Cove nearshore region and SR-105 dike was completed in May 2016.

- 2016 WDoE Combined Topographic and Bathymetric Survey

This survey included a combination of All Terrain Vehicle (ATV) GPS, backpack GPS, boat based LiDAR, and multibeam surveys of Cape Shoalwater and the nearshore region. This survey was performed by Washington State Department of Ecology (WDoE) as part of the monitoring program for the USACE Shoalwater Bay dune restoration project in September 2016.

- 2016 Shoals

NAIP 2017 aerial photography was utilized to digitize the horizontal positions of the surface piercing shoals. An elevation representing the maximum astronomical tide were assigned to these shoal areas.

4.3 Model validation

Two one-month long simulations were utilized to validate the model performance with measured data. These included a strong winter (i.e. the 10 December 2015 storm event) and a typical summer month from August 2016 when incident wave conditions are predominantly out of the northwest. The time series comparisons of measured versus modeled wave height were compared at observation points and were determined to be within an acceptable root mean square error. In general, the model performed well during average wave conditions, but slightly under predicted the nearshore wave heights during large storm events. This is common to linear wave theory models which do not represent the non-linear peaks of waves shoaling in the nearshore region (Figures 31 and 32).

4.4 Wave results

Nearshore wave heights were recorded along the North Cove Shoreline to investigate alongshore variability. Strong gradients in wave height can drive longshore currents in the swash zone capable of transporting sediment alongshore. A series of observation points were specified at the 10 m contour (Figure 33). A large wave height gradient was found to develop between observation point 3 near Tokeland and point 11 near North Cove (Figure 34). Over the month of December 2015, the average significant wave height near North Cove was approximately 3 m (10 ft.), while this was 0.3 m (1 ft.) near Tokeland. The significant wave height at the SR-105 dike ranged from 0.5 to 1.5 m (1.6 to 5 ft.)

4.4 Current results

Currents were investigated near the SR-105 groin and dike and in the North Entrance Channel. Local flow acceleration over the submerged groin and slower velocities on the leeside of the structure during both ebb and flood tides was observed in the simulation. During peak ebb, the maximum currents near the dike were computed at 2 m/s (4 knots). During peak flood, maximum currents computed were 1.5 m/s (3 knots). A gyre also forms in lee of the dike during both ebb and flood tides (Figure 10). A deep trench with depths exceeding -120 feet MLLW exists that also likely contributes to this current pattern. Further to the northwest cross currents of opposing flows were observed adjacent to the North Entrance Channel near North Cove, WA (Figure 35). This is a result of waves driving nearshore circulation while tidal currents generating high flows through the thalweg on an outgoing ebb tide.

4.5 Morphology results

The morphology (or seabed elevation change) over the December 2015 and August 2016 time periods was simulated. The model compares well with historic trends observed in the North Entrance Channel and the North Beach Spit area. During the energetic winter simulations, a significant volume of sand was transported from the shoals into the North Entrance channel. This further promoted scour of the thalweg as well as nearshore erosion near the residential shoreline protection near Tamarack Street. However, significant amount of sand is also observed to bypass the channel as deposit on the North Beach Spit to the northwest (Figure 36). Additionally, sediment is deposited on the ebb tidal delta of the Bar. In the summer months a similar trend is witnessed but at much smaller magnitude (Figure x). This suggests this sediment bypassing process is primarily driven by tidal currents rather the wave driven transport. A unique observation from the summer simulation shows shoal accretion and building up sediment on the flanks of the oceanward North channel sideslopes. This cyclical process is believed to continue driving the North Channel migration northward over time.

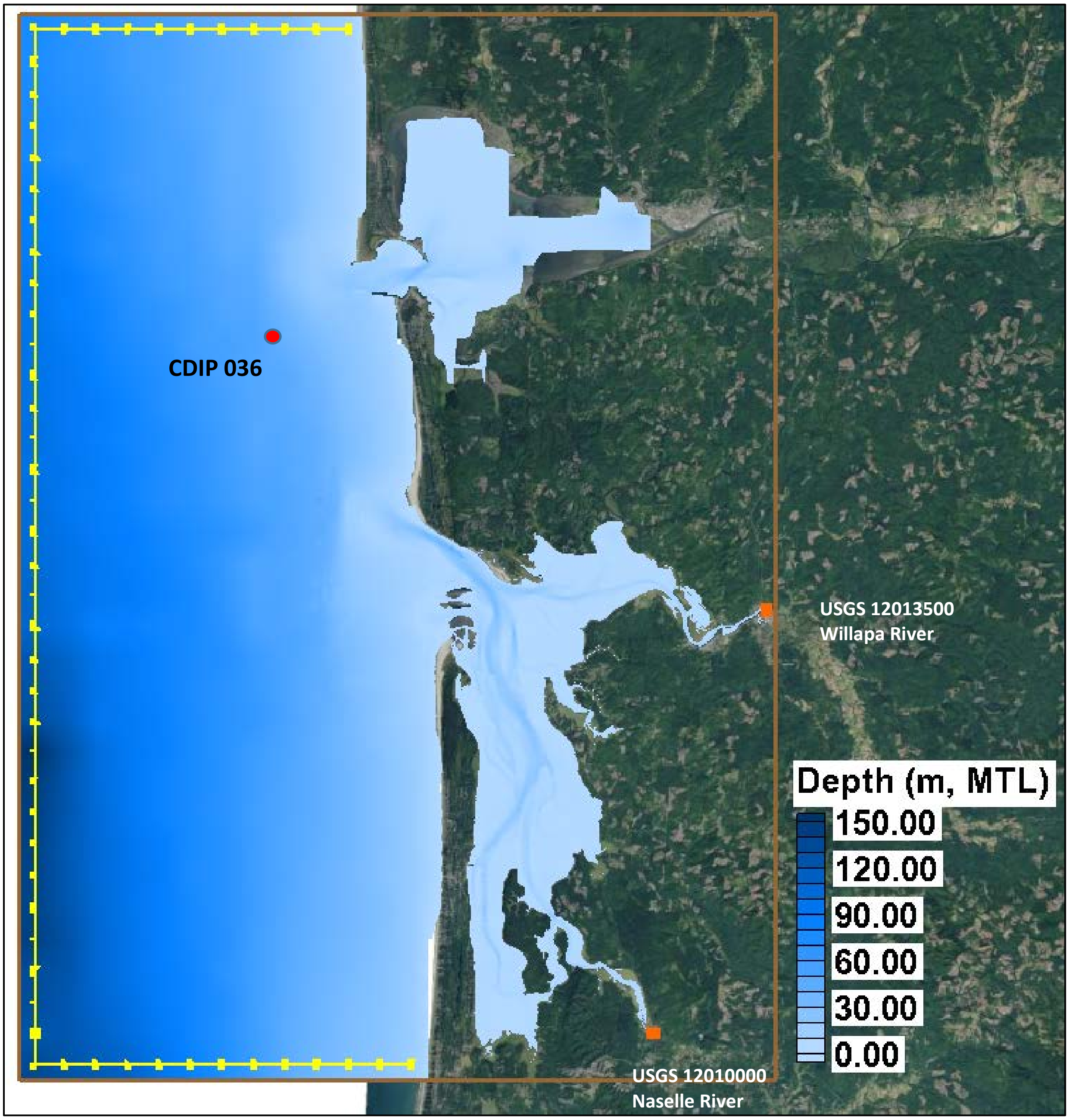


Figure 29: Bathymetry and Boundary Conditions used in Coastal Modeling System (CMS) numerical model.

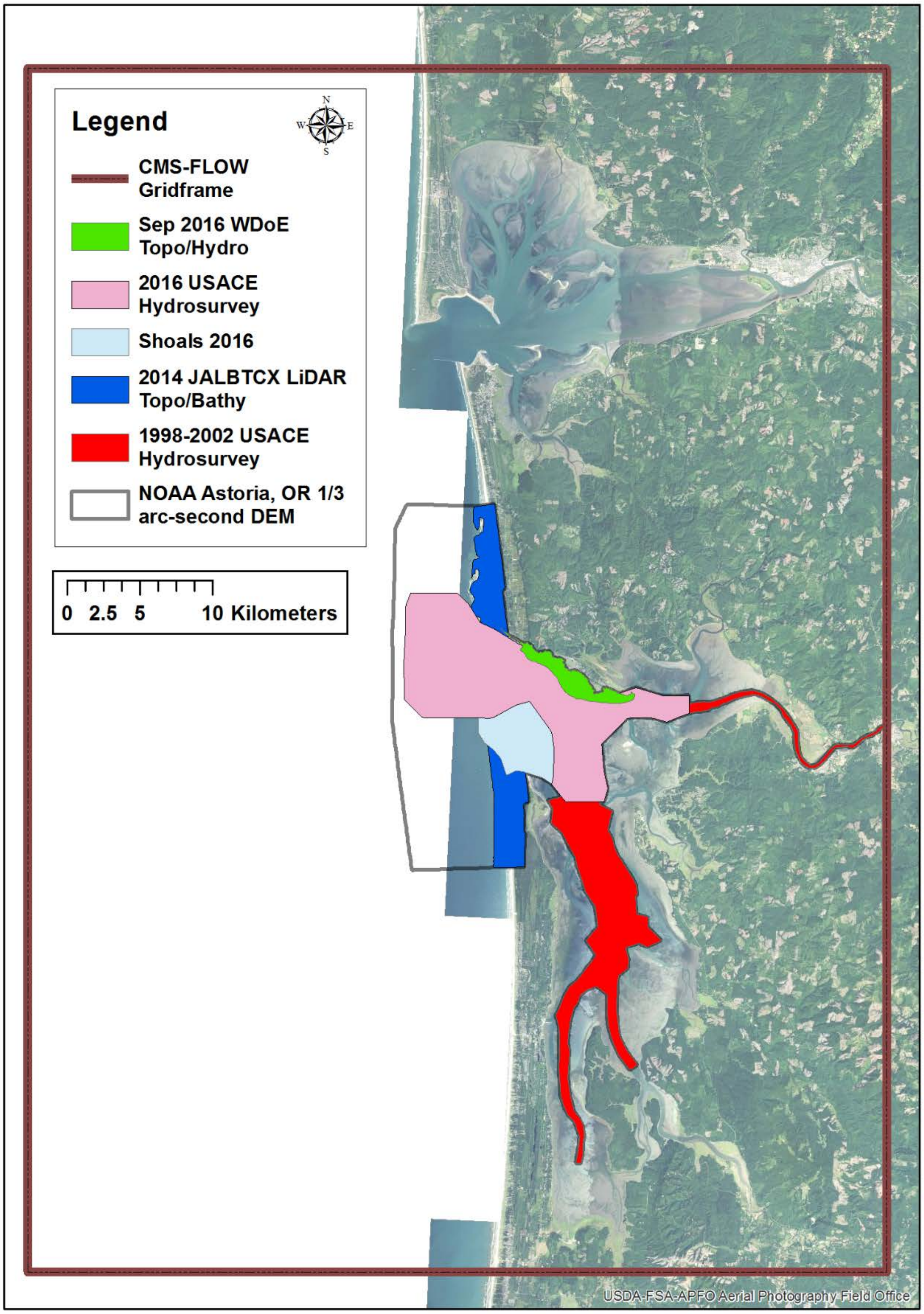


Figure 30. Sources of bathymetry and topography for Willapa Bay numerical model

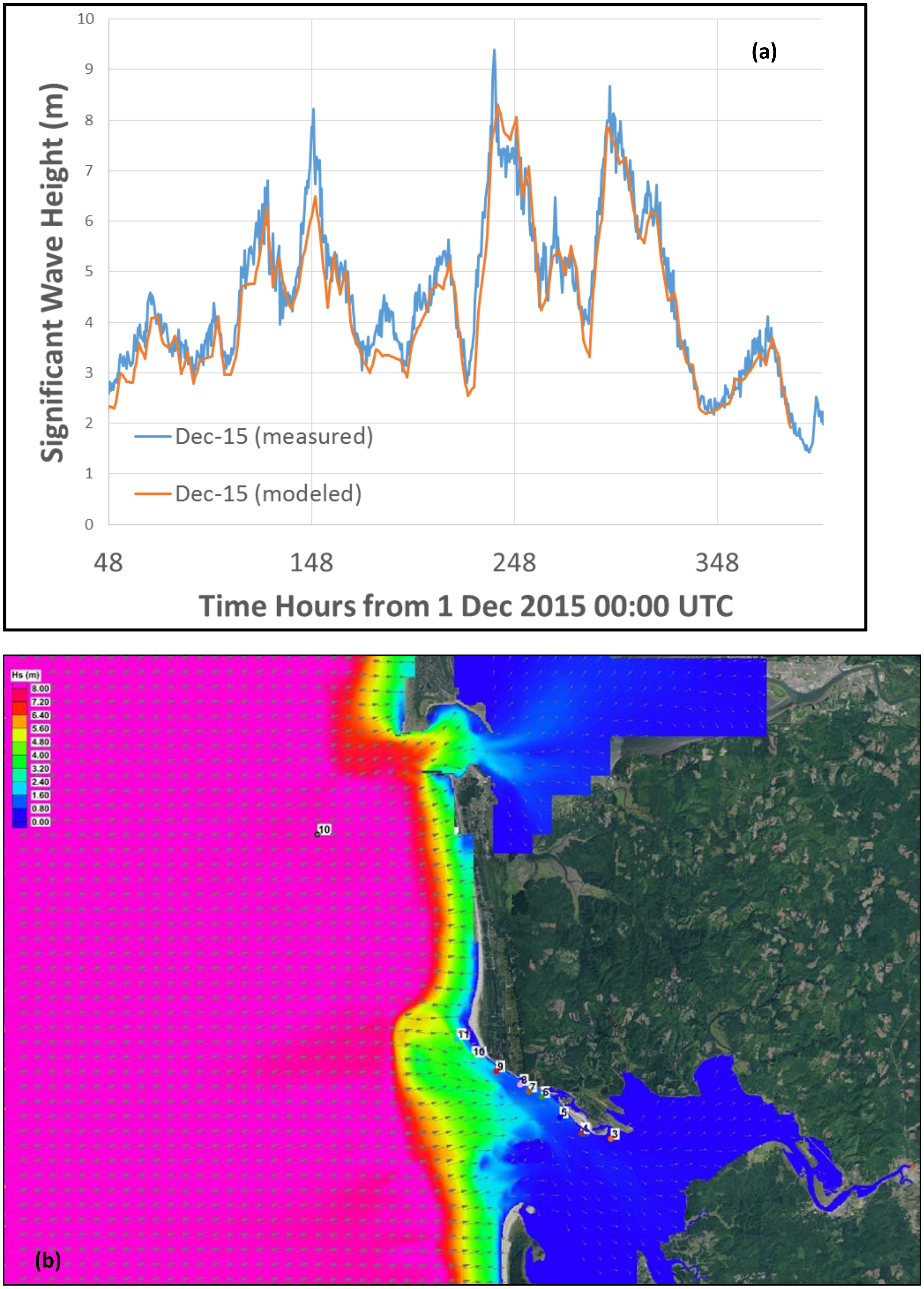
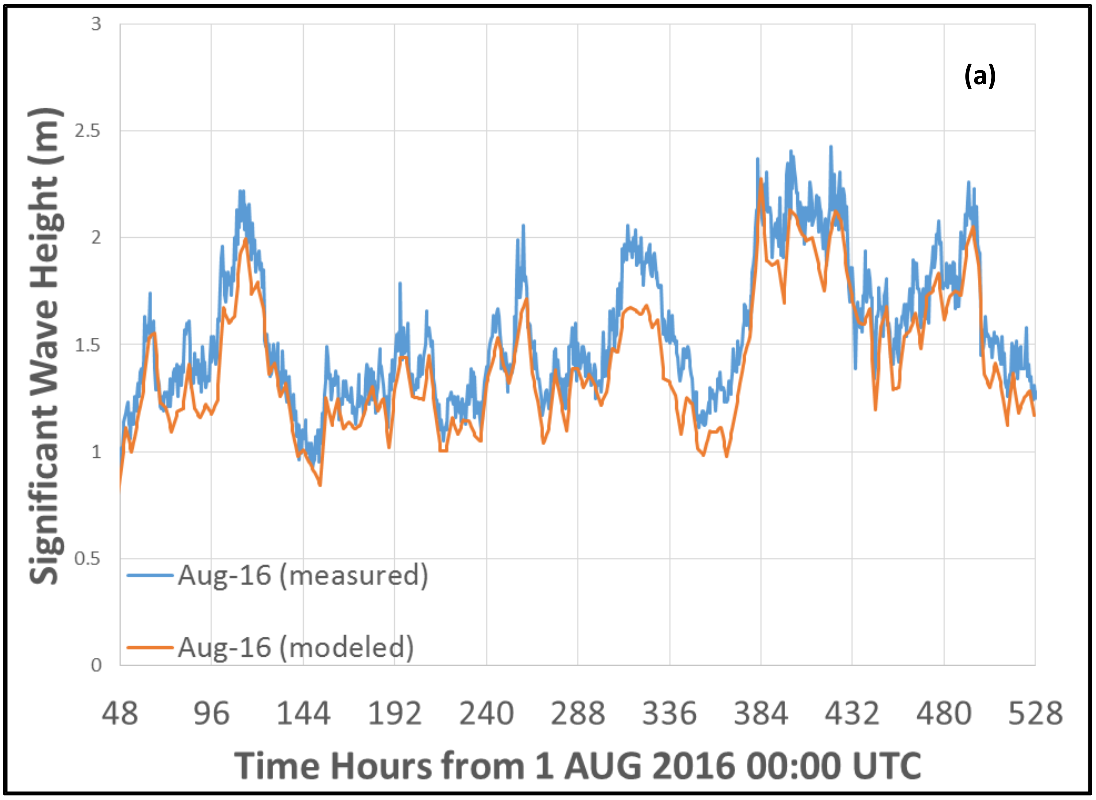


Figure 31: December 2015 (a) wave height time series at point 10 (CDIP 036) (b) Modeled wave height on 10 Dec 2015; vectors denote wave direction



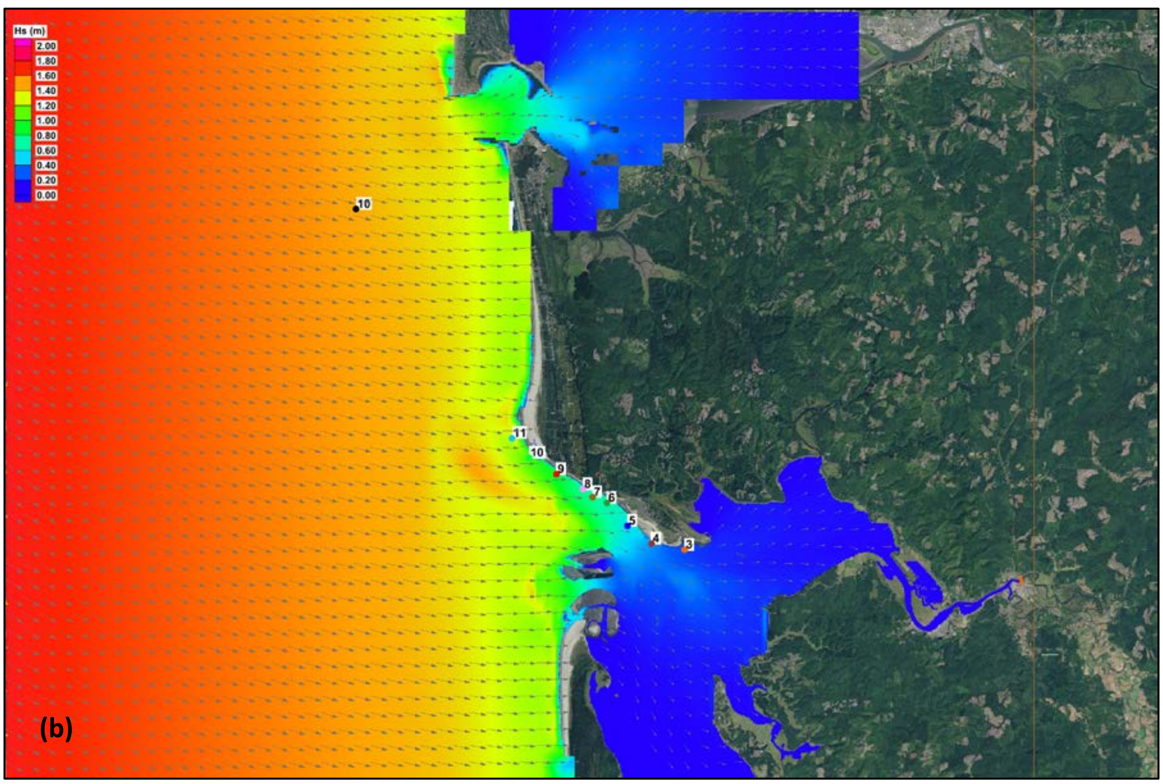


Figure 32: December 2015 (a) wave height time series at point 10 (CDIP 036) (b) Modeled wave height on 10 Dec 2015; vectors denote wave direction

Figure 33: Model observation points along the shoreline from North Cove to Tokeland, WA

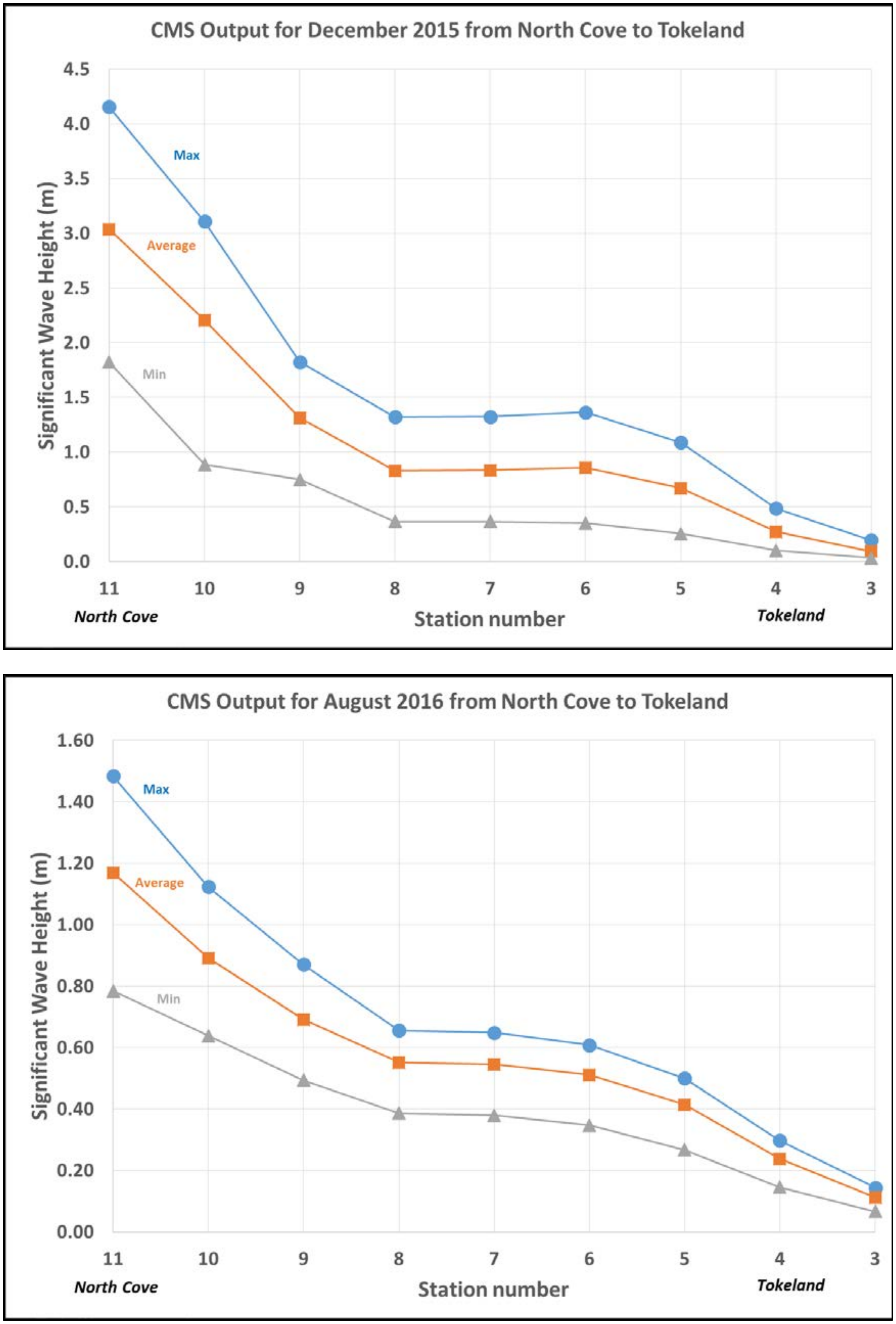


Figure 34: Alongshore variation in wave height for (a) December 2015 (b) August 2016

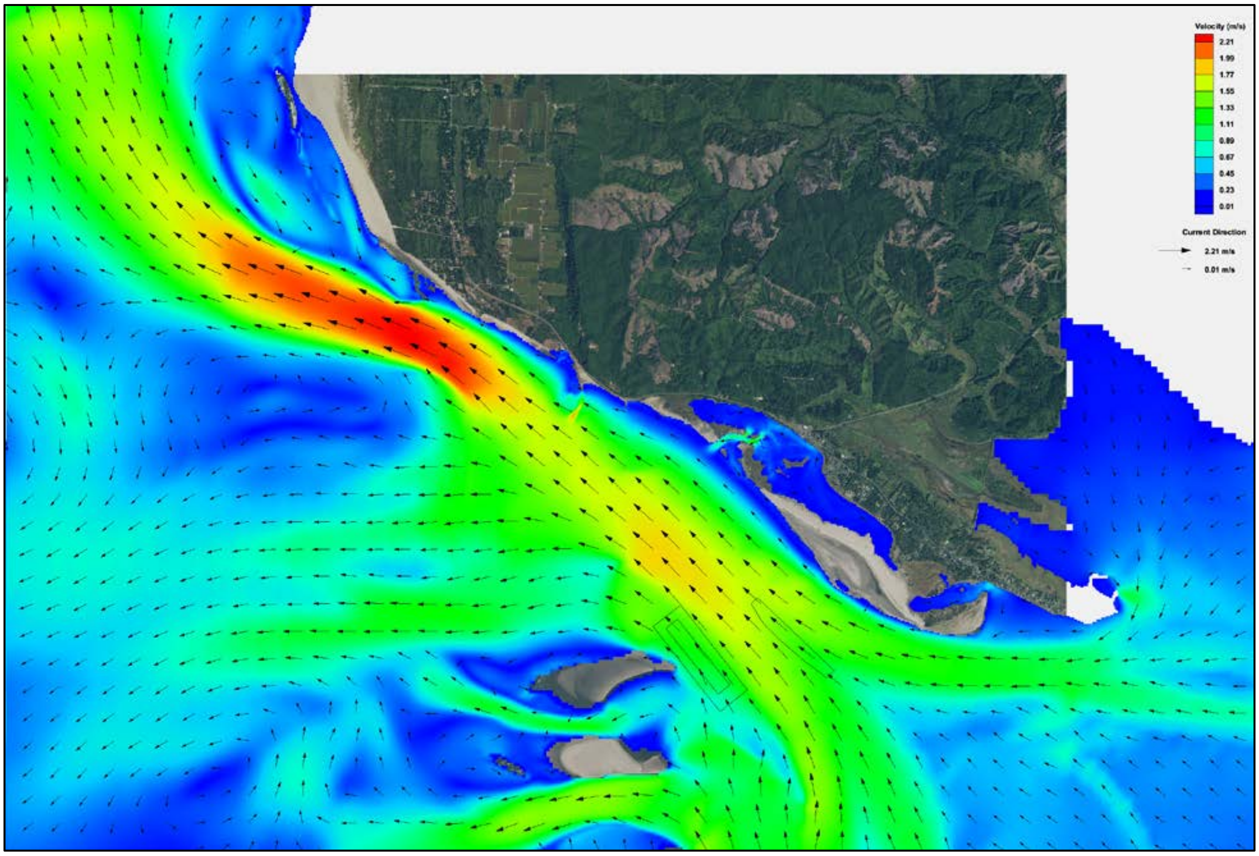


Figure 35. Current field during an ebb tide; note cross currents over the North Beach Spit.

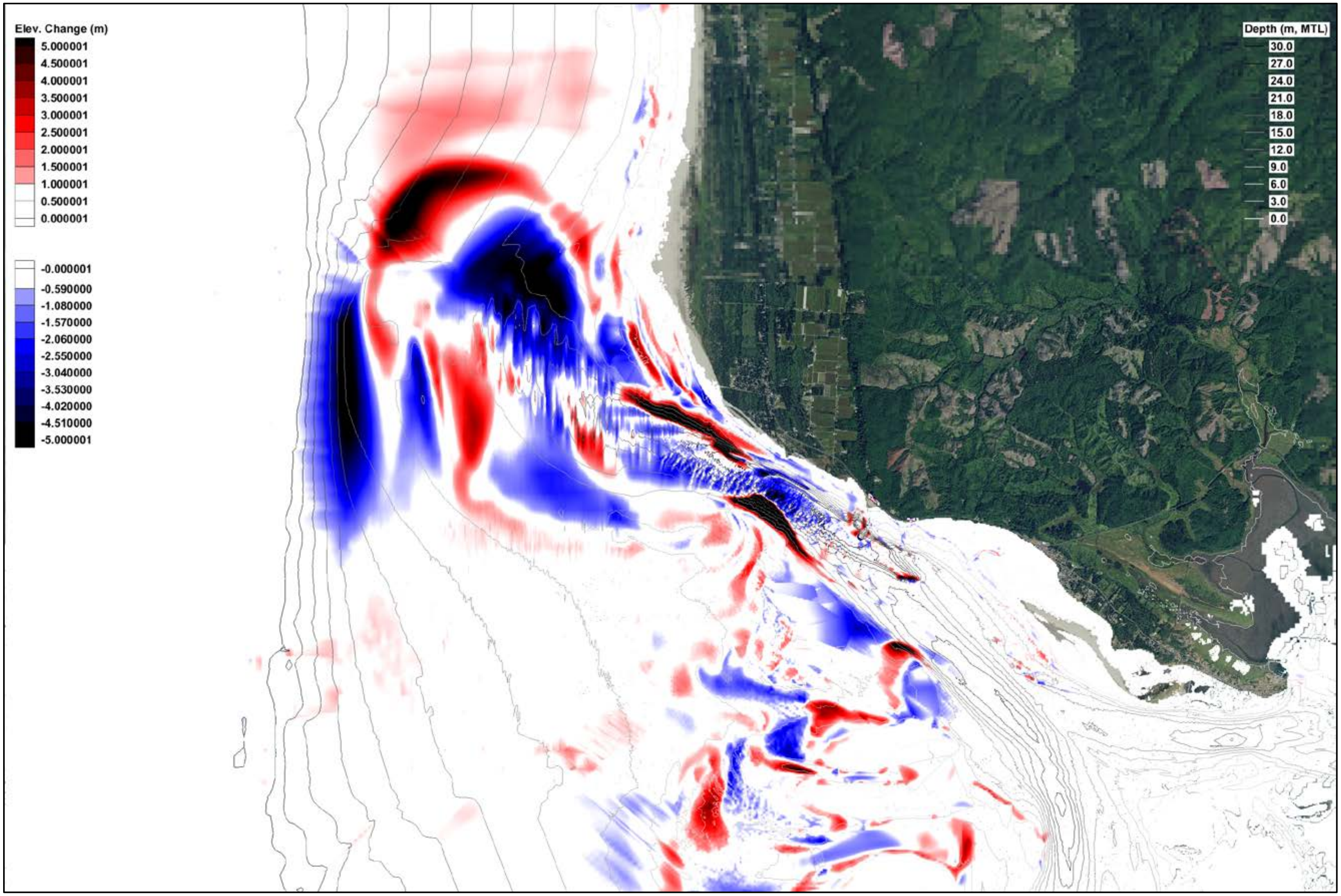


Figure 36. Calculated seabed change over December 2015 simulation

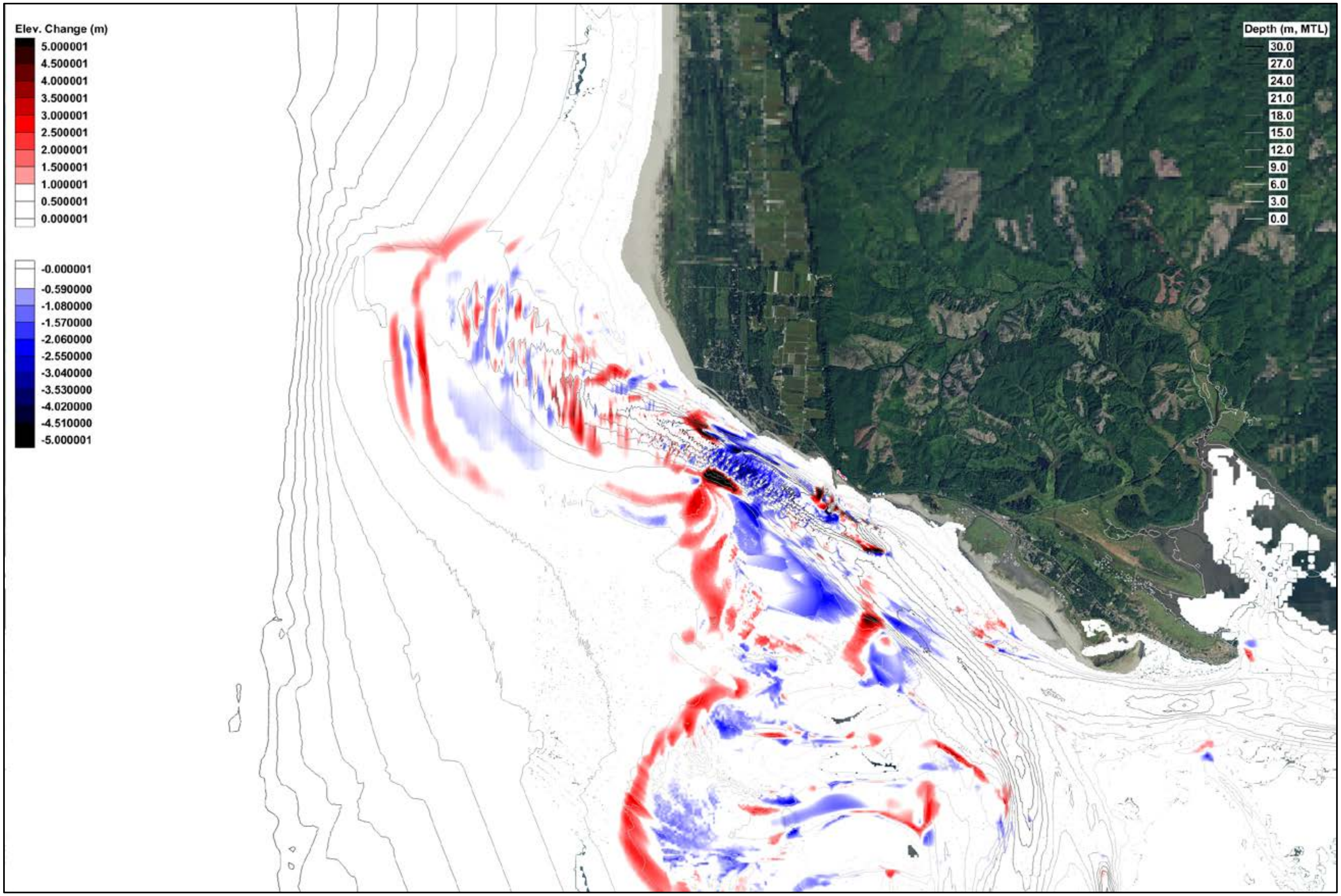


Figure 37. Calculated seabed change over August 2016 simulation

10. Shoreline Change modeling

The CMS model is used to compute nearshore morphology. However, swash zone dynamics, which affect shoreline change, are not intrinsically represented in the model. Standard engineering practice for investigating long-term shoreline change is to utilize a one-line model. The GENCADE model (Frey et al. 2012) is applied utilizing the nearshore wave statistics computed in CMS to force the boundary conditions in the model. GENCADE simulates shoreline change produced by spatial and temporal differences in longshore sand transport. The model is based on the assumption that the beach profile moves parallel to itself, i.e. that is translates showeward and seaward without changing shape in the course of eroding or accreting. 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 geometrical-type assumption is that sand is 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, the so-called 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.

A 10-year period from 2013 to 2023 was simulated to calibrate and validate the model. Hourly wave data from the CDIP buoy 036 was used to generate the wave statistic over a typical year. These statistics were then utilized to develop a randomly generated synthetic time series of the incident wave conditions simulated in the model. The GENCADE model is most applicable to the reaches driven primarily by waves. It should be noted the model does not directly incorporate the result of the North Entrance Channel migration and sediment bypassing from the offshore Shoals to the North Beach Spit area. Therefore, shoreline predictions northwest of Tamarack Street in Reach 1 do not reflect this additional sediment source input into the littoral cell. Shoreline predictions in Reaches 2 and 3 include the response as a result of the alongshore barriers which impede longshore transport. Clear trends of shoreline recession on the downdrift side of the shore protection near Tamarack Street, the SR-105 groin, and the rock outcropping near the land connection of Graveyard Spit are computed by the model (Figure 38). These trends have been observed by successive aerial photography (Figure 28) indicating the model is accurately portraying shoreline long-term change near structures.

Figure 38. GENCADE validation run from 2013 (blue) to 2023 (red).

11. Management Measures

Management measures (MM) are features or activities that can be implemented at a specific site to address one or more planning objectives. The following measures were identified for the study area:

MM1: Improve sediment delivery downdrift SR-105 groin/dike

MM2: Establish a stable shoreline planform shape to minimize erosional hotspots near existing structures

MM3: Reduce flooding, sand overwash and debris on SR-105

MM4: Reduce risk of North Entrance Channel migration to SR-105

The following alternatives were developed using the management measures for each reach identified in the conceptual model. In order to address MM1, modifications to the SR-105 groin or sand bypassing is necessary. Modifications to the groin could include notching a sill near the swash zone at approximately mean tide level. This would facilitate wave driven sediment transport to the downdrift shoreline while still allowing the submerged dike to provide flow diversion away from the shoreline. Similarly, a sand bypassing program could be utilized to achieve this goal. This would involve a hydraulic pump and pipeline to move impounded sand from the spit fronting the drainage ditch to the downdrift side of the groin. In order to address MM2, this involves setting back new shoreline protection near existing longshore barriers. Longshore barriers result in predictable updrift and downdrift planform shoreline shapes. This results in shoreline offset and local hotspot erosion immediately downdrift of the barrier. In the project area, there are three longshore barriers that have resulted in disruption of shoreline uniformity. These include the rock protection around the home near Tamarack Street, the SR-105 groin, and the rock outcroppings near the land connection of Graveyard Spit. A cobble beach, or dynamic revetment has been implemented by local interests within this region and could be enhanced to provide additional protection and longevity. In order to address MM3, low lying areas of SR-105 need protection from storm surge and wave runup. This can be accomplished by absorbing wave energy prior to reaching the roadway or moving the road out of the flood zone. Restoring the historic elevation of Graveyard Spit would be one means to accomplish this goal. A cobble sized beach, or dynamic revetment can be utilized to protect the dune from direct wave attack thereby extending the design life. Similarly, raising the road and constructing a road embankment would also achieve the goal. Finally, in order to address MM4 the North Entrance Channel migration must be addressed. The SR-105 dike has provided shoreline stability approximately 0.5 mile northwest. However, recent analysis indicates outside of this region the North Entrance Channel thalweg is still translating landward. This trend presents the biggest risk to the City of North Cove and ultimately the SR-105 roadway between Tamarack Street and Warrenton-Cannery Road. Protection to this area can be addressed through construction of a new training dike. However, such a structure would need to consider secondary impacts to longshore sediment transport. A sill could be incorporated into the training dike to help mitigate these impacts.

The management measures are applied using the conceptual model to identify specific alternatives for each project area reach.

- *Reach 1* - City of North Cove - shoreline north of rock protection near Tamarack Street

a. Alt 1a - Construct a new 1,300 ft. rock training dike (MM4) – Figure 39

b. Alt 1b - Construct a new 1,300 ft. rock training dike with 200 ft. wide sill in swash zone (MM4) – Figure 39

- *Reach 2* - Cranberry drainage ditch outfall - Shoreline between Tamarack Street and SR-105 groin and dike.

  a. Alt 2a - Construct a 3,600 ft. dynamic revetment (18” minus cobble) and notch 330 ft. of existing SR-105 groin to create sill (MM1, MM2, MM3) – Figure 40

  b. Alt 2b - Construct a 3,600 ft. dynamic revetment (18” minus cobble) and bypass 15,000 cubic yards of impounded sediment north of groin every 5 years (MM1, MM2, MM3) – Figure 41

  c. Alt 2c - Construct dynamic revetment (18” minus cobble) and 30,000 cubic yards of beach fill downdrift of SR-105 groin every 10 years (MM1, MM2, MM3) – Figure 42

- *Reach 3* - Cape Shoalwater – Barrier spits southeast of SR-105 groin and dike

  a. Alt 3a - Restore 3,800 feet of Graveyard Spit dune and maintain with beach fill every 5 years (MM1) – Figure 43

  b. Alt 3b - Restore 3,800 feet of Graveyard Spit dune and construct a protective dynamic revetment (12” minus cobble) on seaward side of dune. Maintain dune and dynamic revetment every 10 years. (MM1, MM2, MM3) – Figure 44

  c. Alt 3c - Extend rock roadway embankment along SR-105 to a total length of 4,500 feet to Tribal boundary (MM3) – Figure 45

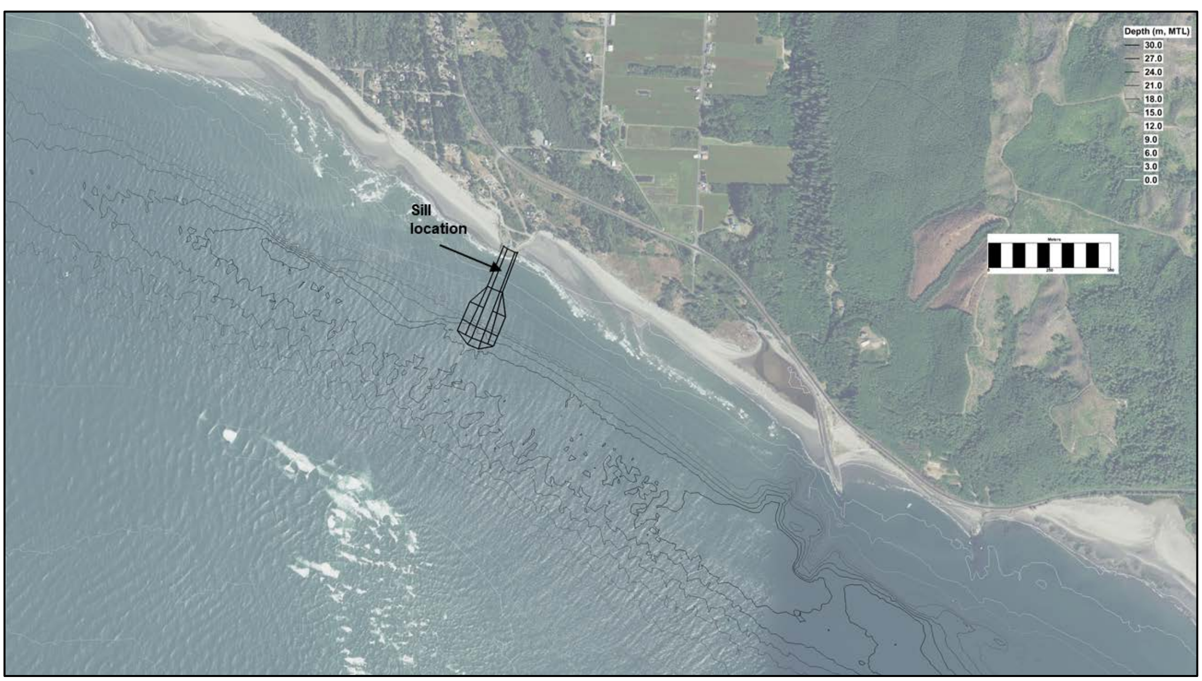


Figure 39. New Training dike - Alternative 1a without sill; Alternative 1b with sill.

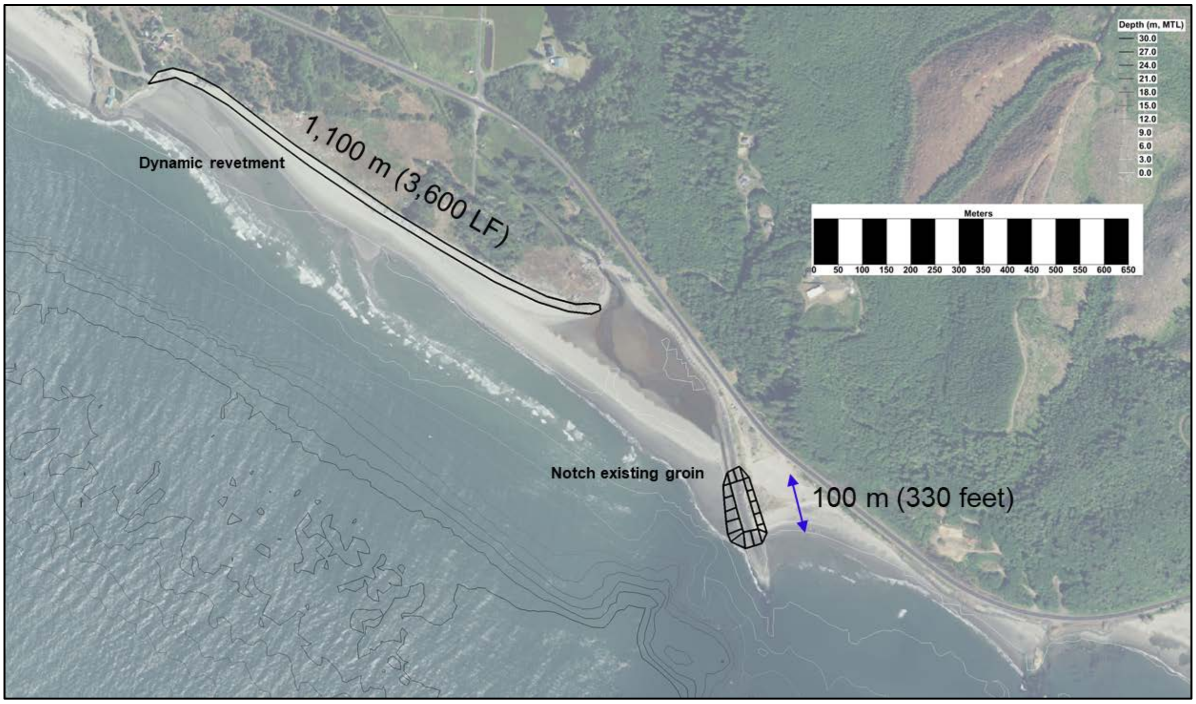


Figure 40. Alternative 2a – Dynamic revetment with SR-105 groin notching

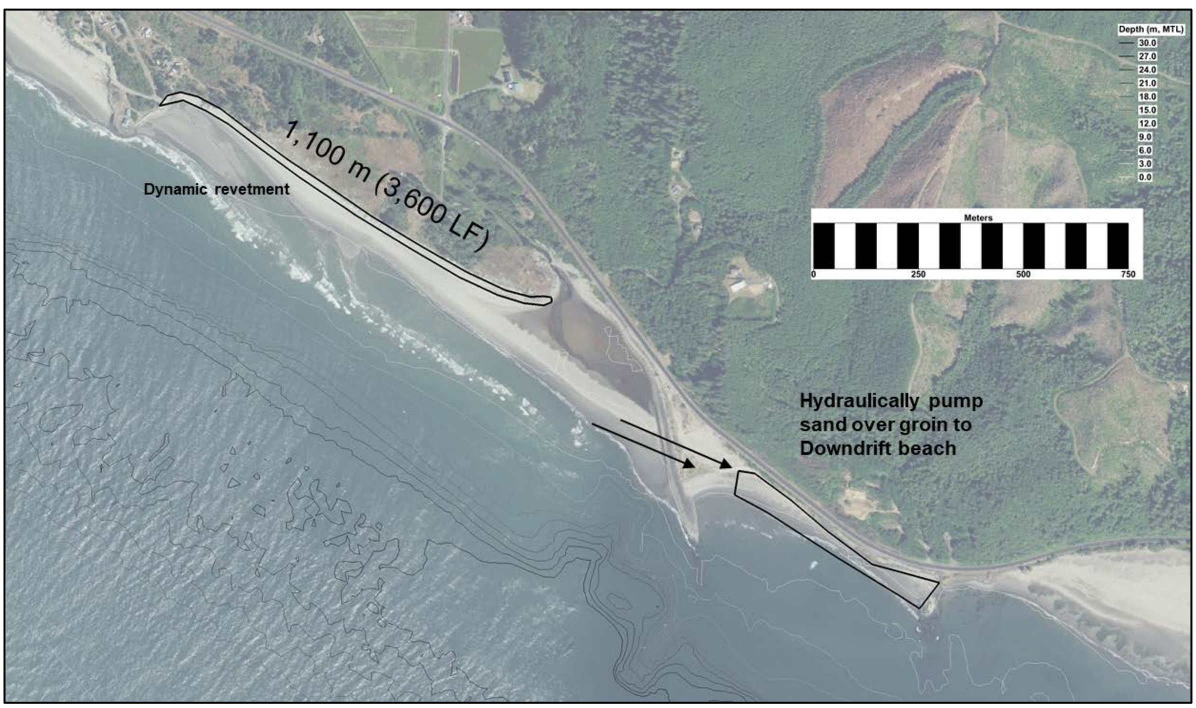


Figure 41. Alternative 2b. Dynamic revetment with sediment bypassing

Figure 42. Alternative 2c. Dynamic revetment with beach fill

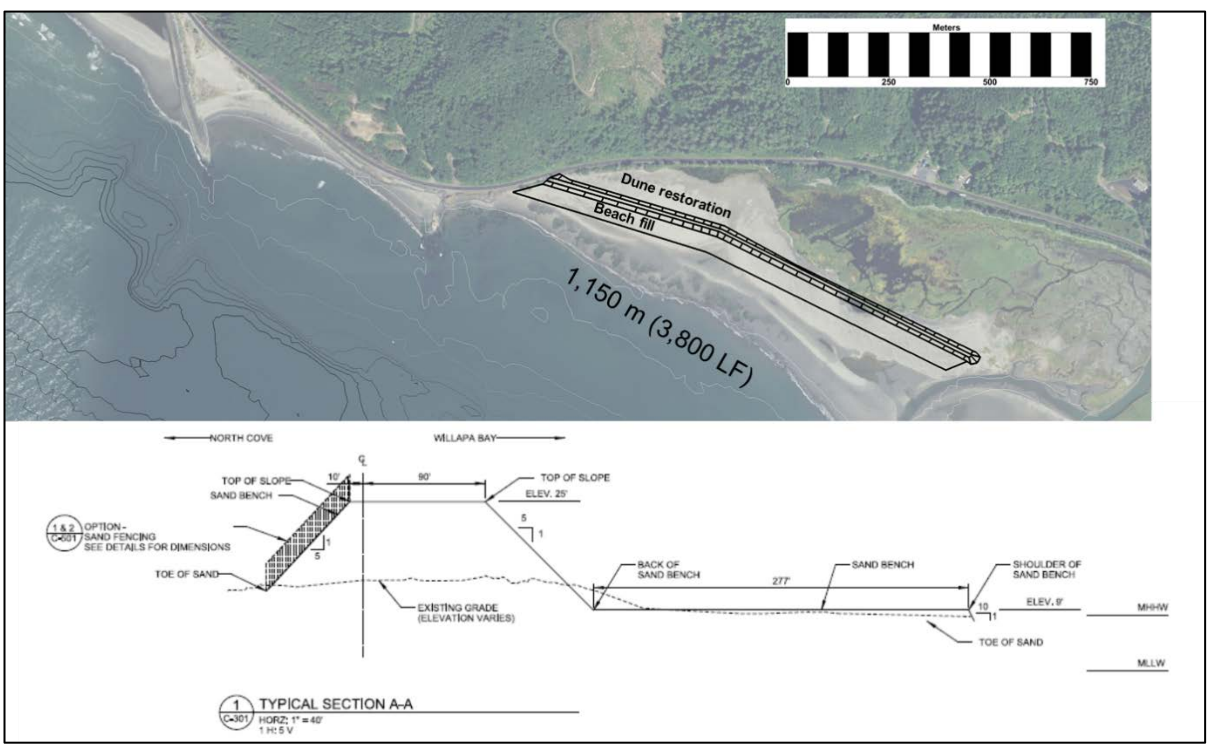


Figure 43. Alternative 3a. Dune restoration with periodic beach nourishment

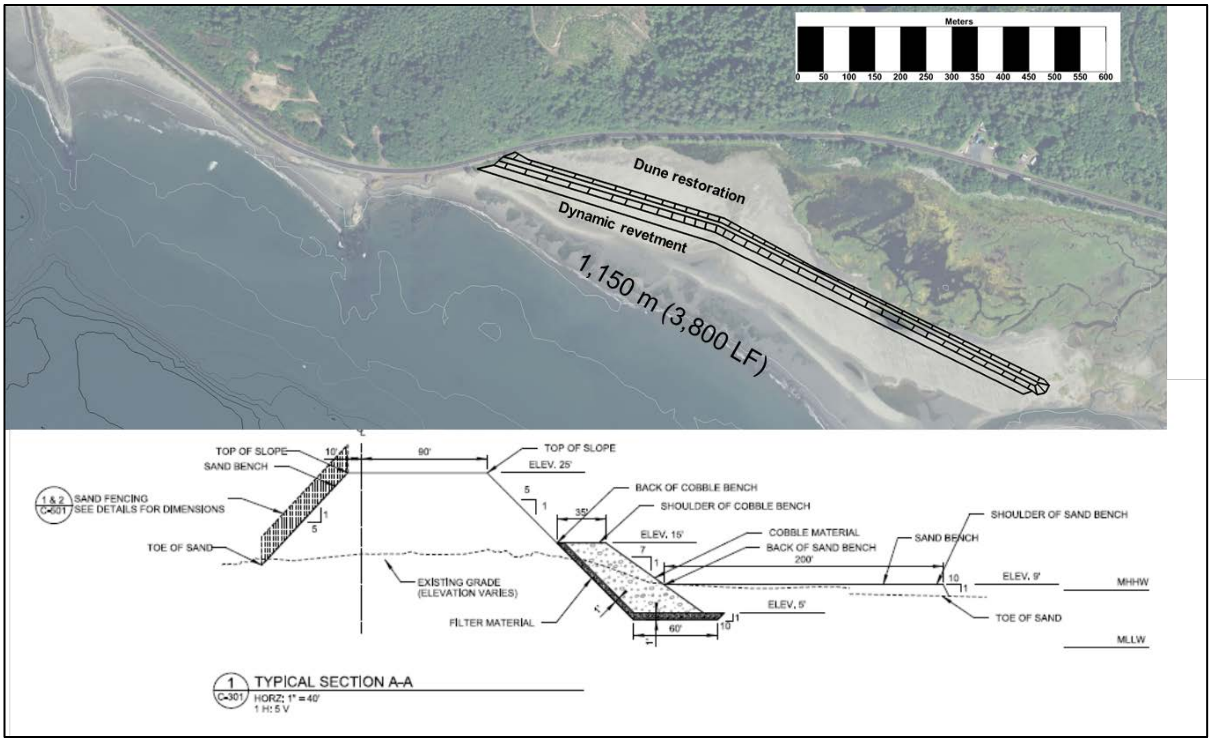


Figure 44. Alternative 3b. Dune restoration with dynamic revetment

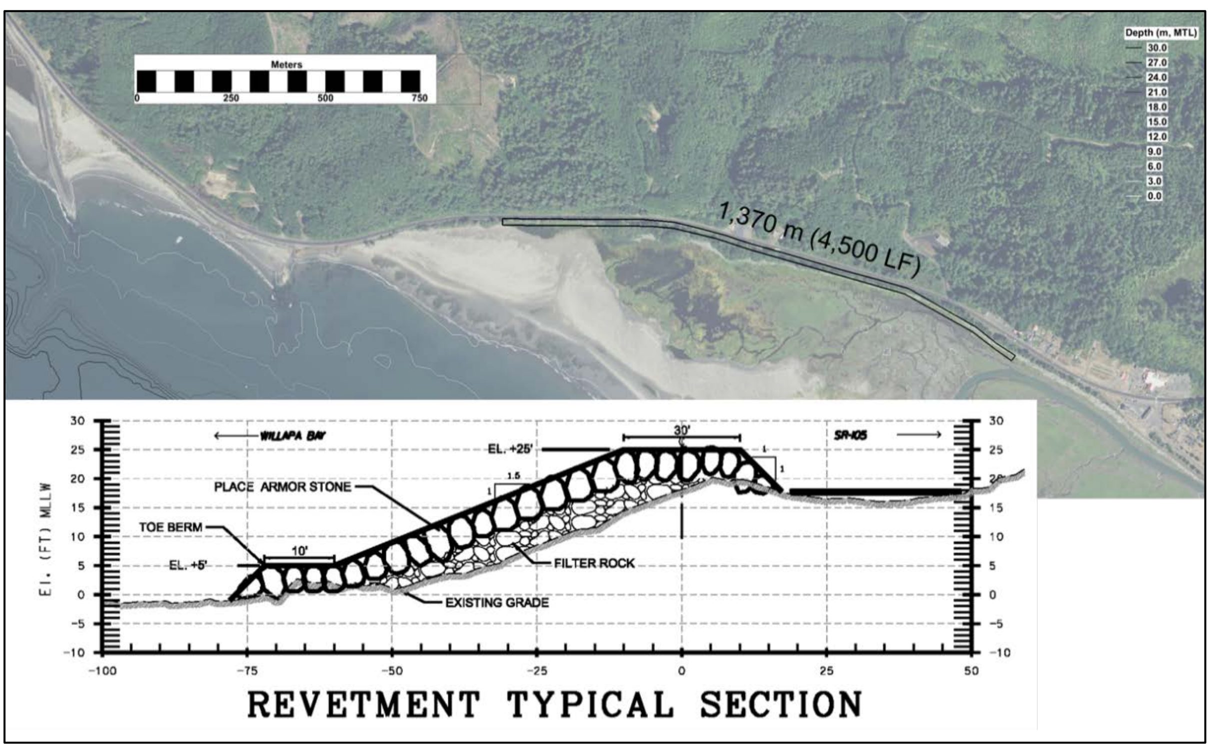


Figure 45. Alternative 3c. Roadway embankment extension

12. Alternative analysis

The performance of each of the alternatives are evaluated by investigating changes in wave height, water level, currents, and morphological response. In order to investigate long-term shoreline change where wave driven transport is the primary mechanism for sediment transport the GENCADE one-line model is used.

The new training dike alternatives show that the structure alters the channel thalweg scouring further offshore near Tamarack Street. As a result, localized erosion in this area has potential to be reduced. The trend of sediment bypassing from the offshore shoals to the North Beach Spit continues to occur, however the results indicate more material is retained in the shoals (Figure 46). Since a new training dike would impeded alongshore transport to Reaches 2 and 3, Alternative 1b was also simulated to determine the sill effectiveness. The results indicate little difference in nearshore evolution indicating the opening would not create significant tidal current flow through the sill (Figure 47).

A dynamic revetment in Reach 2 would reduce runup of the existing dune face and reduce bank erosion. Applying the compute nearshore wave height of Hs = 1.5 m and peak period of T = 16.7 seconds, the Battjes 1974 relationship for wave runup ($R_{u2\%}$) on a cobble beach computes to 1.5 m with a permeability coefficient (Cr) specified as 0.55 for cobble sized sediment according to experimental data.

$$R_{u2\%} = C_r\sqrt{\frac{g}{2\pi}}T\sqrt{H_s}(tan\theta) \qquad (2)$$

Permeability coefficients on sand slopes are on the order of 1 to 2. Therefore, a dynamic revetment can significantly reduce the wave runup by 2 to 4 times subsequent backshore erosion during storm events.

Bypassing sediment through notching the SR-105 groin or actively moving material hydraulically would provide effective nourishment to Cape Shoalwater and the existing WSDOT shoreline protection located downdrift. Shoreline change model results indicate removing the longshore barrier would allow the downdrift shoreline to accrete over time (Figure 48). This could be a long-term O&M benefit to WSDOT to avoid costly repairs to the roadway embankment that were required in 2017.

Rebuilding the barrier dune on Graveyard Spit would reduce the wave height and wave runup, which causes debris and flooding to occur on SR-105 between the groin and the Shoalwater Reservation. Model results indicate total water levels can be reduced below the current road elevation during the December 10, 2015 storm event (Figure 49).

Finally, the last resort of extending the roadway embankment along SR-105 all the way to the Shoalwater Reservation is evaluated. While feasible, this presents a long-term O&M problem as the Graveyard Spit will continue to move landward over time and likely still result in direct wave attack on the embankment. This would result in periodic O&M of the revetment and continued debris management.

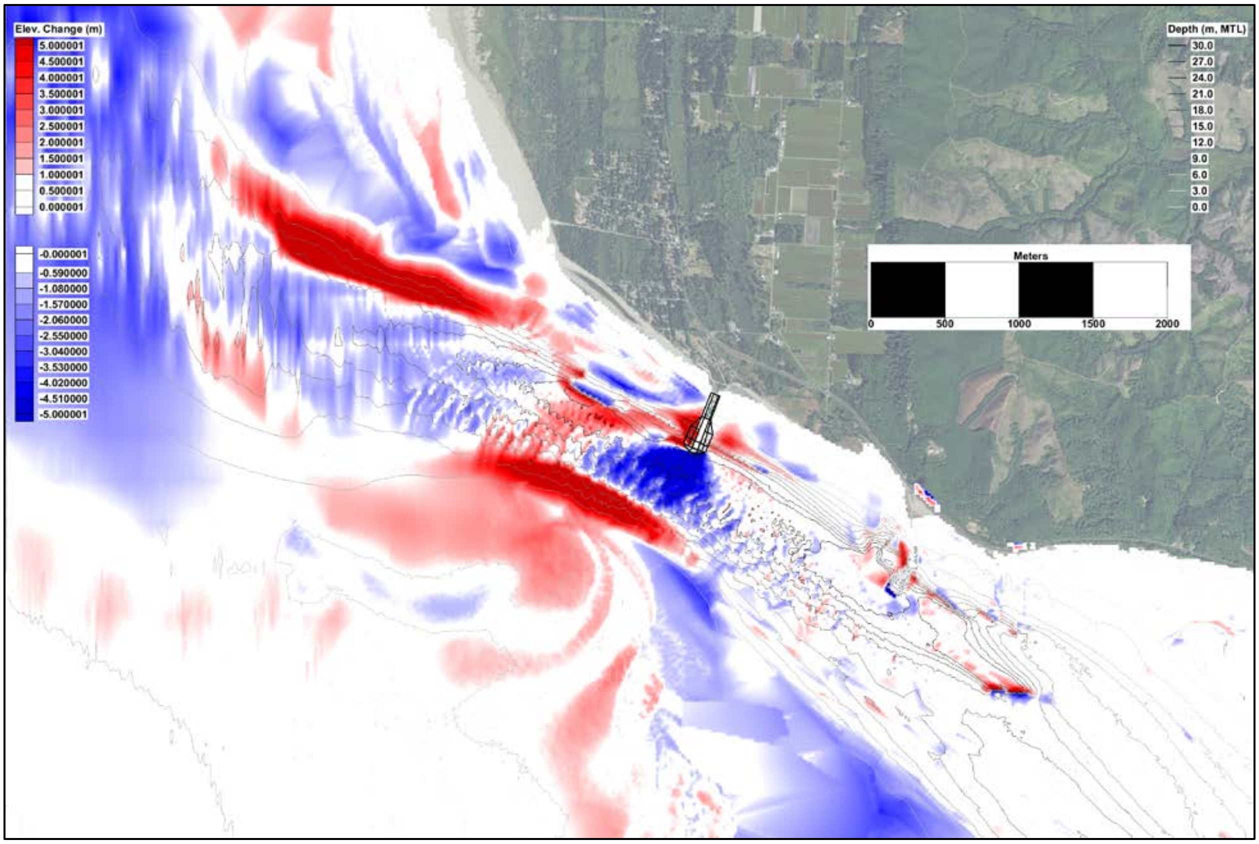


Figure 46. Calculated seabed change for December 2015 with Alternative 1a (new training dike no sill).

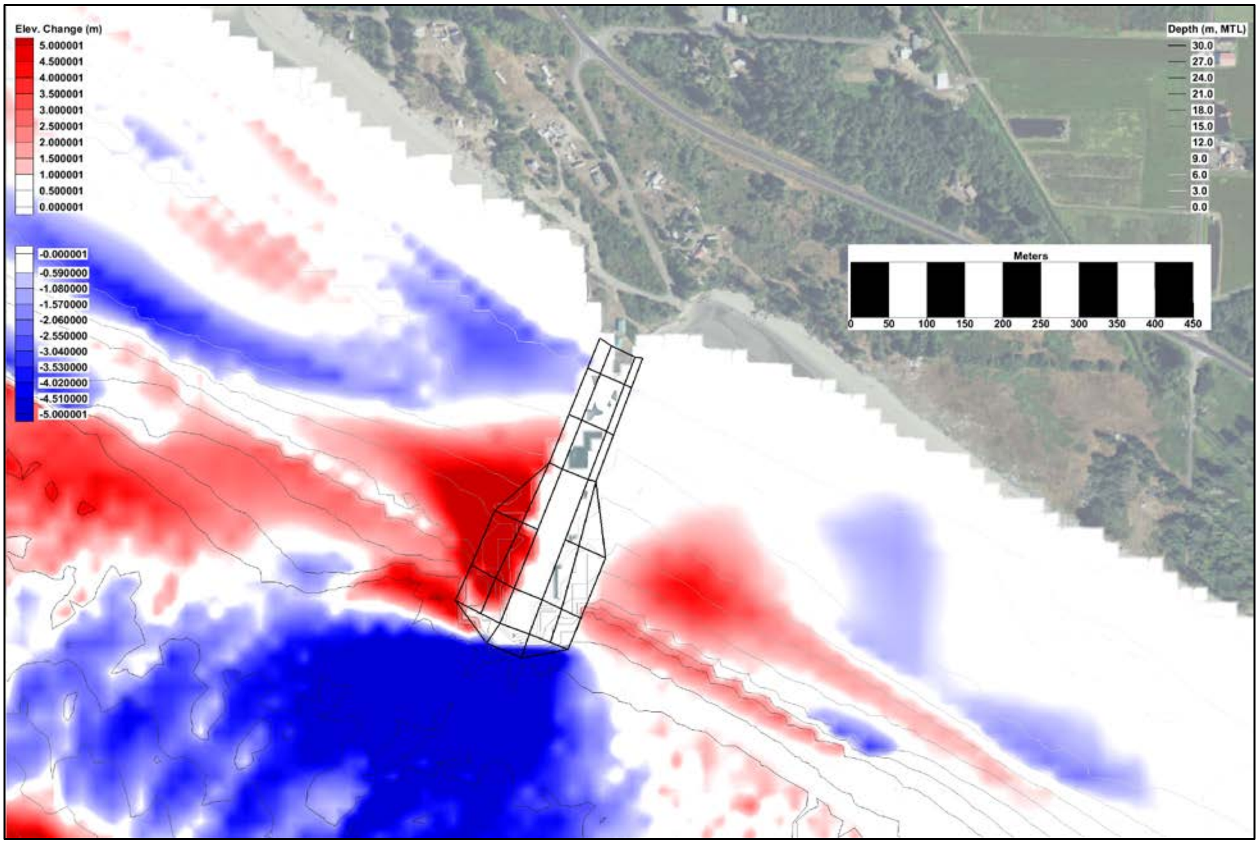


Figure 47. Calculated seabed change for December 2015 with Alternative 1b (new training dike with sill).

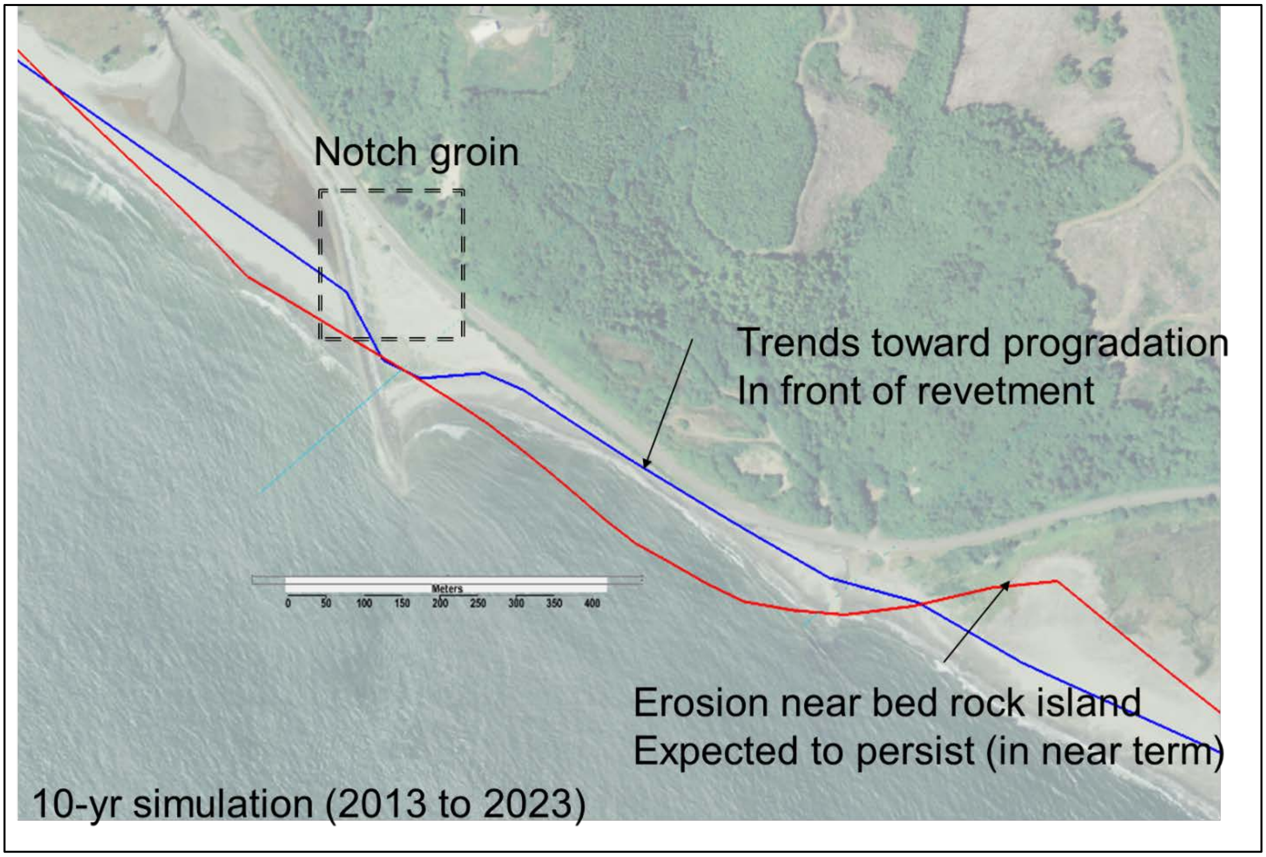


Figure 48. Calculated shoreline change from 2013 to 2023 after implementing a notch in the SR-105 groin/dike.

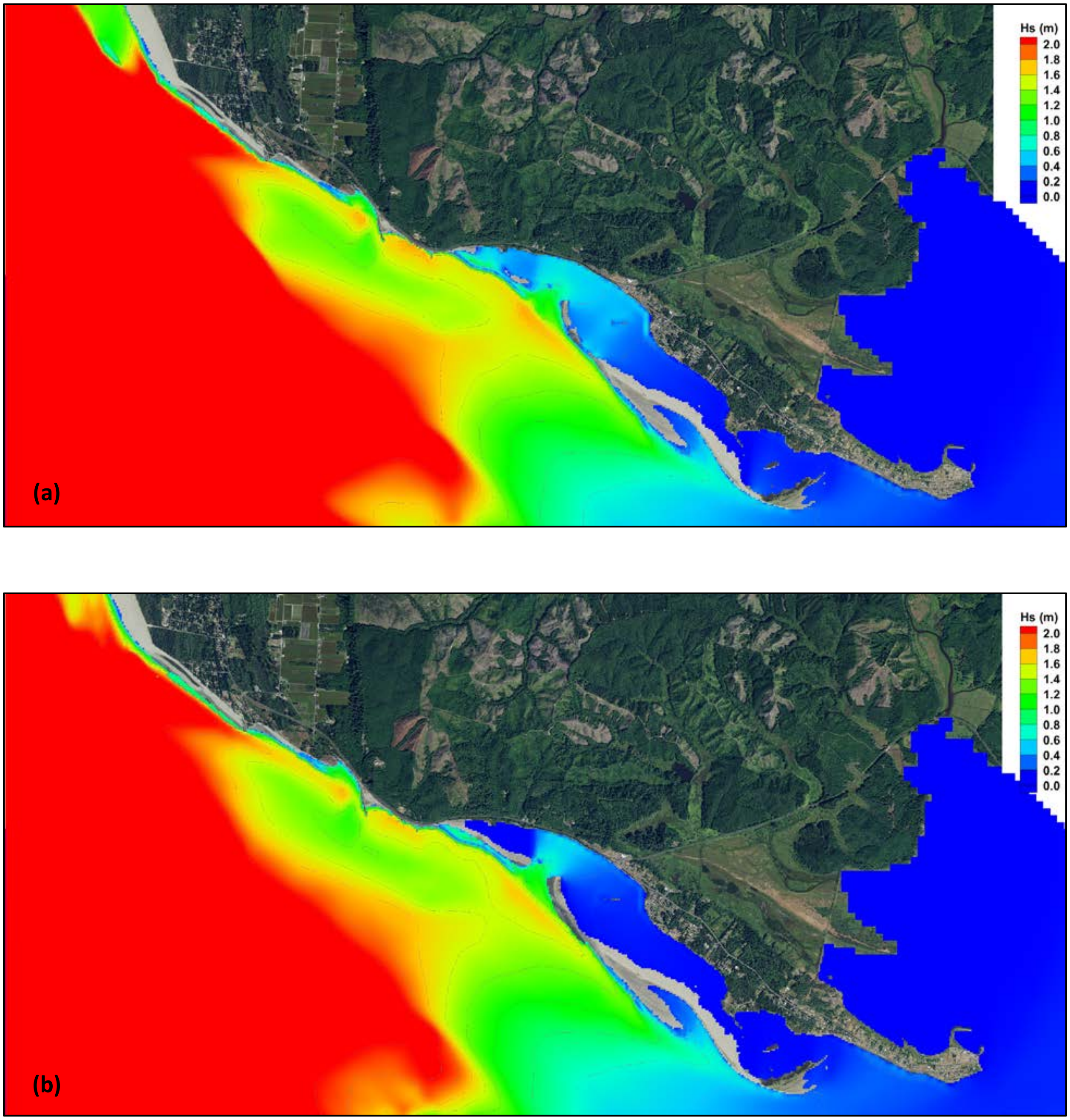


Figure 49. Reduction in wave height through dune restoration of Graveyard Spit (a) Before (b) After

13. Project Cost

In order to compare the merits of implementing each alternative a rough order of magnitude cost estimate is performed for each alternative

Reach 1. City of North Cove - shoreline north of rock protection near Tamarack St.

Assumptions:

- New training dike is approximately 1,300 feet in length. Average crest width is 50 feet.
- Sideslopes are 1:3H.
- Average thickness of armor layer is 20 feet.
- Dike assumes a 5 foot filter layer thickness for bedding material to minimize structure settlement

Table 4. Alternative 1a. Material Quantities for new training dike

	Cross Sectional Area (ft^2)	Length (ft.)	Volume (CY)	Tons/CY	Tons
Training Dike Armor Layer (Full Height)	1900	1,300	91,480	1.4	128,100
Training Dike Filter Layer	850	1,300	40,930	1.5	61,400

Table 5. Alternative 1b. Material Quantities for new Training Dike with sill

	Cross Sectional Area (ft^2)	Length (ft.)	Volume (CY)	Tons/CY	Tons
Training Dike Armor Layer (Full Height)	1900	1,100	77,400	1.4	109,000
Training Dike Armor Layer (Sill Portion)	800	200	6,000	1.4	8,300
Training Dike Filter Layer	850	1,300	41,000	1.5	61,400

Reach 2. Cranberry drainage ditch outfall - Shoreline between Tamarack St. and SR-105 groin and dike.

Assumptions:

- Assume notching a 330 foot length of groin in the swash zone down to +4' MLLW; Existing crest height is currently approx. +16 feet MLLW in this region. The existing groin would excavate the crest down to +4 feet MLLW. Existing stone would be relocated to either side slope.
- Assume maintenance required every 10 years to sill
- Assume dynamic revetment is entrenched to +5' MLLW and has a crest height of +15' MLLW. Berm width is 30 feet. Final sideslope is 1:7H
- Assume 1-foot filter layer beneath the dynamic revetment to minimize settlement
- Assume O&M required every 10 years. Approximately 50% of the cobble and 100% of beach fill

Table 6. Alternative 2a. Dynamic Revetment and notching a sill in SR-105 groin

	Cross Sectional Area (ft^2)	**Length (ft.)**	**Volume (CY)**	**Tons/CY**	**Tons**
Groin Rock Reworking	456	330	5,600	1.4	7,800
Dynamic revetment cover layer	400	3,600	53,300	1.4	74,700
Dynamic revetment filter layer	75.5	3,600	10,100	1.5	15,100
New Groin Rock for O&M	125	400	1,900	1.4	2,600

Table 7. Alternative 2b. Material Quantities for Dynamic Revetment and bypassing of beach fill

	Area (ft^2)	**Length (ft.)**	**Volume (CY)**	**Tons/CY**	**Tons**
Beach Fill	225	1,800	15,000	-	-
Dynamic revetment cover layer	400	3,600	53,300	1.4	74,700
Dynamic revetment filter layer	75.5	3,600	10,100	1.5	15,100

Table 8. Alternative 2c. Material Quantities for Dynamic Revetment and periodic beach fill

	Area (ft^2)	**Length (ft.)**	**Volume (CY)**	**Tons/CY**	**Tons**
Beach Fill	500	1,800	33,333		
Dynamic revetment cover layer	400	3,600	53,333	1.4	74,667
Dynamic revetment filter layer	75.5	3,600	10,067	1.5	15,100

Reach 3. Cape Shoalwater – Barrier spits southeast of SR-105 groin and dike

Assumptions:

- Initial dune nourishment assumes a 100-foot crest width, +25' MLLW crest height, 1:5H sideslopes; Length of dune nourishment = 3,800 feet. Topographic survey from Sep 2017 indicates current Spit height is approximately +10 feet MLLW
- Beach fill berm height of +10' MLLW and width of 200 feet; Average Beach fill thickness is 5 feet
- Assume dynamic revetment is entrenched to +5' MLLW and has a crest height of +15' MLLW. Berm width is 30 feet. Final sideslope is 1:7H
- 1-foot filter layer beneath cobble berm to minimize settlement
- With dune in place barrier spit rollover is halted.
- O&M (sand) is required every 5 years. Approximately 100% of the sand and 25% of dune volume is replaced
- O&M (cobble) is required every 10 years. Approximately 50% of the cobble and 25% of dune volume
- Assume mobilization cost for dune restoration would be shared with USACE project.

Table 9. Alternative 3a - Dune restoration with periodic beach fill

	Cross Sectional Area (ft^2)	**Length (ft.)**	**Volume (CY)**
Dune	2,625	3,800	369,500
Beach	1,000	3,800	140,800

Table 10 Alternative 3b - Dune restoration with dynamic revetment

	Cross Sectional Area (ft^2)	**Length (ft.)**	**Volume (CY)**	**Tons/CY**	**Tons**
Dune	2625	3,800	369,500		
Dynamic revetment cover layer	400	3,800	56,300	1.4	78,800
Dynamic revetment filter layer	75.5	3,800	10,600	1.5	16,000

Table 11. Alternative 3c - Dune restoration with dynamic revetment

	Cross Sectional Area (ft^2)	Length (ft.)	Volume (CY)	Tons/CY	Tons
Revetment	827.5	4,500	138,000	1.4	193,100
Revetment Filter Layer	170	4,500	28,400	1.5	42,500

Tables 4 to 11 provide cost information for each alternative. However, each alternative should be viewed as a modular that can be combined Reach by Reach. For instance, a complete solution that minimizes future residual risk would implement one alternative for Reach 1, 2, and 3. However, applying a cost risk trade-off may focus on implementing an alternative for the Reach with the greatest probability of future consequences. For Instance, combining Alternatives 1b, 2c, and 3b result in the lowest residual risk over the project life cycle but comes in with an average annual cost of $2.16 million. Should it be decided that a moderate risk is tolerable, combining alternatives 1b and 3b would have an annual cost savings of $460,656 (Table 12).

Table 12. Rough Order of Magnitude Costs over 30 year life cycle (Undiscounted)

Alternative	Initial Cost	O&M cost	Total Cost
1a. Reach 1 - New Training Dike	$ 17,850,926	$ 4,002,315	$ 21,853,241
1b. Reach 1 - New Training Dike with sill in swash zone	$ 16,425,000	$ 3,645,833	$ 20,070,833
2a. Reach 2 - Construct Dynamic Revetment and notching of existing SR-105 groin to create sill	$ 5,445,107	$ 8,292,222	$ 13,737,329
2b. Reach 2 - Construct Dynamic Revetment and bypassing of impounded sediment north of groin	$ 5,333,000	$ 9,120,000	$ 14,453,000
2c. Reach 2 - Construct Dynamic Revetment and periodic beach fill downdrift of SR-105 groin	$ 5,474,667	$ 8,345,000	$ 13,819,667
3a. Reach 3 - Dune restoration with periodic beach fill	$ 10,290,509	$ 34,727,431	$ 45,017,940
3b. Reach 3 - Dune restoration with dynamic revetment	$ 13,210,528	$ 17,595,938	$ 30,806,465
3c. Reach 3 - Extend existing roadway embankment toward Shoalwater Reservation	$ 25,410,417	$ 6,033,854	$ 31,444,271
Residual Risk	**Alternative Combination**	**Total Cost**	**Average annual cost**
Lowest	1b + 2c + 3b	$ 64,696,965	$ 2,156,566
Low	1b + 2a + 3b	$ 64,614,628	$ 2,153,821
Moderate	1b + 3b	$ 50,877,299	$ 1,695,910
High	3c	$ 31,444,271	$ 1,048,142
Highest	none	$ -	$ -

14. Conclusions and Recommendations

This study focused on evaluating the feasibility of a suite of alternatives to mitigate the long-term risk to the SR-105 roadway between North Cove and Tokeland, WA on the northern shoreline of Willapa Bay. An analysis of historic engineering activities and harbor morphology was performed. A sediment budget analysis indicates that the Willapa Bay inlet has a net export of sediment of over 5 MCY a year to the shorelines north of North Cove on Grayland Plains. This is driven primarily by the North Entrance Channel migration, which bypasses sediment from the offshore shoals across the channel to the North Beach Spit and Grayland Plains littoral sub-cell.

The shoreline geology was mapped with the best available data including local well logs and aquifer studies conducted for the Shoalwater Tribe. Geologic stratigraphy was reviewed in context with observations in the SR-105 corridor. Analysis suggests that erosion resistant Pleistocene outcrops may exist in regions north of the SR-105 groin and may affect future shoreline response. Additionally, evidence of basement geology exposed in the subtidal regions offshore may be a driver for halting future North Entrance Channel migration. Collectively this evidence suggests a more robust shoreface geology mapping program would provide valuable information for any future engineering design alternatives.

A 2D numerical model with sediment transport was employed to validate the coastal processes using a representative 1-month long simulation from December 2015 and August 2016 respectively. Model results confirmed the observed sediment transport trends and replicated the complex current patterns near the North Beach Spit, which can produce opposing currents between the nearshore and North Entrance Channel thalweg.

Observations and numerical model results were utilized to help frame a conceptual model for the project area. The area was separated into three distinct reaches and defined by the primary divers controlling sediment transport. Reach 1 extending from Tamarack Street northwards, Reach 2 between Tamarack Street and the SR-105 groin, and Reach 3 between the SR-105 groin and the Shoalwater Tribal Reservation in Tokeland, WA.

A series of engineering alternatives were evaluated in each of these Reaches with the 2D sediment transport model and shoreline change model. Analysis suggests that construction of a new training dike near the residential shore protection near Tamarack Street could help mitigate near shore erosion between Tamarack and Warrenton-Cannery Road by altering the sediment bypassing patterns of the offshore Shoals to the North Beach Spit region. The analysis also suggests that modifying existing longshore barriers such as the SR-105 groin could result in positive benefits for the shoreline in Reach 3, which is currently the most invested stretch of shoreline protection by WSDOT with almost 1 mile of roadway embankment.

Finally rough order of magnitude cost estimates were performed on a number of alternatives investigated in each Reach. A basic cost-risk analysis was included to highlight the merits of combining actions in a single Reach or all Reaches. This provided information on what actions may provide maximal risk reduction. Next steps for the study should focus on pursuing more detailed information regarding shoreline geology, prior to investing a significant amount of resources on engineering solutions. This information will help guide engineering design from feasibility level to a detailed design where accurate cost estimates can be performed suitable for construction level planning.

It is recommended that WSDOT also coordinate with Pacific County on a possible USACE Section 103 Continuing Authorities Program project study that has recently been approved for a new start in Federal Fiscal Year 2019. This program could be used as an avenue to receive federal funding to implement a future engineering solution, which could benefit both the City, County, and State.

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