NESKOWIN SHORELINE ASSESSMENT

Coastal Engineering Analysis of Existing and Proposed Shoreline Protective Structures

Prepared for The Neskowin Hazards Committee

March 12th, 2013

High surf during high tide at Neskowin on Jan 9th, 2008.

Source: Armand Thibault (Published in NRC, 2012).

ESA PWA

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Introduction

The purpose of this project is to provide technical and engineering analysis to the Neskowin Coastal Hazard Committee (NCHC) to evaluate various structural alternatives that reduce threats to upland development while maintaining a beach. The project goal is to provide an objective engineering analysis that will provide the community with additional information that they can use to make decisions about how to contend with the current erosion. For purposes of this study, the NCHC requested cost estimates and results to examine potential changes over the next 15 years. ESA PWA has recommended looking out to at least 2050 when various sea level rise (SLR) estimates begin to diverge dramatically.

The specific objectives of this study are to examine a range of alternatives to mitigate erosion in a variety of ways. The first objective is to evaluate the effectiveness of each method at protecting upland properties and maintaining a beach. The second objective is to provide conceptual level cost estimates for various erosion mitigation strategies and lifecycle maintenance costs. The third and final objective is to consider innovative options to mitigate the erosion at Neskowin.

Authors

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Background

There has been a lot of work done by various scientific experts examining the erosion at Neskowin (Figure 1), most notably led by Jonathan Allan of Oregon Department of Geology and Mineral Industries (DOGAMI) and Peter Ruggiero at Oregon State University (OSU). Both researchers have provided a tremendous amount of data and insight from their research efforts, graduate students and expertise.

The intent of this report is not to synthesis this information, but to incorporate those research findings and data sets into the engineering analyses. However, ESA PWA does feel that it is important to document some of the key events and processes that have occurred since they provide a context that should be considered in addition to the engineering analyses when making community management decisions.

Causes of Erosion and Damages

From review of the literature, there appear to be several causes for the erosion at Neskowin: total water levels, littoral cell wide reorientations related to El Niño, rip embayments, and structural effects. The actual cause of the damages is the wave exposure which has increased as the dissipative effects of the beach have diminished with decreasing beach width. Total water level is one measure of wave damage and flood potential which has been reported in the recent literature. Total water level (TWL) is a combination of tides, surge (e.g. El Niño related), dynamic wave setup, wave run-up, and sea level rise (Figure 2a). The high energy Oregon coast typically experiences high TWL on an annual event (>5.5 meters NAVD), however upland development at Neskowin was not subject to significant damage until the 1997-98 El Niño occurred. During the 1997-98 El Niño the south end of the Neskowin littoral cell was starved of sediment as part of the El Niño pattern of littoral cell wide reorientations (Komar, 1997) (which also occurred in the Netarts and Beverly littoral cells). Typically, the littoral cell would rotate back to the south and the beach would subsequently recover. However, this expected recovery has not happened in Neskowin.

Two theories exist as to why this counter rotation has not occurred:

1. The lack of recovery to the south end is possibly related to changes in wave direction influenced by the Pacific Decadal Oscillation (PDO). The PDO is a 20-30 year climate cycle which affects the north - south location of the jet stream and thus the wave-generating storm tracks. A more southerly shift in the wave direction is consistent with the current phase of the PDO (NOAA 2011). This is consistent with the short term shoreline change rates shown in Figure 3 (Ruggiero et al in press). If this theory is correct, then the implication is that the sand north in the system would eventually return to the south end of the littoral cell reducing the levels of erosion and the extent of damages to structures.

Implication if correct: Any strategies implemented to mitigate erosion must not curtail beach recovery by reducing the ability of sand to migrate to the south.

2. The second theory for the continuing erosion at Neskowin is that the impact of several large storm seasons in the past decade (1997, 1998, 1999, 2004, 2006, 2010), have moved substantial amounts of sediment offshore beyond the depth that it is actively moved by wave energy except during the largest storm events. Significant accretion shown in the Neskowin profiles near the mouth of the Nestucca River may provide some evidence that this has occurred (Figure 3).

   Implication if correct: This ebb shoal or offshore location of this sediment may be a suitable source for acquiring beach sand for nourishment in the south end.

Rip embayments, localized areas of erosion (200+ yards wide) that migrate along the beach, are another cause for the erosion and damages in Neskowin. These embayments scour sand (up to 9' from certain sections of beach) and reduce wave breaking, enabling larger wave impacts at the shoreline. Rip embayments typically develop near the north side of Proposal Rock, perhaps either due to creek discharge, lowered beach conditions, or reflected wave energy from the island (Jonathan Allan, personal communication). This rip embayment typically migrates northward with the southwesterly direction of the winter waves and focuses the erosion on or around Pacific Sands (Figure 1).

Implication: The engineering implication of this rip embayment generation zone is that altering this generation zone or stabilizing this rip embayment may reduce the armoring required to protect the upland properties north of Proposal Rock.

The final cause of damages tends to be associated with failures of the shoreline armoring structures. There are several aspects of the existing armoring which may be contributing to the erosion damages. First, the revetment structures have a tendency to settle due to several factors including sand fluidization, scour at the toe from active reflection off of the structure, and the depth to bedrock in the north of the Proposal Rock portion of the study area. An examination of the nearest groundwater wells from the Oregon Water Resources Department in the vicinity of Neskowin (Table 1) show that the depth to bedrock is well below the elevation that any of these structures are likely to be built. Another exacerbating factor to structural damages is the volume of water that overtops the revetments. This volume can saturate soils and the revetment, weakening the structural integrity and fluidizing the sand behind the structure. Such an example occurred on January 5, 2008, overtopping wave water volumes saturated dune sands and contributed to a mass failure of the structure similar to a landslide (Figure 2b, NCHC, personal communication).

Table 1: Groundwater Well Logs Nearest to Neskowin with Depth to Bedrock

Well Log	Address	Depth to Rock	Rock type
Till 1111	47405 Highway101	60'	Basalt
Till 52053	43505 Aeolian Way Loop Rd	31'	Sandstone
Till 51161	NE of Fire Hall	53'	Basalt

The revetments also have a much steeper slope than a natural beach, which contributes to elevating the wave run up elevations. The TWL calculations completed by Baron and Ruggiero for this region have utilized the empirical parameterization of wave run-up of Stockdon (2006). This Stockdon run-up equation, which integrates dynamic wave set up (an integral part to TWL on these high energy beaches), is based on wave run-up on a natural foreshore slope. This application of the Stockdon wave run-up equation to steep slopes is inappropriate and contributes to an underestimate of potential wave run up elevations and storm wave damages. (See methods and results section for composite slope method)

Implication: TWL and damage assessments need to utilize a more appropriate run-up equation.

While it is clear that sea level rise is occurring and will continue to occur for centuries with an increase in damaging coastal events, the positive indication identified by the National Research Council in a recently released report is that the tectonic uplift found along the Oregon coast will not significantly exacerbate current coastal processes. Projections of sea level rise by 2030 are 6.8cm (2.6 inches) and by 2050 17.2cm (6.7 inches) (NRC 2012). Note that the analysis conducted in this report did not factor in sea level rise or other climate change impacts.

Methods

Methods and Approach

- TWL calculation on a composite slope using traditional wave run-up equations. Calculation of run-up on a composite slope will involve estimating the dynamic wave setup and using a depth limited wave height at the toe of the structure.
- Calculate current overtopping potential based on negative freeboard (TWL elevation – structural crest elevation)
- Considerations of additional engineering of toe of structure to minimize settling
- Physical BEACH 10 modeling plotting Beach Width vs. Upland Erosion
- Calculate historic damages to structures based on storm wave events and permit database
- Cost estimates for lifecycle maintenance cost of current structures to 2025 and 2050
- Cost estimates for bolstering of existing structure with additional revetment volume
- Cost estimates for other alternatives and structures

Data Sets Used

Several data sets were acquired from various researchers and project partners. The key data sets used are discussed briefly below.

Waves – A composite time series of waves recorded at northwest buoys was acquired from Dr. Peter Ruggiero (Figure 4, Harris, 2011). This recorded time series was used to generate a future synthetic time series used in the engineering analysis. Additional information on this data set can be found in Harris 2011.

*Coastal Geomorpholog*y – Beach topographic and nearshore profile data was collected by Jonathan Allan at DOGAMI and Dr. Peter Ruggiero at Oregon State. Beach profiles were collected using survey grade GPS equipment. For more details see Allan and Hart, in review. Additional elevation data was extracted from topographic LIDAR collected by the Oregon Department of Geology and Mineral Industries in 2009 (available on the NOAA Digital Coast website).

Future TWL – Projections of existing and future water levels using the wave time series discussed above was completed by Heather Baron (2011) and provided to ESA PWA for use in this project.

Shoreline Change – Short and long term shoreline change rates were provided by Dr. Peter Ruggiero under contract with the United States Geological Survey as part of the National Assessment of Shoreline Change in the Pacific Northwest. For more details see Ruggiero et al in review (Figure 3).

Coastal Structure Physical Conditions Inventory and Permit Database – Structural conditions along the Neskowin Shoreline were surveyed in the field by Tony Stein at Oregon Parks and Recreation Department. Summary tables were examined in the Neskowin Coastal Erosion Adaptation Plan (Tillamook County, 2012). Additionally, OPRD provided a subset of the coastal armoring tracking database which contained specific dates, volumes and lengths of revetment alterations associated with permits in the Neskowin region.

Coastal Structure Elevation information – Structural characteristics were collected from available LIDAR surveys and ground-truthed with DOGAMI survey data to provide a quantitative understanding of the exposure faced by the community to existing and future hazards (Figure 5).

Engineering unit costs and designs – For more details on the unit cost, volumes and assumptions made in the cost estimating, please see Appendix 1.

Erosion Mitigation Strategies for Analyses

The following mitigation strategies were considered for analysis after consultation with the community and previous experience. Upon further review of the site and literature, several of these were selected for more detailed analysis including beach width and upland property effectiveness (BEACH10) and cost estimating.

Those measures selected for detail assessment included:

- Managed retreat – assumes natural erosion
- Altered revetment –concrete cap wall
- Altered revetment –with additional rock revetment cap (Figure 12)
- Structural adaptation – elevate structures on piles to existing floor elevation +10'
- Seawall with return - (e.g., O'Shannessy Seawall, Figure 6)
- Beach nourishment
- Nearshore breakwater (Figure 7)
- Low crested structure or groin to stabilize migrating embayments (only cost estimating)

Analysis rejected for detailed analysis (see discussion in Innovative options):

- Wave tripping structure on the beach (e.g., Taraval wall, Figure 8)

Coastal Structure Elevation Information

To assess the volumes of material needed to elevate the crest of existing structures to deal with existing hazards and rising sea levels, and to understand the existing site characteristics needed to drive the TWL wave run-up analysis, a detailed inventory of existing elevation information on the structures was conducted (Figure 5). This analysis involved extracting the crest elevation from the 2009 LIDAR and then fact checking the crest elevations using the DOGAMI profiles provided by Dr. Jonathan Allan. Results from this analysis are shown in Figure 5.

Composite Slope Wave Run-up Analysis

To assess the required changes to the existing structures and understand the volumes and materials necessary to provide conceptual cost estimates it was important to develop a TWL calculation that was consistent with observations. To do this, wave run-up on a composite slope was modeled using a computer program developed by ESA PWA. The program uses several published methods to assess the extent of wave run-up on beaches and shores with irregular topography and surface conditions. Wave run-up is computed using the method of Hunt (1959) which is based on the Irribarren number (also called the Surf Similarity Parameter), a non-dimensional ratio of shore steepness relative to wave steepness. The program also uses the Direct Integration Method (DIM) to estimate the static and dynamic wave setup and resulting water surface profile (FEMA 2005; Dean and Bender 2006; Stockdon 2006). The methodology is consistent with the FEMA Guidelines for Pacific Coastal Flood Studies for barrier shores, where wave setup from larger waves breaking farther offshore, and wave run-up directly on barriers combine to form the highest total water level and define the flood risk (FEMA 2005). This program also incorporates surface roughness of the structure which acts as friction on the uprush of the waves and uses a composite slope technique as outlined in the Shore Protection Manual (SPM; USACE 1984) and Coastal Engineering Manual (CEM; USACE 2002).

Two cross-shore profiles (called “North” and “South”) were used to estimate the wave run-up at Neskowin, as shown on Figure 9 (Location shown in Figure 1). These profiles were based on nearshore bathymetry and beach surveys collected by the State of Oregon in February 2012 (topography), September 2011 (bathymetry), and LIDAR flown in 2009.

Water levels for the analysis were taken from the nearby Yaquina Bay tide gauge operated by NOAA (ID #9435380). Wave data were taken primarily from the nearby Stonewall Banks, Tillamook, and Washington wave buoys operated by the NOAA (ID # 46050, #46089, and #46005, respectively). The resulting time series of tides, wave periods, and wave heights are shown in Figure 4 (Harris 2011, courtesy of Ruggiero).

Verification of Wave Run-up Model

The run-up calculations were compared to observations of wave run-up provided by the Neskowin community. During a high run-up event in January 9, 2008, wave run-up in excess of 34 feet was observed in the vicinity of Profile “North” at the Pacific Sands Condominiums (Figure 2; personal communication NCHC, Jonathan Allan). The revetment crest in the vicinity is approximately 28 feet in elevation, indicating that the wave run-up overtopped the revetment crest. The wave run-up calculations for this date and location yielded a run-up elevation of 36 feet, as shown in Table 2, with the note “Max R elev.”. The “Max inland limit” indicates the elevation of the landward limit of wave run-up, after overtopping. This calculation indicates that the run-up would extend approximately 200 feet farther inland if not obstructed, which corresponds to a lower elevation on the land side of the development.

Table 2 shows selected wave run-up calculations for January 9, 2009. Visual observations indicate run-up extended to about elevation 33’ at Neskowin, based on our interpretation of the information provided to us by members of the community. These calculations utilize the North Profile.

Table 2: Wave Run-up Calculations for January 9, 2009

Inputs		
SWL(ft)	Hs(ft)	T(s)
9.2	21.8	13.8

Outputs								
DWL (ft)	Hb (ft)	T (sec)	Zb (ft NAVD)	x@Zb (ft)	slope	x-run (ft)	R (ft NAVD)	Notes
14.08	4.26	12.9	8.62	365	0.205	476	32.3	Test run with second highest total water level, one hour earlier than maximum.
14.85	4.86	13.8	8.62	365	0.218	488	36.3	Test run with maximum total water level elevation.

The values in the table are:

DWL = Dynamic Water Level: This is the dynamic wave setup, estimated to be exceeded about 2% of the time.
Hb = Height of the breaking wave that drove the highest total water level
T = Wave period
Zb = the elevation of the bed at the location of Hb. Note that the depth is the DWL minus this bed elevation.
x@Zb = the horizontal coordinate of the breaking location
slope = the composite (average) slope
x-run = the horizontal coordinate of the limit of wave runup
R = the calculated wave runup in terms of the total water level elevation.

The difference (3 feet) indicates the calculation exceeds the observation. However, we believe that this difference is acceptable and verifies that the methodology is sound. It is important to consider that the run-up calculations provide the potential elevation that the run-up would extend if the revetment slope extended high enough. In reality, the wave run-up exceeds the crest of the revetment and the run-up extends inland instead of upward. It is unusual for overtopping to extend contiguously (vs. splash and spray) to an elevation more than about 5' above the crest of a revetment, because the wave momentum rushes inland as a bore. Splash and spray overtopping can take a projectile –like trajectory. These concepts are shown schematically in the Figure 10 (FEMA, 2005). This is consistent with anecdotal observations from Pete Owston as he and his wife were swept off their feet during this calibration event. Therefore we expect the potential run-up elevation to exceed the observed elevation for these overtopping conditions. Also, the calculated wave run-up parameter is called the "2% exceedance" which means that it is the value that is exceeded by only 2% of the individual run-up pulses in an event (1 out of 50 waves) and potentially comparable or greater than that associated with an observed maximum. In summary, we believe that the run-up calculation method is verified to provide reasonable results which may be a bit conservative (calculated higher than actual). Also, wave run-up calculations are not considered to have high accuracy owing to the complex hydrodynamics and empirical basis for these run-up equations.

Application of Composite Slope Wave Run-up Model

There were three primary purposes for the use of the composite slope wave run-up model. The first reason was to calculate the elevations that structural alterations needed to reach to reduce the risk of upland property damages. Secondly, this method was used to calibrate the historic damages to structures based on recorded TWL and project those future conditions to assess future damages using standard CEM practices. Once the wave run-up methodology is verified, run-up time series can be developed using existing wave and water level data. Once these time series are completed, the extreme total water level values can be identified along with their recurrence frequency (e.g. exceed once in 100 years or other time frame).

Table 3 below shows the comparison of the Baron 2011 TWL calculations using the Stockdon formulation with the composite slope method. The January 9, 2008 run-up observations and our composite slope calculations indicate that total water levels in excess of 10 meters are likely to occur more frequently than once every 100 years. Therefore, the total water level values based on Stockdon (Table 3) are too low by a significant amount (at least 3 meters (10 feet) and would not be causing impacts to the homes. However, these lower run-up elevations are also indicative of what may occur if a natural profile forms, which would require either the erosion of the dunes or the widening of the beach by several hundred feet.

Table 3: Future 100-year Total Water Levels using Stockdon Formula (Baron, 2011)

	100 yr TWL (Baron, 2011) *feet NAVD88*	100 yr TWL (current study) *feet NAVD88*
Method	Stockdon, 2006	Composite Slope Method
Present (2009 to 2010)	22.3 +/- 1.1	57.7
2025	22.3 +/- 1.1	N/A
2050	23.0 +/- 1.2	N/A

Note: The 100-year total water level was only estimated for the present. The future 100-year total water level is expected to increase with sea level rise.

BEACH10 Modeling

One of the key project objectives was to evaluate effectiveness of the erosion mitigation strategies at protecting upland and maintaining a beach. To do this, ESA PWA utilized BEACH10, a simple shore profile evolution model that tracks changes to beach dry sand widths (assumed to be between Mean High Water (MHW) and the toe of the revetments) and then compares beach width with changes to upland over time (Figure 11).

To run the BEACH10 model, two input parameters are required – initial beach width and upland width conditions and the historic erosion rates. To identify the beach widths necessary to initialize the model, ESA PWA used the profile #262 located just south of Proposal Rock (Figure 1). This led to an initial beach width and upland distance of 250 feet. To drive the erosion, ESA PWA utilized erosion rates identified in Ruggiero et al unpublished (Table 4). However, due to the uncertainties in littoral cell rotation and the alongshore variability in beach width conditions, 3 separate BEACH10 model runs were conducted.

- Model Run 1 – 250’ beach width, short term erosion rate (to account for changes since 1997)
- Model Run 2 – 250’ beach width, long term erosion rate (to account for a counter rotation in littoral cell)
- Model Run 3 – 100’ beach width, short term erosion rate (existing condition at portions of north end)

Table 4: Historic Erosion Rates (Ruggiero et al, unpublished)

	Short Term Erosion Rates *(feet/year)*	Long term Erosion Rates *(feet/year)*
South End Armored	6.0	1.13
North End Armored*	**6.43**	**1.99**
North End Unarmored	6.99	1.7

** Value used in Beach 10 analyses*

The following assumptions were made for each erosion mitigation strategy in applying the BEACH10 model:

- Managed retreat – rip rap structure is removed and beach width remains constant and upland distance is impacted at erosion rate.
- Altered revetments – assumes placement loss due to footprint of structure is 40’.
- Structural adaptation – same as managed retreat.
- Seawall with return – assumes placement loss due to footprint of structure is 10’.
- Beach nourishment – assumes widens beach by 100’ initially then background erosion rate (ignores diffusion) but that the existing structures remain so upland erosion doesn’t occur.
- Nearshore breakwater - assumes widens beach by 100’ initially then reduces erosion rate to 1/3.
- Low crested structure or groin to stabilize migrating embayments - not completed – needs more sophisticated modeling approach if deemed appropriate by the community.

Cost Estimates

For planning purposes, ESA PWA has provided order of magnitude cost estimates to allow cost comparison of alternatives (Table 10). These cost estimates are intended to provide an approximation of total project costs appropriate for the conceptual level of design. These cost estimates are considered to be approximately -30% to +50% accurate, and include a 35% contingency to account for project uncertainties (such as final design, permitting restrictions and bidding climate). These estimates are subject to refinement and revisions as the design is developed in future stages of the project.

This results table does not include estimated project costs for permitting, design, monitoring and maintenance. Estimated costs are presented in 2012 dollars, and would need to be adjusted to account for price escalation for implementation in future years. This opinion of probable construction cost is based on: ESA PWA’s previous experience, bid prices from similar projects, and consultation with contractors and suppliers.

Lifecycle Cost Estimates

Prior damages to the rock revetment were used to estimate the cost of maintaining the shore protection function. Prior damages were estimated based on information in repair permits. The historic repair costs were then estimated using the permit data, presuming that the repair quantities were representative of prior damages. These estimates of historic repairs provide a baseline life cycle cost under existing conditions.

Future damages were estimated based on historic damages increased to account for future sea level rise and potential shore recession. The result is conceptually an increase in water depth at the toe of the structure, and the related increase in depth-limited wave height breaking on the structure. The increase in wave height breaking on the structure was used to prorate existing damages to future conditions.

Historic Events

Historic damage events were evaluated using eleven (11) permits between 1999 and 2008, with permit issue dates in 1999, 2003, 2007 and 2008 (Table 5). The repair volume was compared to the total revetment volume to develop the estimated “percent damage” as defined in the Shore Protection Manuel (USACE, 1984).

Table 5: Selected OPRD Permit Records for Benchmarks of Percent Damages in Neskowin

FID	**OPRD**	Permit Quantities and Structure Parameters							**DATES**	
		HEIGHT (feet)	**WIDTH (feet)**	**ROCK DIAMETER (feet)**	**SLOPE**	**LENGTH (feet)**	**REPAIR LENGTH (feet)**	**REPAIR VOLUME (cubic yards)**	**ISSUED**	**APPLICATION**
9	BA-443-99	14	6	3	1.5	358	85	240	19990225	2/24/1999
15	BA-466-99	14	20	2	1.5	358	75	729	19990806	4/15/1999
3	BA-464-99	14	14	2.5	2	2804	80	800	19991018	3/22/1999
5	BA-464-99	16	27	2.5	2	2804	60	324	19991018	
19	BA-549-02	9	40	3.5	2	2612	99	300	20030115	2002-11-20
55	BA-548-02	9	40	4	2	2612	88	800	20030115	2002-11-20
56	BA-549-02	8	40	3.5	2	2612	120	200	20030115	2002-11-20
57	BA-549-02	8	40	3.5	2	2612	148	240	20030115	2002-11-20
58	BA-549-02	7	40	3.5	2	2612	92	300	20030115	2002-11-20
79	BA 625-07	7-18'	25-35	3-6'	2H:1V	0	*100**	1000	20071025	2007-07-09
75	BA 631-07	8-10'	43-45'	5'	2H:1V	0	*100**	800	20080128	2007-11-28

Note: Italic numbers with (*) are assumed values of REPAIR VOLUME derived from other information in the permit.

A review of the permit dates along with the emergency status of some of the permits resulted in selection of five (5) damage events that were likely responsible for triggering the repair activity. The contribution of other events and long-term degradation may be important but could not be determined based on the limited data. These damage events are characterized in Table 6.

Table 6: Summary of Wave Conditions and Documented Failures for Select Storm Events

Date	Still Water Level (feet)	Wave Height, H_o (feet)	Period, T (sec)	Damage Volumes (cubic yards)	Length repair (feet)
02/16/1999	1.08	36	20	970	160
03/03/1999	4.24	46	17	1120	140
11/08/2002	8.00	26	20	1840	550
02/04/2006	4.22	44	17	1800	200
12/03/2007	4.45	45	16	650	100

Note: The above wave characteristics are the average of the largest TWL events of that winter season (after Sept 15)

The capacity of the revetment was quantified by calculating the wave height that the revetment could withstand using the Hudson Equation (USACE, 1984). Revetment characteristics were estimated based on information provided to us. The estimated “design wave height” is approximately 2.5 to 3 meters (up to 10 feet). Data on revetment performance indicates that impingement of design waves may result up to 5% of the revetment rocks being moved and slightly displaced. For larger wave heights, the percentage of potential damage increases in proportion to the ratio of actual wave height to design wave height (USACE, 1984). We therefore estimated the wave heights that occurred during damage events, calculated the ratio of actual height to design height, and compared the predicted percent damage to the permit-based damage. As shown in Table 7, the predicted (calculated) damages are higher than the actual (permit-based) estimates of damage by up to about 30%. Given the approximate nature of this calculation, it seems that the “calibration” is reasonable and the methodology for historic damages can be used to estimate future damages.

Table 7: Selected Damage Events and Parameters

Permit Data			Damage Event[1]									Accuracy
FID	*Damage Volume (average cy/ft)*	*Percentage Damage*	*Event*	*DATE*	*T (s)*	*H (m)*	*TIDE (m)*	*Hb_toe (m)*	*Hb_toe /HD*	*Percentage Damage*	*Damage Volume (average cy/ft)*	*Percent difference between calculated and actual*
9	6.06	0.54	1	1999-02-16	20	10.90	0.33	4.49	1.83	0.75	8.46	28%
15	6.06	0.54	1	1999-02-16	20	10.90	0.33	4.49	1.83	0.75	8.46	28%
3	8.03	0.71	2	1999-03-03	16.67	14.15	1.291	4.25	1.74	0.71	8.01	0.3%
5	8.03	0.71	2	1999-03-03	16.67	14.15	1.291	4.25	1.74	0.71	8.01	0.3%
19	8.03	0.71	3	2002-11-08 to 09	20	7.99	2.437	3.21	1.31	0.24	2.71	24%
55	8.03	0.71	3	2002-11-08 to 09	16.67	9.04	1.686	3.21	1.31	0.24	2.71	24%
56	8.03	0.71	3	2002-11-08 to 09	20	6.29	3.064	3.21	1.31	0.24	2.71	24%
57	8.03	0.71	3	2002-11-08 to 09	16.67	5.80	2.657	3.21	1.31	0.24	2.71	24%
58	8.03	0.71	3	2002-11-08 to 09	16.67	5.90	2.876	3.21	1.31	0.24	2.71	24%
79	9.00	0.80	4	2006-02-04	17.39	13.32	1.287	4.30	1.76	0.76	8.57	5%
75	9.00	0.80	4	2006-02-04	17.39	13.32	1.287	4.30	1.76	0.76	8.57	5%
72	6.50	0.58	5	2007-12-05	16	13.83	1.355	3.67	1.50	0.45	5.08	28%

[1] The wave and tide conditions for event 3 were peak values selected from the event period. The H_b and percent damage for event 3 are averages from the five peak values.

The calibration was accomplished by considering the mode of failure. It is our understanding that failure has occurred primarily when a rip current has formed and enlarged to the point of scouring deeply at a particular location, causing the rock to settle. We therefore assumed that the beach elevation at the toe of the structure was lowered during the damage event. We used scour equal to the calculated breaking wave height, measured vertically, and assumed a relatively flat slope of 0.002 (1:50). These parameters were selected to bring the calculated damage closer to the permitted repair volumes. Selecting this damage mechanism and multiple parameters required professional engineering judgment limiting the certainty of the analysis.

Future damages were estimated based on assuming a baseline condition and increased damages due to sea level rise and continued shore recession. For the baseline condition, we assumed that areas not yet repaired could be subject to damage, and the damages would occur roughly at the rate that occurred historically between 1999 and 2008. We then looked at the increase in coastal flood hazard associated with an increase in shore recession, and used this increased exposure to prorate the historic baseline damages to an estimated future damage.

Results

Composite Slope Analysis

The model was successfully verified with observations provided by DOGAMI and NCHC based on anecdotal wave overtopping observations from several events. While additional observations would be helpful, we believe that the methodology is adequate to estimate future damages. The analysis was largely dependent on NCHC and OPRD input for dates of historic events that caused observed failures. From this composite slope analysis it was determined that bolstering of the existing revetments to account for historic events under future rates of sea level rise identified by the National Research Council (2012), that the crest of the revetment should be raised by 8' to about 36' NAVD (Figure 12)

BEACH10 Modeling

The three model runs of BEACH 10 show similar patterns for each alternative (Table 8,Figure 13,Figure 14,Figure 15). In general, the options that maintain a beach width under all of the modeling scenarios for the long term planning horizon (2050) are the managed retreat, structural adaptation, and breakwater plus nourishment strategies. Conversely, upland properties are protected by the altered revetments, seawalls, and breakwaters.

For the wide beach width and high erosion rate, the results of BEACH10 are show in Figure 13. For this scenario the alternatives which retain a beach width out to 2050 are the managed retreat, structural adaptation, and breakwaters with nourishment. The repaired revetments would result in a loss of the beach first, followed by the seawall about 5 years later. The beach nourishment option maintains a beach width greater than or equal to initial conditions until about 2025 at which point it narrows to less than 100' by 2050. Under this modeling scenario upland property is protected by all of the alternatives except for the managed retreat. The structural adaptation options would protect the property, but likely lose the land around the ocean front parcels.

Table 8: Summary of Beach10 Results

		Managed Retreat		Revetment		Seawall		Nourishment		Breakwaters + Sand	
	Year	*Beach*	*Upland*	*Beach*	*Upland*	*Beach*	*Upland*	*Beach*	*Upland*	*Beach*	*Upland*
Present Conditions		250	250	250	250	250	250	250	250	250	250
Low Erosion	Post Action	290	250	250	250	280	250	350	250	350	250
	2025	290	220	218	250	248	250	292	250	317	250
	2050	290	170	160	250	190	250	242	250	262	246
High Erosion	Post Action	290	250	250	250	280	250	350	250	350	250
	2025	290	154	148	250	178	250	254	250	317	250
	2050	290	0	0	250	0	250	94	250	262	237
High Erosion Narrow Beach	Post Action	140	250	100	250	130	250	200	250	200	250
	2025	140	154	0	250	28	250	86	250	167	250
	2050	140	0	0	250	0	250	0	250	112	237

For the wide beach width and low erosion rate, the results of best case BEACH10 are show in Figure 14. In this modeling scenario, all of the options retain some beach width by 2050 providing some evidence that, if the PDO related littoral cell rotation reverses before 2050, there will be both a beach and protected upland until the next large oscillation. It should be noted that the breakwater option is intended to reduce wave energy at the shoreline, and may diminish the ability of the sand associated with the rotation to return to the south end of the littoral cell. This modeling nuance of the breakwater is beyond the resolution of the simple BEACH10 profile model and would require more sophisticated modeling.

For the narrow beach width and high erosion rate, the results of the worst case BEACH10 model run is show in Figure 15. In this modeling scenario, the options that maintain a beach are the managed retreat, structural adaptation, and breakwaters with nourishment. Under this scenario, beaches in Neskowin are gone by 2020 under the existing or altered revetments and by 2025 under the seawall strategy. Nourishment maintains a beach at mean high water until 2040. Upland property remains protected by most of the options except for the managed retreat and the structural adaptation, although the houses would likely survive if appropriately engineered.

Cost Estimates

Please note that in providing opinions of probable construction costs, ESA PWA has no control over the actual costs at the time of construction. The actual cost of construction may be impacted by the availability of construction equipment and crews and fluctuation of supply prices at the time the work is bid. ESA PWA makes no warranty, expressed or implied, as to the accuracy of such opinions as compared to bids or actual costs. For details on the assumptions, please see Appendix 1.

Table 9: Summary of Engineers' Estimates of Construction Costs for Comparison of Alternatives

Alternative	Total Cost[1] (millions of dollars)	Unit Cost (dollars per foot)
Altered Revetment: Rock Cap	$7	$1,000
Altered Revetment: Concrete Wall	$14	$2,000
Structural Modifications to Buildings	$14 - $27	$2,000 - $3,900
Beach Nourishment	$18	$2,600
Nearshore Breakwaters	$38 - $58	$5,500 - $8,300

[1]Assumes a total length of shore of 7,000 linear feet

Life cycle cost estimate results

The rate of damages between 1999 and 2008 were calculated to be about 120 feet of damaged revetment per year, and a repair cost of ~$73,000/year. We do not have actual costs from the community, and therefore cannot verify the accuracy of this estimate. Based on this rate of damage of 120 ft/yr, the remainder of the revetment (the length not repaired) would require repair within about 40 years, for a total life-cycle period of about 50 years. The total additional repair cost anticipated is therefore is about 4 times the 1999-2008 values, which is about $3 Million between now and 2050, in present dollars. This analysis indicates a life cycle cost of about $3.7M over a life of 50 years in addition to the initial construction cost. These are baseline estimates that presume the previously repaired sections will not require repair within the next 40 years. Of interest, the potential total water levels calculated for the five damage events ranged from 44 to 51 feet NAVD88.

Future conditions were estimated by considering the increase in percent exceedence of the total water level due to sea level rise. The direct effect of relative sea level rise on depth limited wave heights at the structure is estimated to be less than a 1%. This is because the relative sea level rise is expected to be only 17 cm by 2050. However, the continued shore erosion will induce deeper depths and larger waves at the revetment. As waves increase in height, the revetment will experience more damage during storms, leading to greater maintenance costs. The damage function developed using historic data indicates that damages increase by about 10% for every 10% increase in actual wave height relative to the design wave height. The resulting increased damages are provided in Table 10.

Table 10: Life Cycle Cost Estimates

Condition	Depth Increase (feet)	Relative Wave Height Increase (feet)	Life Cycle Cost ($/yr)	Life Cycle Cost ($/decade)
Baseline (1999-2008)	-	-	$73,000	$730,000
2030	0.5	0.06	$77,000	$770,000
2050	1.0	0.12	$82,000	$820,000

Additional design considerations for the revetment

We understand that the primary failure mechanism for the revetment is undermining and sloughing of armor when a rip current establishes and intensifies at a particular reach. This implies that the revetment toe is not embedded deep enough and does not include sufficient rock armor (large rocks) volume to accommodate the scour. Typically, this situation is addressed by constructing the revetment to a lower elevation. However, such construction can be very difficult and perhaps not possible without construction of shoring to maintain a deep excavation below tide levels. Of course, shoring in the surf zone is very difficult. The challenge of founding a revetment deeply without shoring can be approached in several ways. One way is to place additional rock as needed, and anticipate that the displaced rock will settle and establish an adequate foundation over time. Another method is to construct a horizontal toe with sufficient volume to accommodate scour by settlement of the "extra" rock placed seaward of the revetment face. Alternatively, a "toe-wall" can be constructed and left in place with rock placed behind it. Such a wall may be constructed of interlocking sheet piles or adjacent pile walls driven into the beach from above.

Discussion of Innovative Options

<u>Breakwaters</u>. Breakwater spacing should be optimized to reduce wave energy and overall alongshore erosion rate. It is anticipated that there may be some mild accretion in direct lee of the structures, but the design objective would be to reduce net transport and reduce background erosion rates to zero. It is important however, given the uncertainty (or likelihood) that a future littoral cell counter rotation to the south would occur, that there are large enough gaps to enable some transport to both 1. Avoid upcoast effects 2. Enable sand to return if the PDO pattern shifts. More sophisticated modeling would be needed to support increasing levels of design to fine tune the specifics of shape, size, length, and offshore distance. If a lower erosion rate is achieved then the volumes of sand needed in the nourishment may be reduced with a subsequent lowering of the nourishment cost.

Wave tripping structure. A wave tripping structure located on the beach such as the Taraval Seawall (Figure 8) was considered but there are many challenges associated with such a design. First, anything temporary such as K rails or concrete blocks would likely fail in one of two ways. First, given the large wave energy, the size of such blocks would likely result in them strewn across the beach with little to no effect on wave energy dissipation. Another likely method of failure should they be anchored together would be for the scour on either side caused by alongshore flow and wave overtopping so that the structures would likely sink into the beach and become useless as an erosion control device. Should a sheet pile structure be constructed with appropriate bracings to withstand the wave loading, the acceleration of trapped longshore currents would both scour the structure and create a safety hazard for beach recreational users with high velocity currents occurring in gaps or at the end of such structures. In general, beach perching/ wave tripping structures raise many safety concerns and are likely to be relatively ineffective during high storm events.

Pile baffle wall. This type of structure could be envisioned as an offshore pier parallel to shore with dense spacing of piles. While this would serve to dissipate wave energy, the potential for debris such as large woody debris to get trapped between piles is high and would increase wave loading that could cause the structure to fail. There is also a consideration of the aesthetics.

T-head groins. This type of structure was considered as a way of limiting creek channel migration and rip channel formation. The "T" refers to a shore parallel oriented structure on the seaward end of the groin, intended to inhibit rip channel formation. However, given the large dynamic setup on Oregon coasts, we are concerned that such a structure may still induce rip channel formation and hence may not perform as intended. This option would preclude any counter rotation of sediment returning to the south end of the littoral cell. For this reason and the likely high cost this is not considered preferred or feasible.

Pile groin. This is a type of groin that allows for sediment and wave to pass through. The use of such a structure would be to partially retain sand and there may be ways to make it more aesthetically pleasing by mimicking the petrified forest. However the real benefit may be the ability to stabilize the rip embayment at the north side of Proposal Rock. However, problems associated with trapped debris causing exceedance of wave load capabilities and the uncertainty at actually stabilizing the rip embayment make this a highly uncertain alternative without more sophisticated fine scale modeling.

Dynamic revetments. Also known as cobble berms, these mimic naturally occurring cobble deposits found along much of the Oregon coast. Although these have been used in nearby locations (Cape Lookout), they are not likely to be effective in Neskowin given the deteriorated beach widths. Inside of Proposal Rock they may have some merit, but concerns about the cobbles becoming projectiles would likely require them to be constrained in some sort of gabion wave tripping device farther on the beach. The placement of such a device would be complicated by the transitory location of Hawk Creek.

Findings and Recommendations

- Beach nourishment provides additional beach width and upland property protection at least through 2030. This nourishment may provide the interim protection for several years until it can be determine whether the littoral cell may counter rotate. Assuming each property lot along the beach is 100 feet, the cost of beach nourishment per lot is about $260,000 per lot.

- Sources for beach nourishment, should this option be pursued, should include detailed investigation of the ebb shoal at the mouth of the Nestucca River.

- Elevating houses balances beach width with private residence protection (but not ground and lawn) but there are additional considerations to providing a refined cost estimate that will require additional engineering information including an inventory of the types of foundations in hazardous areas.

- Potentially reduce hazards at the Hawk Creek Bridge by using a dynamic cobble revetment to knock down wave energy propagating upstream, although it is possible that a gabion matting would be required to reduce likelihood of projectiles affecting the hotels.

- Nearshore breakwaters offer a good balance of upland property protection and maintenance of beach width but are extremely expensive to construct and maintain (assuming they are even permittable).

- The community may wish to form a Geological Hazard Abatement District (GHAD) formation to fund alternatives.

- Interim or seasonal storm response – engineered Krails installation instead of construction of additional wall at the top of the structures could reduce costs, but also may provide a false sense of security.

- Managed retreat is the only option which maintains a beach throughout all of the planning horizons and beyond. It is likely though to lead to extensive damages to the community. However, it should be noted that following an extreme event (100+ year storm or Cascadia subduction zone), it is likely the only solution over the long term. The community should consider developing a post disaster visioning strategy on how and where to rebuild following such an event.

- Anecdotally, large volumes of overtopping water have contributed to structural failures. Developing a storm water management plan that addresses managing both precipitation and wave overtopping volumes may reduce the level of dune sand saturation and reduce the level of structural damages which occur during a large storm event.

- A regional littoral cell approach to dune management planning to the north that reduces dune storage volumes in high dunes dominated by invasive European beach grasses may enable more sediment to be eroded and released during storm events and reduce post storm recovery time periods.

- Stabilization of the rip embayment that forms off of Proposal Rock may enable bolstering of structures in some areas and not entire community as a cost savings, however such stabilizing structures may also limit the return of sediment if the littoral cell counter rotates.

- One way to increase the effectiveness of the existing structure is to increase the surface roughness of the revetment. Such roughness slopes may reduce TWLs and wave run-up elevations. This can be tested with the run-up analysis but reductions of up to 3 feet in total water elevation are expected to be practical. It would be expected to have an increase in associated costs, beach loss, and greater difficulty with vertical beach access.

- The community can anticipate significant future costs to maintain the existing revetment. We estimate average costs of about $700,000 to $1,000,000 per decade, with approximately 1,000 linear feet of damage per decade.

- The revetment could be improved in several ways:

  1. Construction of a deeper foundation by way of:
     a. Extending the revetment toe by excavation and rock placement to lower elevations, and
     b. Construction of a toe-wall to inhibit undermining;

  2. Construction of a “sacrificial toe” consisting of rock placed horizontally with the objective of settlement into scour depressions with less chance of sloughing of the upper, sloped part of the revetment; and,

  3. Placement of an additional layer of armor as an “overlay” to the existing armor to provide additional volume in case of sloughing, and to reduce scour by reducing wave reflection and increasing wave dissipation in the expanded armor voids. This would require alterations to the surface of the existing structure to enable some additional rock to be placed and interlocked into the existing structure.

It should be noted that the above recommendations need to be evaluated against the substantial costs of implementation and the substantial adverse effects to beach width and associated degradation of recreation, ecology, and aesthetics. Moreover the extent to which such actions will be required or permittable is difficult to ascertain and so revetment expansion may be “open ended” if wave energy and structural exposure continue to increase.

References

Allan, J.C. and Hart, R., in review. Assessing the Temporal and Spatial Variability of Coastal Change in the Neskowin Littoral Cell: Developing a Comprehensive Monitoring Program for Oregon Beaches, Oregon Department of Geology and Mineral Industries, Portland, Oregon.

Baron, H., 2011. Coastal Hazards and Community Exposure in a Changing Climate: The Development of Probabilistic Coastal Change Hazard Zones. Master of Science Thesis, Oregon State University.

Komar, P., 1997. El Nino and coastal erosion in the Pacific Northwest. *Oregon Geology*. 60(3) pp. 57-64.

Dean, R.G., and Bender, C.J., 2006, Static wave setup with emphasis on damping effects by vegetation and bottom friction, *Coastal Engineering*, **53**, pp. 149-156.

Federal Emergency Management Agency (FEMA), 2005, Final Draft Guidelines for Coastal Flood Hazard Analysis and Mapping for the Pacific Coast of the United States, Prepared for the U.S. Department of Homeland Security, Washington, D.C., January 2005.

Harris, E., 2011. Assessing Physical Vulnerability of the Coast in Light of a Changing Climate: An Integrated, Multi-hazard, Multi-timescale Approach. Master of Science Thesis, Oregon State University.

Heberger, M., Cooley, H., Herrera, P., Gleick, P.H., and Moore, E., 2011, Potential impacts of increased coastal flooding in California due to sea-level-rise, *Climatic Change*, **109**(S1), pp. 229-249.

Hunt, I.A., 1959, Design of seawalls and breakwaters, *Journal of Waterways and Harbors Division*, ASCE, **85**(3), pp. 123-152.

National Research Council, 2012. Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future. National Academies Press Washington DC. Prepublication 2012.

Peterson, W., C. Morgan, J. Peterson, J. Fisher, B. Burke, and K. Fresh, 2011. Ocean Ecosystem Indicators of Salmon Marine Survival in the Northern California Current. NOAA

Pacific Institute, 2009, The Impacts of Sea-Level Rise on the California Coast, California Climate Change Center, Sacramento, California, Paper CEC-500-2009-024-F, March 2009.

Personal communication Jonathan Allan 5/1.

Philip Williams & Associates (PWA), 2009, California Coastal Erosion Response to Sea Level Rise – Analysis and Mapping, Report prepared for the Pacific Institute, PWA Ref. #1939.00, March 11, 2009.

Revell, D.L., Battalio, R., Spear, B., Ruggiero, P., and Vandever, J., 2011, A methodology for predicting future coastal hazards due to sea-level rise on the California coast, *Climatic Change*, **109**(S1), pp. 251-276.

Ruggiero et al in press – USGS

Stockdon, H.F., Holman, R.A., Howd, P.A., and Sallenger, Jr., A.H., 2006, Empirical parameterization of setup, swash, and runup, *Coastal Engineering*, **53**, pp. 573-588.

Tillamook County Department of Community Development, 2012. Neskowin's Coastal Erosion Adaptation Plan, Draft, revision 1 (April 2012).

U.S. Army Corps of Engineers (USACE), 1984, Shore Protection Manual, 4th ed., 2 Volumes, U.S. Army Engineer Waterways Experiment Station, U.S. Government Printing Office, Washington, DC.

U.S. Army Corps of Engineers (USACE), 2002, Coastal Engineering Manual, Engineer Manual 1110-2-1100, U.S. Army Corps of Engineers, Washington, D.C. (in 6 volumes).

SOURCE: ESA PWA 2012

Neskowin Shoreline Assessment. D211715.00

Figure 1
Study Area

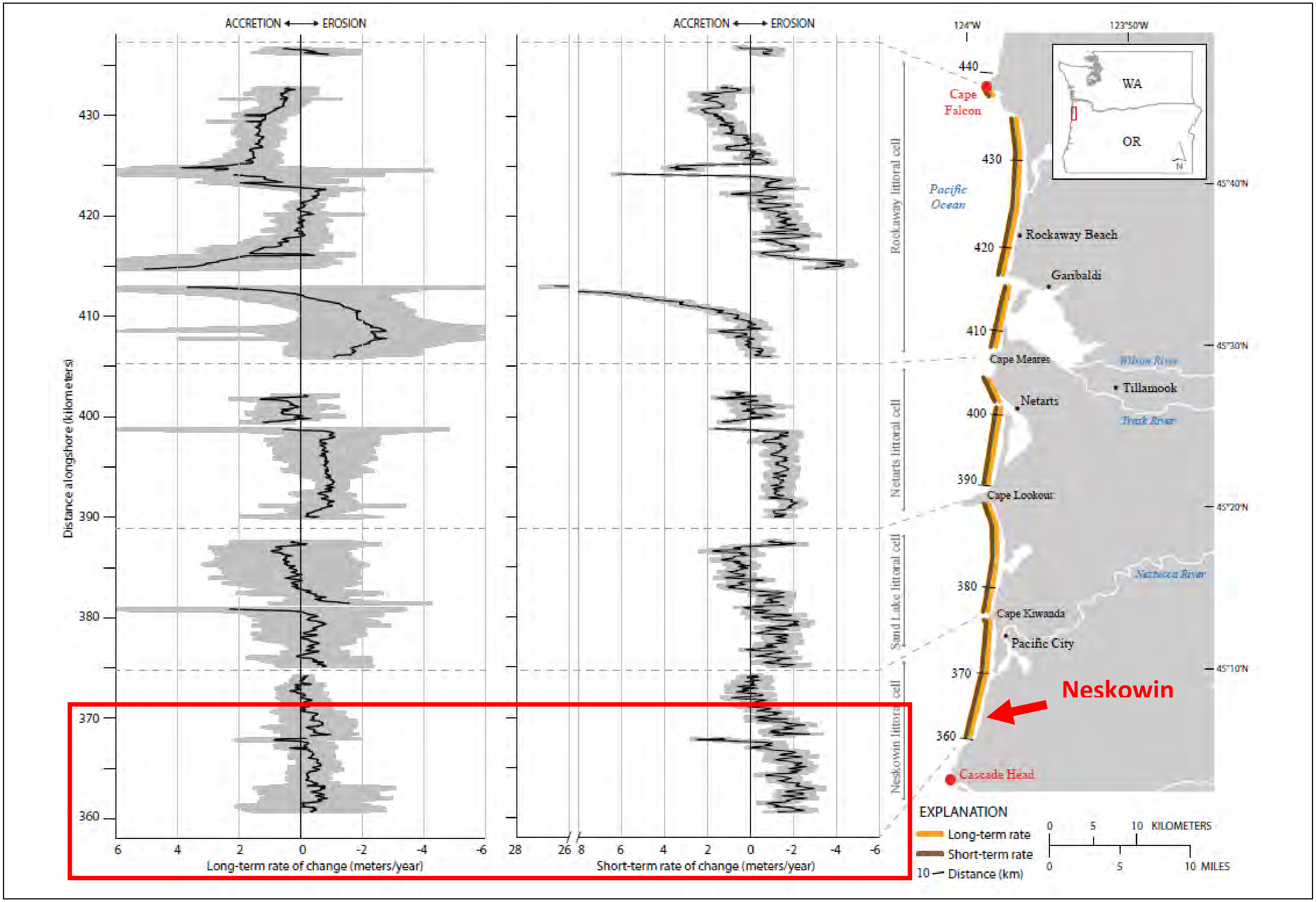
SOURCES:
A - Photo by Armand Thibault, Jan 9, 2008 (Published in the National Research Council's "Sea-Level Rise for the Coasts of California, Oregon, and Washington: Past, Present, and Future." Prepublication. National Academies Press: Washington D.C. 2012),

B - Photo by Pete Owston Neskowin resident January 5, 2008
courtesy of Tony Stein, OPRD

Neskowin Shoreline Assessment. D211715.00

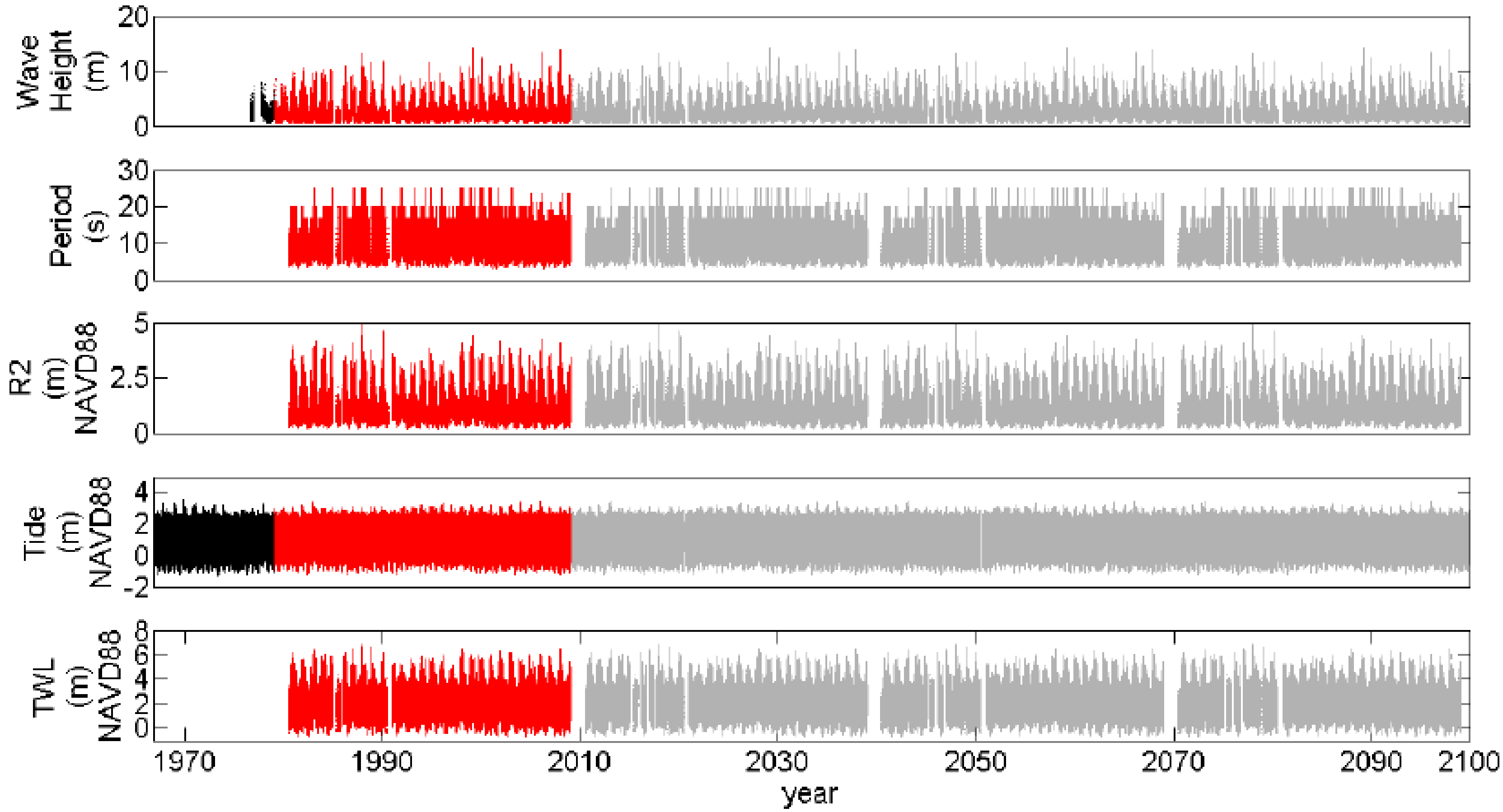
Figure 2

High Surf and Revetment Failure in Neskowin



SOURCE: Ruggiero et al, in press.

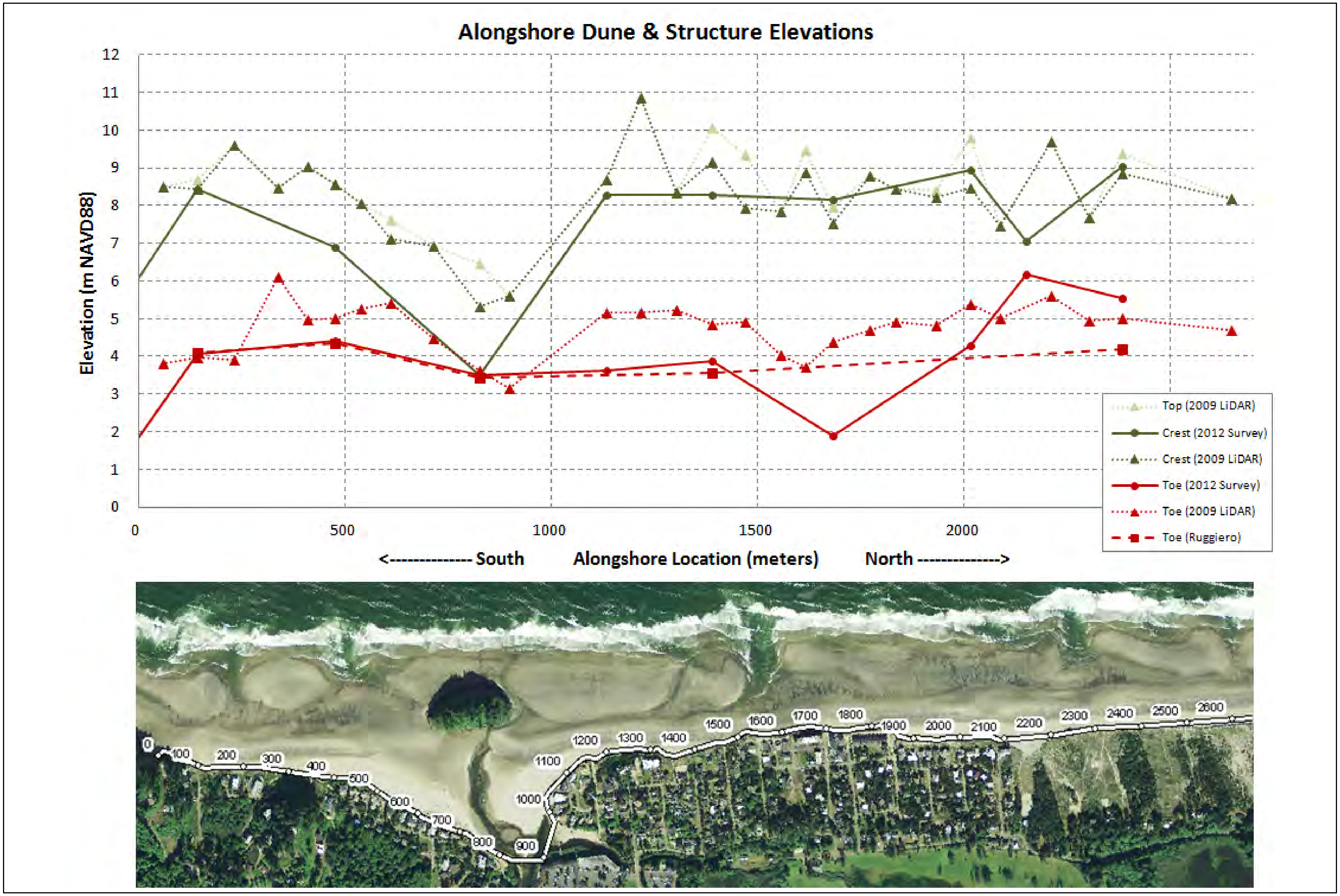
Figure 3
Shoreline Erosion Rates



SOURCE: Figure from Harris, 2011.

Note: Red lines represent observed data, gray represent the synthetic dataset extended to 2100 using red data. Data represented by a black line was not used in the synthetic time series.

Figure 4
Time Series of Waves and Tides



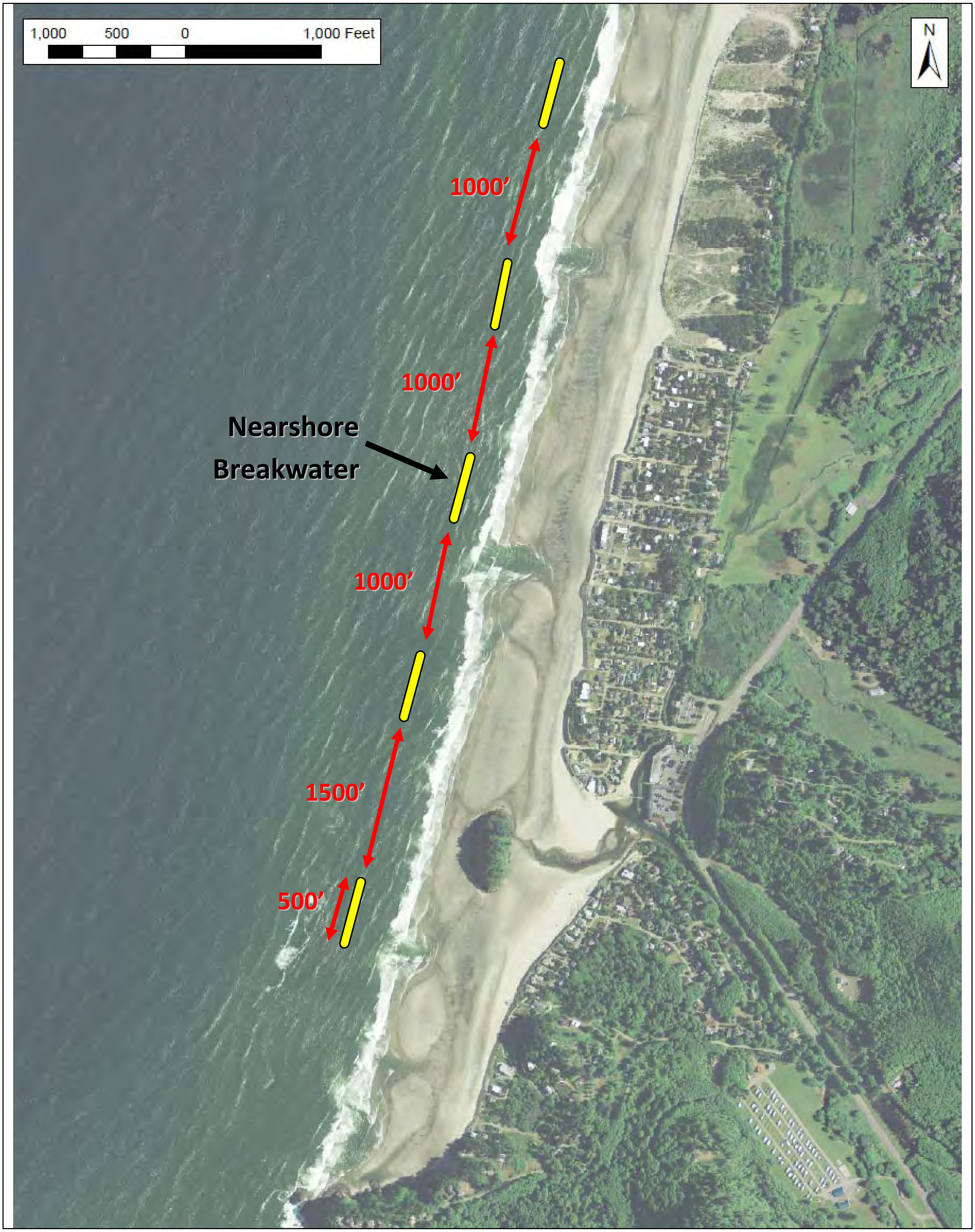
SOURCE: ESA PWA (Figure, profile interpretations), Topo data from the following sources: P. Ruggiero of Oregon State University (Beach profiles), 2009 Oregon Department of Geology and Mineral Industries (DOGAMI) LiDAR, J. Allan of DOGAMI (2012 Survey).

Figure 5
Alongshore Dune and Structure Elevations

SOURCE: Bob Battalio

Neskowin Shoreline Assessment. D211715.00

Figure 6
O'Shannessy Seawall



SOURCE: ESA PWA 2012

Note: Breakwater locations approximate and dimensions not to scale.

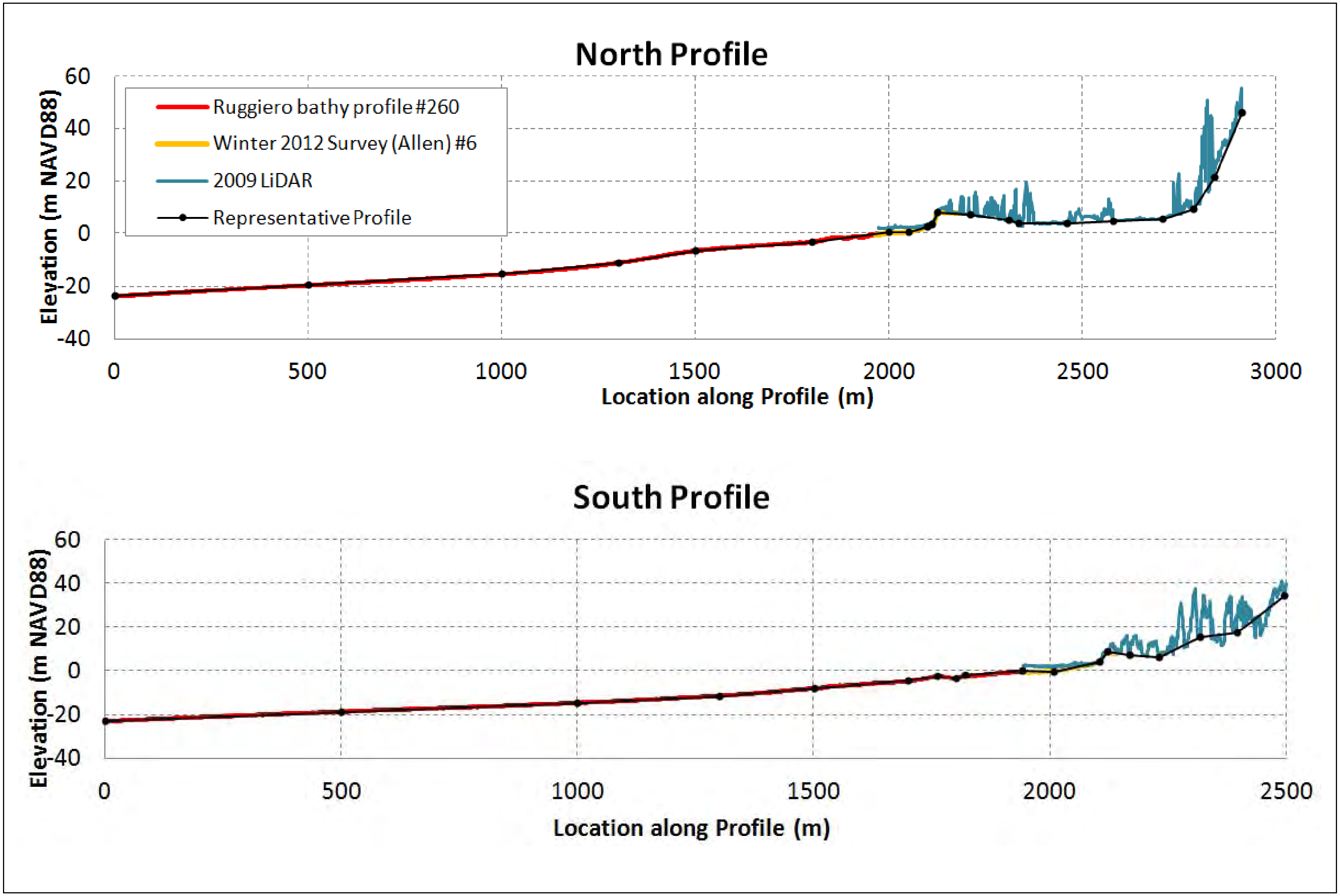
Neskowin Shoreline Assessment. D211715.00

Figure 7

Nearshore Breakwater Schematic

SOURCE: ESA PWA: Photo Left, Elena Vandebroek, 2011; Right: Bob Battalio, 1998.

Figure 8
Taraval Seawall

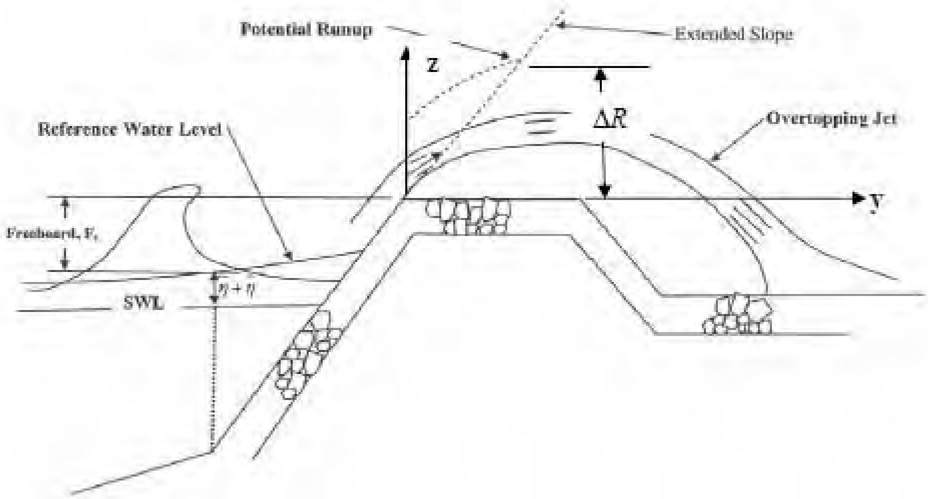


SOURCE: ESA PWA 2012 (Figure, Representative Profiles), Ruggiero et al (Bathymetry), Allan 2012 (Survey), CA Coastal Conservancy LiDAR Project (2009 LiDAR).

Neskowin Shoreline Assessment. D211715.00

Figure 9

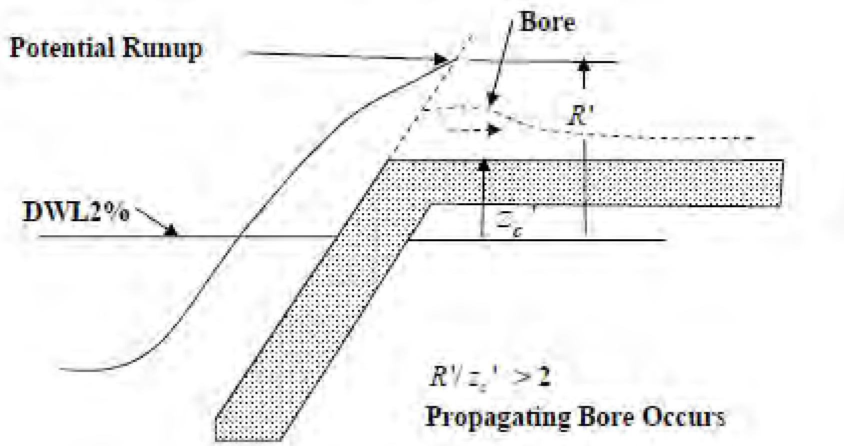
Representative Profiles for Composite Slope Analysis



Parameters Available for Mapping BFEs and Flood Hazard Zones

Overtopping Parameters Used in Hazard Zone Mapping

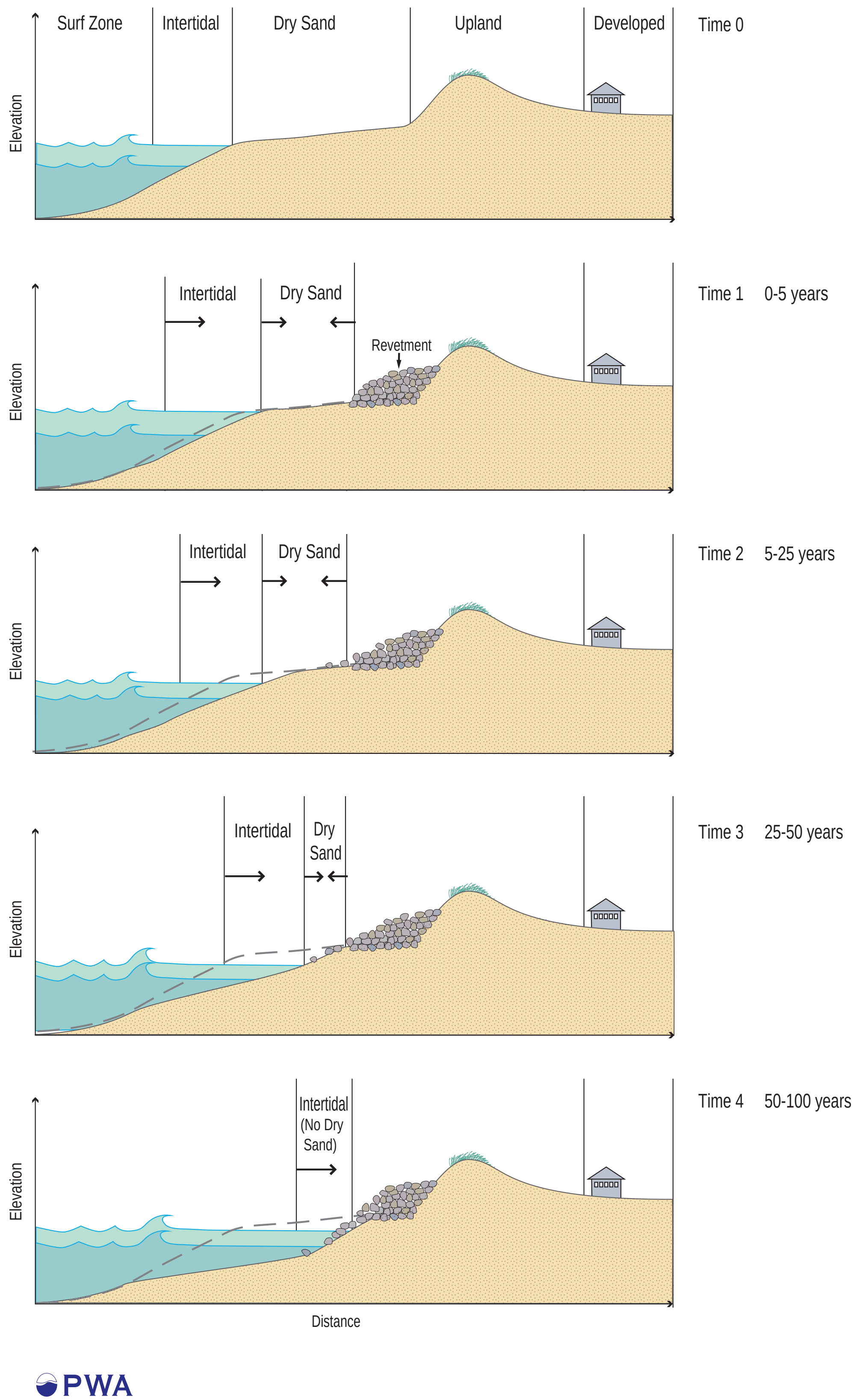
Parameter	Variable	Units
Total potential runup elevation	R	ft
Mean overtopping rate	q	cfs/ft
Landward extent of green water and splash overtopping	$y_{G,Outer}$	ft
Depth of overtopping water at a distance y landward of crest	$h(y)$	ft



Conditions for Bore Propagation Overtopping

SOURCE: FEMA Guidelines for Pacific Coast Flood Studies, 2005.

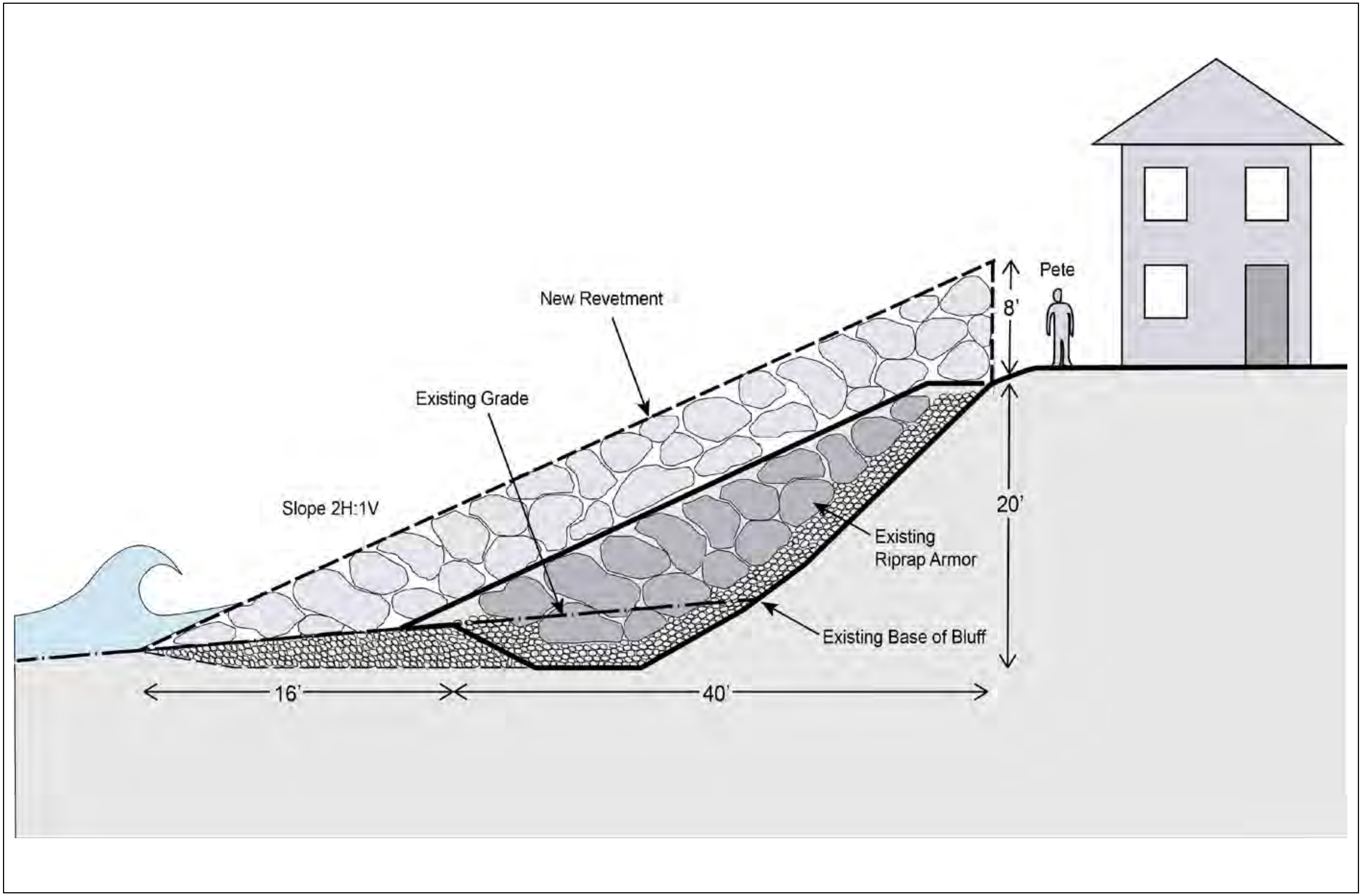
Figure 10
Schematics Defining Wave Run-up and Overtopping Parameters



SOURCE: PWA

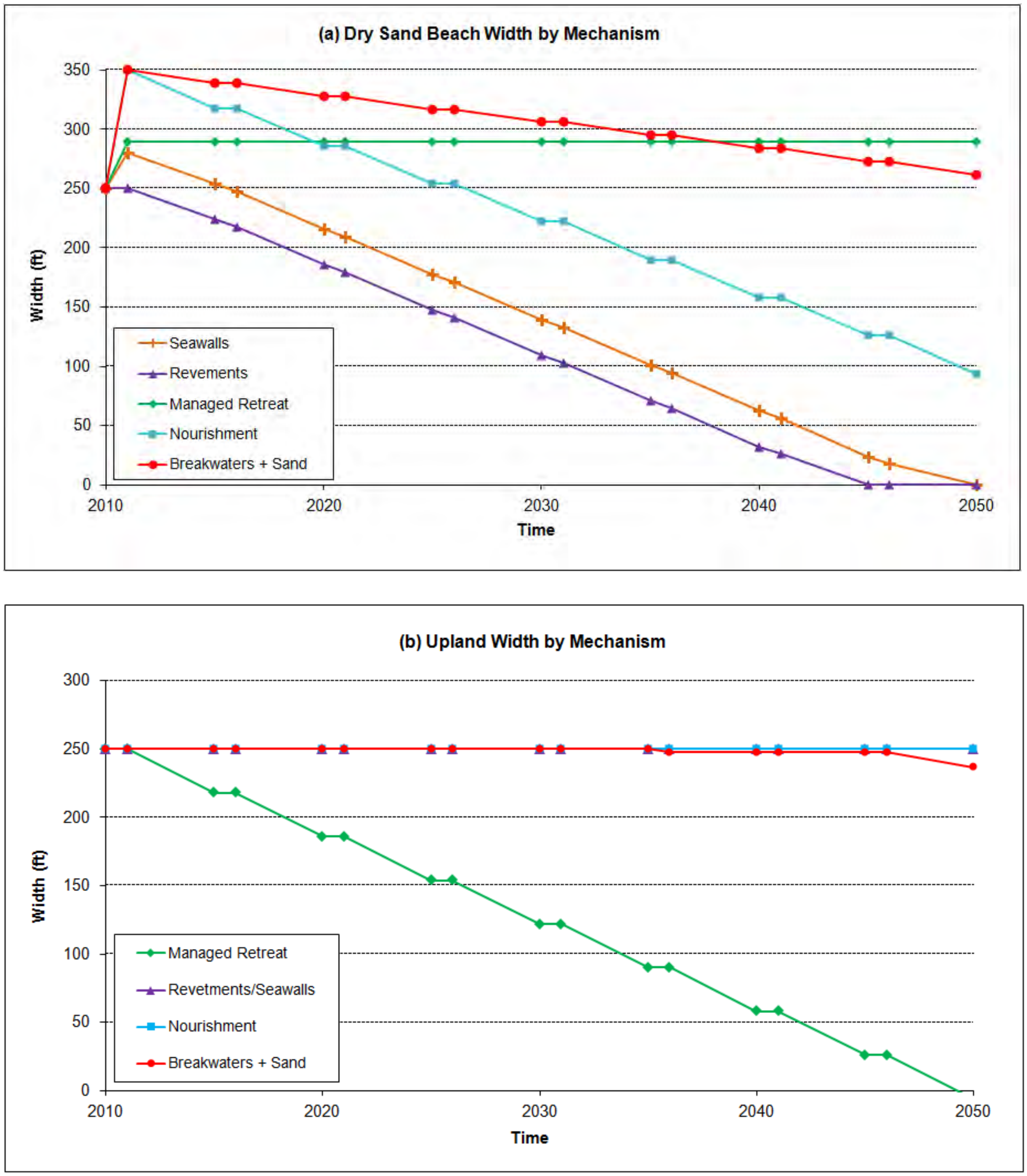
Neskowin Shoreline Assessment. D211715.00

Figure 11
Beach 10 Definitions



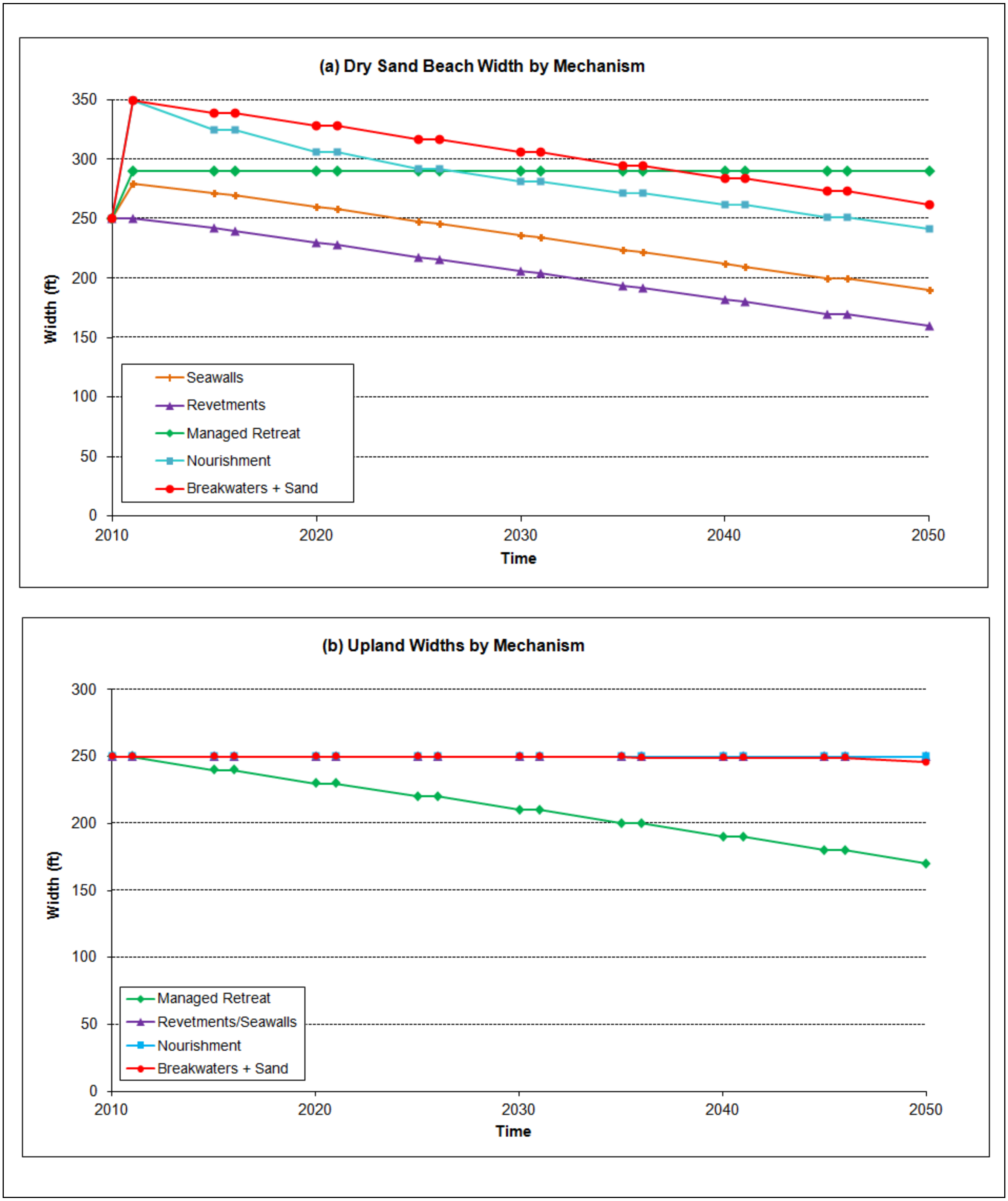
NOTE: Conceptual design not to be used for construction

Figure 12
Conceptual Design for Expanded Revetment



SOURCE: ESA PWA 2012

Figure 13
Beach10 Results: Wide Beach, High Erosion



SOURCE: ESA PWA 2012

Neskowin Shoreline Assessment. D211715.00

Figure 14

Beach10 Results: Wide Beach, Low Erosion

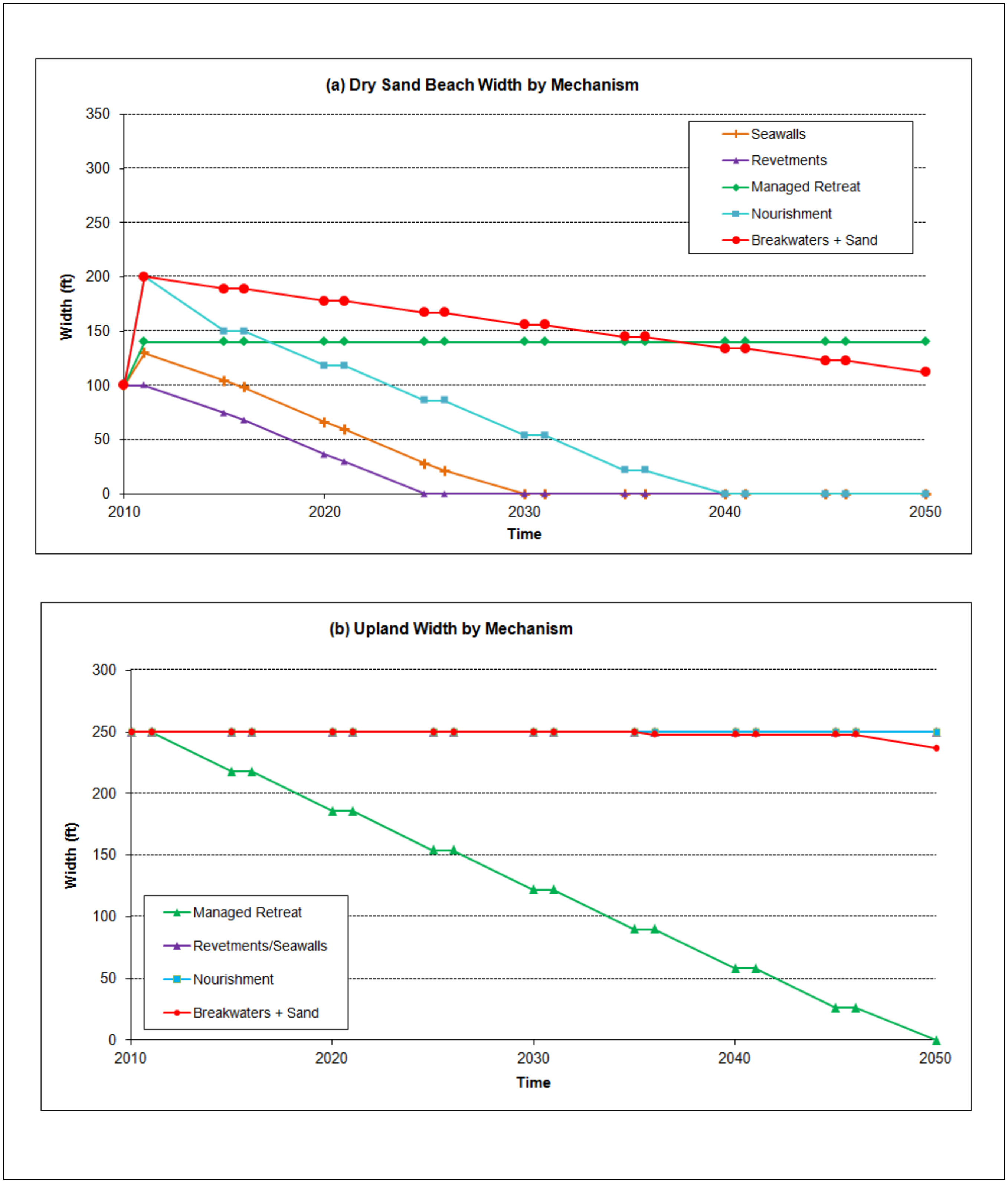


Figure 15
Beach10 Results: Narrow Beach, High Erosion

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memorandum

date	August 10, 2012
to	David Revell (ESA PWA)
from	Louis White, PE (CA)
subject	Appendix 1: Construction Costs of Alternatives for Neskowin Shoreline Assessment

Introduction

This memorandum provides a summary of construction costs of different alternatives identified as part of the Neskowin Shoreline Assessment project. The purpose of presenting the following costs is for comparison of different alternatives to mitigate coastal erosion problems that the local community is presently facing. The work described in this memorandum was accomplished by Louis White, P.E. and Curtis Loeb,P.E., with oversight by Bob Battalio, P.E. (OR).

Level of Cost Estimating

For planning purposes we have provided order of magnitude estimates to allow cost comparison of alternatives. These cost estimates are intended to provide an approximation of total project costs appropriate for the conceptual level of design. These cost estimates are considered to be approximately -30% to +50% accurate, and include a 35% contingency to account for project uncertainties (such as final design, permitting restrictions and bidding climate).

These estimates are subject to refinement and revisions as the design is developed in future stages of the project. This table does not include estimated project costs for permitting, design, monitoring and/or ongoing maintenance. Estimated costs are presented in 2012 dollars, and will need to be adjusted to account for price escalation for implementation in future years. This opinion of probable construction cost is based on: ESA PWA's previous experience, bid prices from similar projects, and consultation with Oregon contractors and suppliers.

Please note that in providing opinions of probable construction costs, ESA PWA has no control over the actual costs at the time of construction. The actual cost of construction may be impacted by the availability of construction equipment and crews and fluctuation of supply prices at the time the work is bid. ESA PWA makes no warranty, expressed or implied, as to the accuracy of such opinions as compared to bids or actual costs.

Alternatives and Assumptions

Construction costs for six alternatives were estimated to a conceptual level for cost comparison purposes. These alternatives include:

- Altered Revetment: Rock Cap
- Altered Revetment: Concrete Wall
- Structural Modifications to Buildings
- Beach Nourishment
- Offshore Breakwaters

Comparative costs per linear foot of beach are presented in addition to the total cost of the alternative. For comparison purposes, a shoreline length of 7,000 feet was assumed. The cost of each alternative, or the elements of each alternative, was estimated on a construction quantity basis. Unit costs of purchasing, transporting, and placing rock and sand were estimated from previous ESA PWA experience, bid sheets, and discussions with local contractors that specialize in seawall construction in coastal Oregon. These costs reflect construction during non-emergency periods. Construction of emergency seawall repair is typically more expensive due to the emergency nature of the work, difficult working conditions, and material and labor constraints.

A summary of each alternative and the assumptions made to estimate the costs follows.

Altered Revetment: Rock Cap

- Increase elevation of existing rock revetment by 8 feet, from elevation +28 ft NAVD to +36 ft NAVD
- Assume rock size is 1-5 ton, approximately 4-7 feet in diameter, with median rock diameter of 5 feet
- Assume crest width is two rocks, or about 10 feet
- Assume sideslope of 2:1 (H:V) on both front and back sides of rock cap
- Calculated unit volume is 7.7 cy/lf; assuming an average revetment density of 1.6 tons/cy (this includes armor stone and bedding), the unit weight is approximately 12.3 tons/lf
- Assume a unit cost of $80 per ton, which includes rock purchase, transport, placing using land-based equipment, and contractor overhead and profit (Morris 2012). This yields a unit cost of $1,000/lf to increase protection with a rock cap, or a total cost of $7M for the whole shore length

Altered Revetment: Concrete Wall

- Increase elevation of protection by 8 feet, from elevation +28 ft NAVD to +36 ft NAVD
- Allow $2,000/lf to construct re-curved reinforced concrete wall, or a total of $14M for the entire shore length

Structural Modifications to Buildings

- Structural modifications to buildings involves raising the buildings vertically up to 10 feet and placed on driven pre-cast concrete piles or cast-in-drilled-hole piles (CIDH piles) to an elevation above the 100-year total water level, per FEMA guidelines.

- A unit cost of $130/sf to raise building up to 10 feet was estimated for structural modification in California (ESA PWA 2012). Due to location difference (Bay Area, CA versus coastal Oregon), we think a unit cost of $65/sf is appropriate. Further investigation into structural modifications methods for the area and building types might warrant an additional decrease in the unit cost for raising a building on piles. However we think a unit cost of $65/sf is appropriate for conceptual cost comparisons.

- Assume total structure length is 60% of total shoreline length: 60% of 7,000 lf = 4,200 lf

- Assume a range in the nominal width (landward) of structures from 50-100 ft

- Range in area is calculated to be 210,000 sf to 420,000 sf

- Total range in cost estimated at $14M to $27M; unit cost range of $2,000 to $3,900/lf for 7,000 lf of shore

Beach Nourishment

- Beach nourishment involves placing sand directly on the shore to widen the beach. This is likely to be accomplished by dredging suitable sand from offshore location, and pumping onshore

- Assume existing top of beach is at elevation +16 feet NAVD (NANOOS 2012)

- Assume depth of closure at elevation -50 feet (-15m) NAVD, estimated visually from measured bathymetry profiles, and personal communication with Peter Ruggiero (OSU 2012)

- Assume unit volume of beach nourishment at 2.5 cy/sf of beach; for widening the beach crest by 100 feet, the unit volume becomes 250 cy/lf of beach, yielding a total volume of 1.8 MCY for 7,000 lf of beach

- Assume a unit cost of $10/cy to pump sand onto the beach from offshore; total cost of project is approximately $18M, or about $2,600/lf of beach

- Assuming each property lot along the beach is 100 feet, the cost of beach nourishment per lot is about $260,000 per lot

Offshore Breakwaters

- Construction of offshore breakwaters is intended to reduce wave energy at the beach. The structures described here are low-crested, and intended to be overtopped by tides and waves, and to allow counter littoral cell-wide rotation which would naturally bring sand back to the south end of the Neskowin beach. Construction of offshore breakwaters should include beach nourishment (see above). Beach nourishment alone will likely need to be repeated periodically, and including offshore breakwaters can reduce the frequency of re-nourishing the beach over time.

- Assume nominal crest length of 500 feet

- Assume spacing of individual offshore breakwater structures to be 1,000-1,500 feet, for a total of 5 structures

- Assume rock size of 10-20 tons, or about 6-8 ft in diameter

- Assume a crest width that is 4-5 rocks wide, or about 30 feet, and sideslopes of 2:1 (H:V)

- Assume the breakwater is constructed on the nearshore bar, from elevation -5 ft NAVD at the bottom to elevation 3.3 ft NAVD at the top

- Assume approximately 2.5 feet of over-excavation to found the structure – this yields a structure that is about 11 feet tall

- Calculated unit volume is 22.4 cy/lf; at 1.6 tons/cy, this yields a unit weight of 36 tons/lf

- Use a unit cost of $200/ton for rock delivery and placement (Moffat & Nichol 2011); use a unit cost of $20/cy for excavation; combining these unit costs yields a construction cost of $8,000/lf per each offshore breakwater

- Due to uncertainties in height and spacing of structures, assume a range in unit cost of $8,000-$16,000/lf

- For 5 breakwaters, the total cost range is $20M-$40M, yielding an approximate unit shoreline cost range of $2,900/lf to $5,700/lf

- Combining the offshore breakwater costs with a one-time beach nourishment (see above) yields a cost range of $38M-$58M, or approximately $5,500/lf to $8,300/lf

Summary Table of Costs

Alternative	Total Cost[1] (millions of dollars)	Unit Cost (dollars per foot)
Altered Revetment: Rock Cap	$7	$1,000
Altered Revetment: Concrete Wall	$14	$2,000
Structural Modifications to Buildings	$14 - $27	$2,000 - $3,900
Beach Nourishment	$18	$2,600
Offshore Breakwaters	$38 - $58	$5,500 - $8,300

[1]Assumes a total length of shore of 7,000 linear feet

References

DOGAMI, 2012, Figures of fluctuations of beach elevations at Neskowin, 1997-2012, available online: www.NANOOS.org, August 9, 2012.

ESA PWA, 2012, Russian River Estuary Management Plan Alternative Evaluation, Memorandum to California Coastal Commission, June 5, 2012.

Moffat & Nichol, 2011, Port Orford Breakwater Major Maintenance Report, prepared for the U.S. Army Corps of Engineers, October 2011.

Morris, L., 2012, personal communication with Larry Morris, Devils Lake Rock Company, Lincoln City, Oregon, August 6, 2012.

OSU, 2012, Bathymetric profiles offshore of Neskowin Beach.