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Dunes and Ocean Front Structures Under Wave Attack

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ABSTRACT

Sand dunes and sandbars which serve as natural protective barriers to ocean front structures on many beaches can undergo geological change due to wave action. Engineered structures such as concrete seawalls, stone revetments, and bulkheads can be damaged or destroyed by storm driven waves. Once these coastal defenses are eroded or destroyed by wave action landward buildings are vulnerable to wave attack. This paper focuses on the relationship of wave runup and setup to dune morphology and how wave forces directed at structures behind the dunes can be quantified. A comparison of wave forces to wind forces is made with the intention of demonstrating the dominance of wave action in producing damage to structures along the coast. Forecast techniques that allow wave action in the velocity zone to be integrated into operational forecasts are presented along with a case study of overwash during the 6-9 March 2013 Atlantic Storm.

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1. Introduction

Coastal areas exposed to the open ocean can sustain substantial damage from large storm driven waves or from long period wave systems arriving from hundreds or even thousands of miles away. These powerful waves can runup onto the shore at high velocity eroding dunes then striking and destroying exposed infrastructures shoreward of them. Damage from wave action can be exceptionally high when coincident with extremely high astronomical tides and long duration storms.

Wave damage along exposed coastal locations is addressed by the National Flood Insurance Program (NFIP) which is under the jurisdiction of the Federal Emergency Management Agency (FEMA). Areas exposed to the most severe wave energy are designated as velocity or V Zones on National Flood Insurance Program (NFIP) Risk Maps (FEMA 2013). Communities that opt to join the NFIP are required to enforce restrictions on construction in these zones. Sand dunes on a beach often represent the separation point of the V Zone from less hazardous zones and serve as natural defensive barriers protecting structures landward of them from wave attack. These barriers are however vulnerable to runup from large waves and can fail.

The first objective of this paper is to quantify the forecasting of wave runup and setup and demonstrate how this information can be utilized to determine if a dune system, which provides protection to infrastructure behind it, could fail under storm conditions. Subsequently, if such a dune system does fail, a demonstration of how to calculate the magnitude of wave forces brought to bear against infrastructure on the landward side of the dune will be made.

An additional goal is to demonstrate that water moving at high velocity is much more destructive than stagnant inundation water. To better exemplify this, the destructive power of wave forces will numerically be compared to wind forces of an equal magnitude.

The National Weather Service coastal flood warnings, watches, advisories, and public information statements commonly focus on storm tide levels. The Extra-tropical Surge and Tide Operational Forecast System (ESTOFS; Feyen et al. 2013), which integrates the extra-tropical storm surge model with the Advanced Circulation Model (ADCIRC), is used to determine storm tide levels during extra-tropical events. The Sea, Lakes, and Overland Surge from Hurricanes (SLOSH) Model (Glahn 2009) is used to determine storm tide levels for tropical events. Information pertaining to wave runup and setup is lacking from current National Weather Service products. While the current NWS storm surge road map addresses wave action in the context of water levels, the problems of wave impact damage in the velocity zone resulting from wave runup are not specifically addressed (http://www.stormsurge.noaa.gov/r_and_d.html). It is therefore the final objective of this paper to demonstrate that it is possible to include effects of wave action in enhanced operational forecasts for specific high impact areas (hot spots).

2. Astronomical Tides

The importance of the astronomical tide cannot be emphasized enough at northern latitudes. Timing of the coincidence of maximum storm conditions and the astronomical tide is of the utmost importance in determining maximum water levels. It is also one the most difficult forecasting issues. Zhang (2001) found that of all the factors responsible for erosion during severe nor'easters that tide level was the most important. Secondly, not all high tides are equal. Spring tides occur when the earth, moon, and sun align with each other (new or full moon) and are much higher than neap tides which occur when the moon is at right angles to the earth and sun alignment. *Perigean* spring tides will produce even higher astronomical tides. They occur when the moon's perigee coincides within ±3.5 days of a new moon. If such a coincidence of the earth's perigee and new moon occurs within ± 5 hours it is referred to as a *Proxigean Spring Tide*. Coincidence of a Proxigean spring tide with maximum storm tide levels and largest waves will result in even greater potential for erosion and damage.

3. Elevated Water Levels

Storm conditions can raise water elevations above normal levels in a number of ways. Elevated water levels resulting from wind and atmospheric pressure are referred to as *storm surge*. If storm surge is combined with the astronomical tide the water elevation is then termed *storm tide*.

In addition to wind and atmospheric pressure, wave action also raises water levels. This process is referred to as *wave setup* and is defined as the super elevation of water along the shoreline resulting from the cross-shore gradient in radiation stress as described by Longuet-Higgins and Stewart (1964). It is a direct result of breaking waves in the surf zone forcing water onto the beach faster than it can retreat. Wave setup is best described as a time averaged water level since it fluctuates quickly relative to tide and storm surge. Additional water elevation resulting from wave setup is excluded from the definition of storm surge and storm tide. Dynamic Water Level (DWL) is composed of the astronomical tide, storm surge, and wave setup.

As large waves break in the surf zone they transition into turbulent bores that make incursions up the beach traveling at high velocity. Such a time varying elevation of water on the beach is known as *wave runup.* All of these reference levels are illustrated in Fig. 1.

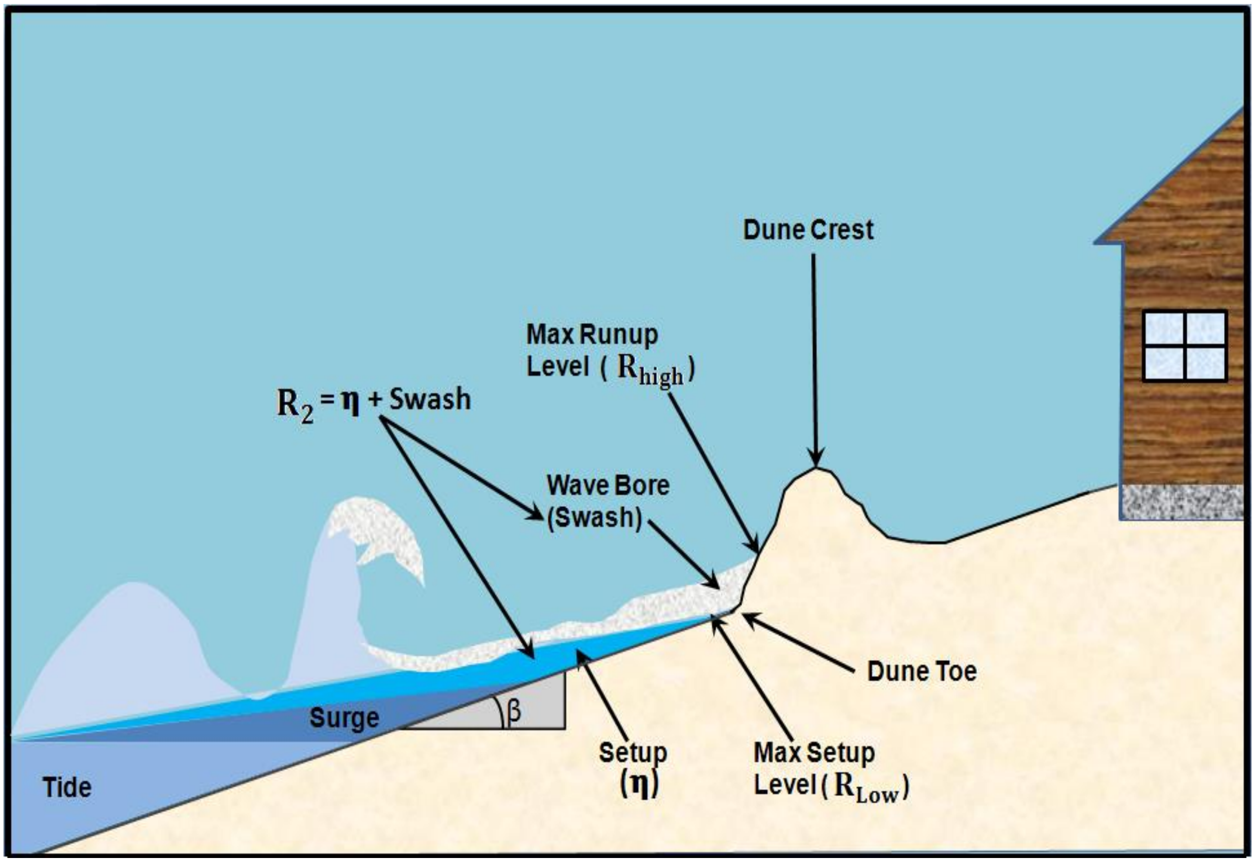

Figure 1. Storm driven water levels on an exposed beach along with wave runup in collision with a sand dune.

4. Beach Dune Structure

Beach dunes are formed through the process of wind and wave action over long periods of time. Carter (1988) has described the dune building process as a result of high energy storm waves depositing materials on the upper portion of beaches. Such a dune building process takes place as a result of the runup and subsequent retreat, or backwash, of individual storm waves. During the runup phase sand is carried up the beach by the surging wave bore. At the same time water is also being absorbed into the coarse ground, which reduces the volume of water contained in the wave bore as it makes its incursion up the beach. As water returns to the ocean during the backwash phase not as much sand can be transported back since water volume has been reduced due to absorption. Sand is therefore left behind contributing to the building of the sand dune.

Davies (1958) described the dune building process in terms of cut and fill. During high energy storm events sand is cut from the dunes and deposited along the foreshore slope or off-shore sand bars by large storm waves. During subsequent long period swell events the newly cut sand is then built back into dunes due to the runup process.

Dune structures along exposed beaches are important since they serve as a line of defense to beachfront buildings, landward of them, from wave attack during intense storms (Marshall 2006). FEMA has developed a so called “540 rule” which can be used as a guideline to determine if a dune can be considered an effective barrier. The 540 rule is illustrated in Fig. 2 (MacArthur 2005). The amount of sand on the seaward side of the dune above the 100 year still water flood level in a two dimensional cross section is determined. If the area of this sand reservoir is greater than 500 square feet, then the dune can be considered to be an effect barrier in which case the landward extent of the V-Zone ends at the heel of the dune as illustrated in Fig. 2.

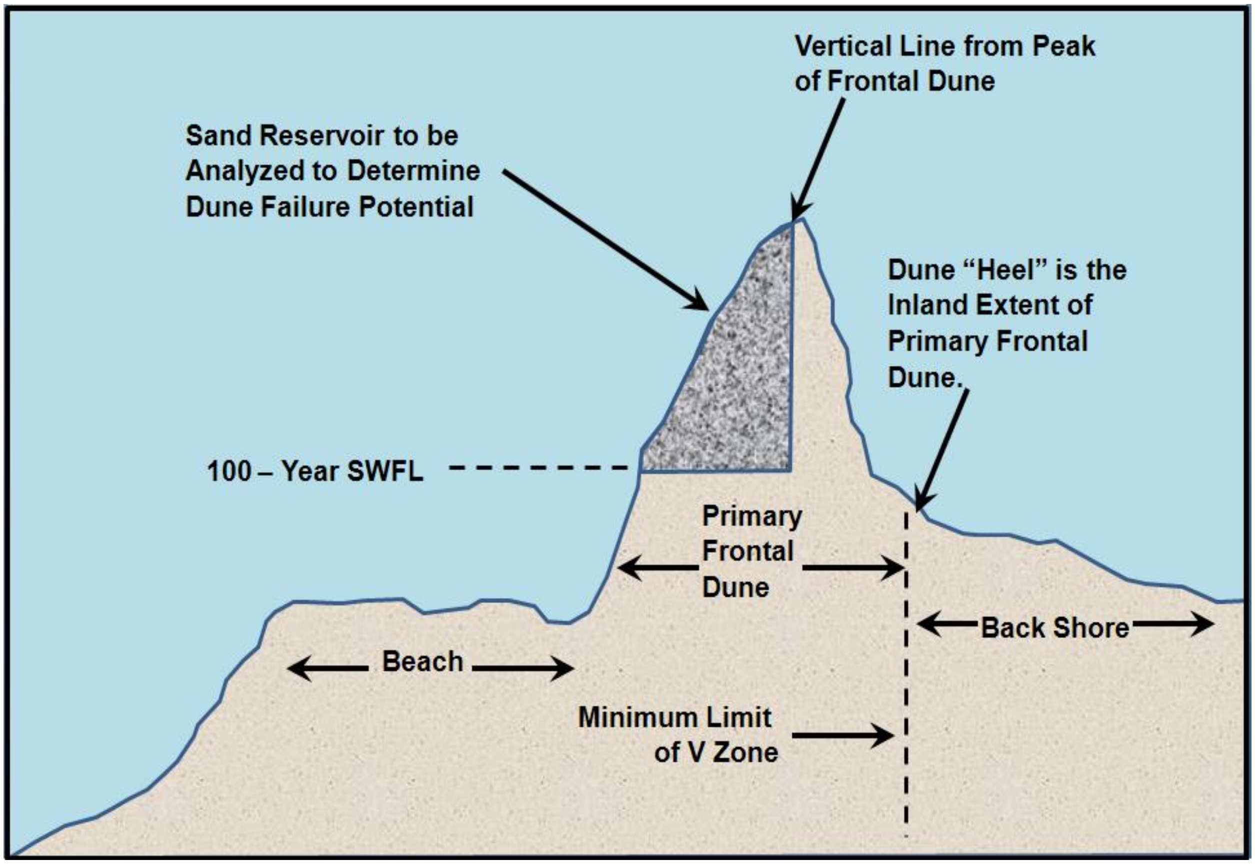

Figure 2. Determining Dune Failure Potential for “540 Rule”.

5. The Dune System under Wave Attack

Most beach front properties with an exposure to the open ocean are constructed behind some type of barrier. These coastal defenses can consist of a natural beach dune, stone revetment, bulkhead or concrete seawall. In this paper we will focus on the natural beach dune since it is a common type of defense from ocean waves on many beaches.

Storm elevated water levels combined with the momentum of large breaking waves will drive incursions of water high up onto the beach. This runup of water is in the form of turbulent bores known as *swash.* The maximum elevation attained by swash up the slope of the beach will be referred to in this paper as the *maximum runup level* $(\mathbf{R_2})$ (Fig. 1).

During a storm a number of different scenarios could occur depending on the elevation of the runup and setup levels. If the runup level were to push far enough up the beach it could either strike the front of the sand dune or completely overtop it. Given an even more intense storm, the setup level itself could overtop the dune.

Dune erosion can occur in a number of ways. Wave impact force against the face of a dune can result in cracks forming in the outer layer of sand which in turn results in this layer becoming unstable and eventually detaching from the main body of the dune. If wave impact is confined to lower face of the dune a notch can develop around the impact area resulting in undermining and eventual collapse of the front layer of the dune.

Sallenger (2000) developed a storm scaling model that addresses how a storm affects a dune structure. This scaling model defines two water levels, the $\boldsymbol{R_{high}}$ level which is a combination of the maximum runup level, the astronomical tide, and storm surge. The $\boldsymbol{R_{Low}}$ level is a combination of astronomical tide, storm surge, and wave setup. Both water levels are illustrated in Fig. 1. The scaling model is based on four regimes which are defined by the elevation of $\mathbf{R_{high}}$ and $\mathbf{R_{Low}}$. They are as follows:

Swash regime: Occurs when the $\mathbf{R_{high}}$ level remains below the toe of the dune and is the least severe in terms of (impact/erosion) of the regimes.

Collision regime: Occurs when the $\mathbf{R_{high}}$ level is above the toe of the dune but does not exceed the dune crest. In this situation erosion occurs as wave bores collide with the seaward side of the dune but its structure remains intact.

Overwash regime: Occurs when the $\mathbf{R_{high}}$ level overtops the dune. This process can transport water and debris to the landward side of the dune resulting in erosion, flooding, and some minor damage to structures behind the dune. Since the dune remains intact the greatest wave energy is blocked by the dune system.

Inundation regime: This regime is the most severe in terms of impact and occurs when the $\mathbf{R_{Low}}$ level rises above the dune crest. In this case the dune itself can be breached allowing water and debris carrying significant wave energy to impact structures on the landward side of the dune.

The challenge then is to predict the setup and runup levels in order to determine whether or not the dune will be breached.

6. Determining Wave Setup and Runup

A number of methods have been used to quantify wave setup and runup such as the Direct Integration Method (DIM), which was developed in conjunction with the Federal Emergency Management Agency sponsored Pacific Coast Guidelines (FEMA 2004). It is used to determine wave setup on beach slopes between Tan β = 0.17 and Tan β = 0.40.

A method developed for the Technical Advisory Committee on Flood Defence (TAW Method) was developed by van der Meer (2002) and is used to determine wave runup on relatively steep slopes.

Mase (1989) developed a method to determine runup on more gently sloped beaches.

Stockdon et al. (2006) developed a parameterization using water level time series collected during 10 separate field experiments on both the Atlantic and Pacific Coasts of the United States and also a location in the Netherlands. This is a much more comprehensive tool in that it simultaneously predicts maximum runup level as a sum of both wave setup and swash. It has been extensively tested in hindcasts of Hurricanes Bonnie, Floyd, Ike, and Ivan. Real time forecasts have been performed with this tool in every season since Hurricane Ivan. Another unique feature is the inclusion of infragravity waves in the derivation of this empirical expression. Infragravity waves are long period waves with a period ranging from 3 to 30 minutes and are generated in areas of differential radiation stress as swell transits from a fetch generation area to the coast. It was for these reasons that the focus of this paper was placed on this parameterization as a tool to compute the maximum runup.

The parameterization is presented in Equation 1 while Equation 2 computes only the wave setup.

$$\boldsymbol{R_2} = 1.1\left(0.35\boldsymbol{B_f}\,(\boldsymbol{H_0}\,\boldsymbol{L_0})^{1/2} + \frac{\left[H_0\,L_0\left(0.563\beta_f^2 + 0.004\right)\right]^{1/2}}{2}\right) \quad \textbf{(1)}$$

$$\boldsymbol{\eta} = 1.1\left(0.35\boldsymbol{B_f}\,(\boldsymbol{H_0}\,\boldsymbol{L_0})^{1/2}\right) \quad \textbf{(2)}$$

Where:

$\boldsymbol{R_2}$ = 2 percent probability of exceedance level
$\boldsymbol{B_f}$ = foreshore beach slope
$\mathbf{H_0}$ = deep water wave height
$\boldsymbol{L_0}$ = wave length
$\boldsymbol{\eta}$ = wave setup

For purposes of simplification in testing the parameterization the foreshore beach slope ($\boldsymbol{B_f}$) is measured from the MHHW tide level to the toe of the dune. In the event that the parameterization was to be modeled this zone can be further refined by using the area between the dune toe and the still water level (SWL). The SWL is defined as the hypothetical water level that would exist in the absence of waves and would consist of the astronomical tide plus storm surge but not wave action. Individual parts of the parameterization are described in Fig. 3.

An operational tool utilizing the Stockdon Parameterization described in this paper has already been incorporated into some National Weather Service work stations as a test program. An Excel program based on the parameterization has also been developed[1]. The purpose of these preliminary programs is to test the parameterization on a variety of beach and storm conditions at different National Weather Service Offices. The ultimate goal is to produce a much more robust runup model that will have the capability of automatically ingesting astronomical tide data, output from storm surge models, output from circulation models, and wave model data. The look of output from such a model is still in the discussion stage however it would most likely be in graphic form with greater detail in enhanced runup areas. Wave setup and runup could be displayed in graphic form either as a deterministic or probabilistic product. Such a product could then be used for predicting areas of high velocity water impact.

[1] For more information on the tool or the parameterization please contact the author at Anthony.mignone@noaa.gov or NWS Eastern Region SSD.

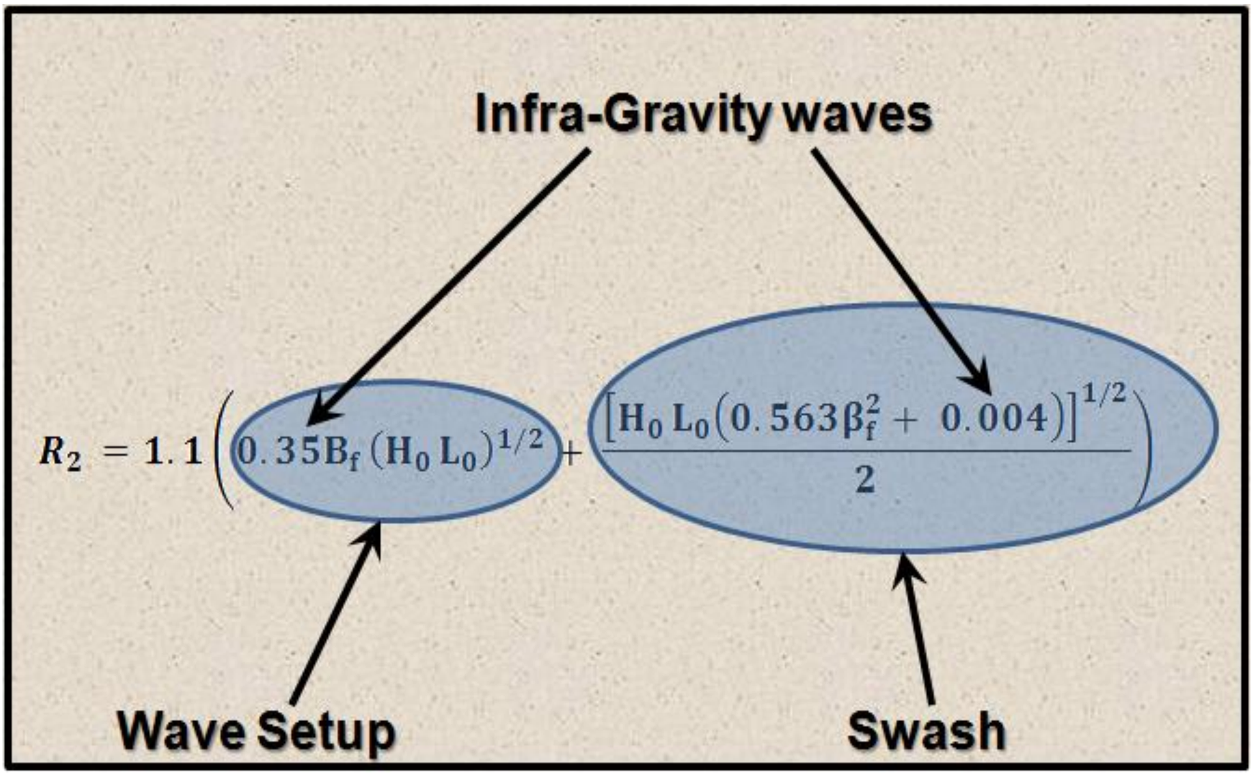

Figure 3. Parts of the Stockdon Parameterization.

7. Wave Loads on Structures

Once a dune system, such as depicted in Fig. 2, is breached, infrastructure located behind the dune is vulnerable to storm waves. After a wave breaks in the surfzone the remaining wave energy travels up the slope of the beach in the form of a broken wave bore. The bore will continue to travel up the beach until its energy is completely dissipated at the $\boldsymbol{R_2}$ level predicted by Equation 1. Approximate force produced by the wave bore striking an intervening structure on the beach can be determined using Equation 3.

When a wave strikes a structure there are a number of different wave forces involved. They include the dynamic force, dynamic overturning moment, hydrostatic force, and hydrostatic overturning moment. Hydrostatic force results from the increase in pressure force with water depth. Since the height of the wave bore striking the structure is relatively shallow both hydrostatic components are also small and therefore can be discounted.

Dynamic overturning moment is the rotation of a structure on its heel (Fig. 4) caused by a force applied to the opposite side of the building. As was the case with the hydrostatic forces the height (h) of the wave bore is relatively shallow and well below the center of gravity of the structure. Therefore dynamic overturning moment can also be neglected.

The main contributor, Dynamic force ($\boldsymbol{F_{surge}}$), is the pressure exerted against the front of the structure resulting from the wave striking it. It can be determined using Equation 3 below from the U.S. Army Corps of Engineers Coastal Engineering Manual (USACE 2002); units are in lbs/ ft^2 per linear foot of a structure.

$$F_{surge} = 0.18WH_b{}^2\left(\frac{1 - x_1 \, Tan\beta}{R_2}\right) \quad (3)$$

W = weight of 1 ft^3 of seawater or 64.0 lbs.
H_b = wave breaking height
β = beach slope
x_1 = horizontal distance from DWL to structure that is being struck by the wave bore
R_2 = Two percent runup level

The runup level R_2 level will be computed with the Stockdon Parameterization.

The broken wave bore height (h_c) at the DWL is computed with Equation 4 (Camfield 1991) and illustrated in Fig. 4. The height of the wave bore as it reaches the structure (h) can be calculated with Equation 5.

$$h_c = .2H_b \quad (4)$$

$$h = h_c\left(1 - \frac{x_1}{x_2}\right) \quad (5)$$

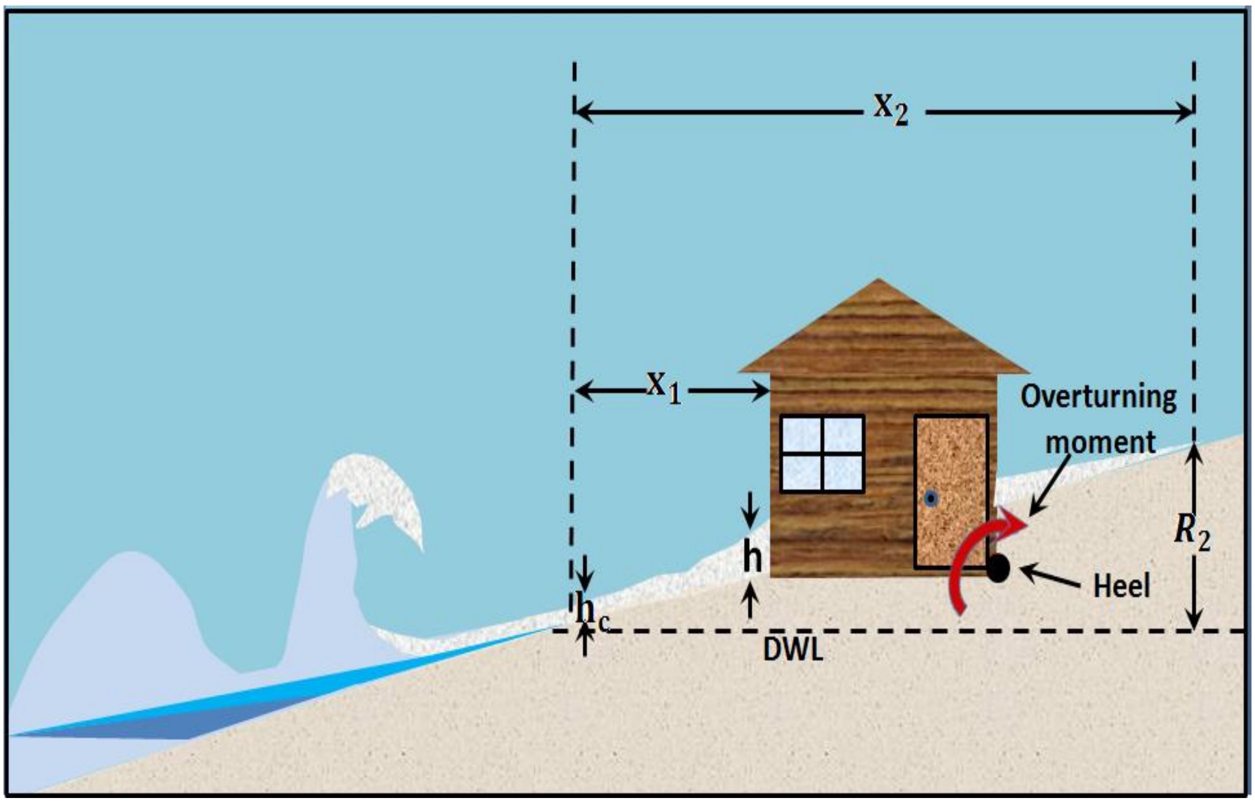

Figure 4. Wave runup on structure after dune destruction.

8. Comparing Wind and Wave Forces on Structures.

For the purpose of demonstration we will compute wave force on a hypothetical structure along the beach that is being struck by waves running up onto the shore. We will consider deep water waves ranging in height from 5 to 15 feet (1.5 to 4.6 m) with a period of 14 seconds. It is assumed that a protective dune system is either not present or has been eroded by wave action. Broken wave bores are surging up the beach and striking the side of the building depicted in Fig. 4 to a height of h.

The Stockdon Parameterization (Eq. 1) is first used to compute the runup level ($\mathbf{R_2}$). Next wave force on the side of a water front building is computed using Equation 3 assuming the breaking wave height is equal to the deep water wave height. Equation 4 is then used to compute the height of the wave bore (**h**) as it is striking the side of the building. Hypothetical wind forces are then computed using techniques described in ASCE (2010). Finally, for comparison and better visualization purposes, wave and hypothetical wind forces are plotted side by side in Fig. 5. Force units are in lbs/foot per length of the building.

As a comparison example, runup from a 14 foot (4.27 m) breaking wave would produce a force of 668 lbs/foot (303 kg/0.31 m) for each foot length of the building while the height of the bore **h** striking the building is only 2 feet (0.61 m) high. Marshall (2006) points out that 1 foot (0.31 m) of water traveling at 10 miles per hour (16.1 km/h) will produce the force equivalent to 280 mph (450.6 km/h) wind. Obviously a small amount of water traveling with any velocity, in comparison requires extremely high wind speed to match its destructive force. The longer the wave period the faster waves travel and therefore carry more kinetic energy up the beach.

Emphasis therefore needs to be placed on the potency of inundation water that is moving at high velocity. This point was best exemplified by Kennedy (2011) during Hurricane Ike. Structures exposed to less than 2 feet (0.6 m) of high velocity water from ocean waves were totally destroyed while others with inundation levels up to the roof survived the hurricane.

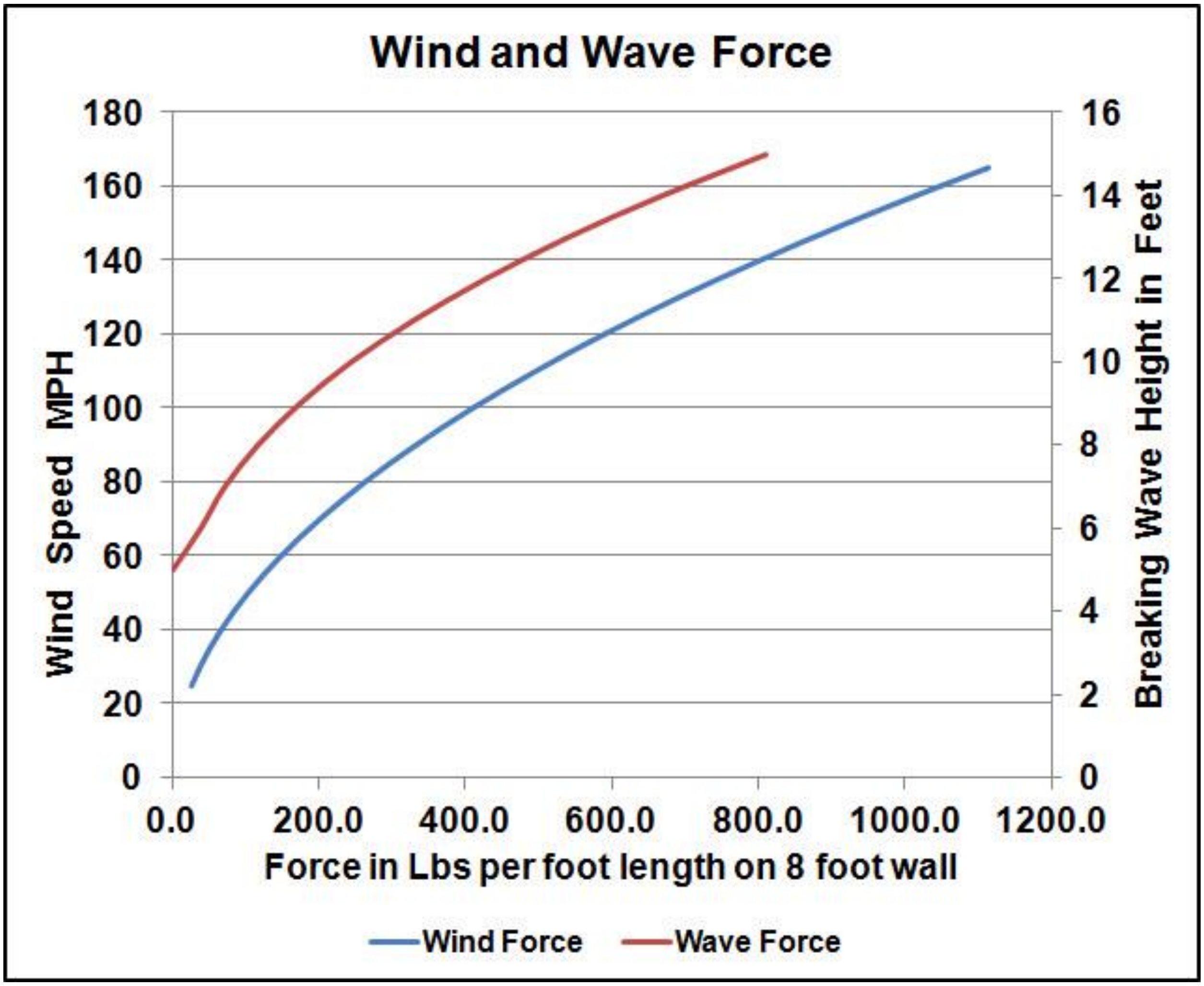

Figure 5. Comparison of wind and wave forces.

9. Overwash along Seawall Road during the 6-9 March 2013 Storm

The following event review describes the operational use of the Stockdon Parameterization to compute runup against Seawall Road which runs adjacent to the ocean. Although the seaward side of the road is armored with cobble stone and therefore not easily susceptible to erosion, a high volume of overtopping water could result in failure of the roadway in a similar way as a dune system.

Seawall Road is located near Southwest Harbor, Maine. Overwash occurred at this site over a two day period at the times of high tide from 0000 UTC 8 March to 0200 UTC 10 March. The Stockdon Parameterization was tested on this event using an excel program. The following is a review of how the predictors needed for this tool were determined and the results of the calculations.

On the morning of 6 March 2013 low pressure developed along the North Carolina Coast. This system would subsequently intensify to 985 mb during the next two days as it moved east northeastward into the Atlantic. The low would remain intense through 9 March 2013. During this same time period strong high pressure would build in a large semi-circle extending from the Mid-Western US, northeastward across Quebec Province, eastward across the Maritimes then southeastward into the Atlantic (Fig. 6). The juxtaposition of these two systems would result in two distinct wave fetch generation areas, one across the Gulf of Maine and a second extending from south of Nova Scotia east-southeastward into the Atlantic.

Astronomically during this period the moon was in transition from the last quarter (5th) to a new moon (11th). Therefore high tide levels were near the highest levels of the month due to an approaching spring tide. Since the moons perigee had already occurred on the 5th, astronomical alignment was not properly in phase for the occurrence of a Perigean Spring tide which would have resulted in even greater potential for runup.

Since output from operational wave models are in terms of combined wave height it is not always suitable to use them directly into calculations for runup. Combined wave height therefore must first be partitioned into its component wave systems to determine the proper wave group to use for calculations. Wave model spectral output was therefore utilized to accomplish this task. The spectral point CAR01 from the Global WAVEWATCH III, which is the closest spectral point to the Seawall Road, was used to partition wave groups for this event. This point is located approximately 43 nm (79.6 km) off the coast to the east-southeast. (Note: An online source for spectral bulletins is available from NCEP (2013). If the reader is not familiar with this product a review of Analyzing Ocean Swell (COMET 2013) is strongly recommended).

Spectral information from CAR01 for 0000 UTC 8 March 2013 (time of the first observed overwash) is depicted in Fig. 7. Combined wave heights of 14.1 feet (4.29 m) are forecasted for the approximate time of high tide. This can be partitioned into two primary wave groups. Group 1 consists of waves 10.6 feet (3.23 m) high that travel in a direction of 236° (056°) with a period of 7.7 seconds. Group 2 consists of waves 9.2 feet (2.81 m) high that travel in a direction of 312° (132°) with a period of 13.9 seconds. (Note: directions from this bulletin are in oceanographic convention).

The two spectral wave groups partitioned above appear to fit well with the synoptic conceptual model. Wave group 1 is generated from the northeasterly fetch in the Gulf of Maine while the source of wave group 2 is the east-southeast fetch area north and east of the low center.

Comparison of model spectral data to observations from nearby directional and spectral equip buoy reports, in this case Jeffrey's Ledge (44098), is then made. Fig. 8 depicts the spectral density (wave energy) plotted against period and direction for 0000 UTC 8 March at Jeffrey's Ledge. Although this buoy is located a significant distance south it is useful in terms of validating the existence of two distinct wave groups of different period and direction which can clearly be seen in the buoy observations.

The final step in the process is to determine which wave group would pose the greatest impact since the effect of multiple wave groups on runup is not cumulative. Intuitively one would select the group with the largest waves but this is not always the correct approach.

Wave group 1 contains the largest waves however the wave period is 7.7 seconds and in addition the wave direction is from 056° which is significant at the buoy site 40 nautical miles off shore but is an off-shore direction at the runup site. Group 2 waves are arriving from an east-southeasterly direction and have a much longer period and therefore travel faster resulting in more wave momentum. Wave group 2 is therefore used to perform the runup calculations. When in doubt, different groups can be tested in the parameterization to determine what yields the highest impact.

The astronomical tide is determined from the NOAA Tides and Currents Web site (NOAA 2013). The tide level from the Bar Harbor Tide Gauge, located only a short distance from this hot spot, was used for calculations.

Storm surge is determined from the Extra-tropical Storm Surge or similar model. As an alternative the Extra-tropical Surge and Tide Operational Forecast System (ESTOFS) which is based on the Advanced Circulation model (ADCIRC) is available as a GFE grid can be used for this portion of the calculations.

Utilizing the techniques described above to obtain input data, calculations for 0000 UTC 8 March, the time of high tide, are entered into the wave calculator. A wave height of 8.4 feet (2.56 m), period of 13.5 seconds, the predicted astronomical tide of 10.55 feet (3.21 m), and surge of 0.03 feet (0.01 m) is entered into the runup calculations and the output is depicted in Fig. 9. Overwash is highlighted in yellow while inundation is highlighted in red.

As the synoptic situation evolved during the next two days, the northeast winds in the Gulf of Maine continued to back into an off-shore flow. At the same time the off-shore fetch south and east of Nova Scotia remained intact generating larger and longer period waves. The wave field from the WAVEWATCH III model analysis at 1200 UTC 8 March is depicted in Fig. 10 and highlights the size of this fetch which will result the arrival of long period waves for the next several days. Calculations for the remainder of the high tide cycles were carried out in a similar manor for this site as described above. A summary of the calculations from the parameterization for each time period and transect at Seawall Road for the next four high tides is presented in Table 1.

The calculations predict overwash of the road in 5 of the 6 transects. Inundation is indicated on the sixth transect. These calculations are consistent with the impact to the roadway structure which was minimal. Overwash of the roadway, whose base is reinforced with baskets of large rocks, did not result in any structural damage. Inundation at transect 6 was potentially problematic since water could have been forced across the road surface at high velocity resulting in the establishment of a standing wave in the revetment on the opposite side of the road. With a predicted setup level of 16.09 feet as opposed to the road elevation at this point of 15.98 feet, the flow across the road was not sufficient to do this. During the high tide cycle on the mornings of the 8th and 9th local officials closed the road at the time of high tide and rocks were cleared off the road surface using snow plows after the tide level receded. During the evening high tides on the 7th, 9th, and 10th only minor overwash occurred with minimal amounts of debris deposited on the road.

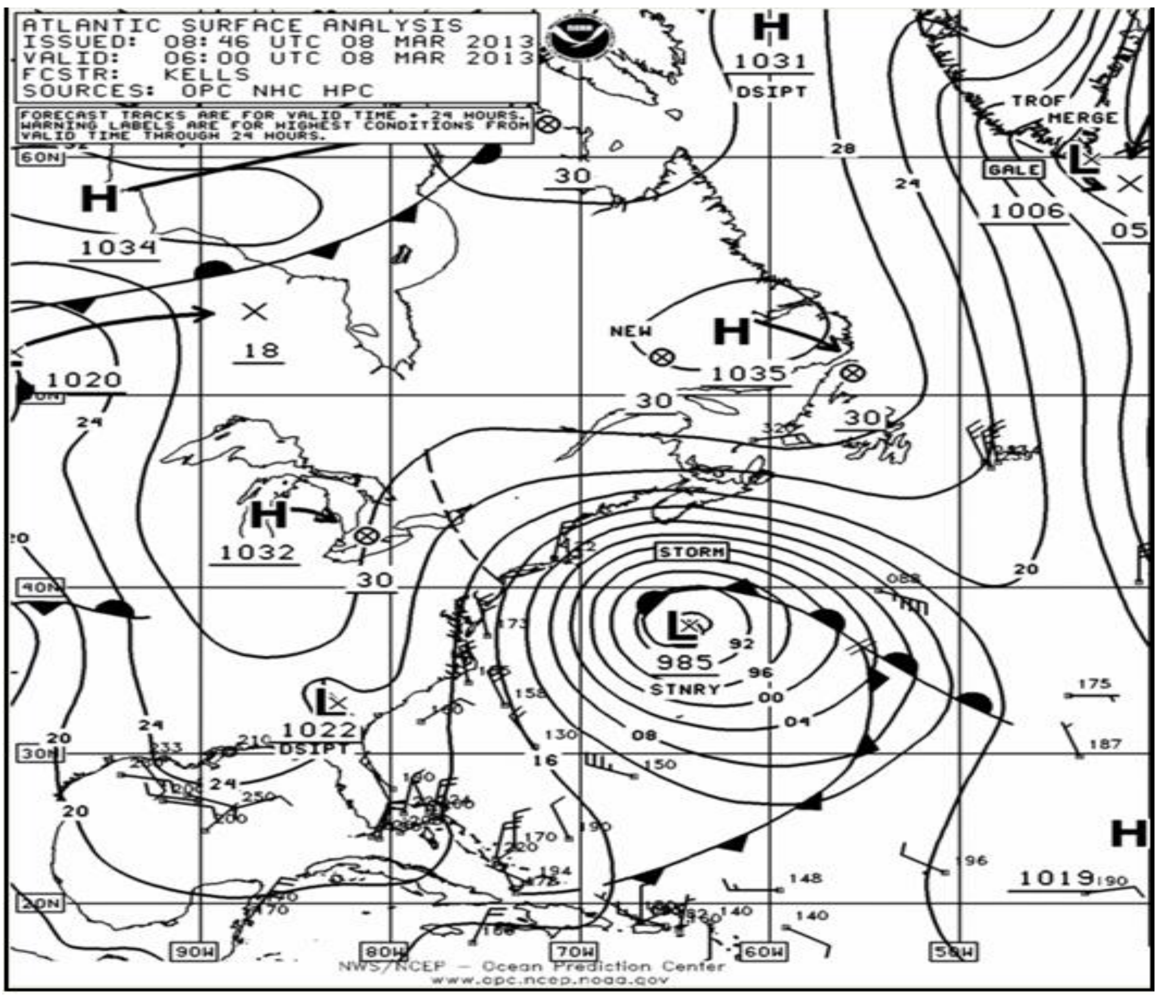

Figure 6. Surface analysis valid at 0600 UTC 8 March 2013 (OPC 2013)

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Location : CAR01      (44.00N  67.50W)
Model    : spectral resolution for points
Cycle    : 20130308  0 UTC
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day & hour	Hst (m)	n x	Hs (m)	Tp (s)	dir (d)	Hs (m)	Tp (s)	dir (d)	Hs (m)	Tp (s)	dir (d)
7 15	3.21	2	* 2.52	6.7	244	2.00	12.4	310			
7 16	3.29	2	* 2.55	6.8	242	2.08	12.5	310			
7 17	3.35	3	* 2.54	6.8	240	0.87	13.8	316			
7 18	3.40	3	* 2.56	6.8	240	0.97	13.8	316			
7 19	3.49	2	* 2.56	6.8	239	2.36	13.2	310			
7 20	3.60	3	* 2.62	6.8	240	2.47	13.4	310	0.25	13.3	8
7 21	3.71	4	* 2.66	6.8	239	2.57	13.5	310	0.18	13.4	357
7 22	3.75	4	* 2.71	6.9	240	2.57	13.5	311	0.17	13.2	355
7 23	3.73	4	* 2.66	6.9	238	2.58	13.5	311			
8 0	3.69	4	* 2.60	6.9	237	2.56	13.5	311	0.40	12.4	17
8 1	3.68	4	* 2.60	6.9	237	2.54	13.6	311	0.48	11.2	15
8 2	3.73	4	* 2.67	6.9	238	2.53	13.6	311	0.50	11.1	15
8 3	3.79	4	* 2.77	6.9	238	2.51	13.7	311	0.55	11.3	14
8 4	3.86	4	* 2.89	7.0	240	2.47	13.9	312	0.55	11.2	15
8 5	3.91	4	* 2.98	7.2	240	2.44	13.9	312	0.55	11.1	15
8 6	3.97	5	* 3.05	7.3	240	2.45	13.8	312	0.32	12.3	20
8 7	4.04	3	* 3.10	7.4	239	2.52	13.8	312			
8 8	4.13	5	* 3.16	7.4	239	2.59	13.7	312	0.36	10.7	16
8 9	4.21	3	* 3.20	7.5	238	2.67	13.8	312			
8 10	4.26	4	* 3.22	7.6	237	2.73	13.9	312	0.39	10.9	18
8 11	4.28	2	* 3.23	7.7	237	2.81	13.8	313			
8 12	4.29	3	* 3.23	7.7	236	2.81	13.9	312	0.32	10.4	19
8 13	4.29	3	* 3.24	7.7	236	2.80	13.8	312	0.28	10.4	18
8 14	4.28	2	* 3.22	7.7	235	2.82	13.9	312			
8 15	4.26	2	* 3.21	7.7	235	2.80	13.9	312			
8 16	4.24	2	* 3.19	7.7	234	2.79	13.9	312			
8 17	4.21	2	* 3.17	7.7	233	2.77	13.9	312			
8 18	4.18	2	* 3.15	7.7	232	2.76	14.0	312			
8 19	4.15	2	* 3.13	7.7	232	2.72	14.0	312			

Figure 7. Output from the WAVEWATCH III text bulletin (NCEP 2013).

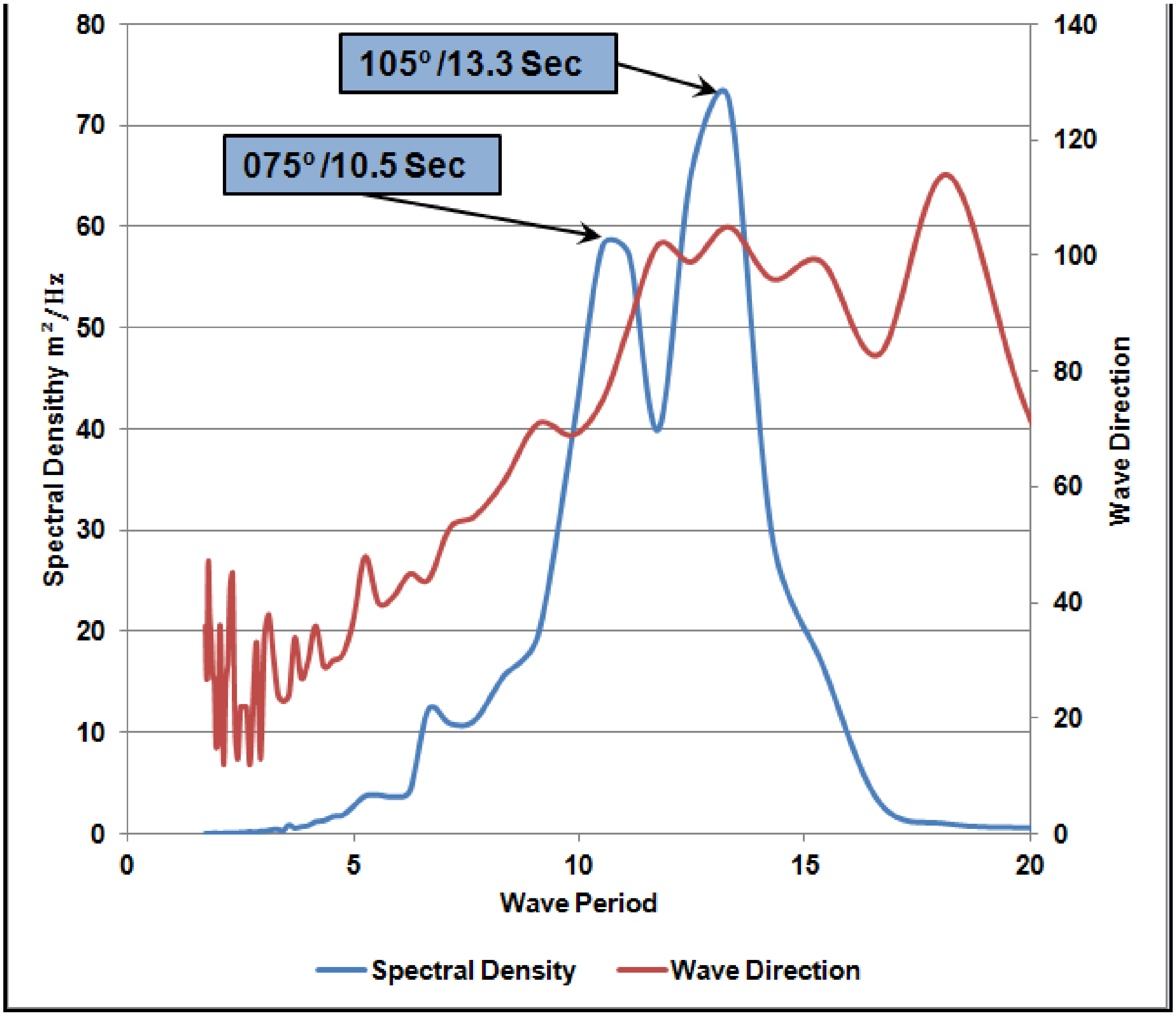

Figure 8. Spectral density from Jeffrey's Ledge (buoy 44098) valid at 0000 UTC 8 March 2013.

Input parameters	Feet
Deep water wave height	9.22
Deep water wave period	13.90
Tide in Feet above MLLW	11.72
Storm Surge in Feet	1.22

Runup and Setup Calculator For the Seawall Road

Transect 1 Elevation of Road = 18.98 ft MLLW		
Max Setup Level	R_{low}	**16.48**
Max Runup Level	R_{high}	**20.69**
Max Runup	$R_{2\%}$	**7.75**
Erosion		**Expected**
Overwash		**Expected**
Inundation		**Not Expected**

Transect 2 Elevation of Road = 22.98 ft MLLW		
Max Setup Level	R_{low}	**16.44**
Max Runup Level	R_{high}	**20.61**
Max Runup	$R_{2\%}$	**7.67**
Erosion		**Expected**
Overwash		**Not Expected**
Inundation		**Not Expected**

Transect 3 Elevation of Road = 16.98 ft MLLW		
Max Setup Level	R_{low}	**16.46**
Max Runup Level	R_{high}	**20.65**
Max Runup	$R_{2\%}$	**7.71**
Erosion		**Expected**
Overwash		**Expected**
Inundation		**Not Expected**

Transect 4 Elevation of Road = 17.98 ft MLLW		
Max Setup Level	R_{low}	**15.95**
Max Runup Level	R_{high}	**19.66**
Max Runup	$R_{2\%}$	**6.72**
Erosion		**Expected**
Overwash		**Expected**
Inundation		**Not Expected**

Transect 5 Elevation of Road = 16.98 ft MLLW		
Max Setup Level	R_{low}	**15.83**
Max Runup Level	R_{high}	**19.43**
Max Runup	$R_{2\%}$	**6.49**
Erosion		**Expected**
Overwash		**Expected**
Inundation		**Not Expected**

Transect 6 Elevation of Road = 15.98 ft MLLW		
Max Setup Level	R_{low}	**16.09**
Max Runup Level	R_{high}	**19.94**
Max Runup	$R_{2\%}$	**7.00**
Erosion		**Expected**
Overwash		**Expected**
Inundation		**Expected**

Figure 9. The CAR wave runup and setup calculator for Seawall Road at 0000 UTC 8 March 2013.

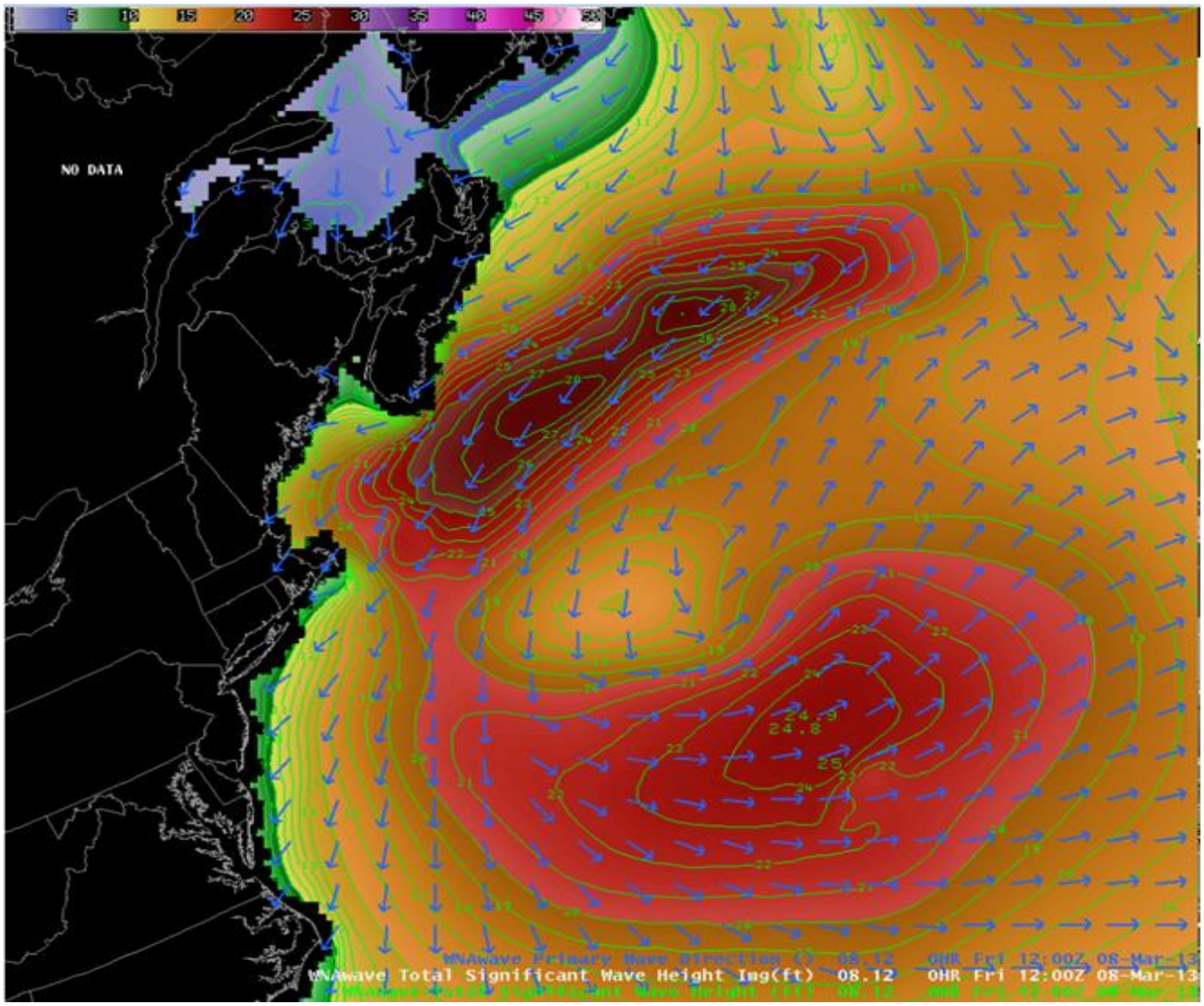

Figure 10. WAVEWATCH III Analysis of wave heights (ft) and primary wave direction valid at 1200 UTC 8 March 2013.

Table 1. Summary of overwash and inundation on Seawall Road from wave calculator as a function of wave height and period along with water elevation.

Date Time	Wave Ht Feet	Period Sec	Tide Feet	Surge Feet	Transects 1	2	3	4	5	6
08/0006Z	8.40	13.50	10.55	0.03			O			O
08/1230Z	9.20	13.90	11.72	1.22	O		O		O	I
09/0112Z	8.01	14.00	11.04	1.05	O		O	O	O	O
09/1330Z	7.05	14.70	12.06	1.01	O		O	O	O	I
10/0200Z	5.28	13.60	11.56	0.16			O			O

Overwash = O

Inundation = I

10. Summary and Recommendations

Importance of the astronomical tide cannot be overemphasized when considering damage potential from wave forces. Coincidence of the time of high tide and arrival of the largest energy waves can make the difference between minor and catastrophic damage.

Duration of the storm is also very important since a long duration event can eclipse multiple tidal cycles. This increases the chance of coincidence of high energy waves with an exceptionally high tide cycle. It also increases the duration of wave attack. Structures which are weakened during the first tidal cycle can be further damaged or destroyed in subsequent tidal cycles.

Wave period is extremely important. Long period waves travel faster than shorter period waves and therefore carry more momentum into the surf zone and consequently carry a much higher impact when running up onto the shore.

As is frequently the case, coastal inundation and resulting wave damage is often maximized in specific areas where there is a large density of structures, minimal natural or manmade defense against wave attack, low elevation relative to ocean level, a narrow beach buffer zones and exposure to large ocean waves. This concept can however be utilized to concentrate wave forecasting resources to these specific problematic areas or hot spots thus allowing enhanced forecasting procedures to be developed for these high impact areas.

Currently at the Caribou Weather Forecast Office we are in the process of testing the techniques in this paper for Seawall Road near Southwest Harbor, Otter Cove in Arcadia National Park, and 8 specific areas that commonly over-wash near Schoodic Point. All of these specific areas were singled out as hot spots based on the high frequency of over-wash or storm surge related flooding over the last several years. As a rule hot spots are often determined by past histories of damage resulting from intense storms. FitzGerald (1994) provides an excellent example of describing high impact areas in Eastern Massachusetts resulting from the Halloween Eve Storm of 1991.

Once a specific hot spot has been designated, a detailed site survey of this area can be made. The site survey consists of differential leveling to determine the beach slope and the elevation of dunes and adjacent structures above reference levels such as the NAVD88 or MLLW datum. Use of Lidar elevation data available from the USGS Web site (USGS 2013) and plotted using ArcGIS is most useful in this process. Unfortunately the LIDAR Data is not available for all coastal areas but the USGS will make available more areas as data is collected for other projects. The LIDAR Data does not replace a site survey as horizontal resolution is not adequate to resolve sharp discontinuities such as seawalls and revetments but it can be used as a quick reference to isolate the most vulnerable areas along with providing beach slope and elevation data for wide areas of coastline. An example of such a Lidar map is presented in Fig. 11 where elevation in feet has been plotted over imagery. During this survey the structural integrity of the natural or manmade defensive structures are also noted.

It is recommended that a local wave model be utilized to compute wave heights as close to the surf zone as possible. This will compensate for loss of wave energy in transit from further off the coast. Once deep water wave height and period have been determined the $\boldsymbol{R_2}$ runup and the $\boldsymbol{\eta}$ setup levels can be computed using the Stockdon Parameterization. The Sallenger Scaling Model can then be employed to evaluate danger to the beach dune system.

Concepts discussed within this paper can also be employed while conducting a site survey after a storm event. A distinction can be made to determine if damage was the result of wave or wind action. If damage is low on the structure the damage will likely be a result of wave action. If it is high on the structure it probably resulted from wind force. Customers may require this information after a storm since insurance policies typically cover wind and wave damage separately (Marshall 2006).

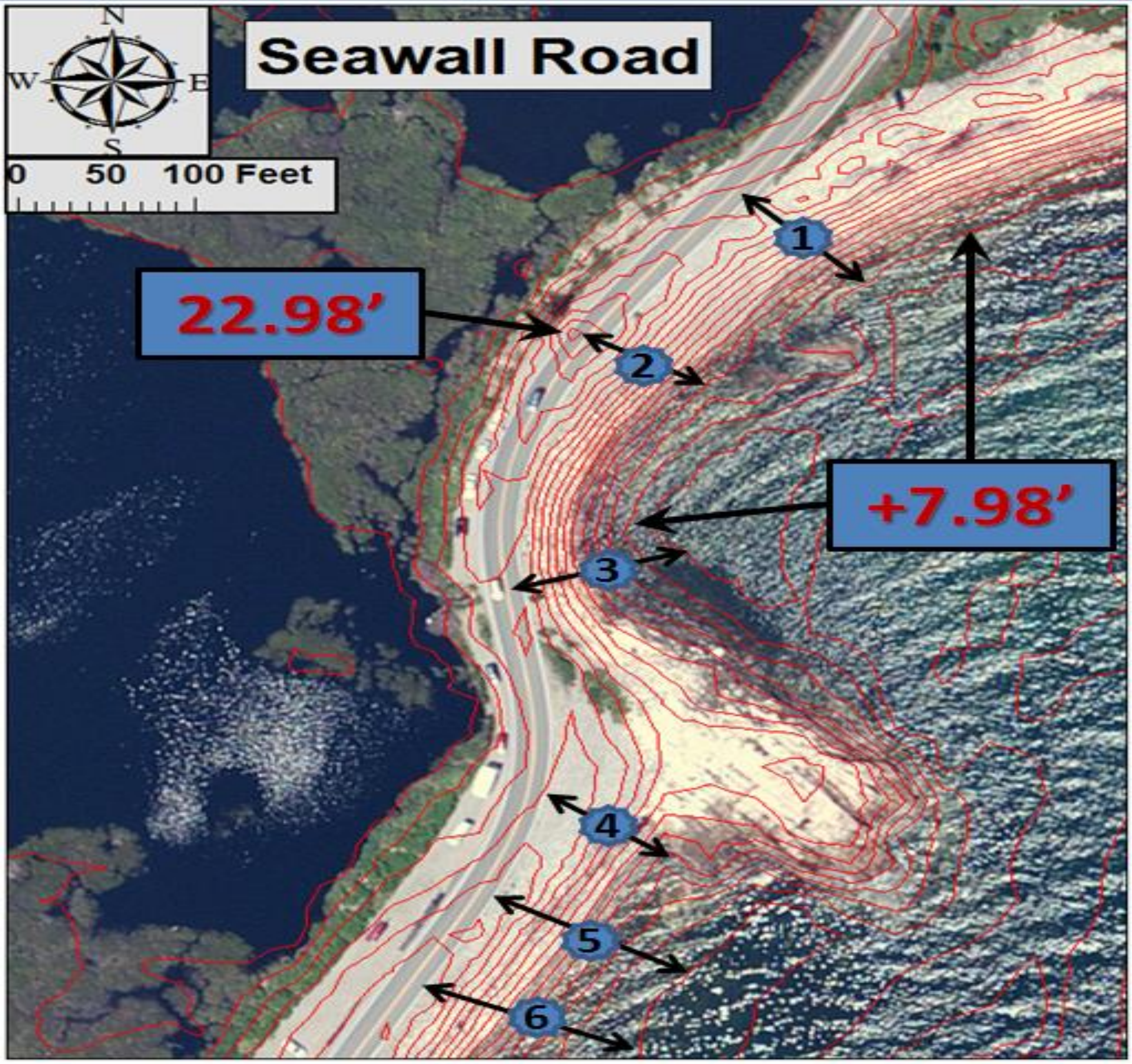

Figure 11. Lidar map of Seawall Road near Southwest Harbor, ME. Elevation in feet and datum in MLLW. Transects labeled 1-6.

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