Acoustic Measurement and Modeling of Waves in Estuarine and Coastal Environments

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ACOUSTIC MEASUREMENT AND MODELING OF WAVES
IN ESTUARINE AND COASTAL ENVIRONMENTS

A Thesis Presented to
The Faculty of the School of Marine Science
The College of William and Mary

In partial fulfillment of the requirements for the degree of
Master of Science

______________________________
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May 14, 2007

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This thesis is submitted in partial fulfillment of the requirements for the degree of

Master of Science

May 2007

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ACOUSTIC MEASUREMENT AND MODELING OF WAVES
IN ESTUARINE AND COASTAL ENVIRONMENTS
Chapter 1

Overview
1 OVERVIEW

Accurate estimates of wave parameters in real-time operational deployments and numerical modeling studies are becoming increasingly important in the coastal zone, not only for navigation and search and rescue operations, but also for recreational boaters and fishermen. Wave climate is important for sediment transport studies (e.g., Boon et al., 1996), the engineering design of structures, nutrient and oxygen exchange, and interactions with aquatic vegetation (e.g., Doyle, 2001; Kobayashi et al., 1993; Mork, 1996; Riis and Hawes, 2003). Resuspension of sediment during high-energy wave events can act to mobilize sediment for subsequent transport by tidal currents, especially in shoal regions (Sanford, 1994). Additionally, waves can act to modify surface drag coefficients, which has important implications for wind-driven flow and turbulent mixing in estuaries.

Locally generated wind waves in estuaries provide a unique challenge for typical methods of wave measurement such as bottom-mounted pressure and velocity sensors because the short period wave signal is quickly attenuated with depth (Pedersen et al., 2005). It is perhaps for this reason that a scarcity of reliable wave data in estuaries exists today. This challenge can be overcome either by making direct measurements of the sea surface elevation or by making remote measurements of surface elevation and wave orbital velocities using acoustic Doppler techniques. The Nortek Acoustic Wave and Current Meter (AWAC) and Teledyne RD Instruments Acoustic Doppler Current Profiler (ADCP) are two such instruments that show promising potential for wave and current measurements in estuarine observing systems.

Much of the historical literature on wave theory and measurement focuses on wave characteristics in coastal or open ocean environments and measurement and
analysis techniques share a similar bias. In contrast to the open ocean, estuaries are typically shallow, narrow, fetch-limited, and display complex and irregular bathymetry and currents – characteristics that make prediction and measurement of estuarine waves difficult (Barthel and Ing, 1982). Estuarine waves typically respond to synoptic-scale frontal systems as opposed to large-scale trade winds or ocean storms (Boon et al., 1994; 1995). As a result, wind and wave events in estuaries coincide so that waves are nearly always “forced” as opposed to freely propagating swell from distant regions, except in the vicinity of the estuary mouth where ocean swell may be important (Boon et al., 1996). These characteristics make estuarine waves inherently different from other environments and a complete description of estuarine wave characteristics is lacking. This study will attempt to address some of these issues regarding the measurement of waves in estuaries and offer a characterization of waves at multiple locations in the York River estuary, a tributary of Chesapeake Bay.

The research presented in this project falls into four sections. The first section is an evaluation of wave height parameter estimates (Chapter 3). This section examines features of the wave field that influence the agreement between the significant wave height derived from the spectrum, $H_{m0}$, and from zero-crossing analysis, $H_{1/3}$, as well as processes governing the relationship between significant and maximum wave height. This section was originally prepared as a stand-alone manuscript for journal publication and retains much of that same structure. As a result, it may contain some redundant discussion of background literature. The second section describes the wind and wave climate of the York River estuary (Chapter 4). The third section deals primarily with characterizing the unique nature of estuarine waves with the goal of assessing the
commonly held assumption that wave heights follow the Rayleigh distribution (Chapter 5). The final section assesses the applicability of a steady-state numerical wave model for use in the Chesapeake Bay tributaries and other similar environments (Chapter 6).
CHAPTER 2

BACKGROUND
2 BACKGROUND

2.1 Ocean Waves

At any point on the ocean’s surface, the observed fluctuation in water level can be thought of most simply as a linear superposition of a large number of independent components. Given the apparent randomness of the process, parameterization of wave characteristics can be achieved only through stochastic analysis of the sea surface, which spans three domains: (1) time, (2) frequency, and (3) probability (Massel, 1996). Zero-crossing analysis of the surface elevation in the time domain provides a direct measure of individual wave heights and periods and allows explicit determination of such parameters as significant wave height \( H_s \), maximum wave height \( H_{\text{max}} \), and mean zero-crossing period \( T_z \). Wave parameters are derived from a record by sorting the individual wave heights and periods and averaging some fraction of the total to obtain parameter estimates. A set of representative wave heights of the form \( H_p \), the average of the \( pN \) largest waves, can be determined, where \( 0 < p \leq 1 \) and \( N \) is the total number of waves. For significant wave height \( H_{1/3} \), \( p = 1/3 \). While this procedure provides insight into the bulk statistics of the wave field, it is incapable of describing more complex features such as spectral shape or multiple wave trains.

The directional spectrum fully describes the way in which wave energy is distributed at various frequencies and directions. Therefore, it offers a more complete description of the wave field and allows one to examine the spectral evolution over time. Using linear wave theory, it is possible to infer surface wave characteristics through indirect means, such as bottom-mounted pressure \( (p) \) and horizontal velocity \( (u,v) \) time
series (i.e. the PUV method). It is then possible to arrive at many of the same parameters as from zero-crossing analysis such as $H_{rms}$, an energy-based significant wave height, and $T_m$, the mean period. Other parameters are unique to the spectral analysis, such as $T_p$ and $D_p$, the period and direction at the spectral peak. Additional parameters such as spectral width and directional spread describe how energy is distributed among different frequencies and directions. One drawback of such measurements is the exponential attenuation of the pressure and velocity signal with depth below the surface. The result is that high frequency components of the wave field are indistinguishable from background noise at depth, so the measurement itself acts as a low-pass filter of the actual sea surface variation. Measurement techniques to avoid these limitations will be discussed later in Section 2.2.

The probability domain treats the particular wave parameters as elementary random variables to arrive at probability density functions and statistical moments that describe the wave field. Longuet-Higgins (1952) first applied the statistics of random signals to ocean waves and showed that for deep-water narrow banded spectra wave amplitudes follow the Rayleigh distribution. The probability density function (pdf) and cumulative distribution function (cdf) are given by:

$$p(\hat{H}) = \frac{2\hat{H}}{H_{rms}^2} \exp \left[ -\left( \frac{\hat{H}}{H_{rms}} \right)^2 \right]$$  \hspace{1cm} (Eq. 2-1)

$$p(H < \hat{H}) = 1 - \exp \left[ -\left( \frac{\hat{H}}{H_{rms}} \right)^2 \right]$$  \hspace{1cm} (Eq. 2-2)

Various investigators have assessed the accuracy of the Rayleigh distribution in describing the observed wave field in and find good agreement except for cases of
shallow water, wave breaking, or wave-current interaction (e.g., Barthel and Ing, 1982; de Lange and Healy, 1990; Chemeva et al, 2005; Goodknight and Russell, 1963; Forristall, 1978; Green, 1994). These same principles will be applied in Chapter 5 to assess the applicability of Eqs. 2-1 and 2-2 to wave height distributions in the York River estuary.

2.2 Acoustic Doppler Technology

The application of acoustic Doppler instruments to current and wave measurement is well established in the fields of coastal engineering and oceanography. Typically, the instrument is deployed as a bottom-mounted, upward looking profiler that uses the Doppler principle to measure velocities in the along-beam directions. Acoustic Doppler instruments provide one alternative to some of the measurement limitations described above, in that they simultaneously make measurements of pressure, orbital velocity near the surface, and sea surface elevation (surface tracking). Since the instrument is located on the seabed, the pressure sensor still suffers from poor resolution of high frequency components, but offers good validation of the surface tracking and velocity-based spectra at lower frequencies (the range of overlapping frequencies depends on the deployment depth). Unlike the traditional $PUV$ method, the velocity measurement is made higher in the water column at bins just below the surface to compensate for the depth-attenuation problem. Surface tracking allows resolution of higher frequency components out to the Nyquist frequency, although it is also depth-dependent due to beam spreading. This has the effect of smearing high frequency waves with wavelengths on the order of the footprint size. For depths greater than
approximately 6 m, the beam spreading effect, not the sampling frequency, limits the high-frequency cut-off of the measurement.

Despite the obvious advantage of making velocity measurements near the surface, the directional measurement of waves using acoustic Doppler current profilers is still depth-limited. This is because the locations of the orbital velocity measurements form a spatial array when projected onto the ocean’s surface due to the beam geometry. As a result, the spacing between measurements at the surface increases with deployment depth. In general, the high frequency cut-off for a directional wave gage is dictated by the spatial separation of the sensors so that the highest resolvable frequency corresponds to a wave with a wavelength equal to twice the sensor spacing.

Later sections will present data collected by acoustic Doppler instruments in coastal and estuarine environments. The limitations and possibilities for the use of these instruments in estuarine environments are discussed in Chapter 4.

2.3 Characteristics of Wave Spectra

Various investigators have examined the spectral shape in different environments and wind conditions (e.g., Hasselmann et al., 1973; Vincent and Resio, 1977; Liu, 1983; Phillips, 1958; Pierson and Moskowitz, 1964). In general, the observed spectrum shape depends on the external forcing conditions such as wind speed, fetch, duration, and storm stage as well as local properties such as water depth and wave-current interaction. Additionally, internal mechanisms such as non-linear wave-wave interaction and energy dissipation due to wave breaking or bottom friction can also act to modify the shape of the spectrum (Massel, 1996). While the range of observed spectra is quite varied, the
fundamental spectrum shape is not arbitrary and some basic characteristics apply for all spectra. The high frequency spectral tail is often called the “saturation range” and represents an equilibrium condition where energy input from the wind is balanced by energy loss due to dissipation and energy transfer to lower frequencies (Phillips, 1958).

Over time, oceanographers and engineers have developed theoretical and empirical representations of wave spectra for numerical modeling and design of engineering structures. Two of the most notable results are the fully developed spectrum of Pierson and Moskowitz (1964) and the fetch-limited spectrum of the JONSWAP experiment (Hasselman et al., 1973).

As previously discussed, the observed sea surface fluctuation at any point can be regarded as the linear superposition of a large number of constituent wave components. In random linear wave theory, this means that the phase is arbitrary and uniformly distributed. By modeling the sea surface in this way, it can be shown that the total variance of the sea surface equals the sum of the variances of its component wave trains. Since the variance is proportional to the average energy per unit area of sea surface, it is useful to consider the distribution of energy (or variance) in frequency-direction space. This is known as the directional spectrum and is often written,

\[ E(f, \theta) = S(f)D(f, \theta) \quad \text{(Eq. 2-3)} \]

where \( S(f) \) is the one-dimensional spectral density function and has units of \( [\text{m}^2/\text{Hz}] \), and the direction-dependent function, \( D(f, \theta) \), is normalized such that it represents the directional distribution of energy,

\[ \iint D(f, \theta) df d\theta = 1 \quad \text{(Eq. 2-4)} \]
It is often useful to consider only the non-directional variance spectrum, \( S(f) \).

Greater insight into the characteristics of the spectrum can be obtained by considering the moments of the spectrum. In general, the spectral moments are defined as:

\[
m_n = \int_0^\infty f^n S(f) \, df \quad \text{for } n = 0, 1, 2, \ldots \quad (\text{Eq. 2-5})
\]

The first several moments are of special importance for the spectral description of ocean waves. The zero-th moment \( (m_0) \) equals the total variance, \( \sigma^2 \), since it simply represents the integration of the variance spectrum over all frequencies. The energy-based definition of significant wave height is taken as \( H = 4 \sqrt{m_0} \). In other words, the significant wave height is approximately four times the standard deviation of the surface elevation time series. The mean frequency is \( \bar{f} = m_l/m_0 \), the mean period is \( T_m = 1/\bar{f} = m_0/m_l \), and the mean zero-crossing period is \( T_z = \sqrt{m_4/m_2} \) and is sometimes written as \( T_{mo2} \) to indicate its derivation from spectral moments.

The concept of spectral width is useful in describing the distribution of energy about the mean frequency. The bandwidth parameter, or normalized radius of gyration, is defined as (Tucker and Pitt, 2001):

\[
\nu = \sqrt{\frac{m_2 m_4}{m_2^2} - 1} \quad (\text{Eq. 2-6})
\]

The bandwidth parameter can be better understood from a mechanics point of view, where the radius of gyration describes the way in which the total cross sectional area is distributed about its centroidal axis. In terms of wave spectra, it describes how wave energy is distributed about the mean frequency. For very narrow bandwidths, \( \nu \) goes to zero and all wave energy is concentrated near the mean frequency and individual waves...
have almost the same frequency with gradually varying amplitudes modulated by the wave envelope. Positive and negative maximum excursions of the wave surface are equal and individual wave heights are approximately equal to twice the wave amplitude. A sample time series is shown in Figure 2-1 \((H_s = 1.04m, T_p = 5.08s, \nu = 0.49)\). A sample broad spectrum time series is shown in Figure 2-1 \((H_s = 0.23m, T_p = 9.4s, \nu = 1.16)\).

Large values of \(\nu\) are associated with wide spectra, when wave energy is broadly distributed among many frequencies. The wave components ride on each other to produce local maxima both above and below the mean sea level. Chapter 3 utilizes the bandwidth parameter to characterize broad and narrow spectra to explain discrepancies between wave height parameter estimates using both zero-crossing and spectral analysis.
Figure 2-1. Example of a narrow (top) and wide (bottom) spectrum from the New Jersey coast. For the narrow spectrum, $H_s = 1.04$ m, $T_p = 5.08$ s, and $v = 0.49$. For the wide spectrum $H_s = 0.23$ m, $T_p = 9.4$ s, $v = 1.16$.
Chapter 3

Evaluation of Wave Height Parameter Estimates in Coastal Environments
3 EVALUATION OF WAVE HEIGHT PARAMETER ESTIMATES IN COASTAL ENVIRONMENTS

3.1 Abstract

This chapter presents comparisons of wave height estimates using data from acoustic Doppler wave gauges in ten coastal and estuarine environments ranging from fetch-limited estuarine systems to high-energy exposed coasts. This chapter examines features of the wave field that influence the agreement between the significant wave height derived from the spectrum, $H_m$, and from zero-crossing analysis, $H_{1/3}$, as well as processes governing the relationship between significant and maximum wave height. Estimates of significant wave height ($H_m$) and $H_{1/3}$ are compared and it is demonstrated that the agreement between significant wave height estimates based on spectral moments ($H_m$) vs. zero-crossing analysis ($H_{1/3}$) is linked to the underlying narrow band assumption. A divergence from theory occurs as spectral width increases with changes in the wave field. Long-term measurements of the maximum to significant wave height ratio, $H_{max}/H_{1/3}$, show a predictable dependence on the site-specific wave climate and sampling scheme. As an engineering tool for other investigators, we present empirically derived equations relating $H_m/H_{1/3}$ and $H_{1/3}/\sqrt{m_0}$ to the spectral bandwidth parameter, $\nu$, and evaluate two procedures to predict $H_{max}$ from the spectrum when the surface elevation time series is unavailable. Comparisons with observations at each site demonstrate the utility of the methods to predict $H_{max}$ within 10% on average. To the author’s knowledge, never before has such a broad synthesis of high quality direct wave
measurements been examined with these objectives. Overall, a total of nearly 7700 wave height parameter estimates from a range of environments are included in the analysis.

### 3.2 Background

The significant wave height \(H_s\) is perhaps the most commonly used parameter to represent the complex sea state (USACE, 2002). Traditionally, \(H_s\) was estimated by visual observations of a trained mariner. Quantitatively, \(H_s\) is found to be most nearly equal to the average height of the \(1/3\) largest waves in a record. Zero-crossing analysis of the surface elevation time series provides a direct measure of individual wave heights and allows explicit determination of parameters such as significant wave height \((H_{1/3})\), \(1/10\) th wave height \((H_{1/10})\), root-mean-square wave height \((H_{rms})\), and maximum wave height \((H_{max})\). Wave parameters are derived from a record by ranking the individual wave heights defined by successive zero-crossings and averaging some fraction of the total to obtain parameter estimates. While this procedure provides some insight into the bulk statistics of the wave field, it is incapable of describing more complex features such as spectral shape or multiple wave trains. The directional spectrum offers a more complete description of the sea surface in that it describes the way in which wave energy is distributed at various frequencies and directions. It is then possible to calculate many of the same parameters as from zero-crossing analysis such as the energy-based significant wave height, \(H_{m0}\), and spectrally defined mean zero-crossing wave period, \(T_{m0}\).

Historically, resolution of high-frequency components of the wave field from bottom-mounted instruments has proven difficult due to the exponential decay of the wave signal with depth (Pedersen et al., 2005). Using linear wave theory, it is possible to
infer low frequency surface wave characteristics via bottom-mounted pressure (p) and horizontal velocity (u,v) time series in relatively shallow water (i.e., the PUV method). The advent of acoustic Doppler wave gauges in the 1980s allowed for measurement of orbital velocities higher in the water column, thus extending the high-frequency cut-off. Additionally, acoustic surface tracking with one or more beams provides an independent measure of the non-directional spectrum by direct ranging of the surface with high temporal resolution. Thus, acoustic Doppler wave gauges provide simultaneous estimates of wave statistics from zero-crossing and spectral methods, making this type of instrumentation ideal for comparisons of wave height parameters.

Longuet-Higgins (1952) first applied the statistics of random signals to ocean waves and demonstrated that for deep-water narrow band spectra, wave amplitudes follow the Rayleigh distribution. Under the assumption of a slowly varying amplitude envelope, the Rayleigh distribution can also be extended to the distribution of wave heights. Field evidence generally supports this claim under most conditions except for cases of shallow water, wave breaking, or wave-current interaction (Thompson and Vincent, 1985; Green, 1994; Barthel and Ing, 1982). One prominent exception, even in deep water, is for the high end of the probability tail where the Rayleigh distribution is found to over-predict the heights of the highest waves (Forristall, 1978). Despite these shortcomings, it is from this foundation that various relationships between wave parameters can be derived for operational use.

For deep-water narrow band spectra, wave heights have been shown to conform to the Rayleigh distribution, and $H_{1/3}$ and $H_{m0}$ are equivalent estimates of significant wave height (Sarpkaya and Isaacson, 1981):
\[
H_{1/3} = (1.416)H_{rms} = (1.416)(2\sqrt{2\sigma^2}) = 4.004\sqrt{\sigma^2} = H_{m_0} \quad \text{(Eq. 3-1)}
\]

where \(H_{rms}\) is the root mean square wave height and \(\sigma^2\) is the sea surface variance and is equal to the zeroth moment, \(m_0\), obtained by integrating the energy density spectrum (see Eq. 3-3). Thus, when the underlying assumptions are satisfied, either estimate (\(H_{1/3}\) or \(H_{m_0}\)) is a valid approximation for \(H_s\). In practice, \(H_{m_0}\) is operationally defined as \(4.004\sqrt{\sigma^2} = 4\sqrt{\sigma^2} = 4\sqrt{m_0}\) regardless of whether or not the wave heights actually follow the Rayleigh distribution. However, the key assumptions are not always valid, especially in shallow water (Thompson and Vincent 1985), and one must exercise caution when applying the term “significant wave height,” as it may imply different meaning depending on the specific method of analysis.

3.3 Methods

Ten datasets were examined from Atlantic and Pacific coastal and estuarine sites: Chesapeake Bay mouth (VA), Lunenburg Bay (Nova Scotia), Tampa Bay (FL), Thames River (CT), Wilmington (NC), York River (VA), York River mouth (VA), Diablo Canyon (CA), Huntington Beach (CA), and Fort Tilden (NY). The site characteristics and locations are summarized in Table 3-1, which lists the number of records, mean water depth, mean bandwidth parameter, mean wave height and period (± one standard deviation), and station coordinates. Data were collected using the Nortek Acoustic Wave and Current Meter (AWAC), a bottom-mounted profiling acoustic Doppler current meter. The AWAC measures pressure at depth and wave orbital velocities along three angled beams at 1 or 2 Hz. The AWAC also uses acoustic surface tracking to directly measure a
time series of surface elevation using a vertical center beam at 2 or 4 Hz. Record lengths were either 512, 1024, or 2048 seconds. Spectral estimates of significant wave height ($H_{m0}$) were calculated from the non-directional energy density spectrum of the sea surface elevation. The zero-crossing estimate of significant wave height ($H_{1/3}$) was calculated from up-crossing analysis of the sea surface elevation time series. The maximum wave height ($H_{\text{max}}$) was defined for each record as the highest individual crest to trough excursion between successive up-crossings. Bad data points were eliminated using an iterative procedure to exclude outliers greater than a threshold number of standard deviations from the mean, and screened data points were linearly interpolated. The outlier bands were narrowed with each iteration and records with greater than 10% data loss were neglected from this analysis. Furthermore, records with $H_{m0} < 0.1$ m were excluded to prevent the dominance of transient waves such as boat wakes during low energy conditions. Of the 8496 initial records, 609 bursts were excluded due to the wave height threshold and 9 bursts were excluded due to excessive outliers (> 10%). Even with a stricter outlier threshold of 5%, only 25 bursts would have been excluded from the analysis. Thus, it is believed that the outlier screening procedure did not bias the estimates of $H_{\text{max}}$ by excluding valid data points.

To relate the degree of agreement between wave height estimates to the validity of the underlying narrow-band assumption, the spectral width was determined for each record. The spectral width parameter applied in this study is the normalized radius of gyration, $v$, which describes the way in which spectral area is distributed about the mean frequency (Tucker and Pitt, 2001):
\[ \nu = \sqrt[4]{\frac{m_m m_2}{m_4^2} - 1} \]  
(Eq. 3-2)

The moments of the spectrum are defined as:

\[ m_n = \int_0^\infty f^n S(f) df \quad \text{for } n = 0, 1, 2, \ldots \]  
(Eq. 3-3)

where \( S(f) \) is the non-directional energy density spectrum. For narrow bandwidths, \( \nu \) approaches zero and all wave energy is concentrated near the mean frequency. Individual waves have nearly the same frequency with gradually varying amplitudes modulated by the wave envelope. Larger values of \( \nu \) are associated with wide spectra, when energy is broadly distributed among many frequencies and the wave components ride on each other to produce local maxima both above and below the mean sea level.

For this application, the normalized radius of gyration, \( \nu \), is preferred relative to an alternate spectral width parameter, \( \epsilon \), defined by Cartwright and Longuet-Higgins (1956). This is because the Cartwright and Longuet-Higgins parameter depends on the fourth moment of the spectrum (\( m_4 \)) and tends to infinity logarithmically with the high-frequency cut-off (Tucker and Pitt, 2001). Rye (1977) showed that while \( \nu \) also suffers from a dependence on the high-frequency cut-off, \( f_c \), the variation appears to be less than 10% for \( f_c/f_p \) greater than about 5, where \( f_p \) is the peak frequency. Given the relatively high cut-off frequency of the acoustic surface tracking measurement (typically \( 1.0 < f_c < 2.0 \) Hz), it is believed that this did not adversely affect the spectral bandwidth calculations.
3.4 Results

3.4.1 Significant Wave Height

As previously discussed, it can be shown that the spectral ($H_{m_0}$) and zero-crossing ($H_{1/3}$) estimates of significant wave height are equivalent when the spectrum is narrow banded and the wave heights are described by the Rayleigh distribution (Eq. 3-1). The agreement between wave height estimates can be evaluated by solving for the coefficient of $\sqrt{m_0}$ from $H_{1/3} = H_{m_0} = 4\sqrt{m_0}$. This coefficient is represented by the non-dimensional ratio $H_{1/3}/\sqrt{m_0}$, and has a theoretical value of approximately 4.0. The average value of the $H_{1/3}/\sqrt{m_0}$ ratio is shown in Table 3-2 for each site. The mean ratio ranged from a minimum of 3.45 at Lunenburg Bay, Nova Scotia to a maximum of 3.76 at Diablo Canyon, CA. The average value of the coefficient for all records was approximately 3.60. This represents a 10% difference relative to the theoretical value of 4.0 typically employed under the narrow band assumption. One possible explanation for this discrepancy is the effect of finite spectral bandwidth.

To evaluate this hypothesis, the ratio $H_{1/3}/\sqrt{m_0}$, was examined as a function of the spectral bandwidth parameter, $v$. $H_{1/3}/\sqrt{m_0}$ was found to be negatively correlated with the spectral bandwidth parameter at all sites. In other words, its value deviated further from the theoretical value as spectral bandwidth increased. To assess the universality of this relationship, data from all sites were combined for analysis. The resulting scatter plot is shown in Figure 3-1. No attempt was made to select records of specific spectral shape or energy level, other than to exclude $H_{m_0} < 0.1 \text{m}$, since the purpose here is to derive a relationship applicable to the broadest possible range of wave
conditions. To reduce scatter and decrease bias introduced by outliers and the over-abundance of mid-range bandwidths, the data were binned in increments of $\Delta \nu = 0.15$. Within each bin, the median and standard deviation were determined for the observed values of $H_{1/3}/\sqrt{m_0}$. A least squares fit (Wunsch 1996) was applied to the binned data points to determine the best-fit slope and intercept for the combined dataset. The best-fit intercept, $\alpha$, for the binned data was found to be $3.95 \pm 0.098$ for a 95% confidence interval; the best-fit slope, $\beta$, for the binned data was found to be $0.537 \pm 0.105$ for a 95% confidence interval:

$$H_{m_o}' = \left[ \alpha - \beta \nu \right] \sqrt{m_0} \quad \text{(Eq. 3-4)}$$

where $H_{m_o}'$ is the newly defined bandwidth-corrected significant wave height, more closely resembling the zero-crossing value, $H_{1/3}$.

For narrow bandwidths, $\nu$ approaches zero and Eq. 3-4 approximates the widely accepted theoretical relation for narrow band spectra, $H_{m_o} = 4\sqrt{m_0}$. The fit was not constrained to a particular intercept at $\nu = 0$ because it is not clear what value of $\nu$ is sufficiently small to constitute a narrow bandwidth. As a result, the exact relationship is not recovered for $\nu = 0$. For larger bandwidths, the value of the coefficient of $\sqrt{m_0}$ can deviate by as much as 25% of the theoretical value (as low as $H_{1/3}/\sqrt{m_0} = 3.0$). A similar procedure was used to apply a least squares fit to the binned data at each individual site to compare the slopes among different environments. The fits were constrained to intersect $H_{1/3}/\sqrt{m_0} = 3.95$ at $\nu = 0$, based on the fit for the combined dataset given above. This was a necessary constraint given that some of the sites display a very narrow range of bandwidths and contain only a few binned data points. The best-
fit slopes are shown in Figure 3-2 and listed in Table 3-2 with 95% confidence intervals for each site. As seen in Figure 3-2, the 95% confidence bands on the slope at each site overlap the 95% confidence interval on the best-fit slope for the combined datasets at eight of the ten sites. This indicates that the majority of the individual site slopes are indistinguishable from the best-fit slope for the combined data, suggesting that the derived relationship between $H_{1/3}/\sqrt{m_0}$ and $\nu$ holds for a wide range of environments.

Closer examination reveals that the individual site slopes exhibit a weak dependence on the local water depth as well. However, when depth is normalized by the wavelength, as would be the expected dependence from theoretical considerations for waves in finite depth, this correlation is no longer observed. Thus, it is believed that the observed relation between site-specific slope and local water depth is not dynamically significant.

The agreement between wave height estimates can also be evaluated in an equivalent manner by simply taking the ratio of the two wave height estimates, $H_{m_0}/H_{1/3}$. While this ratio does not contain any new information not available from the $H_{1/3}/\sqrt{m_0}$ analysis, Eq. 3-5 is included for completeness and may provide a useful tool for investigators, especially when $H_{m_0}$ values of significant wave height have already been computed. The analysis proceeds identically to the description given above. The best-fit intercept, $\alpha$, for the binned data was found to be $0.996 \pm 0.032$ for a 95% confidence interval; the best-fit slope, $\beta$, for the binned data was found to be $0.181 \pm 0.034$ for a 95% confidence interval:

$$\frac{H_{m_0}}{H_{1/3}} = \alpha + \beta \nu$$  \hspace{1cm} (Eq. 3-5)
For narrow bandwidths, $\nu$ approaches zero and Eq. 3-5 approximates the expected relationship, $H_{m_0}/H_{1/3} = 1.0$, but deviates for larger bandwidths. The best-fit slopes for the individual sites are listed in Table 3-2 with 95% confidence intervals.

By examining the spectra, it was observed that the $H_{1/3}/\sqrt{m_0}$ ratio approaches the theoretical value of 4.0 (or equivalently, $H_{m_0}/H_{1/3}$ approaches 1.0) as energy increases and the spectrum narrows and becomes more peaked, but diverges from theory as spectrum width increases under low energy conditions or bimodal structure. This trend is illustrated in Figure 3-3, which shows observed values of (a) $\nu$, (b) $H_{m_0}/H_{1/3}$, and (c) $H_{1/3}/\sqrt{m_0}$ vs. $H_{m_0}$ for two sites: Chesapeake Bay, VA and Diablo Canyon, CA. At each site, the greatest deviations from the theoretical values of the ratios occur for low energy conditions and larger values of the bandwidth parameter.

Thus, the appropriate value of $H_{1/3}/\sqrt{m_0}$ can be determined from Eq. 3-4 to calculate the “bandwidth-corrected” value of the energy-based significant wave height. The result is that $H_{m_0}'$ more closely reflects the value obtained for the traditional significant wave height from zero-crossing analysis ($H_{1/3}$). This is a convenient result for theoretical relationships that require $H_{1/3}$ as opposed to $H_{m_0}$. Tucker and Pitt (2001) provide values of the bandwidth parameter for the Pierson-Moskowitz ($\nu = 0.425$) and JONSWAP ($\nu = 0.39$) spectra. Using these values in Eq. 3-4 with $\alpha = 3.95$ and $\beta = 0.537$, the value of the coefficient of $\sqrt{m_0}$ (i.e., $[\alpha - \beta \nu]$) becomes 3.72 and 3.74 for the P-M and JONSWAP spectra, similar to values reported by other investigators. For comparison, Forristall (1978) found a value of 3.77 for hurricane storm waves in the Gulf of Mexico and Goda (1974) found a value of 3.79 for deep-water waves at Nagoya port.
3.4.2 Maximum Wave Height

The maximum wave height in a record depends fundamentally on the number of waves in the sample, $N$. For each burst, the ratio $H_{\text{max}}/H_{1/3}$ can be treated as a random variable, and there will be a corresponding probability distribution that yields the most probable value of the ratio. Longuet-Higgins (1952) provides a formulation for estimating this ratio given $p$ and $N$ based on the Rayleigh distribution,

$$
\mu \left[ \frac{H_{\text{max}}}{H_p} \right] = \frac{\sqrt{\ln N}}{\sqrt{\ln \frac{1}{p} + 1 + \frac{\sqrt{\pi}}{2 \text{erf} c} \left[ \sqrt{\ln \frac{1}{p}} \right]}}
$$

(Eq. 3-6)

where $H_p$ is the average of the highest $pN$ waves, $0 < p \leq 1$, and $N$ is the number of waves in the record. For significant wave height ($H_{1/3}$), $p = 1/3$ and Eq. 3-6 approximates the more familiar expression, $H_{\text{max}}/H_{1/3} = \sqrt{(\ln N)/2}$. Thus, Eq. 3-6 provides a method for estimating the most probable value of $H_{\text{max}}/H_{1/3}$ for a given value of $N$. Since $N$ can only be determined from zero-crossing analysis, the mean period can be used as a proxy for $N$, where

$$
N = \frac{\text{record length}}{T_{\text{mean}}}
$$

(Eq. 3-7)

where the record length is typically 512, 1024, or 2048 seconds, and $T_{\text{mean}}$ is the reciprocal of the mean frequency estimated from spectral moments ($T_{\text{mean}} = m_0/m_1$). The use of $T_{\text{mean}}$ as opposed to $T_{m_2} = \sqrt{m_0/m_2}$ is recommended in this application to reduce the sensitivity on the high-frequency cut-off. Rye (1977) showed that $T_{\text{mean}}$ appears to be stable for cut-off frequencies greater than about five times the peak frequency. For
example, a cut-off frequency, \( f_c \), of 1.5 Hz would provide a stable estimate of \( T_{\text{mean}} \) for peak periods as short as 3.3 seconds. However, the use of \( T_{\text{mean}} \) or \( T_{\text{mc}} \) provides similar estimates of \( H_{\text{max}} \).

Using Eqs. 3-6 and 3-7, the most probable value of the ratio can be compared to the observed burst-to-burst variation in \( H_{\text{max}}/H_{1/3} \). Figure 3-4 shows time series of predicted vs. observed values of \( H_{\text{max}}/H_{1/3} \) at three sites: (a) Fort Tilden, NY, (b) Diablo Canyon, CA, and (c) Lunenburg Bay, NS. Generally, \( H_{\text{max}}/H_{1/3} \) shows large random variation about the theoretical value that is impossible to predict with exact certainty. This is expected, given that the observed value of the ratio is governed by a probability distribution itself, and not simply a deterministic function of \( N \). However, when averaged over the deployment duration, the mean observed value of \( H_{\text{max}}/H_{1/3} \) at each site more closely matches the theoretical value from Eq. 3-6 using the mean observed \( N \). A comparison of the theoretical curve and mean observed values of \( N \) and \( H_{\text{max}}/H_{1/3} \) is shown in Figure 3-5 for all sites. Recall that the mean observed value of \( N \) depends not only on the wave climate, but also the record length, which varies from 512 to 2048 seconds. As a result, low mean values of \( N \) imply either a short burst duration or a long mean wave period. The data agree favorably with theory and display the general logarithmic increase of \( H_{\text{max}}/H_{1/3} \) with \( N \).

It should be noted that the under-prediction of \( H_{\text{max}}/H_{1/3} \) for high values of \( N \) could be related to the stationary assumption inherent in the analysis or the influence of transient waves during low energy conditions at the estuarine sites (Jerome P.-Y. Maa, personal communication, May 12, 2006). In these complex fetch environments, wave growth is extremely sensitive to the wind direction relative to the dominant fetch.
orientation, so that slight changes in wind magnitude or direction during the sampling could be accompanied by rapid wave field adjustment. For example, a given record will have some observed value of $H_{\text{max}}$ and $H_{1/3}$ that will result in the computed value of $H_{\text{max}}/H_{1/3}$. However, for a non-stationary wave field the significant wave height estimate will be biased low due to the inclusion of smaller waves, yet $H_{\text{max}}$ will be representative of the most energetic conditions. Thus, for non-stationary conditions the observed $H_{\text{max}}/H_{1/3}$ will be biased high relative to the expected value. This highlights the importance of selecting a record length that is appropriate for the wave climate of a particular study site. Thus, for fetch-limited estuarine sites where wave conditions change rapidly in response to wind forcing, short bursts (512 seconds or 8.5 minutes) are recommended. For coasts dominated by long period remote swell, long bursts (2048 seconds or 34.1 minutes) are recommended to observe a sufficient number of waves ($N > 100$) for analysis. At intermediate sites, the typical 1024 second (17.1 minute) burst is sufficient.

### 3.5 Applications

Conceivably, one may wish to estimate the value of $H_{\text{max}}$ when a direct measure of the surface elevation time series is unavailable. This might occur when using the orbital velocity or pressure-based spectra from the acoustic Doppler instruments. For example, when the number of bad detects from the surface tracking time series exceeds a critical threshold one may wish to revert to either the velocity or pressure-based spectrum. In these cases, one must exercise caution when attempting to infer a statistically reasonable estimate of $H_{\text{max}}$ from spectral parameters such as $H_{m_0}$. 

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One method is to assume a constant value of the $H_{\text{max}}/H_{m_0}$ ratio that is consistent with the derivation provided by Longuet-Higgins (1952). Typical values are 1.27 ($H_{1/10}/H_{1/3}$) or 1.67 ($H_{1/100}/H_{1/3}$) (Sarpkaya and Isaacson, 1981). Previous observational studies have assumed a linear relationship between maximum and significant wave height and various investigators have reported observed values of $H_{\text{max}}/H_{1/3}$ for specific study sites: Allan and Kirk (2000) found a mean value of 1.84 for wind waves at Lake Dunstan, NZ, Hastie (1985) found a mean value of 1.56 for ocean swell at Timaru Harbor, NZ, and Myrhaug and Kjeldsen (1986) report a ratio of 1.50 between $H_{\text{max}}$ and $H_{m_0}$ on the Norwegian shelf. However, the observed value of the ratio depends on $N$, which is a function of the record length and the mean wave period so that different investigators may find different values of the ratio at the same site as a result of different sampling schemes or seasonal variations in the wave climate. It should also be noted that while the theoretical coefficients of Longuet-Higgins (1952) represent the ratio between $H_{\text{max}}$ and $H_{1/3}$, most modern estimates of significant wave height are derived from the spectrum ($H_{m_0}$). As previously demonstrated, $H_{1/3}$ and $H_{m_0}$ are only equivalent for narrow bandwidths, which are rarely observed. This makes it difficult to select a single value for the coefficient that is appropriate without first calibrating it to a specific site and sampling scheme.

Here, a method is evaluated that addresses some of the aforementioned problems to predict $H_{\text{max}}$ from the measured spectrum using the extensive dataset that has been assembled. The procedure is outlined as follows:

1. Estimate the bandwidth-corrected significant wave height, $H'_{m_0}$, from Eq. 3-4
2. Estimate the mean period as $T_{\text{mean}} = m_0/m_1$
3. Estimate $N$ from Eq. 3-7

4. Estimate $H_{\text{max}}/H_{1/3}$ from Eq. 3-6 and predict $H_{\text{max}}$

### 3.5.1 Coastal Environments

To illustrate the utility of this procedure, the method was applied to each site and $H_{\text{max}}$ predictions were compared with actual measurements. For each record, the percent error relative to the measured $H_{\text{max}}$ was determined. The mean signed error and mean absolute error are shown in Table 3-3. For each site, the error with and without the bandwidth correction (Eq. 3-4) is given. For comparison, errors are also given for the constant coefficient method of predicting $H_{\text{max}}$ as 1.67 times the significant wave height, as derived from the Rayleigh distribution for the $H_{1/100}$ wave height. For both methods, errors were reduced for a majority of the sites by using the bandwidth-corrected significant wave height, $H_{m}^{'}$, relative to $H_{m}$. For the method outlined above, the mean signed error was less than 5% for eight of ten sites, suggesting that only a slight positive or negative bias is introduced when using the most probable value of the ratio from the Rayleigh distribution (Eq. 3-6). The mean absolute error was less than or equal to 10% for all ten sites. For the constant coefficient method, the mean signed error and mean absolute error were less than or equal to 5% and 10%, respectively, for seven of ten sites.

Over the range $200 < N < 400$, the constant transfer coefficient of 1.67 (i.e. $H_{100}/H_{1/3}$) appears to provide reasonable estimates of $H_{\text{max}}$ that are comparable to Eq. 3-6, but for larger or smaller values of $N$ a substantial positive or negative bias may be introduced into the prediction of $H_{\text{max}}$ if a constant transfer coefficient is used. The sites with the largest deviations for both methods were York River, VA and Thames River, CT.
- both estuarine sites. As previously noted, the estuary sites display relatively high values of the $H_{\text{max}}/H_{1/3}$ ratio given the high number of waves per burst and non-stationary characteristics. For these environments in particular, the use of a constant transfer coefficient is not recommended.

### 3.5.2 York River Estuary: Tropical Depression Ernesto

As previously discussed, one potential application for the proposed method of predicting the maximum wave height in a record is when the number of bad detects from a surface tracking wave gage exceeds a critical threshold. Poor data quality from surface tracking is often associated with overly-steep waves or times of active wave breaking (whitecapping). Without a direct measure of the surface elevation, one cannot accurately estimate the maximum wave height during the record. During these instances, wave statistics are instead derived from pressure or velocity-based spectra. When the remnants of Tropical Storm Ernesto passed over central Virginia on September 1, 2006, winds in excess of 25 m/s generated unusually large waves in the lower reaches of the York River. A Nortek AWAC deployed as part of the Chesapeake Bay Observing System recorded wave and current data near the mouth of the York River throughout the duration of the storm.

The detailed response of the wave field during this storm will be discussed further in Chapter 4. Here, the primary goal is to demonstrate the utility of the method presented in Section 3.5 to predict maximum wave height during extreme events in the Chesapeake Bay tributaries. During the passage of Ernesto, the percentage of bad detects from the surface tracking exceeded 10% for approximately nine hours, thus requiring the
estimation of wave parameters through alternate means. During this time, the significant wave height \( H_{\text{max}} \) was estimated from the velocity-based energy spectrum instead of the acoustic surface tracking (AST) spectrum. Maximum wave height was predicted following the procedure outlined in Section 3.5.

The measured and predicted values of maximum wave height are shown in Figure 3-6 for four days surrounding the Ernesto event. The time series of the parameters relevant to the analysis \( (T_{\text{mean}}, N, H_{\text{max}}/H_{1/3}) \) are displayed in Figure 3-7. \( T_{\text{mean}} \) reached a maximum of 3.8 seconds during the peak of the storm, corresponding to approximately 270 waves per record. The predicted values of \( H_{\text{max}}/H_{1/3} \) ranged from a maximum of 1.9 pre-storm, and reached a minimum of 1.76 during the height of the storm. The resulting predictions seem quite reasonable. The maximum values of significant and maximum wave height were attained for a record around 0900 EST on September 1, 2006 and were 1.67 m and 2.78 m, respectively.

3.6 Discussion

Table 3-2 provides a summary of the mean observed values of \( H_{\text{max}}/H_{1/3} \), \( H_{1/3}/\sqrt{m_0} \), and \( H_{\text{max}}/H_{1/3} \). To illustrate the level of uncertainty in each value, 95% confidence intervals are also given as 1.96 times the standard error (defined as \( s/\sqrt{n} \), where \( s = \) standard deviation of the ratio and \( n = \) total number of records). The generally tight confidence bands indicate that statistically significant differences exist in the value of these ratios at each site. For \( H_{\text{max}}/H_{1/3} \) and \( H_{1/3}/\sqrt{m_0} \), this is due to the ratios' dependence on the spectral bandwidth parameter through the modification of the wave
height distribution as the narrow bandwidth assumption breaks down. The degree of deviation from the theoretical value is related to the magnitude of the spectral bandwidth parameter, \( \nu \). On average, the estuarine sites displayed the narrowest spectra (small \( \nu \)) because wave energy is concentrated primarily at high frequencies characteristic of locally generated wind waves. In contrast, the coastal sites are more susceptible to broad spectra (large \( \nu \)) due to the presence of multiple swell components or the superposition of local wind waves and longer period swell. As a result, it does not seem appropriate to report mean values of these ratios to be taken as universal constants over a broad range of environments. Instead, it is recommended that Eqs. 3-4 and 3-5 are used to estimate approximate values for the ratios given a range of possible \( \nu \) values.

Similarly, it is recommended that Eq. 3-6 be employed to predict expected values of \( H_{\text{max}} / H_{1/3} \) for a given wave climate and sampling scheme. While site-specific mean values of \( H_{\text{max}} / H_{1/3} \) do provide a better approximation of the relationship between \( H_{\text{max}} \) and \( H_{1/3} \) than a universal coefficient, the dependence on the record length and seasonal climatology should not be ignored when predicting maximum wave height for engineering studies.

As previously discussed, the dependence of the spectral bandwidth parameter on the high-frequency cut-off, \( f_c \), of the sensor poses some complications for this type of analysis. In fact, Rye (1977) found that Goda’s “peakedness parameter” (Goda, 1970), \( Q_p \), is the only bandwidth parameter that is not dependent on \( f_c \). It is believed that given the relatively high \( f_c \) characteristic of acoustic surface tracking methods, the computed bandwidth parameters in this study are representative of the true value. Therefore, spectral computations for other sensors with a low \( f_c \) will under-estimate \( H_{\text{max}} \) and \( \nu \) and
over-estimate $T_{mean}$ relative to the true values if substantial energy exists at frequencies above $f_c$. This is the commonly observed low-pass filtering phenomenon associated with bottom-mounted pressure sensors and sub-surface orbital velocity measurements. Often, this deficiency is overcome by extrapolating a high-frequency tail above $f_c$ that is proportional to $f^{-4}$ or $f^{-5}$. Such a procedure is recommended before using the methods and relationships presented in this paper. Another possibility would be to derive similar relationships using Goda’s peakedness parameter since $Q_p$ is independent of $f_c$ for $f_c/f_p$ greater than ~3 or 4. However, it is unclear that $Q_p$ would provide the most appropriate characterization of the spectral shape since the relationships presented here suggest that the emphasis should be placed on the spectrum’s width, not its narrowness.

3.7 Conclusions

In this study, which presents an analysis of wave height parameters from ten environments of varying energy regime, $H_{1/3}/\sqrt{m_0}$ was found to vary at synoptic time scales with changes in energy regime and spectrum shape and was found to be linearly related to the spectrum bandwidth parameter, $v$. The agreement between $H_{m_0}$ and $H_{1/3}$ approached the theoretical Rayleigh distribution at narrow bandwidths, but diverged significantly as spectrum width increased. In general, observations agreed better with theoretical values of $H_{m_0}/H_{1/3}$ and $H_{1/3}/\sqrt{m_0}$ during more energetic conditions when wave spectra became increasingly peaked. The empirical relationships presented in this study can be used in hindcast studies to correct output from spectral numerical wave
models, which typically report $H_{m_0}$, for direct comparison with historical field datasets of $H_{1/3}$ determined from zero-crossing analysis.

$H_{\text{max}}/H_{1/3}$ displayed large random variation from one measurement to the next with a more gradual variation at synoptic time scales, but displayed no clear dependence on $v$. At each site, the mean observed value of $H_{\text{max}}/H_{1/3}$ agreed favorably with the expected value from theory using the mean observed $N$. A procedure was evaluated to estimate $H_{\text{max}}$ in the absence of the surface elevation time series based on characteristics of the wave spectrum or by assuming a universal coefficient. It is believed that this procedure could also be employed to estimate values of $H_{\text{max}}$ based on output from spectral numerical wave models. A comparison between observed and predicted values in a variety of environments demonstrates the utility of the method to predict $H_{\text{max}}$ within 10% on average.
### Table 3-1. Summary of site characteristics and locations.

<table>
<thead>
<tr>
<th>Site</th>
<th>Records</th>
<th>Depth (m)</th>
<th>Bandwidth parameter, $v$</th>
<th>$H_{m_0}$ (m)</th>
<th>$T_{mean}$ (sec)</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chesapeake Bay Mouth, VA</td>
<td>545</td>
<td>19.2</td>
<td>0.64</td>
<td>0.6 ± 0.2</td>
<td>3.6 ± 0.6</td>
<td>36.9589° N 76.0154° W</td>
</tr>
<tr>
<td>Lunenburg Bay, Nova Scotia</td>
<td>1337</td>
<td>21.5</td>
<td>0.83</td>
<td>0.4 ± 0.2</td>
<td>4.2 ± 1.7</td>
<td>44.5527° N 64.1617° W</td>
</tr>
<tr>
<td>Tampa Bay, FL</td>
<td>605</td>
<td>4.2</td>
<td>0.42</td>
<td>0.3 ± 0.1</td>
<td>2.0 ± 0.6</td>
<td>27.6618° N 82.5945° W</td>
</tr>
<tr>
<td>Thames River Estuary, CT</td>
<td>25</td>
<td>3.6</td>
<td>0.46</td>
<td>0.1 ± 0.0</td>
<td>2.3 ± 1.1</td>
<td>41.3717° N 72.0917° W</td>
</tr>
<tr>
<td>Wilmington, NC</td>
<td>176</td>
<td>28.1</td>
<td>0.71</td>
<td>0.8 ± 0.1</td>
<td>3.6 ± 0.7</td>
<td>33.981° N 77.3623° W</td>
</tr>
<tr>
<td>York River Estuary, VA</td>
<td>181</td>
<td>8.5</td>
<td>0.44</td>
<td>0.2 ± 0.1</td>
<td>1.9 ± 0.7</td>
<td>37.2444° N 76.5004° W</td>
</tr>
<tr>
<td>York River Estuary Mouth, VA</td>
<td>1087</td>
<td>10.1</td>
<td>0.41</td>
<td>0.2 ± 0.1</td>
<td>1.7 ± 0.3</td>
<td>37.2347° N 76.3999° W</td>
</tr>
<tr>
<td>Diablo Canyon, CA**</td>
<td>524</td>
<td>25.1</td>
<td>0.66</td>
<td>1.9 ± 0.6</td>
<td>8.1 ± 2.6</td>
<td>35.2038° N 120.8593° W</td>
</tr>
<tr>
<td>Huntington Beach, CA**</td>
<td>1020</td>
<td>22.0</td>
<td>0.76</td>
<td>0.7 ± 0.2</td>
<td>6.7 ± 1.8</td>
<td>33.6229° N 118.0119° W</td>
</tr>
<tr>
<td>Fort Tilden, NY*</td>
<td>2197</td>
<td>9.9</td>
<td>0.71</td>
<td>0.7 ± 0.4</td>
<td>4.5 ± 1.5</td>
<td>40.5527° N 73.8487° W</td>
</tr>
</tbody>
</table>

Note: Wave height and period are given as the mean ± one standard deviation. For all sites, record length was 1024 seconds except where indicated (*512 seconds, **2048 seconds).
Site Slope vs. \( v \) Slope vs. \( \sqrt{m_0} \) Slope vs. \( v \) \( H_{\text{max}}/H_{1/3} \)

<table>
<thead>
<tr>
<th>Site</th>
<th>( H_{mn}/H_{1/3} )</th>
<th>( \text{Slope vs. } v )</th>
<th>( H_{1/3}/\sqrt{m_0} )</th>
<th>( \text{Slope vs. } v )</th>
<th>( H_{\text{max}}/H_{1/3} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chesapeake Bay Mouth, VA</td>
<td>1.09 ± 0.003</td>
<td>0.169 ± 0.025</td>
<td>3.66 ± 0.010</td>
<td>-0.576 ± 0.053</td>
<td>1.69 ± 0.012</td>
</tr>
<tr>
<td>Lunenburg Bay, Nova Scotia</td>
<td>1.17 ± 0.004</td>
<td>0.187 ± 0.010</td>
<td>3.45 ± 0.012</td>
<td>-0.616 ± 0.032</td>
<td>1.78 ± 0.011</td>
</tr>
<tr>
<td>Tampa Bay, FL</td>
<td>1.09 ± 0.005</td>
<td>0.213 ± 0.042</td>
<td>3.67 ± 0.012</td>
<td>-0.763 ± 0.081</td>
<td>1.80 ± 0.015</td>
</tr>
<tr>
<td>Thames River Estuary, CT</td>
<td>1.14 ± 0.024</td>
<td>0.291 ± 0.194</td>
<td>3.49 ± 0.064</td>
<td>-1.071 ± 0.426</td>
<td>2.02 ± 0.097</td>
</tr>
<tr>
<td>Wilmington, NC</td>
<td>1.08 ± 0.004</td>
<td>0.110 ± 0.018</td>
<td>3.71 ± 0.012</td>
<td>-0.412 ± 0.050</td>
<td>1.67 ± 0.019</td>
</tr>
<tr>
<td>York River Estuary, VA</td>
<td>1.11 ± 0.007</td>
<td>0.210 ± 0.072</td>
<td>3.62 ± 0.021</td>
<td>-0.692 ± 0.190</td>
<td>1.90 ± 0.026</td>
</tr>
<tr>
<td>York River Estuary Mouth, VA</td>
<td>1.09 ± 0.002</td>
<td>0.179 ± 0.027</td>
<td>3.65 ± 0.007</td>
<td>-0.763 ± 0.047</td>
<td>1.79 ± 0.009</td>
</tr>
<tr>
<td>Diablo Canyon, CA</td>
<td>1.07 ± 0.003</td>
<td>0.097 ± 0.008</td>
<td>3.76 ± 0.010</td>
<td>-0.360 ± 0.033</td>
<td>1.68 ± 0.013</td>
</tr>
<tr>
<td>Huntington Beach, CA</td>
<td>1.14 ± 0.003</td>
<td>0.180 ± 0.028</td>
<td>3.51 ± 0.009</td>
<td>-0.630 ± 0.101</td>
<td>1.75 ± 0.010</td>
</tr>
<tr>
<td>Fort Tilden, NY</td>
<td>1.11 ± 0.003</td>
<td>0.168 ± 0.026</td>
<td>3.61 ± 0.008</td>
<td>-0.574 ± 0.066</td>
<td>1.62 ± 0.007</td>
</tr>
<tr>
<td>Combined</td>
<td>0.177 ± 0.013</td>
<td>-0.588 ± 0.021</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: Best-fit site slopes and mean values of ratios are given with 95% confidence intervals.

Table 3-2. Summary of statistics for all sites.
Longuet-Higgins (1952) Most Probable Value (Eq. 6) Constant Coefficient $^a$

<table>
<thead>
<tr>
<th>Site</th>
<th>Signed Error (%) $^b$</th>
<th>Absolute Error (%) $^b$</th>
<th>Signed Error (%) $^b$</th>
<th>Absolute Error (%) $^b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chesapeake Bay Mouth, VA</td>
<td>-1.0 / +9.3</td>
<td>6.5 / 10.9</td>
<td>-1.7 / +8.6</td>
<td>6.4 / 10.2</td>
</tr>
<tr>
<td>Lunenburg Bay, Nova Scotia</td>
<td>-3.4 / +10.5</td>
<td>9.2 / 12.6</td>
<td>-3.4 / +10.5</td>
<td>8.2 / 12.1</td>
</tr>
<tr>
<td>Tampa Bay, FL</td>
<td>+1.6 / +7.3</td>
<td>8.5 / 10.7</td>
<td>-4.6 / +1.9</td>
<td>7.4 / 7.2</td>
</tr>
<tr>
<td>Thames River Estuary, CT</td>
<td>-6.6 / +1.0</td>
<td>13.9 / 13.2</td>
<td>-14.0 / -6.9</td>
<td>16.3 / 12.7</td>
</tr>
<tr>
<td>Wilmington, NC</td>
<td>-2.0 / +9.5</td>
<td>6.2 / 10.6</td>
<td>+8.7 / +8.7</td>
<td>10.0 / 10.0</td>
</tr>
<tr>
<td>York River Estuary, VA</td>
<td>-4.2 / +2.7</td>
<td>10.0 / 11.0</td>
<td>-10.9 / -4.5</td>
<td>12.4 / 10.1</td>
</tr>
<tr>
<td>York River Estuary, VA</td>
<td>+3.0 / +10.0</td>
<td>7.1 / 11.4</td>
<td>-4.0 / +2.5</td>
<td>6.7 / 7.0</td>
</tr>
<tr>
<td>Fort Tilden, NY</td>
<td>-5.9 / +4.4</td>
<td>8.2 / 9.1</td>
<td>-4.0 / +6.5</td>
<td>7.7 / 9.1</td>
</tr>
<tr>
<td>Huntington Beach, CA</td>
<td>-0.6 / +12.0</td>
<td>7.9 / 13.2</td>
<td>-2.2 / +10.2</td>
<td>7.2 / 11.5</td>
</tr>
<tr>
<td>Diablo Canyon, CA</td>
<td>-4.7 / +6.8</td>
<td>8.7 / 9.8</td>
<td>+2.9 / +15.4</td>
<td>8.2 / 16.0</td>
</tr>
</tbody>
</table>

$^a$ Wave height predictions were obtained using a statistically reasonable constant coefficient of 1.67 (roughly equivalent to the $H_{1/100}$ wave height).

$^b$ In each column, two error statistics are given. The first is the error using the bandwidth-corrected significant wave height (Eq. 4), the second is the error assuming $H_{m_0} = 4\sqrt{m_0}$.

Table 3-3. Summary of error statistics for $H_{\text{max}}$ predictions at each site.
Figure 3-1. (a) $H_{1/3}/\sqrt{m_0}$ and (b) $H_m/H_{1/3}$ vs. the spectral bandwidth parameter, $v$, for all sites (points). The medians of the binned data points ($\Delta v = 0.15$) are shown as squares with error bars indicating one standard deviation. The least squares best fit to the binned data points is shown as a solid line.
Figure 3-2. Comparison of best-fit slopes at each site (bars) and best-fit slope for combined dataset (solid) for (a) $H_{l/3}/\sqrt{m_0}$ vs. $v$ and (b) $H_{m_0}/H_{l/3}$ vs. $v$. 95% confidence intervals are indicated by error bars for the individual sites and dashed lines for combined dataset.
Figure 3-3. Comparison of observed values of (a) $v$, (b) $H_{m0}/H_{1/3}$, and (c) $H_{1/3}/\sqrt{m_0}$ vs. significant wave height ($H_{m0}$) at two sites: (•) Chesapeake Bay Mouth and (+) Diablo Canyon, CA.
Figure 3-4. Theoretical (solid) vs. observed (dashed) value of the $H_{max}/H_{1/3}$ ratio at three sites: (a) Fort Tilden, NY, (b) Diablo Canyon, CA, and (c) Lunenburg Bay, NS.
Figure 3-5. Comparison of the mean observed value of $H_{\text{max}}/H_{1/3}$ and $N$ at each site (symbols) and the theoretical prediction from Eq. 3-6 (solid).
Figure 3-6. Measured values of significant wave height (-) and maximum wave height (-) with predicted maximum(--) wave height during peak of storm (Tropical Depression Ernesto, 2006) at Goodwin Islands, VA.
Figure 3-7. Time series of (a) $T_{\text{mean}}$, (b) $N$, and (c) $H_{\text{max}}/H_{1/3}$ during the passage of Tropical Storm Ernesto (2006) at Goodwin Islands, VA.
CHAPTER 4

ESTUARINE WAVE CHARACTERISTICS
4 ESTUARINE WAVE CHARACTERISTICS

4.1 Abstract

This chapter focuses on defining the unique nature of waves in the estuarine environment in an attempt to identify distinguishing features of estuarine waves relative to those in the open ocean. To achieve this objective, observing system data from two sites in a Chesapeake Bay tributary were analyzed to determine both the typical and extreme wind and wave conditions characterizing the region. An analysis of the wind climate in the lower York River estuary indicated that conditions favorable for wind wave growth exist only 3-4% of the time. While the wave climate was found to exhibit relatively low energy conditions compared to more exposed coastal environments, the potential exists for high energy conditions during more severe events such as tropical and extratropical storms. The results demonstrate that wave characteristics specific to more coastal open-ocean settings, do not necessarily hold true in estuarine environments. One such finding was the observation of "post-event swell" following large wind events such that the largest observed peak periods are not coupled with the largest values of significant wave height.

4.2 Introduction

Observing system datasets are particularly useful for evaluating both mean and extreme conditions in an environment such as the Chesapeake Bay tributaries where wave events are episodic and short-lived. Infrequent events such as hurricanes and extratropical storms have profound impacts on the Bay’s physics and ecosystem
processes (e.g., Davis and Laird, 1976; Sellner, 2005). However, less extreme events such as summer thunderstorms, sea breezes, and frontal systems dominate the wave response for much of the year. The continuous wind and wave measurements from the VIMS Observing System offer a unique opportunity to address some very basic questions regarding the general characteristics of waves in the York River estuary. It is expected that given the similarities of the York with other Bay tributaries, the results can be expanded and applied to similar systems.

This section will examine wind and waves at two sites in the York River estuary, one within the estuary at Gloucester Point (GP) and the second at the mouth of the estuary north of Goodwin Islands (GI). The site locations are shown in Figure 4-1. The characteristics of basin geometry and wind forcing over the region are discussed first followed by a summary of the observations at each site. Next, the generally accepted Rayleigh distribution for wave heights is evaluated for estuarine waves and possible explanations for non-conformity with theory are discussed.

4.3 Background

4.3.1 Estuarine Field Studies

Given the narrow fetch-limited nature of waves in estuaries, some insight into the physics may be gained by examining the characteristics of waves in similarly sized lakes or other coastal waterways. Allan and Kirk (2000) examined wind-wave characteristics at Lake Dunstan, New Zealand and discuss the episodic and short-lived nature of storm events. The authors note that while most of the records indicate low energy conditions, it is clear that large waves can be experienced on small lakes when the wind forcing is
aligned properly with respect to the dominant fetch orientation. It follows that sites exposed to the longest fetches experience the greatest range of heights and periods.

Fetch geometry and wind direction greatly influence the wave field response. In the open ocean, fetch is defined by the spatial extent of the storm and fetch width and length are often of the same order. However, evidence suggests that narrow fetches reduce wave height growth in comparison to waves generated over broad fetches.

Pettersson (2004) examined the wave field response to winds corresponding to broad, narrow, and slanting fetches in the Gulf of Finland. By non-dimensionalizing the energy spectra, the author demonstrates that fetch geometry affects the characteristic spectral shape in a way that is difficult to predict analytically and counters the assertion that spectrum shape can be represented as a universal function dependent only on fetch, duration, and wind speed alone without considering the effects of fetch geometry.

Hershberger and Ting (1996) show that for extremely confined geometries, such as the Gulf Intracoastal Waterway, wave damping at the banks can also be an important factor and suggest that refraction of along-channel waves by shoals steers waves towards the shoreline where wave breaking and bottom friction represent additional mechanisms for energy dissipation. Such a mechanism could play a role in the narrow upper reaches of the Bay’s tributaries; however, it is expected that over the majority of region this effect is minimal.

4.3.2 Chesapeake Bay Wave Climate

Knowledge of wave characteristics in the Bay has been hampered by the limited amount of observational data available to scientists, and it was not until 1988 that a wave
monitoring program in the lower Bay was established (Boon et al., 1990). The locations of previous field studies are shown on the location map in Figure 4-2 and are summarized in Table 4-1. Boon et al. (1990-1995) summarize the results of the field studies at Thimble Shoal Light (TSL), Thimble Shoal East (TSE), and Wolf Trap Light (WTL). One of the most interesting results of this monitoring program was the confirmation of a bimodal wave climate at the TSL station. The bimodal spectrum represents the combination of the ESE swell from the external oceanic wave field and the northerly local wind sea from within the bay. The two additional lower bay sites (TSE and WTL) display only the individual constituent components, but not both.

At TSL, the results show that variations in wave height, period, and direction in the lower Bay are mainly driven by extratropical frontal disturbances known as nor’easters. Farnsworth (1997) employed Q-mode Factor Analysis to examine the temporal changes in the energy spectra at TSL and found four primary modes of spectrum shape: calm, bimodal, local, and non-local. During periods of light winds, the spectrum is dominated by 7-8 second ocean swell from the SE, since there is little contribution from locally generated wind waves from within the Bay. With the onset of a winter storm, the dominance of the calm mode subsides as locally generated waves become more important. During these events, bay-generated waves dominate the spectrum briefly until shelf-generated waves are added to produce the characteristic bimodal spectrum typically observed at the height of the extratropical storms at TSL.

Wave measurements in the Bay’s tributaries are even more limited than in the main stem. Boon et al. (1996a, 1996b) examined the hydrodynamics of sediment suspension in the littoral zone of the York River at two National Estuarine Research
Reserve (NERR) sites using a directional wave gage in 2 m water depth. In general, observed wave energy was low ($H_{m0} < 0.3$ m, $T_z < 3$ s). However, the authors note two events in March and April where $H_{m0}$ exceeded 0.5 m and individual wave heights approached 0.8 m or more. These events were generally short-lived and associated with strong winds from the W or NW, but demonstrate that large waves can potentially be an important factor in sediment dynamics in the tributaries, especially when the wind direction is aligned with the dominant orientation of the river. The results demonstrate the need for longer-term measurements in deeper channel areas to accurately characterize the York River’s wave climate and further characterize the less frequent events that dominate the wave climate.

4.4 York River Environment

4.4.1 Fetch and Bathymetry

The York River is one of several major estuarine tributaries that enters the Chesapeake Bay on its western boundary. Below the fall line, these tributaries generally display a NW-SE orientation in their upper reaches and shift to a predominately E-W orientation at their junction with the Bay proper. This abrupt shift in topography is due to the lasting effects of the Chesapeake Bay bolide impact event approximately 35 million years ago, which has resulted in differential subsidence over the crater in the southern portion of Chesapeake Bay (USGS, 1998). The wave measurements discussed in this chapter were conducted in the lower reaches of the York River estuary, from Gloucester Point to the mouth of the estuary near Goodwin Islands. This is an area that covers
approximately 45 km² and is roughly 15 km long by 3 km wide. In terms of wind-wave
growth, depth and fetch are two characteristics of the basin geometry that typically
control the observed wave conditions. Given the geologic history of the Chesapeake Bay
tributaries as drowned river valleys, a significant portion of this area is characterized by
shoal regions cut by a narrow, deep channel. In the lower York region bounded by
Gloucester Point to the west and Guinea Marsh (estuary mouth) to the east approximately
30% of the area exhibits a water depth less than 2 m and 60% is less than 5 m. The mean
water depth over this area is approximately 5.5 m. Water depths exceeding 15 m
represent less than 7% by area, but can reach up to 25 m in the deepest parts of the
channel.

Observed wave conditions at Gloucester Point and Goodwin Islands are directly
related to the fetch constraints imposed by the shoreline. At Gloucester Point, wave
growth is most responsive to winds from the east, and fetches exceed 5 km only for wind
directions between 75°-110°, for which fetches are on the order of 40 km and open water
extends from the measurement location across the main stem of the Bay to the Eastern
Shore of Virginia. The Goodwin Islands site is generally more exposed and fetches
exceed 10 km for wind directions between 50°-115° and for a narrow range of directions
nearly due west. Similar to Gloucester Point, easterly fetches extend across the main
stem of the Bay to the Eastern Shore and are on the order of 35-45 km.

It should be noted that for finite-depth wind-wave growth and propagation it is the
combination of depth and fetch that ultimately controls the observed wave height and
period at a given site. At Gloucester Point and Goodwin Islands this means that even
though the fetch computations imply relatively long fetches for easterly directions, the
effective fetch is limited by the presence of extremely shallow shoals in the vicinity of the York River mouth. These shoals may act to dissipate wave energy through bottom friction, refraction, shoaling, and breaking as waves propagate from the main stem of the Bay into the lower York. As a result, simple wave prediction formulas that rely solely on fetch and wind speed may significantly over-predict wave height and period relative to observations. The effect of these shoals in limiting the height of waves propagating into the York River from the main stem of the Bay will be investigated in Chapter 6 using the steady-state spectral wave model, STWAVE.

4.4.2 Wind Climatology

Given the restricted fetch and basin geometry characteristic of the Chesapeake Bay tributaries, there is little influence of ocean swell. Instead, locally generated wind waves typically dominate the observed wave spectra. As such, the wave conditions are directly linked to the local wind forcing and a comprehensive understanding of estuarine waves thus requires an understanding of the regional wind regime. In order to address the wind characteristics of the lower York, meteorological data from the VIMS Observing System Gloucester Point station were analyzed as part of this study. Wind data were collected using a R.M. Young 05106 marine wind monitor mounted approximately 5 m above mean sea level several hundred meters offshore of the Gloucester Point beach. One-minute averages of wind speed and direction were recorded during a period from July 2004 – December 2006. Wind vectors were further averaged to a one-hour time interval to more closely match the wave burst interval and physical time scale for wind wave growth at Gloucester Point and Goodwin Islands. It should be noted
that given the relatively short wind dataset, the purpose here is not to develop a
comprehensive wind climatology, but rather to develop a preliminary understanding of
wind wave forcing that exists in the Chesapeake Bay tributaries. While longer
meteorological datasets do exist in the vicinity of the York River, the Gloucester Point
data has the added advantages of being recorded over water and coincident with wave
measurements, and it is for these reasons that it was chosen for this analysis.

Figure 4-3 shows the monthly mean one-hour wind speed at Gloucester Point,
VA. For future discussions of seasonal trends, the following distinctions will be made for
“fall” (September-November), “winter” (December-February), “spring” (March-May),
and “summer” (June-August). On average, wind energy is highest during the spring and
winter months and lowest during the summer months. These data can be further grouped
to examine the occurrence of winds of varying strength. For this purpose, the following
distinctions will be made for “calm” (0-5 m/s), “moderate” (5-10 m/s), and “strong” (>10
m/s) wind events. Table 4-2 lists the percent occurrence for each level of wind speed for
each season. All seasons exhibit less than 1% occurrence for winds greater than 10 m/s.
Thus, it appears that the moderate wind events will control the bulk of observed waves
since it is expected that very little wave growth will occur during calm conditions. It is
observed that spring exhibits the highest occurrence of moderate wind events, followed
by winter, fall, and summer.

It should be noted that it is not only the wind magnitude that dictates wind wave
growth, but also fetch. Thus, the distribution of wind direction is also important in
controlling the potential for wind wave development in the tributaries, and only during
times of favorable wind speed and direction will significant wave growth occur.
Seasonal histograms of wind direction are shown in Figure 4-4. Fall exhibits the highest occurrence of NE wind events, although there are also more total measurements during these months. The winter months are dominated by a prevailing NW wind. Summer months exhibit a prevailing SW wind and show a high occurrence of SE winds associated with the afternoon sea breeze. The spring months show a transition between winter and summer and display high occurrences of NW winter wind events and SE sea breezes.

As previously discussed, long fetches (> 10 km) result only for wind directions between 75°-110° at Gloucester Point and 50°-115° at Goodwin Islands. Thus, favorable conditions for wave growth will result for these wind directions coupled with moderate or strong winds (i.e. > 5 m/s). For the thirty-month wind dataset examined, this corresponds to approximately 3% of the total time at Gloucester Point and 4% of the total time at Goodwin Islands. Table 4-2 shows the seasonal percentages of favorable conditions for wave growth at both sites. At both Gloucester Point and Goodwin Islands, the lowest percentage of favorable conditions occurred during the winter months. Thus, despite the generally stronger winds during the winter months, the prevailing NW wind in winter results in extremely restricted fetches that limit wave growth at Gloucester Point and Goodwin Islands. The highest percentage of favorable conditions occurred during the summer at Gloucester Point, when wave-generating winds are properly aligned with the dominant fetch orientation, and during the spring at Goodwin Islands.

4.5 Results and Discussion

The following section will present a discussion of observations of waves recorded at two sites in the lower York River: (1) Goodwin Islands, VA and (2) Gloucester Point,
Both datasets were recorded using commercially available profiling acoustic Doppler wave and current meters recording 1024-second wave records at 1-hour intervals.

### 4.5.1 Goodwin Islands

From February 16, 2006 – December 7, 2006 a 1 MHz Nortek Acoustic Wave and Current Meter (AWAC) collected continuous real-time measurements of directional wave spectra and current profiles north of Goodwin Islands, at the mouth of the York River estuary. The deployment site was located at 37° 14.083 N, 76° 23.995 W in approximately 10 m water depth (see Figure 4-1). During this time, the site was exposed to a range of conditions typical of Chesapeake Bay tributaries. The spectral significant wave height \( H_{m0} \) ranged from a minimum of < 0.1 m during calm conditions to a maximum of 1.67 m during Tropical Depression Ernesto. Spectral mean period \( T_{m02} \) ranged from 1.0 to 4.0 seconds. Spectral peak period \( T_p \) ranged from < 1.0 seconds to 5.8 seconds for locally generated wind waves, although some records show evidence of longer period ocean swell (> 10 seconds) during extremely low energy conditions \( H_{m0} < 0.1 \) m. Examination of wave characteristics (period and direction) offshore of Chesapeake Bay mouth confirm favorable conditions for ocean wave propagation to the York mouth vicinity, although this phenomenon was not investigated in detail.

Wave steepness is often regarded as an important parameter for beach erosion and shoreline response (Allan and Kirk, 2000). Here, steepness is calculated as the ratio of significant wave height to wavelength, where the wavelength is determined from the dispersion relation for waves of the peak period in a water depth of 10 m. To simplify
the computation of wave steepness, wavelength was determined using an alternate solution presented by Eckart (1952):

\[ L = \frac{gT_p^2}{2\pi} \sqrt{\tanh \left( \frac{4\pi^2 h}{T_p^2 g} \right)} \]  

(Eq. 4-1)

where \( T_p \) = peak period, \( h \) = water depth, and \( g \) = acceleration due to gravity. A comparison with wavelengths determined directly from the dispersion relation showed excellent agreement.

Joint plots of significant wave height vs. mean and peak period are shown in Figure 4-5. Isolines of wave steepness from Eq. 4-1 are shown as dashed lines from 0.01 to 0.07, in increments of 0.01 increasing in the counter-clockwise direction in the figure. The data indicate an increasing trend of significant wave height and peak period that is bounded by a maximum steepness of 0.06-0.07. This figure could serve as a useful design tool for engineers when determining the range of possible wave heights for a given wave period in the vicinity of Goodwin Islands. As an example, for a three second peak period, there were no occurrences of significant wave heights greater than approximately 0.75 m. Similarly, the highest wave heights (~1.7 m) are observed to occur with moderate peak periods in the range of 4-5 s, while longer period waves seem to occur with lower significant wave heights.

There appear to be two general groups of points displayed in the plots. The first set is represented by waves of maximum steepness that display relatively large wave heights for a given period. These points correspond to times of wave growth where waves propagate under active wind forcing. The second set is represented by waves of minimal steepness that display relatively small wave heights for a given period. These
points correspond to times of swell propagation following a wind event. This
phenomenon is illustrated in Figure 4-6, which shows a four-day time series of wind
speed and significant wave height at Goodwin Islands. The symbols in the lower panel
indicate times of low amplitude waves with relatively long period ($H_m < 0.2$ m and $T_p >$
2.5 seconds). These events typically occur following larger wind events as the wave field
propagates freely as swell. The steepness determined from significant wave height and
peak period is shown in the lower panel multiplied by a factor of 10 to facilitate plotting
on the same scale as wave height. Steepness is shown to track wave height and wind
speed closely and post-event swell waves correspond to times of low steepness (typically
around 0.01).

The physical interpretation of these occurrences is that as the wind decreases
following a moderate event, waves are no longer actively growing and instead propagate
freely as swell. At this point, attenuation by bottom friction and viscosity decreases the
wave height and there is a corresponding decrease in steepness. Steepness and wave
height appear to be closely correlated with wind speed, and wave steepness decreases
rapidly as wind speed declines. Often, the arrival of the longest period waves occurs
after the wind speed falls off so that these waves are associated with lower values of
wave height and steepness.

In addition to joint plots of wave height and period, percentage exceedance curves
provide a useful design tool and provide a method for assessing the likelihood of
observing a wave height or period in excess of some specified value. Figure 4-7 and
Table 4-3 show percentage exceedance values for wave height ($H_m$ and $H_{max}$) and
period ($T_p$ and $T_{max}$) at Goodwin Islands, VA. The results indicate that during much of
the time (> 50%) the wave height ($H_{mw}$) and period ($T_p$) are relatively low, less than 0.17 m and 2.3 seconds, respectively. However, there is a potential for brief periods of high energy. The 1% exceedance significant wave height and peak period are 0.77 m and 5.1 seconds, respectively. During extremely rare events (< 1%), wave height and period can substantially exceed these values.

One such event was on September 1, 2006 when the remnants of Tropical Storm Ernesto passed over eastern Virginia with a maximum 15-minute wind speed of 27 m/s, recorded at Goodwin Islands, and 25 m/s at Gloucester Point. Wind data from the NDBC YKRV2 station near the mouth of the York River are displayed with VIMS Observing System data from Gloucester Point and Goodwin Islands in Figure 4-8. Wind speed increased from the ENE consistently for approximately 48 hours preceding the passage of the storm and then rapidly shifted to a SSE direction as winds subsided. During the peak of the storm, winds were on the order of 25 m/s for a six-hour period on the morning of September 1. It is interesting to note that while there is considerable spatial variability in wind magnitude over the region, the wind direction is remarkably consistent among the stations, which are separated by approximately 20 km.

The wave field response is shown in Figure 4-9. The wave height is observed to track the wind speed very closely. At the height of the storm, significant wave height ($H_{mw}$) reached a maximum of 1.67 m and the maximum predicted individual wave height ($H_{max}$) was 2.78 m, based on the analysis presented in Chapter 3. It is interesting to note that the wave statistics seem to reach equilibrium with the wind as it levels off around 25 m/s (at the NDBC York River mouth station). This suggests that at this time the wave field was in equilibrium with the local wind forcing. This condition can be evaluated by
comparing the observed wave conditions to a simple finite-depth wave prediction equation developed by CERC (1984). For a fetch of 35 km, wind speed of 25 m/s, and mean water depth of 10 m (for a wind direction of 75°), the CERC equations predict a wave height and period of 1.8 m and 4.9 seconds. These predictions are quite close to the observed values for significant wave height and peak period observed during the most intense part of the storm, suggesting that the fully-developed assumption is valid during this brief time.

Figure 4-9 also demonstrates the late arrival of the longest period waves so that the largest observed peak periods are not coupled with the largest values of significant wave height. This is the phenomenon of post-event swell previously discussed. This is an interesting characteristic of locally-generated estuarine waves that differs from coastal environments, where low amplitude long period swell often precedes the arrival of a storm system at the coast. The joint plot of significant wave height and peak period for the 48 hours from August 31 – September 2, 2006 is shown in Figure 4-10. The plot is divided into “growth” and “decay” phases leading up to and following the recorded maximum significant wave height around 0900 EST on September 1, 2006. The growth phase displays a steady increase in wave height and period along the 0.03 steepness isoline, followed by a rapid increase in steepness in wave height during the 25 m/s sustained wind period, where steepness reaches a maximum of 0.07. This is followed by an initially rapid decline in steepness, after which wave height and period decline simultaneously along the 0.02 isoline. The curves display a hysteresis in that the growth and decay phases are not “elastic.” In other words, the decay curve falls below the
growth curve because decaying wave heights are associated with longer period post-event swell waves.

4.5.2 Gloucester Point

From June 2005 – September 2006, a 1200 kHz Teledyne RD Instruments Acoustic Doppler Current Profiler (ADCP) collected continuous measurements of directional wave spectra at Gloucester Point, VA on the York River. This instrument was deployed in a real-time data collection mode as part of the VIMS Observing System and proved to be a valuable test bed for instrument configuration and real-time processing of wave data in an estuarine environment. During this time, two sites at close proximity were maintained and a continuous data record was constructed by alternating between the two sites. At the first site, the instrument was cabled several hundred meters off the VIMS ferry pier and was located in approximately 10-11 m water depth (37° 14.583’ N, 76° 30.013’ W). At the second site, the instrument was cabled to a buoy and located in approximately 7-8 m water depth (37° 14.660’ N, 76° 29.989’ W).

Initial work examined the performance of the ADCP’s three methods for measuring non-directional spectra: (1) orbital velocity, (2) pressure, and (3) surface tracking, in an estuarine environment. Linear wave theory immediately rules out the pressure-based spectra at the present deployment depth due to the attenuation of the short period wave signal at frequencies above 0.2 Hz. Only during extreme events is there significant energy in frequencies below this threshold. Similarly, the orbital velocity measurement, although higher in the water column, also suffers a high frequency cut-off. For the buoy and cabled sites, this threshold is approximately 0.55-0.6 Hz. For waves at
higher frequencies, the magnitude of the orbital velocities approaches the precision of the velocity measurement and the processing cannot distinguish between the wave signal and the background noise. Similarly, there is a lower limit on wave height that corresponds to integrating this spectrum of noise. This limit is typically 0.04-0.07 m and depends on the selected bin size, although visual inspection of the observed spectra suggests that reliable non-directional spectra are only attainable for wave heights above 0.10-0.15 m. Similarly, reliable estimates of spectral parameters such as $f_p$, $T_p$, and $T_{m02}$ can only be obtained for more energetic conditions when a well-defined spectrum is measurable.

The third method, acoustic surface tracking (AST), is perhaps the most unreliable because of its unpredictable nature. This is because during low energy (glassy) conditions, the return echo from the surface is reduced due to forward reflection of the acoustic energy. This makes it very difficult to accurately define the location of the surface for wave heights below ~0.2 m, and outliers artificially increase the energy in the observed spectrum. Thus, times of lowest wave energy may actually correspond to the highest reported wave heights if the wave height is based on surface tracking alone. Because of the uncertainty associated with this method, surface tracking data were not included in this analysis.

From June 2005 – September 2006, the deployment at Gloucester Point was exposed to a range of conditions typical of Chesapeake Bay tributaries. Wave height and period are intimately linked to the wind forcing at synoptic time scales. Significant wave height ranged from 0.1-0.8 m and peak period ranged from 1-5 seconds. Typically, storm conditions in the lower York River display an energy peak in the 2-3 second range and the wave field responds quickly to changing wind conditions. During one event on
October 23, 2005, 10 m/s winds from the east led to very energetic conditions with $H_{m0} = 0.67$ m and $T_p = 4.1$ seconds. Zero-crossing analysis of the surface elevation time series showed individual wave heights in excess of 1.0 m and $H_{max} = 1.24$ m. Perhaps the most interesting feature of this event was the appearance of low-amplitude, long period waves with $H_{m0} = 0.35$ m and $T_p = 5.3$ s during the waning hours of the storm, a similar response to that often observed at Goodwin Islands. This behavior is due to attenuation by bottom friction and viscosity, as well as non-linear energy transfers among frequency components that shift the spectral peak from higher to lower frequencies as waves propagate along the fetch. Additionally, the absence of wind energy input at high frequencies during the waning hours of the storm causes low frequency spectral components to become increasingly important, thus altering the spectral form.

Figure 4-11 shows the percentage exceedance curve for significant ($H_{m0}$) and max ($H_{max}$) wave height (calculated as $H_{1/100} = 1.67H_{m0}$) at Gloucester Point, VA. The results indicate that during much of the time (> ~80%) the significant wave height ($H_{m0}$) is relatively low, less than 0.10 m. However, there is a potential for brief periods of high energy, and the difference between low energy glassy conditions and energetic conditions is even more pronounced at Gloucester Point than at Goodwin Islands. The 1% exceedance significant ($H_{m0}$) and maximum ($H_{max}$) wave heights are 0.30 m and 0.57 m, respectively. During extremely rare events (< 1%), wave height and period can substantially exceed these values as they did during Hurricane Isabel when significant wave height and peak period reached 1.6 m and 5.0 s, respectively.
4.6 Conclusions

This chapter highlights the utility of the wind and wave dataset collected as part of the VIMS Observing System effort to characterize estuarine wave characteristics. At Gloucester Point a sixteen-month wave dataset was analyzed to characterize typical estuarine wave conditions and examine exceedance curves for wave height. Significant wave height ranged from 0.1-0.8 m and peak period ranged from 1-5 seconds. Storm conditions at Gloucester Point displayed an energy peak in the 2-3 second range and the wave field responds quickly to changing wind conditions. At Goodwin Islands, a ten-month wave dataset was analyzed to generate several engineering tools, including joint plots of wave height and period, wave height and period exceedance curves, and to examine wave height distributions in the Chesapeake Bay tributaries. The spectral significant wave height ranged from a minimum of < 0.1 m during calm conditions to a maximum of 1.67 m during Tropical Depression Ernesto. Spectral peak period ranged from < 1.0 second to 5.8 seconds for locally generated wind waves, although some records show evidence of longer period ocean swell (> 10 seconds) during low energy conditions.
### Chapter 4 Tables

<table>
<thead>
<tr>
<th>Station ID</th>
<th>Site</th>
<th>Deployment Dates</th>
</tr>
</thead>
<tbody>
<tr>
<td>PI</td>
<td>Poplar Island</td>
<td>October 26 – November 9, 1995</td>
</tr>
<tr>
<td>TW</td>
<td>Tower Site</td>
<td>July 19-28, 1998</td>
</tr>
<tr>
<td>CC</td>
<td>Calvert Cliffs</td>
<td>October 10-23, 1995</td>
</tr>
<tr>
<td>WTL</td>
<td>Wolf Trap Light</td>
<td>November 6 – August 2, 1990</td>
</tr>
<tr>
<td>TSL</td>
<td>Thimble Shoal Light</td>
<td>Fall 1988 – Spring 1995</td>
</tr>
<tr>
<td>TSE</td>
<td>Thimble Shoal East</td>
<td>Winter – Spring, 1993</td>
</tr>
<tr>
<td>GI</td>
<td>Goodwin Islands, York River</td>
<td>March 22 – July 5, 1995</td>
</tr>
<tr>
<td>CI</td>
<td>Catlett Islands, York River</td>
<td>February – May, 1996</td>
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</table>

Table 4-1. Summary of previous wave studies in Chesapeake Bay.
Table 4-2. Wind statistics at Gloucester Point, VA for July 2004 - December 2006.

<table>
<thead>
<tr>
<th>Season</th>
<th>Calm (%)</th>
<th>Moderate (%)</th>
<th>Strong (%)</th>
<th>GP % Favorable</th>
<th>GI % Favorable</th>
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<tbody>
<tr>
<td></td>
<td>0-5 m/s</td>
<td>5-10 m/s</td>
<td>&gt; 10 m/s</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fall</td>
<td>85</td>
<td>15</td>
<td>&lt; 1</td>
<td>3.5</td>
<td>4.9</td>
</tr>
<tr>
<td>Winter</td>
<td>82</td>
<td>18</td>
<td>&lt; 1</td>
<td>0.9</td>
<td>1.5</td>
</tr>
<tr>
<td>Spring</td>
<td>77</td>
<td>23</td>
<td>&lt; 1</td>
<td>2.5</td>
<td>5.4</td>
</tr>
<tr>
<td>Summer</td>
<td>88</td>
<td>12</td>
<td>&lt; 1</td>
<td>3.6</td>
<td>4.8</td>
</tr>
<tr>
<td>Overall</td>
<td>83</td>
<td>17</td>
<td>&lt; 1</td>
<td>2.7</td>
<td>4.1</td>
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</table>
Table 4-3. Percentage exceedance values for wave height and period at Goodwin Islands, VA for February 16, 2006 - December 7, 2006.

<table>
<thead>
<tr>
<th>% Exceedance</th>
<th>$H_{m0}$ (m)</th>
<th>$H_{max}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$T_{m02}$ (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>0.17</td>
<td>0.28</td>
<td>2.3</td>
<td>1.6</td>
</tr>
<tr>
<td>33</td>
<td>0.22</td>
<td>0.35</td>
<td>2.6</td>
<td>1.7</td>
</tr>
<tr>
<td>10</td>
<td>0.35</td>
<td>0.56</td>
<td>3.3</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>0.42</td>
<td>0.69</td>
<td>3.7</td>
<td>2.2</td>
</tr>
<tr>
<td>1</td>
<td>0.77</td>
<td>1.21</td>
<td>5.1</td>
<td>2.6</td>
</tr>
<tr>
<td>Maximum</td>
<td>1.67</td>
<td>2.78</td>
<td>5.8</td>
<td>4.0</td>
</tr>
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</table>
Figure 4-1. Site locations for York River wind and wave observations (GP = VIMS Gloucester Point wind and wave, GI = VIMS Goodwin Islands wave, GI MET = VIMS Goodwin Islands wind, YKRV2 = NDBC wind station).
Figure 4-2. Station locations for previous wave studies in Chesapeake Bay.
Figure 4-3. Monthly mean one-hour wind speed at Gloucester Point, VA for July 2004 – December 2006.
Figure 4-4. Seasonal histograms of wind direction at Gloucester Point, VA, June 2004-December 2006.
Figure 4-5. Wave height and period at Goodwin Islands, VA with isolines of wave steepness ranging from $H/L = 0.01$ to $H/L = 0.07$, increasing counter-clockwise for February 16, 2006 - December 7, 2007.
Figure 4-6. Illustration of post-event swell propagation indicating times of low amplitude ($H_s < 0.2$ m), long period ($T_p > 2.5$ sec) waves at Goodwin Islands, VA from 24-28 April, 2006.
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Figure 4-10. Joint plot of significant wave height and peak period from August 31 – September 2, 2006 displaying the hysteresis loop for the wave growth (circle, solid) and decay (square, dashed) phases at Goodwin Islands, VA. Isolines of steepness are shown increasing counter-clockwise from 0.01 to 0.07.
Figure 4-11. Percentage exceedance curve for wave height at Gloucester Point, VA for June 2005 - September 2006.
CHAPTER 5

EVALUATION OF WAVE HEIGHT DISTRIBUTIONS
5 EVALUATION OF WAVE HEIGHT DISTRIBUTIONS

5.1 Abstract

This chapter focuses on defining the unique nature of waves in the estuarine environment in an attempt to identify distinguishing features of estuarine waves relative to those in the open ocean. To achieve this objective, observed wave height distributions from a station at the mouth of the York River estuary were compared with the theoretical Rayleigh distribution. The results demonstrate that the Rayleigh assumption is inaccurate in this environment, and that an empirically derived Weibull distribution provides substantially better agreement with observed wave height distributions, especially during times of low wave energy. This finding has important implications for the distribution of wave heights in estuarine environments and predictions of maximum wave height based on the underlying wave height distribution.

5.2 Introduction

Despite the importance of waves in estuarine systems, there is surprisingly little published work on the characteristics and behavior of waves in estuaries or lakes of similar geometry. de Lange and Healy (1990) examined bimodal wave spectra in a New Zealand estuarine lagoon, and found that the average spectrum displayed a low frequency peak at 3.5 seconds, representing wave energy transmitted into the harbor from the external wave field, and a high frequency peak at 1.2 seconds representing local wind waves. Records dominated by the low frequency peak showed good agreement with the Rayleigh distribution whereas those dominated by the high frequency peak deviated.
This suggests that filtering at the harbor mouth resulted in narrower banded spectra that conformed better to the Rayleigh distribution than the finite bandwidth spectra of the locally generated wind waves. Barthel and Ing (1982) examined the height distribution of waves in the Weser estuary using a chi-squared statistical test to find the best correspondence with theoretical distributions and concluded that the most likely distribution of the three examined (normal, log-normal, Rayleigh) was the Rayleigh distribution. The authors examined the role of spatially and temporally varying current fields and found that currents influenced the height distributions such that a better correspondence with the Rayleigh distribution existed when waves and currents traveled in the same direction.

Field evidence generally supports the claim that the Rayleigh distribution accurately describes the observed wave heights under most conditions. One prominent exception is for the high end of the probability tail, where the Rayleigh distribution is found to over-predict the heights of the highest waves (Forristall, 1978). Green (1994) examined the agreement between observational data and the Rayleigh distribution and considered wave breaking as a factor that could limit the occurrence of the highest waves. However, the author could not rule out the equally likely influence of imperfect correlation between crests and troughs for finite bandwidth seas, which would prevent the pairing of the highest crests with the lowest troughs to produce the largest waves (see Figure 2-1 for reference). Instead, a large crest is likely to be followed by an average trough, and vice versa. Given the difficulty in parameterizing the exact physical processes responsible for the deviation from the Rayleigh distribution, Forristall (1978) suggests that empirically derived distributions be employed whenever precise predictions
of maximum wave heights are needed; however, no such parameterizations exist for estuarine environments. Thus, up until now it has been necessary to assume agreement with the theoretical Rayleigh distribution (Longuet-Higgins, 1952).

This chapter will examine wave height distributions in the York River estuary and compare the observed distributions to the theoretical Rayleigh distribution proposed by past investigators. The objectives are as follows: (1) Assess the agreement of observed wave height distributions with the Rayleigh distribution using the chi-squared and Kolmogorov-Smirnov statistical tests, (2) Identify characteristics of the physical forcing that cause deviations from theory, and (3) If the Rayleigh distribution is unsatisfactory in describing the observed distributions, obtain an empirically derived distribution that can be used for future studies in the Chesapeake Bay tributaries and similar estuaries elsewhere.

5.3 Methods

The analysis of wave height distributions requires a detailed record of the sea surface elevation. For the purposes of this study, surface elevation data were collected using a 1 MHz Nortek Acoustic Wave and Current Meter (AWAC) deployed in approximately 10 m water depth near Goodwin Islands, VA (see Figure 4-1). Surface elevation time series were recorded using the instrument’s acoustic surface tracking capability, which serves as an inverted echo-sounder from a bottom-mounted frame. For each burst, 4096 samples were collected at a rate of 4 Hz once per hour. In total, 4664 surface elevation records were recorded from February – September 2006.
Quality of surface tracking data depends on a number of factors, but is primarily related to the strength of the surface return relative to the background noise. Surface tracking with the AWAC relies on a center vertical beam so significant tilts in the instrument mount can affect data quality due to forward reflection of acoustic energy. Typically, surface roughness and ripples can compensate for this effect to some degree. However, as waves grow in size and steepness, a similar problem arises as the surface slope exceeds a critical value and overly steep waves can result in data loss due to “bad detects.” Additionally, times of active wave breaking can pose similar problems for acoustic surface tracking methods due to entrainment of air bubbles near the surface. To a certain degree, bad data points can be screened and interpolated without a substantial loss of information; however, records with substantial data loss should be discarded.

To screen the surface tracking records, a linear trend is removed from the surface elevation record to remove long-term changes in water elevation due to tidal variations. In computing the wave parameters from the surface elevation records, Nortek applies an iterative procedure to remove data points that are greater than 5 standard deviations from the mean and linearly interpolates the removed points, after which the process is repeated. If greater than 10% of the record is removed, the processing software reverts to either the pressure or velocity-based spectral estimates for wave parameters. In this case, parameters that can only be determined from zero-crossing analysis of the sea surface elevation are evaluated using statistically reasonable multiples of the significant wave height ($H_{1/10} = 1.27H_m$, $H_{\text{max}} = 1.67H_m$). The accuracy of using a constant transfer coefficient is discussed in detail in Chapter 3. Thus, there is a continuous record of $H_{\text{max}}$, regardless of whether the maximum wave height was actually determined directly from
the zero-crossing analysis for each burst or as $H_{\text{max}} = 1.67H_{m_0}$. For the purposes of this study, records with greater than 5% data loss were neglected. The screened data records were then analyzed using a zero-upcrossing algorithm to determine the distribution of wave heights. For each record, the empirical exceedance probability distribution was determined from zero-crossing analysis of the surface elevation time series by ranking individual wave heights in ascending order and determining the percentage of waves that exceed each threshold value.

The Rayleigh probability density function and cumulative exceedance probability distribution for the normalized wave heights are defined as follows (Green, 1994):

$$p_R(\xi) = 2\xi \exp(-\xi^2)$$  \hspace{1cm} (Eq. 5-2)

$$P_R(\xi_0) = P(\xi > \xi_0) = \int_{\xi_0}^{\infty} p_R d\xi = \exp(-\xi_0^2)$$  \hspace{1cm} (Eq. 5-3)

where $\xi = H/H_{m0}$ is the normalized wave height. Hu (2002) provides an alternative to the traditional definition by considering the Rayleigh distribution as a special case of the Weibull distribution. The probability density function and cumulative exceedance probability distribution are defined as follows:

$$p(H) = \frac{\alpha H^{\alpha-1}}{\theta^\alpha} \exp \left[ -\left( \frac{H}{\theta} \right)^\alpha \right]$$  \hspace{1cm} (Eq. 5-4)

$$P(H > H_0) = \exp \left[ -\left( \frac{H_0}{\theta} \right)^\alpha \right]$$  \hspace{1cm} (Eq. 5-5)

where $\alpha$ and $\theta$ are shape and scale parameters, respectively. From the statistical moments of the distribution and the definition of $H_{rms}$, the scale parameter can be defined as (Hu, 2002),
\( \theta = \frac{H_{\text{rms}}}{\sqrt{\Gamma\left(\frac{2}{\alpha} + 1\right)}} \)  
(Eq. 5-6)

Thus, for \( \alpha = 2.0 \), the scale parameter, \( \theta \), is equal to \( H_{\text{rms}} \) and Eq. 5-4 reduces to the well-known Rayleigh probability density function (Eq. 2-1),

\[ p(H) = \frac{2H}{H_{\text{rms}}^2} \exp\left[-\left(\frac{H}{H_{\text{rms}}}\right)^2\right] \]  
(Eq. 5-7)

which will be compared to the observed wave height distributions using the chi-squared and Kolmogorov-Smirnov statistical tests, as discussed later.

For empirical fits to wave height distributions in the York River estuary, the flexibility of the Weibull distribution with variable shape and scale parameters makes it possible to obtain an empirically-derived distribution function for waves in the Chesapeake Bay tributaries. Additionally, it may be possible to link the variations in \( \alpha \) and \( \theta \) to other characteristics of the physics such as finite bandwidth effects or steepness-induced breaking. To accomplish this objective, the Weibull distribution is fit to the observed wave height distributions by iteratively solving Eqs. 5-5 and 5-6 for \( \alpha \) and \( \theta \).

Using \( \alpha = 2.0 \) as an initial guess (and therefore \( \theta = H_{\text{rms}} \)), the MATLAB function \texttt{fminzero} was used to solve for a new value of \( \alpha \) to minimize the sum of the squared errors. A new value of \( \theta \) was then estimated from Eq. 5-6 and the process was repeated iteratively to achieve the best fit. It was observed that the best fit could generally be obtained after approximately five iterations.

The fit of observational data to the theoretical Rayleigh distribution is evaluated first using the chi-squared (\( \chi^2 \)) goodness-of-fit test. From sampling theory, if \( N \) outcomes
are divided into $M$ classes, where $X_m$ is the number of outcomes in class $m$ and $p_m$ is the theoretical probability of an outcome being in class $m$, then the random variable,

$$
\chi^2 = \sum_{m=1}^{M} \frac{(X_m - Np_m)^2}{Np_m}
$$

(Eq. 5-8)

has a chi-squared distribution (Chin, 2000). The critical value of the test statistic, $\chi^2_{\alpha, df}$, is defined for a confidence level, $\alpha$, and $df$ degrees of freedom, where $df$ equals the number of data classes, $M$, minus the number of parameters estimated from the data. For the Rayleigh distribution, only one parameter, $H_{rms}$, is estimated from the data. If 

$\chi^2 < \chi^2_{\alpha, df}$, the empirical distribution and theoretical Rayleigh distribution are indistinguishable at the $\alpha$ level. If $\chi^2 > \chi^2_{\alpha, df}$, the deviation of the empirical distribution from the Rayleigh distribution is significant at the $\alpha$ level. For the purposes of this analysis, the statistical tests were conducted at two $\alpha$ levels, 0.01 and 0.05. Since it is harder to reject the null hypothesis that the distributions are indistinguishable at the $\alpha = 0.01$ level, there will be a higher percentage of records that conform to the Rayleigh distribution at this level compared to the $\alpha = 0.05$ level. The tests were conducted at both significance levels because it was unclear from previous studies what level was most appropriate.

An alternative to the chi-squared test is the Kolmogorov-Smirnov test. If the theoretical distribution falls entirely within the confidence band around the empirical distribution, then the two are indistinguishable at the level of the confidence bands (Neter et al., 1978). The alternative hypothesis is accepted if the theoretical distribution does not fall entirely within the confidence bands (Green, 1994). This test was carried out using the "kstest" function in MATLAB 7.
5.4 Results

5.4.1 Rayleigh Distribution

The combined exceedance probability distribution for all records is shown in Figure 5-1 with the theoretical curve for the Rayleigh distribution. When plotted in this manner, it is easy to see the shortcomings of the Rayleigh distribution reported by other investigators, namely the observed excess of mid-range wave heights and the over-prediction of the occurrence of the highest waves (Tayfun, 1981).

The results of the statistical tests for the individual records are shown in Figure 5-2 and Table 5-1, which show the percentage of records conforming to the Rayleigh distribution. The results from the chi-squared test will be discussed first. When considering all bursts, 34% and 49% percent of records are indistinguishable from the Rayleigh distribution at the 0.05 and 0.01 levels, respectively. These percentages seem relatively low considering the widespread application and validation of the Rayleigh distribution in the literature. However, when excluding bursts with significant wave height less than 0.2 m, the percentage of records conforming to the Rayleigh distribution increases to 57% and 75% for $\alpha = 0.05$ and $\alpha = 0.01$. Similarly, percentages are shown for larger significant wave height thresholds in Figure 5-2 with the number of records displaying a significant wave height larger than the minimum threshold height (0.0 m, 0.10 m, 0.20 m, etc.). For example, when considering only the 80 records with $H_{m0} > 0.5$ m, the percentages of records conforming to the Rayleigh distribution are 89% and 94% for $\alpha = 0.05$ and $\alpha = 0.01$. Thus, it appears that the observed wave height distribution
during more energetic times is more likely to be indistinguishable from the Rayleigh
distribution than during low energy times.

Figure 5-2 and Table 5-1 also display the results of the Kolmogorov-Smirnov test.
The results show similar trends in that a higher percentage of records conform to the
Rayleigh distribution with increasing energy level. In general, the K-S test suggests a
higher percentage of records conform to the Rayleigh distribution, which is expected
given that it is a more rigorous statistical test than the chi-squared test. Thus, it is more
difficult to demonstrate that the observed distribution is significantly different than the
Rayleigh distribution.

5.4.2 Weibull Distribution

As previously discussed, the two-parameter Weibull distribution introduces some
additional flexibility when examining observed wave height distributions. The results
from the statistical analysis suggest that a non-Rayleigh distribution may provide better
agreement with the observed wave height distributions. Using Eqs. 5-5 and 5-6, the
Weibull shape and scale parameters were fit to the observed distributions using an
iterative procedure to minimize the sum of the squared errors in order to develop an
empirically-derived wave height distribution appropriate for the Chesapeake Bay
tributaries.

Histograms of the computed values of the shape parameter, \( \alpha \), and the normalized
scale parameter, \( \theta/H_{\text{rms}} \), are shown in Figure 5-3. Note that in the Weibull distribution
(Eqs. 5-4 and 5-5), \( \theta \) essentially replaces \( H_{\text{rms}} \) when normalizing the individual wave
heights. The fit values of \( \alpha \) appear to be normally distributed about a mean value of 2.26.
From Eq. 5-6, this corresponds to a $\theta/H_{\text{rms}}$ value of 1.022. Interestingly, there appears to be little correlation between values of $\alpha$ from record to record. Even at a lag of only one hour, the correlation coefficient has a value of 0.23 and decreases to 0.15 for a two-hour lag (i.e., essentially no correlation).

The value of $\alpha$ is plotted against the significant wave height ($H_{m_0}$) in Figure 5-4 with the linear least squares best fit to the binned data points (binning interval of 0.15 m),

$$\alpha = aH_{m_0} + b$$  \hspace{1cm} \text{(Eq. 5-9)}

where $\alpha =$ Weibull shape parameter and $H_{m_0} =$ spectral significant wave height (meters). The best-fit slope and intercept are $a = -0.2 \pm 0.065$ and $b = 2.30 \pm 0.034$ for the 95% confidence interval. During low energy conditions, there is considerable scatter in the value of $\alpha$, possibly due to the influence of boat wake. As energy level increases, there is less scatter in the data. The low degree of similarity in $\alpha$ between successive records is most likely a result of the large variation in $\alpha$ for low values of significant wave height, which are prevalent in the record. Recall that at Goodwin Islands, significant wave height exceeds ~0.3 m only 10% of the time (Table 4-3). This suggests that the observed value of $\alpha$ varies in a way that is difficult to predict analytically during times dominated by transient waves and that the best approach may be to employ mean values of the shape and scale parameters. The resulting probability density function and cumulative exceedance distribution are given by Eqs. 5-4 and 5-5, with $\theta = 1.022H_{\text{rms}}$ and $\alpha = 2.26$.

Using the same approach as for the Rayleigh distribution, conformity with the empirically-derived Weibull distribution with mean values of $\alpha$ and $\theta$ was tested using the chi-squared and Kolmogorov-Smirnov statistical tests. The results are shown in Figure 5-5 and Table 5-2. In general, there is better agreement between the observed
wave height distributions and the empirical Weibull distribution for all energy levels relative to the Rayleigh distribution (Figure 5-2 and Table 5-1). This is to be expected given that the distribution itself was derived from the observational data; however, it may provide a useful tool for future investigators. The most substantial improvement over the Rayleigh distribution is during periods of low wave energy. For example, when considering the chi-squared test at the 0.05 level, 65% of all records conform to the empirical Weibull distribution while only 34% conform to the Rayleigh distribution. Even when considering only records with significant wave height greater than 0.2 m the discrepancy in conformity is 78% Weibull compared to 57% Rayleigh.

5.5 Discussion

The results for the York River estuary mouth data indicated poor agreement with the Rayleigh distribution for low energy times. One possible explanation for this observation is that higher energy events are associated with a narrow banded spectrum, so the narrow-band assumption inherent in the Rayleigh analysis is more likely to be satisfied. An equally likely explanation for this observation is the influence of non-linear, transient waves such as boat wakes that dominate the wave field during low energy periods. This is because the application of the Rayleigh distribution to ocean waves by Longuet-Higgins (1952) does not apply to regular trains of waves produced by a simple organized mechanism such as the transverse waves produced by a ship. Instead, the observed sea surface oscillations are treated as the linear superposition of a large number of constituents of nearly equal frequency (narrow band) and of random phase. Similarly, it is unclear how these non-linear, transient waves manifest themselves in
methods of analysis based on linear wave theory. Thus, only during more energetic periods, when the wave field is dominated by wind wave energy, should the Rayleigh distribution apply.

Previous investigators provide a broad basis for comparison with the present results. Hu (2002) examined the effects of non-linearity and spectral width on the shape parameter of the Weibull distribution for wave heights using Monte Carlo simulation. Spectral width was quantified using Goda’s peakedness parameter, $Q_p$ (Goda, 1970),

$$Q_p = \frac{2}{m_0} \int_0^\infty f[S(f)]^2 df$$  \hspace{1cm} (Eq. 5-10)

the value of which approaches infinity for extremely narrow-banded spectra and unity for extremely broad banded spectra. Previous investigators have reported values from 2 to 3 for coastal waters (Goda, 1976). For this range of bandwidths, Hu (2002) found that $\alpha$ ranged from 2.13 - 2.15, and approached its theoretical Rayleigh value of 2.0 for narrow banded spectra ($Q_p \sim 11$).

At Goodwin Islands, $Q_p$ ranged from 1 to 3 and displayed a median value of 2.0. These values of $Q_p$ represent spectra that are relatively more broad band than those conditions simulated by Hu (2002), so it would be expected that the shape parameter, $\alpha$, might deviate further from the theoretical Rayleigh value. This is in fact what is observed at Goodwin Islands, recalling that the mean value of $\alpha$ determined from the observed wave height distributions is 2.26. A value of $\alpha$ greater than 2.0 fits the expected mathematical model proposed by Forristall (1978), who suggests that $\alpha$ should be greater than 2.0 to correct for the typical deficiency that observed data tend to fall progressively further below the Rayleigh prediction for higher and higher wave heights. It should be noted from Figure 5-4 that $\alpha$ is negatively correlated with the observed
significant wave height \( (H_{mn}) \), such that its value approaches the theoretical Rayleigh value of 2.0 for larger wave heights. This is consistent with the results of Hu (2002), since higher energy wave events generally correspond to relatively narrow banded spectra. Thus, for larger wave events, a lower value of \( \alpha \) may be applied to more accurately match the observed wave height distribution.

Forristall (1978) examined wave height distributions for hurricane storm data in the Gulf of Mexico, where he highlights the inadequacies of theoretical distributions in predicting the occurrence of the highest waves and suggests the use of empirically derived distributions when highly accurate predictions are required for engineering studies. Similar to the approach taken in this study, a more flexible Weibull-type distribution is fit to the data,

\[
P(X > X_0) = \exp \left( -\frac{1}{\beta} X_0^\alpha \right)
\]  
(Eq. 5-11)

where \( X_0 = H_0 / \sqrt{m_0} \) and \( \alpha \) and \( \beta \) are shape and scale parameters. Forristall found \( \alpha = 2.13 \) and \( \beta = 8.42 \) for 116 hours of data. This formulation is slightly different than that presented by others (Hu, 2002; Green, 1994), in that the wave heights are normalized by \( \sqrt{m_0} \) instead of \( H_{rms} \), although the distribution exhibits some similarities with Eq. 5-5. This suggests that the parameters \( \alpha \) and \( \theta \) from Eq. 5-5 might be converted to their equivalent parameters, \( \alpha \) and \( \beta \), in Eq. 5-11.

Eqs. 5-5 and 5-11 can be re-written and equated to solve for Forristall’s scale parameter, \( \beta \), as follows,

\[
P(H > H_0) = \exp \left[ -\frac{1}{\beta} \left( \frac{H_0}{\sqrt{m_0}} \right)^\alpha \right] = \exp \left[ -\left( \frac{\sqrt{m_0}}{\theta} \right)^\alpha \left( \frac{H_0}{\sqrt{m_0}} \right)^\alpha \right]
\]  
(Eq. 5-12)
\[ \beta = \left( \frac{\theta}{\sqrt{m_0}} \right)^\alpha \]  \hspace{1cm} \text{(Eq. 5-13)}

At Goodwin Islands, the median observed value of \( \theta / \sqrt{m_0} \) is 2.74. For an \( \alpha = 2.26 \), Eq. 5-13 evaluates to a value of 9.76 for \( \beta \). Similar to the behavior of \( \alpha \) during low energy conditions, the computed value of \( \beta \) at low wave heights displays considerably more variability than during higher energy events, where \( \beta \) displays less scatter around the mean value. The empirically derived values of \( \alpha \) and \( \beta \) compare reasonably well with Forristall’s findings, although it is clear that in the Chesapeake Bay tributaries, some fundamentally different processes dictate the agreement with the Rayleigh distribution than for storm waves in the Gulf of Mexico. Krogstad (1985) found values of \( \alpha = 2.38 \) and \( \beta = 12.9 \) from North Atlantic data, suggesting that considerable variability exists among sites. At this point, it is believed that either finite bandwidth effects or the effects of transient waves during low energy conditions may account for the observed differences in the wave height distribution.

We have now obtained an empirically derived wave height distribution appropriate for use in the Chesapeake Bay tributaries. Using the notation presented by Forristall (1978), the exceedance distribution function can be written as:

\[ P(X > X_0) = \exp \left( -\frac{X_0^\alpha}{\beta} \right) \]  \hspace{1cm} \text{(Eq. 5-14)}

where \( X_0 = H_o / \sqrt{m_0} \), \( \alpha = 2.26 \), and \( \beta = 9.76 \). One of the advantages of this functional form is that it can be manipulated to give similar statistics to those presented by Longuet-Higgins (1952) regarding the relationship between significant and maximum wave height (Forristall, 1978). Such a method for predicting the maximum expected wave height
from spectral moments was presented in Chapter 3. Following the approach of Longuet-Higgins (1952), Forristall derives a relationship for the expected value of the maximum normalized wave height in \( N \) waves using Eq. 5-14 in place of the Rayleigh distribution. Using the values of \( a \) and \( \beta \) from Eq. 5-14, the resulting equation can be used for predictions,

\[
E(X_{\text{max}}) = \left[ \beta \ln N \right]^{1/\alpha} \left[ 1 + \frac{\gamma}{\alpha \ln N} \right] \quad \text{(Eq. 5-15)}
\]

where \( X_{\text{max}} = H_{\text{max}} / \sqrt{m_0} \), \( \gamma = 0.5772 \), and \( N \) = number of waves. Table 5-3 displays the expected normalized maximum wave heights for the empirical and Rayleigh distributions and the ratio between the predictions (Weibull divided by Rayleigh).

For reference, recall that the spectrally-derived significant wave height is defined as \( H_{\text{m0}} / \sqrt{m_0} = 4.004 \), based on the Rayleigh distribution. In general, the empirical distribution predicts lower ratios than the Rayleigh distribution. For comparison, a burst length of 1024 seconds and mean period of 4 seconds yields approximately \( N = 250 \) waves, for which the ratio \( H_{\text{max}} / \sqrt{m_0} = 6.107 \). This results in a \( H_{\text{max}} / H_{\text{m0}} \) ratio of 1.53 compared to 1.75 for the Rayleigh distribution. For a significant wave height of 1.5 m, as observed during Tropical Depression Ernesto, this corresponds to a predicted maximum wave height of 2.30 m for the empirical distribution compared to a predicted maximum wave height of 2.63 m for the Rayleigh distribution. Thus, for this example, the Rayleigh distribution over-predicts the expected value of the maximum wave height relative to the empirical distribution by approximately 0.33 m, or 14%.
5.6 Conclusions

This chapter presented an analysis of wave height distributions in the York River estuary, and compared observed distributions with the theoretical Rayleigh distribution. The results demonstrate that typical observations and theories, generally applied to more coastal open-ocean settings, do not necessarily hold true in estuarine environments. Most notably, the commonly held notion that the Rayleigh distribution accurately describes the observed wave height distribution in a range of environments was shown to be inaccurate in the Chesapeake Bay tributaries, and that an empirically derived Weibull distribution provides substantially better agreement with observed wave height distributions, especially during times of low wave energy. The empirical Weibull distribution (Eq. 5-4 and 5-5), with shape and scale parameters of $\alpha = 2.26$ and $\theta = 1.022H_{\text{rms}}$, provided a more accurate fit to wave height distributions than the Rayleigh distribution and provides future investigators with a tool that can be used in engineering practice.
### 5.7 Chapter 5 Tables

<table>
<thead>
<tr>
<th>$H_{\text{rel}}$ height (m)</th>
<th>% Rayleigh $\chi^2, \alpha = 0.01$</th>
<th>% Rayleigh $\chi^2, \alpha = 0.05$</th>
<th>% Rayleigh K-S, $\alpha = 0.01$</th>
<th>% Rayleigh K-S, $\alpha = 0.05$</th>
<th>Number of records</th>
</tr>
</thead>
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<tr>
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<td>96</td>
<td>80</td>
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Table 5-1. Percentage of records conforming to the Rayleigh distribution.
<table>
<thead>
<tr>
<th>$H_{ex}$ exclusion height (m)</th>
<th>% Weibull $\chi^2$, $\alpha = 0.01$</th>
<th>% Weibull $\chi^2$, $\alpha = 0.05$</th>
<th>% Weibull K-S, $\alpha = 0.01$</th>
<th>% Weibull K-S, $\alpha = 0.05$</th>
<th>Number of records</th>
</tr>
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<tbody>
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Table 5-2. Percentage of records conforming to the empirical Weibull distribution
<table>
<thead>
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<th>$N$</th>
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<th>Rayleigh</th>
<th>Empirical/Rayleigh</th>
</tr>
</thead>
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<tr>
<td>5</td>
<td>3.919</td>
<td>4.135</td>
<td>0.948</td>
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<td>10</td>
<td>4.403</td>
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<td>0.882</td>
</tr>
<tr>
<td>200</td>
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<tr>
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<td>7.594</td>
<td>0.865</td>
</tr>
<tr>
<td>1000</td>
<td>6.683</td>
<td>7.744</td>
<td>0.863</td>
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</tbody>
</table>

Table 5-3. Expected normalized maximum wave heights, $H_{\text{max}}/\sqrt{m_0}$ for the empirical Weibull distribution (Eq. 5-14) with $\alpha = 2.26$ and $\beta = 9.76$ and the Rayleigh distribution.
Figure 5-1. Combined exceedance probability curve for all records at Goodwin Islands, VA (points) and theoretical Rayleigh distribution (dashed line).
Figure 5-2. (a) Percentage of records displaying wave height distributions that are indistinguishable from the Rayleigh distribution at the $\alpha$ level for each threshold wave height. (b) The number of records displaying $H_{m0}$ wave heights greater than the threshold wave height.
Figure 5-3. Histograms of the fitted Weibull shape ($\alpha$) and scale ($\theta$) parameters at Goodwin Islands, VA.
Figure 5-4. The Weibull shape parameter ($\alpha$) vs. the significant wave height ($H_{m0}$) at Goodwin Islands, VA. Binned data points (medians) and standard deviations are shown with the linear least squares best-fit to the binned data.
Figure 5-5. (a) Percentage of records displaying wave height distributions that are indistinguishable from the empirically fit Weibull distribution at the $\alpha$ level at each threshold wave height. (b) The number of records displaying $H_{m0}$ wave heights greater than the threshold wave height.
CHAPTER 6

EVALUATION OF A STEADY STATE NUMERICAL WAVE MODEL
6 EVALUATION OF A STEADY STATE NUMERICAL WAVE MODEL

6.1 Abstract

This chapter assesses the suitability of the STWAVE numerical wave model for application to wind wave generation in the Chesapeake Bay tributaries. Using meteorological data from the VIMS Observing System and knowledge of the local wind climatology, model runs were selected to simulate typical and extreme conditions for the region: (1) summer sea breeze, (2) winter wind event, (3) extratropical storm, and (4) tropical storm. Model predictions were compared to observed wave parameters and spectra at two locations in the lower York River. Interpretation of the model results identified the role of shoal regions in the vicinity of the York River mouth in dissipating waves originating in the main stem of the Bay, although this effect appears to be over-emphasized in the model. For moderate winds over relatively short fetches, the modeled wave parameters and spectra agreed quite well with observations; however, for strong winds over long fetches, wave heights were under-estimated. For nearly all cases, peak periods were over-estimated due to the steady-state assumption inherent in the model. Given the mixed model performance compared to the observational data, further research is recommended to identify ways to better implement the model in estuarine environments.

6.2 Background

Several investigators have employed modeling techniques to examine wind-wave growth in Chesapeake Bay. The simplest of these models is the use of predictive wave
equations, which yield estimates of wave height and period for a given wind speed and fetch at a study site. These equations typically assume fetch-limited deep-water wave growth (Bishop, 1983). Examples include the JONSWAP (Hasselmann et al., 1973), Donelan (Donelan, 1980), and SMB equations (CERC, 1977). An additional level of complexity can be added by allowing wave growth to be influenced by water depth, resulting in finite-depth predictive equations, which use the average depth along the fetch as an additional variable (Young and Verhagen, 1996). However, all predictive equations of this form assume steady-state, and thus neglect duration as an important variable in some environments. Background work conducted as part of this study suggests that during some conditions, wind speed and direction (i.e. fetch) may change more rapidly than the wave field can respond, especially when the wind forcing is aligned with long fetches that open to the main stem of the Bay. As a result, these models can overestimate wave height because they assume the wave field is in constant equilibrium with wind forcing (Babineaux et al., 2007). An additional drawback of these equations is that they provide no spectral information, and only one (Donelan, 1980) provides an estimate of mean wave direction other than to assume that it follows the wind direction.

Lin et al. (2002) evaluated the time-dependent GLERL and SWAN wave models and compared the results to field studies in the upper Bay (see Figure 4-2 and Table 4-1). The GLERL model is a parametric deep-water wave model based on the local momentum balance equation. It does not include shallow water wave effects and the energy spectrum is assumed to follow the JONSWAP spectral shape. In comparison, SWAN is a spectral wave model based on the action density balance equation. The model accounts for wind wave growth and propagation, white capping, wave-wave interaction, bottom
friction, and depth-induced breaking. The results for the SWAN model in Chesapeake Bay show that the mean wave direction tends to follow wind direction quickly with a response time of about one hour, at least in the upper Bay, where fetch is generally more constrained. Both models over-predicted the observed wave height, and SWAN predicted wave direction better than GLERL, most likely due to the inclusion finite-depth effects. In conclusion, the authors found that GLERL was a better choice for modeling in the upper Bay, in part because of its computational efficiency.

While the numerical models appear to provide adequate results for wind-generated waves in the upper Chesapeake Bay, they can be complicated and time-consuming to implement and execute. Additionally, their performance was validated with only a relatively small dataset, during which wave energy was low. It is unclear whether the additional work required to implement the GLERL or SWAN models is required to accurately model waves in the Bay's tributaries. Certainly the models provide more robust and complete results than predictive equations alone, but also require substantially more effort. This study will attempt to address this issue by implementing a steady-state spectral wave model for use in the Chesapeake Bay tributaries. In terms of complexity, the steady-state spectral model lies somewhere in between the predictive equations and the time-dependent spectral wave models mentioned above.

6.3 Model Description

For the purposes of this study, the STeady-state spectral WAVE model, STWAVE, is evaluated for application in the Bay’s tributaries. STWAVE is a finite difference model written by the U.S. Army Corps of Engineers Coastal and Hydraulics
Lab. The model is frequently used for wave transformation between offshore and nearshore sites for coastal engineering design, sediment transport studies, and nearshore circulation models. The model is considered a “phase-averaged” approach in that it doesn’t save information concerning the relative phases of the individual spectral wave components, and instead assumes that the phases are random. Grid domains are typically less than 40 km square with cell sizes of 25-100m (Thompson et al., 2004). Model output includes wave parameter fields of steady-state significant wave height, peak period, and mean direction over the model domain and directional spectra for selected output points.

STWAVE simulates depth and current-induced refraction and shoaling, depth and steepness-induced breaking, wind-wave growth, wave-wave interaction, and whitecapping. The governing equation for steady-state conservation of spectral wave action along a wave ray is given by Jonsson (1990):

\[
\begin{align*}
\left(C_{\text{ga}}\right)_x \frac{\partial}{\partial x} C_a C_{\text{ga}} \cos(\mu - \alpha) E(\omega_a, \alpha) \omega_r + \\
\left(C_{\text{ga}}\right)_y \frac{\partial}{\partial y} C_a C_{\text{ga}} \cos(\mu - \alpha) E(\omega_a, \alpha) \omega_r = \sum \frac{S}{\omega_r}
\end{align*}
\]  

(Eq. 6-1)

where \( C_{\text{ga}} \) = absolute group velocity, \( C_a \) = absolute wave celerity, \( \mu \) = wave ray direction, \( \alpha \) = wave crest orthogonal, \( E \) = wave energy density, \( \omega_a \) = absolute angular frequency, \( \omega_r \) = relative angular frequency, and \( S \) = energy source and sink terms (i.e. wind input, nonlinear wave-wave interactions, dissipation, and breaking). Parameters with the subscript “\( r \)” denote a frame of reference moving with the current and the parameters in
the nonmoving (absolute) reference frame are denoted with “a.” Figure 6-1 shows the
definition sketch of wave and current vectors. Without the effect of currents, \( \mu \) and \( \alpha \)
(the wave ray and crest orthogonal) are equal, but with currents (\( U \) in Figure 6-1), the
wave energy moves along the rays, \( \mu \), while the wave direction, \( \alpha \), is defined by the crest
orthogonals (Smith et al., 2001).

STWAVE has several key assumptions that should be considered when adapting
the model to the estuarine environment: (1) mild bottom slope and negligible bottom
friction, (2) depth-uniform current, (3) steady-state wind, waves, and currents. Waves in
the tributaries are typically deep-water waves so that the assumptions regarding bottom
slope and bed friction are reasonable over much of the domain. For example, it is
generally accepted that waves begin to “feel” bottom effects when the water depth is
about one half the wavelength. Thus, for a typical water depth of 8m the threshold
wavelength is equal to 16m, which corresponds to a wave period of 3.2 seconds. Wave
periods in this range are only obtained during more significant wind events so that the
majority of the time bottom effects can be ignored entirely, and may only play a minor
role during other times. One region where the bottom assumptions may come into
question is the shoal region, where depths are typically shallow and the shoal to channel
transition can be quite abrupt. With regard to the assumption of a depth-uniform current,
it is rare that one even has the full three-dimensional current field for modeling
applications. Furthermore, while short period waves are indeed the most sensitive to
modification by currents, the rapid depth attenuation of short-period wave energy makes
the influence of mid and bottom-currents less pronounced so that the use of depth-
averaged currents (or surface currents) should be adequate under most conditions.
The final assumption of steady-state wind, waves, and currents is the most difficult to justify and requires validation with field data. While wind fields in the Bay often show spatial correlations with respect to wind speed and direction over tens of kilometers (Wang and Elliot, 1977; Garfield, 1984), complicated topography and sheltering effects will undoubtedly render the wind field non-uniform across the model domain. The model also assumes a steady-state wind field in time so that winds must remain constant long enough for waves to become fetch-limited. In other words, wind duration is not considered in the wave growth computation (Smith, 2001). This means that the model is only appropriate for conditions that vary more slowly than the time required for the waves to propagate through the model domain. Even for a relatively long fetch of 20 km, a 3.5 second wave in 8m water depth transits the model domain in approximately 2 hours. It is hypothesized that with appropriate averaging of wind data, reasonable results might be achieved through a quasi-time stepping mode.

6.4 Objectives

In this chapter, the STWAVE numerical wave model is evaluated for application to the Chesapeake Bay tributaries. Model output is compared to field data collected at two sites in the York River estuary (Figure 4-1): (1) Gloucester Point (TRDI ADCP) and (2) Goodwin Islands (Nortek AWAC). A series of idealized test cases was run, including: (1) summer sea breeze, (2) winter wind event, and (3) extratropical event (“nor’eastern”) that simulates wind conditions typical of the region. The results are compared to periods in the wind record that match the forcing conditions, which will be described in detail later. The final component of the evaluation is a quasi-time stepping...
two-day simulation of conditions leading up to and encompassing the passage of Tropical Depression Ernesto on September 1, 2006. The goal of these tests is to evaluate how well the model predicts key parameters such as significant wave height ($H_m$), peak period ($T_p$), mean direction ($D_m$), and peak direction ($D_p$) for each wind event.

6.5 Methods

For the purposes of this study, the command line executable version of STWAVE 4.0 was acquired from the U.S. Army Corps of Engineers Coastal and Hydraulics Lab (USACE-CHL, http://chl.erdc.usace.army.mil/). Bathymetry data was obtained from the NOAA National Geophysical Data Center (NGDC) 3 Arc-Second Gridded Coastal Relief Model. The original bathymetry dataset obtained from NGDC covered a latitude range of [36° 50' N, 37° 35' N] and a longitude range of [76° 50' W, 75° 55' W]. Since STWAVE requires a Cartesian coordinate system with uniform x and y spacing, the dataset was converted to UTM coordinates bounded by [340000, 415000] easting and [4080000, 4160000] northing. The bathymetry data were then re-gridded to a 100 m grid using Surfer 8 mapping software employing a Kriging algorithm with a symmetrical search ellipse with a radius of 500 m. The gridded bathymetry domain is shown in Figure 6-2. The spatial limits of the domain were selected to include the portions of the main stem of the Bay that are believed to influence the observed wave heights in the lower York River.

Model forcing is implemented by specifying a wind speed and direction relative to the local coordinate system. Additionally, a water level correction can be specified to simulate the effects of tidal action or storm surge. For the purposes of this study, the water level will be varied only for the extratropical and tropical cyclone cases. Input files
can be set up to run a series of test cases, allowing for different combinations of wind speed, direction, and water level for a single model run. Directional spectra were saved at coordinates corresponding to the Gloucester Point and Goodwin Islands field stations.

6.5.1 Summer Sea Breeze

The wind climatology of the lower York River estuary was discussed in detail in Section 4.3.2. It was noted that the summer months are characterized by a prevailing southwesterly wind that is often interrupted by a southeasterly sea breeze in the late afternoon and early evening hours. A sea breeze is a meteorological phenomenon that results from the differential heating of land and water over the course of a day, especially during the summer months. Figure 6-3 shows a sample six-day period in early September 2006 that consistently displays the southeasterly sea breeze at approximately 5 m/s from 110-120°. To simulate these conditions, a combination of wind speeds and directions were selected to model using STWAVE, and are shown in Table 6-1.

6.5.2 Winter Wind Event

Winter wind events in the Chesapeake Bay tributaries are often characterized by moderate northwesterly winds that occur at synoptic time scales as frontal systems pass through the region. Given the limited fetch for the GP and GI for a NW wind, these wind events typically produce relatively small amplitude, short period waves in the lower York River. However, this is not the case for the upper reach of the York, which is aligned in a NW-SE orientation. Thus, the upper reach of the York is particularly susceptible to wind
forcing from the NW. Since the VIMS Observing System does not, at present, have wave information for this section of the river, numerical modeling, after calibration, may provide a useful tool for predicting wave height and period during these winter events.

An example storm from the record will be used as motivation for the model wind forcing. For the winter storm event, the time period from October 24-26, 2006 was selected. During this time, wind speed ranged from approximately 6-10 m/s and direction was from 280°-315°. The wind speed and direction at the Goodwin Islands meteorological station are shown in Figure 6-4. Table 6-2 shows the selected model forcing for the winter wind events for a moderate (8 m/s) and strong (10 m/s) wind. For completeness, these wind speeds were paired with the following wind directions: (1) NW, (2) WNW, and (3) NNW.

6.5.3 Extratropical Storm

Extratropical storms are mid-latitude cyclones characterized by a central low-pressure system. The storms that influence coastal regions are commonly referred to as “nor’easters” due to the strong northeasterly winds that typically precede the passage of the storm. Extratropical cyclones can occur any time of the year but are usually most intense during the fall and winter months. One such storm influenced the lower Chesapeake Bay region from October 6-9, 2006, producing strong winds and substantial coastal flooding. At the peak of the storm, the average wind speed reached approximately 18 m/s from the NE. The wind speed and direction measured at the Goodwin Islands meteorological station are shown in Figure 6-5.
To examine the wind wave generation processes during extratropical events, this example storm was simulated as an STWAVE case study. The forcing conditions (wind speed, direction, and water level) are shown in Table 6-3. To simulate the effect of storm surge, water level was adjusted to three levels: (1) mean sea level (0.0 m), (2) + 0.5 m above mean sea level, and (3) + 1.0 m above mean sea level. These values were chosen based on observed water level deviations from pressure sensors at Gloucester Point and Goodwin Islands during the October 6-9 event. While several combinations of wind speed, direction, and surge are simulated for completeness, Table 6-3 highlights three particular conditions that follow the observed forcing during this storm. These times are highlighted (in grey) and will be compared directly to observations at Gloucester Point and Goodwin Islands at times that correspond to these forcing conditions.

6.5.4 Tropical Depression Ernesto

Detailed accounts of the wind and wave field response to Tropical Depression Ernesto were discussed in Sections 3.5.2 and 4.5. For the STWAVE simulation, wind data collected at the VIMS Goodwin Islands meteorological station are used for model forcing. The data were averaged in three-hour increments for a 40-hour period from August 31, 2006 to September 2, 2006. The three-hour period was selected to represent the time scale of wave field response to changes in wind forcing and to better satisfy the steady-state assumptions inherent in the model. The wind speed and direction measured at the VIMS Goodwin Islands meteorological station are shown in Figure 6-6, which highlights the simulation period from August 31, 2006 0200 EST to September 1, 2006
1400 EST. This represents a period of steadily increasing wind speed and relatively constant direction.

Table 6-4 shows the values used in the STWAVE model forcing. In addition to wind speed and direction from the Goodwin Islands meteorological station, water elevation data from the NOAA water level station at the U.S. Coast Guard pier (Yorktown, VA) are also included to simulate the effects of storm surge on wind wave generation in the lower York River estuary. At each time step, water elevation (referenced to mean sea level) was averaged over the same three-hour period as for the wind data. Water level is assumed to fluctuate simultaneously over the entire model domain, an assumption that is clearly not true, but required within the constraints of the model. An additional simulation case is implemented for the Ernesto event to examine deep-water wind-wave generation within the lower York River estuary. This objective is accomplished by specifying a uniform water depth of 25 m throughout the model domain. The goal of the deep-water simulation is to assess the importance of bathymetry in controlling wind wave growth in the lower York River estuary.

To assess the sensitivity of the STWAVE model results on the wind forcing, an additional model run was executed using wind observations from the NDBC YKRV2 meteorological station at the York River estuary mouth (see Figure 4-1 for station locations). Figure 4-8 shows a comparison of observed wind speed and direction at the two stations. The YKRV2 data gap on August 31, 2006 was filled in based on an observed mean ratio of wind speed at YKRV2 to Goodwin Islands of 1.23. Additionally, the observed wind speed at YKRV2 was corrected to an equivalent $U_{10}$ wind speed using a simple power law relationship (USACE, 2002),
\[ U_{10} = U_z \left( \frac{10}{z} \right)^{0.17} \]  
(Eq. 6-2)

where \( U_{10} \) = wind speed at 10 m (m/s), \( U_z \) = reference wind speed (m/s), \( z \) = reference elevation (m). Assuming a measurement of 5 m above the sea surface, the corrected \( U_{10} \) wind speed is 1.1 times the observed wind speed. The resulting STWAVE model forcing is shown in Figure 6-7 and Table 6-5.

6.6 Results

In the following sections, results are presented for each of the simulated cases described above. For each event, a summary table is presented containing the simulated significant wave height \( (H_{m0}) \), peak period \( (T_p) \), mean direction \( (D_m) \), and peak direction \( (D_p) \). These simulated values are discussed with respect to typically observed values of the reported parameters. For the higher energy events (winter wind event, extratropical storm, and Tropical Depression Ernesto) simulated spectra are compared to observed spectra at the Goodwin Islands station.

6.6.1 Summer Sea Breeze

Table 6-6 shows the simulated values for significant wave height \( (H_{m0}) \), peak period \( (T_p) \), and mean and peak direction \( (D_m, D_p) \) at Goodwin Islands (GI) and Gloucester Point (GP) for the summer sea breeze events. At GI, simulated wave heights ranged from 0.15-0.23 m and peak period ranged from 2.8-3.8 seconds. Wave heights and periods increased with wind speed and were generally larger for the 110° wind forcing compared to 120°. Mean wave directions were ESE (90-98°) while peak
directions were generally more southerly (75°-115°). However, in some cases peak direction was from the ENE. One possible explanation for this effect could be the refraction of waves around the shoals at Goodwin Islands to a more easterly direction (meteorological sense), despite the SE wind forcing. Wave heights and period were generally larger at GP than at GI due to the sheltering effect to the southeast of the GI site (i.e. Goodwin Islands). At GP, wave height ranged from 0.09-0.29 m and peak period ranged from 1.9-4.2 s. Mean direction was more southerly than at GI, displaying values between 106°-119°, while peak directions were nearly aligned with the wind direction (i.e. 110° and 120°). This suggests that for the SE sea breeze event, bathymetry plays a very minor role in controlling the wave characteristics observed at the GP station.

The observed wave conditions during an example sea breeze event (September 7-9, 2006) displayed similar trends to the model with respect to wave height. At GI, the AWAC recorded significant wave heights ranging from 0.15-0.24 m and peak periods from 2.6-3.3 s during the afternoon and early evening hours. At GP, significant wave heights ranged from 0.15-0.23 m and peak periods ranged from 2.3-2.9 s. It was not possible to accurately resolve wave direction at either site given the limitations of the acoustic Doppler instruments for high frequency waves, but it is expected that wave direction generally followed wind direction.

While the simulated wave conditions seem to reflect the correct total energy in the spectrum quite well, the partitioning of that energy with respect to frequency does not agree with the observed spectra. This is especially true at GP, where the simulated wave periods (on the order of 3.5 s) were greater than at GI, while observations showed the reverse to be true. This is most likely due to the steady-state, fetch-limited assumption.
inherent in the model. At GP, the substantially longer fetch (and hence longer propagation time) would allow for enhanced non-linear energy transfers to lower frequencies in the simulated spectra, thus favoring longer period waves at GP than at GI for an easterly wind. In reality, the combination of long easterly fetch, light winds, and temporally varying wind speed and direction probably do not satisfy the steady-state assumption at Gloucester Point for the summer sea breeze event. Surprisingly, the significant wave height estimates are quite close to observations at Gloucester Point.

The simulated non-directional spectra are shown in Figure 6-8 for GI and GP. Spectra are grouped by site (GI or GP) and wind direction (110° or 120°). All spectra display the trend of increasing energy level and decreasing peak frequency with each incremental increase in wind speed. In general, the GP spectra appear to be considerably more peaked and display an uncharacteristically sharp rear face towards the high frequency end of the spectrum, the origin of which is unclear at this point. It is also interesting to note a secondary, higher frequency (~0.4 Hz), peak in the GP 003 and GP 006 spectra. Some insight into this phenomenon can be obtained by examining the directional spectrum, which indicates that for both cases, the high frequency peak is a secondary wave train aligned with the wind direction. This train represents locally generated wind waves generated along a straight line fetch within the lower section of York, while the low frequency peak represents a wave train that is primarily controlled by non-linear energy transfers along the extensive eastward fetch that extends out into the main stem of the Bay. Interestingly, in terms of total contributions to the sea surface variance, the energy is roughly equal, and the partitioned significant wave heights obtained by integrating over the low and high frequency ranges were 0.20 m and 0.17 m,
respectively. This is an interesting result given that the magnitude of the low frequency peak is greater than that of the high frequency peak by nearly a factor of nine; however, when integrating over the broad secondary peak the sums are roughly equal.

6.6.2 Winter Wind Event

The simulated wave heights, periods, and directions for Goodwin Islands (GI) and Gloucester Point (GP) are shown in Table 6-7. At GI, heights and periods are generally moderate due to the restricted fetch for NW winds (3-5 km). Wave heights ranged from 0.17-0.39 m and are largest and smallest for the WNW and NNW directions, respectively. These directions represent the longest and shortest fetches for the winter wind simulations. Peak periods range from 1.7-3.1 s and display similar trends to wave height. Mean directions are generally about 10° west of the wind direction, thus waves are preferentially aligned along the more extensive westerly fetch, despite the NW wind forcing. Waves tend to follow the wind direction closely at the spectral peak, at least within the directional resolution of the model output (± 5°), but shift to a more westerly direction at higher frequencies.

At GP, wave heights tend to display less sensitivity on wind speed given the extremely restricted fetch lengths, and vary from 0.08-0.25 m. Interestingly, peak periods display more variation than at GI, and range from 1.7-3.6 s. Given the short fetch for NW winds at GP, it appears that wave generation processes in the upper reaches of the estuary control the simulated conditions, especially for the NNW wind direction (Cases 005 and 006), for which the shortest straight line fetch resulted in the longest periods at the GP station.

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This apparent inconsistency can be better understood by examining the non-directional spectra, shown in Figure 6-9. In general, the spectral shapes appear to be better behaved than for the summer sea breeze spectra shown in Figure 6-8. The spectra show the much more typical characteristics of a steep front face with a gradual decay of energy in the saturation range towards higher frequencies. This is especially true for the GI 003 and GI 004 spectra, while the other lower energy cases display a more symmetric shape on the front and rear faces. Qualitatively, this phenomenon matches the observed spectra during moderate energy events at both sites. Out of all cases, the WNW wind (Cases 003 and 004) resulted in the most peaked, highest energy spectra. However, at GP it was the NNW wind that produced the lowest frequency peak, displaying peak periods in excess of 3 seconds for a straight line fetch less than 1 km. It is believed that this low frequency energy represents waves generated in the upper reach of the estuary that propagated towards the SE and through the Gloucester Point constriction. The waves then arrive at the buoy site through a combination of refraction and diffraction processes after passing through the point.

In general, there is very good agreement between the STWAVE results and the comparison storm on October 24-26, 2006. At GI, observed significant wave height ranged from 0.25-0.45 m and peak period ranged from 2.25-3.25 s. Peak directions were between 270° and 300°. At GP, significant wave height ranged from 0.03-0.20 m with a peak period around 3 s, confirming the hypothesis that wave generation in the upper reach of the estuary can affect wave characteristics, at least locally, beyond the change in river orientation at Gloucester Point.
Figure 6-10 shows comparisons of modeled and measured non-directional spectra at the Goodwin Islands site. The measured spectra shown in each panel represent wave bursts taken at times matching each of the prescribed wind conditions shown in Table 6-2, but may not necessarily correspond to the sample storm discussed in the model implementation section. The energy spectra are plotted on a logarithmic axis to accommodate the large range of spectral density values observed. In general, there is good agreement in the so-called saturation range of the spectrum (greater than ~0.6 Hz), although the location and magnitude of the observed spectral peaks vary widely relative to the predicted spectra. It is interesting to note the substantial variability among the observed spectra for each case despite the nearly identical wind forcing.

### 6.6.3 Extratropical Storm

The STWAVE model output for the extratropical storm is shown in Table 6-8. At GI, the simulated wave heights ranged from 0.47-0.55 m for the 40° winds (13 m/s) and from 0.51-0.73 m for the 60° winds (15 and 17 m/s). For all wind conditions, the increase in water level due to the simulated storm surge (+ 0.5 m, + 1.0 m) acted to increase the wave height at the GI site. This is likely due to the effect of wave breaking in shoal regions near the York River mouth, namely the York spit (8 km to the east of GI site) and Guinea Marsh (6 km to the northeast of GI site), where water depths range from 0.5-2.0 m. The largest increase was for the 17 m/s @ 60° case (IDs 007-009), where wave height increased by 0.18 m for the + 1.0 m surge relative to the mean sea level case (+ 0.0 m). Peak period ranged from 2.5-2.9 s for the 40° winds and from 5.0-5.6 s for the 60° winds. The dramatic increase in peak period between these two cases is due to the substantially
longer fetch for the 60° wind direction (40 km vs. 5 km), despite the relatively mild increase in wind speed. Surprisingly, the wave height does not show a concomitant increase – most likely because of active wave breaking in the aforementioned shoal regions.

Figure 6-11 shows a comparison of the observed and simulated conditions. The observed wave conditions at GI displayed similar trends to the simulated conditions in that wave height and period increased with wind speed (13.0 to 17.0 m/s) and fetch (5 km to 40 km). Significant wave height ranged from 0.9 m during the early phases of the storm, and reached a maximum of 1.35 m at its peak. Peak period fluctuated between 3.75-5.0 s and subsided with wind speed following the storm. Wave direction agreed relatively well during the later portion of the storm (corresponding to Cases 005-009), and displayed peak and mean values ranging from 64°-84°. In general, measured directions appear to favor a more easterly orientation.

Despite similarities in the trends observed at GI, the model substantially under-predicts the observed wave height, under-estimating the peak significant wave height by approximately 0.5 m (roughly 45%). This is most likely due to the influence of wave breaking over the shoals to the east and northeast of the GI measurement site. It follows that deeper water over shoals during periods of surge will allow a greater proportion of deep-water wave energy to propagate across the York Spit and Guinea Marsh shoals to the Goodwin Islands site. For comparison, the simulated surge for Case 009 (+ 1.0 m) allows considerably more energy to propagate across the spit, and wave height is reduced by only approximately 15% for this case.
Figure 6-12 displays the simulated non-directional wave spectra for the extratropical event at Goodwin Islands and Gloucester Point. Each panel displays three spectra, one for each water level (+0.0 m, +0.5 m, +1.0 m), for each wind forcing at each site. Comparisons with the observed spectra confirm that the model tends to predict lower peak frequencies than were actually attained during the storm. The effect of wave breaking in reducing the observed wave height at GI is evident in each panel, as the spectral energy density at all frequencies is increased with surge levels. Similar to the GP response to the sea breeze, the spectra display an uncharacteristic sharp rear face towards the high frequency end of the spectrum. Qualitatively, this form does not agree visually with the expected spectral shape for these forcing conditions. In Figure 6-13, the simulated and observed spectra for Cases 001, 005, and 009 are shown. Panels (b) and (c) clearly show the sharp decrease in energy for frequencies above the peak frequency, which accounts for a large portion of the under-prediction in wave height. The under-prediction of energy near the spectral peak in panel (a) will be discussed later.

6.6.4 Tropical Depression Ernesto

In this section, the STWAVE model results for Tropical Depression Ernesto are discussed. The wave field response and comparison with observational data at Goodwin Islands will be presented first, followed by similar comparisons at Gloucester Point. A discussion of the results for applying the model to extreme events in estuarine environments will be presented.

Goodwin Islands
Figure 4-9 provides an idea of the expected values for wave height and period at the Goodwin Islands station. The detailed wave summaries for the no surge, surge, surge with $U_{10}$ wind, and deep-water cases are given in Tables 6-9, 6-10, 6-11, and 6-12, respectively. A first examination of the predicted wave heights for the no surge case immediately reveals substantially lower values of significant wave height than observed during the storm. The maximum $H_{m_0}$ wave height from the model is approximately 1.0 m below the maximum observed $H_{m_0}$ wave height (1.67 m vs. 0.65 m), an error of approximately 60%. Clearly, wave breaking processes in the vicinity of the York River spit are limiting the maximum possible wave heights at Goodwin Islands. This phenomenon was discussed previously with reference to the extratropical storm (Section 6.6.3).

When including the observed surge conditions from the Yorktown dataset, the predictions do not improve substantially. From Table 6-10, we see that the maximum predicted $H_{m_0}$ with surge included is only 0.9 m, approximately 0.7 m below the observed value of 1.67 m at the peak of the storm (45% error). Similarly, the modeled wave heights from the NDBC YKRV2 $U_{10}$ wind forcing fall below the observed wave heights for the duration of the simulation. The maximum predicted $H_{m_0}$ is 1.05 m, approximately 0.6 m below the observed value of 1.67 m at the peak of the storm (37% error). The wave height field in the vicinity of the York River mouth is shown in Figure 6-14 for the VIMS Goodwin Islands wind forcing case. The north-south breaker line in the region of York Spit is clearly visible and represents a significant reduction in wave height as waves propagate into the lower York from the main stem of the Bay. This suggests that a substantially larger surge would be required, or that a large surge be
coupled with a high tide, to allow waves to propagate over the shoals without considerable energy loss due to breaking.

However, it is not only the wave height at the peak of the storm that is under-predicted. Figure 6-15 shows a comparison of the temporal evolution of wave height ($H_m$) and peak period ($T_p$) from the model and the observations. While the $H_m$ wave height is consistently higher for the surge vs. no surge cases, it still falls substantially below the observed wave height ("AWAC" in Figure 6-15). In comparison, the curve representing the uniform deep-water case substantially over-predicts the observed wave height. This confirms the hypothesis that depth-induced breaking over the York mouth spit acts to control the simulated wave heights at the GI station. Even with the stronger wind forcing for the $U_{10}$ surge case, wave heights are still under-predicted. However, there is clearly some energy dissipation that must occur in these shoal regions to reproduce the observed evolution of wave height. In this sense, the deep-water case represents an upper limit for significant wave height when ignoring finite-depth effects.

Peak periods from STWAVE showed little variation between the surge and no surge simulations, and model predictions were consistently higher than observed values. Interestingly, the deep-water peak periods are slightly less than the surge peak periods, a phenomenon that may be related to the implementation of the energy reduction due to wave breaking. The wave period field for the peak of the storm is also shown in Figure 6-14. It is interesting to note that despite the decrease in wave height across the breaker line, peak period continues to increase towards Gloucester Point due to non-linear energy transfers that act to decrease the peak frequency with distance along the fetch. Peak periods were generally over-predicted by approximately 0.3-0.7 s during the early phases.
of the storm and by as much as 1.4 s at the peak of the storm for the surge case with Goodwin Islands wind forcing. For the YKRV2 $U_{10}$ wind forcing, peak period is over-predicted by approximately 1.0 s during the early part of the storm and by 2.0 s at the peak of the storm. This over-prediction is somewhat expected given the steady-state assumption inherent in the model, despite the three-hour averaging implemented for the wind forcing. In other words, the combination of changing wind speed and direction prevented the development of a fetch-limited sea for the observational data, and observed peak periods were lower than the model predictions.

Figure 6-16 shows comparisons of observed and predicted peak and mean directions ($D_p$, $D_m$) at the Goodwin Islands station. The wave directions for the no surge case are not shown for clarity because they are nearly identical to the surge cases. Average wind direction is also shown for reference. As would be expected, peak wave direction closely follows the wind direction for the duration of the storm for the deep-water case, where finite depth effects such as refraction and shoaling play a negligible role. For the surge cases, predicted peak directions were consistently more northerly than both the wind direction and the observed peak direction from the AWAC; however, the data show similar trends, including the clock-wise shift in wind and wave direction following the passage of the storm. For mean direction, the observations, wind, and wave directions are remarkably similar, and all generally agree within ± 10°. This result provides validation not only for the STWAVE model, but also for the directional measurement of the AWAC for significant events in estuarine environments.

Comparisons of observed and modeled non-directional spectra are shown in Figures 6-17 and 6-18 for the Goodwin Islands site. The plots show the measured spectra
from the AWAC, the modeled spectra with surge for the VIMS Goodwin Islands and NDBC YKRV2 $U_{10}$ wind, and the STWAVE deep-water case, for reference. The deep-water case typically over-predicts the energy at the spectral peak, while the peak energy for the surge spectra agree for some times but generally under-predict. For frequencies above the peak, the deep-water spectra display substantially better agreement out to approximately 0.6 Hz, where all four spectra typically converge over the saturation range. Thus, it the region between the peak frequency and ~0.6 Hz that the surge spectra display the greatest discrepancy with observations.

*Gloucester Point*

In many respects, the results at Gloucester Point are similar to those at Goodwin Islands. The detailed wave summaries for the no surge, surge, surge with YKRV2 $U_{10}$ wind, and deep-water cases are given in Tables 6-9, 6-10, 6-11, and 6-12 respectively. At Gloucester Point, wave height predictions tend to agree better with the observational data than at the Goodwin Islands site, although there is still substantial under-prediction of peak significant wave heights even when including the surge effects (Figure 6-19). Wave height is under-estimated at the peak of the storm by approximately 40%. There are times during the storm when predictions agree quite well with observations (within 10-15%). For the YKRV2 $U_{10}$ wind forcing, there appears to be less of an impact on the modeled wave heights at GP than at GI. The modeled significant wave height at the peak of the storm for the YKRV2 $U_{10}$ case is nearly equal to the case with Goodwin Islands wind forcing. The deep-water simulation provides an upper bound for the significant wave height estimates in the absence of finite-depth effects and substantially
over-predicts the observed wave heights during the storm. Peak period is generally over-estimated relative to the observations by about 1.0 s, but predictions follow similar trends as observations. Peak period for the YKRV2 $U_{10}$ wind case is over-predicted by approximately 2.0 s. Thus, it appears that the stronger wind speeds associated with the YKRV2 $U_{10}$ case have a greater effect on the modeled periods than on the modeled wave heights at Gloucester Point.

Figure 6-20 shows the predicted values of peak and mean direction at Gloucester Point and compares predicted peak direction with the observed values. For the surge cases, peak direction tends to follow the wind direction quite closely, suggesting minimal influence of finite-depth effects at GP for this wind forcing. The peak directions associated with the deep water case are nearly due east and agree neither with the wind forcing nor the observations. The observed peak direction from the ADCP is approximately 35° more southerly than predicted by the model, and favors a more ESE direction (~110°) compared to the predominately ENE wind direction (~70°).

6.7 Discussion

When considering all forcing conditions, several trends begin to emerge. In general, wave heights were under-predicted for strong easterly winds associated with the extratropical storm and Tropical Depression Ernesto. It is hypothesized that this under-prediction is related to the wave breaking parameterization in the model. The wave-breaking criterion in STWAVE is applied as a maximum limit on the significant wave height and scales with the relative water depth,
\[
\left( H_{\text{w}} \right)_{\text{max}} = 0.1L \tanh(kh) \quad \text{(Eq. 6-3)}
\]

where \( L \) = wavelength, \( k \) = wave number, and \( h \) = water depth. Thus, the energy in the spectrum is reduced at each frequency and direction in an amount proportional to the pre-breaking energy at that frequency and direction (Smith et al., 2001). As an example, for a peak period of 5.0 s in a water depth of 1.5 m, the limiting significant wave height is 0.88 m. This closely resembles the reduction in wave height observed across the breaker line over York Spit for extratropical Case 007, where wave height is reduced by approximately 45%, from 1.3 to 0.9 m. The result is a significant decrease in modeled wave heights downwind of shoal regions. Lin and Demirbilek (2005) examined the performance of STWAVE at coastal inlets and also found that the model showed excessive dissipation due to wave breaking. The authors speculated that the excessive reduction of energy was related to the wave-breaking criteria implemented in the model, although they did not investigate this phenomenon further.

A second factor that may contribute to under-predictions with regard to wave height relates to the wind direction relative to the grid orientation. As previously stated, STWAVE 4.0 is a half-plane model, which means it can only propagate energy in the positive \( x \)-direction. For an easterly wind, this means that the positive \( x \)-direction is oriented due west. Thus, the directional spectrum is calculated for waves propagating only from easterly quadrants (i.e. \( \pm 85^\circ \) of \( x \)-axis). As the wind direction approaches \( \pm 60^\circ \) relative to the grid orientation, significant decreases in total spectral energy may be observed (Smith et al., 2001). For the cases examined in this study, the wind forcing was generally kept within these constraints except for the winter wind NNW case and the extratropical 40° wind case. For both of these cases, peak frequency and spectral energy...
were substantially under-predicted (see Figures 6-10 e,f and 6-12a), confirming this limitation of the half-plane version of the model. Future investigators should consider application of the full-plane version of STWAVE (Smith and Zundel, 2006) to the Chesapeake Bay tributaries, especially to simulate longer time series where wind direction varies over all compass directions.

Fetch geometry is also believed to play a role in controlling the height of waves arriving at the Goodwin Island station from the main stem of the Bay. By examining model output of mean direction fields, it was noted that for an easterly wind, waves near the north shore of the river displayed southerly directions, while waves near the south shore of the river displayed northerly directions (meteorological convention). Presumably, this phenomenon can be explained by the refraction of waves due to finite-depth effects as they propagate through the York River mouth. An additional dissipation mechanism could be the pseudo-parameterization of diffraction processes, which acts to smooth wave energy laterally as waves propagate from the main stem of the Bay into the lower York. The result is a divergence of wave energy that may act to reduce wave heights within the lower reach of the river. This effect can be seen visually in the wave height field in Figure 6-14, which shows a progressive decay of wave energy moving from east to west, as well as north and south away from the centerline of the river.

Fetch geometry is also known to exert controls wave direction in environments displaying large gradients in fetch such that wave direction can be biased towards the longer fetches (Bishop, 1983). This phenomenon is observed in the model and empirical data at both Gloucester Point and Goodwin Islands. For the sea breeze case at Gloucester Point, it was noted that in addition to the high frequency wave train propagating along the
wind direction, the dominant low frequency wave train displayed a direction aligned more with the dominant fetch direction - southeast. Similarly, observed wave directions at GP during Tropical Depression Ernesto were southeast, despite the ENE wind forcing. At Goodwin Islands, simulated mean wave directions for the winter wind event tended to align with the dominant east-west orientation of the river, while peak directions seemed to follow the northwesterly wind forcing. Similarly, measured peak directions at GI during Ernesto favored a more easterly direction relative to the ENE wind forcing.

Finally, it is also noted that the model does not capture the short-term variations in the wave field displayed in the observational data during the Ernesto simulation. This is most likely attributable to three reasons: (1) averaging of wind data, (2) spatially varying wind field, and (3) statistical variability. Since wave observations are made once per hour, yet wind is averaged over three hour windows, it is expected that small-scale features in the temporal evolution of the wave field may not be captured by the model. Additionally, the effect of assuming uniform wind forcing as opposed to a spatially varying wind field reduces the ability of the model to capture localized variations in wind forcing or features such as topographic control of wind speed and direction. Finally, it is important to realize that there is some degree of uncertainty in the measured data simply due to the inherent statistical variability in the wave field. This certainly can account for some of the apparent burst-to-burst variability, especially in parameters such as peak period that can only be reported at discrete frequency intervals.
6.8 Conclusions

Application of the STWAVE numerical wave model to the York River estuary was determined to have mixed results. For moderate winds over relatively short fetches, the modeled wave parameters and spectra agreed quite well with observations. However, for strong winds over long fetches, wave heights were under-estimated. For nearly all cases, peak periods were over-estimated due to the steady-state assumption inherent in the model. The deviations from observations were significant in some cases. At Goodwin Islands, the peak significant wave height was under-estimated by approximately 45% during Tropical Depression Ernesto and peak period was over-estimated by approximately 1 second for the duration of the storm. It is believed that the wave height under-prediction is related to the wave-breaking formulation in the model, which results in excessive energy dissipation across shoal regions. In the lower York, extensive shoals extend across the mouth of the river and greatly complicate wave prediction for strong easterly winds, and make it difficult to accurately model waves in the vicinity of the river mouth.

Non-directional spectra generally agreed well with observations for the high frequency end of the spectrum (> 0.6 Hz), but deviated in the vicinity of the spectral peak. In some cases, energy levels were substantially below observations for \( f_p < f < 3f_p \), where \( f_p \) = peak frequency. This was especially true for strong easterly winds involving wave breaking over the York mouth shoals. Additionally, non-linear processes, which act to decrease the peak frequency with distance along the fetch, appear to over-estimate the energy transfers for particularly long fetches (>15 km). The result is uncharacteristically sharp spectral peaks at the low frequency end of the spectrum when
wind forcing is aligned with the dominant fetch geometry. The model appeared to give reasonable results for diffraction processes in the vicinity of Gloucester Point, despite the relatively simple smoothing algorithm employed in place of rather complicated physics.

From the results of this study, it is recommended that the half-plane version of the steady-state spectral wave model, STWAVE, be applied with caution in semi-enclosed embayments where wind input is the dominant source of energy. Since the model assumes steady-state, fetch-limited wave conditions, it does not appear to be particularly well suited to sites with large gradients in fetch or long exposed fetches for a narrow range of directions. This is especially true for the Chesapeake Bay tributary mouths, where the ratio of fetch length to fetch width is quite large and long fetches extend eastward across the main stem of the Bay. It is hypothesized that the model would perform significantly better in more sheltered portions of the Bay's tributaries, where the fetch-limited steady-state assumptions would be satisfied.
Table 6-1. STWAVE model forcing for summer sea breeze events.
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Table 6-2. STWAVE model forcing for winter wind events.
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<tr>
<td>005</td>
<td>15.0</td>
<td>60°</td>
<td>0.5</td>
</tr>
<tr>
<td>006</td>
<td>15.0</td>
<td>60°</td>
<td>1.0</td>
</tr>
<tr>
<td>007</td>
<td>17.0</td>
<td>60°</td>
<td>0.0</td>
</tr>
<tr>
<td>008</td>
<td>17.0</td>
<td>60°</td>
<td>0.5</td>
</tr>
<tr>
<td>009</td>
<td>17.0</td>
<td>60°</td>
<td>1.0</td>
</tr>
</tbody>
</table>

*Note: Shaded IDs most closely reflect forcing conditions (wind speed, direction, surge) observed during the comparison storm on Oct. 6-9, 2006.*

Table 6-3. STWAVE model forcing for extratropical storm event – Oct. 6-9, 2006.
<table>
<thead>
<tr>
<th>ID</th>
<th>Date-Time</th>
<th>Wind Speed (m/s)</th>
<th>Wind Direction (deg)</th>
<th>Water Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>08/31/06 02:00</td>
<td>6.7</td>
<td>63</td>
<td>0.60</td>
</tr>
<tr>
<td>002</td>
<td>08/31/06 05:00</td>
<td>7.7</td>
<td>60</td>
<td>0.50</td>
</tr>
<tr>
<td>003</td>
<td>08/31/06 08:00</td>
<td>9.0</td>
<td>55</td>
<td>0.25</td>
</tr>
<tr>
<td>004</td>
<td>08/31/06 11:00</td>
<td>10.3</td>
<td>60</td>
<td>0.35</td>
</tr>
<tr>
<td>005</td>
<td>08/31/06 14:00</td>
<td>10.8</td>
<td>68</td>
<td>0.75</td>
</tr>
<tr>
<td>006</td>
<td>08/31/06 17:00</td>
<td>11.9</td>
<td>69</td>
<td>0.80</td>
</tr>
<tr>
<td>007</td>
<td>08/31/06 20:00</td>
<td>11.7</td>
<td>70</td>
<td>0.55</td>
</tr>
<tr>
<td>008</td>
<td>08/31/06 23:00</td>
<td>12.8</td>
<td>68</td>
<td>0.55</td>
</tr>
<tr>
<td>009</td>
<td>09/01/06 02:00</td>
<td>14.6</td>
<td>67</td>
<td>0.80</td>
</tr>
<tr>
<td>010</td>
<td>09/01/06 05:00</td>
<td>16.0</td>
<td>61</td>
<td>1.00</td>
</tr>
<tr>
<td>011</td>
<td>09/01/06 08:00</td>
<td>19.5</td>
<td>61</td>
<td>1.10</td>
</tr>
<tr>
<td>012</td>
<td>09/01/06 11:00</td>
<td>20.5</td>
<td>71</td>
<td>1.25</td>
</tr>
<tr>
<td>013</td>
<td>09/01/06 14:00</td>
<td>15.2</td>
<td>88</td>
<td>1.40</td>
</tr>
</tbody>
</table>

*Note: Wind speed and direction was measured at the Goodwin Islands meteorological station. Water level was measured at Yorktown, VA and is reference to mean sea level (MSL).

Table 6-4. STWAVE model forcing for Tropical Depression Ernesto from VIMS Goodwin Islands meteorological station, August 31 - September 1, 2006.
<table>
<thead>
<tr>
<th>ID</th>
<th>Date-Time</th>
<th>Wind Speed (m/s)</th>
<th>Wind Direction (deg)</th>
<th>Water Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>08/31/06 02:00</td>
<td>8.3</td>
<td>63</td>
<td>0.60</td>
</tr>
<tr>
<td>002</td>
<td>08/31/06 05:00</td>
<td>10.3</td>
<td>59</td>
<td>0.50</td>
</tr>
<tr>
<td>003</td>
<td>08/31/06 08:00</td>
<td>11.1</td>
<td>57</td>
<td>0.25</td>
</tr>
<tr>
<td>004</td>
<td>08/31/06 11:00</td>
<td>13.4</td>
<td>58</td>
<td>0.35</td>
</tr>
<tr>
<td>005</td>
<td>08/31/06 14:00</td>
<td>14.3</td>
<td>66</td>
<td>0.75</td>
</tr>
<tr>
<td>006</td>
<td>08/31/06 17:00</td>
<td>16.0</td>
<td>68</td>
<td>0.80</td>
</tr>
<tr>
<td>007</td>
<td>08/31/06 20:00</td>
<td>15.8</td>
<td>71</td>
<td>0.55</td>
</tr>
<tr>
<td>008</td>
<td>08/31/06 23:00</td>
<td>16.7</td>
<td>69</td>
<td>0.55</td>
</tr>
<tr>
<td>009</td>
<td>09/01/06 02:00</td>
<td>18.5</td>
<td>68</td>
<td>0.80</td>
</tr>
<tr>
<td>010</td>
<td>09/01/06 05:00</td>
<td>20.4</td>
<td>63</td>
<td>1.00</td>
</tr>
<tr>
<td>011</td>
<td>09/01/06 08:00</td>
<td>23.9</td>
<td>63</td>
<td>1.10</td>
</tr>
<tr>
<td>012</td>
<td>09/01/06 11:00</td>
<td>27.3</td>
<td>67</td>
<td>1.25</td>
</tr>
<tr>
<td>013</td>
<td>09/01/06 14:00</td>
<td>24.5</td>
<td>76</td>
<td>1.40</td>
</tr>
</tbody>
</table>

*Note: Wind speed and direction was measured at the NDBC YKRV2 meteorological station. Water level was measured at Yorktown, VA and is referenced to mean sea level (MSL).*

Table 6-5. STWAVE model forcing for Tropical Depression Ernesto based on NDBC YKRV2 meteorological station (corrected for $U_{10}$ wind speed), August 31 - September 1, 2006.
<table>
<thead>
<tr>
<th>Site</th>
<th>Goodwin Islands</th>
<th>Gloucester Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>ID</td>
<td>Hm0 (m)</td>
<td>Tp (s)</td>
</tr>
<tr>
<td>001</td>
<td>0.15</td>
<td>2.8</td>
</tr>
<tr>
<td>002</td>
<td>0.20</td>
<td>3.3</td>
</tr>
<tr>
<td>003</td>
<td>0.23</td>
<td>3.7</td>
</tr>
<tr>
<td>004</td>
<td>0.13</td>
<td>2.8</td>
</tr>
<tr>
<td>005</td>
<td>0.17</td>
<td>3.4</td>
</tr>
<tr>
<td>006</td>
<td>0.20</td>
<td>3.8</td>
</tr>
</tbody>
</table>

*Note: Hm0 = significant wave height, Tp = peak period, Dm = mean direction (from), and Dp = peak direction (from).*

Table 6-6. STWAVE results for summer sea breeze event at Gloucester Point and Goodwin Islands.
<table>
<thead>
<tr>
<th>Site ID</th>
<th>$H_{m0}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_m/D_p$ (deg)</th>
<th>$H_{m0}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_m/D_p$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>0.23</td>
<td>2.8</td>
<td>307/315</td>
<td>0.15</td>
<td>1.9</td>
<td>301/315</td>
</tr>
<tr>
<td>002</td>
<td>0.29</td>
<td>3.1</td>
<td>308/315</td>
<td>0.19</td>
<td>2.0</td>
<td>301/315</td>
</tr>
<tr>
<td>003</td>
<td>0.30</td>
<td>2.6</td>
<td>291/300</td>
<td>0.20</td>
<td>1.7</td>
<td>292/300</td>
</tr>
<tr>
<td>004</td>
<td>0.39</td>
<td>2.9</td>
<td>291/300</td>
<td>0.25</td>
<td>1.9</td>
<td>291/300</td>
</tr>
<tr>
<td>005</td>
<td>0.17</td>
<td>1.7</td>
<td>325/335</td>
<td>0.08</td>
<td>3.1</td>
<td>315/325</td>
</tr>
<tr>
<td>006</td>
<td>0.24</td>
<td>2.0</td>
<td>325/330</td>
<td>0.10</td>
<td>3.6</td>
<td>315/335</td>
</tr>
</tbody>
</table>

*Note: $H_{m0} =$ significant wave height, $T_p =$ peak period, $D_m =$ mean direction (from), and $D_p =$ peak direction (from).*

Table 6-7. STWAVE results for winter wind event at Gloucester Point and Goodwin Islands.
### Table 6-8. STWAVE model results for extratropical event at Gloucester Point and Goodwin Islands.

<table>
<thead>
<tr>
<th>Site</th>
<th>Goodwin Islands</th>
<th>Gloucester Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>ID</td>
<td>$H_m$ (m)</td>
<td>$T_p$ (s)</td>
</tr>
<tr>
<td>001</td>
<td>0.47</td>
<td>2.5</td>
</tr>
<tr>
<td>002</td>
<td>0.52</td>
<td>2.5</td>
</tr>
<tr>
<td>003</td>
<td>0.55</td>
<td>2.9</td>
</tr>
<tr>
<td>004</td>
<td>0.51</td>
<td>5.0</td>
</tr>
<tr>
<td>005</td>
<td>0.60</td>
<td>5.0</td>
</tr>
<tr>
<td>006</td>
<td>0.68</td>
<td>5.0</td>
</tr>
<tr>
<td>007</td>
<td>0.55</td>
<td>5.3</td>
</tr>
<tr>
<td>008</td>
<td>0.64</td>
<td>5.6</td>
</tr>
<tr>
<td>009</td>
<td>0.73</td>
<td>5.6</td>
</tr>
</tbody>
</table>

*Note: $H_m$ = significant wave height, $T_p$ = peak period, $D_m$ = mean direction (from), and $D_p$ = peak direction (from). Shaded IDs most closely reflect forcing conditions (wind speed, direction, surge) observed during the comparison storm on Oct. 6-7, 2006.
<table>
<thead>
<tr>
<th>Site ID</th>
<th>$H_{\text{m0}}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_m/D_p$ (deg)</th>
<th>Site ID</th>
<th>$H_{\text{m0}}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_m/D_p$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>0.31</td>
<td>3.6</td>
<td>67/60</td>
<td>005</td>
<td>0.43</td>
<td>4.3</td>
<td>68/60</td>
</tr>
<tr>
<td>002</td>
<td>0.33</td>
<td>3.8</td>
<td>66/55</td>
<td>006</td>
<td>0.46</td>
<td>4.5</td>
<td>68/60</td>
</tr>
<tr>
<td>003</td>
<td>0.34</td>
<td>4.2</td>
<td>63/55</td>
<td>007</td>
<td>0.45</td>
<td>4.5</td>
<td>68/60</td>
</tr>
<tr>
<td>004</td>
<td>0.39</td>
<td>4.3</td>
<td>65/55</td>
<td>008</td>
<td>0.48</td>
<td>4.8</td>
<td>67/60</td>
</tr>
<tr>
<td>005</td>
<td>0.43</td>
<td>4.3</td>
<td>68/60</td>
<td>009</td>
<td>0.52</td>
<td>5.0</td>
<td>66/60</td>
</tr>
<tr>
<td>006</td>
<td>0.33</td>
<td>3.8</td>
<td>66/55</td>
<td>010</td>
<td>0.52</td>
<td>5.3</td>
<td>63/55</td>
</tr>
<tr>
<td>007</td>
<td>0.45</td>
<td>4.5</td>
<td>68/60</td>
<td>011</td>
<td>0.59</td>
<td>5.6</td>
<td>61/55</td>
</tr>
<tr>
<td>008</td>
<td>0.48</td>
<td>4.8</td>
<td>67/60</td>
<td>012</td>
<td>0.65</td>
<td>5.9</td>
<td>66/60</td>
</tr>
<tr>
<td>009</td>
<td>0.52</td>
<td>5.0</td>
<td>66/60</td>
<td>013</td>
<td>0.48</td>
<td>5.0</td>
<td>78/75</td>
</tr>
</tbody>
</table>

*Note: $H_{\text{m0}}$ = significant wave height, $T_p$ = peak period, $D_m$ = mean direction (from), and $D_p$ = peak direction (from).*

Table 6-9. STWAVE model results for Tropical Depression Ernesto with no surge simulated.
<table>
<thead>
<tr>
<th>Site ID</th>
<th>$H_{m0}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_{m}/D_p$ (deg)</th>
<th>$H_{m0}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_{m}/D_p$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>0.37</td>
<td>3.6</td>
<td>67/60</td>
<td>0.16</td>
<td>3.8</td>
<td>82/65</td>
</tr>
<tr>
<td>002</td>
<td>0.39</td>
<td>3.8</td>
<td>66/55</td>
<td>0.16</td>
<td>4.0</td>
<td>79/65</td>
</tr>
<tr>
<td>003</td>
<td>0.38</td>
<td>4.2</td>
<td>64/55</td>
<td>0.14</td>
<td>4.3</td>
<td>76/55</td>
</tr>
<tr>
<td>004</td>
<td>0.45</td>
<td>4.3</td>
<td>65/55</td>
<td>0.18</td>
<td>4.5</td>
<td>79/60</td>
</tr>
<tr>
<td>005</td>
<td>0.54</td>
<td>4.3</td>
<td>69/60</td>
<td>0.27</td>
<td>4.5</td>
<td>86/75</td>
</tr>
<tr>
<td>006</td>
<td>0.58</td>
<td>4.5</td>
<td>69/60</td>
<td>0.30</td>
<td>4.8</td>
<td>87/75</td>
</tr>
<tr>
<td>007</td>
<td>0.54</td>
<td>4.5</td>
<td>69/60</td>
<td>0.29</td>
<td>4.8</td>
<td>88/75</td>
</tr>
<tr>
<td>008</td>
<td>0.57</td>
<td>4.8</td>
<td>68/60</td>
<td>0.29</td>
<td>5.0</td>
<td>87/75</td>
</tr>
<tr>
<td>009</td>
<td>0.66</td>
<td>5.0</td>
<td>67/60</td>
<td>0.32</td>
<td>5.3</td>
<td>86/75</td>
</tr>
<tr>
<td>010</td>
<td>0.71</td>
<td>5.3</td>
<td>65/55</td>
<td>0.29</td>
<td>5.6</td>
<td>82/65</td>
</tr>
<tr>
<td>011</td>
<td>0.82</td>
<td>5.9</td>
<td>63/55</td>
<td>0.34</td>
<td>5.9</td>
<td>82/80</td>
</tr>
<tr>
<td>012</td>
<td>0.90</td>
<td>5.9</td>
<td>68/60</td>
<td>0.32</td>
<td>6.2</td>
<td>88/80</td>
</tr>
<tr>
<td>013</td>
<td>0.72</td>
<td>5.0</td>
<td>80/75</td>
<td>0.62</td>
<td>5.3</td>
<td>101/100</td>
</tr>
</tbody>
</table>

*Note: $H_{m0}$ = significant wave height, $T_p$ = peak period, $D_{m}$ = mean direction (from), and $D_p$ = peak direction (from).

Table 6-10. STWAVE model results for Tropical Depression Ernesto with surge included (GI wind forcing).
<table>
<thead>
<tr>
<th>Site</th>
<th>Goodwin Islands</th>
<th>Gloucester Point</th>
</tr>
</thead>
<tbody>
<tr>
<td>ID</td>
<td>$H_m$ (m)</td>
<td>$T_p$ (s)</td>
</tr>
<tr>
<td>001</td>
<td>0.44</td>
<td>4.0</td>
</tr>
<tr>
<td>002</td>
<td>0.48</td>
<td>4.3</td>
</tr>
<tr>
<td>003</td>
<td>0.45</td>
<td>4.5</td>
</tr>
<tr>
<td>004</td>
<td>0.52</td>
<td>5.0</td>
</tr>
<tr>
<td>005</td>
<td>0.65</td>
<td>5.0</td>
</tr>
<tr>
<td>006</td>
<td>0.71</td>
<td>5.3</td>
</tr>
<tr>
<td>007</td>
<td>0.66</td>
<td>5.3</td>
</tr>
<tr>
<td>008</td>
<td>0.69</td>
<td>5.3</td>
</tr>
<tr>
<td>009</td>
<td>0.78</td>
<td>5.6</td>
</tr>
<tr>
<td>010</td>
<td>0.85</td>
<td>5.9</td>
</tr>
<tr>
<td>011</td>
<td>0.91</td>
<td>6.2</td>
</tr>
<tr>
<td>012</td>
<td>1.05</td>
<td>6.7</td>
</tr>
<tr>
<td>013</td>
<td>1.05</td>
<td>6.2</td>
</tr>
</tbody>
</table>

*Note: $H_m$ = significant wave height, $T_p$ = peak period, $D_m$ = mean direction (from), and $D_p$ = peak direction (from).

Table 6-11. STWAVE model results for Tropical Depression Ernesto with surge included (YKRV2 and $U_{10}$ wind forcing).
<table>
<thead>
<tr>
<th>Site ID</th>
<th>$H_{m0}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_m/D_p$ (deg)</th>
<th>$H_{m0}$ (m)</th>
<th>$T_p$ (s)</th>
<th>$D_m/D_p$ (deg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>001</td>
<td>0.59</td>
<td>3.6</td>
<td>69/65</td>
<td>0.27</td>
<td>3.4</td>
<td>83/90</td>
</tr>
<tr>
<td>002</td>
<td>0.67</td>
<td>3.7</td>
<td>67/65</td>
<td>0.28</td>
<td>3.6</td>
<td>81/85</td>
</tr>
<tr>
<td>003</td>
<td>0.75</td>
<td>4.0</td>
<td>65/60</td>
<td>0.28</td>
<td>3.8</td>
<td>78/85</td>
</tr>
<tr>
<td>004</td>
<td>0.93</td>
<td>4.2</td>
<td>67/65</td>
<td>0.39</td>
<td>4.2</td>
<td>81/90</td>
</tr>
<tr>
<td>005</td>
<td>1.03</td>
<td>4.3</td>
<td>72/70</td>
<td>0.54</td>
<td>4.2</td>
<td>85/90</td>
</tr>
<tr>
<td>006</td>
<td>1.16</td>
<td>4.5</td>
<td>72/70</td>
<td>0.63</td>
<td>4.3</td>
<td>85/90</td>
</tr>
<tr>
<td>007</td>
<td>1.14</td>
<td>4.3</td>
<td>73/70</td>
<td>0.64</td>
<td>4.3</td>
<td>86/90</td>
</tr>
<tr>
<td>008</td>
<td>1.26</td>
<td>4.5</td>
<td>71/70</td>
<td>0.65</td>
<td>4.5</td>
<td>85/90</td>
</tr>
<tr>
<td>009</td>
<td>1.45</td>
<td>5.0</td>
<td>71/70</td>
<td>0.74</td>
<td>4.8</td>
<td>84/90</td>
</tr>
<tr>
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*Note: $H_{m0}$ = significant wave height, $T_p$ = peak period, $D_m$ = mean direction (from), and $D_p$ = peak direction (from).*

Table 6-12. STWAVE model results for Tropical Depression Ernesto for idealized deep water wind wave generation.
Figure 6-1. Definition of wave and current vectors for STWAVE (Smith et al., 2001).
Figure 6-2. STWAVE model grid domain. Water depths relative to mean sea level are shown in meters. Grid spacing is 100m.
Figure 6-3. One-hour averaged (a) Wind speed and (b) wind direction measured at Goodwin Islands, VA for the summer sea breeze wind forcing (September 2006). The dashed lines in (b) indicate the simulated range of wind directions (110°-120°).
Figure 6-4. Observed (a) wind speed and (b) wind direction (15-min average) at Goodwin Islands for the winter storm event (October 2006).
Figure 6-5. Observed (a) wind speed and (b) wind direction (15-min average) during an extratropical storm at Goodwin Islands, October 6-9, 2006. Maximum gust and 15-minute average wind speed are shown in top panel. Conditions corresponding to the three cases highlighted in Table 6-3 (001, 005, 009) are shown as vertical dashed lines.
Figure 6-6. Observed (a) wind speed and (b) wind direction at the VIMS Goodwin Islands meteorological station during Tropical Depression Ernesto (2006). Data were averaged in 3-hour increments to apply to STWAVE model forcing. The vertical dashed lines in the figure indicate the time period for model simulation.
Figure 6-7. Corrected $U_{10}$ (a) wind speed and (b) wind direction at the NDBC YKRV2 meteorological station during Tropical Depression Ernesto (2006). Data were averaged in 3-hour increments to apply to STWAVE model forcing. The vertical dashed lines in the figure indicate the time period for model simulation.
Figure 6-8. Simulated non-directional spectra for summer sea breeze for wind directions of 110° at (a) Goodwin Islands and (b) Gloucester Point and 120° at (c) Goodwin Islands and (d) Gloucester Point for wind speeds of 4 m/s (○), 6 m/s (●), and 8 m/s (●). ID numbers refer to forcing conditions outline in Table 6-1.
Figure 6-9. Simulated non-directional spectra during winter wind event for wind speeds of 8 m/s at (a) Goodwin Islands and (b) Gloucester Point and 10 m/s at (c) Goodwin Islands and (d) Gloucester Point for wind directions of NW (○-), WNW (*-), and NNW (-). ID numbers refer to forcing conditions outlined in Table 6-2.
Figure 6-10. Comparison of observed (-) and simulated (o-) non-directional spectra for winter wind event at the Goodwin Islands station. Panels (a) through (f) represent wind conditions 001-006 in Table 6-2.
Figure 6-11. Comparison of observed and simulated (a) significant wave height and (b) peak period at Goodwin Islands during the extratropical storm simulation during October 2006.
Figure 6-12. Simulated non-directional spectra for the extratropical storm forcing: (a,b) 13 m/s @ 40°, (c,d) 15 m/s @ 60°, and (e,f) 17 m/s @ 60° for Goodwin Islands (Gl) and Gloucester Point (GP). Water levels are indicated as follows: (-o) + 0.0 m, (-*) + 0.5 m, and (-.) + 1.0 m.
Figure 6-13. Comparison of observed (-) and simulated (o-) spectra for the extratropical storm at the Goodwin Islands station.
Figure 6-14. Fields of (a) significant wave height (in meters) and (b) peak period (in seconds) for the STWAVE Ernesto surge case (GI wind forcing), ID 012 (09/01/06 11:00 EST). Easting and northing coordinates are referenced to the SW corner of the STWAVE grid (see Figure 6-2).
Figure 6-15. Comparison of measured and simulated (a) significant wave height and (b) peak period at the Goodwin Islands station during the passage of Tropical Depression Ernesto in September 2006.
Figure 6-16. Comparison of observed and simulated (a) peak direction ($D_p$) and (b) mean direction ($D_m$) at the Goodwin Islands station during the passage of Tropical Depression Ernesto in September 2006.
Figure 6-17. Comparison of observed (black, -) and modeled (blue, o-) spectra at the Goodwin Islands station during the passage of Tropical Depression Ernesto. Modeled spectra for deep-water (red, .-) and corrected $U_{10}$ wind speed (green, ★) forcing are shown for reference.
Figure 6-18. Comparison of observed (black, -) and modeled (blue, o-) spectra at the Goodwin Islands station during the passage of Tropical Depression Ernesto. Modeled spectra for deep-water (red, .-) and corrected $U_{10}$ wind speed (green, ★) forcing are shown for reference.
Figure 6-19. Comparison of measured and simulated (a) significant wave height and (b) peak period at Gloucester Point station during the passage of Tropical Depression Ernesto.
Figure 6-20. (a) Comparison of measured and simulated peak direction ($D_p$) at Gloucester Point station. (b) Simulated mean direction ($D_m$) at Gloucester Point station for Tropical Depression Ernesto.
7 SUMMARY

This project has attempted to draw together a large dataset of wave measurements from a variety of regions with the objective of providing a better understanding of wave characteristics in estuarine environments. The first section, "Evaluation of wave height parameter estimates in coastal environments," synthesized acoustic Doppler wave gage field data from 10 sites to better understand how characteristics of the wave spectrum influence computed wave parameters. A methodology was presented to account for these influences and make a statistically reasonable estimate of wave parameters, such as the maximum wave height, when a direct measurement is unavailable. These predictions rely both on theoretical results from the literature and empirical relations presented in this study. Engineers and oceanographers working in the coastal region, especially with ocean observing systems, can apply these results to provide a more complete suite of variables to users in real-time applications.

The second and third sections, "Estuarine wave characteristics" and "Evaluation of wave height distributions," focused on defining the unique nature of waves in the estuarine environment. The analysis relied on field measurements in the York River, VA, a tributary of the Chesapeake Bay. The results demonstrate that estuarine wave height distributions appear to follow the more general Weibull model, as opposed to the typically assumed Rayleigh model for coastal environments. While the wave climate was found to exhibit relatively low energy conditions compared to more exposed coastal environments, the potential exists for high energy conditions during more severe events such as tropical and extratropical storms. Coastal managers and engineers should consider these results during development and flood protection projects along the
Chesapeake Bay shoreline, which is especially susceptible to wave effects during raised water levels associated with hurricanes and nor'easters. In the coming decades, the impact of sea level rise will only exacerbate these issues, and an understanding of the Bay's wave climate will serve to inform the design of marine structures and shoreline restoration in the coastal zone.

The fourth section, "Evaluation of a steady-state numerical wave model," assessed the suitability of the STWAVE numerical wave model for application in the Bay's tributaries. Using meteorological data from the VIMS Observing System and knowledge of the local wind climatology, model runs were selected to simulate conditions typical of the region. Model predictions were compared to observed wave parameters and spectra at two locations in the lower York River. Additionally, the predictions offered insight into aspects of the wave field in regions lacking observations. Interpretation of the model results identified the role of shoal regions in the vicinity of the York River mouth in dissipating waves originating in the main stem of the Bay, although this effect appears to be over-emphasized in the model. This finding highlights the influence of the Bay's recent geologic history and associated drowned river valley morphology in controlling the wave climate of the tributaries. Given the mixed model performance compared to the observational data, further research is recommended to identify ways to better implement the model in these types of environments.


wave growth and swell decay during the Joing North Sea Wave Project (JONSWAP). Dtsch. Hydrogr. Z. Ergangzungheft A12, 95.


