US Army Corps
of Engineers

Oxbow-Hickson-Bakke (OHB) Levee Wind-Wave Analysis

Fargo-Moorhead Metro Flood Risk Management Project

Doc Version: FINAL DRAFT

03 Jan 2014

Contents

Tables

Figures

1 BACKGROUND

1.1 Project Map

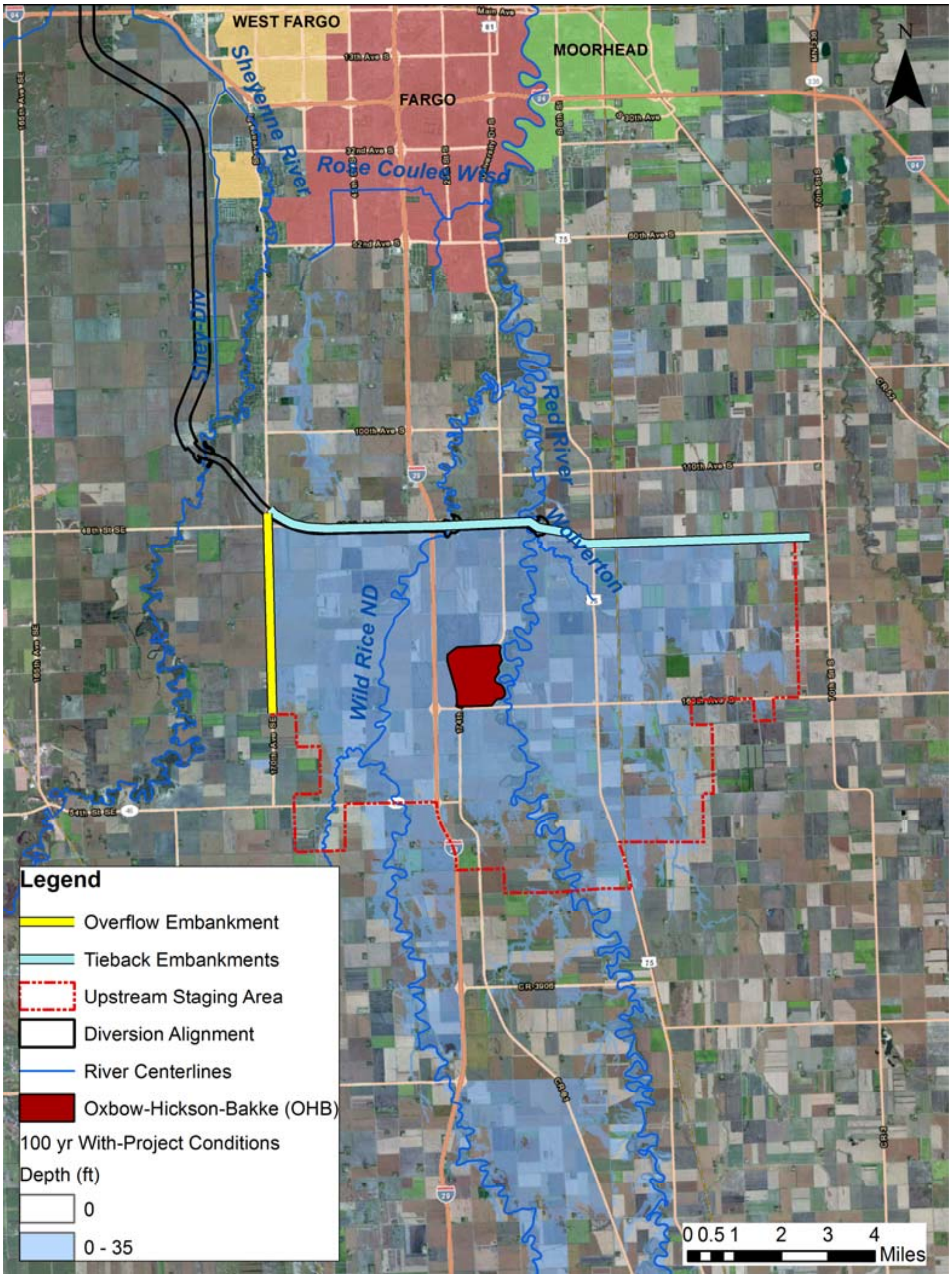

Figure 1 –Map of Fargo-Moorhead Metro Staging Area and Oxbow-Hickson-Bakke (OHB) Ring Levee

1.2 Project Overview

The cities of Oxbow and Hickson and the nearby Bakke Subdivision are located in Eastern North Dakota south of Fargo, ND between the Wild Rice River in North Dakota and the Red River of the North along the border of North Dakota and Minnesota. Due to the flat nature of the Red River Basin, overland flooding as a result of annual spring snowmelt flooding has become a recurring issue for communities along the Red River. The recently authorized Fargo-Moorhead Metro (FMM) Flood Risk Management Project has been developed in recent years to help alleviate Red River flood impacts to the communities of Fargo, ND and Moorhead, MN, among other cities. While the main component of the FMM project is a diversion that routes water to the west around Fargo, an upstream staging area was also included in the project to mitigate for downstream impacts. As part of the operation of this staging area, water surface elevations during flooding will be impacted in upstream areas such as the Oxbow-Hickson-Bakke (OHB) area. To mitigate the increase in water surface elevations and to improve the existing flood fighting measures for the OHB community, a ring-levee is to be constructed before the FMM project is fully operational.

1.3 Analysis Methodology

Simplistic approaches to wave estimation and wind setup estimation have been included in past USACE publications including the Shore Protection Manual (SPM) and the Coastal Engineering Manual (CEM). These approaches tend to be adequate for typical reservoir dams and sometimes even for typical river levees. However, due to the substantial length of levee to be designed (4.6 miles) and the threat of wave growth from any direction around the ring levee, a more sophisticated 2D modeling approach was selected in place of the simplified approach. Along with this analysis, the same approach and model can be used for finalizing the design of the staging area embankment, tieback levees, and road raises as part of the FMM project.

The process to analyze wind-wave growth in a two-dimensional approach involves the coupling of various models and equations. Some definitions of important wind and wave parameters are as follows:

- Annual Chance Exceedance (ACE) or Annual Exceedance Probability (AEP) – the probability of a value (flow, stage, wind speed) exceeding a certain magnitude in any given year.
- Return Period (T_r) or Recurrence Interval (RI) – the inverse of the exceedance probability or the expected value of the frequency of the event (i.e. 1% event would have a 100-yr return period).
- Wind wave – wave generated by wind on the surface of a body of water. Typically, wind waves in lakes and reservoirs are short-crested (crest length is the same order of magnitude as the wavelength and the wave is steep in nature).
- Wave height (H)– the vertical distance from the lowest point (trough) of the wave to the highest vertical point (crest) of the wave. The wave height is twice the wave amplitude.
- Significant wave height (H_s) – traditionally defined as the mean wave height of the highest third of the waves, though currently it is more often defined as four times the standard deviation of the water surface elevation.
- Wave period (T) – the length of time to complete one cycle or wavelength, or, the reciprocal of frequency.
- Peak wave period (T_p) – the wave period with the highest energy in the time series wave spectra.

- Wavelength (*L*) – horizontal distance between corresponding points of the same phase of the wave, such as crest-to-crest or trough-to-trough (= $2\pi/gT^2$)
- Wave steepness (*so*) – ratio of wave height to wave length (= *H/L*)
- Celerity (or Phase velocity) – the rate at which the phase of the wave propagates in space.
- Shoaling – as waves propagate into shallower water the celerity and wavelength of the waves decrease. With the exception of bottom friction, the total energy flux of the wave should remain constant. Therefore, as the wave shoals and the wavelength decreases, the wave height increases.
- Breaking – the amplitude of the wave reaches a critical level as it approaches shallower depth, causing the wave to transform large amounts of its energy into turbulent kinetic energy.
- Surf-similarity parameter (ξ) – also called the breaker parameter, this value defines the type of wave breaking (= $\tan \alpha / so^{1/2}$; where α is the slope of the bathymetry or levee).
- Diffraction – bending of waves around small obstacles and the spreading of waves past small openings.
- Refraction – bending of waves due to differential water depths underneath the wave. As a wave approaches from an angle, the portion of the wave closest to shore slows down first as it approaches shallower depth.
- Reflection – change in direction of a wave due to an obstacle.
- Angle of incidence (β) – angle between the crest of the wave and the crest of the structure. For head-on waves, $\beta = 0^\circ$. For waves propagating parallel to the structure, so that the crests are perpendicular to the structure, $\beta = 90^\circ$.
- Radiation stress – Change in the horizontal momentum of the water column due to the presence of waves.
- Wave setup – increase in water surface elevation due to the presence of waves and their radiation stresses.
- Wind setup – increase in water surface elevation due to the wind forcing. For winds of the same magnitude, shallower bathymetry would result in greater wind setup than deeper bathymetry.
- Still water level (SWL) – average water surface elevation that may include wind setup but excluding variation due to waves. For this analysis, the SWL will be considered the flat pool elevation without any consideration for the wind induced water surface.

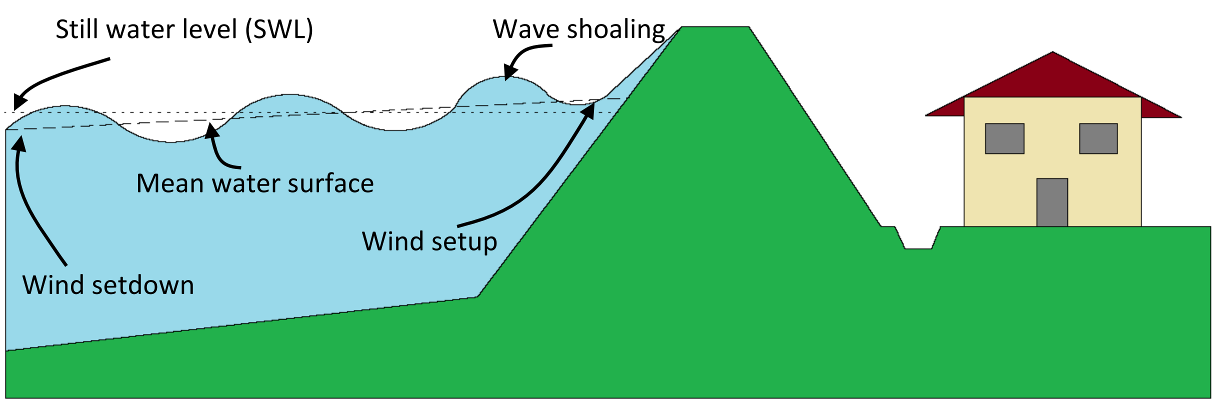

Figure 2 – Diagram of some of the wind-water interactions such as wind setup and wave shoaling.

1.4 Wave Runup vs. Wave Overtopping

The two general methods for determining the required levee height in the presence of waves are as follows:

- Wave runup ($R_{U\%}$) – the vertical distance between the highest point a percentage of waves can reach on a structure and the still water level. While commonly referenced as the 2% wave runup, or $R_{2\%}$, this level is arbitrary in nature and does not inherently describe the threat to the structure or the population behind the structure. See Figure 3 for a diagram of wave runup height.

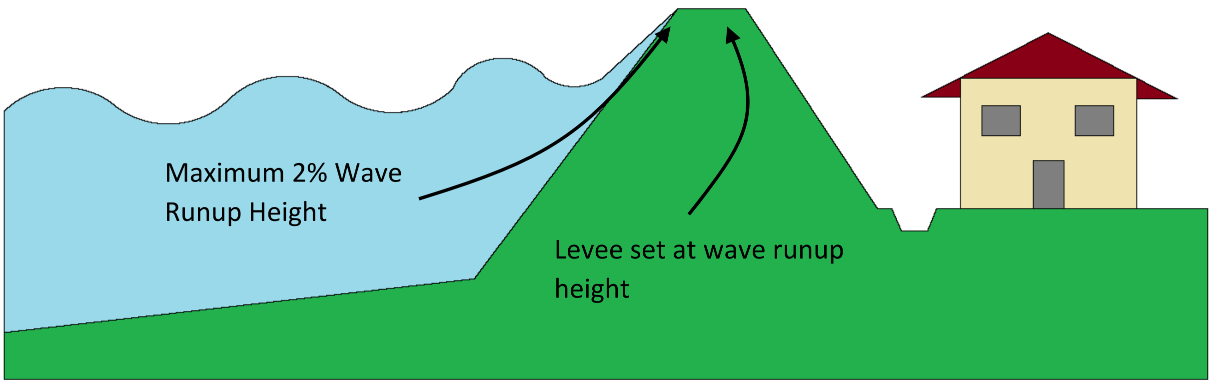

Figure 3 – Diagram of maximum wave runup.

- Wave overtopping (q_o) – average discharge, per unit length of structure, that washes over a structure due to waves (See Figure 4).

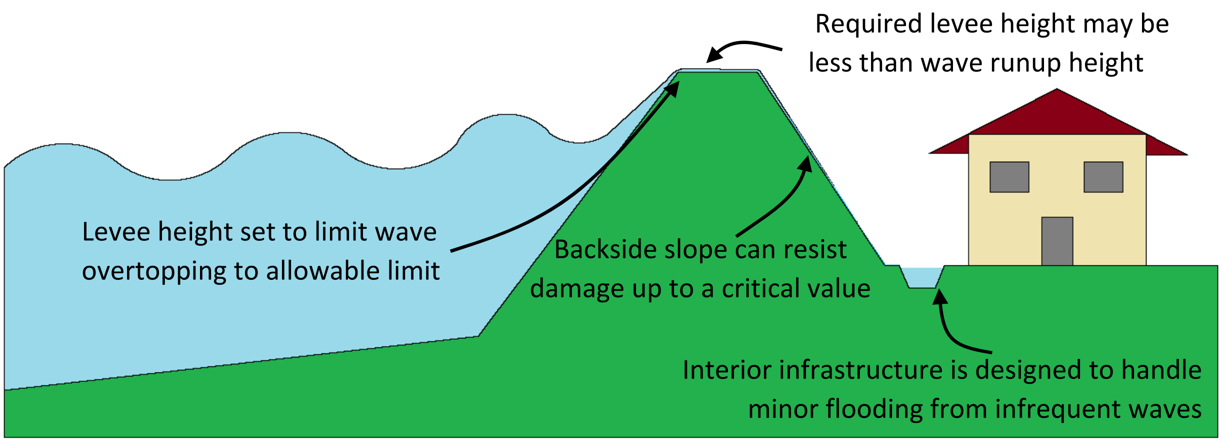

Figure 4 – Diagram of limited allowable wave overtopping.

Allowable overtopping (q_{all}) refers to the various levels of tolerable limits of wave overtopping based on the type of structure and the type of risk. Critical values of average overtopping discharges, summarized from past studies by Goda, van der Meer, and others, can be found in Figure 5.

Table VI-5-6
Critical Values of Average Overtopping Discharges

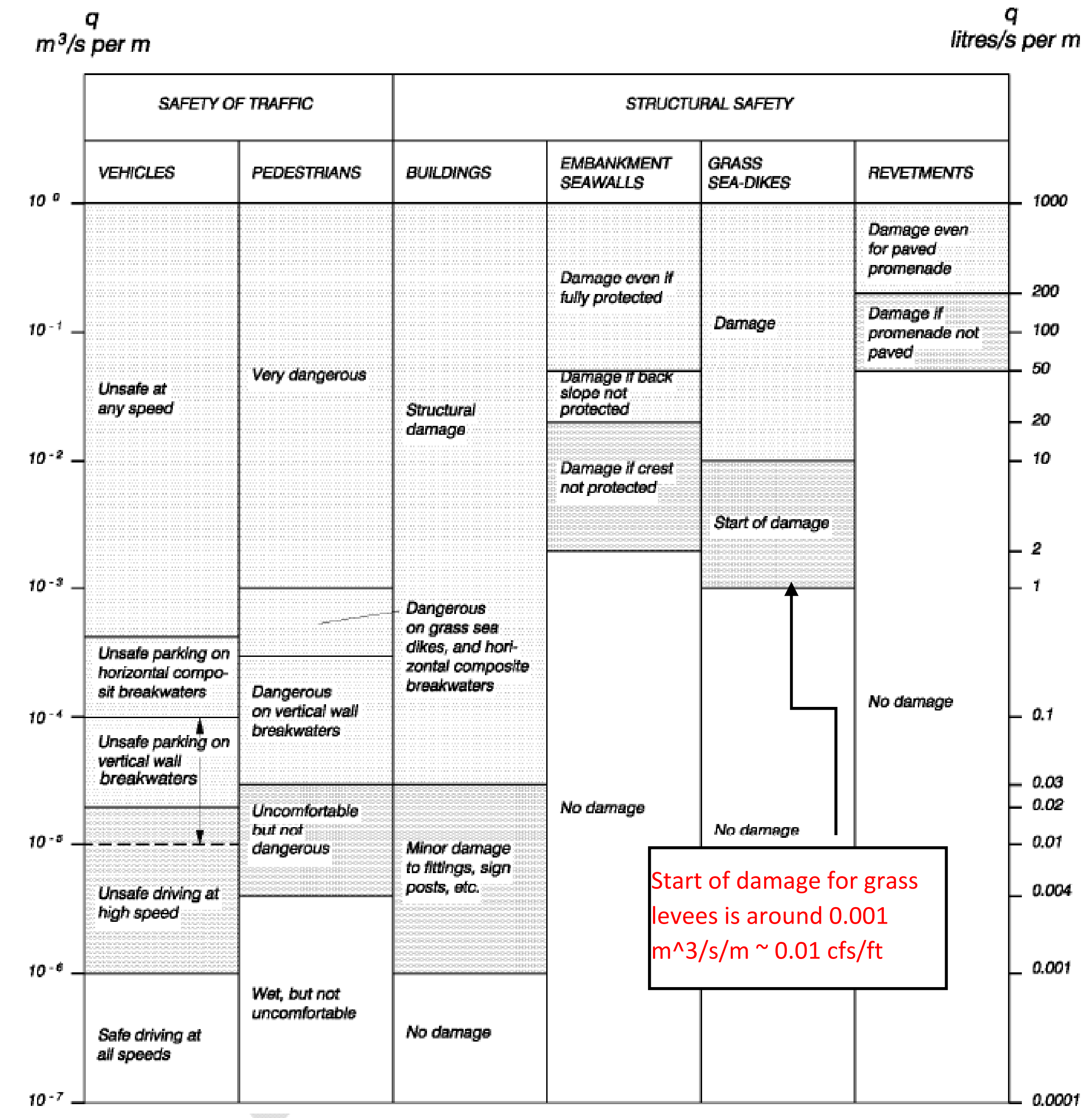

Figure 5 – Table VI-5-6 from the Coastal Engineering Manual – Critical Values of Average Overtopping Discharges

2 WIND DATA

2.1 Data Collection

The closest available wind gage location with a long enough historic record of observations is the gage located at Hector International Airport in Fargo, ND (NOAA 2013). This gage is an overland wind gage that logs the fastest daily wind speeds over various time intervals. These various wind measurements include the fastest 5-second duration wind speed, the fastest 2-minute duration wind speed, and the "the fastest mile" wind speed, which is the average speed of a particle traveling with the wind over the distance of one mile. The 2-minute duration data was selected for use as it is the easiest to convert to a duration long enough to achieve fetch-limited conditions. A plot of the entire record of daily 2-minute wind speeds can be seen in Figure 6 (note that 5 points are considered to be erroneous data (outliers) and will not be included in this analysis). In addition, by summarizing the wind speed data in conjunction with the corresponding wind direction data, a wind-rose was created to show the dominant wind directions (see Figure 7).

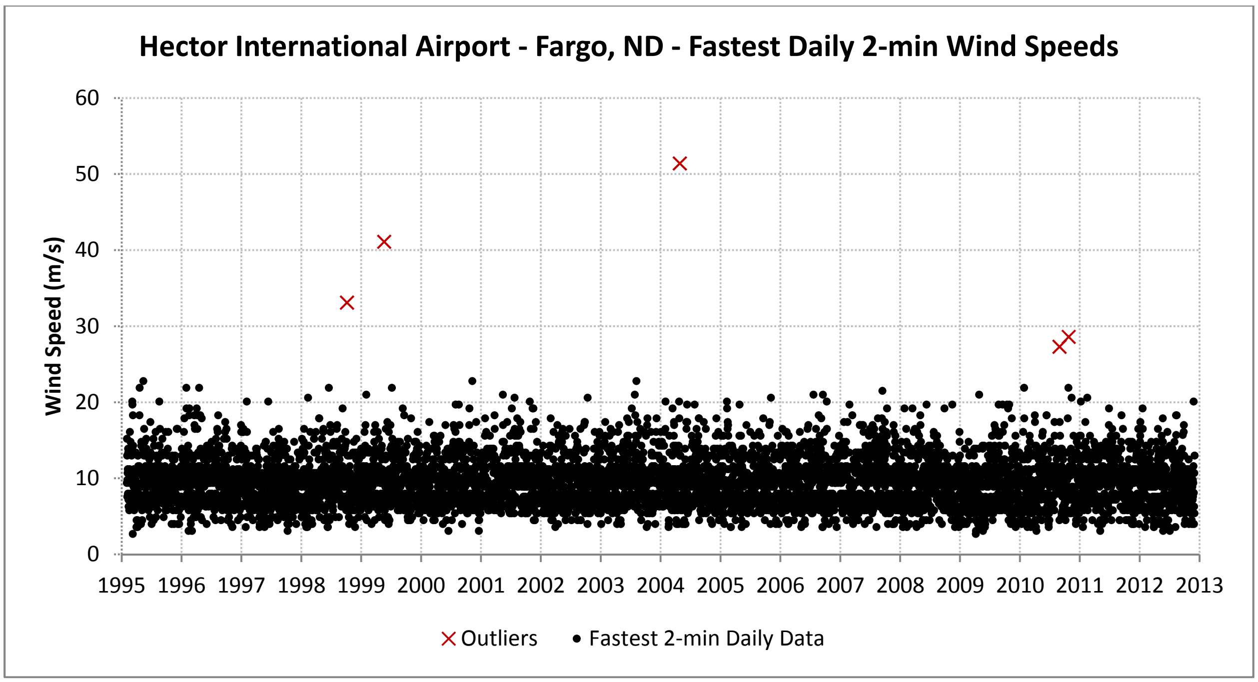

Figure 6 – Fastest 2-min wind speeds for Fargo, ND for entire period of record.

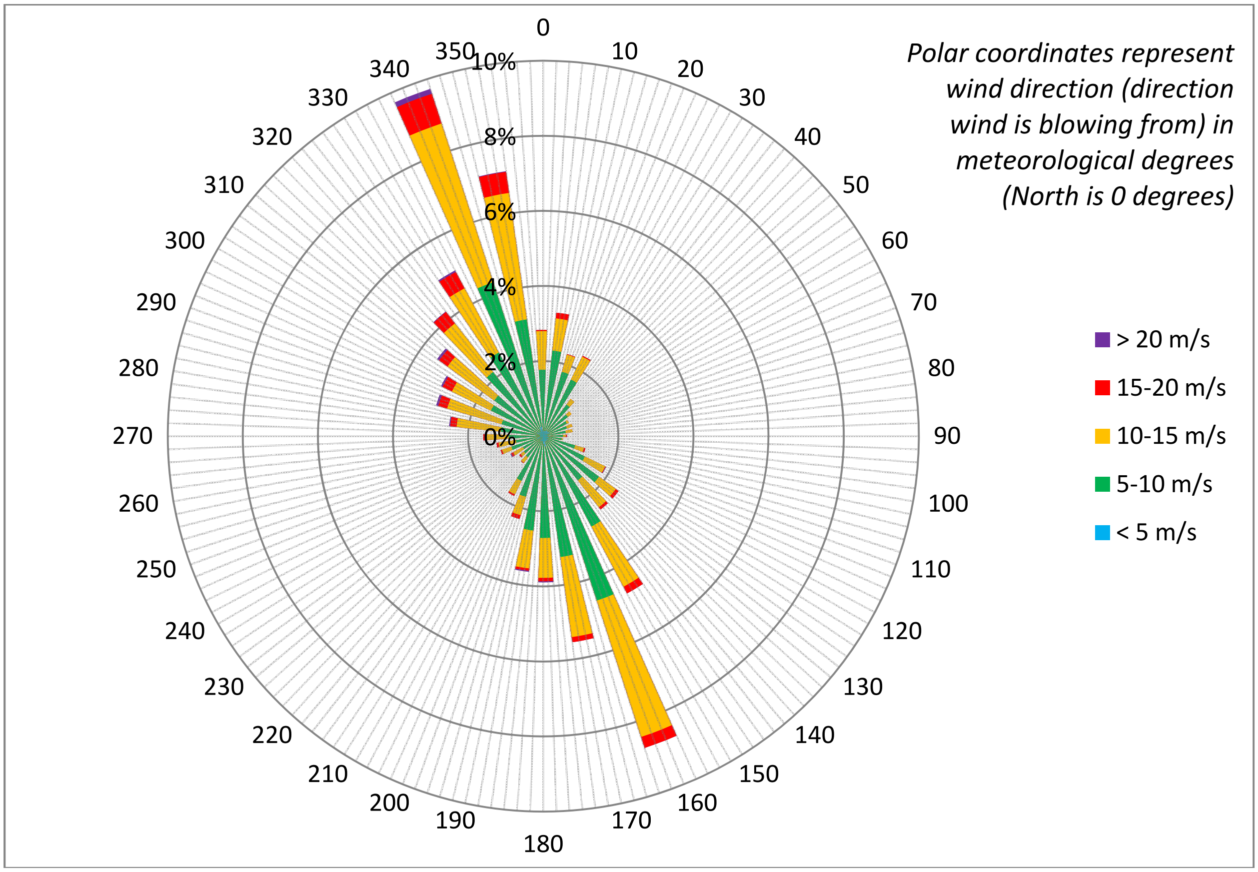

Figure 7 – Wind rose for fastest daily 2-min wind speeds at Fargo, ND.

The 'year-round' data was compared to the 'spring-summer only' data as the months of March through August would be the typical months that the project could be operated. The comparison found little difference in the average values. The year-round data gives higher, more conservative, annual maximum windspeeds, and was therefore used for this analysis. Excluding the erroneous outlier data, the maximum annual 2-minute wind speed data was summarized for the 18 year period of record by performing a Generalized Extreme Value (GEV) Type I (Gumbel Distribution) analysis. The Gumbel Distribution is commonly used to evaluate extreme, low-probability meteorological and hydrological events (Gumbel, 1954). By assigning the maximum values a probability of non-exceedance and plotting the wind speeds against the double-log of the probability, linear regression can be used to estimate values for extreme return periods. This regression analysis is shown in Figure 8. It was recommended by Dr. Jane Smith of the U.S. Army Corps of Engineers, Engineering Research and Development Center, Coastal and Hydraulics Laboratory (USACE-ERDC-CHL), that for an 18 year period of record, the extreme values not be extrapolated beyond the 40-60 year return period. Although a summary of the estimated extreme values shown in Table 1 shows values up to the 100-year (1%) return period, the 50-year (2%) return period will be the maximum value used to calculate maximum wave runup and overtopping for the 1% AEP pool.

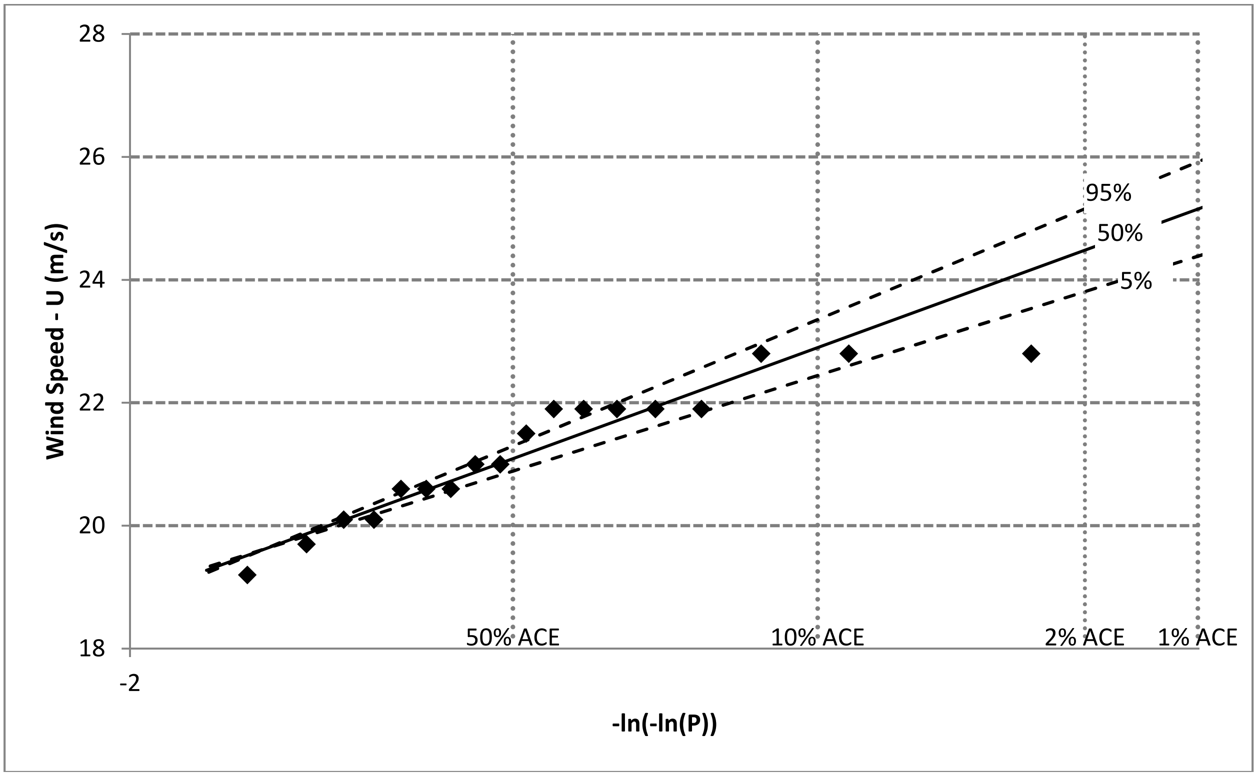

Figure 8 – Generalized Extreme Value (GEV) Type I (Gumbel Distribution) Peak Annual 2-min Wind Data for Fargo, ND

Table 1 – Gumbel Distribution Annual Chance Exceedance 2-min wind speeds (U)

ACE (%)	Tr (yr)	$U_{5\%}$ (m/s)	$U_{50\%}$ (m/s)	$U_{95\%}$ (m/s)
1%	100	24.4	25.2	25.9
2%	50	23.8	24.5	25.2
10%	10	22.4	22.9	23.4
50%	2	20.9	21.1	21.3

In order to analyze the wind-wave response on the ring-levee for a full range of events, 10 simulations were developed to perform the modeling effort. The first 8 simulations were developed to analyze the wind-wave response from any direction. These 8 scenarios apply the maximum recorded 2-minute wind speed for each of the 4 cardinal directions (N, E, S, and W) and 4 ordinal directions (NE, SE, SW, and NW). The last two simulations apply the median 50-year (2%) wind speed in the two predominant wind directions, SSE and NNW. The 50-year (2%) wind speed is extrapolated beyond the annual peak wind speeds using a Generalized Extreme Value distribution, and therefore, greater than any of the observed annual peak wind speeds. For each simulation, an approximate fetch length was estimated from the geometry in order to convert the 2-minute wind speed to a wind speed that would be appropriate for fetch-limited conditions. Conversion equations to adjust wind speeds for duration as well as adjusting overland speeds to overwater speeds and adjusting the wind speeds to a 10 meter height give final maximum wind speeds for each simulation used for modeling. These equations can be found in the

Coastal Engineering Manual (CEM) in Part II – Chapter 2 "Meteorology and Wave Climate" (see Figure 9). A summary of the wind adjustments made for the 10 modeling simulations can be found in Table 2.

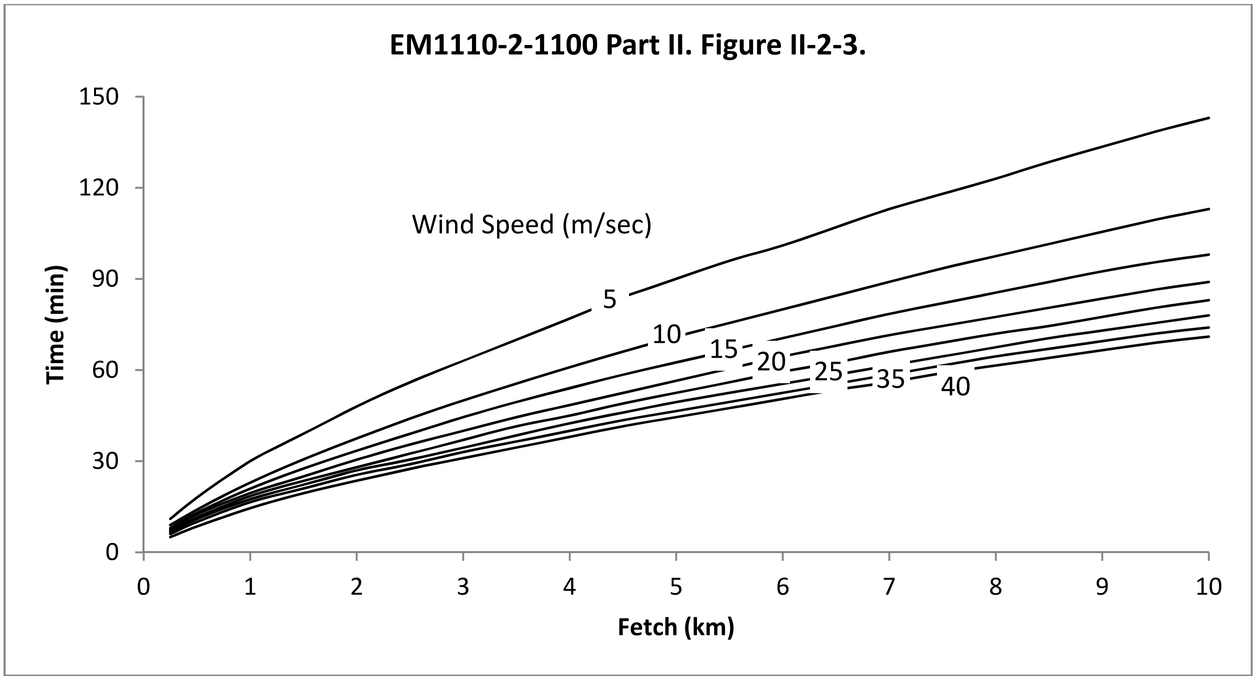

Figure 9 – Equivalent duration for wave generation as a function of fetch and wind speed.

Table 2 – Summary of wind adjustments for all 10 simulated wind speeds and directions

Simulation	Wind Orientation		Gage Wind speed	Adjust to 10m	Adjust to 1hr	Fetch	Duration	Duration Adjusted Speed	Overwater Speed
	(dir.)	(deg.)	(m/s)	(m/s)	(m/s)	(km)	(min)	(m/s)	(m/s)
1	N	0	22.8	23.1	19.8	4	48.7	19.8	23.8
2	NE	45	17.4	17.6	15.1	5.9	63.2	15	18
3	E	90	17.9	18.1	15.5	3.5	44.6	15.6	18.7
4	SE	135	20.1	20.4	17.4	10.7	94	16.9	20.3
5	S	180	22.8	23.1	19.8	10.3	91.7	19.2	23.1
6	SW	225	21	21.3	18.2	10.2	91.1	17.7	21.2
7	W	270	21.9	22.2	19	4.7	54.3	19	22.8
8	NW	315	22.8	23.1	19.8	4.6	53.5	19.8	23.7
9	SSE	160	24.5	24.8	21.2	10.7	87.1	20.7	24.8
10	NNW	340	24.5	24.8	21.2	10.7	87.1	20.7	24.8

3 MODEL SETUP

3.1 Model Process

To accurately estimate the wave-growth and wind-induced setup across the staging area, a more detailed two-dimensional (2D) model approach was selected in place of the one-dimensional (1D) simplified equation approach. This 2D approach involves the coupling of a 2D gridded wave model and a 2D meshed hydrodynamic model. The 2D approach was selected because the models can account for the wave reduction due to the shallow bathymetry and submerged roadways as well as the wave diffraction due to exposed roadways and high ground. Each of the models will be explained in further detail in the following sections.

All model inputs and outputs are reported in meters. Vertical elevation is in NAVD 88 Datum.

3.2 Geometry

Both 2D models utilized LiDAR data collected in 2008 for the construction of the geometry. Roadways were artificially widened to match the resolution of the grid/mesh to allow for stable model runs and low model run-times. The OHB levee alignment was based on the most recent proposed alignment made in October 2013. Three roadways (Interstate-29, County Road 18, and County Highway 75) and one railroad were raised from their current elevation to match the proposed road raises from the latest project plan.

3.3 STWAVE

STWAVE (STeady State spectral WAVE)
The STWAVE model "simulates depth-induced wave refraction and shoaling, current-induced refraction and shoaling, depth- and steepness-induced wave breaking, diffraction, parametric wave growth because of wind input, and wave-wave interaction and white capping that redistribute and dissipate energy in a growing wave field." Figure 10 shows the STWAVE grid.

Number of grid cells in i	= 545
Number of grid cells in j	= 421
Number of grid cells Total	= i x j = 229,445
Grid cell resolution	= 30 m

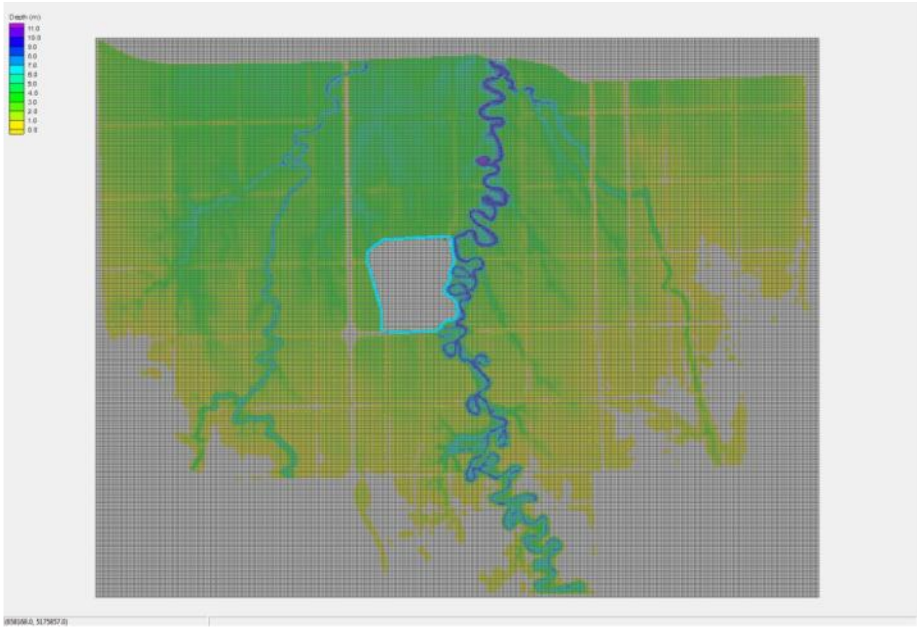

Figure 10 – Plot of STWAVE Grid

3.4 AdH

AdH (Adaptive Hydraulics Modeling)

AdH is an "adaptive finite-element model for one-, two- and three-dimensional flow and transport." The model has capabilities to include such features as "wetting and drying, completely coupled sediment transport, and wind effects." For the purpose of this analysis, AdH will be used to calculate the wind setup. The hydrodynamic inputs in the model were set to remain constant with a level pool at elevation 922.5 ft. For each of the 10 simulations, the wind speed and direction are modified and the radiation stress output from STWAVE is included as an initial condition. Figure 11 shows the AdH mesh. A few of the global inputs and characteristics of the model are as follows:

Elements = 71,514
n-value = 0.035
Kinematic viscosity = 1.52e-006 m^2/s

Two additional runs were made for Simulation 9 to check the sensitivity of the Manning's n-value. Considering an n-value of 0.025 and 0.045, results only differed approximately 10% for the maximum setup. The kinematic viscosity value that was selected was based on the viscosity of water at 5˚ C (41˚ F).

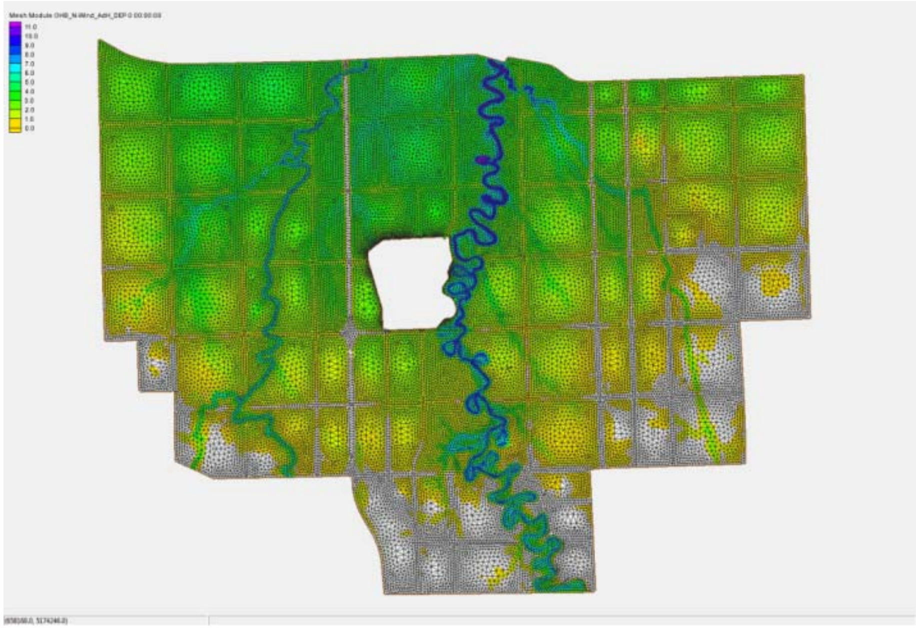

Figure 11 – Plot of ADH Mesh

3.5 Wind Inputs

STWAVE and AdH use standard meteorological coordinates. For STWAVE wind inputs are expected as speed and direction toward which the wind is blowing, measured counterclockwise from East (x). AdH requires the wind to be input as x and y components for the direction using the same coordinate system as STWAVE. Table 3 provides a summary of the wind inputs for each model.

Table 3 – Summary of wind speed and direction inputs for STWAVE and AdH

Simulation	Overwater Wind	X-component	Y-component	STWAVE - Magnitude	STWAVE - Direction	AdH - Vx	AdH - Vy
	(m/s)	-	-	(m/s)	(deg)	(m/s)	(m/s)
1	23.8	0	-1	23.8	270	0	-23.8
2	18	-0.707	-0.707	18	225	-12.8	-12.8
3	18.7	-1	0	18.7	180	-18.7	0
4	20.3	-0.707	0.707	20.3	135	-14.3	14.3
5	23.1	0	1	23.1	90	0	23.1
6	21.2	0.707	0.707	21.2	45	15	15
7	22.8	1	0	22.8	0	22.8	0
8	23.7	0.707	-0.707	23.7	315	16.8	-16.8
9	24.8	-0.342	0.94	24.8	110	-8.5	23.3
10	24.8	0.342	-0.94	24.8	290	8.5	-23.3

4 MODEL RESULTS

4.1 Wave Height

The following figures (Figure 13 through Figure 22) show the modeling results for wave height (in meters) from the STWAVE model. Figure 12 shows a legend for the color contour and corresponding wave height. In general, the highest modeled waves in the staging area generally occur along the tie-back embankments along the north side of the staging area when winds blow across the staging area from a general southerly direction. However, the largest wave heights along the OHB alignment seem to occur when the wind blows from a general northerly direction. This appears to be because winds from the south blow across a shallower pool which limits the ultimate wave height.

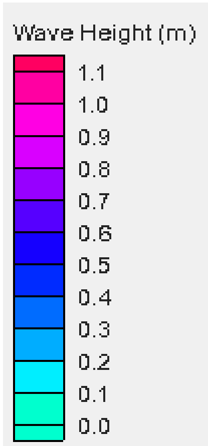

Figure 12 – Legend for Wave Height Results

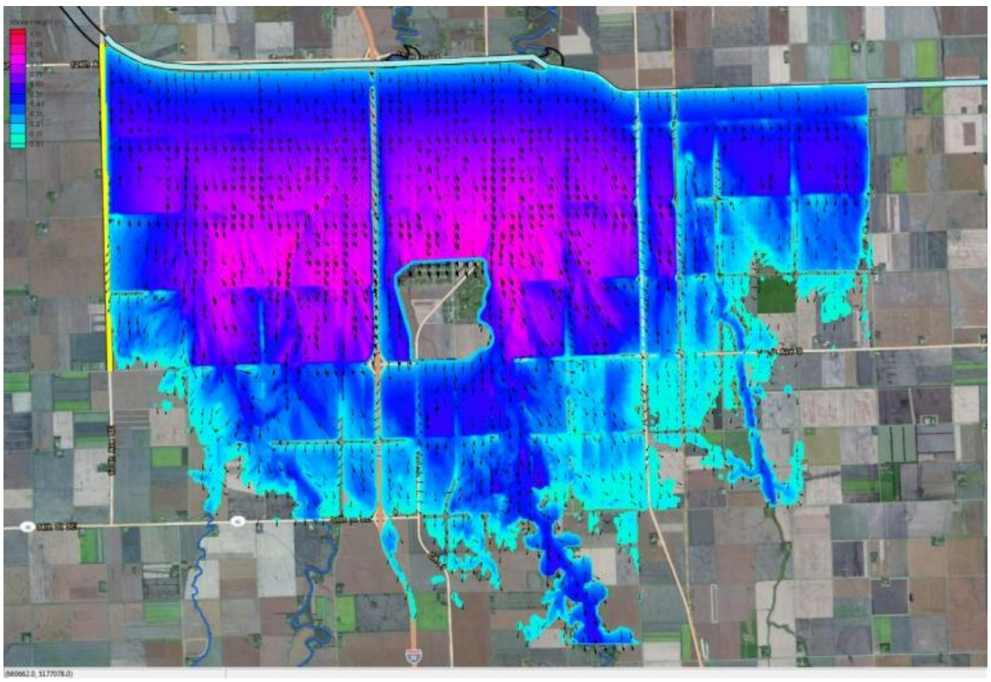

Figure 13 – Wave Height – North wind (N, 0˚) at 23.8 m/s (53.2 mi/hr)

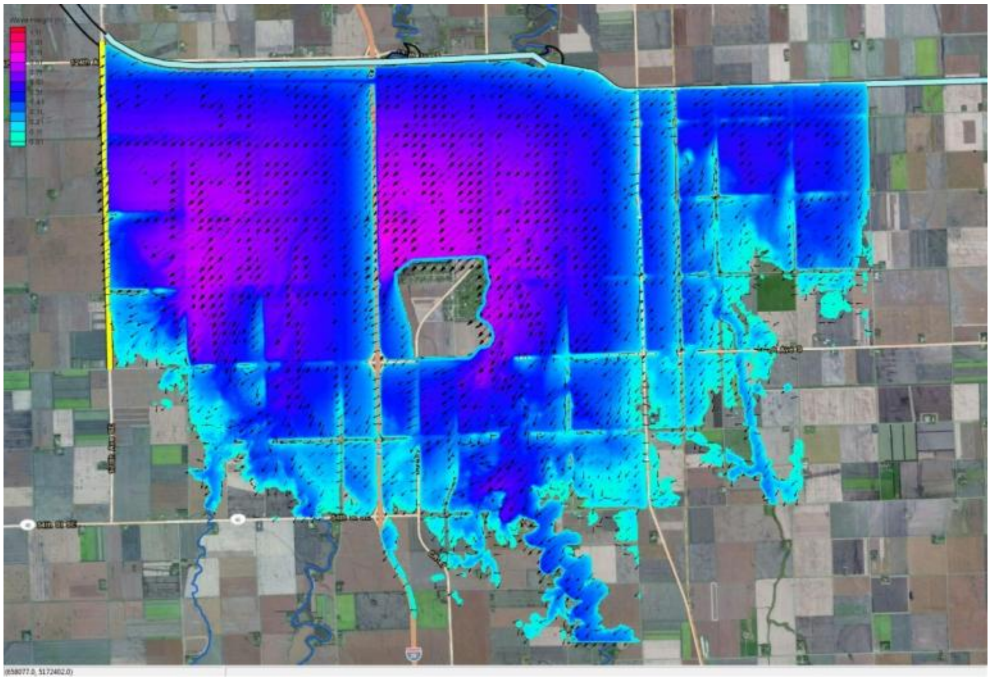

Figure 14 – Wave Height – Northeast wind (NE, 45˚) at 18.0 m/s (40.3 mi/hr)

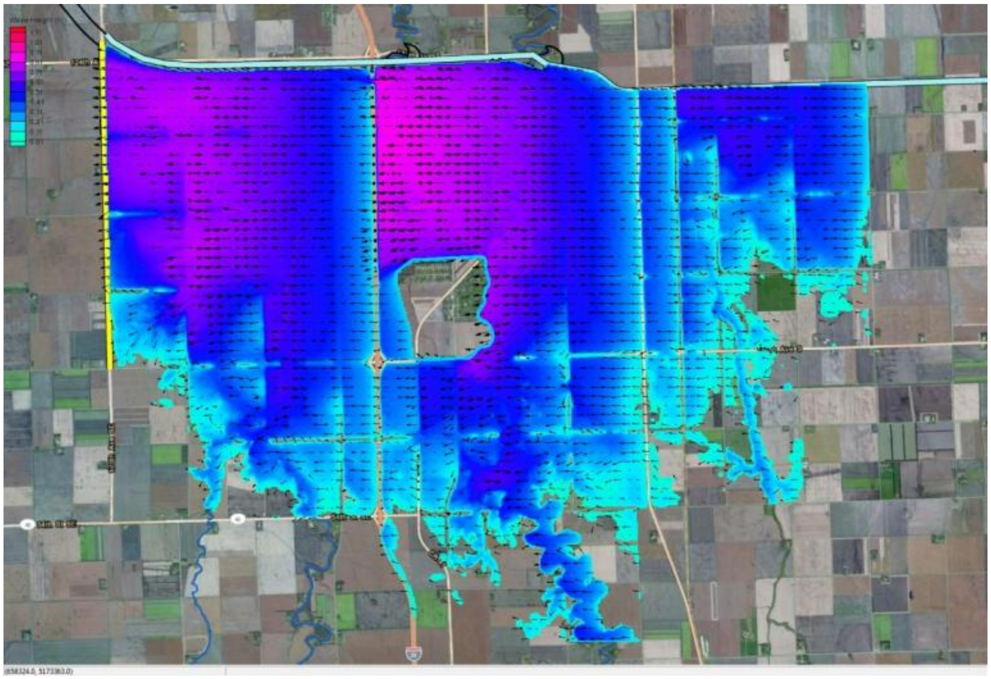

Figure 15 – Wave Height – East wind (E, 90˚) at 18.7 m/s (41.8 mi/hr)

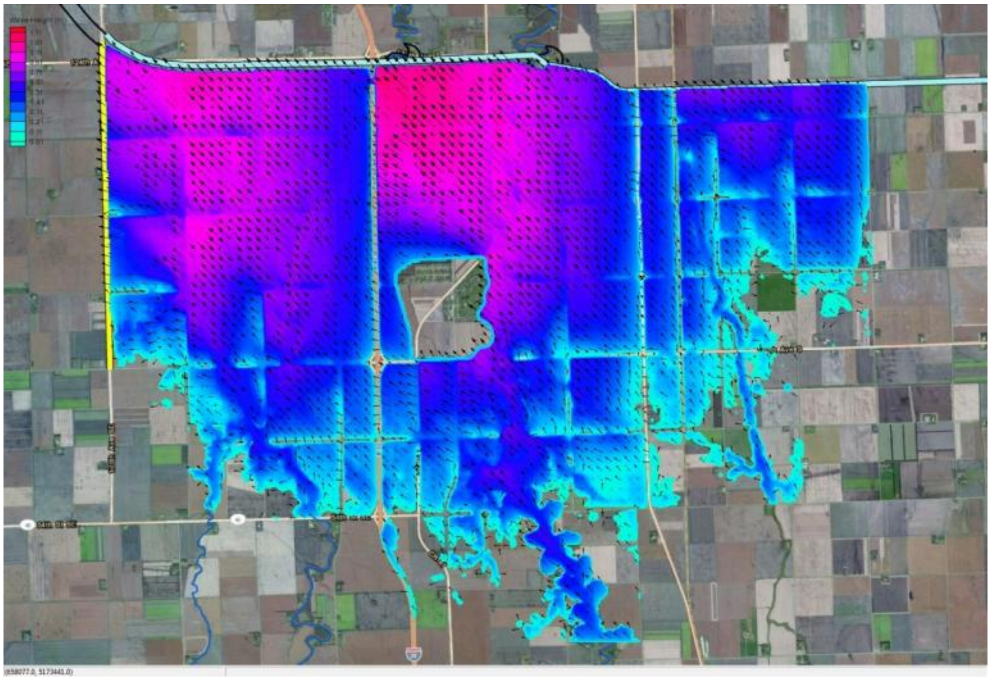

Figure 16 – Wave Height – Southeast wind (SE, 135˚) at 20.3 m/s (45.4 mi/hr)

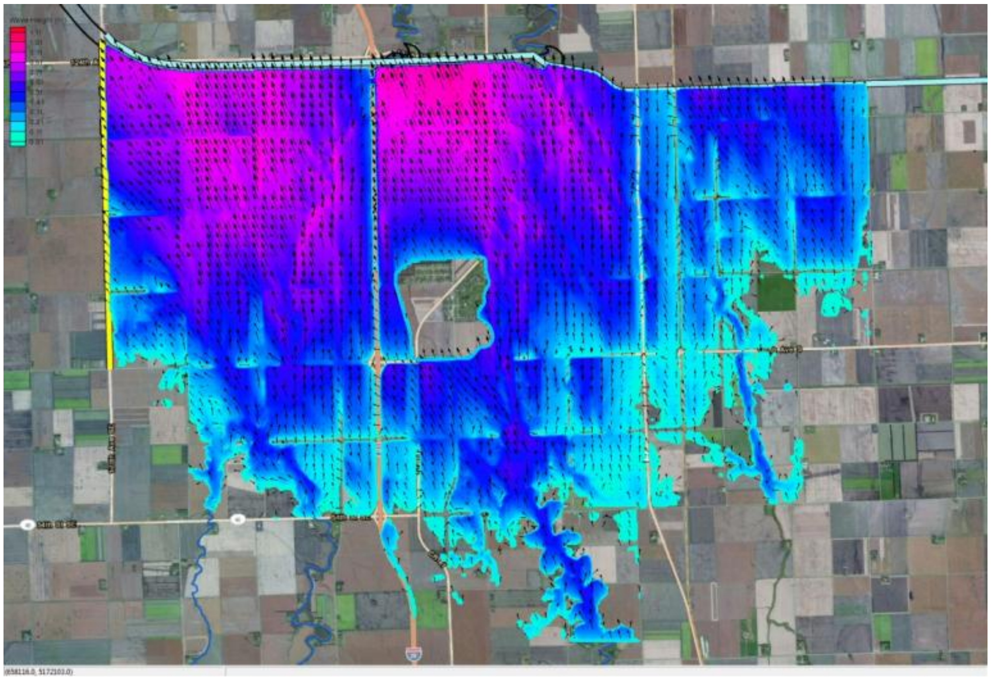

Figure 17 – Wave Height – South wind (S, 180˚) at 23.1 m/s (51.7 mi/hr)

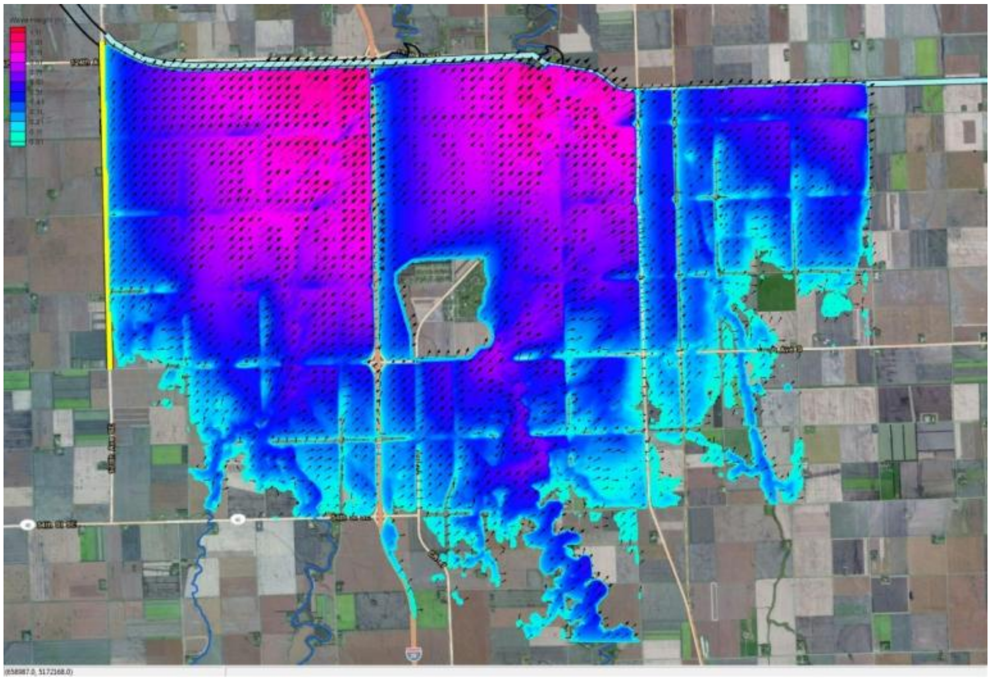

Figure 18 – Wave Height – Southwest wind (SW, 225˚) at 21.2 m/s (47.4 mi/hr)

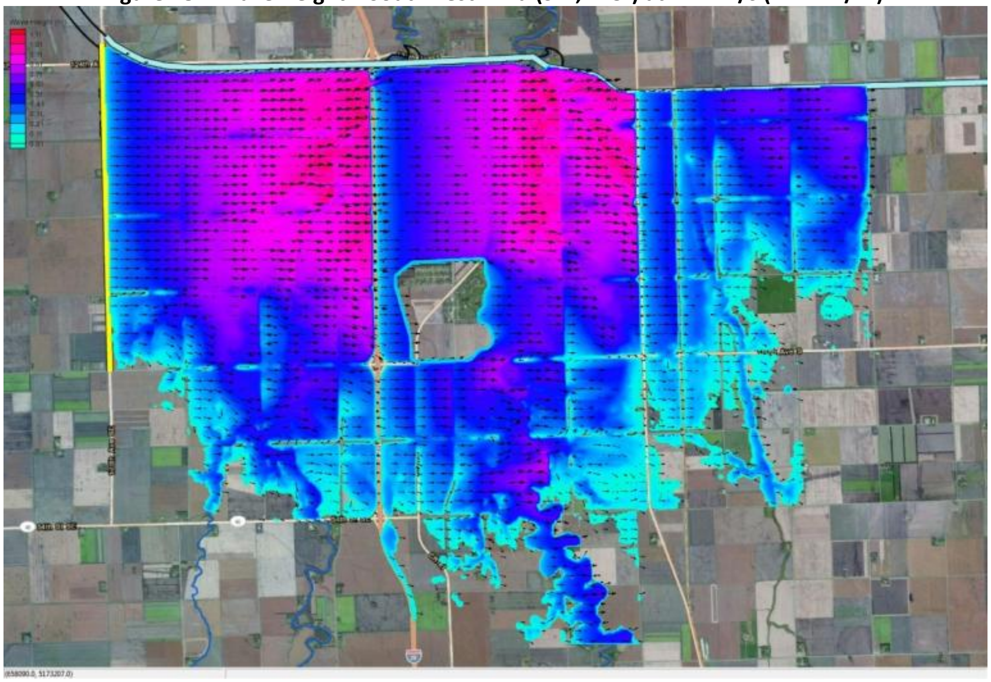

Figure 19 – Wave Height – West wind (W, 270˚) at 22.8 m/s (51.0 mi/hr)

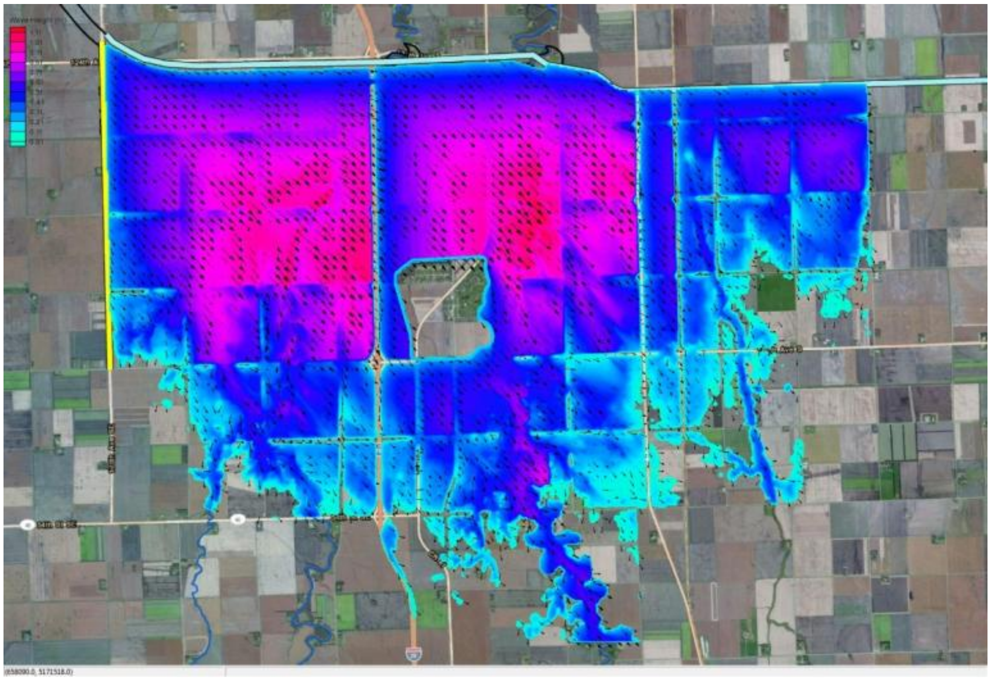

Figure 20 – Wave Height – Northwest wind (NW, 315˚) at 23.7 m/s (53.0 mi/hr)

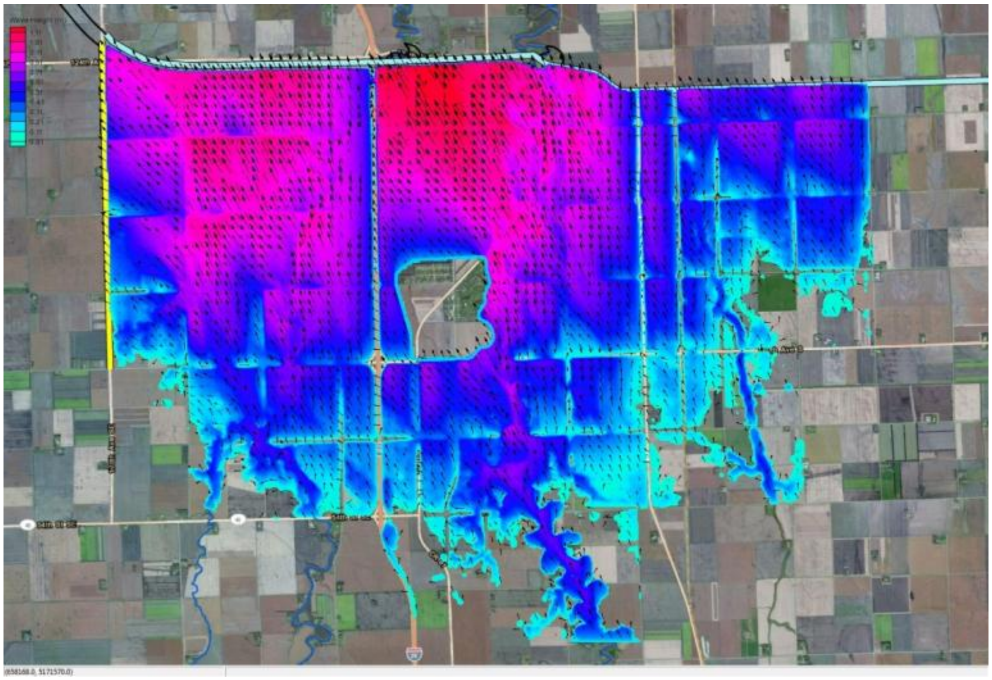

Figure 21 – Wave Height – 50-yr (2%) wind of 24.8 m/s (55.5 mi/hr) in dominant South-Southeast (SSE, 160˚) direction

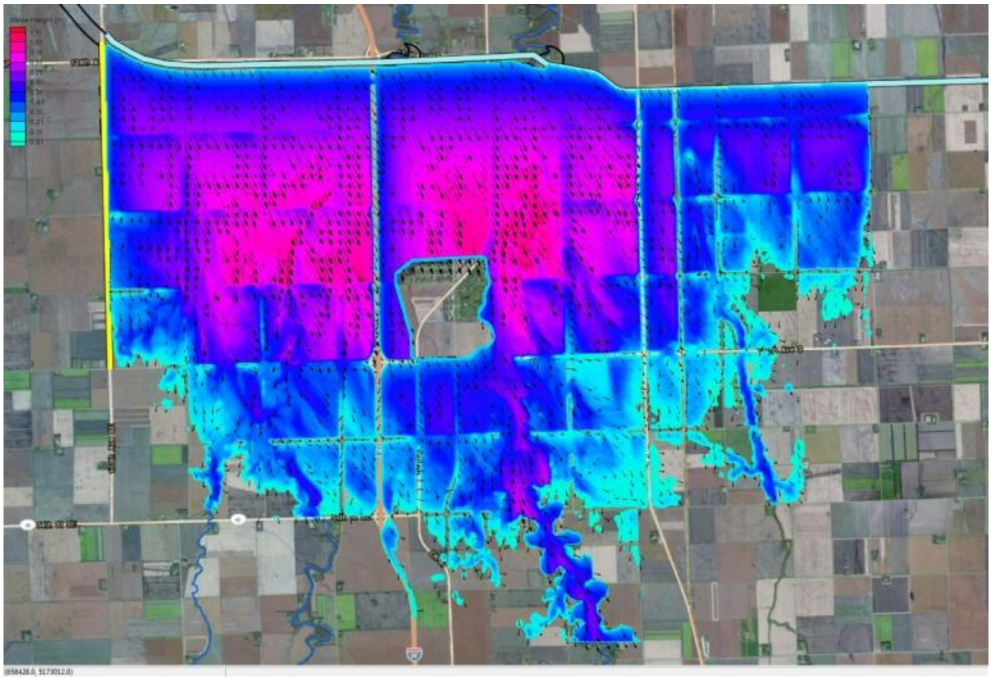

Figure 22 – Wave Height – 50-yr (2%) wind of 24.8 m/s (55.5 mi/hr) in dominant North-Northwest (NNW, 340˚) direction

4.2 Wind and Wave Setup

The following figures (Figure 24 through Figure 33Figure 22) show the modeling results for wind setup (in meters) from the AdH model. Figure 23 shows a legend for the color contour and corresponding change in water surface elevation. In general, since the OHB levee is centrally located in the staging area, no matter which direction the wind blows from, the wind setdown occurs upwind of OHB and the wind setup occurs downwind of OHB. While the wind setup from a southerly wind can be as much as 0.25 meters along the tieback embankments, the typical maximum setup along the OHB levee for any wind is 0.05 meters. The one exception is, due to the road raise of the evacuation route to the south, a northerly wind can cause greater setup (~0.15) at the very southwest corner of the OHB levee.

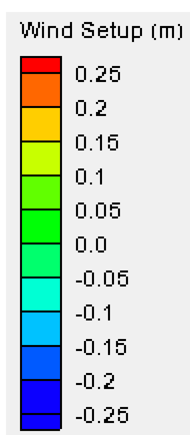

Figure 23 – Legend for Wind Setup (+) and Wind Setdown (-)

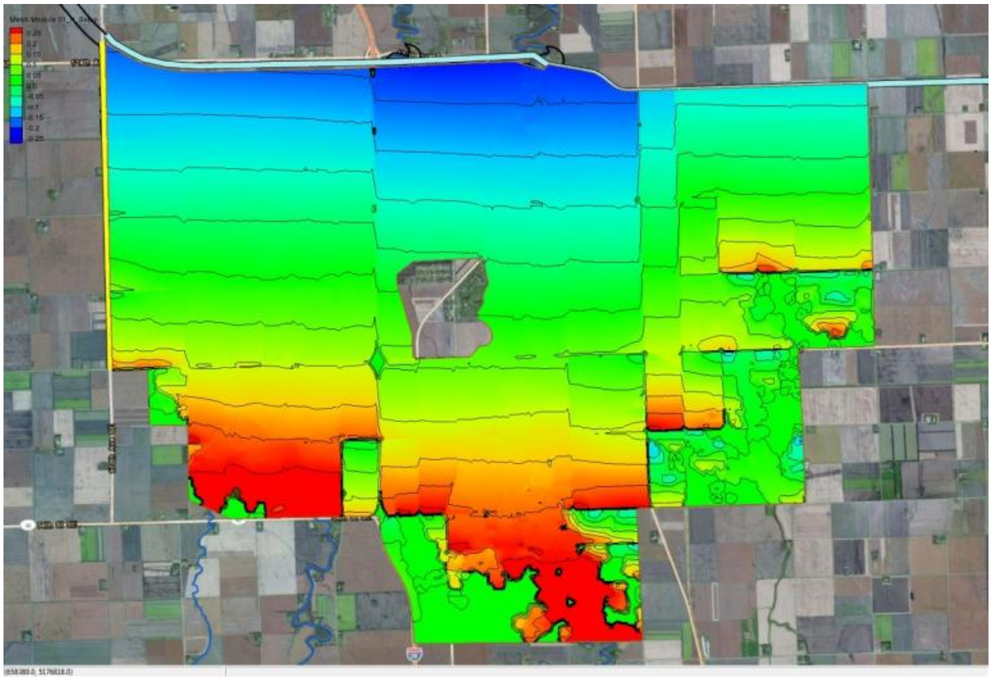

Figure 24 – Wind Setup – North wind (N, 0˚) at 23.8 m/s (53.2 mi/hr)

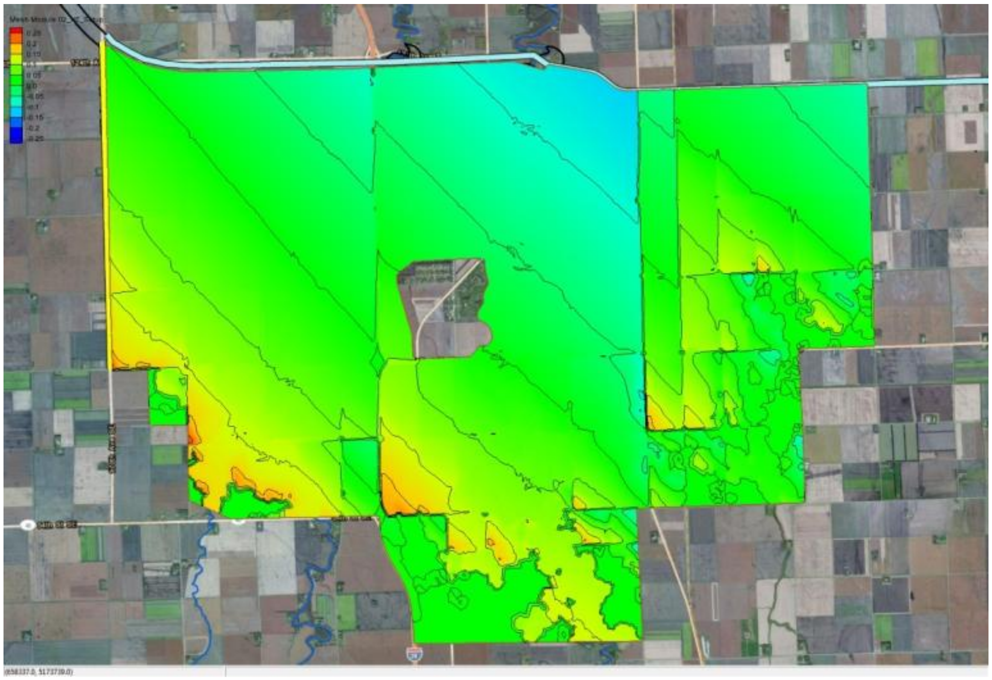

Figure 25 – Wind Setup – Northeast wind (NE, 45˚) at 18.0 m/s (40.3 mi/hr)

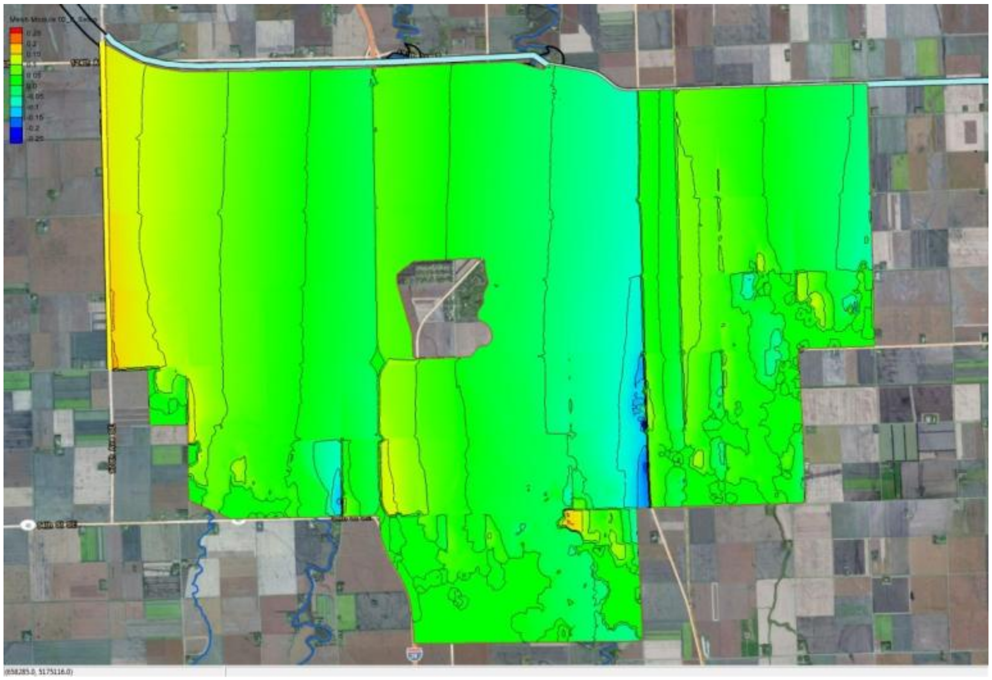

Figure 26 – Wind Setup – East wind (E, 90˚) at 18.7 m/s (41.8 mi/hr)

Figure 27 – Wind Setup – Southeast wind (SE, 135˚) at 20.3 m/s (45.4 mi/hr)

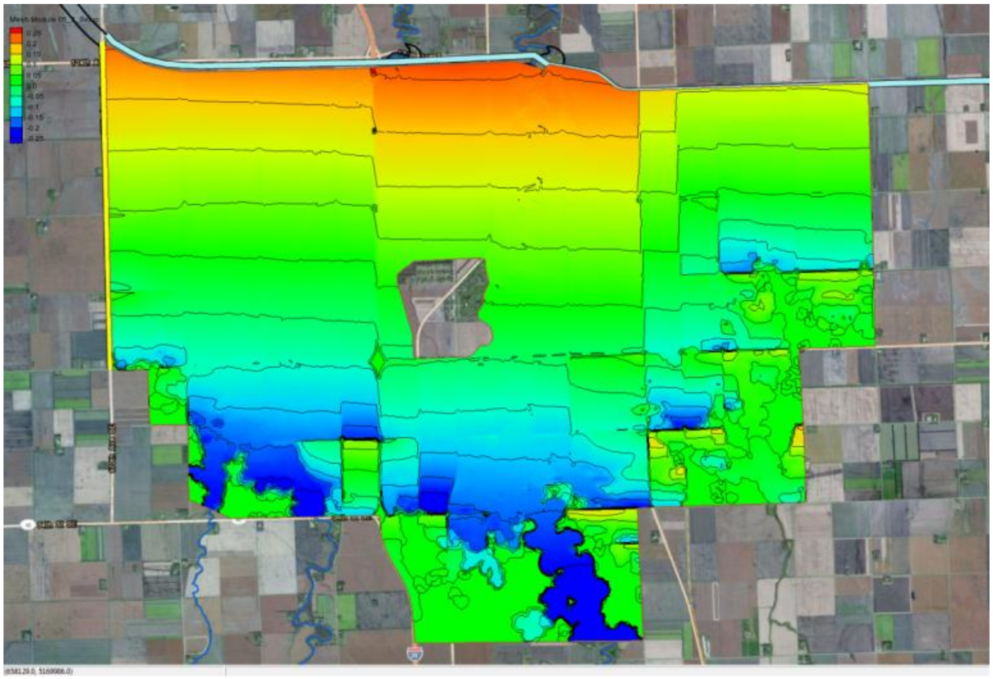

Figure 28 – Wind Setup – South wind (S, 180˚) at 23.1 m/s (51.7 mi/hr)

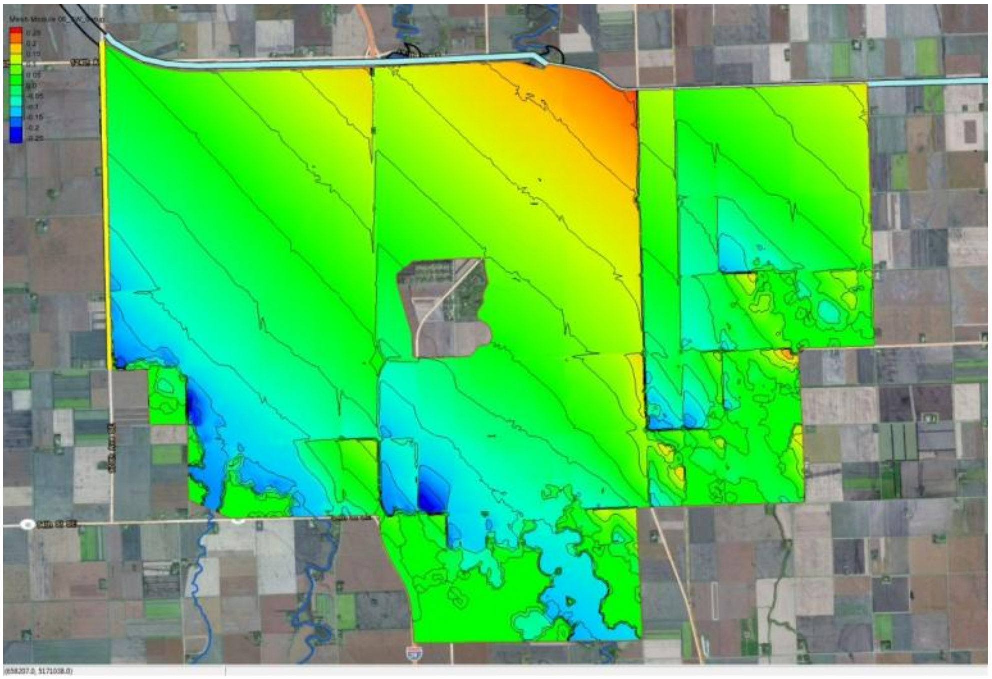

Figure 29 – Wind Setup – Southwest wind (SW, 225˚) at 21.2 m/s (47.4 mi/hr)

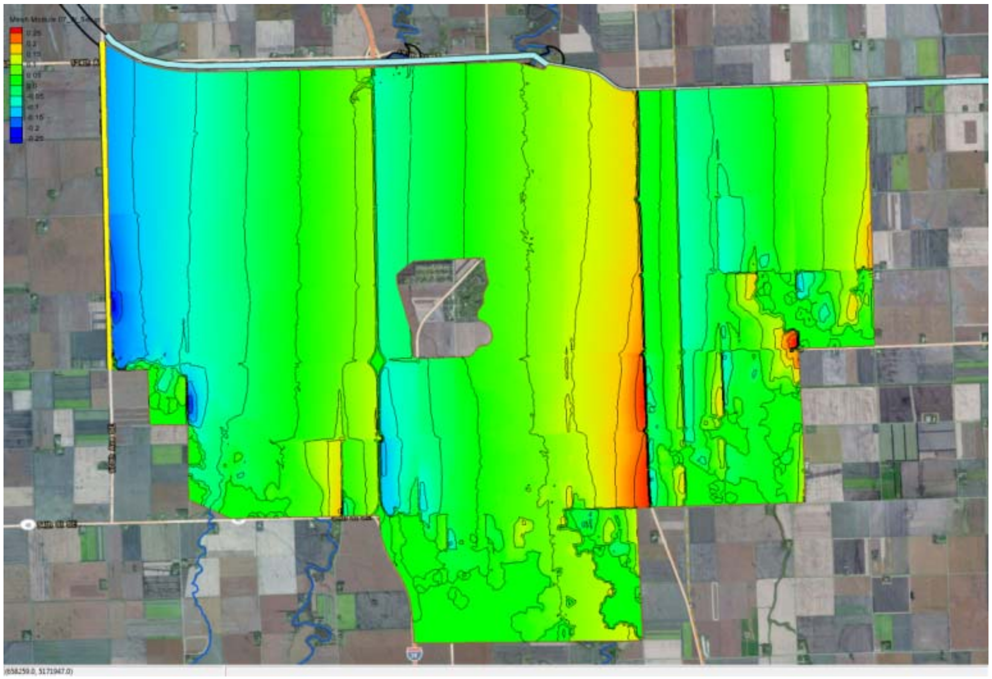

Figure 30 – Wind Setup – West wind (W, 270˚) at 22.8 m/s (51.0 mi/hr)

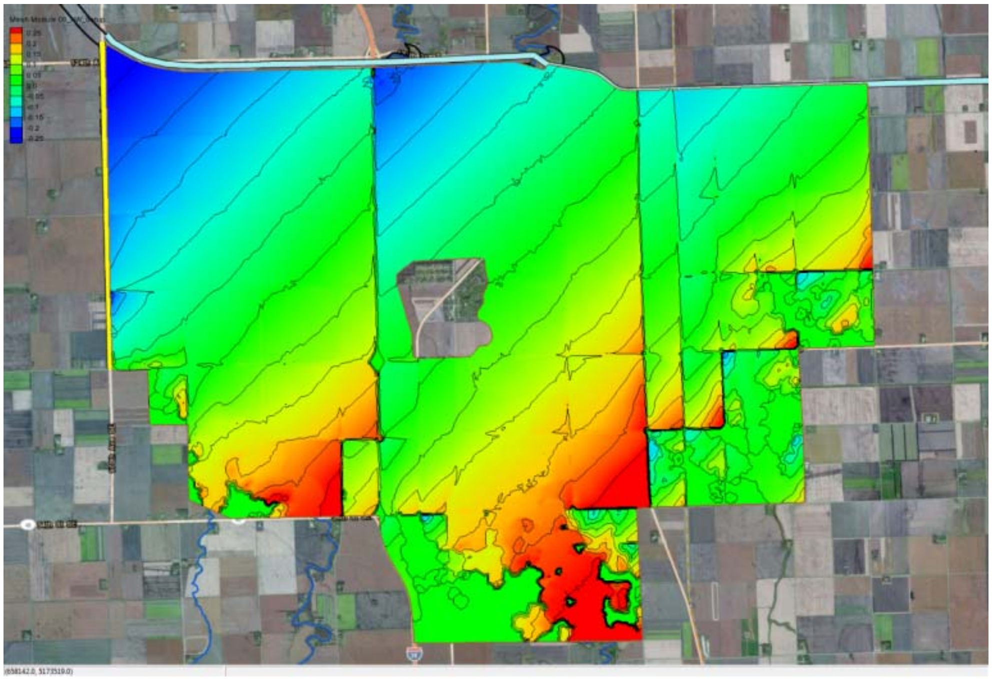

Figure 31 – Wind Setup – Northwest wind (NW, 315˚) at 23.7 m/s (53.0 mi/hr)

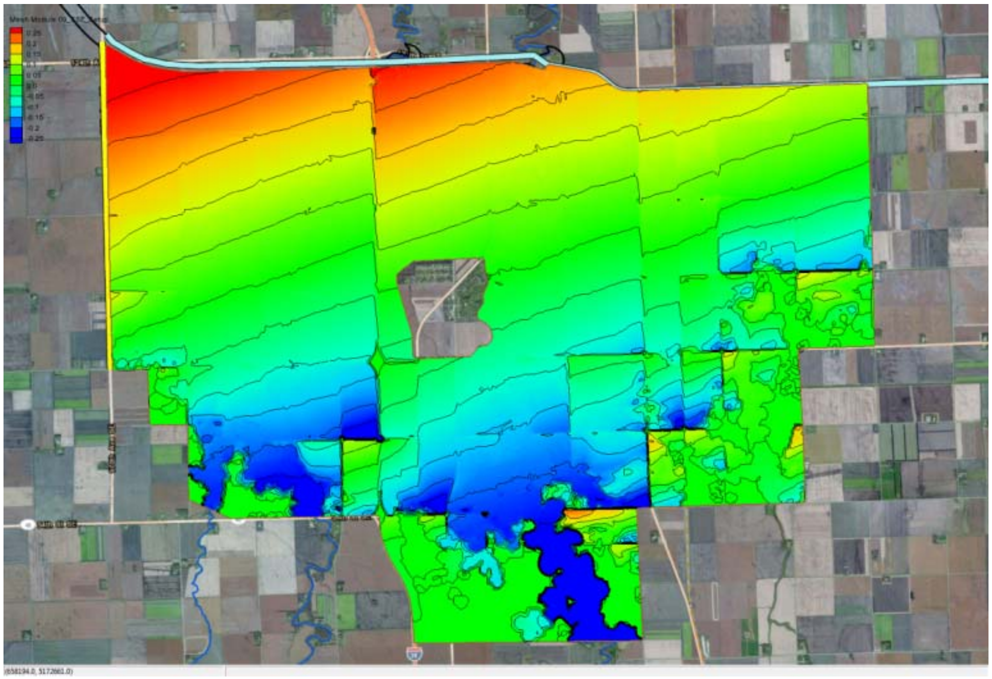

Figure 32 – Wind Setup – 50-yr (2%) wind of 24.8 m/s (55.5 mi/hr) in dominant South-Southeast (SSE, 160˚) direction

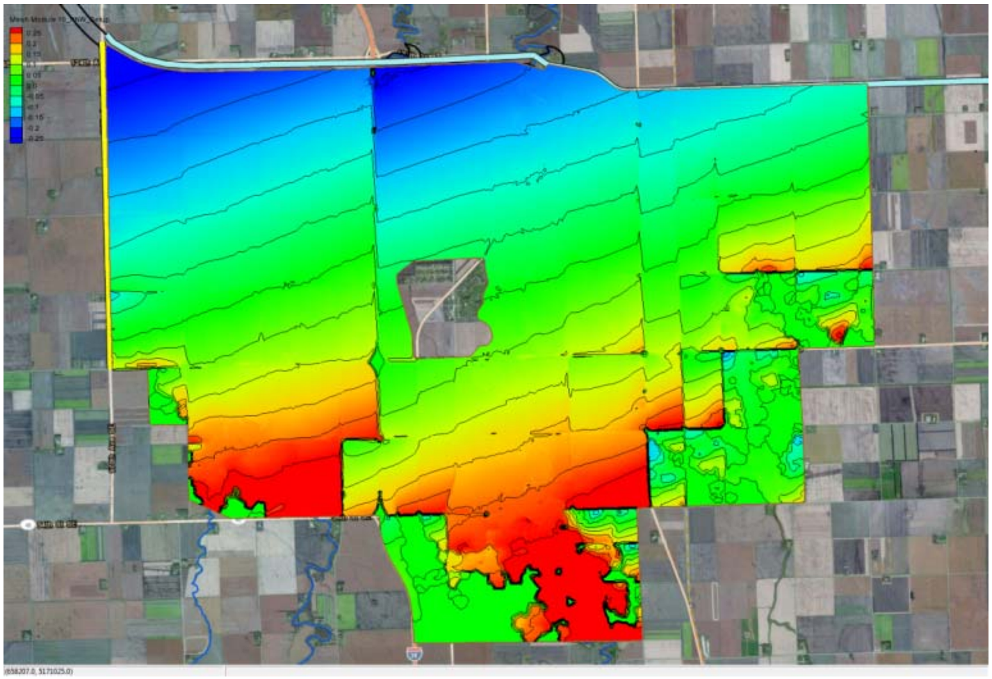

Figure 33 – Wind Setup – 50-yr (2%) wind of 24.8 m/s (55.5 mi/hr) in dominant North-Northwest (NNW, 340˚) direction

5 WIND/WAVE BASED TOP OF LEVEE ELEVATION

For design purposes, all modeling results for wave height and wind setup have been converted from meters to feet. The values given in this section of the report will all be reported in feet. Vertical elevation is in NAVD 88 Datum.

5.1 Design Cross-Section

The preliminary design of the OHB levee assumes the 1% ACE and 0.2% ACE SWL to be maintained at the same elevation of 922.2 ft. The required levee height above the SWL for the current design includes an estimated 4 ft for wind-wave induced height and estimated 0.5 ft for geotechnical settlement of the levee for a total of 4.5 ft. This sets the top of levee at 926.7 ft. The preliminary levee slope design to meet stability and access requirements includes a 1V:4H slope for the exterior side (flood-side) and a 1V:5H slope for the interior of the levee. See Figure 34 for a drawing of a typical levee section.

More recent hydraulic model runs indicate the water surface in the staging area could peak at a slightly higher elevation of 922.5 ft for the 1% and 0.2% ACE events. The wind-wave modeling was performed assuming the water surface was at elevation 922.5 ft and the recommended wind/wave based top of levee assumes a height above this elevation.

One additional important aspect of the design is that the top of levee for OHB must be adequately above the overflow spillway embankment to the west. The preliminary plan for this spillway embankment is to set it at an elevation of 0.5 ft above the 0.2% AEP water surface of 922.5 ft. The OHB design would want to ensure that the ring levee does not get overtopped until an extreme event well beyond the trigger level for the overflow spillway.

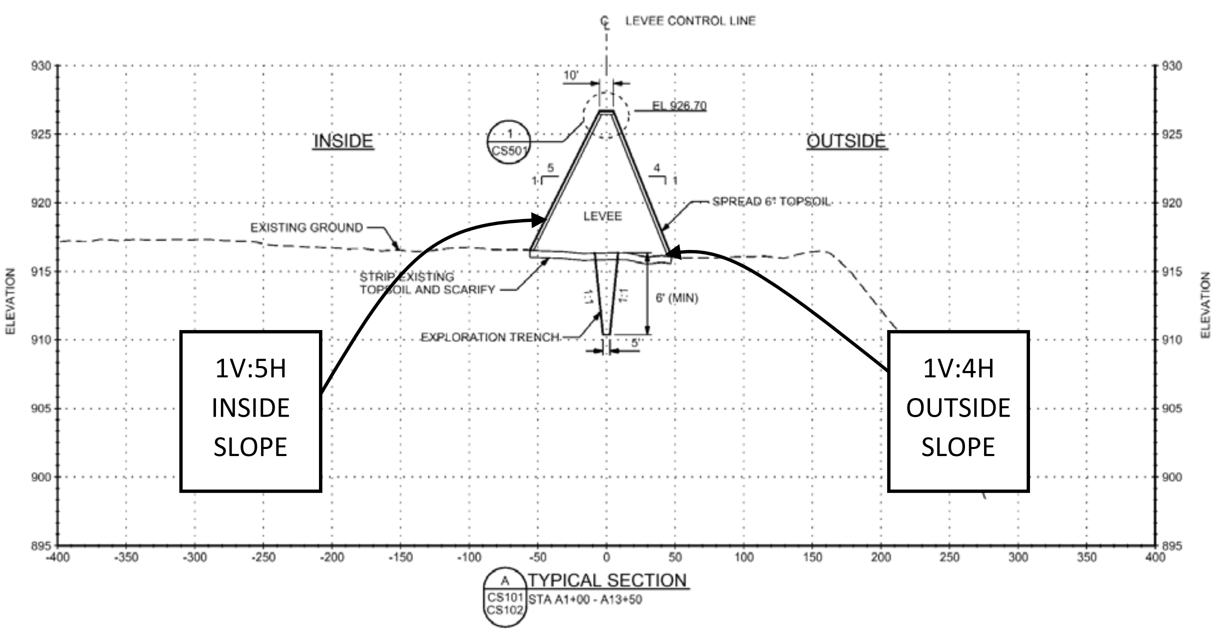

Figure 34 – Preliminary Plan Drawing of a Typical Section of the OHB Levee.

5.2 Wave Runup Method

Wave runup and rundown on impermeable slopes (Battjes 1974) [CEM-VI-5]

$$\frac{R_{ui\%}}{H_s} = (A\xi + C)\gamma_r\gamma_b\gamma_h\gamma_\beta$$

$R_{ui\%}$ = runup level exceeded by *i* percent of incident waves

H_s = significant wave height, calculated in STWAVE – equivalent to H_{mo}

ξ = surf-similarity parameter - ξ_{op} based on the peak wave period

A, C = coefficients dependent on ξ and *i* but related to the reference case of a smooth, straight impermeable slope, long-crested head-on waves and Rayleigh-distributed wave heights

γ_r = reduction factor for influence of surface roughness (1 for smooth slopes)

γ_b = reduction factor for influence of a berm (1 for non-bermed slopes)

γ_h = reduction factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution (1 for Rayleigh distributed waves)

γ_β = factor for influence of angle of incidence *β* of the waves (1 for head-on long-crested waves, i.e., *β* = 0). The influence of directional spreading in short-crested waves is included in the factor as well.

= 1 – 0.0022 *β* (for short-crested waves)

Table 4 – Coefficients for Wave Runup Equation (Ahrens 1981) [CEM-VI-5]

Table VI-5-2 Coefficients for Runup of Long-Crested Irregular Waves on Smooth Impermeable Slopes				
ξ	*Ru*	ξ-Limits	*A*	*C*
ξ_{op}	$R_{u2\%}$	**$\xi_p \le 2.5$***	**1.6**	**0**
		$2.5 < \xi_p < 9$	-0.2	4.5
	R_{us}	$\xi_p \le 2.0$	1.35	0
		$2.0 < \xi_p < 9$	-0.15	3.0

** For all wave conditions near the levee, the surf-similarity parameter is below 2.5*

Wave height and period results from STWAVE and wind setup results from ADH, both at the toe of the levee, have been summarized at an average interval of 50 ft along the levee alignment. The angle of incidence (*β*) was also calculated from the STWAVE results at each location to determine the reduction factor γ_β. Using these model results at each station, and assuming the reduction factors for roughness, berm influence, and wave height distribution are equal to 1.0, the levee height was calculated as the 2% wave runup height ($R_{u2\%}$).

The following figures (Figure 35 to Figure 38) show the required levee height based on wave runup for the entire OHB levee. The first three figures depict the wave runup height radial around OHB, that is, the runup at the northeast corner of OHB would be represented by looking at the northeast direction on the figure. The fourth figure depicts the maximum wave runup for any wind direction around the levee in a typical x-y chart form, considering three different levee slopes.

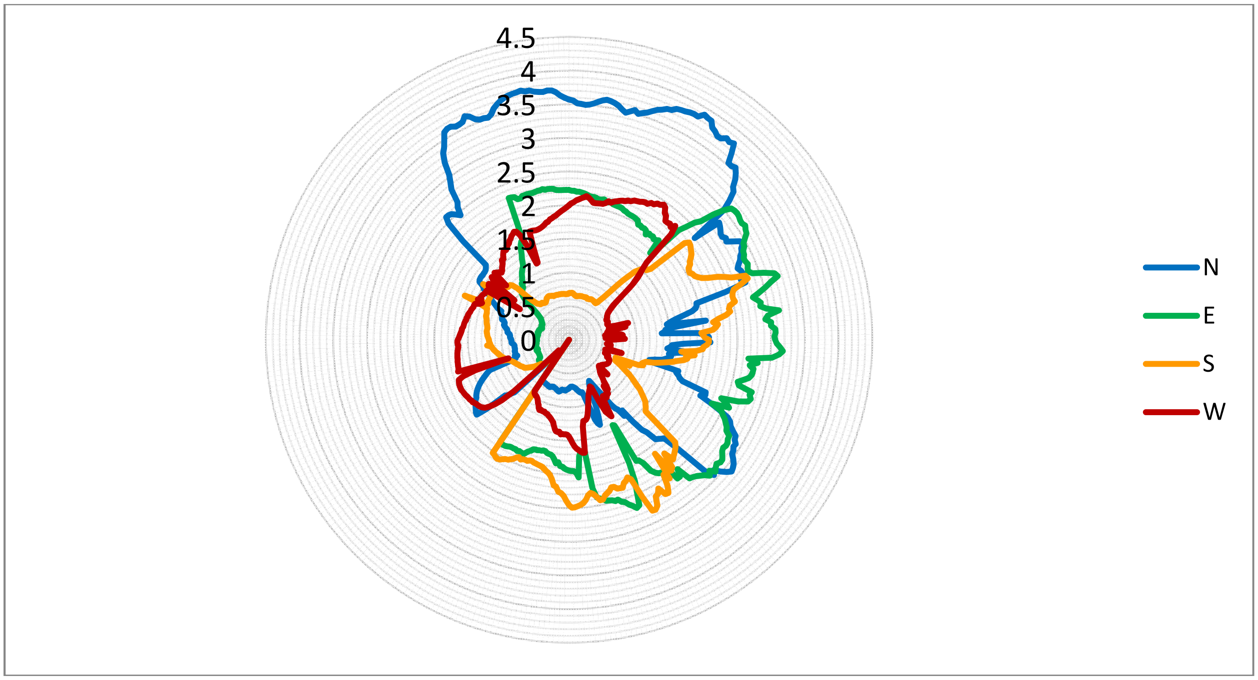

Figure 35 – Maximum 2% wave runup (ft) around OHB levee (1V:4H) for max cardinal direction winds

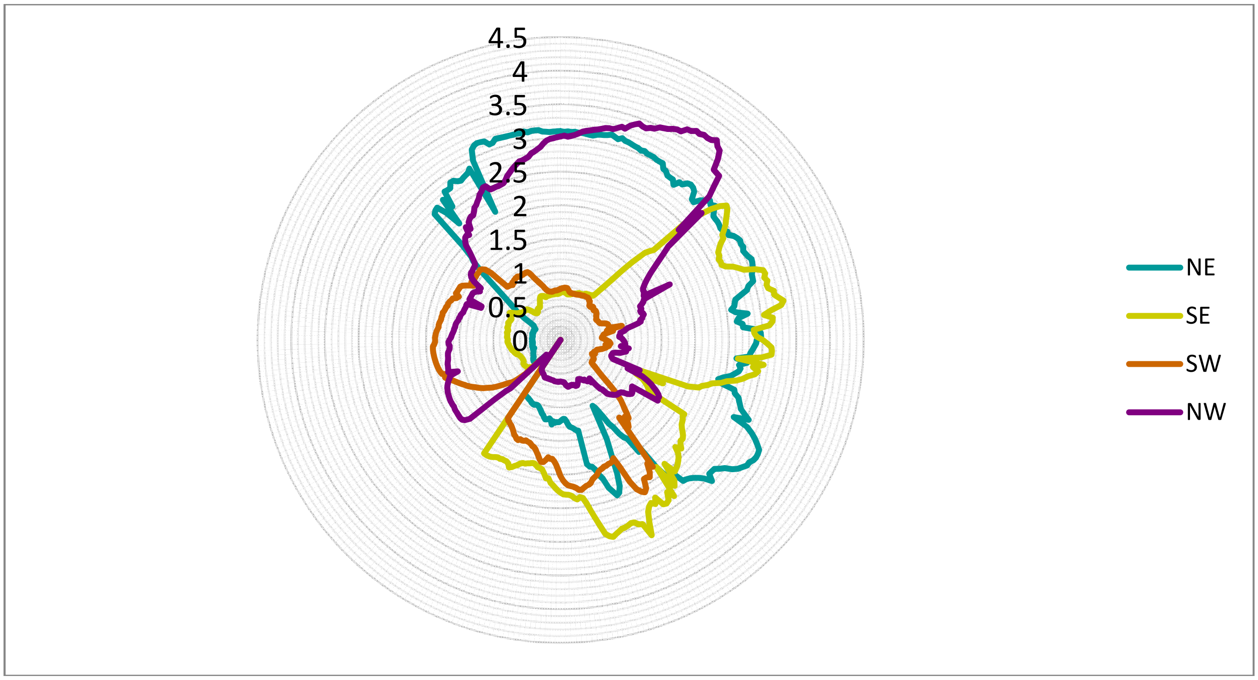

Figure 36 – Maximum 2% wave runup (ft) around OHB levee (1V:4H) for max ordinal direction winds

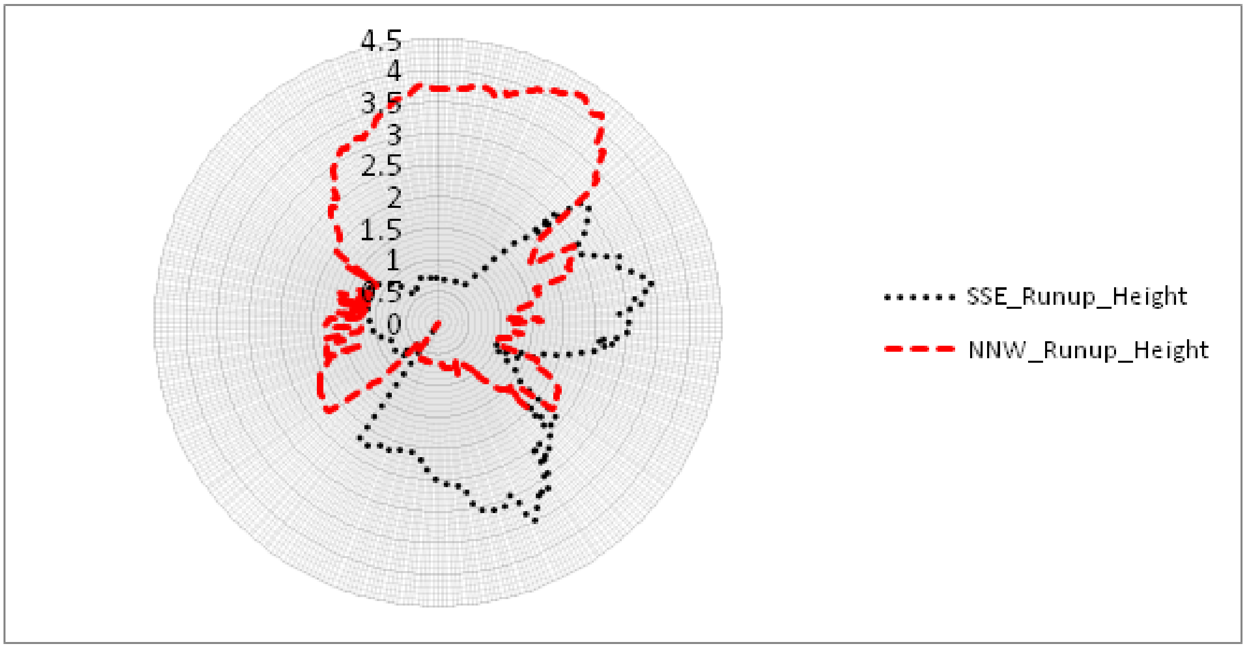

Figure 37 – Maximum 2% wave runup (ft) around OHB levee (1V:4H) for 50-yr return period winds

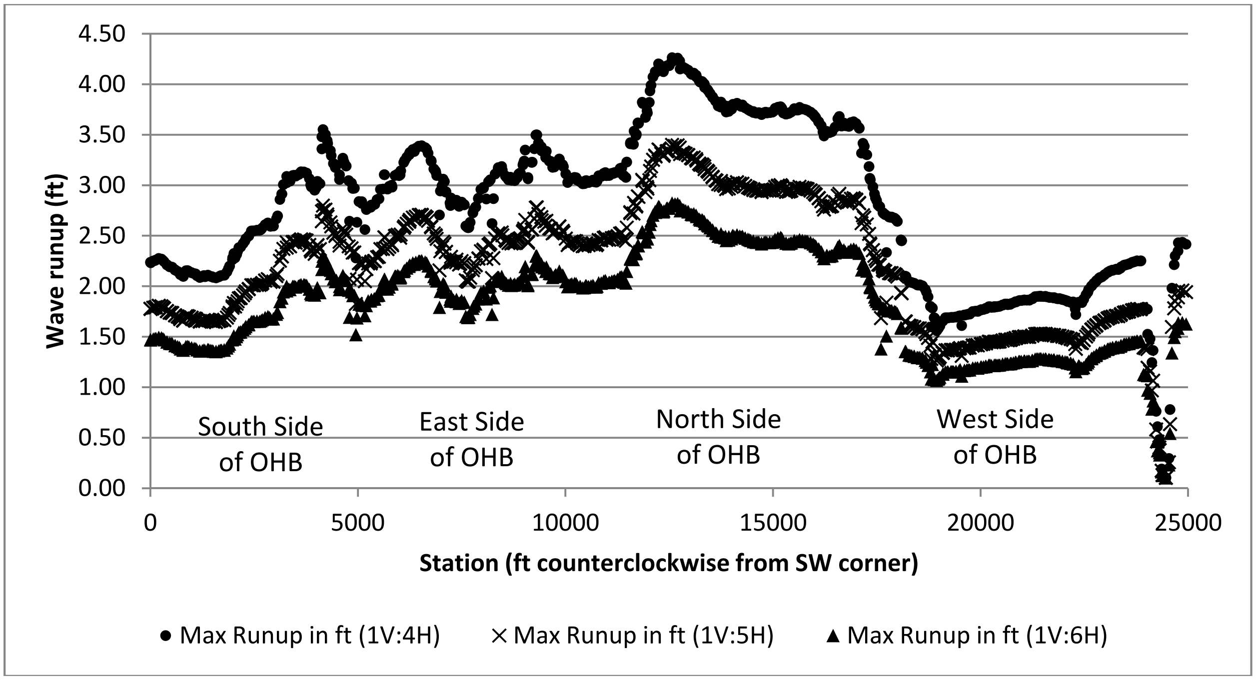

Figure 38 – Maximum 2% wave runup (ft) for all wind directions

5.3 Allowable Overtopping Method

<u>Overtopping Formula by van der Meer and Janssen (1995) [CEM-VI-5]</u>

For "straight and bermed impermeable slopes including influence of surface roughness, shallow foreshore, oblique, and short-crested waves" (CEM, 2013).

q	= overtopping rate (units in volume/time/length)
R_c	= levee height above the SWL (units in length)
$\tan\alpha$	= slope of the levee
s_{op}	= wave steepness

$For\ \xi_{op} < 2$

$$\frac{q}{\sqrt{g\,H_s^3}}\sqrt{\frac{s_{op}}{\tan\alpha}} = 0.06\exp\left(-5.2\frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{\tan\alpha}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right)$$

$$application\ range{:}\ 0.3 < \frac{R_c}{H_s}\frac{\sqrt{s_{op}}}{\tan\alpha}\frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta} < 2$$

$For\ \xi_{op} > 2$

$$\frac{q}{\sqrt{g\,H_s^3}} = 0.2\ \exp\left(-2.6\ \frac{R_c}{H_s}\ \frac{1}{\gamma_r\gamma_b\gamma_h\gamma_\beta}\right)$$

The reduction factors are:

γ_r	= reduction factor for influence of surface roughness (1 for smooth slopes)
γ_b	= reduction factor for influence of a berm (1 for non-bermed slopes)
γ_h	= reduction factor for influence of shallow-water conditions where the wave height distribution deviates from the Rayleigh distribution (1 for Rayleigh distributed waves)
γ_β	= 1 – 0.0033 β (for short-crested waves)

** The minimum value of any combination of the γ-factors is 0.5.*

Wave height and period results from STWAVE and wind setup results from ADH, both at the toe of the levee, have been summarized at an average interval of 50 ft along the levee alignment. The angle of incidence (β) was also calculated from STWAVE results at each location to determine the reduction factor γ_β. Using these model results at each station, and assuming the reduction factors for roughness, berm influence, and wave height distribution are equal to 1.0, the levee height (R_c) was calculated so that the overtopping rate was limited to 0.01 cfs/ft (0.001 m^3/s/m).

The following figures (Figure 39 to Figure 42) show the required levee height to limit wave overtopping for the entire OHB levee. The first three figures depict the levee height radially around OHB, that is, the required levee height at the northeast corner of OHB would be represented by looking at the northeast direction on the figure. The fourth figure depicts the maximums of all the minimum required heights around the levee in a typical x-y chart form, considering three different levee slopes.

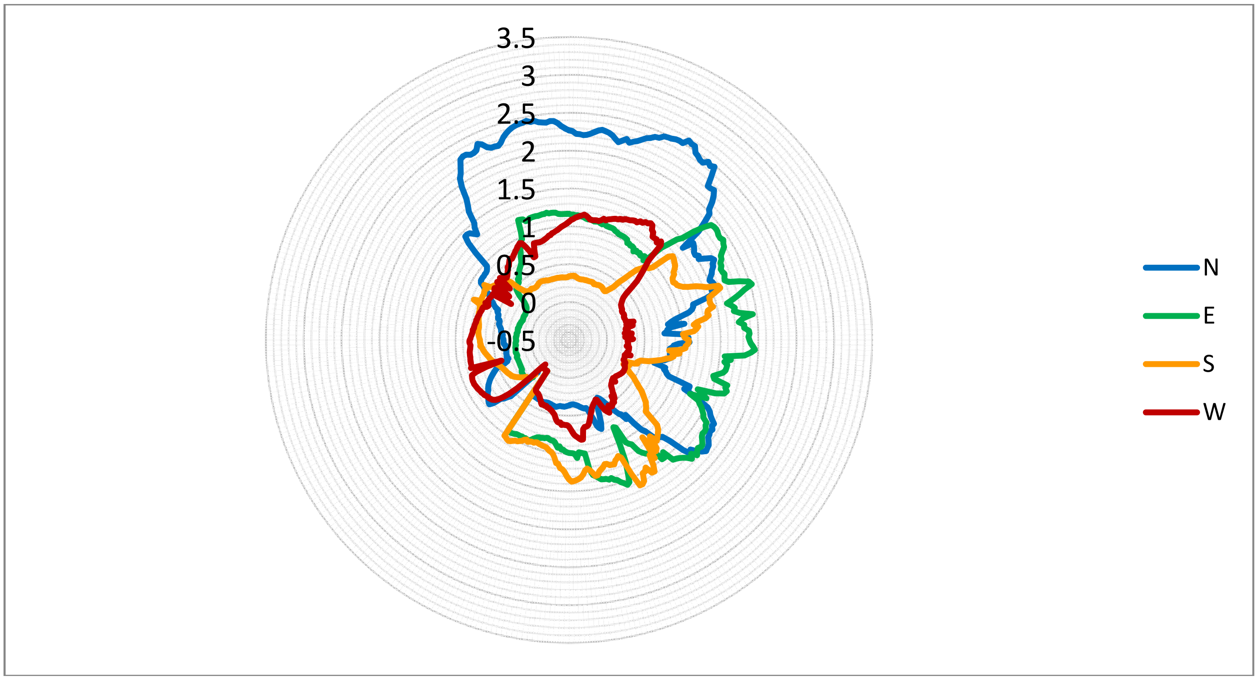

Figure 39 – Min. required height (ft) to limit wave overtopping around OHB Levee (1V:4H) – max cardinal direction winds

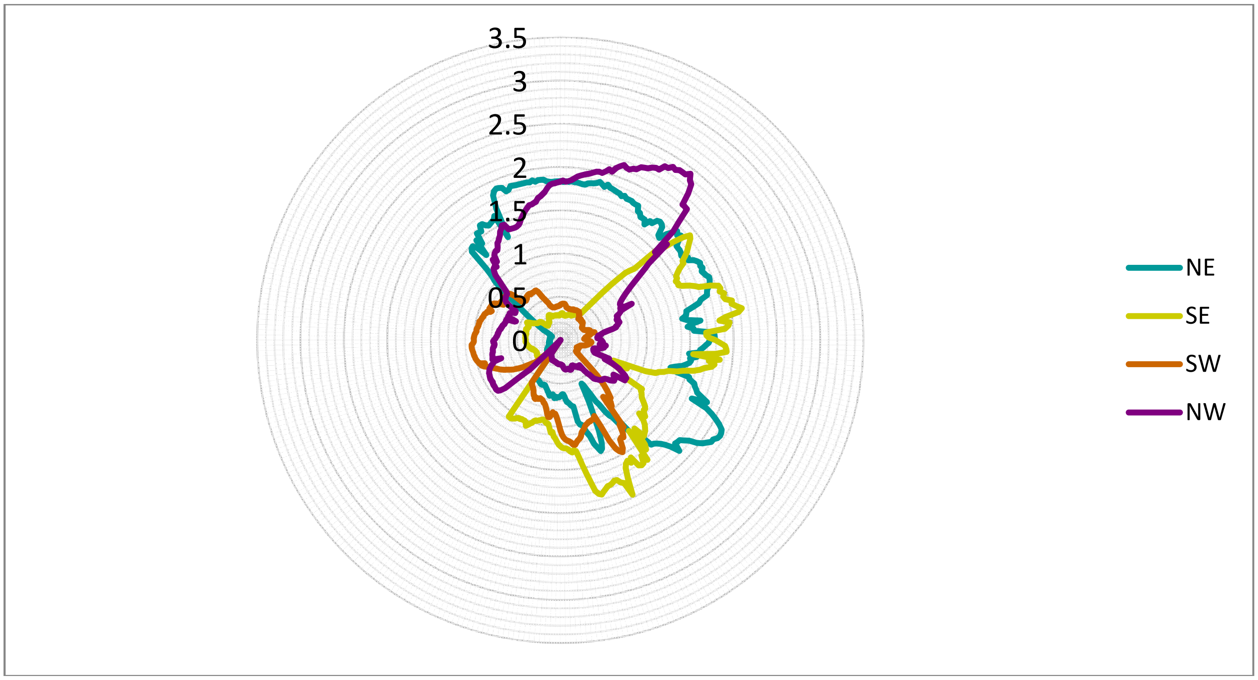

Figure 40 – Min. required height (ft) to limit wave overtopping around OHB Levee (1V:4H) – max ordinal direction winds

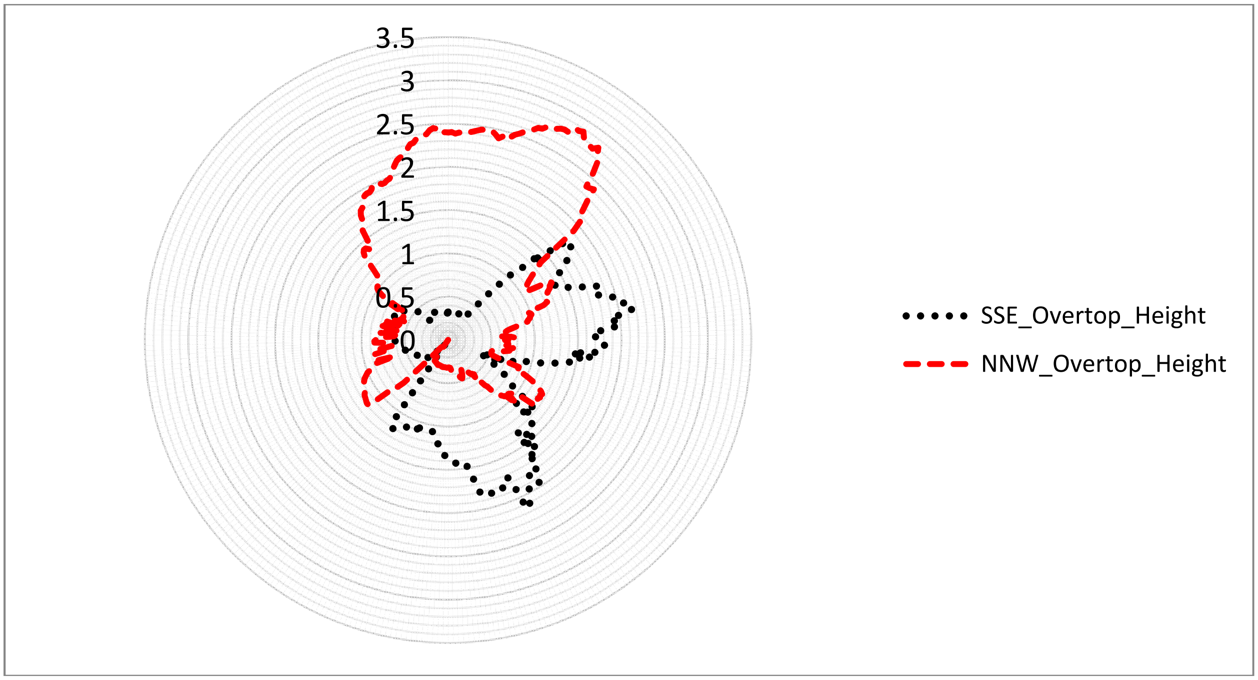

Figure 41 – Min. required height (ft) to limit wave overtopping around OHB Levee (1V:4H) – 50-yr return period winds

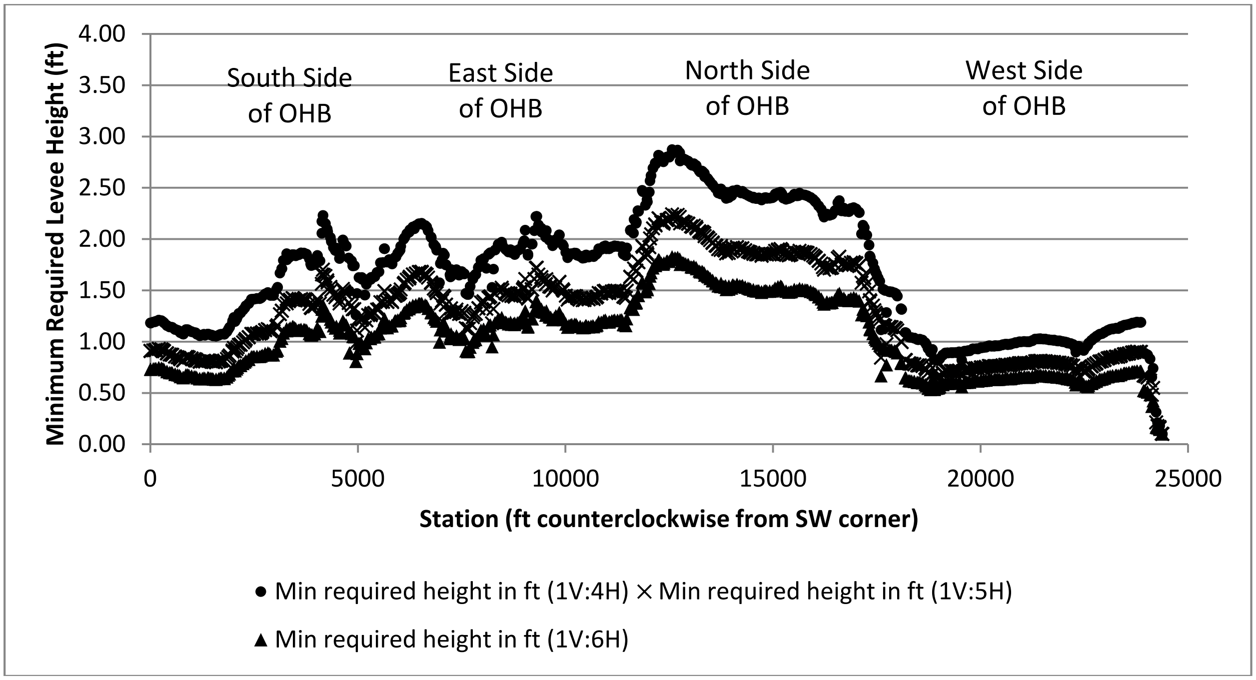

Figure 42 – Minimum required height (ft) to limit wave overtopping around OHB Levee, all wind directions considered

5.4 Conclusions

If the OHB Ring Levee was set at 4 ft above the SWL of 922.5 ft and the exterior levee slope was constructed at 1V:4H, the maximum calculated wave overtopping for the most extreme event simulated (NNW Wind at 55.5 mi/hr) would be:

- *Maximum average overtopping rate = 0.00203 cfs/ft*
- *Maximum total average overtopping = 1.92 cfs for the entire length of levee*

The maximum average overtopping rate is well below the allowable overtopping (0.01 cfs/ft) and the total average overtopping is not a concern for interior flood control.

If the OHB Ring Levee was set at 3 ft above the SWL of 922.5 ft and the exterior levee slope was constructed at 1V:4H, the maximum calculated wave overtopping for the most extreme event simulated (NNW Wind at 55.5 mi/hr) would be:

- *Maximum average overtopping rate = 0.01 cfs/ft*
- *Maximum total average overtopping = 15.85 cfs for the entire length of levee*

The maximum average overtopping rate is right at the allowable overtopping (0.01 cfs/ft) and the total average overtopping could be accommodated by interior flood control (ditches, ponds, and pump stations).

If the design were just to satisfy the allowable overtopping criteria, a 3 ft levee height with 1V:4H slopes would be minimally adequate. A 4 ft levee height with 1V:4H would be more than adequate to limit wave overtopping, however, one location in the northeast corner of OHB would be exceeded by more than 4 ft of wave runup (see Figure 38). If the levee slope at this location were flattened to 1V:5H, the wave runup would not exceed 4 ft. Because the 4 ft levee height (with modified northeast levee slope) can accommodate both wave runup and wave overtopping criteria and because it exceeds the elevation of the overflow spillway by 3.5 ft rather than 2.5 ft, the preferred levee height for hydraulic criteria will be 4 ft.

5.5 Levee System Evaluation Considerations

The purpose of this analysis is to assess the top elevation of the OHB ring levee with regard to EC 1110-2-6067 requirements. All parties involved ultimately want assurance that the ring levee will be 'accredited' by the Federal Emergency Management Agency (FEMA). Two key definitions that are necessary for this discussion are:

- <u>Annual Exceedance Probability (AEP)</u> – the probability that a flood will equal or exceed a given elevation or discharge in any given year. A 1% AEP event has a 1 in 100 chance of being equaled or exceeded in any given year. A 1% AEP event is typically referred to as a "100-yr" event since the long-term expected value for the recurrence interval is 100 years. AEP is equivalent to "percent-annual-chance" and Annual Chance Exceedance (ACE).
- <u>Conditional Non-Exceedance Probability (CNP)</u> – the uncertainty about the AEP stage or discharge, due to the uncertainty in discharge-probability and stage-discharge estimates. The 1%

AEP typically refers to the 1% AEP at a 50% CNP as it has an equal chance of being exceeded or not exceeded based on uncertainties. Sometimes the word "assurance" is used instead of CNP.

EC 1110-2-6067 Top of Levee Elevation Critera:

1) The top of levee must be a minimum of 2 ft above the 1% AEP stage at a 50% CNP.
2) If the top of levee is *less* than 3 ft above the 1% AEP stage at a 50% CNP, the CNP for the top of the levee must be equal to or greater than **95%** for the 1% AEP stage.
3) If the top of levee is *greater* than 3 ft above the 1% AEP stage at a 50% CNP, the CNP for the top of the levee must be equal to or greater than **90%** for the 1% AEP stage.
4) If the top of levee height meets all three of the preceding criteria, it still must be higher than the wind-induced level for the 1% AEP stage with consideration for wave runup or allowable overtopping.

In typical riverine levees, there is uncertainty in the discharge-frequency curve and the stage discharge relationship, both having the potential to greatly reduce the CNP of the 1% AEP stage. Given the capacity and redundancy of outflow structures, it is known that a levee 2 ft above the 50% CNP 1% AEP stage has a CNP of at least 95% as long as waves are not considered. Therefore the wind/wave considerations outweigh hydrology/hydraulic considerations.

5.6 Flood-Side Erosion Resistance of the Levee

One additional concern for the performance of a levee in relation to wind and waves is the ability of the levee to resist erosion from wave attack. Results from full-scale testing of grass-covered clay levee slopes done by Seijffert and Verheij (1998) summarized in the ERDC/CHL Technical Report 10-7 (Hughes, 2010), states the following:

a) Waves up to 0.5 m (1.6 ft) caused no damage to grass covers
b) Waves in the range of 0.5 – 1.5 m (1.6 – 4.9 ft) with a duration between 6 and 24 hours generally did not cause severe damage
c) Waves greater than 1.5 m (4.9 ft) will likely cause severe erosion

Peak wave heights of around 3 ft, as modeled using STWAVE, would only be anticipated to last around one hour. Even if these peak wave heights lasted an order of magnitude longer than expected (~10 hours), the technical report states that they would "generally not cause severe damage." Additional checks using empirical equations developed by Seijffert and Verheij indicate that wave heights of 3 ft lasting for 6 hours or wave heights of 1.6 ft lasting for 24 hours would cause less than a few inches of erosion of the levee.

5.7 Wind Induced Ice Forces on the Levee

The use of the staging area will occur after the initial spring warm-up has melted enough snow to raise flows on the Red and Wild Rice Rivers to above flood stage. Historically, there are some instances where a cold-snap has occurred after a warm-up period, causing low temperature conditions that can lead to the re-formation of ice. The year 1997 is one of these instances, where five consecutive days with high temperatures below freezing occurred after an initial warm-up and ice formation occurred throughout the basin. An analysis of potential ice growth and wind driven ice forces for the 1997 event has been performed as part of the design of the OHB levee.

By extracting the daily temperature data in 1997 from the weather station used for the wind data, an estimation of thermal ice growth can be made using the following equation from the Ice Engineering reference (USACE, 1999):

$$h_j = \alpha\, U_j^{\frac{1}{2}}$$

where:	h_j	=	calculated ice thickness on day *j*
	U_j	=	Accumulated Freezing Degree-Days (AFDDs) recorded between the onset of freezeup (day 1) and day *j*
	α	=	Ice cover condition, 0.47 used for other FMM analyses

A plot of the potential ice growth and recorded daily temperatures can be seen in the following figure, Figure 43.

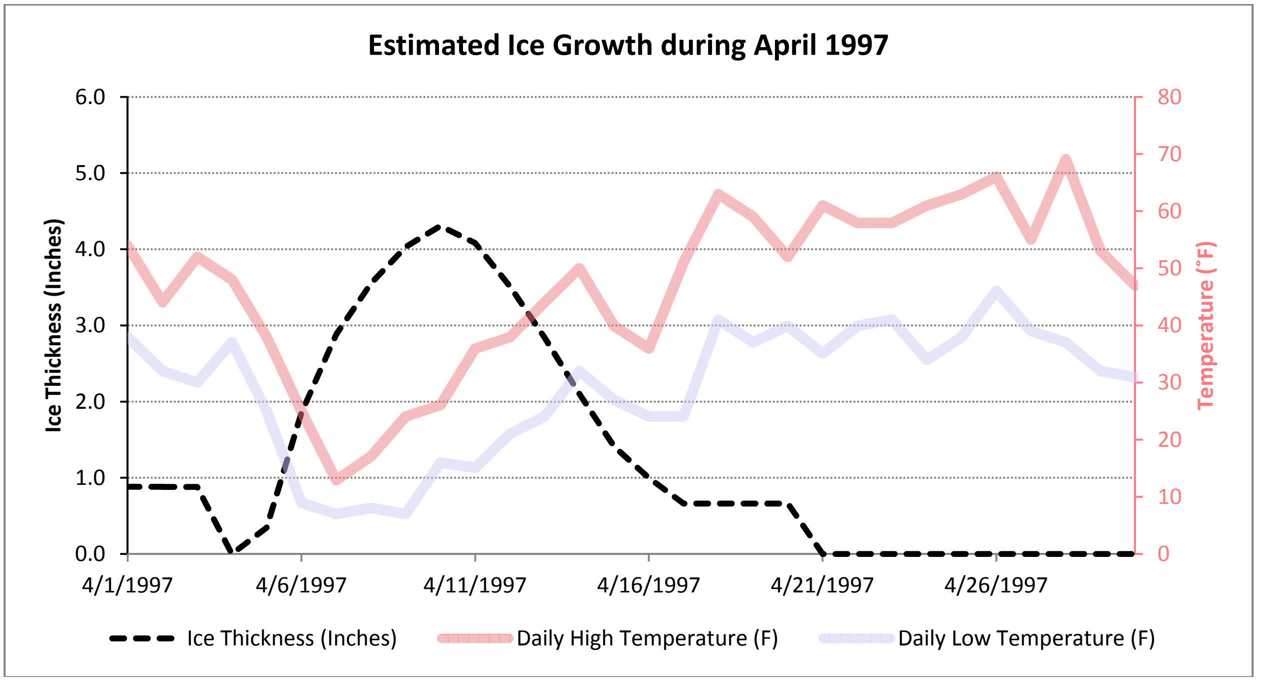

Figure 43 – Estimated Ice Growth during April 1997

The levee has a top width of 100 feet and will be at least 46 feet wide at the 922.5 water surface elevation. Ice that would form and be pushed by wind during project operation will not cause significant damage on an embankment of this size with 1V:4H or flatter slopes. Some turf damage may occur, but the ice will ride up the 1V:4H or flatter slope before it would remove a significant portion of the levee cross-section. It's possible that some ice could even make it to the top of the levee, but the volume of water in the ice would not be enough to cause flooding issues in the interior area, given the ditch that will exist at the interior toe of the levee. The infrequency of such an event warrants simply fixing any levee damage as needed. Interior drainage pipe outlets will not be allowed on the north side of the ring levee due to the wind-driven ice forces that could cause problems for unprotected culvert ends and flap gates.

5.8 Final Recommendations

- Top of Oxbow/Hickson/Bakke Ring Levee Elevation should be set 4 feet above pool elevation of 922.5. This elevation does not include required overbuild.
- Include an overtopping segment that would have controlled overtopping in a Probable Maximum Flood event. The height would be 3.5 feet above pool elevation of 922.5, or 0.5 ft lower than the Top of Levee elevation. The location and length of the segment is still to be determined.
- Erosion Protection: Topsoil and seed only with the intent of establishing and maintaining vegetation on the clay levee.
- Exterior Side Slopes of 1V:4H are acceptable with exception of the NE corner where wave heights would be greatest and the exterior slope should be flattened to 1V:5H.
- Interior Side Slopes of 1V:5H will be used per Local Sponsor request (1V:4H is acceptable from a COE perspective).
- Vegetation Free Zone (VFZ): Extends minimum of 15' from the toe of the levee. Local drainage ditches will be placed outside of the VFZ.
- Use EM 1110-2-1913, "Design and Construction of Levees".

6 REFERENCES

Adaptive Hydraulics Modeling (ADH). Accessed 2013. http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=Links;139

EurOtop Manual. 2007. Overtopping Manual; Wave Overtopping of Sea Defences and Related Structures – Assessment Manual. UK: N.W.H. Allsop, T. Pullen, T. Bruce. NL: J.W. van der Meer. DE: H. Schüttrumpf, A. Kortenhaus. www.overtopping-manual.com.

Gumbel, Emil Julius. Statistical theory of extreme values and some practical applications. U.S. Govt. Print Office, 1954.

Holmes, J.D.. "Prediction of design wind speeds". Accessed 2013. www.hurricaneengineering.lsu.edu/CourseMat/03Lect4DesignWind.ppt

Hughes, Steven A.. "Flood-Side Wave Erosion of Earthen Levees: Present State of Knowledge and Assessment of Armoring Necessity". ERDC/CHL TR-10-7. U.S. Army Corps of Engineers, Coastal and Hydraulics Laboratory, Vicksburg, MS, August 2010.

Mapping of areas protected by levee systems, 44 CFR 65.10 (2010).

NOAA. National Climatic Data Center (NCDC). Daily Summaries Station Details. Fargo Hector International Airport, ND US. Accessed 2013. http://www.ncdc.noaa.gov/cdo-web/datasets/GHCND/stations/GHCND:USW00014914/detail

STWAVE – Steady State Spectral Wave. Accessed 2013. http://chl.erdc.usace.army.mil/chl.aspx?p=s&a=software;9

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

U.S. Army Corps of Engineers. Ice Engineering. Engineer Manual 1110-2-1612, U.S. Army Corps of Engineers, Washington, D.C. 1999.

U.S. Army Corps of Engineers. Engineering Circular No. 1110-2-6067 (EC-1110-2-6067). *USACE Process for the National Flood Insurance Program (NFIP) Levee System Evaluation.* Washington, DC. August 2010.