FINAL REPORT
DEVELOPMENT OF METHODS FOR IMPROVING LEVELS I AND II MET/OCEAN PARAMETER PREDICTIONS

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SI (MODERN METRIC) CONVERSION FACTORS (from FHWA)
APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9	Celsius	^oC

		or (F-32)/1.8		
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS TO SI UNITS

SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
LENGTH				
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
MASS				

g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
TEMPERATURE (exact degrees)				
^oC	Celsius	1.8C+32	Fahrenheit	^oF
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
ILLUMINATION				
lx	lux	0.0929	foot-candles	fc
cd/m^2	$candela/m^2$	0.2919	foot-Lamberts	fl
SYMBOL	WHEN YOU KNOW	MULTIPLY BY	TO FIND	SYMBOL
FORCE and PRESSURE or STRESS				
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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16. Abstract

The prediction of storm surge and wave forces and moments on bridges requires knowledge of design (100-year) water levels and wave heights and periods (met/ocean conditions) as well as bridge dimensions, elevation, orientation, etc. The American Association of State Highway and Transportation Officials (AASHTO) code, *Guide Specification for Bridges Vulnerable to Coastal Storms*, describes three levels of analysis (Levels I, II and III) for obtaining the met/ocean conditions. By necessity Levels I and II analyses over-estimate the combined water levels and wave heights. The objective of this study was to develop a method for adjusting Level I results to bring them more in line with those from a Level III analysis while maintaining a reasonable level of over prediction. Data and information obtained in an FDOT sponsored Pilot Study in the Tampa-Saint Petersburg, Florida area, where Levels I, II and III analyses were performed, were used in the development of Level I modifiers for both design water level and wave heights. The modifiers are in terms of readily available water basin geometric parameters. Extracting this information does, however, require some level of understanding of coastal hydraulics/processes and should be performed by someone knowledgeable in this area. The modified results will retain a degree of over-prediction provided the rules for application of the modifiers, outlined in the report, are followed.

17. Key Word	18. Distribution Statement
Level I Analysis, Met/Ocean Modifiers, Storm Surge, Surge/Wave Forces,	No Restrictions

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Unclassified	Unclassified	54	

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Executive Summary

The prediction of storm surge and wave forces and moments on bridges requires knowledge of design (100-year) water levels and wave heights and periods (met/ocean conditions) as well as bridge dimensions, elevation, orientation, etc. The American Association of State Highway and Transportation Officials (AASHTO 2008) code, *Guide Specification for Bridges Vulnerable to Coastal Storms,* describes three levels of analysis (Levels I, II and III) for obtaining design met/ocean conditions. By necessity Levels I and II analyses provide very conservative results.

Ocean Engineering Associates, Inc. (OEA) was contracted by the Florida Department of Transportation (FDOT) Research Center to develop methods for adjusting Level I results to bring them more in line with those from a Level III analysis, while maintaining a reasonable level of conservatism. A Level I analysis uses existing data and information regarding water depths, 100-year wind speeds and storm surge elevations, etc., at the site of interest. Wave heights and periods are computed using empirical equations in the U.S. Army Corps of Engineers (USACOE) Shore Protection Manual (SPM). Local wind setup/set-down was computed using a methodology developed by OEA for FDOT. There is no way to take phasing of these quantities into consideration, thus the maximum values are assumed to occur at the same time. That is, the maximum storm surge is assumed to occur at the same time as the local wind setup and the maximum wave heights. In general this results in overly conservative met/ocean conditions and therefore predicted forces and moments on the bridge. A Level III analysis eliminates these issues and produces much more accurate results. OEA conducted an FDOT sponsored pilot study in the Tampa-Saint Petersburg, Florida area (FDOT District 7) where Levels I, II and III met/ocean analyses were performed. Information/data from the Pilot Study results were used in this study to develop modifiers for Level I predictions to bring them more in line with those from a Level III analysis. The modifiers are in terms of quantities easily obtained from nautical charts and/or maps of the area. Extracting this information does, however, require some level of understanding of coastal hydraulics/processes and should be performed by someone knowledgeable in this field.

The approach taken in this study was to investigate the correlation between the ratio of Level III to Level I design water elevation at each bridge site and a number of quantities associated with the overall and local water body geometry. The same approach was taken for wave height with the ratio Level III to Level I wave height. Better correlations were found for water elevation than for wave height but both resulted in a method for reducing the Level I values. The modified values of design water level and wave height will still be conservative provided the rules for their application are followed.

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1. Introduction

The recent failure of several bridges in Florida, Mississippi, and Louisiana during hurricanes Ivan and Katrina illustrated the importance of including wave loading in the design of bridge superstructures. Wave loading occurs during hurricanes, when storm surge and wind setup elevates the water surface placing the bridge superstructure within reach of the waves. It should be pointed out that storm surge in this report is defined as the surge that is created offshore and propagates into bay systems. Storm surge, wind setup and wave setup all add to create the total water surface elevation. The combination of elevated water level and waves can exert tremendous horizontal and vertical (uplift) forces and moments on the bridge. Estimating these forces for design or vulnerability analysis requires a description of the 100-year met/ocean conditions (wind speed, storm surge elevation, local wind setup/set-down, and wave conditions). The methods for estimating met/ocean conditions vary in both complexity and accuracy.

1.1 Problem Statement

The *American Association of State Highway and Transportation Officials Guide Specifications for Bridges Vulnerable to Coastal Storms* — referred to herein as the AASHTO Codes — provides three methods for establishing the 100-year met/ocean conditions — Levels I, II, and III. The effort required as well as the accuracy increases with the level of analysis. A Level I analysis uses readily available data for design storm surge and wind speed and empirical equations for computing local wind setup and wave height and period. What is referred to as a Level II analysis can cover a relatively wide range of analysis techniques from slight improvements over a Level I to computer modeling of waves and/or storm surge. A Level III analysis is more sophisticated and requires more effort but produces greater accuracy and significantly more useful information.

Application of the Level III analysis is typically cost effective for urban areas where a large number of bridges can be evaluated with one study — reducing the cost per bridge. In rural areas, where only a few bridges are present, the Level III analysis may not be cost effective. Unfortunately, the Level I and II results are conservative by design and may produce forces that exceed feasible retrofit options. The conservatism of the Level I and II analysis stems (in part) from the assumption that all the met/ocean parameters (storm surge, wind setup and waves) are in phase, i.e., their maximum values occur at the same time. In some cases this may be correct; in most cases, however, it produces results that exceed 100-year probability conditions (i.e., 1% chance of occurrence each year). Phasing of the met/ocean parameters is a function of the local geophysical parameters of the waterway and the bridge's location in the water body. The local geophysical parameters include the fetch length and orientation, the size of the water body (tidal prism), the distance from the inlet, the cross-sectional area of the inlet, etc.

Ocean Engineering Associates Incorporated (OEA) recently completed a Florida Department of Transportation (FDOT) sponsored pilot study of the FDOT District 7 (D7) bridges (Dompe, et al. 2010) (Figure 1-1). In that study, OEA applied all three levels of analyses to the FDOT D7 bridges. In addition to producing surge/wave forces for the district's bridges, the study also provided information of the range and degree of conservatism in the Level I and II analysis. Since the beginning of this study new Flood Insurance Studies (FIS) were published, and a new methodology for calculating local wind setup (OEA 2010) was developed since the original Phase I study. As a result, all Phase I predictions were recalculated based on newer data sources and methodologies as a part of this study. All the Phase I results and figures in this report are the updated results and are different from the original Phase I study.

Figure 1-2 and Figure 1-3 show the water surface elevations (WSE) and significant wave heights calculated by Level I and Level III methods. All WSEs reported in this report are above mean sea level (MSL). As these comparisons illustrate, in many cases Level I estimates are almost double those from the Level III analysis.

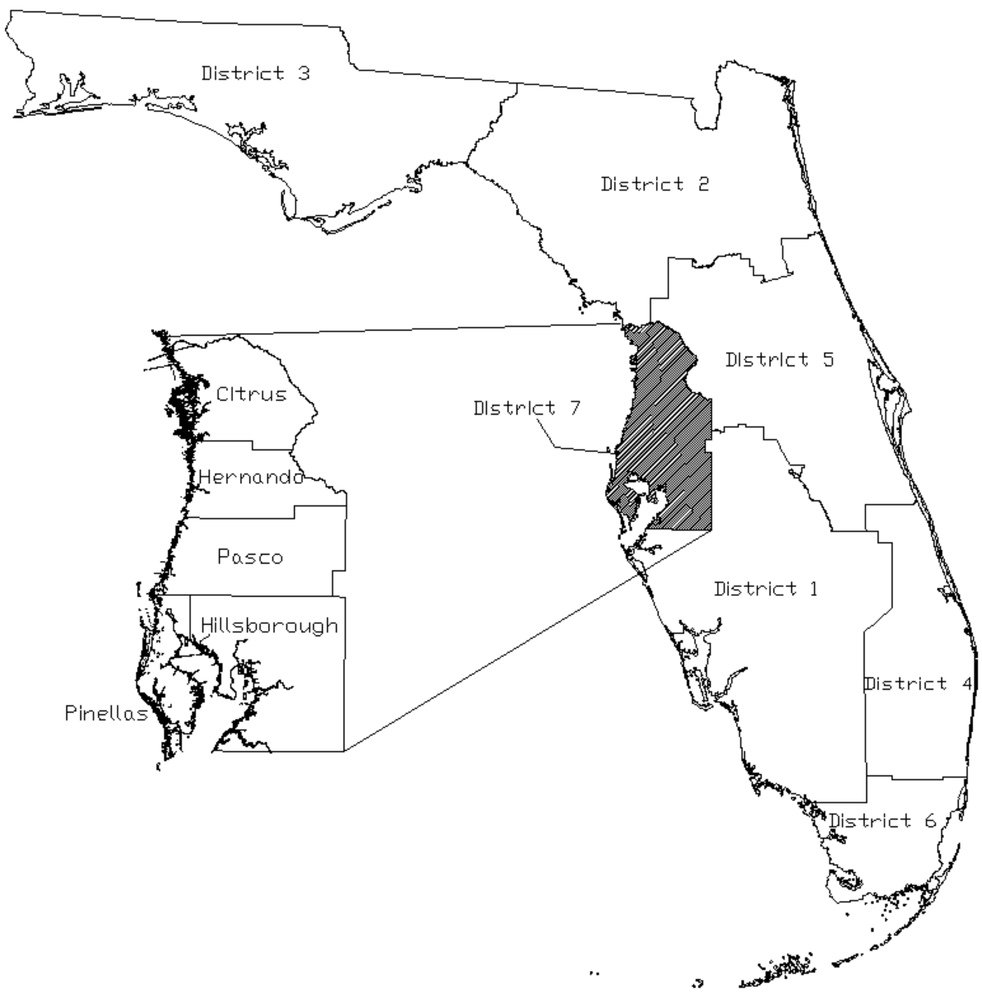

Figure 1-1 Location map for FDOT District 7.

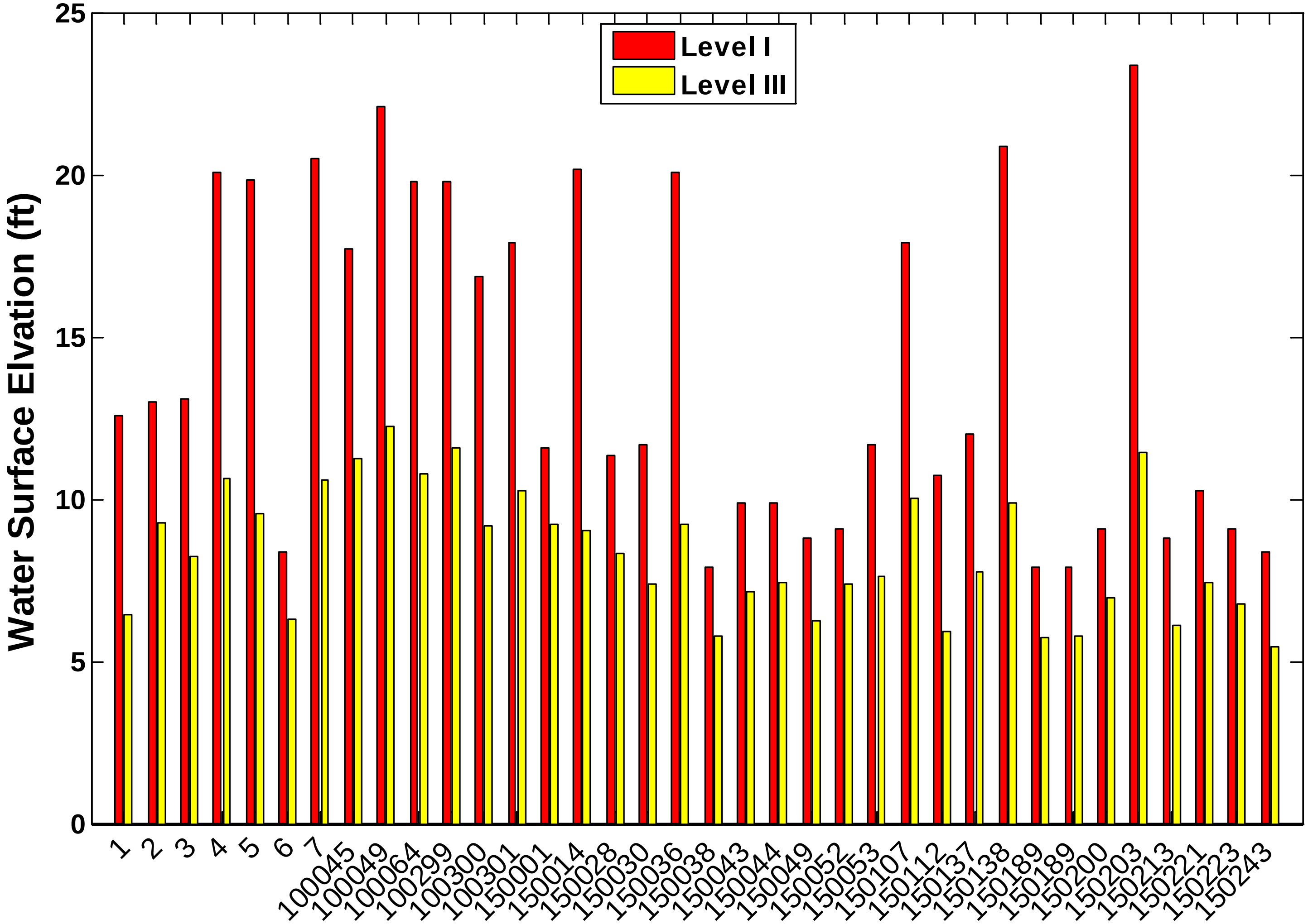

Figure 1-2 Comparison of Level I and Level III predicted design water surface elevations.

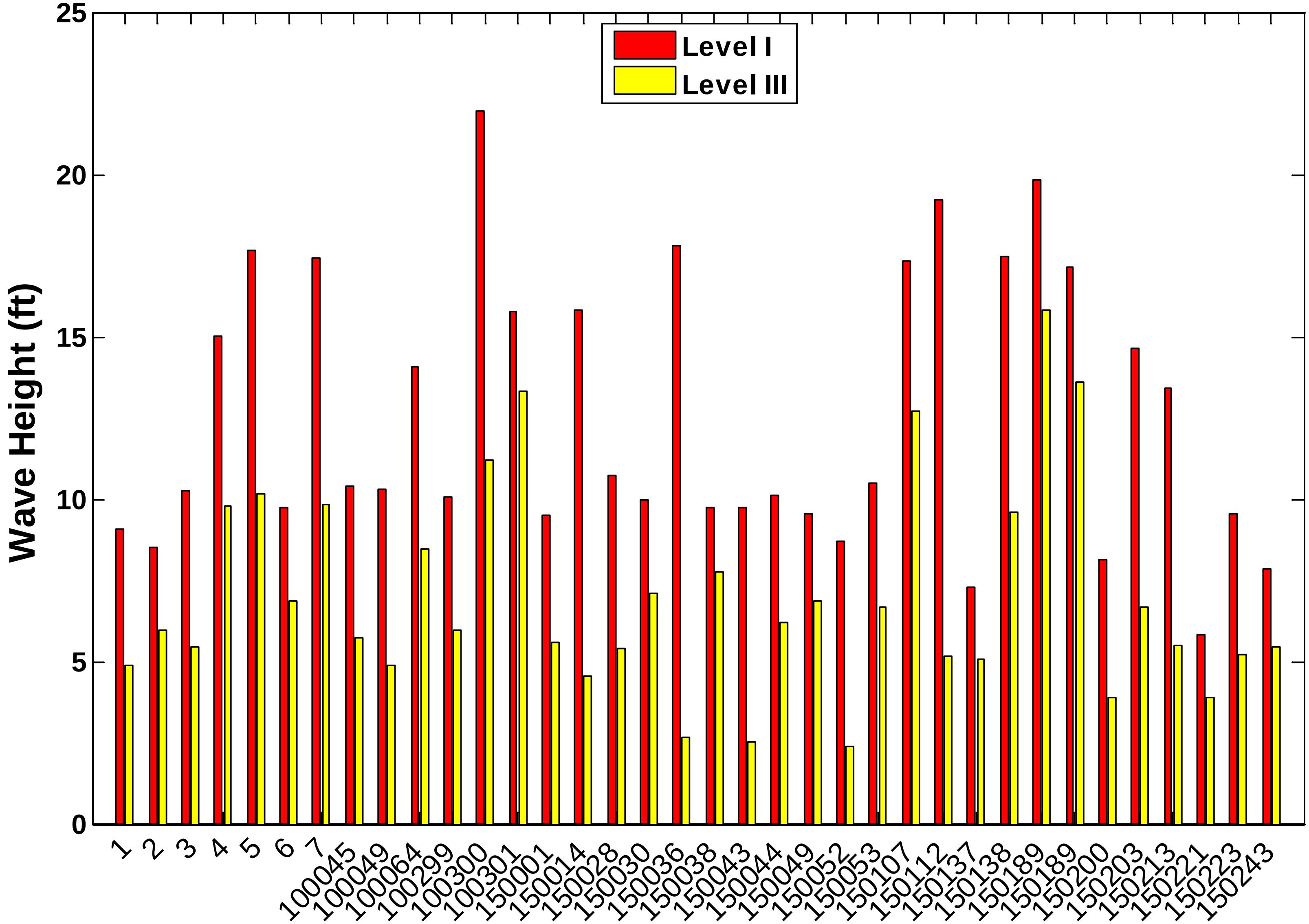

Figure 1-3 Comparison of Level I and Level III predicted design significant wave heights.

1.2 Study Objective

As illustrated above met/ocean data estimated from Level I analyses, in general, produce conservative results. This is due in part to the assumption that the storm surge, local wind setup, and wave heights are in phase, i.e., their maximum values occur simultaneously. Another contributing factor stems from the alignment of the winds. The Levels I and II align the 100-year wind with the longest fetch for both wind setup and the wave parameters. For a bridge crossing a narrow waterway, the maximum wind may never align with the maximum fetch or may align with fetch for only a brief period of time (duration limited). These are two of the contributing factors leading to the conservative met/ocean conditions associated with the Level I and II analyses. There are also differences in FEMA and Level III water surface elevation (WSE) predictions. This is due to differences in methodology and computer model resolution. Level III studies are believed to be more accurate since they use better, more robust numerical models, the spatial resolution is better and they use more recent and detailed bathymetric and topographic information. However, unlike the wind setup, this difference in accuracy does not necessarily mean that the FEMA prediction is more conservative. Although the FEMA mesh is not published, FEMA reports the smallest grid size as 2000 ft. Figure 1-4 illustrates the difference in resolution between the FEMA model and Level III model. In the figure, the inset shows the location of the comparison, the red rectangles show the size of the smallest grid used in the FEMA study, and the darker lines present the ADCIRC grid resolution. As the figure illustrates higher level resolution of the ADCIRC mesh is required to define features such as causeways, inlets and bridge openings.

The objectives of this study are: 1) to continue the work initiated in the previous study to obtain correlations between the differences in Levels I and III water elevation and wave parameter results and the various quantities discussed above and to 2) develop methods for adjusting Level I results to improve their accuracy and bring them more in line with those from the Level III analyses. Due to uncertainties in the accuracy of existing data it is important that the adjustments be such that the final products remain conservative.

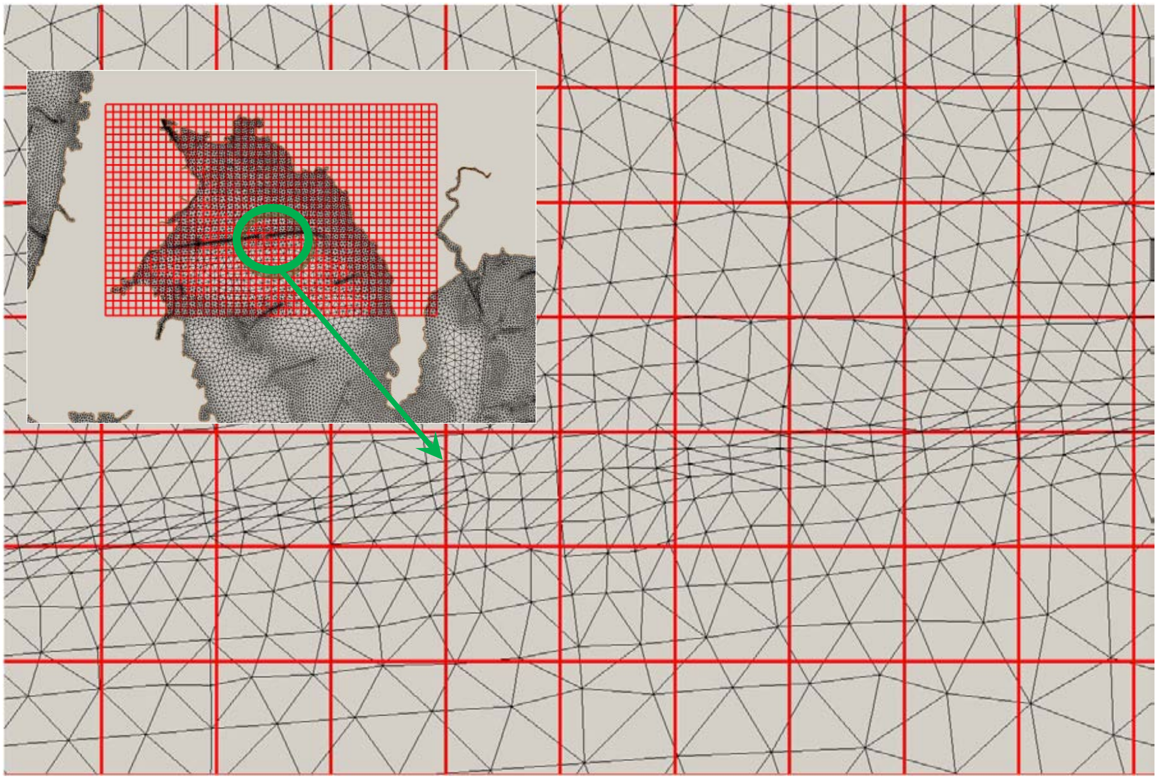

Figure 1-4 Comparison between the Level III ADCIRC grid size (black unstructured) and that used in the FEMA study (red rectangles) for the Pilot Study area. The inset shows the location of the main figure.

1.3 Approach

The study approach used in this study was to: 1) examine the data and information produced by the pilot study (Dompe, Phil, et al. 2010) and determine what additional information had to be extracted from the pilot study solution files, 2) extract the needed information, and 3) analyze the data to determine if correlations between the difference in Level I and Level III water level and wave parameter results and the various site specific parameters discussed above and listed in Table 2-1 exist.

This report presents OEA's application of this approach to develop modifiers designed to improve the accuracy and reduce the overly conservative nature of the Level I met/ocean analysis results. The parameters thought to influence the differences between Levels I and III results are presented and defined in Chapter 2. Chapter 3 compares Level I and Level III wave heights and water surface elevations from the Pilot Study results. The regression analysis that was performed on both the dimensional and the normalized parameters is described in Chapter 4. The verification of the methodology is discussed in Chapter 6 followed by the results of the study and conclusions. Recommendations are presented in Chapter 7.

2. Pertinent Parameters

The phasing of storm surges, local wind setup, and wave heights and periods at a particular site is dependent on a number of water body and storm related parameters. The water body parameters such as the wave and wind setup fetch angles relative to the open coastline, the waterway distance between the bridge of interest and the inlet, etc., are all known quantities and can be obtained from maps and charts of the area. This chapter reports on the efforts to extend work in the Pilot Study to identify the quantities on which the differences between Levels I and III results depend. Table 2-1 lists all the parameters investigated. The non-dimensional groups formed from these parameters are given in Table 2-2. Note that not all of the dimensional parameters are used when creating the non-dimensional groups. This is discussed in section **Error! Reference source not found.**. The values of relevant parameters for all the bridges analyzed are given in Table 3-1 through Table 3-3.

Table 2-1 List of the dimensional parameters investigated.

Wave Fetch Length (ft)
Wave Fetch Band (degrees)
Wave Fetch Orientation (clockwise from Shoreline)
Wave Fetch Length (ft)
Wind Setup Fetch Orientation (clockwise from Shoreline)
Bridge Wind Setup Fetch Length (ft)
Bay Wind Setup Fetch Length (ft)
Wind Setup Fetch Band (degrees)
Distance from Bridge to Inlet (ft)
Inlet Cross-Sectional Area (ft^2)
Flood Volume (ft^3)
Ebb Volume (ft^3)
Tidal Range (ft)
Perpendicular Distance from Bridge to Coast (ft)
Average Water Depth along Fetch(ft)
Level I Wave Height (ft)
Level I Water Surface Elevation (WSE) (ft)
Level I Wind Setup (ft)
Storm Surge (ft)

Table 2-2 List of the non-dimensional groups investigated.

Wind Fetch Band (degrees)
$\frac{\text{Level I WSE}}{\text{Tide Range}}$
$\frac{(\text{Inlet Cross-sectional Area})\ (\text{Tide Range})}{\text{Bay Wind Fetch Length}}$
$\frac{\text{Wind Setup}}{\text{Storm Surge}}$
Wave Fetch Band (degrees)
Wave Fetch Orientation
$\frac{\sqrt{\frac{\text{Water Depth}}{\text{Wave Fetch Length}}}(\text{Storm Forward Velocity})}{(\text{Wave Band})(\text{100 year Wind Speed})}$
$\frac{\text{Level I Wave Height}}{\text{Wave Fetch Length}}$

2.1 Inlet Characteristics

The inlet characteristics, tidal prism and inlet cross-sectional area, play an important role in the phasing of the peak storm surge, local wind setup and wave height. Tidal prism — volume exchanged between a bay/estuary and the ocean from mean low tide to mean high tide — along with the inlet cross-sectional area provides an indication of the inlet's ability to convey and the rate of conveyance of storm surge. The relationship between the tidal prism and inlet cross-sectional area affects phasing. For example, if the tidal prism of an estuary is small compared to the cross-sectional area of the inlet, then the peak storm surge is more likely to be in phase with the peak local wind setup and peak wave heights. By contrast if the tidal prism of an estuary is large compared to the cross-sectional area of the inlet, then it is more likely there will be phase differences.

2.2 Bridge Location

The location of the bridge is measured in two ways; first the distance is measured from the inlet along the waterway to the bridge (channel distance). The second distance is measured as a straight-line from the coast (straight-line distance). Figure 2-1 presents a sketch illustrating the two distances. Like the inlet characteristics, location of the bridge affects phasing of the peak storm surge, local wind setup, and wave height. The difference between the distance the surge travels and the distance the hurricane wind field travels to reach the bridge site affects the alignment of the peak storm surge, local wind setup and wave height. A bridge located near the coastline (short straight-line distance) with a short channel distance is more likely to have the peak storm surge, local wind setup and wave height occur in phase than a bridge located a long distance from the inlet with a short straight-line distance. These measurements are easily scaled from nautical charts.

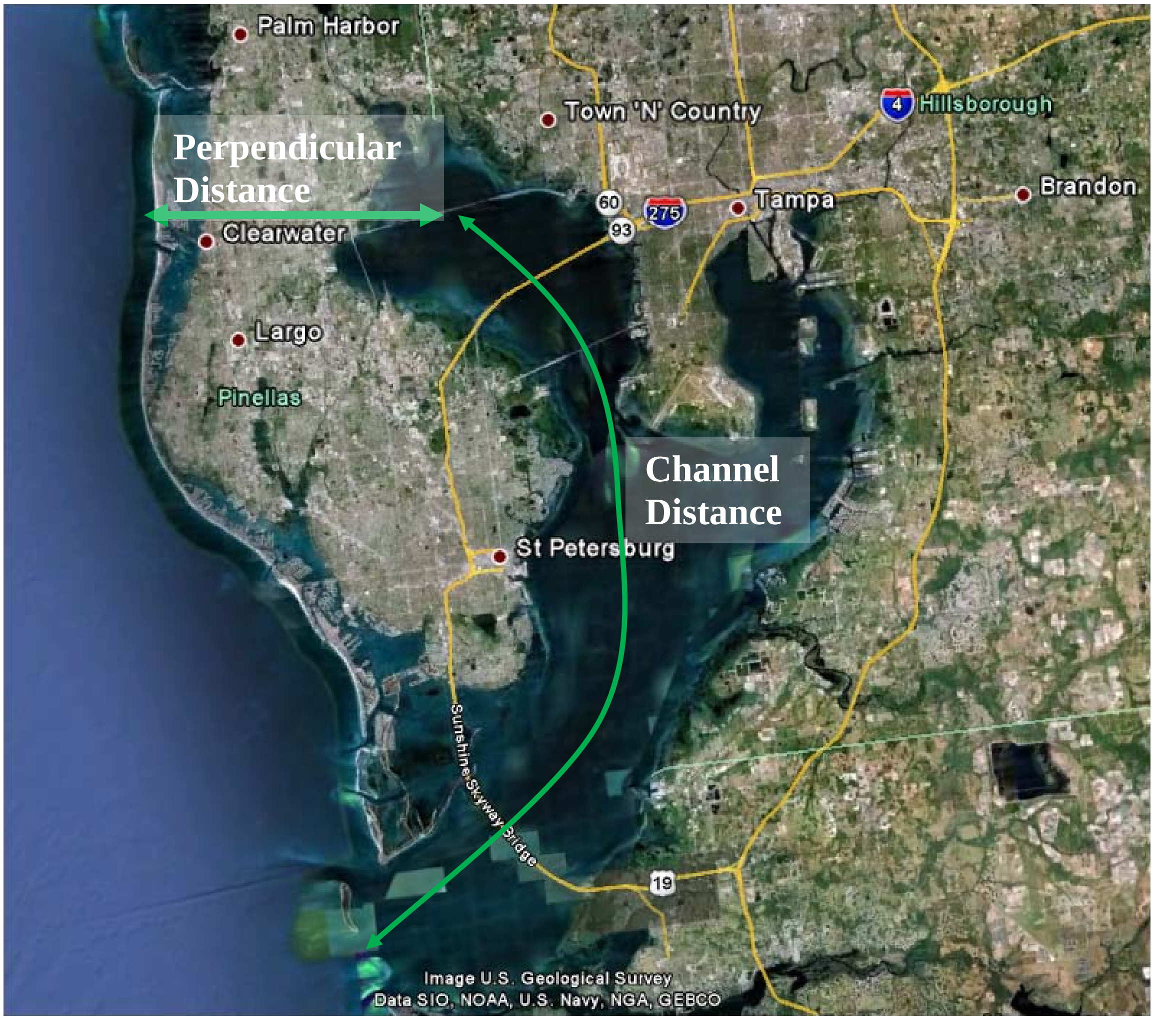

Figure 2-1 Definition sketch for the bridge location parameters.

2.3 Fetch Characteristics

Fetch characteristics (length, orientation relative to the coastline, see Figure 2-2

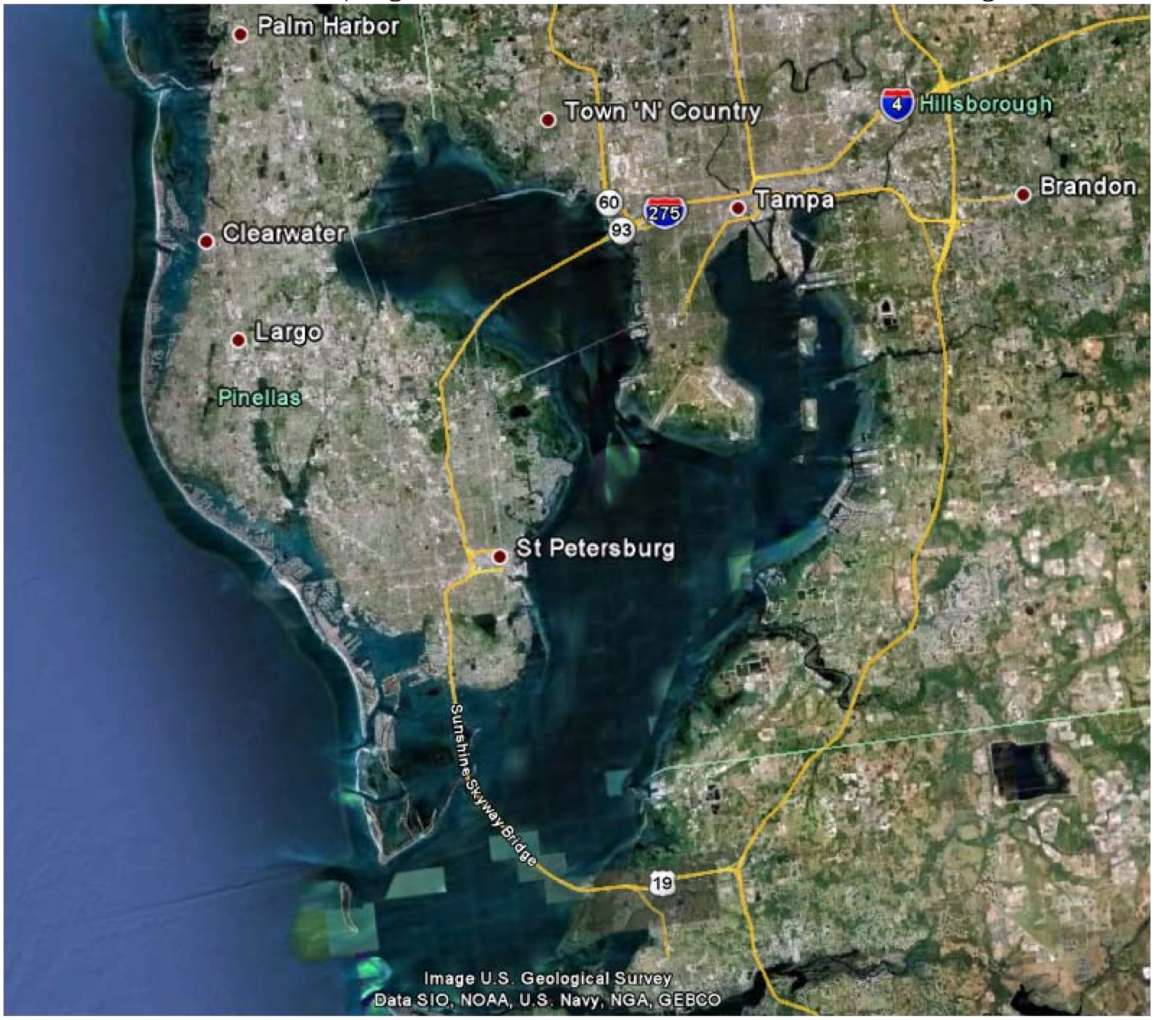

Figure 2-2), and fetch band (angle between the limiting fetch directions, see

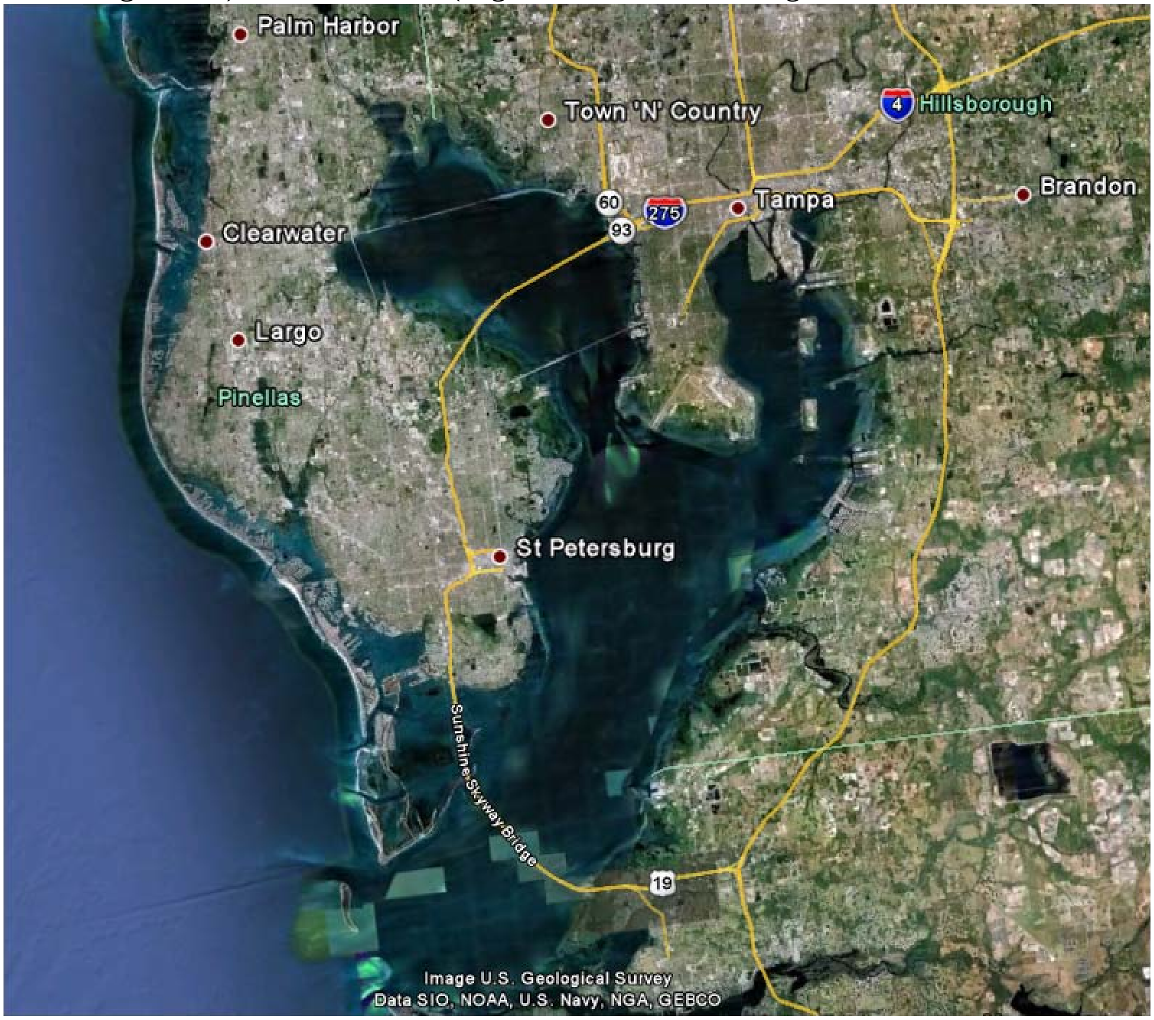

Figure 2-3) can also affect both the magnitude and phasing of the storm surge, local wind setup, and wave height. However, fetch parameters for wave and wind setup are not necessarily the same. For waves the fetch distance is defined as the unobstructed open water distance. For wind setup/set-down the fetch distance can be longer than the unobstructed distance since water can flow around obstructions as indicated in Figure 2-4. When there is wind setup at one end of a closed bay, the water surface is lowered (set-down) at the upwind end. Bay wind setup fetch length is the entire length of the bay and is equal to the wind setup length to the point of interest (bridge) plus the distance from the point of interest to the downwind end of the bay as shown in Figure 2-5.

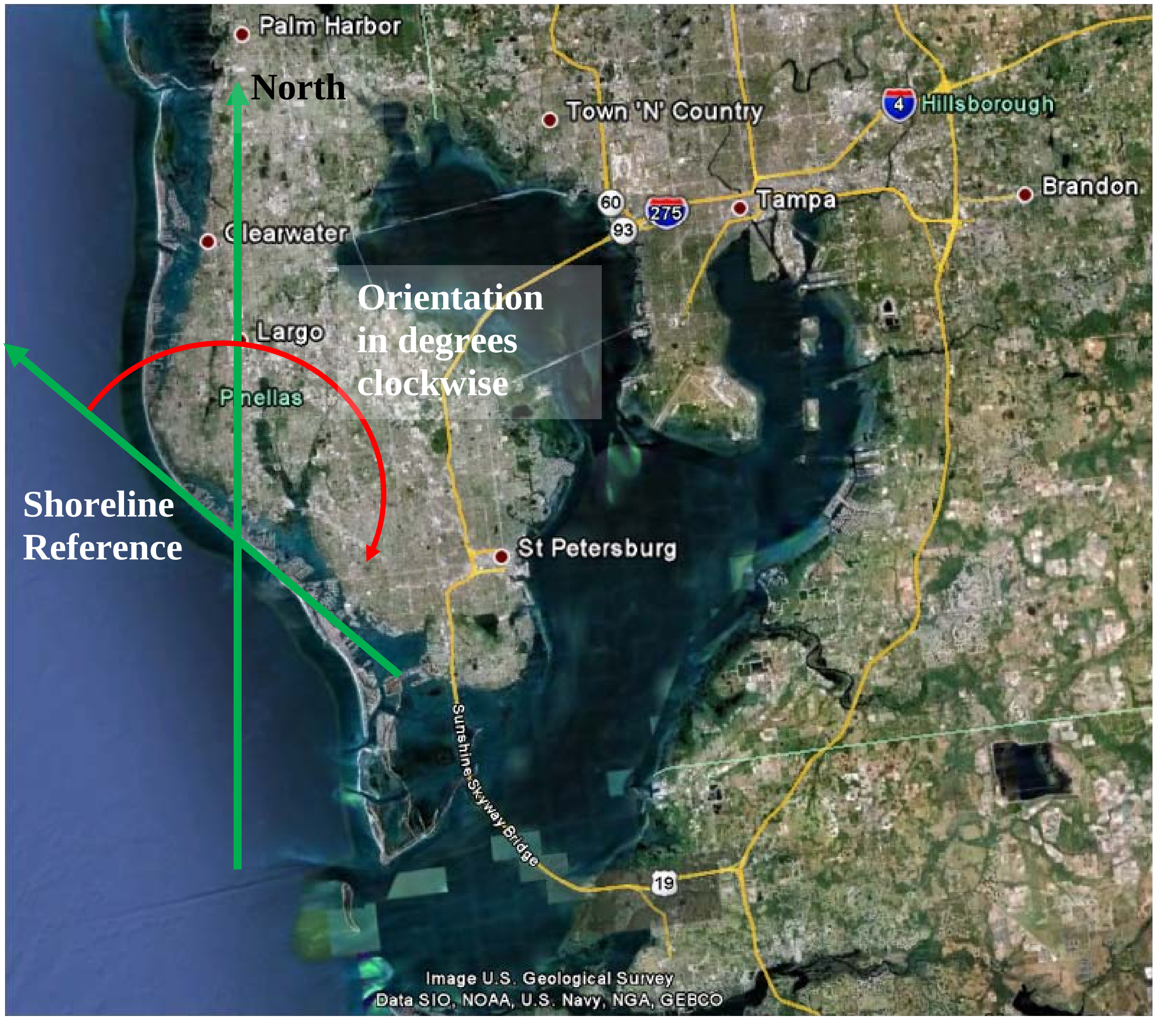

Figure 2-2 Definition sketch for the wave and wind setup fetch orientations.

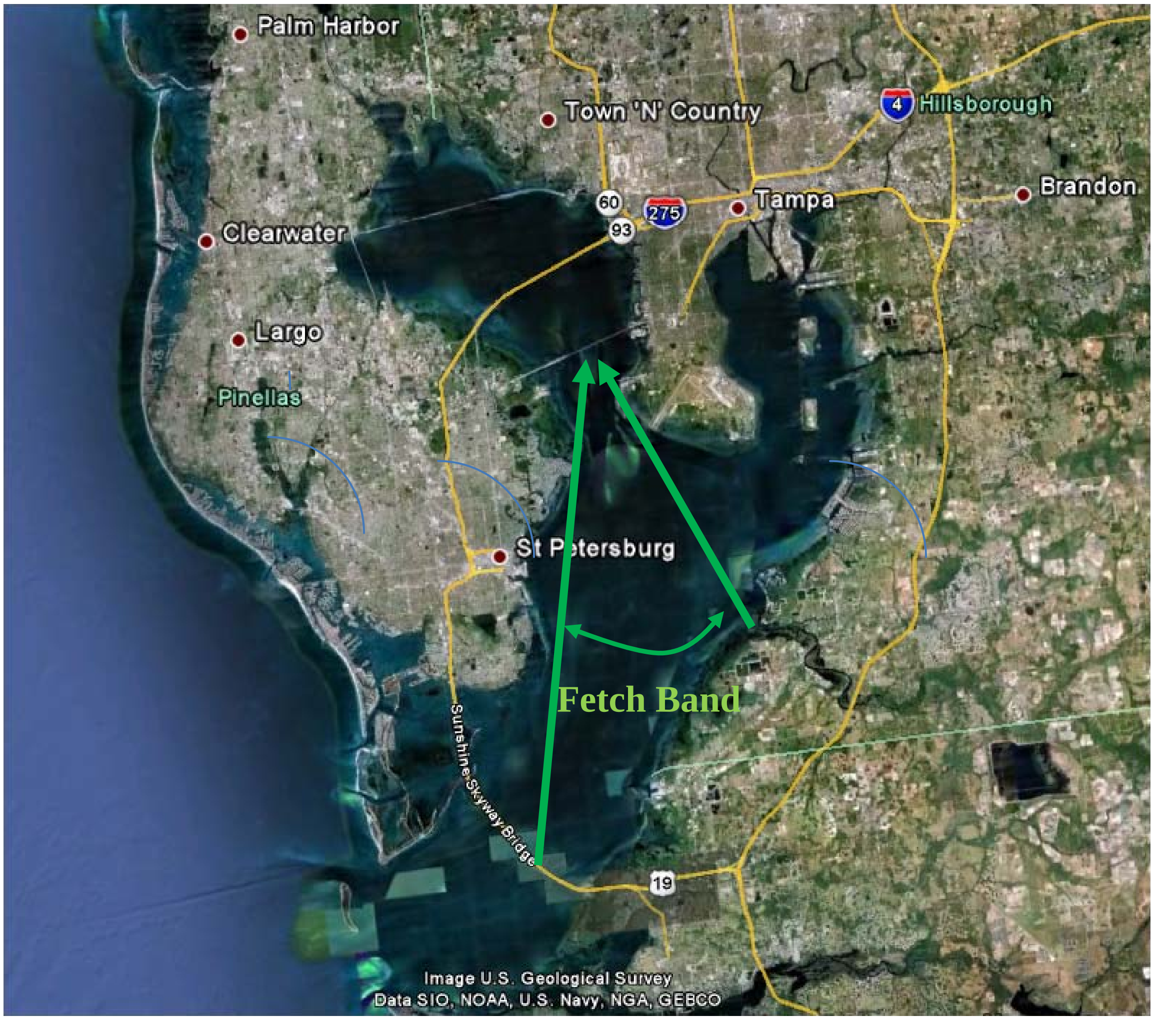

Figure 2-3 Definition sketch for wave fetch band.

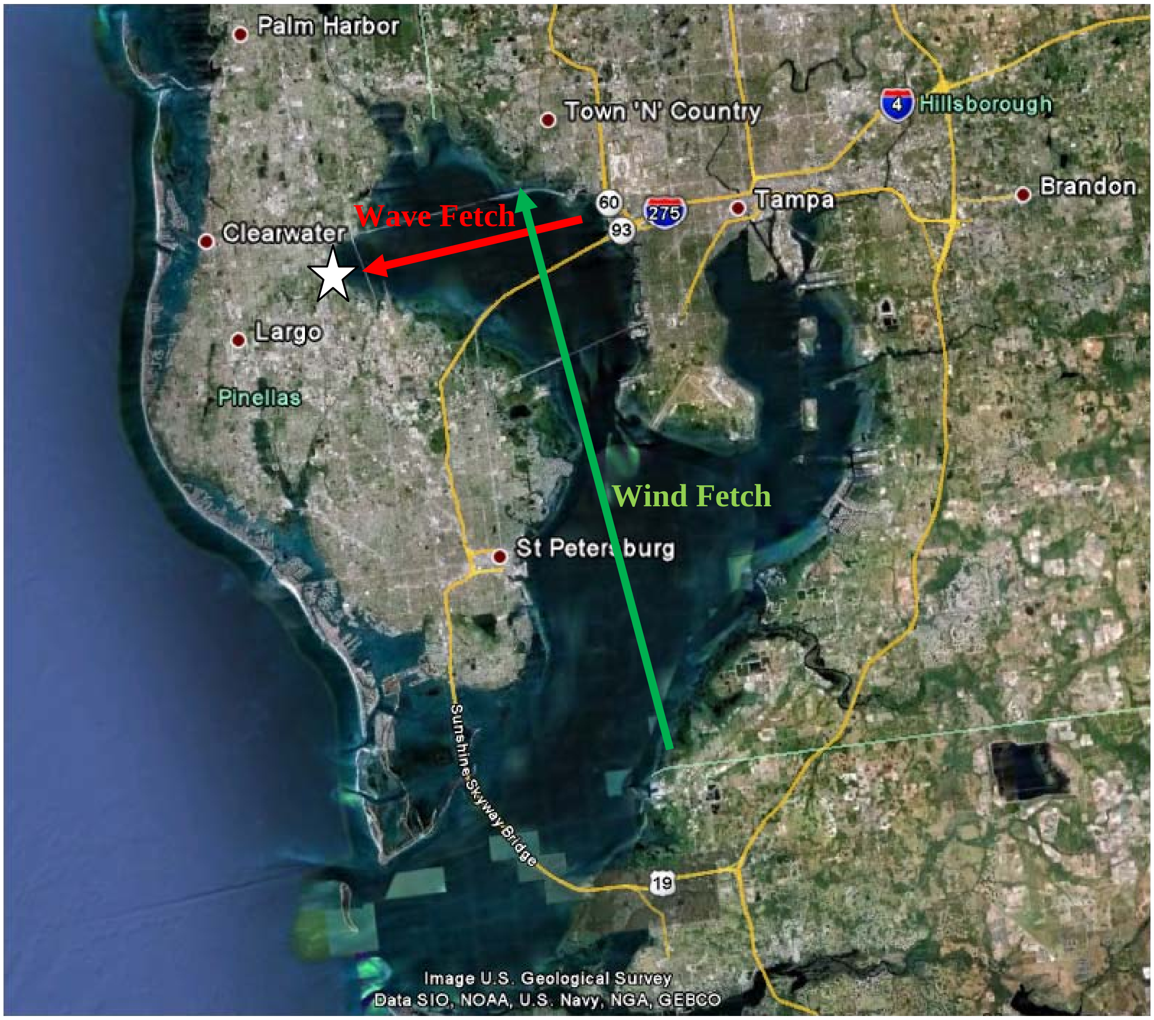

Figure 2-4 Differences between wave and wind setup fetches for a bridge (shown with the white star) located on the west side of Old Tampa Bay.

Figure 2-5 Bay wind setup fetch and bridge wind setup fetch for a bridge (shown with the white star) on Old Tampa Bay.

3. Level I and Level III Met/Ocean Data and Comparisons

A comparison of the two levels quantifies the differences and provides a measure of the magnitude of the conservatism (by both necessity and design) in the Level I analysis. Figure 3-1 and Figure 3-2 present comparisons of water surface elevations and wave heights computed by Level I and Level III analyses. Bridges that were in close proximity to each other and with similar location parameters were not included in this study. Repetitious data does not improve the robustness of the regression analysis. However this raises the problem of independence of observations, i.e., the sampling was not random. The closer the bridges are the more correlation they have in their input parameters. Seven hypothetical bridges were added to provide more data points with more variation in the range of parameters thought to influence the differences be the Level I and III results. As many hypothetical bridge locations as desired can be added, but the additional information diminishes in value as the bridges get closer together.

Figure 3-1 presents a comparison of the Level I and Level III water surface elevations (WSE) at each bridge. In the figure, the diagonal line represents perfect agreement between the two methods and provides a measure of the conservatism. For example, observations above the line indicate that the WSE at that location for the Level I analysis is greater than for the Level III analysis. As the figure illustrates, all of the observations are above the diagonal line indicating that the WSEs from the Level I analysis are always greater than those from the Level III analysis and hence always conservative. Furthermore, the distribution of data indicates the two analyses are highly correlated. Figure 3-2 presents a similar plot comparing wave height for the Level I and Level III analyses. Again the diagonal line represents perfect agreement, where values above the line indicate the Level I analysis predicts larger waves. As the figure illustrates, the Level I analysis produces conservative results for wave height relative to the Level III analysis. Notably, the distribution of the data shows a weaker correlation.

Although results from the Level III analysis produced maximum WSE and associated wave height and maximum wave height and associated WSE, this study by necessity was limited to maximum WSE and maximum wave height since the phasing of these quantities is not known in the Level I analysis.

The pertinent parameters — inlet characteristics, bridge locations and fetch characteristics — described in Chapter 2 are presented in the following tables. Table 3-1 lists the bridge site locations and the fetch parameters. Table 3-2 presents tidal characteristics of the bays over which the bridges are located, water depths and adjusted wind speeds. Table 3-3 lists water surface elevation and wave heights calculated in Levels I and III analyses for all the bridges.

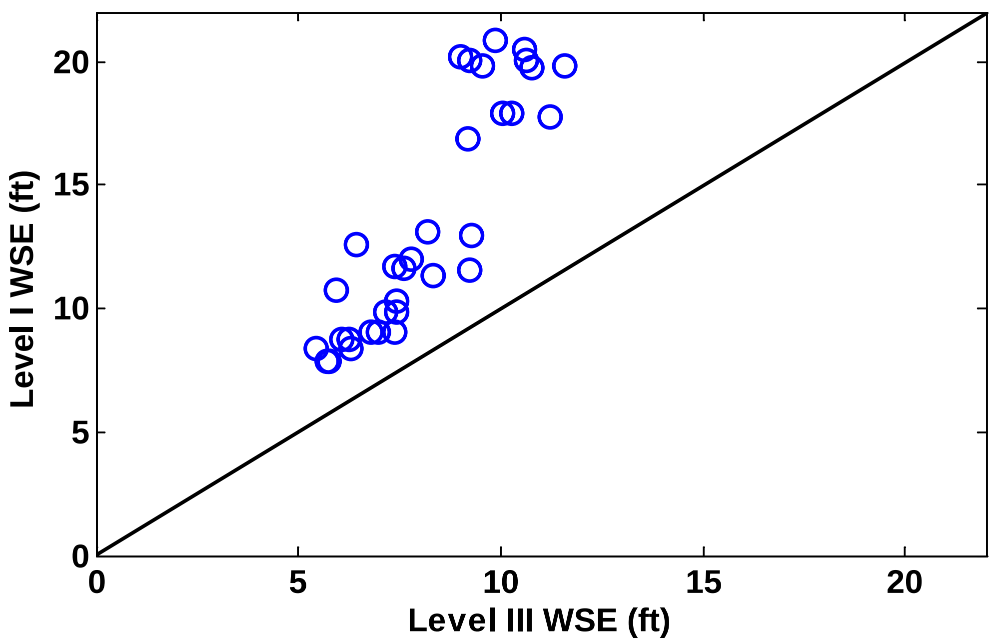

Figure 3-1 Level I versus Level III design water surface elevation (WSE) predictions.

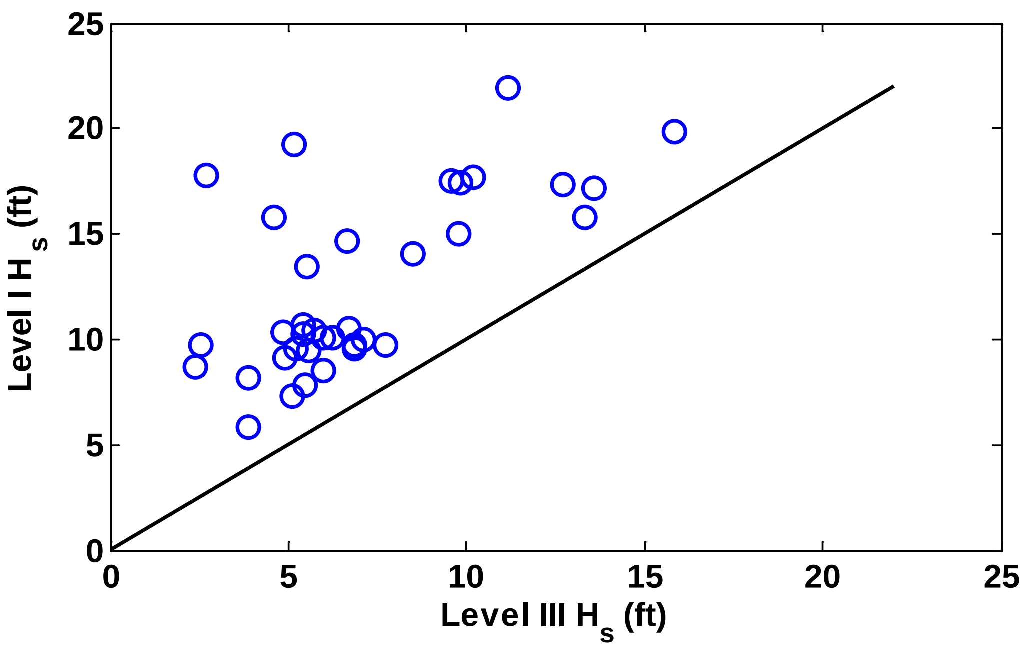

Figure 3-2 Level I versus Level III design significant wave height (H_s) predictions.

Table 3-1 Wind setup and wave fetch characteristics for the bridges analyzed.

Bridge Number	Easting	Northing	Waves			Local Wind Setup			
			Fetch Length (ft)	Fetch Band (degrees)	Fetch Orientation (CW from Shoreline)	Bay Fetch Length (ft)	Bridge Location in Fetch (ft)	Fetch Band (degrees)	Fetch Orientation (CW from Shoreline)
1	319559	3089260	20397	16	42	29685	20397	10	41
2	325684	3079828	7520	13	181.5	34843	18832	30	201
3	327232	3073001	14026	19	6.5	34843	14026	40	246
4	336064	3098356	27766	100	181	168943	156158	47	201
5	331784	3093393	50997	39	125.5	168943	143400	47	201
6	330670	3064537	18409	96	148	27917	19242	0	21
7	347190	3093100	50545	25	281.5	168943	131729	47	201
100045	363803	3082370	11782	20	113	65741	30190	45	251
100049	362108	3091993	10594	20	268	93704	107204	54	246
100064	346396	3094584	23090	80	191	126014	144242	47	201
100107	363803	3082380	11782	20	113	65741	30190	45	251
100299	359222	3090641	10105	60	86	93704	84662	54	246
100300	346115	3085435	109526	58	197	168943	109526	47	201
100301	340170	3094942	20827	150	196	168943	143400	47	201
100585	346115	3085445	109526	58	197	168943	109526	47	201
150001	326229	3077773	10478	35	198.5	25902	15791	30	201
150014	338748	3086742	37320	70	111	168943	118966	47	201
150028	323210	3077106	15600	30	106	20545	20545	35	151
150030	327908	3070311	18289	40	131	18287	18287	50	161
150036	329787	3091298	58929	20	100	168943	134879	47	201
150038	334550	3060754	116600	115	118.5	0	0	0	0
150043	320589	3096050	11530	20	161	0	0	0	0
150044	322396	3094788	20968	55	253.5	0	0	0	0

Bridge Number	Easting	Northing	Waves			Local Wind Setup			
			Fetch Length (ft)	Fetch Band (degrees)	Fetch Orientation (CW from Shoreline)	Bay Fetch Length (ft)	Bridge Location in Fetch (ft)	Fetch Band (degrees)	Fetch Orientation (CW from Shoreline)
150049	330557	3064321	17380	70	136	0	0	0	0
150052	332568	3067063	18194	25	156.5	0	0	0	0
150053	328098	3070635	16527	20	157	18287	18287	50	161
150074	326229	3077763	10478	35	198.5	25902	15791	30	201
150107	343879	3090184	50210	160	181	168943	128450	47	201
150108	338738	3086742	37320	70	111	168943	118966	47	201
150112	318381	3085554	63499	2	240	20200	20200	14	36
150135	328039	3070268	18289	40	131	18287	18287	50	161
150136	328172	3070575	16527	20	157	18287	18287	50	161
150137	328431	3070908	5260	20	219	18287	18287	50	161
150138	334831	3094276	25322	150	196	168943	143400	47	201
150189	336366	3056115	76805	85	148.5	0	0	0	0
150189	335547	3057769	71226	85	148.5	0	0	0	0
150200	331743	3066788	16525	28	156	0	0	0	0
150201	332568	3067053	18194	25	156.5	0	0	0	0
150203	333948	3101584	26520	20	166	168943	166100	47	201
150210	343885	3090184	50210	160	181	168943	128450	47	201
150211	334600	3060754	116600	115	118.5	0	0	0	0
150213	334425	3064340	66807	135	168.5	0	0	0	0
150214	334500	3064340	66807	135	168.5	0	0	0	0
150221	326466	3071397	2799	5	188.5	0	0	0	0
150223	329902	3066213	16799	15	193.5	0	0	0	0
150225	318381	3085544	63499	2	240	20200	20200	14	36
150243	330737	3059516	23609	85	83.5	0	0	0	0
150951	331743	3066798	16525	28	156	0	0	0	0

Table 3-2 Tidal and geographic parameters for the bridges analyzed.

Bridge Number	Inlet	Distance to Inlet (ft)	Inlet CSA (ft^2)	Flood (ft^3)	Ebb (ft^3)	Tide Range (ft)	Perpendicular Distance to Coast (ft)	Depth (ft)
1	Clearwater Pass	21936	7871	4.8E+08	5.2E+08	1.89	540	26
2	Johns Pass	20220	6589	5.3E+08	5.3E+08	1.68	19500	29
3	Johns Pass	15561	6589	5.3E+08	5.3E+08	1.68	19500	29
4	Tampa Bay	178205	298144	1.7E+10	1.7E+10	1.95	64200	52
5	Tampa Bay	179219	298144	1.7E+10	1.7E+10	1.95	42093	52
6	Tampa Bay	10558	298144	1.7E+10	1.7E+10	1.68	17000	27
7	Tampa Bay	154413	298144	1.7E+10	1.7E+10	1.86	80000	53
100045	Tampa Bay	165594	298144	1.7E+10	1.7E+10	2.1	140000	38
100049	Tampa Bay	190623	298144	1.7E+10	1.7E+10	2.04	145500	47
100064	Tampa Bay	172251	298144	1.7E+10	1.7E+10	1.95	84200	53
100107	Tampa Bay	165594	298144	1.7E+10	1.7E+10	2.1	140000	38
100299	Tampa Bay	179560	298144	1.7E+10	1.7E+10	1.89	128000	46
100300	Tampa Bay	142825	298144	1.7E+10	1.7E+10	1.75	90000	54
100301	Tampa Bay	125240	298144	1.7E+10	1.7E+10	1.95	64200	53
100585	Tampa Bay	142825	298144	1.7E+10	1.7E+10	1.75	90000	54
150001	Johns Pass	12959	6589	5.3E+08	5.3E+08	1.68	19500	27
150014	Tampa Bay	167480	298144	1.7E+10	1.7E+10	1.95	68100	53
150028	Johns Pass	14255	6589	5.3E+08	5.3E+08	1.68	9500	26
150030	Blind Pass	15463	4208	1.3E+07	1.2E+07	1.68	15900	24
150036	Tampa Bay	181644	298144	1.7E+10	1.7E+10	1.95	29700	53
150038	Tampa Bay	21194	298144	1.7E+10	1.7E+10	1.55	25400	20
150043	Clearwater Pass	9429	7871	4.8E+08	5.2E+08	1.89	1100	20
150044	Clearwater	8307	7871	4.8E+08	5.2E+08	1.89	7700	19
150049	Johns Pass	10558	6589	5.3E+08	5.3E+08	1.68	17250	18

Bridge Number	Inlet	Distance to Inlet (ft)	Inlet CSA (ft^2)	Flood (ft^3)	Ebb (ft^3)	Tide Range (ft)	Perpendicular Distance to Coast (ft)	Depth (ft)
150052	Tampa Bay	24104	298144	1.7E+10	1.7E+10	1.68	26500	18
150053	Blind Pass	16923	4208	1.3E+07	1.2E+07	1.68	16850	24
150074	Johns Pass	12959	6589	5.3E+08	5.3E+08	1.68	19500	27
150107	Tampa Bay	160584	298144	1.7E+10	1.7E+10	1.86	80000	52
150108	Tampa Bay	167480	298144	1.7E+10	1.7E+10	1.95	68100	53
150112	Clearwater Pass	34744	7871	4.8E+08	5.2E+08	1.68	2100	39
150135	Blind Pass	15463	4208	1.3E+07	1.2E+07	1.68	15900	24
150136	Blind Pass	16923	4208	1.3E+07	1.2E+07	1.68	16850	24
150137	Blind Pass	17884	4208	1.3E+07	1.2E+07	1.68	18250	22
150138	Tampa Bay	179219	298144	1.7E+10	1.7E+10	1.95	50000	54
150189	Tampa Bay	27877	298144	1.7E+10	1.7E+10	1.55	26500	38
150189	Tampa Bay	39728	298144	1.7E+10	1.7E+10	1.55	26500	31
150200	Tampa Bay	19229	298144	1.7E+10	1.7E+10	1.68	23650	16
150201	Tampa Bay	24104	298144	1.7E+10	1.7E+10	1.68	26500	18
150203	Tampa Bay	191033	298144	1.7E+10	1.7E+10	1.92	39400	53
150210	Tampa Bay	160584	298144	1.7E+10	1.7E+10	1.86	80000	52
150211	Tampa Bay	21194	298144	1.7E+10	1.7E+10	1.55	25400	19
150213	Tampa Bay	22953	298144	1.7E+10	1.7E+10	1.55	29000	24
150214	Tampa Bay	22953	298144	1.7E+10	1.7E+10	1.55	29000	24
150221	Blind Pass	6864	4208	1.3E+07	1.2E+07	1.45	12700	14
150223	Tampa Bay	11900	298144	1.7E+10	1.7E+10	1.68	17000	17
150225	Clearwater Pass	34744	7871	4.8E+08	5.2E+08	1.68	2100	39
150243	Tampa Bay	8281	298144	1.7E+10	1.7E+10	1.55	12250	26
150951	Tampa Bay	19229	298144	1.7E+10	1.7E+10	1.68	23650	16

Table 3-3 Level I and Level III met/ocean results for the bridges analyzed.

	WSE Max Level III			Hsig Max Level III			Level I		
Bridge Number	**H_s (ft)**	**T_p (sec)**	**WSE (ft-MSL)**	**H_s (ft)**	**T_p (sec)**	**WSE (ft-MSL)**	**H_s (ft)**	**T_p (sec)**	**WSE (ft-MSL)**
1	3.0	1.6	6.5	4.9	2.1	6.1	9.1	4.0	12.6
2	5.4	2.2	9.3	6.0	2.3	9.2	8.5	3.8	13.0
3	3.5	1.8	8.2	5.4	2.2	8.0	10.3	3.7	13.1
4	9.1	2.8	10.7	9.8	2.9	10.7	15.0	4.6	20.1
5	8.1	2.7	9.6	10.2	3.0	9.4	17.7	5.3	19.8
6	5.6	2.2	6.3	6.9	2.4	6.2	9.7	4.0	8.4
7	9.3	2.8	10.6	9.8	2.9	10.4	17.4	5.3	20.5
100045	4.6	2.0	11.2	5.7	2.2	11.1	10.4	4.3	17.7
100049	3.8	1.8	12.2	4.9	2.1	12.1	10.3	4.1	22.1
100064	7.8	2.6	10.8	8.5	2.7	10.8	14.1	5.1	19.8
100107	4.6	2.0	11.2	5.7	2.2	11.1	10.4	4.3	17.7
100299	4.9	2.1	11.6	6.0	2.3	11.5	10.1	4.2	19.8
100300	9.2	2.8	9.2	11.2	3.1	9.1	22.0	6.6	16.9
100301	11.7	3.2	10.3	13.3	3.4	10.2	13.2	5.0	17.9
100585	9.3	2.8	9.2	11.1	3.1	9.2	19.6	6.6	10.6
150001	4.8	2.1	9.2	5.6	2.2	9.3	9.5	4.2	11.6
150014	3.8	1.8	9.0	4.5	2.0	9.0	15.8	4.9	20.2
150028	4.1	1.9	8.3	5.4	2.2	8.2	10.7	4.7	11.3
150030	6.5	2.4	7.4	7.1	2.5	7.4	10.0	4.0	11.7
150036	1.7	1.2	9.2	2.7	1.5	9.2	17.8	5.4	20.1
150038	6.5	2.4	5.8	7.7	2.6	5.7	9.8	6.0	7.9
150043	2.0	1.3	7.1	2.5	1.5	7.1	9.7	4.2	9.9
150044	5.2	2.1	7.4	6.2	2.3	7.2	10.1	4.2	9.9
150049	5.6	2.2	6.3	6.9	2.4	6.2	9.5	4.0	8.8
150052	1.9	1.3	7.4	2.4	1.4	7.3	8.7	4.0	9.1

	WSE Max Level III			Hsig Max Level III			Level I		
Bridge Number	**H_s (ft)**	**T_p (sec)**	**WSE (ft-MSL)**	**H_s (ft)**	**T_p (sec)**	**WSE (ft-MSL)**	**H_s (ft)**	**T_p (sec)**	**WSE (ft-MSL)**
150053	5.9	2.3	7.6	6.7	2.4	7.6	10.5	3.9	11.7
150074	4.8	2.1	9.2	5.6	2.2	9.3	9.5	4.2	11.6
150107	11.9	3.2	10.0	12.7	3.3	10.0	17.1	5.3	17.9
150108	3.7	1.8	9.0	4.6	2.0	9.0	15.7	4.9	19.9
150112	2.5	1.5	5.9	5.2	2.1	5.3	19.2	5.9	10.7
150135	6.5	2.4	7.4	7.1	2.5	7.4	10.0	4.0	11.7
150136	5.9	2.3	7.6	6.7	2.4	7.6	10.5	3.9	11.7
150137	5.0	2.1	7.8	5.1	2.1	7.8	7.3	3.5	12.0
150138	8.2	2.7	9.9	9.6	2.9	9.8	14.0	4.4	20.9
150189	13.0	3.4	5.7	15.8	3.7	5.6	19.8	6.2	7.9
150189	13.0	3.4	5.8	13.6	3.4	5.7	17.1	5.9	7.9
150200	3.7	1.8	7.0	3.9	1.8	7.0	8.1	3.9	9.1
150201	1.9	1.3	7.4	2.4	1.4	7.3	8.6	4.0	9.1
150203	5.4	2.2	11.5	6.7	2.4	11.6	14.6	4.4	23.4
150210	11.9	3.2	10.0	12.7	3.3	10.0	17.3	5.3	18.8
150211	6.3	2.3	5.8	7.6	2.6	5.8	9.8	6.0	7.9
150213	5.2	2.1	6.1	5.5	2.2	6.2	13.4	5.7	8.8
150214	5.2	2.1	6.1	5.5	2.2	6.2	13.4	5.7	8.8
150221	3.2	1.7	7.4	3.9	1.8	7.5	5.8	3.0	10.3
150223	3.8	1.8	6.8	5.2	2.1	6.8	9.6	3.9	9.1
150225	2.5	1.5	5.9	5.2	2.1	5.3	19.1	5.9	10.3
150243	3.8	1.8	5.4	5.4	2.2	5.3	7.9	4.2	8.4
150951	3.8	1.8	7.0	3.9	1.8	7.0	7.9	3.9	9.1

4. Regression Analysis

The parameters listed in Table 2-1 and Table 2-2 and used in the regression analysis were selected based on physical arguments and a correlation analysis. Once again, using physical reasoning non-dimensional groups were formed with these parameters. Note that these groups are in terms of the variables that influence the wave parameters and the water elevation due to the location, fetch orientation, etc., of the point of interest, rather than the actual physics of their development. They are just defining a search space to probe for correlations between the wave parameters and water elevations and these quantities. This is different from a standard dimensional analysis, where all the parameters thought to define the physical processes are used. There were many potential groups and it is not possible to know, a priori, which are relevant. Two different regression analyses were performed; one using non-dimensional groups and one with dimensional parameters.

Stepwise regression is used to choose the parameters from the list of potential candidates that will be used in the final regression. In this method F-tests are used which compare statistical models that have been fitted to the data set by looking at their variance ratios. Stepwise regression starts with an initial model fit. The p value of an F-statistic is computed to test models with and without a potential term. The p value is the probability of getting a non-zero coefficient by chance even though the true coefficient is zero. At the next step either a term is added or removed whichever has the smallest p value. The procedure stops when terms cannot be moved in and out of the model any more with lower than a predefined confidence (entrance $p=0.05$, exit $p=0.10$). It should be noted that the final set of parameters chosen depends on the initial model and is not unique. Stepwise regression was applied with an initial model of zero parameters and all the parameters. For situations where the two approaches did not converge to the same set of parameters, the set that appeared to have the most physical basis was selected.

Ideally regression analysis is performed using independent, uncorrelated variables. However, this is rarely the case since the parameters are in general correlated to some degree. This might be just an artifact of the data set selection or it might be due to the physical relationship between the parameters. Correlation coefficients shown in Table 4-1 are a measure of the correlation between all the parameters that are candidates for use in the regression analysis. Wave and wind setup orientations are not included in the list since they have circular values and cannot be compared to non-circular values directly. The correlation matrix is symmetric and the diagonal consists of ones since all variables have perfect correlation with themselves. Coefficients located off the diagonal with values greater than 0.9 are highlighted in yellow. Ebb volume, flood volume and cross-sectional area have almost perfect correlation. They are slightly less than one, but appear as 1.00 in the table due to rounding. Only one of these three parameters needs to be chosen for regression analysis to make the analysis more robust. Cross-sectional area was chosen since it is easier to calculate. However, the ratios among these parameters can still be used. Bridge wind setup fetch and bay wind setup fetch are also highly correlated (coefficient = 0.98). This is expected, but if only one of these is used crucial information (where a bridge is located along the fetch length) is lost. So bridge wind setup fetch and the ratio of bridge wind setup fetch length to bay wind setup fetch length are both used.

Distance to the inlet is also correlated with these two parameters. Even though some level of correlation was expected, such a high correlation is probably just an artifact of the specific geography of the Tampa Bay area. Similarly water depth is also correlated with wind setup and wave fetches and distance to the inlet. There does not seem to be an obvious physical reason for these correlations. These coincidental correlations are highly unlikely to hold at other locations and might have an impact on the transportability of the results of this study which were developed using only data from one location.

Table 4-1 Cross correlation coefficients for regression analysis input parameters.

	wave fetch	wave band	bay wind fetch	bridge wind fetch	wind band	dist inlet	XS area	flood	ebb	tide range	dist perp	depth
wave fetch	1.00	0.36	0.18	0.12	-0.12	0.14	0.41	0.41	0.41	-0.20	0.04	0.28
wave band	0.36	1.00	0.28	0.28	-0.01	0.22	0.49	0.49	0.49	0.02	0.18	0.31
bay wind fetch	0.18	0.28	1.00	0.98	0.75	0.91	0.50	0.50	0.50	0.71	0.59	0.94
br. wind fetch	0.12	0.28	0.98	1.00	0.74	0.92	0.51	0.51	0.51	0.72	0.58	0.93
wind band	-0.12	-0.01	0.75	0.74	1.00	0.73	0.11	0.11	0.11	0.63	0.60	0.72
dist inlet	0.14	0.22	0.91	0.92	0.73	1.00	0.61	0.61	0.61	0.79	0.79	0.92
XS area	0.41	0.49	0.50	0.51	0.11	0.61	1.00	1.00	1.00	0.28	0.56	0.54
flood	0.41	0.49	0.50	0.51	0.11	0.61	1.00	1.00	1.00	0.28	0.56	0.54
ebb	0.41	0.49	0.50	0.51	0.11	0.61	1.00	1.00	1.00	0.28	0.56	0.54
tide range	-0.20	0.02	0.71	0.72	0.63	0.79	0.28	0.28	0.28	1.00	0.63	0.67
dist perp	0.04	0.18	0.59	0.58	0.60	0.79	0.56	0.56	0.56	0.63	1.00	0.64
depth	0.28	0.31	0.94	0.93	0.72	0.92	0.54	0.54	0.54	0.67	0.64	1.00

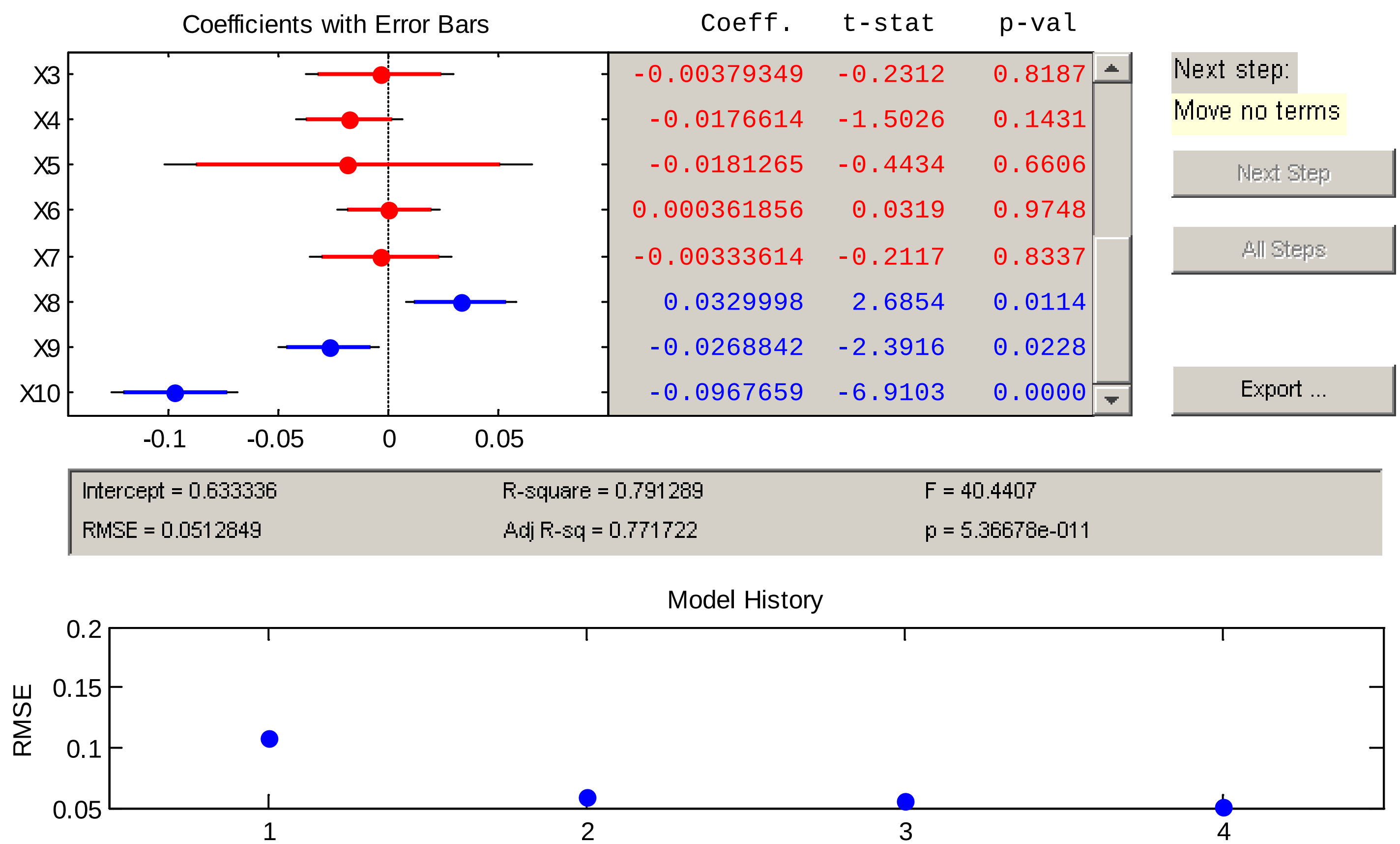

Figure 4-1 Stepwise regression analysis results for dimensional water surface elevation.

4.1 Dimensional Regression for Water Surface Elevation

The parameters chosen for dimensional regression of WSE are bridge wind setup fetch length, ratio of bridge wind setup fetch length to bay wind setup fetch length, wind setup fetch band, distance to the inlet, inlet cross-sectional area, tidal range, perpendicular distance to the ocean, water depth and Level I WSE. Level I WSE is a function of many of the other parameters, but the value used is the result of a FEMA or similar analysis. Figure 3-1 shows that the amount of overprediction increases as the Level I WSE increases. The dependant parameter was chosen as the ratio of Level III WSE to Level I WSE, and this ratio is called the WSE modifier. Having the Level I WSE on both sides of the regression allows for non-linear relationships between Level I and Level III results in this linear regression analysis.

The stepwise regression analysis results are shown in Figure 4-1 above. The top figure shows the coefficients for all the normalized variables with their confidence intervals. The bottom figure shows how the root mean squared error (RMSE) changes as new variables are added. The table shows the values of the coefficients and the p values for the F-test. Variables whose confidence intervals include zero are not chosen. The chosen variables are 8, 9 and 10 which correspond to perpendicular distance to ocean, water depth and Level I WSE, respectively. The predictions are plotted in Figure 4-2 along with R^2 (coefficient of determination) and correlation coefficient (CC) values. The x-axis shows the ratio of Level III to Level I WSE and the y-axis shows the predictions for the same value. The black line is the y = x line showing the perfect fit. The regression equation for the modifier is:

$$\text{WSEMOD(Dim)(Best Fit)} = 0.633 + 0.033\left(\frac{\text{PD} - 40822}{38636}\right) - 0.027\left(\frac{\text{WD} - 21.7}{7.67}\right) - 0.097\left(\frac{\text{L1WSE} - 13.9}{5.07}\right), \quad (1)$$

where PD is the perpendicular distance to ocean in ft, WD is the water depth in ft and L1WSE is the Level I water surface elevation, mean sea level (MSL) in ft. This is a best fit equation, and should not be used for design. The equation that envelopes the data and therefore provides moderately conservative results for use in design is:

$$\boxed{\begin{aligned}&\text{WSEMOD(Dim)(Design)} = 1.25 * \text{WSEMOD}(\text{Dim})\text{(Best Fit)} = \\ &0.791 + 0.041\left(\frac{\text{PD} - 40822}{38636}\right) - 0.034\left(\frac{\text{WD} - 21.7}{7.67}\right) - 0.121\left(\frac{\text{L1WSE} - 13.9}{5.07}\right).\end{aligned}} \quad (2)$$

The predictions made by this equation are shown in Figure 4-3. All of the predictions are above the black line, which means they are conservative (i.e., the modified Level I predictions are greater than the Level III values). However, the equation still decreases the Level I predictions by up to 50% in some cases resulting in significant improvements in Level I water level predictions.

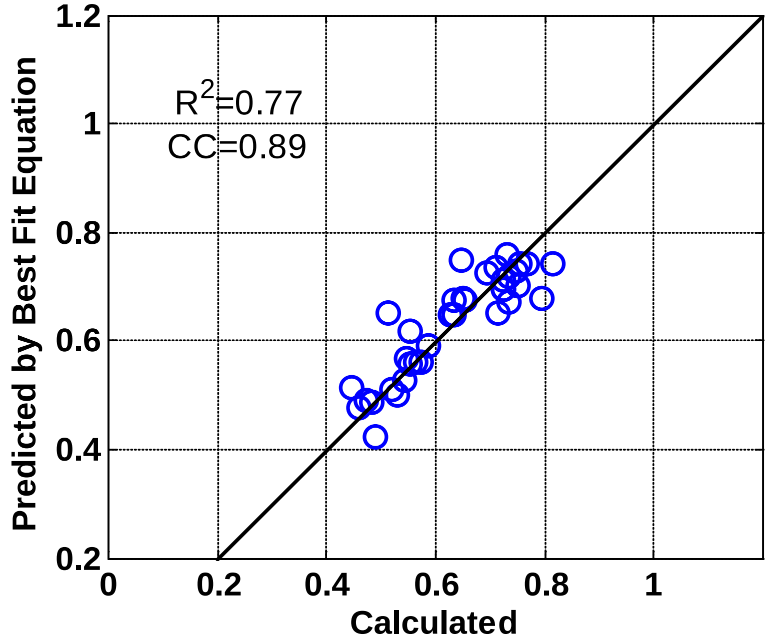

Figure 4-2 Best fit prediction of the ratio of Level III to Level I WSE versus the ratio of Level III to Level I WSE from the Pilot Study. The R^2 error and correlation coefficient (CC) are also shown. The black line is the perfect fit line (y = x).

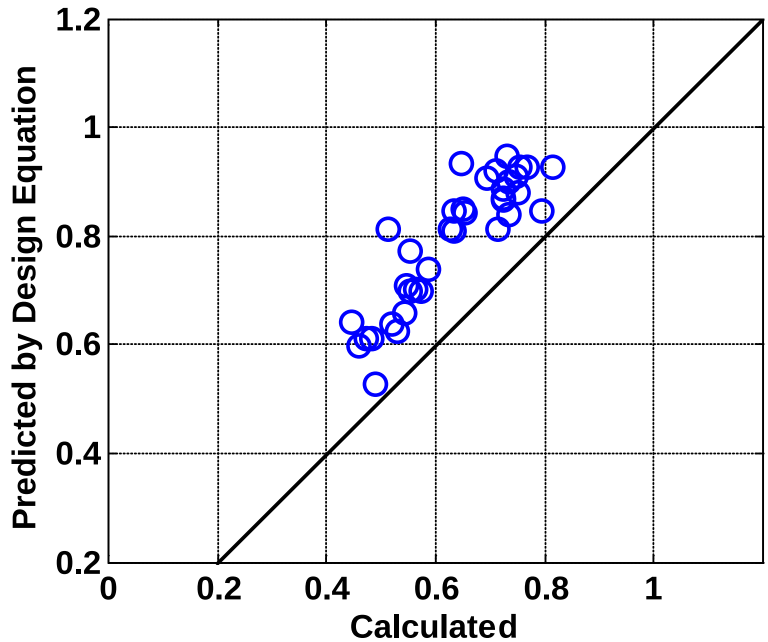

Figure 4-3 Design equation prediction of the ratio of Level III to Level I WSE versus ratio of Level III to Level I WSE from the Pilot Study. The design multiplier is 1.25.

4.2 Non-Dimensional Regression for WSE

Using non-dimensional groups rather than dimensional parameters increases the portability of the results. The non-dimensional groups investigated were wind setup fetch band, bridge wind setup fetch length to bay wind setup fetch length ratio, (Level I WSE)/(Tide range), (inlet cross-sectional area)(Tide range)/(Ebb Volume), and (Bridge Wind Setup)/(Storm surge). Wind Setup and storm surge are the terms that are added to make up the Level I WSE prediction. Standard deviation of the wind setup is much larger than the storm surge, so that term is mainly controlled by the wind setup. Only the last term (Wind Setup)/(storm surge) comes out as a significant term from the stepwise regression analysis. The best fit and design equation predictions are shown in Figure 4-4 and Figure 4-5 respectively. The best-fit equation is:

$$\text{WSEMOD(Non-Dim)(Best Fit)} = 0.633\text{-}0.0931\left(\frac{\frac{\text{Wind Setup}}{\text{Storm Surge}} - 0.513}{0.537}\right), \qquad (3)$$

and the design equation is:

$$\text{WSEMOD(Non-Dim)(Design)} = 1.38\text{WSEMOD(Non-Dim)(Best Fit)} = 0.874 - 0.129\left(\frac{\frac{\text{Wind Setup}}{\text{Storm Surge}} - 0.513}{0.537}\right). \qquad (4)$$

A coefficient of 1.38 was used in the design equation. The non-dimensional equation with just one parameter has an R^2 and correlation coefficient very close to that for the dimensional equation with three parameters. Another feature of the non-dimensional equation is that it only uses the wind setup and storm surge values, which are already needed for Level I analysis, so it does not require additional data collection.

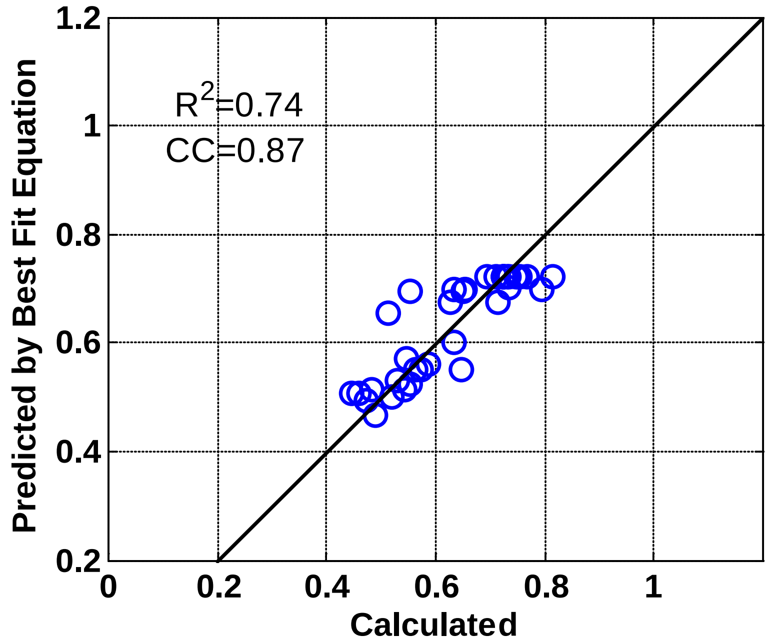

Figure 4-4 Non-dimensional best fit predictions of the ratio of Level III to Level I WSE versus ratio of Level III to Level I WSE from the Pilot Study. The R^2 error and correlation coefficient (CC) are also shown. The black line is the perfect fit line (y = x).

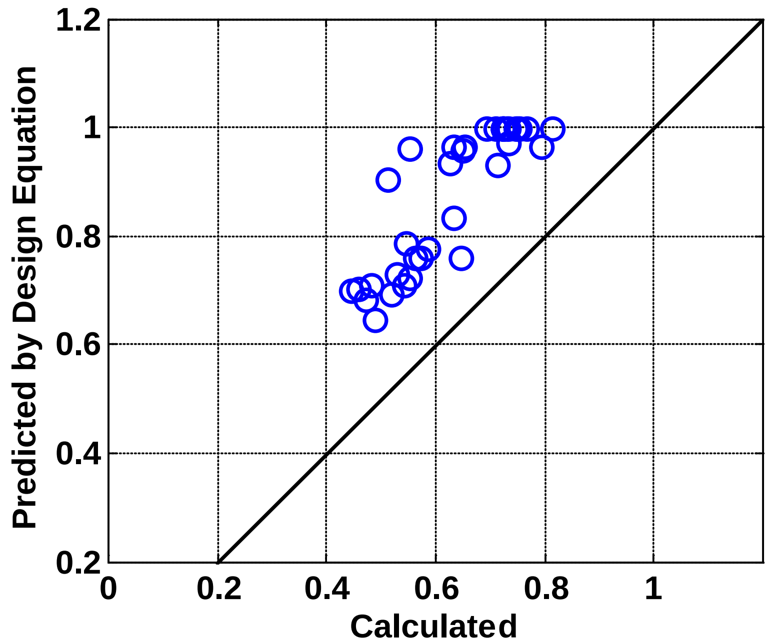

Figure 4-5 Nondimensional design equation prediction of the ratio of Level III to Level I WSE versus ratio of Level III to Level I WSE from the Pilot Study. The design multiplier is 1.38.

4.3 Dimensional Regression for Significant Wave Height

The following parameters were tested for significance in the significant wave height stepwise dimensional regression analysis: wave fetch length, wave fetch band, wave fetch orientation (clockwise from shoreline), water depth, Level I significant wave height, Level I WSE. Wave band was found to be the only significant parameter as a result of the stepwise regression. As the wave band increases, the probability of having the maximum winds along the maximum fetch direction increase, this is in agreement with the positive wave band regression coefficient. The best fit regression equation is:

$$\mathrm{WHMOD}(\mathrm{dim})(\mathrm{bestfit}) = 0.5686 + 0.0636\left(\frac{\text{Wave Fetch Band} - 48.50}{38.76}\right) \tag{5}$$

The fit of the regression equation is not as good as that for WSE (Figure 4-6). This requires a much bigger design multiplier of 1.55 for the design equation (Figure 4-7). The resulting design equation is:

$$\boxed{\begin{aligned}&\mathrm{WHMOD}(\mathrm{dim})(\mathrm{design}) = 1.55\mathrm{WHMOD}(\mathrm{dim})(\mathrm{bestfit}) = \\ &0.8813 + 0.0986\left(\frac{\text{Wave Fetch Band} - 48.50}{38.76}\right).\end{aligned}} \tag{6}$$

This also results in cases where the modifier is greater than 1.0. The modifier can be capped at 1.0, since there were no cases where the Level III significant wave height exceeded that for Level I. This should hold in all cases as long as conservative choices are made for wind speed and fetch length. The percentage of variance explained by wave band is very small as can be seen in Figure 4-6. Therefore using an expression in terms of wave band is only slightly better than using a constant reduction value.

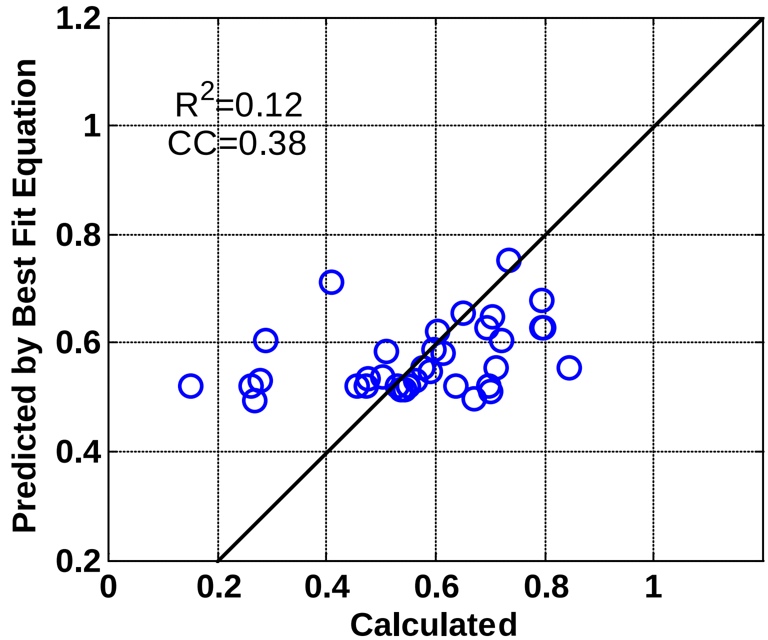

Figure 4-6 Dimensional best fit predictions of the ratio of Level III to Level I H_s versus ratio of Level III to Level I H_s from the Pilot Study. The R^2 error and correlation coefficient (CC) are also shown. The black line is the perfect fit line (y = x).

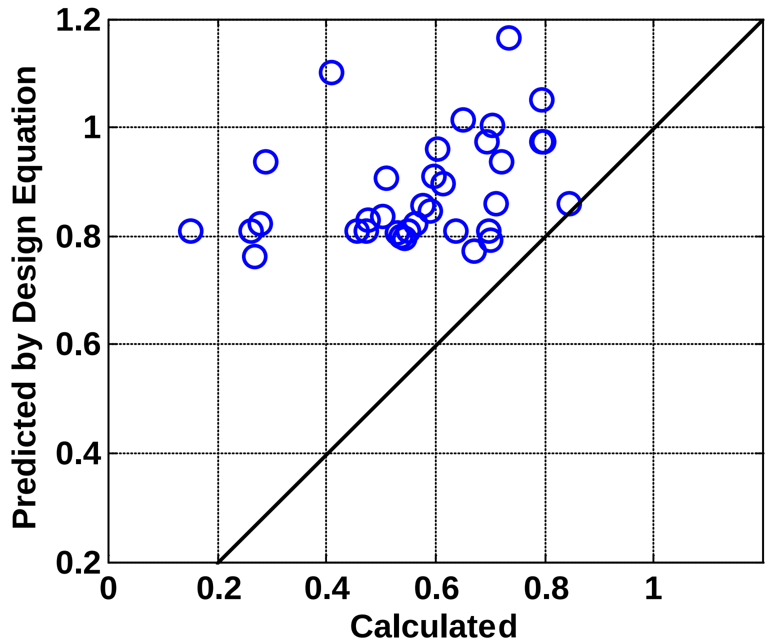

Figure 4-7 Dimensional design equation prediction of the ratio of Level III to Level I H_s versus ratio of Level III to Level I H_s from the Pilot Study.

4.4 Non-dimensional Regression for Significant Wave Height

The following parameters were tested for significance in the significant wave height stepwise non-dimensional regression analysis: wave fetch band (degrees), wave fetch orientation (clockwise from shoreline), $\frac{(\text{Storm Forward Velocity})\sqrt{\frac{\text{Water Depth}}{\text{Wave Fetch Length}}}}{(\text{Wave Band})(\text{100 Year Wind Speed})}$ and (Level I wave height)/(wave fetch length).

Storm forward velocity is the forward velocity of the eye of the storm and it is constant for all bridges in the study area along with 100 year wind speed. The magnitudes of these two are not important for this case since they are constant, but they should be chosen to be representative of a design storm. Wave fetch band and (Level I wave height)/(wave fetch length) terms were found to be the significant terms during the stepwise regression process. The resulting equation is:

$$\text{WHMOD}(\text{non}-\text{dim})(\text{bestfit}) =$$

$$0.569 + 0.086\left(\frac{\text{Wave Band - 48.5}}{38.8}\right) + 0.051\left(\frac{\left(\frac{\text{Level I Wave Height}}{\text{Wave Fetch Length}}\right) - 5.86 \text{ x } 10^{-4}}{3.84 \text{ x } 10^{-4}}\right). \quad (7)$$

The regression fit is slightly better than the analysis with the dimensional parameters (Figure 4-8), but the design multiplier is larger (1.65) due to one outlier (Figure 4-9). The design equation is:

$$\text{WHMOD}(\text{non}-\text{dim})(\text{design}) = 1.65\text{WHMOD}(\text{non}-\text{dim})(\text{bestfit}) =$$

$$0.934 + 0.142\left(\frac{\text{Wave Band - 48.5}}{38.8}\right) + 0.084\left(\frac{\left(\frac{\text{Level I Wave Height}}{\text{Wave Fetch Length}}\right) - 5.86 \text{ x } 10^{-4}}{3.84 \text{ x } 10^{-4}}\right). \quad (8)$$

Level I wave height is a function of fetch length and water depth. It should be noted that even though water depth does not occur in any of the significant parameters, the second term (Level I wave height)/(wave fetch length) is highly correlated with water depth.

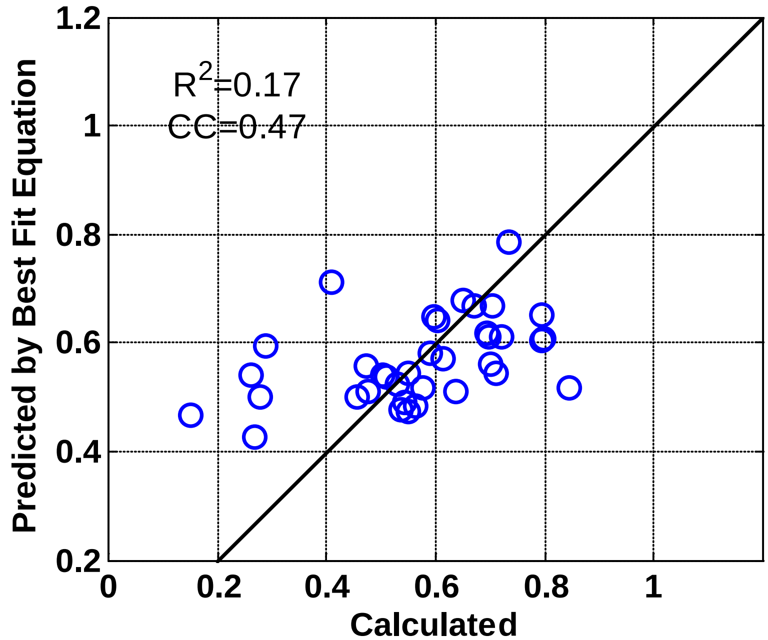

Figure 4-8 Non-dimensional best fit predictions of the ratio of Level III to Level I H_s versus ratio of Level III to Level I H_s from the Pilot Study. The R^2 error and correlation coefficient (CC) are also shown. The black line is the perfect fit line (y = x).

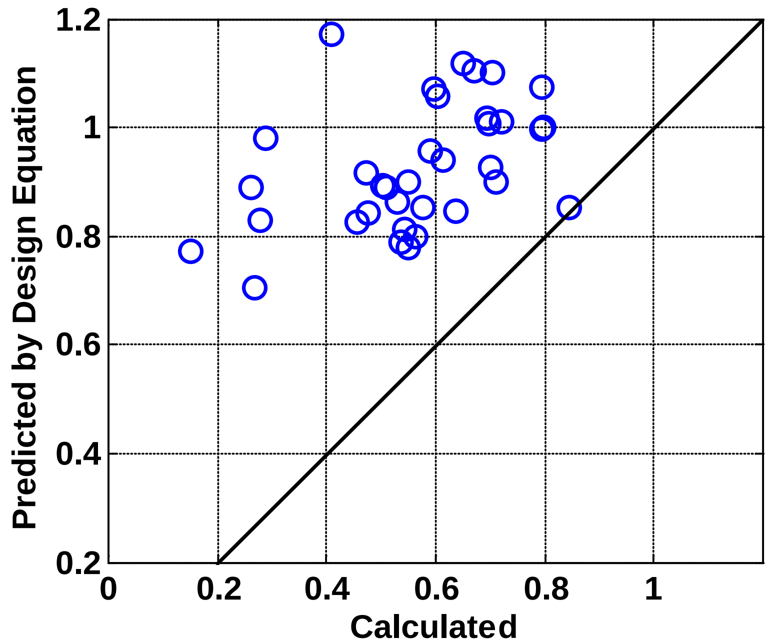

Figure 4-9 Dimensional design equation prediction of the ratio of Level III to Level I H_s versus ratio of Level III to Level I H_s from the Pilot Study. The design multiplier is 1.65.

5. Verification

The initial scope included testing the modifiers with data from the I10-Escambia Bay Bridge. However, after a more careful examination of the available data/information at that site it became obvious that this was not possible. The design conditions for the new I10-Escambia Bay Bridge were based on Hurricane Ivan conditions and not the 100-year values. One-hundred year conditions for Escambia Bay were estimated by scaling the wind and pressure fields from Hurricane Ivan using Ochi's 100-year wind speed at landfall for that area. Neither Level I nor Level III conditions are available for that site.

An alternative methodology was developed to test the methodology. The bridges used in the study were separated into four different groups according to their proximity and shared geographic characteristics as shown in Figure 5-1. One group was removed from the data set (test set) and the regression analysis performed using only the remaining bridges (training set). The new equation was then tested using the excluded bridges. This way the training and test sets were separated. This procedure was repeated four times, excluding a different group each time. It should be pointed out that this is not a test of the final results of the study since it is using different equations. It is, however, a test of the methodology used. It gives some idea about the generalization power of the methodology. This approach does have two drawbacks compared to one with data from another location. First the already limited data set will be made even smaller. Second, the test bridges will be similar to the rest of the data set since they are geographically and meteorologically similar and all of the bridges use the same meteorological input conditions in the Level I analyses.

The results of this testing procedure are presented in Figure 5-2 through Figure 5-5. In these plots the training data are shown with red circles and the test data with blue crosses. The top line is the perfect fit line (y=x) and the line below is y=x/(design multiplier). All points above the second line would be predicted conservatively by the design equation. For WSE the non-dimensional equation performs better. As seen in Figure 5-3 all but one test bridge in group 2 are above the second line and there are no gross over-predictions. As a result the regression equation with the non-dimensional equation is recommended for predicting WSE modification.

Figure 5-4 and Figure 5-5 show the wave height test results. As with the full regression results for wave height the correlation is poor. However there are no underpredictions and the test sets are not behaving significantly different than the training sets. There are no significant differences between the performance of the dimensional and non-dimensional regression. Normally non-dimensional parameters are more desirable than dimensional ones. However, the so-called dimensional regression only includes wave fetch band which is also used in the non-dimensional regression. Also the additional parameter in the non-dimensional regression narrowly passed the significance test. Simpler models are preferred especially when the data is noisy and/or the fits are not good. As a result of these considerations the results from the dimensional regression (which are actually non-dimensional) is recommended for the wave height modifier.

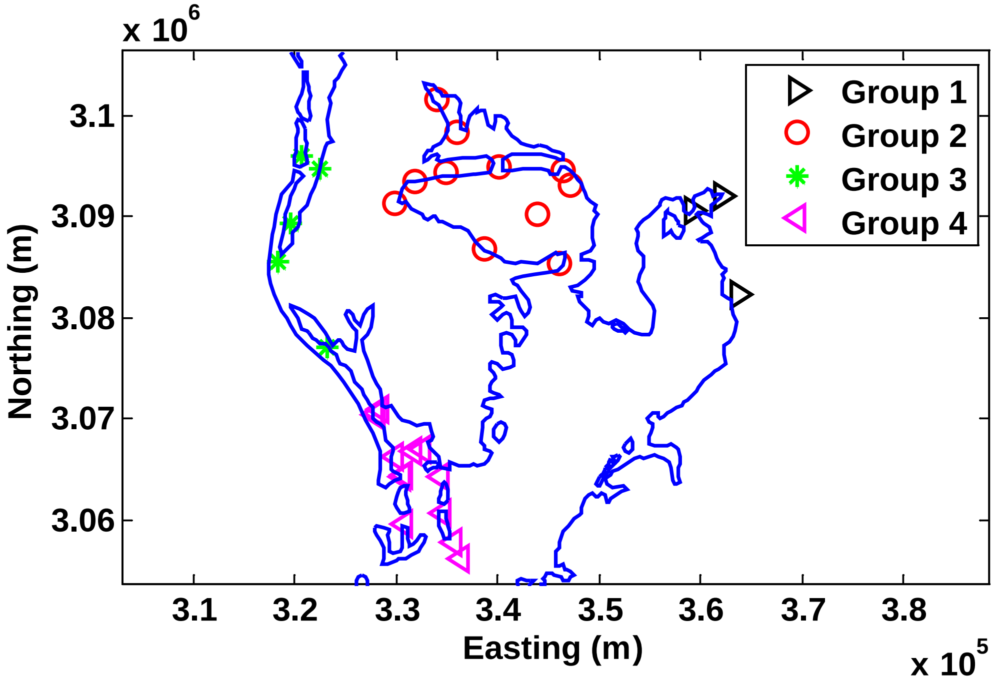

Figure 5-1 Map of the study area showing the four groups of bridges used in the testing procedure.

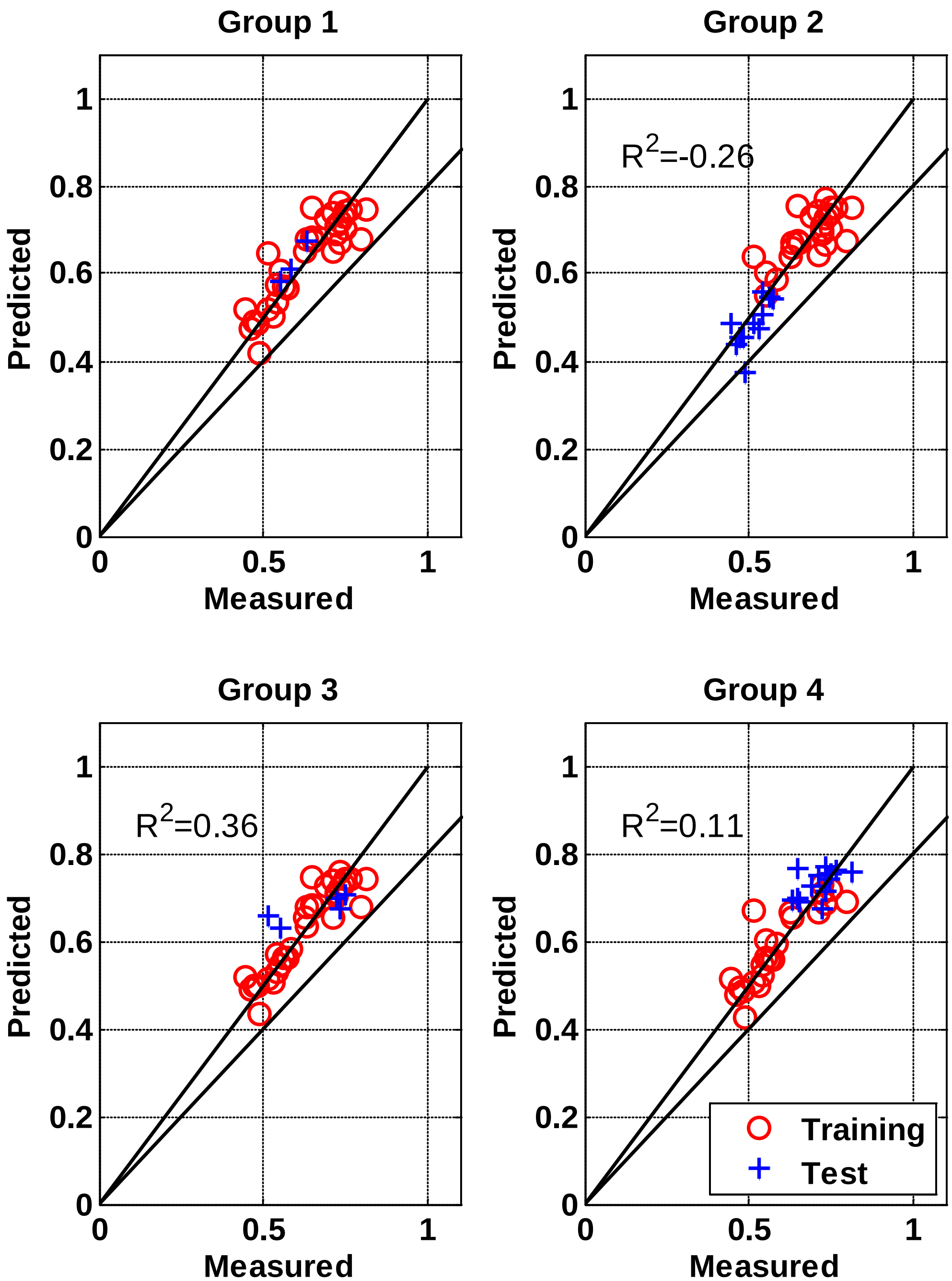

Figure 5-2 Regression test results for dimensional WSE with sub-groups.

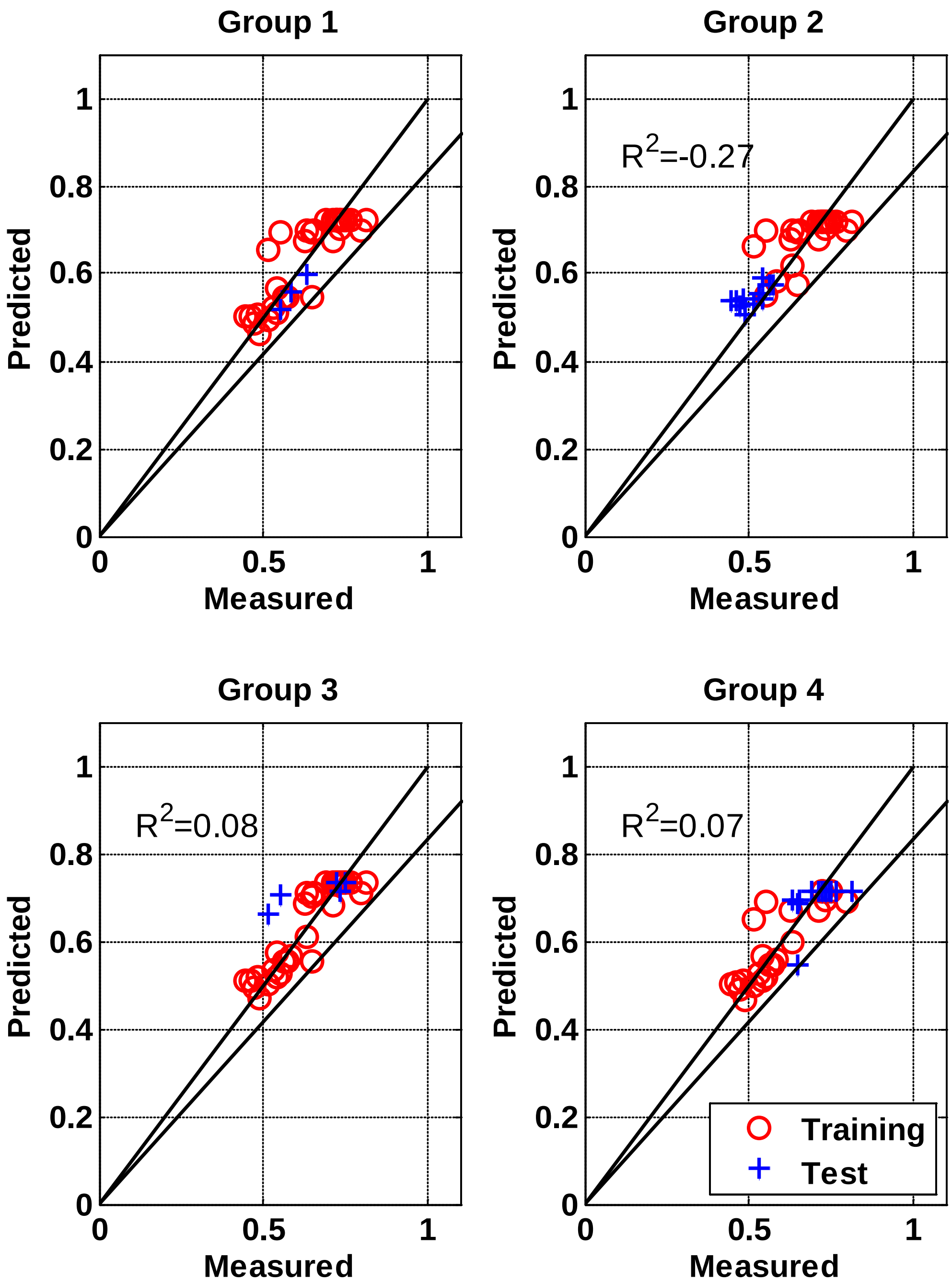

Figure 5-3 Regression test results for non-dimensional WSE with sub-groups.

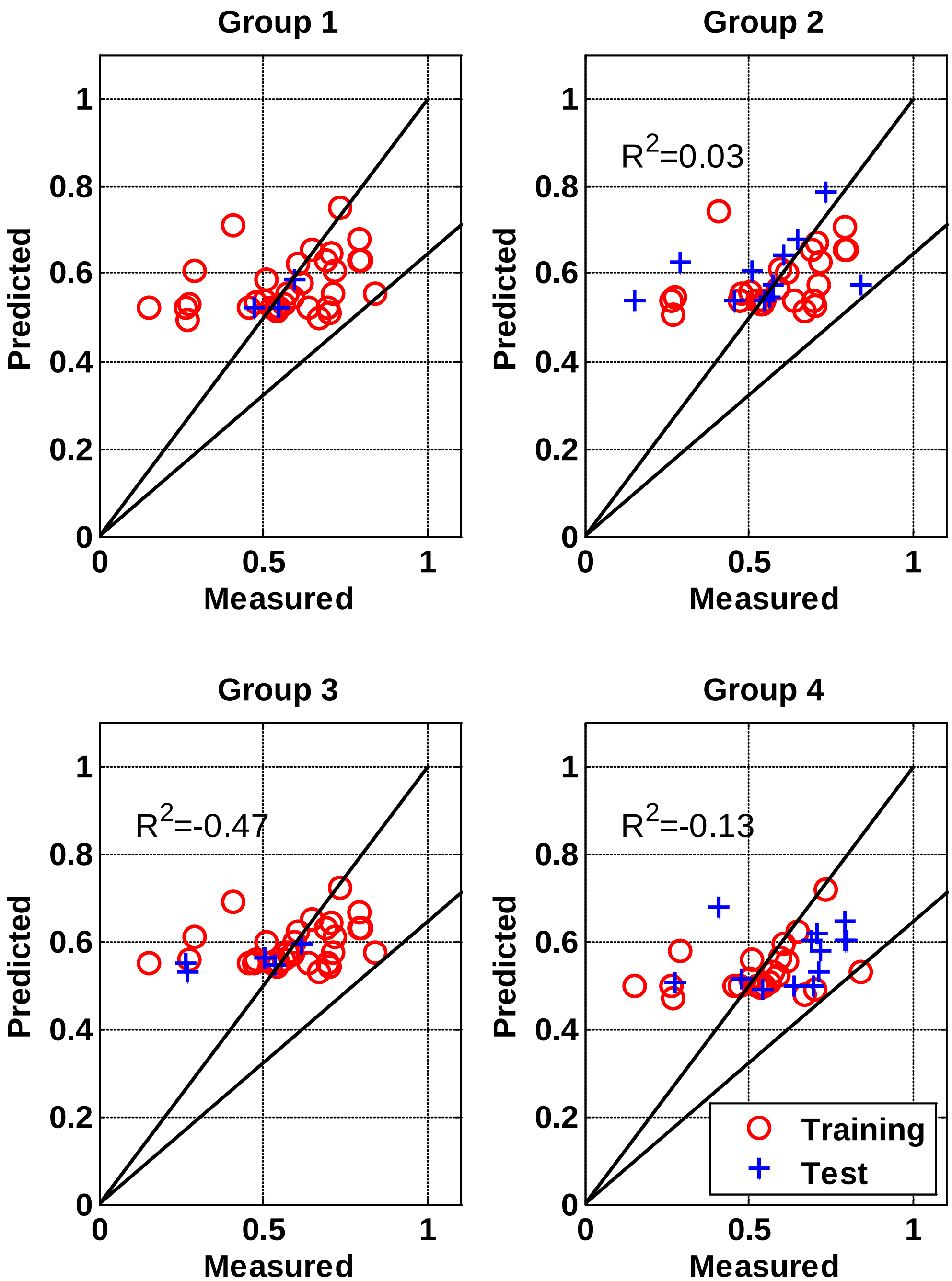

Figure 5-4 Regression test results for dimensional significant wave height with sub-groups.

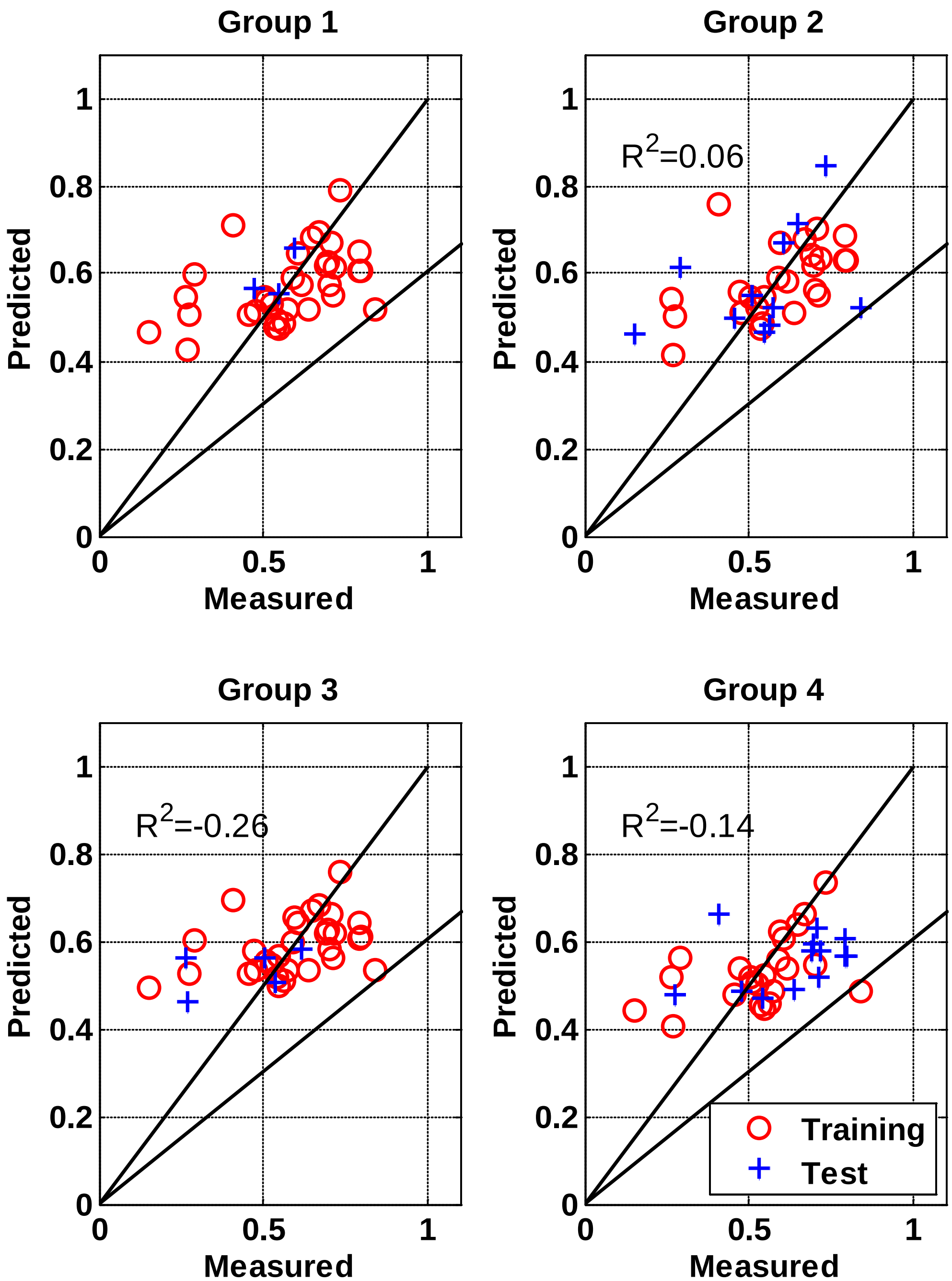

Figure 5-5 Regression test results for non-dimensional significant wave height with sub-groups.

6. Conclusions

The objective of this investigation was to see if there were ways to reduce the differences between Levels I and III predictions of design water levels and wave heights. By necessity Level I predictions are conservative since there is no way to account for the phasing between the various mechanisms that produce both water levels and wave heights in this analysis. There are other parameters such as the location of the point of interest (bridge) in the water body, the orientation and geometric features of the water body in the immediate vicinity of the point of interest, etc., that are also not considered in the Level I analysis. This study used the data generated in the FDOT Pilot Study to see if correlations between the ratio of Level III to Level I design water level (and wave height) and a number of these parameters could be obtained.

Level I predictions are based on existing data and information. The source of the Level I storm surge data for the Pilot Study was the Federal Emergency Management Agency (FEMA). The FEMA storm surge values at that location are conservative relative to the values produced by the Pilot Study Level III analysis. That is, the FEMA storm surge elevations are greater than those from the Level III analysis. At most coastal locations the only 100-year water elevation predictions will be those published by FEMA. The accuracy of these values varies from site to site and it is often difficult to determine if local wind setup was included in their analyses. The water elevation modifiers developed in this study are influenced by the level of conservatism in the storm surge data used for the Level I analysis as well as the phasing of the various components and water basin geometries discussed in the body of this report. It is therefore important that certain rules be followed when applying the modifiers developed in this study to other locations. These rules are listed at the end of the following section. The intent of these rules is to insure that the water levels (storm surge plus local wind setup/set-down) used in the Level I analysis have the approximate level of conservatism as those used in the development of the modifiers.

The regression equations for water surface elevation (WSE) in terms of both the dimensional and non-dimensional quantities gave good fits to the measurements. Attempts to find correlations between the ratio of Level III to Level I wave height and a number of geometric and bathymetric parameters were not as successful. However, it does appear that some reduction in Level I wave heights can be made and still retain their conservative nature.

The results of this study are based on data and information from one Florida coast location, namely the Tampa-Saint Petersburg area (FDOT D7). They appear to work well for this location but it is not known how well they will perform at other locations where the nature of the storms and the water body parameters are different. They may work equally well but they have not been tested at other locations. It should also be pointed out that even though the quantities in the modifier equations are readily available, a certain level of understanding of coastal and ocean processes is needed in order to obtain accurate values.

7. Recommendations

The equations for modifying Level I water surface elevations and wave heights developed in this study are as follows:

Design Water Surface Elevation

$$\text{MWSE} = \text{Level I WSE} * \text{WSEMOD}, \tag{9}$$

where

MWSE = Modified Water Surface Elevation,

Level I WSE= Level I Water Surface Elevation,

WSEMOD = Level I Water Surface Elevation Modifier,

Wind Setup = Wind setup calculated using OEA methodology. Note that wind setup can be negative (i.e., there can be set-down at the bridge),

Storm Surge = Level I storm surge that includes only the effect of offshore surge as it propagates from the open coast into the bay system, and

$$\text{WSEMOD} = \begin{cases} 0.87 - 0.13\left(\dfrac{\dfrac{\text{Wind Setup}}{\text{Storm Surge}} - 0.51}{0.54}\right) & \text{if Wind Setup} > 0 \\ 1.00 & \text{if Wind Setup} \le 0 \end{cases} \tag{10}$$

Design Wave Height

$$\text{MWH}=\text{Level I Hs}*\text{WHMOD}, \tag{11}$$

where

MWH= Modified Significant Wave Height,

Level I H_s= Level I Significant Wave Height,

WHMOD= Level I Significant Wave Height Modifier,

$$\text{WHMOD} = \begin{cases} 0.88 + 0.10\left(\dfrac{\text{Wave Fetch Band} - 49}{39}\right) & \text{if Wave Fetch Band} \le 95 \\ 1.00 & \text{if Wave Fetch Band} > 95 \end{cases}$$

and, Wave Fetch Band = Range of wave fetch angles, in degrees.

Rules for Applications:

Due to the methods and data used in the development of the modifier equations above, certain rules need to be followed for their application. These are listed below:

1. The design (100-year) storm surge used in the Level I analysis must be assumed to not include Local Wind Setup/Set-Down.

2. The Local Wind Setup/Set-Down must be computed using the method developed by OEA for FDOT and described in the FDOT Report entitled “Development of Analytical Equations for Wind Setup for Bays, Estuaries, and Lakes”. This document and the accompanying computer program can be downloaded from the FDOT Drainage website.
3. The methods used to produce the design storm surge used in the Level I analysis (usually FEMA) need to be investigated to assess their potential accuracy. When possible their predicted values should be compared with predictions from other sources, such as state and other federal agencies. It is important that the storm surge values used in the Level I analysis be conservative. The use of non-conservative storm surge values will result in non-conservative force and moment predictions.

8. References

American Association of State Highway and Transportation Officials (AASHTO). (2008). “Guide Specification for Bridges Vulnerable to Coastal Storms.” 1st edition. https://bookstore.transportation.org/item_details.aspx?ID=1365

Dompe, Phil, Huseyin Demir and D. Max Sheppard (2010). “Wave Vulnerability Pilot Study FDOT District 7”. Florida Department of Transportation, Tallahassee, FL.