TASK SUBMISSION

TASK 6 – Guide Specification and Retrofit Manual
Task 6a, 50% Guide Specifications

May 15, 2007

TASK ORDER DTFH61-06-T-70006

FOR THE DEVELOPMENT OF
GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS AND HANDBOOK OF RETROFIT OPTIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

by

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<u>**Possible Organization and Content for Coastal Specifications**</u>

(Article Numbers Relate to LRFD Specifications)

May 15, 2007

Definitions

Nomenclature

2.3.3 Clearances

2.3.3.2

2.3.3.2.1 Overhead Clearance

[Existing 2.3.3.2]

2.3.3.2.2 Storm Clearance

Wherever practical, the vertical clearance of highway bridges should be sufficient to provide at least 1 foot of freeboard over the design wave crest elevation, (which includes the design storm water elevation defined in Article 3.7.X, considered in Article 3.7.4). For bridge spans where this is not possible other design strategies may be considered including those identified in Article 2.7. Bridge spans satisfying this criterion require no further investigation of wave effects or storm water effects on the superstructure. Wave effects on substructure shall still be investigated in accordance with the provisions of Articles 3.7.2 and 3.7.4.

C2.3.3.2.2

Setting vertical elevations to keep as much of the structure as possible above the design wave crest elevation clearly decreases the vertical and horizontal surge and wave induced forces.

2.7 Design Strategies for Coastal Storms

2.7.1 General

2.7.2 Avoidance

The provisions of Article 2.3.3.2.2 shall apply.

2.7.3 Force Mitigation

Where it is not possible to provide the vertical clearance recommended in Article 2.3.3.2.2, the following may be considered to reduce the wave forces acting on the superstructure:

- Setting the vertical elevation as high as practical

- Use of open or sacrificial parapets
- Venting the potential cells that could entrap air creating increased buoyancy forces
- Use of large holes in concrete diaphragms or framed cross-frames and end diaphragms on concrete superstructures to promote venting and the exchange of trapped air between spans
- Use of continuous superstructures to increase the reactive force of individual spans
- The use of solid or voided slab bridges to reduce buoyancy forces
- Etc.

C2.7.3

Some of the force mitigation measures specified in this article are based on observations of the response of structures to coastal hurricanes.

At Lake Pontchartrain Bridge in Louisiana, 14,000 feet of parapet were broken off. This suggests that the use of a sacrificial parapet that would respond inward in a sacrificial manner while still providing the proper traffic barrier resistance outward could reduce the amount of area exposed to the waves. This could also promote inundation which reduces the total wave force to be resisted.

Calculated estimates of the effect on entrapped air on the vertical wave forces on the Lake Pontchartrain I-10 Bridge and the Escambia Bay I-10 Bridge have shown that the vertical force can be substantially reduced if the amount of air entrapped between the beams can be reduced. Calculations based on venting the cavities formed by beams and diaphragms on selected spans from those two bridges indicated that it was not practical to drill deck holes to vent air entrapped by waves. Holes could be effective in reducing the amount of air entrapped when the still water elevation is between the bottom of the beams and the bottom of the deck. This is because the surge effects that create the still water elevation occur over a much longer time frame than wave action.

The use of large holes in concrete diaphragms, framed cross-frames and end diaphragms, or concrete partial depth diaphragms can create large holes which can be effective in venting entrapped air and allowing the exchange of trapped air between spans. The figure below shows the area of hole necessary to permit evacuation of a volume of air in one second, a time period short enough to represent wave attack.

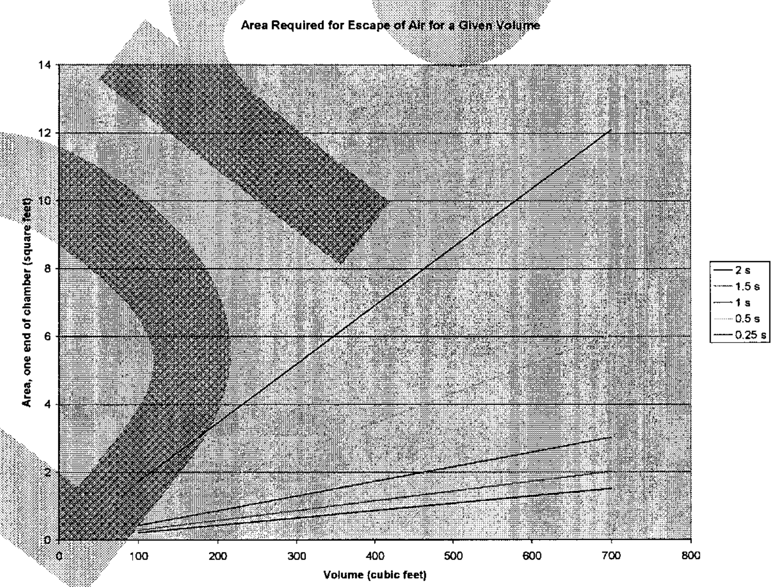

Figure C2.7.3 -- Venting Requirements

Figure to be replaced with one using vertical wave velocity as basis.

Continuous superstructures appear to have benefits due to the three-dimensionality of the waves because storm waves have finite crest lengths and the chance of multiple spans being struck by design waves at the same time is small. Thus, the ability of the structure to resist vertical and horizontal forces are increased through continuous spans.

The use of slab bridges may be especially appropriate for those spans that cannot be raised sufficiently to avoid wave forces such as those near the ends of bridges which have grade constraints.

2.7.4 Force Accommodation

2.7.4.1 General

Design for coastal storms may be based on any of the strategies identified herein. Design and detailing should achieve an engineered response involving avoiding wave loads, accommodating the full loads, accommodating partial loads with superstructure damage or loss above a chosen load, or submergence. The engineered responses other than avoidance should be predicted using design parameters and the methods outlined herein, and designed to protect the substructure so that it can be reused if the superstructure is lost.

C2.7.4.1

In recent cases where superstructure was lost but substructure remained largely re-usable, it was possible to re-open Bridges with either temporary superstructures or permanent superstructures in much less time, and at much lower cost than if the substructure had been functionally destroyed by the combination of forces transmitted from the superstructure and those applied directly to the substructure. Therefore, design to protect the substructure is recommended herein.

Where partial or complete force accommodation is provided, there may be significant upward forces due to hydro-dynamic and hydrostatic (buoyancy) effects, which may cause a reversal of the normal moments and shears. This requires investigation.

2.7.4.2 Design for the Full Wave Loads

The structure may be designed to resist the loads calculated in accordance with the provisions specified in Articles 3.7.4.3 and 3.7.4.4.

2.7.4.3 Fusing for Partial Loads

Where design for the full wave loads specified in Articles 3.7.4.3 and 3.7.4.4 is not justified by the construction cost impacts or the importance of the bridge, the owner may design the superstructure to break away from the substructure at less than the full loads.

C2.7.4.3

Various concepts for fusing parts of structures to dissipate the energy of seismic events have been considered and applied. Many of these applications used plastic bending deformation to create the fuse effect. This is not necessarily applicable to the coastal storm situation as the amount of deformation would have to be considerable. The concepts of fusing that are applicable to the coastal storm situation involve units designed to fail in tension or separate in some manner allowing the superstructure to float free of the substructure preserving the substructure for future use.

2.7.4.4 Sacrificial Superstructure [May be overlapping with earlier articles.]

Superstructures may be designed to separate from the substructure either under the action of vertical forces, horizontal forces, buoyancy or any combination thereof. Past experience has shown that freed superstructure units have caused damage to substructure as they moved away.

C2.7.4.4

Sacrificial superstructures are a variation of the fusing for partial loads specified under Article 2.7.4.3. In some cases where it is not possible to elevate structures or to resist the loads in an economical and safe way, it may be necessary to sacrifice low level spans and to replace them after the storm.

2.7.5 Submersible Superstructures

Spans may be designed to be totally inundated at the design wave crest elevation, provided they can be designed to resist the forces caused by waves with lower wave crest elevations which impinge upon the structure.

C2.7.5

Submersible structures may have application in low level approach structures similar to situations were sacrificial superstructures are also applicable. Wave forces will tend to be smaller once the structure is totally submerged in the water. Submersible heavy structures that close voids thereby reducing the buoyancy effect may be a cost effective solution in some cases.

3.3.2 Add Waves as a Load

3.4.1 Load Combinations

Add Strength Load Combination VI to Table 1 as follows:

$$\gamma_d DC + \gamma_d DD + \gamma_d DW + \gamma_d EL + \gamma_{wave} WA + 1.4 WS$$

Where: WA=wave forces specified in Article 3.7.4

3.4.1.X Wave Force Load Modifiers

Each of the three levels of wave force determinations specified in Article 3.7.X yield loads which shall be modified based on site-specific parameters as specified below:

- For Level I, the load specified in Table 1 may be used either without further modification or may be multiplied by the site adjustment factors specified in Figure 1;
- For Level II, the load specified in Table 1 shall be modified as specified in Table 1 and, in lieu of better site specific data, may be multiplied by the site adjustment factors specified in Figure 1;
- For Level III, the load specified in Table 1 shall be modified dependent on the level of analysis of site specific parameters.

For a Level III analysis the site factor specified in Figure 1 is not applicable because the bridge location in the body of water will be rigorously included in the required modeling.

REPLACE WITH MAX'S FIGURE OF BAY

Figure 3.4.1.X-1 Adjustment Factors Based on Position of Structure within Body of Water.

Table 3.4.1.X-1 Load Factor Modifiers for Level II Determination of Wave Forces.

Table 3.4.1.X-2 Load Factor Modifiers for Level III Determination of Wave Forces.

LOAD ADJUSTMENTS WILL BE BASED ON FDOT PROJECTS BUT PROBABLY NOT BEFORE FEB. '08.

LOAD MODIFIERS COULD BE MOVED TO APPROPRIATE SUBARTICLES IN ARTICLE 3.7.4.

C3.4.1.X

3.7.2 [Add Air Entrapment]

3.7.4 Hydrodynamic Loads and Design Parameters

3.7.4.1 General

The provisions of this article shall be taken to apply to bridges located in areas where they may be impacted by coastal storm events.

Information required for establishment of structure vertical alignment and determination of coastal storm forces on the structure should include as a minimum:

- Bridge location within the water system
- Bridge elevation
- Structure configuration
- Bathymetry
- Fetch length and orientation relative to the open coastline
- Fetch angle segment for waves
- Fetch angle segment for wind setup/setdown
- Design wave height, period and wavelength
- Design wind velocity
- Design water elevation composed of: (1) astronomical tide, (2) storm surge created by reduced atmospheric pressure, wind stress on water surface and wave setup, and (3) local wind set-up/set-down
- Design current velocity

[Figure to illustrate fetch angle segment for waves and wind setup/setdown.]

The determination of the appropriate design parameters may proceed according to the three levels of analysis specified in Articles 3.7.4.4 through 3.7.4.6. Determination of which level to use shall be based on the replacement value and importance of the structure under consideration and site specific parameters such as the complexity of the water boundaries and bathymetry, quantity and quality of met/ocean data for the site, etc.. A Level I analysis (Article 3.7.4.4) may be used initially to determine if a more sophisticated analysis is necessary. Alternatively, Level I may be bypassed when the conditions at a particular site and/or the importance of the bridge clearly indicate that a higher level of analysis is appropriate.

Input from a qualified coastal engineer experienced in the determination of these design parameters shall be obtained for Level I analyses. Level II and Level III analyses shall be performed by a qualified coastal engineer experienced in the determination of these design parameters.

C3.7.4.1

The load factors presented in Section 3.4.1 are based on a design event that is assumed to be a one in one-hundred year (referred to here as one hundred year) event. For the Level I and Level II analyses discussed in Section 3.7.4.4 and Section 3.7.4.5, the initial definition of such an event is the 100-year return period wind velocity combined with the 100-year return period wave height (and period) and the 100-year return period water level and the 100-year return period current speed. However, due to the fact that these parameters are not necessarily 100% correlated for coastal storm events, this definition may yield results that are conservative, and in many cases may be too conservative. How much greater depends primarily on site specific parameters. Therefore, adjustments to the load factors presented in Section 3.4.1 are made based on site specific parameters that are illustrated by examples.

The forces exerted on a bridge superstructure by elevated water levels and waves depend on all the quantities that govern the magnitudes of these parameters as identified in this article, as well as the size, shape and elevation of the superstructure. The most accurate way to estimate 100-year loads for an important or expensive bridge is with a Level III analysis where the forces on the superstructure produced by the most significant storms at that location are recreated (hindcasted) and an extremal analysis performed. The purpose of the Level III analysis is to better ascertain the design parameters. The Level III analysis will require more extensive data collection and the use of more sophisticated computer numerical and / or analytical modeling techniques available to the coastal engineering community as discussed in Section 3.7.4.6.

The criteria to establish suitable credentials in coastal engineering are not fully developed at this time. Until such time as a consensus on certification is reached, the following statement developed by the Florida DOT may be considered.

> "A Coastal Engineer must hold a M.S. or Ph.D. in Coastal Engineering or a related Engineering field and/or have extensive experience (as demonstrated by technical publications in technical journals with peer review) in coastal hydrodynamics, wave mechanics, and/or sediment transport processes. If computer modeling of storm surge, waves, etc. is required, demonstrated expertise/experience in this area is also required."

Conditions that typically require direct attention by a Coastal Engineer are listed below:

- Hydraulic analysis of complex geometry tidal water bodies,
- Hindcasting of historical hurricane events,
- Determination of design wave parameters,
- Analysis of inlet or channel instability, either vertically or horizontally,
- Prediction of potential wave scour at bridges and seawalls,
- Design of countermeasures for wave induced erosion/scour at bridge abutments and approaches,
- Prediction of barrier island overtopping and channel cutting
- Design of countermeasures for inlet instability, wave attack, or channel cutting
- Prediction of global coastal sediment transport or design of countermeasures to control global sediment transport
- Assessment of wave loading on bridges and other structures
- Determination of design hurricane parameters

3.7.4.2 Hydrodynamic Loads on Superstructure

3.7.4.2.1 General

The following contributors to hydrodynamic loads on superstructures shall be considered as appropriate:

- Buoyancy
- Drag and inertia forces

- Forces associated with added mass
- Slamming forces

C3.7.4.2.1

The hydrostatic and hydrodynamic forces acting on bridge superstructures are composed of several components that, in general are not in phase. For most bridge span configurations, the critical loading situation occurs when the vertical forces are at their maximum value. The forces computed using the equations and tables presented below are the total forces on the span when the vertical component of the force is a maximum. That is, the vertical force is the maximum value experienced by the span during the passage of design waves and the horizontal force and moment are the values at the time of maximum vertical force. The forces and moments not only depend on the variables in the equations but on the number of girders, the girder height and the percent air entrapment as well. These additional quantities are accounted for by coefficients presented in tables. All forces are a parameterization of the results from detailed analyses using a modified and extended form of Kaplan's equations.

Vertical hydrostatic forces are imparted to structures once any portion of the structure is submerged. These forces are called buoyancy forces. When there is motion of the water in contact with the structure (relative to the structure) hydrodynamic forces are imparted to the structure. If the water is accelerating, as is the case when waves are present, then additional forces are exerted on the structure. These forces are called inertia forces. For structures, such as bridge spans, subjected to water waves the volume of the submerged structure can change with time. This creates an additional force referred to as the "change in added mass force". With the exception of the buoyancy force (which only acts in the vertical direction) all of these force components act in both the horizontal and vertical directions. That is, both the horizontal and vertical forces are composed of drag, inertia and change in added mass forces. All of these force components have frequencies close to the wave frequency. The sum of these force components has been referred to as the "quasi-static wave force" in the literature.

If the low member elevation of the superstructure is above the elevation of the trough of the design wave then there is yet another wave force imparted to the structure. When the air-water interface strikes the structure, an impulse or slamming force is exerted on the structure. The magnitude of the slamming force can be as large as or greater than the quasi-static force. For design purposes the total vertical force for these situations is the sum of the quasi-static and the slamming force.

3.7.4.2.2 Hydrostatic and Hydrodynamic Forces

3.7.4.2.2a Vertical Forces

The vertical force is composed of two parts referred to herein as 1) the quasi-static and 2) the slamming force. These forces depend on slightly different parameters and thus are presented separately. The vertical slamming force often occurs at the time of the maximum upward vertical quasi-static force and thus the two should be added to achieve the total upward vertical force on the superstructure. There are corresponding horizontal forces and moments about the trailing edge of a bridge cross-section at the time of maximum vertical force. These quantities are presented below as well and should be used in conjunction with the vertical force in the structure response analysis

Quasi-Static Vertical Force.

$$\frac{F_z}{F^*} = a\left(\frac{W}{\lambda}\right)^b \qquad (3.7.4.2.2a\text{-}1)$$

in which:

$$a = C_{z1}\left(\frac{Z_c}{\eta}\right) + C_{z2}, \qquad (3.7.4.2.2a\text{-}2)$$

$$b = \begin{cases} C_{z3}\left(\frac{Z_c}{\eta}\right) + C_{z4} & \text{for } \left(\frac{Z_c}{\eta}\right) \geq 0 \\ C_{z6}\left(\frac{Z_c}{\eta}\right) + C_{z4} & \text{for } \left(\frac{Z_c}{\eta}\right) < 0 \end{cases} \qquad (3.7.4.2.2a\text{-}3)$$

$$F^* = \gamma_w W \beta$$

$$\beta = \begin{cases} 0 & \text{for } (\eta - Z_c) \leq 0 \\ (\eta - Z_c) & \text{for } 0 < (\eta - Z_c) \leq t \\ t & \text{for } (\eta - Z_c) > t \end{cases} \qquad (3.7.4.2.2a\text{-}4)$$

where:

F_Z	=	vertical quasi-static hydrostatic and hydrodynamic force per unit length of the span
t	=	girder height + deck thickness (ft)
λ	=	wave length obtained from Figure 2 (ft
C_{z1}–C_{z6}	=	coefficients for vertical wave forces specified in Table 3.7.4.2.2-1
W	=	bridge width (ft)
Z_c	=	vertical distance from bottom of cross-section to the storm water level, positive if storm water level is below the bottom of the cross-section (ft)
η	=	distance from the storm water level to design wave crest (obtained from Figure 1)
γ_w	=	unit weight water taken as 0.064 (k/ft^3)

The dimensions Z_c and η shall be determined based on the wave and surge heights consistent with level of analysis using the provisions of Articles 3.7.4.4 through 3.7.4.6.

Table 3.7.4.2.2a-1 – Coefficients for Quasi-Static Vertical Loads

Girder Type	100% Entrapped Air Number of Girders																	
	4						8						12					
	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}
Florida Bulb-T 72																		
ASHTO Type II																		
ASHTO Type III																		
Slab Bridges																		
Steel Girders																		

Girder Type	100% Entrapped Air Number of Girders																	
	4						8						12					
	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}
Florida Bulb-T 72																		
ASHTO Type II																		
ASHTO Type III																		
Slab Bridges																		
Steel Girders																		

Girder Type	100% Entrapped Air Number of Girders																	
	4						8						12					
	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}	C_{z1}	C_{z2}	C_{z3}	C_{z4}	C_{z5}	C_{z6}
Florida Bulb-T 72	-0.306	0.406	0.228	-0.963	0	-0.271												
ASHTO Type II																		
ASHTO Type III																		
Slab Bridges																		
Steel Girders																		

Slamming Force

<Equation, Definitions and Tables>

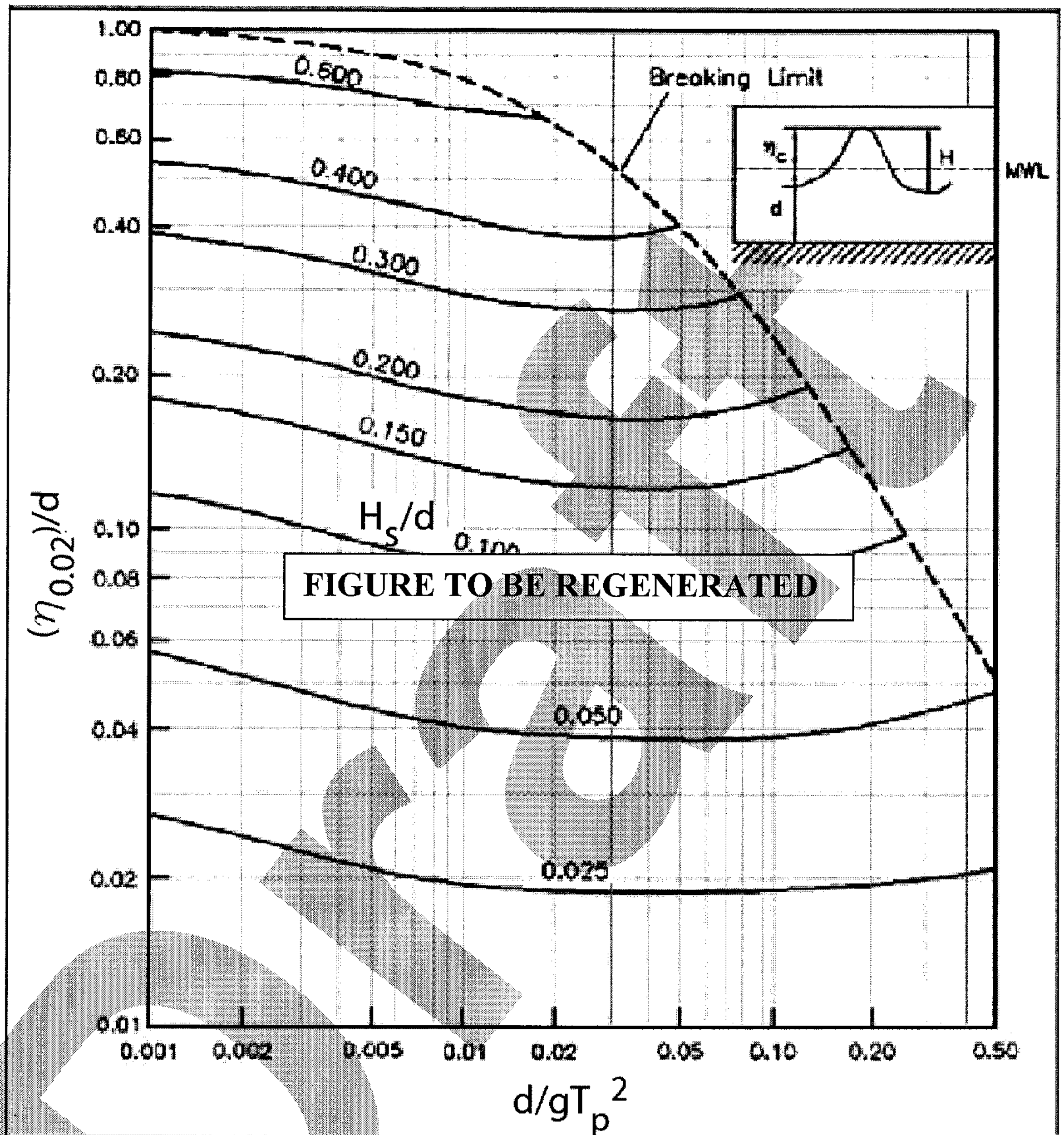

Figure 3.7.4.2.2a-1 This graph can be used to obtain the 2% highest wave crest elevation, $h_{0.02}$, knowing the storm water depth, d, the significant wave height, H_s, and the peak period, T_p.

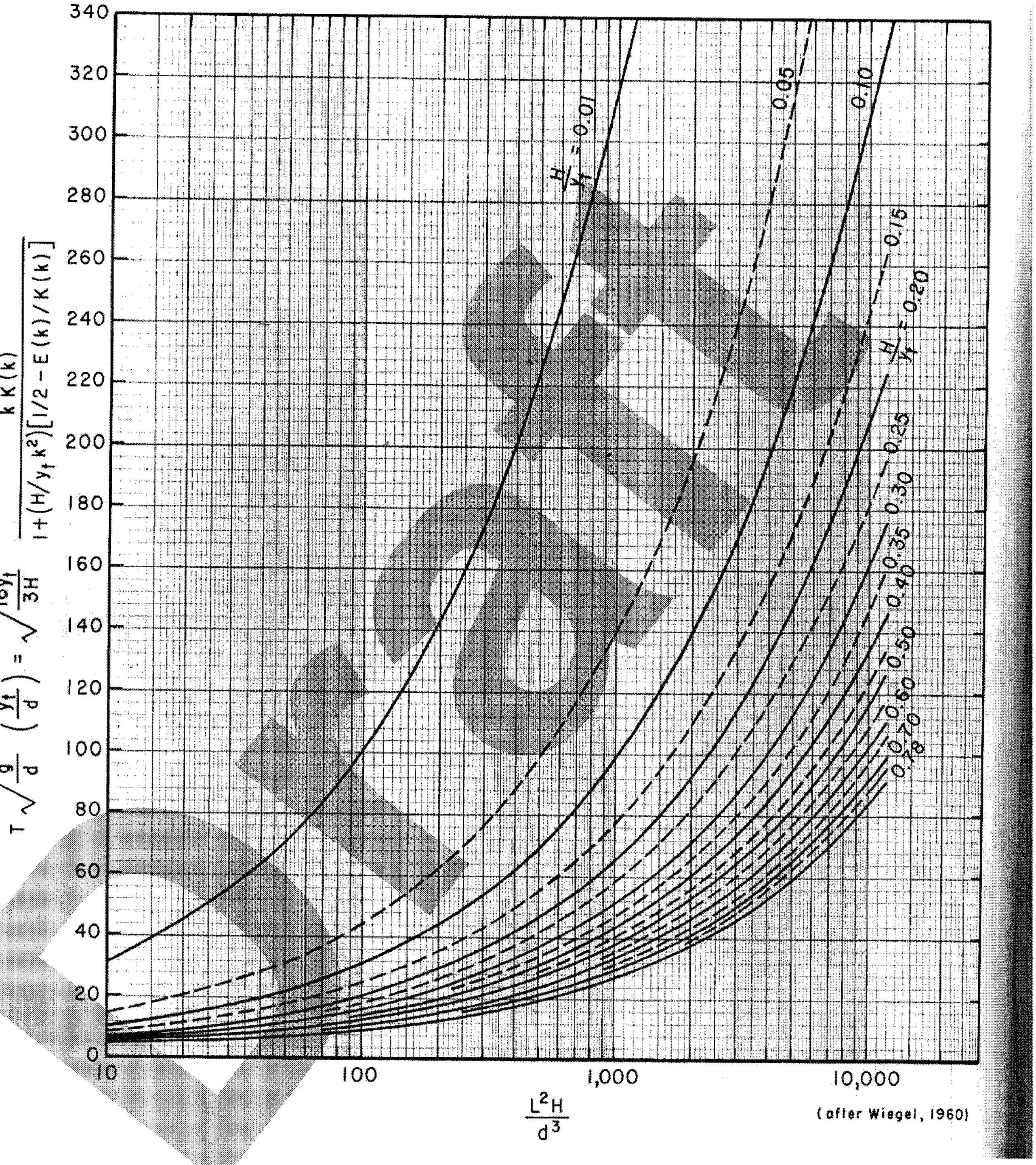

Figure 3.7.4.2.2a-2 (Placeholder) This graph can be used to obtain the wavelength, knowing the storm water depth, d, the significant wave height, H_s, and the peak period, T_p.

C3.7.4.2.2a

Equations 3.7.4.2.2a-1 through 3.7.4.2.2a-3 and Tables ?? through ?? are the result of extensive studies described in Reference X (Project Report). These studies evaluated and compared several methods for predicting hydrostatic and hydrodynamic forces on bridge superstructures due to elevated water levels and waves. The methods were tested with laboratory data from 1/8 scale model wave tank tests at the University of Florida and field data from the I10-Escambia Bay Bridges (damaged during Hurricane Ivan). The criterion used to evaluate the various methods included:

1. Correlation to experimental results: The selected method should result in force magnitude that correlates well with the measures forces in laboratory test.
2. Prediction of forces that led to failure: The calculated forces should exceed the force required to cause the bridge failures observed in past hurricanes.
3. Theoretical Completeness: Preferably, the selected method should be supported by theory.
4. Practicality: The selected method should be easy to interpret and simple to apply.

The Modified Kaplan Method was selected based on its better representation of the physics of the fluid structure interaction and its ability to better predict the laboratory and field results. The equations are, however, complex and difficult to evaluate thus once the equations and methods were refined to the point of producing acceptable results they were used in a series of numerical experiments over a wide range of water depths, wave conditions and structure parameters to compute wave forces. Data from these numerical simulations were then used to produce design curves and equations in terms of dimensionless groups involving water depths, wave conditions, and structure parameters.

When the bottom of the superstructure is above the trough of the wave the structure can experience a slamming force when the water surface strikes the underside of the span. None of the predictive equations, including the Modified Kaplan, compute the slamming force directly. The slamming force in the design equations above are based on the results from the University of Florida model wave tank tests.

As explained in Reference X in more detail, a procedure was developed to calculate discrete forces at several thousand locations on a cross-section due to a time variant wave, impacting and/or passing over the structure using the equations of the modified Kaplan Method. These discrete forces were then used in numerical simulation to determine the net vertical, horizontal and moment applied to the cross-section. It was determined that for the geometries under consideration, the maximum net result occurred when the vertical force was maximized and the corresponding horizontal force and moment were determined at the time and under the condition for which the vertical load was maximized. Non-dimensional plots were developed to express the results of these simulations as shown in Figures C1 through C4 which are a representative group of figures selected from many possibilities. The results of the simulated modified Kaplan equations were then curve fit to produce the results shown in this article.

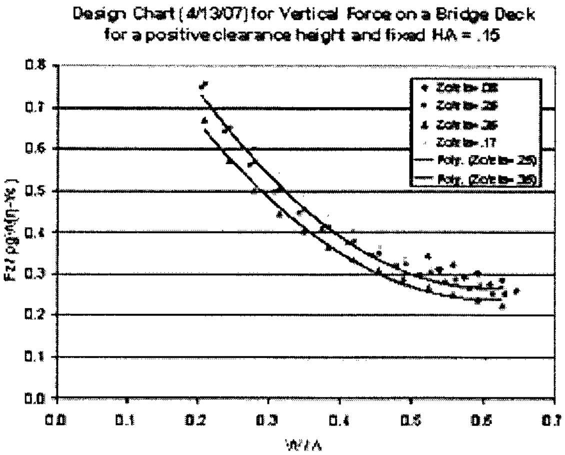

Figure C3.7.4.2.2a-1 -

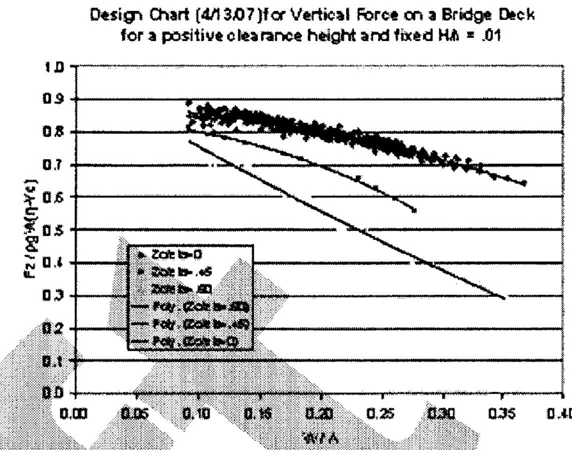

Figure C3.7.4.2.2a-2 -

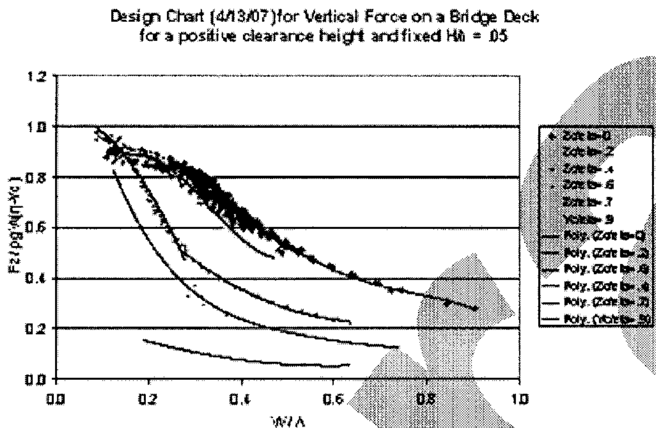

Figure C3.7.4.2.2a-3 -

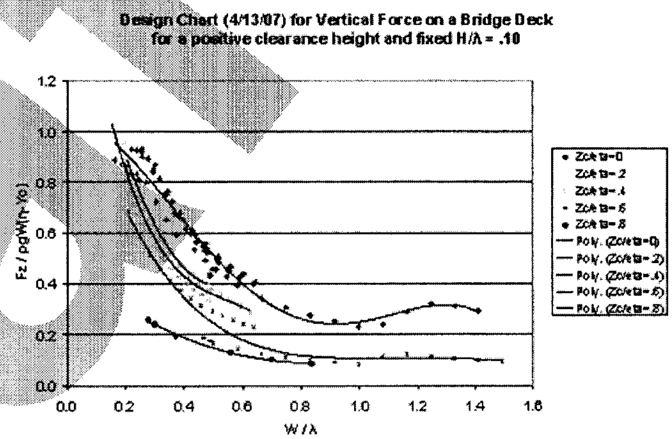

Figure C3.7.4.2.2a-4 -

3.7.4.2.2b Associated Horizontal Force (Similar)

3.7.4.2.2c Associated Moment (Similar)

3.7.4.3 Hydrodynamic Loads on Substructure

3.7.4.3.1 General

Loads which may be imparted to substructure elements such as the piles, pile cap or piers shall be considered. For the purpose of this article, substructure elements can be classified into two categories:

- Small elements whose presence does not strongly disturb the incident wave field. For circular piles they would have a diameter to wavelength ratio of less than 0.2; and
- Large elements that do affect the incident wave field.

3.7.4.3.2 Small Structural Elements

For small dimension structural elements, the Morison equation should be used for determination of forces due to non-breaking waves shall be determined as.

$$F = C_d(\rho/2)Au|u| + C_m(\rho)V\frac{du}{dt} \qquad (3.7.4.3.2\text{-}1)$$

where:

F	=	force on element per unit length
ρ	=	mass density of water taken as 2.0 slugs/ft^3
u	=	horizontal component of water particle velocity
A	=	projected area per unit length (for a circular pile of diameter D, A = D)
V	=	displaced volume per unit length (for a circular pile of diameter D, V = $\pi D^2/4$)
C_d	=	Morison drag coefficient
C_m	=	Morison inertia coefficient
du/dt	=	horizontal acceleration of water particles

The total force shall be calculated taking account of the phase difference between the drag and inertia components. To identify the maximum combined drag and inertia force, the force shall be determined for various time increments during the passage of the wave train.

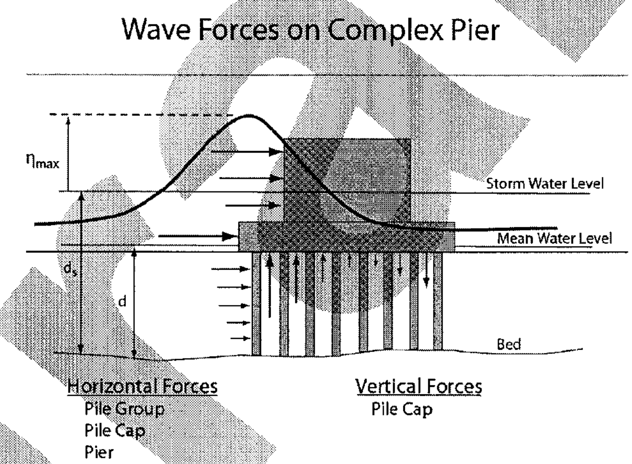

Figure 3.7.4.3.2-1

C3.7.4.3.2

Equation 1 is known as the Morison equation (Morison XXXX).

API RP2A – LRFD recommends, for the case of large waves with UmoTapp / D > 30, the following values for circular cylinders:

Smooth: Cd=0.65, Cm = 1.6
Rough: Cd=1.05, Cm = 1.2

where:

Umo	=	maximum wave induced orbital velocity
Tapp	=	apparent wave period (accounting for design current)
D	=	diameter of the cylinder

For other cases, suggested values are presented in the commentary of API RP2A-LRFD as well as numerous coastal and ocean engineering references.

3.7.4.3.3 Large Structural Elements

For large dimension structural elements the wave pressure on the front of a vertical wall shall be determined as:

$$p_1 = \frac{1}{2}(1+\cos\beta)(\alpha_1)\gamma_w H_{\max} \qquad (3.7.4.3.3\text{-}1)$$

$$p_2 = \frac{p_1}{\cosh(2\pi d/\lambda)} \qquad (3.7.4.3.3\text{-}2)$$

in which:

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi d/\lambda}{\sinh(4\pi d/\lambda)}\right]^2 \qquad (3.7.4.3.3\text{-}3)$$

$$\eta^* = 0.75(1+\cos\beta)H_{\max} \qquad (3.7.4.3.3\text{-}4)$$

where:

β	=	angle between direction of wave approach and a line normal to the structure
H_{max}	=	the highest wave in the design sea state
d	=	water depth
λ	=	wave length
γ_w	=	density of water taken as 0.064 k/ft^3
η^*	=	potential height above the storm water level to which wave pressure could be exerted

The calculation of applied pressure shall be based on pressure prism specified in Figure 1.

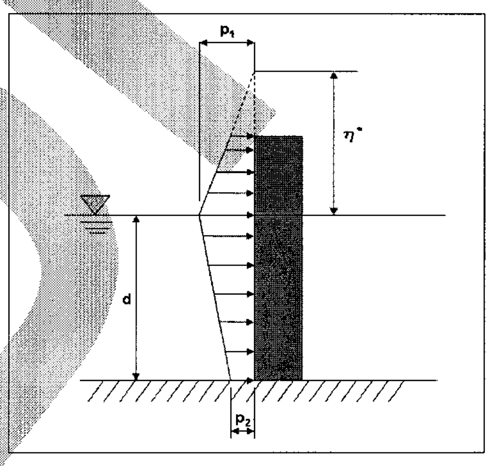

Figure 3.7.4.3.3-1 – Wave Force on Large Element

C3.7.4.3.3

Equations 1 through 4 are known as the Goda equations (Goda XXX). The Goda formulas presented for wave forces on large structural elements represent quasi-static loads. Impact loads and loads from breaking waves may be significantly higher and should be considered on a case-by-case basis.

3.7.4.4 Level I Analysis of Design Parameters [Max]

3.7.4.4.1 General

A Level I analysis for the determination of maximum wave crest elevation should include consideration of the following:

- Bridge location
- 100 year design wind speed
- Maximum fetch length and orientation relative to the open coastline
- 100 year storm surge elevation and the mechanisms considered in its determination
- Bathymetry – submarine topography

C3.7.4.4.1

A Level I analysis:

- requires the least effort of the three levels to perform,
- is the most conservative in the magnitude of the predicted forces, and
- is for the most part, based on readily available information.

The Level I analysis is designed to be conservative due to the lower confidence levels associated with the input parameters for computing design water levels and wave heights and periods. One Hundred year values are used for all the components that make up the design water elevation and the wave parameters. For some situations (open coast, center of a near circular bay, etc.) this combination will produce a 100 year event. However, for most bridge locations (e.g. bridges over long narrow waterways, etc.) the combination of 100 year components will yield a less frequent event. These differences are accounted for in the load factor modifications presented in Table ??.

This procedure is one step above a screening analysis as might be used to identify critical bridges for retrofit and is suitable for eliminating bridge spans from further analysis. In most cases a level 2 analysis should be performed prior to retrofitting.

The information described herein with regard to Level I analysis could lead to a false sense of security regarding the ability for engineers without a coastal background to correctly asses a given situation.

Even at the Level I stage of analysis a review of data and interpretation of results by a coastal engineer is required.

3.7.4.4.2 Design Wind Speed

- If 100 year coastal storm wind speeds exist at the site then these values should be used, otherwise,
- The design wind shall be the 100 yr wind speed determined as 107% of the wind speeds given in Figures 6-1, 6-1A, 6-1B and 6-1C of ASCE Standard 7-05 (ASCE 2005)

Need to say something about wind duration in ASCE—may want 30 minute duration.

C3.7.4.4.2

ASCE 7-05 tabulates winds for a 50 year event. Table C6-7 provides conversion factors for other mean recurrence intervals. The factor for 100 years is 1.07 for the continental U.S. and 1.06 for Alaska. The small difference between 1.07 and 1.06 has been ignored herein.

3.7.4.4.3 Design Water Level

The design water level at a particular site shall be taken as the sum of the 100 year storm surge and 100 year wind setup. The 100 year storm surge elevation should be taken from the best available source (FEMA, NOAA, State Agencies, other reliable sources). The wind setup should be computed using the 100 year wind speed; the most critical fetch, and be determined as:

$$h_s(x) = h\sqrt{1 + \frac{2\,n\,\tau_{wx}\,x}{\gamma_w h^2\,L}} - 1 \qquad (3.7.4.4.3\text{-}1)$$

in which:

$$k = \begin{cases} 1.2 \times 10^{-6}, & |W| \leq 18.4 \text{ ft/s} \\ 1.2 \times 10^{-6} + 2.25 \times 10^{-6}\left(1 - \dfrac{18.4 \text{ ft/s}}{|W|}\right)^2, & |W| > 18.4 \text{ ft/s} \end{cases} \qquad (3.7.4.4.3\text{-}2)$$

and

$$\tau_{wx} = \rho_w\, k\, W|W| \qquad (3.7.4.4.3\text{-}3)$$

where:

s = wind setup measured from the storm still water level
d = average water depth over fetch
n = 1.3
x = fetch length in the direction of the wind from the upwind shore
τ_{wx} = wind shear stress on water surface g = acceleration of gravity = 32.17 ft/s^2
γ_w = unit weight of water taken as 0.064 k/ft^3
L = total length of the water body over which the fetch is measured
W = wind speed at the standard 10 m (32.8 ft) elevation
ρ = mass density of water taken as 2.0 slugs/ft^3

C3.7.4.4.3

Refer to the definition sketch in Figure C1

Where the variable "x" above defines the location of the bridge it is identical to the variable "F" in Article 3.7.4.4.4.

Figure C1

Equation 3 for wind shear stress on the water surface is due to Van Dorn (1953) and is but one of several algorithms published in the literature.

Need to explain why Van Dorn was chosen.

3.7.4.4.4 Design Wave Parameters

Significant wave height and peak period may be estimated using the following empirical equations:

$$\frac{H_s}{U_A^2} = 0.283\tanh\left[0.53\left(\frac{gd}{U_A^2}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{gF}{U_A^2}\right)^{1/2}}{\tanh\left[0.53\left(\frac{gd}{U_A^2}\right)^{3/4}\right]}\right\} \qquad (3.7.4.4.4\text{-}1)$$

$$\frac{gT}{U_A} = 7.54\tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{3/8}\right]\tanh\left\{\frac{0.0379\left(\frac{gF}{U_A^2}\right)^{1/3}}{\tanh\left[0.833\left(\frac{gd}{U_A^2}\right)^{3/8}\right]}\right\} \qquad (3.7.4.4.4\text{-}2)$$

$$\frac{gt}{U_A} = 537\left(\frac{gT}{U_A}\right)^{7/3} \qquad (3.7.4.4.4\text{-}3)$$

in which:

$$U_A = 0.71\,U_s^{0.123} \qquad (3.7.4.4.4\text{-}4)$$

where:

g = gravitational constant (m/sec^2)
U_A = wind-stress factor (m/s)
U_s = surface wind velocity (m/sec)
d = average depth over the fetch (m)
F = fetch length (m)
H_s = height of significant wave from crest to trough
T = wave period (sec)
t = duration of U_s (sec)

Maximum Wave Height ≈ 1.8 H_s, but there are depth and steepness limits that should be checked.

For this level of analysis, required data may be taken from the following sources:

- Wind speed from extremal analyses of hurricane wind speeds at landfall or ASCE 7
- Surge from data available from FEMA, NOAA or other reliable sources
- Current speed and direction

The 100 year value for the parameters storm surge, wind speed, wind setup, wave height and period shall be used in this Level I analysis.

Need to discuss astro tide-mean, max etc. In some cases astro tide has been greater than surge.

Need to discuss current more—how and when does it add or subtract from other contributors.

C3.7.4.4.4

Wave heights and periods at a particular site depend primarily on the wind speed, water depth, fetch length and wind duration. The wind duration required for the wave heights and periods to become independent of time (i.e. to become fetch limited) depend on the wind speed and water depth. Fetch limited wave heights as a function of fetch length for a water depth of 35 ft and wind speed of 100 miles/hr is shown in Figure C1. Also shown in Figure C1 is the approximate wind duration required to achieve a fetch limited conditions for a fetch of 10 miles (for the specified water depth and wind speed).

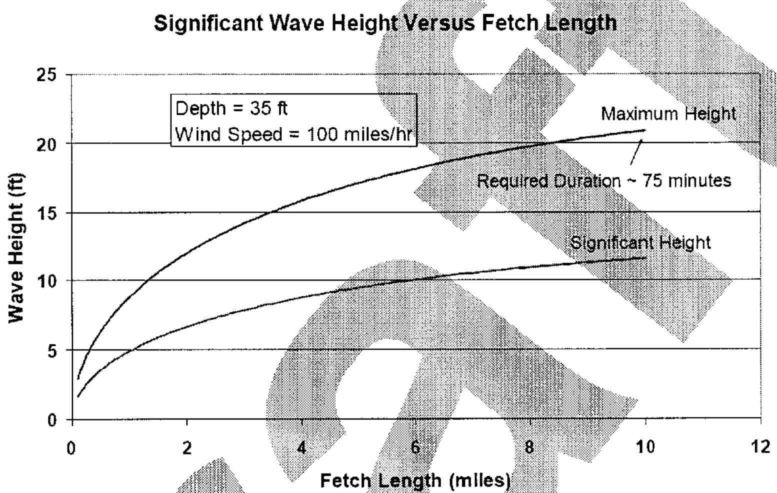

Figure C3.7.4.4.4-1 Affect of Fetch Length on Wave Height for a Particular Site

Equations 1 through 4 are taken from the Shore Protection Manual (COE 1983) and are approximations. They are acceptable for water depths up to 300. The limit of 80% on H_s represents the depth-limited case.

Information on 100 year storm surge elevations is available from several agencies (e.g. FEMA, http://msc.fema.gov) but this information has to be examined carefully to determine if astronomical tides and wave setup have been included in the analysis.

It is important to realize that waves higher than the design wave can occur. The wave height usually derived from statistical analysis of historical hurricanes represents wave condition for a specific probability. The wave conditions are normally represented by the significant wave height, H_s, and peak period, T_p. Assuming a Rayleigh wave height distribution, H_s may be further defined in approximation relation to other height parameters of the statistical wave height distribution in deep water:

$H_{1/3}$ or H_s = average of highest 1/3 of all waves (an alternate definition of H_s sometimes applied is four times the standard deviation of the sea surface elevations, often denoted as H_{mo})

$H_{10} \approx 1.27\ H_s$ = average of highest 10 percent of all waves (C3.7.4.1-1)
$H_5 \approx 1.37\ H_s$ = average of highest 5 percent of all waves (C3.7.4.1-2)
$H_1 \approx 1.67\ H_s$ = average of highest 1 percent of all waves (C3.7.4.1-3)
T_p = period of waves with the greatest energy (period at the peak on a wave energy density versus wave period plot)

Additionally, the maximum practical wave height can be on the order of 1.65 H_s to 1.70 H_s is sometimes conservatively taken as:

$H_{max} = 1.8\ H_s$ (C3.7.4.4-1)

3.7.4.4.5 Maximum Water Elevation (wave and surge)

The design wave crest elevation may be determined as:

η_{max} = storm surge elevation$_{100\ year}$ + wind setup$_{100\ year}$ + wave crest height$_{100\ year}$,

where:

storm surge elevation$_{100\ year}$ includes wave setup and astronomical tides.

Need to introduce wave setup

3.7.4.5 Level II Analysis of Design Parameters

3.7.4.5.1 General

A Level II analysis may be used to improve upon any of the data or analytical techniques / equations used in Level I to develop the following design parameters.

C3.7.4.5.1

The primary difference between Level I and Level II analyses is the accuracy of the information used to compute the design water elevations and wave parameters. Depending on the circumstances a Level II analysis may be performed initially or following a Level I analysis. Where level I analysis has preceded Level II, all quantities used to compute design water elevations and the wave parameters in the Level I analysis should be reassessed and those deemed improvable, reevaluated.

A Level II analysis:

- requires more effort than a Level I analysis,
- is more accurate than a Level I analysis, and

The Level II analysis allows for a wide range of possible improvements over that for a Level I analysis. Additional or more recent measurements may be required for such quantities as bathymetry. Computer models will most likely be needed for accurate wind setup and waves.

Level II analysis may be found to be cost-justifiable as the minimum level of effort needed to obtain the information needed to retrofit an existing bridge.

3.7.4.5.2 Design Wind Velocity

Improvements to the magnitude and directionality of the design wind speed and direction may be obtained through the use of, or acquisition of data from, numerical hindcast models or from site-specific measured data.

C3.7.4.5.2

The storm events that produce significant storm surge elevations are the ones of importance in these investigations. Where the 100 year wind speed for all wind events was sufficient for a Level I analysis it may not be for Level II analysis. It may be necessary to single out storm surge producing wind events and perform an extremal analysis on this data to obtain an accurate design wind speed.

3.7.4.5.3 Design Water Level

Wherever possible, a Level II analysis of design water level should be based on data obtained from several agencies which should be examined and compared. If necessary, the missing mechanisms should be approximated and included in the design storm surge elevation. If a complete reanalysis of storm surge is required then a Level III analysis should be performed as specified in Article 3.7.4.6.

C3.7.4.5.3

A number of state and federal agencies have published the results of storm surge analyses for the Atlantic and Gulf Coasts of the U.S. The mechanisms (astronomical tides, wave setup, etc.) included in these analyses differ greatly from location to location, agency to agency, etc. and to a large extent on when the analysis was performed.

3.7.4.5.4 Design Wave Height and Period

Advanced numerical models may be used to improve upon the magnitude and timing of the design wave height and period. This may include acquisition of improved bathymetry for input to such models or the analytical techniques presented in the Level I analysis.

C3.7.4.5.4

Whereas empirical equations for significant wave height and peak period were adequate for a Level I analysis numerical models will most likely be required for a Level II analysis. This depends on, among other things, the bathymetry and complexity of the shoreline of the water body in the vicinity of the fetch. For example, the empirical equations are more accurate for a uniform depth basin with a simple geometry shoreline. The input parameters for numerical wave models are wind velocities, bathymetry and water boundaries. The accuracy of the wave parameters produced by these models can be no better than the accuracy of the input parameters. Knowledge of the strengths and weaknesses of the model used is important in the interpretation of the results.

Level II and III may need discussion wave refraction and diffraction.

3.7.4.5.5 Design Current Velocities

Advanced numerical models may be used to improve upon the magnitude of the design current velocity. This could include input of time-varying winds to better define the "associated" design current velocities with the either the 100 year design wind or the 100 year design wave parameters.

Discuss currents in other level too.

C3.7.4.5.5

Riverine current velocities for different return intervals are usually available with varying degrees of accuracy. This is not the case for storm surge and/or wind driven currents. These values can, however, be estimated by running a storm surge model for the area provided a 100 year open coast storm surge magnitude and hydrograph are known and available for use.

3.7.4.6 Level III Analysis of Design Parameters

3.7.4.6.1 General

A Level III analysis shall be used to determine design parameters for bridges critical to a region's economy or safety or for bridges where substantial repair and/or replacement costs may be incurred if damaged by a coastal storm event.

C3.7.4.6.1

Level III analyses:

- are more time consuming and costly to perform;
- produce more accurate results than Levels I and II analyses, and
- are necessary for large and/or important bridges deemed susceptible to storm surge and wave loading.

The modeling effort outlined below improves the accuracy of all met/ocean parameters needed for the computation of storm surge and wave loading on bridge sub and super structures. This includes design water elevations, current velocities, and wave heights and periods.

3.7.4.6.2 Design Water Elevation, Current Velocity, and Wave Parameters

A Level III analysis requires an extensive computer modeling and analysis effort and possibly the measurement of bathymetry and model calibration parameters such as water elevations and waves.

C3.7.4.6.6

There are a number of numerical models for computing hurricane generated wind fields, storm surge hydraulics (water elevation, depth averaged current velocity), and wave parameters in use, each with their strengths and weaknesses. The following procedure is one that has been successfully used and can be considered as a guideline for performing a Level III analysis.

1. Perform hurricane windfield hindcasts for as many hurricanes that have impacted the area of interest as time and resources allow. The hindcasts should initiate at least ?? days prior landfall of the hurricane eye. It should be pointed out that hurricane windfield hindcasting is a very specialized discipline and in most cases should be left to those meteorologists that specialize in this area.

2. Perform storm surge and wave hindcasts (coupling wave and surge models) for the hurricanes analyzed in 1) above, using the hindcasted wind fields from 1).

3. Using the water elevation and wave information at the bridge site for each of the hindcasted storms, perform an extremal analysis on these parameters to obtain the values for the desired design return interval.