TASK DOCUMENT

TASK 6 - GUIDE SPECIFICATION AND RETROFIT MANUAL
TASK 6a – 90%GUIDE SPECIFICATIONS

August 17, 2007

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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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Possible Organization and Content for Coastal Specifications

~~(Article Numbers Relate to LRFD Specifications)~~

August 17, 2007

1. SCOPE

These specifications shall apply to bridges deemed to be important by the owner, and may be applied to other structures at the discretion of the owner. In making a decision as to which bridges qualify for treatment under these specifications, the screening concepts of the Handbook of Retrofit Options for Bridges Vulnerable to Coastal Storms should be used as a guide. Evacuation and rescue/recovery of the affected area should be a prime consideration when considering a system of bridges serving a coastal area. The effect on the local economy should also be considered.

2. DEFINITIONS

Not all of the definitions herein have been used in these specifications. They are, however, part of the lexicon of coastal engineering and may be useful when reading literature in the field.

ASTRONOMICAL TIDE
The tidal levels and character which would result from gravitational effects, e.g. of the Earth, Sun and Moon, without any atmospheric influences.

BATHYMETRY
The measurement of water depths in oceans, seas, and lakes; also information derived from such measurements.

BUOYANCY
The resultant of upward forces, exerted by the water on a submerged or floating body, equal to the weight of the water displaced by this body.

DATUM
Any permanent line, plane or surface used as a reference datum to which elevations are referred.

DEPTH
The vertical distance from a specified datum to the sea floor.

DESIGN STORM
A hypothetical extreme storm whose waves coastal protection structures will often be designed to withstand. The severity of the storm (i.e. return period) is chosen in considering the acceptable level of risk of damage or failure.

DESIGN WAVE CONDITION
Usually an extreme wave condition with a specified return period used for the design of coastal works.

DURATION, MINIMUM
The time necessary for steady-state wave conditions to develop for a given wind velocity over a given fetch length.

EBB CURRENT
The movement of a tidal current away from shore or down a tidal stream.

EBB TIDE
The period of tide between high water and the succeeding low water; a falling tide.

FETCH LENGTH
The horizontal distance (in the direction of the wind) over which a wind generates seas or creates a wind setup.

FETCH-LIMITED
Situation in which wave energy (or wave height) is limited by the size of the wave generation area (fetch).

FLOOD CURRENT
The movement of a tidal current toward the shore or up a tidal stream.

FLOOD TIDE
The period of tide between low water and the succeeding high water; a rising tide. (See Figure II-5-16)

HIGHEST ASTRONOMICAL TIDE (HAT)
The highest level of water which can be predicted to occur under any combination of astronomical conditions. This level may not be reached every year.

HINDCASTING
In wave prediction, the retrospective forecasting of waves using measured wind information.

HURRICANE
An intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 33.5 m/sec (75 mph or 65 knots) for several minutes or longer at some points. TROPICAL STORM is the term applied if maximum winds are less than 33.5 m/sec but greater than a whole gale (63 mph or 55 knots). The term is used in the Atlantic, Gulf of Mexico, and eastern Pacific.

IRREGULAR WAVES
Waves with random wave periods (and in practice, also heights), which are typical for natural wind-induced waves.

JOINT PROBABILITY
The probability of two (or more) things occurring together.

JOINT RETURN PERIOD
Average period of time between occurrences of a given joint probability event.

MEAN HIGH WATER SPRINGS (MHWS)
The average height of the high water occurring at the time of spring tides.

MEAN HIGHER HIGH WATER (MHHW)
The average height of the higher high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the result to the equivalent of a mean 19-year value.

MEAN SEA LEVEL (MSL)
The average height of the surface of the sea for all stages of the tide over a 19-year period, usually determined from hourly height readings. MSL is not necessarily equal to MEAN TIDE LEVEL. It is also the average water level that would exist in the absence of tides.

MEAN TIDE LEVEL (MTL)
A plane midway between MEAN HIGH WATER and MEAN LOW WATER. MTL is not necessarily equal to MSL. Also HALF-TIDE LEVEL.

MONOCHROMATIC WAVES
A series of waves generated in a laboratory, each of which has the same length and period.

NUMERICAL MODELING
Refers to analysis of coastal processes using computational models.

OVERTOPPING
Passing of water over the top of a structure as a result of wave runup or surge action.

PARTICLE VELOCITY
The velocity induced by wave motion with which a specific water particle moves within a wave.

PHYSICAL MODELING
Refers to the investigation of coastal or riverine processes using a scaled model.

PROBABILITY
The chance that a prescribed event will occur, represented by a number (p) in the range 0 - 1. It can be estimated empirically from the relative frequency (i.e. the number of times the particular event occurs divided by the total count of all events in the class considered).

REFRACTION (of water waves)
The process by which the direction of a wave moving in shallow water at an angle to the contours is changed: the part of the wave advancing in shallower water moves more slowly than that part still advancing in deeper water, causing the wave crest to bend toward alignment with the underwater contours.

RETURN PERIOD
Average period of time between occurrences of a given event.

RISK ANALYSIS
Assessment of the total risk due to all possible environmental inputs and all possible mechanisms.

SCOUR
Removal of underwater material by waves and currents, especially at the base or toe of a shore structure.

SEAS
Waves caused by wind at the place and time of observation.

SHOALING
Decrease in water depth. The transformation of wave profile as they propagate inshore.

SIGNIFICANT WAVE HEIGHT
The average height of the one-third highest waves of a given wave group.

SIGNIFICANT WAVE PERIOD
An arbitrary period generally taken as the period of the one-third highest waves within a given group.

SOUNDING
A measured depth of water. On hydrographic CHARTS, the soundings are adjusted to a specific plane of reference.

STORM SURGE [TO BE REVISED]
A rise above normal water level on the open coast due to the action of wind stress on the water surface. Storm surge resulting from a hurricane also includes that rise in level due to atmospheric pressure reduction as well as that due to wind stress. See WIND SETUP.

SWELL
Wind-generated waves that have traveled out of their generating area. Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their fetch (SEAS).

TIDE
The periodic rising and falling of the water that result from gravitational attraction of the Moon and Sun and other astronomical bodies acting upon the rotating Earth.

TSUNAMI
A long-period water wave caused by an underwater disturbance such as a volcanic eruption or earthquake. Commonly miscalled "tidal wave."

UPLIFT
The upward water pressure on the base of a structure or pavement.

WATER DEPTH
Distance between the seabed and the still water level.

WATER LEVEL
Elevation of still water level relative to some datum.

WAVE
A ridge, deformation, or undulation of the surface of a liquid.

WAVE CREST
The highest part of a wave.

WAVE DIRECTION
The direction from which a wave approaches.

WAVE FREQUENCY
The inverse of wave period.

WAVE HEIGHT
The vertical distance between a crest and the preceding trough.

WAVE PEAK FREQUENCY
The inverse of wave peak period.

WAVE PEAK PERIOD
The wave period at which the wave energy spectrum reaches its maximum.

WAVE SETUP
Superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

WAVE STEEPNESS
The ratio or wave height to wavelength also known as sea steepness.

WAVE TRANSFORMATION
Change in wave energy due to the action of physical processes.

WAVE TROUGH
The lowest part of a wave form between successive crests. Also that part of a wave below still-water level.

WAVE LENGTH
The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

WEIBULL DISTRIBUTION
A model probability distribution, commonly used in wave analysis.

WIND SETDOWN
Drop in water level below the still water level on the windward ends of enclosed bodies of water and semi-enclosed bays.

WIND SETUP - LOCAL
On reservoirs and smaller bodies of water, the vertical rise in the still-water level on the leeward side of a body of water caused by wind stresses on the surface of the water.

WIND WAVES
(1) Waves being formed and built up by the wind. (2) Loosely, any wave generated by wind.

3. NOMENCLATURE

(To be taken from where lists and consolidated in final draft)

4. GENERAL

4.1 Storm Clearance

Wherever practical, the vertical clearance of highway bridges should be sufficient to provide at least 3 ft. of clearance over the 100-year design wave crest elevation, which includes the design storm water elevation.

For bridge spans where this vertical clearance is not possible, other design strategies may be considered including those identified in Article 4.2. Bridges located with less than 3 ft. of clearance over the design wave crest elevation shall be designed assuming a minimum of 1 ft. intrusion into the wave. Wave effects on substructure shall be investigated in accordance with the provisions of Article 6.2.3.

C4.1

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. Additional freeboard beyond that indicated in this article should be considered due to the large uncertainty in the basic wave and surge data needed to determine the design wave crest elevation.

4.2 Design Strategies for Coastal Storms

4.2.1 General

Regardless of the design strategy chosen for a particular bridge, early input from a coastal engineer to clarify coastal issues and scope for the bridge should be considered.

C4.2.1

Further discussion on the credentials recommended for a coastal engineer, and a list of some conditions which require more extensive involvement of a coastal engineer are provided in C6.2.1.

4.2.2 Avoidance

The provisions of Article 4.1 shall apply.

4.2.3 Force Mitigation

Where it is not possible to provide the vertical clearance recommended in Article 4.1, the following may be considered to reduce the wave forces acting on the superstructure:

- Setting the vertical elevation as high as practical
- Using open or sacrificial parapets
- Venting the potential cells that could entrap air creating increased buoyancy forces
- Using 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
- Using continuous superstructures to increase the reactive force of individual spans
- Using solid or voided slab bridges to reduce buoyancy forces

C4.2.3

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 ft of parapet were broken off. This performance suggests that either a shorter overhang or the use of a parapet that would respond inward in a sacrificial manner, while still providing the required traffic barrier resistance outward, could reduce the amount of area exposed to the waves. This response could also promote inundation which reduces the total wave force to be resisted.

Calculated estimates of the effect of 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 behavior occurs 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 be effective in venting entrapped air and allowing the exchange of trapped air between spans. Figure C1 shows the area of hole necessary to permit evacuation of a volume of air for different times.

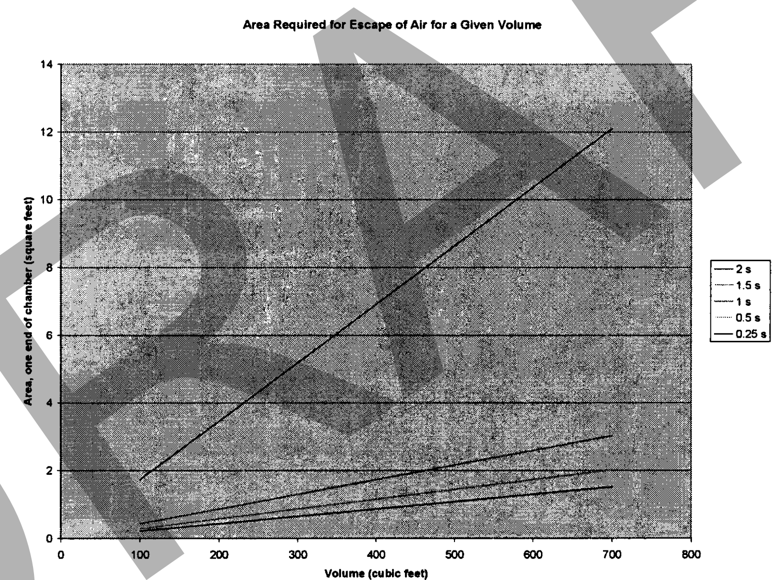


Figure C4.2.3-1 – Venting Requirements

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. Therefore, 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 to raise some spans sufficiently to avoid wave forces such as those near the ends of bridges which have grade constraints.

4.2.4 Force Accommodation

4.2.4.1 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.

C4.2.4.1

In recent cases where the superstructure was lost but the 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.

4.2.4.2 Design for the Full Wave Loads

The structure may be designed to resist the loads calculated in accordance with the provisions specified in Articles 6.2.3 and 6.3.

4.2.4.3 Fusing for Partial Loads

Where design for the full wave loads specified in Articles 6.2.3 and 6.3 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.

C4.2.4.3

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 concept is not necessarily applicable to the coastal storm situation because 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 to allow the superstructure to float free, thus preserving the substructure for future use.

4.2.4.4 Sacrificial Superstructure

Superstructures may be designed to separate from the substructure either under the action of vertical forces, which include buoyancy as determined herein, horizontal forces, or any combination thereof. Where this strategy is used, the design and details shall ensure that separation occurs only when the forces associated with the 100-year design event are exceeded.

C4.2.4.4

Sacrificial superstructures are a variation of the fusing for partial loads specified under Article 4.2.4.3. 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 replace them after the storm.

Past experience has shown that freed, i.e. separated, superstructure units have caused damage to substructure.

4.2.5 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 wave crest elevations (including storm water levels) lower than the 100-year design values.

C4.2.5

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 with small volumes of voids, which reduce the buoyancy, may be a cost effective solution in some cases.

5. LOAD COMBINATIONS

The following Strength Limit State Load Combination shall be considered for bridges vulnerable to wave or surge forces associated with coastal storms:

$$\gamma_d DC + \gamma_d DD + \gamma_d DW + \gamma_d EL + \gamma_{wave} WA \qquad (5\text{-}1)$$

where:

DC	=	dead load of structural components and nonstructural attachments
DD		downdrag
DW	=	dead load of wearing surfaces and utilities
EL	=	accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning
WA	=	wave forces F_v, F_s, F_H and M_t specified in Articles 6.2.2 and 6.2.3
γ_d	=	minimum load factors for dead loads as specified in Article 3.4.1 of the *AASHTO LRFD Bridge Design Specifications*
γ_{wave}	=	load factors on wave forces

For values of η_{wave}-Z_c > 4, the load factor for wave loads, γ_{wave}, shall be taken as 2.25.

Work is ongoing to determine the appropriate value for η_{max}-Zc < 4

C5.1

Since dead loads generally resist the wave loads, consideration should be given to whether DW can be reasonably expected to be in place for design event.

6. FORCES ASSOCIATED WITH COASTAL STORMS

6.1 [Add Air Entrapment]

6.2 Hydrodynamic Loads and Design Parameters

6.2.1 General

The provisions of this article shall be taken to apply to bridges located in areas where they may be impacted by 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 dimensions, shape and orientation relative to the water body
- Bathymetry of the water body
- Fetch length orientation relative to the bridge location
- Fetch and fetch angle segment for waves
- Fetch and fetch angle segment for local wind setup/setdown
- Design wave height and period (wave length)
- Design wind velocity
- Design storm water level 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. - Max]

Determination of the appropriate design parameters may proceed according to the three levels of analysis specified in Article 6.3. 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 meteorological/oceanographic data for the site, etc. A Level I analysis (Article 6.3.2) 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.

C6.2.1

The load factors presented in Article 5 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 Articles 6.3.2 and 6.3.3, 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), 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, load modifiers are presented in Articles 6.2.2.6 and 6.3.3.7 for Level I and Level II, respectively, 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 is 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 Article 6.3.4.

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,

6.2.2 Hydrostatic and Hydrodynamic Forces and Moments on Superstructure

6.2.2.1 General

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

- Buoyancy
- Drag and inertia forces
- Forces associated with added mass
- Vertical slamming forces

The vertical force shall be considered to be sum of two parts referred to herein as 1) the quasi-static force and 2) the slamming force.

The equations for forces and moments given herein were developed around the trailing edge of the girders, as shown in Figure 1, and calculations of force effects on the structure shall start with the forces assumed to be applied at the trailing edge. The value of M_t shall be taken as specified in Article 6.2.2.5.

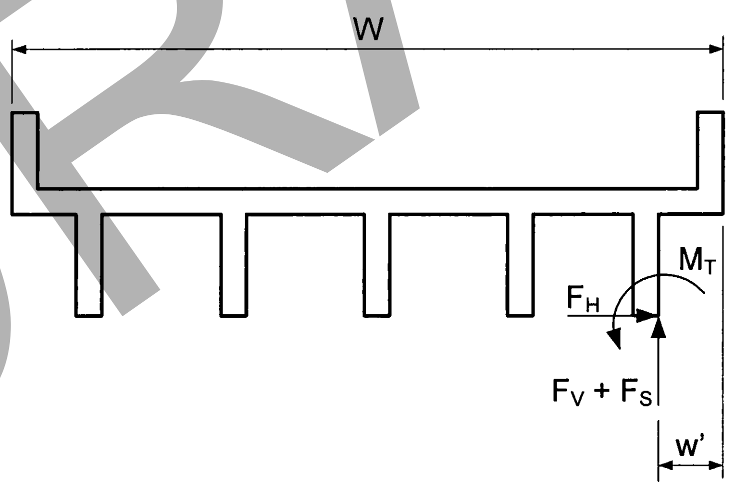


Figure 6.2.2.1-1 – Location of Forces and Moments

Where Equation 1 is not satisfied, the structure is above the wave zone and, therefore, the wave forces need not be determined.

$$\frac{Z_c}{\eta_{max}} \le 1.0 \qquad (6.2.2.1\text{-}1)$$

The equations in wave forces in Articles 6.2.2.2, 6.2.2.3, 6.2.2.4 and 6.2.2.5 should be considered most accurate when the following criteria are satisfied:

$$0.05 \leq \frac{H_{max}}{\lambda} \leq 0.1 \qquad (6.2.2.1\text{-}2)$$

$$0 < \frac{W}{\lambda} < 0.7 \qquad (6.2.2.1\text{-}3)$$

Where these criteria are not satisfied, the wave length, λ, may be arbitrarily limited to the extreme value which would satisfy Equations 2 and 3, i.e. the value at the limit of Equations 2 and 3. Determination of wave forces may then proceed with the adjusted value of λ, albeit with somewhat reduced accuracy.

C6.2.2.1

The hydrostatic and hydrodynamic forces acting on bridge superstructures are composed of several components that, in general, are not in phase. For most bridge configurations, the critical loading situation occurs when the total quasi-static vertical force is at its maximum value. The forces computed using the equations and tables presented in Article 6.2.2.2 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. These additional quantities are accounted for by coefficients presented in tables. All force equations are a parameterization of the results from detailed analyses using a modified and much extended form of Kaplan's equations referred to herein as the Physics Based Model (PBM). The equations developed by Paul Kaplan for wave forces on offshore platform decks were an extension of Morison's equations (for wave forces on vertical piles). Kaplan assumed that the platform decks were thin horizontal structures. Due to significant differences between structure width to wave length ratios and structure shapes between offshore platforms located in deep, open water, and bridge superstructures located over bays and coastal waterways, direct application of Kaplan's equations to bridge superstructures have not provided satisfactory results. The equations developed at the University of Florida and specified herein include the same general components as the Morrison and Kaplan equations but differ in how they are applied to the structure. In the PBM equations, the forces are integrated over the wetted portion of the superstructure at each time step of the computation. This results in significant differences in the magnitude of the change in added mass terms. Another important difference between Kaplan and PBM is the inclusion of finite thickness of the structure and its impact on the added mass. There are other differences regarding trapped air. Since the Kaplan method considers the structure to be a thin plate, trapped air was not an issue. The PBM not only accounts for the percent of trapped air but the air compression during wave impact as well. There are also differences in the way the water overtopping the structure is handled.

The forces given by the equations herein apply to the full longitudinal length of one span of a structure at the same time. The variation across the structure in the transverse direction has been included in the development of the equations. Similarly, the direction of wave attack, shown schematically in Figure C1, has been limited to a right angle attack, i.e. $\theta = 0°$ which may be considered to be conservative for other approach angles. The provided equations are thought to give reasonable results when the wave attack is longitudinal, i.e. $\theta = 90°$, with the bridge length and width are substituted appropriately. Forces at other angles of attack have not been developed as of this writing and may be added at a later date if appropriate research is carried out.

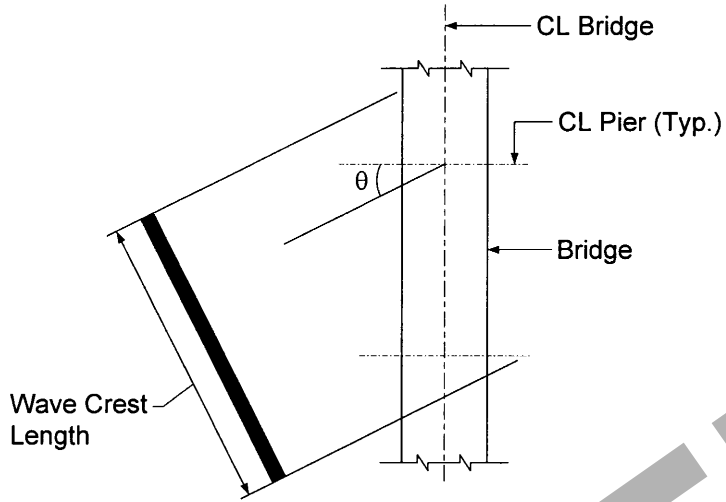


Figure C6.2.2.1-1 – Oblique Wave Attack

Vertical hydrostatic forces are imparted to structures once any portion of the structure is submerged. When there is no water motion, the only force acting is buoyancy. 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 (i.e., inertia forces) are exerted on the structure. For structures, such as bridge spans that subjected to water waves, the volume of the submerged structure can change with time. This phenomenon 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. Thus, at the time of maximum vertical force there are corresponding horizontal forces and moments about the trailing edge of a bridge cross-section. Equations to determine these forces are given in Article 6.2.2.4 and 6.2.2.5. These forces should be used in conjunction with the vertical force in the structure response analysis.

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, but it is of short duration. The vertical slamming force often occurs at the time of the maximum upward vertical quasi-static force, and therefore the two should be added to achieve the total upward vertical force on the superstructure. For design purposes the total vertical force for these situations is the sum of the quasi-static and the slamming force.

The equations presented herein for loads on superstructures do not result in a reduction of load when the bridge becomes submerged. The experimental studies which were used to verify the development process did show some decrease in force with submergence, but the reduction was relatively small. For this first codification of loads from coastal storms it was decided to conservatively ignore this reduction.

The equations in Articles 6.2.2 and 6.2.3 have been developed based on a Florida F32, 32 in. high, solid parapet. From a wave impingement point of view, this parapet may be considered to represent any 32 in. high solid parapet. Future work will include evaluation of other parapet types.

6.2.2.2 Quasi-Static Vertical Force

Subject to the limitations in Article 6.2.2.1, the quasi-static vertical force, including the effect of variable air entrapment, may be determined as:

$$F_v = A\,\gamma_w W \beta \left(\frac{W}{\lambda}\right)^B (\mathrm{TAF}) \tag{6.2.2.2-1}$$

in which:

$$\text{If } 0 \le \frac{Z_c}{\eta_{max}} < 1, \text{ then } A = \frac{C_1 + C_2\left(\frac{H_{max}}{\lambda}\right) + C_3\left(\frac{Z_c}{\eta_{max}}\right)}{1 + C_4\left(\frac{H_{max}}{\lambda}\right) + C_5\left(\frac{Z_c}{\eta_{max}}\right)} \tag{6.2.2.2-2}$$

$$\text{If } -1 \le \frac{Z_c}{\eta_{max}} < 0, \text{ then } A = e^a \tag{6.2.2.2-3}$$

where:

$$a = C_6 + C_7\left[\ln\left(\frac{H_{max}}{\lambda}\right)\right]\sqrt{\left(\frac{H_{max}}{\lambda}\right)} + C_8\left(\frac{Z_c}{\eta_{max}}\right) + C_9\left(\frac{Z_c}{\eta_{max}}\right)^2 \tag{6.2.2.2-4}$$

$$\text{If } -1 \le \frac{Z_c}{\eta_{max}} < 1, \text{ then } B = \frac{1}{k_1 + \dfrac{k_2}{\ln\left(\frac{H_{max}}{\lambda}\right)} + k_3 e^{-\left(\frac{H_{max}}{\lambda}\right)} + k_4\left(\frac{Z_c}{\eta_{max}}\right) + k_5\left(\frac{Z_c}{\eta_{max}}\right)^2 + k_6\left(\frac{Z_c}{\eta_{max}}\right)^3 + k_7 e^{-\left(\frac{Z_c}{\eta_{max}}\right)}} \tag{6.2.2.2-5}$$

$$\text{If } (\eta_{max} - Z_c) \le 0, \text{ then } \beta = 0 \tag{6.2.2.2-6}$$

$$\text{If } 0 < (\eta_{max} - Z_c) \le d_b, \text{ then } \beta = (\eta_{max} - Z_c) \tag{6.2.2.2-7}$$

$$\text{If } (\eta_{max} - Z_c) > d_b, \text{ then } \beta = d_b \tag{6.2.2.2-8}$$

$$\mathrm{TAF} = A_{AIR}(\%\mathrm{AIR}) + B_{AIR} \le 1.0 \tag{6.2.2.2-9}$$

in which:

$$A_{AIR} = 0.0123 - 0.0045 e^{(-Z_c/\eta_{max})} + 0.0014\, ln(W/\lambda) \tag{6.2.2.2-10}$$

$$B_{AIR} = e^{\left[-2.477 + 1.002 e^{(-Z_c/\eta_{max})} - 0.403\, ln(W/\lambda)\right]} \tag{6.2.2.2-11}$$

$$\text{If } 0 < \frac{\eta_{max} - Z_c}{d_g} \le 1, \text{ then \%Air may be selected from the range } 100\left[1 - \left(\frac{\eta_{max} - Z_c}{d_g}\right)\right] \text{ to } 100 \tag{6.2.2.2-12}$$

$$\text{If } \frac{\eta_{max} - Z_c}{d_g} > 1, \text{ then \%Air may be selected from the range 0 to 100} \tag{6.2.2.2-13}$$

The percent of entrapped air, %Air, to be considered in determining the quasi-static vertical force may be selected within the range given by Equations 12 and 13. Where reduced air entrapment is used as a means to lower the quasi-static vertical design force, adequate venting shall be provided.

Refer back to Article 4.2.3 when graph is replaced.

where:

F_v = vertical quasi-static hydrostatic and hydrodynamic force per unit length of the span (kips)
d_b = girder height + deck thickness (ft)
K1–k7 = coefficients for the B term in the vertical wave force equation specified in Table 1
C1–C9 = coefficients for the B term in the vertical wave force equation specified in Table 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_{max} = distance from the storm water level to design wave crest (ft)
λ = wave length (ft)
γ_w = unit weight water taken as 0.064 (k/ft^3)
H_{max} = maximum probable wave height which may be determined by Equation 6.3.2.4-7 for a Level I analysis, and by storm modeling for Level II and III
d_g = girder depth (ft)
TAF = a factor to adjust the vertical quasi-static force for variable amounts of entrapped air (dim)

The dimensions Z_c and η_{max} and parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Articles 6.3 through 6.3.4.

Table 6.2.2.2-1 – Coefficients for Quasi-Static Vertical Load

Coefficients	**AASHTO Type III**	**AASHTO Type IV**	**AASHTO Type VI**	**Florida Bulb-T 72**	**Florida Bulb-T 78**	**21" Voided Slab**	**36" Adjacent Box Girders**
K_1	-77.567				-76.798	-122.754	-77.451
k_2	-27.557				-25.094	-44.126	-27.157
k_3	57.51				57.616	93.366	57.691
k_4	12.166				12.046	18.000	12.682
k_5	-8.336				-7.959	-10.935	-7.771
k_6	3.142				2.505	3.300	3.234
k_7	12.544				12.244	18.238	12.550
C_1	0.252				0.245	0.570	0.331
C_2	-0.023				-0.021	-0.371	-0.071
C_3	-0.145				-0.153	-0.542	-0.324
C_4	-5.580				-5.151	-5.198	-5.086
C_5	-0.033				-0.054	-0.312	-0.033
C_6	-8.152				-8.170	-7.550	-7.981
C_7	-10.355				-10.285	-10.504	-10.399
C_8	-1.92				-2.065	-1.450	-1.951
C_9	-0.995				-1.995	-0.301	-0.371

C6.2.2.2

Some of the variables used in equations herein are illustrated in Figure C-1. The description of d_s in Figure C-1 indicates that it is the depth at or near the bridge. It is the inclusion of the location which distinguishes "d_s" from "d", which is defined as the average depth over the fetch length.

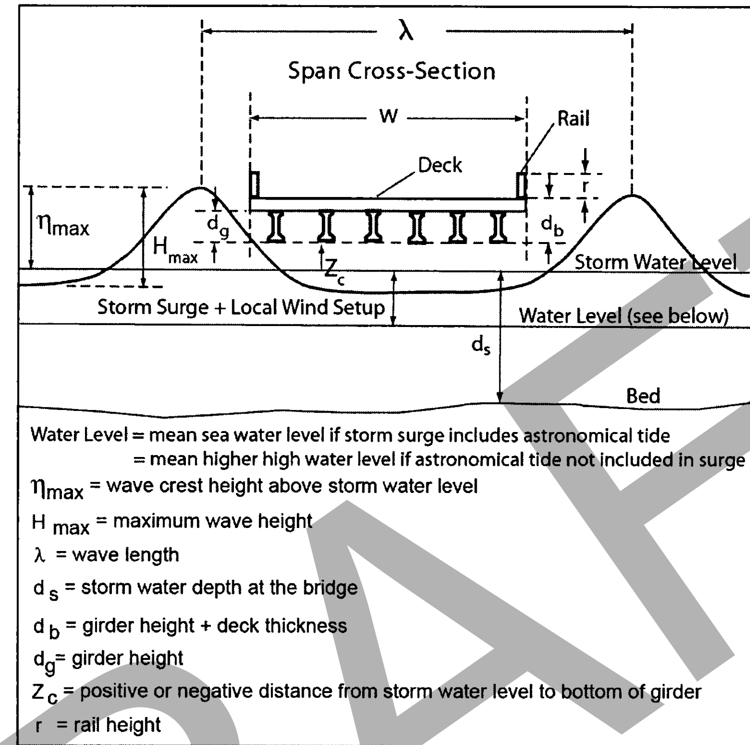


Figure C6.2.2.2-1 – Nomenclature in Equations 1-5

Equations herein and Tables 1 through 6.2.2.5-1 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 criteria 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 tests.
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 PBM 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. Therefore, 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 PBM, compute the slamming force directly. The slamming force in the design equations above is an empirical equation based on the results from the University of Florida model wave tank tests.

As explained in detail in Reference X (project report), a procedure was developed to calculate discrete forces at several thousand locations on a cross-section of a bridge girder due to a time variant wave, impacting and/or passing over the structure using the equations of the PBM. These discrete forces were then used in numerical simulations to determine the net vertical force, horizontal force and moment applied to the cross-section. For the geometries considered, 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. Figures C2 through C7 show time history comparisons of forces measured in the wave tank described in Reference X compared to numerical simulations computed using the PBM. The PBM was then used to conduct numerical experiments that covered a wide range of structure and meteorological/oceanographic conditions. Data from these numerical experiments were then used to develop the parametric equations.

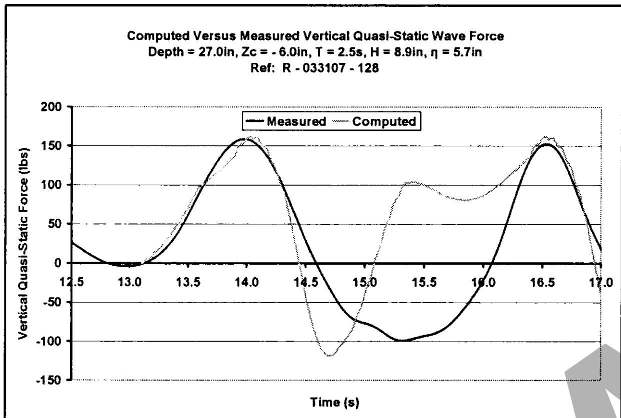


Figure C6.2.2.2-2 – Comparison between computed and measured quasi-steady vertical wave forces for a submerged bridge span.

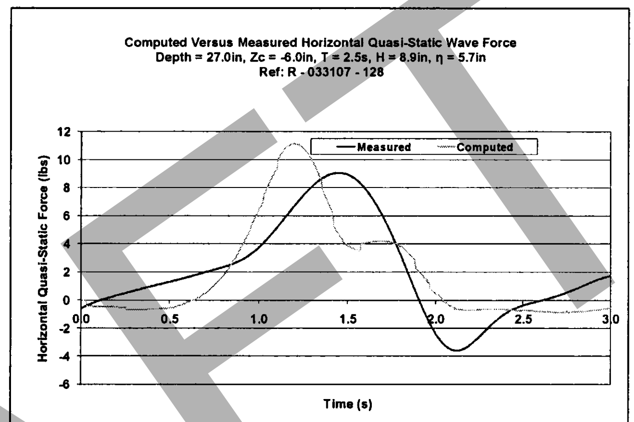


Figure C6.2.2.2-3 – Comparison between computed (Modified Kaplan Equations) and measured horizontal quasi-static wave forces at the time of maximum vertical forces for submerged span.

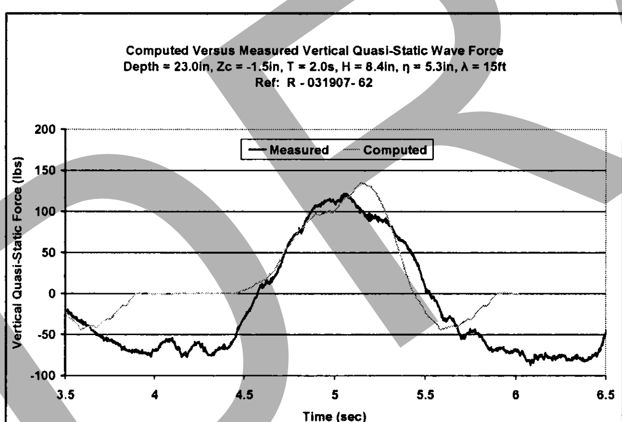


Figure C6.2.2.2-4 – Comparison between computed and measured vertical quasi-static wave forces for partially submerged bridge span.

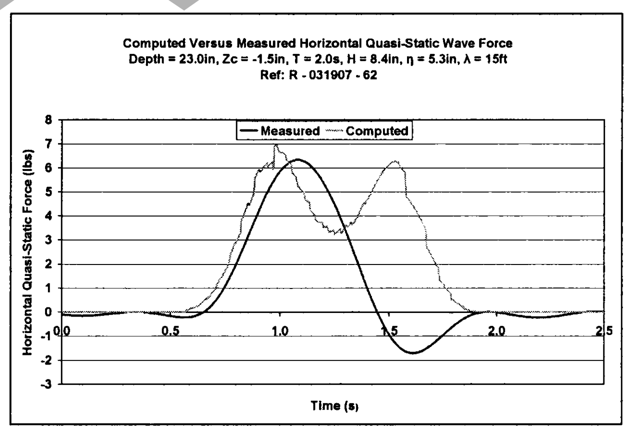


Figure C6.2.2.2-5 – Comparison between computed (Modified Kaplan Equations) and measured horizontal quasi-static wave forces at the time of maximum vertical force for partially submerged span.

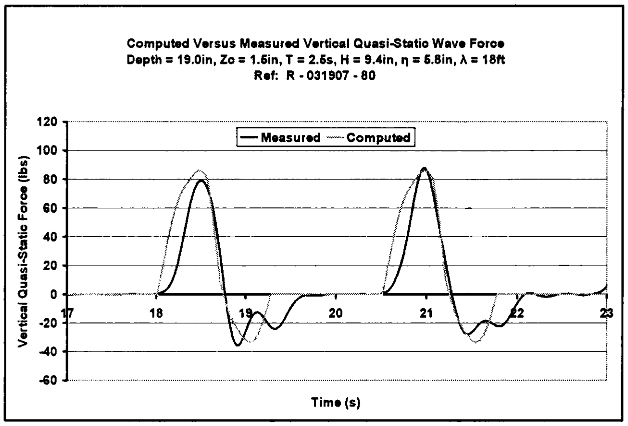


Figure C6.2.2.2-6 – Comparison between computed and measured vertical quasi-static wave forces for sub aerial bridge span.

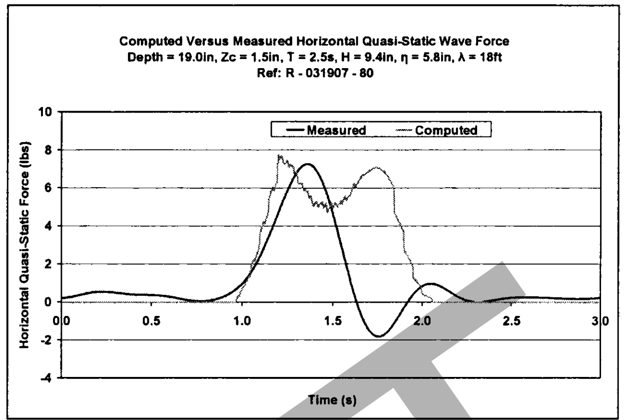


Figure C6.2.2.2-7 – Comparison between computed (Modified Kaplan Equations) and measured horizontal quasi-static wave forces at the time of maximum vertical force for a sub aerial span.

The Trapped Air Factor (TAF) accounts for the effect of trapped air between the girders for girder-type bridge spans. The effect is to alter the buoyancy force which only has a vertical component, thus the TAF is only multiplied times the vertical quasi-static force.

Wave tank data shows that the TAF is primarily a function of (span clearance)/(wave length), Z_c/λ , (span width)/(wave length), W/λ and percent air entrapment, %Air.

The coefficients in A_{AIR} and B_{AIR} were developed for an AASHTO Type 3 girder, but appear to be acceptable for other girder heights. It is possible that these coefficients will need to change for very tall girders.

6.2.2.3 Vertical Slamming Force

Subject to the limitations in Article 6.2.2.1, the vertical slamming found may be determined as:

$$F_s = A\gamma_w H^2_{max}\left(\frac{H_{max}}{\lambda}\right)^B \qquad (6.2.2.3\text{-}1)$$

in which:

$$B = 0.6588\left(\frac{Z_c}{\eta_{max}}\right)^2 + 0.5368\left(\frac{Z_c}{\eta_{max}}\right) - 1.193 \qquad (6.2.2.3\text{-}2)$$

$$\bullet\ \text{If } 0 \le \frac{Z_c}{\eta_{max}} \le 1\text{, then } A = 0.0149\left(\frac{Z_c}{\eta_{max}}\right) + 0.0316 \qquad (6.2.2.3\text{-}3)$$

$$\bullet\ \text{If } \frac{Z_c}{\eta_{max}} < 0\text{, then } A = \left[-1562.9 + 1594.5e^{-\left(\frac{Z_c}{\eta_{max}}\right)}\right]^{-1} \qquad (6.2.2.3\text{-}4)$$

where:

F_s = vertical quasi-static hydrostatic and hydrodynamic force per unit length of the span (kips)
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_{max}	=	distance from the storm water level to design wave crest (ft)
λ	=	wave length (ft)
γ_w	=	unit weight water taken as 0.064 (k/ft^3)
H_{max}	=	maximum probable wave height which may be determined by Equation 6.3.2.4-7 for a Level I analysis, and by storm modeling for Level II and III

The dimensions Z_c and η and parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Articles 6.3 through 6.3.4.

6.2.2.4 Associated Horizontal Quasi-Static Force

Subject to the limitations in Article 6.2.2.1, the associated horizontal quasi-static force may be determined as:

$$F_h = \gamma_w H^2_{max}\left(a_0 + a_1\,(x) + a_2\,(x)^2 + a_3\,(x)^3 + a_4\,(x)^4 + a_5\,(x)^5 + a_6\,ln(y)\right)\left[a_7 + a_8\left(\frac{W}{\lambda}\right)\right] \quad (6.2.2.4\text{-}1)$$

in which:

$$x = \left(\frac{\eta_{max} - Z_c}{d_b}\right) \quad (6.2.2.4\text{-}2)$$

$$y = \frac{H_{max}}{\lambda} \quad (6.2.2.4\text{-}3)$$

where:

F_h	=	horizontal force associated with the vertical quasi-static hydrostatic and hydrodynamic force per unit length of the span (kips/ft)
d_b	=	girder height + deck thickness (ft)
a_0–a_8	=	coefficients specified in Table 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_{max}	=	distance from the storm water level to design wave crest (ft)
λ	=	wave length (ft)
γ_w	=	unit weight of water taken as 0.064 ($kips/ft^3$)
H_{max}	=	maximum probable wave height which may be determined by Equation 6.3.2.4-7 for a Level I analysis, and by storm modeling for Level II and III

The dimensions Z_c and η_{max} and parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Articles 6.3 through 6.3.4.

Table 6.2.2.4-1 – Coefficients for Quasi-Static Horizontal Load

Coefficients	AASHTO Type III	AASHTO Type IV	AASHTO Type VI	Florida Bulb-T 72	Florida Bulb-T 78	21" Voided Slab	36" Adjacent Box Girders
a_0	0.269				0.106	0.1756	0.2418
a_1	0.573				-0.0649	0.7769	0.4200
a_2	-0.419				1.437	-0.9696	-0.3074
a_3	0.0939				-1.446	0.4461	0.0688
a_4	-0.00255				0.489	-0.0889	-0.0019
a_5	-0.00088				-0.0547	-0.0064	-0.0064
a_6	0.0661				0.0665	0.0692	0.0484
a_7	0.628				0.537	0.6886	0.4600
a_8	0.924				0.832	0.3135	0.6770

6.2.2.5 Moment about the Trailing Edge due to the Quasi-Static and Slamming Forces

Subject to the limitations in Article 6.2.2.1, the moment above the trailing edge may be determined as:

$$M_t = F_V W \left[b_1 + b_2 \left(\frac{W}{\lambda}\right) ln\left(\frac{W}{\lambda}\right) + b_3 e^{-\left(\frac{Z_c}{\eta_{max} - Z_c}\right)} \right] - F_s \left(\frac{2}{3} W - w'\right) \qquad (6.2.2.5\text{-}1)$$

where:

M_t = moment about the trailing edge of the bridge span due to the quasi-static vertical and horizontal forces and the slamming force (ft-kips/ft)
F_v = vertical quasi-static hydrostatic and hydrodynamic force per unit length of the span specified in Article 6.2.2.2 (kips/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)
W = bridge width (ft)
λ = wave length (ft)
η_{max} = distance from the storm water level to design wave crest (ft)
b_1-b_3 = coefficients from Table 1
w′ = horizontal projection of overhang (ft)

The dimensions Z_c and η_{max} and parameter λ shall be determined based on the wave and surge heights consistent with the level of analysis using the provisions of Articles 6.3 through 6.3.4.

Table 6.2.2.5-1 – Coefficients for Moment

Coefficients	AASHTO Type III	AASHTO Type IV	AASHTO Type VI	Florida Bulb-T 72	Florida Bulb-T 78	21" Voided Slab	36" Adjacent Box Girders
b_1	-0.521				-0.622	-0.455	-0.495
b_2	1.179				0.593	1.190	1.152
b_3	0.270				0.246	0.288	0.279

C6.2.2.5

The associated horizontal force is much smaller than the vertical force. It is also a bit erratic since a small change in the position of the wave at the time of maximum vertical force can have a big impact on the horizontal force. For this reason, the moment due to the horizontal force was embedded in the equation for moment by adjusting the moment arm for the vertical quasi-static force. That is, the coefficients for the parametric moment equation are based on the PBM computed moments that include the horizontal force component.

6.2.3 Hydrodynamic Loads on Substructure

6.2.3.1 General

Loads which may be imparted to substructure elements such as the piles, pile cap or piers shall be considered as an integral part of the bridge analysis and design. 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.

C6.2.3.1

The equations herein suggest static loading, but the action of waves is clearly repetitive. At this writing the effect of repetitive load on degradation of foundation capacity, especially when that load results in net tension on foundation elements such as piles and shafts, is not known.

The loads on substructure were not subject to same calibration studies as the loads on superstructure. It is assumed that the same load factors apply to substructure and superstructure.

6.2.3.2 Forces on Exposed Piles and Columns

For exposed piles and columns, the Morison equation should be used for determination of forces due to non-breaking waves resulting from the maximum wave height, H_{max}, and they shall be determined as:

$$F = \left[C_d(\rho_w/2)Au|u| + C_m(\rho)V\frac{du}{dt} \right]/1000 \qquad (6.2.3.2\text{-}1)$$

where:

F = force on element per unit length (k/ft)
ρ_w = mass density of water taken as 2.0 slugs/ft^3
u = horizontal component of water particle velocity for H_{max} including current (ft/sec)
A = projected area per unit length (for a circular pile of diameter D, A = D; ft^2/ft)
V = displaced volume per unit length (for a circular pile of diameter D, V = $\pi D^2/4$ ft^3/ft)

C_d = Morison drag coefficient (dim)
C_m = Morison inertia coefficient (dim)
du/dt = horizontal acceleration of water particles for H_{max} (ft/sec^2)

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.

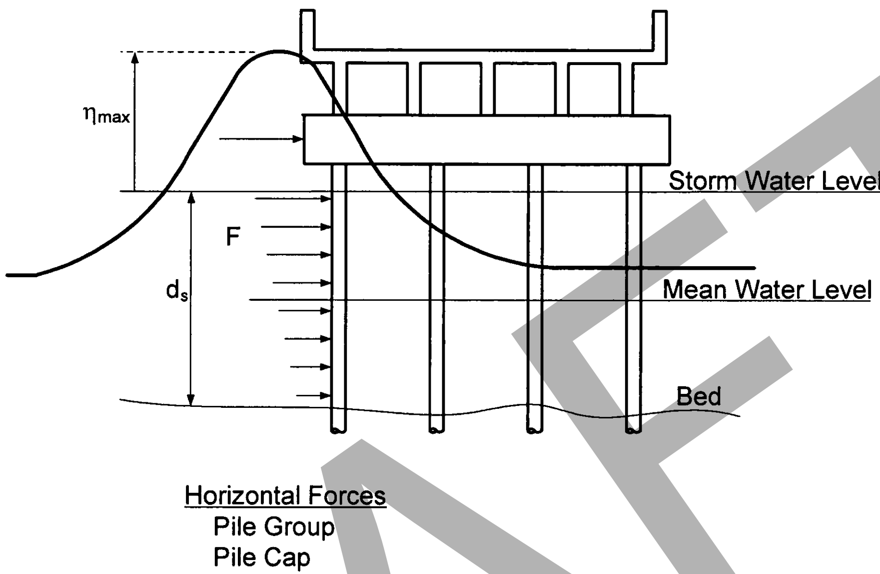


Figure 6.2.3.2-1 – Schematic Showing Hydrodynamic Load Effects on Substructure Elements

C6.2.3.2

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

Horizontal velocity, U, and acceleration, du/dt, of water particles should be calculated using the appropriate wave theory which is dependent on the wave height, period and water depth. Reference is made to Figure II-1-20 of the Coastal Engineering Manual. For most bridge locations and design events, Stream Function theory will be applicable. A Java Applet can be found at http://www.coastal.udel.edu/faculty/rad/streamless.html that allows for calculation of these parameters. Maximum horizontal particle velocity occurs at the crest of the wave while maximum horizontal acceleration occurs near the still water level.

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

Smooth: $C_d = 0.65$, $C_m = 1.6$
Rough: $C_d = 1.05$, $C_m = 1.2$

where:

U_{mo} = maximum wave induced orbital velocity (ft/sec)
T_{app} = apparent wave period (accounting for design current) (sec)
D = diameter of the cylinder (ft)

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

6.2.3.3 Forces on Exposed Pier Shafts and Walls

For forces on exposed pier shafts and walls wave pressure on the front of a vertical wall shall be determined as:

$$p_1 = \frac{1}{2}(1+\cos\theta)(\alpha_1)\gamma_w H_{max} \qquad (6.2.3.3\text{-}1)$$

$$p_2 = \frac{p_1}{\cosh(2\pi d_s / \lambda)} \qquad (6.2.3.3\text{-}2)$$

in which:

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi d_s / \lambda}{\sinh(4\pi d_s / \lambda)}\right]^2 \qquad (6.2.3.3\text{-}3)$$

$$\eta^* = 0.75(1+\cos\theta)H_{max} \qquad (6.2.3.3\text{-}4)$$

where:

p_1 = pressure at storm water level (k/ft^2)
p_2 = pressure at mudline (k/ft^2)
α_1 = coefficient
θ = angle between direction of wave approach and a line normal to the structure
H_{max} = design wave height (ft)
d_s = water depth at or near the bridge including surge, astronomical tide, and local wind setup (ft)
λ = wave length (ft)
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 (ft)

The calculation of applied wave pressure shall be based on pressure prism specified in Figure 1, to which the current forces must be added.

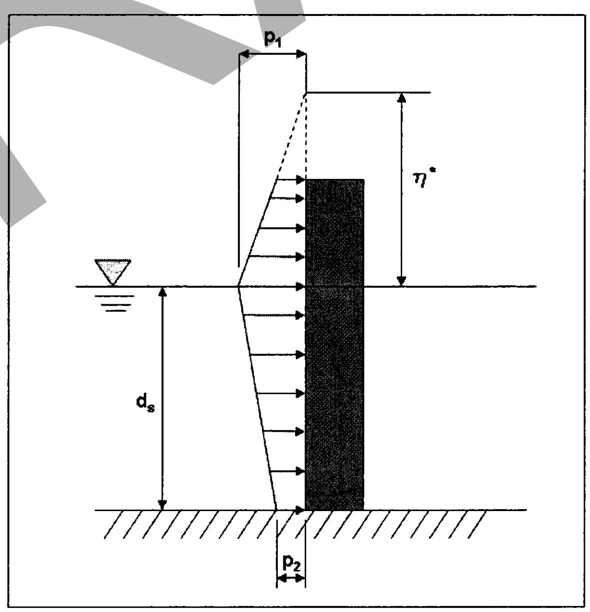


Figure 6.2.3.3-1 – Wave Force on Large Element

C6.2.3.3

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

6.3 Levels of Analysis of Forces from Coastal Storms

6.3.1 General

INCLUDE:

- GENERAL DISCUSSION OF LEVEL I, II & III - JMK

6.3.2 Level I Analysis of Design Parameters

The 100-year value for the parameters storm surge, wind speed, wind setup, current if readily available from previous studies, wave height and period shall be used simultaneously in a Level I analysis.

C6.3.2

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 (e.g., open coast, center of a near circular bay) this combination will produce a 100-year event. However, for most bridge locations (e.g., bridges over long narrow waterways) the combination of 100-year components will yield a less frequent event. These differences are accounted for in the load modifiers presented in Tables 6.2.2.6-1 and 6.2.2.6-2.

A Level I analysis is one step above a screening analysis that might be used to identify critical bridges for retrofit, and the approach is suitable for eliminating bridge spans from further analysis. In most cases, a Level II analysis should be performed prior to retrofitting.

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

Even for a Level I analysis, a review of data and interpretation of results by a coastal engineer is required.

6.3.2.1 Required Information

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

6.3.2.2 Design Wind Velocity

The base design wind velocity may be based on the following:

- If 100-year coastal storm wind speeds exist at the site then these values should be used.
- The design wind shall be the 100-year wind speed determined as 107% of the wind speeds given in Figure 1 taken from ASCE Standard 7-05 (ASCE, 2005).
- The 500-year wind speed shall be determined as 123% of the wind speeds given in Figure 1 taken from ASCE Standard 7-05 (ASCE, 2005).

The base design wind velocity should be adjusted for elevation other than the standard 32.8 ft using Equation 3.8.1.1-1 of the *AASHTO LRFD Bridge Design Specifications*.

C6.3.2.2

ASCE 7-05 tabulates winds for a 50-year event based on a 3 second gust. ASCE 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.

Likewise, the factor for 500 years is 1.23 for the continental U.S. and 1.18 for Alaska. The difference has also been ignored herein.

Adjustments for duration and return interval may also be appropriate using factors and equations herein.

LRFD used rounded elevations of 30’, not 32.8’

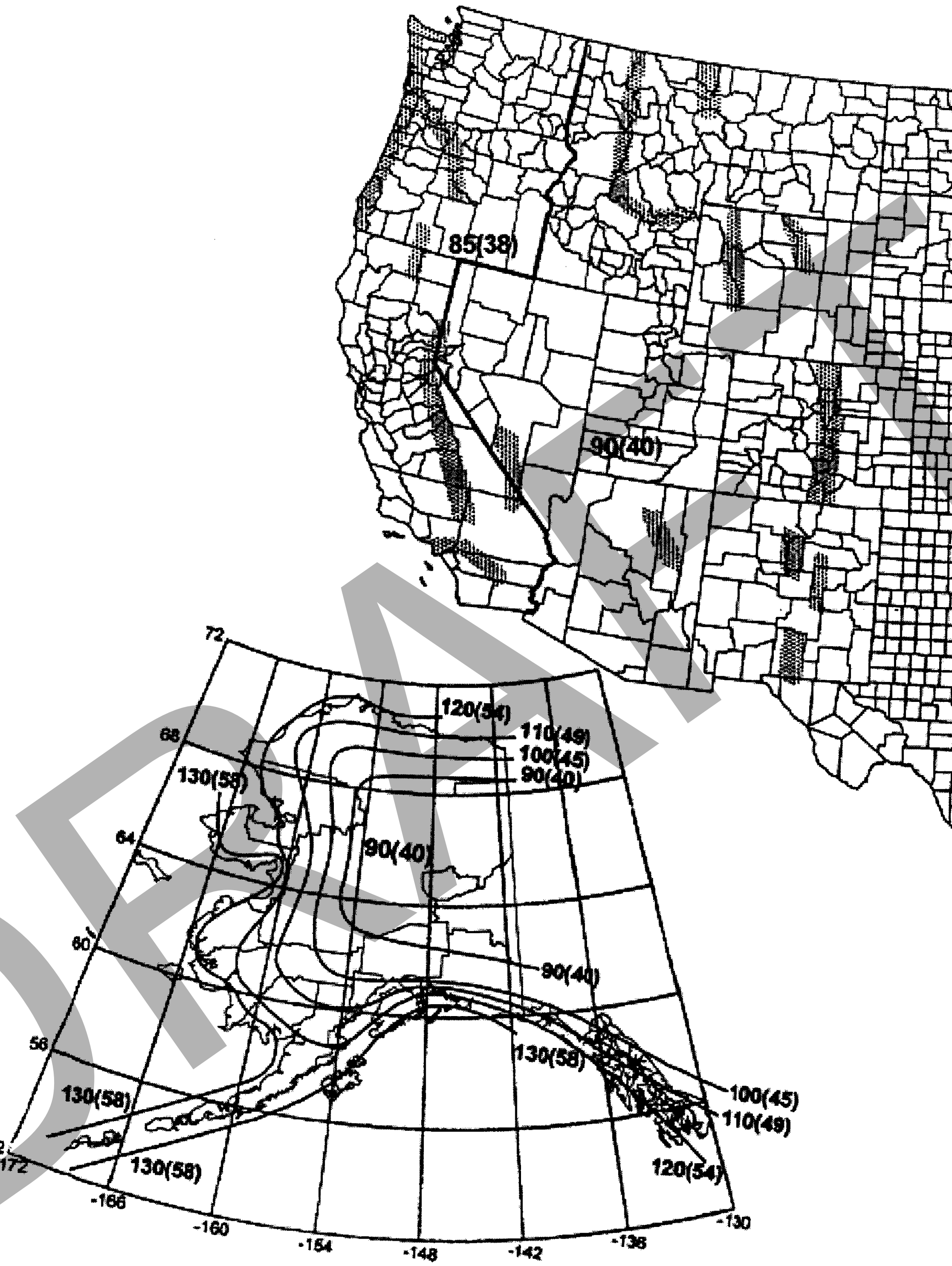
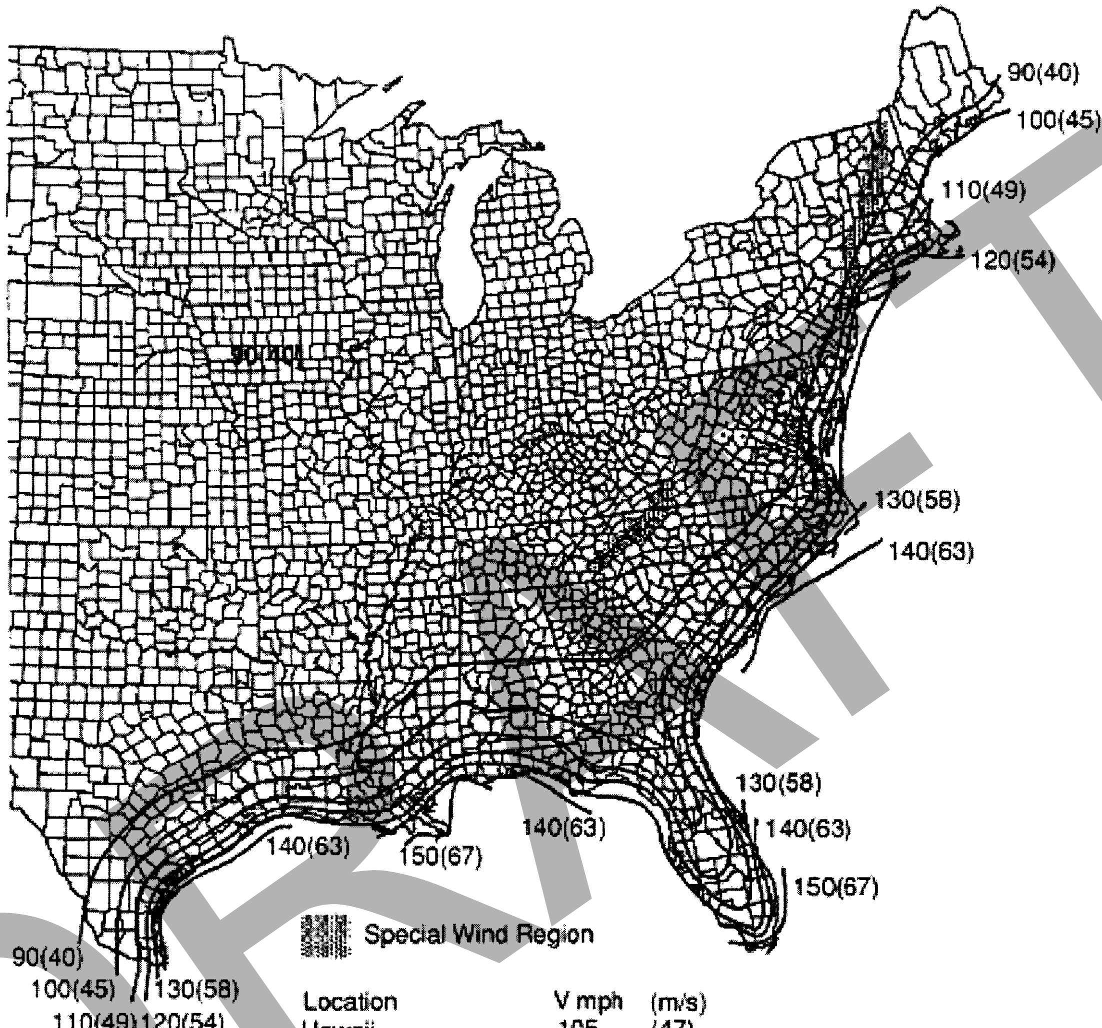


FIGURE 6-1 BASIC WIND SPEED

Figure 6.3.2.2-1 (1 of 5) – 50 Year 3 Second Gust Wind Speeds (ASCE 07-05)



Location	V mph	(m/s)
Hawaii	105	(47)
Puerto Rico	145	(65)
Guam	170	(76)
Virgin Islands	145	(65)
American Samoa	125	(56)

Notes:
1. Values are nominal design 3-second gust wind speeds in miles per hour (m/s) at 33 ft (10 m) above ground for Exposure C category.
2. Linear interpolation between wind contours is permitted.
3. Islands and coastal areas outside the last contour shall use the last wind speed contour of the coastal area.
4. Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

Figure 6.3.2.2-1 (2 of 5)

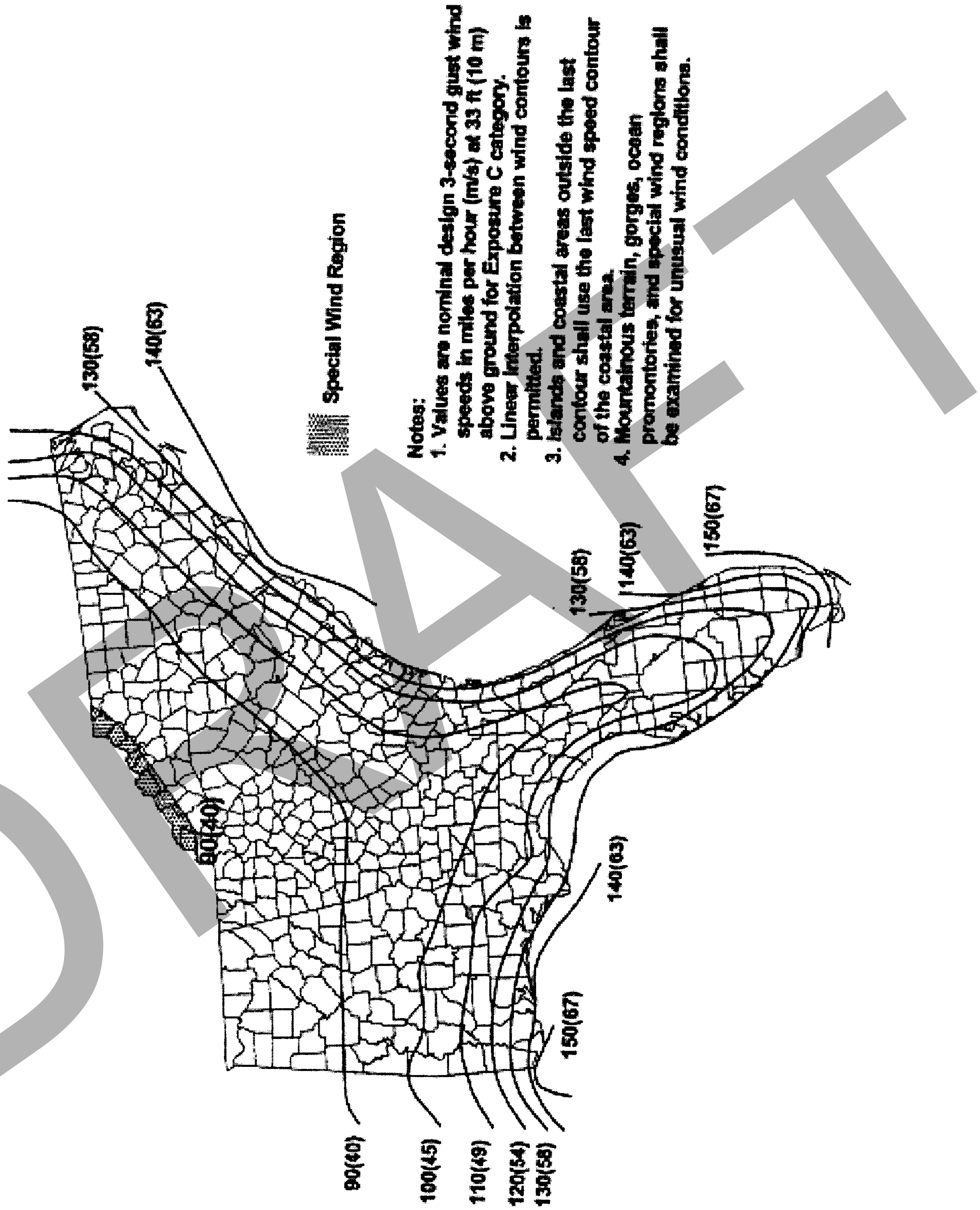


Figure 6.3.2.2-1 (3 of 5)

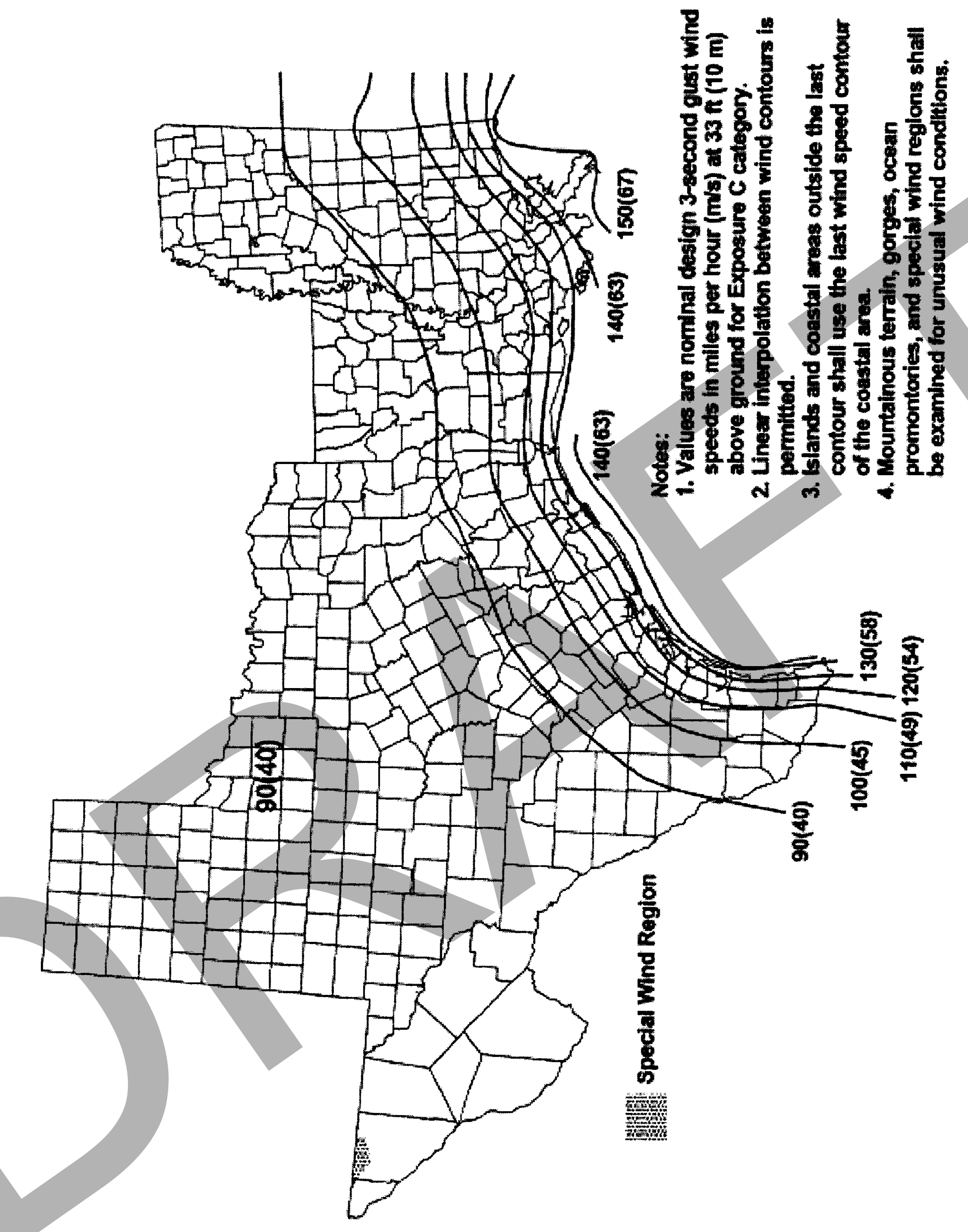


Figure 6.3.2.2-1 (4 of 5)

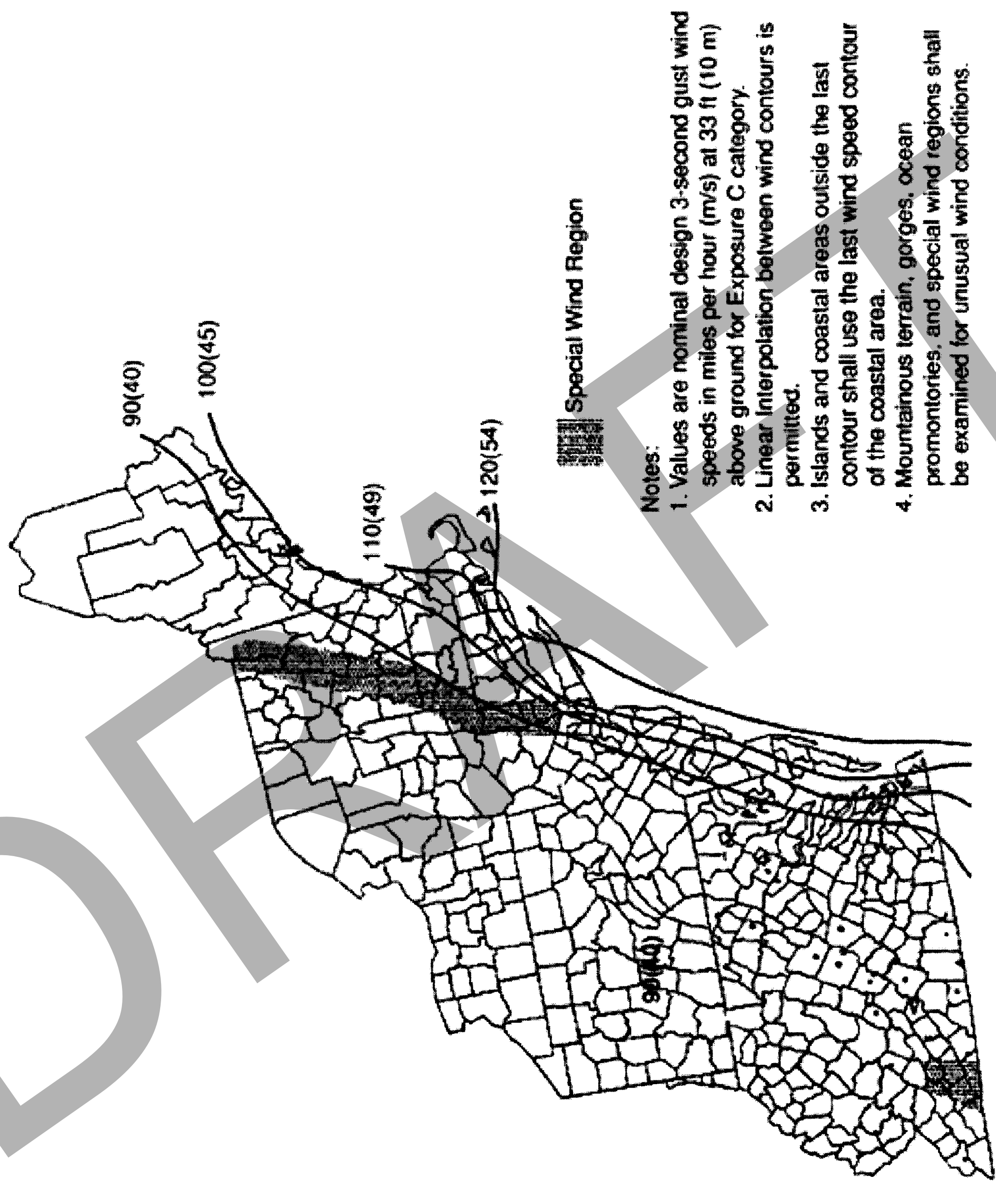


Figure 6.3.2.2-1 (5 of 5)

6.3.2.3 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 local wind setup. The 100-year storm surge elevation should be taken from the best available source (FEMA, NOAA, State Agencies, other reliable sources). If the storm surge source does not explicitly state that the surge level accounts for the joint probability of its occurrence with astronomical tides, then the storm surge value should be added to the Mean Higher Water Level (MHHW) at the site to set the storm surge elevation.

The local wind setup should be computed using the 100-year wind speed; the most critical fetch, and be determined as:

$$S = d^* \left(\sqrt{1 + \frac{2\,n\,\tau_{wx}\,F}{\gamma_w (d^*)^2}} -1 \right) \qquad (6.3.2.3\text{-}1)$$

in which:

- If $U_{10min} \leq 18.4$ ft/s, then $k = 1.2 \times 10^{-6}$ (6.3.2.3-2)

- If $U_{10min} > 18.4$ ft/s, then $k = 1.2 \times 10^{-6} + 2.25 \times 10^{-6} \left(1 - \frac{18.4 \text{ ft/s}}{U_{10min}}\right)^2$ (6.3.2.3-3)

and

$$\tau_{wx} = \rho_w \frac{k\,U_{10min}\left|U_{10min}\right|}{1000} \qquad (6.3.2.3\text{-}4)$$

where:

S	=	local wind setup measured from the storm still water level (ft)
d^*	=	average water depth over the fetch including storm surge and astronomical tide (ft)
n	=	1.3 (dim)
F	=	fetch length in the direction of the wind from the upwind shore (ft)
τ_{wx}	=	wind shear stress on water surface (k/ft^2)
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 (ft)
$U_{10\,min}$	=	wind speed at the standard 32.8 ft elevation for a 10 minute duration (ft/s)
ρ_w	=	mass density of water taken as 2.0 slugs/ft^3
k	=	unitless parameter

The wind speed adjustment provisions of Article 6.3.2.4 should be used to determine $U_{10\,min}$.

C6.3.2.3

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.

As a practical matter, $U_{10\,min} \leq 18.4$ ft/s as required by Equation 2 is such a low wind speed that Equation 2 will seldom apply. It is provided for completeness.

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.

6.3.2.4 Design Wave Parameters

The wind stress factor, U_A, peak period T_p, and time required to reach a fetch limited wave, t, shall be determined using empirical equations 1-5 in the steps indicated, which shall repeated until the value of t converges acceptably:

- The wind-stress factor, U_A, may be determined from the surface wind, U_s as:

$$U_A = 1.4667\left(0.589\right) U_s^{\;1.23} \qquad (6.3.2.4\text{-}1)$$

- The wave period may be determined as:

- $$T_p = 7.54 \tanh\left[0.833\left(\frac{gd}{U_A^{\;2}}\right)^{3/8}\right] \tanh\left\{\frac{0.0379\left(\dfrac{gF}{U_A^{\;2}}\right)^{1/3}}{\tanh\left[0.833\left(\dfrac{gd}{U_A^{\;2}}\right)^{3/8}\right]}\right\}\left(\frac{U_A}{g}\right) \qquad (6.3.2.4\text{-}2)$$

- The time duration required to develop a fetch limited wave may be determined as:

$$t = 537\left(\frac{gT_P}{U_A}\right)^{7/3}\left(\frac{U_A}{g}\right) \qquad (6.3.2.4\text{-}3)$$

- The surface wind speed, U_s, shall be adjusted from its base duration (3 second gust for ASCE 7-05) to a one-hour wind speed using either Equation 4 or 5, and from a one-hour duration to Duration t using either Equation 4 or 5.

- If $1 < t < 3600$ sec, then

$$\frac{U_t}{U_{1hr}} = 1.277 + 0.296 \tanh\left(0.9 \log\left(\frac{45}{t}\right)\right) \qquad (6.3.2.4\text{-}4)$$

- If $3{,}600 < t < 36{,}000$ sec, then

$$\frac{U_t}{U_{1hr}} = -0.15 \log t + 1.5334 \qquad (6.3.2.4\text{-}5)$$

in which:

g = gravitational constant (ft/sec^2)
U_A = wind-stress factor (ft/sec)
U_s = 100-year wind velocity at the standard 32.8 ft elevation unless modified for duration as specified in Article 6.3.2.2 (mph)
d = average water depth over the fetch including surge, astronomical tide, and local wind setup (ft)
F = fetch length in the direction of the wind from the upwind shore (ft)
T_p = wave period (sec)
t = duration of U_s (sec)

Once the value of t has converged and the associated value of T_p calculated, the remaining wave characteristics shall be determined as follows:

- The significant wave height shall be determined as:

$$H_s = 0.283\tanh\left[0.53\left(\frac{gd}{U_A^2}\right)^{3/4}\right]\tanh\left\{\frac{0.00565\left(\frac{gF}{U_A^2}\right)^{1/2}}{\tanh\left[0.53\left(\frac{gd}{U_A^2}\right)^{3/4}\right]}\right\}\left(\frac{U_A^2}{g}\right) \qquad (6.3.2.4\text{-}6)$$

- The wave length may be determined as:

$$\lambda = \frac{gT_p^2}{2\pi}\sqrt{\tanh\left(\frac{4\pi^2}{T_p^2}\frac{d_s}{g}\right)} \qquad (6.3.2.4\text{-}7)$$

The value of λ calculated above should be further adjusted when using equations in Article 6.2.2.

- The assumed maximum wave height shall be determined as:

$$H_{max} = 1.80H_s \qquad (6.3.2.4\text{-}8)$$

- The maximum wave height H_{max} should be limited for depth and for steepness using the lesser of the results of Equations 9 and 10, respectively.

$$H_{max} \le 0.65\,d_s \qquad (6.3.2.4\text{-}9)$$

$$H_{max} \le \lambda/7.0 \qquad (6.3.2.4\text{-}10)$$

- The assumed maximum distance from the storm water level to the design wave crest shall determined as:

$$\eta_{max} = 0.70H_{max} \qquad (6.3.2.4\text{-}11)$$

where:

d_s = water depth at or near the bridge including surge, astronomical tide, and local wind setup (ft)

Alternatively, nonlinear wave theory may be used to determine more accurate values of η_{max} and λ.

C6.3.2.4

The factor 1.4667 is a conversion from mph to fps.

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., 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 are 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).

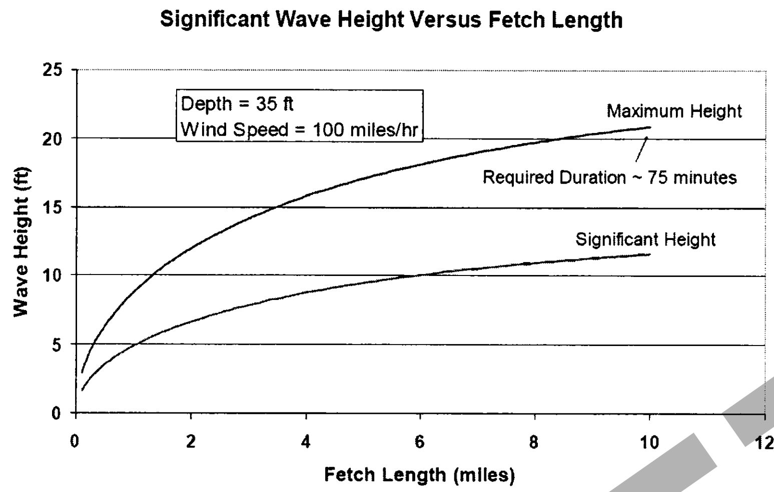


Figure C6.3.2.4-1 Affect of Fetch Length on Wave Height for a Particular Site

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 ft.

While the determination of duration as a function of the wave period which is itself dependent on wind velocity of a given duration is iterative, experience has shown that the procedure convenes quickly; one cycle often suffices.

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 as follows:

$H_{1/3}$ 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% of all waves (C6.3.2.4-1)
$H_5 \approx 1.37\ H_s$ = average of highest 5% of all waves (C6.3.2.4-2)
$H_1 \approx 1.67\ H_s$ = average of highest 1% of all waves (C6.3.2.4-3)
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. 1.80 H_s is sometimes conservatively taken as specified in Equation 8:

The water depth, d_s, should usually be taken as the average depth over the fetch, but site-specific factors could be used to make adjustments. For example, where the water depth decreases significantly as the wave would approach the bridge, consideration may be given to using the average depth over approximately 200 ft each side of the bridge. Similarly, if there is a possibility that general scour or channel migration could result in a change in approach water depth, these factors should also be considered.

The appropriate wave theory to use for improved determination of η_{max} and λ is dependent on the wave height, period and water depth. Reference is made to Figure II-1-20 of the Coastal Engineering Manual. For most bridge locations and design events, stream function theory will be applicable. A Java Applet can be found at http://www.coastal.udel.edu/faculty/rad/streamless.html that allows for calculation of these parameters.

6.3.2.5 Maximum Water Elevation

The design wave crest elevation may be determined as:

$$MWE = \text{elevation of bed} + d_s + \eta_{max} \quad (6.3.2.5\text{-}1)$$

where:

MWE = design wave crest elevation
d_s = water depth at or near the bridge including surge, astronomical tide, and local wind setup (ft)
η_{max} = wave crest height for the 100-year event

6.3.2.6 Load Factor Modification Due to Local Conditions

When sufficient local information exists, the wind velocity determined as specified in Article 6.3.2.2 may be modified for probable approach angle sector and fetch angle relative to the orientation of the coastline using the wind speed reduction factor specified in Table 1.

Table 6.3.2.6-1 – Wind Speed Adjustment Factors for Different Fetch Angles and Fetch Angle Segments.

Angle Segment (deg)	5	10	15	25	50	75	100	125	150	180	225	300	360
Angle (deg)													
0 to ±90													
±91 to ±150													
±151 to ±180													

The modified wind speeds may then be used to determine the local wind setup and wave height as specified in Articles 6.3.2.3 and 6.3.2.4, respectively.

C6.3.2.6

For Level I (and some Level II) analyses the meteorological/oceanographic conditions used in the storm surge/wave force calculations are “worst case” scenarios in that the assumption is that the storm surge, maximum local wind setup and maximum wave conditions all occur at the same time. For many situations the probability of this is much lower than 0.01 (100-year return interval). Since wind directions during the passage of a hurricane can be from any direction and there are a wide range of paths that can produce large storm surge elevations in coastal water bodies, quantifying the reduced probability is difficult. Nevertheless, conservative meteorological/oceanographic parameter reduction factors have been established and are presented in Table 1. The local parameters initially thought to impact the meteorological/oceanographic conditions (i.e. the local wind setup and the wave heights and periods) at the site are:

- Wave fetch direction relative to the coastline direction
- Wave fetch angle segment
- Local wind fetch direction relative to the coastline direction
- Local wind fetch angle segment

Other factors such as 1) the magnitude of bathymetry variations over the fetch, 2) elevation of land masses surrounding the water body, 3) shape of water body, 4) size of the inlet, etc. may be added to this list at a later date.

Note that modifications to the design meteorological/oceanographic parameters will change the storm surge/wave loads on the bridge superstructure. The amount of the change will differ depending on the superstructure shape, dimensions and the low member elevation relative to the storm water level. Adjustments for the site specific conditions are made to the design wind speeds. In general, the wind adjustment will be different for waves and local wind setup due to differences in mean fetch angles and fetch angle segments. The wind reduction factors are presented in Table 1 for both waves and local wind setup.

Local wind setup/setdown depends on wind speed and duration, water depth over the fetch, fetch, water body length and shape and connections to and the size of other water bodies (gulfs, oceans, etc.). For simple shaped water bodies of near uniform depth a conservative estimate of the setup can be obtained using equations such as Equation 6.3.2.3-1. Note that this equation only predicts setup and thus is conservative in many, if not most, cases. For water bodies with complex boundaries, such as that shown in Figure C1, significant setup can be produced by wind from a range of directions as can be seen in Table 1 and Figure C2. For Level I analyses both the local wind fetch and the fetch angle segment will have to be conservative. More rigorous Level II and III analyses will most likely produce reduced setup values. The time required to reach an equilibrium setup is not well documented. A ten minute average wind speed is recommended for computations at this time.

As can be seen in Figure C2 there is no clear cutoff for the fetch angle segment. Wind from a wide range of directions produce setup at the bridge site. The larger the wind setup fetch angle segment the more likely the maximum conditions computed in the Level I analysis will occur and therefore the smaller the water level reduction factor. The choice of wind setup fetch angle will therefore depend on the desired level of conservatism.

Waves can change directions due to refraction, diffraction and reflection but for the purpose of screening and Level I analyses changes in direction greater than about 50 degrees can be ignored when estimating wave fetch and fetch angle segment. Examples of wave fetches and fetch angle segments are shown in Figure C3 for the I-10 Blackwater Bay Bridge near Pensacola, Florida.

The values in Table 1 were determined using a "Delphi Process" in which experienced coastal engineers considered the joint probability of occurrence of maximum wind speed, surge and wave height at a given site, and based on practical considerations and past observations of phenomena selected the given values. At this writing, the Delphi Process was used to compensate for a lack of sufficient data on joint probability. The values in these tables should be reconsidered when better data become available.

No table of load multipliers is provided for Level III analysis as the issue of joint probability and refinements for site nuances are assumed to be included in the refined analysis required for Level III.

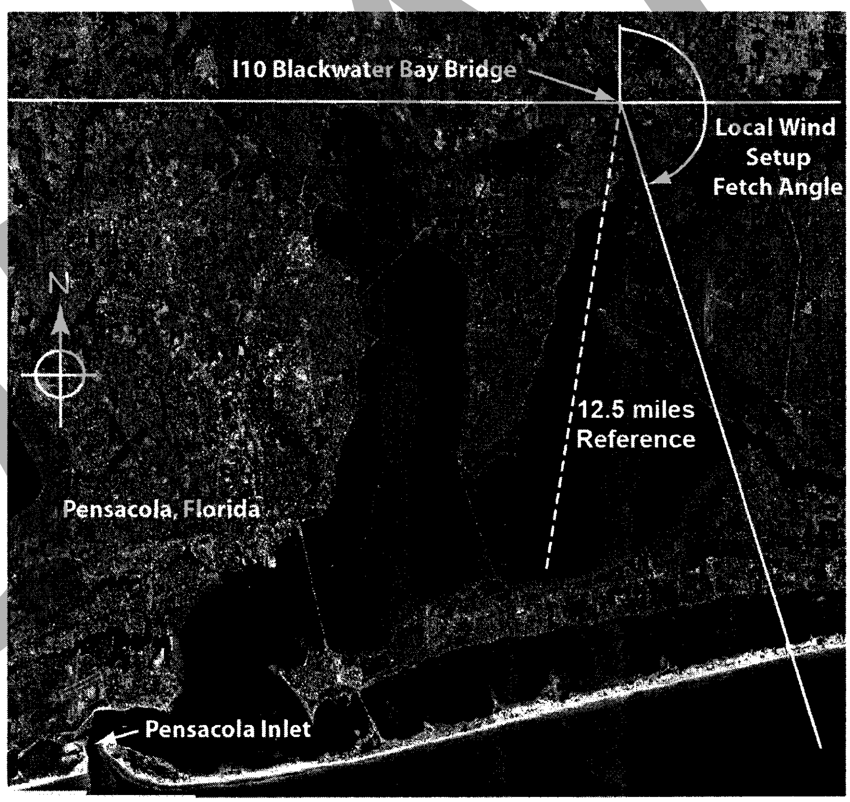


Figure C6.3.2.6-1 – This figure coupled with Table C1 and Figure C2 illustrates the difficulty in estimating local wind setup fetch and fetch angle segment. The wind speed used in the analysis was the predicted 100-year return interval wind (97.4 miles/hour).

Table C6.3.2.6-1 – Equivalent Local Wind Setup Fetch as a Function of Wind Direction (from) for the I-10 over Blackwater Bay Bridge shown in Figure C1.

Fetch Angle[1] (compass angle) (degrees)	**Local Wind Setup at the Bridge Site (ft)**
110	3.3
140	4.2
160	4.9
180	5.5
195	6.3
205	6.8
210	7.2
215	7.4
220	7.7
225	7.8
230	7.9
235	7.9
240	7.7
245	7.6
250	7.2
255	6.8
260	6.4
265	5.6
270	5.1

[1] Angle from which the wind is blowing

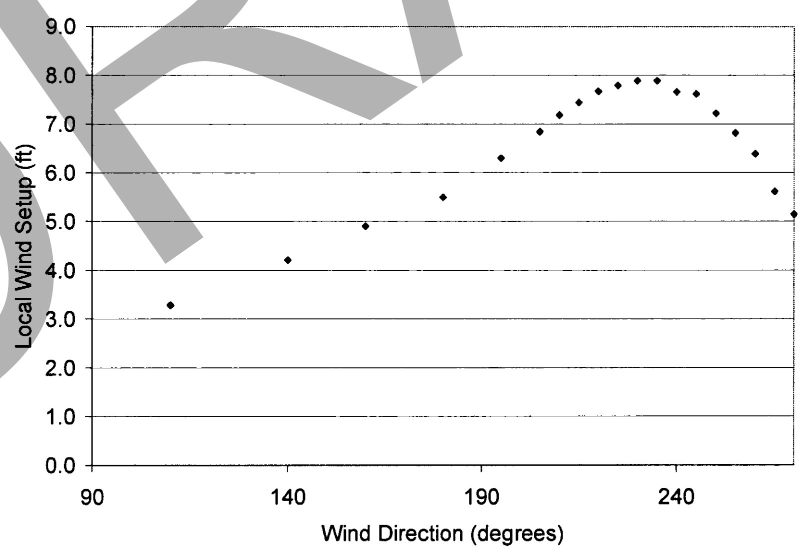


Figure C6.3.2.6-2 – Plot of Local Wind Setup/Setdown (above/below storm water level) as a function wind angle. The wind angle (compass angle) is the direction the wind is FROM.

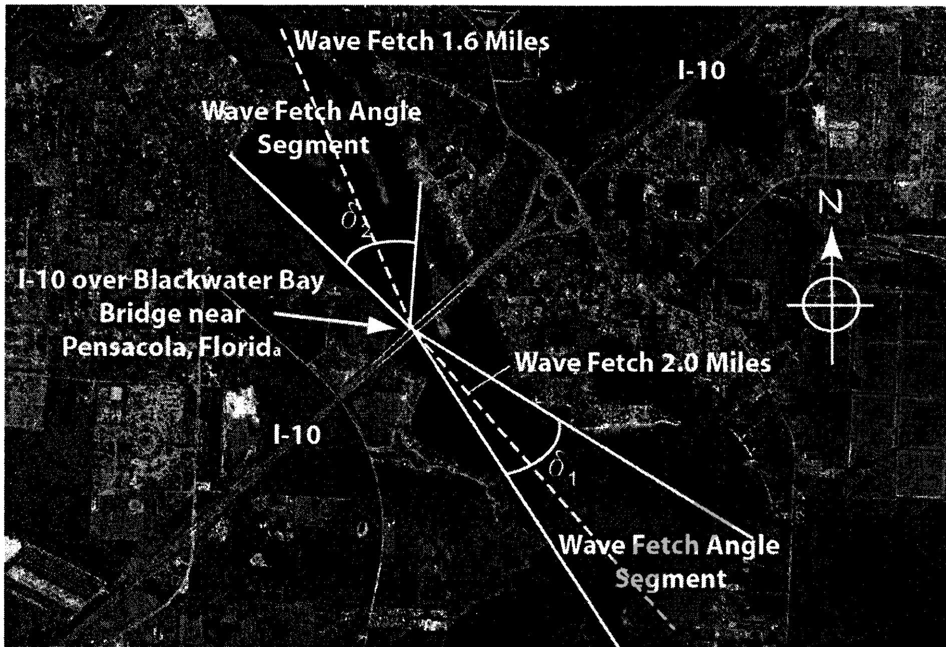


Figure C6.3.2.6-3 – Example of Wave Fetch and Wave Fetch Angle Segment

6.3.3 Level II Analysis of Design Parameters

6.3.3.1 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.

C6.3.3.1

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 a 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 should be 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 compared to 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 reliable estimates of wind setup and wave parameters.

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

6.3.3.2 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. The provisions of Article 6.3.2.6 may also be considered as long as the approach angle and wind sector have not been included in other refinements.

C6.3.3.2

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 sufficient for a 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.

6.3.3.3 Design Water Level

Wherever possible, a Level II analysis of design water level should be based on data obtained from several agencies sources 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 6.3.4.

C6.3.3.3

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 (e.g., astronomical tides, wave setup) included in these analyses differ greatly from location to location, and agency to agency, and to a large extent on when the analysis was performed. The backup reports for this data should be reviewed to determine the complexity of the modeling effort and what physical effects were accounted for.

6.3.3.4 Design Wave Parameters

Advanced numerical models may be used to improve on the magnitude and timing of the design wave height and period. Important effects such as wave refraction or diffraction are not considered in a Level I analysis, but could be addressed using numerical models or analytical techniques in a Level II analysis. Acquisition of improved bathymetry for input to such models or the analytical techniques presented in the Level I analysis may also be appropriate.

The determination of the maximum wave height, H_{max}, shall include consideration of fetch limitation, depth limitation, duration, and wave steepness.

C6.3.3.4

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. Therefore, their application 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 interpreting the results.

6.3.3.5 Maximum Water Elevation

The provisions of Article 6.3.2.5 shall apply.

6.3.3.6 Design Current Velocities

Advanced numerical models may be used to improve on the magnitude of the design current velocity developed for a Level I analysis. Improvements 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.

C6.3.3.6

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.

6.3.4 Level III Analysis of Design Parameters

6.3.4.1 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.

Where sufficient meteorological and oceanographic data exists to consider return periods other than 100 years and 500 years, a multi-level approach with site-specific performance criteria may be considered.

C6.3.4.1

Level III analyses:

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

The modeling effort outlined Articles 6.3.4.2 improves the accuracy of all meteorological/oceanographic parameters needed for the computation of storm surge and wave loading on bridge sub and super structures. This improved accuracy includes design water elevations, current velocities, currents, and wave heights and periods and their joint probabilities.

The determination of the maximum wave height, H_{max}, shall include consideration of fetch limitation, depth limitation, duration, and wave steepness.

6.3.4.2 Design Wind Velocity Storm Water Level, 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.

C6.3.4.2

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. Hurricane windfield hindcasting is a very specialized discipline, so in most cases, these analyses should be accomplished by meteorologists who 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 hindcast storms, perform an extremal analysis on these parameters to obtain the values for the desired design return interval.

7. <u>REFERENCES</u>