

2008

Guide Specifications for Bridges Vulnerable to Coastal Storms

1
EDITION



AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

AVA Documents
Document Sharing Hub

2008

Guide Specifications for Bridges Vulnerable to Coastal Storms

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS



444 North Capitol Street, NW, Suite 249
Washington, DC 20001

www.transportation.org

© 2008 by the American Association of
All rights reserved. Duplication

Full document details: AASHTO BVCS-1-2008

AVA Documents
Document Sharing Hub

Copyright © 2008, by the American Association of State Highway and Transportation Officials. All Rights Reserved. Printed in the United States of America. This book, or parts thereof, may not be reproduced in any form without written permission of the publishers.

EXECUTIVE COMMITTEE 2008–2009

Voting Members

Officers:

President: Allen Biehler, Pennsylvania

Vice President: Larry L. “Butch” Brown, Sr., Mississippi

Secretary-Treasurer: Carlos Bracerias, Utah

Regional Representatives:

REGION I: Carolann Wicks, Delaware, One-Year Term
Joseph Marie, Connecticut, Two-Year Term

REGION II: Larry L. “Butch” Brown, Mississippi, One-Year Term
Dan Flowers, Arkansas, Two-Year Term

REGION III: Kirk T. Steudle, Michigan, One-Year Term
Nancy J. Richardson, Iowa, Two-Year Term

REGION IV: Rhonda G. Faught, New Mexico, One-Year Term
Will Kempton, California, Two-Year Term

Nonvoting Members

Immediate Past President: Pete K. Rahn, Missouri

AASHTO Executive Director: John Horsley, Washington, DC

HIGHWAYS SUBCOMMITTEE ON BRIDGES AND STRUCTURES, 2008

MALCOLM T. KERLEY, *Chair*

KEVIN THOMPSON, *Vice Chair*

M. MYINT LWIN, Federal Highway Administration, *Secretary*

FIRAS I. SHEIKH IBRAHIM, Federal Highway Administration, *Assistant Secretary*

ALABAMA, John F. Black, William F. Conway, George H. Conner

ALASKA, Richard A. Pratt

ARIZONA, Jean A. Nehme

ARKANSAS, Phil Brand

CALIFORNIA, Kevin Thompson, Susan Hida, Barton J. Newton

COLORADO, Mark A. Leonard, Michael G. Salamon

CONNECTICUT, Gary J. Abramowicz, Julie F. Georges

DELAWARE, Jiten K. Soneji, Barry A. Benton

DISTRICT OF COLUMBIA, Nicolas Glados, L.

Donald Cooney, Konjit “Connie” Eskender

FLORIDA, Robert V. Robertson, Jr., Marcus Ansley, Andre Pavlov

GEORGIA, Paul V. Liles, Jr., Brian Summers

HAWAII, Paul T. Santo

IDAHO, Matthew M. Farrar

ILLINOIS, Ralph E. Anderson, Thomas J. Domagalski

INDIANA, Anne M. Rearick

IOWA, Norman L. McDonald

KANSAS, Kenneth F. Hurst, James J. Brennan, Loren R. Risch

KENTUCKY, Allen Frank

LOUISIANA, Hossein Ghara, Arthur D’Andrea, Paul Fossier

MAINE, David Sherlock, Jeffrey S. Folsom

MARYLAND, Earle S. Freedman, Robert J. Healy

MASSACHUSETTS, Alexander K. Bardow

MICHIGAN, Steven P. Beck, David Juntunen

MINNESOTA, Daniel L. Dorgan, Kevin Western

MISSISSIPPI, Mitchell K. Carr, B. Keith Carr

MISSOURI, Dennis Heckman, Michael Harms

MONTANA, Kent M. Barnes

NEBRASKA, Lyman D. Freemon, Mark Ahlman, Hussam “Sam” Fallaha

NEVADA, Mark P. Elicegui, Marc Grunert, Todd Stefonowicz

NEW HAMPSHIRE, Mark W. Richardson, David L. Scott

NEW JERSEY, Richard W. Dunne

NEW MEXICO, Jimmy D. Camp

NEW YORK, George A. Christian, Donald F. Dwyer, Arthur P. Yannotti

NORTH CAROLINA, Greg R. Perfetti

NORTH DAKOTA, Terrence R. Udland

OHIO, Timothy J. Keller, Jawdat Siddiqi

OKLAHOMA, Robert J. Rusch, Gregory D. Allen

OREGON, Bruce V. Johnson, Hormoz Seradj

PENNSYLVANIA, Thomas P. Macioce, Harold C. “Hal” Rogers, Jr., Lou Ruzzi

PUERTO RICO, Jaime Cabré

RHODE ISLAND, David Fish

SOUTH CAROLINA, Barry W. Bowers, Jeff Sizemore

SOUTH DAKOTA, Kevin Goeden

TENNESSEE, Edward P. Wasserman

TEXAS, William R. Cox, David P. Hohmann

U.S. DOT, M. Myint Lwin, Firas I. Sheikh Ibrahim, Hala Elgaaly

UTAH, Richard Miller

VERMONT, William Michael Hedges

VIRGINIA, Malcolm T. Kerley, Kendal Walus, Prasad L. Nallapaneni, Julius F. J. Volgyi, Jr.

WASHINGTON, Jugesh Kapur, Tony M. Allen, Bijan Khaleghi

WEST VIRGINIA, Gregory Bailey

WISCONSIN, Scot Becker, Beth A. Cannestra, Finn Hubbard

WYOMING, Gregg C. Fredrick, Keith R. Fulton

ALBERTA, Tom Loo

NEW BRUNSWICK, Doug Noble

NOVA SCOTIA, Mark Pertus

ONTARIO, Bala Tharmabala

SASKATCHEWAN, Howard Yea

GOLDEN GATE BRIDGE, Kary H. Witt

N.J. TURNPIKE AUTHORITY, Richard J. Raczynski

N.Y. STATE BRIDGE AUTHORITY, William J. Moreau

PENN. TURNPIKE COMMISSION, Gary L. Graham

**SURFACE DEPLOYMENT AND DISTRIBUTION
COMMAND TRANSPORTATION**

ENGINEERING AGENCY, Robert D. Franz

U.S. ARMY CORPS OF ENGINEERS—

DEPARTMENT OF THE ARMY, Paul C. T. Tan

U.S. COAST GUARD, Nick E. Mpras, Jacob Patnaik

**U.S. DEPARTMENT OF AGRICULTURE—
FOREST SERVICE**, John R. Kattell

FOREWORD

In 2004 and 2005, Hurricanes Ivan and Rita caused significant damage to numerous bridges in the Gulf Coast states. Subsequently, the Federal Highway Administration (FHWA) initiated a pooled fund contract DTHF61-06-T-7006 for the “Development of Guide Specifications for Bridges Vulnerable to Coastal Storms and Handbook of Retrofit Options for Bridges Vulnerable to Coastal Storms.” Sponsored by ten states and the FHWA, the project was contracted to the private firm Modjeski and Masters, Inc., with subconsultants Moffatt and Nichol, Inc., Ocean Engineering Associates, Inc., D’Appolonia, Inc., and Dr. Dennis R. Mertz. Dr. John M. Kulicki was the principal investigator for Modjeski and Masters, Inc.

An AASHTO/FHWA Wave Task Force was established to provide technical guidance and oversight for the contractor. The Task Force was initially chaired by William N. Nickas of the Florida Department of Transportation and later by Gregory R. Perfetti of the North Carolina Department of Transportation. Thomas D. Everett of the FHWA served as Vice Chairman of the Task Force. AASHTO State Bridge Engineer representatives included Mitchell K. Carr of the Mississippi Department of Transportation and Hossein Ghara of the Louisiana Department of Transportation and Development. AASHTO State Hydraulic Engineer representatives included Kevin Flora of the California Department of Transportation, David R. Henderson of the North Carolina Department of Transportation, and Rick Renna of the Florida Department of Transportation. FHWA representatives included project manager Dr. Firas Sheikh Ibrahim, Jerry A. DiMaggio, Shoukry Elnahal, Dr. Kornel Kerenyi, and Joseph Krolak. Coastal and ocean engineering subject matter expert representatives included Dr. Robert A. Dalrymple of Johns Hopkins University, Dr. David L. Kreibel of the U.S. Naval Academy, and Spencer M. Rogers, Jr. of North Carolina Sea Grant.

This document contains specifications for the design of bridges vulnerable to coastal storms that were prepared for the FHWA under the aforementioned pooled fund contract. The specifications are comprehensive and embody new concepts which have not been included in previous design provisions. The methods for calculating wave forces on superstructures presented herein are based primarily on numerical simulation of the state of pressure, velocity, and acceleration within the water as a wave passes under or over bridge cross sections. Physical wave tank tests conducted by the Coastal Engineering Laboratory at the University of Florida were used to develop some coefficients needed for the numerical simulation process, known as the Physics Based Method (PBM), and to verify the results. Bridge failures due to storm surge and wave loading in Gulf Coast states during recent years provided limited field data that was used to further verify these methods. Provisions for current-induced forces include the results of recent research conducted at the Turner-Fairbank Highway Research Center.

GUIDE SPECIFICATIONS FOR BRIDGES VULNERABLE TO COASTAL STORMS

TABLE OF CONTENTS

SECTION 1—SCOPE	1
1.1—GENERAL	1
SECTION 2—DEFINITIONS	3
SECTION 3—NOTATION	7
SECTION 4—GENERAL	9
4.1—GENERAL	9
4.2—CLEARANCE REQUIRED TO AVOID WAVE FORCES ON SUPERSTRUCTURE	9
4.3—FORCE MITIGATION	9
4.4—FORCE ACCOMMODATION	10
4.4.1—General.....	10
4.4.2—Design for the Full Wave Loads	10
4.4.3—Fusing for Partial Loads.....	11
4.4.4—Submersible Superstructures.....	11
SECTION 5—LIMIT STATES, LOAD COMBINATIONS, AND DESIGN FORCES.....	13
5.1—LIMIT STATES AND PERFORMANCE OBJECTIVES FOR BRIDGES, OR SPANS OF BRIDGES, THAT CANNOT BE RAISED ABOVE THE WAVE ZONE.....	13
5.2—LOAD COMBINATIONS	14
5.3—MINIMUM LOADS.....	15
SECTION 6—FORCES ASSOCIATED WITH COASTAL STORMS	17
6.1—HYDRODYNAMIC LOADS AND DESIGN PARAMETERS.....	17
6.1.1—General.....	17
6.1.2—Hydrostatic and Hydrodynamic Forces and Moments on Superstructure.....	18
6.1.2.1—General	18
6.1.2.2—Maximum Quasi-Static Vertical Force and Associated Forces and Moment	21
6.1.2.2.1—Maximum Quasi-Static Vertical Force	21
6.1.2.2.1a—Girder Spans	26
6.1.2.2.1b—Slab Spans	27
6.1.2.2.2—Associated Vertical Slamming Force	27
6.1.2.2.3—Associated Horizontal Quasi-Static Wave Force.....	28
6.1.2.2.4—Associated Moment about the Trailing Edge Due to the Quasi-static and Slamming Forces	30
6.1.2.3—Maximum Horizontal Wave Force and Associated Forces and Moments	31
6.1.2.3.1—Maximum Horizontal Wave Force	31
6.1.2.3.2—Associated Quasi-Static Vertical Force	32
6.1.2.3.3—Associated Vertical Slamming Force	33
6.1.2.3.4—Associated Moment about Trailing Edge	33
6.1.2.4—Current Loads on Superstructure	33
6.1.3—Hydrodynamic Loads on Substructure.....	34
6.1.3.1—General	34
6.1.3.2—Forces on Exposed Piles and Columns	34
6.1.3.3—Forces on Exposed Pier Shafts and Walls.....	35
6.2—LEVELS OF ANALYSIS OF FORCES FROM COASTAL STORMS	37
6.2.1—General.....	37
6.2.2—Level I Analysis of Design Parameters.....	37
6.2.2.1—Required Information.....	38
6.2.2.2—Base Design Wind Velocity.....	38
6.2.2.3—Design Water Level	45
6.2.2.4—Design Wave Parameters.....	46
6.2.2.5—Maximum Water Elevation.....	48
6.2.2.6—Design Current Velocity	48
6.2.3—Level II Analysis of Design Parameters	48
6.2.3.1—General	48
6.2.3.2—Design Wind Velocity	49
6.2.3.3—Design Water Level	49
6.2.3.4—Reduction in Wind Velocity and Water Elevation.....	49
6.2.3.5—Design Wave Parameters.....	51

6.2.3.6—Maximum Water Elevation.....	52
6.2.3.7—Design Current Velocities.....	52
6.2.4—Level III Analysis of Design Parameters	52
6.2.4.1—General.....	52
6.2.4.2—Design Wind Velocity Storm Water Level, Current Velocity, and Wave Parameters.....	52

SECTION 1

SCOPE

1.1—GENERAL

Article 5.1 of these Specifications gives Owners the opportunity to apply coastal loads in either the strength limit state or the extreme limit state, depending on the Owner's assessment of the criticality of the bridge. Evacuation and rescue/recovery of the affected area should be a prime factor when considering a system of bridges serving a coastal area. The effect on the local economy should also be considered.

No effect of anticipated climate change has been accounted for herein. Individual Owners may include this feature depending on their jurisdiction's policy in this regard.

SECTION 2

DEFINITIONS

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

Astronomical Tide—Periodic fluctuations in the elevation of coastal and ocean waters due to gravitational forces from the moon, sun, and planets.

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

Buoyancy—The net vertical component of the pressure forces on a body due to the surrounding fluid. Numerically equal to the weight of the fluid displaced by the 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—The storm used to establish the meteorological/oceanographic (met/ocean) loads for which a structure is designed to withstand. The probability of this event (return interval) is based on the level of risk deemed appropriate for that particular structure and situation.

Design Wave Conditions—The wind wave parameters (heights and periods) generated at the point of interest by the design storm. Since wind waves are irregular and consist of a range of wave heights and periods, the design wave height depends on the type of structure, acceptable level of risk for the design, event duration, and depth and steepness limitations. The design wave height is often expressed as a constant times the significant wave height (which is the average height of the one-third highest waves).

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

Ebb Current—Astronomical tide-generated water flow from land to the sea.

Ebb Tide—Phase of the astronomical tide where water is flowing from the land to the sea.

Fetch Length—For a given point in a water body, the fetch length is the horizontal length from that point to the water's edge in the direction from which the wind is blowing.

Fetch Limited Wave—Wind-generated wave heights and periods depend on wind speed, wind duration, and fetch length as well as water depth and wave steepness. Waves whose parameters are limited by the length of the fetch are called fetch limited waves.

Flood Current—Astronomical tide generated water flow from sea to the land.

Flood Tide—Phase of the astronomical tide where water is flowing from the sea to the land.

Highest Astronomical Tide (HAT)—The highest level of water that can be predicted to occur under any combination of astronomical conditions. This level may not be reached every year.

Hindcasting—The process of simulating an event after the fact. Hurricane hindcasting starts with measured and interpolated/extrapolated atmospheric pressure fields within and throughout the duration of the storm. This information is used as input to computer models to recreate the wind velocity, water level, and water velocity at the nodal points of the model meshes at each time step throughout the storm.

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 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 75 mph or 65 knots 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 (Random) Waves—Wind-generated waves are composed of waves with a range of heights and frequencies (periods) and are referred to as irregular or random waves.

Joint Probability—The probability of two (or more) events occurring together.

Joint Return Period—Average period of time between occurrences of a given joint probability event.

Local Sediment Scour (Scour)—When water flows around a structure located in or near an erodible sediment bottom, sediment is removed from the vicinity of the structure. This process is known as local sediment scour and is sometimes referred to simply as “scour.”

Local Wind Set-Down—Decrease in water elevation on the upwind portion of an enclosed or partially enclosed water body due to local wind-induced shear stress on the water surface.

Local Wind Set-Up—Increase in water elevation on the downwind portion of an enclosed or partially enclosed water body due to local wind induced shear stress on the water surface.

Mean High Water (MHW)—The average height of the high waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All high water heights are included in the average where the type of tide is either semidiurnal or mixed. Only the higher high water heights are included in the average where the type of tide is diurnal. So determined, MHW in the latter case is the same as mean higher high water.

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 Low Water (MLW)—The average height of the low waters over a 19-year period. For shorter periods of observation, corrections are applied to eliminate known variations and reduce the results to the equivalent of a mean 19-year value. All low water heights are included in the average where the type of tide is either semidiurnal or mixed. Only lower low water heights are included in the average where the type of tide is diurnal. So determined, MLW in the latter case is the same as mean lower low water.

Mean Lower Low Water (MLLW)—The average height of the lower low 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 MHW and MLW. MTL is not necessarily equal to MSL.

Monochromatic Waves—Waves that have a single frequency (period).

Numerical Modeling—Refers to analysis of coastal processes using computational models.

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 speed at which waves travel (wave celerity) decreases with reduced water depth (once the depth is less than about one half the length of the wave). The direction of a wave, propagating at angles other than 90 degrees to the depth contours, is altered by the depth (once the depth becomes less than about one half the length of the wave). This altering of wave direction is known as wave refraction.

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 (of Water Waves)—When waves propagate into shallow water, their length and celerity decrease and their height increases. This process is called shoaling.

Significant Wave Height—The average height of the one-third highest waves in a (~20-minute) wave record.

Significant Wave Period—The average period of the one-third highest waves in a (~20-minute) wave record.

Sounding—A measured depth of water. On hydrographic charts, the soundings are adjusted to a specific plane of reference.

Storm Surge—As used in this document, storm surge refers to the increase in water elevation due to the far-field storm-induced wind shear stress, reduced central atmospheric pressure, wave set-up, and shoaling. It specifically does not include local wind set-up.

Swell—The longer waves generated in a distant storm propagate beyond the bounds of the wind event that generated the waves. These waves have a narrow range of periods and, prior to reaching very shallow water and breaking, are more sinusoidal in shape.

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.”

Water Depth—Distance between the seabed and the still-water level.

Water Level—Elevation of still-water level relative to some datum.

Water Particle Velocity—The velocity of a small volume of water in a water body.

Water Wave—Oscillations in the water’s surface.

Wave Celerity (Wave Speed)—The speed at which a wave propagates (travels).

Wave Crest—The highest part of a wave.

Wave Direction—The direction from which a wave propagates (travels).

Wave Frequency—The inverse of wave period, i.e., one divided by the wave period or one divided by the time required for one complete wave to travel past a fixed point.

Wave Height—The vertical distance from the trough of a wave to the next wave crest.

Wave Length—The horizontal distance between similar points on two successive waves measured perpendicular to the crest.

Wave Overtopping—When a wave crest or wave run-up elevation exceeds a structure’s elevation, and the wave passes over the structure.

Wave Peak Period—The period of the waves at the highest point on a wave energy density versus wave period plot. The period of the waves with the most energy.

Wave Period—The time required for one complete wave to travel past a fixed point.

Wave Set-Up—Superelevation of the water surface over normal surge elevation due to onshore mass transport of the water by wave action alone.

Wave Steepness—The height of a wave divided by its length.

Wave Transformation—Change in wave energy due to the action of physical processes.

Wave Trough—The lowest part of a wave.

Weibull Distribution—A model probability distribution, commonly used in wave analysis.

Wind Direction—The direction from which the wind is blowing.

Wind Waves—Water waves created by wind-induced stress on the water surface.

SECTION 3

NOTATION

A	=	projected area per unit length of superstructure subjected to current at the storm water level (ft ² /ft); projected area per unit length (for a circular pile of diameter D , $A = D$; ft ² /ft) (6.1.2.4) (6.1.3.2)
a_0 – a_8	=	coefficients specified in Table 6.1.2.2.3-1 (6.1.2.2.3)
a_m , b_m & c_m	=	coefficients (dim) (6.1.2.2.4)
b_0 – b_6	=	coefficients (ft) (6.1.2.2.1a) (6.1.2.2.1b)
C_d	=	drag coefficient taken as 2.5; Morison drag coefficient (dim) (6.1.2.4) (6.1.3.2)
C_m	=	Morison inertia coefficient (dim) (6.1.3.2)
D	=	diameter of the cylinder (ft) (C6.1.3.2)
DC	=	dead load of structural components and nonstructural attachments (5.2)
DD	=	downdrag (5.2)
DW	=	dead load of wearing surfaces and utilities (5.2)
d	=	average water depth over the fetch including surge, astronomical tide, and local wind set-up (ft) (6.2.2.4)
d^*	=	average water depth over the fetch including storm surge and astronomical tide (ft) (6.2.2.3)
d_b	=	girder height plus slab thickness for girder bridges; slab thickness plus deck thickness for slab bridges (ft); combined slab thickness plus deck thickness (ft) (6.1.2.2.1) (6.1.2.2.1b) (6.1.2.2.3) (6.1.2.2.4) (6.1.2.3.1) (6.1.2.3.2) (6.1.2.3.4)
d_g	=	girder height for girder bridges (ft) (6.1.2.2.1a)
d_s	=	water depth at or near the bridge including surge, astronomical tide, and local wind set-up (ft) (6.1.2.2.1) (6.1.3.3) (6.2.2.4) (6.2.2.5)
du/dt	=	horizontal acceleration of water particles for $H_{\max}$ (ft/sec ²) (6.1.3.2)
EL	=	accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning (5.2)
F	=	fetch length in the direction of the wind from the upwind shore (ft) (6.2.2.3) (6.2.2.4)
F_a	=	minimum factored design uplift force (kips) (5.3)
F_{HC}	=	horizontal force resulting from current (kips/ft per unit length of span) (6.1.2.4)
F_{H-AV}	=	horizontal force associated with F_{V-MAX} (kips/ft per unit length of span) (6.1.2.1); associated horizontal quasi-static force specified in Article 6.1.2.2.3 (kip/ft) (6.1.2.2.4) (6.1.2.3.4)
F_{H-MAX}	=	horizontal quasi-static hydrostatic and hydrodynamic force (kips/ft per unit length of span) (6.1.2.1)
F_s	=	vertical slamming force specified in Article 6.1.2.2.2 (kip/ft) (6.1.2.2.4) (6.1.2.3.4)
F_{V-AH}	=	vertical force associated with F_{H-MAX} (kips/ft per unit length of span) (6.1.2.1)
F_{V-MAX}	=	vertical quasi-static hydrostatic and hydrodynamic force (kips/ft per unit length of span) (6.1.2.1)
g	=	gravitational constant (ft/sec ²) (6.2.2.4)
$H_{\max}$	=	maximum probable wave height, which may be determined as specified in Article 6.2.2.4 for a Level I analysis, and by storm modeling for Levels II and III (ft) (6.1.2.1) (6.1.2.2.1) (6.1.2.2.2) (6.1.2.2.3) (6.1.2.2.4) (6.1.2.3.1) (6.1.2.3.2) (6.1.3.3)
H_{mo}	=	an alternative symbol for the significant wave height when that is determined as four times the standard deviation of the sea surface elevation (ft) (C6.2.2.4)
H_s	=	significant wave height determined as the average of the one-third highest of all waves (ft) (6.2.2.4)
k	=	unitless parameter (6.2.2.3)
MWE	=	design wave crest elevation (ft) (6.2.2.5)
M_{T-AH}	=	overturning moment associated with F_{H-MAX} (kips/ft per unit length of span) (6.1.2.1)
M_{T-AV}	=	overturning moment associated with F_{V-MAX} (kips/ft per unit length of span) (6.1.2.1)
n	=	1.3 (dim) (6.2.2.3)
p_1	=	pressure at storm water level (kip/ft ²) (6.1.3.3)
p_2	=	pressure at mudline (kip/ft ²) (6.1.3.3)
r	=	rail height as shown in Figure 6.1.2.2.1-1 (ft) (6.1.2.2.3) (6.1.2.2.4) (6.1.2.3.1) (6.1.2.3.4)
S	=	local wind set-up measured from the storm still-water level (ft) (6.2.2.3)
TAF	=	entrapped air factor specified in Articles 6.1.2.2.1a and 6.1.2.2.1b; a factor to adjust the vertical quasi-static force for variable amounts of entrapped air (dim) (6.1.2.2.1) (6.1.2.2.1a) (6.1.2.2.1b) (6.1.2.3.2)
T_{app}	=	apparent wave period (accounting for design current) (sec) (C6.1.3.2)
T_p	=	period of the waves with the greatest energy exhibited in a spectrum (sec) (6.1.2.1) (6.1.2.2.1) (6.2.2.4)
t	=	duration of U_i (sec) (6.2.2.4)
U	=	horizontal component of water particle velocity for $H_{\max}$ including current (ft/sec) (6.1.3.2)
U_c	=	current velocity taken as specified in Article 6.2.2.6 (ft/sec) (6.1.2.4)

U_t	=	wind speed at the standard elevation for a given duration, t (ft/sec); 100-year design wind velocity at the standard 32.8-ft elevation modified for duration t as specified in Article 6.2.2.2 (ft/sec) (6.2.2.2) (6.2.2.4)
U_t^*	=	wind-stress factor (ft/sec) (6.2.2.4)
$\overline{U}_t$	=	wind speed given for Elevation Z , which is different from the standard elevation (ft/sec) (6.2.2.2)
U_{mo}	=	maximum wave-induced orbital velocity (ft/sec) (C6.1.3.2)
$U_{3 \text{ sec}}$	=	design wind velocity at the standard 32.8-ft elevation averaged for a duration of 3 seconds (ft/s) (6.2.2.3)
$U_{10 \text{ min}}$	=	design wind velocity at the standard 32.2-ft elevation and averaged for a duration of 10 minutes (ft/s) (6.2.2.3)
V	=	displaced volume per unit length (for a circular pile of diameter D , $V = \pi D^2/4 \text{ ft}^3/\text{ft}$) (6.1.3.2)
W	=	total bridge width (ft) (6.1.2.2.1a) (6.1.2.2.3) (6.1.2.2.4) (6.1.2.3.1) (6.1.2.3.2) (6.1.2.3.4)
W'	=	horizontal projection of overhang (ft) (6.1.2.2.4)
W_{dir}	=	angle from the upwind normal to the shoreline to the direction of attack of a hurricane, measured clockwise as shown in Figure C6.2.3.4-1 (deg) (6.2.3.4)
W^*	=	effective bridge width (ft) (6.1.2.2.4)
WA	=	wave forces F_V , F_S , F_H , and M_T as specified in Articles 6.1.2 and 6.1.3 (5.2)
Z	=	the elevation, other than the standard elevation, for which wind speeds are given (ft) (6.2.2.2)
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) (5.3) (6.1.2.2.1) (6.1.2.2.1a) (6.1.2.2.2) (6.1.2.2.3) (6.1.2.3.1) (6.1.2.3.2) (6.1.2.3.4)
α_1	=	coefficient (6.1.3.3)
γ_p	=	minimum load factors for dead loads, as specified in Article 3.4.1 of the <i>AASHTO LRFD Bridge Design Specifications</i> (AASHTO 2007) (5.2)
γ_w	=	unit weight of water taken as 0.064 (kip/ft ³) (6.1.2.2.1) (6.1.2.2.2) (6.1.2.2.3) (6.1.2.3.1) (6.1.2.3.2) (6.1.3.3) (6.2.2.3)
γ_{wave}	=	load factors on wave forces (5.2)
η^*	=	potential height above the storm water level to which wave pressure could be exerted (ft) (6.1.3.3)
η_{max}	=	distance from the storm water level to design wave crest (ft); wave crest height for the 100-year event (ft) (5.3) (6.1.2.2.1) (6.1.2.2.1a) (6.1.2.2.2) (6.1.2.2.3) (6.1.2.2.4) (6.1.2.3.1) (6.1.2.3.2) (6.1.2.3.4) (6.2.2.5)
θ	=	angle between direction of wave approach and a line normal to the structure (deg) (6.1.3.3)
λ	=	wave length (ft) (6.1.2.1) (6.1.2.2.1) (6.1.2.2.1a) (6.1.2.2.2) (6.1.2.2.3) (6.1.2.2.4) (6.1.2.3.1) (6.1.2.3.2) (6.1.3.3)
ρ_w	=	mass density of water taken as 2.0 slugs/ft ³ (6.1.2.4) (6.1.3.2) (6.2.2.3)
τ_{wx}	=	wind-shear stress on water surface (kip/ft ²) (6.2.2.3)
ω	=	an adjusted bridge width (ft) (6.1.2.3.1)
%AIR	=	amount of air trapped in the cells of a bridge superstructure determined as the percent of the cross-section with due regard to the relative crest elevation and any physical characteristics of diaphragms that limit the amount of air that can be entrapped (%) (6.1.2.2.1a)

SECTION 4

GENERAL

4.1—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.

4.2—CLEARANCE REQUIRED TO AVOID WAVE FORCES ON SUPERSTRUCTURE

Wherever practical, the vertical clearance of highway bridges should be sufficient to provide at least 1 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 4.3.

4.3—FORCE MITIGATION

Where it is not possible to provide the vertical clearance recommended in Article 4.2, 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 and adjusting the parameter r in Articles 6.1.2.2.3, 6.1.2.2.4, 6.1.2.3.1 and 6.1.2.3.4
- Venting the cells that could potentially entrap air, resulting in increased buoyancy forces and adjusting the parameter TAF (trapped air factor) in Article 6.1.2.2.1 to account for the venting
- 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 and adjusting the parameter TAF in Article 6.1.2.2.1
- Using continuous superstructures to increase the reactive force of individual spans
- Using solid or voided slab bridges to reduce buoyancy forces

C4.1

Further discussion of the credentials recommended for a coastal engineer and a list of some conditions that require more extensive involvement of a coastal engineer are provided in Article C6.1.1.

C4.2

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. 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.

C4.3

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, approximately 14,000 ft of parapet were broken off during Hurricane Katrina (Volkert, 2005). 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 I-10 Lake Pontchartrain Bridge and the I-10 Escambia Bay 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. Deck 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.

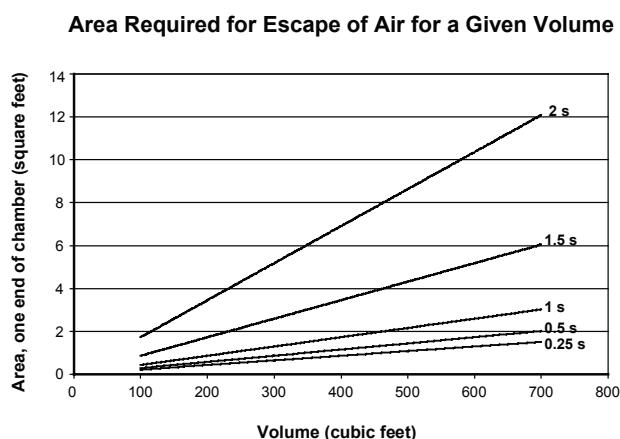


Figure C4.3-1—Venting Requirements

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 is increased through continuous spans.

The use of slab bridges may be useful to lessen superstructure depth and raise the bottom of the superstructure of some spans sufficiently to avoid wave forces. This may be particularly applicable to spans near the ends of bridges that have grade constraints.

4.4—FORCE ACCOMMODATION

4.4.1—General

Design for coastal storms may be based on any of the strategies identified herein. Design and detailing should achieve an engineered response that involves 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 either to carry the load or to protect the substructure so that it can be reused if the superstructure is lost.

Where partial or complete force accommodation is provided, the design shall consider the possibility that there may be significant upward forces due to hydrodynamic and hydrostatic (buoyancy) effects, which may cause a reversal of the normal moments and shears.

4.4.2—Design for the Full Wave Loads

The structure may be designed to resist the loads calculated in accordance with the provisions specified in Section 5 and Articles 6.1.2 and 6.1.3.

C4.4.1

In recent cases where the superstructure was lost but the substructure remained largely reusable, 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 minimize damage to the substructure is recommended herein.

4.4.3—Fusing for Partial Loads

Where design for the full wave loads specified in Article 6.1.2 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. Where this strategy is used, the design and details shall ensure that separation occurs only when the forces associated with the chosen design event are exceeded.

4.4.4—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.

C4.4.3

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 the substructure.

C4.4.4

Submersible structures may have application in low-level approach structures similar to situations where sacrificial superstructures are also applicable. Actual wave forces will tend to be smaller once the structure is totally submerged in the water, so storms with a lower design water level that do not submerge the structure should also be investigated. Submersible heavy structures with small volumes of voids, which reduce the buoyancy, may be a cost-effective solution in some cases.

Where the maximum storm surge submerges the structure, the case in which the surge reaches the bottom of the structure should also be investigated. This will require calculation of the associated wave height using the equations specified in Article 6.2.2 using the same design wind speed as that which created the submergence.