Guide Specifications and Commentary for
Vessel Collision Design of Highway Bridges

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2008–2009

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FOREWORD

The 1980 collapse of the Sunshine Skyway Bridge was a major turning point in awareness and increased concern about vessel collision and the safety of bridges crossing navigable waterways in the United States. Studies initiated as a result of this tragedy led to the 1988 pooled-fund research project sponsored by 11 states and the Federal Highway Administration (FHWA) which developed a proposed design code for use by bridge engineers in evaluating structures for vessel collision. This effort culminated in 1991 with the adoption by the American Association of State Highway and Transportation Officials (AASHTO) of the Guide Specification and Commentary for Vessel Collision Design of Highway Bridges (AASHTO, 1991).

The 1991 AASHTO Guide Specification established design provisions for bridges crossing navigable waterways to minimize their susceptibility to damage from vessel collisions. The provisions applied to both new bridges and to the analysis of existing bridges to determine vulnerability and potential retrofit. The intent of the AASHTO provisions is to provide bridge components with a “reasonable” resistance capacity against ship and barge collisions. In navigable waterway areas where collision by merchant vessels may be anticipated, the Guide Specification requires that bridge structures be designed to prevent collapse of the superstructure by considering the size and type of vessel fleet navigating the channel, available water depth, vessel speed, structure response, the risk of collision, and the operational classification of the bridge.

This Second Edition of the Guide Specification was developed to incorporate lessons learned from the use of the original 1991 Vessel Collision Guide Specification; incorporate the current LRFD Bridge Design methodology; clarify some of the risk procedure elements; make minor modifications and corrections; and discuss, and incorporate where deemed necessary, results from barge and ship collision research conducted since the original vessel collision publication. The use of the Guide Specification procedures to evaluate existing bridges has been highlighted in this revised edition, and a new worked example illustrating the vessel collision risk assessment procedures has been provided.

Compared to more mature and established fields such as wind and earthquake engineering, vessel collision design is in its infancy stages. Although there are a number of important research needs within the discipline, the key areas of ship impact forces; barge impact forces; risk acceptance criteria; physical protection systems; and aids-to-navigation improvements should be highlighted as areas of future research.

This Second Edition was prepared by the consulting firm of Moffatt and Nichol. The principal author was Michael A. Knott, P.E. (who was also the principal author of the original 1991 Guide Specification). Moffatt and Nichol provided their services under contract to HDR Engineering on behalf of the Federal Highway Administration (FHWA).
1.1 PURPOSE

In navigable waterway areas, where vessel collision by merchant ships and barges may be anticipated, bridge structures shall be designed to prevent collapse of the superstructure by considering the size and type of the vessel, available water depth, vessel speed, and structure response. The requirements apply to all bridge types which cross a navigable shallow draft inland waterway or canal with barge traffic, and deep draft waterways with large merchant ships. The provisions are for normal merchant steel-hulled vessels (ships and barges) and are not applicable for waterways whose maritime traffic consists of recreational or other special vessels constructed of wood or fiberglass.

The intent of the vessel collision requirements is to establish analysis and design provisions to minimize bridge susceptibility to catastrophic collapse. The purpose of the provisions is to provide predictable design vessel collision effects in order to proportion bridge components with a reasonable resistance to collapse. The provisions apply to bridges crossing navigable waterways which carry waterborne commerce as established by federal and state agencies. Judgment should be used when applying the criteria to waterways in which no defined navigation channel exists and no commercial maritime traffic can be reasonably anticipated.

Bridges over a navigable waterway meeting the criteria above, whether existing or under design, should be evaluated as to vulnerability to vessel collision in order to determine prudent protective measures. The recommendations listed below summarize the essential elements which should be addressed in developing a program for evaluating bridges and providing pier protection for vessel collision.

1.1.1 Interdisciplinary Team

Vessel collision evaluations of new and existing bridges should be conducted by an interdisciplinary team comprised of structural, geotechnical, and hydraulic engineers. In special cases where benefit/cost analysis of risk reduction is required, an economic specialist should also be part of the team. Representatives and coordination with the U.S. Coast Guard, the Army Corps of Engineers, and other federal and state agencies as appropriate for the bridge location should also be included in the interdisciplinary evaluation.

1.1.2 New Bridges

Vessel collision evaluations of new bridges over navigable waterways should be conducted in accordance with this Guide Specification.
The AASHTO Guide Specifications contain three alternative analysis methods for determining the design vessel for each bridge component in the structure in accordance with two-tiered risk acceptance criteria. Method I is a simple to use semi-deterministic procedure; Method II is a detailed risk analysis procedure; and Method III is a cost-effectiveness of risk reduction procedure (based on a classical benefit/cost analysis). The Guide Specifications require the use of Method II risk analysis for all bridges unless special circumstances exist as described in the code for the use of Methods I and III. Special circumstances for using Method I include shallow draft waterways where the marine traffic consists almost exclusively of barges, and for using Method III include very wide waterways with many piers exposed to collision, as well as existing bridges to be retrofitted.

1.1.3 Existing Bridges

Unless an existing bridge was designed in accordance with the previous 1991 edition of the AASHTO Vessel Collision Specifications, all remaining existing bridges over navigable waterways with commercial barge and ship traffic should be evaluated using a vulnerability assessment in accordance with the Method II risk analysis procedures contained in this current guide specifications. The vulnerability assessments would meet NTSB recommendations to AASHTO, FHWA and other federal agencies for improved bridge safety based on previous vessel collision accidents involving bridge failures.

Based on the vulnerability assessment evaluations of existing bridges within the state system, a screening process based on the estimated annual frequency of collapse can be used to identify and rank high risk bridges, and to prioritize vulnerable structures for potential rehabilitation, retrofit, pier protection countermeasures, or replacement.

AASHTO recognizes the potential that a significant portion of older bridges crossing navigable waterways in the Nation may not meet the risk acceptance criteria for new bridges contained in the AASHTO Specifications adopted since 1991. The intent of performing vessel collision vulnerability assessments on the existing bridge system is to identify those structures that are particularly vulnerable to catastrophic collapse. The vessel collision vulnerability information would provide a framework for States to be aware of high-risk safety needs requiring immediate or short-term action, as well as information to prioritize and budget for the long-term needs for bridge rehabilitation or replacement. The risk assessment of the existing bridges will be used as a part of the prioritization process and allocation of federal funds.

AASHTO recognizes that the cost of retrofitting the potentially large number of existing bridges over navigable waterways to meet the risk acceptance criteria for new bridges may not be realistic based on current
1.2 BACKGROUND

Ship and barge collisions with bridges that are located in coastal areas and along inland waterways represent a growing and serious threat to public safety, port operations, motorist traffic patterns, and environmental protection in many cities throughout the world. In the 42-year period from 1960 to 2002, there have been 31 major bridge collapses worldwide due to ship or barge collision, with a total loss of life of 342 people.

Seventeen of the bridge catastrophes discussed above occurred in the United States, including the 1980 collapse of the Sunshine Skyway Bridge crossing in Tampa Bay, Florida, in which 1,300 feet of the main span collapsed and 35 lives were lost as a result of the collision by an empty 35,000-DWT (deadweight tonnage) bulk carrier.

Recent bridge collapses in the United States include the Queen Isabella Bridge connecting San Padre Island to the Texas mainland, which was hit by a barge in September 2001 (8 fatalities); and the collapse of the I-40 Bridge over the Arkansas River near Webber Falls, Oklahoma, which was hit by a barge in May 2002 (13 fatalities).

It should be noted that there are numerous vessel collision accidents with bridges which cause damage that varies from very minor to significant damage, but do not necessarily result in collapse of the structure or loss of life. A recent U.S. Coast Guard study (May 2003) of towing vessels and barge collisions with bridges located on the U.S. inland waterway system during the 10-year period from 1992 to 2001 revealed that there were 2,692 accidents with bridges. Only 61 of those caused bridge damage in excess of $500,000 (1,702 caused very minor damage with no repair costs to the bridge), and there were no fatalities within the study period. The study concluded that 90 percent of the barge accidents were related to human performance (78 percent to pilot error and 12 percent to other operational factors). Only 5 percent were related to mechanical problems, and for the remaining 5 percent there was insufficient information to assign a cause.

C1.2

Many factors account for the present ship/bridge accident problem confronting many countries around the world. One factor is that a larger number of merchant ships are making more frequent transits past more bridges. Since 1960, the number of bridges across major waterways leading to U.S. coastal ports has increased by one-third. During that same period, the number of vessels in the world fleet has increased three-fold and worldwide seaborne tonnage has increased by more than 255 percent (McDonald, 1983; U.S. Department of Commerce, 1978).

Other factors include poorly sited bridges. Inadequate attention is often given to the bridge's relationship with waterborne traffic with the result that bridges are placed too near tricky bends or turns in the navigation channel, or too near waterfront docks where berthing maneuvers could threaten the bridge. Many bridges today have inadequate spans over the navigation channel for the safe transit of modern ships which regularly exceed 800 feet in length and 100 feet in width. These narrow spans leave little room for error on behalf of the merchant vessel—particularly under adverse wind and hydraulic current conditions. These small spans often result from economic pressure on behalf of the bridge owner and designer to minimize the in-place cost of the substructure and superstructure of the bridge without regard to the potential for ship impact against the structure.

Economic pressures have long been recognized as conflicting with safety. This is true of both the bridge industry and the maritime industry. In the latter, safety concerns are often placed second to the maintenance of ship schedule—with predictably disastrous consequences. Since masters and pilots are often rated on their ability to make schedules, they are sometimes very reluctant to abort transits into harbors even during adverse environmental conditions. This may have been one of the factors involved in the Skyway Bridge accident, where the pilot on-board the empty inbound merchant ship attempted to transit under the bridge during very low visibility, dense rainfall, and high wind conditions. The vessel struck an anchor pier of the bridge located approximately 800 feet from the centerline of the channel.

A comprehensive literature review of the current domestic and foreign practice, experience, and research
1.2.1 AASHTO Guide Specification (1991)

The 1980 collapse of the Sunshine Skyway Bridge was a major turning point in awareness and increased concern for the safety of bridges crossing navigable waterways in the United States. Studies initiated as a result of this tragedy led to the 1988 pooled-fund research project sponsored by 11 states and the Federal Highway Administration (FHWA) which developed a proposed design code for use by bridge engineers in evaluating structures for vessel collision. This effort culminated in 1991 with the adoption by the American Association of State Highway and Transportation Officials (AASHTO) of the Guide Specification and Commentary for Vessel Collision Design of Highway Bridges (AASHTO, 1991).

The AASHTO Guide Specification established design provisions for bridges crossing navigable waterways to minimize their susceptibility to damage from vessel collisions. The provisions applied to both new bridges and to the analysis of existing bridges to determine vulnerability and potential retrofit. The intent of the AASHTO provisions was to provide bridge components with a “reasonable” resistance capacity against ship and barge collisions. In navigable waterway areas where collision by merchant vessels may be anticipated, the Guide Specifications require that bridge structures be designed to prevent collapse of the superstructure by considering the size and type of vessel fleet navigating the channel, available water depth, vessel speed, structure response, the risk of collision, and the operational classification of the bridge.

It should be noted that damage to the bridge (even failure of secondary structural members) is permitted by the code as long as the bridge deck carrying motorists and traffic doesn’t collapse (i.e., sufficient redundancy and alternate load paths exist in the remaining structure to prevent collapse of the superstructure).

When the original 1991 Guide Specification was developed in the late 1980s, most analysis was done by hand calculation; therefore, the specification provisions included some simplifying requirements to minimize the hand analysis effort. With modern personal computers and software programs, the vessel collision risk analysis procedures can be easily programmed. Therefore, some of those earlier simplifications have been removed in this 2008 Edition of the Guide Specifications.

C1.2.1

Since its adoption by AASHTO in 1991, the Guide Specification has been used to design numerous new bridges and to evaluate existing structures for their susceptibility to vessel collision. Because the code was published as a “guide specification,” its use by the State Departments of Transportation (DOTs) was optional, not mandatory.

In general, the use of the code was well received in the engineering community. The major drawbacks in the early implementation of the specifications involved lack of experience in collecting the large amount of vessel fleet data needed to perform the risk analysis for each bridge, as well as a general unfamiliarity of most bridge designers (and bridge owners) in directly using risk concepts in structural design. Historically in the United States, the risk of structural collapse and potential loss of life have been (and to a great extent still are) buried in various “safety factors,” “reliability indexes,” etc., used in structural design equations within the design codes. Similar to most countries, the United States has a great amount of difficulty in dealing directly with engineering risks in a public environment (and this is reflected in our design codes). Defining an acceptable level of risk is a value-oriented process and is by nature subjective. This subjectiveness and the wide range of public opinion concerning risk acceptance levels results in an engineering issue that most bridge designers would rather not address.

The vessel collision code is somewhat unique in the United States in that the acceptable risk of collapse is clearly stated by the Guide Specifications, and risk analysis procedures are directly used to design the structure.

Experience to date has shown that the use of the vessel impact and bridge protection requirements of the AASHTO Guide Specifications for planning and design of new bridges has resulted in a significant change in proposed structure types over navigable waterways. Incorporation of the risk of vessel collision and cost of protection in the total bridge cost has almost always resulted in longer span bridges being more economical than traditional shorter span structures. This is a consequence of bridge designs involving longer spans requiring fewer piers, and therefore fewer pier protection systems, thus producing lower total (bridge plus protection system) costs.

Experience has also shown that it is less expensive to include the cost of protection in the planning stages of
a proposed bridge than to add it after the basic span configuration has been established without considering vessel collision concerns. Typical costs for adding protection, or for retrofitting an existing bridge for vessel collision, have ranged from 25 percent to over 100 percent of the existing bridge costs.

C1.2.1.1 Extreme Event Combinations (Scour)

The 1991 AASHTO Guide Specification recommended a load combination of vessel impact plus dead load for bridge design under ultimate (survivability) conditions. It was not anticipated that scour (or other extreme events) would occur simultaneously with vessel collisions.

It should be noted that the magnitudes and consequences of individual extreme events such as ship and barge collisions; scour due to flooding; earthquakes; ice flows; hurricane-driven storm surge and waves; terrorist attacks, etc., usually govern the design process for new highway bridges. If the simultaneous occurrence of two or more of these events is considered (for example, a ship collision or earthquake occurring on a bridge pier whose foundation had been subjected to scour during a flood event), the combination of these separate extreme events will generally result in a dominating load combination with significant cost consequences.

Design based on superposition of extreme loads (as currently advocated by some engineers and government agencies) can lead to a significant design increase costing millions of dollars on each project. Since a simultaneous occurrence of two or more extreme events with maximum magnitudes is unlikely, a rational design approach must be formulated for use by bridge engineers. Toward this end, the FHWA sponsored a conference in Atlanta, Georgia, on December 1996, entitled the "Design of Bridges for Extreme Events." The conference proceedings contain a collection of papers dealing with vessel collision, scour, and earthquake design for highway bridges (FHWA, 1996). Concerning the possible combination of vessel collision and scour, Nowak and Knott recommended an evaluation of the following two load cases (Nowak and Knott, 1996):

1. Minimum impact loads associated with a drifting empty barge breaking loose from its moorings and hitting a bridge (potentially during storm and high-water conditions). The drifting barge impact loads should be combined with one half of the predicted long-term plus one half of the predicted short-term scour. For this load case, long-term scour should be taken as the sum of the contraction scour portion of live bed scour and scour due to long-term channel degradation. Short-term scour should be taken as the short-term portion of the live bed scour associated with the 100-year storm/flood event.

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2. Maximum impact loads associated with a ship or barge tow striking the bridge while transiting the navigation channel under typical waterway conditions (i.e., not during extreme storm events and high-water conditions). The vessel impact loads should be combined with one half of the predicted long-term scour.

Short-term scour includes contraction, local and live bed scour in which river or bay bottom material (sand, clay, gravel, etc.) is removed as a result of increased water velocities caused by flooding conditions in conjunction with the overall bridge geometry and substructure shape on the hydraulic conductivity of the site.

In the United States, historical data indicates that merchant ships and barge tows will not transit river and harbor areas during periods of high water and flood events which cause abnormal and dangerous water currents in the navigation channel. During such flood events, vessels will normally leave the harbor, tie-up at docks, or anchor in designated areas of the waterway. Following the passage of the flood stages of the waterway, and once currents return to normal levels, merchant shipping will recommence in the waterway. It is anticipated that the short-term (live bed) scour areas near the bridge piers will have been significantly refilled by sediment transport mechanisms in the waterway by that time. Note that no records of scour concerns are reported on any of the 31 major bridge collapses mentioned at the beginning of this report.

At limited locations in the United States, live bed scour conditions do not exist and instead, clearwater scour conditions may exist. In clearwater scour situations, up-river site conditions are such that there is virtually no particulate matter (soil, gravel, etc.) to transport; therefore, river bed material removed by local contraction scour is not replaced after flood-level water velocities subside. Under this special condition, the full depth of scour should be used in the vessel collision analysis.

Long-term scour includes aggradation and degradation scour and refers to scour across the entire waterway width. This is a permanent site condition with a magnitude (depth) that increases with time, and is independent of the presence of a bridge or the structure’s geometry—this scour will occur regardless of the bridge. Long-term scour (if it is present at all) is usually a gradual deterioration of base support across the waterway.

1.2.2 AASHTO LRFD Bridge Design Specifications


C1.2.2

In the LRFD Code, design values of factored load combinations were determined using rigorous statistical analysis procedures and were based on a target beta reliability index of $\beta = 3.5$. However, the statistical analysis was performed only for the basic load
collision requirements are now mandatory for users of the LRFD Bridge Design Code.

The vessel collision force in the LRFD Code (designated as CV) is considered an "Extreme Event II" load combination, in which a load factor of 1.0 is used for the vessel collision force in combination with the dead load, 50 percent of the live load, water loads and stream pressure, earth pressure, and friction (no other extreme events are combined with the vessel collision force).

1.3 BASIC CONCEPTS

Development of the Guide Specifications has been predicated on the following basic concepts:

- hazard to life be minimized,
- risk of bridge service interruption to be minimized,
- importance of bridge to be reflected in required safety level,
- specifications to accept damage of secondary structural members provided bridge service can be maintained,
- specifications to be simple and unambiguous,
- ingenuity of design not to be restricted, and
- provision to be applicable to all of the United States.

1.4 DESIGN ANALYSIS

When the specifications provide for an empirical formula as a design convenience, a rational analysis based on a theory accepted by the Subcommittee on Bridges and Structures of the American Association of State Highway and Transportation Officials, with stresses in accordance with the specifications, or by model testing supported by analysis, will be considered combinations with dead load and live load. Extreme loads and their combinations were not considered in the LRFD calibration because of the lack of statistical data concerning the correlation of such extreme events (vessel collision, scour, earthquake, etc.). Therefore, the development of rational design criteria for extreme load events will require future research and the collection of extensive statistical data. Because of the rare nature and large variability of magnitudes associated with extreme events, some researchers believe that the current bridge design methods, statistical analysis models, and calibration procedures used in the development of LRFD load combinations are inappropriate for application to extreme event design.

The vessel collision force in the LRFD Code (designated as CV) is considered an "Extreme Event II" load combination, in which a load factor of 1.0 is used for the vessel collision force in combination with the dead load, 50 percent of the live load, water loads and stream pressure, earth pressure, and friction (no other extreme events are combined with the vessel collision force).

C1.3

The basic design philosophy embodied in the Specifications is that it is possible to design a bridge in a cost-effective manner which minimizes the risk of catastrophic superstructure collapse due to vessel collision. Bridges may be designed to resist vessel impact loads in either the elastic or plastic range, or protected by a bridge protection system. In the plastic range, significant damage to the bridge substructure is acceptable providing that superstructure collapse does not occur and that the damage is easily repairable. Structural ductility and redundancy are important in preventing superstructure collapse.

One of the basic concepts in developing the Specifications was that it would be applicable to all parts of the United States with navigable waterways, including the inland waterway system as well as the coastal areas. In order to provide flexibility in specifying design provisions, three alternative methods of selecting the design vessel (ranging from simple to complex) were developed. Two operational classifications were defined to classify bridges according to Social/Survival and Security/Defense requirements.

C1.4

The designer is cautioned that many of the equations in the Specifications for vessel collision analysis were derived from physical model studies and analysis methods in which critical assumptions have been made. Therefore, the implied accuracy of the equations in the Specifications is limited, and the use of the equation results to many significant figures is not warranted.

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1.5 FLOW CHARTS

Flow charts outlining the basic steps in the vessel collision design procedures are given in Figure 1 for evaluating bridges. Method II shall be used for all bridge analysis unless the special situations presented in Article 4.1 exist, in which case Method I or III, as appropriate, may be used.

C1.5

Method II of the AASHTO Guide Specifications is a probability-based risk analysis procedure for determining the appropriate vessel impact design loads for a bridge structure. Using Method II procedures, a mathematical risk model is used to estimate the annual frequency of bridge collapse based on the bridge pier/span geometry, ultimate resistance of the pier (or span), waterway characteristics, and the characteristics of the vessel fleet transiting the channel. The estimated risk of collapse is compared to standard acceptance criteria, and the bridge characteristics (span layout, pier strength, etc.) are adjusted until the acceptance criteria are satisfied. The Method II procedure is iterative in nature and is normally performed by specialized computer programs and spreadsheets.
Figure 1.5-1—Design Procedure Flow Chart

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Figure 1.5-1—Design Procedure Flow Chart—Continued
REFERENCES


The following symbols and definitions apply to these Guide Specifications.

\[ a = \text{bow or vessel damage depth used in Article C3.9 and Figure C3.9-3; acceleration as used in Article C7.3.2 (in./s^2)} \]

\[ \bar{a} = \text{average bow damage depth as used in Equation C3.10-1} \]

\[ a_{SH} = \text{bow damage depth of standard hopper barge as determined by Equation 3.13-1 (ft)} \]

\[ AF = \text{annual frequency of bridge element collapse defined in Article 4.8.3 (number of collapses/year)} \]

\[ a_t = \text{bow damage depth of ship as determined by Equation 3.10-1 (ft)} \]

\[ B = \text{loads resulting from buoyancy forces and used in the group load combination of Equation 3.14-1; beam (width) of vessel as shown in Figure C4.8.3.5-1 (ft)} \]

\[ B_B = \text{barge width as defined in Article 3.13 (ft)} \]

\[ B_M = \text{beam (width) of barge, barge tows, and ship vessels used in Articles 3.5 and 4.8.3.3; as shown in Figures 3.5.1-1, 3.5.1-2, and 4.8.3.3-1; and as used in Tables 3.5.2-1 through 3.5.2-3 (ft)} \]

\[ B_P = \text{width of bridge pier used in Figures 4.8.3.3-1 and 8.5.1-1 (ft)} \]

\[ BR = \text{base rate of vessel aberrancy defined in Article 4.8.3.2 (dimensionless)} \]

\[ C = \text{channel width as shown on Figures 4.2.1-1, 4.2.1-2, and 8.5.1-1 (ft); vessel coefficient as used in Equation C7.3.2-1} \]

\[ C_B = \text{vessel block coefficient (dimensionless) as defined in Equation C3.5.2-1} \]

\[ C_C = \text{size of barge based on cargo capacity as defined in Article 3.5 and used in Figure 3.5.1-1 (tonne, ton)} \]

\[ C_H = \text{hydrodynamic mass coefficient defined in Equation 3.8-1 (dimensionless)} \]

\[ CV = \text{vessel collision force used in the group load combination of Equation 3.14-1} \]

\[ D = \text{loads resulting from dead load and used in the group load combination of Equation 3.14-1; diameter of dolphin as shown in Figure C4.8.3.5-1 (ft)} \]

\[ DA = \text{avoidable disruption cost as shown in Figure C4.8.3.5-1 (}) \]

\[ D_B = \text{bow depth of a ship or barge vessel as shown in Figures 3.5.1-1, 3.5.2-3, 3.5.2-4, 3.15.1-2, and 3.15.1-3, and in Tables 3.5.2-1 through 3.5.2-3 (ft)} \]

\[ DC = \text{bridge collapse disruption cost as defined in Article 4.9.3 and as shown in Figure C4.8.3.5-1 (}) \]

\[ D_C = \text{mean draft of an empty vessel (light draft ) as shown in Figure 3.5.1-1 (ft); mean draft of a ballasted vessel as shown in Figure 3.5.2-4 (ft); effective dolphin diameter as shown in Figure C4.8.3.5-1 (ft)} \]

\[ D_EB = \text{draft of ballasted ship bow as shown in Figure 3.5.2-4 and Tables 3.5.2-1 through 3.5.2-3 (ft)} \]

\[ D_ES = \text{draft of ballasted ship stern as shown in Figure 3.5.2-4 (ft)} \]

\[ DF = \text{distribution factor as used in Equation C7.3.2-4} \]

\[ d_f = \text{depth to fixity in Equation C7.3.3-3 (in.)} \]
$D_L$ = mean draft of a fully loaded vessel as shown in Figures 3.5.1-1 and 3.5.2-4 and in Tables 3.5.2-1 through 3.5.2-3 (ft)

$D_M$ = mean vessel draft as defined in Equation C3.5.2-1 (ft)

$D_P$ = depth of vessel as shown in Figure 3.5.1-1 (ft)

$DW$ = size of vessel based on deadweight tonnage as defined in Articles 3.5 and 3.9 (tonne, ton)

$D_x$ = horizontal stiffness as used in Equation C7.3.2-5 (kip-in.)

$D_y$ = vertical stiffness as used in Equation C7.3.2-6 (kip-in.)

$E$ = modulus of elasticity as defined in Equation C7.3.2-4 (ksi); modulus of elasticity of pile section as defined in Equation C7.3.3-4 (psi); absorbed collision energy as defined in Equation C3.8-3 and as shown in Figure C3.9-3 (kip-ft)

$E_b$ = deformation energy as used in Figure C3.12-2 (kip-ft)

$E_{h}, E_{V}$ = loads resulting from earth pressure and used in the group load combination of Equation 3.14-1

$F$ = vessel crushing force as defined in Equation C7.3.1.1-1 (kips)

$F_{R}$ = friction in load combination as used in Equation 3.14-1

$F(s)$ = protective structure force, as a function of deflection, as used in Equation 7.3.1 (kips)

$g$ = acceleration due to gravity as used in Equation C3.8-1 (ft/s²); real annual rate of growth of disruption costs as used in Equation 4.9.2-1 (rate/year)

$GRT$ = gross registered tonnage as defined in Article C3.5 (ft³)

$H$ = ultimate bridge element strength as defined in Article 4.8.3.4 (kips); height of dophin to location of the plastic hinge as used in Equation C7.3.3-6 and Figure C7.3.3-8 (ft)

$h$ = distance from the top of the cell to the plane of maximum interlock stress as defined in Article C7.3.3, used in Equation C7.3.3-2, and shown in Figure C7.3.3-8 (ft)

$H_C$ = depth of barge head-log on its bow as shown in Figure 3.5.1-1 (ft)

$H_p$ = ultimate bridge pier resistance strength as defined in Article 4.8.3.4 (kips)

$H_S$ = ultimate bridge superstructure resistance strength as defined in Article 4.8.3.4 (kips)

$l$ = moment of inertia of pile section as used in Equation C7.3.3-4 (in.⁴)

$i$ = discount rate used in Equation 4.9.2-1 (rate/year)

$l_p$ = moment of inertia of pile as used in Equation C7.3.2-4 (in.⁴)

$l_w$ = water moment of inertia as defined in Article 7.3.2 (in.⁴)

$K$ = equivalent spring constant of pile and fender as used in Equations C7.3.2-1 and C7.3.2-2 (kip/in.)

$K_a$ = active earth pressure coefficient as used in Equation C7.3.3-2

$KE$ = design impact energy of vessel collision as defined in Equations 3.8-1, C3.8-2, 7.3-1, and C7.3.1.1-1 (kip-ft); kinetic energy as defined in Equation C3.8-1 (kip-ft); ship collision energy as defined in Equations 3.10-1, C3.10-1, and C3.10-2 (kip-ft); kinetic energy to be absorbed as used in Equations C3.8-3 and C7.3.1-2 (kip-ft); barge collision energy as defined in Equations 3.13-1 and C3.13-1b (kip-ft)

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\[ K_f = \text{spring constant of fender as used in Equation C7.3.2-2 (kip/in.)} \]
\[ K_p = \text{spring constant of pile as used in Equation C7.3.2-2 (kip/in.)} \]
\[ L = \text{distance of dolphin from pier as shown in Figure C4.8.3.5-1 (ft); length of pile above fixity as used in Equations C7.3.2-4 and C7.3.2-5 (in.)} \]
\[ L_B = \text{length of individual barge as shown in Figure 3.5.1-1 (ft)} \]
\[ L_{CB} = \text{length from bow to collision bulkhead for ships as defined in Figure 3.5.2-3 (ft)} \]
\[ LL = \text{live load as used in load combination Equation 3.14-1} \]
\[ LOA = \text{length overall of ship or barge tow as shown in Figures 3.5.1-2, 3.5.2-3, and 3.5.2-4 and in Tables 3.5.2-1 through 3.5.2-3 (ft)} \]
\[ LP = \text{length of bridge pier as defined in Figure 8.5.1-1 (ft)} \]
\[ L_W = \text{length of vessel at waterline as defined in Equation C3.5.2-1 (ft)} \]
\[ M = \text{mass of vessel as used in Article C7.3.2 (kip-s²/in.)} \]
\[ MHW = \text{mean high-water level of waterway as shown in Figures 3.15.1-1 through 3.15.1-3, 3.16-1, C7.3.1.1, and C7.3.1.2-1 (ft)} \]
\[ MIC = \text{motorist inconvenience cost due to bridge collapse as defined in Article 4.9.3 ($)} \]
\[ MLW = \text{mean low water level as shown in Figures C7.3.1.1-1, C7.3.1.2-1, and C7.3.3-4 (ft)} \]
\[ MSL = \text{mean sea level as shown in Figures C7.3.1.3-1, C7.3.3-6, C7.3.3-7, C7.3.4-6, and C7.3.5-1 (ft)} \]
\[ N = \text{number of one-way passages of vessels transiting under the bridge as defined in Article 4.8.3 (number/year)} \]
\[ n_h = \text{modulus of horizontal subgrade reaction as used in Equation C7.3.3-4 (lb/in.²)} \]
\[ NHW = \text{normal high water level as defined in Article 1.2.3 of the Risk Assessment Example} \]
\[ NRT = \text{net registered tonnage as defined in Article C3.5 (ft³)} \]
\[ P = \text{loads resulting from vessel impact and used in the group load combination of Equation 3.14-1 and shown in Figures C3.9-2 and C3.9-3; applied force to structure as used in Equation C7.3.2-1 (kipx)} \]
\[ P = \text{collision or impact force as defined in Article C4.8.3.4; lateral fill pressure used in Equation C7.3.3-2 (lb/ft²)} \]
\[ P_a = \text{barge collision impact force for head-on collision between barge bow and a rigid object as defined in Article 3.12 (kip)} \]
\[ P_{sh} = \text{ship collision impact force between ship bow and a rigid superstructure as defined in Equation 3.11.1-1 (kip)} \]
\[ P_{sh} = \text{ship collision impact force between ship deckhouse and a rigid superstructure as defined in Equation 3.11.2-1 (kip)} \]
\[ P_{sh} = \text{ship collision impact force between ship mast and a rigid superstructure as defined in Equation 3.11.3-1 (kip)} \]
\[ P_s = \text{ship collision impact force for head-on collision between ship bow and a rigid object as defined in Equations 3.9-1 and 3.10-1 (kip)} \]
$PA$ = probability of vessel aberrancy as defined in Article 4.8.3 (dimensionless)

$PC$ = probability of bridge collapse as determined in Article 4.8.3 (dimensionless)

$PF$ = protection factor as determined in Article 4.8.3

$PG$ = geometric probability of vessel collision with bridge pier/span as determined in Article 4.8.3 (dimensionless)

$PIC$ = port interruption cost as defined in Article 4.9.3 ($)

$PRC$ = pier replacement cost as defined in Article 4.9.3 ($)

$f(t)$ = compression phase of impact over time as used in Article C3.9

$ar{F}(d)$ = mean impact force averaged over damage depth as used in Equations C3.9-1 and C3.10-1 and as shown in Figure C3.9-3 (kip)

$ar{F}(g)$ = impact force averaged over damage depth as used in Equations C3.9-1 through C3.9-3b and as shown in Figures C3.9-2, C3.9-4, and C3.9-5

$PW$ = present worth of disruption cost as determined in Equation 4.9.2-1 ($)

$R$ = area of the density function between ±9 as shown in Figure C4.8.3.5-1

$r$ = dolphin radius as used in Equation C7.3.3-1 (ft)

$R_B$ = ratio of barge width as defined in Equation 3.13-1; $PA$ correction factor for bridge location as defined in Equations 4.8.3.2-1 and 4.8.3.2-2a through 4.8.3.2-2c (dimensionless)

$R_{sd}$ = ratio of exposed superstructure depth to the total ship bow depth as defined in Equation 3.11.1-1 (dimensionless)

$RC$ = $PA$ correction factor for currents parallel to vessel transit path as defined in Equations 4.8.3.2-1 and 4.8.3.2-3 (dimensionless)

$RD$ = $PA$ correction factor for vessel traffic density as defined in Article 4.8.3.2 (dimensionless)

$R_{sd}$ = reduction factor for ship deckhouse collision force as defined in Article 3.11.2 (dimensionless)

$RL$ = rake length of vessel bows as shown in Figures 3.5.1-1 and 3.5.2-3 (ft)

$R_{BC}$ = $PA$ correction factor for crosscurrents acting perpendicular to vessel transit path as defined in Equations 4.8.3.2-1 and 4.8.3.2-4 (dimensionless)

$S$ = bridge main span length over navigable channel as shown in Figure 8.5.1-1 (ft)

$SF$ = loads resulting from stream flow forces and used in the group load combination of Equation 3.14-1

$s_i$ = shear on interlocks as used in Equation C7.3.3-6 (lb)

$Sp$ = vertical pile spacing as used in Article 7.3.2 (in.)

$SRC$ = span replacement cost as defined in Article 4.9.3 ($)

$SW$ = horizontal water spacing as used in Article 7.3.2 (in.)

$T$ = relative stiffness factor for normally loaded clay, granular soils, silt, and peat as used in Equations C7.3.3-3 through C7.3.3-5 (in.)
SECTION 2—SYMBOLS AND DEFINITIONS

\( t \) = stopping time as used in Article C7.3.2 (sec); interlock tension as used in Equation C7.3.3-1 (lb/in.)

\( V \) = design impact speed of vessel as determined in Articles 3.7, 3.8, and 3.9 (ft/s); impact velocity as used in Article C7.3.2 (in/s)

\( V_C \) = waterway current component acting parallel to the vessel transit path as determined in Equation 4.8.3.2-3 (knots)

\( V_s \) = shear on centerline as used in Equation C7.3.3-7 (lb)

\( V_T \) = vessel transit speed in the navigable channel as defined in Article 3.7 (ft/sec)

\( V_{sc} \) = waterway current component acting perpendicular to the vessel transit path as determined in Equation 4.8.3.2-4 (knots)

\( W \) = displacement weight of vessel as defined in Equations C3.5.2-1, 3.8-1, and C3.8-1, (tonne, ton)

\( WA \) = water load and stream pressure used in group load combination Equation 3.14-1

\( W_b \) = ballasted displacement weight of vessel as shown in Figure 3.5.1-1 and Tables 3.5.2-1 through 3.5.2-3 (tonnes)

\( WL \) = waterline as shown in Figures 3.5.2-3, 3.5.2-4, and C7.3.4-10

\( W_L \) = fully loaded displacement weight of vessel as shown in Figure 3.5.1-1 and Tables 3.5.2-1 through 3.5.2-3 (tonnes)

\( W_o \) = deadweight tonnage of largest ship as defined in Article C4.7.2

\( W_w \) = volume of water (34.4 cubic ft per tonne of saltwater; 35.4 cubic ft per tonne of freshwater) as defined in Equation C3.5.2-1

\( x \) = distance to bridge element from the centerline of vessel transit path as shown in Figure 3.7-1 (ft) and Figure 4.8.3.3-1 (ft); distance to bridge element from the centerline of vessel transit path as shown in Figure 3.7-1; deflection of protection structure due to vessel impact as defined in Equation 7.3-1 (ft)

\( x_C \) = distance to edge of channel from centerline of vessel transit path as shown in Figure 3.7-1 (ft)

\( x_L \) = distance equal to \( 3 \times LOA \) from centerline of vessel transit path as shown in Figure 3.7-1 (ft)

\( Y \) = design life of the bridge (in years) as shown in Equation 4.9.2-1 (typically 75 years for a new bridge); maximum system deflection as used in Article C7.3.2 (in.)

\( Y_w \) = distance from pier centerline to edge of outbound channel as shown in Figure 8.5.1-1 (ft)

\( Y_P \) = off-set distance from edge of foundation to pier column as shown in Figures 3.15.1-2 and 3.15.1-3 (ft)

\( Y_w \) = distance from pier centerline to edge of inbound channel as shown in Figure 8.5.1-1 (ft)

\( \alpha \) = impact angle as shown in Figure C3.8-1 (degrees)

\( \gamma \) = average unit of weight of fill as used in Equation C7.3.3-2 and shown in Figure C7.3.3-8 (lb/ft^3)

\( \gamma_p \) = load factor, 1.25 maximum, 0.9 minimum, used in the group load combination of Equation 3.14-1

\( \Delta_p \) = pile deflection due to unit load as used in Equations C7.3.2-3 and C7.3.2-4 (in./kip)

\( \eta \) = portion of absorbed collision energy to initial collision energy as defined in Equation C3.8-3 and shown in Figure C3.8-1
\( \theta \) = angle of channel turn or bend as defined in Article 4.8.3.2 and as shown in Figure 4.8.3.2-1 (degrees); protection angle provided by dolphin as shown in Figure C4.8.3.5-1 (degrees)

\( \phi \) = angle between channel and bridge centerlines as shown in Figures 4.8.3.3-1 and 8.5.1-1 (degrees); angle of internal friction for granular soils as shown in Figure C7.3.3-7 and Equation C7.3.3-8 (degrees)

\( \lambda \) = frequency as used in Article C7.3.2 (sec\(^{-1}\))

\( \mu \) = coefficient of friction as used in Equation C7.3.3-6, Article C3.8, and Figure C3.8-1

\( \sigma \) = standard deviation of normal distribution as defined in Article 4.8.3.3
SECTION 4

DESIGN VESSEL SELECTION

4.1 GENERAL

The requirements of this Section shall control the design vessel selection for collision impact analysis of bridges in navigable waterways.

4.1.1 Design Method

Three alternative design methods are presented in this Section to determine the design vessel for collision impact analysis of the bridge. Method II and its corresponding acceptance criteria in Article 4.8.2 shall be used for all bridge design unless the approval of the Owner and the special situations stated in Article 4.1.2 exist.

4.1.2 Selection of Design Method

4.1.2.1 Method I

Method I is a simple semi-deterministic procedure for selecting the design vessel for collision impact. The procedure is calibrated to normally fulfill the Method II acceptance criteria in Article 4.8.2. However, the procedure is less accurate than Method II and should be used only in simple and uncomplicated situations. Situations in which Method I may be used include:

- Shallow draft waterways where the marine traffic consists almost exclusively of inland barges;
- Waterways where the distribution of vessel sizes (DVFT) using the channel is small (i.e., vessels in the waterway are almost all the same size); and
- Waterways in which accurate vessel traffic data is unavailable or difficult to obtain.

Situations in which Method I should not generally be used include:

- Critical/Essential Bridges,
- Deep draft waterways where large merchant ships comprise a significant portion of the total vessel traffic; and
- Waterways where the distribution of vessel sizes (DVFT) vary over a wide range of vessel types and sizes.

C4.1

Three alternative design methods, designated as Methods I, II, and III, are presented in Section 4 to provide the designer flexibility in determining the design vessel for ship/barge collision. Method II shall be used for all bridge design unless the special situations presented in Article 4.1.2 of the Guide Specifications exist.
4.1.2.2 Method II

Method II is a more complicated probability-based analysis procedure for selecting the design vessel for collision impact.

This method must be used in all situations where a proper documentation of fulfillment of the acceptance criteria in Article 4.8.2 is required.

4.1.2.3 Method III

Method III is a cost-effectiveness analysis procedure for selecting the design vessel for collision impact.

This method may be used in cases where it is not economical or technically feasible to design the bridge structure to comply with the acceptance criteria in Article 4.8.2.

A prerequisite for using Method III is that the annual frequency of bridge collapse is computed in accordance with Method II and brought to the attention of the owner.

Situations in which Method III may be considered include:

- Existing bridges which are evaluated for vulnerability to vessel collision and potential bridge protection retrofit measures; or
- Bridges crossing very wide waterways resulting in many piers exposed to vessel collision.

4.2 WATERWAY CHARACTERISTICS

C4.2

The typical vessel transit path in the waterway where a bridge crossing occurs must be determined by the designer. The approximate track of the vessels can be estimated based on actual observations of vessels using the waterway, discussions with the pilots and vessel operators using the waterway, or estimated based on experience. The location of the centerline of the vessel transit path is very important since it serves as the origin for the distribution of vessel impact speed (Article 3.7), impact distribution (Article 4.5), and the geometric probability (Article 4.8.3.3).

The water depth should be measured from the existing mudline to mean high water. It is recognized that this represents an approximation of the actual maximum water depth at a bridge pier. River flooding and periods of extreme high-water levels due to tropical and extratropical storms may cause water depths to significantly exceed that computed using “mean” high-water levels. Using mean high-water rather than extreme high water is recommended because of the use of annual averages with respect to the statistics on vessel frequency and accident data in developing the basic framework of these Guide Specifications. In those situations in which seasonal flooding or storms represent a significant portion of the
yearly high-water activity, judgment must be used to establish the design water level.

Design values for water currents at the bridge location should be selected based on the same philosophy discussed above for establishing design water levels. The design water currents should represent annual average values rather than the occasional extreme values which could occur under special circumstances.

In situations in which seasonal flooding or storms represent a significant portion of the yearly water current activity, judgment must be used to establish the design current values. For most waterways, the 2 percent flow line elevation is usually available from statistical data and represents the elevation at which the water can be expected to be at or higher 2 percent of the time.

4.2.1 Channel Layout

The geometry of the navigable channel that the bridge crosses shall be established for the waterway including the centerline of the navigable channel. The possibility of future modifications to the channel (deepening, widening, realignment, etc.) should be considered. The centerline of the typical vessel transit path under the bridge shall be determined. One of the following two situations usually exists:

1. For bridge and channel geometry where vessels can only transit one-at-a-time under the bridge, or for those bridge locations where vessels are prohibited from meeting or passing in the vicinity of the bridge, or for bridges located where vessels would rarely meet or pass in the vicinity of the bridge, the centerline of vessel transit path shall be taken as the centerline of the navigable channel as shown in Figure 1.

2. For most other bridges, the navigable channel shall be divided into two equal halves representing inbound and outbound traffic, respectively. The vessel transit path of inbound vessels shall be taken as the centerline of the inbound half of the channel, and the vessel transit path of outbound vessels as the centerline of the outbound half of the channel as shown in Figure 2.

The vessel transit path shall be determined by the designer for any special channel or vessel operating situations not covered by Items 1 and 2 above.
4.2.2 Water Depths

The design water depth for each pier and span element in the waterway shall be determined. As a minimum, the design water depth shall be computed from the bottom of the waterway to the annual mean high water level.

In waterways where seasonal flooding represents a significant portion of the high-water activity, judgment must be used to establish the design water level.

The ability of a vessel to strike a pier or span shall be determined based on the design water depth at the location of the bridge element, and the draft of the vessel.

The water depth at the pier should not include short-term scour. In addition, the water depth should not just be evaluated at the specific pier location itself, but also at locations upstream and downstream of the pier—which may be shallower and would potentially block certain deeper draft vessels from hitting the pier.

4.2.3 Water Currents

Water currents at the bridge location shall be resolved into currents in the direction of vessel movement and cross currents that act perpendicular to the direction of vessel movement.
4.3 BRIDGE CHARACTERISTICS

The alignment and location of the bridge in the waterway shall be determined. The bridge pier and span geometry, including the horizontal and vertical clearances of each pier and span member, shall be established.

4.4 VESSEL CHARACTERISTICS

Vessel types, sizes (in DWT), loading condition (loaded, partly loaded, or ballasted), speed, and number of annual passages for each type shall be determined for the waterway and bridge location. Inbound and outbound vessel characteristics shall be determined.

Barge and ship characteristics shall be based on the actual vessels using the waterway, or estimated from the data provided in Article 3.5.1 for barges and Article 3.5.2 for ships.

Vessel characteristics and the design vessel selection shall include consideration of the possibility of a growth in vessel frequency, distribution, and size over the design life of the bridge as a result of channel improvements in the waterway, or an increase in commerce on the waterway.

4.5 IMPACT DISTRIBUTION

The impact loads from the design vessel determined in accordance with Method I, II, or III shall be applied to the bridge structure for a distance of three times the length overall of the vessel (3 x LOA) on each side of the centerline of the inbound and outbound vessel transit paths in the navigable channel.

Portions of the bridge structure located outside of the 3 x LOA distance on each side of the vessel transit path shall be designed in accordance with the minimum impact loads in Article 3.16.

The LOA shall be based on the dimensions of a vessel selected in accordance with the Method I criteria in Article 4.7.2. The LOA for impact distribution is the same dimension used in Article 3.7 for vessel impact speed and is a constant for Methods I, II, and III. For barge tows, LOA shall be equal to the combined length of all barges in the tow plus the length of the tug/tow vessel as shown in Figure 3.5.1-2.

4.6 DESIGN LOADS

The impact force and energy for the selected design vessel using Method I, II, or III, shall be determined in accordance with Section 3.
4.7 METHOD I

4.7.1 General

Method I is a semi-deterministic analysis procedure for determining the design vessel. Method I requires a minimum amount of input data for the vessel and waterway characteristics.

4.7.2 Design Vessel Acceptance Criteria

The design vessels shall be selected based on the bridge operational classification, vessel characteristics, bridge geometry, and water depths in accordance with the following acceptance criteria:

- Critical/Essential Bridges. The design vessel size shall be determined such that the annual number of vessel passages that involve vessels larger than the design vessel amounts to a maximum of 50 vessel passages, or 5 percent of the total number of merchant vessels per year which could impact the bridge element, whichever is smaller.

- Typical Bridges. The design vessel size shall be determined such that the annual number of vessel passages that involve vessels larger than the design vessel amounts to a maximum of 200 vessel passages or 10 percent of the total number of merchant vessels per year which could impact the bridge element, whichever is smaller.

C4.7

4.7.1

Method I is a semi-deterministic analysis procedure for selecting the design vessel. The intent of Method I is to provide a simple, conservative procedure for determining the design impact loads without having to deal with the large data collection and analysis requirements of Methods II and III.

C4.7.2

The framework of the Method I acceptance criteria was based on the ship impact criteria for bridge design stated in the Common Nordic Regulations (1980) currently in use in Scandinavian countries. The following is quoted from these regulations (Nordic Road Engineering Federation, 1980):

"For waters difficult to navigate the design vessel size shall be determined such that the number of ships that are larger than the design vessel amounts to a maximum of 50 ships or 10 percent of the total number of ships.

For waters easy to navigate the design vessel size shall be determined such that the number of ships that are larger than the design vessel amounts to a maximum of 200 ships or 20 percent of the total number of passing ships.

The design vessel size must not be taken less than $(0.05)W_o$, where $W_o$ is the deadweight tonnage of the largest ship, using the sea lane."

The values quoted above for 50 ships and 200 ships were used for the Critical/Essential and Typical bridge operational classification categories, respectively. The Guide Specification project consultants considered the 10 percent and 20 percent values to be too high in the Nordic Code and lowered the values to 5 percent and 10 percent for the Critical/Essential and Typical operational classification categories, respectively.

4.8 METHOD II

4.8.1 General

Method II is a probability-based analysis procedure for determining the design vessel. Method II requires a significant amount of input data for the vessel, bridge, and waterway characteristics. An idealized mathematical model describing the bridge and the vessel traffic transiting through the bridge is used to estimate the probability of bridge collapse and to determine the design vessel impact forces for elements of the bridge structure.

C4.8

4.8.1

The use of any risk analysis method involves the complex organization of a large body of data into a series of computations based on statistical and probability procedures. Values must be determined for a large number of parameters, often with the designers' judgment as the primary basis of the estimate. Because of this, the outcome of the analysis can be influenced by the design engineer and its integrity depends on the design engineer's experience and abilities.
4.8.2 Design Vessel Acceptance Criteria

The design vessels shall be selected based on the bridge operational classification, vessel, bridge, and waterway characteristics in accordance with the following acceptance criteria for the total bridge:

- **Critical/Essential Bridges.** The acceptable annual frequency of collapse, $AF$, of critical/essential bridges shall be equal to, or less than, 0.01 in 100 years ($AF = 0.0001$).

- **Typical Bridges.** The acceptable annual frequency of collapse, $AF$, of typical bridges shall be equal to, or less than, 0.1 in 100 years ($AF = 0.001$).

The acceptable annual frequency of bridge collapse for the total bridge as determined above shall be distributed over the number of pier and span elements located within the waterway, or within the distance $3 \times LOA$ on each side of the inbound and outbound vessel transit paths if the waterway is wide. This results in an acceptable risk criteria for each pier and span element of the total bridge.

The design vessel for each pier or span element shall be chosen such that the annual frequency of collapse due to vessels equal to, or larger than, the design vessel is less than the acceptance criterion for the element.

The Method II procedure for selecting the design vessel is a probability-based, risk-analysis method. Method II was developed to minimize the number of judgment calls that the designer must make during the analysis. In order to do this, various empirical relationships based on experience and judgment were developed for the Guide Specifications.

C4.8.2

Establishment of risk acceptance criteria for use in Method II for vessel collision with bridges was one of the most difficult elements of the Guide Specification development. A comprehensive literature search and consultation with risk analysis experts was conducted during the Guide Specification development.

Risk can be defined as the potential realization of unwanted consequences of an event (Rowe, 1983). Both a probability of occurrence of an event and the magnitude of its consequence are involved. Risk estimation is the process used for controlling such risks and arriving at an acceptable level of risk. Defining an acceptable level of risk is a value oriented process, and is by nature subjective (Rowe, 1977). Risk estimation purports to be value free, but when rare events (such as ship collisions) are treated, very large levels of uncertainty exist and value judgments of engineers are sometimes used in the absence of hard data. It must be noted that the estimated risk cannot be fully equated with actual risk because probability and consequence estimates that make up a risk estimate may be inexact.

There are many approaches to evaluating risks to determine acceptability (Phillipson, 1983). The most important of these can be grouped into two broad categories: 1) risk comparison approaches, and 2) cost-effectiveness of risk reduction. Risk comparison was used to establish the Method II acceptance criteria, and cost-effectiveness of risk reduction to the Method III acceptance criteria.

Figures C1 and C2 are typical of the type of risk comparison data available in the literature for risks associated with natural events and engineering projects. One of the objectives of the Guide Specifications was to establish a simple criterion defining a single level of risk acceptance for superstructure collapse for each of the two operational classification categories, which would be easily understood and used by bridge designers.

Based on the data available concerning risk comparisons and their judgment, the Guide Specification project consultants established an acceptance criterion of $AF = 0.0001$ per year for critical/essential bridges, and $AF = 0.001$ per year for typical bridges for bridge collapse associated with vessel collision.

The critical/essential bridge acceptance criterion, $AF = 0.0001$ per year, is the same criterion recommended by Modjeski and Masters (1985) for vessel collision in Louisiana waterways. This acceptance criterion has been
used for several recent long span bridges, including the Annacis Island Bridge near Vancouver, Canada (Sexsmith, 1983). As seen in Figure C2 which depicts the risk of failure of selected engineering projects, an $AF = 0.0001$ (i.e., $1 \times 10^{-4}$) is equivalent to the risk of failure of dams.

The typical bridge acceptance criterion, $AF = 0.001$ per year exceeds the risk of failure of foundations and is equivalent to the risk of failure of fixed drill rig structures as shown in Figure C2.

**Figure C4.8.2.1—Risk of Fatalities from Natural Events (Whitman, 1984)**

**Figure C4.8.2.2—Risk of Failure of Selected Engineering Projects (Whitman, 1984)**

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4.8.3 Annual Frequency of Collapse

The annual frequency of bridge element collapse shall be computed by:

\[ AF = (N)(PA)(PG)(PC)(PF) \]  (4.8.3-1)

where:

- \( AF \) = annual frequency of bridge element collapse due to vessel collision;
- \( N \) = annual number of vessels classified by type, size, and loading condition which can strike the bridge element;
- \( PA \) = probability of vessel aberrancy;
- \( PG \) = geometric probability of a collision between an aberrant vessel and a bridge pier or span;
- \( PC \) = probability of bridge collapse due to a collision with an aberrant vessel; and
- \( PF \) = adjustment factor to account for potential protection of the piers from vessel collision due to upstream or downstream land masses, or other structures, that block the vessel.

\( AF \) shall be computed for each bridge element and vessel classification. The summation of all element \( AF \)'s equals the annual frequency of collapse for the entire bridge structure.

4.8.3.1 Vessel Frequency (\( N \))

A vessel frequency distribution shall be determined for the bridge site. The number of vessels, \( N \), passing under the bridge based on size, type, and loading condition and available water depth shall be developed for each pier and span element to be evaluated. Depending on waterway conditions, a differentiation between the number and loading condition of vessels transiting inbound and outbound may also be required.

The vessel frequency distribution for vessels should be developed and modeled using \( DWT \) classification intervals appropriate for the waterway vessel traffic. Guidelines are provided in the Commentary.

C4.8.3

Various types of risk assessment models have been developed for vessel collision with bridges by researchers worldwide (IABSE Colloquium). Practically all of these are based on a similar form of guide specification Eq. 1, which is used to compute the annual frequency of bridge collapse, \( AF \), associated with a particular bridge element. Summation of \( AF \) for each element in the bridge results in the \( AF \) for the entire bridge as a whole. The inverse of the \( AF \) (i.e., \( 1/AF \)) is equal to the return period (in years).

C4.8.3.1

Sources for obtaining vessel frequency data are discussed in Article C3.4. In order to use Method II, a determination of the number of vessels (\( N \)) and their size (\( DWT \)) must be made for each bridge element to be evaluated. The number of vessels that could strike a pier or span is based on the water depth and the draft of the vessel. Ballasted as well as loaded vessels should be included in the analysis.

The designer must use judgment in developing a distribution of the vessel frequency data based on discrete groupings or categories of vessel size by \( DWT \). It is recommended that the \( DWT \) intervals used in developing the vessel distribution not exceed 20,000 \( DWT \) for vessels smaller than 100,000 \( DWT \) and not exceed 50,000 \( DWT \) for ships larger than 100,000 \( DWT \). An example of vessel distribution is shown in Table C1.

In developing the vessel distribution, the designer should first establish the number and characteristics of the vessels using the navigable waterway under the bridge. Since the water depth limits the size of vessel that
could strike a bridge element, the main channel vessel frequency data should be modified as required based on the water depth at each bridge element.

Table C4.8.3.1-1—Vessel Frequency Data for the Dame Point Bridge, Jacksonville, Florida (1984 Fleet) (Greiner Engineering Sciences, Inc., 1984)

<table>
<thead>
<tr>
<th>Vessel Type</th>
<th>DWT</th>
<th>Number of Annual Transits (n)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Loaded</td>
</tr>
<tr>
<td>Barge (Ocean)</td>
<td>15,000</td>
<td>73</td>
</tr>
<tr>
<td>Barge (Ocean)</td>
<td>25,000</td>
<td>67</td>
</tr>
<tr>
<td>Barge (Ocean)</td>
<td>35,000</td>
<td>81</td>
</tr>
<tr>
<td>Barge (Ocean)</td>
<td>50,000</td>
<td>66</td>
</tr>
<tr>
<td>Freighter/Container</td>
<td>10,000</td>
<td>170</td>
</tr>
<tr>
<td>Freighter/Container</td>
<td>18,000</td>
<td>360</td>
</tr>
<tr>
<td>Freighter/Container</td>
<td>26,000</td>
<td>28</td>
</tr>
<tr>
<td>Tanker/Bulk Carrier</td>
<td>20,000</td>
<td>67</td>
</tr>
<tr>
<td>Tanker/Bulk Carrier</td>
<td>30,000</td>
<td>139</td>
</tr>
<tr>
<td>Tanker/Bulk Carrier</td>
<td>40,000</td>
<td>78</td>
</tr>
<tr>
<td>Tanker/Bulk Carrier</td>
<td>60,000</td>
<td>25</td>
</tr>
</tbody>
</table>

Note: Ocean-going barges and the tanker/bulk carriers transit one-way loaded and one-way empty or ballasted. Freighter/Container ships transit loaded in both directions.

4.8.3.2 Probability of Aberrancy ($P_A$)

The probability of aberrancy, $P_A$, is a value related to the statistical probability that a vessel will stray off-course and threaten the bridge. Vessel aberrancy is usually a result of pilot error, adverse environmental conditions, or mechanical failure. Values of $P_A$ vary widely.

The most accurate method of determining $P_A$ for a particular bridge site is based on historical data on vessel collisions, ramming, stranding and groundings in the waterway, and the number of vessels transiting the waterway during the period of accident reporting. From this data, $P_A$ can be computed.

In lieu of the above method, $P_A$ can be estimated for the bridge/waterway location by the following:

$$P_A = BR(R_b)(R_c)(R_{sc})(R_D)$$  \hspace{1cm} (4.8.3.2-1)

where:

- $P_A$ = probability of aberrancy,
- $BR$ = aberrancy base rate,
- $R_b$ = correction factor for bridge location,
- $R_c$ = correction factor for current acting parallel to vessel transit path,
- $R_{sc}$ = correction factor for crosscurrents acting perpendicular to vessel transit path, and

C4.8.3.2

The probability of aberrancy, $P_A$, (sometimes referred to as the causation probability) is a measure of the risk that a vessel is in trouble as a result of a pilot error, adverse environmental conditions, and/or mechanical failure. Examples of these factors are listed below.

1. **Human Errors:**
   - inattentiveness on board the ship,
   - lack of reactivity (drunkenness, tiredness),
   - misunderstanding between captain/pilot/helmsman,
   - incorrect interpretation of chart or notice to mariners,
   - violations of rules of the road at sea, and
   - incorrect evaluation of current and wind conditions, etc.

2. **Adverse Environmental Conditions:**
   - poor visibility (fog, rainstorm),
   - high density of ship traffic,
   - strong current or wave action,
   - wind squalls,
   - poor navigation aids, and
   - awkward channel alignment, etc.

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\[ R_O = \text{correction factor for vessel traffic density.} \]

Based on historical accident data from several U.S. waterways, the base rate, \( BR \), can be estimated as follows:

- For ships \( BR = 0.6 \times 10^{-4} \)
- For barges \( BR = 1.2 \times 10^{-4} \)

The correction factor for bridge location, \( R_b \), can be estimated based on the relative location of the bridge in either of three waterway regions shown in Figure 1 as:

1. **Straight Region**: For a bridge located in a straight region:
   \[
   R_b = 1.0 \quad \text{(4.8.3.2-2a)}
   \]

2. **Transition Region**: For a bridge located in a transition region, \( R_b \) can be computed by:
   \[
   R_b = \left( 1 + \frac{\theta}{90^\circ} \right) \quad \text{(4.8.3.2-2b)}
   \]
   where:
   \[ \theta = \text{angle of the turn (degrees)} \]

3. **Turn/Bend Region**: For a bridge located in a turn or bend region, \( R_b \) can be computed by:
   \[
   R_b = \left( 1 + \frac{\theta}{45^\circ} \right) \quad \text{(4.8.3.2-2c)}
   \]

The correction factor, \( R_C \), for currents acting parallel (i.e., along track) to the vessel transit path in the waterway can be computed by:

\[
R_C = \left( 1 + \frac{V_C}{10} \right) \quad \text{(4.8.3.2-3)}
\]

where:
\[ V_C = \text{current component parallel to vessel path (knots)} \]

The correction factor, \( R_{XC} \), for crosstransversals acting perpendicular to the vessel transit path in the waterway can be computed by:

\[
R_{XC} = \left( 1 + V_{XC} \right) \quad \text{(4.8.3.2-4)}
\]

where:
\[ V_{XC} = \text{current component perpendicular to vessel path (knots)} \]

\[ \]

### 3. Mechanical Failures:
- mechanical failure of engine,
- mechanical or electrical failure of steering, and
- other failures due to poor equipment, etc.

An evaluation of accident statistics indicates that human errors and adverse environmental conditions are the primary reasons for accidents rather than mechanical failures. In the United States, an estimated 60 percent to 85 percent of all vessel accidents have been attributed to human error.

The most accurate procedure for determining \( PA \) is to compute it using long-term vessel accident data (groundings, collisions, stranding, and ramming) in the waterway, and statistics on the frequency of ship/barge traffic in the waterway during the same period of time. Table C1 lists values of \( PA \) developed from accident data for various waterway and bridge locations worldwide. As indicated in Table C1, the abnormally high rate for barges is usually two to three times that measured for ships in the same waterway.

Since the determination of \( PA \) based on actual accident data in the waterway is often a difficult and time-consuming process, an alternative simpler method for estimating \( PA \) is provided in the Guide Specifications. Eqs. 1 through 4 are empirical relationships based on historical accident data. The comparison between the predicted \( PA \) value using these equations and the value determined from the accident statistics in Table C1 is generally in fair agreement, although exceptions do occur.

Note that the procedure for computing \( PA \) using Eq. 1 should not be considered as being either rigorous or exhaustive. Several influences, such as wind, visibility conditions, navigation aids, piloting, etc., were not directly included in the method because their effects were difficult to quantify. Indirectly these influences are included because the empirical equations were developed from accident data in which these influences had a part.

It is anticipated that future research will provide a better understanding of the probability of aberrancy and how to accurately estimate its value. An ongoing (unpublished) study on vessel accident statistics for the proposed Great Belt Bridge in Denmark questions the use of grounding and ramming accident data to predict the probability of aberrancy associated with bridge collisions, and is trying to develop an alternate method of estimating aberrancy values. Future research is also needed to identify methods of reducing the probability of aberrancy in a waterway in order to reduce the risk of collision with a bridge structure. The implementation of advanced vessel traffic control systems using automated surveillance and warning technology should significantly reduce the probability of aberrancy in navigable waterways.
The correction factor for vessel traffic density, $R_D$, in the waterway in the immediate vicinity of the bridge can be estimated by determining whether the bridge is in either a low, medium, or high density area as defined below:

**Low Density**, $R_D = 1.0$—vessels rarely meet, pass, or overtake each other in the immediate vicinity of the bridge.

**Average Density**, $R_D = 1.3$—vessels occasionally meet, pass, or overtake each other in the immediate vicinity of the bridge.

**High Density**, $R_D = 1.6$—vessels routinely meet, pass, or overtake each other in the immediate vicinity of the bridge.

![Diagram of Turn in Channel](image)

![Diagram of Bend in Channel](image)

**Figure 4.8.3.2-1**—Waterway Regions for Bridge Location

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### Table C4.8.3.2.1—Summary of Probability of Aberrancy, P₄, Values

<table>
<thead>
<tr>
<th>Locality</th>
<th>Type of Data</th>
<th>Probability of Vessel Aberrancy (×10⁻⁴)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dover Straits—Collisions (MacDuff, 1974)</td>
<td>Statistics</td>
<td>5 to 7</td>
</tr>
<tr>
<td>Dover Straits—Groundings (MacDuff, 1974)</td>
<td>Statistics</td>
<td>1.4 to 1.6</td>
</tr>
<tr>
<td>Japanese Straits—Groundings (Fujii et al., 1974)</td>
<td>Statistics</td>
<td>0.7 to 6.7</td>
</tr>
<tr>
<td>Japanese Straits—Collisions (Fujii et al., 1974)</td>
<td>Statistics</td>
<td>1.3</td>
</tr>
<tr>
<td>Worldwide (MacDuff and Partners, 1979)</td>
<td>Statistics</td>
<td>0.5</td>
</tr>
<tr>
<td>Tasman Bridge, Australia (Leitch, 1979)</td>
<td>Estimate</td>
<td>0.6 to 1.0</td>
</tr>
<tr>
<td>Great Belt Bridge, Denmark (Cowperconsult, 1978)</td>
<td>Estimate</td>
<td>0.4</td>
</tr>
<tr>
<td>Sunshine Skyway Bridge, Florida (Greiner Engineering Sciences, 1985)</td>
<td>Statistics</td>
<td>1.3 (Ships)</td>
</tr>
<tr>
<td>Sunshine Skyway Bridge, Florida (Greiner Engineering Sciences, 1985)</td>
<td>Statistics</td>
<td>2.0 (Barges)</td>
</tr>
<tr>
<td>Annacis Island Bridge, Canada (UBA/Bucknell and Taylor, 1982)</td>
<td>Estimate</td>
<td>3.6</td>
</tr>
<tr>
<td>Francis Scott Key Bridge and William Preston Lane Bridges, Maryland (Greiner Engineering Sciences, 1983)</td>
<td>Statistics</td>
<td>1.0 (Ships)</td>
</tr>
<tr>
<td>Francis Scott Key Bridge and William Preston Lane Bridges, Maryland (Greiner Engineering Sciences, 1983)</td>
<td>Statistics</td>
<td>2.0 (Barges)</td>
</tr>
<tr>
<td>Dames Point Bridge, Florida (Greiner Engineering Sciences, 1984)</td>
<td>Statistics</td>
<td>1.3 (Ships)</td>
</tr>
<tr>
<td>Dames Point Bridge, Florida (Greiner Engineering Sciences, 1984)</td>
<td>Statistics</td>
<td>4.1 (Barges)</td>
</tr>
<tr>
<td>Lavolette Bridge, Canada (Greiner Engineering Sciences, 1984)</td>
<td>Statistics</td>
<td>0.5</td>
</tr>
<tr>
<td>Centennial Bridge, Canada (Greiner Engineering Sciences, 1986)</td>
<td>Statistics</td>
<td>5.0</td>
</tr>
<tr>
<td>Louisiana Waterways (Philippi, 1983)</td>
<td>Statistics</td>
<td>0.8 to 1.9</td>
</tr>
<tr>
<td>Louisiana Waterways (Philippi, 1983)</td>
<td>Statistics</td>
<td>1.5 to 3.0</td>
</tr>
<tr>
<td>Gibraltar Straits—Strandings, Morocco (Modjeski and Masters Consulting Engineers, 1985)</td>
<td>Statistics</td>
<td>2.2</td>
</tr>
<tr>
<td>Gibraltar Straits—Collision, Morocco (Modjeski and Masters Consulting Engineers, 1985)</td>
<td>Statistics</td>
<td>1.2</td>
</tr>
</tbody>
</table>

#### 4.8.3.3 Geometric Probability (PG)

The geometric probability is defined as the conditional probability that a vessel will hit a bridge pier or span given that it has lost control (i.e., it is aberrant) in the vicinity of the bridge. Based on a review of historical bridge collision data, a normal distribution shall be utilized to model the aberrant vessel transit path near the bridge as shown in Figure 1. The standard deviation, σ, of the normal distribution shall be assumed equal to the LOA of the vessels in the design fleet. The LOA dimension for the normal distribution is the same value used in Article 3.7 for impact speed and Article 4.5 for impact distribution.

The location of the mean of the standard distribution shall be equal to the centerline of the vessel transit path determined in accordance with Article 4.2.1. In the computation of AF, the value of PG shall be computed based on the width (beam), B₉, of each vessel classification category, or it may be computed for all classification intervals using the B₉ of a vessel selected using Method 1 as discussed above.

As shown in Figure 1, the value of PG for a pier represents the area in the normal distribution bounded by the pier width and the width of the vessel on each side of the pier.

#### C4.8.3.3

The geometric probability, PG, is defined as the conditional probability that a vessel will hit a bridge pier or span given that it has lost control (i.e., it is aberrant) in the vicinity of the bridge. The probability of occurrence depends on a great number of factors such as:

- geometry of the waterway;
- water depths of the waterway;
- location of bridge piers;
- span clearances;
- transit path of the vessel;
- maneuvering characteristics and size of vessel;
- location, heading, and velocity of vessel;
- rudder angle at time of failure;
- environmental conditions;
- width, length, and shape of vessel; and
- vessel draft (loaded or ballasted).

The methods used to determine PG varies significantly among researchers. Models to compute PG as developed by Fujii (Fujii and Shiozara, 1978; Fujii et al., 1984), MacDuff (1974), Cowperconsult (1987a and b),
Knott (1983; 1985), and Modjeski and Masters (1985) were evaluated during the Guide Specification development. Their methods range in use from relatively simple (Fujii) to complex (Cowiconsult). A combination of the best features from each of these models was developed into a relatively straightforward risk model for the Guide Specifications.

The geometric probability, $PG$, is computed based on a normal distribution of vessel accidents about the centerline of the vessel transit path as shown in Figure 1. The use of a normal distribution is based on historical ship, bridge accident data, although it must be recognized that the number of data points in the database are very few from a statistical point of view. By definition 68.3 percent of all collisions occur within one standard deviation ($\sigma$) of the mean, 95.5 percent within two standard deviations ($2\sigma$), and 99.7 percent within three standard deviations ($3\sigma$) for a normal distribution. The Guide Specifications recommend that $\sigma = LOA$ of the design vessel for computing $PG$, and that bridge elements beyond $3\sigma$ from the centerline of the vessel transit path of the largest vessels in the design fleet not be included in the analysis (other than the minimum impact requirement).

Table C1 provides the accident data used to develop the recommended value of $\sigma = LOA$. The use of $LOA$ as the standard by which $\sigma$ is computed, was a recommendation by the project consultants and is considered preferable to criteria based on channel width, or by simply using a fixed distance for $\sigma$, since the value of $PG$ is influenced by the size of the ships and barges passing under the bridge.

The 1991 Guide Specifications (Article 3.7 on Design Impact Speed and Article 4.8.3.3 Geometric Probability) required the use of a vessel length overall ($LOA$) selected in accordance with the Method I criteria for use in estimating the impact speed and geometric probability for all vessel classifications. This provision has been revised in the new Guide Specifications to allow for the $LOA$ of each specific vessel category to be used in determining the vessel speed distribution and geometric probability associated with that specific vessel category.

The accident data in Table C1 primarily represents ship vessels. Although barge accidents occur relatively frequently in U.S. waterways, there has been little published research concerning the distribution of barge accidents over a waterway. Until such data and research become available, the Guide Specification project consultants recommend that the same $\sigma = LOA$ developed for ships be applied to barges with the barge $LOA$ equal to the total length of the barge tow, including the towboat.
4.8.3.4 Probability of Collapse (PC)

The probability of bridge collapse, PC, once a bridge element has been struck by an aberrant vessel is a function of many variables, including vessel size, type, forepeak ballast and shape, speed, direction of impact, and mass. It is also dependent on the ultimate lateral capacity of the pier, HP, and span, HS, to resist collision impact loads. Based on damage sustained during ship-ship collisions, which have been correlated to the bridge-ship collision situation, PC shall be computed as follows:

For $0.0 \leq HP < 0.1$, PC shall be computed as:

$$PC = 0.1 + 9 \left( 0.1 - \frac{HP}{P} \right)$$  \hspace{1cm} (4.8.3.4-1a)

For $0.1 \leq HP < 1.0$, PC shall be computed as:

$$PC = \left( 1 - \frac{HP}{P} \right)$$  \hspace{1cm} (4.8.3.4-1b)

### Table C4.8.3.3-1—Computation of Standard Deviation for Normal Distribution of Historic Collisions with Bridges

<table>
<thead>
<tr>
<th>Bridge Name</th>
<th>x</th>
<th>$(x-x_m)^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sidney Lanier</td>
<td>0.57</td>
<td>0.325</td>
</tr>
<tr>
<td>Taunton Bridge</td>
<td>1.47</td>
<td>2.161</td>
</tr>
<tr>
<td>Fraser River Bridge</td>
<td>0.31</td>
<td>0.096</td>
</tr>
<tr>
<td>Benjamin Harrison</td>
<td>0.69</td>
<td>0.476</td>
</tr>
<tr>
<td>Tidewater Bridge</td>
<td>0.33</td>
<td>0.109</td>
</tr>
<tr>
<td>Second Narrows RR</td>
<td>0.43</td>
<td>0.185</td>
</tr>
<tr>
<td>Second Narrows RR</td>
<td>0.66</td>
<td>0.436</td>
</tr>
<tr>
<td>Almont (Te) Bridge</td>
<td>0.89</td>
<td>0.792</td>
</tr>
<tr>
<td>Sunshine Skyway</td>
<td>1.31</td>
<td>1.716</td>
</tr>
<tr>
<td>Newport Bridge</td>
<td>1.07</td>
<td>1.145</td>
</tr>
<tr>
<td>Sensen Bridge</td>
<td>0.82</td>
<td>0.672</td>
</tr>
<tr>
<td>Outerbridge (NY)</td>
<td>0.78</td>
<td>0.608</td>
</tr>
<tr>
<td>Outerbridge (NY)</td>
<td>0.52</td>
<td>0.270</td>
</tr>
<tr>
<td>Outerbridge (NY)</td>
<td>0.50</td>
<td>0.250</td>
</tr>
<tr>
<td>Richmond/San Rafael</td>
<td>2.13</td>
<td>4.537</td>
</tr>
</tbody>
</table>

$s = 13.778$

where:

$$\sigma = \frac{s}{\sqrt{(n-1)}}$$

where:

$x = \frac{\text{ratio of the approximate vessel impact distance from centerline of vessel transit to the LOA of the vessel}}{x_m}$

$x_m = \text{mean of distribution} = 0.0$, and

$n = \text{number of collisions} = 15$.

C4.8.3.4

The probability that the bridge will collapse, PC, once it has been struck by an aberrant vessel is very complex and is a function of the vessel size, type, configuration, speed, direction, mass, and the nature of the collision. It is also dependent on the stiffness, resistance, and stability characteristics of the bridge pier and span to resist the collision impact loads.

The Guide Specification methodology for estimating the probability of bridge collapse was derived from studies performed by Fuji in Japan (1978) using historical information about damage to ships colliding at sea. The curves in Figure C1 are reproduced from Fuji's paper where the following definitions are used:

$x = \text{the damage rate is defined as the ratio between the estimated damage cost to the ship (excluding the loss of cargo) and the estimated value of the ship}$

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For $H/P > 1.0$

\[ PC = 0.0 \quad (4.8.3.4-1c) \]

where:

- $PC =$ probability of collapse;
- $H =$ ultimate bridge element resistance, $H_D$ or $H_S$ (kip); and
- $P =$ vessel impact force, $P_D$, $P_{BH}$, $P_{DH}$, or $P_{MT}$ (kip).

Figure 1 is a plot of the above probability of collapse relationships. From Figure 1, the following results are evident:

- in cases where the pier or span impact resistance capacity exceeds the vessel collision impact force of the design vessel, the bridge collapse probability becomes zero;
- in cases where the pier or span impact resistance is in the range 10 to 100 percent of the collision force of the design vessel, the bridge collapse probability varies linearly between zero and 0.10;
- and in cases where the pier or span impact resistance capacity is below 10 percent of the collision force, the bridge collapse probability varies linearly between 0.10 and 1.0.

$y =$ GRT ratio is defined as the ratio between GRT of "the other ship" to the ship to which $x$ is related.

To equate Fuji's results with the size of the collision force, $p$, a damage rate is defined as:

\[ x = \frac{P}{P_{\text{max}}} \quad (C4.8.3.4-1) \]

For $x = 1.0$, the actual impact force, $p$, is the same as the maximum possible impact force, and the vessel has been totally damaged.

The damage to bridge piers is estimated based on the information on ship damage since damage for collisions with bridges is relatively scarce. Cowiconsult (1987a and b) developed the probability density function shown in Figure C2b for the relative magnitude of the collision force using Fuji's results and the following assumptions:

- The pier is considered as a large collision object relative to the ship (i.e., the GRT ratio $y = 10$ to 100).
- The relative magnitude of the collision force ($p/p_{\text{max}}$) is related to the damage rate, $x$.
- From Figure C1 for $p/p_{\text{max}} \geq 0.1$, the probability is approximately 0.1.
- The probability density function for $p/p_{\text{max}}$ has been simplified to be uniform in each of the intervals 0 to 0.1 and 0.1 to 1.

The distribution function, $F$ for $p/p_{\text{max}} \geq x_m$, shown in Figure C2b was derived by integrating $f$ from the upper end in Figure C2a. Figure 1 in the Guide Specification is the same as Figure C2b except that the nomenclature for the terms was changed to agree with the Guide Specification terminology.
Figure C4.8.3.4-1—Fujii’s Distribution Function for Damage Rate to Ships (Fujii, 1978)
The concept of the protection factor was indirectly included in the 1991 AASHTO Guide Specification, but presented some confusion. The inclusion of $PF$ in this Guide Specification clarifies the concept and makes it explicit. The recommended procedure for estimating values for $PF$ are shown in Figure C1 which illustrates a simple model developed to estimate the effectiveness of dolphin protection on a bridge pier.
\( PF \) should be computed as:

\[
PF = 1 - (\% \text{ Protection Provided}/100) \quad (4.8.3.5-1)
\]

If no protection of the pier exists, then \( PF = 1.0 \). If the pier is 100 percent protected, then \( PF = 0.0 \). If the pier protection (for example a dolphin system) provides 70 percent protection, then \( PF \) would be equal to 0.3. Values for \( PF \) may vary from pier to pier and may vary depending on the direction of the vessel traffic (i.e., vessel traffic moving inbound versus traffic moving outbound).

\[
D_E = D + 0.75(B)
\]

\[
\theta = \sin^{-1}\left[\frac{D_E}{2L}\right]
\]

where

- \( \theta \) = Protection angle provided by dolphin,
- \( D \) = Diameter of dolphin (ft)
- \( B \) = Beam (width) of vessel (ft)
- \( L \) = Distance of dolphin from pier (ft)
- \( D_E \) = Effective dolphin diameter (ft)

**a. Plan of Dolphin Protection**

\[
D_A = R(DC)
\]

where

- \( D_A \) = Avoidable disruption cost ($)
- \( R \) = Area of the density function between \( \pm \theta \)
- \( DC \) = Disruption cost ($)

**b. Normal Distribution of Vessel Collision Trajectories around Bridge Pier (\( \alpha \) assumed = 30°)**

Figure 4.8.3.5-1—Illustrative Model of the Protection Factor (PF) of Dolphin Protection around a Bridge Pier

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4.9 METHOD III

4.9.1 General

Method III is a cost-effectiveness analysis procedure for determining the design vessel. Method III can also be used to determine the design capacity of bridge members or indicate the appropriate level of protection for the bridge. In certain cases, the risk acceptance criteria defined in Methods I and II cannot be fulfilled due to unreasonable or prohibitively high costs. These cases might include bridges crossing very wide waterways with many piers exposed to vessel collision, the retrofit of existing piers found to be vulnerable to vessel collision, or piers located in very deep water.

For those situations, the economics associated with the cost-effectiveness of risk reduction using Method III can be used to determine the design vessel, the design resistance of bridge members, or the appropriate level of protection for the bridge.

4.9.2 Design Vessel Acceptance Criteria

The design vessel and the design resistance of the bridge or the type of protection to be provided shall be selected based on a cost-effectiveness acceptance criteria (such as a benefit/cost analysis) where the cost of bridge strengthening or bridge protection systems is compared against the benefits of risk reduction.

The analysis methodology used to test economic feasibility and desirability shall be a conventional benefit/cost, B/C, ratio calculation in which the present worth of avoidable disruption cost, $PW$, for each year of the analysis period is compared against the total present worth of the costs to build, maintain, and operate the protection system or bridge strengthening required to provide those benefits. The present worth of the costs and benefits of the protected bridge shall be computed over a specific time period in order to identify incremental costs and benefits attributable to the protection system. The present worth is the cumulative present value of a series of costs and benefits occurring over time, and is derived by applying to each cost or benefit in the series an appropriate discount factor, which converts each cost or benefit to present value.

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costs, benefits, and other values shall be expressed in constant dollars. Growth of the disruption cost over time shall be considered in the analysis.

The approximate benefit used to compare against the cost of strengthening, retrofitting, or adding a pier protection system to a bridge can be estimated as follows:

\[
P_{W} = (AF)(DC)\left[\frac{(1+g)}{(1-g)}\right]\left[1-\left(\frac{(1+g)}{(1+i)}\right)^{Y}\right]
\]

(4.9.2-1)

where:

\(P_{W}\) = present worth of the disruption cost,
\(AF\) = annual frequency of bridge collapse,
\(DC\) = disruption cost associated with bridge collapse,
\(g\) = real annual rate of growth of disruption costs (as a decimal, 2 percent/yr = 0.02),
\(i\) = discount rate (as a decimal, 4 percent/yr = 0.04), and
\(Y\) = design life of the bridge (years).

In addition to the benefit/cost (B/C) ratio, other measures of cost-effectiveness may also be included in the economic analysis such as, net present value (NPV), payback period, and rate of return (ROR). Cost-effectiveness of a protection system is indicated by a B/C ratio greater than 1.0, a NPV greater than zero, a payback period which occurs during the useful life of the project, or a ROR greater than the discount rate.

### 4.9.3 Disruption Cost

The disruption cost associated with bridge collapse can be computed as:

\[
DC = PRC + SRC + MIC + PIC
\]

(4.9.3-1)

where:

\(DC\) = disruption cost,
\(PRC\) = pier replacement cost,
\(SRC\) = span replacement cost,
\(MIC\) = motorist inconvenience cost, and
\(PIC\) = port interruption cost.

Additional costs such as environmental, business, social, and loss of life costs may often be incurred in a catastrophic bridge collapse. Since these costs are

C4.9.3

The disruption cost, \(DC\), determined in accordance with Eq. 1 of the Guide Specifications, represents the estimated losses associated with the collapse of a bridge due to vessel collision. Evaluating the cost factors in Eq. 1 requires the establishment of accident scenarios for each pier or span element of the bridge risk analysis. For each pier or span element which collapses as a result of a vessel collision, it must be determined which adjacent pier or span elements would also be destroyed or damaged. The level of damage to bridge elements located away from the immediate area of vessel impact is primarily a function of the structure type and continuity.

As an example, for some types of long span bridges, the loss of the anchor pier would be sufficient to cause severe damage and collapse of the entire main span unit. When computing the disruption cost of the collapse of such an anchor pier, the cost and losses associated with the entire main span unit would be required. Table C1 illustrates the estimated disruption cost associated with
usually subjective and therefore difficult to estimate, they are normally not included in computing DC.

Pier replacement costs (PRC) and span replacement costs (SRC) are those costs associated with the replacement of bridge piers and spans damaged by a given accident. For each pier and span component, an estimate of PRC and SRC shall be made including the damage caused to adjacent piers and spans caused by the collapsed bridge element. For bridges with a high level of continuity, damage to one pier/span component may require the repair/replacement of portions of the structure located relatively far away from the collapse location. An estimate of the length of bridge outage required to repair or replace the damaged structure must be made for each pier/span component.

Motorist inconvenience costs (MIC) include costs incurred by motorists who would be forced to use a detour route for the period of bridge outage. For toll bridges, it also includes revenues lost by the owner. Estimates of MIC require identification of detour routes, collection of traffic volume data, and calculation of incremental vehicle operating costs using prescribed AASHTO standard methodologies. In some cases, the MIC costs can be quite large—particularly if there is no nearby alternative route, or if the bridge repair time is lengthy.

Port interruption costs (PIC) include costs associated with the temporary closure of port facilities caused by bridge debris in the navigable ship/barge channel. Interruption of port commerce in a busy U.S. waterway for even a short period of time can cause very large disruption costs. The computation of port interruption costs requires knowledge of merchant shipping operation limitations, marine transport cost structures, cargo values and the capabilities of alternative port facilities. Factors to be included in estimating PIC are:

- The duration of navigable channel blockage (how long it would take to clear wreckage and reopen the channel);
- The number of vessels carrying cargoes that would be delayed or trapped due to the bridge collapse, and for what length of time;
- Cargoes that would be foregone (rerouted to other ports, or shipped by alternative modes); and
- Opportunities that may exist for establishing a temporary channel under adjacent undamaged spans of the bridge, and if so, which vessels could and would use such a channel.

The discount rate, \( i \), is used to bring back future costs and benefits to present value. For future costs and benefits calculated in constant dollars, only the real cost of capital should be represented in the discount rate.

The rate of growth of disruption costs, \( g \), accounts for increasing disruption costs over time due to increasing the collapse of one of the main piers of the Dame Point Bridge (in Florida), a cable-stayed structure with a 1,300-ft main span (Greiner Engineering Sciences, Inc., July 1984).

<table>
<thead>
<tr>
<th>Item</th>
<th>Year—1987</th>
<th>Year—2037</th>
</tr>
</thead>
<tbody>
<tr>
<td>PRC</td>
<td>$8,948,000</td>
<td>$8,948,000</td>
</tr>
<tr>
<td>SRC</td>
<td>27,038,000</td>
<td>27,038,000</td>
</tr>
<tr>
<td>PIC</td>
<td>21,000,000</td>
<td>21,000,000</td>
</tr>
<tr>
<td>MIC</td>
<td>75,810,000</td>
<td>375,480,000</td>
</tr>
<tr>
<td>DC</td>
<td>$132,796,000</td>
<td>$432,466,000</td>
</tr>
</tbody>
</table>

The pier and span replacement costs (PRC and SRC) should be based on estimates of the costs to rebuild the bridge components which would be destroyed in the accident scenario. Included in PRC and SRC should be the costs associated with debris removal from the waterway, and engineering and construction inspection costs.

The disruption cost must include any motorist inconvenience costs, MIC, which may occur with bridge outage. In some cases, these costs can be quite large, particularly if there is no nearby alternative route or if the repair time is lengthy. The detour costs are typically found in two main categories: 1) additional vehicle operating costs incurred by motorists who must take a longer, more congested, or less efficient route; and 2) lost revenues for toll bridges. Estimates of MIC require identification of detour routes, collection of traffic volume data, and calculation of incremental vehicle operating costs, using standard methodologies prescribed by AASHTO (1977). Future growth in motorist traffic must be considered in the analysis since it can have a significant impact on the disruption cost as illustrated in Table C1.

Another factor in Eq. 1 for which a detailed accident scenario is required is the port interruption cost, PIC. The importance of a major seaport's contribution to the regional economy is well documented. In terms of jobs and income created in direct, indirect, and port related industries, the average U.S. seaport can be found to add nearly a billion dollars per year to the economy of its region. An interruption of port commerce such as would occur with bridge wreckage in a navigable channel can create an enormously adverse economic impact.

The key factors to be considered in the estimation of PIC are discussed in this Article. The establishment of the port interruption scenario requires an understanding of merchant shipping operation limitations, marine transport cost structures, cargo values, capabilities of alternative port facilities, and several other factors. Even at that, there are some costs which are certain in principle to occur, but which are not easily quantified. Therefore, the value of PIC should always be conservative in the analysis.
ing vessel traffic under the bridge due to port growth, and to increasing motorist traffic on the bridge due to growth in the community. The influence on $g$ for motorist traffic can be computed using future ADT volumes estimated for the bridge. The influence due to port growth can be estimated based on historical long-term port growth for the waterway, or from other procedures.

Other costs which are not easily quantified include environmental, business, social, and loss of life costs. Since subjective value judgments lead to widely differing costs for these categories, they are usually not directly included in the disruption cost analysis. For these disruption categories, qualitative consideration and judgment must be exercised to include these concerns in the decision-making process.
REFERENCES


IABSE. 1983. IABSE Colloquium on Ship Collision with Bridges and Offshore Structures. 3 Vols. (Introductory, Preliminary, and Final Reports). International Association for Bridge and Structural Engineering, Copenhagen, Denmark.


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8.1 GENERAL

This Section provides general guidelines for planning a new bridge crossing a navigable waterway.

These guidelines are based on historical bridge accident data and represent recommendations from the viewpoint of minimizing vessel collision with bridges only. Other constraints, including costs, roadway geometry and alignment, and environmental impacts, may result in different bridge geometries than those recommended in this Section.

C8.1

The planning data for new bridges in Section 8 of the Guide provides guidance to the bridge designer based on historical accident data and experience. Judgment must be exercised in determining the appropriate use of the guidelines and their application to a particular bridge site.

In general, the use of the Guide Specifications requirements and planning guidelines will result in relatively long span and high clearance bridges to reduce the risk and consequences of a vessel collision. Using cost-effectiveness techniques, the higher cost of longer span bridges with a lower present worth of avoidable disruption costs, must be balanced against the lower cost of shorter span bridges with a higher present worth of avoidable disruption costs. The minimization of the sum of the cost of bridge protection and the present value of the avoidable disruption cost is one method of providing an optimal bridge solution for vessel collision as described by Sexsmith (1983).

The geometry and water depths of the waterway are a significant planning consideration for bridges. Water depths may be such that vessels cannot impact piers beyond the navigation channel without running aground; therefore, shorter approach spans could be used that otherwise may not have been advisable.

The horizontal span clearance data in Article 8.5.1 was developed primarily from studies performed by Shoji and Iwai (1985), and Shoji and Wakao (1986). Figure C8.5.1-1 shows the relation between ship length, LOA, and main span length, S, for actual ship/bridge accident data. From Figure C8.5.1-1 it can be seen that bridges with main spans less than approximately 300 feet are relatively vulnerable to collision by even small ships. The relationship between the colliding ship's size, DWT, and the main span, S, for bridge accidents is shown in Figure C8.5.1-2. From Figure C8.5.1-1, it can be seen that the probability of ship collision with the bridge is increased when the main span is less than two or three times the ship length. The bridge accident data included in Figure C8.5.1-2 is shown in Table C8.5.1-1 (Shoji and Wakao, 1986).

Research reported by Shoji and Iwai (1985) indicates that environmental conditions of current and wind can be a major indirect cause of vessel accidents for bridges if the main piers are located near the edge of the navigable channel within a distance less than two or three times the width of the pier. This is caused by the flow of the current or wind which must curve around the pier. These curved flowlines can induce transverse forces on a passing vessel causing it to deviate from its original course.
8.2 LOCATION OF CROSSING

The location of a bridge crossing a navigable waterway is a key factor in determining the risk of vessel collisions.

To the extent possible, bridges should be located in straight regions of the navigable waterway and away from bends and turns. Bridges located near or in turns/bends will have a higher probability of vessel collision as discussed in Article 4.8.3.2.

8.3 BRIDGE ALIGNMENT

Bridges crossing navigable waterways should be aligned perpendicular to the direction of vessel traffic passing through the bridge and perpendicular to the direction of current flow wherever possible. Skewed bridge alignments, and those which are located in regions where crossovertants exist, have a higher risk of vessel collision.

8.4 TYPE OF BRIDGE

The type of bridge crossing a navigable waterway should be selected to minimize the risk of vessel collision in accordance with the requirements of these Guide Specifications.

The primary area of vessel collision risk to the bridge is the region near the navigable waterway as modeled by the normal distribution discussed in Article 4.8.3.3. Within this area (3 × LOA on each side of the inbound and outbound centerline of vessel transit paths), the bridge type should be developed to minimize the number of piers supporting the superstructure, and to maximize the horizontal clearances between piers and the vertical clearance to the superstructure.

8.5 NAVIGATION SPAN CLEARANCES

8.5.1 Horizontal Clearances

Figure 1 depicts the typical relationship between a vessel transiting the waterway and the main (navigation) span of a bridge. Using historical vessel collision data, the following guidelines for planning the navigation span of a new bridge have been developed:

- Bridges with main spans, S, less than two or three times the design vessel length, LOA, are particularly vulnerable to vessel collision.
- Bridges with main spans, S, less than two times the channel width, C, are particularly vulnerable to vessel collision.
- Piers located less than two or three times the pier width, Bp, from the edge of channel, Yv and Yw, are particularly vulnerable to collision.

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- The centerline of the navigable channel should coincide with the centerline of the main span. The maximum offset between the centerline of the channel and main span length, $S$, of the bridge should not exceed 10–15 percent.

![Diagram showing Bridge/Waterway Planning Geometry]

**Figure 8.5.1-1—Bridge/Waterway Planning Geometry**

**Table C8.5.1-1—Main Span Versus LOA for Historical Bridge Collisions (Shoji and Walko, 1986)**

<table>
<thead>
<tr>
<th>Date of Accident</th>
<th>Bridge Name</th>
<th>Location</th>
<th>Main Span (ft)</th>
<th>LOA (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1963</td>
<td>Sorsund</td>
<td>Norway</td>
<td>328</td>
<td>354</td>
</tr>
<tr>
<td>1972</td>
<td>Sidney Lanier</td>
<td>USA</td>
<td>246</td>
<td>571</td>
</tr>
<tr>
<td>1975</td>
<td>Fraser</td>
<td>Canada</td>
<td>384</td>
<td>656</td>
</tr>
<tr>
<td>1977</td>
<td>Benjamin Harrison</td>
<td>USA</td>
<td>236</td>
<td>613</td>
</tr>
<tr>
<td>1977</td>
<td>Tromso</td>
<td>Norway</td>
<td>262</td>
<td>134</td>
</tr>
<tr>
<td>1979</td>
<td>Second Narrows RR</td>
<td>Canada</td>
<td>498</td>
<td>574</td>
</tr>
<tr>
<td>1980</td>
<td>Almo (Tjorn)</td>
<td>Sweden</td>
<td>912</td>
<td>564</td>
</tr>
<tr>
<td>1980</td>
<td>Sunshine Skyway</td>
<td>USA</td>
<td>860</td>
<td>610</td>
</tr>
<tr>
<td>1981</td>
<td>Jordfallet</td>
<td>Sweden</td>
<td>144</td>
<td>157</td>
</tr>
</tbody>
</table>
Figure C8.5.1-1—Colliding Ship's LOA Versus Main Span of Bridge(s) (Shoji and Wakao, 1986)

Figure C8.5.1-2—Colliding Ship's Size (DWT) Versus Main Span of Bridge(s) (Shoji and Iwai)
8.5.2 Vertical Clearances

Vertical clearances for a proposed bridge should be established to permit the passage of the vessel using the waterway with the highest vertical clearance requirements traveling in a ballasted condition at periods of high water levels. The vertical clearance requirements shall be established from data on the actual and proposed vessels using the waterway, and through coordination with the U.S. Coast Guard. Typical vertical clearance heights for ship mast and deckhouses are shown in Figures 3.5.2-5 and 3.5.2-6. Typical vertical clearance heights for a ship's bow can be determined from the bow height and draft data in Figure 3.5.2-4 and Tables 3.5.2-1, 3.5.2-2, and 3.5.2-3.

8.6 APPROACH SPANS

Approach spans and their supporting piers should be established using the requirements of these Guide Specifications. Based on historical ship collision data, twice as many accidents have occurred with approach piers as have occurred with the main piers and navigation spans.

The use of the Guide Specification criteria will usually result in an increase in approach span lengths in order to minimize the number of piers located in the central area of vessel collision vulnerability.

8.7 PROTECTION SYSTEMS

The cost associated with protecting a bridge from catastrophic vessel collision can be a significant portion of the total bridge cost and must be included as one of the key elements in establishing a bridge's type, size, location, and geometry.

The following protection alternatives should be evaluated in order to develop a cost-effective solution to a new bridge project:

- Design the bridge piers, foundations, and superstructure such that the vessel collision impact force and energy can be withstood.
- Design a pier fender system to reduce the impact force and energy to a level below the capacity of the pier and foundation.
- Locate piers in shallow water, out-of-reach from large vessels in order to reduce the magnitude of the impact force and energy for design of the pier.
- Protect piers from vessel collision by means of protective islands, dolphins, or other structures which are designed to redirect, withstand, or absorb the design impact force and energy.

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8.8 PLANNING PROCESS

Vessel collision with highway bridges crossing navigable waterways is only one of a multitude of factors involved in the planning process for a new bridge. The designer must balance a variety of needs including political, social, and economic in arriving at an optimal bridge solution for a new crossing. Depending on the waterway characteristics and the type and frequency of motorist and merchant vessels using and passing under the bridge, the vessel collision factor may range from insignificant to very significant in the bridge planning process.

8.8.1 Route Location Study

The potential for vessel collision should be a key factor in performing route location studies for a new bridge crossing. The guidelines for crossing location and bridge alignment in Articles 8.2 and 8.3 should be followed to the extent possible.

8.8.2 Bridge Type, Size, and Location Study

For a given route across the waterway, a bridge type, size, and location (T, S, and L) study should be performed which includes a detailed evaluation of the potential for vessel collision with the structure. Alternative bridge types, sizes, and geometrics should be evaluated based on planning, engineering, and economic factors. The provisions of Section 8 and the flow charts in Section 1, provide a basis of incorporating the vessel collision loads in evaluating alternative bridge configurations in the T, S, and L study. All of the basic decisions regarding bridge type, layout, clearances, pier locations, design loads, and bridge protection method should be determined during the T, S, and L study and before detailed preliminary and final design of the structure begins.

The goal in the T, S, and L study is to develop the least cost total structure—including protection costs. After the development of a protection system for a particular bridge, a comparison should be made between the total cost of the proposed alignment and span lengths with protection, to the total cost of an alternate structure with revised bridge characteristics (i.e., longer spans, stronger piers, alternate alignment, etc.). The methodology is an iterative process using the flow chart procedures in Article 1.5.

C8.8

The AASHTO Guide Specifications requirements for vessel collision design have provided a rational methodology for determining the risk of vessel collision with bridges, and for the development of structures with improved resistance to catastrophic collapse due to ship and barge impacts. Currently, Bridge Engineers’ experience using the AASHTO vessel collision codes has been reasonably positive.

It is recognized that vessel collision is but one of a multitude of factors involved in the planning process for a new bridge. The designer must balance a variety of needs including political, social, and economic in arriving at an optimal bridge solution for a proposed highway crossing. Due to the relatively high bridge costs associated with vessel collision design for most waterway crossings, it is important that additional research be conducted to improve our understanding of vessel impact mechanics, the response of the structure, and the development of cost-effective protection systems.
8.8.3 Preliminary and Final Design

The preliminary and final design phases are performed based on the design criteria of the selected structure established during the T, S, and L study. The impact forces determined in Sections 3 and 4 are applied to the bridge structure or protection system (Section 7) as one of the AASHTO group loading conditions. Minor refinement of the design criteria and the use of model studies to evaluate design assumptions usually occurs in the preliminary design phase of the project.
REFERENCES

