Guide Specification and Commentary for Vessel Collision Design of Highway Bridges

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PREFACE

This document contains specifications for the vessel collision design of highway bridges. The specifications are the recommendations of a team of internationally recognized experts, composed of consulting engineers, state highway engineers and federal agency representatives from throughout the United States. The specifications are comprehensive in nature and embody new concepts which have not been included in previous design provisions. They are based on both the observed performance of bridges during past vessel collisions and on recent research conducted in the United States and abroad. A commentary documenting the basis for the specifications and examples illustrating their use are included.

This document was prepared for the Federal Highway Administration (FHWA), Office of Research, Structures Division, under pooled fund contract DTFH61-88-C-00011 and was sponsored by eleven (11) States. The Specifications were developed by Greiner, Inc., Irving, Texas, a consulting engineering firm under contract to the FHWA. The principal investigator for Greiner was Mr. Michael A. Knott. Subconsultants to Greiner were Cowiconsult Ltd., Copenhagen, Denmark (Mr. Ole Damgaard Larsen); Rowe Research and Engineering, Inc., Alexandria, Virginia (Dr. William Rowe); and Mr. Gerhard Woisin, a consulting naval architect from Hamburg, West Germany.

To ensure representative input and adequate consideration of the many factors involved, two technical committees were established to review and comment on the Specification as it was being developed.

The first review committee consisted of FHWA representative Mr. Eric Munley, and members from each of the pooled-fund sponsoring states as listed below:

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SECTION 1
INTRODUCTION

1.1 PURPOSE

This Guide Specification establishes the design provisions for bridges crossing navigable waterways to minimize their susceptibility to damage from vessel collisions. The purpose of the provisions is to provide bridge components with a reasonable resistance capacity against vessel collision. The provisions specified are design values only and should not be interpreted as covering all conceivable cases of vessel collision.

1.2 BACKGROUND

The 1980 collapse of the Sunshine Skyway Bridge crossing Tampa Bay in Florida was a major turning point in the development of vessel collision design criteria for bridges in the United States. As a result of the collision by an empty 35,000 DWT bulk carrier with one of the bridge's anchor piers, 1,300 feet of the southbound main span collapsed and 35 lives were lost in vehicles which fell into the bay.

In the period 1965-1989, an average of one catastrophic accident per year involving bridge collisions by merchant vessels have been recorded worldwide. More than 100 persons died in these accidents and very large economic losses were incurred in repair/replacement costs, lost transportation service, and other damages. More than half of these bridge collisions occurred in the United States.

As a result of these accidents, increased concern over the safety of bridges crossing navigable waterways has arisen and research into the vessel collision problem has been initiated in several countries of the world. In 1983, a "Committee of Ship/Bridge Collisions" appointed by the Marine Board of the National Research Council, Washington, D.C. examined the risks and consequences of ship and barge collisions with bridges in the United States. Included in this committee's report were the following observations:

- No agency or unit of government is responsible for the safety of over water bridges against ship collisions.
- No standards have been developed for the design and construction of bridges to resist ship collisions (with the exception of criteria for fenders to protect railroad bridges).
- Regulatory and institutional activities address parts of the ship-bridge-waterway system, but none addresses the functioning of the system as a whole.

In 1988, a pooled-fund research project sponsored by 11 states and administered by the Federal Highway Administration (FHWA) was initiated to begin addressing the above concerns by establishing a design specification for ship and barge collisions with highway bridges crossing navigable waterways. The basis of the project was the published literature from the "1983 Colloquium on Ship Collisions with Bridges and Offshore Structures" held in Copenhagen, Denmark by the International Association of Bridge and Structural Engineers (IABSE), and the results of in-depth ship collision studies performed for several bridge projects by consultants worldwide.

1.3 BASIC CONCEPTS

Development of the Specification 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
- 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 as compliance with the specifications.

1.5 FLOW CHARTS

Flow charts outlining the basic steps in the vessel collision design procedures are given in Figures 1.5-1 and 1.5-2 for evaluating bridges. Method II shall be used for all bridge analysis unless the special situations presented in Section 4.1 exist, in which case Method I or III as appropriate may be used.

The Commentary provides background information and examples to assist the user in understanding the intent of the Specification.

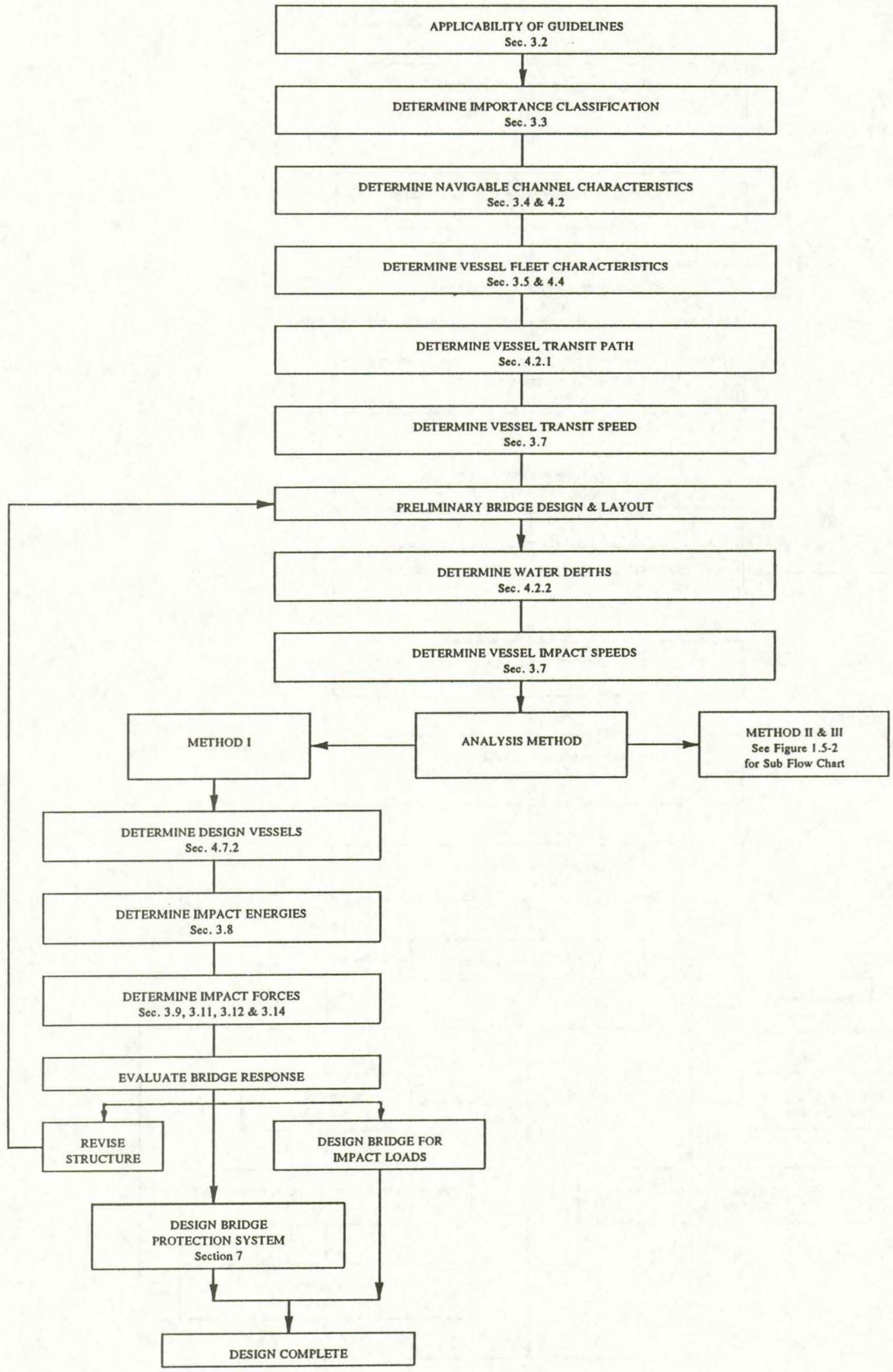


Figure 1.5-1. Design Procedure Flow Chart.

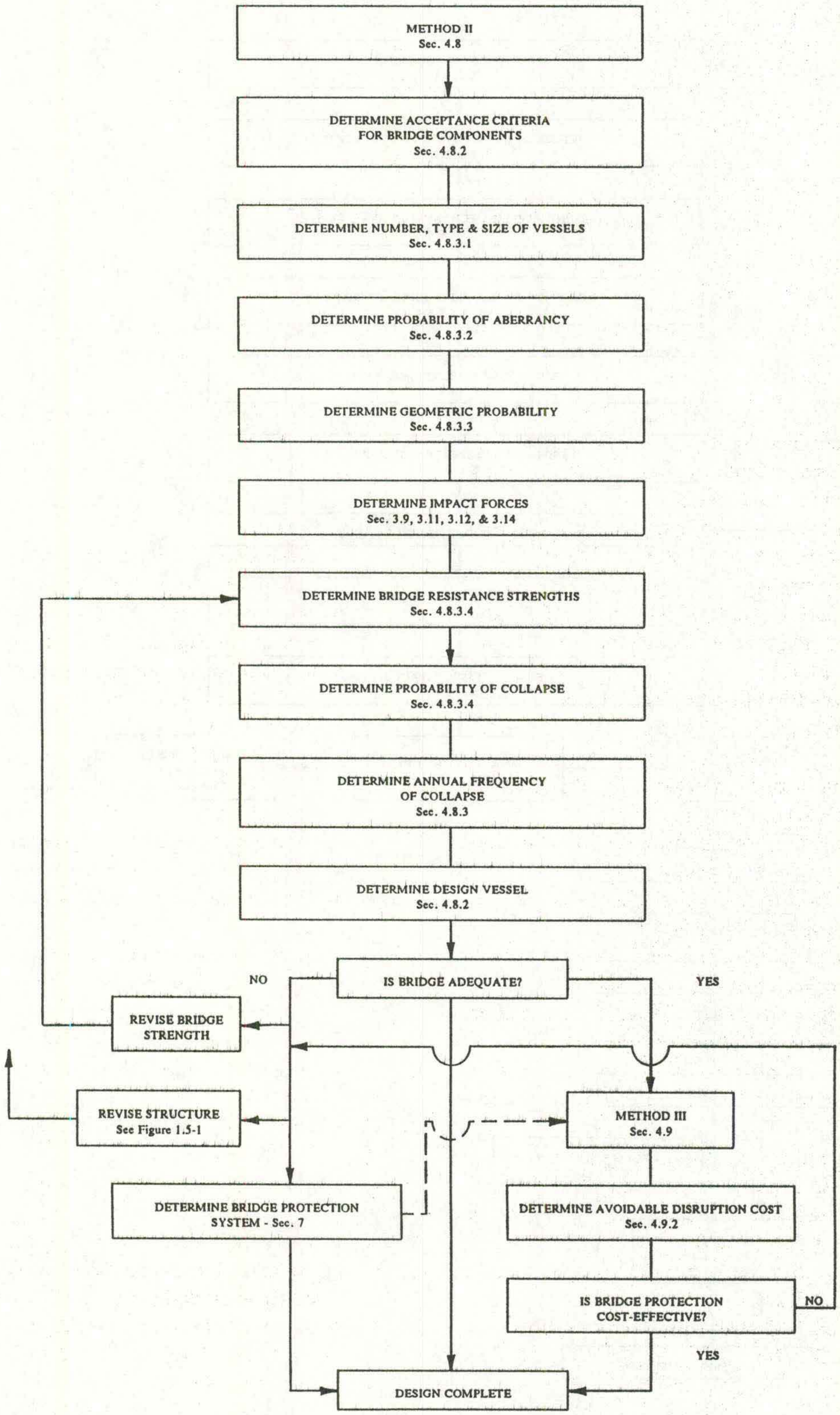


Figure 1.5-2. Sub Flow Chart for Methods II and III.

SECTION 2

SYMBOLS AND DEFINITIONS

The following symbols and definitions apply to the Guide Specification.

a_B = bow damage depth of standard hopper barge as determined by Equation 3.13-1 (feet)

a_S = bow damage depth of ship as determined by Equation 3.10-1 (feet)

AF = annual frequency of bridge element collapse defined in Section 4.8.3 (number of collapses/year)

B = loads resulting from buoyancy forces and used in the group load combination of Equation 3.14-1

B_M = beam (width) of barge, barge tows, and ship vessels used in Sections 3.5 and 4.8.3.3 (feet)

B_P = width of bridge pier used in Figures 4.8.3.3-1 and 8.5.1-1 (feet)

BR = base rate of vessel aberrancy defined in Section 4.8.3.2 (dimensionless)

C = channel width as shown on Figures 4.2.1-1, 4.2.1-2, and 8.5.1-1 (feet)

C_C = size of barge based on cargo capacity as defined in Section 3.5 (1 ton = 2,000 pounds)

C_H = hydrodynamic mass coefficient defined in Equation 3.8-1 (dimensionless)

D = loads resulting from dead load and used in the group load combination of Equation 3.14-1

D_B = bow depth of a ship or barge vessel as shown in Figures 3.5.1-1 and 3.5.2-3 (feet)

D_E = mean draft of a ballasted vessel as shown in Figure 3.5.2-4 (feet)

D_{ES} = draft of ballasted ship stern as shown in Figure 3.5.2-4

D_{EB} = draft of ballasted ship bow as shown in Figure 3.5.2-4

D_L = mean draft of a fully loaded vessel as shown in Figure 3.5.2-4 (feet)

DA = avoidable bridge collapse disruption cost as defined in Section 4.9.2 ($)

DC = bridge collapse disruption cost as defined in Section 4.9.3 ($)

DWT = size of vessel based on deadweight tonnage as defined in Section 3.5 (one tonne = 2,205 pounds)

E = loads resulting from earth pressure and used in the group load combination of Equation 3.14-1

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

g = real annual rate of growth of disruption costs as used in Equation 4.9.2-1 (rate/year)

H = ultimate bridge element strength as defined in Section 4.8.3.4 (kips)

H_L = depth of barge head-log on its bow as shown in Figure 3.5.1-1 (feet)

H_P = ultimate bridge pier resistance strength as defined in Section 4.8.3.4 (kips)

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

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

KE = design impact energy of vessel collision as defined in Equation 3.8-1 (kip-feet)

L_{CB} = length from bow to collision bulkhead for ships as defined in Figure 3.5.2-3 (feet)

L_B = length of individual barge as shown in Figure 3.5.1-1 (feet)

L_P = length of bridge pier as defined in Figure 8.5.1-1 (feet)

LOA = length overall of ship or barge tow as shown in Figures 3.5.1-2 and 3.5.2-4 (feet)

MHW = mean high water level of waterway (feet)

MIC = motorist inconvenience cost due to bridge collapse as defined in Section 4.9.3 ($)

N = number of one-way passages of vessels transiting through the bridge as defined in Section 4.8.3 (number/year)

P = loads resulting from vessel impact and used in the group load combination of Equation 3.14-1

P_B = barge collision impact force for head-on collision between barge bow and a rigid object as defined in Section 3.12 (kips)

P_{BH} = ship collision impact force between ship bow and a rigid superstructure as defined in Equation 3.11.1-1 (kips)

P_{DH} = ship collision impact force between ship deckhouse and a rigid superstructure as defined in Equation 3.11.2-1 (kips)

P_{MT} = ship collision impact force between ship mast and a rigid superstructure as defined in Equation 3.11.3-1 (kips)

P_S = ship collision impact force for head-on collision between ship bow and a rigid object as defined in Equation 3.9-1 (kips)

PA = probability of vessel aberrancy as defined in Section 4.8.3.2 (dimensionless)

PC = probability of bridge collapse as determined in Section 4.8.3.4 (dimensionless)

PG = geometric probability of vessel collision with bridge pier/span as determined in Section 4.8.3.3 (dimensionless)

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

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

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

R_B = PA correction factor for bridge location as defined in Equation 4.8.3.2-2 (dimensionless)

R_C = PA correction factor for currents parallel to vessel transit path as defined in Equation 4.8.3.2-3 (dimensionless)

R_D = PA correction factor for vessel traffic density as defined in Section 4.8.3.2 (dimensionless)

R_L = rake length of vessel bows as shown in Figures 3.5.1-1 and 3.5.2-3 (feet)

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

R_{DH} = reduction factor for ship deckhouse collision force as defined in Section 3.11.2 (dimensionless)

R_{XC} = PA correction factor for crosscurrents acting perpendicular to vessel transit path as defined in Equation 4.8.3.2-4 (dimensionless)

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

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

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

V = design impact speed of vessel as determined in Section 3.7 (ft/sec)

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

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

V_{XC} = 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 Equation C3.8-1 (tonnes, 1 tonne = 2,205 pounds)

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

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

x = distance to bridge element from the centerline of vessel transit path as shown in Figure 3.7-1 and Figure 4.8.3.3-1 (feet)

x = deflection of protection structure due to vessel impact as defined in Equation 7.3-1 (feet)

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

x_L = distance equal to 3xLOA from centerline of vessel transit path as shown in Figure 3.7-1 (feet)

Y_N = distance from pier centerline to edge of outbound channel as shown in Figure 8.5.1-1 (feet)

Y_W = distance from pier centerline to edge of inbound channel as shown in Figure 8.5.1-1 (feet)

Y_P = offset distance from edge of foundation to pier column as shown in Figures 3.15.1-2 and 3.15.1-3 (feet)

θ = angle of channel turn or bend as shown in Figure 4.8.3.2-1 (degrees)

ϕ = angle between channel and bridge centerlines as shown in Figures 4.8.3.3-1 and 8.5.1-1 (degrees)

σ = standard deviation of normal distribution as defined in Section 4.8.3.3

SECTION 3
GENERAL PROVISIONS

3.1 GENERAL

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 in accordance with the Guide Specification criteria.

3.2 APPLICABILITY OF SPECIFICATION

These specifications are for the design of new bridges and for the evaluation of existing bridges to resist the effect of collision impacts from merchant vessels.

The specifications apply to all bridge types which cross a navigable shallow draft inland waterway with barge traffic, and deep draft waterways with large merchant ships. The provisions are applicable to normal merchant vesseis, either steel hulled ship or barge vessels.

The specifications are not applicable to special purpose vessels, wood, or fiberglass constructed vessels, ships smaller than 1,000 DWT, naval vessels, nor to recreational vessels. Vessel impact requirements for bridges located in waterways characterized by significant usage of these special vessels shall be established by the bridge Owner.

The specifications apply to bridges crossing waterways which have defined navigation channels as established by federal or state agencies. Judgment must be used when applying the Guide Specification criterion to waterways in which no defined navigation channel exists.

The provisions specified in the Specification are minimum requirements.

3.3 IMPORTANCE CLASSIFICATION

An Importance Classification (IC) shall be assigned for all bridges crossing a navigable waterway for purposes of determining the risk acceptance and subsequent design vessels in Sections 4.7.2 and 4.8.2 as follows:

1) CRITICAL BRIDGES
2) REGULAR BRIDGES

Bridges shall be classified on the basis of Social/Survival and Security/Defense requirements.

Critical bridges are those that must continue to function after impact from a design vessel whose probability of occurrence is smaller than other, regular, bridges. The Social/Survival evaluation is primarily concerned with the need for civil defense, police, fire department, or public health agencies to respond to an emergency situation which might exist on the opposite side of the waterway. Bridges which provide the only continuous transportation route for such emergency situations should be classified as critical. Bridges which serve as important links in the Security/Defense roadway network based on the 1973 Federal-Aid Highway Act should be classified as critical. Additional guidelines for determining the importance classification are provided in the Commentary.

3.4 DATA COLLECTION

Data shall be collected as appropriate for the analysis method utilized for the bridge design. Essential data includes description of the vessel traffic passing under the bridge, vessel transit speeds, vessel loading characteristics, waterway and navigable channel geometry, water depths, environmental conditions, and bridge geometry. Data sources are presented in the Commentary.

3.5 VESSEL TYPE AND CHARACTERISTICS

The vessel types using the waterway shall be determined. Vessels shall be classified as either, 1) inland waterway barges, or 2) ships. The first category includes barge vessels using shallow draft inland waterways, including the tugs and tows which push/pull them. The second category includes ships which use deep draft waterways.

Ship size shall be determined based on the vessel's deadweight tonnage (DWT). DWT is the weight of cargo that the vessel can carry when fully loaded (1 tonne = 2,205 pounds).

The relations between DWT and other units of measurements of vessel size which might be encountered by the designer such as Gross Registered Tonnage (GRT), Net Registered Tonnage (NRT), and Displacement Tonnage (W) are explained in the Commentary.

Barge size shall be determined based on the vessel's cargo carrying capacity (C_C) in tons (1 ton = 2,000 pounds).

3.5.1 Barge Vessels

Typical inland waterway barge and towboat characteristics are shown in Tables 3.5.1-1 and 3.5.1-2. Figure 3.5.1-1 shows detailed characteristics of three (3) common barge types using the inland waterways. Variations from these typical dimensions exist and the designer should verify the applicability of the data for the specific waterway and bridge location being evaluated.

Barges are often towed (pushed) in groups of two or more; therefore, their dimensions and drafts tend to be standard in order to provide hydrodynamic efficiency. In addition, standardized barge dimensions facilitate the establishment of tow configurations through locks on river systems. Typical tow configurations are shown in Figure 3.5.1-2.

3.5.2 Ship Vessels

Ship characteristics vary considerably depending on the size, draft, and type of cargo being carried by the vessel. Figure 3.5.2-1 contains three broad classes of ships typical of U.S. waterways. The ship classes include bulk carriers, product carriers/tankers, and freighter/container vessels. Figures 3.5.2-2 thru 3.5.2-6 and Tables 3.5.2-1, 3.5.2-2, and 3.5.2-3 contain data on typical sizes, dimensions, drafts, bow shapes, and vertical clearances for these ship classes. Variations from these typical dimensions exist and the designer should verify the applicability of the data for the specific waterway and bridge location being evaluated. In particular the designer should determine if partly loaded vessels due to channel depth restrictions, use the waterway.

3.5.3 Special Vessels

A variety of special ship and barge vessels transit U.S. waterways. These include ocean-going barges, dredges, offshore industry transports, jack-up boring rigs, barge mounted cranes, passenger ships, LASH vessels, Liquefied Natural Gas (LNG) vessels, and naval vessels. The applicability of the Specification is limited with respect to these vessel types and judgment must be exercised in evaluating their influence on the vessel collision problem.

3.6 DESIGN VESSEL

A design vessel shall be determined for each bridge element exposed to collision. The design vessel shall be selected in accordance with the requirements of Section 4.1 using Method II and its corresponding acceptance criteria in Section 4.8.2 unless the approval of the Owner and the special situations stated in Section 4.1.2 exist.

3.7 DESIGN IMPACT SPEED

The design impact speed for each exposed bridge element in the waterway shall be determined based on the typical vessel transit speed within the navigable channel limits, the distance to the location of the bridge element from the centerline of vessel transit path, and the vessel length overall (LOA). The centerline of vessel transit path shall be determined according to Section 4.2.1. The typical vessel transit speed (V_T) shall represent the typical speed at which the design vessel is transiting the waterway in the vicinity of the bridge under normal environmental circumstances. A different transit speed for inbound and outbound vessels may be required depending on water current conditions in the waterway.

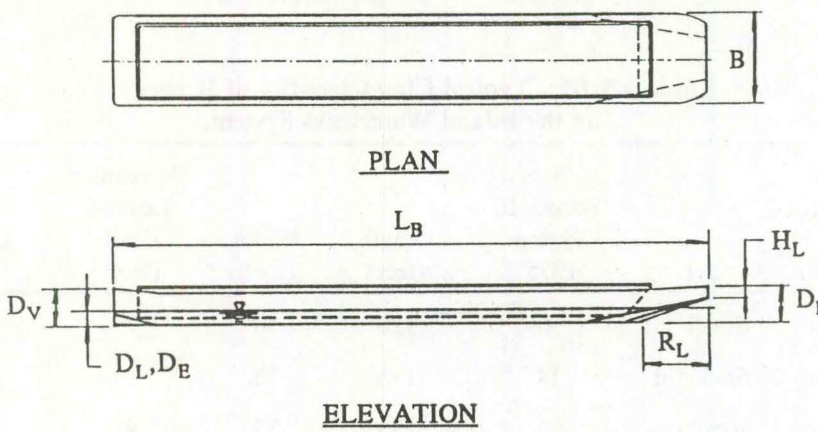
The design impact speed for each bridge element in the waterway shall be determined as shown in Figure 3.7-1. The design impact speed distribution shall be based on the geometry of the bridge, the navigable channel width, and the length overall (LOA) of a vessel selected in accordance with the Method I criteria in Section 4.7.2. Once determined, the LOA dimension for impact speed distribution shall be considered a constant and applicable to Method I, II, and III procedures. For barge tows, the length overall shall be equal to the total length of the tow including the barges and tug/tow boat vessel as shown in Figure 3.5.1-2.

Table 3.5.1-1. Typical Characteristics of Barges on the Inland Waterways System.

Barge Type	Size	% of Barges In System 1975	Length (feet)	Width (feet)	Maximum Loaded Draft (feet)	Capacity (tons)
Open Hopper	Small	6	120	30	7	630
Open Hopper	Standard	14	175	26	9	1,060
Open Hopper	Jumbo	27	195	35	9	1,700
Open Hopper	Oversize	1	245	35	10	2,400
Covr'd Hopper	Jumbo	22	195	35	9	1,700
Deck Barge	Small	10	100/150	26/32	6	350/600
Deck Barge	Jumbo	2	195	35	9	1,700
Deck Barge	Oversize	2	200	50	9	2,050
Tank Barge	Small	3	135	40	9	1,300
Tank Barge	Jumbo	4	195	35	9	1,700
Tank Barge	Oversize	9	185/290	53	9	2,530/3,740

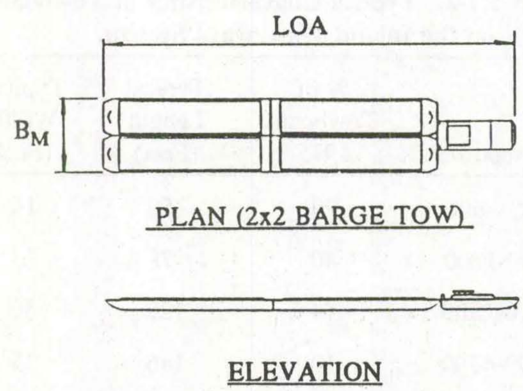
Table 3.5.1-2. Typical Characteristics of Towboats on the Inland Waterways System.

Towboat Type	Horsepower	% of Towboats 1975	Typical Length (Feet)	Typical Width (Feet)	Typical Draft (Feet)
Harbor Boat	< 600	29	50	16	4-1/2
Line Haul	600-1200	40	78	23	7
Line Haul	1200-2500	14	120	30	9
Line Haul	2500-4300	10	146	35	9
Line Haul	4300-8400	7	160	45	9
Line Haul	> 8,400	1	185	55	9



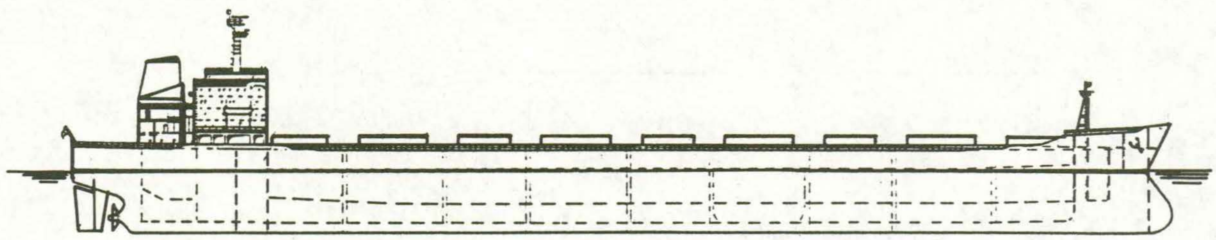
			Jumbo Hopper	Oversize Tank	Special Deck
L_B	=	length (feet)	195	290	250
B_M	=	width (feet)	35	53	72
D_V	=	depth of vessel (feet)	12	12	17
D_E	=	empty [light] draft (feet)	1.7	1.7	2.5
D_L	=	loaded draft (feet)	8.7	8.7	12.5
D_B	=	depth of bow (feet)	13	13	18
R_L	=	bow rake length	20	25	30
H_L	=	head log height (feet)	2-3	2-3	3-5
C_C	=	cargo capacity (tons)	1700	3700	5000
W_E	=	empth displacement (tons)	200	600	1300
W_L	=	loaded displacement (tons)	1900	4300	6300

Figure 3.5.1-1. Typical Barge Characteristics.

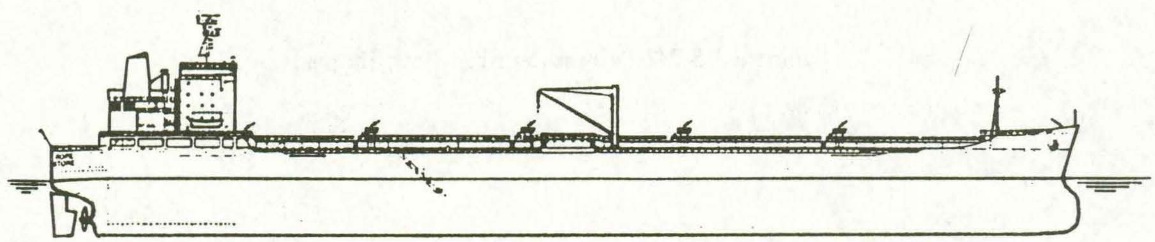


Barge Type	Average Size of Barge in Tow	Average Number of Barges Along the Length of Tow
Jumbo Hopper	35x195	3
Oversize Tank	53x290	2
Special Deck	72x250	1

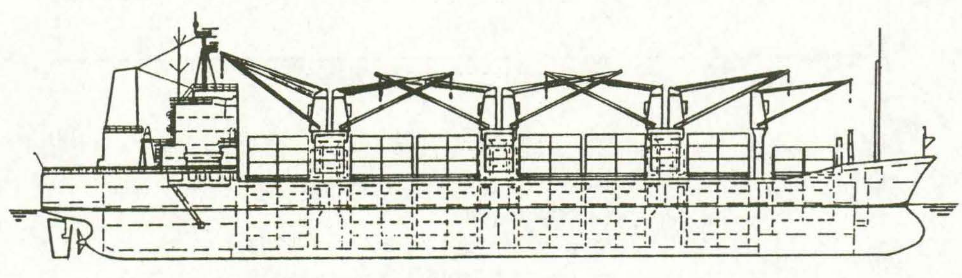
Figure 3.5.1-2. Typical Barge Tow Configurations.



a. 81,400 DWT Bulk Carrier (length = 855 ft).



b. 67,000 DWT Product Carrier/Tanker (length = 800 ft).



c. 22,000 DWT Multi-Purpose Freighter/Container Ship (length = 670 ft).

Figure 3.5.2-1. Typical Ship Profiles.

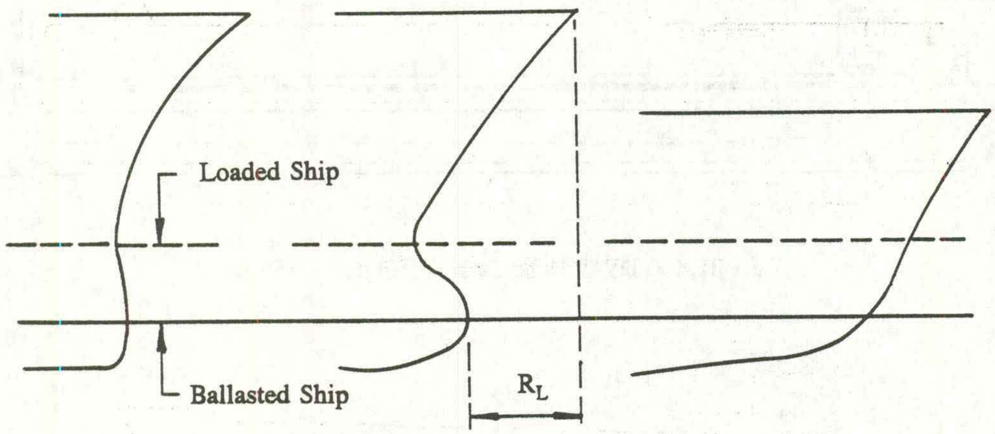
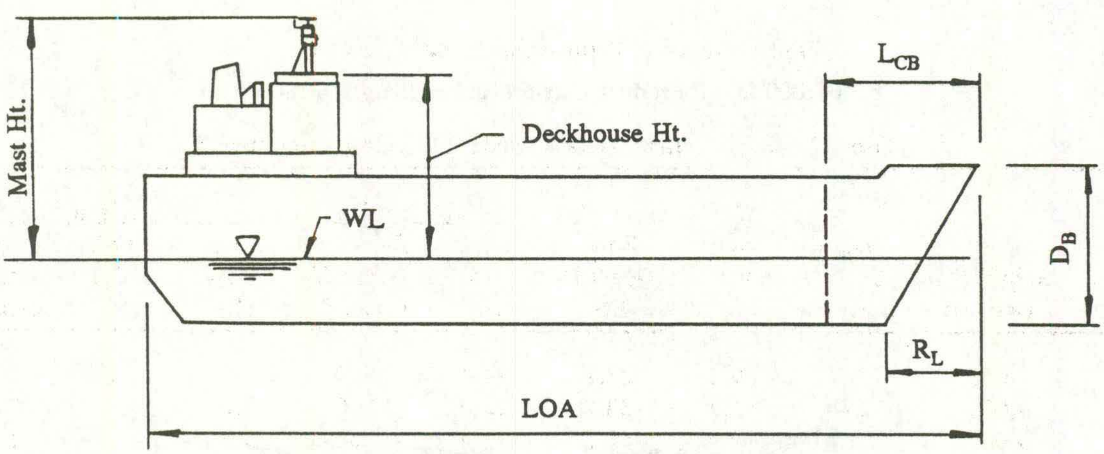
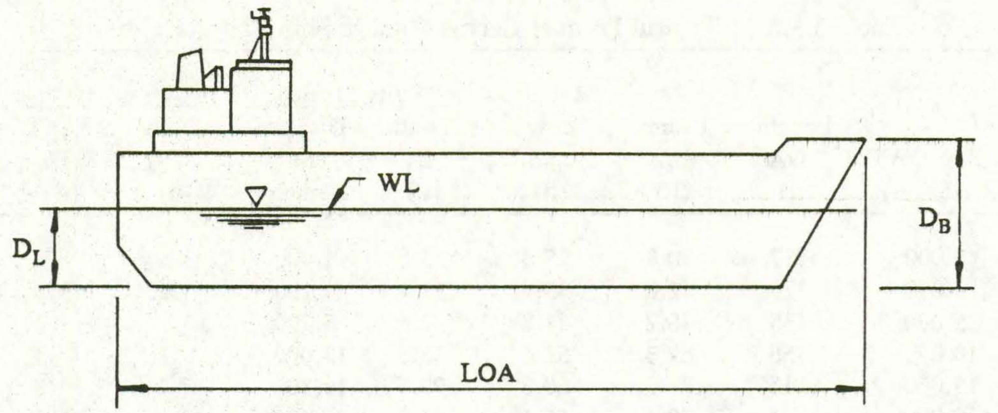


Figure 3.5.2-2. Common Ship Bow Shapes.

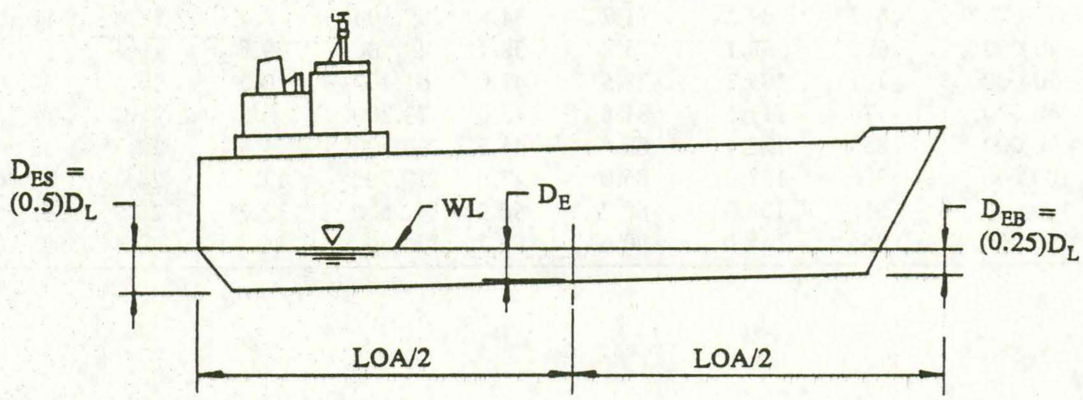


D_B = bow depth
R_L = rake length of bow ≅ 0.25 D_B
L_{CB} = length to collision bulkhead ≅ 0.1(LOA)
LOA = length overall of ship
WL = waterline

Figure 3.5.2-3. Typical Ship Bow and Vertical Clearance Dimensions.



a. Loaded Ship Profile.



b. Ballasted Ship Profile.

Figure 3.5.2-4. Typical Ship Characteristics.

Table 3.5.2-1. Typical Bulk Carrier Ship Characteristics.

				Fully Loaded		Ballasted		
Ship DWT (tonnes)	Length LOA (ft)	Beam B_M (ft)	Bow Depth, D_B (ft)	Draft D_L (ft)	Displacement, W_L (tonnes)	Draft D_{EB} (ft)	Draft D_{ES} (ft)	Displacement, W_E (tonnes)
1,000	200	29.2	27.2	14.1	1,500	3.5	7.1	600
3,000	289	41.7	38.2	22.3	4,200	5.6	11.2	1,600
5,000	341	48.9	45.2	21.3	6,800	5.3	10.7	2,600
10,000	459	61.4	57.6	26.6	13,100	6.7	13.3	4,900
15,000	515	70.5	64.2	29.5	19,300	7.4	14.8	7,200
20,000	558	77.8	68.4	31.5	25,500	7.9	15.8	9,600
25,000	577	82.4	70.8	32.2	31,500	8.1	16.1	11,800
30,000	630	89.6	74.1	34.8	37,500	8.7	17.4	14,100
40,000	682	99.1	77.8	37.4	49,400	9.4	18.7	18,500
50,000	728	107.0	80.2	39.0	61,100	9.8	19.5	22,900
60,000	771	109.3	83.7	40.4	72,800	10.1	20.2	27,300
80,000	850	120.1	86.2	43.3	95,800	10.8	21.7	35,900
100,000	902	137.8	92.8	52.8	118,600	13.2	26.4	44,500
150,000	1027	146.0	99.7	59.1	174,700	14.8	29.6	65,500

Table 3.5.2-2. Typical Product Carrier/Tanker Ship Characteristics.

				Fully Loaded		Ballasted		
Ship DWT (tonnes)	Length LOA (ft)	Beam B_M (ft)	Bow Depth, D_B (ft)	Draft D_L (ft)	Displacement, W_L (tonnes)	Draft D_{EB} (ft)	Draft D_{ES} (ft)	Displacement, W_E (tonnes)
1,000	187	30.8	25.0	13.8	1,400	3.5	6.9	500
3,000	279	42.0	35.4	19.4	4,100	4.9	9.7	1,500
5,000	335	48.2	41.8	22.6	6,700	5.7	11.3	2,500
10,000	456	62.3	53.6	26.6	13,000	6.7	13.3	4,900
15,000	515	71.2	60.2	29.5	19,300	7.4	14.8	7,200
20,000	561	78.1	65.1	32.2	25,400	8.1	16.1	9,500
25,000	577	83.7	68.7	33.1	31,500	8.3	16.6	11,800
30,000	637	89.2	71.7	34.8	37,500	8.7	17.4	14,100
40,000	692	98.1	75.8	38.4	49,500	9.6	19.2	18,600
50,000	741	105.3	78.5	41.0	61,400	0.3	20.5	23,000
60,000	774	111.5	81.8	42.0	73,200	0.5	21.0	27,500
80,000	853	122.4	83.6	45.6	96,500	11.4	22.8	36,200
100,000	886	128.0	85.0	47.9	119,700	12.0	24.0	44,900
120,000	915	138.9	88.2	50.9	142,600	12.7	25.5	53,500
150,000	955	145.0	90.6	58.7	176,800	14.7	29.4	66,300

Table 3.5.2-3. Typical Freighter/Container Ship Characteristics.

				Fully Loaded		Ballasted		
Ship DWT (tonnes)	Length LOA (ft)	Beam B_M (ft)	Bow Depth, D_B (ft)	Draft D_L (ft)	Displacement, W_L (tonnes)	Draft D_{EB} (ft)	Draft D_{ES} (ft)	Displacement, W_E (tonnes)
1,000	190	31.2	23.0	13.8	1,400	3.5	6.9	500
3,000	282	43.3	39.0	19.4	4,200	4.9	9.7	1,600
5,000	338	50.5	44.9	22.3	7,000	5.6	11.2	2,600
7,000	423	57.7	52.8	24.6	9,700	6.2	12.3	3,600
10,000	472	63.6	58.0	26.9	3,800	6.7	13.5	5,200
12,000	499	65.9	60.8	28.9	16,600	7.2	14.5	6,000
16,000	617	84.3	76.2	30.8	24,800	7.7	15.4	9,300
20,000	643	90.6	80.4	34.4	31,600	8.6	17.2	11,850
24,000	697	98.4	82.0	34.4	36,700	8.6	17.2	13,800
27,000	717	102.4	86.0	36.7	42,200	9.2	18.4	15,800
33,000	863	105.6	86.5	37.7	51,600	9.4	18.9	19,400
49,700	950	106.0	94.8	36.1	77,000	9.0	18.1	28,900
54,500	903	129.2	96.4	41.0	84,500	10.3	20.5	31,700

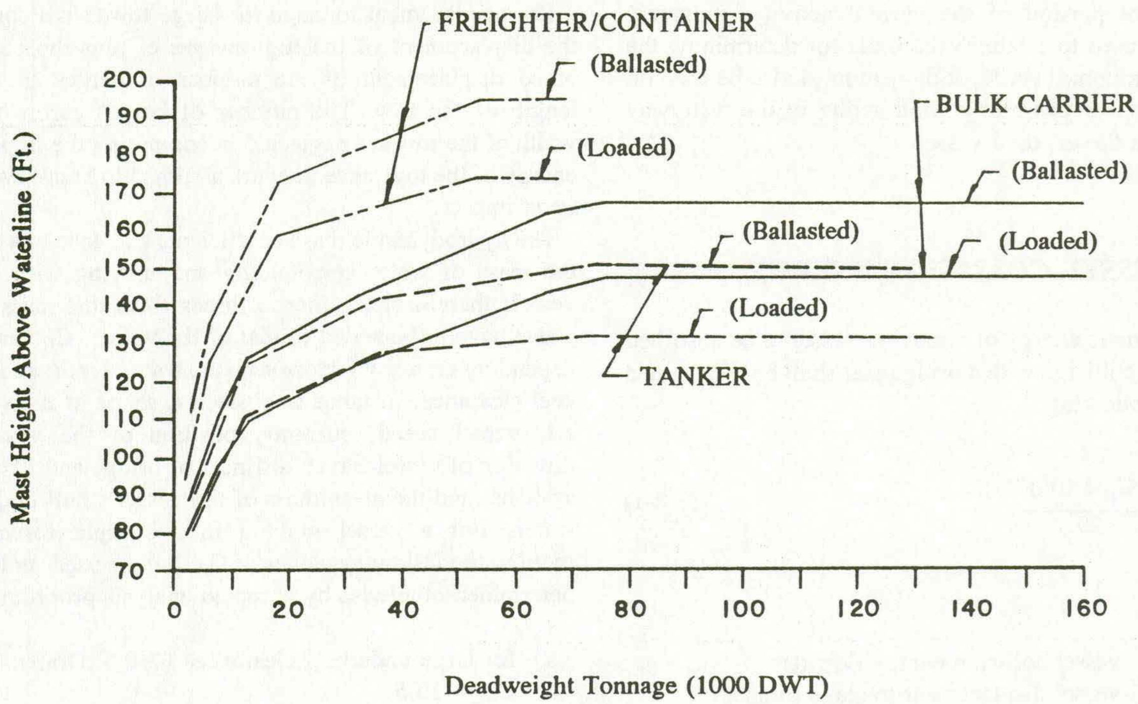


Figure 3.5.2-5. Typical Ship Mast Clearance Heights.

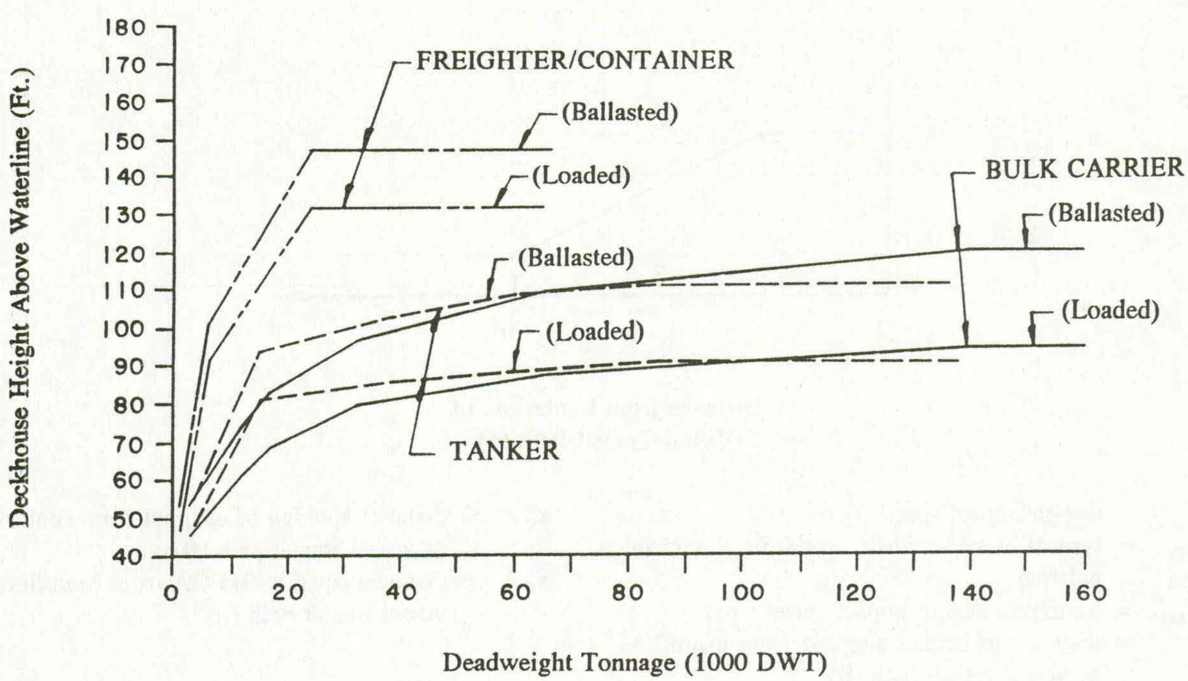


Figure 3.5.2-6. Typical Ship Deckhouse Clearance Heights.

As a minimum, the design impact speed shall be equal to the yearly mean current for the waterway location. In waterways where seasonal flooding represents a significant portion of the current activity, judgment must be used to establish the basis for determining the minimum impact speed. Judgment must also be used on the effects of prevailing wind acting in the waterway and upon the exposed vessel.

3.8 VESSEL COLLISION ENERGY

The kinetic energy of a moving vessel to be absorbed during a collision with a bridge pier shall be determined by the following:

$$KE = \frac{C_H W (V)^2}{29.2} \qquad (3.8\text{-}1)$$

where

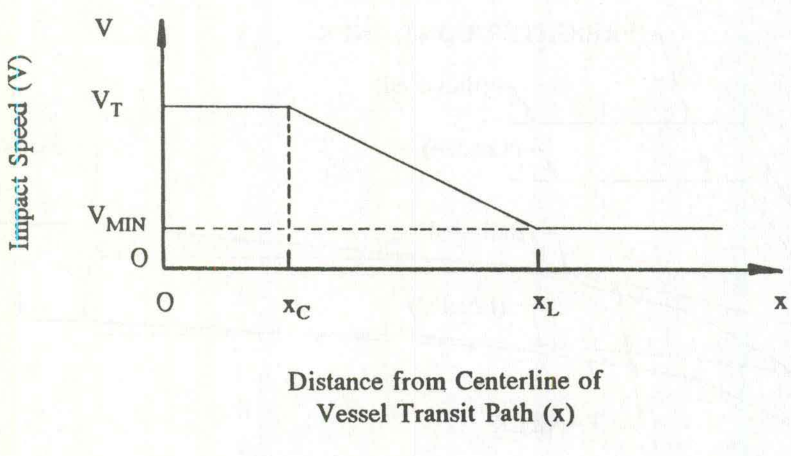
KE = vessel collision energy (kip-ft)
W = vessel displacement tonnage (tonnes)
V = vessel impact speed (fps)
C_H = hydrodynamic mass coefficient

The vessel displacement tonnage, W, equals the weight of the vessel when empty plus the weight of the ballast and cargo (DWT) being carried by the vessel.

The displacement tonnage for barge tows shall equal the displacement of the tug/tow vessel plus the combined displacement of the number of barges in the length of the tow. The number of barges across the width of the tow are neglected in computing the impact energy of the tow since they are assumed to break away upon impact.

The hydrodynamic mass coefficient, C_H, accounts for the mass of water surrounding and moving with the vessel; therefore, the inertia forces from this mass of water have to be added to that of the vessel. C_H varies depending on many factors such as, water depth, underkeel clearance, distance to obstacles, shape of the vessel, vessel speed, currents, position of the vessel, direction of vessel travel, stiffness of bridge and fender systems, and the cleanliness of the vessel's hull underwater. For a vessel moving in a straight forward motion, the following values of C_H shall be used, unless determined otherwise by accepted analysis procedures:

1) for large underkeel clearances (≥ 0.5xDraft), $C_H = 1.05$
2) for small underkeel clearances (≤ 0.1xDraft), $C_H = 1.25$



V = design impact speed (fps)
V_T = typical vessel transit speed in the channel(fps)
V_{MIN} = minimum design impact speed (fps)
x = distance to bridge element from centerline of vessel transit path (ft)
x_C = distance to edge of channel from centerline of vessel transit path (ft)
x_L = distance equal to 3xLOA from centerline of vessel transit path (ft)

Figure 3.7-1. Design Impact Speed.

The underkeel clearance is the distance between the bottom (keel) of a vessel and the bottom of the waterway. C_H for underkeel clearances between the large and small limits discussed above may be estimated by interpolating.

3.9 SHIP COLLISION FORCE ON PIER

The ship collision equivalent static impact force associated with a head-on collision with a rigid object shall be computed by the following for Product Carrier/Tanker, Bulk Carrier, and Freighter/Container vessels:

$$P_S = 220(DWT)^{1/2}\left[\frac{V}{27}\right] \qquad (3.9\text{-}1)$$

where

P_S = equivalent static ship impact force (kips)
DWT = deadweight tonnage of ship (tonnes)
V = ship impact speed (fps)

A more rigorous generally accepted dynamic analysis procedure may be used in lieu of the recommended static analysis procedure. Such procedure shall be based on an accepted impact force/damage length relationship established by model testing or structural analysis. If transient or permanent deflections or movements of the bridge component are introduced in the analysis, the force reducing effect must be documented by a dynamic analysis.

3.10 SHIP BOW DAMAGE DEPTH

The depth of the ship's bow crushed during impact with a rigid object shall be computed by:

$$a_S = 1.54\left[\frac{KE}{P_S}\right] \qquad (3.10\text{-}1)$$

where

a_S = ship damage depth (ft)
KE = ship collision energy (kip-ft)
P_S = equivalent static ship impact force (kips)

3.11 SHIP COLLISION FORCE ON SUPERSTRUCTURE

3.11.1 Bow Collision

The ship collision impact force between the bow of the design ship and an exposed superstructure shall be computed by:

$$P_{BH} = (R_{BH})(P_S) \qquad (3.11.1\text{-}1)$$

where

P_{BH} = ship bow impact force on the exposed superstructure (kips)
R_{BH} = ratio of exposed superstructure depth to the total bow depth (Section 3.5.2)
P_S = ship impact force from Section 3.9

3.11.2 Deckhouse Collision

The superstructure collision impact force between the deckhouse of the design ship and an exposed superstructure shall be computed by:

$$P_{DH} = (R_{DH})(P_S) \qquad (3.11.2\text{-}1)$$

where

P_{DH} = ship deckhouse impact force (kips)
R_{DH} = reduction factor
P_S = ship impact force from Section 3.9

For ships greater in size than 100,000 DWT, R_{DH} = 0.10. For ships smaller than 100,000 DWT, R_{DH} shall be computed as:

$$R_{DH} = 0.2 - \left[\frac{DWT}{100{,}000}\right] 0.10 \qquad (3.11.2\text{-}2)$$

3.11.3 Mast Collision

The superstructure collision impact force between the mast of the design ship and an exposed superstructure shall be computed by:

$$P_{MT} = 0.10(P_{DH}) \quad (3.11.3\text{-}1)$$

where

P_{MT} = ship mast impact force (kips)
P_{DH} = ship deckhouse impact force from Equation 3.11.2-1

3.12 BARGE COLLISION FORCE ON PIER

The barge collision impact force associated with a head-on collision shall be determined by the following:

for $a_B < 0.34$,

$$P_B = 4112(a_B)(R_B) \quad (3.12.1\text{-}1a)$$

for $a_B \geq 0.34$,

$$P_B = [1349 + 110(a_B)]R_B \quad (3.12.1\text{-}1b)$$

where

P_B = equivalent static barge impact force (kips)
R_B = ratio of $B_B/35$
B_B = barge width (ft)
a_B = barge bow damage depth (ft)

3.13 BARGE BOW DAMAGE DEPTH

The barge bow damage depth shall be computed as:

$$a_B = \left[\left(1 + \frac{KE}{5672}\right)^{1/2} - 1 \right] \left(\frac{10.2}{R_B} \right) \quad (3.13\text{-}1)$$

where

a_B = barge bow damage depth (ft)
KE = barge collision energy (kip-ft)
R_B = ratio of $B_B/35$
B_B = barge width (ft)

3.14 IMPACT LOAD COMBINATION

The vessel impact loading for each bridge component shall be computed as:

$$\text{Group Load} = \gamma\,(1.0D + 1.0P + 1.0B + 1.0SF + 1.0E) \quad (3.14\text{-}1)$$

where

γ = load factor = 1.0, for all design methods
D = dead load
P = vessel collision impact force
B = buoyancy
SF = stream flow pressure
E = earth pressure

Each component of the structure shall be designed to withstand the forces resulting from each load combination according to the AASHTO Standard Specifications for Highway Bridges, current edition adopted by AASHTO. In addition, the structure shall be designed for the Group Loading given by Equation 3.14-1 and the requirements of this Guide Specification.

Under the application of the group loading in Equation 3.14-1, the piers, substructures, and connections to the superstructure shall be proportioned to prevent the collapse of the superstructure. Damage or local collapse of substructure and superstructure elements is permitted to occur provided that, 1) sufficient redundancy of the remaining structure, or multi-load paths, exist in the ultimate limit state to safely prevent superstructure collapse, 2) that the design vessel has been completely stopped or redirected so that no significant damage to the superstructure will result, and 3) that the structure element can be visually inspected and repaired in a relatively straightforward manner.

As an alternative to this ultimate state design, each component of the structure may be designed to withstand the forces resulting from the group loading in Equation 3.14-1. Either Service Load Design or Load Factor Design may be used. For Service Load Design a 50 percent increase is permitted in the allowable stresses for structural steel and a 33.33 percent increase for reinforced concrete. For Load Factor Design no additional load magnification factors are required, the results of Equation 3.14-1 may be applied directly.

As an additional alternative, pier protection may be provided for the bridge structure to eliminate or reduce the group loading in Equation 3.14-1 to acceptable levels.

3.15 LOCATION OF IMPACT FORCES

3.15.1 Substructure Design

For substructure design, the design impact force shall be applied as an equivalent static force transverse to the substructure in a direction parallel to the alignment of the centerline of the navigable channel. Fifty percent (50%) of the transverse load shall be applied as a longitudinal force to the substructure. These transverse and longitudinal impact forces shall not be taken to act simultaneously.

All portions of the bridge pier or substructure exposed to physical contact by any portion of the design vessel's hull or bow, shall be proportioned to resist the applied loads in accordance with these Guide Specifications. The bow overhang, rake, or flair distance, of ship and barge vessels shall be considered in determining the portions of the pier and substructure exposed to contact by the vessel. Unless determined otherwise by a detailed investigation of the actual vessel traffic using the waterway, the typical data in Section 3.5 shall be used to determine the bow overhang distances. Crushing of the vessel's bow causing contact with any setback portion of the pier or substructure shall also be considered.

The design impact force shall be applied to the pier in accordance with the following criteria:

1) The design impact force shall be applied as a concentrated force on the substructure at the mean high water level of the waterway to design the substructure for overall stability as shown in Figure 3.15.1-1.
2) The design impact force shall be applied as a vertical line load equally distributed along the ship's bow depth to design the pier and substructure for local collision forces as shown in Figure 3.15.1-2. The ship's bow shall be considered to be raked forward when determining the potential contact area of the impact force on the pier or substructure. For barge impact, the local collision force shall be taken as a vertical line load equally distributed on the depth of the head block as shown in Figure 3.15.1-3.

3.15.2 Superstructure Design

For superstructure design, the design impact force shall be applied as an equivalent static force transverse to the superstructure member in a direction parallel to the alignment of the centerline of the navigable channel.

3.16 MINIMUM IMPACT REQUIREMENT

All bridge elements in a navigable waterway crossing located in design water depths (Section 4.2.2) equal to or greater than 2.0 feet for which these specifications are applicable (Section 3.2), shall be designed for vessel impact. The minimum design impact force for pier design shall be computed using an empty hopper barge drifting at a speed equal to the yearly mean current for the waterway location. The empty hopper barge characteristics shall represent the typical barge size using the waterway. A single 35 x 195 foot barge with an empty displacement of 200 tons would be typical for most waterways. The drifting barge impact force shall be applied to the bridge according to Section 3.15, or as a broadside collision force as shown in Figure 3.16-1. The minimum design impact force for superstructure design in deep draft waterways, shall be the ship mast impact force in Section 3.11.3.

3.17 BRIDGE PROTECTION SYSTEMS

Bridge protection systems may be provided to reduce or eliminate the exposure of bridge components to vessel collision. Bridge protection system design shall be in accordance with the requirements of Section 7.

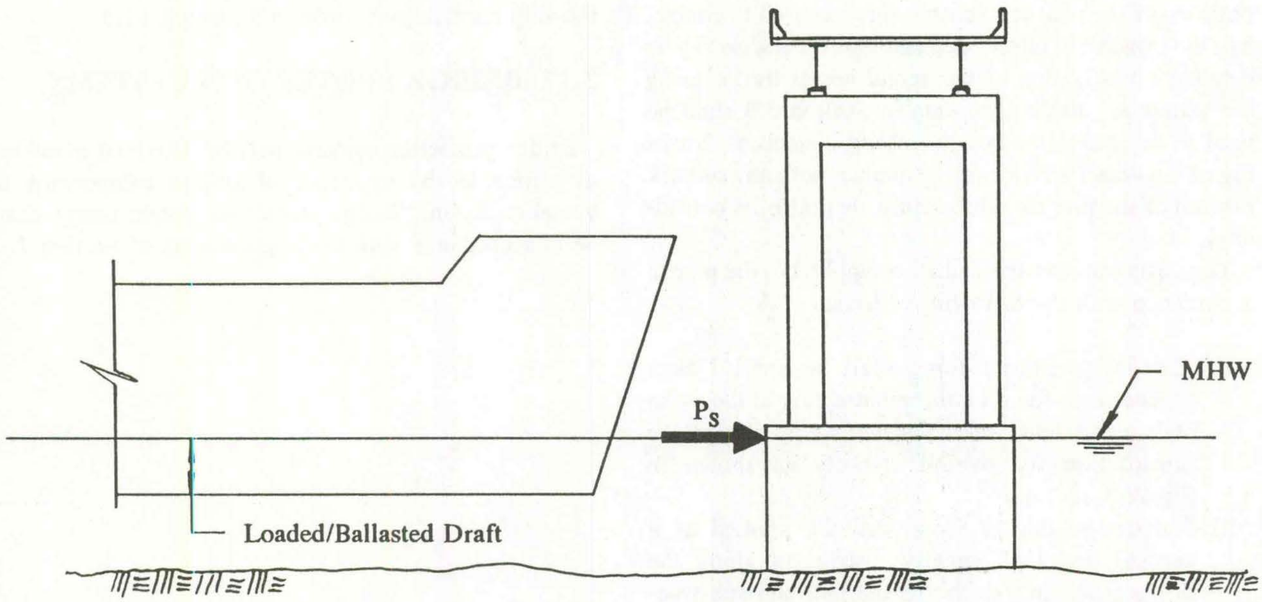


Figure 3.15.1-1. Ship Impact Concentrated Force on Pier (For Foundation Design & Overall Stability).

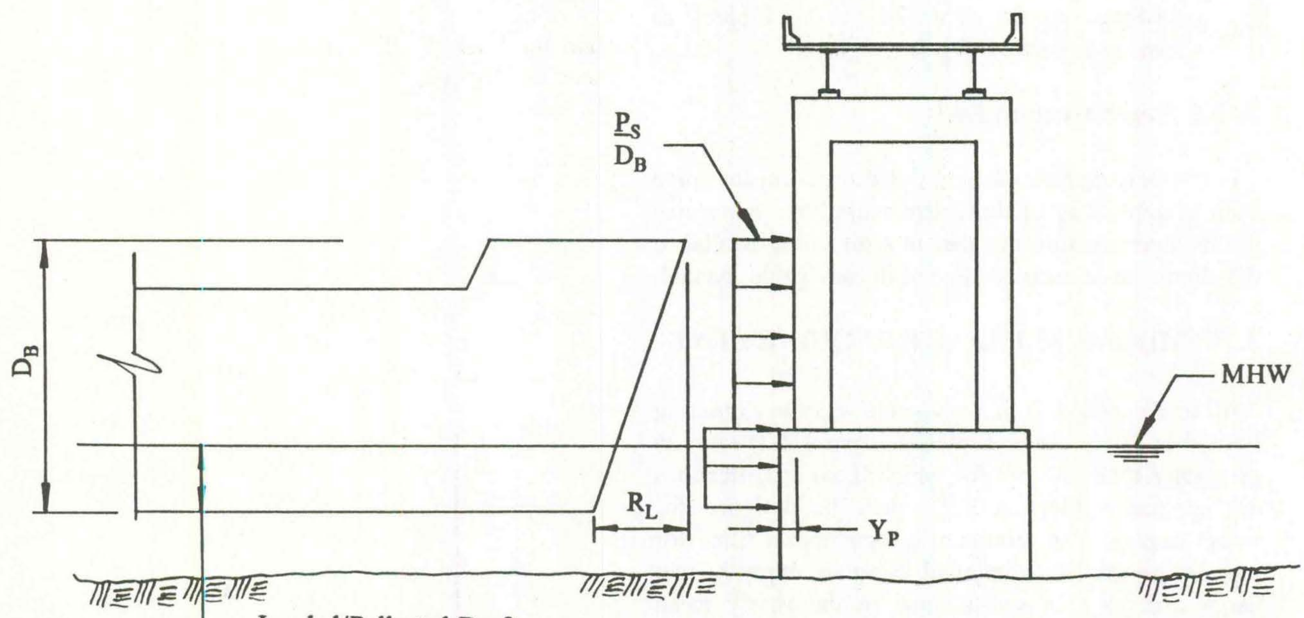


Figure 3.15.1-2. Ship Impact Line Load for Local Collision Force on Pier (For Structure Check & Design).

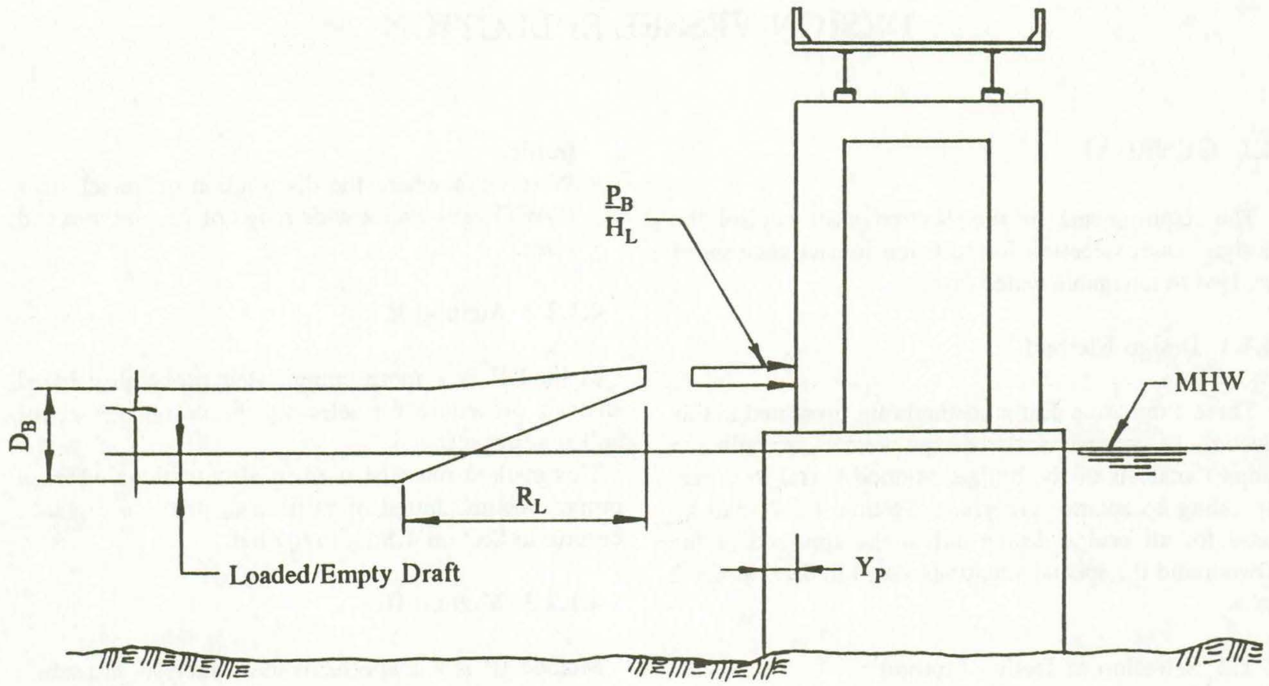


Figure 3.15.1-3. Barge Impact Line Load for Local Collision Force on Pier (For Structure Check & Design).

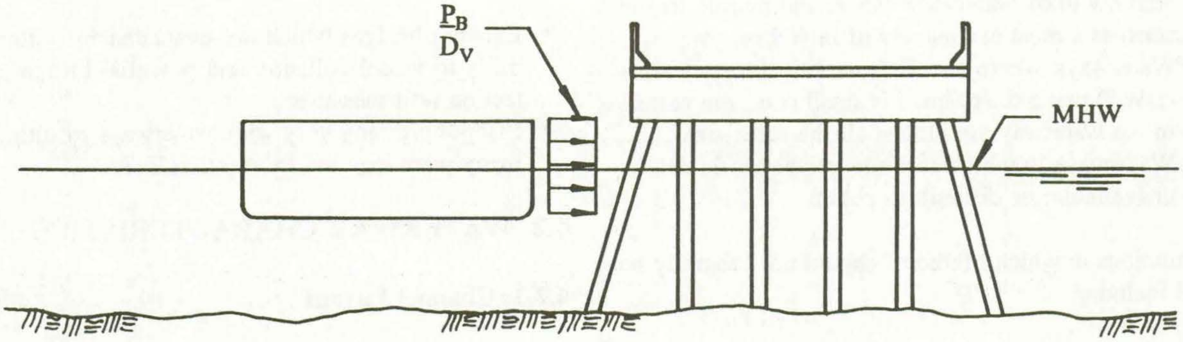


Figure 3.16-1. Broadside Barge Impact on Pier.

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 Section 4.8.2 shall be used for all bridge design unless the approval of the Owner and the special situations stated in Section 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 Section 4.8.2. However, the procedure is less accurate than Method II and should be used only in simple and un-complicated 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 (DWT) using the channel is small (i.e., the vessels in the waterway are almost all the same size).
- 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 Bridges.
- Deep draft waterways where large merchant ships comprise a significant portion of the total vessel traffic.
- Waterways where the distribution of vessel sizes (DWT) vary over a wide range of vessel types and sizes.

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 Section 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 Section 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 refit measures.
- Bridges crossing very wide waterways resulting in many piers exposed to vessel collision.

4.2 WATERWAY CHARACTERISTICS

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 exist:

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 4.2.1-1.
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 4.2.1-2.

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.

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 which 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 Section 3.5.1 for barges, and Section 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 3xLOA 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 3xLOA distance on each side of the vessel transit path shall be designed in accordance with the minimum impact loads in Section 3.16.

The length overall (LOA) shall be based on the dimensions of a vessel selected in accordance with the Method I criteria in Section 4.7.2. The LOA for impact distribution is the same dimension used in Section 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.

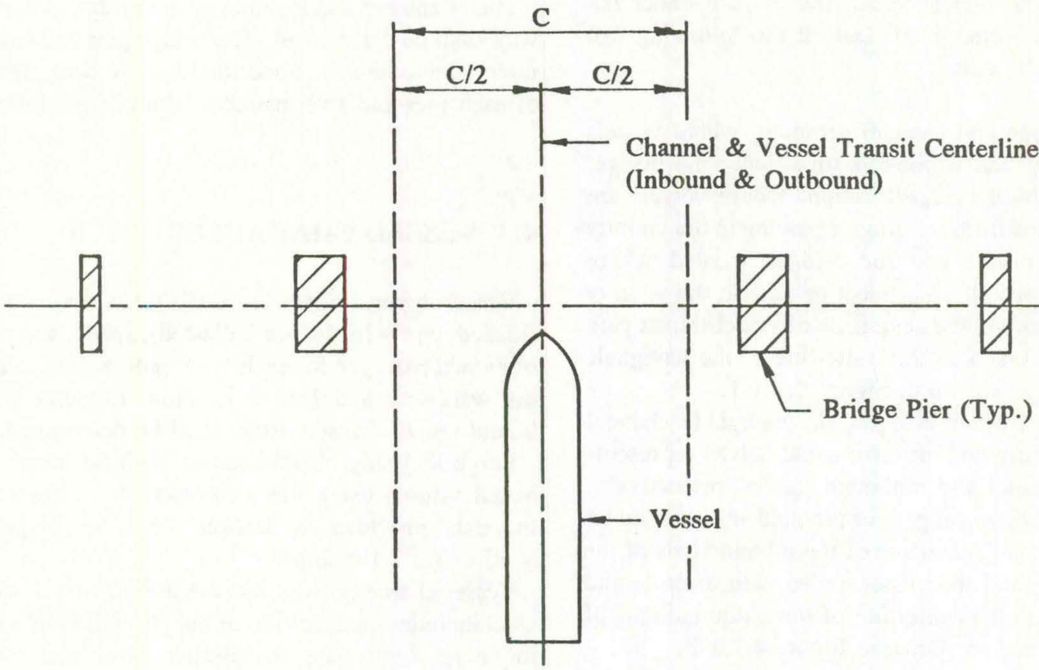


Figure 4.2.1-1. Single Transit Path in Channel Through Bridge.

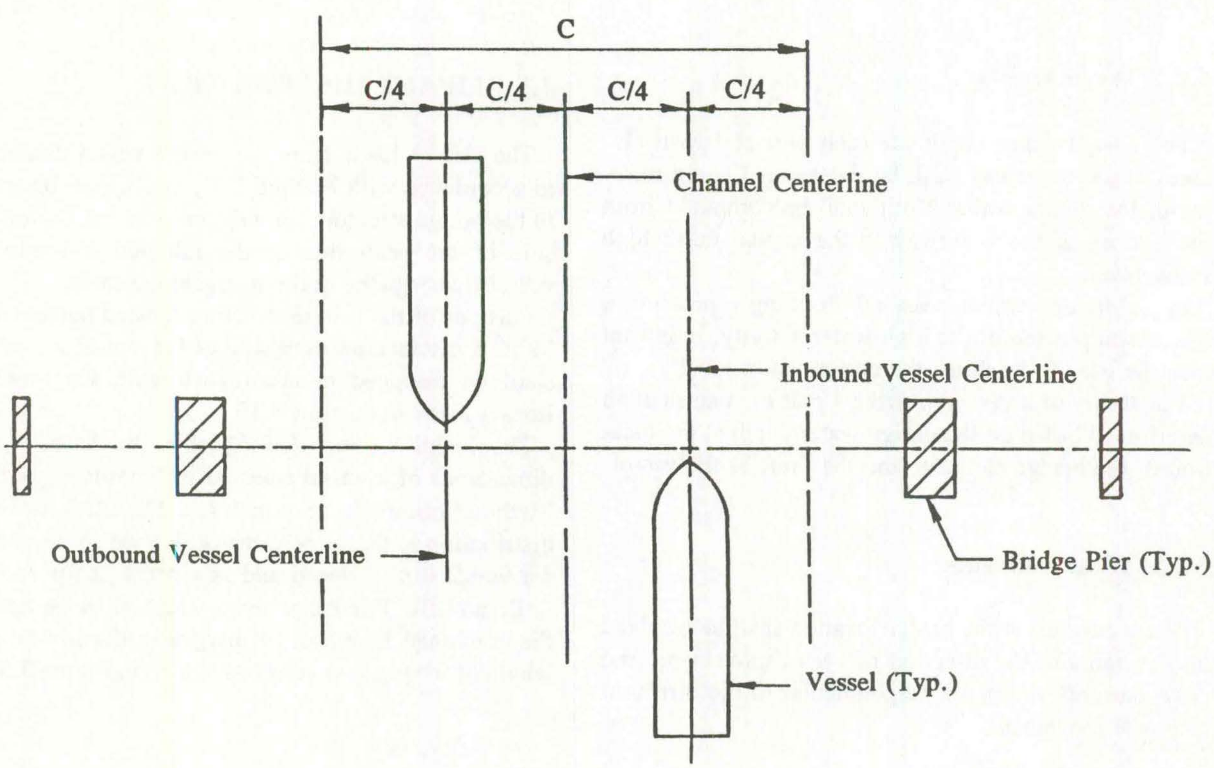


Figure 4.2.1-2. Passing Vessel Transit Paths in Channel Through Bridge.

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 importance classification, vessel characteristics, bridge geometry, and water depths in accordance with the following acceptance criteria:

- CRITICAL 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 five percent (5 percent) of the total number of merchant vessels per year which could impact the bridge element, whichever is smaller.
- REGULAR 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 ten percent (10 percent) of the total number of merchant vessels per year which could impact the bridge element, whichever is smaller.

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.

4.8.2 Design Vessel Acceptance Criteria

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

- CRITICAL BRIDGES. The acceptable annual frequency of collapse, AF, of critical bridges shall be equal to, or less than, 0.01 in 100 years (AF=.0001).
- REGULAR BRIDGES. The acceptable annual frequency of collapse, AF, of regular bridges shall be equal to, or less than, 0.1 in 100 years (AF=.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 3xLOA 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, and larger than, the design vessel is less than the acceptance criterion for the element.

4.8.3 Annual Frequency of Collapse

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

$$AF = (N)(PA)(PG)(PC) \qquad (4.8.3\text{-}1)$$

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

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

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.

4.8.3.2 Probability of Aberrancy

The probability of aberrancy, PA, 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 PA vary widely.

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

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

$$PA = BR(R_B)(R_C)(R_{XC})(R_D) \qquad (4.8.3.2\text{-}1)$$

where

- PA = 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_{XC} = correction factor for crosscurrents acting perpendicular to vessel transit path
- R_D = 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 4.8.3.2-1 as:

1) Straight Region: For a bridge located in a straight region:

$$R_B = 1.0 \qquad (4.8.3.2\text{-}2a)$$

2) Transition Region: For a bridge located in a transition region, R_B can be computed by:

$$R_B = \left[1 + \frac{\theta}{90°}\right] \qquad (4.3.3.2\text{-}2b)$$

where

Θ = 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:

$$R_B = \left[1 + \frac{\theta}{45°}\right] \qquad (4.3.3.2\text{-}2c)$$

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

$$R_C = \left[1 + \frac{V_c}{10}\right] \qquad (4.8.3.2\text{-}3)$$

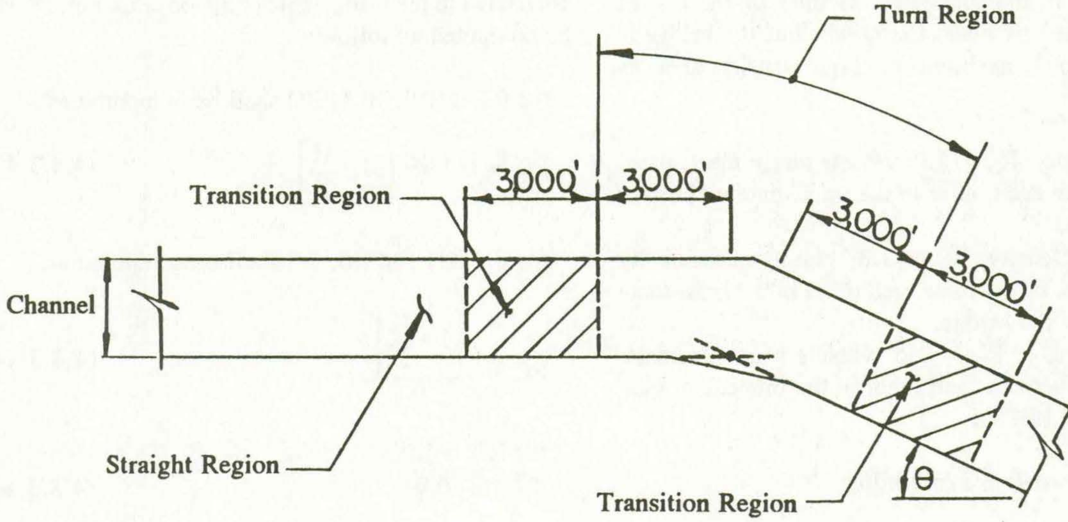
where

V_C = current component parallel to vessel path (knots)

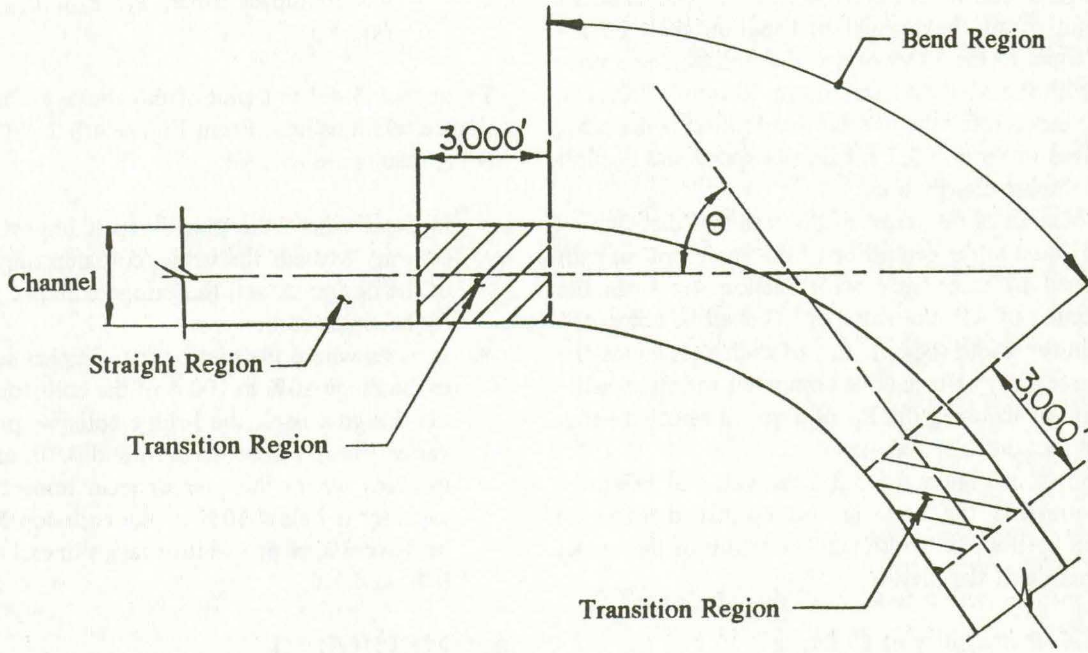
The correction factor, R_{XC}, for crosscurrents acting perpendicular to the vessel transit path in the waterway can be computed by:

$$R_{XC} = (1 + V_{XC}) \qquad (4.8.3.2\text{-}4)$$

where



a. Turn in Channel.



b. Bend in Channel.

Figure 4.8.3.2-1. Waterway Regions for Bridge Location.

V_{XC} = current component perpendicular to vessel path (knots)

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.

4.8.3.3 Geometric Probability

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 sailing path near the bridge as shown in Figure 4.8.3.3-1. The standard deviation, σ, of the normal distribution shall be assumed equal to the LOA of a vessel selected in accordance with the Method I criteria in Section 4.7.2. The LOA dimension for the normal distribution is the same value used in Section 3.7 for impact speed and Section 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 Section 4.2.1. In the computation of AF, the value of PG shall be computed based on the width (beam), B_M, of each vessel classification category, or it may be computed for all classification intervals using the B_M of a vessel selected using Method I as discussed above.

As shown in Figure 4.8.3.3-1 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.

4.8.3.4 Probability of Collapse

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 strength of the pier, H_P, and span, H_S, to resist collision impact loads. Based on collision damage sustained during ship-ship collision accidents which has been correlated to the bridge-ship collision situation, PC shall be computed as follows:

for $0.0 \leq H/P < 0.1$, PC shall be computed as:

$$PC = 0.1 + 9\left[.1 - \frac{H}{P}\right] \quad (4.8.3.4\text{-}1a)$$

for $0.1 \leq H/P < 1.0$, PC shall be computed as:

$$PC = \frac{\left[1 - \frac{H}{P}\right]}{9} \quad (4.8.3.4\text{-}1b)$$

for $H/P > 1.0$

$$PC = 0.0 \quad (4.8.3.4\text{-}1c)$$

where

PC = probability of collapse
H = ultimate bridge element strength, H_P or H_S (kips)

P = vessel impact force, P_S, P_{BH}, P_{DH}, or P_{MT} (kips)

Figure 4.8.3.4-1 is a plot of the above probability of collapse relationships. From Figure 4.8.3.4-1, the following results are evident:

- in cases where the pier or span impact strength 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 strength is in the range 10% to 100% 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 strength capacity is below 10% of the collision force, the bridge collapse probability varies linearly between 0.10 and 1.0.

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

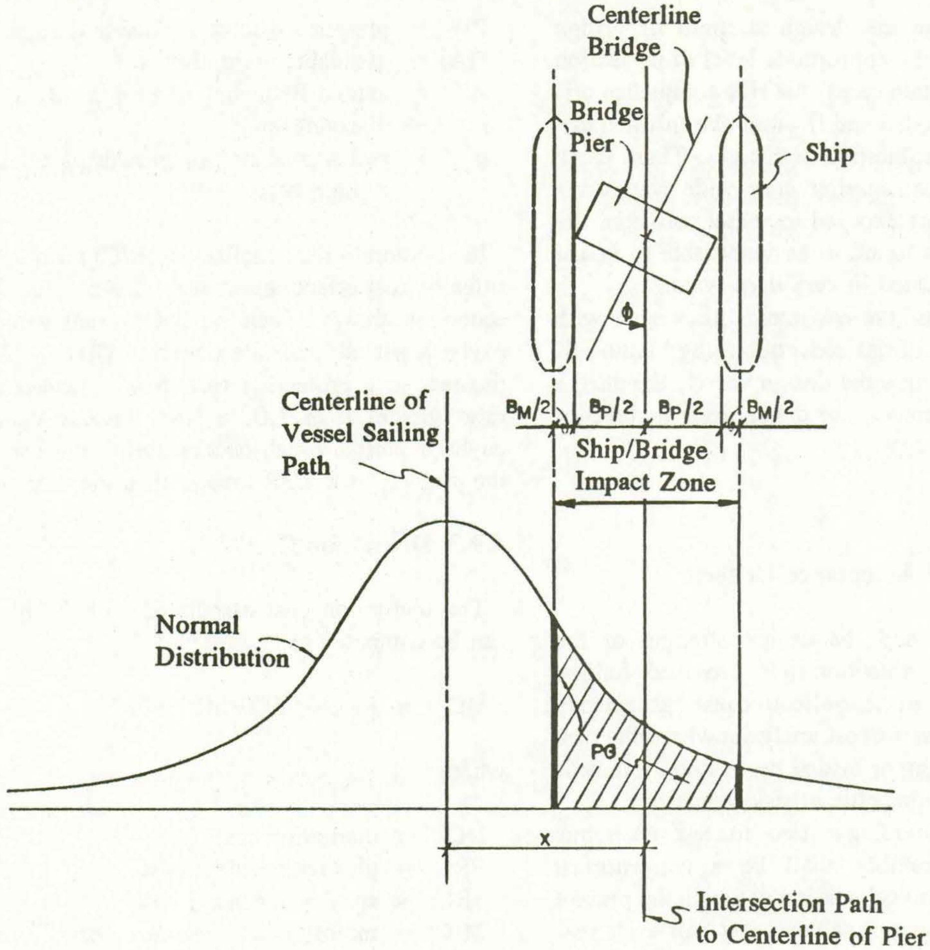


Figure 4.8.3.3-1. Geometric Probability of Pier Collision.

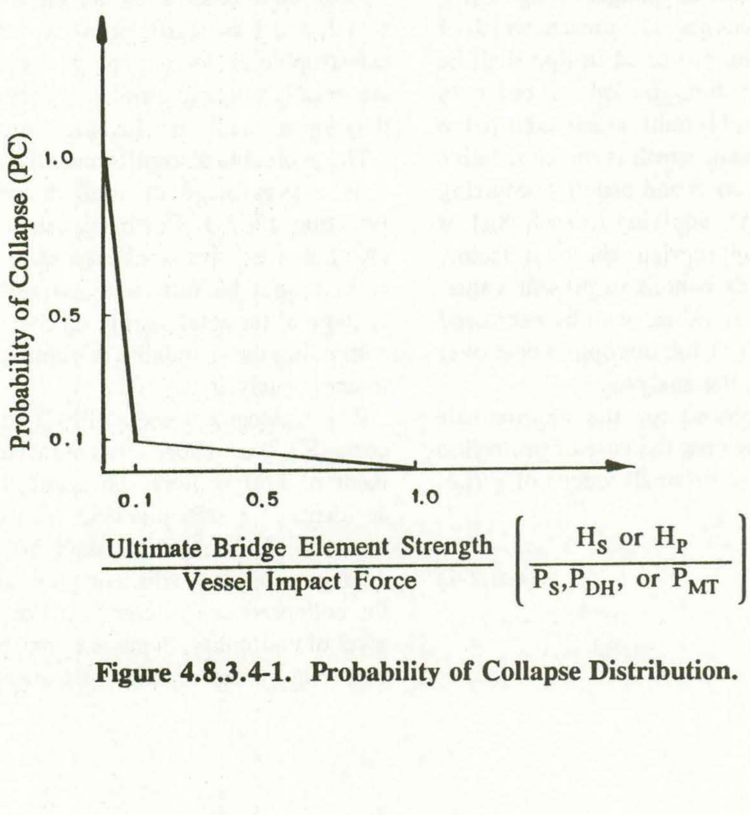


Figure 4.8.3.4-1. Probability of Collapse Distribution.

be used to determine the design strength 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 prohibitive high costs. These cases might include bridges crossing very wide waterways resulting in many piers exposed to vessel collision, the refit 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 strength 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 strength 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. All costs, benefits, and other values shall be expressed in constant dollars. Growth of the disruption cost over time shall be considered in the analysis.

In lieu of the above procedure, the approximate benefits used to compare against the cost of protection can be estimated as follows for small values of g (i.e. $g \leq i/5$):

$$PW = \frac{AF(DA)}{(AF+1-g)} \qquad (4.9.2\text{-}1)$$

where

PW = present worth of avoidable disruption cost
DA = avoidable disruption cost
AF = annual frequency of bridge collapse
i = discount rate
g = real annual rate of growth of avoidable disruption costs

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:

$$DC = PRC + SRC + MIC + PIC \qquad (4.9.3\text{-}1)$$

where

DC = disruption cost
PRC = pier replacement cost
SRC = span replacement cost
MIC = motorist inconvenience cost
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; however, since these cost are usually subjective and therefore difficult to estimate, they are normally not included in computing DC.

The avoidable disruption cost, DA, in Equation 4.9.2-1 is a percentage of total disruption cost, DC, in Equation 4.9.3-1. For many situations, DA will equal DC; however, situations exist where the total disruption costs cannot be fully avoided and some smaller percentage of the total disruption costs must be utilized in computing the avoidable disruption cost for cost-effectiveness analysis.

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. It also includes toll revenues lost by the out-of-service facility owner, if it is a toll bridge. 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 collapsed 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 what cargoes, that would be delayed or trapped due to the bridge collapse, and for what length of time;
- what cargoes would be foregone (rerouted to other ports, or shipped by alternative modes); and
- what opportunities 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 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.

SECTION 5
SUBSTRUCTURE PROVISIONS

5.1 GENERAL

This section includes only those substructure requirements related to the design of bridge substructures to resist vessel impact loading without causing superstructure collapse. It assumes the substructure has been adequately designed for each load combination according to the *AASHTO Standard Specifications for Highway Bridges* current edition adopted by AASHTO.

This section's requirements are not applicable to the substructure design of sacrificial protection systems.

5.2 ANALYSIS

To achieve a cost effective design, the substructure and superstructure may be analyzed as a unit, thus allowing adjacent substructure elements to participate in resisting the vessel impact force. Sound principles of structural mechanics must be followed in this analysis. Only positive connections of the superstructure to the substructure shall be considered effective in this analysis for transfer of vessel impact force to adjacent substructure elements.

All structural components and their connections in the load path must be adequately proportioned to withstand the impact force.

5.3 FOUNDATION DESIGN

Ultimate strength or service load design methods should be used to resist vessel impact forces for any foundation element which cannot be visually inspected and repaired in a relatively straight-forward manner. If ultimate strength methods are used, the capacity of an axially loaded pile shall be limited to the ultimate strength of the pile as a structural element or the ultimate strength of the foundation material, whichever controls. If analysis indicates that piles will be loaded in tension by vessel impact forces, the designer must determine that the piles as installed have adequate pullout resistance.

Transient foundation uplift or rocking involving separation from the subsoil of an end bearing foundation pile group or the contact area of a foundation footing is permitted under impact loading providing sufficient consideration is given to the structural stability of the substructure. Consideration shall be given to the magnitude of foundation settlement that the bridge can withstand when subjected to vessel impact loading.

SECTION 6
CONCRETE AND STEEL DESIGN

6.1 GENERAL

This section includes only those structural requirements that are specifically related to the design of bridge elements to resist vessel impact loading without causing superstructure collapse. It assumes compliance with each load combination according to the *AASHTO Standard Specifications for Highway Bridges*, current edition adopted by AASHTO.

This section is not applicable to the design of sacrificial protection system elements.

6.2 REINFORCED CONCRETE

Design and construction of cast-in-place monolithic reinforced concrete or prestressed, precast, concrete columns, pier footings and connections shall conform to the requirements of the *AASHTO Standard Specifications for Highway Bridges* and to the additional requirements of this Section. Either ultimate limit state, service load or load factor design may be used. If service load design is used, the allowable stresses are permitted to increase by 33⅓ percent.

If plastic hinges are to form in columns, consideration shall be given for adequate confinement at all plastic hinges locations. The minimum transverse reinforcement requirements and spacing of such reinforcement and splices shall be in accordance with Sections 8.4.1(D), 8.4.1(E), and 8.4.1(F) respectively, of the *AASHTO Guide Specifications for Seismic Design of Highway Bridges*.

6.3 STRUCTURAL STEEL

Design and construction of structural steel elements and connections shall conform to the requirements of the *AASHTO Standard Specifications for Highway Bridges* and to the additional requirements of this Section. Either ultimate limit state, service load or load factor design may be used. If service load design is used the allowable stresses are permitted to increase by 50 percent.

SECTION 7

BRIDGE PROTECTION DESIGN PROVISIONS

7.1 GENERAL

This section contains the requirements for the design of bridge piers and spans to protect them from collapse due to vessel collision. As discussed in Section 3, the bridge elements can be designed to withstand the impact loads, or a fender or protection system can be developed to prevent, redirect, or reduce the impact loads to non-destructive levels.

7.2 DESIGN LOADS

The design vessel for each substructure or superstructure element shall be determined by Method II unless the special situations in Section 4.1.2 exist for using Method I or Method III. The design impact force and energy associated with the design vessel shall be computed in accordance with Section 3 requirements.

The design impact force shall be applied to the bridge elements as an equivalent static force. The impact force shall be applied in accordance with Section 3.15.

If the bridge cannot safely withstand the design impact loads, a protection system must be developed to reduce the bridge loads to an acceptable level, or to absorb the loads before they reach the bridge.

7.3 PHYSICAL PROTECTION SYSTEMS

Physical protection systems are protective structures provided on a bridge to fully or partially absorb the design impact loads. The protective structures may be located directly on the bridge (such as a bridge pier fender), or independent of the bridge (such as a dolphin). The geometric configuration of the protective structure can be developed to deflect or redirect the aberrant vessel away from the bridge. The protective structure geometry should be developed to prevent the rake (overhang) of the design vessel's bow from striking and causing damage to any exposed portion of the bridge above the protective structure with the protective structure in its deflected or collapsed position.

Protective structures shall be designed in accordance with accepted engineering practice using either energy or force-acceleration ($F = ma$) methods. Protective structures designed using energy methods shall be in accordance with;

$$KE = \int F(x)dx \qquad (7.3\text{-}1)$$

where

KE = kinetic energy of design vessel (k-ft)
$F(x)$ = protective structure force, F(kips), as a function of deflection, x(ft)

Protective structures shall be designed in accordance with one of the following sets of alternative criteria:

- The total design impact energy, KE, shall be absorbed by the design vessel. The impact energy shall be absorbed by the elastic and plastic deformation (crushing) of the vessel's bow. The bridge or protective structure shall be designed to withstand the design impact loads without significant damage or collapse.
- The total design impact energy, KE, shall be absorbed by a protective system. The impact energy shall be absorbed by the elastic and plastic deformation of the protection system structure without causing significant damage or collapse of the bridge. The vessel absorbs no energy, and no significant vessel damage occurs.
- The design impact energy is absorbed both by the vessel and the protective system. The impact energy is absorbed by the elastic and plastic deformation of both the ship and the protective structure without causing significant damage or collapse of the bridge.

The analysis and design of bridge protection structures requires the use of engineering judgment to arrive at a reasonable solution. In the following sections, the various types of protective structures commonly used for bridges will be briefly discussed.

7.3.1 Fender System

7.3.1.1 Timber Fenders

Timber fenders are composed of vertical and horizontal timber members in a grillage geometry attached to the face of the bridge pier, or erected as an independent structure adjacent to the pier. Energy is absorbed by elastic deformation and crushing of the timber members. Because of their relatively low cost, timber fenders have frequently been used on bridge projects for protecting piers from minor vessel impact forces. However, for the relatively large collision impact loads associated with the design vessels in this Specification, the resulting timber fenders would have to be extremely large, and might be uneconomical in most circumstances.

7.3.1.2 Rubber Fenders

Rubber fenders are commercially available in a wide variety of extruded and built-up shapes. Impact energy is absorbed through the elastic deformation of the rubber elements either in compression, bending, shear deformations, or a combination of all three.

7.3.1.3 Concrete Fenders

Concrete fenders consist of hollow, thin-walled, concrete box structures attached to the bridge pier. Usually, a timber fender is also attached to the outer face of the concrete box fender. Impact energy is absorbed by the buckling and crushing of the concrete walls composing the fender system.

7.3.1.4 Steel Fenders

Steel fenders consist of thin-walled membranes and bracing elements composed in a variety of box-like arrays and assemblies attached to the bridge pier. Impact energy is absorbed by compression, bending, and buckling of the steel elements in the fender. Timber facing should be attached to the steel fender to prevent sparks resulting from direct contact with steel hulled vessels.

7.3.2 Pile Supported Systems

Pile supported structures can be used to absorb collision impact loads. Pile groups connected together by rigid caps provide one method of generating high levels of protection resistance to vessel impact forces. Free standing piles and piles connected by relatively flexible caps are also used for bridge protection. The pile groups may consist of vertical piles, which primarily absorb energy by bending, or batter piles which absorb energy by compression and bending. As a result of the high impact design loads associated with vessel collision, plastic deformation and crushing of the pile structure is permitted provided that the vessel is stopped before striking the pier, or the resulting impact is below the resistance strength of the pier and foundation.

The pile supported protection structures may be either free-standing away from the pier, or attached to the pier itself. Fender systems may be attached to the pile structure to help resist a portion of the impact loads. Timber, steel, or concrete piles may be utilized depending on site conditions, impact loads, and economics.

7.3.3 Dolphin Protection

Large diameter dolphins may be used for protection of bridge piers. Dolphins are typically circular cells constructed of driven steel sheet piling, filled with rock or concrete, and topped by a concrete cap. Dolphins may also be constructed of precast concrete sections, or precast entirely off-site and floated into final position. Driven pilings are sometimes incorporated in the cell design. Design procedures for dolphins are usually based on an estimate of the energy changes that take place during the design impact loading. Energy-displacement relationships are typically developed for the following energy dissipating mechanisms:

- crushing of the vessel's bow
- lifting of the vessel's bow
- friction between the vessel and the dolphin
- friction between the vessel and the river bottom
- sliding of the dolphin
- rotation of the dolphin
- deformation of the dolphin

Deformation of the vessel/dolphin system is assumed to follow a path of least energy. For each potential displacement configuration of dolphin and vessel, a deformation path can be developed. Deformation stops when all the kinetic energy of the impact has been absorbed. For purposes of design, it is recommended that the maximum dolphin deformation be limited to less than one-half the diameter of the cell. Under design loading conditions, the cell is permitted to undergo large plastic deformation and partial collapse.

7.3.4 Island Protection

Protective islands around vulnerable bridge piers provide highly effective vessel collision protection. Islands typically consist of a sand or rock core which is protected by outer layers of heavy rock armor to provide protection against wave, currents, and ice actions. The island geometry should be developed in accordance with the following criteria:

1) vessel impact force transmitted through the island to the bridge pier must not exceed the lateral capacity of the pier and pier foundation
2) island dimensions are such that vessel penetration into the island during a collision will not result in physical contact between the vessel and any part of the bridge pier

The requirement of Item (2) above is particularly critical for empty or ballasted ships and barges which can slide up on the slopes of an island and travel relatively large distances before coming to a stop. In sizing the island, consideration must also be given to the overhang, or rake, distance of a ship or barge's bow which should be added to the required stopping distance of the vessel. The design of the surface armor protection of the islands for wave and current attack shall be based on methodologies used for rubble mound breakwater design established by the U.S. Army Corps of Engineers Shore Protection Manual.

The following items have been identified as sources of energy absorption/dissipation during a vessel impact with an island:

1) Ship
   - change in potential energy of the ship due to change in the vertical position of its center of gravity
   - crushing of the hull of the ship
2) Water
   - generation of water waves and turbulence
3) Island
   - change in potential energy of island material
   - displacement, shear and compaction of the island material
   - friction between the ship and the island
   - generation of shock waves in the island
   - crushing of particles of island material

Inclusion of these items in a design analysis is difficult since their effects are only partially understood; however, simplifying assumptions are made. Physical model studies, as well as mathematical simulations, are usually performed when protective islands are designed.

7.3.5 Floating Protection Systems

Various types of floating protective systems have been used and may be considered by the engineer. Several of these systems include:

- Cable Net Systems: vessels are stopped by a system of cables anchored to the waterway bottom and suspended by buoys located in front of the bridge piers.
- Anchored Pontoons: large floating pontoons anchored to the waterway bottom in front of the piers absorb vessel impact.
- Floating Shear Booms: floating structures anchored to the waterway bottom that deflect aberrant vessels away from piers and absorb impact energy.

Special consideration for corrosion protection must be made for all systems involving underwater steel cables and anchorages.

Special considerations shall be given to function and vulnerability/durability of floating systems during winter time in waters subject to icing or ice drift.

Floating systems are vulnerable to being overridden by vessels with sharply raked bows.

7.4 MOVABLE BRIDGE PROTECTION

Movable bridges are particularly susceptible to interrupted service as a result of vessel collision because even minor impact on the substructure or superstructure can cause mechanical equipment to jam or fail. Movable bridge piers which house mechanical equipment or support movable machinery should be fully protected from vessel contact by aberrant vessels. There should be no contact of the vessel with the pier when the protection system is in the fully deformed position and the vessel has been stopped. Special consideration must be included for the overhang of raked bows on ships and barges.

The navigation spans of all movable bridges should provide a protection system which prevents vessels from laterally contacting the pier or navigation channel superstructure while the vessel is transiting through the bridge. There should be no contact between the vessel and the pier or span while the protection system is in the deformed position.

The superstructure of the movable spans on bascule and swing bridges should be fully protected when they are in an open position. The protection system along the sides of the navigation channel should prevent contact between the vessel and the span in the open position. This is a special concern for bascule bridges in which the movable span leaves in the open position may overhang the pier and are vulnerable to contact by a vessel's superstructure.

Electrical power cables, including submarine cables, should be positioned and supported so as to be fully protected from damage by impact from marine traffic.

Bascule bridge spans are subject to impact damage by marine vessels when spans are in either the open or closed position. Bascule leaves, when in the full open position, should be designed such that an aberrant vessel cannot come into contact with the structure. Although it is impractical to design closed bascule leaves such that marine vessel contact cannot occur, leaf designs should minimize the resultant leaf damage from impacts occurring when the bridge is in the closed or partially open position.

The bridge tender's house should be located such that marine vessel impact will not endanger the bridge tender or bridge controls and operating system.

Mechanical, hydraulic, or electrical systems should not be located on walls susceptible to vessel impact. Precautions should be taken to locate and/or protect drive systems, such as hydraulic systems, drive gearing, motors, and electrical power and control systems from possible damage due to direct or indirect impact damage from marine vessels.

Drive systems for movable bridges should be evaluated to identify single point failures which may result from impact damage. Consideration should be given to providing redundant system elements for single point failures. For those system elements which cannot be fully protected against impact damage, critical items should be conveniently replaceable.

7.5 MOTORIST WARNING SYSTEMS

Motorist warning systems may be used on bridges to minimize the loss of life which may occur in the event of a catastrophic collapse of a bridge during a vessel collision. Motorist safety system components can be grouped into three categories of function: 1) devices to detect hazards, either environmental or man-made, 2) devices to verify hazards or problems, and 3) devices to control traffic and/or pass information to drivers. The motorist safety system for a bridge should be developed using the appropriate items from three separate categories which will interact to address the specific problems anticipated on the bridge.

7.5.1 Hazard Detection Systems

As they relate to ship/bridge collision hazards, devices in this functional category include the following:

- Ship Impact Vibration Detectors - Placed on bridge piers, these vibration sensors would be capable of distinguishing between normal structural vibrations and movements associated with substantial ship impacts.
- Continuity Circuits - This electrical system would utilize pairs of conductors terminating with end-of-line devices attached to the bridge superstructure. Collapse of some portion of the bridge deck would interrupt the circuit continuity.
- VHF Radio Link - The use of this device would be in advance of imminent danger, as foreseen by the pilot or master of a vessel which had, for instance, lost steerage. If the mariner anticipated a possible ship/bridge collision, he would radio the bridge toll booth personnel, or other appropriate agency, via VHF channel 16 (marine emergency channel) in order to halt motorist traffic on the bridge.

Either of the first two of the above devices could activate traffic control/information systems automatically or through a machine-man-machine interface with the human intermediary verifying hazards before interrupting traffic.

VHF radio units are readily convenient in the deckhouse of virtually every merchant vessel. The use of such a system would require the installation of a relatively inexpensive VHF set in a bridge toll plaza, or other appropriate agency, and would require continued monitoring by the Owner's personnel who could make appropriate traffic control decisions.

Detectors for other than ship collision hazards include:

- Weather instrumentation, particularly to measure wind velocity, and
- Electronic loops in the bridge deck to detect non-movement of traffic, indicative of a disabled vehicle or a traffic accident.

The wind velocity data can be used in conjunction with criteria to restrict high surface exposure vehicles and/or to close the bridge to traffic altogether. Detection by the loops of traffic stoppage can automatically activate traffic control/information systems or can alert bridge personnel to verify problems and take appropriate action.

7.5.2 Verification Devices

Virtually any detection device can be electronically linked to traffic control/information equipment to automatically warn or stop traffic. Theoretically, such automatic links are relatively simple in design; however, in actual practice, considerable difficulty can be experienced with false alarms and unnecessary interruptions of traffic.

In many instances, agencies implementing motorist safety systems have opted to place a "verification" function into the systems whereby initial alerts from the detection equipment are checked by personnel before traffic control actions are taken. Included among possible verification methods are:

- Closed Circuit Television (CCTV) - Permanently installed, rotational, remote controlled, closed circuit cameras can be placed strategically to allow personnel at a monitor site to view the bridge main span, the ship channel, the roadway, or any other feature desired.
- Visual Delineation - In this relatively simple system, the top of the bridge parapet or guardrail would be fitted with a series of reflectors or lights. A collapse of a portion of the bridge superstructure would appear to system control personnel as a discontinuity in the series of delineators.
- Motorist Call Boxes - Numerous states have implemented systems of motorist aid call stations along roadways and bridges in recent years. Among systems in place are code radio transmitting devices on which the motorist selects push-button options specific to the type of assistance desired, telephone systems which allow two-way communication between the motorist and system control personnel or law enforcement dispatchers, and two-way radio communication providing similar service as telephone systems, but without dependence on phone company lines.

Using either of the first two of the above systems, alarms received from ship accident detector systems can be verified and decisions made as to appropriate action to be taken. However, there are difficulties and shortcomings to any verification system, including the requirement to establish a monitoring site or sites; maintaining skilled, salaried personnel on an around-the-clock basis; and technical problems of using CCTV for visual verification in darkness or low visibility.

When visual pictures can be acquired via CCTV, it is often difficult to determine the specific type of assistance required (e.g., tow vehicle, ambulance, fire-rescue).

7.5.3 Traffic Control and Information Devices

Whether the hazard detector device information is used automatically or is manually verified, the ultimate function of a motorist safety system is to appropriately control traffic or inform motorists of hazards. The following devices can be used to accomplish this function:

- Variable Message Signs - Virtually any message can be transmitted via this device, including warnings of catastrophic bridge failure, environmental hazards, traffic congestion, construction/maintenance activity, etc.
- Flashing Beacons - Used in conjunction with standard format warning signs (diamond-shape, black legend and border, yellow back-ground), this device can be used to bring attention to a warning message.
- Movable Gates - Usually fitted with flashing red lights and an audio alarm (siren or bell), this device can be lowered across traffic lanes to halt motorists (as at railroad crossings).

7.6 AIDS TO NAVIGATION ALTERNATIVES

Improvements in navigation within the navigable channel at a bridge location will often result in a significant reduction in the vulnerability of a bridge to vessel collision. Since 60 to 85 percent of all vessel collision accidents are attributed to pilot error, it is important that all aspects of the bridge design, siting, and aids to navigation with respect to the navigation channel be carefully evaluated with the purpose of improving or maintaining safe navigation in the waterway in the vicinity of the structure.

The bridge designer is very limited in his ability to require any modifications which affect operations on a navigable waterway since the responsibility and authority for implementing such navigation improvements within the U.S. waterways belongs to the U.S. Coast Guard. Regardless of the question of design responsibility, the following discussion will highlight various aspects of navigation alternatives which should be considered in light of vessel collision with bridges.

7.6.1 Operational Alternatives

Operational alternatives should be considered as a means of improving safety near bridges, reducing the consequences of a collision, or reducing the required level of pier protection. Some of these alternatives are:

- imposing speed limits for vessels
- mandatory pilotage requirements
- mandatory tug assistance requirements
- minimum weather standards to transit under the bridge
- restrict vessel passage during high currents
- restrict large vessels to daylight transits only
- require empty vessels to take on ballast for a minimum draft
- impose traffic separation schemes
- impose advanced VTS systems (Vessel Traffic Service)
- require radiotelephone communication between ship and bridge personnel

7.6.2 Standard Navigation Alternatives

Alternatives to be considered include:

- placement of ranges on inbound/outbound channels near the bridge
- additional buoys and buoy placement near the bridge
- radar reflectors and lights on all buoys near the bridge
- high intensity light beacons on the bridge structure
- sound devices (fog horns) on the bridge
- RACON device on the bridge structure main span at the centerline of the channel for improved radar image on the vessel

7.6.3 Electronic Navigation Systems

The use of advanced navigation systems for vessels transiting under a bridge structure have shown significant reductions in the probability of aberrancy by pilots under simulator conditions. These include both advanced shorebased radar VTS systems with real-time surveillance capabilities, as well as small portable navigation units carried on board by the master or pilot of the vessel.

SECTION 8

BRIDGE PROTECTION PLANNING GUIDELINES

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.

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 Section 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 crosscurrents 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 this Guide Specification.

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 Section 4.8.3.3. Within this area (3xLOA 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 and vertical clearances between piers.

8.5 NAVIGATION SPAN CLEARANCES

8.5.1 Horizontal Clearances

Figure 8.5.1-1 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 2 or 3 times the design vessel length, LOA, are particularly vulnerable to vessel collision.
- Bridges with main spans, S, less than 2 times the channel width, C, are particularly vulnerable to vessel collision.
- Piers located less than 2 or 3 times the pier width from the edge of channel, Y_N and Y_W, are particularly vulnerable to collision.
- The centerline of the navigable channel should coincide with the center of the main span. The maximum offset between the centerline of the channel and of the bridge should not exceed 10-15% of the main span length, S.

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 this Guide Specification. Based on historical ship collision data, twice as many accidents have occurred with approach piers spans 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; and
- 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

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.

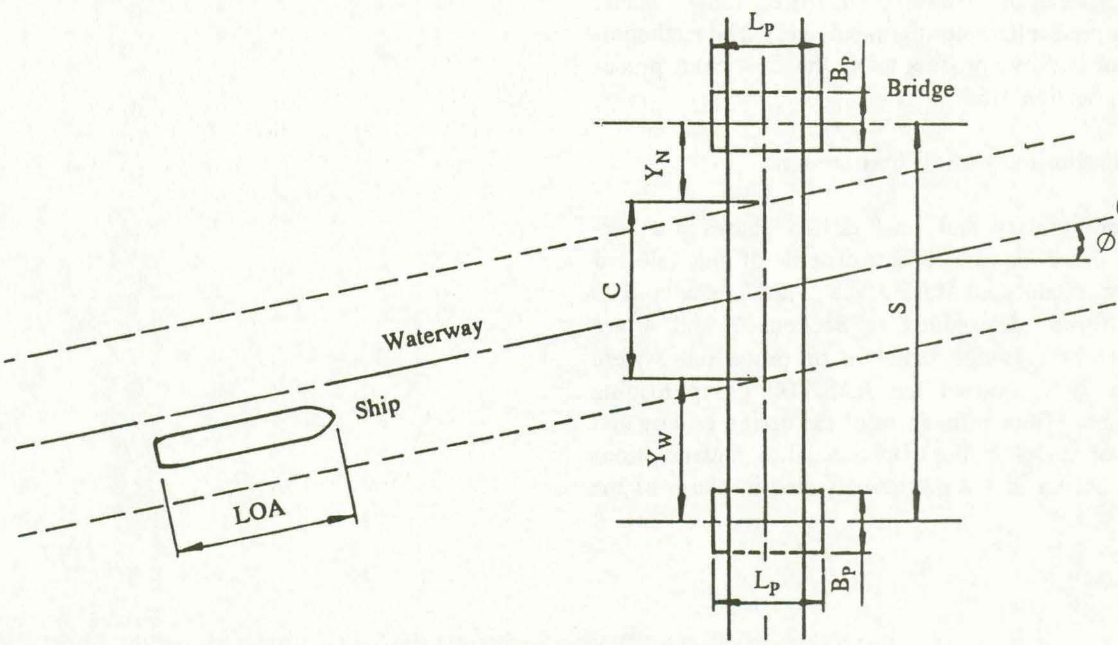


Figure 8.5.1-1. Bridge/Waterway Planning Geometry.

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 Sections 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&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&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&L study and before detailed preliminary and final design of the structure begins.

The goal in the T,S&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 Section 1.5.

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&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.

COMMENTARY
SECTION 1 - INTRODUCTION

BACKGROUND

Ship and barge collisions with bridges represent a growing and serious threat to public safety, port operations, motorist traffic patterns, and environmental protection in many cities throughout the world that are located in coastal areas and along inland waterways. The direct inclusion of ship and barge impact loads on bridge structures have historically been neglected in bridge design. Reasons for neglecting these potential forces have included a belief that the probability of such a catastrophic collision was very small; that vessel collisions are an "act-of-God" and can therefore be excluded; that it's impossible to protect bridges from impact by large merchant vessels; that it's uneconomical to design bridges to resist impact forces; and that no suitable theories for determining vessel impact forces existed. It is of interest to note that similar reasons were sometimes applied to earthquake loadings on bridge structures prior to the development of seismic design specifications.

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 at 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 [1,2].

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.

HISTORICAL COLLISIONS

A study of river towboat collisions with bridges located on the U.S. inland waterway system during the period 1970-1974 revealed that there were 811 accidents with bridges costing 23-million dollars in damages and 14 fatalities [3].

In a paper given at the International Association for Bridge and Structural Engineers (IABSE) Colloquium on Ship Collision with Bridges and Offshore Structures [4], Copenhagen, 1983, Frandsen summarizes the account of 22 catastrophic ship/bridge accidents from the period 1960-1982 [5]. Each accident resulted in a collapse of a portion of the bridge involved with the aggregate loss of life being almost 100 people. The list of accidents is quite cosmopolitan and includes the 1964 collapse of the Maricaibo Bridge, Venezuela; the 1964 and 1974 collapses of the Pontchartrain Bridge, Louisiana, USA (9 fatalities); the 1975 collapse of the

Tasman Bridge, Hobart, Australia (15 fatalities and the ship sank); the 1979 collapse of the Second Narrows Railway Bridge, Vancouver, Canada; the 1980 collapse of the Almo Bridge, Almosund, Sweden (8 fatalities); the 1980 collapse of the Sunshine Skyway Bridge, Florida, USA, in which there were 35 fatalities; and the 1982 collapse of a gas pipeline bridge across the Mosel River, Lorraine, France, in which there were seven fatalities due to the escaping gas from the destroyed pipeline. Frandsen notes that the annual rate of catastrophic collisions increased from 0.5 bridges per year in the decade 1960-1970 to 1.5 bridges per year in the period 1971-1982.

Since 1982, there have been several additional serious accidents. In July, 1983, between 100 and 200 people were killed when a passenger ship rammed into a railway bridge across the Volga River near the City of Ulyanovsk in the Soviet Union. The exact number of fatalities in this accident is not known because of Soviet policy against outside publication of such disasters. According to news reports, those killed were watching a film in a hall on the upper deck of the ship when it was torn off during the collision with the bridge. In June 1984, a portion of the Pontchartrain Bridge in Louisiana, USA, again collapsed when rammed by a tug/barge. In December 1985, the St. Louis Bridge across the St. Lawrence Seaway near Valleyfield, Quebec, Canada, was partially destroyed when a freighter rammed into one of the side spans of the lift bridge structure. The Seaway was temporarily closed because of the collision.

The above overview does not include the numerous incidents of collisions with bridges which do not lead to total collapse, inland waterway bridge collisions, or to the many "near-misses" which occur with uncomfortable regularity along the world's busy waterways.

DATA BASE

A comprehensive literature review of the current domestic and foreign practice, experience, and research findings available on the subject of vessel collision with bridges was performed during the development of the Guide Specification. Information from this review is provided in the Commentary as background data for the design methodologies presented. Data utilized as the basis of this Specification is contained in the References section of this Commentary.

An important source of data was the 1983 International Association of Bridge and Structural Engineers (IABSE) Colloquium on Ship Collisions with Bridges and Offshore Structures, Copenhagen, Denmark [4]. The Marine Board Report [6] on ship/bridge collisions, and the 1981 Conference on Bridge Protection Systems at Stevens Institute of Technology [7] also contained valuable information. Additional data were obtained from personal contacts with researchers and government officials worldwide, the AASHTO Committees overseeing the development of the Guide Specification, and from the experience of the project consultants.

DESIGN PHILOSOPHY

The basic design philosophy embodied in the Specification 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 Specification 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 defined. Two importance classifications were defined to classify bridges according to Social/Survival and Security/ Defense requirements.

The Specification and design philosophy have been structured in accordance with the recent edition of the *AASHTO Standard Specifications for Highway Bridges* [8], and in many areas to reflect the principles utilized in the *AASHTO Guide Specifications for Seismic Design of Highway Bridges* [9]. The Importance Classification (IC) in particular was taken almost directly from the Seismic Specification.

SYMBOLS AND DEFINITIONS

The symbols and definitions which apply to the Commentary are defined in the Commentary and are not included in Section 2.

ACCURACY

The designer is cautioned that many of the Specification equations 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 Specification equations is limited, and the use of the equation results to many significant figures is not warranted. Engineering judgment should be used to round the equation results to establish design values to be used in applying the Specification provisions.

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COMMENTARY

SECTION 3 - GENERAL PROVISIONS

C3.2 APPLICABILITY OF SPECIFICATION

The Guide Specification presents vessel collision design requirements applicable to the majority of highway bridges crossing navigable waterways to be constructed in the United States. The Specification was developed for steel-hulled merchant vessels and barges and is not applicable to vessels constructed of other materials, recreational vessels, or ships smaller than 1,000 DWT.

The Specification specifies minimum requirements. More sophisticated design and/or analysis techniques may be utilized if deemed appropriate by the design engineer and approved by the Owner.

It must be emphasized at the outset that the specification of vessel impact loads and bridge protection requirements cannot be achieved solely by following a set of scientific principles. First, the causes of vessel collision are not well understood, and experts do not fully agree on how available knowledge should be interpreted to specify the impact loads for use in design. Second, to achieve workable bridge design provisions it is necessary to simplify the enormously complex matter of vessel impact occurrence, vessel impact forces and motions, and bridge response. Finally, any specification of vessel impact loadings and pier protection requirements involves balancing the risk of that impact occurring against the cost to society of requiring that structures be designed to withstand that loading. Therefore, judgment, engineering experience, and political wisdom are as necessary as scientific knowledge.

The recommended vessel impact methodologies are the work of the project consultants and are based upon the best scientific knowledge available in 1990, adjusted and tempered by experience. Throughout the following sections, explanations for the various recommendations are provided as a guide both for the user of the Specification and to those who will improve the Specification in the future.

A great deal of future research is required to understand the complicated processes involved in a vessel collision with bridge structures. It is expected that the methodology and key assumptions used in the Specification will change with time as the profession gains more knowledge about vessel collision loads, probability of collision, and bridge response, and as society gains greater insight into the process of establishing acceptable risk criterion.

C3.3 IMPORTANCE CLASSIFICATION

The Importance Classification (IC) is used in conjunction with the acceptable risk criteria to establish the design vessel used to determine vessel impact loadings for bridges. Two Importance Classifications are specified, 1) Critical Bridges, and 2) Regular Bridges.

Critical bridges are those that must continue to function after impact from a design vessel whose probability of occurrence is smaller than other, regular, bridges. The determination of the Importance Classification is necessarily subjective. Consideration should be given to the Social/Survival and Security/Defense requirements discussed below. Additional considerations should be the availability of alternate detour routes and the average annual daily traffic. The latter consideration provides an indirect means of assessing the loss of life which might occur in the event of collapse of portions of the bridge superstructure.

The Social/Survival evaluation is largely concerned with the need for roadways connecting the communities located on opposite sides of the waterway together. In order for civil defense, police, fire department or public health agencies to respond to an emergency situation which might exist on the opposite side of the waterway a continuous route must be provided. Bridges on such routes should be classified as critical.

Transportation routes to essential facilities such as hospitals, police and fire stations and communications centers must continue to function and bridges required for this purpose should be classified as critical.

The well-being of the community is another major concern. Bridges which carry very high volumes of motorist traffic and those which provide routes to such facilities as schools, arenas, power installations, water treatment plants, etc., should suffer little or no damage and bridges on such routes should be classified as critical.

The importance evaluation of a bridge for Social/ Survival significance in an emergency or disaster situation depends on the range of options available and the possibility of a bridge being in parallel or series with other bridges in a roadway network. Discussion may be required with highway, civil defense and police officials.

An example of Social/Survival consequences was dramatically illustrated by the collapse of the Tasman Bridge, Hobart, Australia, in 1975 as a result of a ship collision. The Tasman Bridge was closed for 33 months while repairs were made to the structure. The nearest alternative river crossing was located 30 miles away. During the repair time, investigators found an increase in suicide rates, divorce, bankruptcy, crime, and illness in the bedroom community on the eastern shore of the Derwent River which had been severed from all hospital, medical, police, and other social services located in the City of Hobart on the western shore and linked by the bridge.

A basis for the Security/Defense evaluation is the 1973 Federal-Aid Highway Act which required that a plan for defense highways be developed by each state. This plan had to include, as a minimum, the Interstate and Federal-Aid Primary routes; however, some of these routes can be deleted when such action is considered appropriate by a state. The defense highway network provides connecting routes to important military installations, industries and resources not covered by the Federal-Aid Primary routes and includes:

- military bases and supply depots and National Guard installations;
- hospitals, medical supply centers and emergency depots;
- major airports;
- defense industries and those that could easily or logically be converted to such;
- refineries, fuel storage, and distribution centers;
- major railroad terminals, railheads, docks, and truck terminals;
- major power facilities and hydroelectric centers at major dams;
- major communication centers;
- other facilities that the state considers important from a national defense viewpoint or during emergencies resulting from natural disasters or other unforeseen circumstances.

Bridges serve as important links in the Security/ Defense roadway network and such bridges should be classified as critical.

C3.4 DATA COLLECTION

Essential data for use of the specification methodologies includes a description of the vessel traffic passing under the bridge, vessel transit speeds and environmental conditions. Sources for obtaining data on these items include the following:

- Waterborne Commerce of the United States (WCUS), Parts 1 thru 5 [1]. This document contains statistics on the commercial movement of foreign and domestic cargo on a calendar year basis. Included are detailed data by commodity and number of vessel trips for U.S. harbors and waterways. The number of vessel trips is arranged by vessel draft rather than vessel size (DWT) which is a limitation of these publications.
- Waterborne Transportation Lines of the United States [2]. This document contains information of vessel operations of American flag vessels operating in U.S. waterways on a calendar year basis. Included are vessel operators and their addresses; vessel description based on type, construction, net registered tonnage (NRT) length, breadth, draft, horsepower, vertical clearance, etc. Also, included is a description of operations, type of service, principal commodities carried and localities served. This document is an excellent source of data for inland waterway barge tow operations. It is of limited use for ship data since foreign flag vessels (which constitutes the large majority of coastal port usage) are not included.
- Lock Performance Monitoring (LPM) Reports [3]. The report data contains valuable information on vessels using the inland waterway system. The data consists of information describing vessel traffic through locks as well as physical dimensions of locks, significant weather, and navigation conditions. Vessel data includes vessel name, flotilla dimensions, barge types, number, and tonnage.
- U.S. Army Corps of Engineers (COE) District Offices. The Navigation Department for the local COE District may have publications on past or proposed channel modifications in the waterway which might contain valuable vessel data statistics.
- U.S. Coast Guard, Marine Safety Office (MSO). The local MSO maintains daily arrival/departure

logs for the waterways within its jurisdiction. Included information in the logs are the vessels names and berths while in the harbor. This allows a determination as to whether the vessel called a facility upstream or downstream of the proposed bridge site. The arrival/departure log for each vessel trip must be reviewed which can be very time consuming.

- Port Authorities. Local port authorities and operators often maintain logs of vessel trip data for their waterway area. This data can be useful in developing vessel frequency data for the specific bridge location.
- Pilot Associations. Discussion with local harbor pilots and towboat operators is essential for determining vessel operating conditions, speed, and transit paths for the waterway. The local pilots and towboat operators are regular users of the waterway and have valuable information on the local wind and current conditions, waterway geometry, traffic density, and vessel transit speeds which are critical factors in the vessel impact methodologies.
- National Oceanic and Atmospheric Administration (NOAA) of the U.S. Department of Commerce [4]. The following NOAA documents are available for most waterways; Tide Tables, Tidal Current Tables, Tidal Current Charts, United States Coast Pilots, Distance Tables, and Nautical Charts. The Nautical Charts are particularly useful for establishing the channel geometry and water depths in the waterway.

C3.5 VESSEL TYPE AND CHARACTERISTICS

The two basic vessel types identified for use in the Specification are ocean-going ships and inland barges. The basic unit of measurement in defining the vessel size shall be the vessels' deadweight tonnage (DWT). This, and other measurement criteria that might be encountered by the bridge designer, are listed below:

- Deadweight Tonnage (DWT) is the weight of cargo, fuel, water and stores necessary to submerge a vessel from her light draft to her loaded draft. This tonnage should not be confused with the weight of the ship. It is the "dead" weight of the cargo, etc. in distinction to the "live" weight of the ship.
- Displacement Tonnage (W) is the weight of the vessel. The weight of the vessel including fuel, stores, cargo, etc. when she is floating at her deepest possible draft is known as the loaded displacement (W_L). When the vessel is completely empty, her weight is known as light displacement (W_E).
- Gross Registered Tonnage (GRT) is calculated by measuring in cubic feet the total internal volume of a vessel (less certain exempted spaces) and dividing by 100. GRT is not a measurement of weight - but of volume. One "registered" ton, by law, represents 100 cubic feet of a vessels' internal space.
- Net Registered Tonnage (NRT) is computed by deducting from GRT most spaces which are not used for the carriage of cargo or passengers. As in GRT, NRT is a measure of volume and not of weight.

Deadweight tonnage (DWT) and displacement tonnage for ships (W) are expressed in tonnes (2,205 pounds). Deadweight tonnage and displacement tonnage for inland barges are expressed in tons (2,000 pounds). The designer should exercise care when researching vessel traffic and commodity data since some Federal, State and local port agencies use the short ton (2,000 pounds) to report vessel cargo statistics.

C3.5.1 Barge Vessels

Three basic types of barges are in use on the inland waterway system; hopper (open and covered), deck, and tank barges. Barge sizes vary widely depending on the type of cargo and the waterway characteristics (including navigation locks). The barge data in Tables 3.5.1-1 and 3.5.1-2 were adapted from [5]. The three typical barge sizes and tow data shown in Figures 3.5.1-1 and 3.5.1-2 were based on research conducted on the inland waterways of Louisiana (including the Mississippi River) by the consulting firm of Modjeski and Masters [6]. The applicability of the Figure 3.5.1-1 barge sizes should be verified by the bridge designer for other locations. As an example, the typical standard hopper barge on the upper tributaries of the Ohio River is 175 feet by 26 feet due to locks smaller than the modern standard of 600 feet by 110 feet.

Barges are pulled/pushed by towboats. Towboats vary in size by their function and the reach of the inland waterways system on which they operate. Towboats typically push tows of 4 to 40 barges between major terminals and port areas. The larger tows are found on the Lower Mississippi River below St. Louis where river width and depth permit their operation [6]. Tows of 15 or more barges are found on other major rivers on the inland system. The Missouri, the upper reaches of the Mississippi, and several tributaries of the Ohio have small locks and/or restrictive channels which limit tow size to less than 10 barges. Small harbor tugs are often used to move barges (usually up to four) within a port or harbor area. The designer must establish typical tow sizes for the waterway/bridge location in order to use the specification methodology.

A source for barge tow data is the U.S. Army Corps of Engineers, Water Resources Support Center, Fort Belvoir, Virginia, which keep records at all of its lock locations on barge tow activity, and the American Waterway Operators, Inc., Alexandria, Virginia, which is a trade association representing the inland waterway transportation industry. Logbooks maintained at moveable bridge locations by State Departments of Transportation are often helpful in obtaining statistics on vessel activity in a waterway.

C3.5.2 Ship Vessels

Ship characteristics vary considerably depending on the size, draft, and type of cargo being carried by the vessel. The three broad classes of bulk carriers, product carriers/tankers, and freighter/container vessels presented in the Guide Specification cover the majority of vessels using U.S. Waterways. Special ships such as passenger ships, LASH vessels, LNG carriers, and naval vessels are not included in the data presented in the Guide Specification. Data for such special ships will require additional research and judgment by the designer if they form a significant portion of the vessel traffic in a particular waterway.

Whenever possible, the ship characteristics associated with the particular vessels using a waterway should be developed by the designer. If such data are not readily available, the typical ship data shown in Figures 3.5.2-2, 3.5.2-3, and 3.5.2-4, and Tables 3.5.2-1, 3.5.2-2, and 3.5.2-3 should be utilized. The ship data in these figures and tables were developed from [7], [8], and [9] along with supplemental data by the project consultants.

One of the most difficult statistics to obtain for merchant ships is vertical clearance data. Practically all published records on ship data do not include this vital statistic which is so important to bridge designers. The information in Figures 3.5.2-5, and 3.5.2-6 for ship mast and deckhouse clearances was developed from scaled vessel drawings representing over 2,300 merchant ships provided by Scott in [7] and [8]. Figure C3.5.2-1 is a plot of the data for mast height clearances for freighter/ container vessels illustrating the typical scatter in vessel dimensions.

Another difficult statistic to obtain, and one which significantly affects the vulnerability of a bridge to vessel collision and vertical clearance requirements, is the ballasted draft of vessels which are transiting the waterway in either an empty or partially loaded condition. All merchant ships have the capability to pump water into special compartments located in the bow, and along the sides of the ship in order to minimize exposed freeboard and increase the vessel draft for proper steering and maneuverability when the vessel is traveling either empty or partially loaded. The degree to which the ship is ballasted depends solely upon the ships' master or pilots' decision given the wind and current conditions in a particular waterway. Unless otherwise determined from the actual vessels using the waterway, a ballasted ship geometry as shown in Figure 3.5.2-4, is recommended for design based on observations and discussions with ship operators and pilots.

The ability of a ship to strike a bridge pier or superstructure element is limited primarily by the water depths in the waterway and navigable channel and the geometry of the bridge and the ship. The determination of ship draft (loaded and ballasted) and available water depth are key factors in the vessel impact analysis. In utilizing the typical ship data in Section 3.5.2, the bridge designer should be aware that very large merchant ships frequently transit U.S. waterways in partly loaded conditions due to draft limitations in the navigable channel. Modifications to the typical data must be made to account for such vessels. As an example, an 80,000 DWT bulk carrier with a fully loaded draft of 45.6 feet, would typically transit partly loaded with a draft of only 33.0 feet in a channel whose limiting water depth was 35.0 feet. In this condition the mast height and bow height above the waterline would be 10.6 feet higher than then fully loaded.

For vessels transiting in other than a fully loaded condition, the displacement weight can be estimated by:

$$W = \frac{(C_B)(L_W)(B_M)(D_M)}{W_W} \qquad \text{(C3.5.2-1)}$$

where

- W = displacement weight of vessel (tonnes);
- C_B = block coefficient (dimensionless);
- L_W = length of vessel waterline (ft);
- D_M = mean draft (ft);
- B_M = mean breadth (ft);
- W_W = 34.4 (cubic feet per tonne of salt water); 35.4 (cubic feet per tonne of fresh water).

The block coefficient, C_B, is the ratio of the immersed section of the vessel to a block having the same length, width, and depth. C_B can be estimated by substituting W_L and other values for the fully loaded

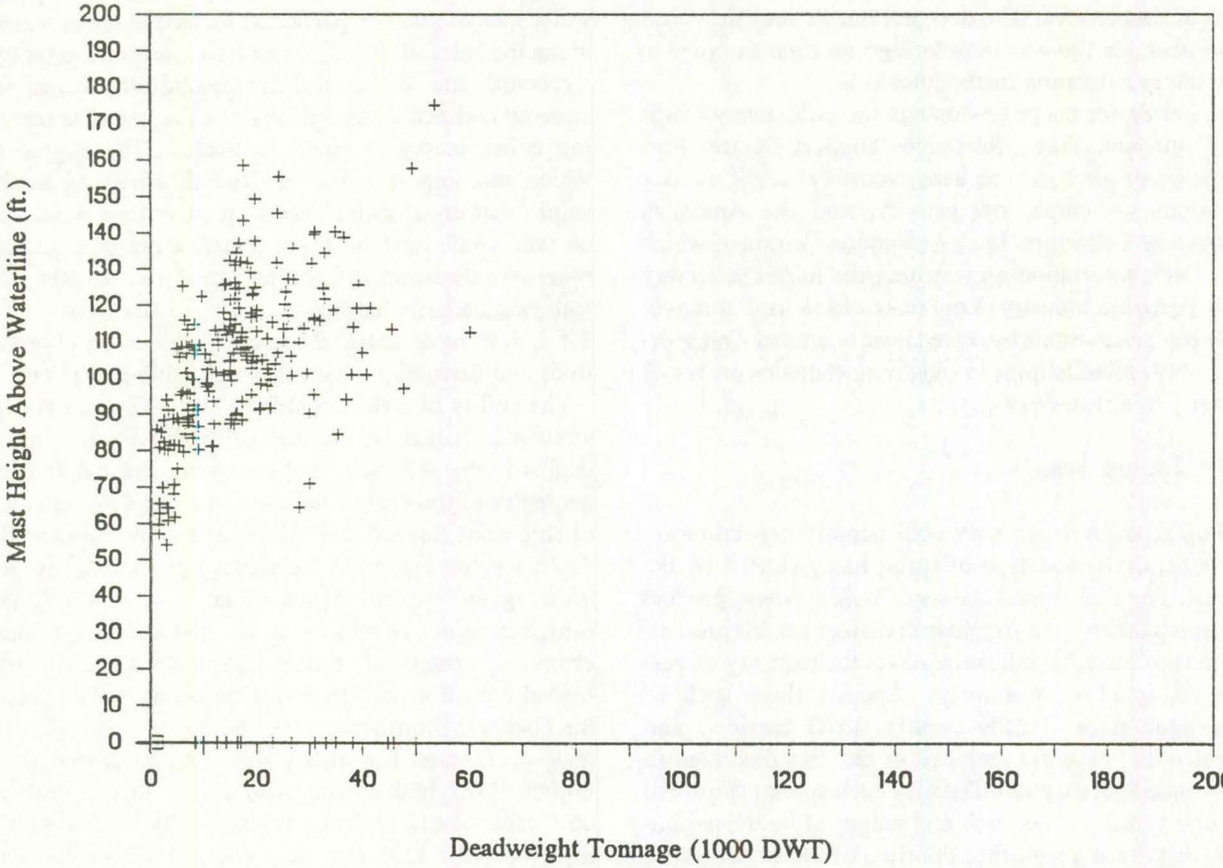


Figure C3.5.2-1. Typical Mast Height Clearance Data for Loaded Freighter/Container Ships. (See Figure 3.5.2-5)

vessel into Equation C3.5.2-1. Using the same value of C_B, the displacement weight of the vessel for other drafts representing different loading conditions and drafts can be estimated using Equation C3.5.2-1. Setting L_W = LOA is usually assumed in using Equation C3.5.2-1 since data on L_W is difficult to obtain. The use of LOA results in no significant error in computing C_B and W.

C3.6 DESIGN VESSEL

Three alternative methods for selecting the design vessel for collision impact are presented in the Guide Specification. Method II shall be used for all bridge design unless approval by the Owner and the special situations stated in Section 4.1.2 exist. Methods I, II, and III vary from relatively simple to use to relatively complex. All of the methods are suitable for manual computation. A brief overview discussion of each method is presented below. A detailed discussion of each method is presented in the Commentary on Section 4.

1) Method I is a semi-deterministic procedure for selecting the design vessel for collision impact. Method I is the simplest of the three methods to use, but is also the most conservative, resulting in higher impact forces than those developed in Method II.
2) Method II is a probability based (risk) analysis procedure for selecting the design vessel for collision impact. Significantly more complicated than Method I, Method II requires a relatively large amount of data to conduct the analysis. The use of the Method II probability procedures results in a more realistic assessment of the risk of vessel collision with a bridge structure, and therefore a

more accurate selection of the appropriate collision impact loads.

3) Method III is a cost-effectiveness analysis procedure for selecting the design vessel for collision impact. The determination of annual frequency of bridge collapse, AF, required in Method III shall be computed using Method II. The disruption costs associated with a potential bridge collapse are evaluated using standard benefit/cost (B/C) analysis to determine the cost-effectiveness of bridge strengthening or bridge protection measures.

C3.7 DESIGN IMPACT SPEED

The selection of the design impact speed is one of the most significant design parameters associated with the vessel collision Specification. Judgment must be exercised by the designer in determining the appropriate design speed for a vessel transiting the waterway. The chosen speed should reflect the "typical" transit speed of the design vessel under "typical" conditions of wind, current, visibility, opposing traffic, waterway geometry, etc. A different vessel speed may be requirèd for inbound vessels than for outbound vessels given the presence of currents which may exist in the waterway.

In general, the design speed should not be based on extreme values representing extreme events such as flooding, hurricanes, and other extreme environmental conditions. Vessels transiting under these conditions are not representative of the "annual average" situations reflecting the typical transit situations.

The use of a triangular distribution of vessel impact speed across the length of the bridge and centered on the centerline of the vessel transit path (Figure 3.7-1), reflects a departure from previous models of vessel collision risk assessment. The recommended use of a triangular distribution was based on the project consultants review of accident case histories during the Guide Specification development. While the data is certainly sparse, it seems clear that aberrant ships and barges which collide with bridge piers further away from the channel are moving at reduced speeds than those piers located closer to the navigable channel limits. Aberrant vessels located at large distances from the channel are usually drifting with the current. Aberrant vessels located very near the channel are moving at speeds approaching the speeds of ships and barges in the main channel.

The exact distribution of the speed reduction is unknown. However, a triangular distribution was chosen for the Specification because of its simplicity, as well as its reasonableness in modeling the aberrant vessel speed situation. As shown in Figure 3.7-1, the typical vessel transit speed in the waterway is constant to the edge of the channel at which point it decreases to the minimum design speed value at a distance 3xLOA from the centerline of vessel transit path. The use of the distance 3xLOA to define the limits at which the design speed becomes equal to the water current was based on the observation that very few accidents (other than drifting vessels) have historically occurred beyond that boundary. Additional discussion of historical accident data is contained in Section C4.8.3.3.

C3.8 VESSEL COLLISION ENERGY

Equation 3.8-1 in the Guide Specification was developed using the standard relationship for computing the kinetic energy, KE, of a moving body as:

$$KE = \frac{m(V)^2}{2} = \frac{W(V)^2}{2g} \qquad \text{(C3.8-1)}$$

where

m = mass of the vessel;
g = acceleration of gravity;
W = vessel displacement tonnage;
V = vessel impact speed.

Expressing KE in kip-feet, W in tonnes (1 tonne = 2.205 kips), V in fps, g = 32.2 fps^2, and including the hydrodynamic mass coefficient, C_H, in Eq. C3.8-1 results in the Specification equation:

$$KE = \frac{2.205(C_H)(W)(V)^2}{(2)(32.2)}$$

$$= \frac{(C_H)(W)(V)^2}{29.2} \qquad \text{(3.8-1)}$$

Included in this equation is a hydrodynamic mass coefficient, C_H, to account for the influence of the surrounding water upon the moving vessel.

It is difficult to find a single value for C_H because of the many factors which influence its magnitude. Reference [10] provides an extensive discussion of the various investigations which have been conducted to measure and compute C_H associated with vessel berthing and fender design, and discusses the wide scatter of the reported results. On the basis of its investigation, Reference [10] states that unless the designer has good reasons to adopt other values, to assume C_H to range between 1.5 (for large underkeel clearances) and 1.8 (for small underkeel clearances) for computing the kinetic energy associated with ship berthing. These values apply to ships which are approaching a berthing wharf from a lateral (broadside) direction. During such lateral motions a relatively large mass of water moves with the vessel. For vessels moving in a forward direction however, a smaller mass of water moves with the vessel, and therefore the values of C_H are smaller than those encountered in berthing maneuvers.

One of the basic concepts of the Guide Specification is that the impact loadings represent the worst-case, head-on collision situation with the vessel moving in a forward direction at relatively high speed. For acceleration in the direction of the ship's length, and for waterways with large underkeel clearances, a constant value of $C_H = 1.05$ may be used according to Saul and Svensson [11]. For waterways with small underkeel clearances, the 1.05 value was increased by the ratio (1.8/1.5) to the approximate value of $C_H = 1.25$, which is similar to the increase in hydrodynamic mass discussed in the previous paragraph for vessel berthing.

While not a requirement in the Specification, the ability to compute the impact energy due to an oblique collision is often times useful. The collision energy, KE, to be absorbed by either the vessel or the bridge structure during a collision event, E, is a function of the impact angle, α, and the coefficient of friction, μ, between the colliding vessel and the bridge structure. Research by Saul and Svensson [11] indicates the following relationship:

$$E = \eta\,(KE) \qquad \text{(C3.8-1)}$$

Values of η are shown in Figure C3.8-1 as a function of the impact angle and coefficient of friction based on research by Woisin, Saul, and Svensson.

C3.9 SHIP COLLISION FORCE ON PIER

The determination of the impact load on a bridge structure during a ship collision is extremely complex and depends on many factors such as the structural type and shape of the ship's bow, the degree of water ballast carried in the forepeak of the bow, the size and speed of the ship, the geometry of the collision, and the geometry and strength characteristics of the bridge pier.

European, Japanese, and U.S. experimentation utilizing physical and mathematical models for collision tests of various types of vessels have resulted in the development of several empirical relationships for estimating the crushing load of a ship's bow, which is an upper limit for the collision force on the bridge [13].

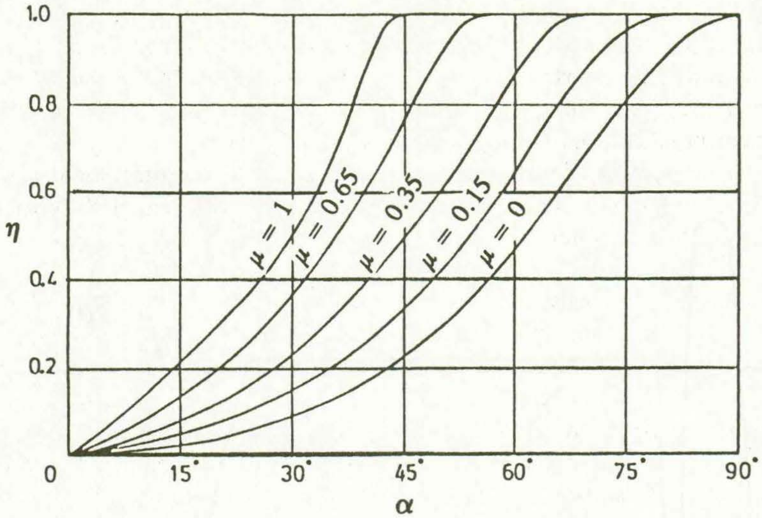
Equation 3.9-1 of the Guide Specification was primarily developed from research conducted by Woisin in Hamburg, West Germany, in 1967-76 [14] to generate collision data to protect the reactors of nuclear powered ships from collisions with other ships. The ship collision data resulted from a total of 24 collision tests with 12 pair of physical ship models at scales 1:12 and 1:7.5 as shown in Figure C3.9-1. Woisin's results have been found to be in good agreement with research conducted by other ship collision investigators worldwide [13].

A schematic representation of the typical dynamics of impact force over time is shown in Figure C3.9-2 for Woisin's model tests. An oscillation of the striking mass and impact force occurred during the initial phase of the impact with a duration of 0.1 to 0.2 seconds for a real ship. During this phase the amplitude sometimes increased to twice the mean value of the impact force. Unfortunately, however, accurate collision force-time histories were not obtained during the testing due to electronic measuring difficulties in the instrumentation and induced vibrations in the model test set-up. As a result, it was not possible to evaluate the compression phase of the impact over time, P(t). Woisin did compute the mean impact force, $\overline{P}(a)$, averaged over the bow damage depth, a, by dividing the loss in kinetic energy, KE, by the bow damage depth. A typical plot of impact force and energy over the bow damage depth is shown in Figure C3.9-3. Based on theoretical and model test results, Woisin developed the following relationship between the mean impact force averaged over time, $\overline{P}(t)$, and the mean impact force averaged over the damage depth, $\overline{P}(a)$;

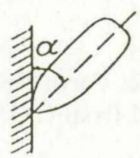
$$\overline{P}(t) = (1.25)\overline{P}(a) \qquad \text{(C3.9-1)}$$

The major influences affecting the mean impact force arranged in order of decreasing importance by Woisin were; 1) ship size (DWT), 2) type of ship, 3) shape and structure of bow, 4) amount of ballast water in the bow, and finally 5) the impact speed. Because of these varying influences, a ± 50 percent scatter in the ship impact force was measured as shown in Figure C3.9-4. The scatter in impact forces about the mean force, $\overline{P}(t)$, approximately followed a triangular probability density distribution as shown in Figure C3.9-5.

From Woisin's data the following basic equation for the mean ship impact force (first reported by Saul and Svensson [11]) was developed for bulk carriers larger than 40,000 DWT colliding with a rigid body at a speed of approximately 16 knots:



$$\eta = \frac{\text{absorbed collision energy}}{\text{initial ship's energy}}$$



Coefficient of Friction (μ)		
Steel - steel	~	0.15
Steel - concrete	~	0.35
Steel - wood	~	0.65

Figure C3.8-1. Portion of Collision Energy to be Absorbed by the Ship or Bridge Structure in Relation to the Collision Angle and the Coefficient of Friction [11].

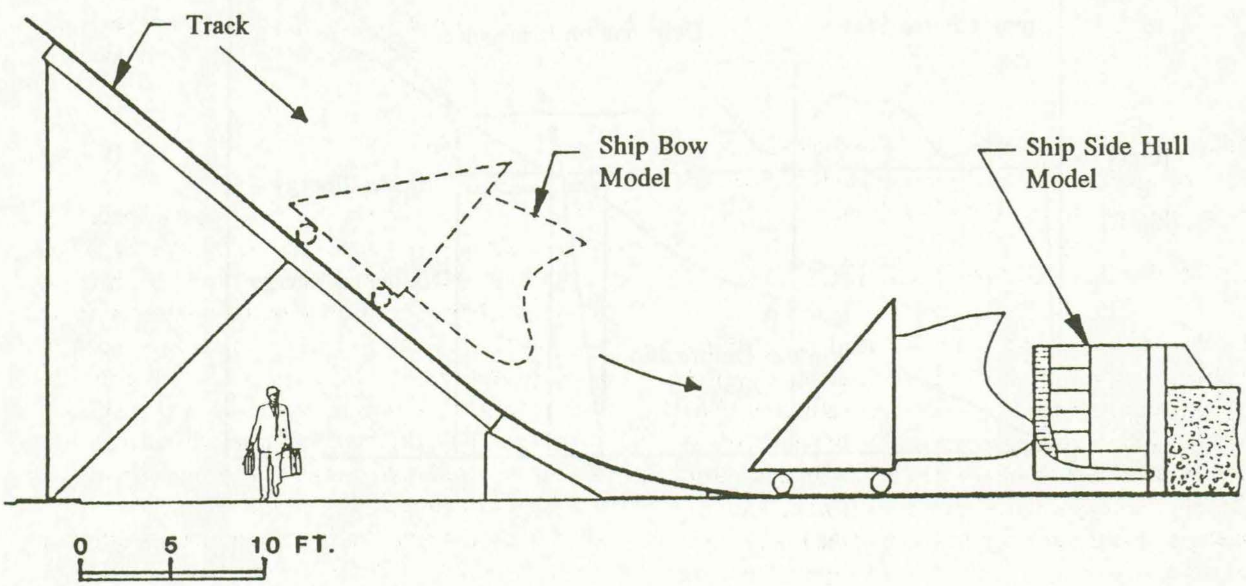


Figure C3.9-1. Elevation View of Set-up for Woisin's Ship Model Collision Tests at Howldtswerke-Deutsche Werft, Hamburg [12].

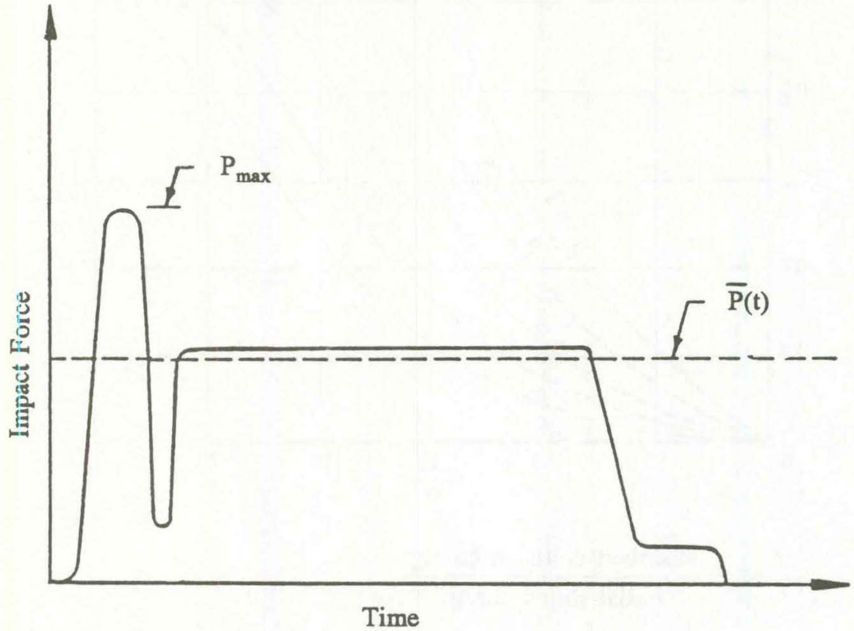


Figure C3.9-2. Schematic Representation of Ship Impact Force History from Collision Tests Conducted by Woisin, adapted from [15].

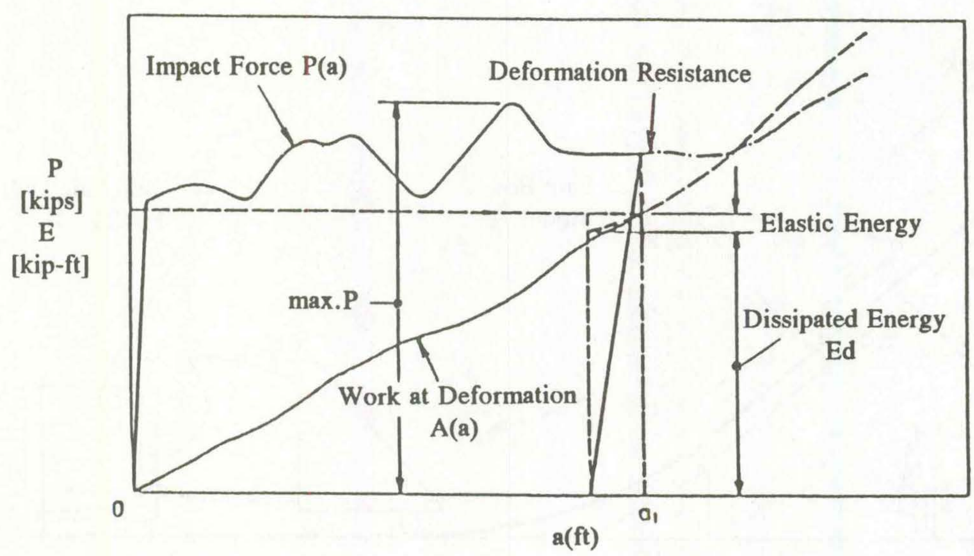


Figure C3.9-3. Impact Force, P, and Energy, E, in Relation to the Vessel Damage Depth, a, adapted from [16].

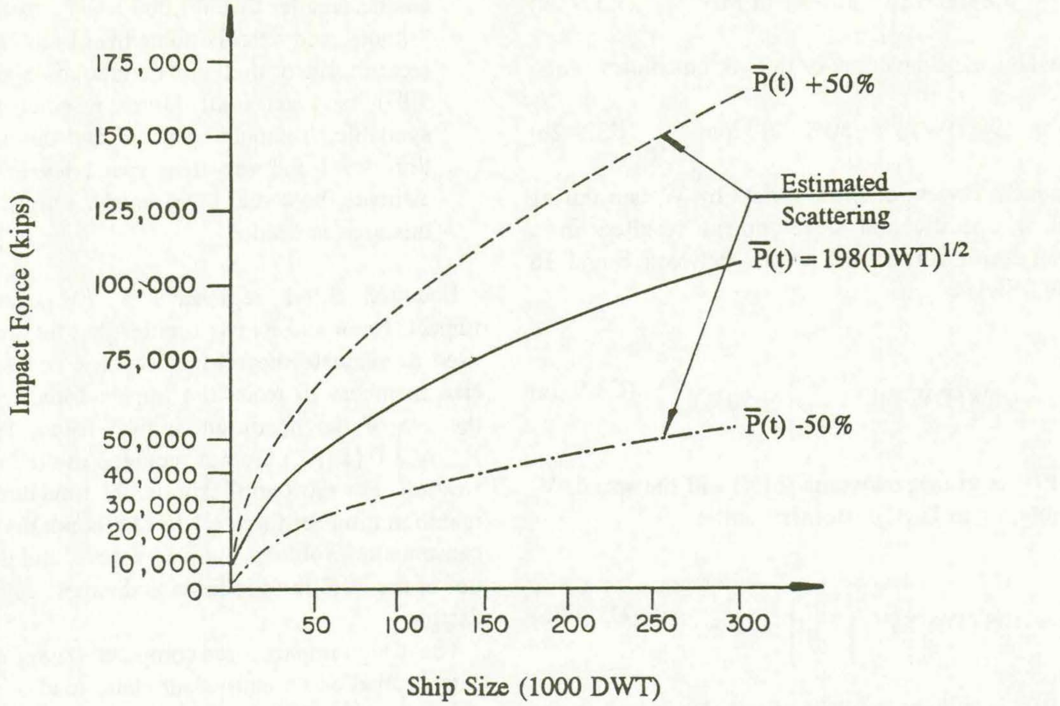


Figure C3.9-4. Average Impact Force, $\overline{P}(t)$, for Bulk Carriers, adapted from [12].

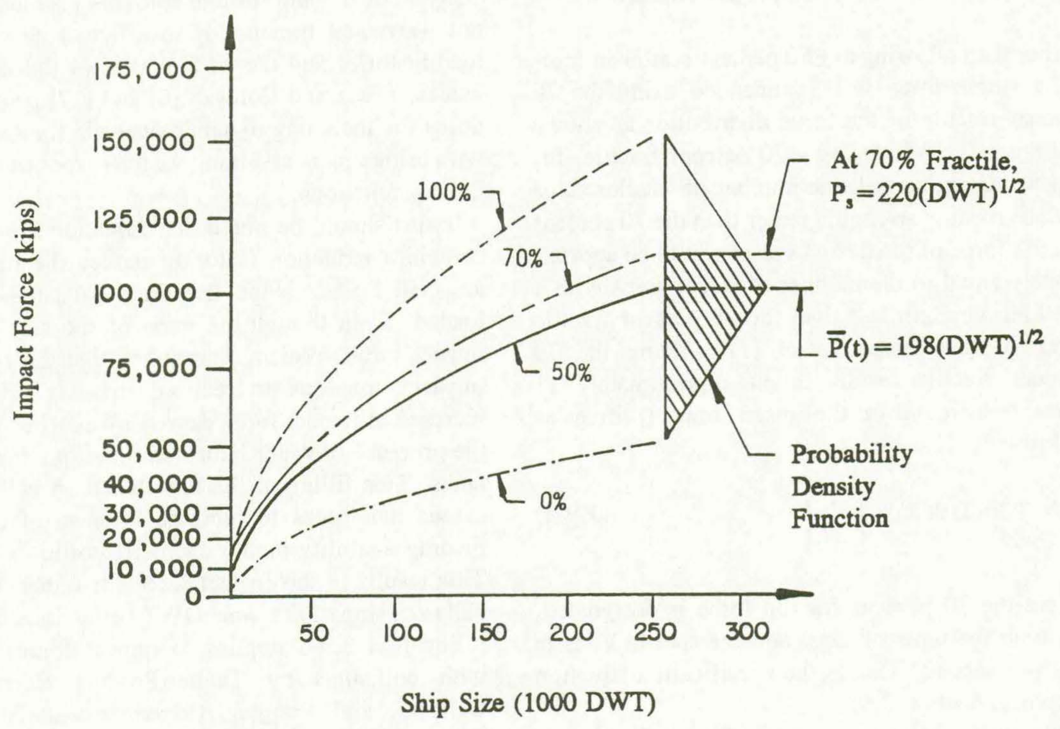


Figure C3.9-5. Probability Density Function of Impact Force Showing 70% Fractile Used for P_s, adapted from [12].

$$\overline{P}(t) = 0.88(DWT)^{1/2} \pm 50\%, \text{ in MN} \qquad (C3.9\text{-}2a)$$

where MN is meganewtons, or in U.S. customary units:

$$\overline{P}(t) = 198(DWT)^{1/2} \pm 50\%, \text{ in kips} \qquad (C3.9\text{-}2b)$$

Subsequent review of the test data by Woisin during the Guide Specification development resulted in a reduction factor for impact speeds between 8 and 16 knots as follows:

$$\overline{P}(t) = 0.88(DWT)^{1/2} \left[\frac{V}{16}\right] \pm 50\% \qquad (C3.9\text{-}3a)$$

where $\overline{P}(t)$ is in meganewtons (MN) and the speed, V, is in knots, or in U.S. customary units:

$$\overline{P}(t) = 198\,(DWT)^{1/2} \left[\frac{V}{27}\right] \pm 50\% \qquad (C3.9\text{-}3b)$$

where $\overline{P}(t)$ is in kips, and the speed, V, is in feet per second.

Equation 3.9-1 of the Guide Specification was developed from Eq. C3.9-3b with the following modifications as recommended by the project consultants:

- Rather than allowing a ± 50 percent scatter in forces, a single force is recommended using the 70 percent fractile of the force distribution as shown in Figure C3.9-5. Using a 70 percent fractile, for a given design vessel, the number of smaller ships with a crushing strength greater than the 70 percent fractile force of the design vessel would be approximately equal to the number of larger ships with a crushing strength less than the 70 percent fractile force of the design vessel [12]. Using the 70 percent fractile results in an approximately 11 percent increase in the mean impact force as follows:

$$P_S = 220(DWT)^{1/2} \left[\frac{V}{27}\right] \qquad (3.9\text{-}1)$$

  where the 70 percent fractile force is designated, P_S, with the units of kips, and the speed, V, is in feet per second. This is the specification equation shown in Section 3.9.
- Woisin's limitations on size of vessel, type of vessel, and a minimum impact speed of 8 knots were not imposed. In order to provide flexibility to the designer in estimating ship impact forces for vessels smaller than 40,000 DWT, speeds less than 8 knots, and vessels other than bulk carriers, it is recommended that the extrapolation of Equation 3.9-1 be used until future research results are available. It should be noted that the use of Equation 3.9-1 for very low speed levels may under estimate the actual force levels. Future research in this area is needed.

Equation 3.9-1 represents a 70 percent fractile impact force and its use implies that this force will be used to evaluate the bridge response to impact and to size members to resist the impact forces. Values and the use of the maximum impact force, P_{max}, (where $P_{max} = 2.0\,[\overline{P}(t)]$) are not included in the Guide Specification. For most applications, the time duration of the maximum impact force (.1 to .2 seconds) is too brief to cause major problems to the structure, and therefore the use of the 70% fractile force is the most appropriate for design.

The design impact force computed from Equation 3.9-1 is applied as an equivalent static load to the bridge. Since the model testing used to derive Equation 3.9-1 were based on dynamic tests, its use incorporates some to the influence of the ship dynamics in its empirical formulation. The authors believe the use of dynamic analysis of the ship/bridge collision problem is usually not warranted because of insufficient data on impact load histories and the wide scatter of the impact force values. Prucz and Conway [6] and [17] provide procedures for including dynamic analysis for ship collision with bridge piers assuming various types of impact load history functions.

It also should be noted that Equation 3.9-1 does not contain a reduction factor to reduce the impact force associated with ships traveling ballasted, or partly loaded. Even though the mass of the ship affects the impact force, Woisin determined that the reduction in impact force due to reduced mass is offset by the increase in impact force caused by a stiffer bow due to the presence of water ballast in the ship's forepeak/bow tanks. This filling of the bow tanks on ballasted ships causes the forces to increase because of the water's incompressibility in the enclosed portion of the bow. This results in the impact force for either a loaded or ballasted ship of the same DWT being essentially equal.

Equation 3.9-1 applies to impact forces associated with collisions by Tanker/Product Carriers, Bulk Carriers, and Freighter/Container vessels. Although Tanker/Product Carrier and Bulk Carrier vessels usually have relatively soft, deformable bows as compared to the relatively hard rigid bows of Freighter/Container vessels, the total average collision forces given by Equation 3.9-1 for Tanker/Product Carriers and Bulk Carriers remains unchanged because the larger contact area and the broader deck and bottom structure in the bow counteract the local softening.

C3.10 SHIP BOW DAMAGE DEPTH

The average bow damage depth, a, is computed based on impact force averaged against the work path, P(a), rather than averaged against impact duration, P(t), such that:

$$\bar{a} = \frac{KE}{\bar{P}(a)} \qquad \text{(C3.10-1)}$$

For a constant level of impact energy, KE, the ship bow damage length increases for lower values of the average impact force, $\bar{P}(a)$. In order to provide a level of safety consistent with the 70 percent fractile used to compute the design impact force, P_S, the bow damage depth, a_S, should be estimated as:

$$a_S = \frac{(1.25)\ (1.11)\ KE}{(.9)\,P_S} = 1.54 \left[\frac{KE}{P_S}\right] \qquad \text{(3.10-1)}$$

where the factor 1.25 accounts for the increase in average impact force over time versus damage depth, the factor 1.11 accounts for the increase in impact force due to the 70 percent design fractile, and the factor 0.9 represents an increase in the damage depth (11 percent) to provide a similar level of design safety as that used to compute P_S.

C3.11 SHIP COLLISION FORCE ON SUPERSTRUCTURE

Limited data exists on the collision forces between ship superstructure (bow, deckhouse, and mast) and bridge superstructure elements. Forces developed by Cowiconsult [18] during the 1970 Great Belt Bridge Investigation in Denmark for deckhouse collision with a bridge superstructure were:

P_{DH} = 1,200 kips for the deckhouse collision of a 1,000 DWT freighter ship;

P_{DH} = 6,000 kips for the deckhouse collision of a 100,000 DWT tanker ship.

Based roughly on these values, the empirical relationship of Equation 3.11.2-1 was developed for selecting superstructure collision design impact values for deckhouse collision.

Very little data on mast impact forces exist in the published literature. Equation 3.11.3-1 was developed by estimating the impact forces based on bridge girder and superstructure damage from several historical mast impact accidents.

C3.12 BARGE COLLISION FORCE ON PIER

Compared to ship collision data, very little research has been reported in the published literature concerning barge collision impact forces. The barge collision impact force determined by Equation 3.12.1-1 was developed from research conducted by Meir-Dornberg in West Germany in 1983 on behalf of the Water and Shipping Directorate Southwest-Saar District [19]. The experimental and theoretical studies performed by Meir-Dornberg were performed to study the deformation force and the deformation when barges collide with lock entrance structures and with bridge piers. Meir-Dornberg's investigation also studied the direction and height of climb of the barge upon bank slopes and walls due to skewed impacts and groundings along the sides of the waterway.

Meir-Dornberg's study [19] included dynamic loading with a pendulum hammer on three barge bottom models in Scale 1:4.5, static loading on one bottom model in Scale 1:6, and numerical computations. The results for the standard European Barge, Type IIa (Figure C3.12-1) are shown in Figure C3.12-2 for barge deformation and impact loading. No significant difference was found between the static and dynamic forces measured during the study.

Using metric units of meganewton (MN) for force, P_B, and meter (m) for bow damage length, a_B, Meir-Dornberg developed the following equations:

For $a_B < 0.1$m,
$$P_B = 60(a_B), \text{ in MN} \qquad \text{(C3.12.1-1a)}$$

For $a_B \geq 0.1$m,
$$P_B = 6 + 1.6(a_B), \text{ in MN} \qquad \text{(C3.12.1-1b)}$$

Converting these equations to U.S. customary units yields:

For $a_B < 0.34$ feet,
$$P_B = 4112\ (a_B), \text{ in kips} \qquad \text{(C3.12.1-2a)}$$

For $a_B \geq 0.34$ feet,
$$P_B = 1349 + 110(a_B), \text{ in kips} \qquad \text{(C3.12.1-2b)}$$

The European Barge Type IIa has a bow width x depth dimensions of 37.4x15.4 feet which compares

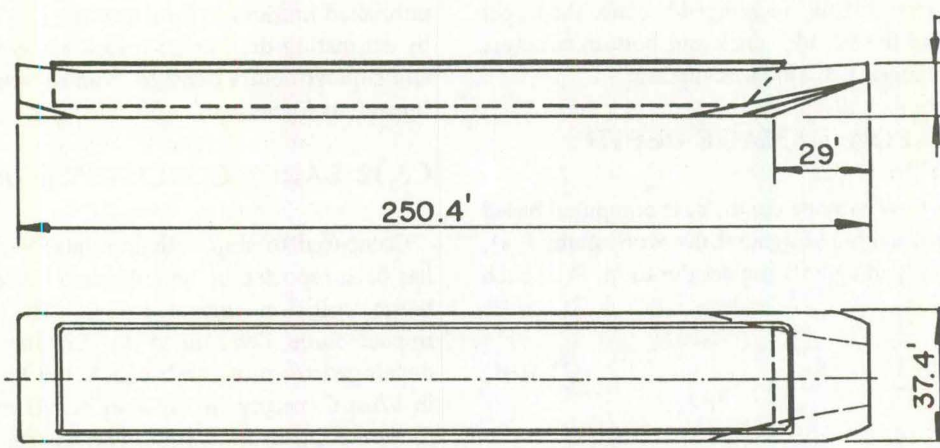


Figure C3.12-1. Dimensions of European Barge Type IIa, adapted from [9].

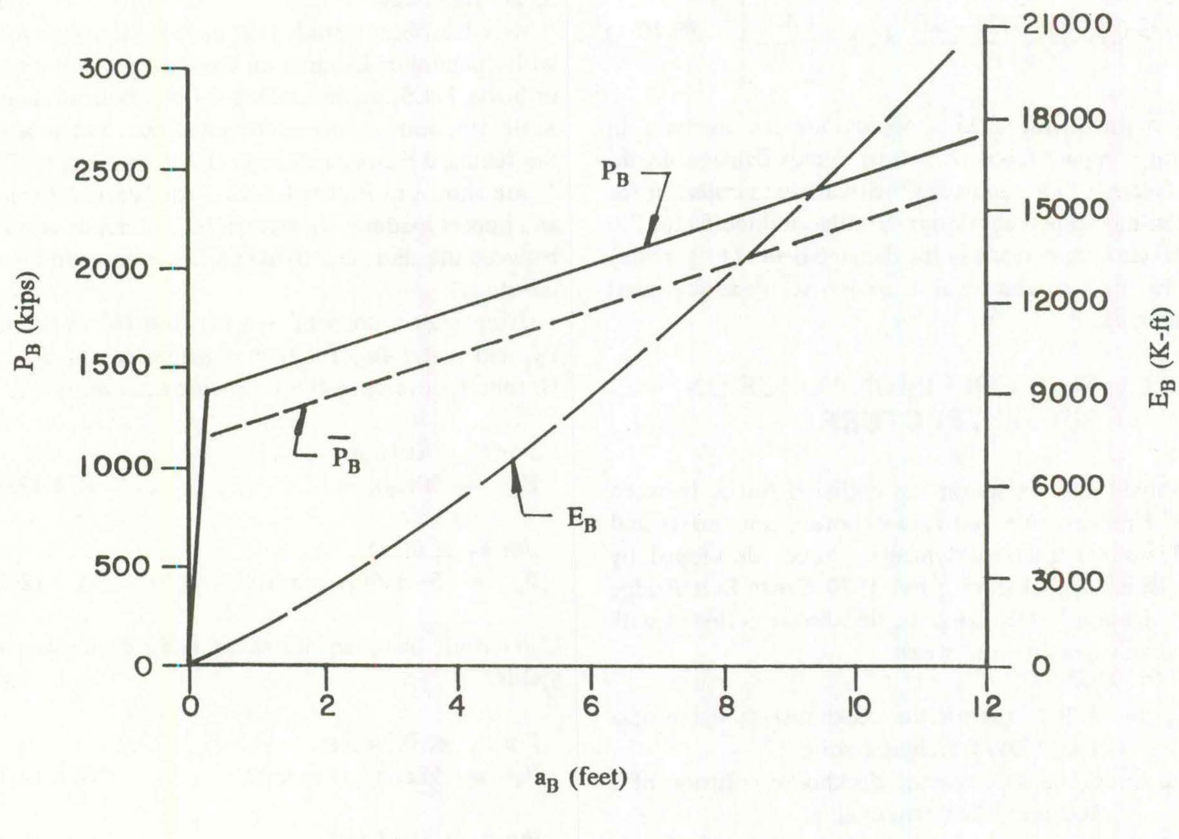


Figure C3.12-2. Barge Impact Force (P_B) and Deformation Energy (E_B) Versus Damage Length (a_B) for European Barges Types II and IIa, adapted from [9].

relatively closely with the Jumbo Hopper Barge bow dimensions of 35.0x13.0 feet as shown in Figure 3.5.1-1. The Jumbo Hopper Barge size is the most frequent barge size utilizing the U.S. inland waterway system. Due to their similar barge bow shapes, the Guide Specification recommends using the Meir-Dornberg results for computing the impact load for U.S. inland barges with a width of 35 feet. In Equation 3.12.1-1 a modification factor, $R_B = B_B/35$, was introduced into the basic Meir-Dornberg equation to modify the impact force for barges whose width, B_B, was different than 35.0 feet.

C3.13 BARGE DAMAGE LENGTH

The relationship for barge damage length, a_B, was developed from the same research conducted on barge collisions by Meir-Dornberg [19] as discussed in Section C3.12 above. From the test data, Meir-Dornberg developed the following equation for barge deformation, a_B, and impact deformation energy, E_B, using metric units of meters (m) and meganewton-meters (MNm):

$$a_B = \left(\left[1+.13(E_B)\right]^{1/2}-1\right)(3.1) \qquad \text{(C3.13-1a)}$$

Converting this equation to U.S. customary units and substituting the kinetic impact energy, KE, for the deformation energy yields:

$$a_B = \left[\left(1+\frac{KE}{5672}\right)^{1/2}-1\right] 10.2 \qquad \text{(C3.13-1b)}$$

where a_B is in feet, and KE is in units of kip-feet. Equation 3.13-1 of the Guide Specification was developed from C3.13-1b with the incorporation of the width modification factor, R_B, as discussed in Section C3.12 for barges with a width different than 35 feet.

C3.14 IMPACT LOAD COMBINATION

The vessel collision impact forces are combined with those from other loads and the group loading combination is the same format as that used in the current AASHTO Specifications for Seismic Design [20] with all gamma and beta factors equal to 1.0. Either limit state, load factor, or service load method of design according to the Standard Specification for Highway Bridges, current edition adopted by AASHTO [21] can be used with the specified forces.

The intent of the loading is to prevent superstructure collapse with its resulting disruption of motorist traffic. Loss of life concerns are only indirectly considered. Under the group loading of Equation 3.14-1, partial or local failure of bridge elements may occur provided that sufficient redundancy exists in the ultimate state of the remaining structure to safely support the superstructure.

C3.15 LOCATION OF IMPACT FORCES

Applying the vessel impact forces to the structure based on the structure geometry and the geometry of the ship or barge is an important consideration of the Guide Specification requirements.

C3.15.1 Substructure Design

Two cases must be evaluated in designing the bridge substructure for vessel impact loadings; 1) the overall stability of the substructure and foundation assuming that the vessel impact acts as a concentrated force at the waterline, and 2) the ability of each member of the substructure to withstand any local collision force associated with a vessel impact.

The need to apply local collision forces on bridge piers and substructure exposed to contact by overhanging portions of a ship or barge's bow is well documented by accident case histories. The Sunshine Skyway Bridge (which collapsed in 1980 due to a ramming by a ballasted 35,000 DWT bulk carrier) collapsed as a result of the ship's bow impacting a pier column at a point 42 feet above the waterline as shown in Figure C3.15.1-1. Ship and barge bow rake lengths (overhangs) are often large enough that they can even extend over protective fender systems and contact vulnerable bridge elements as shown in Figures C3.15.1-2 and C3.15.1-3. Bow shapes and dimensions vary widely and the designer may need to perform special studies to establish vessel bow geometry for a particular waterway location. Typical bow geometry data is provided in Section 3.5.

C3.15.2 Superstructure Design

The ability of various portions of a ship or barge to impact a span or superstructure element depends on the available vertical clearance under the structure, the water depth, vessel type and characteristics, and the loading condition of the vessel. Section 3.5 contains

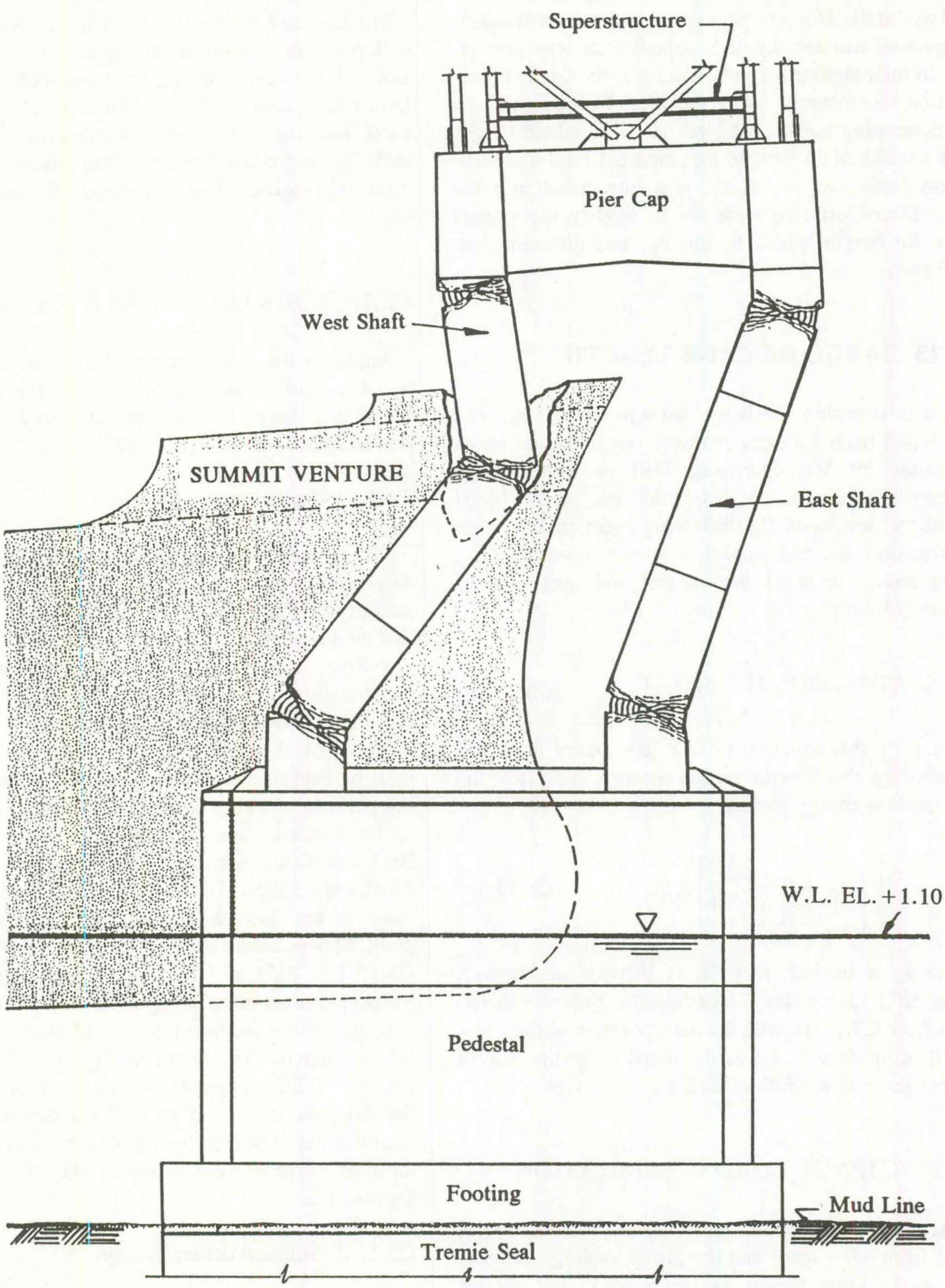


Figure C3.15.1-1. Collapse of Pier 2S of the Sunshine Skyway Bridge Subsequent to Impact by the Bow Overhang of the M/V Summit Venture, adapted from [22].

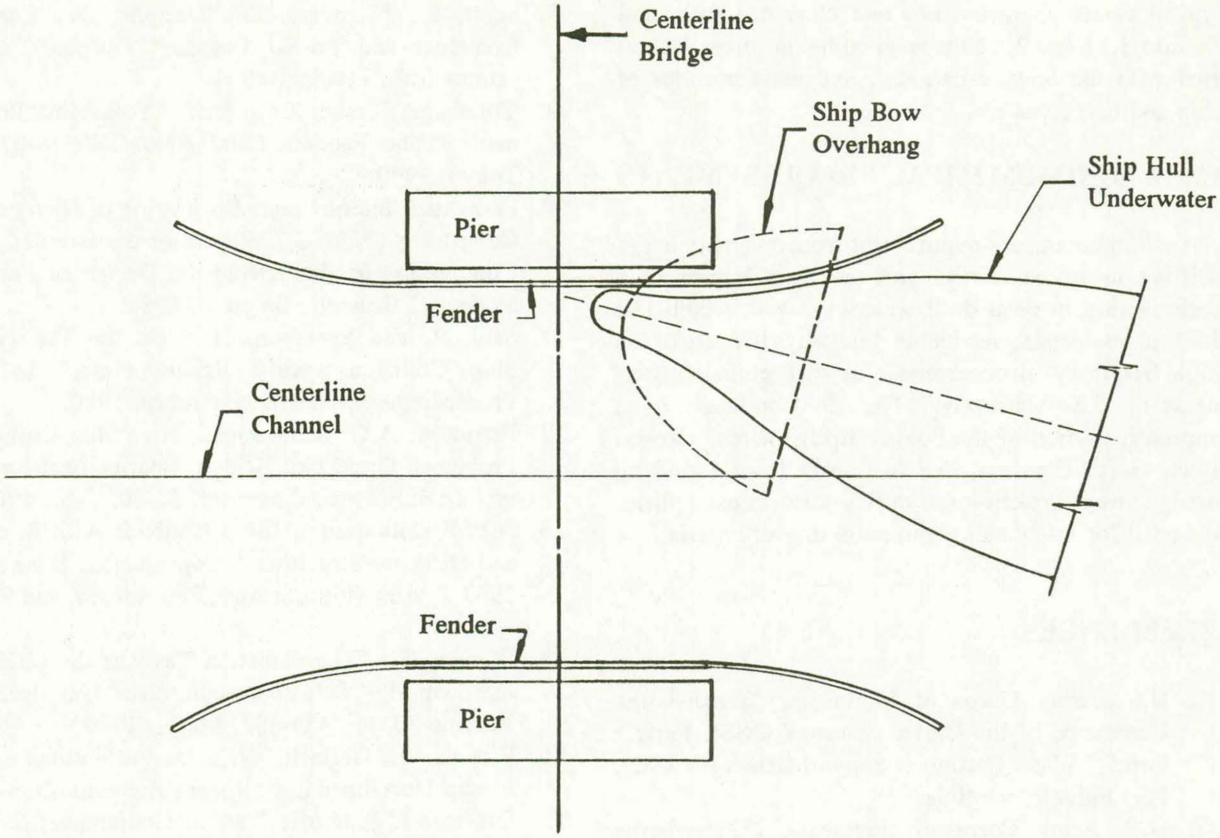


Figure C3.15.1-2. Plan of Ship Bow Overhang Impacting Pier Behind Fender.

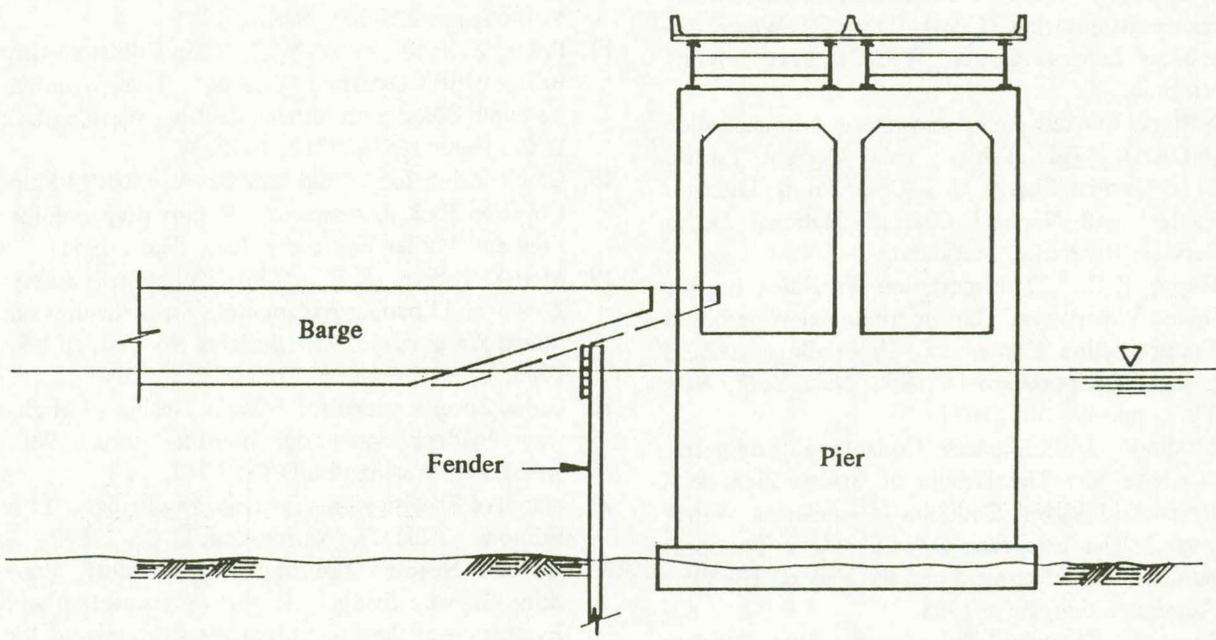


Figure C3.15.1-3. Elevation of Barge Overhang Impacting Pier Behind Fender.

typical vessel characteristics and clearance data, and Section 3.11 and 3.12 the span collision forces associated with the bow, deckhouse, and mast portions of ship and barge vessels.

C3.16 MINIMUM IMPACT REQUIREMENTS

A minimum impact requirement from an empty barge drifting in all waterways and the mast impact of a drifting ship in deep draft waterways was established for bridges crossing navigable waterways because of the high frequency of occurrences of such collision accidents in U.S. waterways. The 1990 collapse of an approach portion of the Bonner Bridge across Oregon Inlet, North Carolina, due to impact from a drifting dredge vessel broken loose during a storm exemplifies the need for establishing minimum impact criteria.

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COMMENTARY

SECTION 4 - DESIGN VESSEL SELECTION

C4.1 GENERAL

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 Section 4.1.2 of the Guide Specification exist.

C4.2 WATERWAY CHARACTERISTICS

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 (Section 3.7), impact distribution (Section 4.5) and the geometric probability (Section 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 extra-tropical 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 the Guide Specification. 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.

The selection of 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 those 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.

C4.5 IMPACT DISTRIBUTION

Based on historical accident data, the primary area of concern for vessel collision with a bridge structure is within the central area near the navigable channel. This central area is defined as an area within a distance 3xLOA on each side of the inbound and outbound vessel transit paths in the channel as discussed in Section C4.8.3.3. Beyond the central area, bridge elements should meet the minimum impact requirements of Section 3.16. Within the central area, a design speed in accordance with Section 3.7, and a design vessel in accordance with Method I, II, or III must be determined in order to establish vessel impact design forces for the bridge.

C4.7 METHOD I

C4.7.1 General

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 Design Vessel Acceptance Criteria

The framework of the Method I acceptance criteria was based on the ship impact criteria for bridge design stated in the Common Nordic Regulations [1] currently in use in Scandinavian countries. The following is quoted from these regulations [1]:

"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 and Regular bridge importance 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 and Regular importance classification categories respectively.

C4.8 METHOD II

C4.8.1 General

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 his experience and abilities.

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 Specification.

C4.8.2 Design Vessel Acceptance Criteria

Establishment of a 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 [2]. 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 [3]. 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 [5]. The most important of these can be grouped into two broad based 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 C4.8.2-1 and C4.8.2-2 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 Specification was to establish a simple criterion defining a single level of risk acceptance for superstructure collapse for each of the two importance classification categories, which could 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 = .0001 per year for critical bridges, and AF = .001 per year for regular bridges for bridge collapse associated with vessel collision.

The critical bridge acceptance criterion, AF = .0001 per year, is the same criterion recommended by Modjeski and Masters [18] for vessel collision in

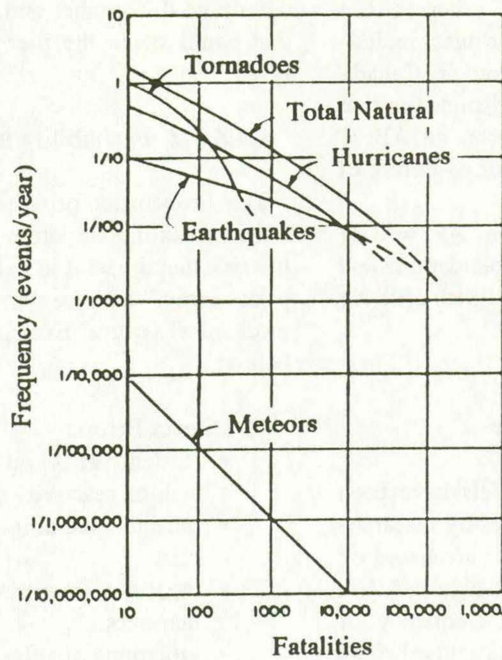

Figure C4.8.2-1. Risk of Fatalities from Natural Events [4].

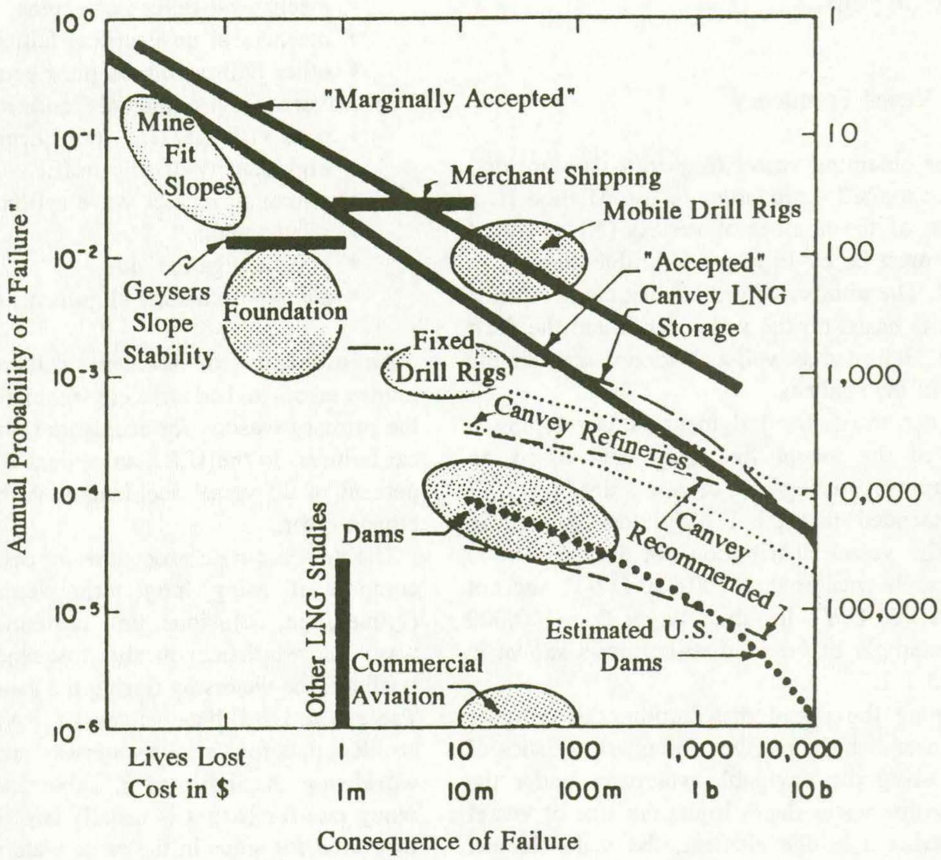

Figure C4.8.2-2. Risk of Failure of Selected Engineering Projects [4].

Louisiana waterways. This acceptance criterion has been used for several recent long span bridges, including the Annacis Island Bridge near Vancouver, Canada [25]. As seen in Figure C4.8.2-2 which depicts the risk of failure of selected engineering projects, an AF = .0001(i.e., $1x10^{-4}$) is equivalent to the risk of failure of dams.

The regular bridge acceptance criterion, AF = .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 C4.8.2-2.

C4.8.3 Annual Frequency of Collapse

Various types of risk assessment models have been developed for vessel collision with bridges by researchers worldwide [6]. Practically all of these are based on a similar form of Guide Specification Equation 4.8.3-1, 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 Vessel Frequency

Sources for obtaining vessel frequency data are discussed in Section 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 exceeding 50,000 DWT for ships larger than 100,000 DWT. An example of vessel distribution is shown in Table C4.8.3.1-1.

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 to determine the number and characteristics of the vessels that could strike the pier or span element being analyzed.

C4.8.3.2 Probability of Aberrancy

The probability of aberrancy, PA, (sometimes referred to as the causation probability) is a measure of the risk that a vessel is in trouble as a result of either a pilot error, adverse environmental conditions, 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
   - incorrect evaluation of current and wind conditions, etc.
2) Mechanical Failures:
   - mechanical failure of engine
   - mechanical or electrical failure of steering
   - other failures due to poor equipment, etc.
3) Adverse Environmental Conditions:
   - poor visibility (fog, rainstorm)
   - high density of ship traffic
   - strong current or wave action
   - wind squalls
   - poor navigation aids
   - awkward channel alignment, 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 U.S., 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, and rammings) in the waterway, and statistics on the frequency of ship/barge traffic in the waterway during the same period of time. Table 4.8.3.2-1 lists values of PA developed from accident data for various waterway and bridge locations worldwide. As indicated in Table 4.8.3.2-1, the aberrancy 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

Table C4.8.3.1-1. Vessel Frequency Data for the Dame Point Bridge, Jacksonville, Florida (1984 Fleet) [7].

Vessel Type*	DWT	No. of Annual Transits (n) Loaded	Ballasted
Barge (Ocean)	15,000	73	73
Barge (Ocean)	25,000	67	67
Barge (Ocean)	35,000	81	81
Barge (Ocean)	50,000	66	66
Freighter/Container	10,000	170	0
Freighter/Container	18,000	360	0
Freighter/Container	26,000	28	0
Tanker/Bulk Carrier	20,000	67	67
Tanker/Bulk Carrier	30,000	139	139
Tanker/Bulk Carrier	40,000	78	78
Tanker/Bulk Carrier	60,000	25	25

*** 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.**

consuming process, an alternative simpler method for estimating PA is provided in the Guide Specification. Equations 4.8.3.2-1, 4.8.3.2-2, 4.8.3.2-3, and 4.8.3.2-4 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 4.8.3.2-1, is generally in fair agreement, although exceptions do occur.

It should be noted that the procedure for computing PA using Equation 4.8.3.2-1 should not be considered as being either rigorous, or exhaustive. Several influences, such as wind, visibility conditions, navigation aids, pilotage, 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.

C4.8.3.3 Geometric Probability

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;
- sailing path of the vessel;
- maneuvering characteristics and size of vessel;
- location, heading, and velocity of vessel;

Table C4.8.3.2-1. Summary of Probability of Aberrancy, PA, Values.

Locality	Type of Data	Probability of Vessel Aberrancy ($x10^{-4}$)
Dover Straits - Collisions[8]	Statistics	5 to 7
Dover Straits - Groundings[8]	Statistics	1.4 to 1.6
Japanese Straits - Groundings[9]	Statistics	0.7 to 6.7
Japanese Straits - Collisions[9]	Statistics	1.3
Worldwide[10]	Statistics	0.5
Tasman Bridge, Australia[11]	Estimate	0.6 to 1.0
Great Belt Bridge, Denmark[12]	Estimate	0.4
Sunshine Skyway Bridge, Florida[13]	Statistics Statistics	1.3 (Ships) 2.0 (Barges)
Annacis Island Bridge, Canada[14]	Estimate	3.6
Francis Scott Key Bridge & Wm. Preston Lane Bridges, Maryland[15]	Statistics	1.0 (Ships) 2.0 (Barges)
Dames Point Bridge, Florida[7]	Statistics	1.3 (Ships) 4.1 (Barges)
Laviolette Bridge, Canada[16]	Statistics	0.5
Centennial Bridge, Canada[17]	Statistics	5.0
Louisiana Waterways[5]	Statistics	0.8 to 1.9 (Ships) 1.5 to 3.0 (Barges)
Gibraltar Straits - Strandings, Morocco[18]	Statistics	2.2
Gibraltar Straits - Collision, Morocco[18]	Statistics	1.2

- rudder angle at time of failure;
- environmental conditions;
- width, length, and shape of vessel;
- vessel draft (loaded or ballasted).

The methods used to determine PG varies significantly among researchers. Models to compute PG as developed by Fujii [20,21], MacDuff [8], Cowiconsult [22], Knott [18], and Modjeski and Masters [18] 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 Specification.

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 4.8.3.1-1. 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 data base are very few from a statistical point of view. By definition 68.3 percent of all collisions occur within one standard deviation (σ)of the mean, 95.5 percent within two standard deviations (2σ), and 99.7 percent within three standard deviations (3σ) for a normal distribution. The Guide Specification recommends that σ = LOA of the design vessel for computing PG, and that bridge elements beyond 3σ from the centerline of the vessel transit path not be included in the analysis (other than the minimum impact requirement).

Table C4.8.3.3-1 provides the accident data used to develop the recommended value of σ = LOA. The use of LOA as the standard by which σ 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 σ, since the value of PG is influenced by the size of the ships and barges passing under the bridge. For reasons of simplicity, a LOA value equal to a vessel selected using the Method I criteria was recommended for determining the distribution of impact speed and the geometric probability.

The accident data in Table C4.8.3.3-1 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 Specification project consultants recommend that the same σ = LOA developed for ships be applied to barges with the barge LOA equal to the total length of the barge tow, including the towboat.

C4.8.3.4 Probability of Collapse

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/strength characteristic 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 Fujii in Japan [20] using historical damage caused between two colliding ships at sea. The curves in Figure C4.8.3.4-1 are reproduced from Fujii's paper where the following definitions are used:

x = 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

y = G.R.T. ratio is defined as the ratio between G.R.T. of "the other ship" to the ship to which x is related

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

$$x = \frac{p}{p_{max}}$$

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 [22] developed the probability density function shown in Figure C4.8.3.4-2 for the relative magnitude of the collision force using Fujii's results and the following assumptions:

- The pier is considered as a large collision object relative to the ship (i.e., the G.R.T. ratio y = 10 to 100).
- The relative magnitude of the collision force (p/p_{max}) is related to the damage rate, x.
- From Figure C4.8.3.4-1 for $p/p_{max} \geq 0.1$, the probability is approximately 0.1.
- The probability density function for p/p_{max} has been simplified to be uniform in each of the intervals 0 to 0.1 and 0.1 to 1.

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

Bridge Name	x	$f(x-x_m)^2$
Sidney Lanier	0.57	0.325
Tasman Bridge	1.47	2.161
Fraser River Bridge	0.31	0.096
Benjamin Harrison	0.69	0.476
Tingstad Bridge	0.33	0.109
Second Narrows RR	0.43	0.185
Second Narrows RR	0.66	0.436
Almo (Tjorn) Bridge	0.89	0.792
Sunshine Skyway	1.31	1.716
Newport Bridge	1.07	1.145
Sorsund Bridge	0.82	0.672
Outerbridge (NY)	0.78	0.608
Outerbridge (NY)	0.52	0.270
Outerbridge (NY)	0.50	0.250
Richmond/San Rafael	2.13	4.537
	s =	13.778

where

σ = standard deviation

$\sigma = [s/(n-1)]^{1/2} = [13.778/14]^{1/2} = 0.992$

where

x = ratio of the approximate vessel impact distance from centerline of vessel transit to the LOA of the vessel

x_m = mean of the distribution = 0.0

n = number of collisions = 15

The distribution function, F for $p/p_{max} \geq x_o$, shown in Figure C4.8.3.4-2b was derived by integrating f from the upper end in Figure C4.8.3.4-2a. Figure 4.8.3.4-1 in the Guide Specification is the same as Figure C4.8.3.4-2b except that the nomenclature for the terms was changed to agree with the Guide Specification terminology.

C4.9 METHOD III

C4.9.1 General

The Method III procedure was developed for those situations in which risk criteria alone might be inadequate in establishing the acceptable risk levels for a bridge. These situations might include bridges crossing very wide waterways resulting in many piers exposed to vessel collision, and the refit of existing piers found to be vulnerable to vessel collision. For these types of circumstances the economics associated with the cost-effectiveness of risk reduction can be brought into consideration. One aspect of this type of approach is the benefit/cost (B/C) analysis, where the cost of protection is compared against the benefits of risk reduction. Figure C4.9.1-1 indicates the typical relationship between the risk cost (also termed the exposure cost) and the cost of risk reduction.

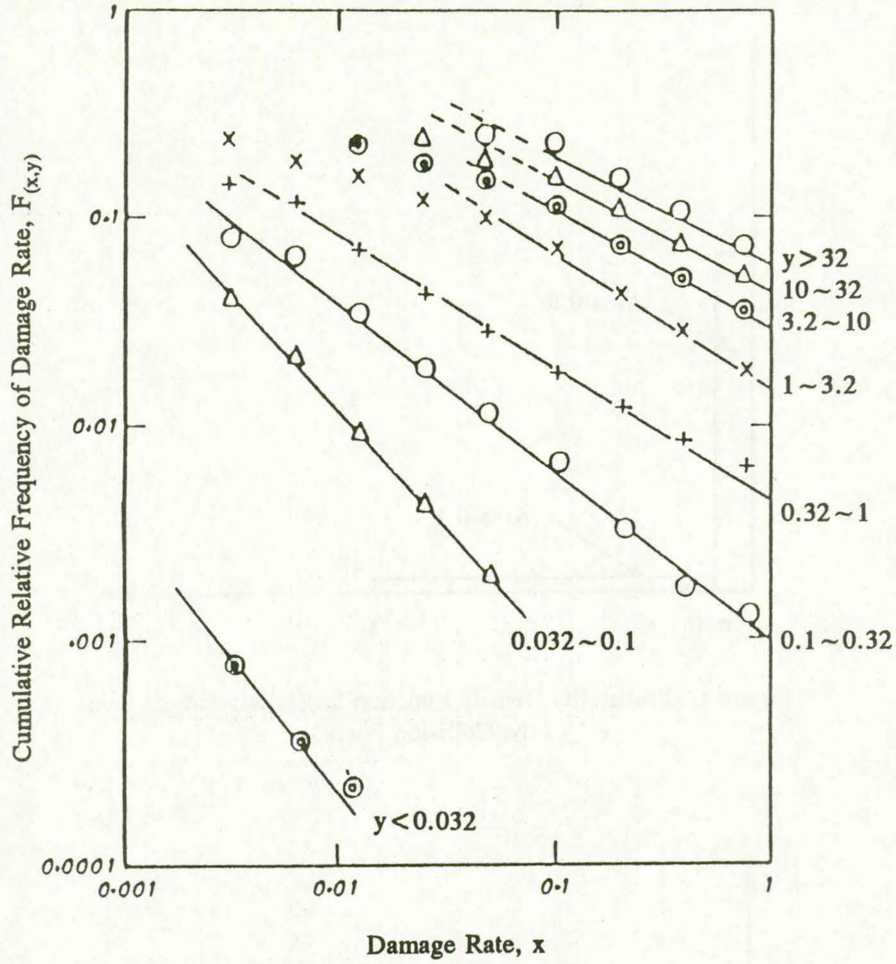

Fig. a. - Cumulative Relative Frequency, $F_{(x,y)}$, of the Damage Rate for Various Values of the Gross Ratios.

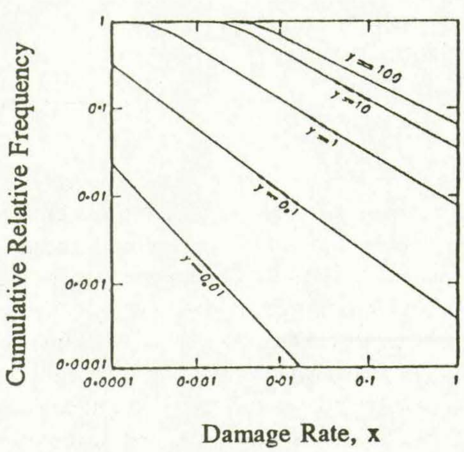

Fig. b. - Damage Rate as A Function of G.R.T. Ratio.

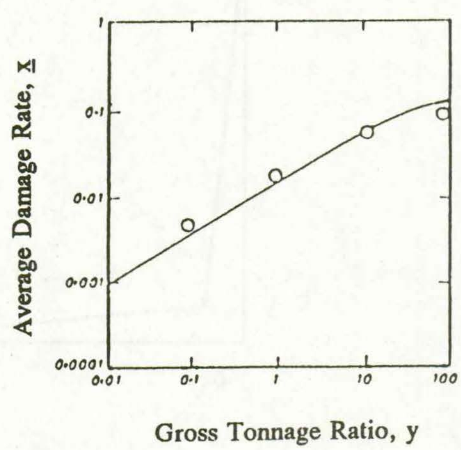

Fig. c. - Average Damage Rate and G.R.T. Ratio.

Figure C4.8.3.4-1. Fujii's Distribution Function for Damage Rate for Ships [2].

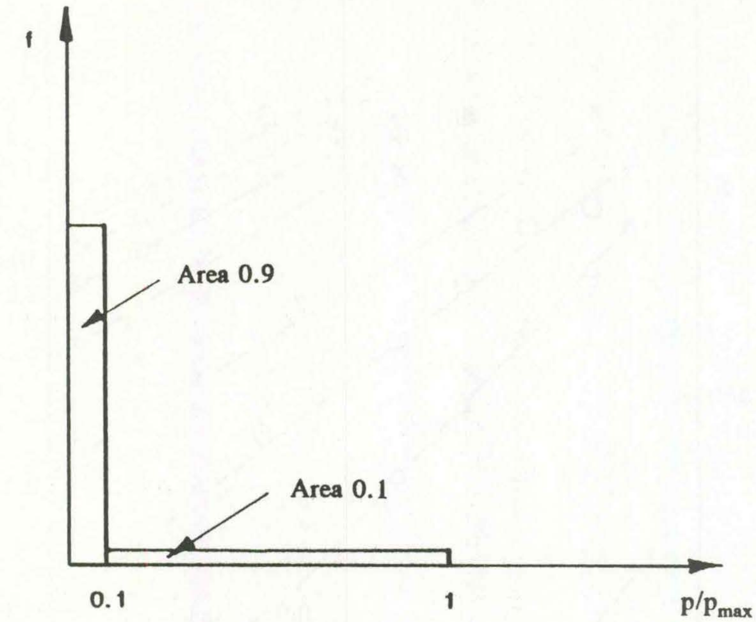

Figure a. Probability Density Function for Relative Magnitude of the Collision Force.

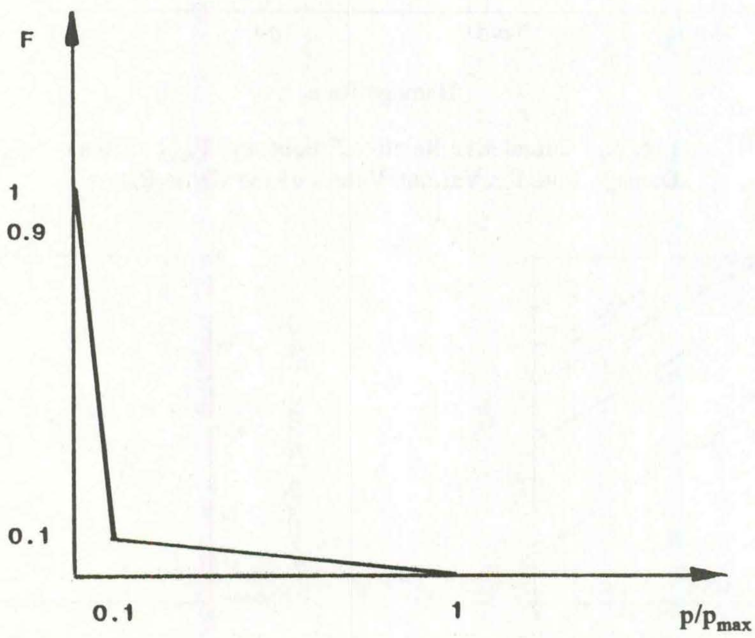

Figure b. Distribution Function for P/P_{max} Exceeds a Given Level.

Figure C4.8.3.4-2. Distribution Function for Relative Magnitude of the Collision Force for Ships [22].

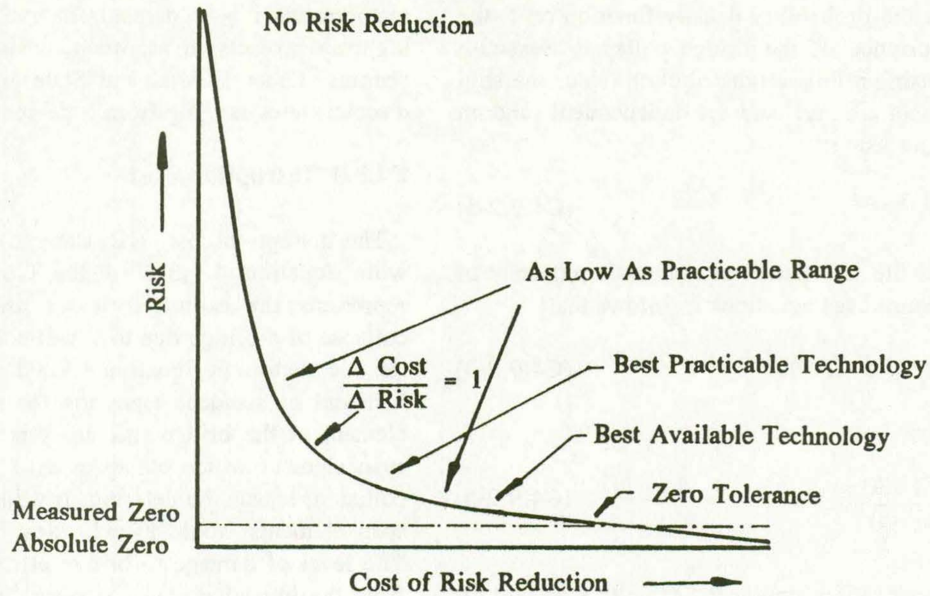

Figure C4.9.1-1. Typical Criteria for Acceptance Levels of Cost Effectiveness of Risk Reduction [2].

C4.9.2 Design Vessel Acceptance Criteria

The design vessel selection using Method III is determined based on a cost-effectiveness analysis. The cost-effectiveness analysis (CEA) methodology used to test economic feasibility and desirability should be a conventional benefit/cost (B/C) ratio calculation wherein the total present value of the benefits (avoidable disruption cost) for each year of the analysis period is compared against the total present value of the costs to build, maintain, and operate the system required to provide those benefits. Traditionally, cost effectiveness is indicated by a B/C ratio greater than 1.0, and a rate-of-return greater than the discount rate.

The benefits used to compare against the cost of protection is the present worth of the avoidable disruption cost associated with the protection, PW. The recommended method of performing the cost-effectiveness analysis in the Specification is by a detailed analysis procedure using standard engineering economic principles.

The alternative approximate method using Equation 4.9.2-1 was developed from research reported by Sexsmith [25] and Leslie [26]. According to Sexsmith [25], the present value of a loss which will occur at a definite time in the future is;

$$PW = (DA)e^{-it} \qquad \text{(C4.9.2-1)}$$

where

PW = present value of future loss;
DA = future disruption cost in present value;
i = discount rate;
t = time to the future loss.

Since the consequences of a catastrophic failure due to a vessel collision are expected to have return periods in the order of hundreds or thousands of years, the present value of the consequences of a second loss in a series will be negligible compared with those of the first loss in the series. Sexsmith therefore recommends that the problem be limited to consideration of the first occurrence of catastrophic collision only. Since the time to occurrence of the loss is a random variable, the present value based on Equation C4.9.2-1 becomes:

$$PW = DA \int e^{-it} f(t)dt \qquad (C4.9.2\text{-}1)$$

where f(t) is the probability density function on t, the time to occurrence of the bridge collapse. Sexsmith models f(t) using a Poisson distribution (since the ship collision events are rare and are independent random events in time) using:

$$f(t) = (AF)e^{-(AF)t} \qquad (C4.9.2\text{-}3)$$

where AF is the rate parameter (annual frequency of collapse). From these equations it follows that:

$$PW = DA \int e^{-it} (AF)e^{-(AF)t} \qquad (C4.9.2\text{-}4)$$

and, therefore

$$PW = \frac{AF(DA)}{(AF + i)} \qquad (C4.9.2\text{-}5)$$

Leslie's work [26] is similar to Sexsmith's, except for the inclusion of a growth factor, g, in computing the present value. The growth factor represents the real annual rate of growth of disruption costs associated with the bridge. An increase in disruption costs over the life of the bridge are usually a result of an increase in vessel traffic passing under the bridge (causing an increase in AF and port interruption costs), or an increase in motorist vehicles using the bridge (causing an increase in motorist interruption costs). Inclusion of the rate of growth of disruption cost, g, similar to Leslie [26] into Equation C4.9.2-5 results in the Guide Specification equation:

$$PW = \frac{AF(DA)}{(AF + i - g)} \qquad (C4.9.2\text{-}1)$$

Equation 4.9.2-1 is valid only for small values of g (i.e., $g \leq i/5$). Case histories for several U.S. waterways indicate that values for g generally range between 0 percent and 4 percent (g = .01 to .04). The rate of growth can be determined by:

$$g = \left[(PW_F / PW_C)^{1/n} - 1\right] \qquad (C4.9.2\text{-}6)$$

where, PW_C and PW_F are the present worth of avoidable disruption costs for the current and future year, respectively, and n is the number of years (usually the lifetime of the bridge). Equation C4.9.2-5 is used to compute PW_C and PW_F in order to determine g.

The discount rate, i, is usually established by the Owner. Typically, a range of discount values are evaluated in the cost-effectiveness analysis in order to determine the impact of its variability. Reference [27] recommends i = 4 percent for evaluating alternative highway projects in accordance with AASHTO procedures. Other Federal and State agencies have used discount rates ranging from 5 percent to 10 percent.

C4.9.3 Disruption Cost

The disruption cost, DC, determined in accordance with Equation 4.9.3-1 of the Guide Specification, represents the estimated losses associated with the collapse of a bridge due to vessel collision. Evaluating the cost factors in Equation 4.9.3-1 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 C4.9.3-1 illustrates the estimated disruption cost associated with the collapse of one of the main piers of the Dame Point Bridge, a cable-stayed structure with a 1,300-foot main in Florida [7].

Table C4.9.3-1. Main Pier Collapse Disruption Cost Example [7].

Cost Item	Disruption Costs (1984 Constant $) Yr. 1987	Yr. 2037
PRC	$ 8,948,000	$ 8,948,000
SRC	27,038,000	27,038,000
PIC	21,000,000	21,000,000
MIC	75,810,000	375,480,000
DC	$132,796,000	$432,466,000

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 due to collapsed bridge sections, 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) toll revenues lost by the out-of-service facility owner, if it is a toll bridge. 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 [27]. 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 C4.9.3-1.

Another factor in Equation 4.9.3-1 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 PIC are discussed in Section 4.9.3 of the Guide Specification. 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 conservatively understated in the analysis.

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.

In using the Method II cost effectiveness procedures to select the appropriate design vessel and pier protection system, care must be taken to make a distinction between the avoidable disruption cost, DA, and the disruption cost, DC. The avoidable disruption cost, DA, is the benefit derived by incorporating bridge protection to reduce, or eliminate the disruption costs caused by a catastrophic vessel collision. The value of DA is dependant on the level of protection provided. For 100 percent (total) protection, DA = DC; however, for anything less than 100 percent, DA would equal some percentage of DC. For example, an island around a bridge pier would provide 100 percent protection, whereas a single dolphin located in front of the same pier might provide a 60 percent level of protection. In the latter situation, DA = (0.6)DC for use in Equation 4.9.2-1. The distinction between the levels of protection provided by various bridge protection systems is critical when performing a B/C analysis for comparison of the protection alternatives.

Determination of DA is a matter of engineering judgment based on the characteristics of the waterway, vessel traffic, bridge, and protection system properties. As an example, Figure C4.9.3-1 illustrates a simple model developed to estimate the effectiveness of dolphin protection on a bridge pier.

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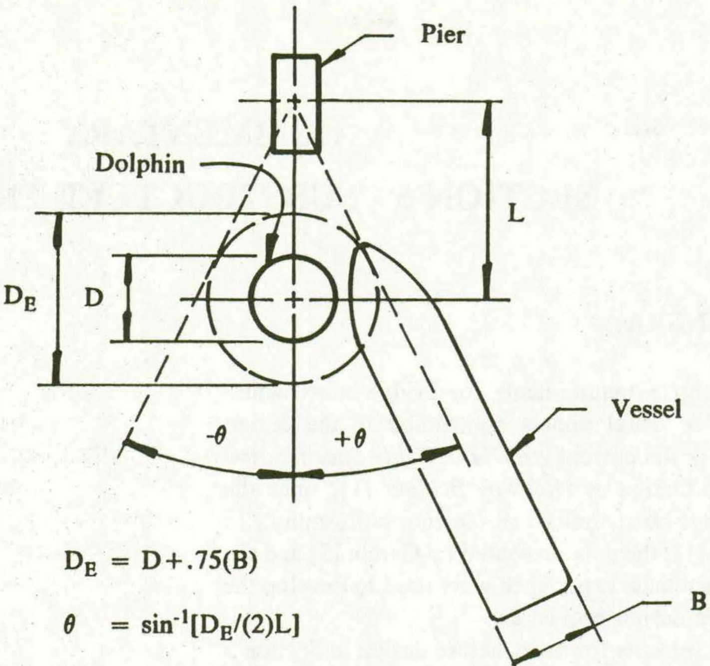


$$D_E = D + .75(B)$$

$$\theta = \sin^{-1}[D_E/(2)L]$$

where,

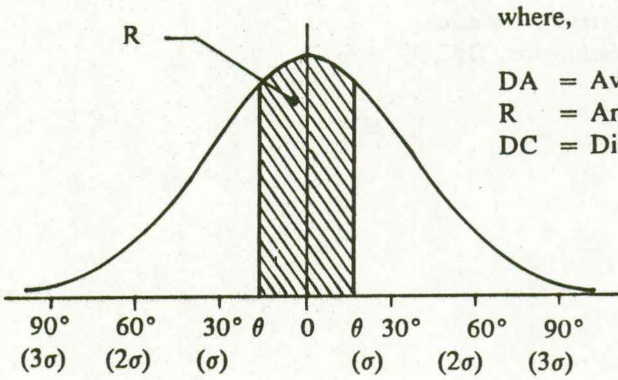
θ = Protection angle provided by dolphin
D = Diameter of dolphin (ft)
B = Beam (width) of vessel (ft)
L = Distance of dolphin from pier (ft)
D_E = Effective dolphin diameter (ft)

a. Plan of Dolphin Protection.

$$DA = R(DC)$$

where,

DA = Avoidable disruption cost ($)
R = Area in the density function between $\pm\theta$
DC = Disruption cost ($)



b. Normal Distribution of Vessel Collision Trajectories Around Bridge Pier (σ assumed = 30°).

Figure C4.9.3-1. Illustrative Model of the Effectiveness of Dolphin Protection Around a Bridge Pier.

COMMENTARY

SECTION 5 - SUBSTRUCTURE PROVISIONS

C5.1 GENERAL

The Section 5 requirements for bridge substructure design under vessel impact are similar to the design provisions in the current *AASHTO Guide Specification for Seismic Design of Highway Bridges* [1], since the two loadings are similar in design philosophy. In addition to [1], the data presented by Garcia [2] and the project consultants experience were used to develop the substructure design provisions.

The requirements for substructure design in Section 5 are applicable only to the design of bridge substructures to withstand vessel impact loading without causing collapse of the superstructure. The requirements are not applicable to the design of sacrificial protection structures which are presented in Section 7.

REFERENCES

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COMMENTARY
SECTION 6 - CONCRETE AND STEEL DESIGN

C6.1 GENERAL

The design philosophy for vessel collision is similar to that used for seismic design of highway bridges. The 33 1/3 percent increase in allowable concrete stress and the 50 percent increase in allowable steel stress for service load design in Section 6 for the vessel collision loading are the same allowable increases permitted in the current *AASHTO Guide Specification for Seismic Design of Highway Bridges* [1].

Similarly, the requirements in [1] for the formation of plastic hinges in concrete or steel members, shall also apply to bridge members subject to vessel impact forces in which plastic hinges are allowed to form as discussed in Section 6.

The requirements for concrete and steel design in Section 6 are applicable only to the design of bridge members to withstand vessel impact loading without causing collapse of the superstructure. The requirements are not applicable to the design of sacrificial protection structures which are presented in Section 7.

REFERENCES

1. American Association of State Highway and Transportation Officials (AASHTO), "Guide Specifications for Seismic Design of Highway Bridges, 1983, including Interim Specifications through 1988," Washington, D.C.

COMMENTARY

SECTION 7 - BRIDGE PROTECTION DESIGN PROVISIONS

C7.1 GENERAL

The development of bridge protection alternatives of vessel collisions generally follow three approaches: 1) reduction in the annual frequency of collision events (for example, by improving navigation aids near a bridge), 2) reducing the probability of collapse (for example, by imposing vessel speed restrictions in the waterway), and 3) by reducing the disruption costs of a collision (for example, by physical protection and motorist warning systems). Since modifications to navigation aids in the waterway and vessel operating conditions are normally beyond the designers ability to implement, the primary area of bridge protection to be considered by the designer are physical protection and motorist warning systems.

The requirements of the Guide Specification provide two basic protection options to the bridge designer. The first involves designing the bridge to withstand the impact loads in either an elastic or plastic manner. If plastic, the design must insure that the superstructure does not collapse by incorporating redundancy in the structure, or by other means. The second option allows the designer to provide a protection system of fenders, pile supported structures, dolphins, islands, etc. to either reduce the magnitude of the impact loads to within the allowable strength of the bridge pier or spans, or to independently protect the bridge elements.

The Specification requirements for either of these two options are general in nature since the actual design procedures that could be utilized vary considerably in the engineering profession. This is particularly true for plastic design. Since little information is available on the behavior of the plastic deformation of materials and structures during the type of dynamic impacts associated with vessel impact, assumptions based on experience and sound engineering practice must be substituted. In this Section of the Commentary, the various types of protection systems commonly used for bridges will be discussed, and case histories of their use will be presented.

C7.2 DESIGN LOADS

The Guide Specification requires that exposed bridge elements either be designed to withstand the required impact forces without bridge collapse, or that physical protection be provided.

The ability of adequately designed bridge piers to withstand major collision forces was dramatically illustrated by the 1981 collision of a fully loaded 31,800 DWT oil tanker (M/V Gerd Maersk) with one of the main tower piers of the Newport Suspension Bridge crossing Narragansett Bay, Rhode Island. As reported by Kuesel [1], the ship struck the pier head-on with an estimated speed of six knots (approximately 10 fps) while navigating in a dense fog. The bridge pier was relatively undamaged whereas the ship's bow was crushed in approximately 11 feet. Figure C7.2-1 depicts a profile of the surface spalling damage caused by the ship's bow impacting the pier. The ship came to a complete stop after crashing into the pier and then drifted off. Although the vessel took on some water through sprung plates, no oil was spilled, and the ship was never in danger of sinking.

Supporting a 1,600-foot main span, the Newport Bridge main piers were located in water depths of approximately 98 feet. The concrete piers which supported the steel towers were of "Potomac Type" caisson construction, founded on 512 steel H-piles driven into sands that fill the glacial gorge under the bay. Using Equation 3.9-1, the estimated average impact force on the pier would have been approximately,

$$P_S = 220(31,800)^{1/2}(10/27) = 14,500 \text{ kips}$$

This compares very favorably with the average impact force, P_S, computed by dividing the ship impact energy by the measured bow crushing depth, a_B, of 11 feet. The displacement of the ship, W, was approximately 45,000 tonnes. Since the underwater keel clearance of 45 feet is greater than 0.5xDraft (23ft), the hydro-

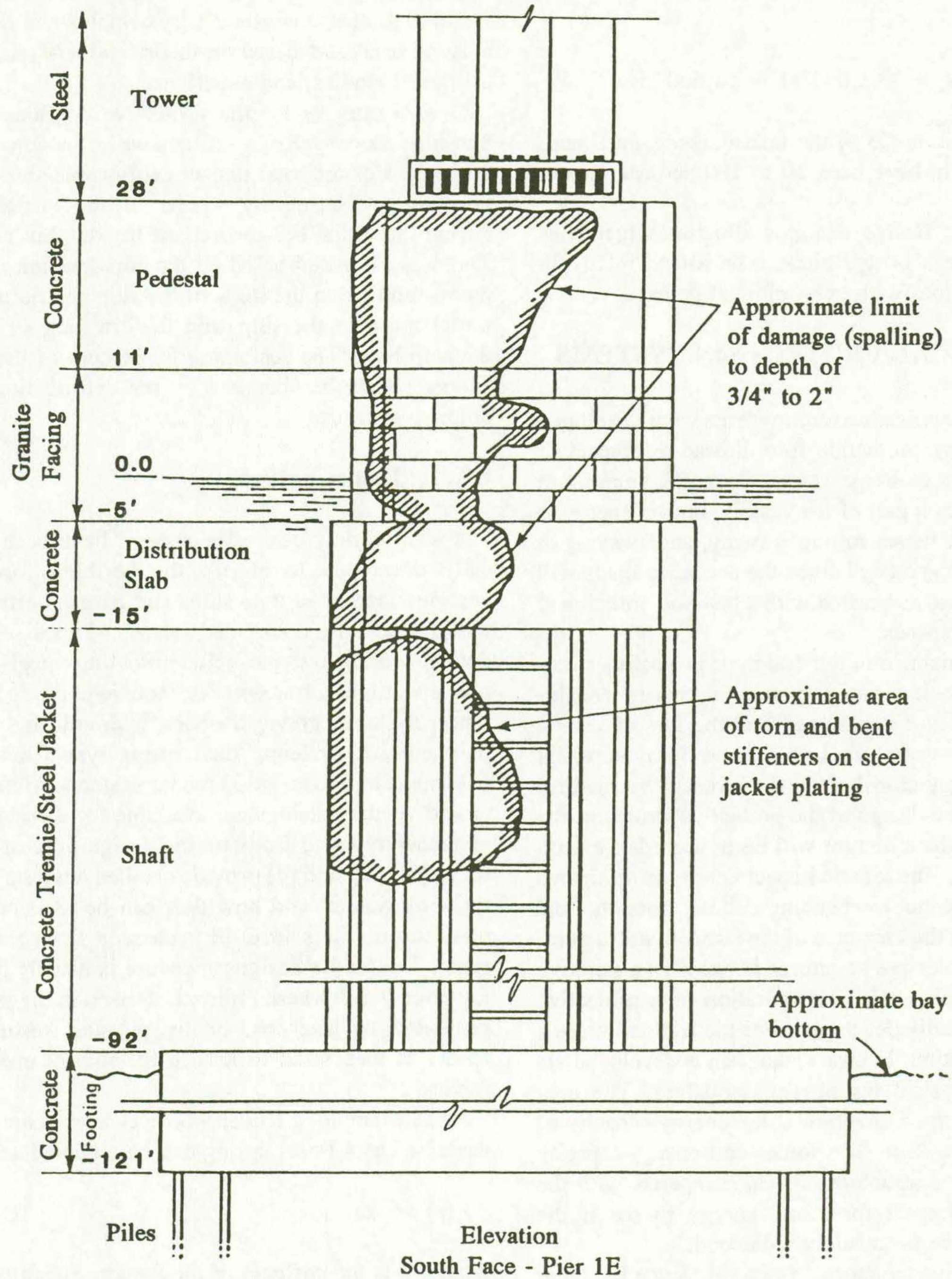


Figure C7.2-1. Damage to the Newport Bridge Main Pier After Collision with the M/V Maersk [1].

dynamic coefficient, C_H, equals 1.05. From Equation 3.8.1,

$$KE = (1.05)(45{,}000)(10)^2/29.2 = 161{,}000 \text{ kip-ft}$$

and

$$P_S = KE/a_B = 161{,}000/11 = 14{,}600 \text{ kips.}$$

As noted in Section C3.9, the instantaneous maximum force level might have been 50 to 100 percent greater than this.

The Newport Bridge example illustrates that it is possible to design bridge piers to withstand relatively large impact forces with only minimal damage.

C7.3 PHYSICAL PROTECTION SYSTEMS

The Guide Specification requirements were developed to provide bridge protection from a head-on impact of an aberrant ship or barge vessel. Eccentric impacts in which a significant part of the vessel's impact energy is absorbed by the vessel rolling, yawing, and swaying in the water is not specified since the eccentric loads will be less than those associated with a head-on collision at relatively high speeds.

The current practice in the design of protective structures is almost invariably based on energy considerations. In these it is assumed that the loss of kinetic energy of the vessel is transformed into an equal amount of energy absorbed by the protective structure. Regardless of the design of the protective structure, the work done by the structure will be in accordance with Equation 7.3-1. The kinetic impact energy is dissipated by the work done by bending, shear, torsion, and displacement of the members of the protective structure.

Design of protective structures is usually an iterative process in which a trial configuration of a protective structure is initially developed. For the trial structure a force vs. deflection, F(x)vs.x, diagram is developed via analysis or physical testing and modeling. The area under the F(x) vs. x diagram is the energy capacity of the protective system. The forces and energy capacity of the protective structure is then compared with the design vessel impact force and energy to see if the vessel loads have been safely withstood.

If the protective structure's force resistance is higher than the vessel impact force, then the vessel's bow will crush and the impact energy will be primarily absorbed by crushing of the vessel's bow. If the vessel impact force is higher than the protective structures resistance, then the impact energy will be primarily absorbed by the deflection and crushing of the protection system.

For the case where both crushing of the bow and deformation of the protective structure are to be included in the design, the designer must determine the portion of the impact energy to be apportioned to the vessel. The percentage of the energy absorbed by the vessel in such an analysis is very complex and judgment must be exercised based on theoretical analysis, physical model studies, and experience.

As an example, for the protective dolphins for the Sunshine Skyway Bridge discussed in Section C7.3.3, 20 percent of the total impact energy was absorbed by crushing approximately 4 feet of the vessel's bow during the initial 0.3 seconds of the dolphin collision. This was estimated based on the conservation of linear momentum given the mass of the ship and dolphin, the initial speed of the ship, and the crushing strength of the ship bow. The remaining 80 percent of the impact energy had to be absorbed by the deformation of the dolphin structure.

C7.3.1 Fender Systems

A wide variety of fender systems have been historically developed to absorb the berthing forces and energies associated with ships and barges berthing and mooring against docks and wharfs. Types of typical dock fendering systems include: floating camels, timber pile and timber frameworks, concrete piling, rubber fender systems, gravity fenders, hydraulic and hydraulic-pneumatic systems, steel spring type fenders, and pneumatic and foam filled fender systems. Manufacturer and vender catalogs are available for a wide variety of fender types to facilitate the design process. References [2], [3], and [4] provide detailed analysis of these types of systems and how they can be used on bridge piers to provide a level of protection from ship collisions. The fender design procedure is usually based on Equation 7.3-1 where force vs. deflection diagrams are generated by analysis, or by physical testing. The fender is then sized to absorb the impact energy and forces.

As an example, a fender whose characteristics can be described as a linear spring can be modeled as,

$$F(x) = kx \qquad \text{(C7.3.1-1)}$$

where, k is the stiffness of the fender. Substituting this expression into Equation 7.3-1 and integrating results in,

$$KE = \frac{1}{2}kx^2 \qquad \text{(C7.3.1-2)}$$

For a given value of KE and deformation, x, the required fender stiffness can be computed; or for a given stiffness the required deformation to absorb the energy can be computed.

A computer program has been developed for the elastic analysis of a variety of fender systems to facilitate the design of protection systems. The program is available from the Bridge Division, FHWA, Washington, D.C. The program does not include plastic or large deformation analysis for protective structures.

In general, fender systems are adequate to absorb the collision energy and loads associated with medium to small vessels at low impact speeds and at oblique angles. For larger vessels and higher impact speeds, other types of protection are usually required. Exceptions occur for those bridges with very massive pier structures and high shear and overturning resistance.

C7.3.1.1 Timber Fenders

Timber fenders are frequently used for bridge protection because of their relatively low cost and good energy absorption characteristics. Timber fenders are also placed on most other types of protection systems, such as pile supported structures and dolphins, in order to provide a rubbing and anti-sparking surface to avoid metal-to-metal contact with steel hulled vessels. This is particularly important for protective structures with exposed steel elements such as plates, walers, and bolts. In 1970 an accident involving loss of life occurred in Port Arthur, Texas in which a fender system with steel walers was hit by a gasoline barge. The barge of gasoline was ripped open on the steel fenderworks igniting the fuel [5]. It is important that all exposed steel hardware (bolts, plates, etc.) be either countersunk or placed behind the timber fender.

An example on the use of a framed timber protection system is discussed by Yiu [6] for the main piers of the Commodore John Barry Bridge, a cantilever truss bridge with a main span of 1,644 feet near Bridgeport, New Jersey crossing the Delaware River. The timber system shown in Figure C7.3.1.1-1 was developed to resist a "large ship" impact under the following conditions:

- Case I - Impact speed of 1.5 knots at 10 degrees angle to transverse axis of pier.
- Case II - Impact speed of 6 knots at 10 degrees angle to transverse axis of pier.
- Case III - Impact speed of 6 knots head-on with the longitudinal face of the pier.

Designed using the kinetic energy method, the analysis found that neither the ship or fender system were damaged in a Case I collision; that the fender would suffer damage during a Case II collision; and that the fender system would collapse under a Case III collision. Under Case III, the bridge pier would safely resist the resulting impact force computed by:

$$F = KE/x \qquad (C7.3.1.1\text{-}1)$$

where, F is the vessel crushing force (pier impact load), KE the kinetic energy of the collision, and x was taken as the 5 ft - 8 in depth of the six layer timber cribbing framework.

C7.3.1.2 Rubber Fenders

The high elasticity inherent in rubber results in relatively high energy absorption characteristics for rubber fender systems. When compared to timber, rubber fenders also have the advantage of low maintenance cost, high durability (lifespan several times that of timber fenders), superior physical and chemical properties (resistant to aging, oil, friction wear and tear, water and marine bore attack), plus ease of handling [5]. A disadvantage is their relatively high initial cost when compared to timber fenders.

An example on the use of rubber fender systems for bridge piers is provided by Shintaku [7] for pier-mounted rubber fenders on the main piers of the new Passyunk Avenue Bridge over the Schuylkill River in Philadelphia, Pennsylvania. The design conditions were for a 23,000 DWT ship traveling at 5 knots and impacting the fender line at an angle of 27 degrees. The impact energy to be absorbed by the fender system was computed to be 3,720 ft-kips. The analysis assumed that the energy would be absorbed by the deflection of four rubber arch fenders as shown in Figure C7.3.1.2-1. The impact reaction force resulting from the fender deflection to be applied to the pier was estimated to be approximately 2,400 kips.

C7.3.1.3 Concrete Fenders

Crushable concrete box fenders offer an effective method of absorbing vessel collision energy. By varying the box dimensions, wall thickness, and geometric layout of interior walls and diaphragms, a wide range of energy absorption capabilities can be achieved. The primary drawback to the fender is the difficulty in

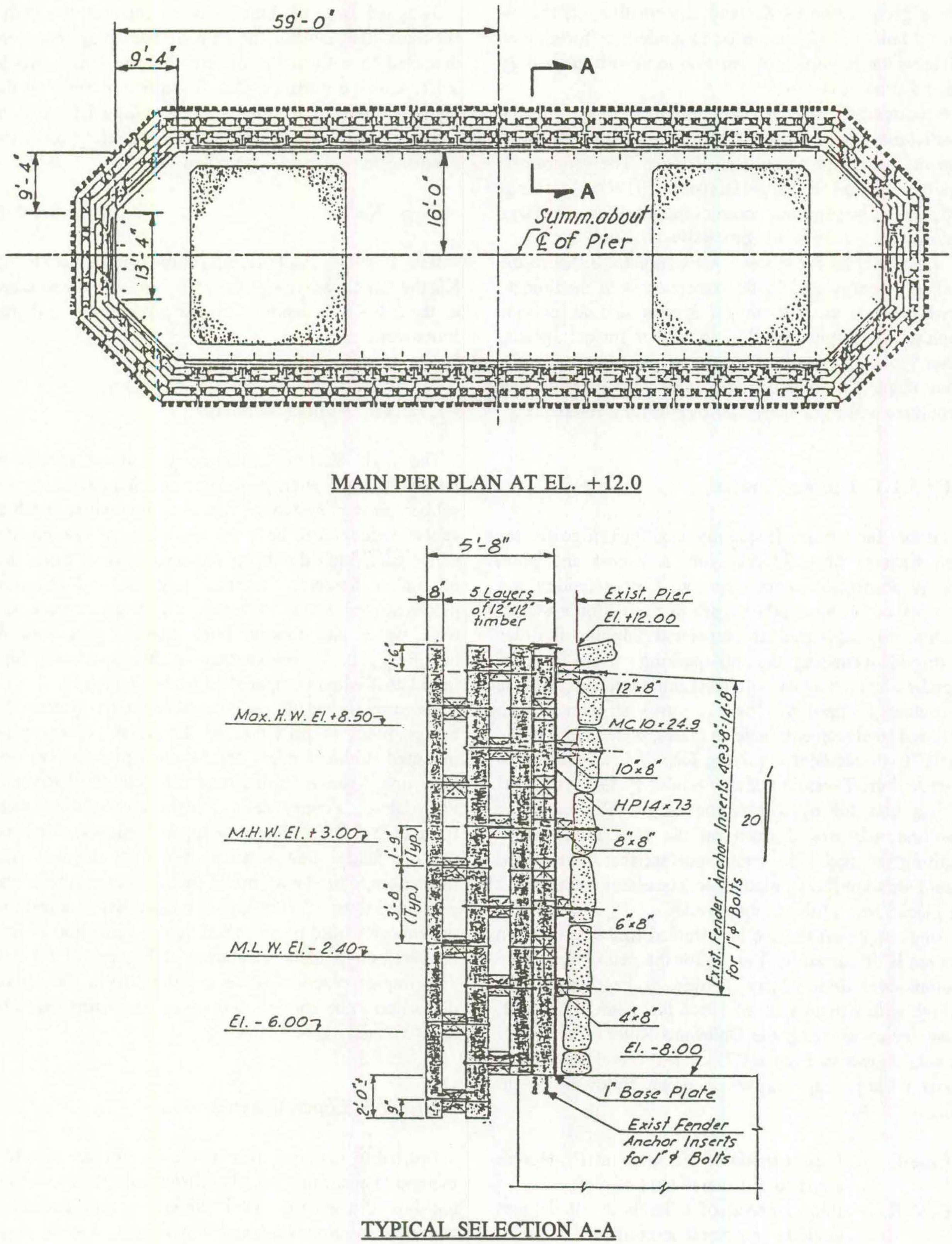
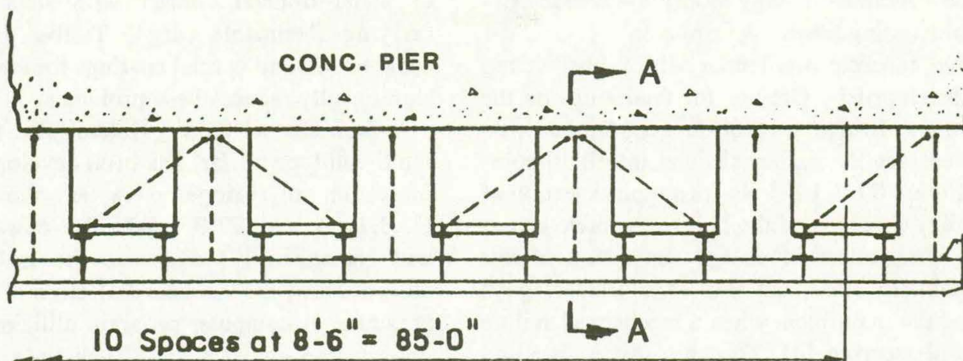
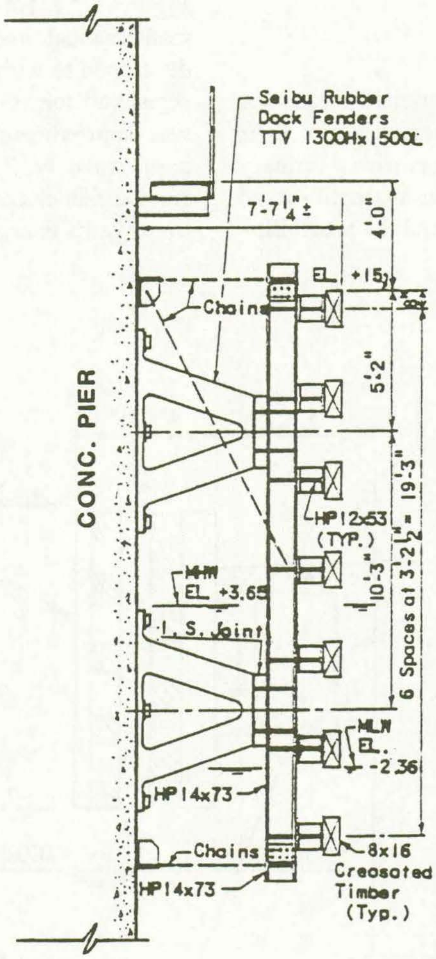


Figure C7.3.1.1-1. Timber Fender System on the Commodore John Barry Bridge, New Jersey [6].



PARTIAL PLAN OF MAIN PIER



SECTION A - A

Figure C7.3.1.2-1. Rubber Fender System of Passyunk Avenue Bridge, Philadelphia [7].

analyzing the structures' energy absorption characteristics while undergoing plastic deformation.

A crushable concrete box fender with timber facing strips was developed by Greiner for fenderings on the Francis Scott Key Bridge: a 1,200-foot continuous truss span which crosses the harbor channel into Baltimore, Maryland. Figure C7.3.1.3-1 shows a typical section of the box fender. Crushing of the hollow concrete box is the primary mechanism of energy absorption of this type of system. In 1980, the ship M/V Blue Negoya struck one of the main piers when a mechanical failure caused loss of steering [8]. The ship drifted head-on into the pier destroying the concrete box fender and impaling its bow on the A-frame pier columns. Only minor surficial spalling of the concrete occurred on the main pier columns due to the vessel bow overhang. In stopping the ship, the concrete and timber fender was totally destroyed and had to be replaced.

C7.3.1.4 Steel Fenders

Steel framed fenders provide an efficient means for absorbing relatively high impact energies due to their elastic and plastic deformation properties. Primary disadvantages to steel fenders are their susceptibility to corrosion in salt water environments and the possibility of metal-to-metal contact with steel hulled vessels carrying flammable cargo. Timber facing, concrete encasement, and special coatings for steel members can significantly reduce these problems.

A framed steel fender system (also referred to as a multi-cell type buffer) has been developed in Japan for protection of bridge piers as shown in Figures C7.3.1.4-1 and C7.3.1.4-2. The research by Namita and Nakanishi [9] discusses the method of energy absorption by the inelastic deflection of the framed steel structure. A computer program utilizing inelastic large deformation analysis for a steel truss framework was developed to compute the strain energy absorbed by the fender structure during the process of collapse. Using this approach Matsuzaki and Jin [10] developed the framed fender system design specifications for the main piers of the Bisan-Seto Bridge with a 3,600-foot suspension span near Honshu, Japan. The fender shown in Figures C7.3.1.4-1 and C7.3.1.4-2 were analyzed using mathematical and physical models. The fender was developed to withstand an 8 knot impact of a 500 gross registered ton (GRT) fishing vessel. The impact force was approximately 800 kips and the impact energy approximately 7,500-foot-kips. The interior of the framed fender can be filled with dense foam to further improve its energy absorption properties.

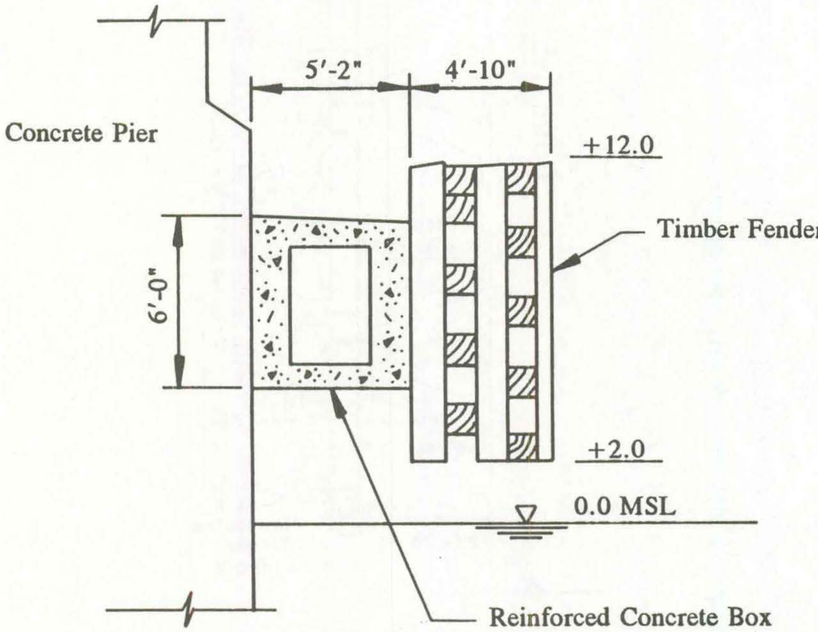


Figure C7.3.1.3-1. Crushable Concrete Box Fender on the Francis Scott Key Bridge Main Piers, Baltimore, Maryland.

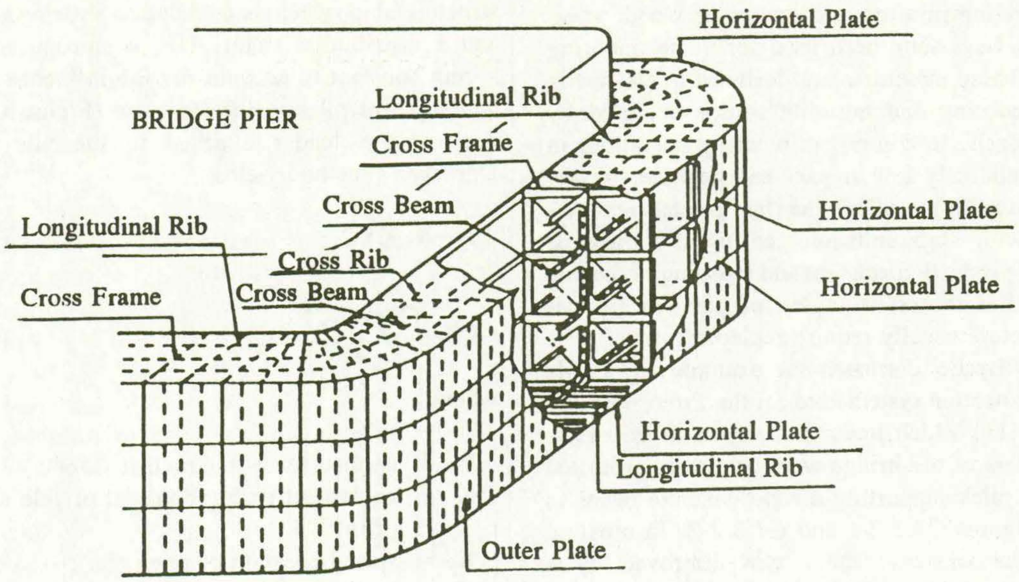


Figure C7.3.1.4-1. Framed Steel Fender System for Bisan-Seto Bridge, Japan [9].

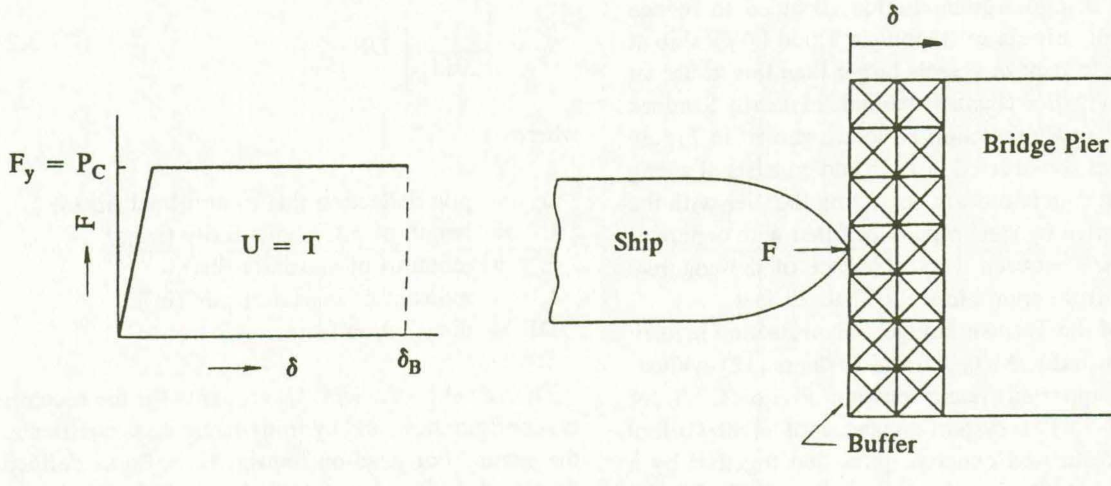


Figure C7.3.1.4-2. Load-Deformation Relationship of Framed Steel Fender Wall [10].

C7.3.2 Pile Supported Systems

Single standing piles or pile groups of wood, steel, and concrete have long been used for vessel mooring operations. These structures are designed to elastically resist the mooring and berthing forces imparted by merchant vessels. In contrast to mooring operations in which the relatively low impact energies can be absorbed elastically by piles, the far greater energies associated with ship collision can usually only be absorbed by plastic deformation and crushing of the pile structure. After the collision, all or parts of the destroyed structure usually require replacement.

P. Tambs-Lyche discusses an example of a pile supported protection system used for the Tromso Bridge in Norway [11] which has a main span of 260 feet. The main piers of the bridge were originally protected by concrete piles supporting a rigid concrete beam as shown in Figures C7.3.2-1 and C7.3.2-2. In separate accidents, the western fender was destroyed by a collision with a 10,000 DWT vessel in 1961, and the eastern fender was destroyed by collision of a 1,560 DWT ore-ship in 1963. The capacity of the original fenders was estimated to stop a 10,000 DWT ship drifting at a speed of 1 knot.

Following these accidents, an investigation recommended that the protection system be replaced with a stronger pile supported structure capable of stopping a 12,000 DWT ship impact at a speed of 8 knots. The construction costs were so expensive however that the Norwegian Bridge Administration decided to reduce the protection criteria to stopping a 7,000 DWT ship at 8 knots and to require vessels larger than this to use an alternate navigation channel available in nearby Sandnes Sound. The new protection structure shown in Figure C7.3.2-3 was constructed in 1975 and consists of a ring shaped rigid concrete beam encircling the pier with the beam supported by steel pipe piles filled with concrete. The clearance between the inside face of the concrete ring varied from approximately 17 to 23 feet.

As part of the Tasman Bridge pier protection investigation in Australia, Maunsell and Partners [12] evaluated the pile supported system shown in Figures C7.3.2-4 and C7.3.2-5. The system consisted of eight 10-foot diameter prestressed concrete piles tied together by a rigid cap beam. During the design impact of a 35,000 DWT ship at 8 knots, the piles would form plastic hinges at the top and bottom to absorb the impact energy through rotational deformation.

Derucher [2] developed the following dynamic analysis method for the design and analysis of pile supported protective structures. The analysis assumes that the pile and fenders remain in the elastic range, and that the ship is a non-deformable rigid body. The pile structure/ship system is modeled as a spring and weight and a distribution factor, DF, is introduced into the spring constant to account for the influence of walers and adjacent piles in the structure (Figure C7.3.2-6). Assuming a fender attached to the pile structure, Derucher's method yields;

$$P = KYC \qquad \text{(C7.3.2-1)}$$

$$K = \frac{(K_p)(K_f)}{(K_p + K_f)} \qquad \text{(C7.3.2-2)}$$

where

- P = applied force to structure (kips);
- K = equivalent spring constant of pile and fender (k/in);
- K_p = spring constant of pile (k/in);
- K_f = spring constant of fender (k/in);
- Y = maximum system deflection (in);
- C = vessel coefficient.

The stiffness of a cantilevered pile with a unit lateral load on top can be computed by:

$$K_p = \frac{1}{\Delta_p} \qquad \text{(C7.3.2-3)}$$

$$\Delta_p = \left[\frac{L^3}{3EI_p}\right] DF \qquad \text{(C7.3.2-4)}$$

where

- Δ_p = pile deflection due to unit load (in/k);
- L = length of pile above fixity (in);
- E = modulus of elasticity (ksi);
- I_p = moment of inertia of pile (in^4);
- DF = distribution factor.

The vessel coefficient, C, accounts for the eccentricity, configuration, and hydrodynamic mass coefficient of the vessel. For head-on impact, $C = C_H$ as defined in Section 3.8. The distribution factor, DF, developed by Derucher can be computed as;

$$DF = \left[-6.0 \times 10^{-7}\,(D_x) + F\right] L^{-.006} \qquad \text{C7.3.2-5)}$$

where

$$F = -3.5\text{x}10^{-13}(D_y)^2 + 3.1\text{x}10^{-7}(D_y) + .335; \qquad \text{(C7.3.2-6)}$$

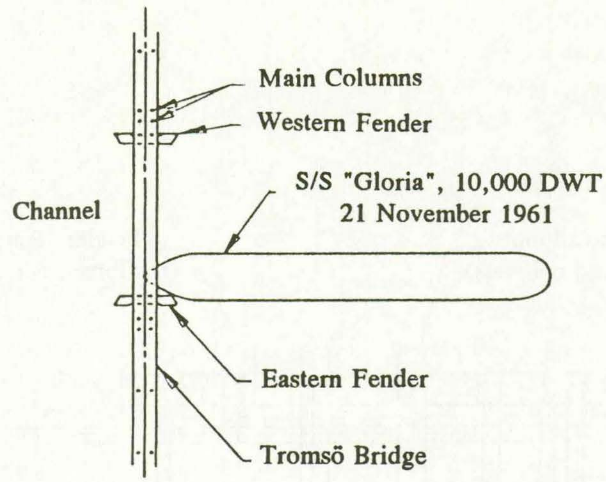


Figure C7.3.2-1. Plan of 1961 Ship Collision with the Tromso Bridge, Norway [11].

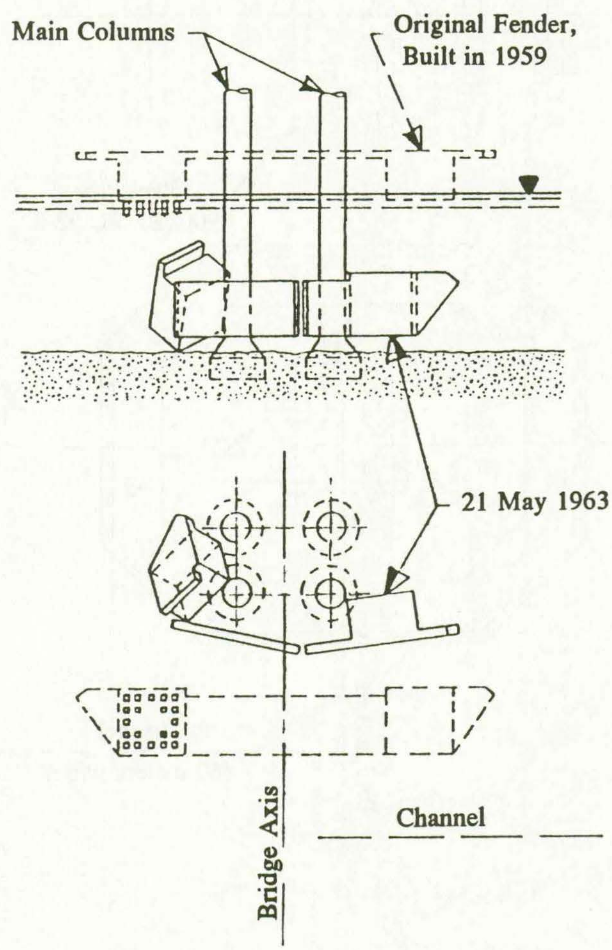


Figure C7.3.2-2. Detail of Destroyed Pile Supported Fender of the Tromso Bridge due to a 1963 Ship Collision [11].

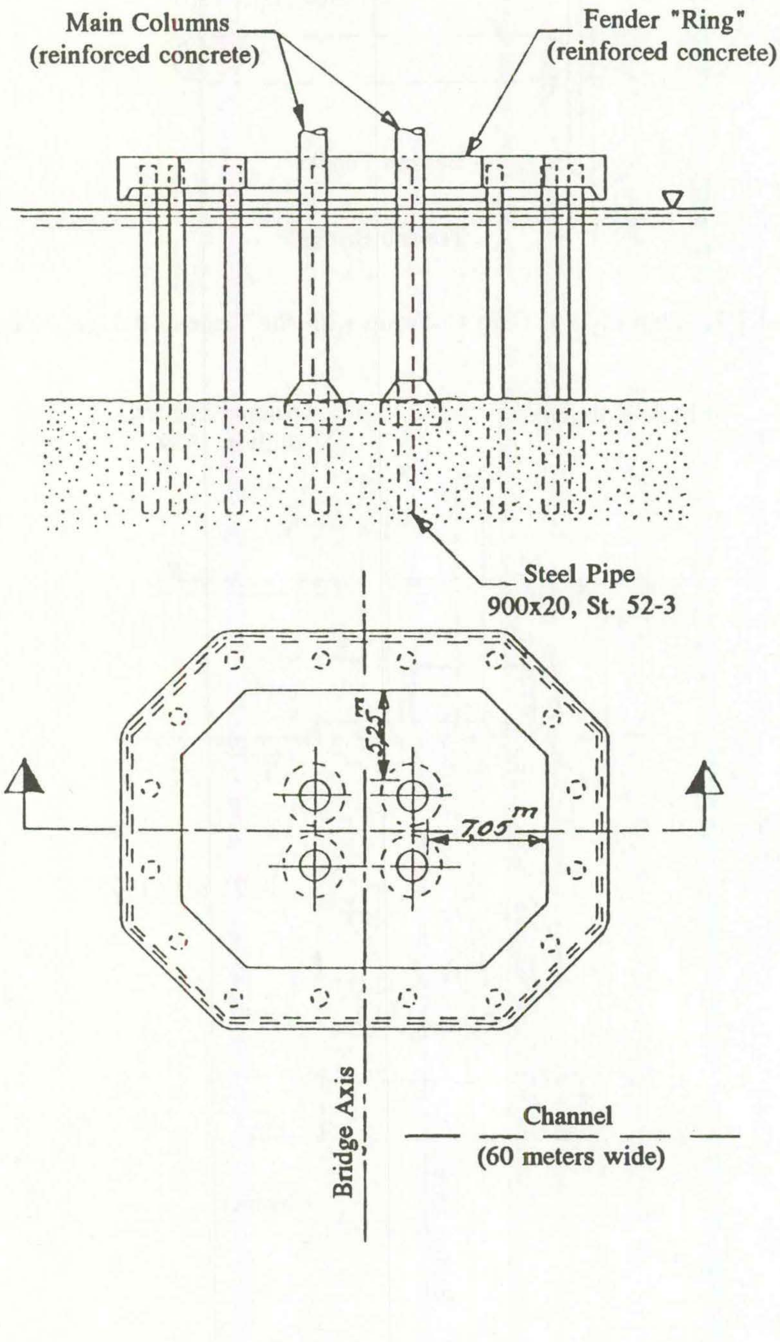


Figure C7.3.2-3. Pile Supported Protection System for the Tromso Bridge, Norway [11]. (All Units are Metric)

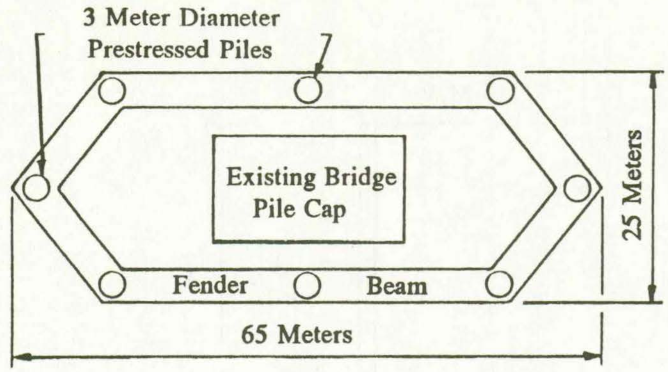


Figure C7.3.2-4. Plan of Pile Supported Pier Protection System Evaluated for the Tasman Bridge, Australia [12]. (All Units are Metric)

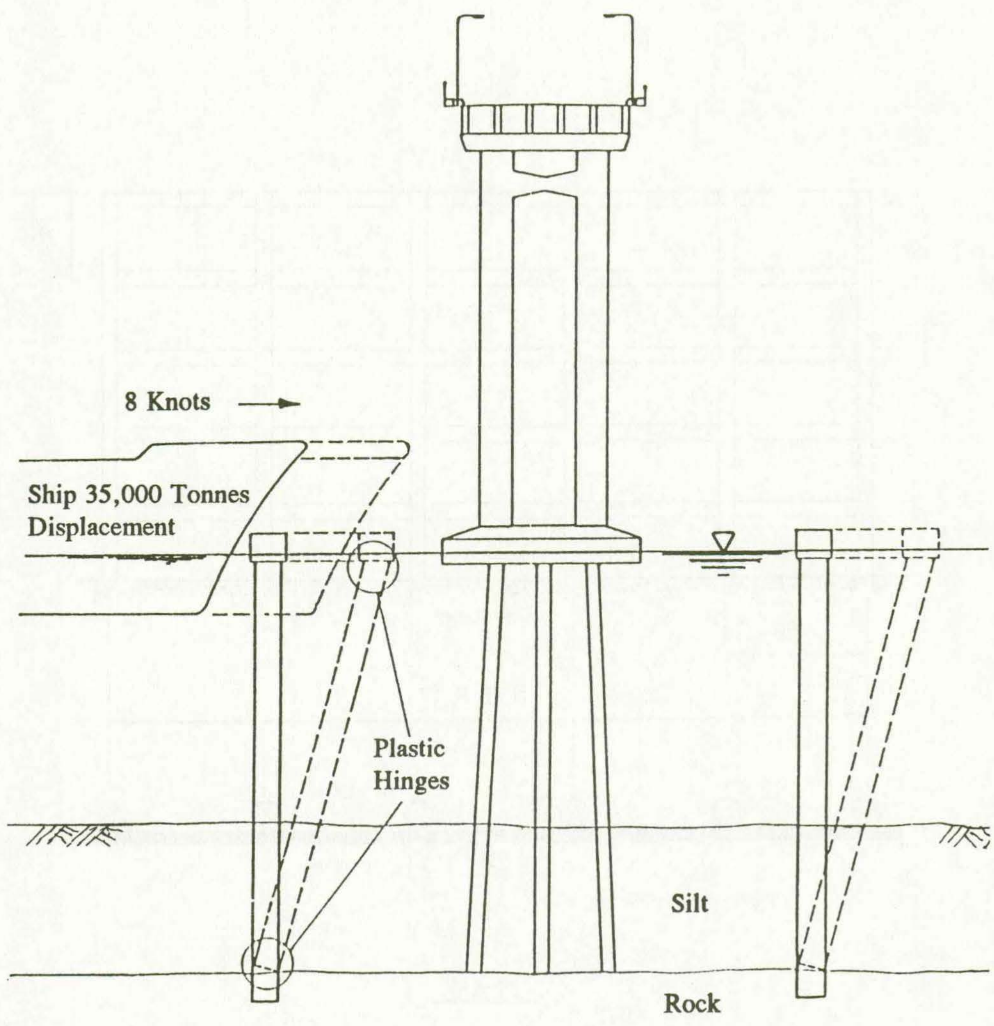
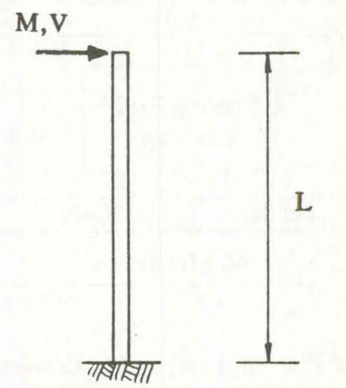
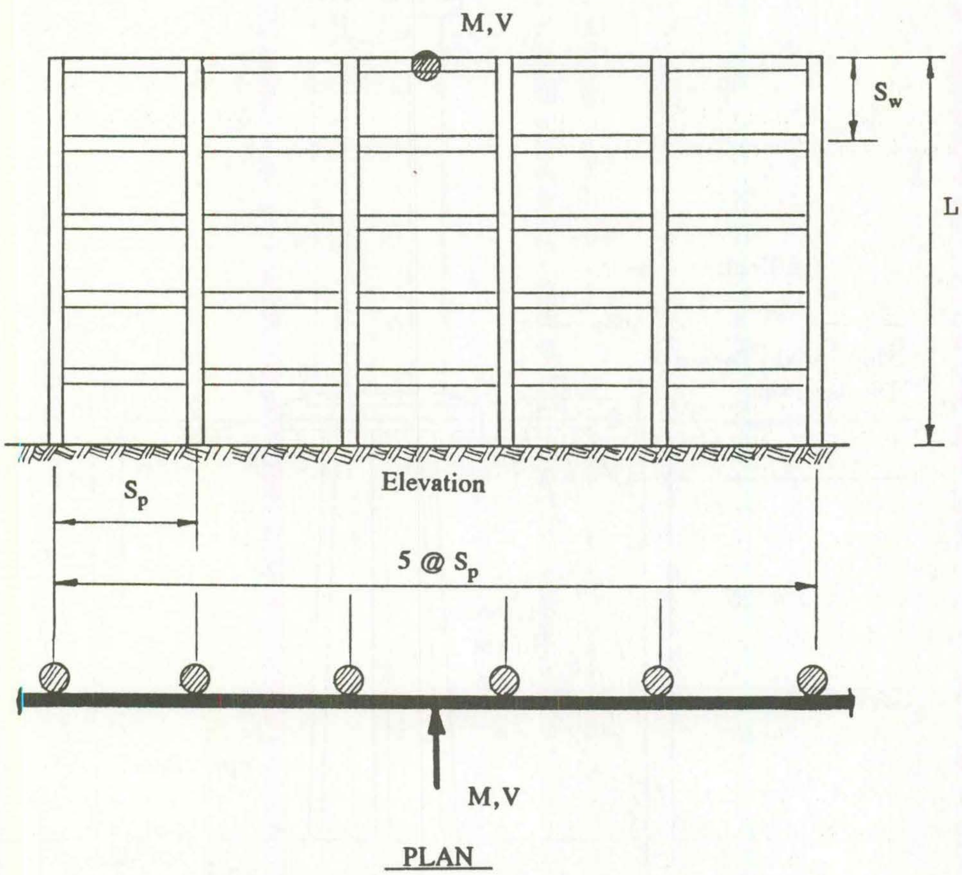


Figure C7.3.2-5. Section of Pile Supported Pier Protection System Evaluated for the Tasman Bridge [12].



a. Single Pile Elevation.



b. Multiple Pile Fender Structure.

Figure C7.3.2-6. Typical Pile Structure Geometry for Derucher's Dynamic Analysis [2].

D_y = vertical stiffness = EI_p/S_p; (C7.3.2-7)
S_p = vertical pile spacing (in);
D_x = horizontal stiffness = EI_w/S_w; (C7.3.2-8)
S_w = horizontal waler spacing (in);
I_w = waler moment of inertia (in^4).

The maximum system deflection, Y (inches), and period, λ, can be computed as:

$$Y = V/\lambda \text{ (inches)} \quad (C7.3.2\text{-}9)$$
$$= (K/M)^{1/2} \quad (C7.3.2\text{-}10)$$

where

V = impact velocity (in/sec);
M = mass of vessel ($k\text{-}in/s^2$).

The acceleration, a, and stopping time, t, can be determined as follows:

$$a = V\lambda \text{ } (in/s^2) \quad (C7.3.2\text{-}11)$$
$$t = (\pi/2\lambda) \text{ (sec)} \quad (C7.3.2\text{-}12)$$

C7.3.3 Dolphin Protection

Large diameter dolphins have frequently been used in the U.S. and Canada for protection of bridge piers, dock structures, and for mooring of relatively large vessels. The circular cells are typically constructed of driven steel sheet piling, filled with rock or sand, and topped by a concrete cap. Timber or rubber fenders are usually placed on the outer perimeter of the dolphin to act as an anti-sparking surface to prevent metal-to-metal contact in the event of collision with a steel hulled vessel carrying flammable products. Existing examples of dolphin protection in the U.S. include; the Outerbridge Crossing, New York; the Betsy Ross Bridge across the Delaware Bay; the Dame Point Bridge in Jacksonville, Florida; and the Sunshine Skyway Bridge across Tampa Bay, Florida.

The circular shape of the dolphins can help deflect aberrant vessels away from the pier. The cell should, however, be designed for the maximum loading case of a head-on impact. If the dolphin is stronger than the vessel, then the vessel will absorb most of the impact energy through crushing of its bow. If the dolphin is weaker than the vessel, then the dolphin absorbs most of the energy by large translational (sliding) and rotational deformations. An example of the former situation occurred in 1986 when a small 200 ton fishing vessel rammed one of the massive 60-foot diameter dolphins on the Skyway Bridge. The vessel was severely damaged and sank almost instantly whereas the dolphin was completely undamaged. An example of the latter situation occurred during the 1979 ship collision with the Outerbridge Dolphin No. 4 which is discussed later in this section.

A balance between the cost and safety associated with these two conditions is usually sought during design since the larger the dolphin the higher the construction cost; and the smaller the dolphin the increased risk that the vessel will not be stopped before hitting a bridge pier. Figure C7.3.3-1 illustrates the case where the collision energy is absorbed by both the cell and the ship. For those situations where large plastic deformations are permitted, it is recommended that the maximum displacement of the top of the dolphin be limited to one-half of the cell diameter under the design impact. In addition, the sheet piling should be embedded a sufficient distance into the waterway bottom that they will not pull out past the mudline if the dolphin rotates.

Design computations for dolphins are usually based on a consideration of the energy changes that take place during an impact. Force-displacement relationships are typically developed for the following forces:

- crushing of the vessel's bow
- lifting of the vessel's bow
- friction between the vessel and dolphin
- friction between vessel and bay bottom
- sliding of the dolphin
- rotation of the dolphin

The area under the force-displacement diagram equals the energy absorption capacity of the item. It is assumed that the deformation of the ship/dolphin system will follow a path of least energy. For each configuration of dolphin and ship impact, a deformation path can be developed. Deformation stops when all the impact energy has been absorbed.

Case histories of several dolphin protection systems for bridges are presented below:

In 1961, a 45-foot diameter mooring dolphin located in the Port of Philadelphia was struck head-on by a loaded 35,000 DWT ore-carrier at an estimated speed of 8 knots [13]. The dolphin was located in 36 feet of water and was constructed of steel sheet piling filled with sand and gravel and topped with a 4-foot thick concrete cap. The top of the dolphin extended 12 feet out of the water and had a timber fender on the outside. The upper 4 feet of the sheet pile interlocks were welded throughout the circumference. The sheet piling extended approximately 20 feet into the river bottom. As a result of the collision, the dolphin tilted over, the top moving 12 feet, but without being overturned. The

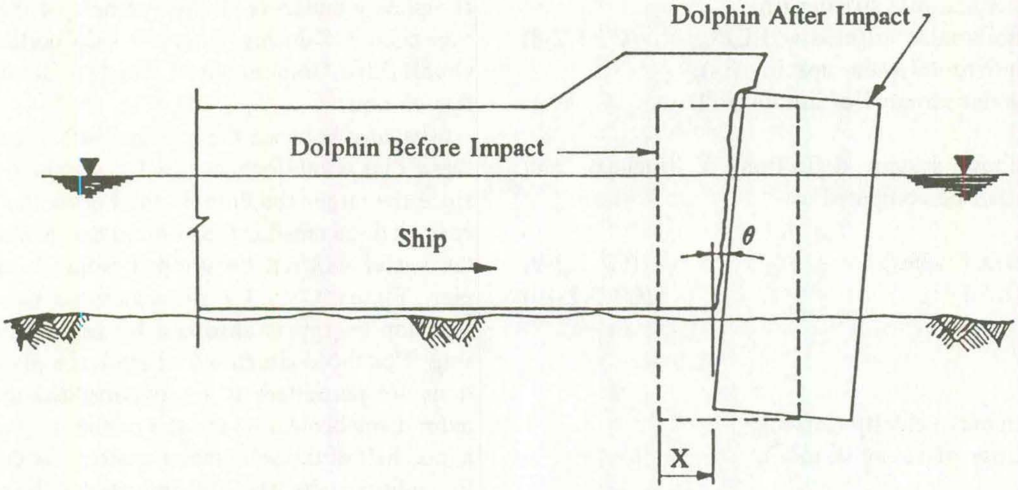


Figure C7.3.3-1. Collision Energy Absorbed by Dolphin Rotation and Sliding, and by Crushing of Vessel Bow.

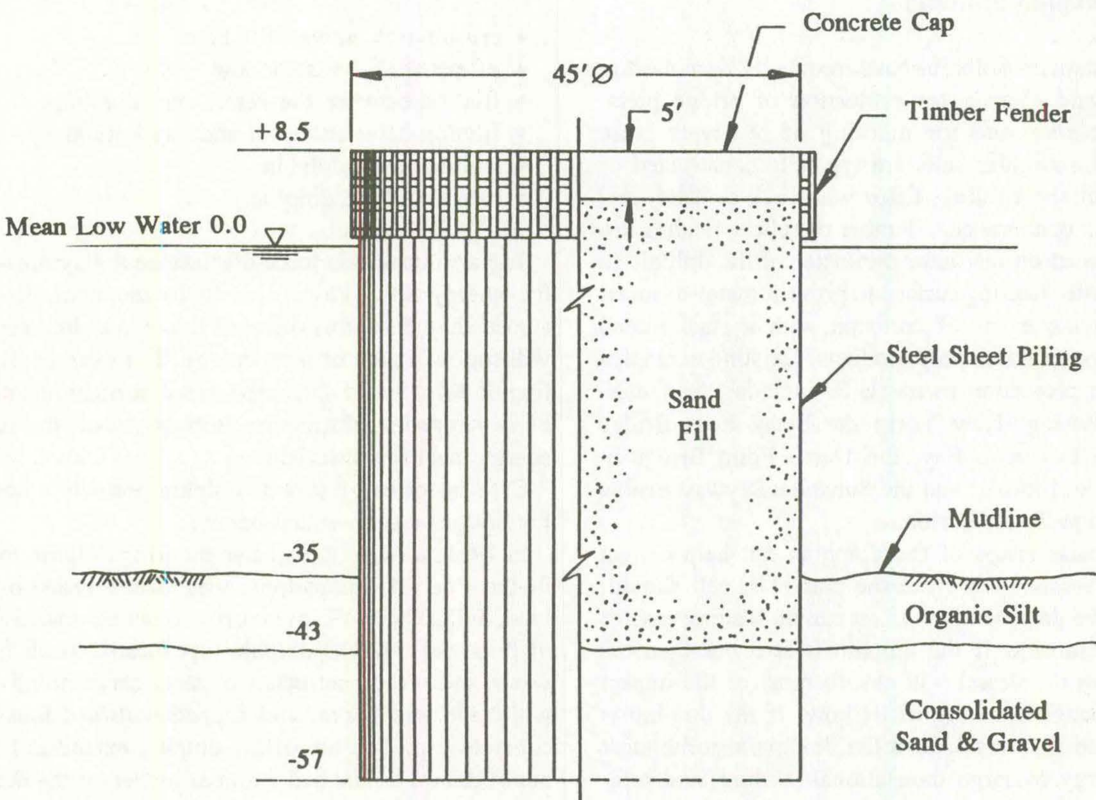


Figure C7.3.3-2. Typical Dolphin Protective Cell on the Outerbridge Crossing, New York, after [4].

sheet piles on the outside, near the point of impact, were lifted up and the piling on the opposite side of the cell buckled due to compressive forces. The welded connections of reinforcement on the inside of the sheet piles broke. The bow of the ship was crushed in several feet with the shape of the deformation matching the circular dolphin shape.

Rama [14] and Englot [15] describe the dolphin protection of the Outerbridge Crossing the Arthur Kill waterway near New York City. Constructed in 1928, the steel cantilever truss bridge has a main span of 750 feet with the main piers located in approximately 35 feet of water. Figures C7.3.3-2, C7.3.3-3, and C7.3.3-4 show the bridge and dolphins used to protect the structure. Originally, the bridge had only a timber fender on the main piers, however, subsequent to a minor collision of a 10,000 DWT tanker in 1960 with one of the main piers, the bridge owners (the Port Authority of New York and New Jersey) decided to construct the dolphin protection shown in Figure C7.3.3-3. Similar to the Philadelphia mooring dolphin, the Outerbridge Crossing steel sheet pile protection cells were 45 feet in diameter, filled with coarse sand, and topped by a 5-foot thick concrete cap (Figure C7.3.3-2). The pilings were driven 8 feet through organic silt and 14 feet into a consolidated sand and gravel layer underlying the river bottom. As seen in Figure C7.3.3-3, the south side of Pier "D" is protected by a cluster of three dolphins because of its geometric vulnerability to ship collision from that direction.

In 1979, the dolphin protection system was put to the test when a loaded 45,000 DWT tanker struck Cell No. 4 head-on in front of main pier "D" at an estimated collision speed of 2.5 knots. The collision took place during dense fog conditions and while the vessel was proceeding upstream under a two-tug escort. Only the center dolphin was hit during the impact, and the ship continued forward 50 feet before it was dragged to a stop. The ship suffered only minor damage to its bow (unlike the 1960 collision in which a 100-foot long gash was torn in the ship's hull). An inspection after the accident revealed that the steel sheet piling of Dolphin No. 4 had burst open, spilling out sand, and that the piling on the ship impact side had been pulled out. The remaining pilings of the cell were bent over at the river bottom. The 45-foot diameter by 5-foot thick concrete cap was found completely intact but displaced 50 feet due to the collision. After two years of delay due to processing the necessary environmental permits, a replacement cell was constructed in front of the destroyed Cell No. 4.

In 1987, the dolphin protection of the same main pier of the Outerbridge Crossing was again tested when a 48,000 DWT tanker collided with, and destroyed Cell No. 5. The cell was 45 feet in diameter and similar to Cell No. 4 described above. Damage to the cell is shown in Figure C7.3.3-4. The top of the cell was displaced laterally approximately 15 feet. Englot [15] identified three basic dolphin deformations due to ship impact with the cell:

1) The sheet pile shell elongates in tension on the side being impacted and buckles or crushes in compression on the opposite side, as the side walls also deform in shear, all commensurate with the lateral displacement of the concrete cap which serves as a rigid diaphragm and remains essentially intact.
2) Upon impact the local inward compression of the sheet pile wall displaces the sand filling which causes hoop tension forces on the sheet pile interlocks and pressure on the underside of the cap. As deformation and rupturing of the cell wall occurs, sand is lost at the perimeter of the cap. During the 1979 and 1987 collisions, Englot states that the concrete cap remained lodged within the sheet pile cell and was pushed down below the water.
3) There is local rupturing of the steel sheet piling at the point of vessel impact which also controls the level of damage to the impacting vessel.

Following the 1987 collision and destruction of Dolphin No. 5, the Port Authority declared an emergency situation and immediately brought a contractor on board to construct a new 60-foot diameter cell around the destroyed 45-foot diameter cell as shown in Figure C7.3.3-3. By placing the new cell in the same location as the originally permitted structure, and given the emergency nature of the construction, the Port Authority was able to quickly complete a replacement cell for the vulnerable pier within eight months after the accident.

The Sunshine Skyway Bridge pier protection system developed by Greiner, Inc. [16] for the Florida Department of Transportation utilizes a combination of dolphin and island protection as shown in Figure C7.3.3-5. The main piers are protected by islands whereas the five approach piers on each side of the main piers are protected by a dolphin system. The use of dolphins to protect the high level approach piers was a result of the risk analysis [17] which indicated that the high level approach piers were vulnerable to a catastrophic vessel collision. Figure C7.3.3-6 is a typical cross section of the Skyway dolphins. The 60-foot diameter cells were designed to withstand a collision from either a loaded

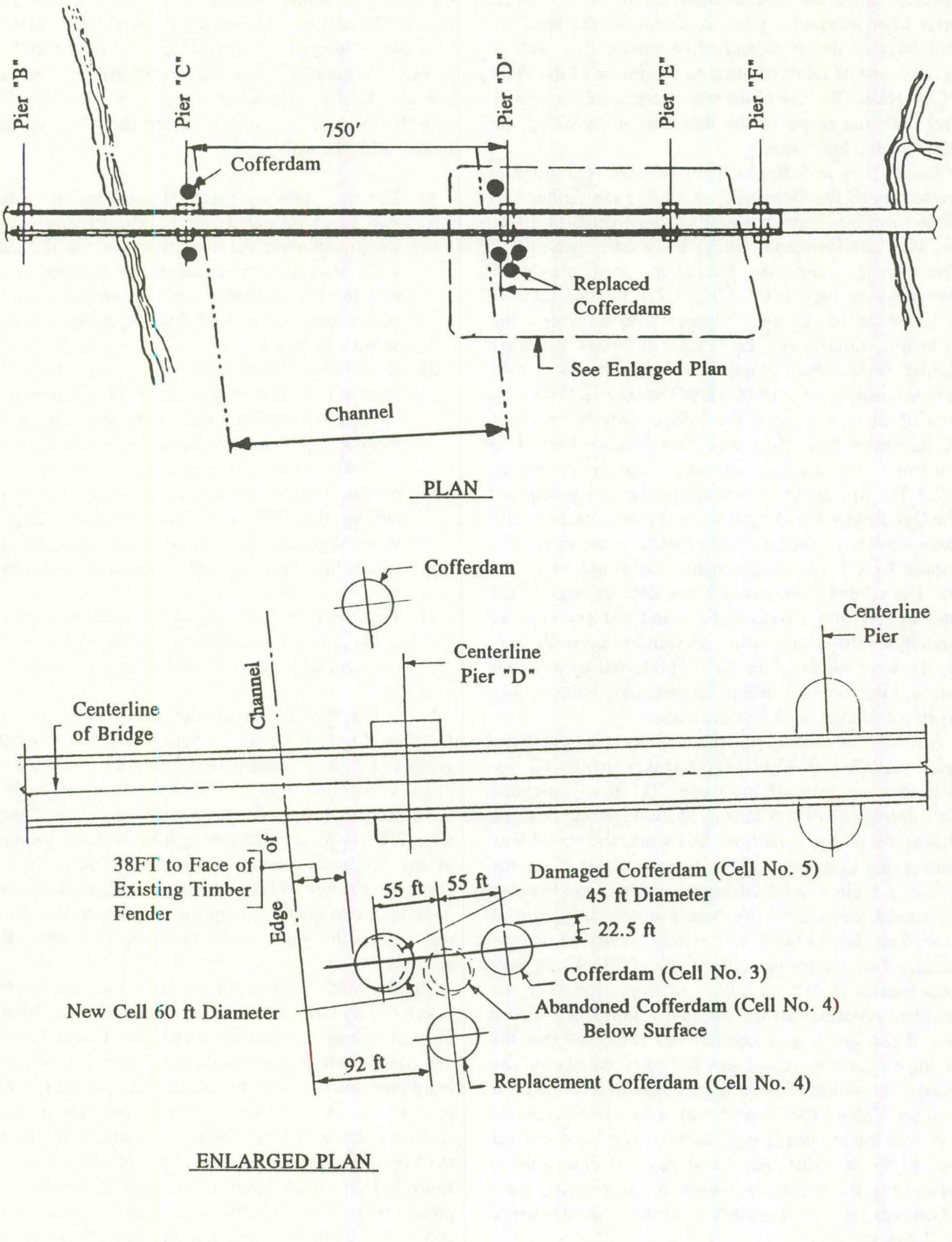


Figure C7.3.3-3. Plan of Dolphin Protection System for the Outerbridge Crossing of the Arthur Kill Waterway, New York [5].

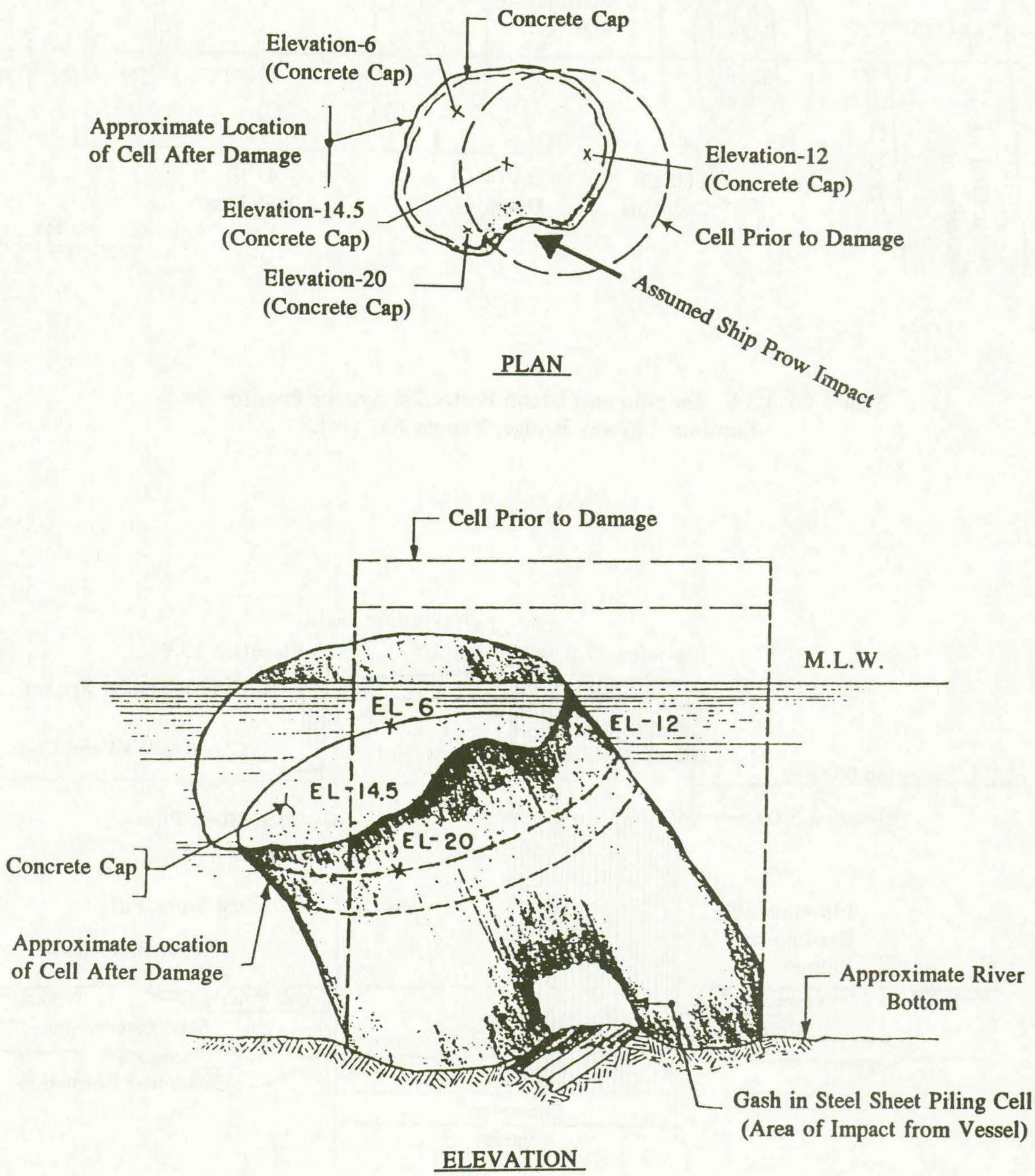


Figure C7.3.3-4. Damage to Dolphin No. 5 of the Outerbridge Crossing due to Ship Collision in 1987 [15].

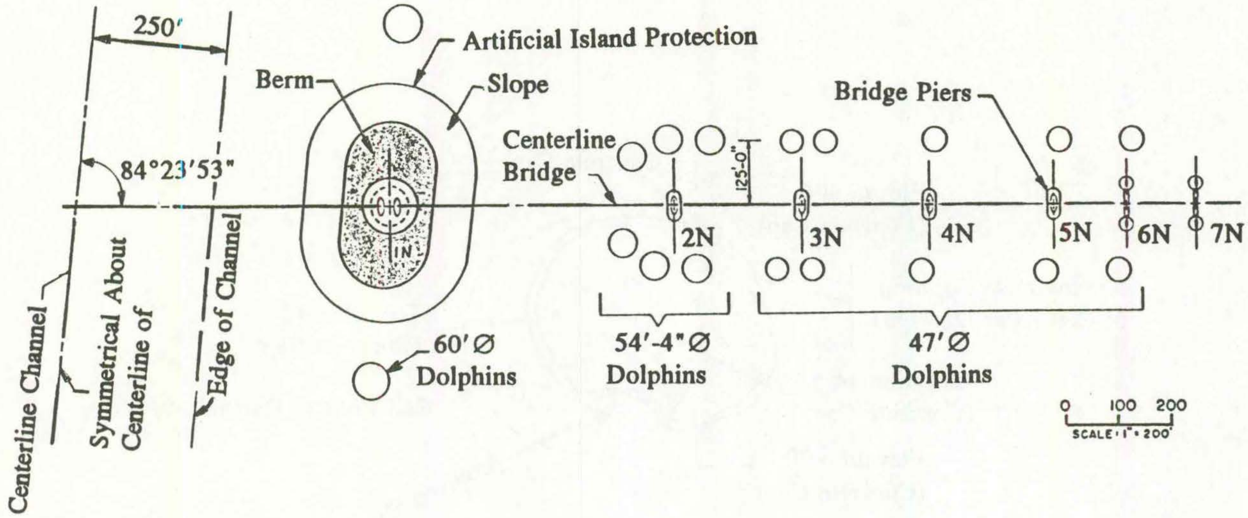


Figure C7.3.3-5. Dolphin and Island Protection System Plan for the Sunshine Skyway Bridge, Tampa Bay [16].

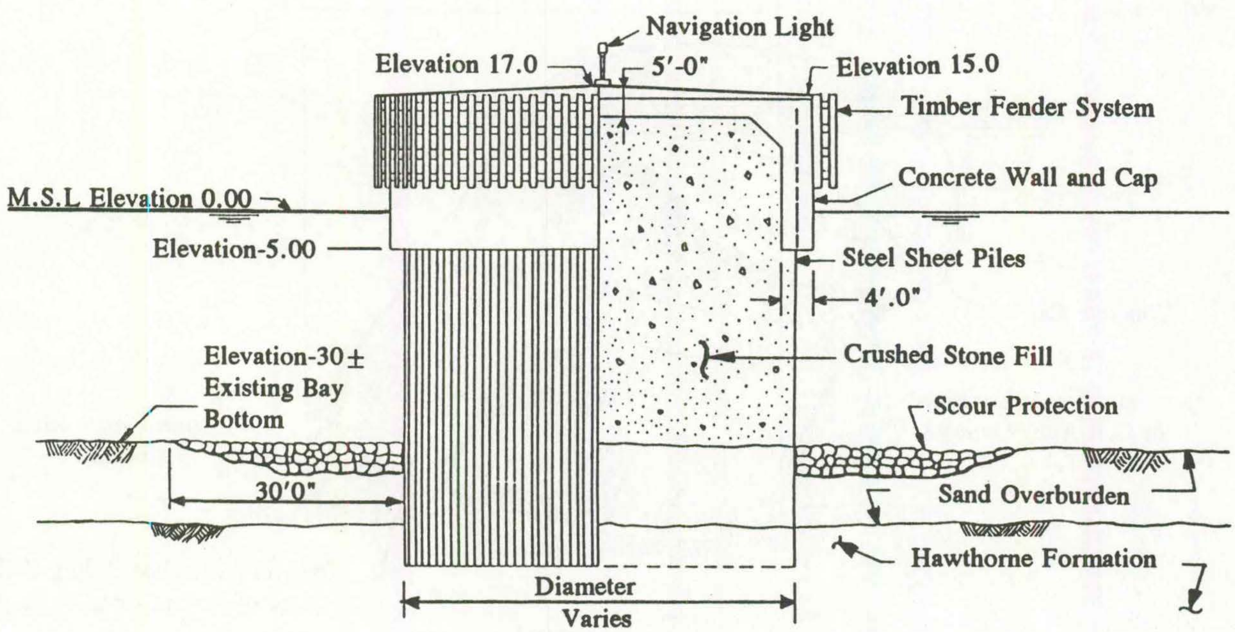


Figure C7.3.3-6. Typical Dolphin Details for the Sunshine Skyway Bridge [16].

23,000 DWT or an empty 87,300 DWT bulk carrier; the 54 ft - 4 in diameter dolphins from impacts with a loaded 25,000 DWT barge, or an empty 70,000 DWT vessel; and the 47-foot diameter cells to withstand impacts from a loaded 15,000 DWT barge or an empty 35,000 DWT ship. All design impact speeds were 10 knots. The Skyway sheet piling were driven through a sand overburden (10-40 feet thick) and then 5 to 10 feet into a staff limestone stratum known as the Hawthorne Formation.

Utilizing the information from some of the previous dolphin collisions discussed above, the Skyway dolphins incorporated several modifications to the one shown in Figure C7.3.3-2. In [16], Knott recommends that the key to a dolphin's ability to absorb a major ship collision is to tie the top of the sheet piling rigidly together with the concrete cap. This is accomplished by using high strength sheet pile interlocks, welding the steel sheet pile interlocks together near the top of the cell, and by enclosing the top of the dolphin with a thick reinforced concrete cap with the reinforcing steel penetrating through holes in the sheet piling. A thick concrete wall encircles the top side of the cell with additional reinforcing steel to help carry the high hoop stresses which occur during the collision. These structural details effectively "fix" the top of the dolphin causing a rigid diaphragm which holds the cell together during the 1 to 3 second collision impact interval. After a major collision the cell would be destroyed and would require replacement.

Although never built, the typical dolphin shown in Figure C7.3.3-7 was developed for protection of the Zarate-Brazo Largo Bridges in Argentina [18]. Located in 100-foot deep water, the dolphins consisted of an approximately 85-foot diameter precast concrete hollow cylinder which was supported by a ring of 6.5-foot diameter drilled concrete shafts. The walls of the dolphin were approximately 10 feet thick and were also partially hollow. The top cap of the dolphin was triangular shaped to help deflect aberrant vessels away from the bridge piers behind the cells. The design vessel for the dolphin system was a 20,000 DWT vessel impacting at 4 knots.

Methods for designing steel sheet pile dolphins have been developed by Parkinson [19] and Heins [20] for cells which stay in the elastic range. Dolphins which act elastically are analyzed for two general requirements; 1) internal stability (such as interlock tension, interlock slippage, and shear failure of the cell), and 2) external stability (such as sliding and overturning) under vessel impact. Figure C7.3.3-8 and the elastic analysis method presented below are summarized from Parkinson's paper [19].

The maximum interlock tension in a steel sheet pile dolphin is typically computed by,

$$t = pr/12 \qquad \text{(C7.3.3-1)}$$
$$p = (\gamma_f)(h)(K_a) \qquad \text{(C7.3.3-2)}$$

where

t = interlock tension (lb/in);
p = lateral fill pressure (lb/ft^2);
γ_f = average unit weight of fill (lb/ft^3);
K_a = active earth pressure coefficient;
r = dolphin radius (ft).

The distance, h(ft), from the top of cell to the plane of maximum interlock stress is taken equal to a third of the distance up from the plane of fixity of the piles, where a plastic hinge develops from the lateral loading. Parkinson recommends the distance from the mudline to the plane of fixity be computed by,

$$d_f = (3.1)T \qquad \text{(C7.3.3-3)}$$
$$T = (EI/n_h)^{1/5} \qquad \text{(C7.3.3-4)}$$

where

d_f = depth to fixity;
E = modules of elasticity of pile section (psi);
I = moment of inertia of pile section (in^4);
n_h = modules of horizontal subgrade reaction (lb/in^3).

For fixity of the pile to develop, the piles should be embedded to a minimum depth, d_{min}, of,

$$d_{min} = (5)T \qquad \text{(C7.3.3-5)}$$

The effect of vessel impact on the dolphin interlock tension are unknown, however, work by Shroeder and Maitland [21] indicates that increases in interlock tension due to lateral loads are on the order of 25 percent at the mudline.

Lateral loads on dolphins are also resisted by the shear resistance in the sheet pile interlocks. The shear on the interlocks, s_i, can be computed by,

$$s_i = (1/2)\gamma_f H^2 r\mu \qquad \text{(C7.3.3-6)}$$

where

H = height of dolphin to the location of the plastic hinge (ft);

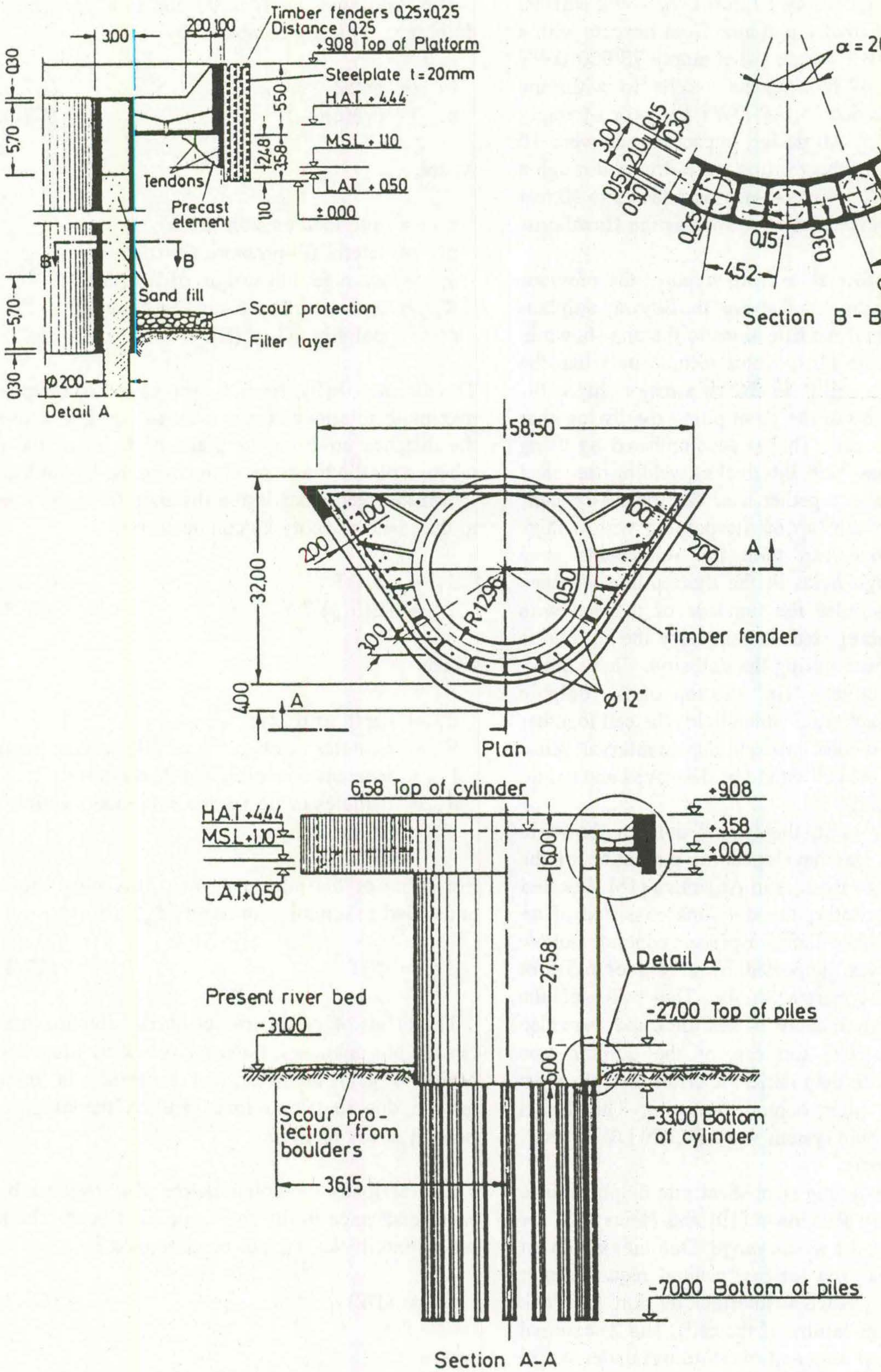
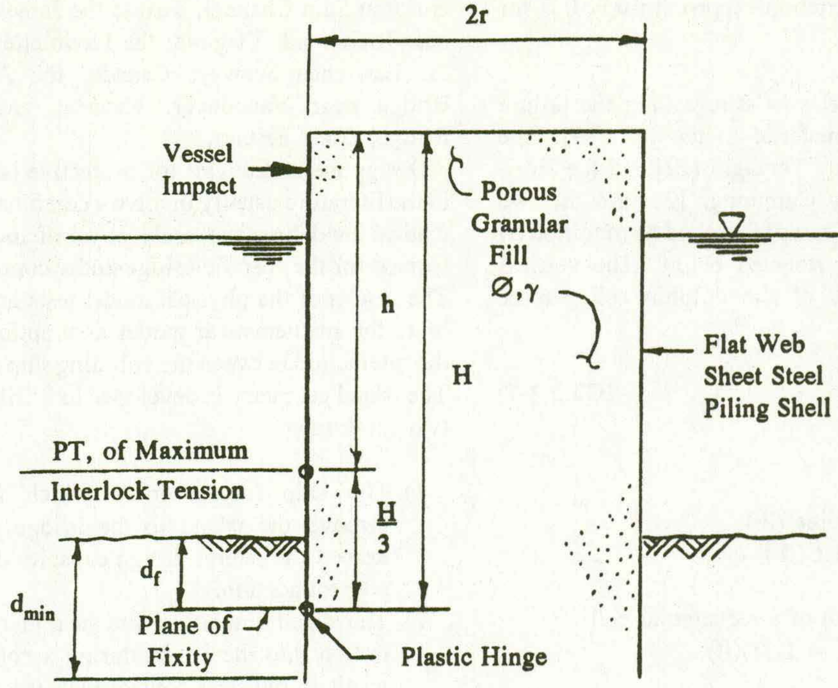
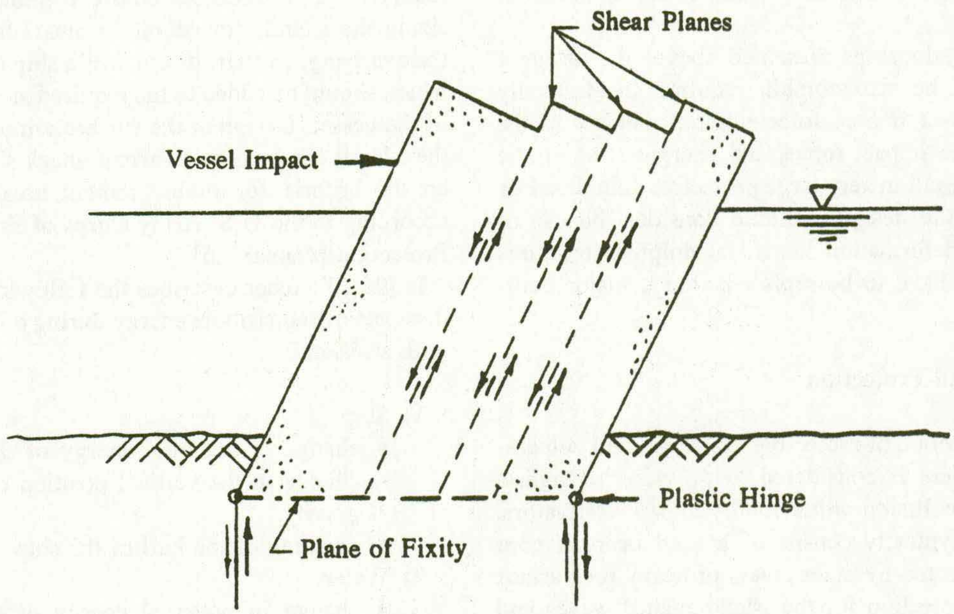


Figure C7.3.3-7. Dolphins Evaluated for use on the Zarate-Brazo Largo Bridge, Argentina [18]. (All Units are Metric)



a. Typical Dolphin Cell.



b. Vertical Shear Failure of Dolphin Cell.

Figure C7.3.3-8. Typical Dolphin Structure Geometry for Elastic Analysis [9].

μ = coefficient of friction (approximately 0.3 for steel-on-steel).

Several different theories exist regarding the failure mechanism of the fill material in the cell [19]. The Vertical Shear Method by Terzaghi [22] and the Horizontal Shear Method by Cummings [23] are the two most widely used. The Terzaghi method as modified by Shroeder [21] will be presented below. The vertical shear on the centerline of the dolphin cell can be determined by,

$$V_s = 3M/2b \qquad \text{(C7.3.3-7)}$$

where

V_s = shear on centerline (lb);
M = resisting moment (ft-lb);

b = equivalent width of a rectangular cell (approximately = 1.7r)(ft).

The maximum resisting moment, M_{max}, (the moment capacity of the cell) can be estimated by,

$$M_{max} = (1/3)\gamma_f bH^2(\tan\phi + \mu) \qquad \text{(C7.3.3-7)}$$

where ϕ = angle of internal friction of the fill material.

Using the relationships presented above, the designer can estimate the size dolphin required to elastically resist the vessel impact force without damage to the cell. For large impact forces and energies, the elastic method will result in very large protective dolphins. For those cases, the designer should consider the use of plastic/large deformation sacrificial dolphin structures which would have to be replaced after a major collision.

C7.3.4 Island Protection

The construction of protective islands around vulnerable bridge piers is considered to provide the highest level of ship collision protection by most investigators. The islands typically consist of a sand or rock core which is protected by outer layers of heavy rock armor to provide protection for the island against waves and currents.

Protective islands have been provided for bridge protection against vessel collision on a number of U.S. and worldwide bridges. Recent projects which have incorporated protective islands include; the Sunshine Skyway Bridge, Florida; the Baytown Bridge across the Houston Ship Channel, Texas; the James River Bridge near Richmond, Virginia; the Laviolette Bridge on the St. Lawrence Seaway, Canada; the Annacis Island Bridge near Vancouver, Canada; and the Orwell Bridge, Great Britain.

Design methodologies for protective islands reported in the literature usually involve a combination of mathematical modeling and scale physical model tests performed for the specific bridge under consideration [24]. The results of the physical model tests are used to calibrate the mathematical model assumptions concerning the interaction between the colliding ship and the island. The island geometry is developed to fulfill the following two conditions:

1) The ship impact force which is transmitted through the island to the bridge pier must not exceed the lateral design capacity of the pier and pier foundation.
2) The island dimensions are such that the ship penetration into the island during a collision will not result in physical contact between the vessel and any part of the bridge pier.

The requirement of Item (2) above is particularly critical for empty or ballasted ships and barges which can slide up on the slopes of an island and travel relatively large distances before coming to a stop. In sizing the island, consideration must also be given to the overhang, or flair, distance of a ship or barge's bow which should be added to the required stopping distance of the vessel. Design of the surface armor protection of the islands for wave and current attack should be based on the criteria for rubble mound breakwater design according to the U.S. Army Corps of Engineers Shore Protection Manual [25].

In [26], Fletcher describes the following items which dissipate or redistribute energy during a vessel collision with an island:

1) Ship
   - change in potential energy of the ship due to change in the vertical position of its center of gravity
   - crushing of the hull of the ship
2) Water
   - change in potential energy of the water displaced by the ship
   - generation of water waves and turbulence
3) Island
   - change in potential energy of island material
   - displacement, shear and compaction of the island material

- friction between the ship and the island
- generation of shock waves within the island
- crushing of particles of island material

The inclusion of these items in a design analysis is difficult since their effects are only partially understood, however, simplifying assumptions and engineering judgment (such as that shown in Figures C7.3.4-1 and C7.3.4-2 are made. Experimental model tests are usually performed as a check of the validity of the mathematical assumptions.

The most extensive tests conducted to date on island protection were performed by the Danish Hydraulic Institute (DHI) in a consortium with Danish investigators for a proposed bridge crossing the Great Belt in Denmark during the late 1970's. The mathematical and physical modeling for the Great Belt is described by Brink-Kjaer, et. al. in [27]. The mathematical modeling was relatively complex since all degrees of freedom of the vessel were allowed as well as the three-dimensional geometry of the island. A computer program solving the numerous simultaneous equations was developed for solution of the collision forces and intrusion into the island. The mathematical model was calibrated and verified from the results of approximately 500 tests using 1:94 and 1:79 scale models of the islands and 250,000 DWT, 150,000 DWT, and 50,000 DWT ship models. Sample results of the testing for the Great Belt Bridge islands are shown in Figures C7.3.4-3, C7.3.4-4, and C7.3.4-5 from [27]. One of the major conclusions of the study was that the island shape should be developed to maximize the deflective characteristics of an impact since the horizontal collision forces between the vessel and the structure decreases rapidly when the vessel is deflected, and that the horizontal force ultimately transferred to the bridge pier is also reduced.

Physical and mathematical models of collisions with islands were also performed during the design of the pier protection system for the new Sunshine Skyway Bridge. A plan and cross section of the Skyway islands is shown in Figures C7.3.4-5 and C7.3.4-6. Located in approximately 30 feet of water, the protection was designed to stop or deflect a 10 knot ship collision from either a loaded 23,000 DWT bulk carrier with a 30 foot draft, a 87,300 DWT bulk carrier partly loaded to a 30 foot draft (61,000 ton displacement), or an empty 87,300 DWT bulk carrier (20,000 ton displacement) which had been trimmed such that the draft at the bow was level with the water surface. The experimental set-up for the testing performed by Hydro Research Science, Inc. [28] is shown in Figure C7.3.4-7. Typical results from the physical model testing is shown in Table C7.3.4-1. As can be seen from these tables, loaded vessels are stopped on the slope of the island whereas empty vessels can slide on top of the island for a considerable distance.

The mathematical model utilized for the Skyway island described by Havno and Knott [24] and was an extension of the DHI Great Belt computer model to include additional research results developed in the early 1980's as well as the Skyway physical modeling. A plot of measured versus computed collision results is shown in Figure C7.3.4-8 where the results are reasonably close. However, since the physical model results were used to calibrate the mathematical model, the comparisons were expected to be close. Figure C7.3.4-9 shows an example of the mathematical simulations performed for the Skyway Bridge [24].

Physical modeling performed for the Orwell Bridge protection islands in England is described by Fletcher, et. al. in [26]. The project consists of 1:100 and 1:50 scale models of the design vessels which were:

1) Loaded 11,000 ton displacement ship with 19.7 foot draft.
2) Vessel (1) ballasted to 9,000 ton displacement with a 16.4 foot draft.
3) A 1,000 ton displacement vessel with 6.6 foot draft.

All collision speeds were equal to 8 knots. Typical results of the testing is shown in Figure C7.3.4-10. The tests indicated that the distance penetrated by a vessel increases as its speed is increased and as the water level relative to the top of the island is increased. It was also found that the shallower draft 1,000 ton vessel penetrated further than the larger deep draft vessels. The final design of the island section is shown in Figure C7.3.4-11 for the Orwell Bridge.

The island protection for the recently completed Annacis Island Bridge is shown in Figure C7.3.4-12. Described by Sexsmith in [29], the design vessel was a 60,000 DWT vessel impacting at 12 knots with a collision energy of 885,200 ft.-kips. By placing the tower piers of the 1,500 foot main span near the river's edge, the designers were able to extend the adjacent river bank out and around the new piers to create the protective embankment. This same approach has also been recently used for the new James River Bridge in Richmond, Virginia and for one of the main piers of the new Dame Point Bridge in Jacksonville, Florida.

The primary means of collision energy absorption with an island system is the deformation and displacement of the island materials. Deformation of the ship's bow will also occur, however, the deformation is expected to be considerably less than would occur with

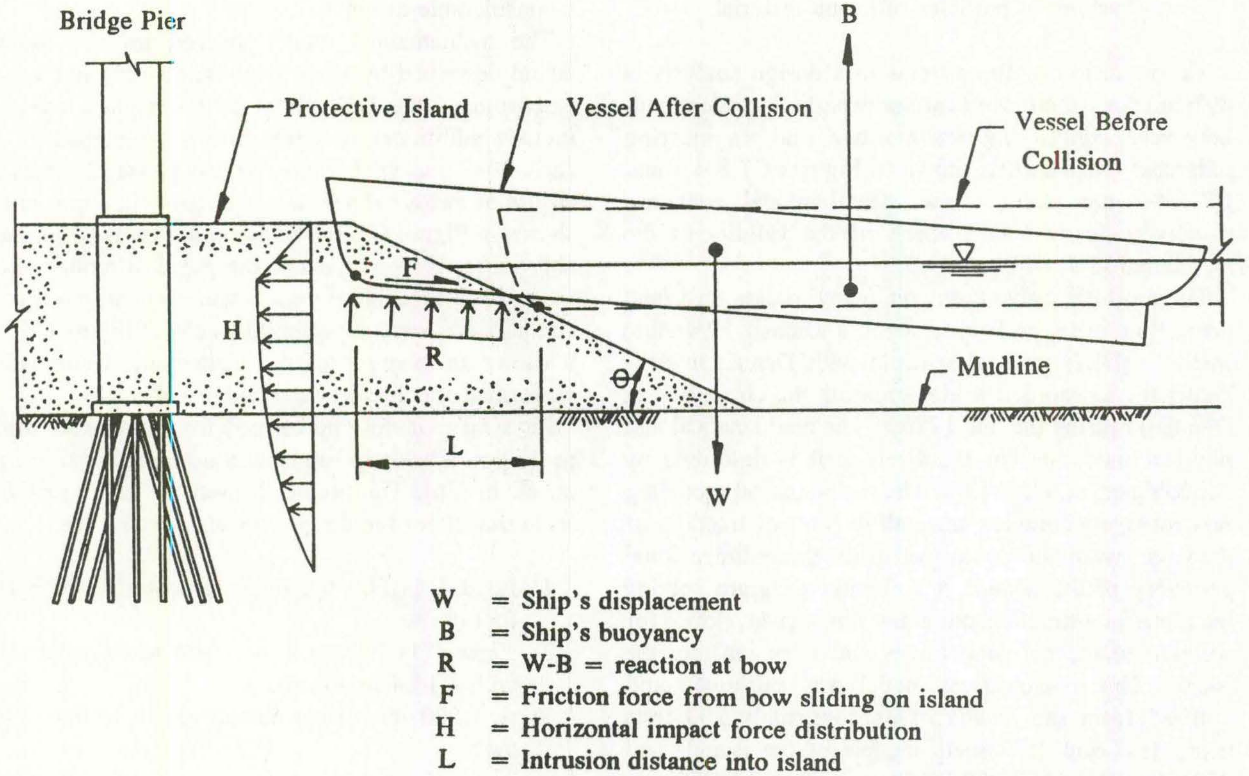


Figure C7.3.4-1. Vertical Force Distribution of Ship Impact Force Through Protective Island [24].

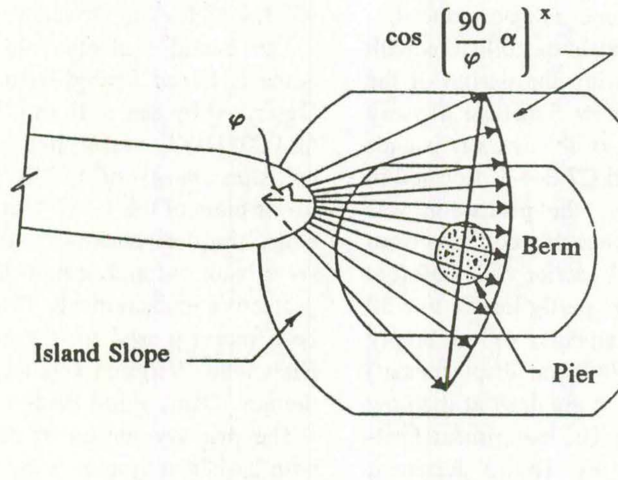


Figure C7.3.4-2. Horizontal Distribution of Ship Impact Force Through Protective Island [24].

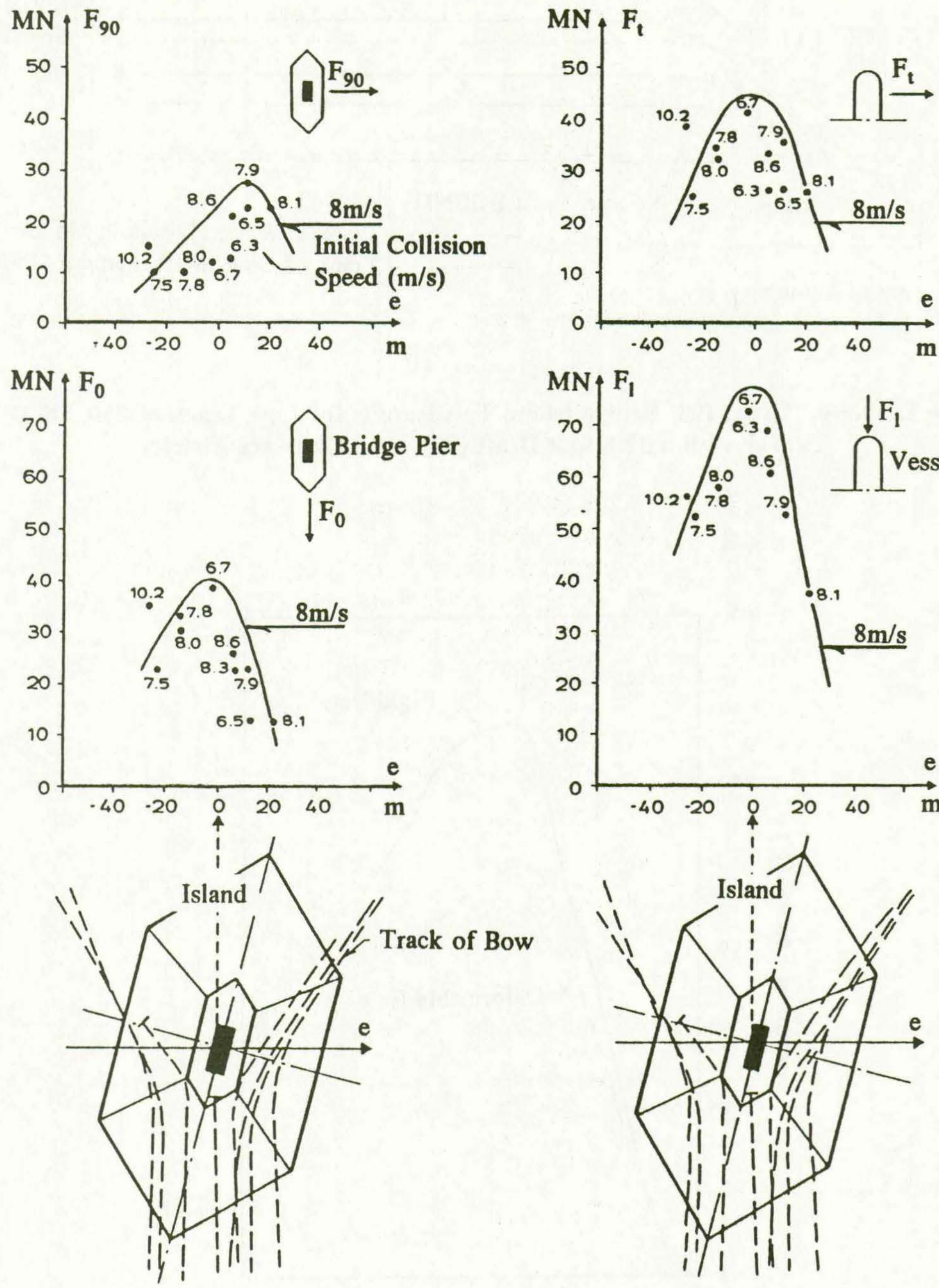


Figure C7.3.4-3. Island Collision Forces on Vessel and Bridge Pier from Impact of 150,000 DWT Tanker with 32.8-foot Draft. Great Belt Bridge Model Results [27]. (All Units are Metric)

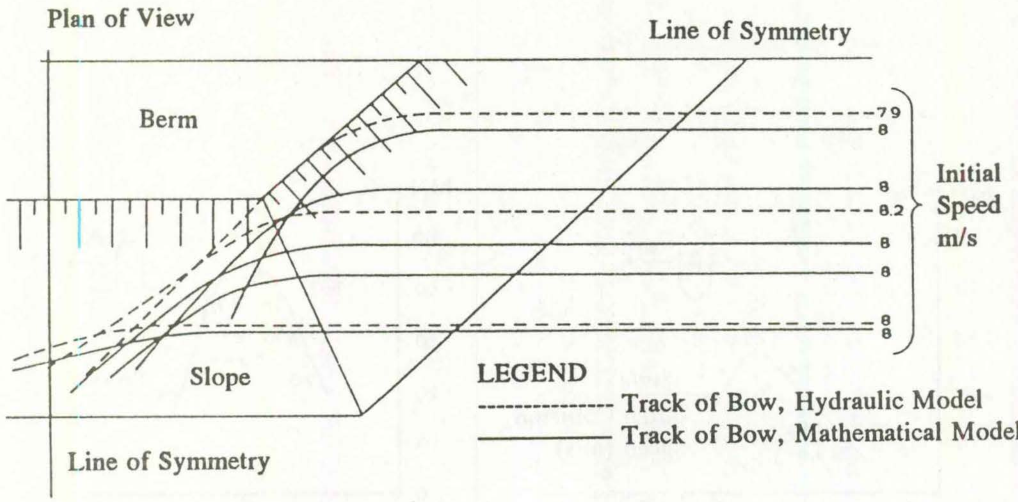


Figure C7.3.4-4. Great Belt Bridge Island Test Results for Bow Track of 250,000 DWT Tanker with a 32.8-foot Draft [27]. (All Units are Metric)

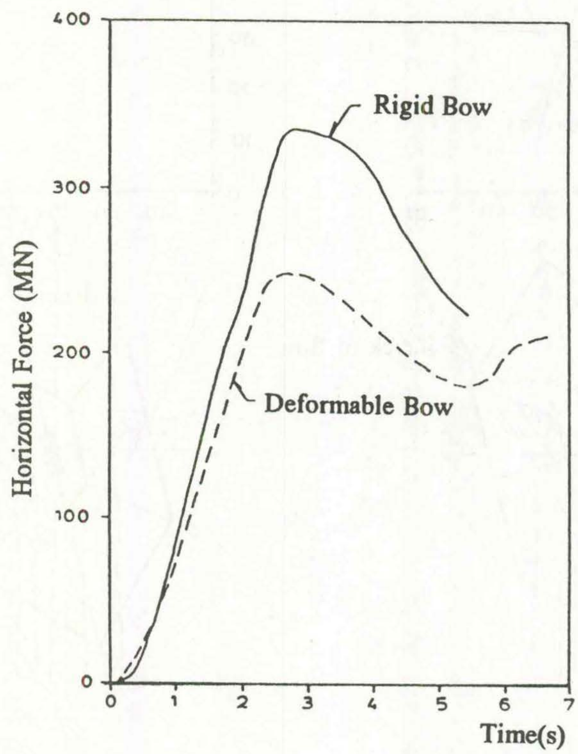


Figure C7.3.4-5. Comparison of Island Horizontal Forces for Rigid and Deformable Bow Models of 250,000 DWT Tanker Head-on Collision. Great Belt Bridge Study [27]. (All Units are Metric)

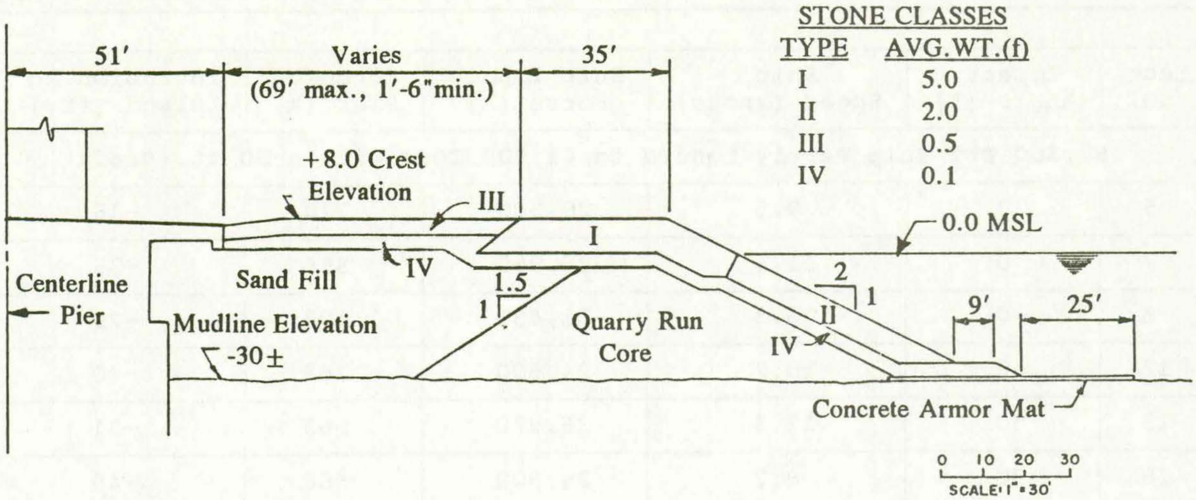


Figure C7.3.4-6. Sunshine Skyway Bridge Protective Island Typical Section [24].

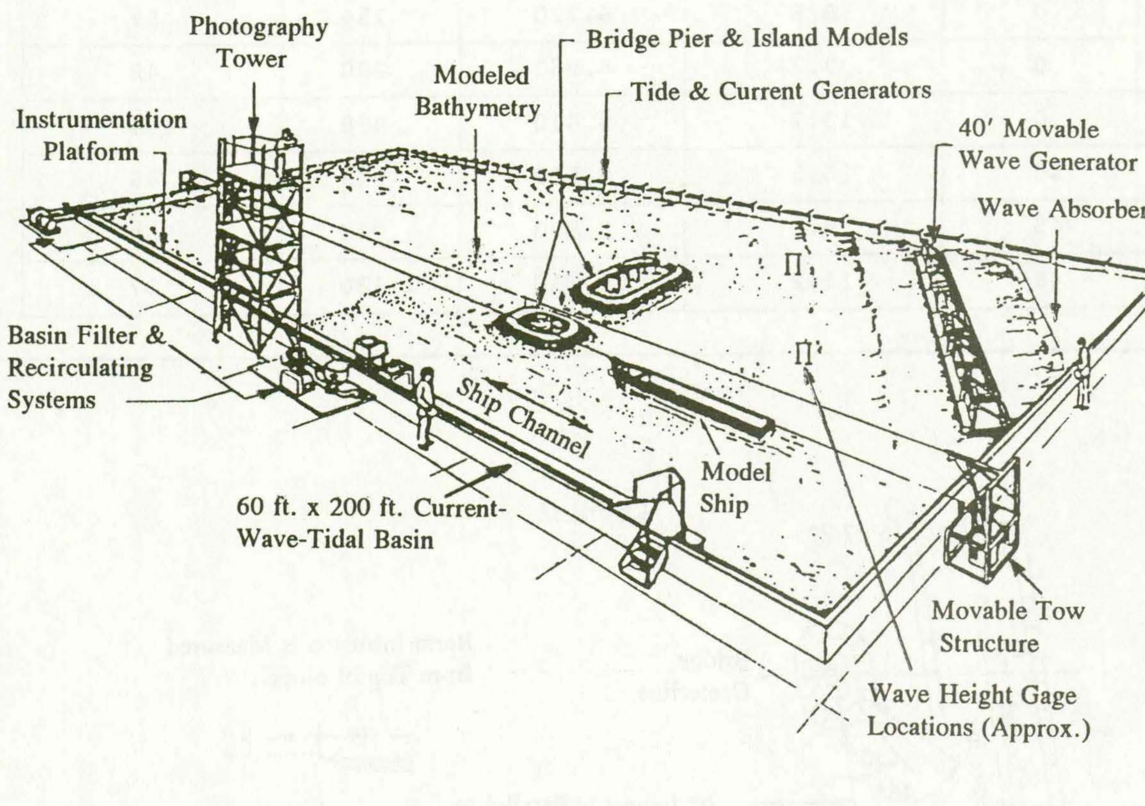
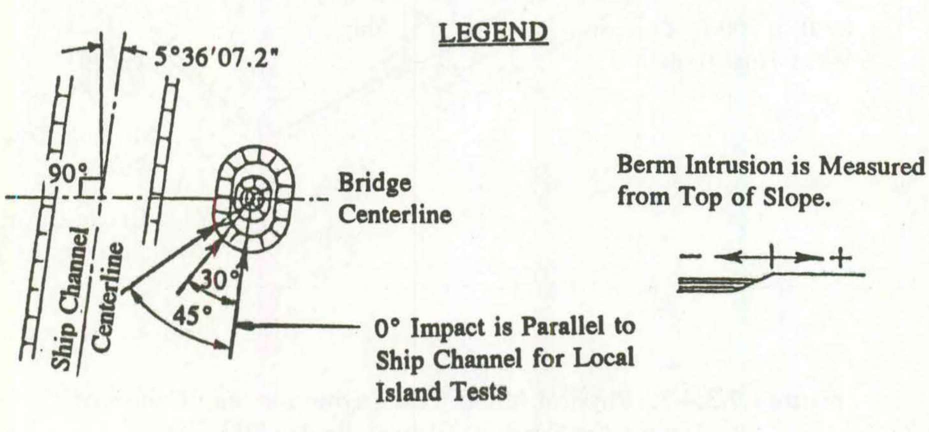


Figure C7.3.4-7. Physical Model Test Layout for Ship Collisions Against the Sunshine Skyway Bridge [28].

Table C7.3.4-1. Sample Results of 1:50 Scale Model Impact on Skyway Bridge Protection Island [28].

Test No.	Impact Angle (°)	Ship Speed (knots)	Ship Impact Force (k)	Force on Pier (k)	Intrusion in Island (ft.)
87,300 DWT Ship Partly Loaded to 61,000 Tons with a 30 ft. Draft					
5	0	9.5	20,500	220	-45
7	0	11.6	25,960	856	-35
8	0	13.3	31,430	1,103	-25
12	0	10.2	24,600	243	-40
13	30	11.3	36,890	863	-31
18	30	8.2	24,600	568	-45
19	45	9.5	38,260	1,229	-40
20	45	12.0	40,990	2,169	-27
21	45	15.9	71,050	3,486	- 5
87,300 DWT Ship Ballasted to 20,000 Tons with a 0.0 ft. Bow Draft					
1	0	8.6	6,720	154	32
2	0	9.2	8,962	290	42
4	0	13.7	9,410	418	104
14	30	10.3	8,510	649	45
15	45	8.6	5,830	717	35
16	45	11.2	10,310	1,130	37



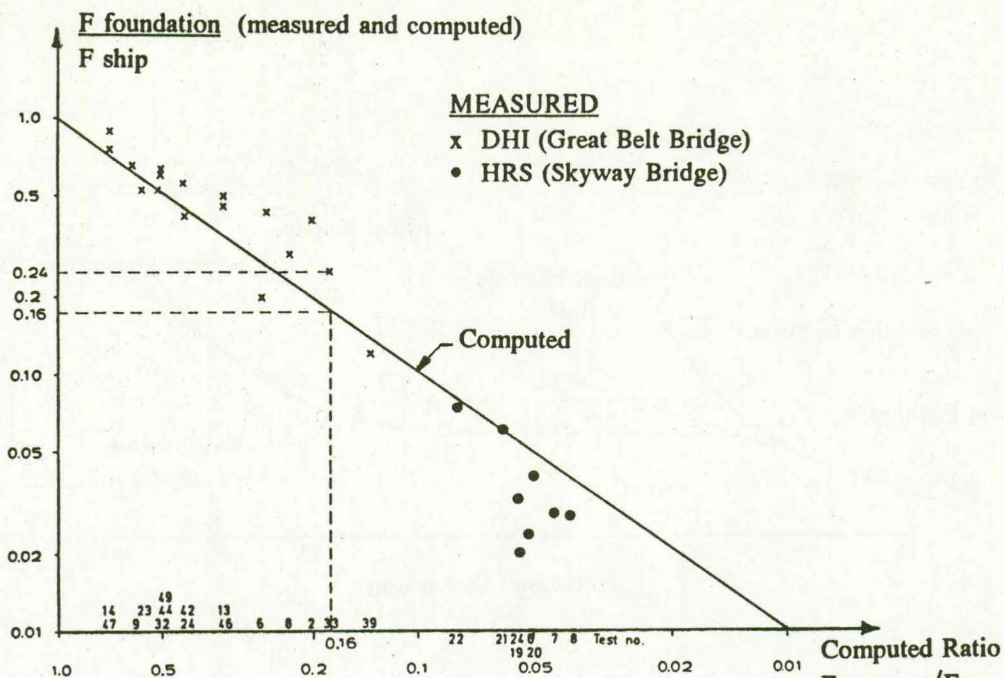


Figure C7.3.4-8. Comparison of Great Belt and Sunshine Skyway Island Collision Test Results [24].

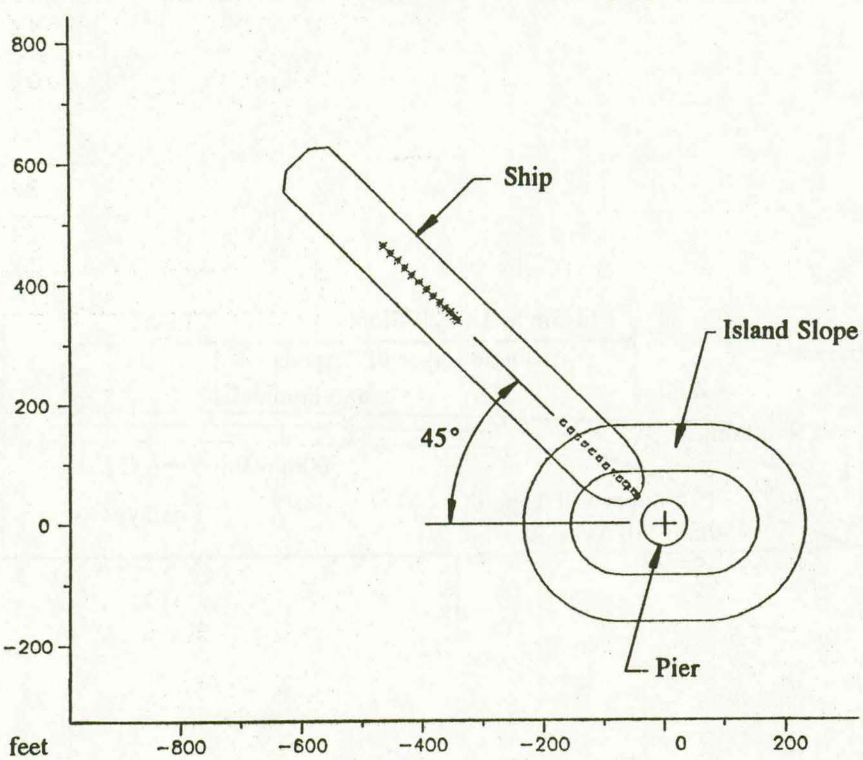


Figure C7.3.4-9. Mathematical Model Result of an Empty, Trimmed, 850,000 DWT Vessel Impacting the Skyway Bridge Island at 10 Knots in Extreme High Water [24].

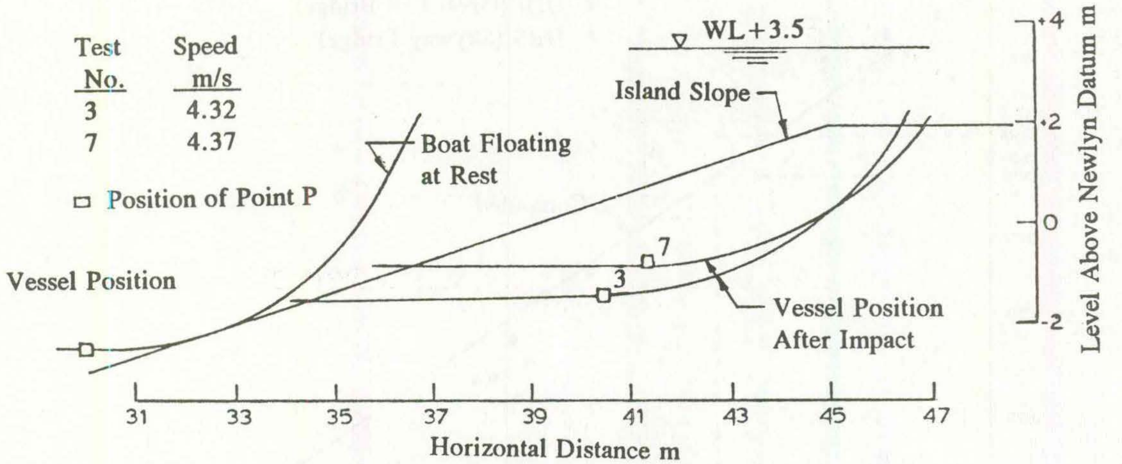


Figure C7.3.4-10. Physical Model Results of 11,000 Ton Vessel Impact with the Orwell Bridge Protective Island [26]. (All Units are Metric)

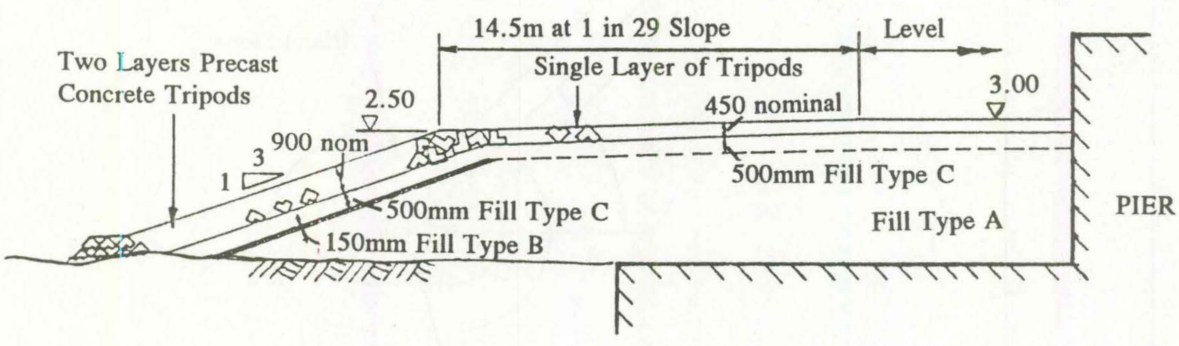
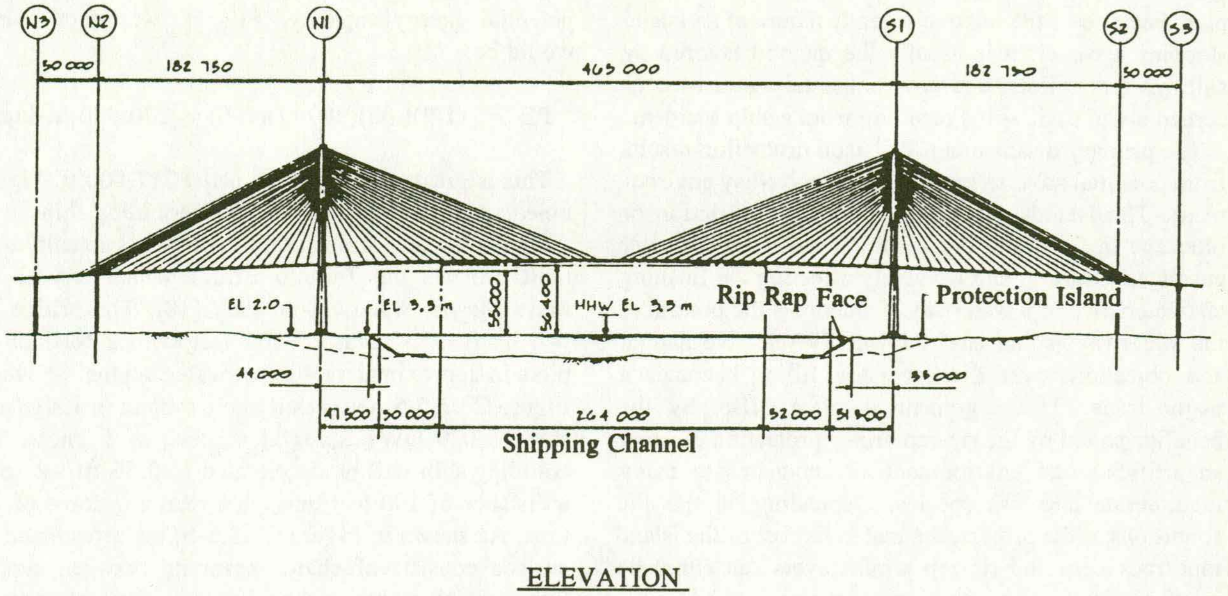


Figure C7.3.4-11. Protective Island Typical Section for the Orwell Bridge, England [26].



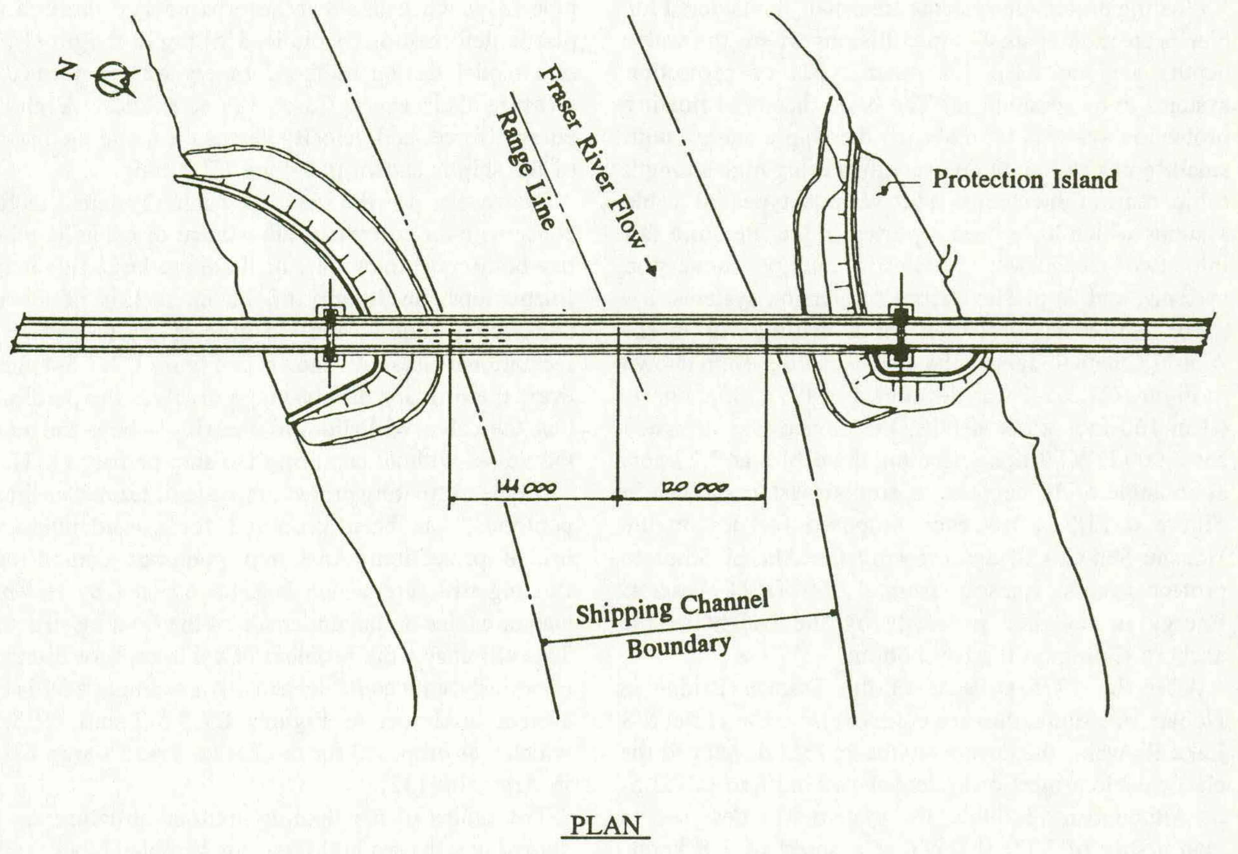


Figure C7.3.4-12. Plan and Elevation of Annacis Island Bridge Protection Island System, Vancouver, Canada [29].

either a dolphin system, or fenders mounted on a rigid pier. Because of the relatively gentle nature of an island stopping a vessel, it is usually the method favored by ship masters, pilots, and environmental agencies concerned about toxic spills resulting from a ship accident.

The primary disadvantage of island protection results from potential adverse impacts to the river/bay environment. The islands can create a serious restriction or blockage in the waterway resulting in increased water currents, scouring, and adversely affecting the flushing characteristics of a waterway. Sometimes the bottom of the waterway is an environmentally sensitive habitat and objections over the necessary filling becomes a major issue. This argument is often offset by the benefits gained by the rip-rap armor protection creating an artificial reef environment advantageous to many invertebrate and fish species. Depending on the site conditions of the project, the materials cost of the island sand/rock core and rip-rap armor layers can either be relatively inexpensive or very expensive.

C7.3.5 Floating Protection Systems

Floating protection systems are usually considered for pier protection against ship collisions where the water depths are too deep for other types of protection systems to be economical. The basic theory of floating protection systems is to absorb the ship's energy with small forces and large deformations using high strength cable tension members. The various types of cable systems which have been reported in the literature fall into two categories: 1) elastic energy conversion systems, and 2) plastic energy conversion systems.

For temporary protection of a drilling rig in the Akashi Channel, Japan, the elastic cable system shown in Figure C7.3.5-1 was developed in 1973 [30]. Located in 160-foot water depths, the device was designed for 2,000 DWT ships impacting the cables at 9.7 knots at an angle of 15 degrees. A similar system, shown in Figure C7.3.5-2, has been proposed for use on the Honshu-Shikoku Bridge crossing the Akashi Strait to protect against impacts from 1,000 DWT vessels. Energy is absorbed primarily by the weight of the anchors sliding on the bay bottom.

After the 1975 collapse of the Tasman Bridge in Hobart, Australia, due to a collision from the vessel S/S Lake Illawara, the design engineers [31] developed the elastic cable protection system shown in Figure C7.3.5-3. Although never built, the system was designed to stop a ship of 35,000 DWT at a speed of 7.8 knots. After a relatively forceless deformation of about 100 feet, the nylon anchor cables can be stretched by roughly 35 percent of their 980-foot length creating a resistance force of 790 kips per cable. The elastic potential energy capacity, PE, of two nylon cables would be:

$$PE = (1/2)(.35)(980)(2)(790) = 270{,}970 \text{ ft.-kips}$$

This is greater than the estimated 217,000 ft.-kips of kinetic energy associated with the colliding ship.

One of the few cable systems which has actually been built protects the Taranto Bridge which crosses the Mare Piccolo waterway in Italy [18]. The bridge has two navigation spans of 500 feet with a total of six piers in approximately 40 foot water depths. Shown in Figure C7.3.5-5, the plastic cable system is designed to stop 15,000 DWT ships at a speed of 6 knots. The colliding ship will be decelerated at 0.66 ft./sec^2 over a distance of 100 feet through a retaining force of 720 kips. As shown in Figure C7.3.5-5, the arrestor on the surface consists of chains spanning between support buoys which in turn are anchored to concrete foundations on the bay bottom using chains. The ship's energy is absorbed by the five lead anchors located on each chain. The 16.4 foot-long dampers consist of a steel pipe in which a drawbar absorbs energy through the plastic deformation of the lead filling in the pipe. Full size model testing of the dampers was performed to measure their energy/force characteristics. A plot of energy, force, and velocity versus the stopping distance of the ship is shown in Figure C7.3.5-6.

Drawbacks to the use of cable systems include concerns over corrosion, adjustment of cable lengths if bay bottom depths scour, or there are large tide height fluctuations, uncertainty of the interaction of the soil and anchor sliding, and blocking of the waterway for recreational uses. As shown in Figure C7.3.5-4, however, the primary disadvantage involves the possibility that the cable will slide off the ship's bow and under the vessel without capturing the ship or barge [31].

A type of floating protection system, termed anchored pontoons, has been proposed for consideration for bridge protection. Anchored pontoons consist of a floating structure which is held in place by fastening anchor cables on the underside of the floating structure. This eliminates the problem of cable capture discussed previously with cable systems. An example of this type system is shown in Figures C7.3.5-7 and C7.3.5-8 which was proposed for the Zarate-Brazo Largo Bridge in Argentina [32].

The cables of the floating pontoon structure are anchored into the sea bottom using weighted blocks, piles, or other types of methods. Similar to cable systems, the impact is absorbed by either elastic or plastic deformation of the cables and anchor system. In addition,

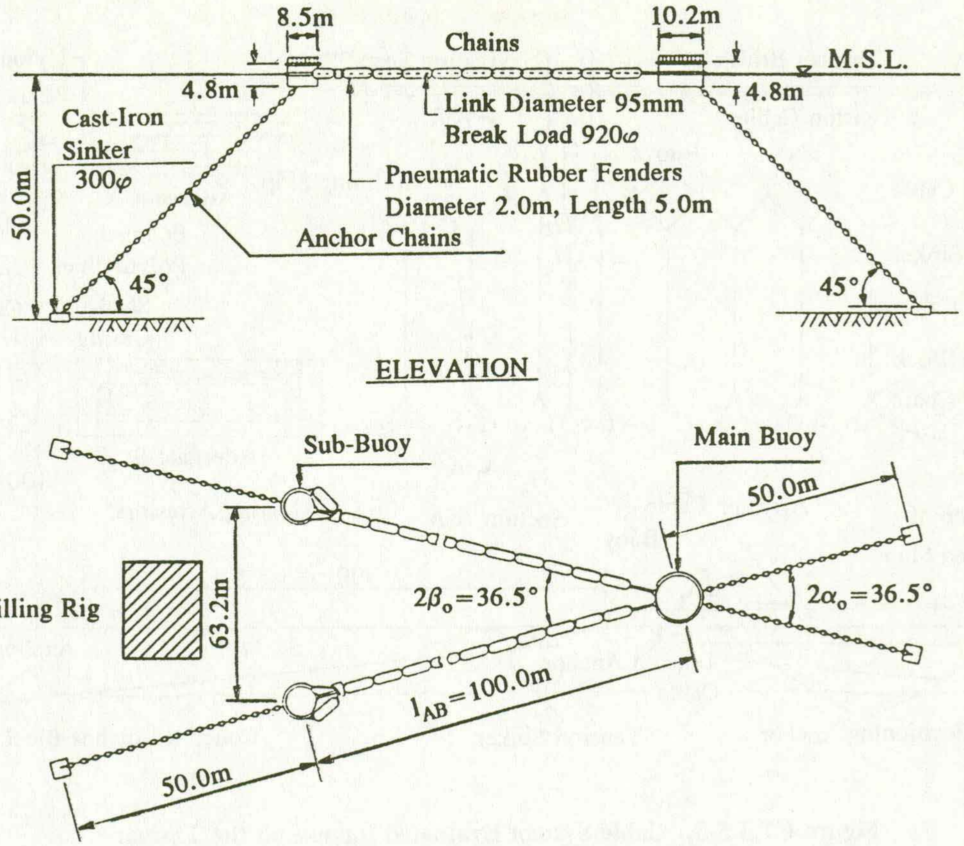


Figure C7.3.5-1. Cable System Protection of Temporary Drilling Rig in the Akashi Channel, Japan [30].

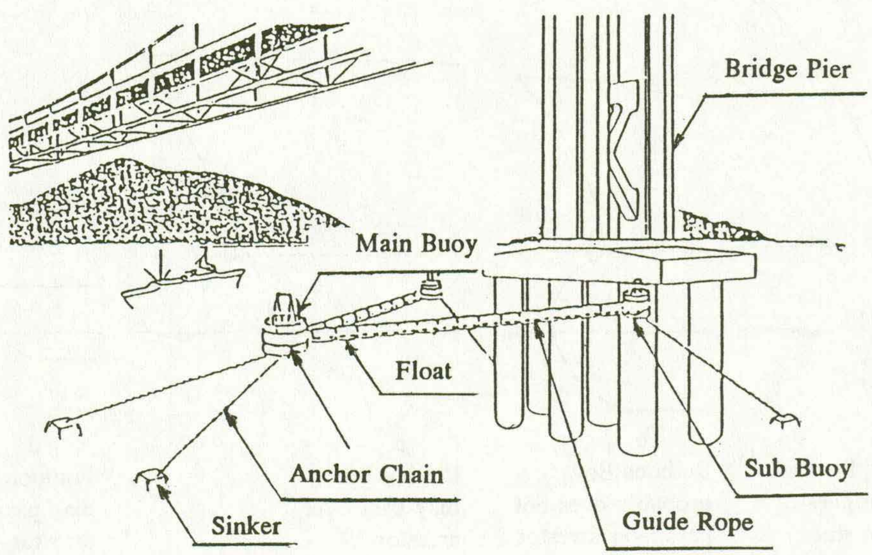


Figure C7.3.5-2. Cable System Protection Proposed for the Honshu-Shikoku Bridge Piers Across the Akashi Straits, Japan.

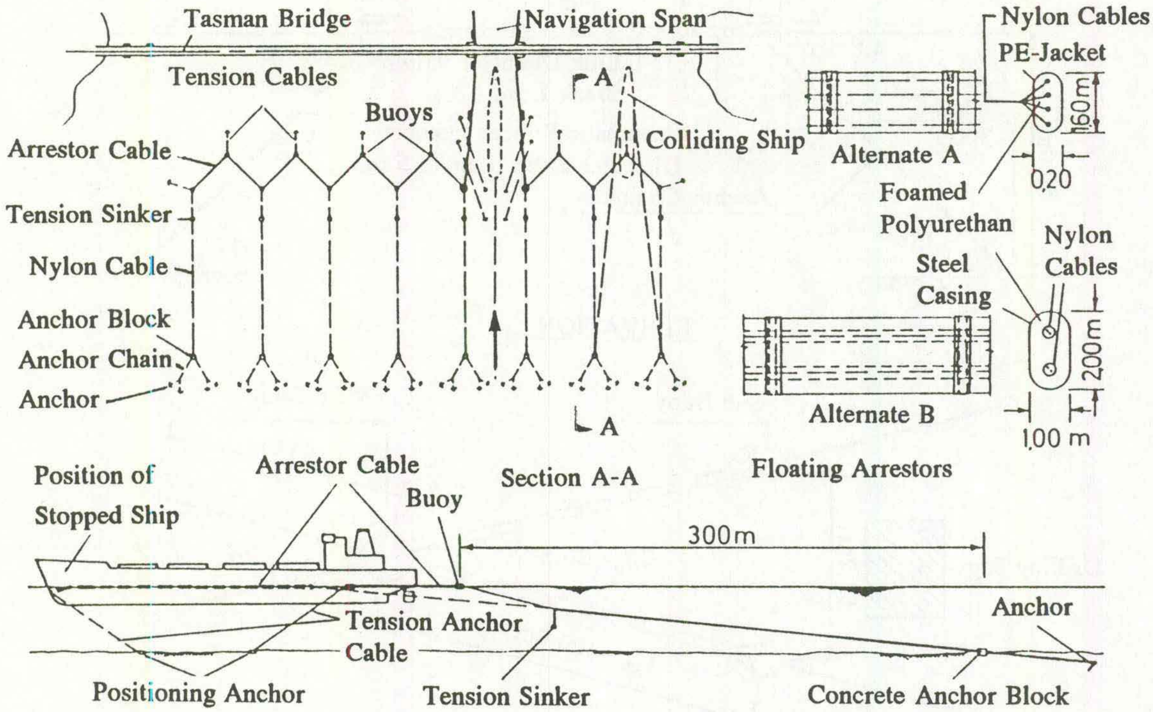


Figure C7.3.5-3. Cable System Evaluated for use on the Tasman Bridge, Australia [31]. (All Units are Metric)

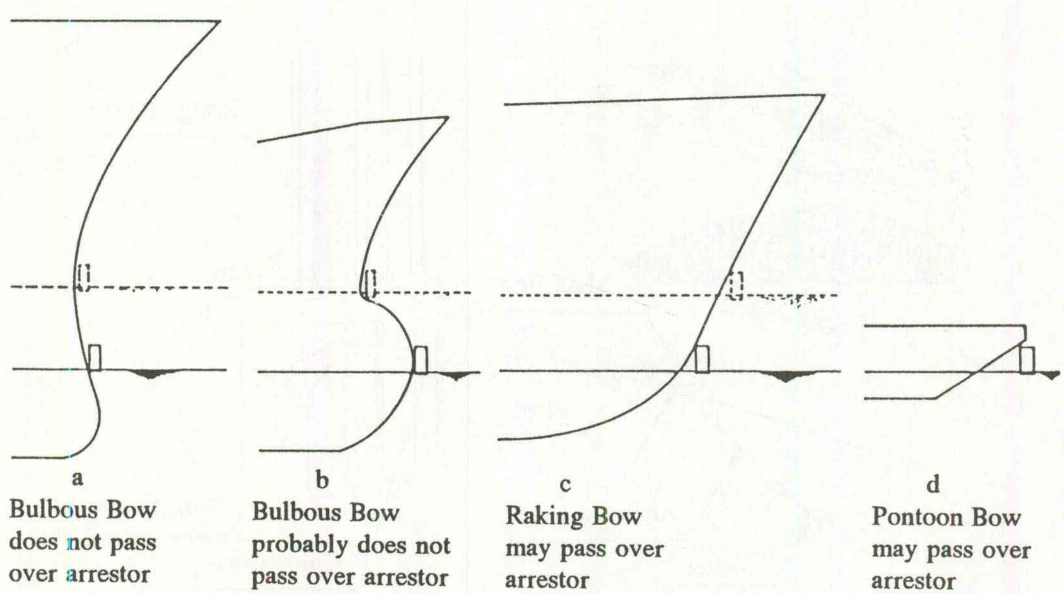


Figure C7.3.5-4. Cable Capture of Vessel Depends Upon the Shape of the Vessel Bow [31].

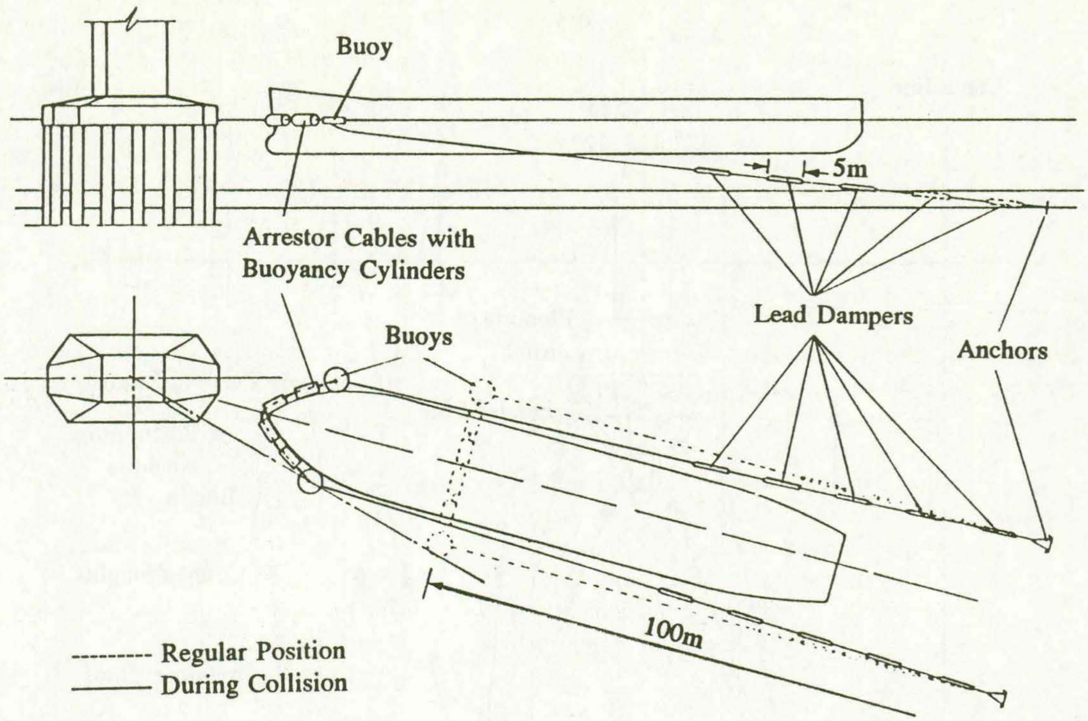


Figure C7.3.5-5. Cable System Protecting Piers of the Taranto Bridge Across the Mare Piccolo, Italy [18]. (All Units are Metric)

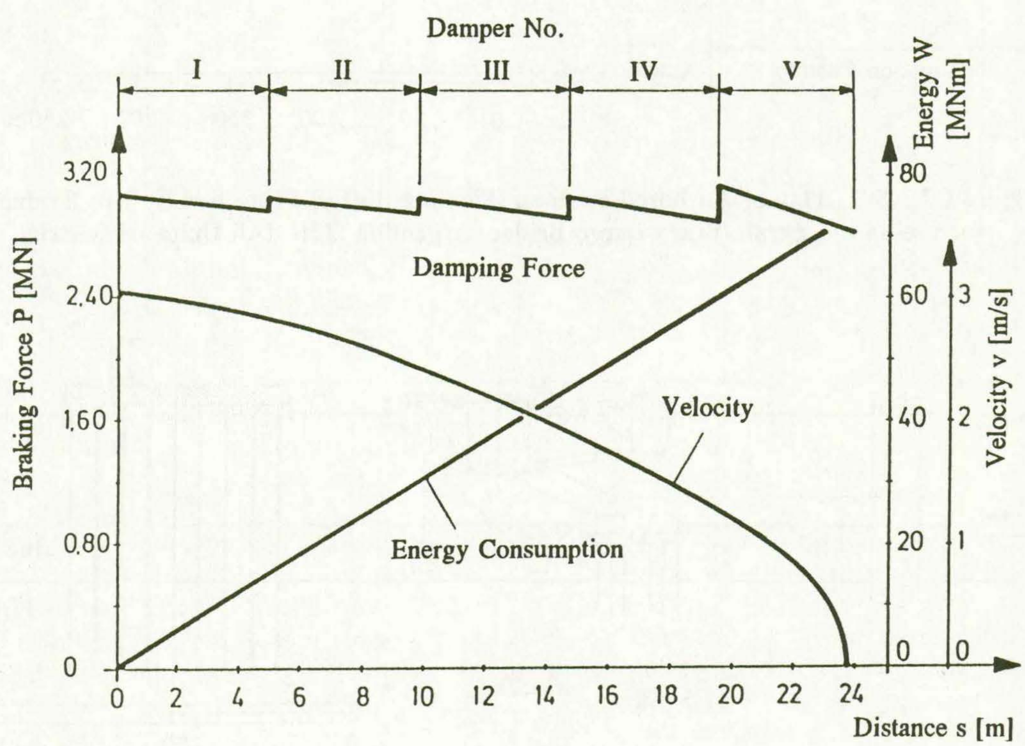


Figure C7.3.5-6. Force, Speed, Energy Relationships of the Taranto Bridge Cable Protection System [18]. (All Units are Metric)

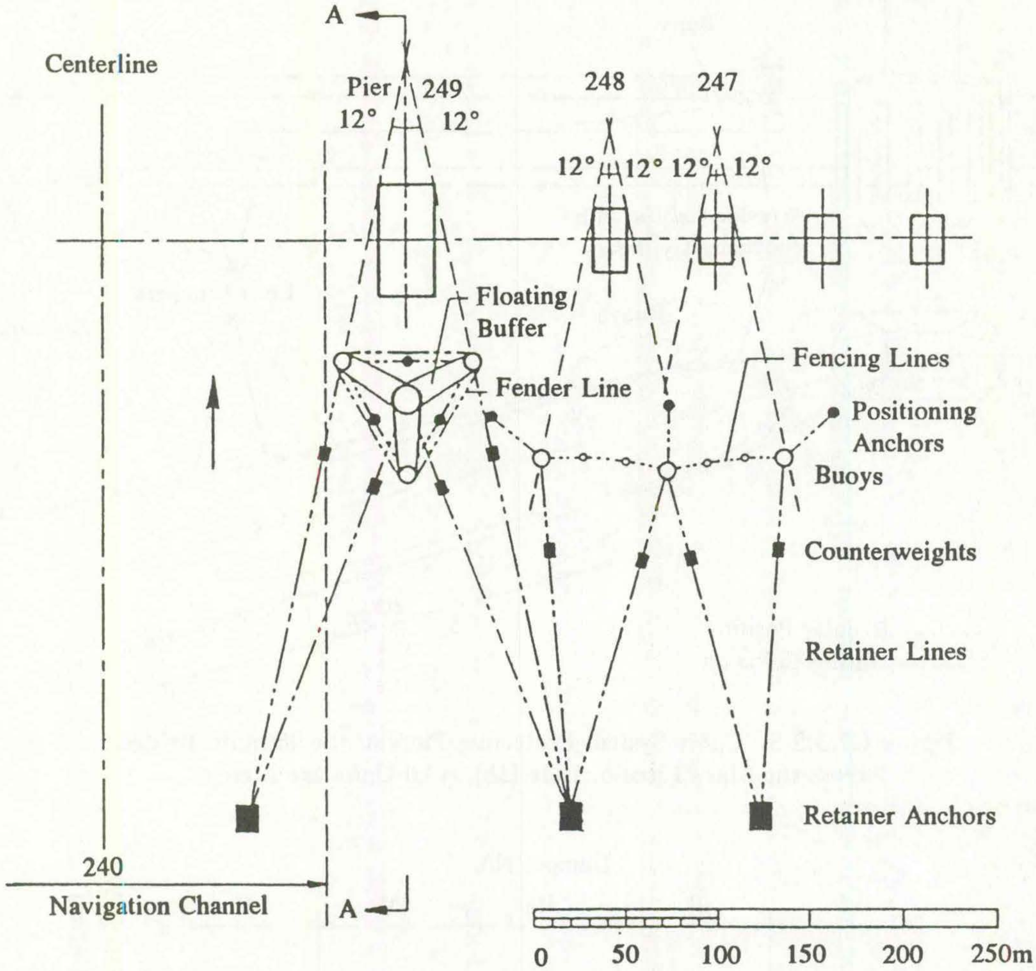


Figure C7.3.5-7. Plan of Anchored Pontoon (Floating Buffer) Protection System Evaluated for use on the Zarate-Brazo Largo Bridge, Argentina [32]. (All Units are Metric)

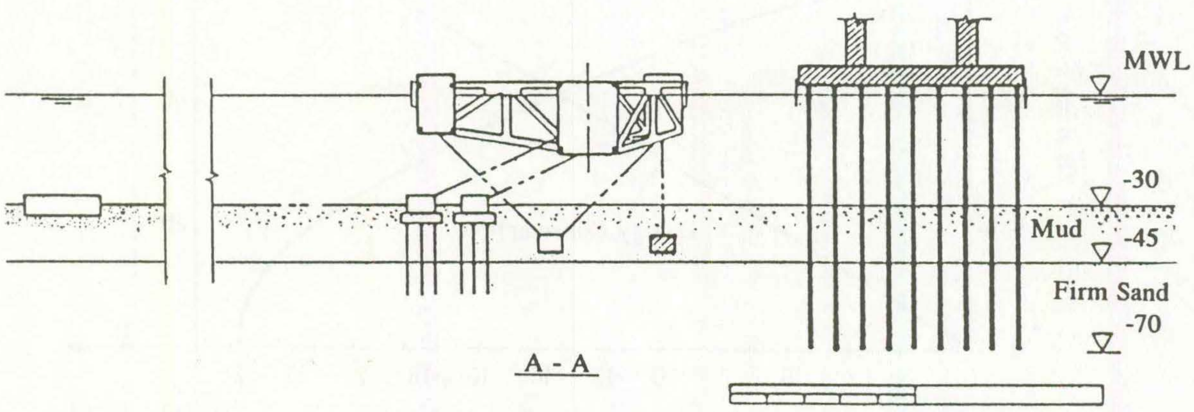


Figure C7.3.5-8. Section of Anchored Pontoon (Floating Buffer) Protection System for the Zarate-Brazo Largo Bridge [32]. (All Units are Metric)

fenders on the floating pontoon can absorb part of the impact energy. A disadvantage of the floating pontoon system is its susceptibility to displacement or damage during severe storms. This would be similar to the anchored concrete pontoons of the Hood Canal Bridge near Seattle, Washington where 13 large segments of the structure sank during a severe storm in 1979. To date there have been no known uses of this system for use as pier protection for bridges.

C7.4 MOVABLE BRIDGE PROTECTION

The special Guide Specification requirements for the protection of movable bridges were developed because of the numerous accidents that have occurred on these bridge structures. Many of the movable bridges in the U.S. were designed and built in the late 1800's and early 1900's when both the frequency and size of vessels using the waterways were very small compared to the ship and barge vessels today. As a result of their relatively narrow horizontal spans, and the increase in size and frequency of vessels in most waterways today, many movable bridges have a relatively high risk of vessel collision. The machinery in most movable bridges is relatively sensitive to impact, vibrations, and deflections in both the substructure and superstructure. As a result, even minor (non-catastrophic) vessel impacts can disrupt the bridge operations causing bridge closure until repairs are made. The requirements of the Guide Specification were developed to give designers specific guidelines in protecting these structures.

C7.5 MOTORIST WARNING SYSTEMS

The greatest loss of life in catastrophic ship/bridge collisions has resulted from the continuation of highway traffic after the span has been severed. Following the investigation of the Sunshine Skyway Bridge collapse, the National Transportation Safety Board [33] recommended that standards be developed for the design, performance, and installation of systems to detect highway bridge span failures and to warn motorists.

The FHWA issued a technical advisory [34] in 1983 describing the investigation and results of the warning systems evaluated for the Sunshine Skyway Bridge by the Florida Department of Transportation. The Guide Specification provides data for the designer to consider in developing a motorist warning system for a bridge structure. Experience in the effectiveness motorist warning systems for vessel collision is limited. One example of an effective warning system is the Tasman Bridge in Hobart, Australia.

The Tasman Bridge collapsed due to a ship collision in 1975. Because the cost of protection for the structure was so expensive, the bridge Authority decided to construct a second bridge crossing upstream of the Tasman to act (essentially) as a backup bridge in the event of a future collision with the Tasman Bride. Although vulnerable to a vessel collision, a motorist warning system was installed to protect the public motorists [35]. The restored bridge, which carries 50,000 vehicles per day, has computer controlled traffic lights on gantries for tidal flow of traffic in peak hours. This system was modified to enable the bridge to be used in a manner similar to a railroad at-grade crossing. In peak road traffic periods, ships are not permitted to navigate the bridge. At all other times the bridge deck is completely cleared of all traffic while a ship passes beneath the bridge. The traffic delay is about 3 minutes.

C7.6 AIDS TO NAVIGATION ALTERNATIVES

Since 60 to 85 percent of all vessel collision accidents are attributed to pilot error, it is important that all aspects of the bridge design, siting, and aids to navigation with respect to the navigation channel be carefully evaluated with the purpose of improving or maintaining safe navigation in the waterway in the vicinity of the structure. The bridge designer is very limited in his ability to require any modifications which affect operations on a navigable waterway since the responsibility and authority for implementing such navigation improvements within U.S. waterways belongs to the U.S. Coast Guard and is protected under Federal Regulations. In some states, the State Government has the responsibility to license and regulate state pilots on merchant vessels, and thru this responsibility can exercise some regulatory privileges affecting navigation within its jurisdiction.

Of the operational alternatives listed in the Guide Specification, the implementation of radio-telephone communication between the ship and bridge operators or toll personnel, is one of the most effective and least expensive alternatives toward improving the safety of bridges. High intensity light beacons, sound devices, and placement of a RACON device on the bridge are alternatives which the bridge owner can implement. RACON devices are typically mounted on the bridge at the centerline of the navigable channel. The RACON sends out a signal which is received by the merchant vessel's radar causing an image to appear on the radar screen identifying the bridge centerline location to the mariner. This allows the vessel's pilot to know the location of the navigable channel under the bridge even in severe weather conditions.

The use of advanced navigation systems for vessels transiting under a bridge structure have shown significant reductions in the probability of aberrancy by pilots under simulator conditions [36]. These include both advanced shorebased VTS systems with real-time surveillance capabilities, as well as small portable navigation units carried on board by the master or pilot of the vessel.

An example of the later is described by Knott [16] where, as part of the Skyway Bridge pier protection project, the Florida Department of Transportation conducted a feasibility study for a portable, lightweight, differential LORAN-C receiver to be carried on board vessels in Tampa Bay by the local harbor pilots. The units would provide digital information to the pilot regarding his vessel's position, and also a visual display on an electronic map. A shorebased receiver would also monitor the vessel's position through the pilot's unit and would initiate motorist warning devices on the bridge to stop traffic if the vessel went out of the navigation channel. Although feasible on an engineering and technology basis, the system has not been implemented due to unresolved legal issues concerning liability.

A totally shore based remote sensing system has been proposed by Greneker et. al. [37] for use on bridges to stop motorist traffic on a bridge structure if the vessel is on a collision path with the bridge. Greneker proposed his system for the Sidney Lanier Bridge near Brunswick, Georgia, following the bridge's collapse and loss of life due to a vessel collision in 1972.

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COMMENTARY

SECTION 8 - BRIDGE PROTECTION PLANNING GUIDELINES

The planning data for new bridges in Section 8 of the Guide Specification 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 Specification 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 [1].

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 Section 8.5.1 was developed primarily from studies performed by Shoji and Iwai [2], and Shoji and Wakao [3]. Figure C8-1 shows the relation between ship length, LOA, and main span length, S, for actual ship/bridge accident data. From Figure C8-1 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 C8-2. From Figure C8-2, it can be seen that the probability of ship collision with the bridge is increased when the main span is less than 2 or 3 times the ship length. The bridge accident data included in Figure C8-2 is shown in Table C8-1 [3]. when the main span is less than 2 or 3 times the ship length. The bridge accident data included in Figure C8-2 is shown in Table C8-1 [3].

Table C8-1. Main Span vs. LOA for Historical Bridge Collisions [3].

Date of Accident	Bridge Name	Location	Main Span(ft)	LOA (ft)
1963	Sorsund	Norway	328	354
1972	Sidney Lanier	USA	246	571
1975	Fraser	Canada	384	656
1977	Benjamin Harrison	USA	236	613
1977	Tromso	Norway	262	134
1979	Second Narrows RR	Canada	498	574
1980	Almo (Tjorn)	Sweden	912	564
1980	Sunshine Skyway	USA	860	610
1981	Jordfallet	Sweden	144	157

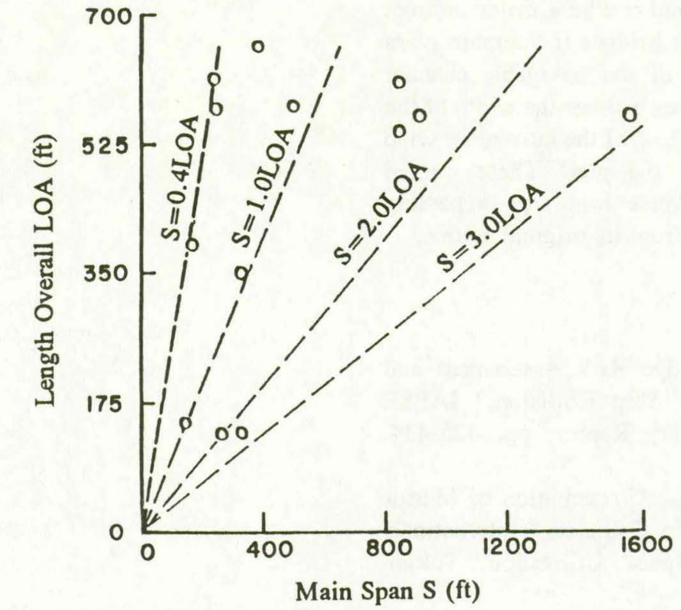

Figure C8-1. Colliding Ship's LOA Versus Main Span of Bridge(s) [3].

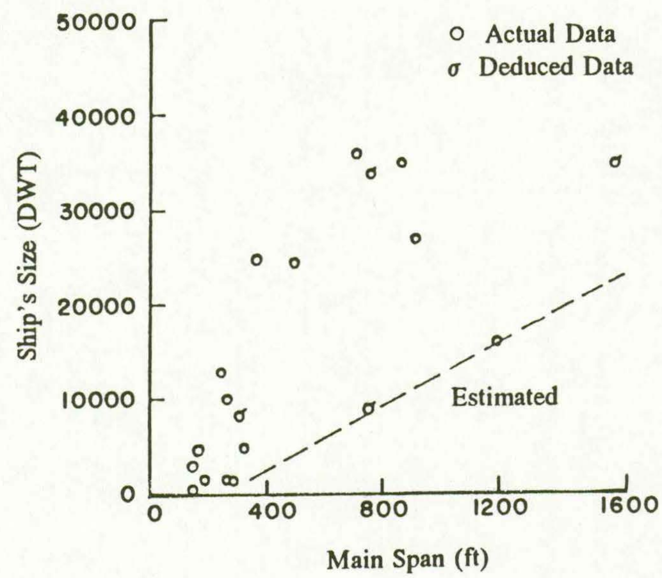

Figure C8-2. Colliding Ship's Size (DWT) Versus Main Span of Bridge(s) [2].

Research reported in [2] 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 2 or 3 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.

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APPENDICES A & B

WORKED EXAMPLES USING THE SPECIFICATION

The two examples presented in Appendices A and B are intended to illustrate the application of the Specification. The two examples are prefixed by the letter "A" or "B" respectively. Each section of the Specification that applies is identified by a corresponding number in the examples for easy reference and identification. Comments are also included to assist in interpretation of the Specification and to clarify assumptions made relative to the analysis.

The state-of-the-art in vessel collision design of bridges has not yet progressed to the point where exact solutions are available. The number of significant figures used in the following examples should not be interpreted as an exact theoretical answer or infer that the same number of significant figures be used in design. They are used to avoid confusion in the use of the Guide Specification.

The two examples provided in the Appendices are not actual case histories, and any resemblance to proposed or existing bridges is purely coincidental. The circumstances and data provided for each example are purely fictional and are provided for illustration only.

APPENDIX A

METHOD I WORKED EXAMPLE

A1 EXAMPLE BRIDGE DESCRIPTION

A high level bridge crossing is proposed across the Atlantic Intracoastal Waterway. The new bridge will be a fixed span structure and will replace an existing movable bridge. The existing movable bridge is over 60 years old and needs replacement. The movable bridge main span and guide pile/fender system have been hit by barges several times in the last 10 years, once causing closure of the bridge for 6 months for repairs to the main span and machinery.

Using Method I, establish the vessel impact design criteria for the piers of the new bridge.

A3.3 IMPORTANCE CLASSIFICATION

The proposed bridge connects one of the Atlantic coast barrier islands to the mainland. The bridge is the only transportation linkage to the mainland for a community of 20,000 people who live on the island. Because it is the only transportation link to mainland facilities such as fire protection, police, hospitals, etc., the bridge importance classification should be classified as Critical.

A4.2 WATERWAY CHARACTERISTICS

The navigable channel in the waterway is 90 feet wide. Although the actual water depth in the channel is about 20 feet, the limiting water depth for merchant vessels is 9 feet due to upstream and downstream shoaling and sedimentation in other parts of the waterway system. Figure A4.2-1 depicts the waterway and channel layout. Figure A4.2-2 depicts the water depths in the waterway.

A4.4 VESSEL CHARACTERISTICS

The primary merchant vessel usage of the waterway is barge tows. From the most recent edition of the "Waterborne Commerce of the United States," the Non-Self Propelled (i.e., Barge) data in Table A4.4-1 was developed. In addition to the barge tows, approximately 1,000 fishing trawler and 12,000 recreational vessel passages occur each year at the bridge location.

Table A4.4-1. Annual Barge Frequency Data for the Waterway.

Draft (ft)	No. of Passages
9	1,400
8	150
7	50
6 and less	1,800
Total	3,400

From Table A4.4-1 it can be seen that about one-half of the barges in the waterway are transiting fully loaded at the limiting channel draft of 9.0 feet. The barges in the "6 and less" category are probably those same barges making the return trip in the waterway while transiting empty with a draft of approximately 2.0 feet.

Discussions with the U.S. Coast Guard, the bridge tender at the existing movable bridge, and vessel operators using the waterway indicate that the barge traffic typically consists of a 1,700 horsepower towboat pushing two 195x35-foot hopper barges as shown in Figure A4.4-1.

A4.2.1 VESSEL TRANSIT PATH

Discussions with the bridge tender and waterway operators indicates that all barge tows transit thru the bridge on the centerline of the navigable channel. No meeting of oncoming barge tows occurs in the vicinity of the existing movable bridge. The centerline of the navigable channel will, therefore, be the origin of the vessel impact speed distribution.

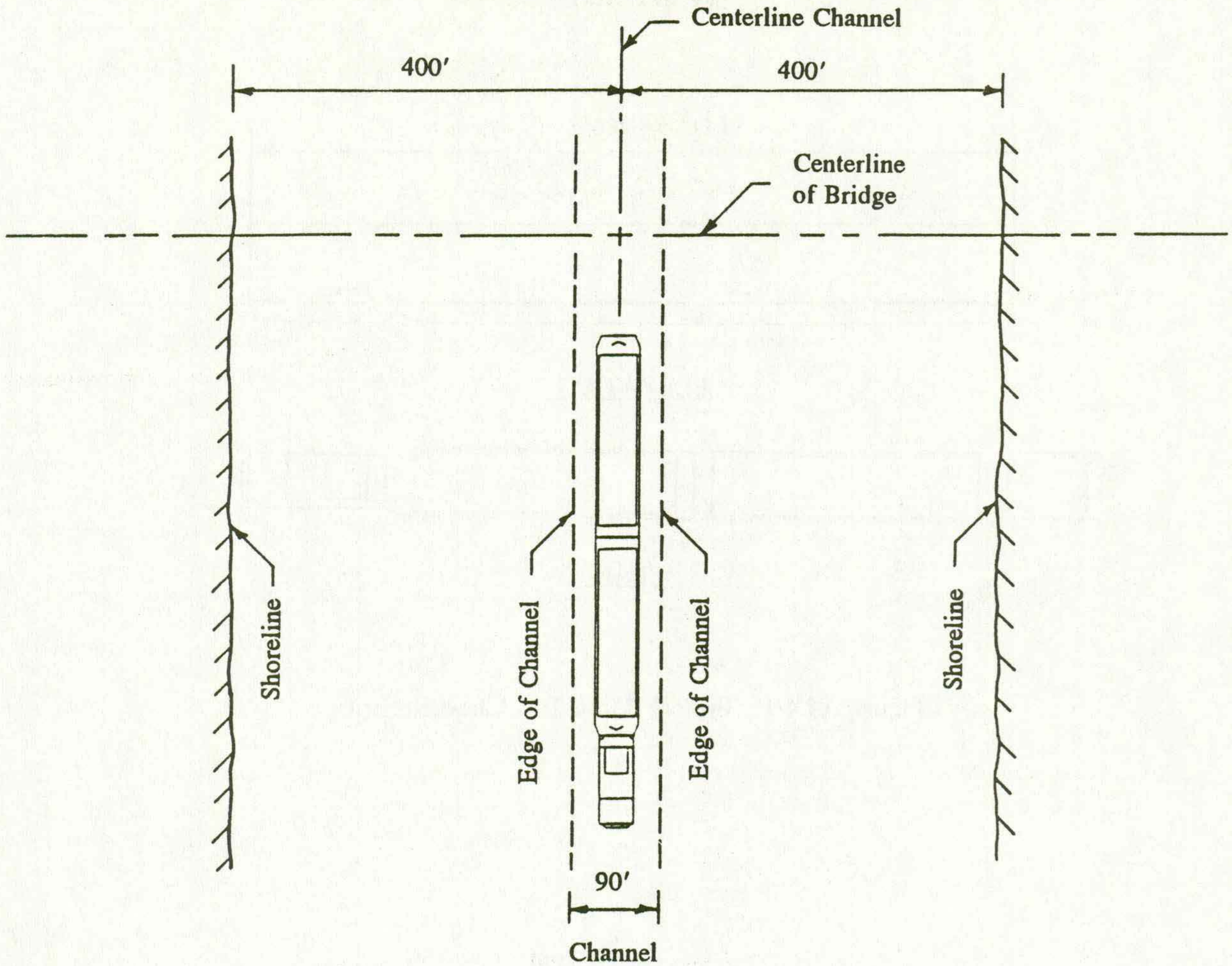

Figure A4.2-1. Plan of Waterway/Channel/Bridge Geometry.

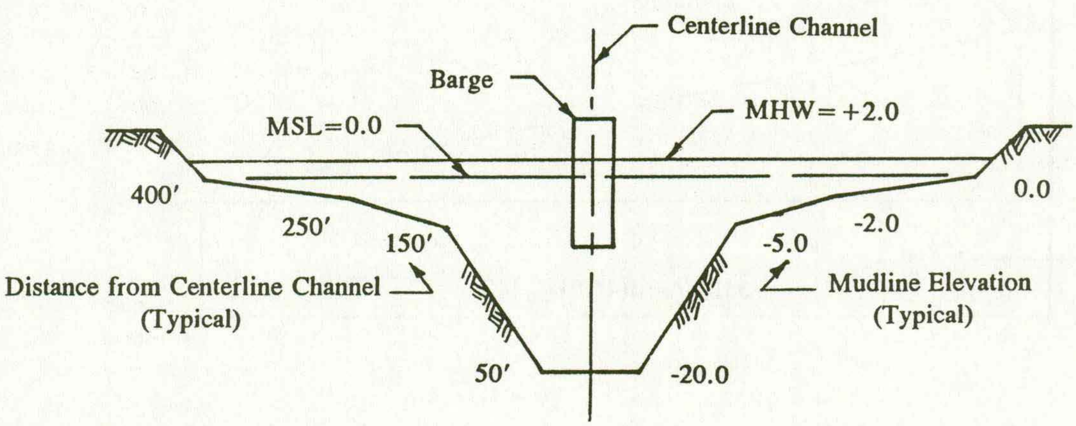

Figure A4.2-2. Profile of Water Depths in Waterway.

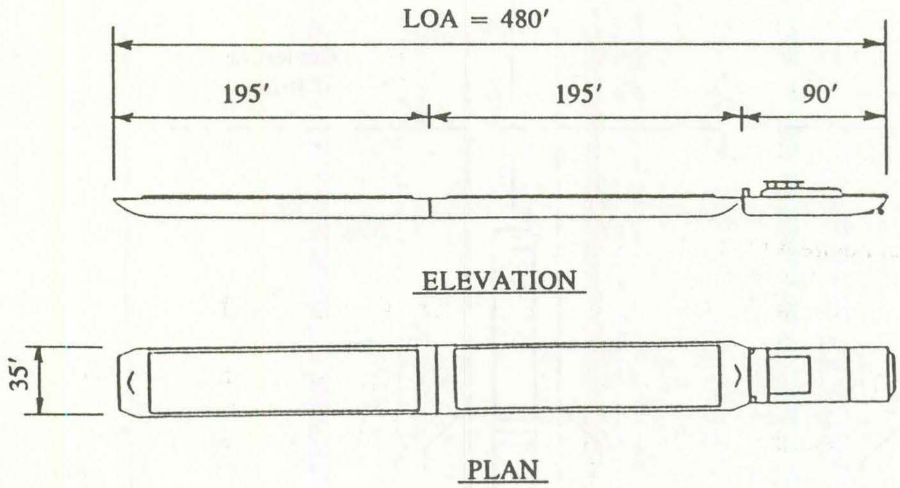

Figure A4.4-1. Typical Barge Tow Characteristics.

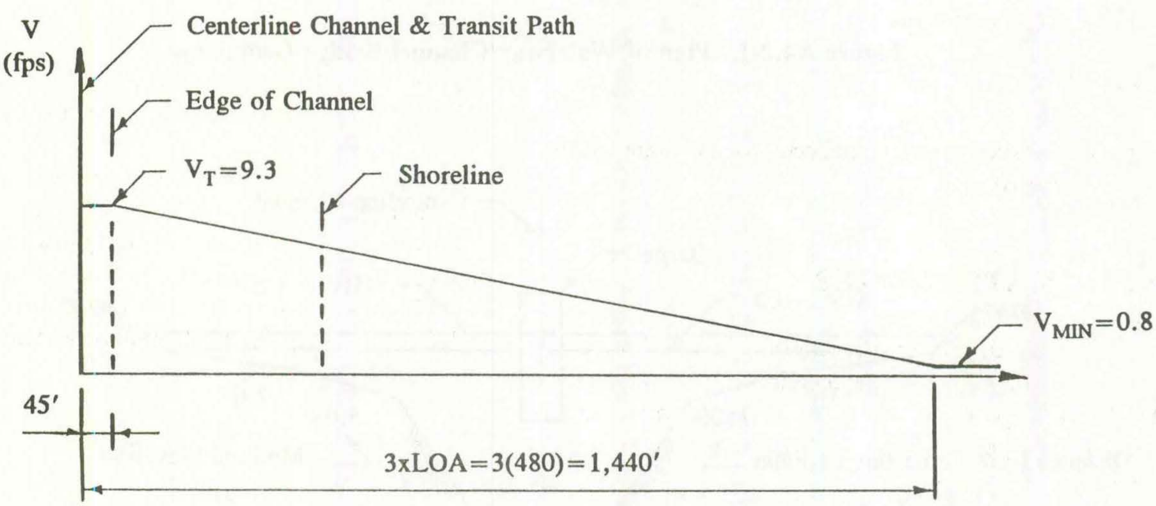

Figure A3.7-1. Design Impact Speed Distribution.

A3.7 DESIGN IMPACT SPEED

Discussions with the barge operators using the waterway indicates that the typical barge tow speed in this section of the intracoastal waterway varies from 4.5 to 6.0 knots. Based on this, the typical vessel transit speed in the channel, V_T, will be set equal to 5.5 knots (9.3 fps).

As shown in Figure A4.4-1, the LOA of the typical barge tow is equal to 480 feet. The design impact speed as a function of the distance from the centerline of the channel, x, is shown in Figure A3.7-1 using the criteria of Section 3.7 and Figure 3.7-1. The minimum design impact speed, V_{MIN}, occurs at the location 3xLOA (3x480 ft = 1,440 ft) from the centerline of the vessel transit path, and is equal to the yearly mean water current of 0.5 knots (0.8 fps). The impact velocity as a function of the distance from the centerline of vessel transit path (i.e., the centerline of the channel in this example) can be computed as follows:

for $x < 45$;

$$V = V_T = 9.3 \text{ fps} \quad \text{(A3.7-1a)}$$

for 45 ft $\leq x \leq$ 1,440 ft;

$$V = 9.3 - \left[(x-45) \left[\frac{9.3-0.8}{1440-45} \right] \right] \quad \text{(A3.7-1b)}$$

for $x >$ 1,440 ft;

$$V = V_{MIN} = 0.8 \text{ fps}$$

where

x = distance from channel centerline and the vessel transit path (ft).

A4.2.2 WATER DEPTHS

The water depths of the waterway are shown in Figure A4.2-2 for the bridge location. The water depths for barge impact are computed from the mean high water (MHW) elevation of +2.0, to the elevation of the mudline.

A4.7.2 DESIGN VESSEL

For a critical bridge, the design vessel will equal the maximum of 50 vessel passages per year, or five percent of the total number of merchant vessels per year, whichever is less. From Table A4.4-1, N=3,400 passages per year, therefore:

$$(.05)N = .05(3{,}400) = 170 \text{ passages}$$

Since this is greater than the maximum allowable of 50 vessels, the 5 percent criteria does not control. Counting back 50 passages from the maximum vessel size in Table A4.4-1 results in the typical barge tow with a 9-foot draft as shown in Figure A4.4-1.

A3.8 VESSEL COLLISION ENERGY

The impact energy shall be computed using Equation 3.8-1. The hydrodynamic mass coefficient for all vessels will be assumed equal to an average value of C_H=1.10. The impact velocity will be computed from Equation A3.7-1.

The displacement of a single 195x35-foot jumbo hopper barge from Figure 3.5.1-1 is 1900 tons (1720 tonnes) loaded, and 200 tons (180 tonnes) empty. The loaded and empty drafts associated with these two displacements are 8.7 and 1.7 feet respectively. For barges partially loaded between these two drafts, the displacement, W_B, can be estimated as;

$$W_B = 180 + (D_E - 1.7) \left[\frac{1720-180}{7} \right] \quad \text{(A3.8-1)}$$

where

D_E = partially loaded draft (ft);
W_B = partially loaded displacement (tonnes).

As a conservative assumption, it will be assumed that at water depths between 8.7 and 1.7 feet, the barge (and towboat) will be transiting partly loaded at a draft equal to the available water depth. The displacement of the towboat is estimated equal to 230 tonnes. The total displacement, W, of the towboat plus the two barges in the tow becomes,

$$W = 230 + 2(W_B) \cdot \text{(tonnes)} \quad \text{(A3.8-2)}$$

Using the above equations, the kinetic impact energy, KE, as a function of available water depth and distance from the centerline of the channel can be computed. As an example, at a distance x = 150 feet, and a design water depth of 7.0 feet, KE becomes:

$$W_B = 180 + (7.0 - 1.7)\left[\frac{1720-180}{7}\right] \quad \text{(A3.8-1)}$$
$$= 1{,}346 \text{ tonnes}$$

$$W = 230 + 2(1{,}346) = 2{,}922 \text{ tonnes} \quad \text{(A3.8-2)}$$

$$V = 9.3 - \left[(150-45)\left[\frac{9.3-0.8}{1440-45}\right]\right] \quad \text{(A3.7-1b)}$$
$$= 8.7 \text{ fps}$$

$$KE = (1.1)(2{,}922)(8.7)^2/29.2$$
$$= 8{,}332 \text{ k-ft} \quad \text{(3.8-1)}$$

A summary of the KE for other distances along the proposed bridge centerline are shown in Table A3.12-1.

A3.13 BARGE DAMAGE DEPTH

In order to compute the barge impact force, the barge damage depth must be computed using Equation 3.13-1. The design barge is 35 feet in width, therefore, R_B=1.0. For the same kinetic energy computed in A3.8, the following bow damage depth can be computed:

for x = 150 ft & KE = 8,332 k-ft

$$a_B = \left[\left(1 + \frac{8332}{5672}\right)^{1/2} - 1\right](10.2) \quad \text{(3.13-1)}$$
$$= 5.8 \text{ feet}$$

A summary of a_B for other distances along the proposed bridge centerline are shown in Table A3.12-1.

A3.12 BARGE COLLISION FORCE

For the proposed bridge, the piers will be designed to withstand the design impact force of the barge. The design impact force, P_B, shall be computed using Equation 3.12.1-1. For the same barge damage depth computed in A3.13, the following impact force can be computed:

for x = 150 ft & a_B = 5.8 ft,

$$P_B = [1349 + 110(5.8)] \quad \text{(3.12.1-1b)}$$
$$= 1{,}987 \text{ kips}$$

Using the same process illustrated above, the design impact forces shown in Table A3.12-1 were computed for the proposed bridge location. The designer would develop several trial combinations of pier locations and span lengths using these collision impact forces in the substructure design of the new bridge. A cost estimate of the alternative bridge types developed would indicate the least cost solution for a bridge at this location.

It should be noted that a minimum navigation span length of twice the channel width (2x90 = 180 feet), would be recommended using the guidelines in Section 8.5.1 unless special circumstances indicate otherwise.

Table A3.12-1. Barge Impact Force Distribution Summary.

x (ft)	Water Depth(ft)	W_B (tonnes)	W (tonnes)	V (fps)	KE (ft-kips)	a_B (ft)	P_B (kips)
0	22.0	1,720	3,670	9.3	11,958	7.8	2,207
45	22.0	1,720	3,670	9.3	11,958	7.9	2,207
75	18.3	1,720	3,670	9.1	11,449	7.9	2,174
100	14.5	1,720	3,670	9.0	11,199	7.5	2,163
125	10.8	1,720	3,670	8.8	10,706	7.1	2,130
150	7.0	1,346	2,922	8.7	8,332	5.8	1,987
175	6.3	1,192	2,614	8.5	7,115	5.1	1,910
200	5.5	1,016	2,262	8.4	6,157	4.5	1,844
225	4.8	862	1,954	8.2	4,950	3.8	1,767
250	4.0	686	1,602	8.1	3,960	3.1	1,690
300	3.3	532	1,294	7.7	2,890	2.3	1,602
350	2.8	422	1,074	7.4	2,216	1.8	1,547
400	2.0	180	590	7.1	1,120	1.0	1,459

APPENDIX B

METHOD II WORKED EXAMPLE

B1 EXAMPLE BRIDGE DESCRIPTION

The fictional suspension bridge shown in Figure B1-1 shall be used to illustrate some of the computations of the Method II risk analysis procedure. For the existing bridge, the following two items will be evaluated:

1) What is the annual frequency of collapse, AF, of one of the main piers of the suspension bridge; and
2) If AF is unacceptable, what level of impact force should be used to develop pier protection alternatives.

B4.2 WATERWAY CHARACTERISTICS

The following waterway characteristics shall be assumed for the sample analysis:

- navigable channel is 800 feet wide
- two-way traffic for merchant vessels exist, therefore, the centerline of vessel transit is located 200 feet on each side of the channel centerline
- water depth at the main piers is greater than 65 feet
- annual mean water current is approximately 0.3 knots (i.e. V_{MIN} = 0.5 fps)

B4.3 BRIDGE CHARACTERISTICS

The following main pier characteristics shall be assumed for the sample analysis:

- pier width is 60 feet
- pier resistance to impact equals 20,000 kips
- bridge importance classification = Critical

B4.8.3.1 Vessel Frequency

The bridge is to be evaluated for a 50-year period of time. The number of vessels using the waterway is increasing annually. The forecasted vessel traffic data for the merchant fleet 50 years in the future is shown in Table B4.8.3.1-1. Using the 50-year forecast for the number of vessel transits will result in a conservative (i.e., over estimation) of the current risk.

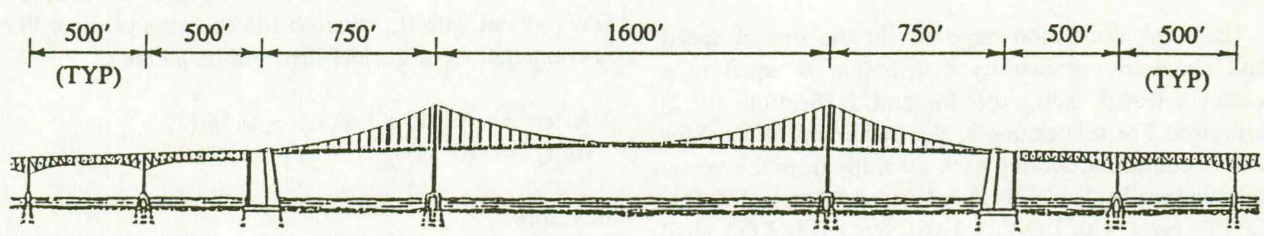

Figure B1-1. Bridge Profile for Method II Example.

Table B4.8.3.1-1. Ship Frequency Data (Yr. 2040)

Size(DWT)	Type*	No. of Passages N (per year)
10,000	F/C	3,000
20,000	F/C	2,000
40,000	B/T	100
60,000	B/T	100
80,000	B/T	300
100,000	B/T	100
150,000	B/T	60
TOTAL		5,660

* F/C = Freighter/Container Vessels
B/T = Bulk Carrier/Tanker Vessels

B4.4 VESSEL CHARACTERISTICS

The vessel characteristics shown in Table B4.4-1 were estimated for each ship classification using the data in Section 3.5.2.

Table B4.4-1. Ship Characteristics

Size(DWT)	Type	Length, LOA (ft)	Beam, B_M (ft)	Draft, D_L (ft)
10,000	F/C	472	64	26.9
20,000	F/C	643	91	34.4
40,000	B/T	682	99	37.4
60,000	B/T	771	109	40.4
80,000	B/T	850	120	43.3
100,000	B/T	902	138	52.8
150,000	B/T	1,030	146	59.1

The LOA dimension required for the impact speed and geometric probability distribution is equal to a vessel selected using the Method I (Section 4.7.2) criterion. For this example, the maximum of 50 ships would control. Counting back 50 ships from the vessel distribution in Table B4.8.3.1-1 results in a 150,000 DWT vessel with LOA = 1,030 feet. This LOA shall be considered a constant and applicable to all vessel classifications in accordance with Sections 3.7 and 4.8.3.3.

B3.7 DESIGN IMPACT SPEED

Discussion with the local pilot's association who are responsible for navigating all foreign flagged ships into the harbor, indicates that the typical transit speed, V_T, for ships in the portion of the waterway where the bridge is located is 13 knots (22 fps). From Figure B3.7-1 the design impact speed, V, for the main pier was computed as,

$$V = 22.0 - \left[(800-400)\left[\frac{22.0-0.5}{3090-400}\right]\right]$$
$$= 18.8 \text{ fps}$$

B4.8.3.2 PROBABILITY OF ABERRANCY

The probability of vessel aberrancy, PA, is estimated using Equation 4.8.3.2-1 as shown below:

$BR = 0.6\text{x}10^{-4}$, for ships
$R_B = 1.0$, since bridge is in a Straight Region
$R_C = 1+(0.3/10) = 1.03$, for $V_C = 0.3$ knots
$R_{xc} = 1.0$ (no cross-currents at site)
$R_D = 1.6$, for High Density vessel traffic

therefore

$$PA = BR(R_B)(R_C)(R_{xc})(R_D) \quad (4.8.3.2\text{-}1)$$
$$= (.6\text{x}10^{-4})(1.0)(1.03)(1.0)(1.6)$$
$$\cong 1.0\text{x}10^{-4}$$

B4.8.3.3 GEOMETRIC PROBABILITY

The geometric probability of ship collision, PG, is computed using the normal distribution shown in Figure B4.8.3.3-1. The standard deviation of the normal distribution is σ = LOA = 1,030 feet. For the 150,000 DWT vessel with B_M equal to 146 feet, the value of PG is computed from normal distribution tables as,

At $x_1 = (0.41)\sigma$; $1\text{-}F(x_1) = 0.3409$
At $x_2 = (0.75)\sigma$; $1\text{-}F(x_2) = 0.2266$

therefore

$$PG = [1\text{-}F(x_1)]\text{-}[1\text{-}F(x_2)]$$
$$= (0.3409\text{-}0.2266) = 0.1143$$

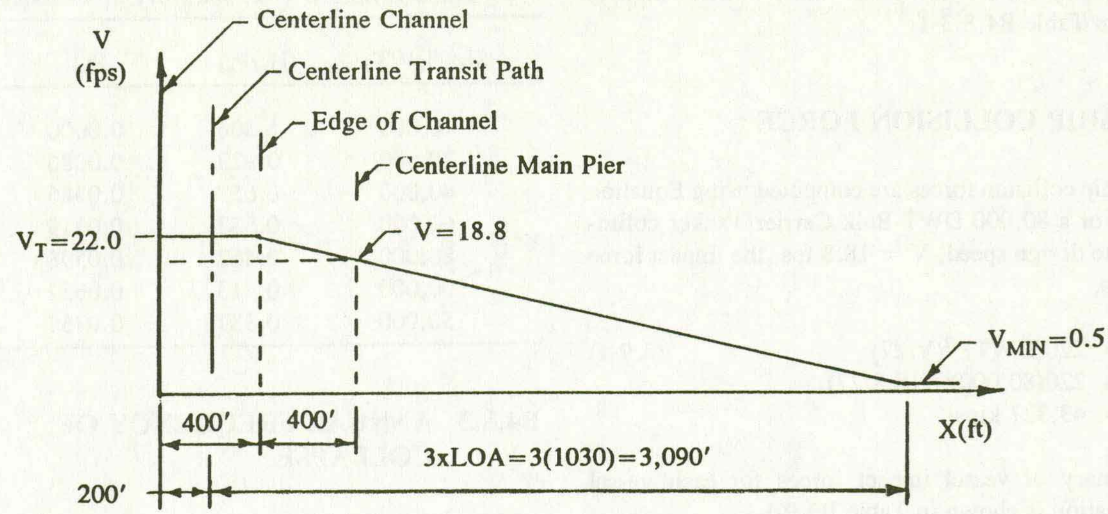

Figure B3.7-1. Main Pier Design Impact Speed.

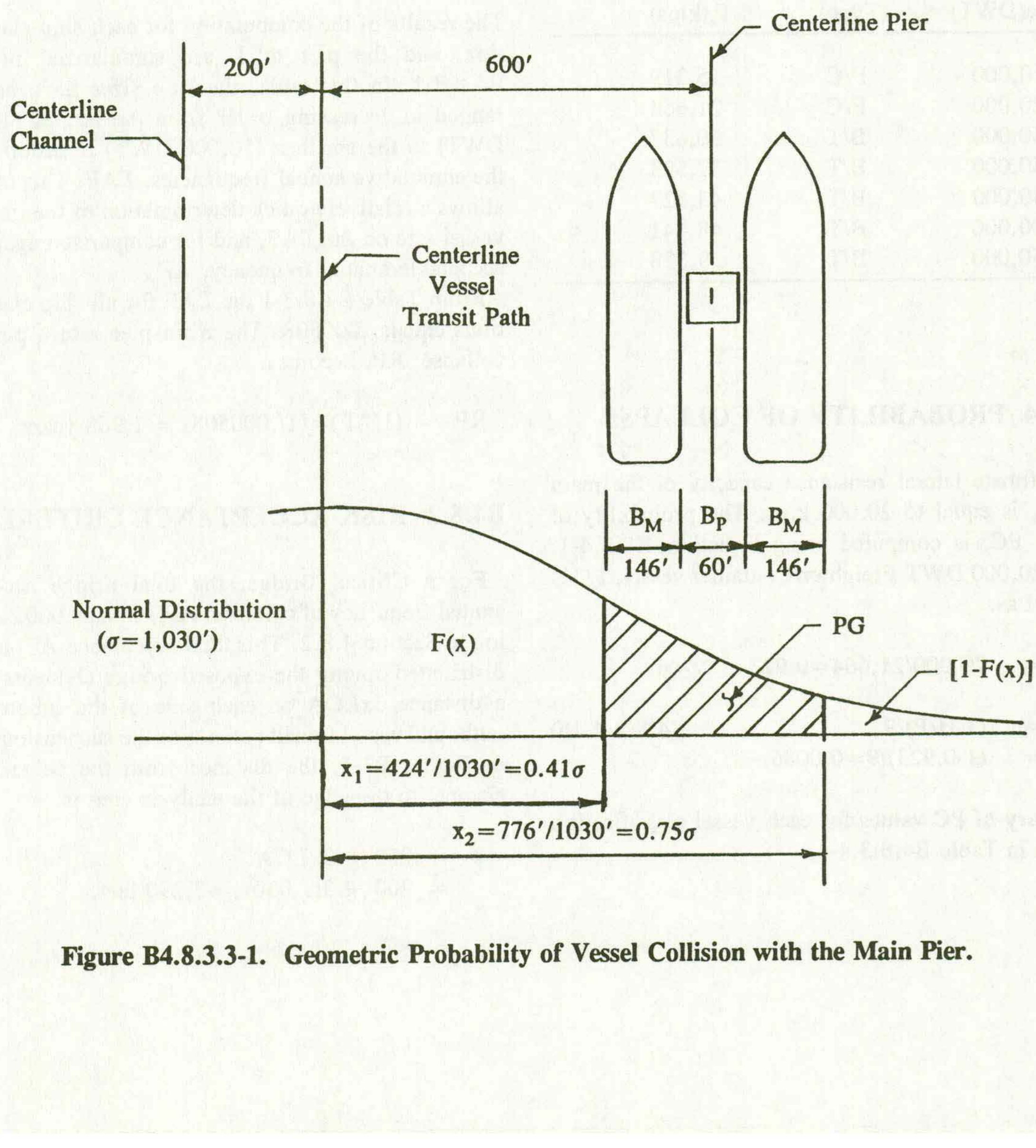

Figure B4.8.3.3-1. Geometric Probability of Vessel Collision with the Main Pier.

As summary of PG for each vessel classification is shown in Table B4.8.3-1.

B3.9 SHIP COLLISION FORCE

The ship collision forces are computed using Equation 3.9-1. For a 80,000 DWT Bulk Carrier/Tanker colliding at the design speed, V = 18.8 fps, the impact force becomes,

$$P_S = 220(DWT)^{1/2}(V/27) \quad (3.9\text{-}1)$$
$$= 220(80{,}000)^{1/2}(18.8/27)$$
$$= 43{,}327 \text{ kips}$$

A summary of vessel impact forces for each vessel classification is shown in Table B3.9-1.

Table B3.9-1 Ship Collision Forces

Size(DWT)	(Type)	P_s(kips)
10,000	F/C	15,319
20,000	F/C	21,664
40,000	B/T	30,637
60,000	B/T	37,523
80,000	B/T	43,327
100,000	B/T	48,441
150,000	B/T	59,328

B4.8.3.4 PROBABILITY OF COLLAPSE

The ultimate lateral resistance capacity of the main pier, H_P, is equal to 20,000 kips. The probability of collapse, PC, is computed using Equation 4.8.3.4-1. For the 20,000 DWT Freighter/Container vessel, PC is computed as:

$$H_P/P_S = 20{,}000/21{,}664 = 0.923$$

$$PC = (1\text{-}H/P)/9 \quad (4.8.3.4\text{-}1b)$$
$$= (1\text{-}0.923)/9 = 0.0086$$

A summary of PC values for each vessel classification is shown in Table B4.8.3.4-1.

Table B4.8.3.4-1 Probability of Collapse

Size(DWT)	(H_P/P_S)	PC
10,000	1.306	0.0000
20,000	0.923	0.0086
40,000	0.653	0.0386
60,000	0.533	0.0519
80,000	0.462	0.0598
100,000	0.413	0.0652
150,000	0.337	0.0737

B4.8.3 ANNUAL FREQUENCY OF COLLAPSE

The annual frequency of main pier collapse, AF, is computed using,

$$AF = (N)(PA)(PG)(PC) \quad (4.8.3\text{-}1)$$

The results of the computation for each ship classification, and the pier total, are summarized in Table B4.8.3-1. In this table, the ship sizes have been arranged in decreasing order from the largest (150,000 DWT) to the smallest (10,000 DWT) in order to sum the cumulative annual frequencies, ΣAF. This ordering allows a relatively quick determination of the impact of vessel size on the ΣAF, and for comparison against the acceptable annual frequency, AF_p.

From Table B4.8.3-1 the ΣAF for all ship classifications equals .000508. The main pier return period of collapse, RP, becomes,

$$RP = (1/AF) = (1/.000508) = 1{,}968 \text{ years}$$

B4.8.2 RISK ACCEPTANCE CRITERIA

For a Critical Bridge, the total bridge acceptable annual frequency of collapse, AF_c, equals .0001 according to Section 4.8.2. This total acceptance AF_c must be distributed among the exposed bridge elements within a distance 3xLOA on each side of the inbound and outbound vessel transit paths. For the suspension bridge in Figure B1-1, the distance from the centerline of channel to the edge of the analysis area is,

$$x = 200 + 3xLOA$$
$$= 200 + 3(1{,}030) = 3{,}290 \text{ feet}$$

Within this distance are 5 piers on each side of the channel centerline for a total of 10 piers. The distribution of AF_c among these 10 piers is determined by the designer. One method would be to equally spread the acceptable risk among all piers (i.e., $AF_c/10$), however, this is not desirable since it fails to take into account the importance and higher cost associated with the main pier of the suspension bridge. A better method would be to apportion the risk to each pier based on its percentage value of the replacement cost of the structure in the central analysis area. For the sample problem, the two main piers and the superstructure they support represents 50 percent of the replacement cost of the bridge in the central analysis area. Each main pier will, therefore, be apportioned 25 percent of the total acceptable annual frequency, so that;

$$AF_p = (.25)(AF_c) = (.25)(.0001) = .000025$$
$$RP_p = (1/AF_p) = 40{,}000 \text{ years}$$

Comparing the allowable value of AF_p = .000025 with the actual value of ΣAF=.000508 in Table B4.8.3-1 indicates that under these future year traffic conditions, the main pier will be relatively vulnerable to catastrophic vessel collision. Also from a comparison with Table B4.8.3-1, it can be seen that even the 150,000 DWT vessels with AF = .000051 exceed the acceptance criteria. The vulnerability is a result of the relatively weak resistance (H_P = 20,000 kips) of the main pier compared to the magnitude and frequency of the vessel impact forces.

B4.8.3 REVISED ANNUAL FREQUENCY

What would the pier strength have to be in order to meet the acceptance criteria? Or restated in another way, what should the impact force be (neglecting the Method III cost-effective procedure) to develop a pier protection system to protect the main pier? To answer this question requires a trial and error process in which values of H_P are assumed, new AF's are computed, and a comparison with AF_P is made.

For the first trial, assume H_P = 46,000 kips. This value of H_P is approximately equal to the impact force, P_S, of a 90,000 DWT ship. The revised annual frequency of collapse is shown in Table B4.8.3-2.

The new ΣAF = .000023 from Table B4.8.3-2 is less than the AF_P = .000025 acceptance criteria, therefore, a 90,000 DWT design vessel would be required for the design of a pier protection system for the main pier using the Method II procedure.

Table B4.8.3-1 Annual Frequency of Main Pier Collapse(H_P=20,000)

Ship(DWT)	N	PA	PG	PC	AF	ΣAF
150,000	60	$1.0X10^{-4}$	.1143	.0737	.000051	.000051
100,000	100	$1.0X10^{-4}$	.1076	.0652	.000070	.000121
80,000	300	$1.0X10^{-4}$	.0973	.0598	.000175	.000296
60,000	100	$1.0X10^{-4}$	.0906	.0519	.000047	.000343
40,000	100	$1.0X10^{-4}$	.0839	.0386	.000032	.000375
20,000	2,000	$1.0X10^{-4}$	.0772	.0086	.000133	.000508
10,000	3,000	$1.0X10^{-4}$	.0607	.0000	----	----

Table B4.8.3-2 Annual Frequency of Main Pier Collapse(H_P=46,000)

Ship(DWT)	N	PA	PG	PC	AF	ΣAF
150,000	60	$1.0X10^{-4}$	.1143	.0250	.000017	.000017
100,000	100	$1.0X10^{-4}$	.1076	.0056	.000006	.000023
80,000	300	$1.0X10^{-4}$	.0973	.0000	----	----
60,000	100	$1.0X10^{-4}$	.0906	.0000	----	----
40,000	100	$1.0X10^{-4}$	.0839	.0000	----	----
20,000	2,000	$1.0X10^{-4}$	.0772	.0000	----	----
10,000	3,000	$1.0X10^{-4}$	.0607	.0000	----	----