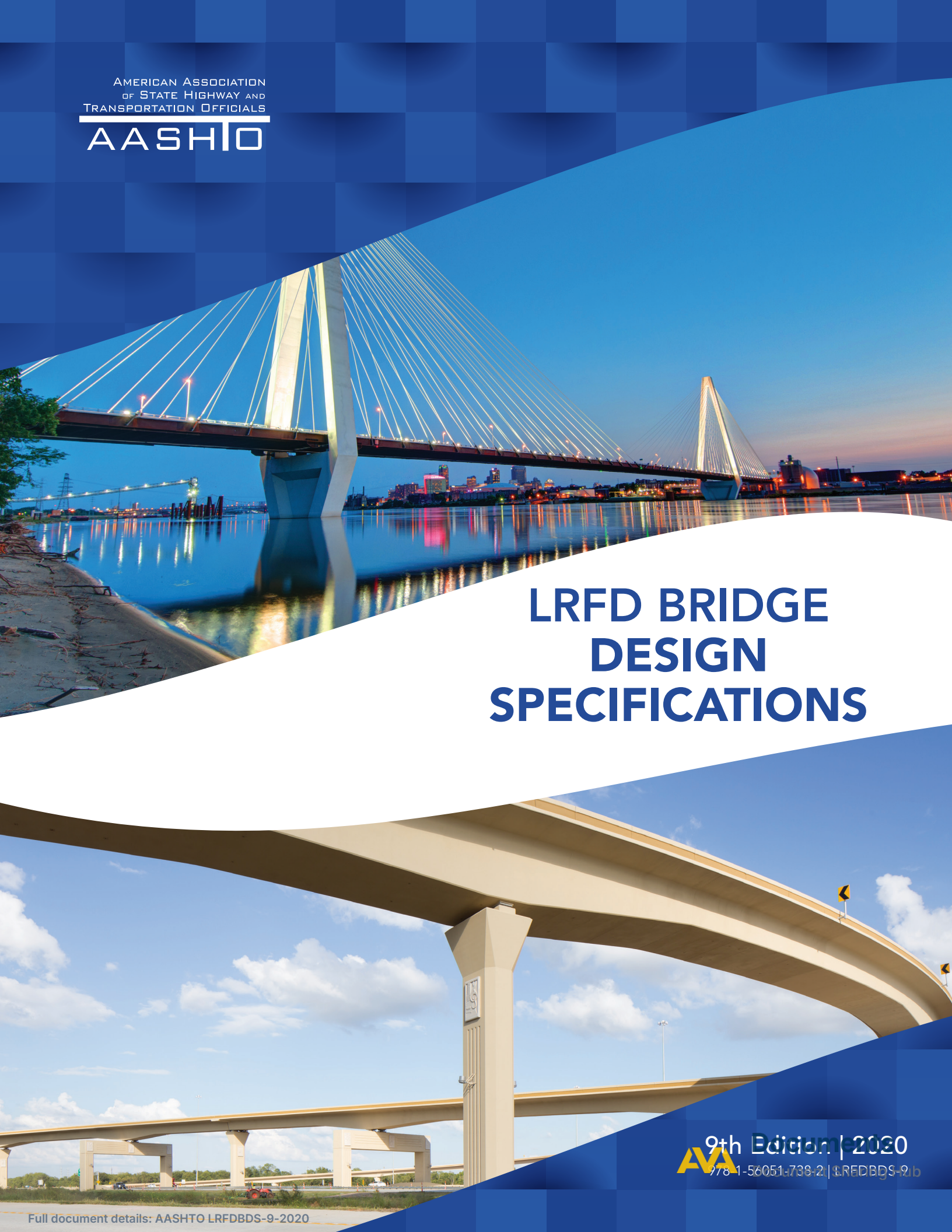


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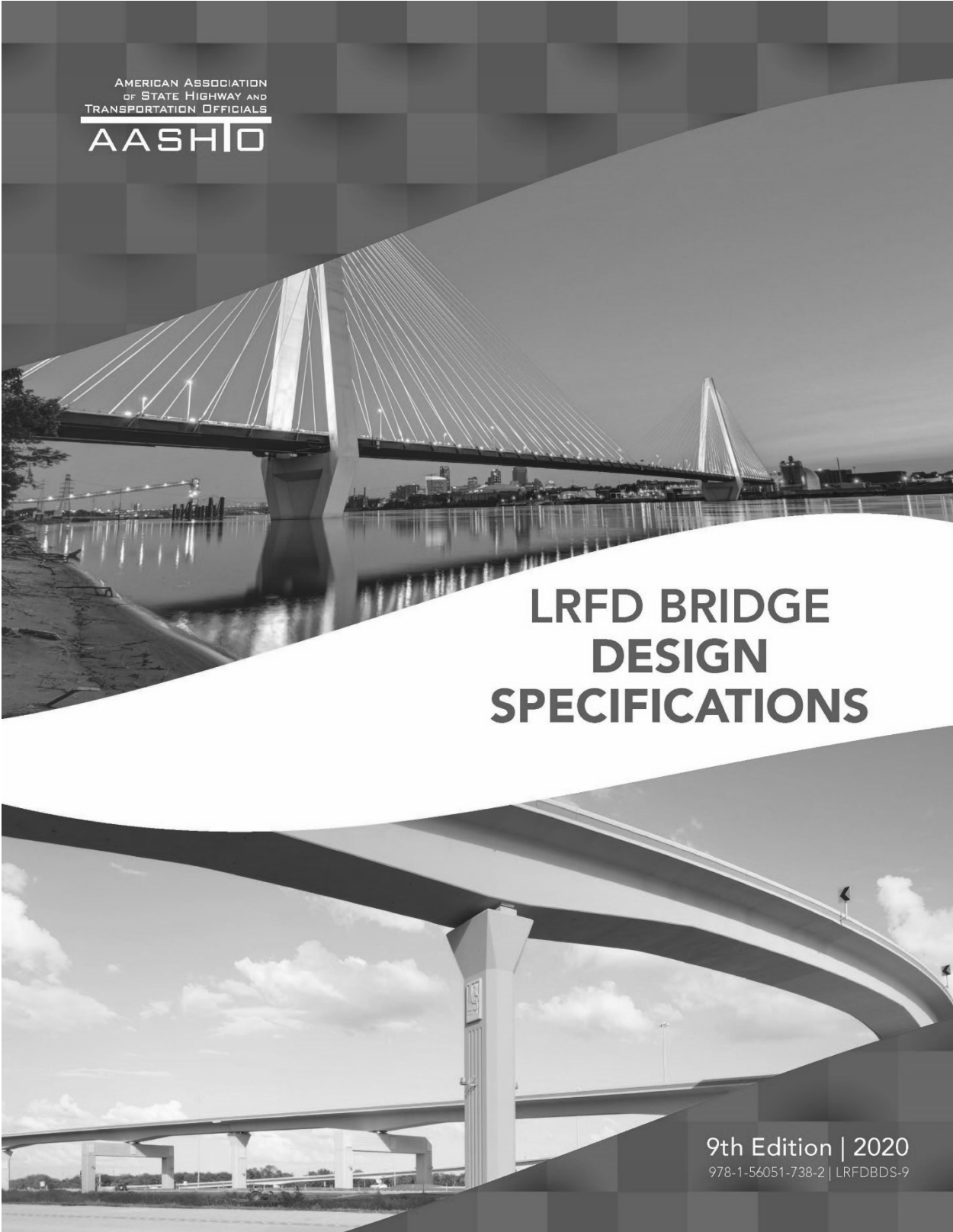
LRFD BRIDGE DESIGN SPECIFICATIONS

9th Edition | 2020
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FOREWORD

The first broadly recognized national standard for the design and construction of bridges in the United States was published in 1931 by the American Association of State Highway Officials (AASHO), the predecessor to AASHTO. With the advent of the automobile and the establishment of highway departments in all of the American states dating back to just before the turn of the century, the design, construction, and maintenance of most U.S. bridges was the responsibility of these departments and, more specifically, the chief bridge engineer within each department. It was natural, therefore, that these engineers, acting collectively as the AASHTO Highway Subcommittee on Bridges and Structures (now the Committee on Bridges and Structures), would become the author and guardian of this first bridge standard.

This first publication was entitled *Standard Specifications for Highway Bridges and Incidental Structures*. It quickly became the *de facto* national standard and, as such, was adopted and used by not only the state highway departments but also other bridge-owning authorities and agencies in the United States and abroad. Rather early on, the last three words of the original title were dropped and it has been reissued in consecutive editions at approximately four-year intervals ever since as *Standard Specifications for Highway Bridges*, with the final 17th edition appearing in 2002.

The body of knowledge related to the design of highway bridges has grown enormously since 1931 and continues to do so. Theory and practice have evolved greatly, reflecting advances through research in understanding the properties of materials, in improved materials, in more rational and accurate analysis of structural behavior, in the advent of computers and rapidly advancing computer technology, in the study of external events representing particular hazards to bridges such as seismic events and stream scour, and in many other areas. The pace of advances in these areas has, if anything, stepped up in recent years.

In 1986, the Subcommittee submitted a request to the AASHTO Standing Committee on Research to undertake an assessment of U.S. bridge design specifications, to review foreign design specifications and codes, to consider design philosophies alternative to those underlying the Standard Specifications, and to render recommendations based on these investigations. This work was accomplished under the National Cooperative Highway Research Program (NCHRP), an applied research program directed by the AASHTO Standing Committee on Research and administered on behalf of AASHTO by the Transportation Research Board (TRB). The work was completed in 1987, and, as might be expected with a standard incrementally adjusted over the years, the Standard Specifications were judged to include discernible gaps, inconsistencies, and even some conflicts. Beyond this, the specification did not reflect or incorporate the most recently developing design philosophy, load-and-resistance factor design (LRFD), a philosophy which has been gaining ground in other areas of structural engineering and in other parts of the world such as Canada and Europe.

From its inception until the early 1970s, the sole design philosophy embedded within the Standard Specifications was one known as working stress design (WSD). WSD establishes allowable stresses as a fraction or percentage of a given material's load-carrying capacity, and requires that calculated design stresses not exceed those allowable stresses. Beginning in the early 1970s, WSD began to be adjusted to reflect the variable predictability of certain load types, such as vehicular loads and wind forces, through adjusting design factors, a design philosophy referred to as load factor design (LFD).

A further philosophical extension results from considering the variability in the properties of structural elements, in similar fashion to load variabilities. While considered to a limited extent in LFD, the design philosophy of load-and-resistance factor design (LRFD) takes variability in the behavior of structural elements into account in an explicit manner. LRFD relies on extensive use of statistical methods, but sets forth the results in a manner readily usable by bridge designers and analysts.

Starting with the Eighth Edition of the *AASHTO LRFD Bridge Design Specifications*, interim changes to the Specifications were discontinued, and new editions are published on a three-year cycle. Changes are balloted and approved by at least two-thirds of the members of the Committee on Bridges and Structures. AASHTO members include the 50 State Highway or Transportation Departments, the District of Columbia, and Puerto Rico. Each member has one vote. The U.S. Department of Transportation is a non-voting member.

Orders for Specifications may be placed by visiting the AASHTO Store, store.transportation.org; calling the AASHTO Publication Sales Office toll free (within the U.S. and Canada), 1-800-231-3475; or mailing to P.O. Box 933538, Atlanta, GA 31193-3538. A free copy of the current publication catalog can be downloaded from the AASHTO Store.

For additional publications prepared and published by the Committee on Bridges and Structures and by other AASHTO Committees, please look online in the AASHTO Store (store.transportation.org) under “Bridges and Structures.”

Suggestions for the improvement of the *AASHTO LRFD Bridge Design Specifications* are welcomed, just as they were for the *Standard Specifications for Highway Bridges* before them, at www.transportation.org.

The following have served as chair of the Committee on Bridges and Structures since its inception in 1921: E. F. Kelley, who pioneered the work of the Committee; Albin L. Gemeny; R. B. McMinn; Raymond Archiband; G. S. Paxson; E. M. Johnson; Ward Goodman; Charles Matlock; Joseph S. Jones; Sidney Poleynard; Jack Freidenrich; Henry W. Derthick; Robert C. Cassano; Clellon Loveall; James E. Siebels; David Pope; Tom Lulay; Malcolm T. Kerley; Gregg Fredrick; and Carmen Swanwick. The Committee expresses its sincere appreciation of the work of these individuals and of those active members of the past, whose names, because of retirement, are no longer on the roll.

The Committee would also like to thank John M. Kulicki, Ph.D., and his associates at Modjeski and Masters for their valuable assistance in the preparation of the *AASHTO LRFD Bridge Design Specifications*.

PREFACE AND ABBREVIATED TABLE OF CONTENTS

The *AASHTO LRFD Bridge Design Specifications*, Ninth Edition contains the following 15 sections and an index:

1. Introduction
 2. General Design and Location Features
 3. Loads and Load Factors
 4. Structural Analysis and Evaluation
 5. Concrete Structures
 6. Steel Structures
 7. Aluminum Structures
 8. Wood Structures
 9. Decks and Deck Systems
 10. Foundations
 11. Abutments, Piers, and Walls
 12. Buried Structures and Tunnel Liners
 13. Railings
 14. Joints and Bearings
 15. Design of Sound Barriers
- Index

Detailed Tables of Contents precede each section. The last article of each section is a list of references displayed alphabetically by author.

Figures, tables, and equations are denoted by their home article number and an extension, for example 1.2.3.4.5-1 wherever they are cited. In early editions, when they were referenced in their home article or its commentary, these objects were identified only by the extension. For example, in Article 1.2.3.4.5, Eq. 1.2.3.4.5-2 would simply have been called “Eq. 2.” The same convention applies to figures and tables. Starting with this edition, these objects are identified by their whole nomenclature throughout the text, even within their home articles. This change was to increase the speed and accuracy of electronic production (i.e., CDs and downloadable files) with regard to linking citations to objects.

Please note that the AASHTO materials standards (starting with M or T) cited throughout the LRFD Bridge Design Specifications can be found in *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*, adopted by the AASHTO Highway Subcommittee on Materials. The individual standards are also available as downloads on the AASHTO Store, <https://store.transportation.org>. Unless otherwise indicated, these citations refer to the current edition. ASTM materials specifications are also cited and have been updated to reflect ASTM’s revised coding system, i.e., spaces removed between the letter and number.

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CHANGED AND DELETED ARTICLES, 2020

SUMMARY OF AFFECTED SECTIONS

The revisions included in the *AASHTO LRFD Bridge Design Specifications*, Ninth Edition affect the following sections:

1. Introduction
3. Loads and Load Factors
4. Structural Analysis and Evaluation
5. Concrete Structures
6. Steel Structures
8. Wood Structures
10. Foundations
11. Walls, Abutments, and Piers
12. Buried Structures and Tunnel Liners
15. Design of Sound Barriers

SECTION 1 REVISIONS

Changed Articles

The following Articles in Section 1 contain changes or additions to the specifications, the commentary, or both:

1.3.5

Deleted Articles

No Articles were deleted from Section 1.

SECTION 3 REVISIONS

Changed Articles

The following Articles in Section 3 contain changes or additions to the specifications, the commentary, or both:

3.3.1	3.6.1.2.6a	3.6.5.2	3.11.5.6	3.11.5.9
3.4.1	3.6.5.1	3.11.5.4	3.11.5.8.2	3.16

Deleted Articles

No Articles were deleted from Section 3.

SECTION 4 REVISIONS

Changed Articles

The following Articles in Section 4 contain changes or additions to the specifications, the commentary, or both:

4.5.3.2.2b	4.6.2.2.1	4.6.2.2.2b	4.6.2.10.2	4.9
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Deleted Articles

No Articles were deleted from Section 4.

SECTION 5 REVISIONS

Changed Articles

The following Articles in Section 5 contain changes or additions to the specifications, the commentary, or both:

5.3	5.7.2.1	5.8.4.3.5	5.10.4.3	5.12.9.5.2
5.4.3.1	5.7.2.8	5.9.4.3.3	5.10.8.2.5	5.14.1
5.4.6.2	5.7.3.3	5.9.4.5	5.10.8.5.1	5.14.4
5.5.3.1	5.7.3.5	5.9.5.6.1	5.10.8.5.2	5.15
5.5.4.3	5.7.3.6.2	5.10.1	5.12.3.2.1	

Deleted Articles

No Articles were deleted from Section 5.

SECTION 6 REVISIONS

Changed Articles

The following Articles in Section 6 contain changes or additions to the specifications, the commentary, or both:

6.1	6.9.4.1.2	6.10.11	6.12.2.2.2d	6.14.4.4
6.2	6.9.4.1.3	6.10.11.1	6.12.2.2.2e	6.14.4.5
6.3	6.9.4.2	6.10.11.1.1	6.12.2.2.2f	6.14.4.6
6.4.9	6.9.4.2.1	6.10.11.2.2	6.12.2.2.2g	6.16.1
6.5.3	6.9.4.2.2	6.10.11.2.4b	6.12.2.2.3	6.16.2
6.5.4.2	6.9.4.2.2a	6.10.11.3	6.12.2.2.4a	6.16.4.1
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6.6.1.2.3	6.9.4.2.2c	6.10.11.3.3	6.12.2.2.4c	A6
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6.6.2.1	6.9.4.4	6.11.1.1	6.12.2.2.4e	A6.2.1
6.6.2.2	6.9.4.5	6.11.3.2	6.12.2.2.5	A6.2.2
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6.7.4.4.2	6.10.1.10.1	6.12.1	6.13.2.7	C6.5.1
6.7.4.4.3	6.10.1.10.2	6.12.1.1	6.13.2.9	C6.5.2
6.7.4.5	6.10.2.2	6.12.1.2.1	6.13.2.10.2	D6.2.1
6.7.8	6.10.3.3	6.12.1.2.2	6.13.2.11	D6.3.1
6.8.2.2	6.10.3.4.1	6.12.1.2.3	6.13.3.6	E6.1
6.8.2.3	6.10.3.4.2	6.12.1.2.3a	6.13.3.7	E6.1.1
6.8.2.3.1	6.10.5.2	6.12.1.2.3b	6.13.6.1.3a	E6.1.2
6.8.2.3.2	6.10.6.1	6.12.1.2.4	6.13.6.1.3b	E6.1.3
6.8.2.3.3	6.10.6.2.3	6.12.2	6.13.6.1.3c	E6.1.4
6.8.6.2	6.10.8.1.1	6.12.2.1	6.13.6.1.4	E6.1.5
6.9.2.2	6.10.8.2.3	6.12.2.2.2	6.14.2.4	E6.1.5.1
6.9.2.2.1	6.10.8.3	6.12.2.2.2a	6.14.4.1	E6.1.5.2
6.9.2.2.2	6.10.9.1	6.12.2.2.2b	6.14.4.2	
6.9.4.1.1	6.10.10.2	6.12.2.2.2c	6.14.4.3	

Deleted Articles

6.12.1.2.3c

SECTION 8 REVISIONS

Changed Articles

The following Articles in Section 8 contain changes or additions to the specifications, the commentary, or both:

8.2	8.4.1.2.1	8.4.1.2.3	8.4.4.9	8.14
8.4.1.1.4	8.4.1.2.2	8.4.1.3.1	8.13	

Deleted Articles

No Articles were deleted from Section 8.

SECTION 10 REVISIONS

Changed Articles

The following Articles in Section 10 contain changes or additions to the specifications, the commentary, or both:

10.3	10.5.3.1	10.6.2.4.2b	10.6.3.5	10.9.3.5.4
10.5.2.1	10.5.5.1	10.6.2.4.2c	10.7.2.1	10.10
10.5.2.2	10.5.5.2.1	10.6.2.4.4	10.7.3.1	
10.5.2.2.1	10.6.2.1	10.6.3.1.2a	10.7.8	
10.5.2.2.2	10.6.2.4.1	10.6.3.1.2c	10.8.3.5.1b	
10.5.2.4	10.6.2.4.2a	10.6.3.2.1	10.8.3.5.2b	

Deleted Articles

10.5.2.3	10.6.2.5
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SECTION 11 REVISIONS

Changed Articles

The following Articles in Section 11 contain changes or additions to the specifications, the commentary, or both:

11.1	11.6.5.1	11.10.5.2	11.10.6.4.1	11.10.10.3
11.2	11.8.3	11.10.5.6	11.10.6.4.2a	11.10.11
11.3.1	11.8.4.1	11.10.6.1	11.10.6.4.2b	11.11.4.6
11.5.2	11.8.6.1	11.10.6.2	11.10.6.4.3b	11.12
11.5.3	11.9.3	11.10.6.2.1	11.10.6.4.4a	11.12.7.2
11.5.4.2	11.9.4.3	11.10.6.2.1a	11.10.6.4.4b	11.12.9
11.5.5	11.9.4.4	11.10.6.2.1b	11.10.7.2	11.13
11.5.7	11.10.1	11.10.6.2.1c	11.10.7.3	A11.5.3
11.5.8	11.10.2.3.1	11.10.6.2.1d	11.10.7.4	B11.1
11.6.2	11.10.4	11.10.6.2.1e	11.10.8	B11.2
11.6.3.1	11.10.4.2	11.10.6.2.2	11.10.10.1	B11.3
11.6.3.7	11.10.4.3	11.10.6.3.2	11.10.10.2	

Deleted Articles

11.6.2.3	11.8.3.2	11.9.3.2
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SECTION 12 REVISIONS

Changed Articles

The following Articles in Section 12 contain changes or additions to the specifications, the commentary, or both:

12.1	12.5.4	12.7.2.7	12.10.2.1	12.12.3.10.2b
12.2	12.6.4	12.8.5.3	12.10.4.3.1	12.16
12.4.2.3	12.6.9	12.8.9.3.1	12.12.2.1	A12
12.4.2.10	12.7.2.1	12.10.1	12.12.3.5	

Deleted Articles

No Articles were deleted from Section 12.

SECTION 15 REVISIONS

Changed Articles

The following Articles in Section 15 contain changes or additions to the specifications, the commentary, or both:

15.9.3	15.9.7	15.9.8
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Deleted Articles

15.9.7.1

AASHTO Publications Staff
April 2020

SECTION 1: INTRODUCTION

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SECTION 1

INTRODUCTION

Commentary is opposite the text it annotates.

1.1—SCOPE OF THE SPECIFICATIONS

C1.1

The provisions of these Specifications are intended for the design, evaluation, and rehabilitation of both fixed and movable highway bridges. Mechanical, electrical, and special vehicular and pedestrian safety aspects of movable bridges, however, are not covered. Provisions are not included for bridges used solely for railway, rail-transit, or public utilities. For bridges not fully covered herein, the provisions of these Specifications may be applied, and augmented with additional design criteria where required.

These Specifications are not intended to supplant proper training or the exercise of judgment by the Designer, and state only the minimum requirements necessary to provide for public safety. The Owner or the Designer may require the sophistication of design or the quality of materials and construction to be higher than the minimum requirements.

The concepts of safety through redundancy and ductility and of protection against scour and collision are emphasized.

The design provisions of these Specifications employ the Load and Resistance Factor Design (LRFD) methodology. The factors have been developed from the theory of reliability based on current statistical knowledge of loads and structural performance.

Methods of analysis other than those included in previous Specifications and the modeling techniques inherent in them are included, and their use is encouraged.

Seismic design shall be in accordance with either the provisions in these Specifications or those given in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

The commentary is not intended to provide a complete historical background concerning the development of these or previous Specifications, nor is it intended to provide a detailed summary of the studies and research data reviewed in formulating the provisions of the Specifications. However, references to some of the research data are provided for those who wish to study the background material in depth.

The commentary directs attention to other documents that provide suggestions for carrying out the requirements and intent of these Specifications. However, those documents and this commentary are not intended to be a part of these Specifications.

Construction specifications consistent with these design specifications are the *AASHTO LRFD Bridge Construction Specifications*. Unless otherwise specified, the Materials Specifications referenced herein are the *AASHTO Standard Specifications for Transportation Materials and Methods of Sampling and Testing*.

The term “notional” is often used in these Specifications to indicate an idealization of a physical phenomenon, as in “notional load” or “notional resistance.” Use of this term strengthens the separation of an engineer’s “notion” or perception of the physical world in the context of design from the physical reality itself.

The term “shall” denotes a requirement for compliance with these Specifications.

The term “should” indicates a strong preference for a given criterion.

The term “may” indicates a criterion that is usable, but other local and suitably documented, verified, and approved criteria may also be used in a manner consistent with the LRFD approach to bridge design.

1.2—DEFINITIONS

Bridge—Any structure having an opening not less than 20.0 ft that forms part of a highway or that is located over or under a highway.

Collapse—A major change in the geometry of the bridge rendering it unfit for use.

Component—Either a discrete element of the bridge or a combination of elements requiring individual design consideration.

Design—Proportioning and detailing the components and connections of a bridge.

Design Life—Period of time on which the statistical derivation of transient loads is based, which is 75 years for these Specifications.

Ductility—Property of a component or connection that allows inelastic response.

Engineer—Person responsible for the design of the bridge and/or review of design-related field submittals such as erection plans.

Evaluation—Determination of load-carrying capacity of an existing bridge.

Extreme Event Limit States—Limit states relating to events such as earthquakes, ice load, and vehicle and vessel collision, with return periods in excess of the design life of the bridge.

Factored Load—The nominal loads multiplied by the appropriate load factors specified for the load combination under consideration.

Factored Resistance—The nominal resistance multiplied by a resistance factor.

Fixed Bridge—A bridge with a fixed vehicular or navigational clearance.

Force Effect—A deformation, stress, or stress resultant (i.e., axial force, shear force, or torsional or flexural moment) caused by applied loads, imposed deformations, or volumetric changes.

Limit State—A condition beyond which the bridge or component ceases to satisfy the provisions for which it was designed.

Load and Resistance Factor Design (LRFD)—A reliability-based design methodology in which force effects caused by factored loads are not permitted to exceed the factored resistance of the components.

Load Factor—A statistically-based multiplier applied to force effects accounting primarily for the variability of loads, the lack of accuracy in analysis, and the probability of simultaneous occurrence of different loads, but also related to the statistics of the resistance through the calibration process.

Load Modifier—A factor accounting for ductility, redundancy, and the operational classification of the bridge.

Model—An idealization of a structure for the purpose of analysis.

Movable Bridge—A bridge with a variable vehicular or navigational clearance.

Multiple-Load-Path Structure—A structure capable of supporting the specified loads following loss of a main load-carrying component or connection.

Nominal Resistance—Resistance of a component or connection to force effects, as indicated by the dimensions specified in the contract documents and by permissible stresses, deformations, or specified strength of materials.

Owner—Person or agency having jurisdiction over the bridge.

Regular Service—Condition excluding the presence of special permit vehicles, wind exceeding 55 mph, and extreme events, including scour.

Rehabilitation—A process in which the resistance of the bridge is either restored or increased.

Resistance Factor—A statistically-based multiplier applied to nominal resistance accounting primarily for variability of material properties, structural dimensions and workmanship, and uncertainty in the prediction of resistance, but also related to the statistics of the loads through the calibration process.

Service Life—The period of time that the bridge is expected to be in operation.

Service Limit States—Limit states relating to stress, deformation, and cracking under regular operating conditions.

Strength Limit States—Limit states relating to strength and stability during the design life.

1.3—DESIGN PHILOSOPHY

1.3.1—General

Bridges shall be designed for specified limit states to achieve the objectives of constructibility, safety, and serviceability, with due regard to issues of inspectability, economy, and aesthetics, as specified in Article 2.5.

Regardless of the type of analysis used, Eq. 1.3.2.1-1 shall be satisfied for all specified force effects and combinations thereof.

1.3.2—Limit States

1.3.2.1—General

Each component and connection shall satisfy Eq. 1.3.2.1-1 for each limit state, unless otherwise specified. For service and extreme event limit states, resistance factors shall be taken as 1.0, except for bolts, for which the provisions of Article 6.5.5 shall apply, and for concrete columns in Seismic Zones 2, 3, and 4, for which the provisions of Articles 5.11.3 and 5.11.4.1.2 shall apply. All limit states shall be considered of equal importance.

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \quad (1.3.2.1-1)$$

in which:

For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \quad (1.3.2.1-2)$$

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \quad (1.3.2.1-3)$$

C1.3.1

The limit states specified herein are intended to provide for a buildable, serviceable bridge, capable of safely carrying design loads for a specified lifetime.

The resistance of components and connections is determined, in many cases, on the basis of inelastic behavior, although the force effects are determined by using elastic analysis. This inconsistency is common to most current bridge specifications as a result of incomplete knowledge of inelastic structural action.

C1.3.2.1

Eq. 1.3.2.1-1 is the basis of LRFD methodology.

Assigning resistance factor $\phi = 1.0$ to all nonstrength limit states is a default, and may be overridden by provisions in other Sections.

Ductility, redundancy, and operational classification are considered in the load modifier η . Whereas the first two directly relate to physical strength, the last concerns the consequences of the bridge being out of service. The grouping of these aspects on the load side of Eq. 1.3.2.1-1 is, therefore, arbitrary. However, it constitutes a first effort at codification. In the absence of more precise information, each effect, except that for fatigue and fracture, is estimated as ± 5 percent, accumulated geometrically. This is a clearly subjective approach, and a rearrangement of Eq. 1.3.2.1-1 may be attained with time. Such a rearrangement might account for improved quantification of ductility, redundancy, and operational classification, and their interactions with system reliability in such an equation.

where:

- γ_i = load factor: a statistically based multiplier applied to force effects
- ϕ = resistance factor: a statistically based multiplier applied to nominal resistance, as specified in Sections 5, 6, 7, 8, 10, 11, and 12
- η_i = load modifier: a factor relating to ductility, redundancy, and operational classification
- η_D = a factor relating to ductility, as specified in Article 1.3.3
- η_R = a factor relating to redundancy, as specified in Article 1.3.4
- η_I = a factor relating to operational classification, as specified in Article 1.3.5
- Q_i = force effect
- R_n = nominal resistance
- R_r = factored resistance: ϕR_n

1.3.2.2—Service Limit State

The service limit state shall be taken as restrictions on stress, deformation, and crack width under regular service conditions.

1.3.2.3—Fatigue and Fracture Limit State

The fatigue limit state shall be taken as restrictions on stress range as a result of a single design truck occurring at the number of expected stress range cycles.

The fracture limit state shall be taken as a set of material toughness requirements of the *AASHTO Materials Specifications*.

1.3.2.4—Strength Limit State

Strength limit state shall be taken to ensure that strength and stability, both local and global, are provided to resist the specified statistically significant load combinations that a bridge is expected to experience in its design life.

The influence of η on the girder reliability index, β , can be estimated by observing its effect on the minimum values of β calculated in a database of girder-type bridges. Cellular structures and foundations were not a part of the database; only individual member reliability was considered. For discussion purposes, the girder bridge data used in the calibration of these Specifications was modified by multiplying the total factored loads by $\eta = 0.95, 1.0, 1.05$, and 1.10 . The resulting minimum values of β for 95 combinations of span, spacing, and type of construction were determined to be approximately 3.0, 3.5, 3.8, and 4.0, respectively. In other words, using $\eta > 1.0$ relates to a β higher than 3.5.

A further approximate representation of the effect of η values can be obtained by considering the percent of random normal data less than or equal to the mean value plus $\lambda \sigma$, where λ is a multiplier, and σ is the standard deviation of the data. If λ is taken as 3.0, 3.5, 3.8, and 4.0, the percent of values less than or equal to the mean value plus $\lambda \sigma$ would be about 99.865 percent, 99.977 percent, 99.993 percent, and 99.997 percent, respectively.

The Strength I Limit State in the *AASHTO LRFD Design Specifications* has been calibrated for a target reliability index of 3.5 with a corresponding probability of exceedance of $2.0E-04$ during the 75-year design life of the bridge. This 75-year reliability is equivalent to an annual probability of exceedance of $2.7E-06$ with a corresponding annual target reliability index of 4.6. Similar calibration efforts for the Service Limit States are underway. Return periods for extreme events are often based on annual probability of exceedance, and caution must be used when comparing reliability indices of various limit states.

C1.3.2.2

The service limit state provides certain experience-related provisions that cannot always be derived solely from strength or statistical considerations.

C1.3.2.3

The fatigue limit state is intended to limit crack growth under repetitive loads to prevent fracture during the design life of the bridge.

C1.3.2.4

The strength limit state considers stability or yielding of each structural element. If the resistance of any element, including splices and connections, is exceeded, it is assumed that the bridge resistance has been exceeded. In fact, there is significant elastic reserve capacity in almost all multigirder bridges beyond such a load level. The live load cannot be positioned to maximize the force effects on

1.3.2.5—Extreme Event Limit States

The extreme event limit state shall be taken to ensure the structural survival of a bridge during a major earthquake or flood, or when collided with by a vessel, vehicle, or ice floe, possibly under scoured conditions.

1.3.3—Ductility

The structural system of a bridge shall be proportioned and detailed to ensure the development of significant and visible inelastic deformations at the strength and extreme event limit states before failure.

Energy-dissipating devices may be substituted for conventional ductile earthquake resisting systems and the associated methodology addressed in these Specifications or in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design*.

For the strength limit state:

$$\begin{aligned}\eta_D &\geq 1.05 \text{ for nonductile components and connections} \\ &= 1.00 \text{ for conventional designs and details} \\ &\quad \text{complying with these Specifications} \\ &\geq 0.95 \text{ for components and connections for which} \\ &\quad \text{additional ductility-enhancing measures have} \\ &\quad \text{been specified beyond those required by these} \\ &\quad \text{Specifications}\end{aligned}$$

For all other limit states:

$$\eta_D = 1.00$$

all parts of the cross-section simultaneously. Thus, the flexural resistance of the bridge cross-section typically exceeds the resistance required for the total live load that can be applied in the number of lanes available. Extensive distress and structural damage may occur under strength limit state, but overall structural integrity is expected to be maintained.

C1.3.2.5

Extreme event limit states are considered to be unique occurrences that may have severe operational impact and whose return period may be significantly greater than the design life of the bridge.

The Owner may choose to require that the extreme event limit state provide restricted or immediate serviceability in special cases of operational importance of the bridge or transportation corridor.

C1.3.3

The response of structural components or connections beyond the elastic limit can be characterized by either brittle or ductile behavior. Brittle behavior is undesirable because it implies the sudden loss of load-carrying capacity immediately when the elastic limit is exceeded. Ductile behavior is characterized by significant inelastic deformations before any loss of load-carrying capacity occurs. Ductile behavior provides warning of structural failure by large inelastic deformations. Under repeated seismic loading, large reversed cycles of inelastic deformation dissipate energy and have a beneficial effect on structural survival.

If, by means of confinement or other measures, a structural component or connection made of brittle materials can sustain inelastic deformations without significant loss of load-carrying capacity, this component can be considered ductile. Such ductile performance shall be verified by testing.

In order to achieve adequate inelastic behavior, the system should have a sufficient number of ductile members and either:

- joints and connections that are also ductile and can provide energy dissipation without loss of capacity; or
- joints and connections that have sufficient excess strength so as to assure that the inelastic response occurs at the locations designed to provide ductile, energy absorbing response.

Statically ductile but dynamically nonductile response characteristics should be avoided. Examples of this behavior are shear and bond failures in concrete members and loss of composite action in flexural components.

Past experience indicates that typical components designed in accordance with these provisions generally exhibit adequate ductility. Connection and joints require special attention to detailing and the provision of load paths.

The Owner may specify a minimum ductility factor as an assurance that ductile failure modes will be obtained. The factor may be defined as:

$$\mu = \frac{\Delta_u}{\Delta_y} \quad (\text{C1.3.3-1})$$

where:

Δ_u = deformation at ultimate

Δ_y = deformation at the elastic limit

The ductility capacity of structural components or connections may either be established by full- or large-scale testing or with analytical models based on documented material behavior. The ductility capacity for a structural system may be determined by integrating local deformations over the entire structural system.

The special requirements for energy dissipating devices are imposed because of the rigorous demands placed on these components.

1.3.4—Redundancy

Multiple-load-path and continuous structures should be used unless there are compelling reasons not to use them.

For the strength limit state:

$$\begin{aligned} \eta_R &\geq 1.05 \text{ for nonredundant members} \\ &= 1.00 \text{ for conventional levels of redundancy,} \\ &\quad \text{foundation elements where } \phi \text{ already accounts for} \\ &\quad \text{redundancy as specified in Article 10.5} \\ &\geq 0.95 \text{ for exceptional levels of redundancy beyond} \\ &\quad \text{girder continuity and a torsionally-closed cross-} \\ &\quad \text{section} \end{aligned}$$

C1.3.4

For each load combination and limit state under consideration, member redundancy classification (redundant or nonredundant) should be based upon the member contribution to the bridge safety. Several redundancy measures have been proposed (Frangopol and Nakib, 1991).

Single-cell boxes and single-column bents may be considered nonredundant at the Owner's discretion. For prestressed concrete boxes, the number of tendons in each web should be taken into consideration. For steel cross-sections and fracture-critical considerations, see Section 6.

The Manual for Bridge Evaluation (2018) defines bridge redundancy as "the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members." System factors are provided for post-tensioned segmental concrete box girder bridges in Section 6A.5.11.6 of the Manual.

System reliability encompasses redundancy by considering the system of interconnected components and members. Rupture or yielding of an individual component may or may not mean collapse or failure of the whole structure or system (Nowak, 2000). Reliability indexes for entire systems are a subject of ongoing research and are

anticipated to encompass ductility, redundancy, and member correlation.

For all other limit states:

$$\eta_R = 1.00$$

1.3.5—Operational Importance

The Owner may declare a bridge or any structural component and connection thereof to be of operational priority.

C1.3.5

Such classification should be done by personnel responsible for the affected transportation network and knowledgeable of its operational needs. The definition of operational priority may differ from Owner to Owner and network to network. Guidelines for classifying critical or essential bridges are as follows:

- Bridges that are required to be open to all traffic once inspected after the design event and be usable by emergency vehicles and for security, defense, economic, or secondary life safety purposes immediately after the design event.
- Bridges that should, as a minimum, be open to emergency vehicles and for security, defense, or economic purposes after the design event, and open to all traffic within days after that event.

For the strength limit state:

$$\eta_I \geq 1.05 \text{ for critical or essential bridges}$$

$$= 1.00 \text{ for typical bridges}$$

$$\geq 0.95 \text{ for relatively less important bridges.}$$

Owner-classified bridges may use a value for $\eta < 1.0$ based on ADTT, span length, available detour length, or other rationale to use less stringent criteria.

For all other limit states:

$$\eta_I = 1.00$$

1.4—REFERENCES

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SECTION 2

GENERAL DESIGN AND LOCATION FEATURES

Commentary is opposite the text it annotates.

2.1—SCOPE

C2.1

Minimum requirements are provided for clearances, environmental protection, aesthetics, geological studies, economy, rideability, durability, constructability, inspectability, and maintainability. Minimum requirements for traffic safety are referenced.

Minimum requirements for drainage facilities and self-protecting measures against water, ice, and water-borne salts are included.

In recognition that many bridge failures have been caused by scour, hydrology and hydraulics are covered in detail.

This Section is intended to provide the Designer with sufficient information to determine the configuration and overall dimensions of a bridge.

2.2—DEFINITIONS

Aggradation—A general and progressive buildup or raising of the longitudinal profile of the channel bed as a result of sediment deposition.

Check Flood for Bridge Scour—The flood resulting from storm, storm surge, tide, or some combination thereof having a flow rate in excess of the design flood for scour, but in no case a flood with a recurrence interval exceeding the typically used 500 years. The check flood for bridge scour is used in the investigation and assessment of a bridge foundation to determine whether the foundation can withstand that flow and its associated scour and remain stable with no reserve. See also superflood.

Clear Zone—An unobstructed, relatively flat area beyond the edge of the traveled way for the recovery of errant vehicles. The traveled way does not include shoulders or auxiliary lanes.

Clearance—An unobstructed horizontal or vertical space.

Degradation—A general and progressive lowering of the longitudinal profile of the channel bed as a result of long-term erosion.

Design Discharge—Maximum flow of water a bridge is expected to accommodate without exceeding the adopted design constraints.

Design Flood for Bridge Scour—The flood flow equal to or less than the 100-year flood that creates the deepest scour at bridge foundations. The highway or bridge may be inundated at the stage of the design flood for bridge scour. The worst-case scour condition may occur for the overtopping flood as a result of the potential for pressure flow.

Design Flood for Waterway Opening—The peak discharge, volume, stage, or wave crest elevation and its associated probability of exceedence that are selected for the design of a highway or bridge over a watercourse or floodplain. By definition, the highway or bridge will not be inundated at the stage of the design flood for the waterway opening.

Detention Basin—A storm water management facility that impounds runoff and temporarily discharges it through a hydraulic outlet structure to a downstream conveyance system.

Drip Groove—Linear depression in the bottom of components to cause water flowing on the surface to drop.

Five-Hundred-Year Flood—The flood due to storm, tide, or both having a 0.2 percent chance of being equaled or exceeded in any given year.

General or Contraction Scour—Scour in a channel or on a floodplain that is not localized at a pier or other obstruction to flow. In a channel, general/contraction scour usually affects all or most of the channel width and is typically caused by a contraction of the flow.

Hydraulics—The science concerned with the behavior and flow of liquids, especially in pipes and channels.

Hydrology—The science concerned with the occurrence, distribution, and circulation of water on the earth, including precipitation, runoff, and groundwater.

Local Scour—Scour in a channel or on a floodplain that is localized at a pier, abutment, or other obstruction to flow.

Mixed Population Flood—Flood flows derived from two or more causative factors, e.g., a spring tide driven by hurricane-generated onshore winds or rainfall on a snowpack.

One-Hundred-Year Flood—The flood due to storm, tide, or both having a 1 percent chance of being equaled or exceeded in any given year.

Overtopping Flood—The flood flow that, if exceeded, results in flow over a highway or bridge, over a watershed divide, or through structures provided for emergency relief. The worst-case scour condition may be caused by the overtopping flood.

Relief Bridge—An opening in an embankment on a floodplain to permit passage of overbank flow.

River Training Structure—Any configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank to deflect current, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimens of the stream.

Scupper—A device to drain water through the deck.

Sidewalk Width—Unobstructed space for exclusive pedestrian use between barriers or between a curb and a barrier.

Spring Tide—A tide of increased range that occurs about every two weeks when the moon is full or new.

Stable Channel—A condition that exists when a stream has a bed slope and cross-section that allows its channel to transport the water and sediment delivered from the upstream watershed without significant degradation, aggradation, or bank erosion.

Stream Geomorphology—The study of a stream and its floodplain with regard to its land forms, the general configuration of its surface, and the changes that take place due to erosion and the buildup of erosional debris.

Superelevation—A tilting of the roadway surface to partially counterbalance the centrifugal forces on vehicles on horizontal curves.

Superflood—Any flood or tidal flow with a flow rate greater than that of the 100-year flood but not greater than a 500-year flood.

Tide—The periodic rise and fall of the earth's oceans that results from the effect of the moon and sun acting on a rotating earth.

Watershed—An area confined by drainage divides, and often having only one outlet for discharge; the total drainage area contributing runoff to a single point.

Waterway—Any stream, river, pond, lake, or ocean.

Waterway Opening—Width or area of bridge opening at a specified stage, and measured normal to principal direction of flow.

2.3—LOCATION FEATURES

2.3.1—Route Location

2.3.1.1—General

The choice of location of bridges shall be supported by analyses of alternatives with consideration given to economic, engineering, social, and environmental concerns as well as costs of maintenance and inspection associated with the structures and with the relative importance of the above-noted concerns.

Attention, commensurate with the risk involved, shall be directed toward providing for favorable bridge locations that:

- fit the conditions created by the obstacle being crossed;
- facilitate practical cost-effective design, construction, operation, inspection, and maintenance;
- provide for the desired level of traffic service and safety; and
- minimize adverse highway impacts.

2.3.1.2—Waterway and Floodplain Crossings

Waterway crossings shall be located with regard to initial capital costs of construction and the optimization of total costs, including river channel training works and the maintenance measures necessary to reduce erosion. Studies of alternative crossing locations should include assessments of:

- the hydrologic and hydraulic characteristics of the waterway and its floodplain, including channel stability, flood history, and, in estuarine crossings, tidal ranges and cycles;
- the effect of the proposed bridge on flood flow patterns and the resulting scour potential at bridge foundations;
- the potential for creating new or augmenting existing flood hazards; and
- environmental impacts on the waterway and its floodplain.

Bridges and their approaches on floodplains should be located and designed with regard to the goals and objectives of floodplain management, including:

- prevention of uneconomic, hazardous, or incompatible use and development of floodplains;
- avoidance of significant transverse and longitudinal encroachments, where practicable;
- minimization of adverse highway impacts and mitigation of unavoidable impacts, where practicable;

C2.3.1.2

Detailed guidance on procedures for evaluating the location of bridges and their approaches on floodplains is contained in Federal Regulations and the Planning and Location Chapter of the *AASHTO Drainage Manual* (see Article C2.6.1). Engineers with knowledge and experience in applying the guidance and procedures in the *AASHTO Drainage Manual* should be involved in location decisions. It is generally safer and more cost effective to avoid hydraulic problems through the selection of favorable crossing locations than to attempt to minimize the problems at a later time in the project development process through design measures.

Experience at existing bridges should be part of the calibration or verification of hydraulic models, if possible. Evaluation of the performance of existing bridges during past floods is often helpful in selecting the type, size, and location of new bridges.

- consistency with the intent of the standards and criteria of the National Flood Insurance Program, where applicable;
- long-term aggradation or degradation; and
- commitments made to obtain environmental approvals.

2.3.2—Bridge Site Arrangement

2.3.2.1—General

The location and the alignment of the bridge should be selected to satisfy both on-bridge and under-bridge traffic requirements. Consideration should be given to possible future variations in alignment or width of the waterway, highway, or railway spanned by the bridge.

Where appropriate, consideration should be given to future addition of mass transit facilities or bridge widening.

2.3.2.2—Traffic Safety

2.3.2.2.1—Protection of Structures

Consideration shall be given to safe passage of vehicles on or under a bridge. The hazard to errant vehicles within the clear zone should be minimized by locating obstacles at a safe distance from the travel lanes.

Pier columns or walls for grade separation structures should be located in conformance with the clear zone concept as contained in Chapter 3 of the AASHTO *Roadside Design Guide*. Where the practical limits of structure costs, type of structure, volume and design speed of through traffic, span arrangement, skew, and terrain make conformance with the AASHTO *Roadside Design Guide* impractical, the pier or wall should be protected by the use of guardrail or other barrier devices. The guardrail or other device should, if practical, be independently supported, with its roadway face at least 2.0 ft from the face of the pier or abutment, unless a rigid barrier is provided.

The face of the guardrail or other device should be at least 2.0 ft outside the normal shoulder line.

2.3.2.2.2—Protection of Users

Railings shall be provided along the edges of structures conforming to the requirements of Section 13.

C2.3.2.1

Although the location of a bridge structure over a waterway is usually determined by other considerations than the hazards of vessel collision, the following preferences should be considered where possible and practical:

- Locating the bridge away from bends in the navigation channel. The distance to the bridge should be such that vessels can line up before passing the bridge, usually eight times the length of the vessel. This distance should be increased further where high currents and winds are prevalent at the site.
- Crossing the navigation channel near right angles and symmetrically with respect to the navigation channel.
- Providing an adequate distance from locations with congested navigation, vessel berthing maneuvers, or other navigation problems.
- Locating the bridge where the waterway is shallow or narrow and the bridge piers could be located out of vessel reach.

C2.3.2.2.1

The intent of providing structurally independent barriers is to prevent transmission of force effects from the barrier to the structure being protected.

C2.3.2.2.2

All protective structures shall have adequate surface features and transitions to safely redirect errant traffic.

In the case of movable bridges, warning signs, lights, signal bells, gates, barriers, and other safety devices shall be provided for the protection of pedestrian, bicycle, and vehicular traffic. These shall be designed to operate before the opening of the movable span and to remain operational until the span has been completely closed. The devices shall conform to the requirements for “Traffic Control at Movable Bridges,” in the *Manual on Uniform Traffic Control Devices* (MUTCD) or as shown on plans.

Where specified by the Owner, sidewalks shall be protected by barriers.

Protective structures include those that provide a safe and controlled separation of traffic on multimodal facilities using the same right-of-way.

Special conditions, such as curved alignment, impeded visibility, etc., may justify barrier protection, even with low design velocities.

2.3.2.2.3—Geometric Standards

Requirements of the AASHTO publication *A Policy on Geometric Design of Highways and Streets* shall either be satisfied or exceptions thereto shall be justified and documented. Width of shoulders and geometry of traffic barriers shall meet the specifications of the Owner.

2.3.2.2.4—Road Surfaces

Road surfaces on a bridge shall be given antiskid characteristics, crown, drainage, and superelevation in accordance with *A Policy on Geometric Design of Highways and Streets* or local requirements.

2.3.2.2.5—Vessel Collisions

Bridge structures shall either be protected against vessel collision forces by fenders, dikes, or dolphins as specified in Article 3.14.15, or shall be designed to withstand collision force effects as specified in Article 3.14.14.

C2.3.2.2.5

The need for dolphin and fender systems can be eliminated at some bridges by judicious placement of bridge piers. Guidance on use of dolphin and fender systems is included in the AASHTO *Highway Drainage Guidelines*, Chapter 7, “Hydraulic Analyses for the Location and Design of Bridges.”

2.3.3—Clearances

2.3.3.1—Navigational

Permits for construction of bridges over navigable waterways shall be obtained from the U.S. Coast Guard and/or other agencies having jurisdiction. Navigational clearances, both vertical and horizontal, shall be established in cooperation with the U.S. Coast Guard.

C2.3.3.1

Where bridge permits are required, early coordination should be initiated with the U.S. Coast Guard to evaluate the needs of navigation and the corresponding location and design requirements for the bridge.

Procedures for addressing navigational requirements for bridges, including coordination with the Coast Guard, are set forth in the Code of Federal Regulations, 23 CFR, Part 650, Subpart H, “Navigational Clearances for Bridges,” and 33 USC 401, 491, 511, et seq.

2.3.3.2—Highway Vertical

The vertical clearance of highway structures shall be in conformance with *A Policy on Geometric Design of Highways and Streets* for the functional classification of the highway or exceptions thereto shall be justified.

C2.3.3.2

The specified minimum clearance should include 6.0 in. for possible future overlays. If overlays are not contemplated by the Owner, this requirement may be nullified.



Possible reduction of vertical clearance, due to settlement of an overpass structure, shall be investigated. If the expected settlement exceeds 1.0 in., it shall be added to the specified clearance.

The vertical clearance to sign supports and pedestrian overpasses should be 1.0 ft greater than the highway structure clearance, and the vertical clearance from the roadway to the overhead cross bracing of through-truss structures should not be less than 17.5 ft.

2.3.3.3—Highway Horizontal

The bridge width shall not be less than that of the approach roadway section, including shoulders or curbs, gutters, and sidewalks.

Horizontal clearance under a bridge should meet the requirements of Article 2.3.2.2.1.

No object on or under a bridge, other than a barrier, should be located closer than 4.0 ft to the edge of a designated traffic lane. The inside face of a barrier should not be closer than 2.0 ft to either the face of the object or the edge of a designated traffic lane.

2.3.3.4—Railroad Overpass

Structures designed to pass over a railroad shall be in accordance with standards established and used by the affected railroad in its normal practice. These overpass structures shall comply with applicable federal, state, county, and municipal laws.

Regulations, codes, and standards should, as a minimum, meet the specifications and design standards of the American Railway Engineering and Maintenance of Way Association (AREMA), the Association of American Railroads, and AASHTO.

2.3.4—Environment

The impact of a bridge and its approaches on local communities, historic sites, wetlands, and other aesthetically, environmentally, and ecologically sensitive areas shall be considered. Compliance with state water laws; federal and state regulations concerning encroachment on floodplains, fish, and wildlife habitats; and the provisions of the National Flood Insurance Program shall be assured. Stream geomorphology, consequences of riverbed scour, removal of embankment stabilizing vegetation, and, where appropriate, impacts to estuarine tidal dynamics shall be considered.

Sign supports, pedestrian bridges, and overhead cross bracings require the higher clearance because of their lesser resistance to impact.

C2.3.3.3

The usable width of the shoulders should generally be taken as the paved width.

The specified minimum distances between the edge of the traffic lane and the fixed object are intended to prevent collision with slightly errant vehicles and those carrying wide loads.

C2.3.3.4

Attention is particularly called to the following chapters in the *Manual for Railway Engineering* (AREMA, 2003):

- Chapter 5—Track, Section 8—Highway/Railway Grade Crossings;
- Chapter 7—Timber Structures;
- Chapter 8—Concrete Structures and Foundations;
- Chapter 15—Steel Structures; and
- Chapter 28—Clearances.

The provisions of the individual railroads and the AREMA Manual should be used to determine:

- clearances,
- loadings,
- pier protection,
- waterproofing, and
- blast protection.

C2.3.4

Stream, i.e., fluvial, geomorphology is the study of the structure and formation of the earth's features that result from the forces of water. For purposes of this Section, this involves evaluating the streams, potential for aggradation, degradation, or lateral migration.

2.4—FOUNDATION INVESTIGATION

2.4.1—General

A subsurface investigation, including borings and soil tests, shall be conducted in accordance with the provisions of Article 10.4 to provide pertinent and sufficient information for the design of substructure units. The type and cost of foundations should be considered in the economic and aesthetic studies for location and bridge alternate selection.

2.4.2—Topographic Studies

Current topography of the bridge site shall be established via contour maps and photographs. Such studies shall include the history of the site in terms of movement of earth masses, soil and rock erosion, and meandering of waterways.

2.5—DESIGN OBJECTIVES

2.5.1—Safety

The primary responsibility of the Engineer shall be providing for the safety of the public. The Owner may require a design objective other than structural survival for an extreme event.

2.5.1.1—Structural Survival

The structure shall not collapse under the design event. The structure may undergo considerable displacement, settlement, or inelastic deformation. Elements of the structural system may be designated as sacrificial.

2.5.1.2—Limited Serviceability

The structure shall remain stable under designated emergency vehicular live loads.

2.5.1.3—Immediate Use

The structure may be reopened to all traffic after inspection following an extreme event. All load-carrying members of the structure should remain essentially elastic.

C2.5.1

Minimum requirements to ensure the structural safety of bridges as conveyances are included in these Specifications. The philosophy of achieving adequate structural safety is outlined in Article 1.3.

C2.5.1.1

The structural repairs may be extensive and require the structure to be replaced or out-of-service for an extended period of time.

C2.5.1.2

Limited displacement, limited plastic deformation in steel members, and spalling in concrete columns may all occur. Sacrificial members may need to be replaced. Structural repairs may be extensive.

C2.5.1.3

Minor spalling of concrete columns may occur.

2.5.2—Serviceability

2.5.2.1—Durability

2.5.2.1.1—Materials

The contract documents shall call for quality materials and for the application of high standards of fabrication and erection.

Structural steel shall be self-protecting, or have long-life coating systems or cathodic protection.

Reinforcing bars and prestressing strands in concrete components, which may be expected to be exposed to airborne or waterborne salts, shall be protected by an appropriate combination of epoxy and/or galvanized coating, concrete cover, density, or chemical composition of concrete, including air-entrainment and a nonporous painting of the concrete surface or cathodic protection.

Prestress strands in cable ducts shall be grouted or otherwise protected against corrosion.

Attachments and fasteners used in wood construction shall be of stainless steel, malleable iron, aluminum, or steel that is galvanized, cadmium-plated, or otherwise coated. Wood components shall be treated with preservatives.

Aluminum products shall be electrically insulated from steel and concrete components.

Protection shall be provided to materials susceptible to damage from solar radiation and/or air pollution.

Consideration shall be given to the durability of materials in direct contact with soil, water, or both.

2.5.2.1.2—Self-Protecting Measures

Continuous drip grooves shall be provided along the underside of a concrete deck at a distance not exceeding 10.0 in. from the fascia edges. Where the deck is interrupted by a sealed deck joint, all surfaces of piers and abutments, other than bearing seats, shall have a minimum slope of 5 percent toward their edges. For open deck joints, this minimum slope shall be increased to 15 percent. In the case of open deck joints, the bearings shall be protected against contact with salt and debris.

Wearing surfaces shall be interrupted at the deck joints and shall be provided with a smooth transition to the deck joint device.

Steel formwork shall be protected against corrosion in accordance with the specifications of the Owner.

C2.5.2.1.1

The intent of this Article is to recognize the significance of corrosion and deterioration of structural materials to the long-term performance of a bridge. Other provisions regarding durability can be found in Article 5.14.

Other than the deterioration of the concrete deck itself, the single most prevalent bridge maintenance problem is the disintegration of beam ends, bearings, pedestals, piers, and abutments due to percolation of waterborne road salts through the deck joints. Experience appears to indicate that a structurally continuous deck provides the best protection for components below the deck. The potential consequences of the use of road salts on structures with unfilled steel decks and unprestressed wood decks should be taken into account.

These Specifications permit the use of discontinuous decks in the absence of substantial use of road salts. Transverse saw-cut relief joints in cast-in-place concrete decks have been found to be of no practical value where composite action is present. Economy, due to structural continuity and the absence of expansion joints, will usually favor the application of continuous decks, regardless of location.

Stringers made simply supported by sliding joints, with or without slotted bolt holes, tend to “freeze” due to the accumulation of corrosion products and cause maintenance problems. Because of the general availability of computers, analysis of continuous decks is no longer a problem.

Experience indicates that, from the perspective of durability, all joints should be considered subject to some degree of movement and leakage.

C2.5.2.1.2

Ponding of water has often been observed on the seats of abutments, probably as a result of construction tolerances and/or tilting. The 15 percent slope specified in conjunction with open joints is intended to enable rains to wash away debris and salt.

In the past, for many smaller bridges, no expansion device was provided at the “fixed joint,” and the wearing surface was simply run over the joint to give a continuous riding surface. As the rotation center of the superstructure is always below the surface, the “fixed joint” actually moves due to load and environmental effects, causing the wearing surface to crack, leak, and disintegrate.

2.5.2.2—Inspectability

Inspection ladders, walkways, catwalks, covered access holes, and provision for lighting, if necessary, shall be provided where other means of inspection are not practical.

Where practical, access to permit manual or visual inspection, including adequate headroom in box sections, shall be provided to the inside of cellular components and to interface areas, where relative movement may occur.

2.5.2.3—Maintainability

Structural systems whose maintenance is expected to be difficult should be avoided. Where the climatic and/or traffic environment is such that a bridge deck may need to be replaced before the required service life, provisions shall be shown on the contract documents for:

- a contemporary or future protective overlay,
- a future deck replacement, or
- supplemental structural resistance.

Areas around bearing seats and under deck joints should be designed to facilitate jacking, cleaning, repair, and replacement of bearings and joints.

Jacking points shall be indicated on the plans, and the structure shall be designed for jacking forces specified in Article 3.4.3. Inaccessible cavities and corners should be avoided. Cavities that may invite human or animal inhabitants shall either be avoided or made secure.

2.5.2.4—Rideability

The deck of the bridge shall be designed to permit the smooth movement of traffic. On paved roads, a structural transition slab should be located between the approach roadway and the abutment of the bridge. Construction tolerances, with regard to the profile of the finished deck, shall be indicated on the plans or in the specifications or special provisions.

The number of deck joints shall be kept to a practical minimum. Edges of joints in concrete decks exposed to traffic should be protected from abrasion and spalling. The plans for prefabricated joints shall specify that the joint assembly be erected as a unit.

Where concrete decks without an initial overlay are used, consideration should be given to providing an additional thickness of 0.5 in. to permit correction of the deck profile by grinding, and to compensate for thickness loss due to abrasion.

2.5.2.5—Utilities

Where required, provisions shall be made to support and maintain the conveyance for utilities.

C2.5.2.2

The *Guide Specifications for Design and Construction of Segmental Concrete Bridges* requires external access hatches with a minimum size of 2.5 ft × 4.0 ft, larger openings at interior diaphragms, and venting by drains or screened vents at intervals of no more than 50.0 ft. These recommendations should be used in bridges designed under these Specifications.

C2.5.2.3

Maintenance of traffic during replacement should be provided either by partial width staging of replacement or by the utilization of an adjacent parallel structure.

Measures for increasing the durability of concrete and wood decks include epoxy coating of reinforcing bars, post-tensioning ducts, and prestressing strands in the deck. Microsilica and/or calcium nitrite additives in the deck concrete, waterproofing membranes, and overlays may be used to protect black steel. See Article 5.14.5 for additional requirements regarding overlays.

2.5.2.6—Deformations

2.5.2.6.1—General

Bridges should be designed to avoid undesirable structural or psychological effects due to their deformations. While deflection and depth limitations are made optional, except for orthotropic plate decks, any large deviation from past successful practice regarding slenderness and deflections should be cause for review of the design to determine that it will perform adequately.

If dynamic analysis is used, it shall comply with the principles and requirements of Article 4.7.

For straight skewed steel girder bridges and horizontally curved steel girder bridges with or without skewed supports, the following additional investigations shall be considered:

- Elastic vertical, lateral, and rotational deflections due to applicable load combinations shall be considered to ensure satisfactory service performance of bearings, joints, integral abutments, and piers.

C2.5.2.6.1

Service load deformations may cause deterioration of wearing surfaces and local cracking in concrete slabs and in metal bridges that could impair serviceability and durability, even if self-limiting and not a potential source of collapse.

As early as 1905, attempts were made to avoid these effects by limiting the depth-to-span ratios of trusses and girders, and starting in the 1930s, live load deflection limits were prescribed for the same purpose. In a study of deflection limitations of bridges (ASCE, 1958), an ASCE committee found numerous shortcomings in these traditional approaches and noted, for example:

The limited survey conducted by the Committee revealed no evidence of serious structural damage that could be attributed to excessive deflection. The few examples of damaged stringer connections or cracked concrete floors could probably be corrected more effectively by changes in design than by more restrictive limitations on deflection. On the other hand, both the historical study and the results from the survey indicate clearly that unfavorable psychological reaction to bridge deflection is probably the most frequent and important source of concern regarding the flexibility of bridges. However, those characteristics of bridge vibration which are considered objectionable by pedestrians or passengers in vehicles cannot yet be defined.

Since publication of the study, there has been extensive research on human response to motion. It is now generally agreed that the primary factor affecting human sensitivity is acceleration rather than deflection, velocity, or the rate of change of acceleration for bridge structures, but the problem is a difficult subjective one. Thus, there are as yet no simple definitive guidelines for the limits of tolerable static deflection or dynamic motion. Among current specifications, the *Canadian Highway Bridge Design Code* (2014) contains comprehensive provisions regarding vibrations tolerable to humans.

Horizontally-curved steel bridges are subjected to torsion resulting in larger lateral deflections and twisting than tangent bridges. Therefore, rotations due to dead load and thermal forces tend to have a larger effect on the performance of bearings and expansion joints of curved bridges.

Bearing rotations during construction may exceed the dead load rotations computed for the completed bridge, in particular at skewed supports. Identification of this temporary situation may be critical to ensure the bridge can be built without damaging the bearings or expansion devices.

- Computed girder rotations at bearings should be accumulated over the Engineer's assumed construction sequence. Computed rotations at bearings shall not exceed the specified rotational capacity of the bearings for the accumulated factored loads corresponding to the stage investigated.
- Camber diagrams shall satisfy the provisions of Article 6.7.2 and may reflect the computed accumulated deflections due to the Engineer's assumed construction sequence.

2.5.2.6.2—Criteria for Deflection

The criteria in this Article shall be considered optional, except for the following:

- The provisions for orthotropic decks shall be considered mandatory.
- The provisions in Article 12.14.5.9 for precast reinforced concrete three-sided structures shall be considered mandatory.
- Metal grid decks and other lightweight metal and concrete bridge decks shall be subject to the serviceability provisions of Article 9.5.2.

In applying these criteria, the vehicular load shall include the dynamic load allowance.

If an Owner chooses to invoke deflection control, the following principles may be applied:

- When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally;
- For curved steel box and I-girder systems, the deflection of each girder should be determined individually based on its response as part of a system;
- For composite design, the stiffness of the design cross-section used for the determination of deflection and frequency should include the entire width of the roadway and the structurally continuous portions of the railings, sidewalks, and median barriers;
- For straight girder systems, the composite bending stiffness of an individual girder may be taken as the stiffness determined as specified above, divided by the number of girders;
- When investigating maximum relative displacements, the number and position of loaded lanes should be selected to provide the worst differential effect;

C2.5.2.6.2

These provisions permit, but do not encourage, the use of past practice for deflection control. Designers were permitted to exceed these limits at their discretion in the past. Calculated deflections of structures have often been found to be difficult to verify in the field due to numerous sources of stiffness not accounted for in calculations. Despite this, many Owners and designers have found comfort in the past requirements to limit the overall stiffness of bridges. The desire for continued availability of some guidance in this area, often stated during the development of these Specifications, has resulted in the retention of optional criteria, except for orthotropic decks, for which the criteria are required. Deflection criteria are also mandatory for lightweight decks comprised of metal and concrete, such as filled and partially filled grid decks, and unfilled grid decks composite with reinforced concrete slabs, as provided in Article 9.5.2.

Additional guidance regarding deflection of steel bridges can be found in Wright and Walker (1971).

Additional considerations and recommendations for deflection in timber bridge components are discussed in more detail in Chapters 7, 8, and 9 in Ritter (1990).

For a straight girder system bridge, this is equivalent to saying that the distribution factor for deflection is equal to the number of lanes divided by the number of beams.

For curved steel girder systems, the deflection limit is applied to each individual girder because the curvature causes each girder to deflect differently than the adjacent girder so that an average deflection has little meaning. For curved steel girder systems, the span used to compute the deflection limit should be taken as the arc girder length between bearings.

- The live load portion of Load Combination Service I of Table 3.4.1-1 should be used, including the dynamic load allowance, IM;
- The live load shall be taken from Article 3.6.1.3.2;
- The provisions of Article 3.6.1.1.2 should apply; and
- For skewed bridges, a right cross-section may be used, and for curved and curved skewed bridges, a radial cross-section may be used.

In the absence of other criteria, the following deflection limits may be considered for steel, aluminum, and/or concrete vehicular bridges:

- Vehicular load, general Span/800,
- Vehicular and pedestrian loads Span/1,000,
- Vehicular load on cantilever arms Span/300, and
- Vehicular and pedestrian loads on cantilever arms Span/375.

For steel I-shaped beams and girders, and for steel box and tub girders, the provisions of Articles 6.10.4.2 and 6.11.4, respectively, regarding the control of permanent deflections through flange stress controls, shall apply. For pedestrian bridges, i.e., bridges whose primary function is to carry pedestrians, bicyclists, equestrians, and light maintenance vehicles, the provisions of Section 5 of AASHTO's *LRFD Guide Specifications for the Design of Pedestrian Bridges* shall apply.

In the absence of other criteria, the following deflection limits may be considered for wood construction:

- Vehicular and pedestrian loads Span/425, and
- Vehicular load on wood planks and panels (extreme relative deflection between adjacent edges) .. 0.10 in.

The following provisions shall apply to orthotropic plate decks:

- Vehicular load on deck plate Span/300,
- Vehicular load on ribs of orthotropic metal decks Span/1000, and
- Vehicular load on ribs of orthotropic metal decks (extreme relative deflection between adjacent ribs) 0.10 in.

Other criteria may include recognized deflection-frequency-perception requirements such as that specified in the *Canadian Highway Bridge Design Code* (CSA, 2006). Application of the CSA criteria is discussed in Kulicki et al. (2015), including statistical data for live load based on WIM data, a load factor for the HL-93 live load, and a target reliability index.

From a structural viewpoint, large deflections in wood components cause fasteners to loosen and brittle materials, such as asphalt pavement, to crack and break. In addition, members that sag below a level plane present a poor appearance and can give the public a perception of structural inadequacy. Deflections from moving vehicle loads also produce vertical movement and vibrations that annoy motorists and alarm pedestrians (Ritter, 1990).

Excessive deformation can cause premature deterioration of the wearing surface and affect the performance of fasteners, but limits on the latter have not yet been established.

The intent of the relative deflection criterion is to protect the wearing surface from debonding and fracturing due to excessive flexing of the deck.

The 0.10-in. relative deflection limitation is tentative.

2.5.2.6.3—Optional Criteria for Span-to-Depth Ratios

Unless otherwise specified herein, if an Owner chooses to invoke controls on span-to-depth ratios, the limits in Table 2.5.2.6.3-1, in which S is the slab span length and L is the span length, both in feet, may be considered in the absence of other criteria. Where used, the limits in Table 2.5.2.6.3-1 shall be taken to apply to overall depth unless noted.

For curved steel girder systems, the span-to-depth ratio, L_{as}/D , of each steel girder should not exceed 25 when the specified minimum yield strength of the girder in regions of positive flexure is 50.0 ksi or less, and:

- When the specified minimum yield strength of the girder is 70.0 ksi or less in regions of negative flexure, or
- When hybrid sections satisfying the provisions of Article 6.10.1.3 are used in regions of negative flexure.

For all other curved steel girder systems, L_{as}/D of each steel girder should not exceed the following:

$$\frac{L_{as}}{D} \leq 25 \sqrt{\frac{50}{F_{yt}}} \quad (2.5.2.6.3-1)$$

where:

F_{yt} = specified minimum yield strength of the compression flange (ksi)

D = depth of steel girder (ft)

L_{as} = an arc girder length defined as follows (ft):

- arc span for simple spans;
- 0.9 times the arc span for continuous end-spans;
- 0.8 times the arc span for continuous interior spans.

C2.5.2.6.3

Traditional minimum depths for constant depth superstructures, contained in previous editions of the *AASHTO Standard Specifications for Highway Bridges*, are given in Table 2.5.2.6.3-1 with some modifications.

A larger preferred minimum girder depth is specified for curved steel girders to reflect the fact that the outermost curved girder receives a disproportionate share of the load and needs to be stiffer. In curved skewed bridges, cross-frame forces are directly related to the relative girder deflections. Increasing the depth and stiffness of all the girders in a curved skewed bridge leads to smaller relative differences in the deflections and smaller cross-frame forces. Deeper girders also result in reduced out-of-plane rotations, which may make the bridge easier to erect.

An increase in the preferred minimum girder depth for curved steel girders not satisfying the conditions specified herein is recommended according to Eq. 2.5.2.6.3-1. In such cases, the girders will tend to be significantly more flexible and less steel causes increased deflections without an increase in the girder depth.

A shallower curved girder might be used if the Engineer evaluates effects such as cross-frame forces and bridge deformations, including girder rotations, and finds the bridge forces and geometric changes within acceptable ranges. For curved composite girders, the recommended ratios apply to the steel girder portion of the composite section.

Table 2.5.2.6.3-1—Traditional Minimum Depths for Constant Depth Superstructures

Superstructure		Minimum Depth (Including Deck)	
		When variable depth members are used, values may be adjusted to account for changes in relative stiffness of positive and negative moment sections	
Material	Type	Simple Spans	Continuous Spans
Reinforced Concrete	Slabs with Main Reinforcement Parallel to Traffic	$\frac{1.2(S+10)}{30}$	$\frac{S+10}{30} \geq 0.54 \text{ ft}$
	T-Beams	$0.070L$	$0.065L$
	Box Beams	$0.060L$	$0.055L$
	Pedestrian Structure Beams	$0.035L$	$0.033L$
Prestressed Concrete	Slabs	$0.030L \geq 6.5 \text{ in.}$	$0.027L \geq 6.5 \text{ in.}$
	CIP Box Beams	$0.045L$	$0.040L$
	Precast I-Beams	$0.045L$	$0.040L$
	Pedestrian Structure Beams	$0.033L$	$0.030L$
	Adjacent Box Beams	$0.030L$	$0.025L$
Steel	Overall Depth of Composite I-Beam	$0.040L$	$0.032L$
	Depth of I-Beam Portion of Composite I-Beam	$0.033L$	$0.027L$
	Trusses	$0.100L$	$0.100L$

2.5.2.7—Consideration of Future Widening*2.5.2.7.1—Exterior Beams on Girder System Bridges**C2.5.2.7.1*

Unless future widening is virtually inconceivable, the load-carrying capacity of exterior beams shall not be less than the load-carrying capacity of an interior beam.

This provision applies to any longitudinal flexural members traditionally considered to be stringers, beams, or girders.

2.5.2.7.2—Substructure

When future widening can be anticipated, consideration should be given to designing the substructure for the widened condition.

2.5.3—Constructability**C2.5.3**

Constructability issues should include, but not be limited to, consideration of deflection, strength of steel and concrete, and stability during critical stages of construction.

An example of a particular sequence of construction would be where the designer requires a steel girder to be supported while the concrete deck is cast, so that the girder and the deck will act compositely for dead load as well as live load.

Bridges should be designed in a manner such that fabrication and erection can be performed without undue difficulty or distress and that locked-in construction force effects are within tolerable limits.

When the designer has assumed a particular sequence of construction in order to induce certain stresses under dead load, that sequence shall be defined in the contract documents.

Where there are, or are likely to be, constraints imposed on the method of construction, by environmental

considerations or for other reasons, attention shall be drawn to those constraints in the contract documents.

Where the bridge is of unusual complexity, such that it would be unreasonable to expect an experienced contractor to predict and estimate a suitable method of construction while bidding the project, at least one feasible construction method shall be indicated in the contract documents.

If the design requires some strengthening and/or temporary bracing or support during erection by the selected method, indication of the need thereof shall be indicated in the contract documents.

Details that require welding in restricted areas or placement of concrete through congested reinforcing should be avoided.

Climatic and hydraulic conditions that may affect the construction of the bridge shall be considered.

2.5.4—Economy

2.5.4.1—General

Structural types, span lengths, and materials shall be selected with due consideration of projected cost. The cost of future expenditures during the projected service life of the bridge should be considered. Regional factors, such as availability of material, fabrication, location, shipping, and erection constraints, shall be considered.

2.5.4.2—Alternative Plans

In instances where economic studies do not indicate a clear choice, the Owner may require that alternative contract plans be prepared and bid competitively. Designs for alternative plans shall be of equal safety, serviceability, and aesthetic value.

Movable bridges over navigable waterways should be avoided to the extent feasible. Where movable bridges are proposed, at least one fixed bridge alternative should be included in the economic comparisons.

2.5.5—Bridge Aesthetics

Bridges should complement their surroundings, be graceful in form, and present an appearance of adequate strength.

An example of a complex bridge might be a cable-stayed bridge that has limitations on what it will carry, especially in terms of construction equipment, while it is under construction. If these limitations are not evident to an experienced contractor, the contractor may be required to do more prebid analysis than is reasonable. Given the usual constraints of time and budget for bidding, this may not be feasible for the contractor to do.

This Article does not require the designer to educate a contractor on how to construct a bridge; it is expected that the contractor will have the necessary expertise. Nor is it intended to restrict a contractor from using innovation to gain an edge over the competitors.

All other factors being equal, designs that are self-supporting or use standardized falsework systems are normally preferred to those requiring unique and complex falsework.

Temporary falsework within the clear zone should be adequately protected from traffic.

C2.5.4.1

If data for the trends in labor and material cost fluctuation are available, the effect of such trends should be projected to the time the bridge will likely be constructed.

Cost comparisons of structural alternatives should be based on long-range considerations, including inspection, maintenance, repair, and/or replacement. Lowest first cost does not necessarily lead to lowest total cost.

C2.5.5

Significant improvements in appearance can often be made with small changes in shape or position of structural members at negligible cost. For prominent bridges, however, additional cost to achieve improved appearance is often justified, considering that the bridge will likely be a feature of the landscape for 75 or more years.

Comprehensive guidelines for the appearance of bridges are beyond the scope of these Specifications.



Engineers should seek more pleasant appearance by improving the shapes and relationships of the structural components themselves. The application of extraordinary and nonstructural embellishment should be avoided.

The following guidelines should be considered:

- Alternative bridge designs without piers or with few piers should be studied during the site selection and location stage and refined during the preliminary design stage.
- Pier form should be consistent in shape and detail with the superstructure.
- Abrupt changes in the form of components and structural type should be avoided. Where the interface of different structural types cannot be avoided, a smooth transition in appearance from one type to another should be attained.
- Attention to details, such as deck drain downspouts, should not be overlooked.
- If the use of a through structure is dictated by performance and/or economic considerations, the structural system should be selected to provide an open and uncluttered appearance.
- The use of the bridge as a support for message or directional signing or lighting should be avoided wherever possible.
- Transverse web stiffeners, other than those located at bearing points, should not be visible in elevation.
- For spanning deep ravines, arch-type structures should be preferred.

Engineers may refer to such documents as the Transportation Research Board's *Bridge Aesthetics Around the World* (Gottemoeller, 1991) for guidance.

The most admired modern structures are those that rely for their good appearance on the forms of the structural component themselves:

- Components are shaped to respond to the structural function. They are thick where the stresses are greatest and thin where the stresses are smaller.
- The function of each part and how the function is performed is visible.
- Components are slender and widely spaced, preserving views through the structure.
- The bridge is seen as a single whole, with all members consistent and contributing to that whole; for example, all elements should come from the same family of shapes, such as shapes with rounded edges.
- The bridge fulfills its function with a minimum of material and minimum number of elements.
- The size of each member compared with the others is clearly related to the overall structural concept and the job the component does, and
- The bridge as a whole has a clear and logical relationship to its surroundings.

Several procedures have been proposed to integrate aesthetic thinking into the design process (Gottemoeller, 1991).

Because the major structural components are the largest parts of a bridge and are seen first, they determine the appearance of a bridge. Consequently, engineers should seek excellent appearance in bridge parts in the following order of importance:

- Horizontal and vertical alignment and position in the environment;
- Superstructure type, i.e., arch, girder, etc.;
- Pier placement;
- Abutment placement and height;
- Superstructure shape, i.e., haunched, tapered, depth;
- Pier shape;
- Abutment shape;
- Parapet and railing details;
- Surface colors and textures; and
- Ornament.

The Designer should determine the likely position of the majority of viewers of the bridge, then use that information as a guide in judging the importance of various elements in the appearance of the structure.

Perspective drawings of photographs taken from the important viewpoints can be used to analyze the appearance of proposed structures. Models are also useful.

The appearance of standard details should be reviewed to make sure they fit the bridge's design concept.



2.6—HYDROLOGY AND HYDRAULICS

2.6.1—General

Hydrologic and hydraulic studies and assessments of bridge sites for stream crossings shall be completed as part of the preliminary plan development. The detail of these studies should be commensurate with the importance of and risks associated with the structure.

Temporary structures for the Contractor's use or for accommodating traffic during construction shall be designed with regard to the safety of the traveling public and the adjacent property owners, as well as minimization of impact on floodplain natural resources. The Owner may permit revised design requirements consistent with the intended service period for, and flood hazard posed by, the temporary structure. Contract documents for temporary structures shall delineate the respective responsibilities and risks to be assumed by the highway agency and the Contractor.

Evaluation of bridge design alternatives shall consider stream stability, backwater, flow distribution, stream velocities, scour potential, flood hazards, tidal dynamics where appropriate, and consistency with established criteria for the National Flood Insurance Program.

C2.6.1

The provisions in this Article incorporate improved practices and procedures for the hydraulic design of bridges. Detailed guidance for applying these practices and procedures is contained in the *AASHTO Drainage Manual*. This document contains guidance and references on design procedures and computer software for hydrologic and hydraulic design. It also incorporates guidance and references from the *AASHTO Highway Drainage Guidelines*, which is a companion document to the *AASHTO Drainage Manual*.

Information on the National Flood Insurance Program is contained in 42 USC 4001-4128, the *National Flood Insurance Act* (see also 44 CFR 59 through 77, *National Flood Insurance Program*) and 23 CFR 650, Subpart A, *Location and Hydraulic Design of Encroachment on Floodplains*.

Hydrologic, hydraulic, scour, and stream stability studies are concerned with the prediction of flood flows and frequencies and with the complex physical processes involving the actions and interactions of water and soil during the occurrence of predicted flood flows. These studies should be performed by an Engineer with the knowledge and experience to make practical judgments regarding the scope of the studies to be performed and the significance of the results obtained. The design of bridge foundations is best accomplished by an interdisciplinary team of structural, hydraulic, and geotechnical engineers.

The *AASHTO Drainage Manual* also contains guidance and references on:

- Design methods for evaluating the accuracy of hydraulic studies, including elements of a data collection plan;
- Guidance on estimating flood flow peaks and volumes, including requirements for the design of Interstate highways as per 23 CFR 650, Subpart A, "Encroachments;"
- Procedures or references for analysis of tidal waterways, regulated streams, and urban watersheds;
- Evaluation of stream stability;
- Use of recommended design procedures and software for sizing bridge waterways;
- Location and design of bridges to resist damage from scour and hydraulic loads created by stream current, ice, and debris;
- Calculation of magnitude of contraction scour, local scour, and countermeasures thereto;
- Design of relief bridges, road overtopping, guide banks, and other river training works; and
- Procedures for hydraulic design of bridge-size culverts.

2.6.2—Site Data

A site-specific data collection plan shall include consideration of:

- Collection of aerial and/or ground survey data for appropriate distances upstream and downstream from the bridge for the main stream channel and its floodplain;
- Estimation of roughness elements for the stream and the floodplain within the reach of the stream under study;
- Sampling of streambed material to a depth sufficient to ascertain material characteristics for scour analysis;
- Subsurface borings;
- Factors affecting water stages, including high water from streams, reservoirs, detention basins, tides, and flood control structures and operating procedures;
- Existing studies and reports, including those conducted in accordance with the provisions of the National Flood Insurance Program or other flood control programs;
- Available historical information on the behavior of the stream and the performance of the structure during past floods, including observed scour, bank erosion, and structural damage due to debris or ice flows; and
- Possible geomorphic changes in channel flow.

2.6.3—Hydrologic Analysis

The Owner shall determine the extent of hydrologic studies on the basis of the functional highway classification, the applicable federal and state requirements, and the flood hazards at the site.

The following flood flows should be investigated, as appropriate, in the hydrologic studies:

- For assessing flood hazards and meeting floodplain management requirements—the 100-year flood;
- For assessing risks to highway users and damage to the bridge and its roadway approaches—the overtopping flood and/or the design flood for bridge scour;
- For assessing catastrophic flood damage at high risk sites—a check flood of a magnitude selected by the Owner, as appropriate for the site conditions and the perceived risk;
- For investigating the adequacy of bridge foundations to resist scour—the check flood for bridge scour;
- To satisfy agency design policies and criteria—design floods for waterway opening and bridge scour for the various functional classes of highways;
- To calibrate water surface profiles and to evaluate the performance of existing structures—historical floods; and
- To evaluate environmental conditions—low or base flow information, and in estuarine crossings, the spring and tide range.

C2.6.2

The assessment of hydraulics necessarily involves many assumptions. Key among these assumptions are the roughness coefficients and projection of long-term flow magnitudes, e.g., the 500-year flood or other superfloods. The runoff from a given storm can be expected to change with the seasons, immediate past weather conditions, and long-term natural and man-made changes in surface conditions. The ability to statistically project long recurrence interval floods is a function of the adequacy of the database of past floods, and such projections often change as a result of new experience.

The above factors make the check flood investigation of scour an important, but highly variable, safety criterion that may be expected to be difficult to reproduce, unless all of the Designer's original assumptions are used in a post-design scour investigation. Obviously, those original assumptions must be reasonable given the data, conditions, and projections available at the time of the original design.

C2.6.3

The return period of tidal flows should be correlated to the hurricane or storm tide elevations of water as reported in studies by FEMA or other agencies.

Particular attention should be given to selecting design and checking flood discharges for mixed population flood events. For example, flow in an estuary may consist of both tidal flow and runoff from the upland watershed.

If mixed population flows are dependent on the occurrence of a major meteorological event, such as a hurricane, the relative timing of the individual peak flow events needs to be evaluated and considered in selecting the design discharge. This is likely to be the case for flows in an estuary.

If the events tend to be independent, as might be the case for floods in a mountainous region caused by rainfall runoff or snow melt, the Designer should evaluate both events independently and then consider the probability of their occurrence at the same time.

Investigation of the effect of sea level rise on tidal ranges should be specified for structures spanning marine/estuarine resources.

2.6.4—Hydraulic Analysis

2.6.4.1—General

The Engineer shall utilize analytical models and techniques that have been approved by the Owner and that are consistent with the required level of analysis.

2.6.4.2—Stream Stability

Studies shall be carried out to evaluate the stability of the waterway and to assess the impact of construction on the waterway. The following items shall be considered:

- Whether the stream reach is degrading, aggrading, or in equilibrium;
- For stream crossing near confluences, the effect of the main stream and the tributary on the flood stages, velocities, flow distribution, vertical and lateral movements of the stream, and the effect of the foregoing conditions on the hydraulic design of the bridge;
- Location of favorable stream crossing, taking into account whether the stream is straight, meandering, braided, or transitional, or control devices to protect the bridge from existing or anticipated future stream conditions;
- The effect of any proposed channel changes;
- The effect of aggregate mining or other operations in the channel;
- Potential changes in the rates or volumes of runoff due to land use changes;
- The effect of natural geomorphic stream pattern changes on the proposed structure; and
- The effect of geomorphic changes on existing structures in the vicinity of, and caused by, the proposed structure.

For unstable streams or flow conditions, special studies shall be carried out to assess the probable future changes to the plan form and profile of the stream and to determine countermeasures to be incorporated in the design, or at a future time, for the safety of the bridge and approach roadways.

2.6.4.3—Bridge Waterway

The design process for sizing the bridge waterway shall include:

- The evaluation of flood flow patterns in the main channel and floodplain for existing conditions, and
- The evaluation of trial combinations of highway profiles, alignments, and bridge lengths for consistency with design objectives.

Where use is made of existing flood studies, their accuracy shall be determined.

2.6.4.4—Bridge Foundations

2.6.4.4.1—General

The structural, hydraulic, and geotechnical aspects of foundation design shall be coordinated and differences resolved prior to approval of preliminary plans.

C2.6.4.3

Trial combinations should take the following into account:

- Increases in flood water surface elevations caused by the bridge,
- Changes in flood flow patterns and velocities in the channel and on the floodplain,
- Location of hydraulic controls affecting flow through the structure or long-term stream stability,
- Clearances between the flood water elevations and low sections of the superstructure to allow passage of ice and debris,
- Need for protection of bridge foundations and stream channel bed and banks, and
- Evaluation of capital costs and flood hazards associated with the candidate bridge alternatives through risk assessment or risk analysis procedures.

C2.6.4.4.1

To reduce the vulnerability of the bridge to damage from scour and hydraulic loads, consideration should be given to the following general design concepts:

- Set deck elevations as high as practical for the given site conditions to minimize inundation by floods. Where bridges are subject to inundation, provide for overtopping of roadway approach sections, and streamline the superstructure to minimize the area subject to hydraulic loads and the collection of ice, debris, and drifts.
- Utilize relief bridges, guide banks, dikes, and other river training devices to reduce the turbulence and hydraulic forces acting at the bridge abutments.
- Utilize continuous-span designs. Anchor superstructures to their substructures where subject to the effects of hydraulic loads, buoyancy, ice, or debris impacts or accumulations. Provide for venting and draining of the superstructure.
- Where practical, limit the number of piers in the channel, streamline pier shapes, and align piers with the direction of flood flows. Avoid pier types that collect ice and debris. Locate piers beyond the immediate vicinity of stream banks.
- Locate abutments back from the channel banks where significant problems with ice/debris buildup, scour, or channel stability are anticipated, or where special environmental or regulatory needs must be met, e.g., spanning wetlands.
- Design piers on floodplains as river piers. Locate their foundations at the appropriate depth if there is a likelihood that the stream channel will shift during the life of the structure or that channel cutoffs are likely to occur.

2.6.4.4.2—Bridge Scour

As required by Article 3.7.5, scour at bridge foundations is investigated for two conditions:

- For the design flood for scour, the streambed material in the scour prism above the total scour line shall be assumed to have been removed for design conditions. The design flood storm surge, tide, or mixed population flood shall be the more severe of the 100-year events or an overtopping flood of lesser recurrence interval.
- For the check flood for scour, the stability of the bridge foundation shall be investigated for scour conditions resulting from a designated flood storm surge, tide, or mixed population flood not to exceed the 500-year event or from an overtopping flood of lesser recurrence interval. Excess reserve beyond that required for stability under this condition is not necessary. The extreme event limit state shall apply.

If the site conditions, due to ice or debris jams, and low tail water conditions near stream confluences dictate the use of a more severe flood event for either the design or check flood for scour, the Engineer may use such flood event.

Spread footings on soil or erodible rock shall be located so that the bottom of footing is below scour depths determined for the check flood for scour. Spread footings on scour-resistant rock shall be designed and constructed to maintain the integrity of the supporting rock.

Deep foundations with footings shall be designed to place the top of the footing below the estimated contraction scour depth where practical to minimize obstruction to flood flows and resulting local scour. Even lower elevations should be considered for pile-supported footings where the piles could be damaged by erosion and corrosion from exposure to stream currents. Where conditions dictate a need to construct the top of a footing to an elevation above the streambed, attention shall be given to the scour potential of the design.

When fendering or other pier protection systems are used, their effect on pier scour and collection of debris shall be taken into consideration in the design.

- Where practical, use debris racks or ice booms to stop debris and ice before it reaches the bridge. Where significant ice or debris buildup is unavoidable, its effects should be accounted for in determining scour depths and hydraulic loads.

C2.6.4.4.2

A majority of bridge failures in the United States and elsewhere are the result of scour.

The added cost of making a bridge less vulnerable to damage from scour is small in comparison to the total cost of a bridge failure.

The design flood for scour shall be determined on the basis of the Engineer's judgment of the hydrologic and hydraulic flow conditions at the site. The recommended procedure is to evaluate scour due to the specified flood flows and to design the foundation for the event expected to cause the deepest total scour.

The recommended procedure for determining the total scour depth at bridge foundations is as follows:

- Estimate the long-term channel profile aggradation or degradation over the service life of the bridge;
- Estimate the long-term channel plan form changes over the service life of the bridge;
- As a design check, adjust the existing channel and floodplain cross-sections upstream and downstream of bridge as necessary to reflect anticipated changes in the channel profile and plan form;
- Determine the combination of existing or likely future conditions and flood events that might be expected to result in the deepest scour for design conditions;
- Determine water surface profiles for a stream reach that extends both upstream and downstream of the bridge site for the various combinations of conditions and events under consideration;
- Determine the magnitude of contraction scour and local scour at piers and abutments; and
- Evaluate the results of the scour analysis, taking into account the variables in the methods used, the available information on the behavior of the watercourse, and the performance of existing structures during past floods. Also consider present and anticipated future flow patterns in the channel and its floodplain. Visualize the effect of the bridge on these flow patterns and the effect of the flow on the bridge. Modify the bridge design where necessary to satisfy concerns raised by the scour analysis and the evaluation of the channel plan form.

Foundation designs should be based on the total scour depths estimated by the above procedure, taking into account appropriate geotechnical safety factors. Where necessary, bridge modifications may include:

The stability of abutments in areas of turbulent flow shall be thoroughly investigated. Exposed embankment slopes should be protected with appropriate scour countermeasures.

2.6.4.5—Roadway Approaches to Bridge

The design of the bridge shall be coordinated with the design of the roadway approaches to the bridge on the floodplain so that the entire flood flow pattern is developed and analyzed as a single, interrelated entity. Where roadway approaches on the floodplain obstruct overbank flow, the highway segment within the floodplain limits shall be designed to minimize flood hazards.

Where diversion of flow to another watershed occurs as a result of backwater and obstruction of flood flows, an evaluation of the design shall be carried out to ensure compliance with legal requirements in regard to flood hazards in the other watershed.

- relocation or redesign of piers or abutments to avoid areas of deep scour or overlapping scour holes from adjacent foundation elements,
- addition of guide banks, dikes, or other river training works to provide for smoother flow transitions or to control lateral movement of the channel,
- enlargement of the waterway area, or
- relocation of the crossing to avoid an undesirable location.

Foundations should be designed to withstand the conditions of scour for the design flood and the check flood. In general, this will result in deep foundations. The design of the foundations of existing bridges that are being rehabilitated should consider underpinning if scour indicates the need. Riprap and other scour countermeasures may be appropriate if underpinning is not cost-effective.

Available technology has not developed sufficiently to provide reliable scour estimates for some conditions, such as bridge abutments located in areas of turbulence due to converging or diverging flows.

C2.6.4.5

Highway embankments on floodplains serve to redirect overbank flow, causing it to flow generally parallel to the embankment and return to the main channel at the bridge. For such cases, the highway designs shall include countermeasures where necessary to limit damage to highway fills and bridge abutments. Such countermeasures may include:

- relief bridges,
- retarding the velocity of the overbank flow by promoting growth of trees and shrubs on the floodplain and highway embankment within the highway right-of-way or constructing small dikes along the highway embankment,
- protecting fill slopes subject to erosive velocities by use of riprap or other erosion protection materials on highway fills and spill-through abutments, and
- use of guide banks where overbank flow is large to protect abutments of main channel and relief bridges from turbulence and resulting scour.

Although overtopping may result in failure of the embankment, this consequence is preferred to failure of the bridge. The low point of the overtopping section should not be located immediately adjacent to the bridge, because its failure at this location could cause damage to the bridge abutment. If the low point of the overtopping section must be located close to the abutment, due to geometric constraints, the scouring effect of the overtopping flow should be considered in the design of the abutment. Design studies for overtopping should also include evaluation of any flood hazards created by changes to existing flood flow patterns or by flow concentrations in the vicinity of developed properties.