

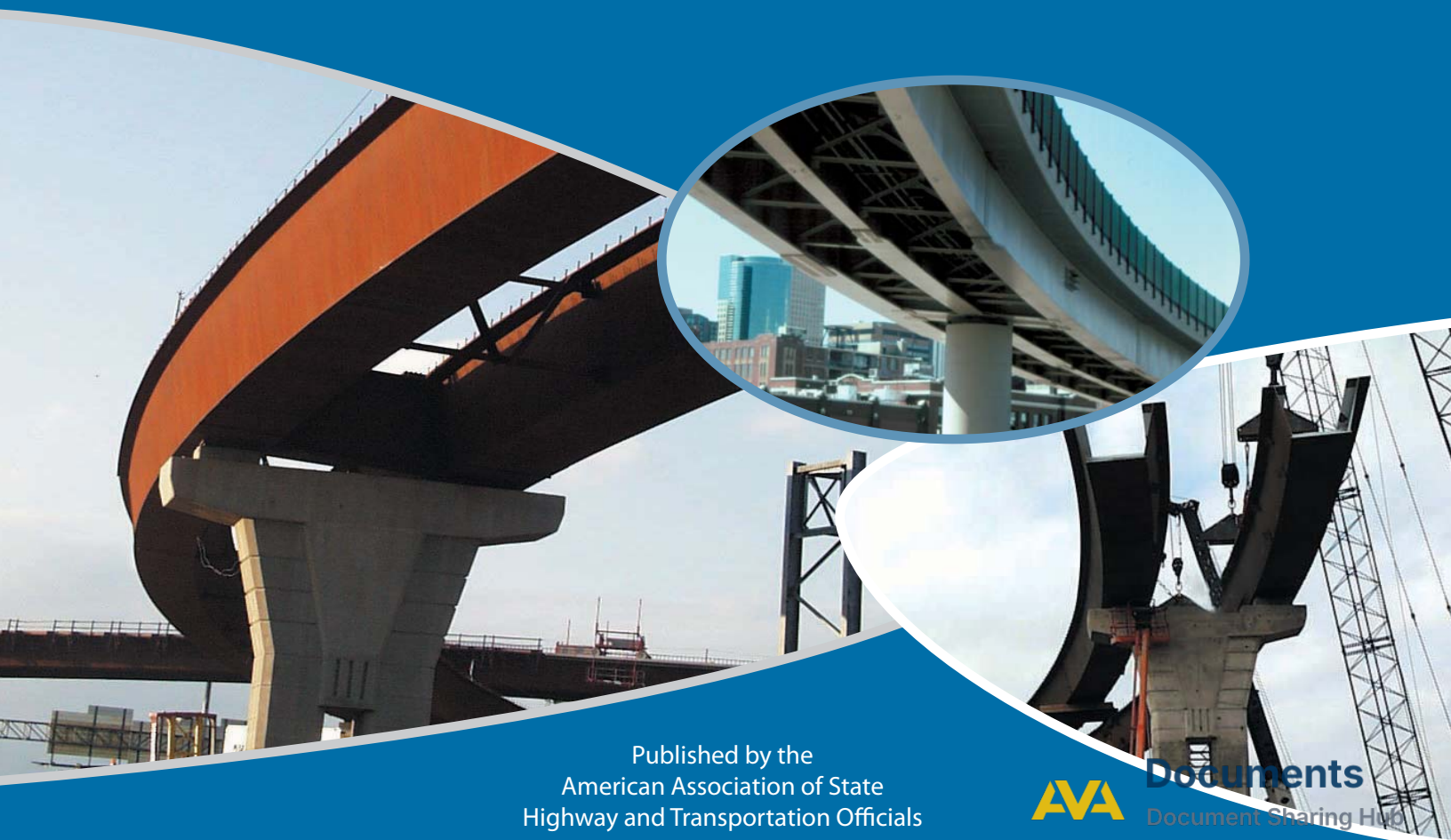


AASHTO Guide Specifications for

Horizontally Curved Steel Girder Highway Bridges

2003

With Design Examples for
I-Girder and Box-Girder Bridges



Published by the
American Association of State
Highway and Transportation Officials



Documents
Document Sharing Hub

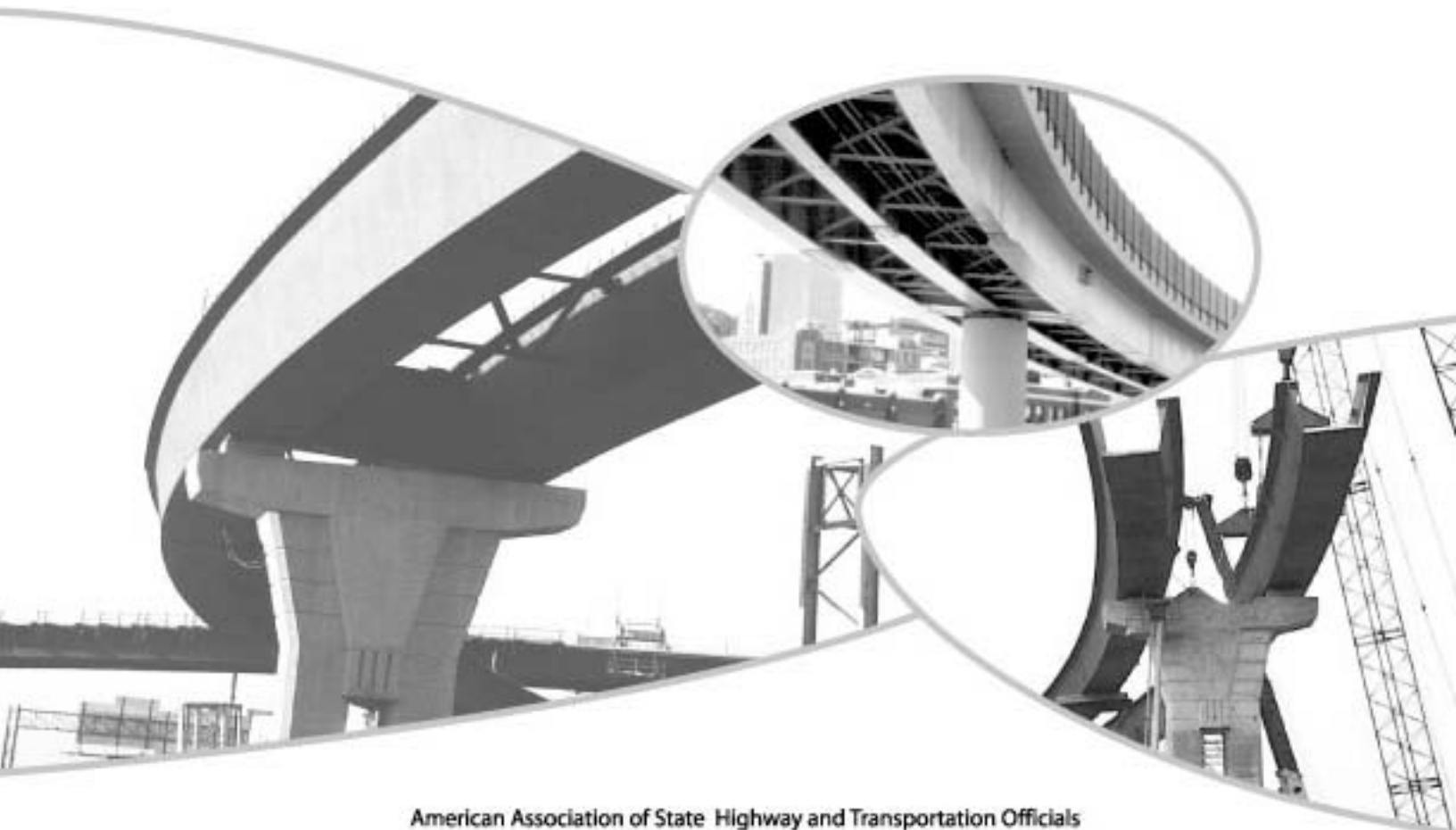
AASHTO Guide Specifications for



Horizontally Curved Steel Girder Highway Bridges

2003

With Design Examples for
I-Girder and Box-Girder Bridges



American Association of State Highway and Transportation Officials
444 North Capitol Street, NW, Suite 249
Washington, DC 20001
(202) 624-5800 (tel) • (202) 624-5806 (fax) • www.transportation.org

Copyright ©2003 by the American Association of State
Highway and Transportation Officials. All Rights Reserved. Printed in the United States.
This book, or parts thereof, may not be reproduced in any form without written permission of the publishers.
Cover photos courtesy of Brian Chen, the University of Texas at Austin.



Documents
Document Sharing Hub

ISBN: 1-56051-165-6
GHC-4

EXECUTIVE COMMITTEE

2002-2003

Voting Members

Officers:

President: James C. Codell, III, Kentucky

Vice President:

Secretary-Treasurer: Larry King, Pennsylvania

Regional Representatives:

REGION I: Carol Murray, New Hampshire, One-Year Term
Brian Searles, Vermont, Two-Year Term

REGION II: Fred Van Kirk, West Virginia, One-Year Term
Whittington Clement, Virginia, Two-Year Term

REGION III: Henry Hungerbeeler, Missouri, One-Year Term
Mark Wandro, Iowa, Two-Year Term

REGION IV: Tom Stephens, Nevada, One-Year Term
Sleeter Dover, Wyoming, Two-Year Term

Nonvoting Members

Immediate Past President:

AASHTO Executive Director: John Horsley, Washington, D.C.

HIGHWAY SUBCOMMITTEE ON BRIDGES AND STRUCTURES 2002

TOM LULAY, OREGON, *Chairman*

SANDRA LARSON, *Vice Chairman*

JAMES D. COOPER, Federal Highway Administration, *Secretary*

ALABAMA, William F. Conway, George H. Connor

ALASKA, Richard A. Pratt

ARIZONA, F. Daniel Davis

ARKANSAS, Phil Brand

CALIFORNIA, Richard Land

COLORADO, Mark A. Leonard

CONNECTICUT, Gordon Barton

DELAWARE, Doug Finney, Dennis O'Shea

D.C., Donald Cooney

FLORIDA, William N. Nickas

GEORGIA, Paul Liles, Brian Summers

HAWAII, Paul Santo

IDAHO, Matthew M. Farrar

ILLINOIS, Ralph E. Anderson

INDIANA, Mary Jo Hamman

IOWA, Norman L. McDonald

KANSAS, Kenneth F. Hurst, Loren R. Risch

KENTUCKY, Stephen E. Goodpaster

LOUISIANA, Hossein Ghara, Mark J. Morvant

MAINE, James E. Tukey

MARYLAND, Earle S. Freedman

MASSACHUSETTS, Alexander K. Bardow

MICHIGAN, Steve Beck

MINNESOTA, Dan Dorgan, Kevin Western

MISSISSIPPI, Harry Lee James

MISSOURI, Shyam Gupta

MONTANA, William S. Fullerton

NEBRASKA, Lyman D. Freemon

NEVADA, William C. Crawford, Jr.

NEW HAMPSHIRE, Mark Richardson

NEW JERSEY, Harry A. Capers, Jr., Richard W.
Dunne

NEW MEXICO, Jimmy D. Camp

NEW YORK, James O'Connell, George A. Christian

NORTH CAROLINA, Gregory R. Perfettie

NORTH DAKOTA, Terry Udland

OHIO, Timothy Keller

OKLAHOMA, Robert J. Rusch, Veldo Goins

OREGON, Mark E. Hirota

PENNSYLVANIA, R. Scott Christie

PUERTO RICO, Jamie Cabre

RHODE ISLAND, Kazem Farhoumand

SOUTH CAROLINA, Randy R. Cannon, Jeff Sizemore

SOUTH DAKOTA, John Cole

TENNESSEE, Ed Wasserman

TEXAS, Mary Lou Ralls

U.S. DOT, Nick E. Mpras

UTAH, David Nazare

VERMONT, James McCarthy

VIRGINIA, Malcolm T. Kerley

WASHINGTON, Jerry Weigel, Tony M. Allen

WEST VIRGINIA, James Sothen

WISCONSIN, Stanley W. Woods

WYOMING, Gregg C. Fredrick, Keith R. Fulton

ALBERTA, Dilip K. Dasmohapatra, Bob Ramsey

MANITOBA, Ismail Elkholy

NORTHERN MARIANA ISLANDS, John C.
Pangalanan

NEW BRUNSWICK, David Cogswell

NORTHAMPTON, R. T. Hughes

NORTHWEST TERRITORIES, John Bower

NOVA SCOTIA, Alan MacRae, Mark Pert

ONTARIO, Vacant

SASKATCHEWAN, Hervé Bachelu

MASS. METRO. DIST. COMM., David Lenhardt

N.J. TURNPIKE AUTHORITY, Richard Raczynski

N.Y. STATE BRIDGE AUTHORITY, William
Moreau

PORT AUTHORITY OF N.Y. AND N.J., Joseph
J. Kelly, Joseph Zitelli

BUREAU OF INDIAN AFFAIRS, Wade Casey

**MILITARY TRAFFIC MANAGEMENT
COMMAND**, Robert D. Franz

**U.S. ARMY CORPS OF ENGINEERS-
DEPARTMENT OF THE ARMY**, Paul C. T.
Tan

U.S. COAST GUARD, Jacob Patnaik

**U.S. DEPARTMENT OF AGRICULTURE-
FOREST SERVICE**, Nelson Hernandez

PREFACE

AASHTO first published Guide Specifications for Horizontally Curved Highway Bridges in 1980. These guide specifications included Allowable Stress Design (ASD) provisions developed by the Consortium of University Research Teams (CURT) and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures in November 1976. CURT consisted of Carnegie-Mellon University, the University of Pennsylvania, the University of Rhode Island and Syracuse University. The 1980 guide specifications also included Load Factor Design (LFD) provisions developed in American Iron and Steel Institute (AISI) Project 190 and approved by ballot of the AASHTO Highway Subcommittee on Bridges and Structures in October 1979. The guide specifications covered both I-girders and box girders.

Changes to the 1980 guide specifications were included in the AASHTO Interim Specifications—Bridges for the years 1981, 1982, 1984, 1985, 1986, and 1990. A new version of the Guide Specifications for Horizontally Curved Highway Bridges was published in 1993. It included these interim changes, and additional changes, but did not reflect the extensive research on curved-girder bridges that has been conducted since 1980, or many important changes in related provisions of the straight-girder specifications.

The present specifications and commentary were developed to correct these deficiencies. They reflect the current state-of-the-art and are consistent with present straight-girder specifications, except as noted. They were developed in National Cooperative Highway Research Program (NCHRP) Project 12-38 and are fully documented in NCHRP Report 424: Improved Design Specifications for Horizontally Curved Steel Girder Highway Bridges.

The following terms are used to identify particular specifications:

- **Bridge Welding Code** refers to the 1996 edition of ANSI/AASHTO/AWS Bridge Welding Code and Interim Specifications,
- “previous curved-girder specifications” or **Guide Spec** refers to the 1993 AASHTO Guide Specifications for Horizontally Curved Highway Bridges,
- **AASHTO** or **AASHTO LFD/ASD** refers to the 1996 AASHTO Standard Specifications for Highway Bridges, 16th Edition and Interim Specifications,
- **AASHTO LRFD** refers to the 1998 AASHTO LRFD Bridge Design Specifications, Second Edition and Interim Specifications.

Curved girder specifications based on the present AASHTO LFD specifications will be incorporated into the AASHTO Load and Resistance Factor Design (LRFD) specifications under NCHRP Project 12-52. An extensive theoretical and experimental research program is also being conducted on curved-girder bridges under sponsorship of the Federal Highway Administration (FHWA). This program should permit further future improvements in the present curved girder specifications.

TABLE OF CONTENTS

PREFACE.....	v
--------------	---

AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges

DEFINITIONS.....	3
------------------	---

NOTATIONS.....	7
----------------	---

DIVISION I—DESIGN

SECTION 1: GENERAL	13
--------------------------	----

1.1 SCOPE	13
-----------------	----

1.2 PRINCIPLES	14
----------------------	----

1.2.1 Statics	14
---------------------	----

1.2.2 Stability.....	14
----------------------	----

1.2.3 Strength of Materials	15
-----------------------------------	----

1.2.4 Small-Deflection Elastic Behavior.....	15
--	----

1.2.5 Large-Deflection Inelastic Behavior.....	16
--	----

SECTION 2: LIMIT STATES.....	17
------------------------------	----

2.1 GENERAL	17
-------------------	----

2.2 STRENGTH	17
--------------------	----

2.3 FATIGUE	17
-------------------	----

2.4 SERVICEABILITY	18
--------------------------	----

2.4.1 General	18
---------------------	----

2.4.2 Deflection	18
------------------------	----

2.4.3 Concrete Crack Control	18
------------------------------------	----

2.5 CONSTRUCTIBILITY.....	18
---------------------------	----

2.5.1 General	18
---------------------	----

2.5.2 Stresses	19
----------------------	----

2.5.3 Deflections and Rotations.....	19
--------------------------------------	----

SECTION 3: LOADS.....	21
-----------------------	----

3.1 GENERAL	21
-------------------	----

3.2 DEAD LOADS	21
----------------------	----

3.3 CONSTRUCTION LOADS	21
------------------------------	----

3.4 WIND LOADS	21
----------------------	----

3.5 LIVE LOADS.....	22
---------------------	----

3.5.1 General	22
---------------------	----

3.5.2 Centrifugal Force.....	22
------------------------------	----

3.5.3 Permit Loads.....	22
-------------------------	----

3.5.4 Overload	23
----------------------	----

3.5.5 Sidewalk Load	23
---------------------------	----

3.5.6 Impact.....	23
-------------------	----

3.5.6.1 I-Girders	23
-------------------------	----

3.5.6.2 Closed Box and Tub Girders	23
--	----

3.5.6.3 Fatigue.....	24
----------------------	----

3.5.7 Fatigue	24
---------------------	----

3.5.7.1 General	24
-----------------------	----

3.5.7.2 Application	24
---------------------------	----

3.6 THERMAL LOADS	25
SECTION 4: STRUCTURAL ANALYSIS	27
4.1 GENERAL	27
4.2 NEGLECT OF CURVATURE EFFECTS.....	27
4.2.1 I-Girders.....	27
4.2.2 Closed Box and Tub Girders.....	28
4.3 METHODS.....	28
4.3.1 Approximate Methods.....	28
4.3.2 Refined Methods.....	29
4.4 UPLIFT	30
4.5 CONCRETE	30
4.5.1 General.....	30
4.5.2 Stresses in Composite Sections.....	30
4.5.3 Prestressed Concrete	31
4.6 CONSTRUCTION STAGES	31
SECTION 5: FLANGES WITH ONE WEB.....	33
5.1 GENERAL	33
5.2 PARTIALLY BRACED COMPRESSION FLANGES	34
5.2.1 Compact Flanges.....	34
5.2.2 Non-Compact Flanges.....	35
5.3 PARTIALLY BRACED TENSION FLANGES	36
5.4 CONTINUOUSLY BRACED FLANGES.....	36
SECTION 6: WEBS.....	39
6.1 GENERAL	39
6.2 UNSTIFFENED WEBS	39
6.2.1 Bending Stresses	39
6.2.2 Shear Strength.....	40
6.3 TRANSVERSELY STIFFENED WEBS	41
6.3.1 Bending Stresses	41
6.3.2 Shear Strength.....	42
6.4 LONGITUDINALLY AND TRANSVERSELY STIFFENED WEBS	42
6.4.1 Bending Stresses	42
6.4.2 Shear Strength.....	43
6.5 TRANSVERSE WEB STIFFENERS.....	43
6.6 LONGITUDINAL WEB STIFFENERS	44
6.7 BEARING STIFFENERS	45
SECTION 7: SHEAR CONNECTORS.....	47
7.1 GENERAL	47
7.2 I-GIRDERS.....	47
7.2.1 Strength.....	47
7.2.2 Fatigue	49
7.3 CLOSED BOX AND TUB GIRDERS.....	50
7.3.1 General.....	50
7.3.2 Strength.....	50
7.3.3 Fatigue	50

SECTION 8: BEARINGS	51
8.1 GENERAL	51
8.2 FORCES	51
8.2.1 Vertical Forces	51
8.2.2 Horizontal Forces	51
8.2.3 Shear Forces	51
8.2.4 Prestressing Forces	51
8.3 MOVEMENTS	51
8.4 BEARING REPLACEMENT	52
SECTION 9: I-GIRDERS	53
9.1 GENERAL	53
9.2 VARIABLE DEPTH GIRDERS	53
9.3 CROSS FRAMES AND DIAPHRAGMS	54
9.3.1 General	54
9.3.2 Arrangement	54
9.3.3 Load Effects	55
9.4 FLANGE LATERAL BRACING	55
9.5 PERMANENT DEFLECTION	56
9.6 FATIGUE	56
9.6.1 General	56
9.6.2 Flanges	56
9.6.3 Webs and Stiffeners	57
9.6.4 Cross Frames, Diaphragms, and Flange Bracing	57
SECTION 10: CLOSED BOX AND TUB GIRDERS	59
10.1 GENERAL	59
10.2 FRAMING	59
10.2.1 Bearings	59
10.2.2 Internal Bracing and Diaphragms	59
10.2.2.1 General	59
10.2.2.2 Internal Diaphragms at Supports	59
10.2.2.3 Intermediate Internal Bracing	60
10.2.3 External Bracing	61
10.2.4 Bracing of Tub Flanges	61
10.2.5 Access Holes	62
10.3 STRESS DETERMINATIONS	62
10.3.1 General	62
10.3.2 Variable Depth Girders	62
10.4 STRENGTH OF BOX FLANGES	63
10.4.1 General	63
10.4.2 Non-Composite Box Flanges	63
10.4.2.1 General	63
10.4.2.2 Shear	64
10.4.2.3 Tension Flanges	64
10.4.2.4 Compression Flanges	65
10.4.2.4.1 Unstiffened Flanges	65
10.4.2.4.2 Longitudinally Stiffened Flanges	66
10.4.3 Composite Box Flanges	66
10.4.3.1 General	66
10.4.3.2 Tension Flanges	67
10.4.3.3 Compression Flanges	67



10.4.3.4 Shear	67
10.4.3.5 Shear Connectors	67
10.5 PERMANENT DEFLECTION	68
10.6 FATIGUE	68
10.6.1 Flanges and Webs	68
10.6.2 Bracing and Diaphragms	69
SECTION 11: SPLICES AND CONNECTIONS	71
11.1 GENERAL	71
11.2 BOLTED CONNECTIONS	72
SECTION 12: DEFLECTIONS	75
12.1 GENERAL	75
12.2 SPAN-TO-DEPTH RATIO	75
12.3 DEAD LOAD DEFLECTIONS	75
12.4 LIVE LOAD DEFLECTIONS	75
SECTION 13: CONSTRUCTIBILITY	77
13.1 GENERAL	77
13.2 STEEL	77
13.3 CONCRETE	77
13.4 DEFLECTIONS	77
13.5 SHIPPING	78
13.6 STEEL ERECTION	78
13.7 DECK PLACEMENT	78
13.8 DECK OVERHANGS	78
REFERENCES	81

DIVISION II—CONSTRUCTION

SECTION 1: GENERAL	87
1.1 GENERAL	87
1.2 SCOPE	87
1.3 CONSTRUCTION PLAN	87
SECTION 2: FABRICATION	89
2.1 GENERAL	89
2.2 HANDLING	89
2.3 GIRDERS	89
2.3.1 Rolled I-Girders	89
2.3.2 Welded I-Girders	89
2.3.3 Welded Box and Tub Girders	89
2.4 WEB ATTACHMENTS	89
2.4.1 Transverse Stiffeners	89
2.4.2 Cross-Frame and Diaphragm Connection Plates	90
2.4.3 Longitudinal Stiffeners	90
2.4.4 Cross Frames and Diaphragms	90
2.5 BOLT HOLES	90
2.6 TOLERANCES	90
2.6.1 Welded Web Flatness	90
2.6.2 Camber	90
2.6.3 Sweep	92

2.6.4 Girder Lengths	92
2.7 FIT-UP	93
2.7.1 General	93
2.7.2 Girder Section Field Splices	93
SECTION 3: TRANSPORTATION PLAN	95
SECTION 4: STEEL ERECTION	97
4.1 GENERAL	97
4.2 FALSEWORK	97
4.3 BEARINGS	97
4.4 I-GIRDERS	97
4.5 CLOSED BOX AND TUB GIRDERS	98
SECTION 5: DECK	99
5.1 FORMS	99
5.1.1 General	99
5.1.2 Overhangs	99
5.1.3 Tub Girders	99
5.2 PLACEMENT OF CONCRETE	99
SECTION 6: REPORTS	101
REFERENCES	103

HORIZONTALLY CURVED STEEL I-GIRDER DESIGN EXAMPLE

SECTION 1: OBJECTIVES	107
1.1 OBJECTIVES	107
SECTION 2: DESIGN PARAMETERS	109
2.1 DESIGN PARAMETERS	
SECTION 3: STEEL FRAMING	111
3.1 GENERAL	111
3.2 GIRDER SPACING	111
3.3 GIRDER DEPTH	111
3.4 MINIMUM PLATE SIZES	111
3.5 CROSS FRAMES	112
3.6 FIELD SECTION SIZES	112
SECTION 4: FRAMING FOR FINAL DESIGN	113
4.1 ALTERNATIVE FRAMING SCHEMES	113
4.2 CROSS FRAMES	113
4.3 FIELD SECTIONS	114
SECTION 5: FINAL DESIGN	115
5.1 LOADS	115
5.1.1 Non-Composite Dead Load	115
5.1.2 Constructibility	115
5.1.3 Superimposed Dead Load	115
5.1.4 Live Load	115
5.2 ANALYSES	115
5.2.1 Loading Combinations	

5.2.2 Three-Dimensional Finite Element Analyses	116
5.2.3 Comparison of Analyses	116
5.3 LIMIT STATES	117
5.3.1 Strength	117
5.3.2 Constructibility	117
5.3.3 Permanent Deflection	117
5.3.4 Fatigue	117
5.3.5 Live Load Deflection	117
5.4 DESIGN	118
5.4.1 Section Properties	118
5.4.2 Flanges	118
5.4.3 Webs	120
5.4.4 Shear Connectors	120
5.4.5 Bearing Orientation	121
5.4.6 Details	121
5.4.7 Erection	121
5.4.8 Wind	122
5.4.8.1 Loading	122
5.4.8.2 Analysis	122
5.4.8.3 Construction	123
5.4.9 Deck Staging	123
5.5 SAMPLE CALCULATIONS	123

APPENDICES

A. GIRDER FIELD SECTIONS	125
B. GIRDER MOMENTS AND SHEARS AT TENTH POINTS	137
C. COMPARISON OF ANALYSES	155
C.1 MODELING FOR GRID ANALYSIS	155
C.1.1 General	155
C.1.2 Coordinates	155
C.1.3 Kinematic Degrees of Freedom	155
C.1.4 Boundary Conditions	155
C.1.5 Dead Load	155
C.1.6 Live Load	155
C.2 MODELING FOR V-LOAD ANALYSIS	156
C.2.1 General	156
C.2.2 Coordinates	156
C.2.3 Boundary Conditions	156
C.2.4 Dead Load	156
C.2.5 Live Load	156
D. SELECTED DESIGN FORCES AND GIRDER 4 SECTION PROPERTIES	163
E. SAMPLE CALCULATIONS	177
E.1 GIRDER STRESS CHECK SECTION 0-0 G4 NODE 4 TRANSVERSELY STIFFENED WEB—SHEAR STRENGTH—WEB	177
E.2 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB— CONSTRUCTIBILITY—TOP FLANGE	179
E.2.1 Overhang Bracket Load	179



E.3 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB—	183
CONSTRUCTIBILITY—WEB	183
E.4 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB—	185
CONSTRUCTIBILITY—DECK	184
E.5 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB—	
STRENGTH—TOP FLANGE	185
E.6 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB—	
BENDING STRENGTH—WEB	186
E.7 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB—	
STRENGTH—BOTTOM FLANGE	187
E.8 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB—	
FATIGUE—BOTTOM FLANGE	188
E.9 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 TRANSVERSELY STIFFENED WEB—	
FATIGUE—SHEAR CONNECTORS	189
E.9.1 Method 1	189
E.9.2 Method 2	190
E.10 GIRDER STRESS CHECK SECTION 3-3 G4 NODE 64 TRANSVERSELY STIFFENED WEB—	
FATIGUE—SHEAR CONNECTORS	191
E.10.1 Method 1	191
E.10.2 Method 2	191
E.11 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 TRANSVERSELY STIFFENED WEB—	
STRENGTH TOP AND BOTTOM FLANGE	193
E.11.1 Top Flange	193
E.11.2 Bottom Flange	193
E.12 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 TRANSVERSELY STIFFENED WEB—	
OVERLOAD—BOTTOM FLANGE	196
E.13 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 TRANSVERSELY STIFFENED WEB—	
FATIGUE—TOP FLANGE	198
E.14 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 TRANSVERSELY STIFFENED WEB—	
SHEAR STRENGTH—WEB	199
E.15 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 TRANSVERSELY STIFFENED WEB—	
BENDING STRENGTH—WEB	200
E.16 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 TRANSVERSELY STIFFENED WEB—	
FATIGUE—SHEAR CONNECTORS	201
E.16.1 Method 1	201
E.16.2 Method 2	201
E.17 GIRDER STRESS CHECK G4 SPAN 1 TRANSVERSELY STIFFENED WEB—STRENGTH—SHEAR CONNECTORS ..	203
E.18 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 UNSTIFFENED WEB—CONSTRUCTIBILITY—WEB	205
E.19 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 UNSTIFFENED WEB—BENDING STRENGTH—WEB	206
E.20 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 UNSTIFFENED WEB—OVERLOAD—WEB	207
E.21 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 UNSTIFFENED WEB—BENDING STRENGTH—WEB	208
E.22 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 UNSTIFFENED WEB—SHEAR STRENGTH—WEB	209
E.23 GIRDER STRESS CHECK SECTION 0-0 G4 NODE 4 TRANSVERSELY STIFFENED WEB—	
BEARING STIFFENER DESIGN	210
E.24 GIRDER STRESS CHECK SECTION 6-6 G1 NODE 97 TRANSVERSELY STIFFENED WEB—	
BEARING STIFFENER DESIGN	211
E.25 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 LONGITUDINALLY STIFFENED WEB—	
CONSTRUCTIBILITY—WEB	212
E.26 GIRDER STRESS CHECK SECTION 2-2 G4 NODE 44 LONGITUDINALLY STIFFENED WEB—	
BENDING STRENGTH—WEB	213
E.27 GIRDER STRESS CHECK SECTION 3-3 G4 NODE 64 LONGITUDINALLY STIFFENED WEB—	



FATIGUE—TOP AND BOTTOM FLANGE	214
E.28 GIRDER STRESS CHECK SECTION 3-3 G4 NODE 64 LONGITUDINALLY STIFFENED WEB—	
CONSTRUCTIBILITY—WEB	216
E.29 GIRDER STRESS CHECK SECTION 3-3 G4 NODE 64 LONGITUDINALLY STIFFENED WEB—	
CONSTRUCTIBILITY—DECK	217
E.30 GIRDER STRESS CHECK SECTION 3-3 G4 NODE 64 LONGITUDINALLY STIFFENED WEB—	
BENDING STRENGTH—WEB	218
E.30.1 Positive Live Load Bending Case	218
E.30.2 Negative Live Load Bending Case	218
E.31 GIRDER STRESS CHECK SECTION 3-3 G4 NODE 64 LONGITUDINALLY STIFFENED WEB—OVERLOAD—WEB	220
E.31.1 Positive Live Load Bending Case	220
E.31.2 Negative Live Load Bending Case	220
E.32 GIRDER STRESS CHECK SECTION 4-4 G4 NODE 76 LONGITUDINALLY STIFFENED WEB—	
CONSTRUCTIBILITY—WEB	222
E.33 GIRDER STRESS CHECK SECTION 4-4 G4 NODE 76 LONGITUDINALLY STIFFENED WEB—	
BENDING STRENGTH—WEB	223
E.33.1 Positive Live Load Bending Case	223
E.33.2 Negative Live Load Bending Case	223
E.34 GIRDER STRESS CHECK SECTION 4-4 G4 NODE 76 LONGITUDINALLY STIFFENED WEB—OVERLOAD—WEB	225
E.34.1 Positive Live Load Bending Case	225
E.34.2 Negative Live Load Bending Case	225
E.35 GIRDER STRESS CHECK SECTION 5-5 G4 NODE 88 LONGITUDINALLY STIFFENED WEB—	
BENDING STRENGTH—WEB	227
E.36 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 LONGITUDINALLY STIFFENED WEB—	
BENDING STRENGTH—WEB	228
E.37 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 LONGITUDINALLY STIFFENED WEB—	
OVERLOAD—WEB	229
E.38 GIRDER STRESS CHECK G4 LONGITUDINALLY STIFFENED WEB – LONGITUDINAL STIFFENER DESIGN	230
E.39 GIRDER STRESS CHECK SECTION 6-6 G4 NODE 100 LONGITUDINALLY STIFFENED WEB—	
SHEAR STRENGTH—WEB	231
E.40 GIRDER STRESS CHECK G4 FIELD SECTION 2 (SPAN 1) TRANSVERSELY	
STIFFENED WEB—TRANSVERSE STIFFENER DESIGN	232
E.41 GIRDER STRESS CHECK G4 SPANS 1 AND 2 LONGITUDINALLY STIFFENED WEB—	
TRANSVERSE STIFFENER SPACING	234
E.42 BOLTED FIELD SPLICE DESIGN SECTION 8-8 G4 NODE 124	237
E.42.1 Design Action Summary and Section Information	237
E.42.2 Bolt Capacities	237
E.42.3 Top Flange Bolts – Constructibility and Overload	238
E.42.4 Bottom Flange Bolts – Constructibility and Overload	240
E.42.5 Top and Bottom Flange Bolts – Strength	242
E.42.6 Web Bolts – Constructibility and Overload	245
E.42.7 Web Bolts – Strength	249
E.42.8 Web Splice Plates	250
E.42.9 Flange Splice Plates	251
E.43 GIRDER STRESS CHECK G4 NODE 99-100 CROSS FRAME DIAGONAL—STRENGTH AND CONNECTION	253
E.44 CENTRIFUGAL FORCE CALCULATIONS	255
F. STUDY OF TWO ERECTION SCHEMES	257
F.1 INTRODUCTION	257
F.2 STEEL ERECTION STUDY	258
F.2.1 Steel Erection without Temporary Supports in End Spans	259
F.2.2 Steel Erection with Temporary Supports in End Spans	264



F.3 WIND ON STEEL DURING ERECTION	266
F.3.1 Girder Stresses Including Lateral Flange Bending	267
F.3.2 Bracing Members	267
G. FIELD SECTION PROFILES	269

HORIZONTALLY CURVED STEEL BOX-GIRDER DESIGN EXAMPLE

SECTION 1: OBJECTIVES	275
1.1 OBJECTIVES	275
SECTION 2: DESIGN PARAMETERS	277
2.1 DESIGN PARAMETERS	277
SECTION 3: STEEL FRAMING	279
3.1 GENERAL	279
3.2 GIRDER DEPTH	279
3.3 INTERNAL AND EXTERNAL BRACING	280
3.4 BRACING OF TUB FLANGES	280
3.5 LONGITUDINAL FLANGE STIFFENER	284
3.6 FIELD SECTIONS	284
SECTION 4: ANALYSES	285
4.1 LOADING COMBINATIONS	285
4.2 THREE-DIMENSIONAL FINITE ELEMENT ANALYSES	285
4.3 COMPARISON OF ANALYSES	286
SECTION 5: LOADS	287
5.1 DEAD LOAD	287
5.2 LIVE LOAD	287
SECTION 6: LIMIT STATES	289
6.1 STRENGTH	289
6.2 CONSTRUCTIBILITY	289
6.3 FATIGUE	289
6.4 LIVE LOAD DEFLECTION	290
6.5 PERMANENT DEFLECTION	290
SECTION 7: DESIGN	291
7.1 SECTION PROPERTIES	291
7.2 SHEAR CONNECTORS	291
7.3 FLANGES	291
7.4 WEBS	291
7.5 DIAPHRAGMS	291
7.6 SAMPLE CALCULATIONS	292

APPENDICES

A. GIRDER FIELD SECTIONS	293
B. GIRDER MOMENTS, SHEARS, AND TORQUES AT TENTH-POINTS	299
C. COMPARISON OF ANALYSES.....	307
C.1 MODELING FOR GRID ANALYSIS	307
C.1.1 General	307
C.1.2 Coordinates.....	307
C.1.3 Kinematic Degrees of Freedom.....	307
C.1.4 Boundary Conditions.....	307
C.1.5 Dead Load	307
C.1.6 Live Load	307
C.2 MODELING FOR M/R METHOD ANALYSIS	308
C.2.1 General	308
C.2.2 Coordinates.....	308
C.2.3 Boundary Conditions.....	308
C.2.4 Dead Load	308
C.2.5 Live Load	308
D. SELECTED DESIGN FORCES AND GIRDER 2 SECTION PROPERTIES	315
E. SAMPLE CALCULATIONS	321
E.1 GIRDER STRESS CHECK SECTION 1-1 G2 NODE 10 CONSTRUCTIBILITY—WEB	321
E.2 GIRDER STRESS CHECK SECTION 1-1 G2 NODE 10 CONSTRUCTIBILITY—TOP FLANGE.....	322
E.2.1 Overhang Bracket Load.....	322
E.3 GIRDER STRESS CHECK SECTION 1-1 G1 NODE 9 CONSTRUCTIBILITY—TOP FLANGE.....	326
E.4 GIRDER STRESS CHECK SECTION 1-1 G2 NODE 10 FATIGUE—BOTTOM FLANGE	329
E.5 GIRDER STRESS CHECK SECTION 1-1 G2 NODE 10 FATIGUE—SHEAR CONNECTORS.....	330
E.6 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 STRENGTH—BOTTOM FLANGE	331
E.7 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 LONGITUDINAL FLANGE STIFFENER	337
E.8 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 OVERLOAD—WEB.....	338
E.9 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 DESIGN OF INTERNAL DIAPHRAGM	339
E.10 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 DESIGN OF BEARING STIFFENERS.....	342
E.11 GIRDER STRESS CHECK G2 SPAN 1 BAY 1 TOP FLANGE BRACING MEMBER DESIGN.....	344
E.12 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 TRANSVERSE BENDING STRESS.....	346
E.13 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 COMPOSITE BOTTOM FLANGE OPTION	358
E.14 GIRDER STRESS CHECK SECTION 4-4 G2 NODE 32 COMPOSITE BOTTOM FLANGE OPTION	362
E.15 GIRDER STRESS CHECK SECTION 5-5 G2 NODE 36 COMPOSITE BOTTOM FLANGE OPTION DESIGN OF SHEAR CONNECTORS—STRENGTH	365
E.16 GIRDER STRESS CHECK SECTION 2-2 G2 NODE 20.3 STRESSES	368
E.17 GIRDER STRESS CHECK SECTION 2-2 G2 NODE 20.3 STRENGTH—BOTTOM FLANGE	370
E.18 BOLTED SPLICE DESIGN SECTION 2-2 G2 NODE 20.3	372
E.18.1 Design Action Summary and Section Information	372
E.18.2 Bolt Capacities.....	372
E.18.3 Top Flange Bolts—Constructibility and Overload	373
E.18.4 Bottom Flange Bolts—Constructibility and Overload.....	375
E.18.5 Top and Bottom Flange Bolts—Strength.....	377
E.18.6 Web Bolts—Constructibility and Overload.....	381
E.18.7 Web Bolts—Strength.....	384
E.18.8 Web Splice Plates	386
E.18.9 Flange Splice Plates.....	387

I-GIRDER

LIST OF FIGURES

Figure 3.2.1 I-Girder Bridge Cross Section	111
Figure 4.2.1 Framing Plan and Nodal Numbering	114
Figure B.1 Dead Load (Structural Steel) Moment	138
Figure B.2 Dead Load (Concrete Deck) Moment	139
Figure B.3 Dead Load (Superimposed Dead Load) Moment	140
Figure B.4 Dead Load (Structural Steel) Shear	141
Figure B.5 Dead Load (Concrete Deck) Shear	142
Figure B.6 Dead Load (Superimposed Dead Load) Shear	143
Figure E.1 Overhang Bracket Loading	179
Figure E.2 Factored Shear and Transverse Stiffener Spacing—Span 1—Longitudinally Stiffened Web	235
Figure E.3 Factored Shear and Transverse Stiffener Spacing—Span 2—Longitudinally Stiffened Web	236
Figure E.4 Bolt Patterns for Top and Bottom Flanges	239
Figure E.5 Web Bolt Pattern	246
Figure E.6 Centrifugal Force and Superelevation	255
Figure F.1 Deformed Shapes	260
Figure G.1 Field Section 1	269
Figure G.2 Field Section 2	270
Figure G.3 Field Section 3	271

LIST OF TABLES

Table 5.3.5.1 Preferred Maximum Live Load Deflections in Center Span (in)	118
Table 5.3.5.2 Computed Maximum Live Load Deflections in Center Span (in)	118
Table 5.4.2.1 Comparison of Lateral Flange Moments from 3D Analysis and Eq. (4-1)	119
Table 5.4.2.2 Comparison of Lateral Flange Moments from 3D Analysis and Eq. (4-1)	120
Table C.1 Dead Load (Structural Steel) Analysis Comparison	157
Table C.2 Dead Load (Concrete Deck) Analysis Comparison	158
Table C.3 Dead Load (Superimposed Dead Load) Analysis Comparison	159
Table C.4 Live Load (Truck) Analysis Comparison	160
Table C.5 Live Load (Lane) Analysis Comparison	161
Table D.1 Girder 4 Selected Moments (k-ft) and Web Shears (kips)	163
Table D.2 Shear (kips), Girder 4, Span 1	164
Table D.3 Shear (kips), Girder 4, Span 2	165
Table D.4 Load Combinations for Cross Frame Member 99 top-100 bottom	166
Table D.5 Load Combinations for Cross Frame Member 99 bottom-100 top	167
Table D.6 Load Combinations for Cross Frame Member 97 top-98 bottom	168
Table D.7 Load Combinations for Cross Frame Member 97 bottom-98 top	169
Table D.8 Load Combinations for Vertical Reaction Bearing 98	170
Table D.9 Load Combinations for Tangential Reaction Bearing 98	171
Table D.10 Load Combinations for Radial Reaction Bearing 98	172
Table D.11 Selected Girder 4 Section Properties—Transversely Stiffened Web	173
Table E.42.1.1 Unfactored Actions	237
Table E.42.1.2 Cross Section	237
Table F.2.1.1 Difference between Maximum Vertical Deflections for RUN1A and RUN1 (in)	261
Table F.2.1.2 Comparison of Vertical Reactions (kips) Between RUN1 and RUN1A	261
Table F.2.1.3 Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN1	262
Table F.2.1.4 Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN1A	262
Table F.5 Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN2	262
Table F.6 Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN3B	262

Table F.7	Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN3C	263
Table F.8	Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN1+RUN3B+RUN3C Compared to RUN2	263
Table F.9	Net Vertical Deflections (in) at Nodes 37-40 Without Temporary Supports No Cross Frames Between Girders 2 and 3 in Field Section 3	263
Table F.10	Net Vertical Deflections (in) at Nodes 145-148 Without Temporary Supports No Cross Frames Between Girders 2 and 3 in Field Section 3	264
Table F11	Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN1D.....	264
Table F12	Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN3E	265
Table F13	Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN3G.....	265
Table F14	Differences in Top and Bottom Girder “X” Deflections (in) at Splice for RUN1D+RUN3E+RUN3G Compared to RUN2	265
Table F15	Net Vertical Deflections (in) at Nodes 37-40 with Temporary Supports	266
Table F16	Net Vertical Deflections (in) at Nodes 145-148 With Temporary Supports	266

BOX GIRDER**LIST OF FIGURES**

Figure 1	Box Girder Bridge Cross Section.....	279
Figure 2	Node Numbers.....	281
Figure 3	Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Double-Diagonal Bracing, Inclined Webs.....	282
Figure 4	Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Single-Diagonal Bracing, Inclined Webs.....	282
Figure 5	Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Entire Deck Weight with Overhang Brackets, Vertical Webs.....	283
Figure 6	Lateral Flange Moments (k-ft) and Bracing Forces (kips) Due to Cast #1 with Overhang Brackets, Single-Diagonal Bracing, Inclined Webs.....	283
Figure 7	Lateral Flange Moments (k-ft) Due to Entire Deck Weight with Overhang Brackets, Double-Diagonal Bracing, Vertical Webs.....	284
Figure E1	Overhang Bracket Loading.....	322
Figure E2	Internal Diaphragm and Bearing Stiffeners at Pier of Girder 2 Looking Upstation.....	342
Figure E3	Composite Box Cross Section, G2.....	346
Figure E4	Effective Width of Web Plate, d_o , Acting with Transverse Stiffener.....	347
Figure E5	Shear Studs On Composite Bottom Flange.....	366
Figure E6	Bolt Patterns for Top and Bottom Flange.....	374
Figure E7	Web Bolt Pattern.....	382

LIST OF TABLES

Table 6.4.1	Preferred Maximum Live Load Deflections (in).....	290
Table C.1	Dead Load (Structural Steel) Analysis Comparison.....	309
Table C.2	Dead Load (Concrete Deck) Analysis Comparison.....	310
Table C.3	Dead Load (Superimposed Dead Load) Analysis Comparison.....	311
Table C.4	Live Load (HS25 Truck) Analysis Comparison.....	312
Table C.5	Live Load (Lane) Analysis Comparison.....	313
Table D.1	Girder 2 Selected Moments (k-ft) and Web Shears (kips).....	315
Table D.2	Shear (kips), Girder 2, Span 1.....	316
Table D.3	Girder 2, Selected Torques (k-ft).....	317
Table D.4	Top Flange Bracing Forces, Girder 2, Span 1 (kips).....	318
Table D.5	Selected Girder 2 Section Properties.....	319
Table E.1	Section Properties of G2 with 8 inches of 6,000 psi Concrete in Bottom Flange.....	358
Table E.2	Strength Limit State at 100 feet from Left Abutment.....	369
Table E.3	Overload At 100 Feet From Left Abutment.....	369
Table E.4	Constructibility Limit State at 100 feet from Left Abutment.....	369
Table E.5	Unfactored Actions.....	372
Table E.6	Tub Cross Section.....	372

AASHTO GUIDE SPECIFICATIONS FOR HORIZONTALLY CURVED STEEL GIRDER HIGHWAY BRIDGES

DEFINITIONS

For a consistent application of the specifications, it is necessary that terms be defined where they have particular meanings in the specifications. The following definitions are given for use in application of these specifications only and do not always correspond to ordinary usage:

Action—A generalized force in a structural member or component including moment, shear, axial force and externally applied load. Deflections and rotations are also considered as actions.

Arc Span—Distance between centers of adjacent bearings, or other points of support, measured horizontally along the centerline of a horizontally curved member.

Aspect Ratio—Ratio computed by dividing the length by the width of a rectangular panel.

Beam—A structural member whose primary function is to transmit loads to the supports through flexure.

Box Flange—A flange that is connected to two webs. The flange may be a flat unstiffened plate, a stiffened plate or a flat plate with reinforced concrete attached to the plate with shear connectors.

Bracing Member—A member intended to brace a girder against lateral movement or distortion.

Closed Box Girder—A bending member having a cross section composed of two webs which has at least a completely enclosed cell. A closed section girder is effective in resisting applied torsion by developing non-vanishing shear flow in the web(s) and flanges.

Compact Flange—A partially or fully braced flange that can sustain sufficient strains so that the entire flange can be assumed to be at the yield stress.

Component—A constituent part of a structure.

Composite Girder—A steel I-shaped or box-shaped bending member connected to a concrete slab so that the steel element and the slab, or the longitudinal reinforcement within the slab, respond to bending loads as a unit.

Connection—A weld or arrangement of bolts that transfers normal and/or shear stresses from one element to another.

Construction Plan—A plan created by the Contractor that details the procedures for fabrication, erection, and deck placement. The plan may be based on the construction scheme shown in the Design Plans, or developed entirely by the Contractor.

Contractor—An organization that agrees to build the bridge for the Owner according to the terms of a contract executed by the Owner and the Contractor.

Cracked Section—A composite section in which the concrete is assumed to carry no stress.

Cross Frame—A vertically oriented transverse truss framework connecting adjacent longitudinal bending members.

Cross Section Distortion—Distortion of the cross section of a box or tub girder due to torsional loading.

Curved Girder—An I-, closed box, or tub girder that is curved in a horizontal plane.

Deck—The roadway composed of reinforced concrete.

Design—Proportioning and detailing the components and connections of a bridge to satisfy these specifications.

Design Plans—A set of plans developed by a licensed structural engineer to be used as the basis for the construction contract.

Diaphragm—A vertically oriented solid transverse member connecting adjacent longitudinal girders or inside a closed box or tub girder.

Distortion-Induced Fatigue—Fatigue caused by secondary stresses that are not normally evaluated in the typical design and analysis of a bridge.

Engineer—A licensed structural engineer responsible for design of the bridge or review of the bridge construction.

Erector—Usually a sub-contracting organization that agrees to construct the structural steel portion of the bridge superstructure according to the Construction Plan prepared by the Contractor.

Fabricator—Usually a sub-contracting organization that agrees to manufacture and provide the structural steel components of the bridge superstructure according to the Construction Plan prepared by the Contractor.

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

Fatigue—The initiation/or propagation of cracks in steel members due to repeated variation of strain.

Fatigue Resistance—The maximum stress range that can be sustained for a specified number of cycles without failure.

Finite Element Method—A method of analysis in which the structure is divided into elements connected at nodes. The shape of the element displacement field is assumed, partial or complete compatibility is maintained among the element interfaces, and nodal displacements are determined by using energy principles or equilibrium methods.

First-Order Analysis—An analysis based on small-deflection theory under linearly elastic constitutive equations.

Flange Concrete—Reinforced concrete acting with a box flange design to resist vertical bending and torsion.

Girder Radius—The radius of the circumferential centerline of a segment of a curved girder.

Grid Method—A method of analysis in which all or part of the superstructure is discretized into components in a two-dimensional plane that represent the characteristics of the structure.

Hooke's Law—Stress in a material is proportional to strain in the elastic range.

Inelastic Behavior—A condition in which deformation is not fully recovered upon removal of the load. Strain exceeds the proportional limit.

I-Girder—A bending member that has an I-shaped cross section.

Lane—The area of deck receiving one vehicle or one uniform load line.

Large Deflection Theory—Any method of structural analysis in which the influence of deformation on force is considered.

Lateral Bending Stress—Bending stress caused by lateral flange bending.

Lateral Bracing—A truss placed in a horizontal plane between two I-girders or two flanges of a tub girder.

Lateral Flange Bending—Flexural action of a flange in the plane of the flange with respect to the vertical axis through the flange. Lateral flange bending may be due to lateral loads applied to the flange and/or nonuniform torsion in the member. In these provisions lateral flange bending moments refer to those at brace points.

Load Effect—Moment, shear, axial force, or torque induced in a member by loads applied to the structure.

Load and Resistance Factor Design (LRFD)—A reliability-based design method in which factored design forces caused by factored service loads are not permitted to exceed the nominal strengths of the members.

Main Member—A primary member designed to carry the internal forces determined from an analysis.

Model—A mathematical idealization of a structure or component used for analysis.

M/R Method—An approximate method for the analysis of curved box girders in which the curved girder is treated as an equivalent straight girder to calculate bending effects and as a corresponding straight conjugate beam to calculate the concomitant St. Venant torsional moments due to curvature.

Müller-Breslau Principle—The influence surface for an action at a preselected point of a linear elastic structure is to the same scale as the deflected shape of the structure when the structure is given a unit deformation at the preselected point corresponding to the action.

Navier's Hypothesis—In any bent beam, sections which were plane before bending remain plane after bending. Also known as the Bernoulli-Euler assumption.

Negative Moment—Moment producing tension at the top of a bending element.

Non-compact Flange—A compression flange that can sustain a peak normal strain at the flange tip equal to the yield strain.

Nonuniform Torsion—An internal resisting torsion in thin-walled sections producing shear stresses and normal stresses, and under which cross sections do not remain plane. Non-uniform torsion is also known as warping torsion. Members developing nonuniform torsion resist the externally applied torsion by warping torsion and St. Venant torsion. Each of these components of internal resisting torsion varies along the member length, although the externally applied concentrated torsion may remain uniform along the member between two adjacent points of torsional restraint. Warping torsion is dominant over St. Venant torsion in members having open cross sections, whereas St. Venant torsion is dominant over warping torsion in members having closed cross sections.

Open Section—A bending member cross section which has no enclosed cell. An open section member resists torsion primarily by nonuniform torsion, which causes normal stresses at the flange tips.

Owner—A person or an organization who pays for the construction of the structure and assumes the responsibility of its operation and maintenance.

Pier—A column or connected group of columns or other configuration designed to be an interior support for a bridge superstructure.

Pitch—The distance between adjacent bolts or shear connectors in the direction of the force.

Positive Moment—Moment producing tension at the bottom of a bending element.

Primary Member—Any member designed to carry the loads applied to the structure.

Refined Methods of Analysis—Methods of structural analysis that consider the entire superstructure as an integral unit and provide the required deflections and actions.

Second-Order Analysis—An analysis that accounts for geometric and/or material nonlinearity.

Secondary Member—A member in which stress is not normally evaluated in the analysis.

Shear Flow—Shear force acting on the cross section of a plate element divided by the thickness or width of the element; shear flow has the units of force per unit width of the plate.

Shear Lag—Nonlinear distribution of normal stress across an element due to shear distortions.

Shear Strength—Shear capacity of a component as computed using these provisions.

Small-Deflection Theory—Any method of structural analysis in which the influence of deformation on force effects is neglected.

Slab—Deck composed of concrete and reinforcement.

Span Length—See Arc Span.

Splice—A group of bolted connections, or a welded connection, sufficient to transfer the moment, shear, axial force, or torque between two girder components.

St. Venant Torsion—Internal resisting torsion producing pure shear stresses on a cross section. Torsion in members such as pipes or solid round bars, where only St. Venant torsion resists the externally applied torsion, is referred to as either pure torsion or uniform torsion.

Stress Range—The algebraic difference between the extreme values in a stress cycle in a fatigue check.

Stringer A longitudinal bending member of open section such as an I-girder. This term is frequently used interchangeably with I-girder.

Strong-Axis—The centroidal axis about which the moment of inertia is a maximum.

Superposition—The stresses in a member due to one loading can be added to the stresses in the member due to another loading if the load-stress relationship is linearly elastic, the sum is still within the elastic range and the stiffness of the structure is not changed.

Through-Thickness Stress—Bending stresses in a web or box flange induced by distortion of the cross section.

Torsional Shear Stress—Shear stress induced by St. Venant torsion.

Tub Girder—An open-topped steel girder which is composed of a bottom flange plate, two web plates and an independent top flange attached to the top of each web. The top flanges are connected with lateral bracing members.

Unbraced Length—The arc distance between brace points resisting the mode of buckling, lateral bending or distortion under consideration.

Uncracked Section—A composite section in which the concrete is assumed to be fully effective in tension and compression.

Uniform Live Load—Loading consisting of a uniform load intensity and specified concentrated loads placed in critical positions longitudinally and transversely. These loads are referred to in **AASHTO** as lane loads.

Vertical Bending Moment—Bending moment about the horizontal axis, usually the strong-axis.

V-Load Method—An approximate method for the analysis of curved I-girder bridges in which the curved girders are represented by equivalent straight girders and the effects of curvature are represented by vertical and lateral forces applied at cross frame locations. Lateral flange bending at brace points due to curvature is estimated.

Von Mises Yield Criterion—A theory which states that inelastic action begins at a point subjected to biaxial or triaxial stresses when the strain energy of distortion per unit volume absorbed at that point is equal to the strain energy of distortion per unit volume absorbed at any point on a bar loaded to its yield strength under uniaxial tensile stress. This theory is also called the maximum strain-energy-of-distortion theory.

Warping Normal Stress—Normal stress induced by non-uniform torsion is referred to as warping stress.

Web Slenderness—The distance along a web between flanges divided by the web thickness.

Wheel Load Distribution Factor—The fraction of half of a truck or lane load assumed to be carried by an individual girder.

Yield Stress—The specified minimum yield stress for the steel.

NOTATIONS

A_{bot}	=	area of bottom flange (in ²) (7.2.2)
A_o	=	enclosed area of box (in ²) (C10.4.2.2)
A_s	=	area of steel girder (in ²) (7.2.1)
a	=	aspect ratio of transverse stiffeners (6.5) (6.6)
b	=	minimum flange width (in) (5.1) (Division II, 3)
b_d	=	effective deck width (in) (7.2.1) (10.4.3.5)
b_f	=	flange width (in) (5.2.1) (5.2.2) (C9.3.2) (10.4.2.4.1) (10.4.2.4.2) (10.4.3.5)
b_s	=	stiffener width (in) (6.5)
b_s	=	distance between longitudinal stiffeners (in) (10.4.2.4.2)
C	=	ratio of the elastic shear-buckling strength to the shear-yield strength (6.2.2)
C	=	equivalent moment factor (6.7)
D	=	dead load (3.5.1)
D	=	distance along the web between flanges (in) (6.2) (6.2.1) (6.2.2) (6.3) (6.3.1) (6.3.2) (6.4) (6.4.1) (6.5) (6.6)
D	=	web depth (in) (4.2.1) (C4.2.1) (C9.1)
D_c	=	depth of web in compression (in) (6.2.1) (6.3.1) (6.4.1) (C9.1)
d	=	depth of steel girder (in) (12.2)
d	=	required distance between transverse stiffeners (in) (6.3.2) (6.5)
d_o	=	actual distance between transverse stiffeners (in) (6.3) (6.3.2) (6.5)
d_s	=	distance between longitudinal stiffener and compression flange (in) (6.4.1)
E	=	modulus of elasticity of steel (ksi) (5.2.1) (5.2.2) (6.2.1) (6.2.2) (6.3.1) (6.4.1) (6.5)
F	=	uniform radial force (k/ft) (C13.8)
F_{bs}	=	critical average normal stress in the compression flange of an equivalent straight girder (ksi) (5.2.1) (5.2.2)
F_{CR}	=	net range of cross frame axial force at the top flange (kip) (7.2.2)
F_{cr}	=	critical average flange stress (ksi) (5.1) (5.2.1) (5.2.2) (5.4) (10.4.2.3) (10.4.2.4.1) (10.5)
F_{cr}	=	critical compressive longitudinal stress in the web (ksi) (6.2.1) (6.3.1) (6.4.1)
F_{cr1}	=	critical average flange stress calculated from reduction factors (ksi) (5.2.1) (5.2.2) (5.3)
F_{cr2}	=	critical average flange stress determined from partial yielding (ksi) (5.2.1) (5.2.2) (5.3)
F_{fat}	=	radial fatigue shear range/unit length (k/in) (7.2.2) (7.3.3) (10.4.3.5)
$\overline{F}_p$	=	radial force in the concrete slab at point of maximum positive live load moment (kip) (7.2.1)
$\overline{F}_t$	=	radial force in the concrete slab at point of greatest negative live load moment (kip) (7.2.1)
F_v	=	critical torsional shear stress in a box flange (ksi) (10.4.2.2)
F_y	=	specified minimum yield stress of steel (ksi) (5.1) (5.2.1) (5.2.2) (5.3) (5.4) (6.1) (6.2.1) (6.2.2) (6.3.1) (6.4.1) (6.7) (7.2.1) (9.5) (10.4.2.2) (10.4.2.3) (10.4.2.4.1) (10.5) (12.2)
F_y	=	specified minimum yield stress of stiffener (ksi) (6.5) (6.7)
F_y	=	specified minimum yield stress of box bottom flange (ksi) (C10.4.2.1)
F_y	=	specified minimum yield stress of box-girder diaphragm (ksi) (C10.2.2.2)
f_b	=	calculated factored average flange stress at the section under consideration (ksi) (5.1) (5.2.1) (5.2.2) (C9.3.2)
f'_c	=	specified 28-day compressive stress of concrete (ksi) (7.2.1) (10.4.3.3) (13.3)
f_ℓ	=	calculated total factored lateral flange bending stress at the section under consideration (ksi) (5.1) (5.2.1) (5.2.2) (5.3) (C9.3.2) (C13.8)
f_r	=	modulus of rupture (ksi) (2.4.3)
f_v	=	factored torsional shear stress in a box flange (ksi) (C10.4.2.1) (10.4.2.3) (10.4.2.4.1)
f_v	=	shear stress in a box-girder diaphragm (ksi) (C10.2.2.2)
h	=	depth between mid-thickness of top and bottom flange (in) (C4.2.1)
I	=	impact allowance (3.5.1)
$I_{\ell s}$	=	required moment of inertia of longitudinal web stiffener (in ⁴) (6.6)
I_s	=	actual moment of inertia of one longitudinal flange stiffener about an axis at the base of the stiffener (in ⁴) (10.4.2.4.2)

I'_s	=	required moment of inertia of a longitudinal flange stiffener about an axis at the base of the stiffener (in ⁴) (10.4.2.4.2)
I_{ts}	=	moment of inertia of a transverse web stiffener with respect to the mid-plane of the web (in ⁴) (6.5) (6.6)
J	=	the required ratio of rigidity of one transverse web stiffener to that of the web plate (6.5)
K	=	effective length factor (6.7) (9.3.1)
k	=	coefficient of web bend-buckling (6.2.1) (6.3.1) (6.4.1)
k	=	plate-buckling coefficient under normal stress (10.4.2.4.1) (10.4.2.4.2)
k_s	=	plate-buckling coefficient under shear stress (10.4.2.4.1) (10.4.2.4.2)
k_w	=	shear-buckling coefficient (6.2.2) (6.3.2)
L	=	arc girder length between bearings (in) (12.4)
L_{as}	=	arc span (ft) (4.2.1) (4.2.2) (12.2)
L_c	=	maximum cantilever length of girder during shipping (in) (Division II, 3)
L_n	=	arc length between a point of maximum positive live load moment and an adjacent point of greatest negative moment (ft) (7.2.1)
L_p	=	arc length between a point of zero moment and an adjacent point of maximum positive live load moment (ft) (7.2.1)
LL	=	live load (3.5.1)
ℓ	=	arc length of the unsupported flange between cross frames or diaphragms (ft) (4.2.1) (5.1) (5.2.1) (5.2.2) (7.2.2) (C9.3.2) (C13.8)
M	=	vertical bending moment (k-ft) (4.2.1)
M_{lat}	=	lateral flange bending moment (k-ft) (4.2.1) (C13.8)
M_p	=	plastic moment (k-ft) (C5.2.1)
M_y	=	yield moment (k-ft) (C5.2.1)
N	=	required number of shear connectors for strength (7.2.1)
n	=	modular ratio (4.5.2) (13.3)
n	=	number of shear connectors in a cross section (7.2.2)
n	=	number of equally spaced uniform longitudinal box flange stiffeners (10.4.2.4.2)
P	=	concentrated lateral force (kip) (C13.8)
P	=	force in deck (kip) (7.2.1) (7.2.2) (7.3.2)
$\overline{P}_n$	=	smaller of P_{1n} or P_{2n} (kip) (7.2.1)
$\overline{P}_p$	=	longitudinal force in the deck or flange concrete at point of maximum positive live load moment equal to the smaller of P_{1p} or P_{2p} (kip) (7.2.1)
$\overline{P}_T$	=	longitudinal force in the deck or flange concrete at point of greatest negative live load moment equal to $\overline{P}_p + \overline{P}_n$ (kip) (7.2.1)
P_{1n}	=	longitudinal force in the girder at point of greatest negative live load moment (kip) (7.2.1)
P_{2n}	=	longitudinal force in the deck at point of greatest negative live load moment (kip) (7.2.1)
P_{1p}	=	longitudinal force in the girder at point of maximum positive live load moment (kip) (7.2.1)
P_{2p}	=	longitudinal force in the deck at point of maximum positive live load moment (kip) (7.2.1)
R	=	girder radius (ft) (C4.2.1) (C9.3.2)
R	=	minimum girder radius within a panel or over a specified girder length (ft) (5.1) (5.2.1) (5.2.2) (6.2) (6.3) (6.5) (7.2.1) (7.2.2)
R_1, R_2	=	reduction factors for stresses in a box flange (10.4.2.4.1) (10.4.3.5)
r_σ	=	desired bending stress ratio (C9.3.2)
S	=	girder spacing (ft) (4.2.1)
SF	=	shear flow (k/in) (C10.4.2.2)
S_u	=	ultimate strength of one shear connector (kip) (7.2.1)
T	=	torque (k-in) (C10.4.2.2)
t_d	=	average concrete deck thickness (in) (7.2.1)
t_f	=	flange thickness (in) (5.2.1) (5.2.2) (10.4.2.4.1) (10.4.2.4.2)
t_s	=	transverse web stiffener thickness (in) (6.5) (6.6)
t_w	=	web thickness (in) (6.2) (6.2.1) (6.2.2) (6.3) (6.3.1) (6.4) (6.4.1) (6.5) (6.6) (6.7)
V_{cr}	=	elastic shear-buckling or shear-yield strength of a web (kip) (6.2.2) (6.3.2) (6.4.2)
V_{fat}	=	longitudinal fatigue shear range/unit length (k/in) (7.2.2) (7.3.3)



V_p	=	plastic shear capacity of a web (kip) (6.2.2)
V_{sr}	=	horizontal shear range between deck and steel for fatigue design of shear connectors (k/in) (7.2.2)
w	=	effective length of deck (in) (7.2.2) (9.3.3)
X	=	modification factor to J (6.5)
Z	=	curvature parameter (6.5) (6.6)
Z_r	=	shear fatigue strength of an individual shear connector (7.2.2)
β	=	position parameter for a longitudinal web stiffener (6.6)
Δ	=	reduction factor for the maximum stress in a box flange (10.4.2.3) (10.4.2.4.1)
λ	=	slenderness parameter (5.2.1)
ϕ	=	strength reduction factor applied to modulus of rupture (= 0.9) (2.4.3)
ϕ	=	coefficient for I'_s (10.4.2.4.2)
ϕ_{sc}	=	reduction factor for strength of shear connectors (= 0.85) (7.2.1)
ρ_b	=	curvature factor for non-compact flange strength (5.2.2)
ρ_w	=	curvature factor for non-compact flange strength (5.2.2)
ρ_{w1}	=	curvature factor for non-compact flange strength (5.2.2)
ρ_{w2}	=	curvature factor for non-compact flange strength (5.2.2)
ρ_b	=	curvature factor for compact flange strength (5.2.1) (5.3)
ρ_w	=	curvature factor for compact flange strength (5.2.1) (5.3)
σ_{flg}	=	range of fatigue stress in bottom flange (ksi) (7.2.2)
σ_1	=	critical maximum principal stress in a box-girder diaphragm (ksi) (C10.2.2.2)
σ_2	=	critical minimum principal stress in a box-girder diaphragm (ksi) (C10.2.2.2)
σ_x	=	average longitudinal stress in a box bottom flange due to vertical bending (ksi) (C10.4.2.1)
σ_y	=	stress in a box bottom flange due to vertical bending of the diaphragm over the bearing sole plate (ksi) (C10.4.2.1)
σ_y	=	stress in a box-girder diaphragm due to vertical bending of the diaphragm over the bearing sole plate (ksi) (C10.2.2.2)
σ_z	=	stress in a box-girder diaphragm due to bending of the diaphragm about its longitudinal axis (ksi) (C10.2.2.2)
τ_y	=	shear yield stress (ksi) (C10.2.2.2)

This page is intentionally left blank.

DIVISION I

DESIGN

SECTION 1

GENERAL

1.1 SCOPE

These provisions are to be used in conjunction with the Standard Specifications for Highway Bridges (AASHTO) and the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD). Articles referenced from those specifications are in bold type. These provisions shall not invalidate any provisions of other AASHTO specifications. However, when there is a conflict between the provisions of this specification and **AASHTO** and **AASHTO LRFD** provisions in the design of horizontally curved steel girders, the provisions herein shall govern.

These provisions apply to the design and construction of highway superstructures with horizontally curved steel I-shaped or single-cell box-shaped longitudinal girders with spans up to 300 feet and with radii greater than 100 feet. There shall be at least two I-girders or one box-shaped girder in each bridge cross section. Any bridge superstructure containing a curved girder segment shall be designed according to these provisions. The structural steel shall have a specified minimum yield stress not greater than 100 ksi.

Both closed box and tub girders are permitted. When tub girders are used, the top flanges shall be connected together with adequate diagonal shear bracing to form a pseudo-box.

Girders may be constant or variable depth. All components of each girder cross section shall be homogeneous with respect to steel grade, but the steel grade may vary along the length of a girder.

The following framing arrangements are permitted under these provisions: simple and continuous spans; constant and variable girder spacing; normal and skewed support lines; bifurcated alignments; different girder stiffnesses within a bridge cross section; discontinuous girders; girders having non-concentric radii in a cross section; girders having kinked alignments; integral pier cap beams; and integral abutments.

The superstructure shall have either a composite or a non-composite cast-in-place or precast concrete deck. Composite girders shall be designed to be composite along their entire length. The deck cross section may have a constant thickness or it may be vaulted (variable thickness). Longitudinal and/or transverse prestressing of the deck is permitted. These provisions do not apply to superstructures with steel orthotropic decks.

C1.1

Although the current **AASHTO (1996)** is applicable to spans up to 500 feet and the **AASHTO LRFD** provisions have no span limit, these provisions are limited to 300-foot spans because of the history of construction problems associated with curved bridges with spans greater than 300 feet. Large girder self-weight may cause critical stresses and deflections during erection when the steel work is incomplete. Large lateral deflections and girder rotations associated with longer spans tend to make it difficult to fit up cross frames. Large curved bridges have been built successfully; however, these bridges deserve special considerations such as the possible need of more than one temporary support in large spans.

Approximations made in several of the formulas defining I-girder strength and impact have not been studied for radii smaller than 100 feet (*Christiano, 1967; Hall and Yoo, 1996a; and McManus, 1971*).

A minimum of two I-girders is required in a cross section to provide static equilibrium without unusual torsional restraints. Single box girders are permitted because torsional equilibrium can be established with two bearings at some supports.

Any bridge superstructure containing a curved-girder segment is to be designed according to these provisions. The effects of curvature on the support reactions and girder deflections, as well as the effects of lateral flange bending, usually extend beyond the curved segment. This requirement also ensures that the bridge is designed according to the provisions of a single specification, which also simplifies future load rating or evaluation.

The provisions do not apply to multi-cell box girders because there has been little published research in the United States regarding these members. Analysis of this bridge type involves consideration of shear flow in each cell. However, the provisions do not prohibit multi-cell box girders.

Bridges beyond these limitations can be designed by following principles given in Article 1.2.

The top flanges of tub girders need to be braced so that the girder acts as a pseudo-box for non-composite loads before the deck hardens. Top flange bracing working with internal cross bracing retains the box shape and resists lateral force induced by inclined webs and torsion.

These provisions do not provide for hybrid girders. The 1993 Guide Specifications for Horizontally Curved Highway Bridges (Guide Spec) provided for hybrid I-girders, but not for hybrid box girders. Design of hybrid girders is based on the assumption that a portion of the web yields. There have been only a few

1.2 PRINCIPLES

Classical structural principles have been applied in the development of this specification. These principles may be applied to the design of complex bridges beyond those covered by these provisions. When inconsistencies between these provisions and the engineering principles, described herein, are discovered, the principles shall predominate.

1.2.1 Statics

The structure shall satisfy static equilibrium. Any section taken through the bridge shall be in equilibrium under each load group.

1.2.2 Stability

General stability of the structure and the individual components during construction as well as when completed shall be ascertained for each critical load condition.

tests of curved hybrid girders; and there has been no study of these test results with regard to hybrid action.

Although a mix of box and I-girders within a cross section are not treated, there may be cases where this arrangement is desirable and such an arrangement is not prohibited.

Kinked girders exhibit the same actions as curved girders except that the effect of the non-collinearity of the flanges is concentrated at the kinks.

Curvature of a single curved girder in a cross section affects all girders in the cross section, whether the others are straight or curved because the forces required for equilibrium of the curved girder(s) are resisted by the adjacent girders. Skewed supports can cause girder torsion. Removal of highly stressed cross frames, particularly near obtuse corners, releases the girders torsionally and is often beneficial as long as girder rotation is not excessive.

Girder-stringer-floor beam framing is not forbidden, although it is usually not desirable for use on curved girder bridges because floor beams must be designed both to support the stringers and to stabilize the curved girders.

C1.2

Although structural principles are implicit in structural design specifications, these provisions differ from most in that they are intended to be used for widely diverse framing that cannot be categorized or predefined. Indiscriminate application of rules to unusual framing may violate the basic principles. The practice of good engineering is ensured by requiring structural principles to be predominant over the provisions rather than simply following rules.

Each component must be designed for the actions to which it will be subjected. If the superstructure is to perform as intended, all components of the superstructure including the deck, shear connectors, bracing and bearings need to be analyzed and designed to the same level of refinement as the girders and connections.

The Engineer may use these principles to determine whether an analysis is rational for a particular structure.

C1.2.1

Statics is fundamental to bridge analysis. Any free body taken within the structure must satisfy six equations of statics for all load conditions. Free body checks should be used to confirm results from any analysis. If the analysis results do not satisfy statics, the analysis may need further refinement.

C1.2.2

The Engineer is responsible for ensuring that the girders are constrained against tangential displacement with respect to any planes orthogonal to the girder axis.

Stability of girders having curvature and section parameters within defined limits shall be ensured by limiting the stresses due to factored loads computed from small-deflection theory to critical stresses defined herein.

Stability of bracing members, connection plates and bearing stiffeners shall be ensured by limiting the computed stresses due to factored loads to the critical stress for compression members defined herein.

Local stability of all components shall be ensured by limiting the computed stresses due to factored loads to the critical stress corresponding to the width-to-thickness ratio of the element.

1.2.3 Strength of Materials

Hooke's law shall apply to structural analyses. The modulus of elasticity of steel and concrete shall be determined from **AASHTO**. Poisson's ratio of steel may be taken as 0.3 and Poisson's ratio of concrete may be taken as 0.16. Navier's hypothesis shall apply independently to web and flange components for vertical bending and lateral flange bending, respectively.

Distortion of the cross section need not be considered in the structural analysis.

1.2.4 Small-Deflection Elastic Behavior

Analyses may be made using small-deflection elastic theory unless deemed inappropriate by the Engineer.

The principle of superposition may be employed for analyses of the completed structure. The principle

For example, a pair of curved I-girders only braced by cross frames may not be stable without lateral flange bracing. It may be advisable to use bottom flange bracing in at least one location in such situations to ensure stability when erecting pairs of I-girders.

Small-deflection theory is appropriate when the translation of the forces or loads due to the change in the shape of the structure is insignificant. Small-deflection theory is appropriate for analysis of completed bridges. However, during erection, when all bracing may not be present, deflections may be large enough that small-deflection theory is not applicable and stability can be ascertained only with the application of large-deflection theory. Thus, the Engineer should consider the magnitude of deflections when performing analysis of the construction conditions when all bracing may not be effective.

Lateral torsional buckling of partially braced curved I-girders is not a bifurcation phenomenon in the classical sense. Instead, out-of-plane deflection occurs incipiently with vertical loading.

Local buckling of flanges is controlled in the provisions by limiting the compression flange width-to-thickness ratio. Research indicates that curvature may contribute to local buckling of I-girder flanges (*Culver and Nasir, 1969; Nakai and Yoo, 1988; and Yoo, 1996*). However, the flange width-to-thickness ratios used in these provisions are appropriate.

Bracing of curved members is more critical than that for straight members. The provisions require that forces in bracing members be computed and considered in design of these members.

C1.2.3

The normal assumptions associated with strength of materials methods are assumed appropriate in these provisions.

Although it is appropriate to assume in the analysis that the girder cross section does not distort, it has been shown that cross section distortion has an important effect on the strength of curved I-girders and was considered in the development of the strength equations (*McManus, 1971*). Cross section deformation of steel box girders may have a significant effect on torsional behavior, but this effect is limited by the provision of sufficient internal cross bracing.

Classical methods of analysis usually are based on strength of materials assumptions that do not recognize cross section deformation. Finite element analyses that model the actual cross section shape of the I- or box girders can recognize cross section distortion and its effect on structural behavior.

C1.2.4

Small-deflection elastic behavior permits the use of the principle of superposition and efficient analytical solutions. These assumptions are typically used in bridge analysis for this reason. The behavior of the

of superposition shall not be applied for analyses of construction processes that include changes in the structure stiffness.

1.2.5 Large-Deflection Inelastic Behavior

Steel is considered to be elastic-perfectly-plastic.

Concrete creep may be considered by using a reduced modulus of elasticity for concrete subjected to long-term stress.

members assumed in these provisions is generally consistent with this type of analysis.

Moments from non-composite and composite analyses may not be added for the purpose of computing stresses. The addition of stresses and deflections due to non-composite and composite actions computed from separate analyses is appropriate.

C1.2.5

The provisions do not require inelastic analyses; however, such analyses are likely to give better results for curved girders with extremely large lateral deflections than analyses based on small-deflection theory. Second-order analysis may be used to predict girder behavior including stability during construction when spacing of braces is greater than that permitted by the provisions.

Since no redistribution of loads is permitted in these provisions, there is no need for the designer to consider material inelastic behavior in the analysis.

SECTION 2

LIMIT STATES

2.1 GENERAL

Designs shall satisfy each limit state specified herein for the appropriate factored load groups.

2.2 STRENGTH

Strength of the completed bridge shall be determined on an element-by-element basis. The superstructure shall be considered to have adequate strength if the stresses and actions in each element for the appropriate load combination meet requirements specified herein.

Load combinations for the strength limit state are defined in Article 3.1.

2.3 FATIGUE

Base metal and details subjected to a net computed tensile stress under two times the factored fatigue live load defined in Article 3.5.7.1 plus impact shall be investigated for fatigue, with the impact allowance taken as defined in Article 3.5.6.3.

The provisions of **AASHTO LRFD Article 6.6.1** shall be applied as modified in Articles 3.5.7, 9.6 and 10.6. The load factor for fatigue shall be taken as 0.75.

C2.1

Limit states are the conditions for which the bridge is designed to satisfy minimum acceptable levels of performance and strength. The limit states specified here are intended to provide for a buildable, serviceable bridge, capable of safely carrying design loads for a specified lifetime.

C2.2

The strength limit state considers stability, or yielding, of each structural element; or plastification of compact I-girders. If the strength of any element, including splices and connections, is exceeded, it is assumed that the bridge capacity has been exceeded. In fact, there is significant elastic reserve capacity in all multi-girder bridges beyond such a load level. The live load cannot be positioned to apply the design live load to all girders simultaneously. Thus, the bending capacity of the bridge cross section, defined as the sum of the capacity of each girder, exceeds the capacity required for the total live load that can be applied in the number of lanes available.

C2.3

The fatigue limit state is based on the assumption that the fatigue life of each component and detail is a function of the number of stress cycles due to the passage of a single vehicle. Each truck traversing the bridge is assumed to travel in the same critical location for a particular component or detail.

When proper detailing practices are not followed, fatigue cracking also has been found to occur due to strains not normally computed in the design process. This type of fatigue cracking is called “distortion-induced fatigue.” Distortion-induced fatigue often occurs in the web near a flange at a welded connection plate for a cross frame. These provisions require computation of forces in cross frames and other bracing members that are often considered to be “secondary members” in straight bridges. The computed forces in cross frames must be transferred to the girder flanges. Bracing members and their connections are designed for computed stress ranges according to these provisions, whereas they may be detailed with no stress computations according to **AASHTO** since they are not considered to be primary load carrying members in straight bridges.

To be consistent with the intent of the **AASHTO LRFD** provisions, a heavy loading is used to determine if a net tensile stress occurs requiring fatigue to be considered.

2.4 SERVICEABILITY

2.4.1 General

Serviceability limit states provide for the proper performance of the bridge over its expected life.

2.4.2 Deflection

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

Permanent deflection under overload defined in Article 3.5.4 shall be controlled by limiting the factored average flange stress due to vertical bending to the limiting stress specified in Article 9.5 or 10.5, as applicable.

Vertical live load deflections preferably shall be limited according to Article 12.4.

2.4.3 Concrete Crack Control

Longitudinal reinforcement shall be placed in the concrete deck when the longitudinal tensile stress in the deck due to factored construction loads or due to overload exceeds ϕf_r , where ϕ is a strength reduction factor equal to 0.9 and f_r is the modulus of rupture as defined in **AASHTO Article 8.15.2.1**. Reinforcement shall be No. 6 bars, or smaller, spaced at not more than 12 inches. Reinforcement shall be placed in top and bottom layers of the deck with concrete cover as specified by the Owner. The area of longitudinal reinforcement shall be preferably not less than one percent of the total cross sectional area of the deck. The provisions of **AASHTO Article 10.38.4.3** shall be applied on placement of longitudinal reinforcement.

2.5 CONSTRUCTIBILITY

2.5.1 General

The constructibility limit state considers deflection, strength of steel and concrete and stability during critical stages of construction.

C2.4

C2.4.1

Safety margins are not directly applicable to serviceability. Instead, limits are, to a large extent, based on experience.

C2.4.2

Curved bridges are subjected to torsion resulting in larger lateral deflections and twisting than normal tangent bridges with less than 20 degrees skew from radial. 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.

Excessive permanent set in the bridge generally does not indicate structural failure but is not desirable. The overload condition is investigated to ensure that localized yielding of the girders and cross frames is controlled to provide good riding quality.

Vertical acceleration of the bridge is thought to be the principal cause of discomfort. Acceleration can be controlled by limiting the live load deflection according to Walker and Veletos (1966).

C2.4.3

Concrete crack control is important to protract the service life of the deck. Although cracks will occur in concrete decks, properly placed reinforcing steel controls the size of the cracks.

Recognition of the tensile strength of the concrete is appropriate at the fatigue, serviceability and constructibility limit states. The **AASHTO ASD** provisions assume a resistance factor of 0.21 against the modulus of rupture. These provisions provide an effective factor of safety of approximately $\frac{1.4}{0.9} = 1.6$ against

the modulus of rupture at the constructibility limit state. By controlling crack size at overload and due to the factored construction loads, the deck can be considered effective in tension at the fatigue, serviceability and constructibility limit states. Thermal stresses and shrinkage stresses are not considered.

Placement of the deck in stages produces negative moments in regions of adjacent continuous spans that are typically subjected to positive moment. Critical placement of live load on the completed bridge can produce negative moments throughout continuous span bridges. The point of dead load contraflexure is not significant.

C2.5

C2.5.1

During construction, curved I-girder bridges are often at the greatest risk because curved I-girder bridges are dependent on adjacent girders to provide

One scheme to construct the bridge within the constraints of the site shall be shown on the Design Plans. The scheme shall include a sequence of girder and deck placement and the location of any necessary temporary supports as specified in Article 13.6. Stresses and deflections during construction shall be investigated to ensure that the design meets the limits specified in Section 13.

2.5.2 Stresses

Stresses due to factored construction loads defined in Article 3.3 in any steel component shall not exceed the specified minimum yield stress, nor shall they exceed the elastic buckling stress, during any stage of construction.

Bolts in load-resisting connections shall be checked against the slip-critical capacity for the factored loads during construction.

Strength of hardened portions of the concrete deck due to factored loads shall be checked against the provisions of Article 13.3.

2.5.3 Deflections and Rotations

Computed girder rotations at bearings and vertical girder deflections shall be accumulated over the 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 may reflect the computed accumulated deflections due to both the erection sequence and the deck casting sequence.

equilibrium through shear forces in the connecting cross frames. The constructibility limit state is to be met at design time and refined by the Contractor prior to actual construction.

One construction scheme must be shown on the Design Plans to ensure that at least one method of construction within site limitations has been considered during the design process. Division II provides for the Contractor to consider other construction schemes. The Contractor is always responsible for the final Construction Plan as determined in Division II and for execution of construction according to that Plan.

C2.5.2

The factored steel stresses are limited to the specified minimum yield stress during each critical stage of erection to ensure permanent set or deformation is controlled. Factored forces in bolted joints of load carrying members are limited to the slip-critical capacity to ensure no damage to the connections and that the correct geometry of the structure is maintained.

C2.5.3

Computation of girder vertical and lateral deflections for each stage of erection is needed to ensure proper fit-up of subsequent steel sections. Deflections and bearing rotations during construction may exceed the dead load deflections and rotations computed for the completed bridge. Identification of this temporary situation may be critical to ensure the bridge can be built without damaging the bearings or expansion devices.

This page is intentionally left blank.

SECTION 3

LOADS

3.1 GENERAL

For the strength limit state, the effect of each load shall be factored and combined according to the groups defined in **AASHTO Table 3.22.1A**, except as modified herein.

Loads and load factors for other limit states are defined herein.

3.2 DEAD LOADS

Dead loads include the weight of components that form the permanent structure. The sequence of application of dead load shall be considered in the analyses. Material densities shall be taken as specified in **AASHTO Article 3.3.6**.

Lighter dead loads may be uniformly distributed according to **AASHTO Article 3.23.2.3.1.1**. Parapets, sidewalks, barriers and other heavy line loads preferably shall be assumed applied at their actual location on the bridge. Wearing surface and other distributed loads may be assumed uniformly distributed to each girder in the cross section.

3.3 CONSTRUCTION LOADS

Construction loads shall include design dead loads such as self-weight of steel, deck forms, deck, haunches, parapets and construction equipment. A load factor of 1.4 shall be applied to loads when computing actions for the constructibility limit state. A load factor of 1.0 shall be applied to construction loads when computing deflections.

The bridge shall be investigated for uplift under each critical construction stage. Dead loads and construction loads that resist uplift shall be factored by 0.9, and loads that tend to cause uplift shall be factored by 1.2.

3.4 WIND LOADS

The intensity of wind load may be taken from **AASHTO Article 3.15** or specified by the Owner. Wind load shall be applied unidirectionally to the projected area of the bridge including barriers and sound

C3.1

AASHTO Table 3.22.1A provides factored loads that must be considered in the design of each component.

The loads defined in these provisions differ from **AASHTO Table 3.22.1A** by the addition of construction and uplift loads. Actions determined for the constructibility limit state need not be added to the strength load groups.

C3.2

The self-weight of structural steel is usually assumed to be applied to the completed non-composite structure, which implies that the steel has been erected without the impediment of gravity.

C3.3

Construction loads are dead loads and temporary loads that act on the structure during construction. Construction loads include the weight of equipment such as deck finishing machines or loads applied to the structure through falsework or other temporary supports. Often the construction loads are not accurately known at design time; however, the magnitude and location of these loads considered in the design should be noted on the Plans.

The load factor of 1.4 is an average of the load factors of 1.25 and 1.5 that are specified in **AASHTO LRFD Article 3.4.2**.

The load factor for loads causing uplift is less than the normal load factor for constructibility because the occurrence of a slight uplift is not critical to most structures and the cost to overcome uplift may be large. However, when uplift would cause instability or excessively different deflections, the Engineer may wish to use a larger load factor for uplift.

C3.4

Instead of defining wind as either perpendicular or longitudinal with respect to the structure as done in **AASHTO**, wind is defined unidirectionally. The critical direction(s) is determined for different members.

walls. The direction(s) of wind that causes critical load effects in girders, cross frames and bearings shall be determined by the Engineer. Horizontal wind load shall be applied to the top and bottom of the girders.

A load path through which wind loads are assumed to be transmitted to the substructure shall be identified. Members and connections along this path shall be designed for the combined effects of wind load and the appropriate loads according to Article 3.1.

Uplift wind loads shall be as defined in **AASHTO Article 3.15.3**.

The load factor for wind during construction shall be 1.4.

3.5 LIVE LOADS

3.5.1 General

The design vehicular live load for checking the strength limit state shall be an AASHTO HS loading specified by the Owner. Design lane widths and number of traffic lanes shall be taken as specified in **AASHTO Articles 3.7 through 3.12**.

Potential of uplift shall be investigated for the loading: $D + 2.0 (LL + I)$. The potential absence of any future-wearing surface shall be considered when checking for uplift.

3.5.2 Centrifugal Force

Centrifugal force shall be determined in accordance with **AASHTO Article 3.10**. A load path to carry the radial force to the substructure shall be provided.

The overturning effect of centrifugal force on vertical wheel loads shall be considered.

3.5.3 Permit Loads

Permit loads and load factors used to check the strength limit state shall be as specified by the Owner.

Permit loads may be placed in the critical lane with or without factored design vehicular live loading in the remaining lanes, as directed by the Owner. Alternatively, permit loads may be placed in all traffic lanes.

Wind force is determined as the wind pressure times the projected bridge area.

After the deck has hardened, wind load is mainly resisted by horizontal deck shear. Wind load applied to the bottom of the girder causes force in vertically inclined cross bracing which transfers the force to the deck. Cross frame bottom chords attached to the next girder experience nominal wind force because the bottom flange of the adjacent girder is less rigid in the lateral direction than is the deck. The wind load is transferred from the deck down through end-cross frames or diaphragms and through the bearings to the substructure. The end-cross frames or diaphragms may be critical for wind load and should be checked. This is true even when in-plane bottom flange bracing is present. In-plane bottom flange bracing receives little horizontal wind load after the deck has hardened because the deck is much more rigid.

The superstructure should be treated as an integral unit with the proper horizontal and vertical bearing restraints. The horizontal shear stiffness of the deck should be recognized.

Uplift due to wind is generally critical only during construction.

C3.5

C3.5.1

Live load is placed in positions to create critical responses in each element. Influence surfaces which cover the deck area, where traffic is permitted, can be used to position vehicular loads to determine critical live load responses.

C3.5.2

Centrifugal force creates a radial load that is transmitted from the deck through the end cross frames or diaphragms and the bearings to the substructure.

Centrifugal force also causes an overturning effect because the radial force is applied 6 feet above the top of the deck. Thus, centrifugal force tends to cause an increase in the vertical wheel loads toward the outside of the bridge and an unloading of wheel loads toward the inside of the bridge.

C3.5.3

A single lane of permit load with the remaining lanes filled with normal design traffic is possible when a refined analysis is used, and may be considered as an additional design live load to be used with the strength limit state.

Permit load(s) may be limited to the "striped lanes" or permitted to "roam" transversely on the roadway.

3.5.4 Overload

The factored loading group shall be taken as defined in **AASHTO Article 10.57**. Impact shall be taken as defined in Article 3.5.6.

3.5.5 Sidewalk Load

Sidewalk live load shall be taken from **AASHTO Article 3.14** or specified by the Owner. Sidewalk live load shall be considered to act in combination with vehicular live load. When vehicular live load is permitted on the sidewalk, sidewalk live load shall not be considered concurrently.

The same load factors shall be used for sidewalk live load as for vehicular live load.

3.5.6 Impact

Vehicular live loads shall be increased by the factors specified herein to account for dynamic amplification of live loads.

An impact allowance need not be applied to sidewalk live load.

3.5.6.1 I-Girders

Vehicular live loads for strength and serviceability limit states shall be increased by the factors specified in Table 3.5.6.1 to account for dynamic amplification.

Table 3.5.6.1 Impact for I-Girders

Load Effect	Impact Factor	
	Vehicle	Lane
Girder bending moment, torsion and deflections	0.25	0.20
Reactions, shear, cross frame and diaphragm actions	0.30	0.25

3.5.6.2 Closed Box and Tub Girders

Vehicular live loads for strength and serviceability limit states shall be increased by the factors specified in Table 3.5.6.2.

Restraint to the striped lanes usually provides a greater computed bridge capacity for the permit loads.

C3.5.4

Overload is defined as the dead load plus the largest live load the bridge is likely to experience a number of times. Overload is invoked to ensure that permanent set does not occur after a number of heavy vehicles traverse the bridge.

C3.5.5

When a rational analysis is used, the transverse position of loads can be recognized. If the assumption is made that vehicular live load can mount the sidewalk, vehicular live load may be considered over the sidewalk area and sidewalk live load should not be considered to occupy the sidewalk. The number of lanes of vehicular live load is usually determined from the roadway width excluding the sidewalk.

The load factor for the combined sidewalk live load and vehicular live load remains unchanged in these provisions because by considering the transverse positioning of the loads, the proper reduction in live load is obtained automatically.

C3.5.6

Although **AASHTO Article 3.14.1** is silent with regard to impact on the sidewalk loading, these provisions do not apply impact factors to sidewalk live load.

C3.5.6.1

The values in Table 3.5.6.1 are based on the factors specified in the **Guide Spec** and have been simplified.

C3.5.6.2

Table 3.5.6.2 is based on Schelling, et al. (1992). Generally, impact factors for box girders are higher than for I-girders.



Table 3.5.6.2 Impact for Closed Box and Tub Girders

Load Effect	Impact Factor	
	Vehicle	Lane
Girder bending moment, torsion and deflections	0.35	0.30
Reactions, shear, cross frame and diaphragm actions	0.40	0.35

3.5.6.3 Fatigue

The vehicular fatigue load specified in Article 3.5.7 shall be increased 15 percent to account for dynamic amplification.

3.5.7 Fatigue

3.5.7.1 General

The fatigue live load shall be taken as defined in **AASHTO LRFD Article 3.6.1.4** with a load factor of 0.75. The fatigue truck is shown in Figure 3.5.7.1.

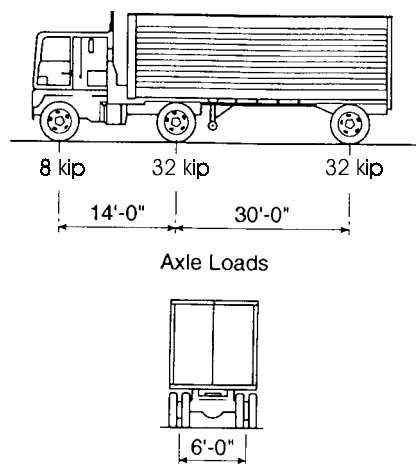


Figure 3.5.7.1 Fatigue Truck

C3.5.6.3

The impact factor, 0.15, has been taken from **AASHTO LRFD Article 3.6.2.1**.

C3.5.7

C3.5.7.1

Metal fatigue is a function of both the magnitude of the range of strain and number of repetitions of that strain. Thus, fatigue design requires a specified limit on the range of strain (stress) and a specified number of its repetitions. The number of repetitions is a function of the expected frequency of truck traffic and the design life of the bridge. The permitted range of strain is also dependent on the category specified for each detail.

Welding causes residual tensile stresses near the yield stress in the weld material and in the heat-affected zone of the base metal. Unless these stresses are relieved, fatigue cracking can occur when the cyclic stress and the number of cycles are large enough. Fatigue cracking will be limited to the region of high residual tensile stresses when the applied cyclic principal stress plus dead load stress is compressive. These provisions and **AASHTO LRFD** explicitly require that fatigue be considered only when the stress caused by the largest expected live load plus unfactored dead load stress is tensile.

The **AASHTO LRFD** fatigue truck is essentially an HS20 truck with a constant rear axle spacing of 30 feet and with a load factor of 0.75 applied. The fatigue truck is intended to produce an average stress cycle.

Heavier trucks will traverse the bridge producing tensile stresses that may initiate fatigue cracks that grow under the smaller stress range caused by the fatigue truck. Thus, any location that undergoes a net tensile stress when subjected to the heaviest load expected is defined as fatigue sensitive; and fatigue must be considered. Thus, locations that are subjected to a net tensile stress due to two times the factored fatigue truck must be checked for fatigue using the fatigue truck loading.

3.5.7.2 Application

One cycle of stress range in girders shall be computed as the algebraic difference between the minimum and maximum stress due to a single passage of the factored fatigue truck along any path vehicular traffic is permitted.

One cycle of stress for transverse members shall be defined as seventy-five (75) percent of the stress range determined by the passage of the factored fatigue truck in two different transverse positions. However, in no case shall the range of stress in transverse members be less than the stress range due to a single passage of the factored fatigue truck. If the maximum stress in a transverse member is caused by a single axle, the number of cycles of stress range shall be equal to two times the number of truck passages.

3.6 THERMAL LOADS

Load effects in the superstructure shall be determined for uniform temperature changes as specified in **AASHTO Article 3.16**.

A uniform temperature difference of 25 degrees Fahrenheit between the deck and the girders shall be considered when the width of the deck is less than one-fifth of the longest span. The load effects due to the temperature differential shall be added to the effects due to the temperature changes specified in **AASHTO Article 3.16**.

C3.5.7.2

The maximum bending stress in girders is usually produced with the vehicle placed near the point under consideration; the minimum stress is usually produced with the vehicle placed in an adjacent span. The fatigue truck is placed in the critical transverse position; but it is not permitted to double back to a different transverse position.

The maximum shear range is usually produced with the fatigue truck placed to the left and to the right of the point under consideration. The vehicle is often positioned in opposing directions to compute the shear range, although this convention implies two passages of the vehicle.

Cross frames and diaphragms connecting adjacent girders are stressed when one girder deflects with respect to the adjacent girder connected by the cross frame or diaphragm. The sense of stress is reversed when the vehicle is placed over the adjacent girder. These two transverse positions of the vehicle usually create the largest stress range in transverse members. To cause such a stress cycle, two vehicles traverse the bridge in separate lanes; one vehicle leading the other. The provisions account for such a cycle by computing the stress range assuming two transverse positions and applying a factor of 0.75 to account for the probability of two vehicles being located in the critical relative position. In no case is the computed range of stress to be less than the stress range due to a single passage of the factored fatigue truck. There is no allowance for the fact that two trucks are required. When the design fatigue life is less than the fatigue limit, the Engineer may wish to consider a reduction in the number of cycles.

C3.6

Although temperature changes in a bridge do not occur uniformly, bridges generally are designed for an assumed uniform temperature change. The orientation of bearing guides and the freedom of bearing movement is important to thermal forces. Sharp curvature and sharply skewed supports can cause excessive lateral thermal forces at supports if only tangential movement is permitted. Wide bridges are particularly prone to large lateral thermal forces because the bridge expands radially as well as longitudinally.

Orienting bearing guides toward a "fixed point" and allowing the bridge to move freely along rays emanating from the fixed point causes thermal forces to be zero if the structure changes temperature uniformly. However, there are other load conditions that may affect decisions regarding bearing orientation.

The requirement to consider a 25 degree Fahrenheit temperature difference between deck and steel addresses the relative time lag that occurs with regard to temperature changes. Since the steel is less massive than the concrete, it changes temperature faster (*Krauss and Rogalla, 1996*). This temperature difference is

particularly important in narrow bridges where a significant difference in vertical reactions may occur.

There is evidence that this temperature differential may contribute to early deck transverse cracking (*Krauss and Rogalla, 1996*). The engineer may wish to include the deck stress from this loading when checking the need for reinforcement according to Article 2.4.3.

SECTION 4

STRUCTURAL ANALYSIS

4.1 GENERAL

The moments, shears and other load effects required to proportion the superstructure components shall be based on a rational analysis of the entire superstructure.

The entire superstructure, including bearings, shall be considered as an integral structural unit. Analyses may be based on elastic small-deflection theory, unless more rigorous approaches are deemed necessary by the Engineer.

Analyses shall consider bearing orientation and restraint of bearings afforded by the substructure. These load effects shall be considered in designing bearings, cross frames, diaphragms, bracing and deck.

4.2 NEGLECT OF CURVATURE EFFECTS

4.2.1 I-Girders

The effects of curvature may be ignored in the determination of vertical bending moment in I-girder bridges when the following three conditions are met:

- Girders are concentric,
- Bearing lines are not skewed more than 10 degrees from radial, and
- The arc span divided by the girder radius is less than 0.06 radians where the arc span, L_{as} , shall be taken as follows:

For simple spans:

$$L_{as} = \text{arc length of the girder,}$$

For end spans of continuous members:

$$L_{as} = 0.9 \text{ times the arc length of the girder,}$$

For interior spans of continuous members:

$$L_{as} = 0.8 \text{ times the arc length of the girder.}$$

C4.1

Since equilibrium of curved I-girders is developed by the transfer of load between girders, the analysis must recognize the integrated behavior of all structural components. Equilibrium of curved box girders may be less dependent on the interaction between girders. Bracing elements are considered primary members in curved bridges since they transmit forces necessary to provide equilibrium.

The deck acts in bending, vertical shear and horizontal shear. Torsion increases the horizontal deck shear, particularly in box girders. The lateral restraint of the bearings may also cause horizontal shear in the deck.

Small-deflection theory is adequate for the analysis of most curved girder bridges. However, curved I-girders are prone to deflect laterally when the girders are insufficiently braced during erection. This behavior may not be well recognized by small-deflection theory.

C4.2

This article applies only to vertical bending moment. It does not apply to lateral flange bending, torsion or shear that should always be examined with respect to curvature.

C4.2.1

If a line girder analysis is appropriate, the wheel load distribution factor of $S/5.5$ is to be used because the new methods in the 1994 AASHTO Guide Specifications for Distribution of Loads for Highway Bridges and in **AASHTO LRFD** may underestimate live load moments in the girder on the outside of the curve (*Hall and Yoo, 1996b*). Non-composite dead load preferably is to be distributed to the girders using a simple span assumption since the deck is not effective. Certain dead loads applied to the composite bridge may be distributed uniformly to the girders as provided in **AASHTO Article 3.23.2.3.1.1**. However, heavier concentrated line loads such as parapets, sidewalks, barriers or sound walls should not be distributed equally to the girders. Engineering judgment must be used in determining the distribution of these loads. Often the largest portion of the load on an overhang is assigned to the exterior girder. The exterior girder on the outside of the curve is often critical in curved girder bridges.

The effect of curvature on the torsional behavior of a girder must be considered regardless of the amount of curvature since stability and strength of curved girders is different from that of straight girders, (*Hall and Yoo, 1996b*). If a line girder analysis is appropriate and the

An I-girder in a bridge satisfying these criteria may be analyzed as an individual straight girder with span length equal to the arc length. Live load shall be applied to each I-girder assuming a wheel-load distribution factor of $S/5.5$ for computing vertical bending moment, shear and vertical deflection. Non-composite dead loads preferably shall be applied to each girder by assuming a simple-span distribution of load between girders.

The effect of curvature on strength and stability shall be considered for all curved I-girders.

Cross frame members shall be designed in accordance with Articles 9.3 and 11.1 for forces computed by rational means.

Cross frame spacing shall be set to limit lateral flange bending in the girders.

If a line girder analysis is appropriate, Equation (4-1) may be appropriate for determining the lateral bending moment in I-girder flanges due to curvature.

$$M_{\text{lat}} = \frac{6 M \ell^2}{5RD} \quad \text{Eq. (4-1)}$$

where:

M_{lat}	=	lateral flange bending moment (k-ft)
M	=	vertical bending moment (k-ft)
ℓ	=	unbraced length (ft)
R	=	girder radius (ft)
D	=	web depth (in)

4.2.2 Closed Box and Tub Girders

The effect of curvature may be ignored in the determination of vertical bending moment in box-girder bridges when the following three conditions are met:

- Girders are concentric,
- Bearings are not skewed, and
- The arc span divided by the girder radius is less than 0.3 radians and the girder depth is less than the width of the box at mid-depth where the arc span, L_{as} , shall be taken as defined in Article 4.2.1

The effect of curvature on strength and stability shall be considered for all curved box girders

4.3 METHODS

4.3.1 Approximate Methods

Approximate analysis methods may be used for analysis of curved girder bridges. The Engineer shall ascertain that the approximate analysis method used is appropriate by confirming that results from the analysis are consistent with the principles defined in Article 1.2.

cross frame spacing is nearly uniform, Equation (4-1) may be appropriate for determining the lateral flange moment in I-girder flanges due to curvature. Although the depth to be used in computing the lateral flange moment from Equation (4-1) is theoretically equal to the depth, h , between the mid-thickness of the top and bottom flanges, for simplicity, the web depth, D , is conservatively used in Equation (4-1). The Engineer may substitute the depth, h , for D in Equation (4-1), if desired.

Other conditions that produce torsion, such as skew, should be dealt with by other analytical means which generally involve a refined analysis.

Bridges with even slight curvature may develop large radial forces at the abutment bearings. Therefore, thermal analysis of all curved bridges is recommended.

C4.2.2

Although box-shaped girders have not been examined as carefully as I-girders with regard to approximate methods, bending moments in closed girders are less affected by curvature than are I-girders (*Tung and Fountain, 1970*). However, torsion is much greater so web shears are affected by torsion due to curvature, skew or loads applied away from the shear center of the box. Double bearings resist significant torque compared to a box-centered single bearing.

If the box is haunched or tapered, the shallowest girder depth should be used in conjunction with the narrowest width of the box at mid-depth in determining whether the effects of curvature may be ignored in calculating the vertical bending moment.

C4.3

C4.3.1

The V-load method (*U.S. Steel Corporation, 1984*) has been a widely used approximate method for analyzing horizontally curved I-girder bridges. The method assumes that the internal torsional load on the bridge—resulting solely from the curvature—is resisted by self-equilibrating sets of shears between flanges

cent girders. The V-load method does not directly account for sources of torque other than curvature, and the method does not account for the horizontal shear stiffness of the concrete deck. The method is only valid for loads such as normal highway loadings. For exceptional loadings, a more refined analysis is required. The method assumes a linear distribution of girder shears across the bridge section; thus, the girders at a given cross section should have approximately the same vertical stiffnesses. The V-load method is also not directly applicable to structures with reverse curvature or to a closed-framed system with horizontal lateral bracing near or in the plane of one or both flanges. The V-load method does not directly account for girder twist; thus, lateral deflections, which become important on bridges with large spans and/or sharp skews, and vertical deflections, may be significantly underestimated. In certain situations, the V-load method may not detect uplift at end bearings. The method is best suited for preliminary design, but may also be suitable for final design of structures with radial supports or supports skewed less than approximately 10 degrees.

The M/R method provides a means to account for the effect of curvature in curved box-girder bridges. The method and suggested limitations on its use are discussed by Tung and Fountain (1970).

Vertical bearing reactions at interior supports on the concave side of continuous-span bridges may be significantly underestimated by both of the above methods.

Strict rules and limitations on the applicability of both of these approximate methods do not exist. The Engineer must determine when approximate methods of analysis are appropriate.

4.3.2 Refined Methods

Refined methods of analysis shall be used unless the Engineer determines that approximate methods referred to in Article 4.3.1 are acceptable.

If stresses, rather than girder actions such as moments and shears are determined from an analysis, the stresses may be compared directly to permitted service stresses or permitted fatigue stress ranges.

C4.3.2

Refined analysis methods are generally computer based. The finite strip and finite element methods have been the most common. The finite strip method is less rigorous than the finite element method and has fallen into disuse with the advent of more powerful computers. Finite element programs may provide for grid analyses using a series of beam elements connected in a plane. Refinements of the grid model may include offset elements. Frequently, the torsional warping degree of freedom is not available in beam elements. The finite element method may be applied to a three-dimensional model of the superstructure. A variety of elements may be used in this type of model. The three-dimensional model may be made capable of recognizing warping torsion by modeling each girder cross section with a series of elements.

The stiffness of supports, including any lateral restraint such as that offered by integral abutments or integral piers, should be recognized in the analysis. Since bearing restraint is offset from the neutral axis of the girders, large lateral forces at the bearings often occur and may create significant bending in the girders.

The restraint of girders at supports may be modeled