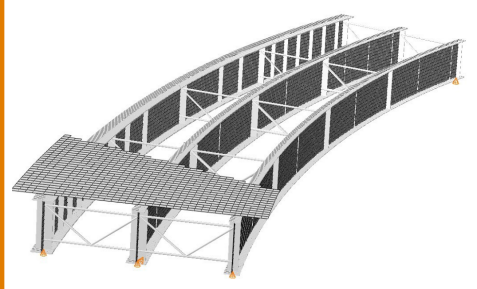


Guidelines for Steel Girder Bridge Analysis

G13.1-2019



AMERICAN ASSOCIATION
OF STATE HIGHWAY AND
TRANSPORTATION OFFICIALS
AASHTO



American Association of State Highway and
Transportation Officials
National Steel Bridge Alliance
AASHTO/NSBA Steel Bridge Collaboration

AVA Documents
Document Sharing Hub

PREFACE

This document is a standard developed by the AASHTO/NSBA Steel Bridge Collaboration. The primary goal of the Collaboration is to achieve steel bridge design and construction of the highest quality and value through standardization of the design, fabrication, and erection processes. Each standard represents the consensus of a diverse group of professionals.

It is intended that Owners adopt and implement Collaboration standards in their entirety to facilitate the achievement of standardization. It is understood, however, that local statutes or preferences may prevent full adoption of the document. In such cases Owners should adopt these documents with the exceptions they feel are necessary.

Cover graphics courtesy of HDR.

DISCLAIMER

The information presented in this publication has been prepared in accordance with recognized engineering principles and is for general information only. While it is believed to be accurate, this information should not be used or relied upon for any specific application without competent professional examination and verification of its accuracy, suitability, and applicability by a licensed professional engineer, designer, or architect.

The publication of the material contained herein is not intended as a representation or warranty of the part of the American Association of State Highway and Transportation Officials (AASHTO) or the National Steel Bridge Alliance (NSBA) or of any other person named herein, that this information is suitable for any general or particular use or of freedom from infringement of any patent or patents. Anyone making use of this information assumes all liability arising from such use.

Caution must be exercised when relying upon other specifications and codes developed by other bodies and incorporated by reference herein since such material may be modified or amended from time to time subsequent to the printing of this edition. The authors and publishers bear no responsibility for such material other than to refer to it and incorporate it by reference at the time of the initial publication of this edition.



Copyright © 2019 by the AASHTO/NSBA Steel Bridge Collaboration
All rights reserved.

ISBN: 978-1-56051-744-3

PUB CODE: NSBASGBA-3

AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS

444 North Capitol Street, N.W. Suite 249

Washington, D.C. 20001

EXECUTIVE COMMITTEE

2019–2020

OFFICERS:

PRESIDENT: Patrick McKenna, Missouri*

VICE PRESIDENT: Victoria Sheehan, New Hampshire*

SECRETARY–TREASURER: Scott Bennett, Arkansas

EXECUTIVE DIRECTOR: Jim Tymon, Washington, D. C.

REGIONAL REPRESENTATIVES:

REGION I: Vacant
Diane Gutierrez-Scaccetti, New Jersey

REGION II: Melinda McGrath, Mississippi
Kevin Thibault, Florida

REGION III: Mark Lowe, Iowa
Craig Thompson, Wisconsin

REGION IV: Kyle Schneweis, Nebraska
James Bass, Texas

IMMEDIATE PAST PRESIDENT: Carlos Braceras, Utah

***Elected at the 2019 Annual Meeting in St. Louis, Missouri**

This page intentionally left blank.

AASHTO COMMITTEE ON BRIDGES AND STRUCTURES, 2019

CARMEN SWANWICK, *Chair*

SCOT BECKER, *Vice Chair*

JOSEPH L. HARTMANN, Federal Highway Administration, *Secretary*

PATRICIA J. BUSH, *AASHTO Liaison*

ALABAMA, Eric J. Christie, William “Tim”

Colquett, Randall B. Mullins

ALASKA, Leslie Daughtery, Elmer E. Marx, Richard

A. Pratt

ARIZONA, David B. Benton,

David L. Eberhart, Pe-Shen Yang

ARKANSAS, Charles “Rick” Ellis, Mike Hill, Joe

Santini

CALIFORNIA, Thomas A. Ostrom, Gedmund

Setberg, Dolores Valls

COLORADO, Behrooz Far, Stephen Harelson,

Jessica Martinez

CONNECTICUT, Mary E. Baker, Timothy D. Fields

DELAWARE, Jason Arndt, Jason Hastings, Craig A.

Stevens

DISTRICT OF COLUMBIA, Donald L. Cooney,

Konjit C. “Connie” Eskender, Richard Kenney

FLORIDA, Sam Fallaha, William Potter, Jeff

Pouliotte

GEORGIA, Clayton Bennett, Bill DuVall, Steve

Gaston

HAWAII, James Fu, Kevin Murata, John Williams

IDAHO, Matthew Farrar

ILLINOIS, Tim A. Armbrrecht, Carl Puzey, Jayme

Schiff

INDIANA, Andrew Fitzgerald, Jeremy Hunter, Anne

M. Rearick

IOWA, Ahmad Abu-Hawash, James S. Nelson

KANSAS, Mark E. Hoppe, John P. Jones, Curt F.

Niehaus

KENTUCKY, Bart Asher, Andy Barber, Marvin

Wolfe

LOUISIANA, Arthur D’Andrea, Paul Fossier,

Zhengzheng “Jenny” Fu

MAINE, Jeffrey S. Folsom, Wayne Frankhauser,

Michael Wight

MARYLAND, Maurice Agostino, Jesse Creel, Jeffrey

L. Robert

MASSACHUSETTS, Alexander K. Bardow, Thomas

Donald, Joseph Rigney

MICHIGAN, Matthew Chynoweth, Rebecca Curtis,

Richard E. Liptak

MINNESOTA, Arielle Ehrlich, Ed Lutgen,

Kevin Western

MISSISSIPPI, Aaron Cagle, Justin Walker, Scott

Westerfield

MISSOURI, Dennis Heckman, Greg E. Sanders,

Scott Stotlemeyer

MONTANA, Amanda Jackson, Dustin E. Rouse

NEBRASKA, Mark Ahlman, Fouad Jaber, Mark J.

Traynowicz

NEVADA, Troy Martin, Jessen Mortensen

NEW HAMPSHIRE, Robert Landry, David L. Scott

NEW JERSEY, Xiaohua “Hannah” Cheng, Eddy

Germain, Dave D. Lambert

NEW MEXICO, Kathy Crowell, Jeff C. Vigil

NEW YORK, Brenda Crudele, Ernest Holmberg,

Richard Marchione

NORTH CAROLINA, Brian Hanks,

Scott Hidden, Girchuru Muchane

NORTH DAKOTA, Jon D. Ketterling, Jason R.

Thorenson

OHIO, Alexander B.C. Dettloff,

Timothy J. Keller, Jeffrey Syar

OKLAHOMA, Steven Jacobi, Walter Peters, Tim

Tegeler

OREGON, Bruce V. Johnson, Albert Nako,

Tanarat Potisuk

PENNSYLVANIA, James M. Long, Thomas P.

Macioce, Lou Ruzzi

PUERTO RICO, (Vacant)

RHODE ISLAND, Georgette Chahine, Keith Gaulin

SOUTH CAROLINA, Terry B. Koon, Jeff Sizemore

SOUTH DAKOTA, Steve Johnson, Dave Madden,

Todd S. Thompson

TENNESSEE, John S. Hastings, Ted A. Kniazewycz

TEXAS, Bernie Carrasco, Jamie F. Farris, Gregg A.

Freeby

U.S. DOT, Joseph L. Hartmann

UTAH, Rebecca Nix, Carmen Swanwick, Cheryl
Hersh Simmons

VERMONT, Kristin M. Higgins, Jim Lacroix

VIRGINIA, Prasad L. Nallapaneni,
Kendal R. Walus, Andrew M. Zickler

WASHINGTON, Tony M. Allen,
Mark A. Gaines, Bijan Khaleghi

WEST VIRGINIA, Ahmed Mongi, William Varney

WISCONSIN, Scot Becker,
William C. Dreher

WYOMING, Jeff R. Booher, Paul G. Cortez,
Gregg C. Frederick, Michael E. Menghini

DELAWARE RIVER AND BAY AUTHORITY,
Shoukry Elnahal

MDTA, Dan Williams

MULTNOMAH COUNTY
TRANSPORTATION DIVISION,
Jon Henrichsen

N.Y. STATE BRIDGE AUTHORITY, William Moreau

U.S. ARMY CORPS OF ENGINEERS—
DEPARTMENT OF THE ARMY, Phillip
W. Sauser

U.S. COAST GUARD, Kamal Elnahal
U.S. DEPARTMENT OF AGRICULTURE—
FOREST SERVICE, John R. Kattell

ALBERTA, Lloyd Atkin

KOREA, Eui-Joon Lee, Sang-Soon Lee

SASKATCHEWAN, Howard Yea

TRANSPORTATION RESEARCH
BOARD, Waseem Dekelbab

AASHTO/NSBA STEEL BRIDGE COLLABORATION TASK GROUP 13, ANALYSIS OF STEEL GIRDER BRIDGES

DEANNA NEVLING, Michael Baker International, *Chair*
DOMENIC COLETTI, HDR, *Editor, G13.1*

Shane Beabes, AECOM
Bradley Bell, Michael Baker International
Allan Berry, RS&H
Frank Blakemore, Garver
Matthew Bunner, HDR
Travis Butz, Burgess&Niple
Terry Cakebread, LUSAS
Nicholas Cervo, HDR
Brandon Chavel, NSBA
Bob Cisneros, High Steel
Domenic Coletti, HDR
Rob Connor, Purdue University
Allen Crozier, HDR
Jason Doughty, Modjeski&Masters
Jason Dreyer, Oates Associates
Lian Duan, CALTRANS
Jamie Farris, Texas DOT
Karl Frank, Consultant
Christina Freeman, Florida DOT
Chung Fu, Optimate
Michael Garlich, Collins Engineering
Christopher Garrell, NSBA
Walter Gatti, Tensor Engineering
Dennis Golabek, WSP
Michael Grubb, MA Grubb&Associates
Jawad Gull, WSP
Todd Helwig, University of Texas at Austin
Dongzhou Huang, Atkins
Russell Jeck, Perini
Greg Kochersperger, HDR
Travis Konda, HNTB
Brian Kozy, Federal Highway Administration
Scott Krause, High Steel

Dan Linzell, University of Nebraska Lincoln
Bob Magliola, Parsons
Natalie McCombs, HNTB
Bill McEleney, Consultant
Ronnie Medlock, High Steel
Duane Miller, Lincoln Electric
Thomas Murphy, Modjeski&Masters
Glenn Myers, Atkins
Walid Najjar, WSP
Deanna Nevling, Michael Baker International
Thanh Nguyen, RS&H
Josh Orton, CDM Smith
Duncan Paterson, HDR
Stephen Percassi, Bergman
Anthony Ream, HDR
Sougata Roy, Rutgers University
Frank Russo, Michael Baker International
Andres Sanchez, Adstren
Kevin Sear, AECOM
Tony Shkurti, HNTB
Jason Stith, Michael Baker International
Jeff Svatora, HDR
Todd Ude, Parsons
John Vogel, Houston Ship Channel Bridge
Steve Walsh, Upstate Detailing
Dayi Wang, Federal Highway Administration
Wagdy Wassef, WSP
Donald White, Georgia Tech University
Brian Wolfe, Maryland DOT
John Yadlosky, HDR
Quihong Zhao, Tianjin University
Yuan Zhao, Burns & McDonnell

This page intentionally left blank.

FOREWORD

The First Edition of *G13.1, Guidelines for Steel Girder Bridge Analysis* was originally published in 2011 and represented a comprehensive treatment of issues related to steel girder bridge analysis, but the guidance presented was largely qualitative.

In the time after the writing of the First Edition, the National Cooperative Highway Research Program (NCHRP) completed NCHRP Research Project 12-79, the results of which are documented in NCHRP Report 725, *Guidelines for Analysis Methods and Construction Engineering of Curved and Skewed Steel Girder Bridges*. The research included extensive analytical studies of over 70 different steel girder bridges, comparing the accuracy results of a variety of one-dimensional (1D), two-dimensional (2D), and three-dimensional (3D) analysis methods, and leading to recommendations on appropriate levels of analysis based on the geometric complexity of a given steel girder bridge. Important findings about the nature of current 2D analysis methods were also reported, along with recommended improvements to the modeling of I-girder torsional stiffness and truss-type cross-frame stiffness to increase the accuracy of 2D methods and a recommended method for estimating I-girder flange lateral bending stresses in straight, skewed bridges analyzed using 2D methods. Preliminary findings concerned with the estimation of locked-in force effects and fit-up forces associated with the chosen cross-frame detailing method were also presented. The key findings reported in NCHRP Report 725 were summarized and included as revisions to G13.1 and formed the majority of the changes in the Second Edition. Other changes included: clarifications to the text on prediction of deflections and load rating analyses, incorporation of recommendations on the impact of connection stiffness on cross-frame stiffness, incorporation of recommendations on global second-order amplification of structural response and narrow system stability analysis, and other minor editorial corrections.

The revisions incorporated in the Third Edition of G13.1 again largely reflect summaries of advances in the state of knowledge gained since the publication of the Second Edition. These advances include improved knowledge of steel I-girder fit resulting from the findings and recommendations of NCHRP Research Project 20-07, Task 355, improved methods for evaluating global stability and global second-order amplification of structural responses, the implementation of minimum strength and stiffness requirements for stability bracing of steel I-girder bridges (particularly in straight bridges with little or no skew), improved guidance regarding the effect of connection eccentricity on the stiffness of cross-frames, and significant advancements and refinements in the classification of members as System Redundant Members or Internally Redundant Members, providing objective methods to avoid the costs and complications associated with classification of bridges as fracture-critical. Other editorial changes and clarifications of previous text were also addressed.

AASHTO/NSBA Steel Bridge Collaboration Task Group 13, Fall 2019

This page intentionally left blank.

TABLE OF CONTENTS

SECTION 1—Modeling Description.....	1-1
1.1—Introduction	1-1
1.2—Hand Analysis Methods.....	1-1
1.2.1—Beam Charts	1-1
1.2.2—Line Girder Analysis Method.....	1-1
1.2.3—V-Load Method.....	1-1
1.2.4—M/R-Load Method.....	1-2
1.3—Finite Element Method	1-2
1.3.1—Basic Concept.....	2-2
1.3.2—D Grid Analysis Method	2-2
1.3.3—Plate and Eccentric Beam Analysis Methods.....	2-2
1.3.4—Generalized Grid Analysis Method.....	2-3
1.3.5—3D FEM Analysis Methods.....	2-3
1.3.6—Element Types.....	2-4
SECTION 2—HISTORY OF STEEL GIRDER BRIDGE ANALYSIS	2-1
2.1—Introduction	2-1
2.2—History of Change in the Complexity of Bridges	2-1
2.3—Bridge Design Code Advancements.....	2-2
2.4—Bridge Analysis Advancements.....	2-3
2.5—Concurrent Initiatives Parallel to This Guideline Document.....	2-3
SECTION 3—ISSUES, OBJECTIVES, AND GUIDELINES COMMON TO ALL STEEL GIRDER BRIDGE ANALYSES	3-1
3.1—Behavior Considerations.....	3-1
3.1.1—Behavior of Tangent, Non-Skewed Steel Girder Bridges	3-1
3.1.2—Behavior of Curved and/or Skewed Steel Girder Bridges.....	3-1
3.1.2.1—Torsional Stress Effects.....	3-1
3.1.2.2—Flange Lateral Bending	3-4

3.1.2.3—Torsional Deformation Effects.....	3-5
3.1.2.4—Load Shifting.....	3-5
3.2—Section Property Modeling Considerations	3-7
3.2.1—Section Properties as Related to Loading Levels in New Construction	3-7
3.2.1.1—Level 1: Erection of Structural Steel Framing.....	3-7
3.2.1.2—Level 2: Placement of Structural Deck Slab.....	3-8
3.2.1.3—Level 3: Placement of Appurtenances	3-9
3.2.1.4—Level 4: Bridge In-Service Condition	3-10
3.2.2—Section Properties as Related to Phased Construction	3-10
3.2.3—Section Properties as Related to Evaluation of Existing Bridges	3-11
3.3—Loads on the Permanent Structure	3-11
3.3.1—Dead Load—Weight of Structural Steel	3-13
3.3.2—Dead Load—Weight of Deck Forming System	3-13
3.3.3—Dead Load—Weight of Concrete Deck	3-13
3.3.4—Dead Load—Barriers and Sidewalks	3-14
3.3.5—Dead Load—Future Wearing Surface	3-14
3.3.6—Dead Load—Utilities and Other Appurtenances.....	3-15
3.3.7—Live Loads.....	3-15
3.3.8—Construction Loads	3-16
3.3.9—Dynamic Load Allowance (Impact)	3-16
3.3.10—Centrifugal Force.....	3-17
3.3.11—Braking.....	3-17
3.3.12—Wind on Structure	3-17
3.3.13—Wind on Live Load	3-18
3.3.14—Uniform Thermal Contraction or Expansion.....	3-18
3.3.15—Thermal Gradient	3-18
3.4—Strength Design	3-18
3.5—Inelastic Design	3-19
3.6—Fatigue Analysis and Evaluation	3-20
3.7—Superstructure Live Load Reactions for Substructure Design.....	3-21

3.8—Constructability—Analysis Issues.....	3-21
3.8.1—Erection of Steel Framing.....	3-21
3.8.1.1—General Guidance.....	3-22
3.8.1.2—Analysis Techniques	3-22
3.8.1.3—Investigation of Steel Erection Sequence.....	3-23
3.8.2—Deck Placement Sequence	3-26
3.8.3—Overhang Analysis and Effects on Girders	3-27
3.8.3.1—Selection of Overhang Width versus Beam Spacing.....	3-28
3.8.3.2—Selection of Overhang—Effect on Forming.....	3-28
3.8.3.3—Overhang Effects on Structural Design.....	3-29
3.8.4—Wind Loading during Construction.....	3-31
3.8.5—Live Loads during Construction.....	3-32
3.8.5.1—Bridge Construction	3-33
3.8.5.2—Bridge Rehabilitation or Demolition.....	3-33
3.8.6—Miscellaneous Construction Loading.....	3-33
3.8.7—Stability Analysis during Various Stages of Construction.....	3-34
3.8.8—Analysis for Incremental Launching of Steel Girders.....	3-35
3.9—Prediction of Deflections	3-35
3.9.1—General	3-35
3.9.2—Skewed Bridges.....	3-37
3.9.3—Curved Bridges.....	3-38
3.9.4—Final Deflections (Vibration, Dynamic Response)	3-38
3.10—Detailing of Cross-Frames and Girders for the Intended Erected Position	3-40
3.10.1—Detailing Methods	3-43
3.10.2—Straight, Skewed I-Girder Bridges	3-48
3.10.3—Curved I-Girder Bridges.....	3-49
3.10.4—Tub Girders	3-51
3.10.5—Calculation of Locked-In Force Effects Due to Cross-Frame Detailing.....	3-52
3.10.6—Estimation of Fit-Up Forces	3-53
3.11—Cross-Frame Modeling (2D versus 3D)	3-54

3.11.1—Impact of Connection Stiffness on Cross-Frame Stiffness.....	3-55
3.11.2—Simplified Euler–Bernoulli Approximations of the Stiffness of Truss-Type Cross-Frames	3-56
3.11.3—Shear-Deformable (Timoshenko) Beam Approach for Modeling the Stiffness of Truss-Type Cross-Frames	3-58
3.11.4—“Exact” Modeling the Stiffness of Truss-Type Cross-Frames	3-61
3.12—Modeling the Torsional Stiffness of I-Girders.....	3-61
3.13—Deck Modeling (e.g., Deck Effective Width, Composite Action).....	3-62
3.14—Bearings, Substructures, and Boundary Conditions for Models.....	3-64
3.14.1—Rollers versus Pins	3-64
3.14.2—Fixed versus Guided versus Free Bearings	3-68
3.14.3—Unusual Substructures and the Effect of Variable Substructure Stiffness	3-71
3.14.3.1—Straddle Bents and Other Variable Substructure Stiffness Effects.....	3-71
3.14.3.2—Integral Straddle Bents	3-74
3.14.3.3—Integral Pier Caps	3-76
3.14.3.4—Integral Abutments.....	3-79
3.15—Roadway/Structure Geometry Considerations.....	3-83
3.15.1—Suggestions to Simplify Structure Geometry in Curved and/or Skewed Bridges.....	3-84
3.15.2—Superelevation Effects on Loading	3-88
3.16—Second-Order Effects	3-90
3.16.1—Second-Order Analysis Methods.....	3-91
3.16.2—Analysis Assumptions versus Observed Behavior	3-92
3.16.3—Application.....	3-92
3.16.3.1—Straight Steel Girders, Trusses, and Arches	3-94
3.16.3.2—Curved Steel Girders	3-94
3.16.3.3—Narrow Steel Girder Systems	3-95
3.16.3.4—Global Second-Order Amplification and Global System Buckling	3-95
3.16.3.5—Local Second-Order Amplification of Flange Lateral Bending between Cross-Frames.....	3-100
3.17—Phased Construction, Redecking, and Widening	3-101
3.17.1—Consideration of Permanent (Dead Load) Deflections	3-101

3.17.2—Consideration of Transient (Live-Load) Deflections during Phased Construction...	3-102
3.17.3—Stability of Narrow Sections during Interim Phases of Construction	3-103
3.17.4—Considerations for Cross-Frames between New and Existing Sections.....	3-103
3.18—Temperature Effects	3-105
3.18.1—Uniform Temperature Changes	3-105
3.18.2—Temperature Gradients	3-105
3.18.3—Temperature Effects during Construction	3-106
3.18.4—Wide and/or Highly-Skewed Bridges.....	3-106
3.19—Analyzing Older Bridges	3-106
3.19.1—Bridge Inspections.....	3-106
3.19.2—Load Rating.....	3-107
3.19.3—Fatigue Evaluation.....	3-108
3.19.3.1—Load-Induced Fatigue.....	3-108
3.19.3.2—Distortion-Induced Fatigue.....	3-108
3.19.4—Bridges with Hinges	3-108
3.19.5—Load Testing as Part of Analysis of Existing/Older Bridges	3-108
3.20—Discontinuities in Structures.....	3-109
3.20.1—Stress Concentrations—Need for Detailed Stress Analysis	3-109
3.20.2—Access Openings	3-110
3.21—References to Benchmark Analysis Problems	3-110
SECTION 4—ANALYSIS GUIDELINES FOR SPECIFIC TYPES OF STEEL GIRDER BRIDGES .	4-1
4.1—Plate Girders—General Issues.....	4-1
4.1.1—Cross-Frame Analysis	4-1
4.1.1.1—Minimum Strength and Stiffness Requirements for Stability Bracing.....	4-1
4.1.1.2—Two-Dimensional Analysis Techniques	4-2
4.1.1.3—Three-Dimensional Analysis Techniques.....	4-3
4.1.2—Lateral Bracing.....	4-4
4.1.3—Narrow Systems, Stability Analysis.....	4-4
4.1.4—Narrow Systems, Redundancy Analysis	4-5
4.1.5—Variable-Depth Girders	4-7

4.1.6—Width-to-Span Ratios and Influence on Secondary Effects	4-7
4.1.7—Improved Calculation of I-Girder Flange Lateral Bending Stresses from 2D Grid Analysis	4-8
4.2—Tangent Steel Plate Girders or Rolled Beams	4-13
4.2.1—No Skew or Limited Skew (<20 Degrees)	4-13
4.2.2—Significantly Skewed.....	4-14
4.2.3—Multiple Different Skews	4-16
4.2.4—Through Girder Bridges	4-17
4.3—Curved Steel Plate Girders or Rolled Beams.....	4-17
4.3.1—Methods of Analysis.....	4-17
4.3.1.1—V-Load Analysis	4-18
4.3.1.2—Grid or Grillage Analysis	4-19
4.3.1.3—3D Finite Element Analysis	4-20
4.3.2—Skewed and Curved I-Girder Bridges	4-22
4.4—Tub Girders—General Issues	4-22
4.4.1—Analysis of Internal Framing (Internal Intermediate Cross-Frames, Top Flange Lateral Bracing)—Grid Analysis and Hand Calculations versus 3D.....	4-24
4.4.2—Consideration of External Intermediate Cross-Frames	4-25
4.4.3—Cross-Frame Modeling (Shear Stiffness, Flexural Stiffness).....	4-25
4.4.4—Narrow Systems, Stability Analysis.....	4-26
4.4.5—Narrow Systems, Redundancy Analysis	4-27
4.4.6—Variable Depth Girders	4-28
4.4.7—Recent Improvements to Simplified Analysis Methods for Tub Girders.....	4-28
4.5—Tangent Steel Tub or Box Girders.....	4-29
4.5.1—No Skew or Limited Skew (<10 Degrees)	4-29
4.5.2—Significantly Skewed.....	4-30
4.5.3—Multiple Different Skews	4-30
4.6—Curved Steel Tub or Box Girders	4-30
4.6.1—Skewed and Curved Steel Tub or Box Girder Bridges.....	4-32
4.7—Bridges with Significantly Complex Framing.....	4-33
4.7.1—Variable Girder Spacing.....	4-33

4.7.2—Discontinuous Girders.....	4-34
4.7.3—Transfer Girders	4-35
4.7.4—Girder-Substringer Systems	4-36
4.7.5—Elevated T-Intersections.....	4-37
4.7.6—Single Point Urban Interchanges	4-38
REFERENCES	R-1
GLOSSARY	G-1
APPENDIX A—SURVEY OF CURRENT PRACTICE	A-1
A1—Background.....	A-1
A2—Participation in the Survey.....	A-1
A3—AASHTO RESPONSES	A-2
A3.1—Overall Trends and Comments	A-2
A3.2—Responses To General Questions	A-2
A3.3—Responses To Specific Questions.....	A-3
A3.3.1—Responses to Questions about Tangent (Straight) Steel Plate Girders or Rolled Beams with No Skew or Limited Skew (Skewed Less than 20 Degrees from Non-Skewed)	A-3
A3.3.2—Responses to Questions about Tangent (Straight) Steel Plate Girders or Rolled Beams, Significantly Skewed (Skewed More than 20 Degrees from Non-Skewed)	A-3
A3.3.3—Responses to Questions about Curved Steel Plate Girders or Rolled Beams with No Skew or Limited Skew (Skewed Less than 20 Degrees from Non-Skewed)	A-4
A3.3.4—Responses to Questions about Curved Steel Plate Girders or Rolled Beams, Significantly Skewed (Skewed More than 20 Degrees from Non-Skewed)	A-5
A3.3.5—Responses to Questions about Tub Girders or Box Girders.....	A-5
A3.3.6—Responses to Questions about Bridges with Significantly Complex Framing Plans (e.g., Variable Girder Spacing, Bifurcation of Girders, Elevated T-Intersections, Single Point Urban Interchanges)	A-6
A4—AREMA RESPONSES.....	A-6
A5—OTHER RESPONSES.....	A-6
APPENDIX B—RECOMMENDATIONS ON METHODS OF ANALYSIS	B-1
B1—INTRODUCTION	B-1
B2—I-GIRDER BRIDGES	B-1
B2.1—I-Girder Bridge Level of Analysis Example 1	B-5

B2.2—I-Girder Bridge Level of Analysis Example 2.....	B-6
B2.3—I-Girder Bridge Level of Analysis Example 3.....	B-7
B3—TUB GIRDER BRIDGES.....	B-8
B3.1—Tub Girder Bridge Level of Analysis Example.....	B-11

SECTION 1—MODELING DESCRIPTION

1.1—INTRODUCTION

The methods used in steel bridge analysis can generally be classified in one of two categories: hand analysis or computer-based numerical analysis—finite element method (FEM). Any analysis or design method that can be performed completely by hand (even if programmed into a spreadsheet or computer program) can be categorized as a hand analysis method. Examples include the line girder method and the V-Load method. The finite element method is the most common numerical method in structure analysis and design but appears in various forms (e.g., grid or grillage analysis versus full three-dimensional finite element analysis).

In reviewing (and choosing) the methods listed below, one should keep in mind the value of performing independent verification analyses. As a general rule, it is good practice to perform some kind of simplified verification of the results of more complex analysis models by means of simpler analysis models, hand calculation, or both. Though this may sometimes seem to be easier said than done (due to the level of complexity of the structure), nonetheless, these types of checks are extremely valuable in that they allow the designer an opportunity to better understand the anticipated behavior of the structure and a method to validate the correctness of the more complicated analysis. It is also advisable for designers to perform a number of simple check calculations directly based on the analysis results. For instance, when performing a 3D FEM analysis, the designer should check that the summation of dead load reactions equals the summation of the applied dead loads, and that the distribution of dead load reactions among the various support points matches the anticipated internal load distribution in the structure.

1.2—HAND ANALYSIS METHODS

1.2.1—Beam Charts

There are a number of standard beam design charts and other design aids which can be of use to the designer. For example, the AISC *Steel Construction Manual* (AISC, 2017) includes a table of beam shear, moment, deflection, and reaction graphs and formulas for the cases of uniform load and point load. While these patterns of loading are typically too simplified to be of direct benefit to the practicing bridge engineer, these design aids can still serve a valuable purpose by providing a handy resource for finding approximate analysis methods for use in preliminary design or in the checking of more complicated analyses.

1.2.2—Line Girder Analysis Method

The line girder analysis method uses load distribution factors to isolate a single girder from the rest of the superstructure system and evaluates that girder individually. The load distribution factors can be simply determined by approximate formulae for both straight bridges (AASHTO, 2017) and curved bridges (Kim et al., 2007) (Zhang et al., 2005).

1.2.3—V-Load Method

The V-Load method (Richardson, Gordon, & Associates, 1976) (United States Steel Corporation, 1984) is 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 shear responses (referred to as secondary) between adjacent girders. The final response in the curved girder is the sum of the secondary response and the respective straight girder primary response.

1.2.4—M/R-Load Method

The M/R Method (Tung and Fountain, 1970) is a method for analyzing horizontally curved steel box girders. This method, similar to the V-Load method, is based on the principles of statics and can be used to estimate the torsional load and associated twist deformations in a curved box girder.

1.3—FINITE ELEMENT METHOD

1.3.1—Basic Concept

The finite element method is also an approximate structural analysis method (Ziemien, 2010). In this method, structures are subdivided generally into a large number of small finite elements. The deformation within each finite element is represented by approximate displacement functions, which are typically simple polynomials. The model is assembled by considering the equilibrium and displacement compatibility conditions at the nodes. The accuracy of the finite element solution is increased generally by increasing the number of elements, or in other words, the finite element mesh density. Engineers should realize that using too few finite elements in bridge analysis may induce a significant error. There are a number of different applications of the finite element method used in bridge analysis; these include 2D grid analysis methods, plate and eccentric beam analysis methods, generalized grid analysis methods, and 3D FEM analysis methods. Each is described in subsequent sections.

1.3.2—2D Grid Analysis Method

This method is also referred to as plane grid or grillage analysis method. This method is often used in steel bridge design and analysis. In this method the structure is divided into plane grid elements. These elements often feature three degrees of freedom at each node (the vertical displacement, and the rotation angles about the longitudinal and transverse axes); if a 2D grid analysis is performed using general FEM software, the elements may feature six or more degrees of freedom at each node (see also Section 1.3.4) although the primary response to gravity loads in a plane grid analysis is addressed by only the three degrees of freedom mentioned above. Modeling parameters are often set following simplified guidelines such as those provided in Articles 4.6.3.3.1 and C4.6.3.3.1 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017). Live load distribution is typically involves the use of live load distribution factors (as provided in Section 4 of the *AASHTO LRFD Bridge Design Specifications*). See also Sections 3.11 and 3.12 of this document for further discussion of recent and future developments in 2D grid modeling.

1.3.3—Plate and Eccentric Beam Analysis Methods

This is a variant of a 2D grid/grillage analysis model. The deck is modeled using plate or shell elements, while the girders and cross-frames are modeled using beam elements offset from the plate elements to

represent the offset of the neutral axis of the girder or cross-frame from the neutral axis of the deck. This approach is discussed in Article C4.6.3.3.1 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017). The offset length is typically equal to the distance between the centroids of the girder and deck sections. This method is somewhat more refined than the traditional 2D grid method in terms of both the stiffness model and the ability of the model to distribute live load based on relative stiffness rather than through live load distribution factors. For this modeling approach, beam and plate element internal forces need to be eccentrically transformed to obtain the composite girder internal forces (bending moment and shear) used in the bridge design.

1.3.4—Generalized Grid Analysis Method

This is a modification of a 2D grid analysis, where more degrees of freedom are modeled and used in the analysis. Some typical enhancements that separate the generalized grid method from the 2D grid method include modeling of cross-frames with consideration of shear deformation in addition to flexural deformation, modeling of girder supports, lateral bracing, cross-frames at their physical elevation within the structure, or combination thereof (Chang et al., 2005) (Huang and Zhang, 2007). In this type of analysis, the warping of open cross-section members (such as I-shaped girders) is included explicitly in the derivation of the element used to model girders. This is an important attribute of this type of analysis, particularly for the analysis of structures where torsion is a significant consideration (such as curved and skewed I-girder bridges). At this time, the Generalized Grid Analysis method is used only for academic research projects.

1.3.5—3D FEM Analysis Methods

The category “3D FEM analysis methods” is meant to encompass any analysis or design method that includes a computerized structural analysis model where the superstructure is modeled fully in three dimensions, including: modeling of girder flanges using line/beam elements or plate/shell/solid type elements; modeling of girder webs using plate/shell/solid type elements; modeling of cross-frames using line/beam, truss, or plate/shell/solid type elements (as appropriate); and modeling of the deck using plate/shell/solid elements. Though this method is arguably deemed the “most accurate” analysis method available to most practicing bridge design engineers, this method is also typically time-consuming and complicated, and is arguably deemed most appropriate for use for complicated bridges (e.g., bridges with severe curvature or skew, or both, unusual framing plans, unusual support/substructure conditions, or other complicating features). 3D analysis methods are also useful for performing refined local stress analysis of complex structural details.

There are also some complicating factors associated with 3D analysis methods. For instance, in a 3D analysis, girder moments and shears are not directly calculated; instead, the model reports stresses in flanges, webs, and deck elements. If the designer wishes to consider girder moments and shears, some type of conversion/integration of the stresses over the depth of the girder cross section will be required. This can be a significant undertaking, particularly with regard to proper proportioning of deck stresses and deck section properties to individual girders.

When and how to use refined 3D FEM analysis for engineering design is a controversial issue, and such an approach has not been fully incorporated into the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) to date. The typical AASHTO methodology for design is generally based on assessment of nominal (average) stresses calculated without consideration of shear lag (e.g., axial stress is calculated as the total axial force divided by the cross-sectional area), and not localized peak stresses obtained by shell- or solid-based finite element models. Refined analysis can provide substantially more

detailed and accurate information about the stress state of the structure and allow for more cost-effective and reliable design, but this often comes with increased engineering effort and increased potential for error. The results are often more sensitive to the input parameters and the mathematical assumptions which are employed by the software. For instance, a given element will have a unique formulation, interpolation, and integration and software implementation, all of which will affect results. The engineer must understand the assumptions and limitations to ensure correct application. These results are often difficult for the engineer to verify directly by independent calculations, and so special procedures must often be employed to verify accuracy of modeling.

1.3.6—Element Types

There is a wide range of different element types used in general finite element practice. Cook et al. (2002) provide a reasonably comprehensive overview. In short, the following definitions are used for purposes of discussion in these guidelines:

Truss Element—A 2D or 3D element in which the responses are solely axial tension/compression along the length of the component. 2D truss elements typically have two translational degrees of freedom at each node, and 3D truss elements typically have three translational degrees of freedom at each node.

Beam Element—A 2D or 3D element in which the responses involve both the axial tension/compression, as in truss elements, as well as structural member flexure and, in the case of 3D elements, structural member torsion. This type of beam element is sometimes referred to as a frame element. Typical beam elements have six degrees of freedom (DOFs) at each node: three translational DOFs and three rotational DOFs. However, additional advanced beam elements can include other DOFs to represent the warping of an open thin-walled cross-section (such elements are not commonly available in professional software applications at the present time). In some circumstances, the term “beam element” may be reserved to represent a 2D element that represents only flexural effects. However, a more general definition involving axial, torsion, and flexure effects is used here.

Plate Element—A 2D element that consists typically of three to nine nodes. The internal element responses generally consist of moments and shears. The result values are usually per unit length of the plate. Plate elements can have various combinations of nodal degrees of freedom.

Shell Element—A 3D element that combines the effects of plate bending as well as membrane effects. Shell elements can be either flat or curved. Small flat shell elements can be used to form curved surfaces.

Brick Element—A 3D element supporting three translational degrees of freedom per node. The number of nodes can range from four to twenty or more. Brick elements generally have three translational degrees of freedom at each node.

It must be stressed that not all elements in a given category perform equally and that differences exist depending on the theories and numerical implementation used by the FEM software developer. Engineers should review the theory manual and verify that the element selected is appropriate to accurately understand and respond to the demands placed on it. Engineers need not be mathematicians and computer programmers, but they must understand that all FEMs have inherent approximations and that some finite element analysis fundamentals, such as element formulation, interpolation, integration, and software implementation, can influence the structural analysis results.

The element formulation refers to the mathematical theory used to define the element's behavior. For instance, shell problems generally fall into one of two categories: thin shell problems or thick shell problems. For a detailed discussion on different shell formulations, as well as proper integration order for the integration of their stiffness matrices, the reader is referred by Bathe (1996). Thick shell problems assume that the effects of transverse shear deformation are important to the solution. Thin shell problems, on the other hand, assume that transverse shear deformation is small enough to be neglected. Thin shell elements provide solutions to shell problems that are adequately described by classical (Kirchhoff) plate theory, thick shell elements yield solutions for structures that are best modeled by shear flexible (Mindlin) plate theory. Mindlin theory-based shell elements are sometimes used in thin shell analysis because they have a less strict continuity requirement on element interpolation functions.

The interpolation refers to the displacement functions that are assumed in the element formulation for describing the deformed shape between the element nodes. It also refers to the approximation and mathematical simplification of the original shape of the structure under investigation. In most cases, the interpolation order is either linear or quadratic. Linear interpolations are arguably the most common type; however, quadratic or higher-order elements are very efficient as the complexity of the domain shape and deformation increases. Quadratic elements are more accurate on a per-element basis; however, their use comes at an increased computational expense since additional nodes are required to adequately describe their shape.

The element integration refers to the number of discrete points within each element that are utilized to calculate the internal strain energy in the deformed configuration, which affects element stiffness matrices following the energy principle in solid mechanics. Shell elements can be either fully integrated or use reduced integration. For full integration, standard Gauss quadrature is typically employed, which results in four integration points for a four-node quadrilateral and three integration points for a three-node triangular element. For reduced integration, only a single integration point is used for each of these elements. Reduced integration elements are attractive because they reduce computational expense while providing a means for mitigating shear-locking effects that become pronounced when shear-deformable shell formulations are used in situations where the through-thickness dimension is small. However, reduced integration elements often exhibit another numerical problem called “hourglassing,” in which the element can deform in certain ways with the internal strain energy remaining zero. Fully-integrated finite elements can in some cases exhibit numerical locking, which is a phenomenon in which the numerical approximation leads to element responses that are so over-stiff that the element becomes practically useless in approximating certain types of response. Some finite element programs have formulations based on the use of separate interpolation functions for internal stresses, displacements, and other specialized procedures. These elements are typically aimed at providing improved accuracy while avoiding spurious zero-energy modes and locking.

Once the element type is selected and verified for use, the engineer should perform a mesh convergence study to ensure that the model is sufficiently refined to yield accurate results.

This page intentionally left blank.

SECTION 2—HISTORY OF STEEL GIRDER BRIDGE ANALYSIS

2.1—INTRODUCTION

Steel girder bridge analysis has evolved in concert with advancement in materials, construction methods, and equipment and with the changing needs of the traveling public and the agency with jurisdiction over the bridge. This section provides a brief discussion of the history of steel highway bridges and introduces the evolution of bridge design codes and analysis techniques currently in use for steel bridge design.

2.2—HISTORY OF CHANGE IN THE COMPLEXITY OF U.S. BRIDGES

The invention of the automobile in the late 1800s and the introduction of mass production techniques had a significant effect on the needs for roadway facilities and the bridges required to cross natural obstacles, such as waterways. The automobile gave the American public the ability to travel greater distances faster and with greater ease than with traditional transportation methods, without the restrictions of location near existing or proposed rail lines.

Medium-span highway bridges from the early 1900s through the 1940s were fairly simple and typically consisted of either single-span or multispans straight bridges aligned normal to the obstacle being crossed in order to minimize the span length of the bridge. Acquisition of right-of-way for transportation facilities was not as difficult or as expensive as it is today, allowing roadways to be aligned in a manner that maximized the economy of the bridge. Spans were typically short and, in many instances, the waterway was constricted with causeway bridge approaches and numerous substructure units within the channel. Many of these bridges consisted of timber and structural steel beam-deck superstructures, as well as masonry arch or cast-in-place concrete superstructures.

Structural steel connection methodologies for highway bridges saw technological advances in the 1940s that allowed greater economy in the production and field construction of steel bridges. Hot-driven rivets that had been utilized since the advent of steel were being replaced by high-strength bolts and welding, made possible with the development of weldable and higher strength steels.

The 1950s saw unprecedented growth of the highway transportation network, driven by urban sprawl, the move toward the suburbs, and the need for an expanded roadway network fueled by increasing interstate commerce and requirements for national security mobility. The most influential federal program during this time period was the birth of the Interstate System. The ever-increasing congestion on highways resulted in an expanding number of grade separation structures used to keep traffic flowing without the constrictions caused by signalized intersections, a necessity for the Interstate System and other expressways under development. As the requirements for increased open spaces under bridges became more prevalent, span lengths increased dramatically, being met by structural steel and the recently introduced precast, prestressed concrete superstructure systems.

As right-of-way became more difficult and expensive to acquire, bridges were sometimes confined by available space, resulting in increased span lengths and usage of skewed and curved steel superstructure. Curved steel structures became more prevalent in direct connections between expressway facilities. Also, as expressways became more crowded and cloverleaf interchange configurations became obsolete due to inadequate accommodation of weaving between entering and exiting traffic, curved steel ramp connections grew in usage to eliminate these weaving conflicts. These

direct connection ramp structures (also known as flyover bridges) allow multiple roadways to cross each other with ease.

Until the 1960s, the typical steel beam deck superstructure consisted of either rolled beams or fabricated steel plate girders. The 1960s saw the advent of the steel box girder section shown by research to have superior torsional properties for bridges on curves and even on tangent bridges due to improved distribution of loads within the bridge. For most highway bridges, the natural application is to use a composite concrete deck as the top of the closed girder system. However, the departments of transportation (DOTs) of some states in the U.S. had concerns about the ability to redeck and inspect tub girders, and so their use was not embraced equally across the country. Later developments included the use of top flange lateral bracing to create a quasi-closed section in the non-composite condition to address some of these concerns.

While the basic configurations of steel bridge superstructures have not changed dramatically over the years, significant advances have been made through research to gain a better knowledge of the behavior of steel bridges of all types. For example, advances in the research of bridge fracture and fatigue characteristics have resulted in the material toughness requirements and stress range design philosophy in the current AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2017), which improved the performance of steel bridges significantly. The development of high-performance steels with higher strength and greater toughness and ductility will also provide greater steel bridge performance now and in the future.

2.3—BRIDGE DESIGN CODE ADVANCEMENTS

With the development of the automotive industry in the early 20th century, bridge design requirements had to be developed to account for this new type of loading. Original highway design codes were synthesized from railroad bridge design practice, where the principle objectives were to ensure the materials remained elastic to control deflections and to ensure a safe, serviceable design. These design methods applied a factor of safety to material yield strength to ensure they were in the elastic range of response during service, and was referred to as the Working (or Allowable) Stress Method. Strength requirements were rarely investigated. These codes included span–depth ratios, which were also adopted by the first design specification developed by AASHTO in 1925 (AASHTO, the American Association of State Highway Officials, was AASHTO's name until 1973).

With the birth of the interstate system, the political need to maximize construction economy and economic competition due to advancements in other materials drove the need for new methods of design. The Load Factor Design method was developed to predict maximum loads through the application of various load factors, then comparing to a reduced strength of the member, assuming some degree of difference between the design specifications and the as-built structure. Certain serviceability checks were also included to ensure serviceability, assessing fatigue, deflection, and other criteria.

The Load Factor Design methodology, however, did not provide the same level of safety for all spans and bridge types. The concept of reliability, using probabilistic methods to provide more uniform strength and serviceability, culminated in the development of the first edition of the Load and Resistance Factor Design (LRFD) used in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 1994). This design methodology establishes load factors and strength reduction factors developed through statistical means, resulting in more uniform assessment of safety for all structure types, no matter what material is used in construction.

As these codes evolved, provisions within these documents also incorporated increased knowledge of the specific behavior of construction materials gleaned from research; the analysis of structural failures; advancements in material strength and performance, e.g., high-performance steel; and the ability to perform more advanced calculations to assess structural performance and characteristics.

2.4—BRIDGE ANALYSIS ADVANCEMENTS

Advancements in the analysis of steel bridges have evolved to provide the engineering background and justification for innovative structural steel systems as they have come into the construction market. Advancement in analytical tools, whether computational methods or computational technology advancements, have allowed the analysis of bridges to be performed in greater detail, allowing more efficient, cost-effective designs to be developed.

Early bridges, being primarily straight and simple-span, were designed using techniques that could easily be accommodated with the computational tools of the day, e.g., slide rules and logarithmic and sine tables. Computational methods were developed from procedures utilized for railroad bridges, particularly for the analysis of moving loads. As the use of continuous spans became more prevalent, tables providing the basis for the development of moving load influence lines were developed as design aids, greatly simplifying the analysis. Another technique used on early bridges to simplify analysis was what some call forced determinate structures. Bridges were designed and detailed with pins or rollers, or both, to allow the assumption of statically determinate behavior, which permitted simple manual computations of forces.

As indeterminate structures became more prevalent, the need for analytical tools for indeterminate, curved, and other structures resulted in the development of procedures such as the V-Load method in the 1960s and the M/R method in the 1970s. These methods allowed statics-based hand methods to be utilized. The advent of computers, consisting of large mainframes, allowed for the development of computational tools to simplify this analysis. These methods were typically utilized for simpler structural configurations (e.g., constant girder spacing) and were very complicated and less accurate for more complex superstructure framing systems; however, these methods allowed designers to avoid complicated 2D grid or 3D finite element analysis that required extensive computer time and money, as most programs were only available as third-party time-share services.

Beginning in the 1970s, the increased usage of computers 1) saw the development of programs to perform the computational analysis of structural steel frames utilizing 2D and 3D structural modeling techniques, and 2) allowed more sophistication in the analysis of simple and complex structural framing systems. Several programs (e.g., SIMON, Merlin-Dash) were developed to perform straight line girder analysis. Other programs (e.g., DESCUS, MDX) were developed to perform frame analysis for curved steel girder bridges.

The 1990s saw the introduction of finite element modeling (e.g., STAAD, SAP, GTSTRUDL) to practical design use, particularly for the more complex structural arrangements.

2.5—CONCURRENT INITIATIVES PARALLEL TO THIS GUIDELINE DOCUMENT

Sometimes when a simplified analysis is performed, certain secondary members of the bridge are omitted and their effect on structural response is approximated using assumptions justified by successful past experience. The consequence of omitting these members from the analytical model can be adverse if complex bridge geometry exists. But the questions remain: How significant is the effect of the modeling choices on the accuracy of the analysis results when compared to the actual behavior

of the structure? For what types of structures, at what span lengths, at what degrees of curvature, and at what severities of skew, are more refined analysis models warranted?

Ongoing and future research efforts will continue to provide more answers to these types questions. Engineers are encouraged to keep abreast of new research findings and design guidance as they appear. Engineers are also encouraged to monitor, or participate in, the activities of various organizations working to provide improved guidance.

One organization specifically dedicated to the better understanding of steel bridge behavior and analysis is the Transportation Research Board (TRB) Committee AFF20(1)—Methods of Analysis of Steel Bridges. TRB is a program unit of the National Academies of Science, Engineering, and Medicine. TRB has numerous committees, including AFF20, the Committee on Steel Bridges. Within AFF20 is a subcommittee, AFF20(1), dedicated to Methods of Analyzing Steel Bridges. This subcommittee meets at least once a year during TRB's Annual Meeting in Washington, DC, and also carries on business during the course of the year via email. AFF20(1) sponsors presentations of current research into steel bridge analysis issues during their annual meeting as well as at AFF20(1)-sponsored workshops and also helps to solicit topics for future research in steel bridge analysis.

In addition to the continuing work of AFF20 and AFF20(1), there are specific research efforts, such as National Cooperative Highway Research Program (NCHRP) Research Project 12-79, Guidelines for Analysis and Construction Engineering of Curved and Skewed Steel Girder Bridges. The associated research report by White et al. (NCHRP Report 725) was published in 2012 (White et al.). The stated goals of this project were to:

1. Quantify when line girder versus grid versus 3D analysis methods are more appropriate,
2. Provide guidelines on when out-of-plumbness or locked-in stresses, or both, should be considered in the analysis, a topic further studied by the later NCHRP Research Project 20-07, Task 355, Reliable Fit-Up of Steel Girder Bridges (White et al., 2015), and
3. Provide guidelines on erection engineering analysis, erection plan detail, review of erection plans, and associated minimum level of erection engineering analysis.

As of the writing of the 3rd edition of this document, various subsequent research efforts are underway or are being discussed to better understand the influence of locked-in stresses, to further clarify aspects such as the influence of the overall bridge geometry, the structural framing, and the type of detailing of the bridge cross-frames on the ease of fitting the components together during steel erection, and to provide improved guidance for the design of cross-frames in terms of their fatigue performance and their function as stability bracing for girders.

Finally, there are several excellent training documents and courses available to designers. Some examples include:

- National Highway Institute (NHI) Courses 130081 and 130095, which are multiple-day courses in LRFD design of straight and curved/skewed steel girder bridges, respectively. Contact the NHI for more details. These courses also feature excellent, very comprehensive reference Manuals (Grubb et al., 2010) (Grubb et al., 2015).
- The Federal Highway Administration (FHWA) *Steel Bridge Design Handbook*, a comprehensive multivolume collection of guideline documents and complete design examples for straight and curved steel I-girder and tub girder superstructures (FHWA, 2015).

SECTION 3—ISSUES, OBJECTIVES, AND GUIDELINES COMMON TO ALL STEEL GIRDER BRIDGE ANALYSES

3.1—BEHAVIOR CONSIDERATIONS

This section provides background information that should be familiar to all bridge engineers before they undertake the design of a steel girder bridge, particularly when the bridge features some level of complexity (e.g., skew, curvature). However, all engineers may benefit from reading it and understanding some of the nuances of the behavior of more complicated bridges.

3.1.1—Behavior of Tangent, Non-Skewed Steel Girder Bridges

All bridges are subject to shear and bending moment effects, as well as vertical deflections, and major axis bending rotations. These effects are familiar to bridge engineers so an extensive discussion of these effects is not warranted here. However, it is important to mention them since they are essential components in the total equation of stress and deformation for curved and/or skewed steel girder bridges (discussed in Article 3.1.2).

3.1.2—Behavior of Curved and/or Skewed Steel Girder Bridges

The behavior of curved and skewed steel girder bridges can be broadly divided into two categories:

The Basics—Curved or skewed steel girder bridges, or both, experience the same effects of gravity loading (dead load and live load) as straight girder bridges (as described in Article 3.1.1).

Curvature and Skew Effects—Torsional St. Venant shear and warping normal stresses, flange lateral bending, load shifting, and twisting deformations.

In the following sections, many of these effects will be characterized as effects of curvature. However, one should note that similar twisting and flange lateral bending effects occur in skewed steel girder bridges as will be described in detail later in these Guidelines.

3.1.2.1—Torsional Stress Effects

In addition to the basic vertical shear and bending effects described in Article 3.1.2, a curved girder also will be subject to torsional effects. In a curved girder, the torsion arises because 1) gravity loads are applied along the length of the girder and 2) these loads are offset from a chord line drawn between the supports for that span. Due to this offset of the loading, resultant from the chord line drawn between the resultant vertical support reactions, a net torsional reaction is required at the supports to satisfy overall equilibrium of the span. This description can also be extended to describe the development of an overall torque on the bridge cross section.

Torsion in steel girders causes normal stresses and shear stresses. I-shaped girders and box-shaped girders carry these stresses in different ways so it is worthwhile to consider them separately.

Because I-shaped girders have low St. Venant torsional stiffness, they carry torsion primarily by means of warping. The total state of normal stress in an I-shaped girder is a combination of any axial stress, major axis bending stress, lateral bending stress, and warping normal stress (Figure 3.1.2.1-1). The total state of shear stress in an I-shaped girder is a combination of vertical shear stress, horizontal shear

stress, some St. Venant torsional shear stress (typically relatively small), and warping shear stress (Figure 3.1.2.1-2).

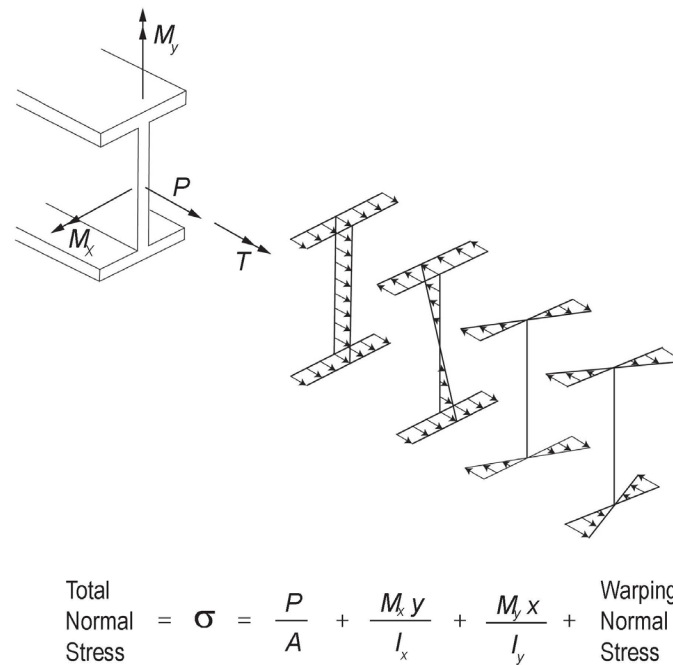


Figure 3.1.2.1-1. Illustration of the general I-girder normal stresses, which can occur in a curved or skewed I-shaped girder.

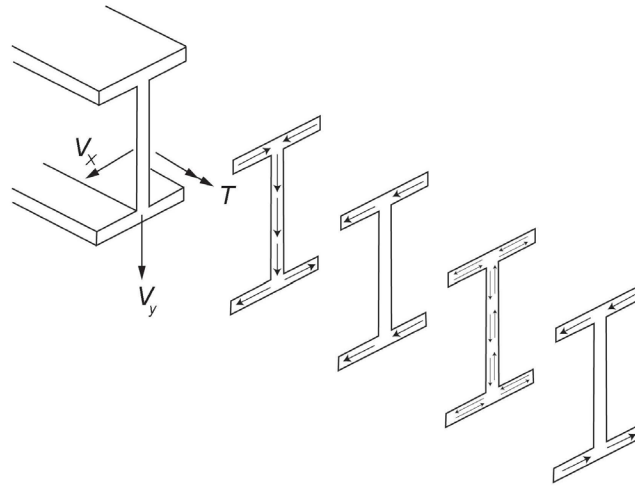
The relatively low St. Venant torsional stiffness of I-shaped girders is a result of their open cross-sectional geometry. The St. Venant torsional shear flow around the perimeter of the cross section can only develop relatively small force couples. Without significant force couples, the ability of I-shaped girders to carry torque via St. Venant torsional response is low.

Box-shaped girders, on the other hand, are closed section members. Closed cells are extremely efficient at carrying torsion by means of St. Venant torsional shear flow because the shear flow around the circumference of the box has relatively large force couple distances (Figure 3.1.2.1-4). For this reason, a box-shaped girder can carry relatively large torques with relatively low shear flows. The shear flow around the circumference of the box follows a consistent direction (clockwise or counterclockwise) at any given location along the length of the girder. As a result, when combined with vertical shear in the webs, this shear flow is always subtractive in one web and additive in the other.

As in an I-shaped girder, the total state of normal stress in a box-shaped girder is a combination of any axial stress, major axis bending stress, lateral bending stress, and warping normal stress (Figure 3.1.2.1-3). The total state of shear stress in a box-shaped girder is a combination of vertical shear stress, horizontal shear stress, St. Venant torsional shear stress and warping shear stress (Figure 3.1.2.1-4).

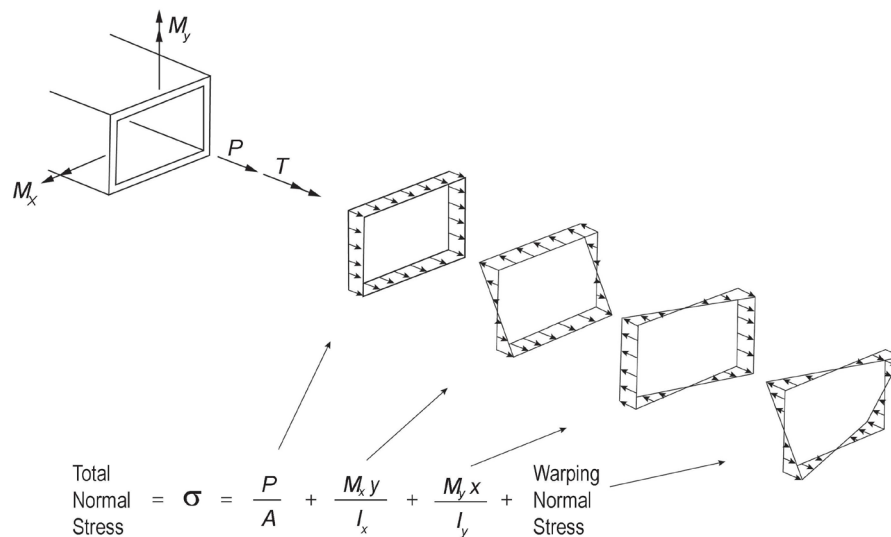
In addition, box girders are subject to cross-sectional distortion when subject to eccentric loading such as overhang loads and eccentrically applied live loads. This cross-sectional distortion results in out-of-plane bending stresses in the webs and full-width flanges of the box cross section. The effects of cross-sectional distortion are typically controlled by providing adequately spaced internal intermediate cross-

frames. Cross-sectional distortion, the resulting stress effects, and the design of internal intermediate cross-frames are discussed in detail in Fan and Helwig (2002).



$$\text{Total Shear Stress} = \tau = \frac{V_y Q_x}{I_x t} + \frac{V_x Q_y}{I_y t} + \text{St. Venant Torsion} + \text{Warping Torsion}$$

Figure 3.1.2.1-2. Illustration of the general I-girder shear stresses, which can occur in a curved or skewed I-shaped girder.



$$\text{Total Normal Stress} = \sigma = \frac{P}{A} + \frac{M_x y}{I_x} + \frac{M_y x}{I_y} + \text{Warping Normal Stress}$$

Figure 3.1.2.1-3. Illustration of the general tub girder normal stresses, which can occur in a curved or skewed box-shaped girder.

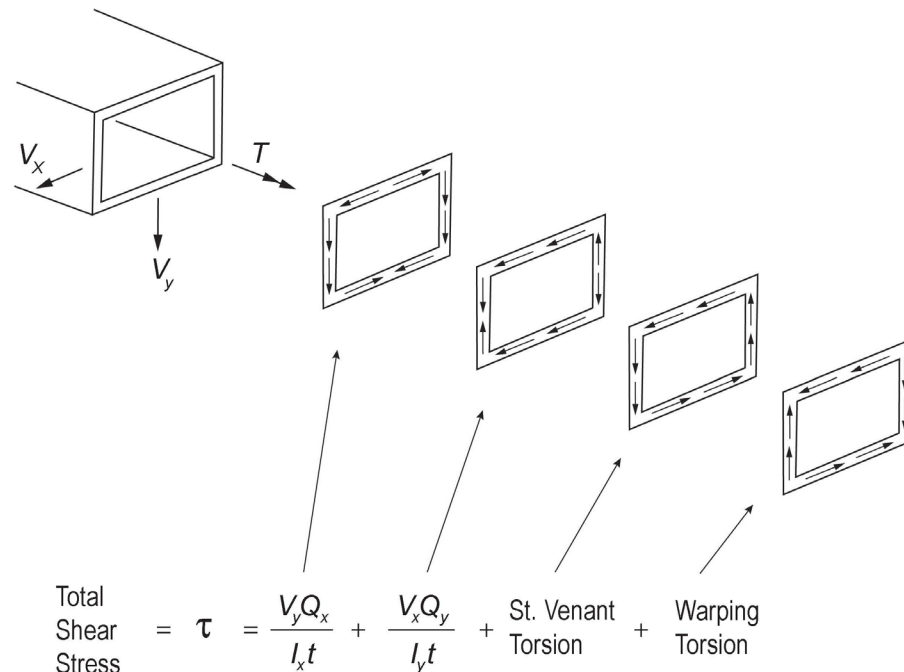


Figure 3.1.2.1-4. Illustration of the general tub girder shear stresses, which can occur in a curved or skewed box-shaped girder.

It should be noted that all box girders, even tangent box girders, are subject to torsional loading. Torsion is caused not only by curvature but by overhang loads, eccentrically located live loads, and construction loads (discussed further in Articles 3.8.3 and 3.8.7).

See Article 3.11.3 for discussion of the modeling of the torsional stiffness of I-girders.

3.1.2.2—Flange Lateral Bending

Many practical effects result from the way girders carry torsion. For example, the warping normal stresses for I-girders caused by torsion represent one source of what are called flange lateral bending stresses. These are an important part of the design equations for flange stresses in I-girders. Most curved I-girder analysis techniques include, as a key feature, some method of calculating flange lateral bending stresses and most formulae for girder design (applied loads/stresses versus load/stress capacity) include flange lateral bending contributions to the flange stresses.

It should be pointed out that curvature is not the only source of flange lateral bending stresses. Other causes include wind pressure and seismic events, both of which can induce lateral loads that cause flange lateral bending. Construction loads such as the loads applied to exterior girders by cantilever overhang formwork brackets can also contribute to flange lateral bending stresses. Of greater importance to this discussion, though, is the effect of skew in causing flange lateral bending moments. The effects of flange lateral bending in tangent but skewed steel girder bridges are often neglected by designers but it may be unconservative to do so. See further discussion in Article 4.2.2.

3.1.2.3—Torsional Deformation Effects

In addition to causing significant stresses in both I-shaped and box-shaped girders, torsion also causes significant deformations. Curved girders not only deflect vertically; they also twist. They not only experience major-axis bending rotations; they also warp (Figure 3.1.2.3-1). Depending on the severity of the curvature, the length of the spans, the framing of the bridge, and the magnitude of the loads, these deformations can become very large, sometimes large enough to be a serious consideration affecting the contractor's ability to assemble adjacent girders and their connecting cross-frames in the field.

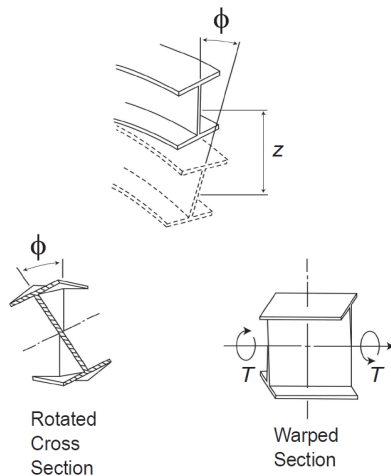


Figure 3.1.2.3-1. Illustration of the vertical deflection, twisting deformation, and warping deformation experienced by curved steel I-shaped girders.

Keep in mind also that curved girder bridges are systems, not just collections of individual girders. The sequence of erection, as well as the number of girders in place and connected by cross-frames at any given time during erection, will affect their response to loading. Many owner agencies require that contract plans clearly indicate the assumed erection sequence and designers should be ready to assess different erection sequences during shop drawing review if the contractor chooses to erect the girders in a different way. Even if it is not required to show the assumed erection sequence in the plans, designers are encouraged to consider how their bridge could be erected to ensure that at least one feasible erection scheme exists. The 8th Edition of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) introduced language suggesting that designers assess these deformations, address them as appropriate on their plans, and indicate the assumed erection sequence and intended positions of the girders at various stages of construction. Research (Shura and Linzell, 2006) discusses the magnitudes of these deformations and the ability of various analysis techniques to quantify them.

Again, note that these deformation issues are not exclusively limited to curved girders. Skewed bridges experience many of the same phenomena.

3.1.2.4—Load Shifting

As mentioned previously, systems of curved girders experience torsion because their center of loading (center of gravity) is offset from the chord line drawn between their supports. This behavior manifests itself as a load-shifting effect whereby girders on the outside of the curve carry different loads than those on the inside.

This effect is similar to how groups of piles carry vertical loads and overturning moments in a pile-supported footing. Another analogy is the way bolts carry loads in an eccentrically-loaded bolt group. In all cases, the model used is a rigid-body model in which the applied moment is resolved into force couples that are additive to the applied loads in some elements of the system (i.e., additive to the loads in some piles, additive to the loads in some bolts, or additive to the loads in some girders) and relieving in other elements. As an example, this behavior in a simple-span curved girder bridge results in girders on the outside of the curve carrying more load (see Figure 3.1.2.4-1).

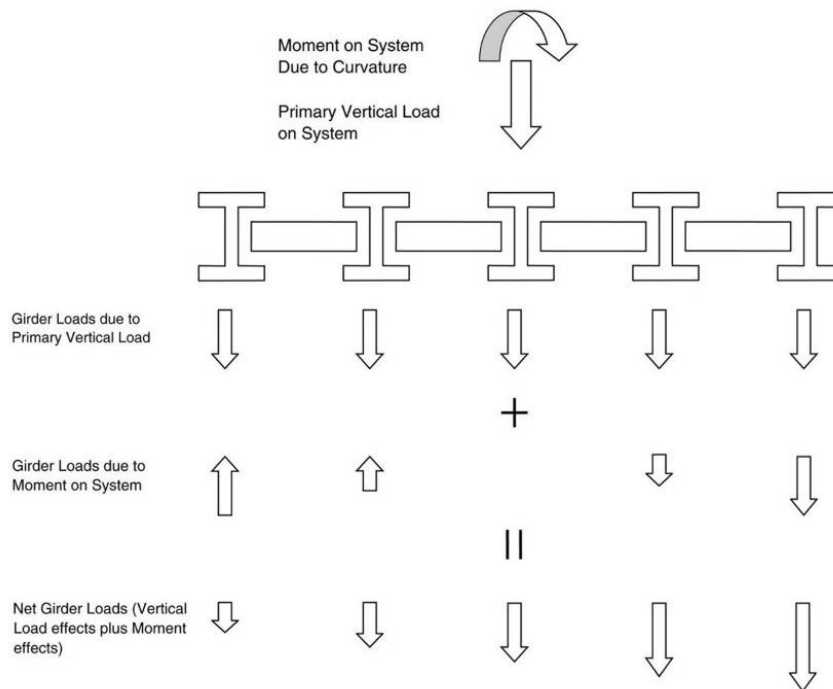


Figure 3.1.2.4-1. Illustration of the load-shifting phenomenon experienced by curved girders in multiple-girder bridges. The analogy of an eccentrically-loaded pile group or bolt group is apparent in this illustration.

This behavior characteristic generally holds for most curved girder bridges, but designers should be advised to watch carefully for variations in the direction of this type of behavior depending on characteristics such as the span length balance in multiple-span continuous girder bridges and the skew of the supports, as shown by Domalik et al. (2005). Designers should also be aware that there are other contributors to global overturning effects, which occur in both curved and tangent girder bridges, such as eccentric live load and overhang loads during phased construction.

Note that bridges with straight girders may also experience this type of behavior to some level when subject to eccentrically applied transverse force effects, such as wind loads and centrifugal forces due to a curved roadway. In many cases, these effects can be neglected in the analysis since the nature of the AASHTO load combinations is such that limit states where these loads are considered typically do not control the design of the girders when compared to limit states such as Strength I or Service II, where the effects of gravity loads (dead load and live load) predominate.

Not only is this load-shifting phenomenon itself significant but the specific load path for effecting this load shifting is also important. Loads are transferred from one girder to the next through the cross-frames, which are thus primary load carrying members and must be designed as such.

3.2—SECTION PROPERTY MODELING CONSIDERATIONS

During the development of any structural analysis, the boundary conditions and connectivity of the various components that comprise the structural system being studied are critical to the analysis results. The section properties that are assigned to the members are of equal importance. The distribution of forces through the system is highly dependent on member stiffness parameters such as EI_x , EI_y , GJ , and EC_w (the warping stiffness parameter). In particular, EC_w is rarely included in structural analysis methods based on beam theory and is not typically included in analysis programs most often employed for bridge design. Without consideration of EC_w , the local twisting responses of the girders cannot be modeled accurately, since EC_w is often the dominant contributor to the individual girder torsional stiffness. Full 3D FEM analysis circumvents the need for the modeling of warping stiffness within the single beam element used to model the girder in 2D analysis approaches. The degree to which this issue becomes a significant consideration in evaluating the accuracy of model responses between 2D and 3D methods varies depending on a number of specific parameters unique to each bridge and conclusive quantitative guidance is not yet available. In many cases, the overall torsional stiffness of the bridge cross section may often be dominated by the differential major-axis bending of the girders illustrated in Figure 3.1.2.4-1 such that 2D methods would yield adequate analysis results.

See Article 3.11.3 for discussion of the modeling of torsional stiffness of I-girders.

3.2.1—Section Properties as Related to Loading Levels in New Construction

In steel girder bridge analysis, whether using hand calculations and a single line-girder analysis or using a more complex computer analysis methods such as grid, plate and eccentric-beam, or 3D analysis, application of the appropriate section properties is essential. The various loading stages that take place during the construction and during the service life of a steel girder bridge directly correlate to the physical cross-sectional characteristics of the girders. For discussion purposes, four general loading levels can be described for steel multi-girder bridges with cast-in-place concrete decks. These conditions of loading can be defined, in general terms, as:

Level 1: Erection of structural steel framing (girders and cross-frames),

Level 2: Placement of the structural deck slab (wet concrete),

Level 3: Placement of appurtenances (e.g., barriers, railings, overlays), and

Level 4: Bridge in-service condition (e.g., carrying live loads; vehicular, rail, pedestrian).

For each of the loading conditions described, a distinct set of section properties exists and must be used respectively in the analysis in order to properly ascertain design forces and deflections to evaluate strength and serviceability criteria.

3.2.1.1—Level 1: Erection of Structural Steel Framing

During the girder erection process, the only gravity loads present from a design perspective are the self-weight of the girders and, if they are in place, the self-weight of the cross-frames. The non-composite (steel girder only) section properties apply to this loading condition when determining steel dead load

moments, shears, and cambers, and when evaluating code requirements associated with constructability.

It is worth mentioning that during this period of the construction of a steel girder bridge, depending on girder depth(s), span length(s), span arrangement, bracing and cross-frame configuration, and other characteristics, the steel framing could be susceptible to high stresses due to wind loading. Without the deck in place to act as a rigid diaphragm, the steel framing must be capable of resisting wind loads with only the non-composite section properties.

3.2.1.2—Level 2: Placement of Structural Deck Slab

Once the steel framing is entirely erected, the deck formwork is set and the deck placement operation begins. The non-composite girder section resists the gravity load of the wet concrete while it is placed, including the weight of any formwork. For a simple-span bridge, the non-composite section should be used for all analyses at this loading stage, including but not limited to, constructability evaluations.

However, when a multispan continuous girder bridge deck is placed, generally, a deck placement sequence is specified on the design drawings. In the most common case for a typically sized bridge, the placement sequence requires that positive bending moment regions be poured first, followed by the negative moment zones. The length of the positive moment zone pour is usually defined as being approximately between points of dead load contraflexure. Then, after the positive moment zones have sufficiently cured (after several days), the concrete for the deck in the negative moment regions is poured. The goal of this technique of using separate pours is to minimize deck cracking over the piers. The majority of the girder rotations at the bearings occur when the concrete in the positive zones is placed. Since two separate deck placements are made, two separate analyses are required in order to determine the bending moments, shears, and deflections due to the weight of the concrete deck.

In the case of particularly long or wide bridges, positive moment zones may be poured and cured individually, followed by individually pouring and curing each negative moment zone. In larger or more complicated bridges, the deck placement sequence may be even more complex.

Further complicating the consideration of composite action in the modeling of section properties is the issue of early age concrete composite action. Concrete begins to harden to some degree almost immediately upon placement and it gains strength and stiffness continuously over time. Traditionally, most designers have ignored this behavior and instead address the onset of composite action at discrete time intervals (e.g., 3 days, 7 days). In many cases this type of approximation is adequate but, in some cases (such as continuous placement of an entire bridge deck, particularly if it is very large), it may be prudent to investigate the effects of early composite action. Topkaya et al. (2004) present research on this topic.

In all cases, it is important to model the deck placement sequence accurately; the analysis model should be run in stages with each new section of cured deck included in the model in a manner reflecting the construction sequence, with locked-in non-composite stresses accounted for accurately in the analysis.

The first analysis uses the non-composite properties across the full length of the bridge in conjunction with the loads from the wet concrete placed in the positive moment zones. After adequate cure time, composite behavior between the girders and the deck is established in the positive moment regions. However, the non-composite section still exists in the negative moment regions. At this point, a second analysis is performed using both non-composite and composite section properties at the appropriate segments along the length of the girder and the loads due to the wet concrete being placed in the

negative moment zones. The results of these two analyses should then be superimposed in order to determine the design bending moments, shears, and deflections caused by the placement of the deck slab in its entirety. Most jurisdictions require a placement sequence to be shown on the design drawings and specify that the computation of all girder cambers shall be based on the placement sequence shown on the plans.

Some practitioners do not use this dual analysis and superposition method but instead simply apply the wet concrete dead load to the non-composite section for the entire girder length. It is suggested that this dual analysis and superposition method (accounting for the prior stages of concrete placement) be used to establish cambers and also be used to determine the design bending moments and shears in the girders at this loading stage. This produces consistent analyses for the determination of both forces and deflections.

One of the reasons why this stage of girder analysis is so critical is related to girder stability. While the concrete is wet and the non-composite section properties exist, the girder top flange is unbraced between cross-frames and, depending on the girder proportions, the girder could become unstable as load is added due to placement of the wet concrete. The girder top flange is not continuously braced until the deck concrete cures.

Though it is typical for a placement sequence to be used as previously described, some owners, depending on pour volumes, concrete placement rates, bridge width, and other parameters, permit continuous-girder bridge decks to be placed in a single operation (a continuous pour). When considering the use of a continuous placement, the designer must be cognizant of the section properties that develop along the length of the girder or girders. Concrete placed earlier in the continuous placement partially cures and, as a result, causes that portion of the girder or girders to be quasi-composite while portions placed later during the continuous placement remain non-composite. This variation in section properties needs to be accounted for when computing stresses or resistances and, possibly more importantly, when determining deflections/cambers. In some cases, the construction specifications may require the use of set-retarding concrete additives to keep the concrete fluid longer and avoid the need to consider any potential for early partial strength/stiffness gain.

Proper analysis of stresses and deflections for deck placement is discussed further in Articles 3.4 and 3.8.2.

3.2.1.3—Level 3: Placement of Appurtenances

At this point in the construction of the bridge, the steel framing is fully composite since the entire bridge deck is sufficiently cured (assuming properly designed shear connectors are present). Now, railings, barriers, raised medians, fencing, overlays, trackwork, temporary construction barriers, bridge lighting, and other bridge appurtenances are placed. Composite section properties are used for determining the design bending moments and shears, and for computing the deflections/cambers due to these loadings. The long-term composite section properties should be used at this level of the analysis. Note that the term “long-term composite section properties” refers to the section properties calculated using a modular ratio calculated as three times the nominal modular ratio (i.e., $3n$, where n is E_s/E_c , the ratio of the modulus of elasticity of steel versus concrete) to reflect the effects of creep of the concrete under sustained composite dead load effects. Conversely, the short-term composite section properties are the section properties calculated using the nominal modular ratio, n .

It is worth noting that many jurisdictions require the designer to neglect a discrete thickness of the deck slab at the top of the deck when calculating composite section properties. This thickness is usually

referred to as a sacrificial wearing surface and usually accounts for the depth of concrete disturbed by deck grooving, lost by wear overtime, or both. During the service life of the deck, portions of the top of the deck will wear from vehicular loading and will potentially split and spall. The reduction to the structural slab thickness accounts for this likelihood. The reduction to the overall slab thickness should be accounted for in the composite section properties but obviously not discounted from the self-weight of the slab.

In addition, it is worth noting that most jurisdictions require that designers neglect the dimension of the haunch (the depth of concrete between the top flange of the girder and the bottom of the deck) in section property calculations. The purpose of the haunch is to accommodate variations in camber and deflection of the girders and other construction tolerances without the need to adjust the roadway profile, and thus its contribution to the depth of the composite section in section property calculations and strength calculations should be conservatively neglected during design. If accurate measurements of the haunch depth are available after construction is complete, it may be acceptable to include consideration of the haunch dimension in later load rating analyses.

3.2.1.4—Level 4: Bridge In-Service Condition

Upon completion of the appurtenances, the structure will be opened to traffic. Short-term composite section properties should be used for all live load analyses. For continuous girder bridges, longitudinal distribution of live load is a function of girder stiffness. In the negative moment regions, the composite section for calculation of girder capacity is typically assumed to include only the girder and the longitudinal reinforcing steel embedded in the deck. However, from a stiffness standpoint, Article 6.10.1.5 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) requires that the concrete deck be assumed effective over the entire span length in the analysis to determine force effects due to composite loads. The associated commentary discusses that field tests of composite continuous bridges have shown that there is considerable composite action in the negative moment regions, even if shear connectors are not provided in those regions. In other words, for the purposes of determining live load force effects, the composite section may consist of the girder and the deck throughout the length of the bridge. In addition, Articles 6.6.1.2.1 and 6.10.4.2.1 of the same specifications permit the concrete deck to be considered fully effective in tension when determining stresses due to composite loads at the fatigue and service limit states if certain conditions are met.

In cases where shear connectors are not provided in the negative moment region, shear continuity between the girder and the deck should not be assumed for strength capacity calculations. In this situation, since the deck is assumed to be cracked in the negative moment region and since there is no shear connectivity between the girders and the deck, the common assumption is to neglect the deck in the calculation of the girder section properties. Designers are advised to consider any owner guidance in regard to this issue; many owners have specific requirements regarding the use of shear connectors and the associated analysis and design assumptions.

And, as discussed above, conservative assumptions regarding the contributions of sacrificial wearing surfaces and haunch dimensions is prudent in composite section property calculations.

3.2.2—Section Properties as Related to Phased Construction

For steel girder bridges built using phased construction techniques, the same sets of section properties as described based on the various loading conditions apply. However, some girders within a given construction stage might temporarily have a reduced composite section due to the proximity of a

longitudinal construction joint in the deck (stage line) reducing the effective slab width contributing to the composite section properties. Though in the final bridge cross section, a particular interior girder would appear to have the same sectional properties and effective widths as all other interior girders, the girders adjacent to the stage line(s) may temporarily support different effective widths and have different section properties as the bridge construction progresses. The designer must account for these differences in section properties (and loads) when evaluating strength and serviceability and also, maybe most importantly, must account for these differences when estimating girder deflections/cambers.

3.2.3—Section Properties as Related to Evaluation of Existing Bridges

When load rating or studying the feasibility of widening or redecking an existing bridge, the as-inspected (in-situ) section properties are critical to the analysis results. Section loss measurements allow for effective section properties to be estimated by the designer. Generally, section loss is not significant enough on steel girder bridges to alter the distribution of loads to the primary members. However, section loss can significantly affect the section properties at a localized region of a girder resulting in a potential high-stress zone.

Another section property related consideration when evaluating existing steel girder bridges is to consider whether the existing bridge is behaving as a composite bridge. Older bridges in the U.S. bridge inventory might have some type of “shear connectors” shown on record plans. However, it is not always obvious as to whether the girders were originally designed as composite sections. The designer must determine whether the girder is, in fact, a composite girder or not. Furthermore, if a particular bridge owner requires their bridge rehabilitation projects to upgrade the existing structure capacity to a more current-day live load, the live load shear forces in the girder could potentially be increased from those used in the original bridge’s design. The designer must then determine whether, for instance, under a redecking scenario, additional shear connectors are required to ensure composite action (if it is determined that composite action is required for a satisfactory design).

In some cases, existing steel girder bridges that are not detailed with shear connectors can still behave as composite sections. Research has shown that the bond strength alone between the steel girder top flange and the concrete deck provides some amount of shear transfer, which means a composite section exists. However, the designer should use caution when relying solely on concrete bond strength to provide adequate shear transfer. The designer should also be aware that in cases where physical shear connectors are not provided, shear continuity due to bond strength can only be relied on up to the point of first slip. Once the bond between the steel and concrete is broken, the shear continuity drops to zero and all benefits of composite action are lost, including any benefits that may have been assumed at lower loading levels. See Article 3.19 for more discussion of the analysis of older bridges.

3.3—LOADS ON THE PERMANENT STRUCTURE

Numerous different loads are typically considered for the design of steel girder bridges, some permanent in nature, others variable or transient. Some loads are applied to the non-composite section (structural steel framing only) while others are applied after the deck is hardened and acting in a composite manner with the steel girders. Most loads considered in steel girder bridge superstructure designs are gravity loads, applied in a downward, vertical direction but some loads, such as centrifugal force, wind, and thermal effects, act in other directions. Some of the most commonly considered of these loads are listed below.

Table 3.3-1. Commonly Considered Bridge Design Loads

Load	AASHTO LRFD Load Category Abbreviation	Permanent or Transient?	Applied to Non-composite or Composite Structure?	Primary Direction of Action?
Dead Load—Self-weight of Structural Steel	DC1	Permanent	Non-composite	Vertical
Dead Load—Weight of Deck Forming System	DC1	Permanent (SIP forms) or Transient (removable forms)	Non-composite	Vertical
Dead Load—Weight of Concrete Deck	DC1	Permanent	Non-composite	Vertical
Dead Load—Barriers and Sidewalks	DC2	Permanent	Composite	Vertical
Dead Load—Future Wearing Surface	DW	Permanent	Composite	Vertical
Dead Load—Utilities and Other Appurtenances	DW	Permanent	Composite (sometimes Non-composite)	Vertical
Live Load	LL	Transient	Composite	Vertical
Construction Loads	See Note	Transient	Non-composite and Composite	Horizontal and Vertical
Dynamic Load Allowance (Impact)	IM	Transient	Composite	Vertical
Centrifugal Force	CE	Transient	Composite	Horizontal and Vertical
Braking	BR	Transient	Composite	Horizontal and Vertical
Wind on Superstructure and Substructure	WS	Transient	Non-composite and Composite	Horizontal and Vertical
Wind on Live Load	WL	Transient	Composite	Horizontal and Vertical

Uniform Thermal Contraction or Expansion	TU	Transient	Non-composite and Composite	Horizontal
Thermal Gradient	TG	Transient	Non-composite and Composite	Horizontal and Vertical

Note: See *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017), Section 3 for more discussion of current load factors and other considerations for treatment of construction loads.

3.3.1—Dead Load—Weight of Structural Steel

The self-weight of structural steel should be applied to the non-composite section; when using automated computer programs, be sure to understand exactly how the program is applying this load. For basic girder design, it is usually appropriate for both stress and deflection calculations to assume that the entire steel superstructure is in place before the structural steel self-weight is applied. Most computer programs apply the steel self-weight to the entire superstructure. However, when performing an erection analysis, the actual sequence of erection of the various sections of each girder and cross-frames should be considered. It is not uncommon that a critical stress case occurs at some point during the construction of the superstructure before all the framing is in place. Furthermore, deflections (and rotations), which occur at various stages of erection, can be critical parameters when it comes to evaluating fit-up and constructability of the superstructure—often temporary shoring towers or temporary holding cranes, or both, are required to control excessive deflections to allow for proper fit-up of subsequent sections of the superstructure.

3.3.2—Dead Load—Weight of Deck Forming System

In most cases, from a global standpoint, it is appropriate to assume that the weight of the deck forming system is applied to the completed, non-composite, structural steel framing system and can be applied as a simple, uniformly distributed line load on each girder. The type of forming system (permanent, stay-in-place forms versus removable forms) will affect the nature of the effects of this loading. When permanent forming is used, typically the effects of its weight are approximated using a simplified calculation or based on an approximate percentage of the weight of the deck; many owner agencies have different recommendations for this calculation, which should be followed. The twisting effect of overhang formwork is usually analyzed locally on the exterior girders. The specific overhang formwork system and any temporary backup structure (bracing) should be considered and understood, and the effect of loads applied to the girders by the overhang falsework system should be accounted for in the girder design. Typically overturning effects from the left and right overhang are assumed to balance each other in terms of system behavior; in such cases, it may be reasonable to limit consideration of overhang falsework loads to the effects on the exterior girders only. However, in cases of bridge widening or phased construction, special care should be taken to understand the full nature of the loading applied by the deck forming system (as well as the related construction loads and the weight of the wet concrete deck). Without proper bracing, these asymmetrical loading cases may cause stability problems.

3.3.3—Dead Load—Weight of Concrete Deck

The application of concrete to a steel girder bridge represents a fairly complicated analysis problem, particularly on bridges that are longer, wider, or both. When initially placed on the steel girders, the wet concrete offers no structural capacity or stiffness to the system and represents nothing more than a

gravity load. However, as the concrete begins to cure, it begins to develop stiffness and affect the overall stiffness of the structural system. Ongoing research (particularly at the University of Texas at Austin) is currently evaluating the effects of early stiffness gain in deck concrete for steel girder bridges; however, in most cases of reasonable size deck pour stages, this effect of partial early stiffness gain can be neglected.

More commonly, though, the effect of placing the deck in stages does need to be considered on a stage-by-stage basis. See Article 3.2 for a detailed discussion of sequenced deck placement.

The self-weight of the wet concrete deck has historically often been assumed to be distributed to the girders based on a tributary deck width assumption. However, due to the presence of cross-frames connecting adjacent girders and forcing them to act together as a system, when performing a line-girder analysis of straight girder bridges with little or no skew (skew not exceeding 20 degrees from normal), a more accurate assumption would be to distribute the self-weight of the deck uniformly among all girders, as permitted under Article 4.6.2.2.1 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017). Conversely, when performing a refined analysis (i.e., a 2D or 3D analysis, where the entire structural steel framing system is modeled), the calculated distribution of the self-weight of the deck for application to the girders in the model can and should be based on a tributary deck width assumption; the model will then determine the actual force effects in each girder based on relative stiffness. In addition, in refined analysis models where the deck is explicitly modeled (e.g., 2D plate-and-eccentric beam models or 3D models), the self-weight of the deck should be applied as direct loading to the girders, rather than using the density of the deck elements to represent the weight of the deck, since the deck has no stiffness at the non-composite stage of the analysis and should not be included in the model at that point.

Note that much of the previous discussion of loads associated with deck forming systems also applies directly to the consideration of loads due to the weight of the concrete deck.

3.3.4—Dead Load—Barriers and Sidewalks

The weight of barrier rails, median barriers, and sidewalks typically represent simple dead loads applied to the long-term composite section. Care should be taken when distributing these loads to various girders. The *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) provide guidance on the distribution of these types of loads if performing a simplified analysis; owner agencies often have similar or competing guidance. If performing a refined analysis which explicitly models the deck (e.g., plate and eccentric-beam grid analysis or 3D analysis), the analysis model may be used to determine the distribution of these loads based on the stiffness of the modeled structural elements. Note that there is some research suggesting that barrier rails may provide additional stiffness and load resistance; however, many owners have not yet adopted policies allowing consideration of the barriers as part of the structural section.

3.3.5—Dead Load—Future Wearing Surface

The weight of any possible future wearing surfaces represents simple dead loads applied to the long-term composite section. Most owner agencies have guidelines on what to assume for the weight of a future wearing surface. These are typically based on the assumption of 2 in. to 3 in. of future wearing surface, usually resulting in loads in the 20 to 30 psf range. In most cases (at least in cases where the barrier rails are relatively narrow compared to the overall bridge width and where sidewalks are not

provided), the weight of the future wearing surface is often assumed to be distributed equally to all girders; however, some designers use tributary width assumption based on girder spacing.

3.3.6—Dead Load—Utilities and Other Appurtenances

In some cases, utilities, lighting, signs, or other items are attached to bridges. The nature and location of the item and how it is attached to the bridge directly affects how the loads resulting from the presence of these items should be included in the analysis of the bridge. In extreme cases, these attachments can have dramatic effects on a steel girder bridge, but in most cases the effects are minor. Designers are encouraged to keep in mind the magnitude of these additional loads in relation to the overall loading of the bridge; in most cases a simplified, slightly conservative approach to the treatment of these loads is appropriate and encouraged. Usually the treatment of these loads in the analysis model can be similar to that of barriers and sidewalks.

3.3.7—Live Loads

The treatment of live loads can be one of the most complicated aspects of steel bridge analysis. Live loads are applied to the short-term composite section, but that is where the simplicity ends. Live loads are moving loads that need to be applied in various patterns moving both longitudinally and transversely over the bridge. The AASHTO LRFD HL-93 live load includes both a lane load component (to be applied in patterns over the bridge to determine the most critical loading conditions) and truck (point load) components. There are also multiple presence factors that represent modifications to the loads, which reflects the lower probability that multiple lanes will all be fully loaded simultaneously.

There are two primary methods for calculation of the transverse distribution of live load effects to individual girders: empirical live load distribution factors, and refined analysis based on relative stiffness. Empirical live load distribution factors are provided in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) and are appropriate for many types of bridges, particularly tangent girder bridges, when they are analyzed using a line girder analysis model, the VLOAD method, or many types of traditional 2D grid analysis models. If a more refined analysis is used (e.g., a plate- and eccentric beam analysis or a 3D analysis), the use of empirical live load distribution factors is not necessary and inappropriate; instead live load distribution may be accomplished by means of relative stiffness analysis within the model. In more complicated structures such as curved girder bridges, relative stiffness analysis is preferred if the analysis model can perform this type of analysis; otherwise, modifications to the empirical live load distribution factor approach may be required (AASHTO, 2017).

It should be noted that the use of the empirical live load distribution factors presented in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) is limited to tangent girder bridges with span lengths not exceeding 240 ft. For spans longer than 240 ft, using a limiting value of $L=240$ ft in the AASHTO live load distribution factor equations produces a conservative distribution factor. However, at longer span lengths, the contribution of live load to the total load is less, since the magnitude of live load per linear foot of bridge is essentially constant, while the magnitude of dead load per linear foot of bridge increases with span length, so the impact of this conservatism on the economy of the design is reduced.

There are two primary ways to handle live load modeling for bridge structures. First is what can be called the brute force method, which involves running analyses of multiple live load cases. In computer analysis techniques, this is accomplished using a live load generator—a computer routine that produces literally hundreds or thousands of live load cases, each representing a different load (e.g., truck load,

lane load, combinations of multiple truck or lane loads) applied at different positions along the structure. For each live load case, the analysis model is fully calculated; therefore, shear and moment results for all key members are developed. The multiple live load case method generates a huge pool of numbers, developing the force envelopes for various members in the structure.

An alternative to the multiple live load case method is the influence line or influence surface method. An influence surface is an influence line approach applied in two dimensions rather than just one dimension. A full explanation of the influence surface method is beyond the scope of this guideline document, but a summary description is warranted:

In this approach to live load modeling, the response of a given point in the model (e.g., a point on a girder, deck, cross-frame) is calculated for all possible positions of a unit load. Instead of presenting these responses in terms of the results of multiple iterative analyses, however, the responses are directly presented in terms of the maximum and minimum response. Thus, the influence surface approach to modeling live load effects allows the designer to quickly zero in on the maximum loading responses of the structure at given locations. The amount of output from an influence surface analysis is much less and the designer can focus on the critical loading effects rather than spending substantial time collating thousands or millions of numbers to determine envelope results.

Note that multiple presence factors are often more efficiently applied to the load effects from the analysis, rather than to the applied loads in the analysis.

3.3.8—Construction Loads

Construction loads include loads from construction equipment (such as deck screeding machines), construction workers, and construction materials stored on the bridge. Typically, the most critical case to consider is during deck placement when the deck screeding machine is moving over the bridge and applying concentrated loads on the deck overhangs, with simultaneous construction worker loading (modeled using a uniform pressure). However, care should be taken to limit or to carefully analyze the effects of loads from stored materials on bridges. Depending on the nature and timing of the loading, construction loads may be applied to the non-composite or the composite section. Construction loads should be considered on a case-by-case basis; however, in some cases, owner agencies may specify the nature and magnitude of construction loads based on what is permitted in their construction specifications. In the absence of other requirements, some guidance can be found in the *AASHTO Guide Design Specifications for Bridge Temporary Works* (AASHTO, 2017). See also Section 3 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017).

3.3.9—Dynamic Load Allowance (Impact)

Impact is typically applied to the bridge analysis as a simple factor increasing the magnitude of the live load and is meant to model the effects of trucks and other vehicles “bouncing” on the bridge when they hit irregularities on the deck surface. Previous AASHTO bridge specifications used a formula for calculating the effects of impact that varied with span length, but the current *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) have simplified this and, for primary structure members, a single value of 15 percent for the Fatigue and Fracture Limit State or 33 percent for all other Limit States is specified; note that this dynamic load allowance factor is only applied to the load effects from the truck or tandem load, not the lane load.

3.3.10—Centrifugal Force

Vehicles traveling on curved roadway alignments develop a centrifugal force. The *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) provide guidance on the calculation of this force and the location of its application (6 ft above the deck). Typically, the horizontal component of this force has negligible effects on the bridge design; the deck forms a large shear diaphragm that carries this load to the supports. In some cases (extreme curvature and longer span, wider structures), the magnitude of the horizontal component of the force may be large enough that it should be considered when designing cross-frames at supports (the load carried through the deck to the support locations is transferred to the bearings through the end cross-frames). More commonly for the superstructure design is consideration of the overturning effects due to applying the horizontal load 6 ft above the deck. Some analysis programs include consideration of this overturning effect as it affects the distribution of loads through the girders, causing more load to be distributed to the girders on the outside of the curve. A discussion of this effect is included in Richardson, Gordon, & Associates (1976) and Article 3.15.2. Also discussed in Article 3.15.2 is the beneficial counteracting effects of superelevation on the overturning moment due to centrifugal force. It should be noted that the centrifugal force is applied to the design truck or tandem only (and not the lane loads) and also to the fatigue live load.

3.3.11—Braking

The *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) provide guidance on the calculation of vehicular braking forces. These forces are typically neglected in superstructure design but are included in the bearing and substructure design. The effects of a longitudinally applied force on the superstructure are negligible since they are easily transferred through the deck to the support locations and down through the girders to the bearings, all via very robust shear diaphragm load paths. The effects of overturning due to the application of braking forces 6 ft above the deck as specified by the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017), is similarly negligible. Considering these forces on a per truck basis, a series of force couples between the front and rear axles (i.e., a series of upward and downward pairs of forces on the superstructure) is developed on the span. The effect of this series of force couples is the development of a series of small peaks and valleys in the shape of the shear and moment diagrams. The magnitude of these local peaks and valleys is negligible relative to the overall magnitude of the main vertical loading effects.

3.3.12—Wind on Structure

The effects of wind on the superstructure are of more significant concern during construction. However, the effects of wind loads on the completed structure are also important. The effects of wind loads on the girders in the completed structure are typically considered in terms of increases in the flange lateral bending moments in the exterior girders. A simple way of addressing this is to assume that the wind pressure on the bottom half of the web of the exterior girder results in a uniform horizontal load on the bottom flange, with the flange spanning between cross-frame locations. The reaction at each cross-frame is then treated as a load in the cross-frame, which must transfer the load up to the deck. Once in the deck, these wind loads (as well as the load on the top half of the web, on the barrier, and on other deck-mounted elements) are carried by the deck to the support points by means of the deck acting as a shear diaphragm. At the support points, the resulting wind loads are then transferred down to the bearings through the cross-frames at supports.

3.3.13—Wind on Live Load

The effects of wind on live load can be treated in a manner similar to the way that centrifugal forces are treated.

3.3.14—Uniform Thermal Contraction or Expansion

See Article 3.18.

3.3.15—Thermal Gradient

See Article 3.18.

3.4—STRENGTH DESIGN

In order to satisfy the provisions outlined in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) for strength design, the designer must extract output from the analysis model and perform a series of checks at various locations along the girder. The type of output required depends on the geometry of the system and the girders themselves. Design strength of a steel girder specified in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) is based on whether a section is considered compact or noncompact. A compact section is assumed to be able to develop a fully plastic section, and all of the strength checks for that girder are based on the ultimate moment capacity of the section. A noncompact section is assumed to be limited by the yielding of the flanges, and its capacity is limited by the yield stresses in the section. The *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) consider all sections in curved girders to be noncompact.

When designing noncompact sections using the stress-based provisions of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017), it is important for the designer to remember the superposition of stresses from the analysis model. It is important for the designer to carefully superimpose various stresses from the analysis model. It is incorrect to take the total maximum moment in the girder and calculate stresses based on the composite section for unshored construction.

In addition, when a deck slab placement sequence is specified, the designer should calculate the noncomposite stresses in the girder at each stage of construction based on the loading and section properties of the girder during that stage. The designer could consider the stresses associated with a specific slab placement sequence and track the specific locked-in stresses for use in the final strength checks for the girder, but this is probably overkill in all but the most highly unusual situations. Many of the commercially available girder design programs will analyze the slab placement sequence to check the strength of the noncomposite section before the deck hardens, but then for final design they only consider the locked-in noncomposite stresses calculated from an analysis which considers placement of the entire deck at once. Rigorous tracking of the stress history of the girder through erection, slab placement, composite dead load placement, and in-service condition may produce different results, but these differences are typically fairly minor and in most cases are negligible.

An additional consideration in the provisions of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) is the effect of lateral bending moments in girder flanges. This lateral bending component can be the result of wind load on the structure, curvature, or even skew. The wind load effect can be included in the analysis model as a distributed load applied to the flanges or simply hand calculated and superimposed on the primary bending forces. The lateral effects of curvature will be captured directly in a 3D FEM model but most grid analysis programs rely on the M/R relationship for

calculation of lateral bending moments. Even when using a 3D analysis, many designers will defer to the M/R formulas as this approach is typically conservative and easy to apply. Determining lateral bending effects due to skew is highly dependent on intermediate cross-frame spacing and orientation. There is no accurate method for approximating the effects of skew on flange lateral bending and, if this is a concern to the designer, a 3D FEM model should be considered.

3.5—INELASTIC DESIGN

Inelastic Design generally refers to structural design with consideration of behaviors, such as material yielding or cracking, and their effects on load distribution and resistance mechanisms. Increases in design strength provided by inelastic actions were first incorporated into the AASHTO *Standard Specifications for Highway Bridges* (2002) for Load Factor Design (LFD) in a limited, empirical way; specifically, two simple provisions were incorporated. First, the moment capacity of compact sections was increased to the plastic strength, M_p , rather than being limited to the yield moment, M_y . Second, it was permitted to shift 10 percent of the peak negative elastic moments in compact continuous-span members to positive bending regions before the bending strengths are checked. This second provision was intended to account, in an approximate way, for the redistribution of moments that occurs due to inelastic behavior.

Comprehensive inelastic procedures were first adopted for highway bridge design with the advent of the AASHTO *Guide Specifications for Alternate Load Factor Design Procedures for Steel Beam Bridges Using Braced Compact Sections* (ALFD) (AASHTO, 1986). These procedures, originally called autostress designs, were applicable only to compact sections. They were primarily intended to eliminate the need for 1) additional cover plates on rolled beam sections and 2) multiple flange thickness transitions in welded beams, which were often required in LFD.

In 1993, Galambos et al. proposed inelastic rating procedures for highway bridges. These methods utilize the same rating vehicles and load and resistance factors as the rating procedures adopted in the AASHTO *Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* (AASHTO, 1989), but define the strength limit state as either shakedown (deflection stability) or a specified maximum permanent deflection. The inelastic rating procedures allow a bridge owner to take advantage of the reserve strength inherent in continuous-span bridges that are structurally deficient.

The inelastic procedures from the ALFD guide specifications were incorporated into the first edition of the AASHTO *LFRD Bridge Design Specifications* (AASHTO, 1994) with minor modifications and additions, as an optional design approach. However, these provisions were still limited to the design of compact beams—although preliminary research results at that time showed that sections with noncompact and slender webs perform in a similar fashion, but at a reduced level of moment. This limited the potential savings that could be realized with these methods, as economical continuous-span bridges often have noncompact or slender webs at their interior supports.

Schilling et al. (1997) recognized that the above design provisions were prohibitive for bridge engineers to use because they required significant additional engineering effort and did not apply to all possible cross-section configurations. Therefore, a simplified procedure was developed based on the shakedown limit state (ASCE, 1971) rather than plastic collapse. As a result, the designer uses the same elastic moment envelopes as calculated in elastic analysis-design. The simplified procedure avoids complex iterations or special calculations to find the plastic collapse mechanism for the Strength I load level or to determine the extent of permanent deflections under the Service II load level. Also, Schilling et al. (1997) point out that the shakedown limit state—which defines the maximum load at which increments

of plastic deflection will cease after a few repeated passages of excessive loads, with the subsequent response being fully elastic—is more appropriate for bridge design than the plastic collapse limit state. In their proposed procedure, the section response at the pier sections is quantified by an effective plastic moment, $M_{pe}(\theta_{pm})$, defined as the moment resistance of the pier section at a predetermined upper-bound estimate of the maximum required plastic rotation demand, θ_{pm} . Based on trial design studies by Schilling (1986) and Schilling, et al. (1997), the estimated upper-bound plastic rotation demands at general interior pier locations are 30 mrad under the AASHTO LRFD Strength I loading conditions and 9 mrad under Service II loading conditions (AASHTO, 1998). The shakedown-based design procedure involves only a very simple modification of elastic design or rating procedures, effectively a simple calculation of the percentage of the elastic pier section moment that can be redistributed to the positive moment region of an I-girder. The design or rating of the positive moment region is handled essentially in the same way as in elastic design methods.

Also, Schilling, et al. (1997) addressed general unstiffened and transversely stiffened cross-sectional I-shapes in their simplified procedure. This is a significant extension of the prior procedures since, as noted above, many I-girder bridges have noncompact or slender webs. They defined a new type of cross section, termed an ultra-compact compression flange section. This type of section has a compression flange that satisfies the traditional plastic design slenderness limit as well as having stiffeners placed at $\leq D/2$ on each side of the pier. However, the web can be compact, noncompact, or slender. By use of a stocky compression flange and closely spaced transverse stiffeners in the vicinity of the support, improved moment-rotation characteristics are obtained even with a slender web. Improved moment-rotation characteristics are also obtained if the web is restricted to traditional plastic design limits on its slenderness. Sections with these types of webs and with an ultra-compact compression flange are also considered as ultra-compact.

Barth, et al. (2001) have developed improved effective plastic moment expressions that are applicable for I-sections satisfying certain flange proportioning and unbraced length limits. Enhanced strength equations that predict capacities greater than the yield moment, M_y , and up to the plastic moment capacity, M_p , are suggested. These equations are derived from $M-\theta_p$ expressions developed by Barth and White (1998) and White and Barth (1998). These $M-\theta_p$ expressions were in turn derived through regression analysis of the results from extensive finite element parametric studies and correlation with experimental results. Both non-composite and composite I-girders in negative bending are considered in the development of the equations. These equations are incorporated in the optional Appendix B6 of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017), which permits inelastic moment redistribution from interior-pier sections in straight continuous-span bridges.

3.6—FATIGUE ANALYSIS AND EVALUATION

The fatigue provisions of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) have been significantly modified from previous specifications. The Fatigue Limit State can control the design of girders, particularly the design of the bottom flange in positive moment regions. In addition, the Fatigue Limit State very often controls the design of cross-frame, especially in curved and/or severely skewed structures, where the cross-frames are primary load carrying members and are subject to live load effects. When designing cross-frames for fatigue, the designer should carefully consider the calculation of the stress range in cross-frame members and fully understand how these stress ranges are determined in the analysis model. The *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) include specific guidance in this regard in Article C6.6.1.2.1. In addition, research is currently underway (NCHRP Research Project 12-113, Proposed Modification to AASHTO Cross-Frame

Analysis and Design) to provide more rational and comprehensive guidance with regard to the fatigue evaluation of cross-frames.

3.7—SUPERSTRUCTURE LIVE LOAD REACTIONS FOR SUBSTRUCTURE DESIGN

Oftentimes, substructure elements are being designed based on reactions provided by the steel superstructure analysis. This is helpful output for substructure and foundation design but the designer must be cognizant of what the output really means to avoid being overly conservative in the substructure design. Live load reactions at each girder are typically output as an envelope of absolute maximum reactions at each girder. To simply apply these maximum live load reactions to the substructure would be grossly over-estimating the load on the substructure. In order to properly apply the provision of the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) for substructure design, the designer must break down the reactions and determine the effects at each girder per lane loaded. In addition to reporting the maximum live load reaction at a particular girder, some analysis programs will report concurrent live load reactions at adjacent girders due to the controlling lane pattern. The designer can use this detailed output to back out the effects by lane and more accurately apply loads to the substructure.

In addition, designers should be aware of the guidelines in the *AASHTO LRFD Bridge Design Specifications* (AASHTO, 2017) regarding the application of dynamic load amplification of live load (a.k.a. impact) for substructure design, where the dynamic load allowance must be applied, versus for foundation design (“foundation components that are entirely below ground”), where dynamic load allowance need not be applied.

3.8—CONSTRUCTABILITY—ANALYSIS ISSUES

3.8.1—Erection of Steel Framing

The erection of the steel framing should be given consideration during the design of the bridge. Also, and probably more importantly, the steel erection should be given significant attention by an engineer who is working for the steel erector. In the design phase, for example, consideration should be given to girder field piece lengths and weights, field splice locations, possible temporary support locations, and potential unbraced lengths that may occur during erection. Several agencies require that a conceptual erection sequence be shown in the contract plans, regardless of the complexity of the design. This conceptual erection plan will require some forethought, analysis, and calculations by the designer as to how the bridge can be built. Typically, the conceptual erection plan will not be as detailed as the erection plan developed by the contractor’s engineer.

Some agencies require that an erection sequence, including plans, procedures, and calculations that are stamped and sealed by a professional engineer, be submitted by the contractor or steel erector, or both, prior to the start of any steel erection. The plans and procedures for the steel erection should be based on calculations performed by an engineer, which may include girder erection analysis; girder stability checks; girder or bridge jacking measures, or both; or the design of temporary support structures; or combination thereof. It is often the case that the bridge designer is responsible for reviewing the erection plan, procedures, and calculations submitted by the contractor. Therefore, it is important that the bridge designer understand what should be shown in the erection plan, procedures, and calculations.

This section is intended to present the key issues that should be investigated through various analyses with regard to the erection of the steel framing and takes the approach from the standpoint of an engineer responsible for developing the erection plan, procedures, and calculations for a contractor.

For a more detailed discussion of the elements of an erection engineering submittal and their associated requirements, and the roles of the various parties involved in the development, review, and execution of the erection engineering plans, procedures, and calculations, please see the AASHTO/NSBA Steel Bridge Collaboration's S10.1, *Steel Bridge Erection Guide Specification* (AASHTO/NSBA, 2019).

3.8.1.1—General Guidance

Procedures required for the general steel erection of highway bridges are provided in S10.1, the *Steel Bridge Erection Guide Specification* developed through the AASHTO/NSBA Steel Bridge Collaboration (AASHTO/NSBA, 2019). This document highlights minimum requirements for the development of steel erection procedures, including steel erection drawings and calculations. For example, at a minimum, the steel erection drawings should provide:

- 1) a plan of the work area;
- 2) erection sequence, including a narrative of the procedure, for all superstructure components, noting the use of temporary supports, lifting cranes, and holding cranes, as required;
- 3) details of temporary support structures, tie-down devices, and blocking for the bearings; and
- 4) details of jacking devices, spreader beams, and attachments, as well as the lifting weight of girder pieces including weights of the rigging and lifting attachments.

The contractor's engineer may be required to submit calculations that provide the basis for the details and procedures provided on the erection drawings. For example, the calculations will provide the basis for the given erection sequence, the design of the temporary support structures, and the design of spreader beams. These calculations should verify:

- 1) the load capacity of the lifting and holding cranes,
- 2) the load capacity and stability of the temporary support structures,
- 3) the structural adequacy and stability of the girders for each stage of the erection sequence, and
- 4) the load capacity of spreader beams, beam clamps, stiffening trusses, or tie-down devices.

Additional information, especially with regard to the erection of horizontally curved steel I-girder bridges, can be found in Chavel et al. (2009) and Stith et al. (2009).

3.8.1.2—Analysis Techniques

The engineer working for the contractor will need to investigate the erection sequence of the steel framing through some type of analysis. Depending on the complexity of the steel framing and the proposed erection sequence, the level of analysis required can range from simple hand calculations to 3D finite element modeling. Unless specified by the bridge owner or the contract plans, the engineer must decide what level of analysis is appropriate for the given steel structure.

In general, for a simple framing plan such as a simple-span bridge with no skew, hand calculations may be the sufficient level of analysis. On the other hand, for a curved girder bridge where vertical and lateral displacements may be of concern to ensure proper fit-up, or where lateral bending stresses at certain stages of erection may be of concern, a full 3D finite element analysis may be warranted. The reader is referred to the findings published in NCHRP Report 725 (White et al., 2012) for detailed discussion of some of these considerations. One of the main goals of NCHRP Report 725 was to provide