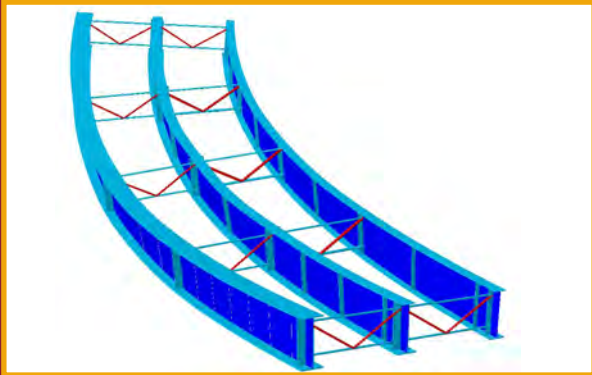


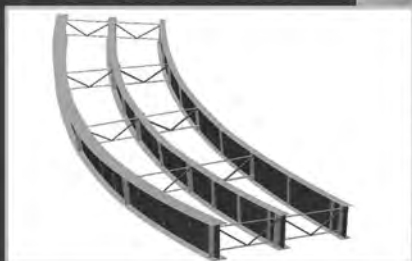
# ***G13.1 Guidelines for Steel Girder Bridge Analysis***

***2<sup>nd</sup> Edition***



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*2<sup>nd</sup> Edition*



AMERICAN ASSOCIATION OF  
STATE HIGHWAY AND  
TRANSPORTATION OFFICIALS  
**AASHTO**  
ADVANCING THE ART OF TRANSPORTATION



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

## 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.

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## 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 interim since 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 form the majority of the changes in the Second Edition. Other changes include clarifications to the text on prediction of deflections and load rating analyses, incorporation of recent recommendations on the impact of connection stiffness on cross frame stiffness, incorporation of recent recommendations on global second-order amplification of structural response and narrow system stability analysis, and other minor editorial corrections.

*AASHTO/NSBA Steel Bridge Collaboration Task Group 13, August 2014*





## TABLE OF CONTENTS

1	MODELING DESCRIPTION .....	1-1
1.1	Hand Analysis Methods .....	1-1
1.1.1	Beam Charts.....	1-1
1.1.2	Line Girder Analysis Method .....	1-1
1.1.3	V-Load Method .....	1-1
1.1.4	M/R-Load Method .....	1-1
1.2	Finite Element Method .....	1-2
1.2.1	Basic Concept .....	1-2
1.2.2	2D Grid Analysis Method .....	1-2
1.2.3	Plate and Eccentric Beam Analysis Methods .....	1-2
1.2.4	Generalized Grid Analysis Method .....	1-2
1.2.5	3D FEM Analysis Methods .....	1-3
1.2.6	Element Types .....	1-3
2	HISTORY OF STEEL BRIDGE ANALYSIS.....	2-1
2.1	History of Change in the Complexity of Bridges .....	2-1
2.2	Bridge Design Code Advancements.....	2-2
2.3	Bridge Analysis Advancements .....	2-2
2.4	Concurrent Initiatives Parallel to This Guideline Document .....	2-3
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 .....	3-4
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
3.2.2	Section Properties as Related to Phased Construction .....	3-10
3.2.3	Section Properties as Related to Evaluation of Existing Bridges .....	3-10
3.3	Loads on the Permanent Structure.....	3-11
3.3.1	Dead Load—Weight of Structural Steel .....	3-12
3.3.2	Dead Load—Weight of Deck Forming System .....	3-12
3.3.3	Dead Load—Weight of Concrete Deck .....	3-13
3.3.4	Dead Load—Barriers and Sidewalks.....	3-13
3.3.5	Dead Load—Future Wearing Surface .....	3-13
3.3.6	Dead Load—Utilities and Other Appurtenances.....	3-14
3.3.7	Live Loads .....	3-14
3.3.8	Construction Loads.....	3-15
3.3.9	Dynamic Load Allowance (Impact) .....	3-15
3.3.10	Centrifugal Force .....	3-15
3.3.11	Braking .....	3-15
3.3.12	Wind on Structure.....	3-16
3.3.13	Wind on Live Load.....	3-16
3.3.14	Uniform Thermal Contraction or Expansion .....	3-16
3.3.15	Thermal Gradient .....	3-16
3.4	Strength Design .....	3-16
3.5	Inelastic Design .....	3-17
3.6	Fatigue Analysis and Evaluation .....	3-18

3.7	Superstructure Live Load Reactions for Substructure Design .....	3-19
3.8	Constructibility—Analysis Issues .....	3-19
3.8.1	Erection of Steel Framing .....	3-19
3.8.1.1	General Guidance .....	3-20
3.8.1.2	Analysis Techniques.....	3-20
3.8.1.3	Investigation of Steel Erection Sequence .....	3-21
3.8.2	Deck Placement Sequence.....	3-23
3.8.3	Overhang Analysis and Effects on Girders .....	3-25
3.8.3.1	Selection of Overhang Width versus Beam Spacing.....	3-25
3.8.3.2	Selection of Overhang—Effect on Forming.....	3-25
3.8.3.3	Overhang Effects on Structural Design .....	3-26
3.8.4	Wind Loading during Construction .....	3-28
3.8.5	Live Loads during Construction .....	3-29
3.8.5.1	Bridge Construction .....	3-29
3.8.5.2	Bridge Rehabilitation or Demolition .....	3-29
3.8.6	Miscellaneous Construction Loading .....	3-30
3.8.7	Stability Analysis during Various Stages of Construction .....	3-30
3.9	Prediction of Deflections .....	3-31
3.9.1	General .....	3-31
3.9.2	Skewed Bridges .....	3-32
3.9.3	Curved Bridges .....	3-33
3.9.4	Final Deflections (Vibration, Dynamic Response).....	3-34
3.10	Detailing of Cross-Frames and Girders for the Intended Erected Position .....	3-36
3.10.1	Detailing Methods .....	3-38
3.10.2	Straight, Skewed I-Girder Bridges .....	3-42
3.10.3	Curved I-Girder Bridges .....	3-43

3.10.4	Tub Girders .....	3-45
3.10.5	Calculation of Locked-In Force Effects Due to Cross Frame Detailing .....	3-45
3.10.6	Estimation of Fit-Up Forces .....	3-46
3.11	Cross Frame Modeling (2D versus 3D) .....	3-47
3.11.1	Impact of Connection Stiffness on Cross Frame Stiffness .....	3-48
3.11.2	Simplified Euler-Bernoulli Approximations of the Stiffness of Truss-Type Cross-Frames .....	3-49
3.11.3	Shear-Deformable (Timoshenko) Beam Approach for Modeling the Stiffness of Truss-Type Cross-Frames.....	3-51
3.11.4	“Exact” Modeling the Stiffness of Truss-Type Cross-Frames .....	3-53
3.12	Modeling the Torsional Stiffness of I-Girders.....	3-54
3.13	Deck Modeling (e.g., Deck Effective Width, Composite Action) .....	3-55
3.14	Bearings, Substructures, and Boundary Conditions for Models .....	3-56
3.14.1	Rollers versus Pins .....	3-56
3.14.2	Fixed versus Guided versus Free Bearings.....	3-61
3.14.3	Unusual Substructures and the Effect of Variable Substructure Stiffness .....	3-63
3.14.3.1	Straddle Bents and Other Variable Substructure Stiffness Effects.....	3-63
3.14.3.2	Integral Straddle Bents .....	3-65
3.14.3.3	Integral Pier Caps .....	3-67
3.14.3.4	Integral Abutments .....	3-70
3.15	Roadway/Structure Geometry Considerations.....	3-73
3.15.1	Suggestions to Simplify Structure Geometry in Curved and/or Skewed Bridges .....	3-74
3.15.2	Superelevation Effects on Loading.....	3-78
3.16	Second-Order Effects.....	3-80
3.16.1	Second-Order Analysis Methods.....	3-80
3.16.2	Analysis Assumptions versus Observed Behavior .....	3-81
3.16.3	Application .....	3-81

3.16.3.1	Straight Steel Girders, Trusses, and Arches .....	3-82
3.16.3.2	Curved Steel Girders .....	3-82
3.16.3.3	Narrow Steel Girder Systems .....	3-83
3.16.3.4	Global Second-Order Amplification .....	3-84
3.16.3.5	Second-Order Amplification of Flange Lateral Bending between Cross-Frames.....	3-86
3.17	Phased Construction, Redecking, and Widening .....	3-87
3.17.1	Consideration of Permanent (Dead Load) Deflections .....	3-87
3.17.2	Consideration of Transient (Live Load) Deflections during Phased Construction .....	3-88
3.17.3	Stability of Narrow Sections during Interim Phases of Construction.....	3-89
3.17.4	Considerations for Diaphragms/Cross-Frames between New and Existing Sections .....	3-89
3.18	Temperature Effects .....	3-90
3.18.1	Uniform Temperature Changes .....	3-90
3.18.2	Temperature Gradients .....	3-90
3.18.3	Temperature Effects during Construction .....	3-91
3.18.4	Wide and/or Highly Skewed Bridges .....	3-91
3.19	Analyzing Older Bridges .....	3-91
3.19.1	Bridge Inspections .....	3-91
3.19.2	Load Rating .....	3-92
3.19.3	Fatigue Evaluation .....	3-93
3.19.3.1	Load-Induced Fatigue.....	3-93
3.19.3.2	Distortion-Induced Fatigue.....	3-93
3.19.4	Bridges with Hinges .....	3-93
3.19.5	Load Testing as Part of Analysis of Existing/Older Bridges.....	3-94
3.20	Discontinuities in Structures.....	3-94

3.20.1	Stress Concentrations—Need for Detailed Stress Analysis .....	3-94
3.20.2	Access Openings .....	3-95
3.21	References to Benchmark Analysis Problems .....	3-95
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 Modeling (Shear Stiffness, Flexural Stiffness).....	4-1
4.1.1.1	Two-Dimensional Analysis Techniques .....	4-1
4.1.1.2	Three-Dimensional Analysis Techniques .....	4-2
4.1.2	Lateral Bracing .....	4-2
4.1.3	Narrow Systems, Stability Analysis .....	4-3
4.1.4	Narrow Systems, Redundancy Analysis .....	4-4
4.1.5	Variable Depth Girders .....	4-4
4.1.6	Width to Span Ratios and Influence on Secondary Effects .....	4-4
4.1.7	Improved Calculation of I-Girder Flange Lateral Bending Stresses from 2D-Grid Analysis .....	4-5
4.2	Tangent Steel Plate Girders or Rolled Beams .....	4-10
4.2.1	No Skew or Limited Skew (<20°) .....	4-10
4.2.2	Significantly Skewed .....	4-11
4.2.3	Multiple Different Skews .....	4-13
4.2.4	Through Girder Bridges .....	4-14
4.3	Curved Steel Plate Girders or Rolled Beams.....	4-14
4.3.1	Methods of Analysis .....	4-14
4.3.1.1	V-Load Analysis .....	4-15
4.3.1.2	Grid or Grillage Analysis .....	4-15
4.3.1.3	3D Finite Elements Analysis .....	4-17
4.3.2	Skewed and Curved I-Girder Bridges.....	4-18



4.4	Tub Girders—General Issues.....	4-18
4.4.1	Analysis of Internal Framing (Internal Intermediate Diaphragms, Top Flange Lateral Bracing)—Grid Analysis and Hand Calculations versus 3D.....	4-20
4.4.2	Consideration of External Intermediate Diaphragms .....	4-21
4.4.3	Cross Frame Modeling (Shear Stiffness, Flexural Stiffness).....	4-21
4.4.4	Narrow Systems, Stability Analysis .....	4-22
4.4.5	Narrow Systems, Redundancy Analysis .....	4-23
4.4.6	Variable Depth Girders .....	4-24
4.4.7	Recent Improvements to Simplified Analysis Methods for Tub Girders .....	4-24
4.5	Tangent Steel Tub or Box Girders.....	4-24
4.5.1	No Skew or Limited Skew (<10°).....	4-24
4.5.2	Significantly Skewed .....	4-26
4.5.3	Multiple Different Skews .....	4-26
4.6	Curved Steel Tub or Box Girders.....	4-26
4.6.1	Skewed and Curved Steel Tub or Box Girder Bridges .....	4-27
4.7	Bridges with Significantly Complex Framing .....	4-28
4.7.1	Variable Girder Spacing .....	4-28
4.7.2	Discontinuous Girders .....	4-29
4.7.3	Transfer Girders .....	4-30
4.7.4	Girder-Substringer Systems.....	4-31
4.7.5	Elevated T-Intersections .....	4-32
4.7.6	Single Point Urban Interchanges.....	4-33
	REFERENCES .....	R-1
	GLOSSARY .....	G-1

APPENDIX A	SURVEY OF CURRENT PRACTICE .....	A-1
A.1	Background.....	A-1
A.2	Participation in the Survey .....	A-1
A.3	AASHTO Responses .....	A-1
A.3.1	Overall Trends and Comments .....	A-1
A.3.2	Responses to General Questions .....	A-2
A.3.3	Responses to Specific Questions .....	A-3
A.3.3.1	Responses to Questions about Tangent (Straight) Steel Plate Girders or Rolled Beams with No Skew or Limited Skew (Skewed Less than 20° from Non-Skewed) .....	A-3
A.3.3.2	Responses to Questions about Tangent (Straight) Steel Plate Girders or Rolled Beams, Significantly Skewed (Skewed More than 20° from Non-Skewed).....	A-3
A.3.3.3	Responses to Questions about Curved Steel Plate Girders or Rolled Beams with No Skew or Limited Skew (Skewed Less than 20° from Non-Skewed) .....	A-3
A.3.3.4	Responses to Questions about Curved Steel Plate Girders or Rolled Beams, Significantly Skewed (Skewed More than 20° from Non-Skewed) .....	A-4
A.3.3.5	Responses to Questions about Tub Girders or Box Girders .....	A-5
A.3.3.6	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-5
A.4	AREMA Responses .....	A-5
A.5	Other Responses .....	A-6
APPENDIX B	.....	B-1
B.1	Recommendations on Methods of Analysis .....	B-1
B.2	I-Girder Bridges .....	B-1
B.2.1	I-Girder Bridge Level of Analysis Example 1 .....	B-4
B.2.2	I-Girder Bridge Level of Analysis Example 2 .....	B-5
B.2.3	I-Girder Bridge Level of Analysis Example 3 .....	B-6

B.3	Tub Girder Bridges .....	B-7
B.3.1	Tub Girder Bridge Level of Analysis Example .....	B-11

## 1 MODELING DESCRIPTION

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. Any analysis/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 finite element method (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.

### 1.1 Hand Analysis Methods

#### 1.1.1 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 Manual 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.

#### 1.1.2 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 (8) and curved bridges (65, 97).

#### 1.1.3 V-Load Method

The V-Load method (73, 83) 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.

#### 1.1.4 M/R-Load Method

The M/R Method (81) 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.

## 1.2 Finite Element Method

### 1.2.1 Basic Concept

The finite element method is also an approximate structural analysis method (98). 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 of this document.

### 1.2.2 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 1.2.4) 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* (8). 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 Section 3.11 and Section 3.12 of this document for further discussion of recent and future developments in 2D grid modeling.

### 1.2.3 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* (8). 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.

### 1.2.4 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 or diaphragms with consideration of shear deformation in addition to flexural deformation, modeling of girder supports, lateral bracing, cross-frames or diaphragms at their physical elevation within the structure, or combination thereof (29, 62). 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.

### 1.2.5 3D FEM Analysis Methods

The category 3D FEM Analysis methods is meant to encompass any analysis/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 or diaphragms 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.

Also, there are 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 specifications to date. The typical AASHTO methodology for design is generally based on assessment of nominal (average) stresses calculated by simplified methods, such as  $P/A$  or  $Mc/I$ , 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.

### 1.2.6 Element Types

There is a wide range of different element types used in general finite element practice. Cook, et al. (41) 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 finite element methods have inherent approximations and that some FE 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 to the book by Bathe (20). 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.

## 2 HISTORY OF STEEL BRIDGE ANALYSIS

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.

### 2.1 History of Change in the Complexity of Bridges

The invention of the automobile in the 1890s 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 multispan 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, some DOTs 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 design specifications, which improved the performance of the 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.

## 2.2 Bridge Design Code Advancements

With the development of the automotive industry in the early 20<sup>th</sup> 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 known design specification developed by AASHO in 1925 (AAHSO, 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 Load and Resistance Factor Design used in the *LRFD AASHTO LRFD Bridge Design Specifications* (5). 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.

## 2.3 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, 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.

## 2.4 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 by rule-of-thumb. 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 methods warranted?

Comprehensive quantitative answers to these questions are not yet available. In many cases there is no consensus in the industry about the answers to these questions, but some hints of the magnitude of these effects are beginning to appear thanks to recent research. At the time of the writing of this document, there are several initiatives that directly parallel the efforts of Task Group 13 and of this guideline document.

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 an arm of the National Research Council, which is jointly administered by the National Academy of Sciences, the National Academy of Engineering, and the Institute of Medicine. TRB has numerous committees, including AFF20, the Committee on Steel Bridges. Within AFF20 is a subcommittee, AFF20(1), dedicated to methods of analysis of 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 the recently completed 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 (88). 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, 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 2<sup>nd</sup> edition of this document, various subsequent research efforts are underway to better understand the influence of locked-in stresses, and 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.

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. Extensive and comprehensive Reference Manuals are provided. Contact the NHI for more details.
- The National Steel Bridge Alliance (NSBA) *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.

### 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.

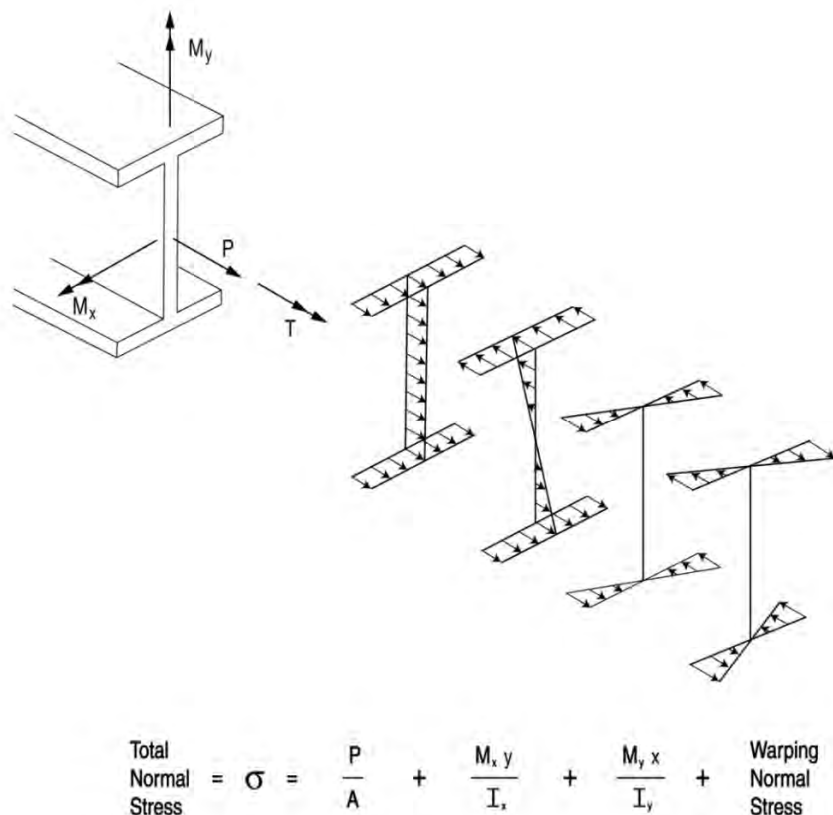
In the following sections, many of these effects will be characterized as effects of curvature. However, one should note that similar twisting and flange lateral bending effects occur in skewed steel girder bridges as will be described in detail later in this guide.

###### 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.

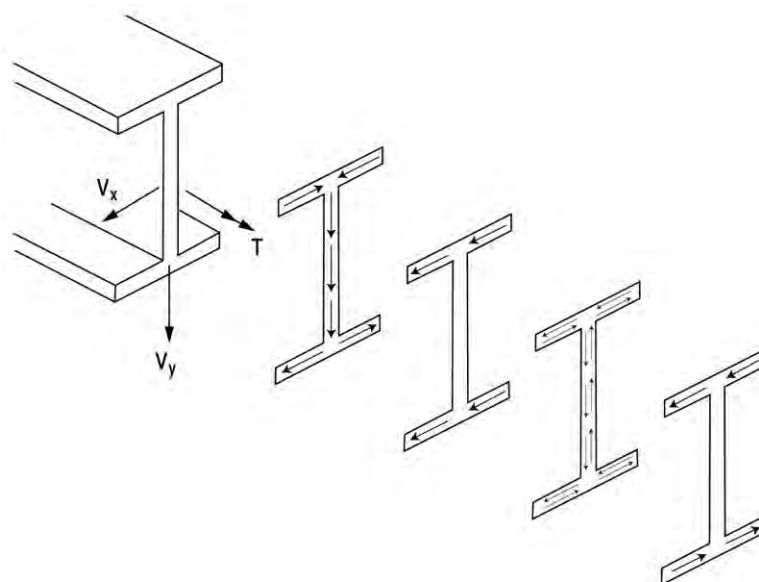
Box-shaped girders, on the other hand, are closed section members. Closed cells are extremely efficient at carrying torsion by means of St. Venant torsional shear flow because the shear flow around the circumference of the box has relatively large force couple distances (Figure 3.1.2-4). For this reason, 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 3.1.2-3). 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

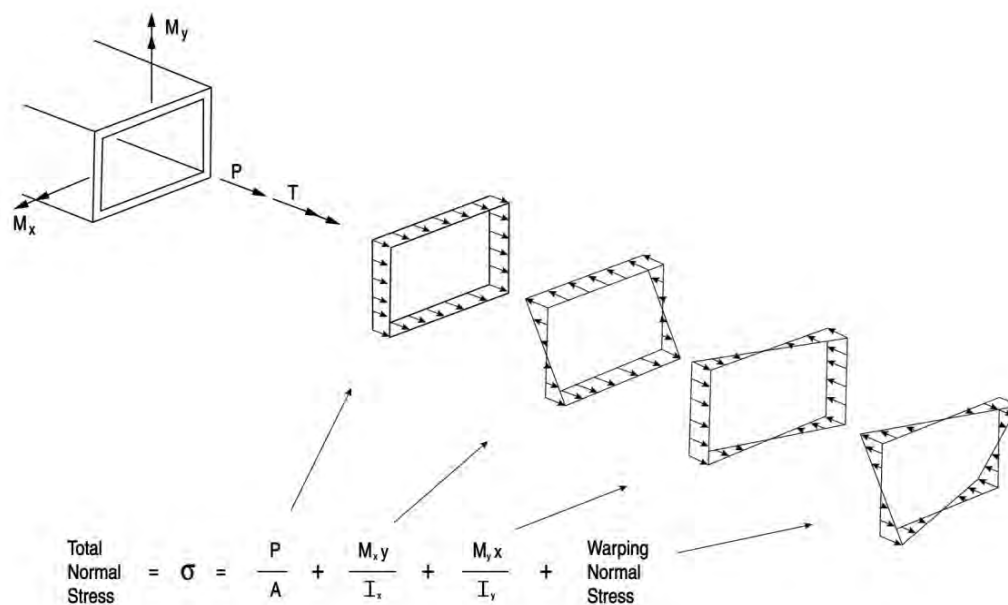


intermediate diaphragms. Cross-sectional distortion, the resulting stress effects, and the design of internal intermediate diaphragms are discussed in detail in Fan and Helwig (48).



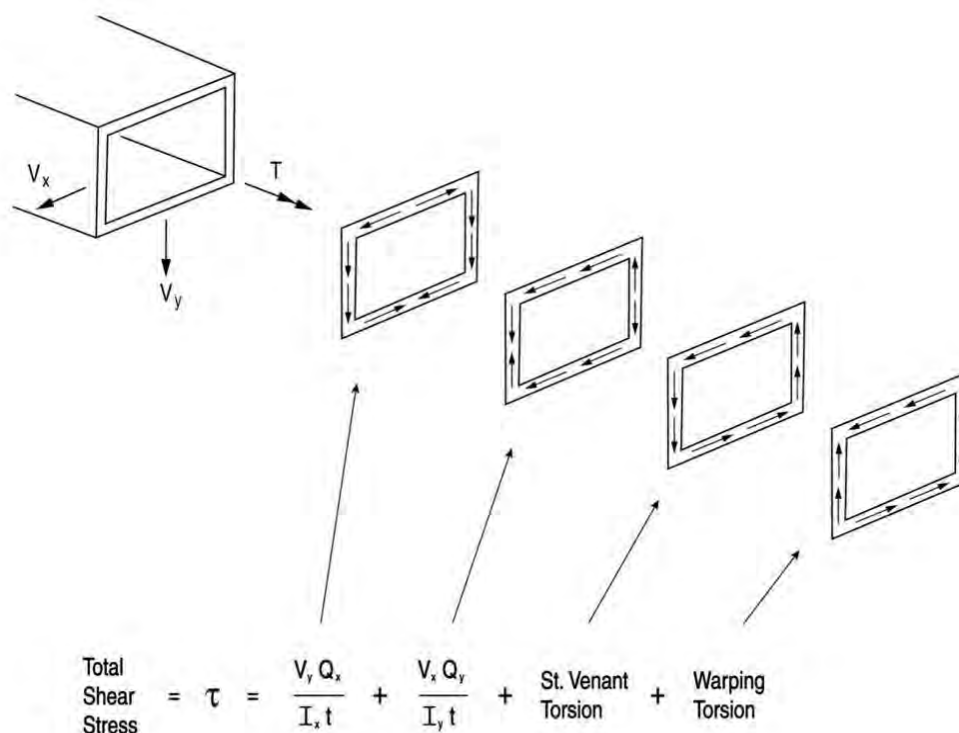
$$\text{Total Shear Stress} = \tau = \frac{V, Q_x}{I_x t} + \frac{V, Q_y}{I_y t} + \text{St. Venant Torsion} + \text{Warping Torsion}$$

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.



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

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, 3.8.4, 3.8.5, 3.8.6, 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

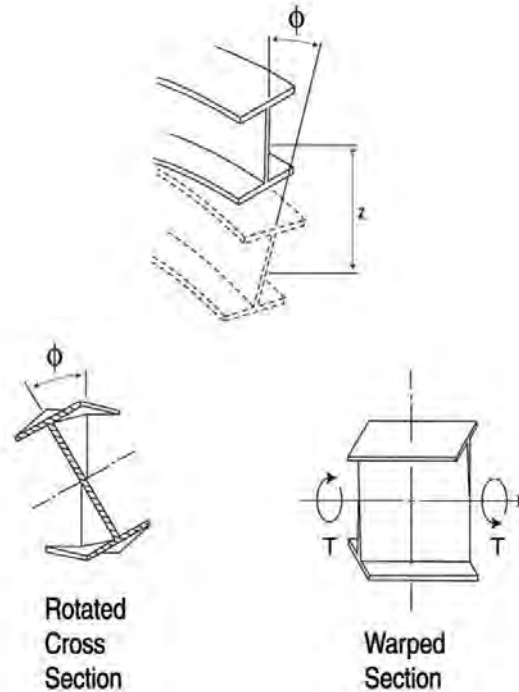
Many practical effects result from the way girders carry torsion. For example, the warping normal stresses for I-girders caused by torsion represent one source of what are called flange lateral bending stresses. These are an important part of the design equations for flange stresses in I-girders. Most 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. Recently, the *AASHTO LRFD Bridge Design Specifications* (8) were revised to suggest 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. Recent research (77) discusses the magnitudes of these deformations and the ability of various analysis techniques to quantify them.

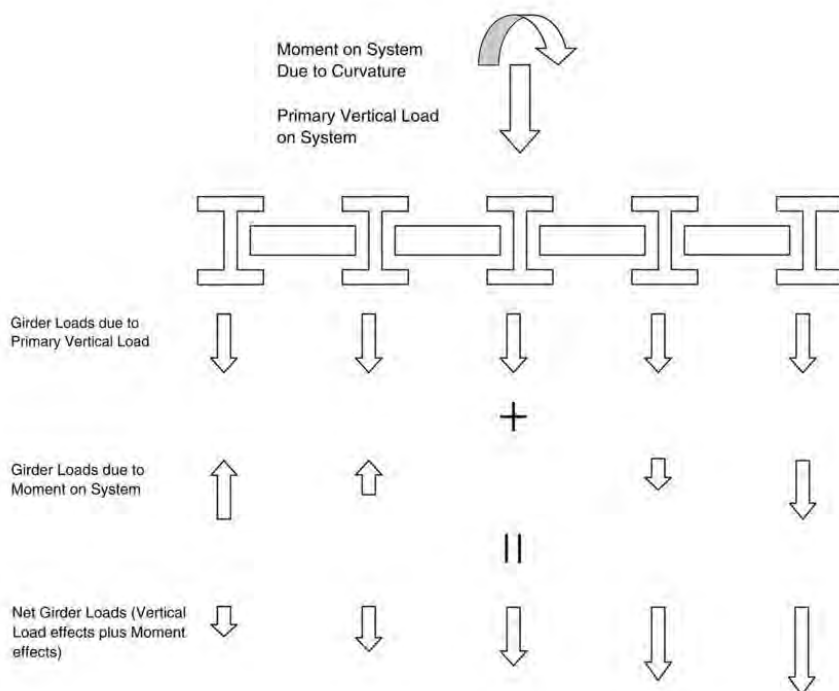
Again, note that these deformation issues are not exclusively limited to curved girders. Skewed bridges experience many of the same phenomena.

#### 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 (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 and Shura (42). Designers should also be aware that there are other contributors to global overturning effects, which occurs 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.

##### *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 constructibility.

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 diaphragm 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.

#### *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, constructibility 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 then 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. (80) 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 diaphragms 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.

#### *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.

#### *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, recent revisions to the AASHTO specifications and commentary permit calculations of moments to be based on the fully composite section based on observed behavior in numerous field tests, which suggest that some level of composite action occurs even when shear connectors are not provided. 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 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.

### **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 nation'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. 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**

<b>Load</b>	<b>AASHTO LRFD Load Category Abbreviation</b>	<b>Permanent or Transient?</b>	<b>Applied to Non-composite or Composite Structure?</b>	<b>Primary Direction of Action?</b>
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	Note 1	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* (8) 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 constructibility 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 longer or wider bridges, 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.

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 specifications 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 (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 or refined analysis based on relative stiffness. Empirical live load distribution factors are provided in the *AASHTO LRFD Bridge Design Specifications* (8) and are appropriate for many types of bridges, particularly tangent girder bridges. If a more refined analysis is used (e.g., a plate- and eccentric beam analysis or a 3D analysis), 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 (73, 83).

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.

### 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* (9). See also Article 3.8.

### 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 editions of the AASHTO specifications used a formula for calculating the effects of impact that varied with span length, but the current *AASHTO LRFD Bridge Design Specifications* (8) has simplified this and uses a single value of 33 percent for primary structure members.

### 3.3.10 Centrifugal Force

Vehicles traveling on curved roadway alignments develop a centrifugal force. The AASHTO LRFD specifications 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 the end diaphragms or cross-frames (the load carried through the deck to the support locations is transferred to the bearings through the end cross-frames or diaphragms). 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, et al. (73) and Article 3.15.2.

### 3.3.11 Braking

The AASHTO LRFD specifications 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* (8)) 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 end cross-frames or diaphragms.

### 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 of AASHTO LRFD strength design, the designer must extract output from that 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* (8) is based on whether a girder is considered compact or noncompact. A compact girder 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. AASHTO considers curved girders to be noncompact.

When designing noncompact girders using the stress-based provisions of the *AASHTO LRFD Bridge Design Specifications* (8), it is important for the designer to remember the superposition of stresses from the analysis model. It is important for the designer to carefully superposition 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. The designer must calculate the stresses in the girder at each stage of construction based on the loading and section properties of the girder during that stage, as well as take the summation of those stresses. In addition, the designer must consider the slab placement sequence and take into account any locked-in stresses that may be present. Many of the commercially available girder design programs will analyze the slab placement sequence but then do not consider the locked-in stresses for final design. The best approach is a rigorous tracking of the stress history of the girder through erection, slab placement, composite dead load placement, and in-service condition.

An additional consideration in the AASHTO provisions 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 diaphragm 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 specifications for highway bridges in a limited empirical way. Specifically, two simple provisions were incorporated into the *AASHTO Standard Specifications for Highway Bridges (14)* for Load Factor Design (LFD). 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, the *AASHTO Standard Specifications for Highway Bridges (14)* permitted 10 percent of the peak negative elastic moments in compact continuous-span members to be shifted 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) (10)*. 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. Plastic analysis methods were used to evaluate the strength for the maximum load combination, which was called the “Strength Limit State” in the first publication of the *AASHTO LRFD Bridge Design Specifications (5)*. Inelastic analysis-design principles were used to limit permanent deflections caused by local yielding at the interior supports under the overload condition, which was called the “Service Limit State Control of Permanent Deflection” in the first publication of the *AASHTO LRFD Bridge Design Specifications (5)*.

In 1994, the inelastic procedures from the ALFD guide specifications were incorporated into the new *AASHTO LRFD Bridge Design Specifications (5)* with minor modifications and additions. 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.

In 1993, Galambos, et al. proposed inelastic rating procedures for highway bridges (51). These methods utilize the same rating vehicles and load and resistance factors as the rating procedures adopted in the 1989 *AASHTO Guide Specifications for Strength Evaluation of Existing Steel and Concrete Bridges* 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.



Schilling, et al. (76) 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, ASCE developed a simplified procedure based on the shakedown limit state (18) 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. (76) 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  $\theta_{pm}$ . Based on trial design studies by Schilling (75) and Schilling, et al. (76), 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 (7). 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. (76) 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.

Most recently, Barth, et al. (20) 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 (19) and White and Barth (89). 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.

### 3.6 Fatigue Analysis and Evaluation

The fatigue provisions of the *AASHTO LRFD Bridge Design Specifications* (8) have been significantly modified from previous specifications, and while they do not often control the primary girder design, they may affect the cross frame design, 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 consider taking the stress range as 75 percent of the total stress range calculated in the analysis as described in Article C6.6.1.2.1. The peak stress range in the cross-frames from a 3D FEM or grid analysis is caused by two trucks in

adjacent lanes, likely outside of the striped lanes, and the 75 percent factor is to account for the unlikelihood of this event over the life of the structure.

### 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* (8) 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* (8) regarding the application of dynamic load amplification of live load (a.k.a. impact) for substructure design (consideration of impact is required) versus for foundation design (impact can be neglected).

## 3.8 Constructibility—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 the 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 of the document 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.

### 3.8.1.1 General Guidance

Procedures required for the general steel erection of highway bridges are provided in the *Steel Bridge Erection Guide Specification* developed through the *AASHTO/NSBA Steel Bridge Collaboration (16)*. 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. (36) and Stith, et al. (79).

### 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 (88) for detailed discussion of some of these considerations. One of the main goals of NCHRP Report 725 is to provide guidelines for the level of analysis required for investigating the erection of the steel framing. Various recommendations from NCHRP Report 725 have been incorporated into the 2<sup>nd</sup> edition of this document. Furthermore, see Section 4 for further details regarding guidelines for level of analysis, as these typically apply to the analysis of the steel erection as well. Regardless, sound engineering judgment is required in developing the model associated with the appropriate level of analysis, as well as in regard to investigating the erection sequence.

### 3.8.1.3 Investigation of Steel Erection Sequence

The engineer investigating the steel erection sequence on behalf of the contractor should investigate, at a minimum, the critical stages of erection. For each critical stage, girder stresses, vertical and out-of-plane girder displacements and rotations, cross frame forces, temporary support loads, and girder stability may need to be checked. These behaviors will be required to determine the viability of the chosen erection sequence and methods. Again, engineering judgment will be required to determine what stages may be critical prior to performing any analysis.

Critical scenarios at various stages of erection may include (but are not limited to) some of the following:

- A single girder is erected with few brace points provided, in which lateral torsional stability will be critical.
- A condition in which few brace points are provided in a multiple-girder system, again creating lateral torsional buckling concerns.
- A condition in which girders cantilever a significant length beyond a support.
- Out-of-plane rotations caused by a significant skew or curvature may make fit-up of cross-frames difficult.
- Wind loading that may cause instabilities of an incomplete steel framing system.

If a computer analysis is employed to investigate the steel erection sequence, whether it is a 2D or 3D model, it is typically easier to investigate all stages of erection and not just those that may be deemed critical ahead of time. A 2D or 3D finite element model can be employed to evaluate the step-by-step erection sequence, whereas 1D models or hand calculations may only focus on critical stages. The 2D or 3D model can be created for the completed steel framing, and then reconstructed stage-by-stage in accordance with the proposed erection sequence. In fact, some commercially available software programs have construction staging features that can aid in the modeling of the steel erection sequence.

For the analysis of the steel erection sequence, dead loads and construction loads need to be determined and applied to the appropriate elements in the model. Dead loads typically include the self-weight of the structural members and detail attachments. Wind loads must be considered by the engineer in the analysis of the steel erection sequence. The calculation of the wind load is typically performed in accordance with the governing specification or owner guidelines. Provisions should be made by the contractor's engineer to ensure that girders are stable in high wind events. These provisions may include lateral bracing, tie-downs at the supports, specifying a certain number of cross-frames to be erected, or providing limits on the wind speed when girder field pieces can be lifted into place. Hand calculations can be performed or a finite element model of the steel erection sequence can be used to determine the effects of wind, including member stresses and deflections at each stage of steel erection.

Increasingly, engineers are being required to evaluate the stability of steel members under partial stages of completion, for instance, the behavior of a beam suspended by a crane or spreader beams during lifting or the behavior of partly completed spans during erection with beams cantilevered past a support or partly suspended by holding cranes. Coupled with the use of higher strength steels, predominant use of composite construction, and liberalization of design criteria through the years, steel members are increasingly slender yet the design specifications lend little practical guidance on the evaluation of the strength of partly completed systems. Given the need to understand these potentially critical stages of construction, a brief discussion on available hand analysis methods is provided along with cross references dealing with the subject in greater detail. For other scenarios not covered here, eigenvalue buckling or nonlinear collapse analysis by 3D FEM should be considered.

### *Stability of I-Beams under Self-Weight During Lifting*

Unlike traditional buckling solutions, which presume that members have lateral and torsional restraint at the supports, a beam suspended by beam clamps, and a spreader beam is completely unsupported at its ends. Additionally, the points of support during lifting provide only a vertical reaction; no lateral or torsional restraint is provided by the lifting mechanism. Thus, conventional solutions cannot be used to analyze this situation. Engineers should not apply traditional LTB equations which assume an unbraced length equal to the distance between the lifting lugs; this will result in an unconservative prediction of strength. The apparent brace point effect comes from the counter-balancing effect of the torque contributed by the vertical component of the forces from the holding or lifting ropes, cables, or straps acting about the member shear center when the section starts to twist at the hold points. This effect is not the same as that of a genuine lateral or torsional brace support. In addition, moving the lifting or holding positions inward along the length of the beam from the ends reduces the maximum bending moments up to a certain point, so that the member is less prone to buckling based on load but not based on stability bracing. For a prismatic member held/lifted by vertical cables at two points, the weight that can be held/lifted without buckling of the member is maximized when the hold/lift points are located approximately at the quarter points of the member length.

The *Guide to Stability Design Criteria for Metal Structures*, 6th edition (98) provides specific direction on the design of beams for stability acting under the action of self-weight and suspended by two equidistant lifting points symmetric about midspan. The formulation is based on the work of Essa and Kennedy (45) and results in prediction of a critical buckling weight (to be compared to self-weight) in order to assess a factor of safety against buckling of the element. Essa and Kennedy find that the buckling resistance of a suspended load is greatest when the lifting points are near the beam's quarter points, and thus provide equations and graphs for the prediction of strength. They also find that the buckling strength is highly sensitive to the position of the cables along the girder length and recommend that the field conditions closely match the designed pick points, particularly if the lift location is in the vicinity of the quarter points. In addition to the evaluation of the strength of the suspended load, Essa and Kennedy provide an expression for the buckling strength of a spreader beam loaded with two vertical loads at its ends; expressions for spreader beams subjected to inclined sling loads are not provided by Essa and Kennedy but may be found in Dux and Kitipornchai (43).

### *Stability of Cantilevered Beams*

Another common scenario encountered in steel bridge erection is the assessment of the strength and stability of cantilevered sections. It is common to progressively build a bridge from one end to the other, inevitably leaving the “pier section” cantilevered past the pier and out to the next splice. The stability of a cantilevered continuous beam is not the same as a cantilevered beam with built-in fixed ends since the buckling strength of the anchor span influences the strength of the cantilever. The *Guide to Stability Design Criteria for Metal Structures*, 6th edition (98) provides a recommended procedure by Essa and Kennedy (46) dealing with various overhang loading and tip/backspan restraint conditions. In Article 5.2.4 of the older *Guide to Stability Design Criteria for Metal Structures*, 5th edition (50), an alternate method, based on the work of Nethercot, is provided. The strength of the section is given as:

$$M_{cr} = \frac{\pi}{KL} \sqrt{EI_y GJ} \sqrt{1 + \frac{\pi^2 EC_w}{(KL)^2 GJ}} \quad (3.8.1.3-1)$$

where the effective length of the cantilever,  $KL$ , is a function of the restraint at the pier, the restraint at the tip, the nature of the loading, and recommendations are described. A summary of the Nethercot design approach, design example using the SSRC guidelines, and a chart of the relevant “K” factors for various restraint conditions can be found in Eskildsen (44).

### *Temporary Supports*

Temporary supports should be adequately considered during the investigation of the steel erection sequence. When temporary supports are used, particular attention should be paid to the reaction imparted on the temporary support. This reaction will vary throughout the steel erection, and it may be the case that an intermediate stage of erection may govern the loads applied to the temporary support.

### *Stability throughout Erection Sequence*

Lastly, stability must be ensured throughout the steel erection sequence. Even though a girder has been designed to accommodate all possible loading conditions in its final condition, the strength of the girder may not be adequate for temporary conditions that can arise during the erection of the bridge. The final bridge will have typical cross frame spacings in the range of 15 to 25 ft. Temporary conditions, however, can include cross frame spacings several times larger, and in some cases, such as the first girder being erected, will not have any functioning cross-frames within its span. These long unbraced lengths due to the lack of installed cross-frames can have a tremendous effect on the buckling capacity of the girder. So, although the loading may be many times lower than what the bridge will see in its final condition, the buckling strength may be reduced by a significantly larger and critical amount. See Article 3.8.7 for additional details regarding stability during construction.

## **3.8.2 Deck Placement Sequence**

Deck placement effects must be considered an integral part of the design of steel bridges and not considered simply the responsibility of the contractor. Although the contractor is ultimately responsible for deciding how to pour the deck in a steel bridge, and may request changes to a sequence specified shown in the design documents, this must be done with forethought and insight as to the implications of deck placement on the behavior of steel bridges.

In Article 3.2, the section properties related to steel bridge construction are introduced. The Level 2 analysis discussion provides a description of the sequential slab placement analysis for steel bridges. The concept of staged placement is introduced. The sequence of deck pours is selected to achieve the following: minimize the potential for uplift, minimize the potential for slab cracking, and prevent overstress of the steel structure.

Two approaches to deck placement analysis have been used—one that assumes simultaneous placement and another that considers multiple sequential pours. In an analysis that assumes that the entire weight of the wet slab is applied simultaneously, the distribution of forces along the length of the girder is a function only of the girder moment of inertia. However, this simultaneous placement is frequently impractical and slabs are more typically poured in at least two and, many times, more stages.

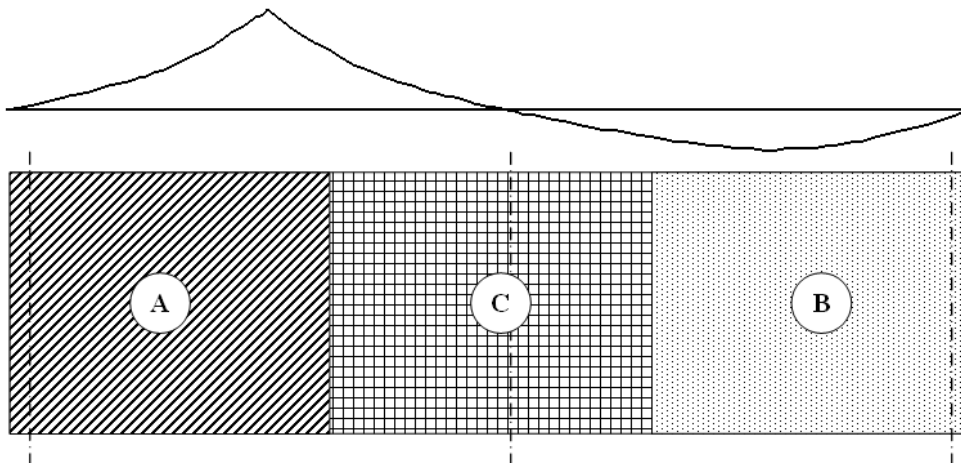
When slabs are placed in multiple stages, several days typically elapse between sequential pours—enough so that composite action is attained between the girder and previously poured slab sections. This affects the moment of inertia in the previously poured sections and greatly increases the stiffness of the section. When sequential pours are placed from pour two to the final pour, the distribution of forces is affected by the previously poured composite sections. The eventual accumulated moments, shears, and deflections at a given point are different from a staged analysis than from an analysis assuming simultaneous placement.

An additional aspect of slab pour sequencing is the effect on instantaneous forces at a section as opposed to final forces. Figure 3.8.2-1 is used for illustration.

A typical two-span bridge slab placement sequence is shown. When Pour “A” (see Figure 3.8.2-1) is placed, it loads the positive moment portion of the influence line for the midspan region of Span 1.

This pour is then allowed to cure for several days. In the next stage, Pour “B” is placed; two things occur. First, the positive moment in Span 1 is reduced since the load is placed in an adjacent span. Secondly, due to the application of negative moments to a region of the bridge with a previously poured slab, there is a possibility of slab cracking under these negative moments. Finally, to minimize the amount of cracking over the piers, these sections (Pour “C”) are placed last.

In the case of instantaneous placement of the slab, the positive moments in Span 1 will be less than when only Pour “A” is placed. There is usually a significant difference in the positive moment obtained from these results, and the engineer is advised to perform the constructibility checks of steel structures based on the peak moments obtained from a placement sequence analysis and not based on the final moments assuming simultaneous placement of the loads.



**Figure 3.8.2-1. Illustration of slab placement sequence and effects on instantaneous forces at a section as opposed to final forces—influence line for the positive moment at the cross section corresponding to the cusp in the diagram.**

The deck placement sequence also has an effect on other aspects of bridge behavior including uplift, deflections, and bearing rotations. Uplift can be a concern in deck placement sequences and should be evaluated. For relatively light steel framing and heavy concrete loads in adjacent spans, uplift can occur under some circumstances. If the presence of uplift is indicated in an analysis, the analysis is technically invalid since it assumes a bearing with tension capacity. To use the model, one must either provide bearings with tie downs, temporarily ballast the uplift bearings to add dead load at the lightly loaded locations, or revise the model to remove the degree of freedom constraint in the vertical direction. The allowance of uplift is discouraged. Revised placement sequence limits or temporary ballasting of the bearings is recommended.

The *AASHTO LRFD Bridge Design Specifications* (8) specifically address the possibility of deck cracking during the placement sequence. Continuous steel bridges have traditionally required that the longitudinal steel in a deck slab equal at least 1 percent of the cross-sectional area of the deck, yet no guidance on the limits of placement were provided. Engineers would frequently use a dead load point of contraflexure to establish these “negative moment regions,” yet the point of dead load contraflexure is of little meaning to the possible tension stress in a slab. AASHTO 6.10.1.7 dictates that if the factored stress in the deck slab due to the slab placement sequence or an in-service condition exceeds 90 percent of the modulus of rupture of the deck concrete, then the 1 percent steel requirement applies to the section in question. Thus the designer is advised not only to check the girders for strength during the pour sequence analysis but also to track the deck slab stresses as well to aid in placing the required slab reinforcing.

### 3.8.3 Overhang Analysis and Effects on Girders

The loads stemming from bridge deck overhangs have a unique effect on steel bridges that must be considered as part of the design. The selection of the overhang must first be made as part of the proportioning of the transverse section of the bridge. The selection of the overhang must be done with consideration for load balancing between exterior and interior girders, with consideration of loads on the overhang forming system, and with special consideration to local flange stresses that are generated by the use of traditional closely spaced overhang form brackets.

#### 3.8.3.1 Selection of Overhang Width versus Beam Spacing

One of the first considerations in selection of a deck overhang width is to choose an overhang width that results in relatively similar final beam sizes for interior and exterior stringers, the focus herein being for common I-beam bridges with constant girder spacing.

Load sharing amongst steel stringers in a traditional bridge is complex, the behavior becoming more complex with the introduction of skew or curvature, or both. However, even for the basic case, many factors enter into the relative load distribution between stringers. It can often be seen in refined analysis models that all girders in the cross section tend to deflect as a system under the weight of the wet concrete deck rather than as individual girders; the cross-frames serve to equalize the deflections and thus the internal loading. There has also been recent research supporting similar conclusions (49, 70). However, dead loads applied to the overhang itself, such as cast-in-place railings, are distributed somewhat more complexly to the various beams. Live load distribution to the beams is also influenced by the overhang geometry, particularly by the width of the overhang and the distance from the gutter line to the centerline of the exterior beam. Thus, the selection of the overhang width has a pronounced effect on the total loads applied to the exterior beam of a multi-girder steel beam bridge.

Analysis of the distribution of non-composite dead loads, composite dead loads, and live loads will reveal a range of overhangs that result in the total demand being comparable for interior and exterior stringers and thus attaining the greatest efficiency of the total system. Past experience suggests that overhang widths in the range of  $\frac{1}{4}$  to  $\frac{1}{3}$   $S$  ( $S$  is the beam spacing) result in reasonable load sharing between interior and exterior stringers.

#### 3.8.3.2 Selection of Overhang—Effect on Forming

A typical exterior overhang forming system is depicted in Figure 3.8.3.2-1. Trussed overhang forming brackets and temporary wood form materials are used to cast the overhangs for steel bridges. These forms and brackets must be able to support the weight of the wet concrete (a function of thickness and overhang length), the weight of construction workers on the temporary work platform, and frequently the weight of the finishing machine, which is usually supported from the brackets. As overhang width increases, the cost and complexity of this forming system goes up. No specific recommendation is given here since it is the contractor's engineer who will eventually provide the design of the brackets and forming. However, understand that larger overhangs lead to more expensive forming. It is the suggestion of this guide that if the engineer adheres to the recommendations of overhangs being on the order of  $\frac{1}{4}$  to  $\frac{1}{3}$   $S$ , traditional forming materials will be able to be used without much difficulty or increase in expense.

The design of steel bridges has traditionally ignored the force effects arising from the effects of overhang loading and the use of finishing machines. Unfortunately, these loads can apply a significant amount of torsion and flange lateral bending on the exterior girder. With I-sections being noticeably weak in the out-of-plane direction, the combination of major axis bending (primary moments during deck construction) plus flange lateral bending (from the overhang brackets) can create a serious stress condition that must be evaluated. High axial stress in a compression flange coupled with lateral bending further tends to destabilize the compression flange and results in



amplification of stresses similar to the moment magnification due to  $P-\Delta$  effects on steel beam-columns; a method to address this effect is provided in Section 6 of the *AASHTO LRFD Bridge Design Specifications* (8).

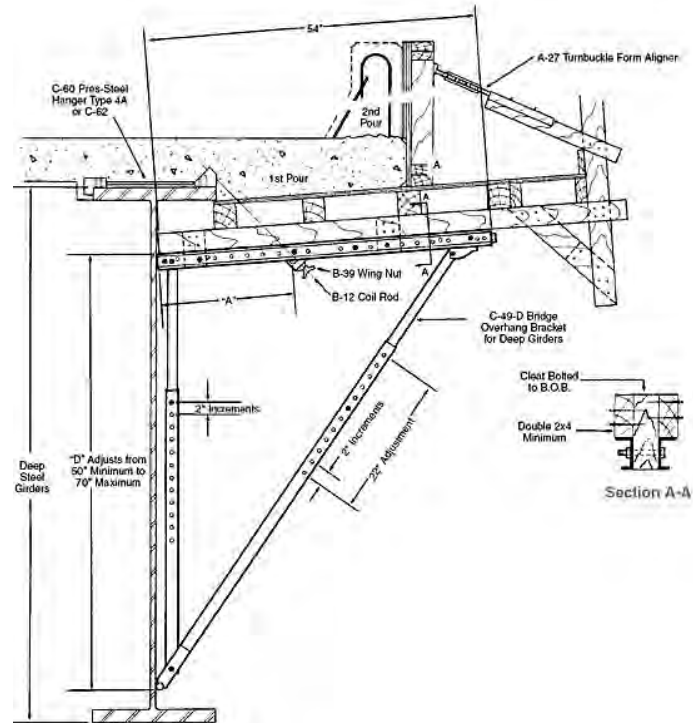
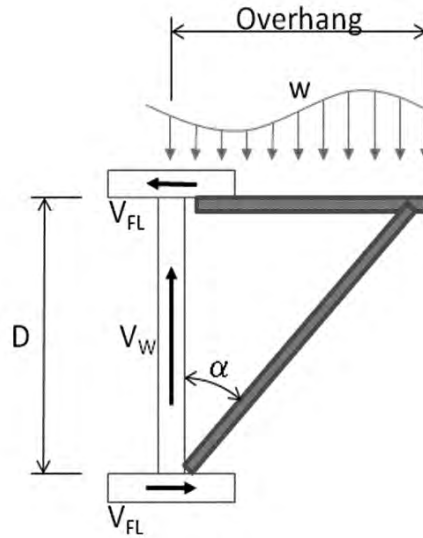


Figure 3.8.3.2-1. Typical overhang falsework bracket (courtesy Dayton Superior).

### 3.8.3.3 Overhang Effects on Structural Design

The lateral force on a flange ( $V_{FL}$ ) from vertical loads applied to the overhang ( $w$ ) is depicted in a simple static force diagram shown in Figure 3.8.3.3-1. Due to the eccentricity of the vertical loading and the method of connection of the form bracket to the flange, a large lateral component is generated with most common bracket geometries and overhangs. The problem of lateral flange loading is made worse by either wide overhangs or shallow beam depths—the worst case being a bridge with both of these features. As the angle  $\alpha$  increases, so does the lateral component  $V_{FL}$ .



**Figure 3.8.3.3-1. Free-body diagram of a typical overhang bracket system.**

The unified design provisions for steel bridges can be used to assess the force effects from overhangs. Using the “ $1/3$  rule equations” for flange stress interaction, an engineer should assess the strength of flanges subjected to combined major-axis bending and flange lateral bending. Guidance is provided in the *AASHTO LRFD Bridge Design Specifications* (8) to approximate the flange lateral bending stress in the girder flanges for flanges subjected to a uniform lateral flange loading (self-weight of overhang concrete and forming system can be included in this category) or flanges subjected to a discrete out-of-plane loading in between brace points (e.g., a set of wheels from a finishing machine). The equations are as follows for uniform loading and point loading, respectively. These are recognized as equations for fixed end moments for a beam with a uniformly distributed load and for a point load at midspan, respectively:

$$M_\ell = \frac{F_\ell L_b^2}{12} \quad (3.8.3.3-1)$$

$$M_\ell = \frac{P_\ell L_b}{8}$$

The length of interest,  $L_b$ , is the spacing between cross-frames, which effectively restrains the flange lateral bending. The moments from the uniform and discrete loadings are provided by the given formulas. It is then a simple matter to use the section modulus of the flange in the out-of-plane direction to convert these moments into flange tip stresses.

Once the flange tip stresses are determined, the designer must determine whether the first-order results, those obtained by statics alone, can be used or if amplification is required due to moment magnification effects. Amplification of flange lateral bending moments is only required in the compression flange; when checking the tension flange, moment amplification does not apply.

The resulting flange lateral stress (first- or second-order) in the compression flange is combined with the flange major axis bending stress using the  $1/3$  rule equations and the flange is checked for strength under this combined loading. When checking a tension flange, the designer should apply the peak flange lateral bending stress ( $1.0 f_{lat}$ ) to prevent yielding of the flange tips, as specified in the *AASHTO LRFD Bridge Design Specifications* (8).

These construction stresses are typically considered a temporary stress condition. When the overhang bracket is removed, bottom flange rebounds. The top flange is continuously braced by the hardened

deck in the final constructed condition of the bridge and so the stress effects in the top flange are typically neglected in the final condition.

In summary, the selection of bridge deck overhangs should: 1) be done with consideration of relative balance of primary forces in the interior and exterior beams (the selection of overhang as a fraction of beam spacing), 2) consider the forces in the overhang forming system, 3) consider the local flange bending effects induced by the overhang forming system in the exterior girder flanges, and 4) consider the level of transverse deck reinforcing required in the overhang.

### 3.8.4 Wind Loading during Construction

Design of bridges for wind loading has traditionally focused narrowly on the design of cross-frames to brace flanges in the final condition. In this scenario, a tributary length of flange is loaded with wind. Wind loading on the lower half of the girder is transferred to the cross-frame, which transmits this load to the deck. Wind load on the upper half of the girder is assumed to be transferred direction to the deck supporting the top flange. However, during construction, the load path is dramatically different.

The designer is referred to *AASHTO LRFD Bridge Design Specifications* (8) Section 4.6.2.7 for specific guidance on wind load force determination. Three wind load paths are provided.

In the basic bridge, the third load path is most common, which AASHTO states as:

Lateral bending of the flange subjected to the lateral forces and all other flanges in the same plane, transmitting the forces to the ends of the span, for example, where the deck cannot provide horizontal diaphragm action, and there is no wind bracing in the plane of either flange.

In this model, the wind on an entire span results in lateral bending of the girder system. The global moment is taken evenly by all beams. Additional local bending of the exterior girder between cross-frames also occurs and is additive to the global span-based moments. With a trend toward deeper girders, longer spans, and fewer beams in a cross section, the flange lateral bending stresses caused by wind loading are increasingly significant and can influence member proportions that are otherwise sufficient.

The *AASHTO LRFD Bridge Design Specifications* (8) recommend that construction conditions be analyzed using the acting loads combined according to the Strength Load combinations I through V. The calculation of service wind load based on 100 mph design speed with adjustments for height and exposure is unchanged for the construction condition, though it is recognized that the Strength III factor on wind can be reduced from 1.40 to 1.25. This is functionally equivalent to using a reduced wind speed and recognizes the short duration over which the bare steel framing is exposed to wind.

Some owners maintain alternate or additional criteria for checking wind load during construction. For instance, the Pennsylvania Department of Transportation developed standards for wind bracing evaluation for steel bridges based on a design wind pressure of 25 psf and an analysis of stresses based on the standard specifications. Additionally, the need for lateral bracing is based on lateral deflection of the bridge exceeding  $L / 150$ . If deflections exceed  $L / 150$ , the designer is directed to include permanent lateral bracing in the as-designed plans. This lateral bracing system will effectively restrain flange lateral bending and serve as the primary lateral wind load resisting system. Other agencies may provide different guidance; designers are cautioned to check for any owner-specific guidance on this issue.

Designers are encouraged to perform an analysis of lateral wind loading on the completed steel framing. The analysis should assume that the loads consist of the weight of steel plus any stay-in-place forming and reinforcing steel but prior to deck placement. The wind load procedures of the LRFD

specifications should be followed unless superseded by an agency directive or policy to the contrary. The steel framing should be checked for the vertical and lateral force effects arising from the combined loading. It is considered unreasonable to also include the weight of slab and finishing machines in this analysis since it is impossible to pour a slab in the 100 mph wind conditions assumed as part of the base wind pressure analysis. If the combined stresses in the girders are not in compliance with specification requirements, the flanges can be resized to reduce the flange stress or a lateral bracing system can be added to increase the lateral stiffness of the structure.

### 3.8.5 Live Loads during Construction

#### 3.8.5.1 Bridge Construction

Typically, live loads are not applied to the steel superstructure during erection of the steel framing. Live loads during construction usually occur during the placement of the concrete deck. These live loads during construction can consist of the finishing machine loads, which typically apply load to rails placed along the overhang brackets. The weights of these finishing machines are often available in manufacturer catalogues, and care should be taken in making sure the weight coincides with the width of finishing machine required. As discussed previously, the finishing machine will impart a vertical load on the overhang bracket, which will then impart reaction loads on the top and bottom flanges of the girder if the bracket is deep enough to bear on the girder near the bottom flange web junction. If the bracket does not bear at this location, it will impart an out-of-plane force on the web that needs to be considered. The design engineer should consider the reactions imparted to the top and bottom flanges of the girder from the overhang bracket as part of the girder constructibility checks specified in the *AASHTO LRFD Bridge Design Specifications* (8).

#### 3.8.5.2 Bridge Rehabilitation or Demolition

Construction live loads should also be considered during rehabilitation and/or concrete deck removal projects as well. In some cases, large loads may be placed on a superstructure during a rehabilitation project, which can include crane loads, hauling equipment, temporary supports, or any other type of equipment that may be used to perform the rehabilitation.

The consideration of live loads during deck removal is also quite important. When removing the deck, the deck slab is typically saw-cut into manageable pieces to allow removal of each piece. An excavator is often placed on the deck to pick the slab pieces. When the excavator or any other equipment is on the deck, it must be ensured that the deck slab is adequate to handle the weight of the equipment, including consideration of any saw cuts and other damage. In some cases, the tracks of the excavator can be kept directly over the beams to limit the load on the deck.

The steel superstructure (typically girders, floor beams, or stringers) must be examined to ensure it can adequately support the excavator and other equipment on the bridge (such as a triaxle truck hauling debris away). Whether or not the deck slab is composite with the girders is an important feature for the girder checks. When the deck slab is saw-cut transversely, the composite action between the girders and deck slab is lost if the girder was originally composite. For this reason, non-composite girders are often easier to remove since cutting the deck slab does not theoretically affect the strength of the girders (although some level of composite action will most likely be present). For composite girders, however, cutting the deck slab significantly reduces the girder capacity and often requires making the transverse saw cuts as the excavator progresses off the bridge rather than making all saw cuts up front. In addition, as the deck slab is removed, the lateral support it provides disappears and lateral buckling of the girders becomes an issue. Therefore, the procedure for saw cutting deck pieces and the loads applied during deck removal and their location should be carefully considered in deck removal projects.

### 3.8.6 Miscellaneous Construction Loading

Every bridge construction and rehabilitation project is unique; therefore, thorough consideration of loads to be applied to the particular structure is required. Additional dead loads such as the stockpile of raw materials are another load that should be considered, especially during rehabilitation projects. The consideration of these stockpile loads has been advised by the FHWA based on the collapse of the I-35W bridge in Minneapolis, Minnesota in 2007. The reader should refer to the FHWA guidance regarding the consideration of the stockpiling of materials on bridges. These loads should always be considered and applied appropriately during the analysis of the structure should they occur during the construction or rehabilitation of a bridge.

### 3.8.7 Stability Analysis during Various Stages of Construction

The stability of the steel superstructure needs to be considered at various stages of construction, including steel erection, concrete deck placement, and during structure demolition. Within this section of the document, consideration of stability during steel erection and deck placement will be discussed. In many cases, using static, linear elastic analysis models will be sufficient in obtaining girder demands that can then be compared with buckling capacity provisions in the *AASHTO LRFD Bridge Design Specifications* (8) (local buckling, lateral torsional buckling). However, in some cases, a linearized eigenvalue buckling analysis may be warranted, should the stability of the structure at the given construction stage be of concern to the engineer.

The stability of girders during picking and lifting operations should be considered by the engineer employed by the contractor to investigate the erection of the steel framing. The girder being lifted has not necessarily been designed for the loading and support condition associated with the lifting operation. Consideration should be given to the flange stresses resulting from the vertical bending moment resulting from the particular girder pick; for curved girders, torsional stresses should also be considered.

The erection of the steel framing, whether the bridge is straight or curved, is one of the most critical stages with regard to ensuring stability. The girders during erection will have much longer unbraced lengths than that of the final constructed structure. A longer unbraced length significantly reduces the lateral torsional buckling capacity of the girder, and the girder is susceptible to buckling under its own self-weight. Lateral torsional buckling capacity can be determined in accordance with the *AASHTO LRFD Bridge Design Specifications* (8). Girder flange stresses can be determined via the appropriate level of analysis, with consideration given to wind loading as necessary, and flange lateral bending stress amplification in accordance with Article 6.10.1.6 of the *AASHTO LRFD Bridge Design Specifications* (8).

With two girders erected in a span and enough cross-frames installed to ensure that single-girder buckling between the cross-frames will not occur, the two girders may still be unstable when examined as a system. This phenomenon, referred to as global buckling, is not a typical design requirement specified in the Specifications (such as yielding, local buckling, and lateral torsional buckling). Failures have occurred in the past as a result of global buckling. For example, a single box girder bridge in Marcy, New York failed during deck placement from global buckling, as reported by Yura and Widiyanto (96). Global buckling is the two-girder system buckling laterally as a group. Yura, et al. (95) have studied this global buckling phenomenon and have developed a method to determine the global buckling capacity of a multi-girder system. Although the most critical application for this equation is for two-girder structures during deck placement, temporary two-girder systems during girder erection should be examined to ensure that the system is stable. Alternatively, the global buckling capacity of a two-girder system can be determined with the use of a finite element model of the subject structure, as discussed by Kozy and Tunstall (66). An eigenvalue buckling analysis can be

performed and the load at which the system will buckle can be determined and compared to the load applied to the given structural configuration.

Sufficient bracing needs to be provided at each stage of steel erection to prevent instabilities of the framing system. A sufficient number of cross-frames (diaphragms) are needed to provide the torsional stiffness necessary to prevent buckling of the girder system. The contractor typically prefers to save as much of the cross frame installation as possible until after the bulk of the girder erection has been completed. The engineer, however, needs to balance this goal with the need for cross-frames to provide girder brace points to prevent lateral torsional buckling. In addition to preventing buckling, enough cross-frames must be installed to ensure that the girders are not overstressed due to lateral wind loads. Methods have been developed by Yura, et al. (95) that can be used to initially investigate the stiffness provided by cross-frames. Due consideration should be given by the engineer to the assumptions in developing these methods. If the actual stiffness provided is calculated to be near the minimum stiffness required, it may be appropriate to investigate the particular stage of erection through an eigenvalue buckling analysis in which the critical load can be determined for the particular portion of steel framing.

The placement of the concrete deck also needs to be considered with regard to stability. The deck placement sequence is discussed in detail in Article 3.8.2. The flange major axis bending stresses resulting from the weight of the wet concrete and the flange minor axis bending stresses resulting from the reactions applied to the girder by the overhang bracket need to be considered by the design engineer as part of the constructibility design checks. Flange lateral bending stress amplification should also be considered in accordance with Article 6.10.1.6 of the *AASHTO LRFD Bridge Design Specifications* (8). The flange stresses should be compared to the local flange buckling and lateral torsional buckling capacities. In most cases for the deck pour sequence, local stability issues will need to be checked, such as local flange buckling and/or lateral torsional buckling, between cross-frames. However, in some cases global buckling may be a concern, as discussed by Yura and Widiyanto (96) with regard to the Marcy Bridge collapse during the concrete deck placement.

### 3.9 Prediction of Deflections

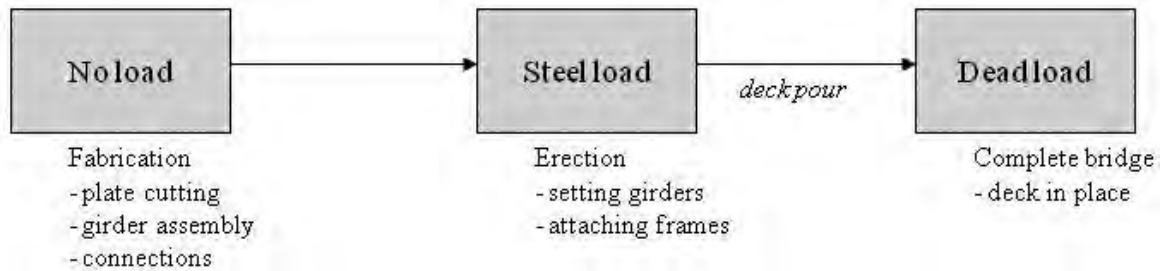
#### 3.9.1 General

It is well understood that girders deflect under their own weight as well as under the weight of the deck. However, deflection behavior of skewed and curved steel girder systems is complex and not readily understood without experience.

Girders can exist in three conditions with respect to vertical deflections:

1. No load—There is no weight on the girder, either from itself or from the concrete deck. This occurs when the girder is blocked to the no-load condition.
2. Steel load—The girder is under its own weight but not the weight of the deck. This occurs when the girder is vertical and erected within the steel framing system but without the concrete deck.
3. Dead load—The girder is under its own weight and the weight of the deck. This occurs once the deck is poured.

Girders transition from no load, through steel load, and to dead load by fabrication through construction, as shown in Figure 3.9.1-1:



**Figure 3.9.1-1. Three conditions girders experience from inception of fabrication to completion of bridge in place in the field.**

When evaluating deflections of steel girder bridges, designers are encouraged to give some consideration to the behavior of the entire steel framing as a system, as opposed to only considering girders as isolated structural elements. Even in relatively simple tangent girder bridges with no skew, research has shown that there is some measure of system behavior that affects the distribution of the weight of the wet concrete deck and thus affects the dead load deflections of the girders (49, 70).

In addition, designers should consider the effects of the anticipated deck placement sequence on dead load deflections. In some cases there is not a significant difference between deflections predicted assuming the entire deck is placed instantaneously versus deflections predicted assuming the deck is placed in sections following a pour sequence. However, in other cases, the deflection predictions can be significantly different (as much as several inches). There are no simple rules of thumb to guide designers as to when this effect may be significant; often, the only way to know is to perform the analysis both ways and compare the results. At a minimum, it is advisable that the deflections shown on the plans are based on an analysis that is consistent with how the plans direct the contractor to place the deck.

Dead load deflections are used extensively by the detailer, fabricator, and the contractor for a number of purposes, including the establishment of web/girder camber geometry, cross frame detailing, deck form setting, etc. It is important that the dead load deflection information shown on the plans be correct and accurate. However, it is also important that designers recognize that no analysis is perfect and that variables beyond the control of the designer may render their predicted deflections slightly inaccurate. Designers are advised to take reasonable prudent steps to provide for construction tolerances with regard to dead load deflections; one such recommendation is to consider the possible variations in dead load deflections, web camber geometry, etc., and provide additional haunch (or build-up) depth between the girder and the deck.

### 3.9.2 Skewed Bridges

As individual pieces, girder deflection behavior is simple: girders move down vertically as self-weight and, in theory, as deck weight is added (in theory because deck is not actually added to individual girders).

When girders are joined together by cross-frames and diaphragms to form the bridge framing system, the girders no longer deflect as individual pieces. Rather, the framing system deflects as a unit. First, the framing system deflects under steel load, including the weight of the girders plus the weight of the additional framing elements; then later the framing system deflects under the weight of the concrete deck.

On skewed bridges, girders twist in response to deflections. See detailed discussion and figures in Article 4.2.2. In a steel girder bridge framing system, I-girders are relatively flexible and cross-frames are relatively stiff. When girders deflect, the cross-frames translate down with the girders; however,

on a skewed bridge with cross-frames perpendicular to the girders, the cross-frames do not translate the same amount on either side because the deflections of the girders to which they are joined are not the same. The in-plane shear stiffness of the cross-frames is much greater than the torsional stiffness of straight I-girders. As such, the majority of the response to the differential deflection is exhibited as twisting of the girders rather than racking of the cross-frames.

The twisting is a normal and simple phenomenon, but there are some important implications that must be understood:

- The twisting causes the girders to be out-of-plumb prior to deck placement (i.e., when subject to steel dead load only); this out-of-plumb condition is normal, but will be a cause for concern for field personnel who do not understand this behavior.
- During erection, the girders are twisted into the steel load shape by the contractor; due to the relative flexibility of the girders, this is readily accomplished for most systems using come-alongs or similar means.
- The amount of twist does not need to be calculated; the amount of twist is a function of what is needed to accommodate girder deflections and keep the cross-frames square. Therefore, the cross-frames naturally set the girders to the correct amount of twist.
- At the connections between the girders and cross-frames, the bolts must be tightened before the deck concrete is poured so that the frames can properly preset the girders and also properly cause the girders to rotate back to plumb as the deck is poured.

The points above apply when the steel has been detailed for girders plumb under total dead load when construction is complete, which is the way that straight (tangent) girder bridges (skewed or not skewed) are typically detailed.

From an analysis standpoint, this phenomenon is generally inconsequential. Because the girders rotate back to plumb under the weight of the deck, there are no girder twists in the final condition and so there are no associated construction loads to consider. However, there is an exception: girders plumb at steel load.

As an alternative to twist-to-plumb behavior described above, it is possible to detail the bridge such that the girders will be plumb prior to the deck pour. However, under this approach the girders will twist when the deck is poured due, once again, to the deflection differences at each cross-frame; therefore, in the final condition, the girders will be twisted out-of-plumb. This twist introduces a final, permanent lateral load that the designer may want to consider in analysis.

### 3.9.3 Curved Bridges

As with skewed bridges, deflections complicate girder behavior on curved bridges. Deflections are different at each cross frame location and, like skewed bridges, must be accommodated during erection.

On curved bridges with radial piers, deflection differences occur because the girders in a curved span are different lengths. If the piers are not radial but skewed, the skew causes still further deflection differences.

In general, girder flexibility accommodates the different deflections in a manner similar to the skewed bridges discussed above, but there are some complicating factors:

- Long spans—Long spans may have deflections that are so large that adjacent girders cannot readily be attached during erection. In such cases, the erector will reduce deflections by use of a holding crane or shoring tower.



- Continuity in multiple-span bridges—On continuous bridges, girder deflections are influenced by adjacent spans. Just as the presence of girders in one span reduces the deflections in the adjacent spans, when the girders in an adjacent span are not present, deflections are greater. To accommodate this, erectors may leave cross frame bolts loose in a given span until girders are present two spans down. Note, however, that as with skewed bridges bolts must be tightened before the deck is placed.
- Global stability—The erector must ensure that a given span is stable without the presence of the deck concrete. If not, shoring must be used. Shoring for global stability will have the added effect of reducing deflection differences and thereby facilitating assembly of the girder/cross frame superstructure framing system.

Generally, these erection phenomena are independent of the final condition bridge analysis because the erector will assemble the girders such that deflections and rotations from deck concrete will put the girders in the desired position. Certainly, an analysis may be chosen to determine deflections and check global stability during erection.

### 3.9.4 Final Deflections (Vibration, Dynamic Response)

The designer must not only be aware of strength, serviceability, and constructibility design criteria when evaluating and sizing steel girders but must also acknowledge the limitations imposed by the bridge owner related to live load girder deflections. In some circumstances, satisfying live load deflection limitations can govern the girder design.

As outlined in other sections of this document, controlling dead load deflections in an effort to minimize differential dead load deflections between adjacent girders is important as related to fit-up of cross-frames. Reasonable estimation of deflections is necessary in order to report the proper camber dimensions on the design drawings. Aside from these considerations, the final deflections resulting from live load are also important to consider. This section will discuss girder deflections that result from the occurrence of live loads.

The AASHTO *Standard Specifications for Highway Bridges* (14) include two different live load deflection limits. The first,  $L/800$ , is applicable to highway bridges that do not support pedestrian sidewalks. The other,  $L/1000$ , applies to highway bridges that carry pedestrian sidewalks. The parameter “L” in both expressions represents the span length of the girder. The more stringent criterion was established in an effort to diminish the sensation of a bridge deflecting or bouncing, as perceived by pedestrians. In addition to these two live load deflection limits, the standard specifications also prescribe minimum span-to-depth ratios for steel girder bridges. In most cases, by proportioning a girder to satisfy the minimum span-to-depth ratios, the live load deflection limits will also be satisfied. Nonetheless, these deflection limits need to be checked.

The latest version of the AASHTO *LRFD Bridge Design Specifications* (8) also includes suggested span-to-depth ratios and live load deflection limits. However, one of the key differences between the AASHTO *Standard Specifications for Highway Bridges* and the AASHTO *LRFD Bridge Design Specifications* is that the AASHTO *LRFD Bridge Design Specifications* identify the span-to-depth ratios and the live load deflection limitations as being optional criteria. Some jurisdictions require that both sets of these criteria be met in design, while others require only one or the other to be satisfied. For jurisdictions that take no position, the designer will need to decide if either or both of these optional criteria will be used.

The span to depth ratio criteria are intended to provide structures with sufficient depth to allow for strength and serviceability criteria to be easily met during final design. Many structures have been successfully designed with much shallower depths. The shallower the design, the more care the

designer should take in his or her analysis to make sure that secondary effects are adequately addressed; in some cases, this may suggest the need for a more refined analysis.

The live load deflection criteria are intended to provide sufficient stiffness in the structure to avoid harmful vibration response under dynamic loading (such as live load). If these criteria are adopted for a given design, the designers are cautioned to carefully read the code provisions and commentary regarding specific application of the criteria, particularly with regard to how the deflections should be calculated.

One prudent approach to this question may be as follows: use the traditional span-to-depth ratios for bridge type, size, and location studies to ensure adequate structure depth is provided by highway engineers as they set vertical geometry for bridges and set minimum vertical clearances over roadways, railroads, or waterways but do not be constrained by the span to depth ratio limits during final design. This approach affords the final design some flexibility in structure depth to accommodate any potential unanticipated loads.

Highway bridges are, in general, not subject to final deflection criteria in addition to those already mentioned. However, transit bridges, such as light rail transit bridges, can be subject to not only the live load deflection limits analogous to those prescribed for highway bridges but also additional live load deflection limits and live load vibration limits.

One such example of an additional live load deflection criterion that can be unique to transit bridges is related to a requirement stipulated by the Americans with Disabilities Act (ADA). For the case of an aerial structure carrying light rail vehicles (LRVs) adjacent to an aerial station platform, in addition to satisfying the “standard” live load deflection limits, the designer must also limit the differential live load deflection between the station platform’s finished walking surface and the LRV’s floor. At the time of this writing, the maximum permitted differential live load deflection is limited to 0.625 in. Several live load scenarios need to be evaluated to confirm this criterion is satisfied, including but not limited to, a fully-loaded LRV entering the station area beside an empty platform or an empty LRV entering the station area beside a fully loaded station platform. The structural stiffness of the LRV structure, the platform structure, and the LRV vehicle’s suspension system need to be tuned to meet this differential deflection criterion. Meeting this criterion is critical to the functionality of the station platform and to the safety of the transit system’s patrons. The transit agency typically would outline these types of deflection requirements in the system’s structural design criteria. The designer needs to be familiar with these types of guidelines and must consider that live load deflections could, potentially, govern the overall girder design.

Another design criterion that is somewhat unique to steel girder transit bridges is a requirement for the structure to be designed to have a certain minimum natural frequency. The natural frequency (i.e., the frequency of the structure in its fundamental (first) mode of vibration) is related to structure stiffness and mass. A bridge with a low natural frequency has a tendency to deflect and vibrate more due to the passage of live load than a bridge with a high natural frequency. A bridge with a higher natural frequency tends to have lower live load deflections (and less vibration), which indicates the bridge is relatively stiff. The purpose of such a design criteria is primarily related to user perception. Passengers riding on an LRV or an elevated subway car that is bouncing and listing as the vehicle crosses a bridge can become uncomfortable. In order to maintain a safe level of rider comfort, historically, minimum natural frequency values have been established for transit bridges. On a recent light rail transit project, the required minimum natural frequency was 2.5 Hz. As such, after the steel girders on an aerial structure were sized for strength, serviceability, and live load deflections, an additional analysis was required. The vibration analysis (an eigenvalue analysis) was then performed in order to confirm that the final response of the structure satisfied the required frequency limit. Several commercial software packages are available that are capable of performing eigenvalue analyses. One technique that can be used is to construct a plate- and eccentric-beam model and have

the computer program perform the eigenvalue analysis. The designer must be cognizant of mass inputs in the model. Often the structural models used for estimating design moments and shears may not include all of the mass-contributing components that exist on the bridge. Including the proper mass of the system in the eigenvalue analysis is critical to the results, especially as related to the natural frequency. The eigenvalue analysis will report the bridge's frequency in the first mode of vibration (and other modes) and, typically, will plot the corresponding deflected shape of the structure (the mode shape). It is worth noting that higher level detailed dynamic response models used to evaluate vibration characteristics of bridges would need to include not only the structure's mass and stiffness but also characteristics of the vehicles using the bridge such as their speed of travel and their suspension system properties. The designer must determine whether a complex and detailed analysis is warranted.

Though the discussion provided in this section is simplified, it can be seen that the final deflections of steel girder bridges can play a significant role in the final outcome of a girder design. The designer, depending on the bridge type (a highway bridge versus a transit bridge, for example), must be aware of the variety of potential final deflection limitations that may apply.

### 3.10 Detailing of Cross-Frames and Girders for the Intended Erected Position

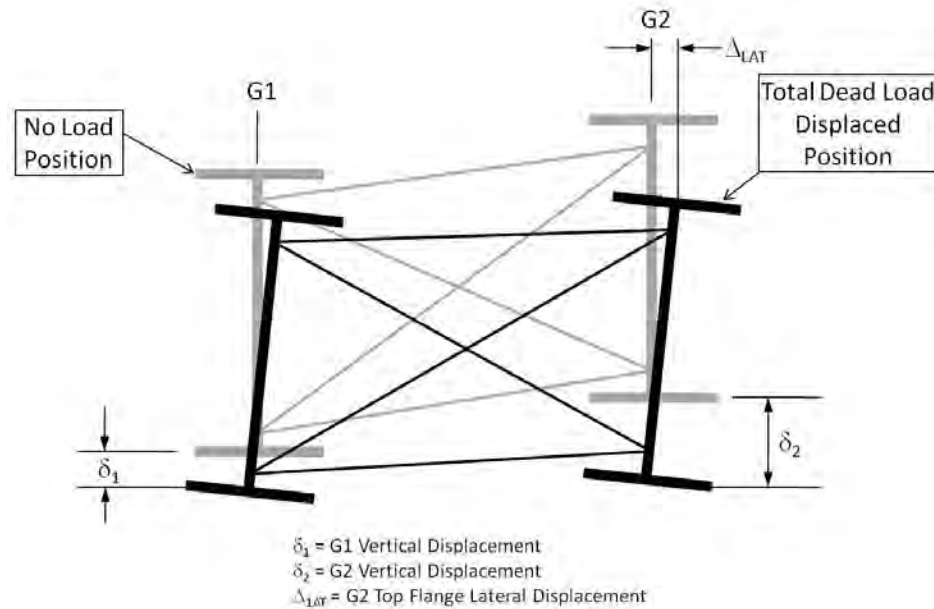
Per Article 6.7.2 of the *AASHTO LRFD Bridge Design Specifications* (8), for straight I-girder bridges with skewed supports and for curved I-girder bridges with or without skewed supports, the contract documents should clearly state an intended erected position (web plumb or web out-of-plumb) of the girders and the loading condition (no load, steel dead load, total dead load) under which that position should be theoretically achieved. Therefore, the bridge designer needs to give careful consideration to the detailing that he or she specifies for these bridge types and to the effects the specified method can have on the behavior of the structure in its final state as well as during steel erection.

The detailing of diaphragms and I-girders in straight bridges with skewed supports and horizontally curved steel girder bridges is more complicated than the typical detailing associated with straight girder bridges without skew. As such, the bridge designer should give consideration to the detailing of these bridge types. Reference should be made to Article 6.7.2 of the *AASHTO LRFD Bridge Design Specifications* (8) and the following discussion, which elaborates on this particular issue.

For straight I-girder bridges with skewed supports, and horizontally curved steel I-girder bridges with or without skewed supports, there are two theoretical positions in which the girders can be at different points during erection:

- Girder webs vertical, otherwise known as web plumb, or
- Girder webs not vertical, otherwise known as web out-of-plumb.

As discussed in previous sections, a curved girder bridge not only displaces vertically, but rotates out of plane due to the torsional effects of the eccentrically applied gravity loads (dead loads) acting on the curved geometry. This out-of-plane rotation, as shown in Figure 3.10-1, will cause lateral deflection of the top and bottom flanges in the radial direction. Straight I-girder bridges with skewed supports and non-skewed cross-frames exhibit a similar behavior, in which the girder webs will rotate out of plane. In situations where non-skewed intermediate cross-frames are used in skewed bridges, the cross-frames connect adjacent girders at different points along the span of each girder. As a result, the cross-frames connect adjacent girders at points of different vertical displacement. The girders try to force a racking distortion of the cross-frames, but the in-plane racking stiffness of the cross-frames is quite large, so instead the cross-frames rotate and force the girders to rotate out of plane. In a curved I-girder without skewed supports, the out-of-plane rotation will occur within the spans and with no out-of-plane rotation at the supports. For bridges with skewed supports, the out-of-plane rotation will typically occur within the span and at the supports.



**Figure 3.10-1. Out-of-plane rotation experienced by a curved girder bridge including superelevation effects (skewed bridge exhibits similar behavior).**

In either of these particular cases, the girder webs in these bridge types can only be theoretically plumb under only one loading condition. These loading conditions are typically referred to as:

- No load,
- Steel dead load, and
- Total (or full) dead load condition.

The no load condition refers to the condition in which the girders and cross-frames are erected under a theoretically zero stress condition, where there are no deformations. In other words, it is the geometric configuration that the structure assumes once assembled, but not acted upon by any external forces, including gravity. This no load condition is practically approached during steel erection with the proper use of temporary supports and/or hold cranes. In the field, an absolute no load condition cannot be achieved unless the structure is fully supported. However, a condition that will closely resemble the no load condition can be achieved with very few properly used temporary supports. Furthermore, in a fabrication shop the girders can be blocked to their desired camber to simulate the no load condition during the shop assembly process or assembly can be accomplished with the girders lying down (hence the term *laydown*), which is also a no load condition.

The steel dead load condition is the theoretical state of the assembled steel superstructure under the action of gravitational forces caused by the self-weight of the steel members. This condition occurs after all of the steel members have been erected and all temporary supports have been removed. In a curved I-girder bridge or a bridge with skewed supports and non-skewed cross-frames, the girders will displace vertically and rotate out of plane due to steel dead load condition, as shown in Figure 3.10-1.

The total dead load condition refers to the condition after the full non-composite dead load, including the concrete deck, is applied to the steel superstructure. Therefore, the total dead load condition includes the weight of the steel members and the weight of the concrete deck. In a curved I-girder

bridge or a bridge with skewed supports and non-skewed cross-frames, the girders will displace vertically and rotate out of plane due to total dead load condition.

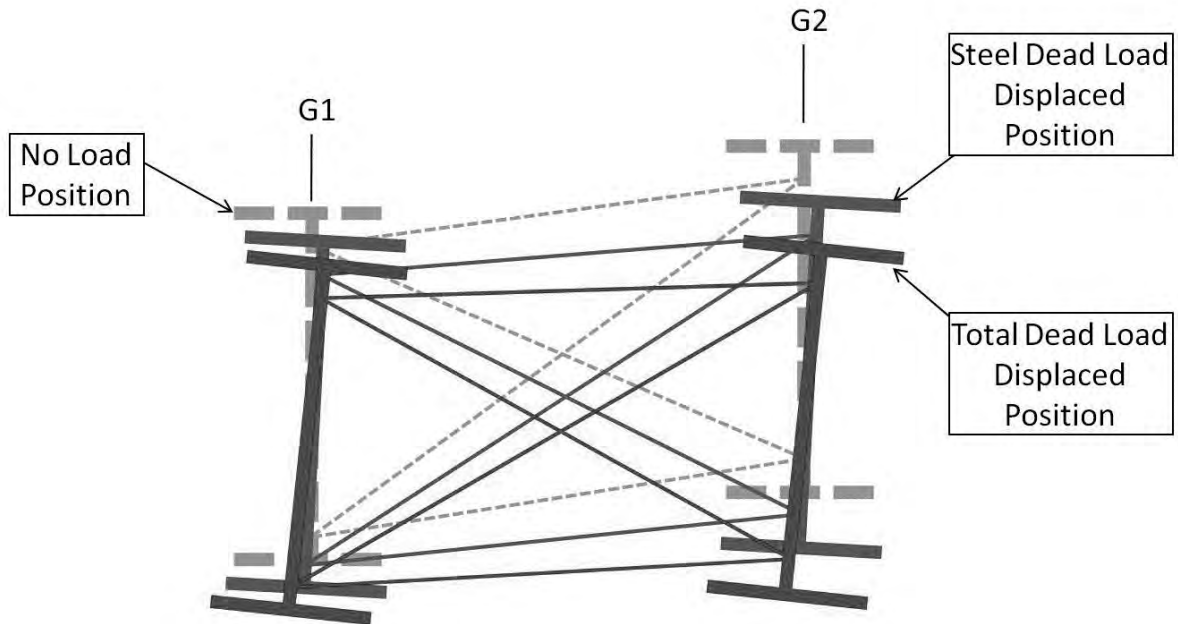
### 3.10.1 Detailing Methods

In straight I-girder bridges with skewed supports and curved I-girder bridges with or without skew, the cross-frames may be detailed to fit-up with the girders in a theoretically web-plumb position at the no load, steel dead load, or total dead load condition. These are commonly referred to as no load fit, steel dead load fit, and total dead load fit detailing. Each of these is defined as follows:

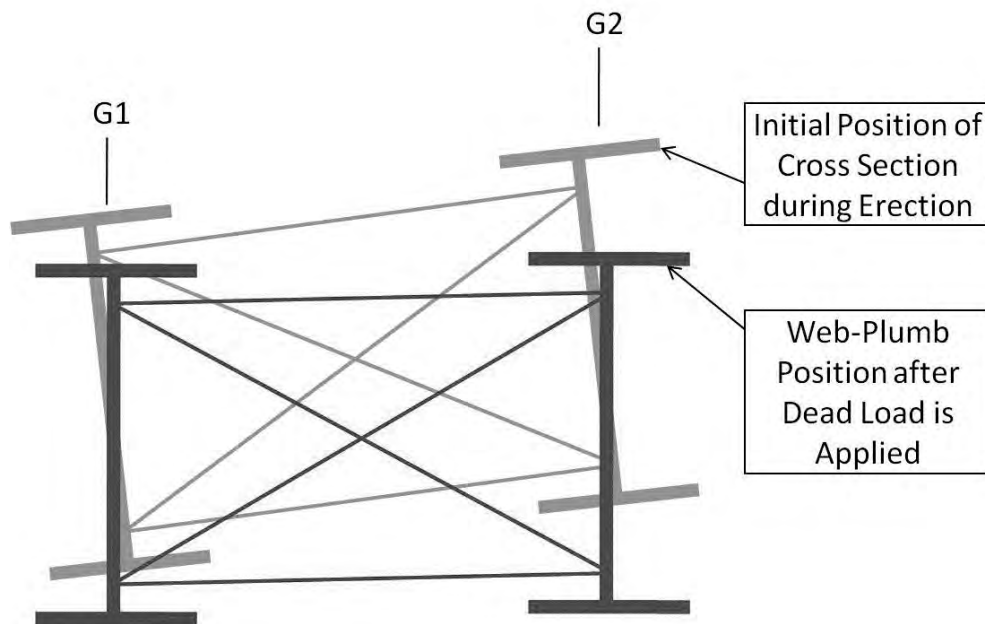
- *No Load Fit (NLF)*—diaphragm members are detailed to fit girders with the webs vertically plumb as though no dead load deflections have taken place, including those due to steel self-weight.
- *Steel Dead Load Fit (SDLF)*—diaphragm members are detailed to force the girder webs to be theoretically plumb once the girders have displaced due to steel dead load.
- *Total Dead Load Fit (TDLF)*—diaphragm members are detailed to force the girder webs to be theoretically plumb once the girders have displaced due to total non-composite dead load (steel and concrete).

If the NLF method is employed, the girders will rotate out of plane upon loading and the girders will be webout-of-plumb under steel dead load and total non-composite dead load, as shown in Figure 3.10.1-1 (superelevation effects are not included). NLF detailing can also be referred to as a consistent detailing method, as the girder and cross-frames would be detailed for the web-plumb position at the same loading condition. In this case, girders and cross-frames are both detailed to be in the web-plumb position at the no load condition.

The SDLF and TDLF methods of detailing are similar, with the only difference being the particular load condition, so they will be described together within this discussion. In each case, diaphragm members are detailed with the intent of forcing the girders to be web-plumb at the given loading condition, as shown in Figure 3.10.1-2. SDLF and TDLF also require that the girders be out-of-plumb during steel erection but out-of-plumb in the opposite direction of what they will rotate due to dead load. Therefore, once dead load is applied, the girders will theoretically rotate to a web-plumb position.



**Figure 3.10.1-1.** Displaced position (magnified) of a curved I-girder bridge or a skewed I-girder bridge employing NLF detailing.

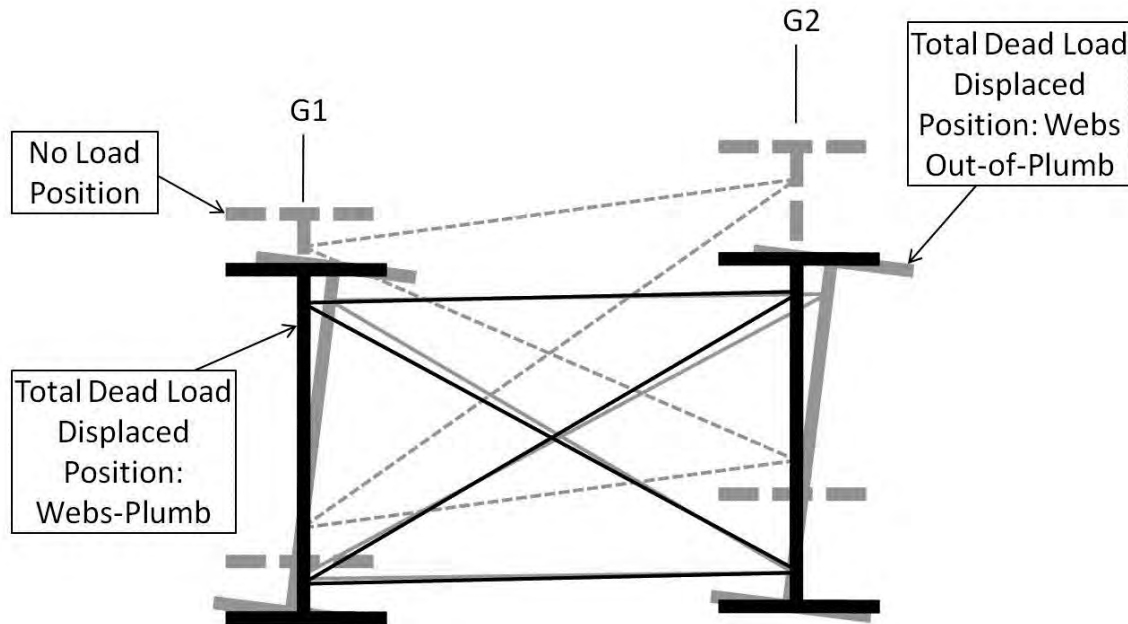


**Figure 3.10.1-2.** Intent of SDLF and TDLF detailing is to have the girder web-plumb at steel dead load or total dead load, respectively.

SDLF and TDLF detailing utilize diaphragm members that are detailed for the cross section to be web-plumb at the given dead load condition and girders that are detailed to be web-plumb at the no load condition. Therefore, because the components are detailed for the same web-plumb position but

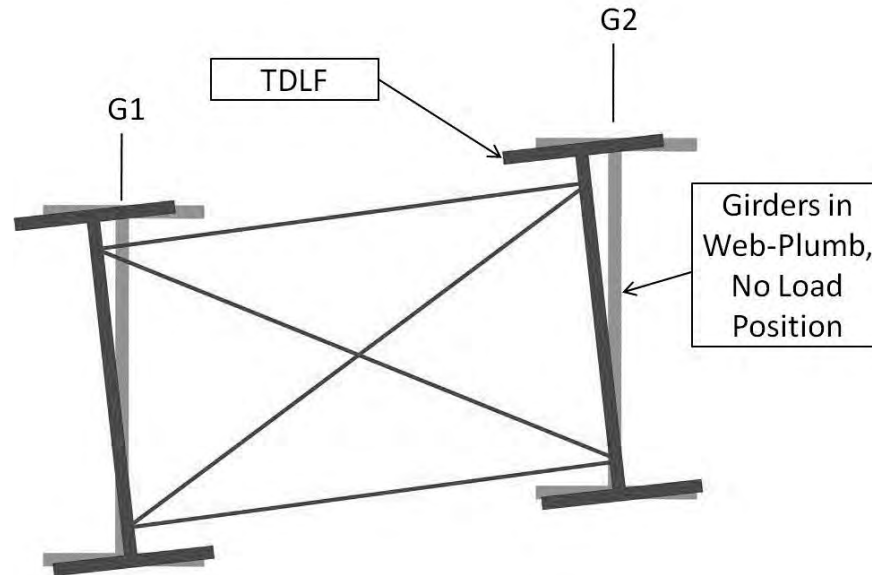
under different loading conditions, the components may not necessarily fit together at the no load position during erection. This will be further explained in the following discussion and figures, assuming TDLF detailing in a curved I-girder bridge.

To determine the diaphragm member lengths for TDLF, the first step is to determine the displacements of the cross section due to total dead load, as shown previously in Figure 3.10.1-1. The second step is to assume that the girders are web-plumb under total dead load, schematically removing any out-of-plane rotation of the cross section, as shown in Figure 3.10.1-3. From the girders in this position, diaphragm member lengths can be determined, using the same work point locations on the girder webs as would be used for NLF detailing. Similarly, the detailer and/or fabricator will use the vertical dead load displacements given in the camber tables on the contract plans to determine the diaphragm member lengths. In understanding this, it is key to remember that the diaphragms are very stiff (i.e., they have a very high in-plane shear stiffness) and very resistant to racking deformations, while the girders are relatively flexible and can more easily twist to accommodate the given shape of the diaphragms. Also keep in mind that whenever the girders are forced to twist in order to accommodate the given shape of the diaphragms, load is induced in the system, both in the form of diaphragm member loads and flange lateral bending moments in the girders.



**Figure 3.10.1-3.** TDLF detailing, determining the diaphragm member lengths assuming that the girders are web-plumb at total dead load.

The next step in TDLF detailing is to consider the position of the girders during steel erection. It is assumed that the girders will be close to the no load position during steel erection, which can be accomplished with the use of temporary supports or hold cranes, or both. As shown in Figure 3.10.1-4 with TDLF detailing, the girders will need to be forced out of plumb during steel erection in the opposite direction of the total dead load out-of-plane rotation. This is due to the fact that the girders are detailed for web-plumb at no load, while the diaphragms are detailed for web-plumb at total dead load.

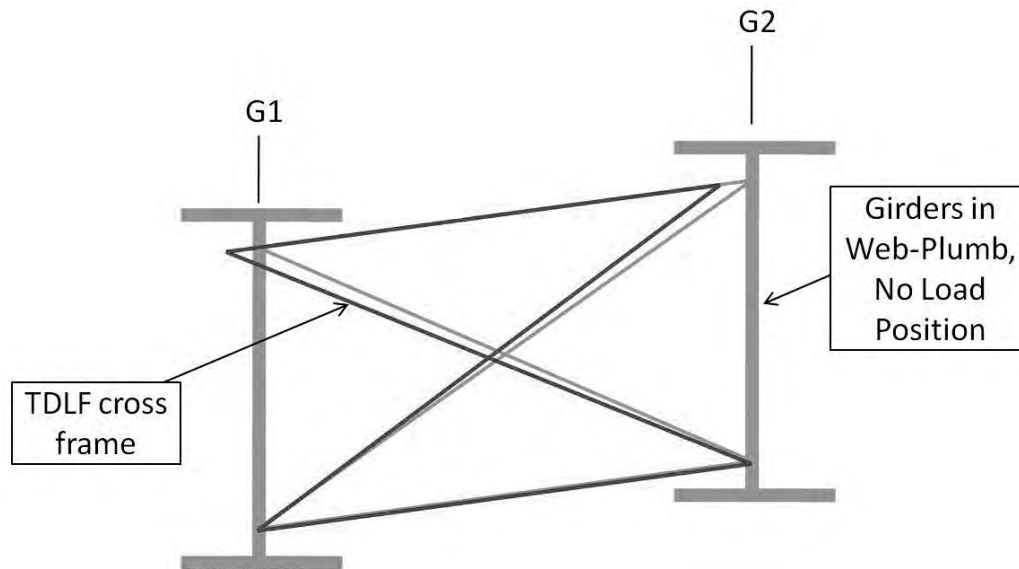


**Figure 3.10.1-4. TDLF during erection, assuming girders are held close to no load position with temporary supports or holding cranes, or both.**

It can be seen from Figure 3.10.1-4 that the diaphragms for TDLF do not necessarily fit with the girders detailed for NLF. This is more clearly shown in Figure 3.10.1-5. For this X-type diaphragm, TDLF detailing results in diagonal members that are slightly too long or too short to fit the same work points on the girders detailed to be web plumb at no load. There is little or no change in the top and bottom chord length dimensions. The difference in diagonal member lengths is due to the fact that there is an inconsistency in the assumed load condition of the girders and diaphragms for the given detailing position—i.e., diaphragms detailed for web-plumb at total dead load, and girders detailed for web-plumb at no load.

Hence, this type of detailing is sometimes referred to as inconsistent detailing, since the cross frame members are detailed for web-plumb at total (or steel) dead load condition, while the girders are detailed for the web-plumb at the no load condition (35). In curved I-girder bridges and straight I-girder bridges with skewed supports, the girders can only be web-plumb at single load condition. Since the girders are often erected at a web-plumb position at the no load condition, the diaphragm members that are detailed for the web-plumb position at the total (or steel) dead load condition will need to be forced to fit in between the girders. The amount of force required to fit up the cross-frames and girders is dependent on the amount of out-of-plane rotation that is intended to be reduced by SDLF and TDLF detailing.





**Figure 3.10.1-5. TDLF detailing, where cross-frames are detailed for web-plumb at total dead load and girders detailed for web-plumb at no load.**

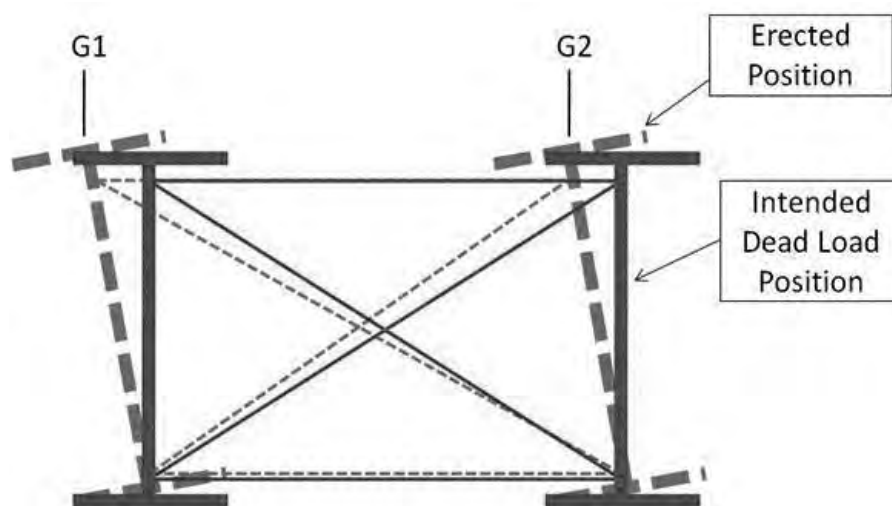
Alternatively, for girder webs to be theoretically plumb at steel dead load or total dead load, the girder webs can be fabricated for the no load position with a twist about the tangential axis of the girder for the particular load condition. However, this particular practice of detailing is more costly and has been used on an extremely limited basis, if at all, and is typically not recommended. Theoretically, this method of detailing would not induce any locked-in stresses and achieve girder webs plumb at the given load condition. For this case, the girder flanges would be welded square with respect to the webs and the cross-frames would be detailed for the desired load condition to correspond with the twist in the web. This combination of detailing can also be referred to as a consistent detailing method, as the girder and cross-frames would be detailed for the web-plumb position at the same loading condition.

It is currently industry practice that the geometry of 2D and 3D bridge analysis models is based on NLF. However, when SDLF or TDLF is used for detailing of the structure, the engineer should be aware that the supplemental stresses and deflections generated (lack-of-fit effects) will have an effect on the structure that may warrant further consideration by the engineer. Assessment of the magnitude and effect of these supplemental stresses and deflections currently requires engineering judgment and needs to be evaluated on a case-by-case basis. In any event, the engineer should recognize that the typical analysis model based on the assumption of NLF is not necessarily an equivalent representation of a bridge that is detailed and constructed using SDLF or TDLF.

### 3.10.2 Straight, Skewed I-Girder Bridges

In a straight I-girder bridge with skewed supports, with the cross-frames detailed for NLF, the girder webs will be out-of-plumb after the steel erection and rotate further out of plumb after the concrete deck is placed. At the support locations, this out-of-plane rotation may exceed the allowable rotation of the bearing. If the allowable rotation is exceeded, an alternate detailing method such as SDLF or TDLF may need to be considered by the bridge designer. For the SDLF or TDLF to be employed, the cross-frames are detailed so that it will be necessary to twist the girder during erection to an out-of-

plumb position that is opposite of the out-of-plane rotation due to dead load. Therefore, upon application of dead load (steel or total), the girders will rotate to a web-plumb position at the bearings, as shown in Figure 3.10.2-1.



**Figure 3.10.2-1. SDF and TDLF detailing for a skewed I-girder bridge, girder positions at the bearings.**

Beckman et al. (23, 24) recommend that for straight, skewed bridges, the girders should be vertically plumb, within a reasonable tolerance, when the construction is complete—i.e., after the deck has been placed. In this case, the girders will be forced out of plumb in the opposite direction to which they will rotate upon load application through the use of TDLF detailing. As load is applied, the girder will rotate or untwist to a theoretically vertically web-plumb position. The detailing and fabricating of the cross-frames such that the girders will be plumb under the application of total dead load facilitates the alignment of the adjacent deck segments at expansions joints (23, 24).

### 3.10.3 Curved I-Girder Bridges

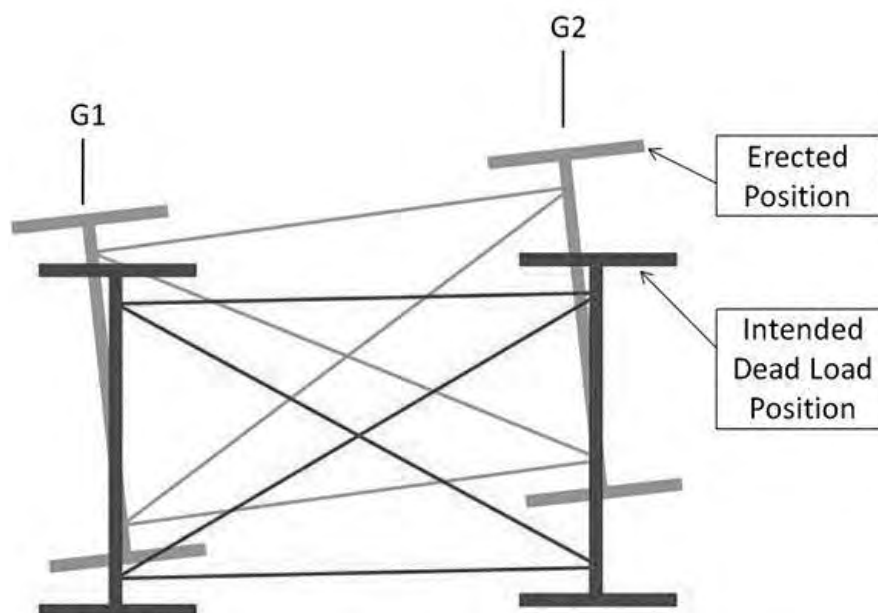
The implementation of SDF or TDLF in straight, skewed I-girder bridges can usually be accomplished without inducing significant locked-in stresses in the girder flanges or cross-frames, caused by the force fit of the cross-frames. The amount of locked-in stress in the system is dependent on the amount of force required to fit the cross-frames.

In a horizontally curved I-girder bridge in which NLF detailing is employed, the girder webs will be out-of-plumb at steel dead load and at total dead load. However, at non-skewed supports (radial), the girder out-of-plane rotation is typically close to zero and the girder webs will be near plumb. Within the span of the curved bridge, the girder webs will be out-of-plumb. The amount of web out-of-plumbness is dependent on the geometry of the bridge. For a curved I-girder bridge with skewed supports, there will be out-of-plane rotation similar to that discussed previously for straight, skewed I-girder bridges.

Since a curved I-girder bridge that employs NLF detailing will be out-of-plumb at steel and total dead load, this out-of-plumbness may need to be considered in the detailing of the concrete deck or the bearings as needed. In some cases, the additional flange stresses caused by the rotated cross section may need to be considered by the bridge designer as well. Through a series of analytical investigations, Howell and Earls (59) and Chavel (32) show that there is an increase in flange tip stresses caused by the girders' being out-of-plumb when NLF detailing is employed. Domalik et al.

(42) provide a simplified method for the consideration of the additional flange stresses caused by the out-of-plumb girders.

In some cases, the amount of out-of-plane rotation associated with NLF detailing may be considered to be too large by the bridge designer or owner, in which case either party may desire to have the webs of the girders plumb at either the steel or total dead load condition. It should be noted that various owner-agencies have different requirements with regard to the amount of web out-of-plumbness that is acceptable. To have girder webs theoretically plumb under either steel or total dead load in a curved I-girder bridge, the cross-frames must again be detailed for the particular load condition and force fit into girders detailed to be web-plumb at the no load condition. In other words, SDLF or TDLF detailing will need to be employed to theoretically achieve web plumbness at the particular load, as shown in Figure 3.10.3-1, assuming girder G2 has a larger radius than girder G1.



**Figure 3.10.3-1. SDLF and TDLF detailing for a curved I-girder bridge within a span.**

For SDLF and TDLF in a curved I-girder bridge, as the cross-frames are forced into place and the girders are twisted during erection, the curved girder flanges act to resist the twist and the induced change in their radii. Therefore, the implementation of SDLF or TDLF can result in potentially problematic locked-in stresses in the girder flanges and cross-frames that may need to be considered by the bridge designer. The decision as to when these locked-in stresses need to be considered is currently a matter of engineering judgment that needs to be evaluated on a case-by-case basis.

In some cases, such as in larger structures where the out-of-plane rotation and the differential displacements are significant, the use of SDLF and TDLF detailing can lead to problems during steel erection (34, 35). However, in cases where the out-of-plane rotation and differential displacements are small, SDLF and TDLF detailing may be employed with few problems during steel erection. Again, the amount and significance of fit-up problems in the field needs to be considered on a case-by-case basis.

Furthermore, it has been shown that SDLF or TDLF detailing will indeed result in an increase in flange tip stresses at diaphragm locations but may also affect the vertical and lateral displacements of the girders. In the structure that was studied, it was observed that SDLF detailing caused the interior girder to be at a higher elevation than predicted for the structure detailed for NLF, thus possibly affecting the haunch thickness. Additionally, a study conducted by Ozgur et al. (72) shows that there

is a slight decrease in the structural capacity of the girders when TDLF is employed in the given bridge, but in many typical cases this reduction caused by second-order effects will be small. It is important to choose the correct analysis method to develop the most effective design. It is also important to clearly state how the design was intended to be fabricated and erected. There is a method of fabrication and erection that most closely matches the assumptions made while developing the design.

#### **3.10.4 Tub Girders**

In discussing detailing of tub girders for no load fit, total dead load fit, and steel dead load fit, it should first be pointed out that tub girders are generally much stiffer than I-girders. It is much more difficult to adjust the position of a tub girder in the field than it is to adjust the position of an I-girder. For this reason, great care should be taken in making decisions regarding fit-up detailing of tub girders. Consideration should be given to the likely erection scheme, including the possibility of using temporary shoring towers, hold cranes, or other means of temporary support. Since tub girders are stiff and difficult to move or adjust in the field, it may be advisable to detail tub girders for no load fit or steel dead load fit (with consideration given to possible temporary shoring or hold cranes; if sufficient shoring or temporary support is provided, detailing for no load fit may be more appropriate). This should help facilitate erection of the girders. Later, when the deck is placed, the girders will deflect and rotate and may end up out-of-plumb, but, since tub girders are so stiff and their deflections and rotations are typically much less pronounced than I-girders, designers can usually take comfort that the deflections and rotations that occur during deck placement will probably be much less severe than what is typically seen in I-girder bridges and that any out-of-plumbness will probably be minor.

#### **3.10.5 Calculation of Locked-In Force Effects Due to Cross Frame Detailing**

White et al. provide an extensive discussion of the calculation of locked-in force effects due to cross frame detailing in Sections 3.2.5 and 3.3.4 of NCHRP Report 725 (88). Additional research has been proposed to provide further quantitative guidance on this topic. In the interim, readers are referred to White et al. (88) for guidance on when locked-in forces, caused by various detailing methods, should be considered in the design of a bridge. In addition, the AASHTO/NSBA Steel Bridge Collaboration is developing a qualitative document to provide interim guidance on this topic.

As a general rule, the locked-in forces in the cross-frames tend to be opposite in sign to the cross frame dead load forces in straight-skewed bridges. That is, the cross frame locked-in forces tend to offset or cancel-out the cross frame dead load forces for these bridge geometries. However, for curved radially-supported bridges, the opposite is true; in these structures, the locked-in forces due to the cross frame detailing are typically additive with the cross frame member forces coming from the dead load. Generally, in understanding locked-in forces due to cross frame detailing, it is very important to realize that the so-called “locked-in” forces are the forces solely due to the effect of the cross-frames not fitting with the girders in the fabricated no-load geometry of the steel. These forces are influenced by the lack-of-fit between the cross-frames and the girders due to the cross frame detailing, which is a function of the girder vertical camber diagrams. However, the locked-in forces are otherwise independent of the bridge dead load. The “dead-load forces” in the cross-frames are the forces one can determine in the cross-frames by building a structural analysis model and simply “turning gravity on.” These forces do not include any locked-in force effects. The additive (or subtractive) combination of (1) the internal locked-in forces and (2) the dead load forces produces the total cross frame forces for a given stage of the construction.

White et al. (88) made significant contributions to the better understanding of locked-in force effects, and have demonstrated that these force effects can be significant. However, comprehensive design guidelines on how to consider locked-in force effects due to cross frame detailing in a production

design setting are not yet currently available. Traditionally, this issue has only been addressed in rare and infrequent situations, and typically only in an indirect fashion by construction engineers from the perspective of quantifying the magnitude of force needed to assemble the bridge, rather than evaluating the permanent effects on the structure. It is hoped that proposed research will soon provide better guidance on this issue. In the interim, designers are encouraged to: a) understand the nature of locked-in force effects; b) take prudent steps to avoid situations where these effects would be magnified (for example, avoiding specifying total dead load fit detailing for curved girder bridges), and; c) when faced with situations where it appears that locked-in force effects may be significant, consider quantifying the force effects. Discussion with senior bridge design engineers, owners, construction engineers, and bridge erectors is strongly encouraged.

### 3.10.6 Estimation of Fit-Up Forces

Generally, the bridge erector will have to apply certain magnitudes of force to various elements in order to align and connect the components of the steel structure during erection. These are called “fit-up forces.” The fit-up forces are by definition specifically the temporary forces the erector needs to apply via cranes, come-alongs, jacks, cable bracing, etc. during the steel erection. These forces can be influenced significantly by the geometric attributes of the structure, the framing of the steel (particularly the arrangement of the cross-frames between the girders in an I-girder bridge), and the type of detailing of the cross-frames. Basically, if say a steel girder is brought in and needs to be connected by cross-frames to a portion of the bridge that has already been erected, both the steel girder and the previously erected portion of the bridge deflect under their self-weight plus any temporary support conditions at the specific erection stage. These deflections are different than the final deflections of the steel once the structural system is completed, and the steel geometry at a given erection stage is generally different than the no-load geometry of the various components. The incompatibilities between the different pieces at the time of the steel erection are influenced by these deflections, as well as by any initial lack-of-fit between the components in their fabricated no-load geometry (due to the type of cross frame detailing). The fit-up forces are related to, but they are not the same as, the “locked-in” internal forces in the steel, which are discussed in the previous section. In addition, the actual fit-up forces encountered by the steel erector can depend on numerous intricacies of the steps taken in erecting the steel, the temperatures and temperature gradients of the different steel elements at the time of erection, minor connection tolerances and how any accumulated effects of these tolerances are controlled by the erector, etc. Many of these aspects are impossible for the design engineer to predict. However, the design engineer can make reasonable estimates of the general magnitude of the fit-up forces, and where they may be of some potential concern, based on specific structural analysis models that are relatively easy to create.

Although the precise fit-up forces that will be experienced by the erector cannot be predicted, the general magnitude of the fit-up forces can be estimated. This is because the fit-up forces are closely related to the internal forces in the structure under the steel dead load once any portion of the bridge has been erected. In fact, as explained by White et al. (88) in Section 3.3.5 of their report, the most practical way to estimate fit-up forces at a given steel erection stage is to create a structural analysis model of the steel framing at the completion of that stage, and then analyze this portion of the structure for the combined internal steel dead load forces plus any significant locked-in forces from the cross frame detailing at that stage. The specific support conditions at the end of the targeted erection stage must be included in the model. Given these analysis results, the internal forces in any given component at the end of the targeted erection stage (under the combination of any significant lack-of-fit between the components in their initial fabricated no-load geometry, the steel dead load, and the specific support conditions at that stage) can be used as an indicator of the fit-up forces the erector will need to apply to install that specific component just prior to that stage. (This is because the force in a targeted component is zero before it is installed, and it is effectively equal to the force determined from the above analysis once the component is installed.) In addition, the forces in the

different members of a given cross-frame can be combined from the above structural analysis to provide an indicator of the magnitude of the forces that would be required to install the cross-frame.

Various research studies are still under way at the time of the writing of the 2<sup>nd</sup> Edition of G13.1, and additional substantive research has been proposed to provide further quantitative guidance on the estimation of the ease or difficulty of fit-up as a function of the bridge geometry, the bridge framing (specifically the arrangement of the cross-frames) and the type of cross frame detailing. Until these research efforts are completed, the recommendations by White et al. (88) provide some guidance on this topic.

Discussion of the issue of estimation of fit-up forces goes hand-in-hand with the discussion of locked-in force effects (Section 3.10.5 of this document). Traditionally, the estimation of fit-up forces has been the responsibility of the construction engineer (contractor's engineer), and the estimates have been approximate. There have been cases in which the magnitude of the forces required to assemble a bridge have proven to be impractical to achieve, resulting in delays, claims, changes to the erection sequence and/or temporary shoring provisions, or other undesirable consequences. Designers should be cognizant of the issues related to fit-up forces so that they do not inadvertently develop designs or specify requirements that could result in bridges that may be difficult to assemble, or designs that might experience significant locked-in force effects. It is hoped that proposed research will soon provide better guidance on this issue. In the interim, designers are encouraged to: a) understand the nature of fit-up force requirements; b) take prudent steps to avoid situations where fit-up forces may be excessive (for example, avoiding specifying total dead load fit detailing for curved girder bridges); and c) when faced with situations where it appears that fit-up forces may be significant, consider quantifying the force effects. Discussion with senior bridge design engineers, owners, construction engineers, and bridge erectors is strongly encouraged.

### 3.11 Cross Frame Modeling (2D versus 3D)

Cross-frames and diaphragms perform several functions in steel girder bridges. In even the simplest tangent, non-skewed bridges, the cross-frames or diaphragms serve to brace the girder compression flanges and they also participate to some degree in live load distribution. In addition, although traditionally seldom accounted for, cross-frames and diaphragms participate in distribution of non-composite dead loads such as the dead load of the wet concrete slab. In more complicated structures, such as curved girder bridges and cross-frames, diaphragms are considered primary load-carrying members because they constitute an essential part of the overall structural system. Similarly, in skewed bridges, the cross-frames can carry significant loads, as they resist differential deflection of adjacent girders and form secondary load paths (24, 39, 40, 47, and 65).

In a typical line girder analysis, the effects of cross-frames and diaphragms as part of the structural system usually are neglected. Dead loads are typically assumed to be distributed to individual girders based on tributary load width assumptions, while live loads are typically assumed to be distributed to individual girders based on empirical live load distribution factors. When these assumptions are used prudently and with care, they generally lead to safe, conservative designs from a strength standpoint. However, anecdotal accounts of problems with poor prediction of interior versus exterior girder dead load deflections and recent research have suggested that these simplifying assumptions may be inadequate for prediction of dead load deflections. A number of problems with poor prediction of dead load deflections have been reported, including misalignment of cross frame connections and difficulty achieving correct deck slab thicknesses during deck screeding (49, 63, 70).

Recent experimental and analytical studies by Fisher (49) have demonstrated that there is significant secondary stiffness provided by cross-frames and diaphragms, as well as by stay-in-place (SIP) metal deck forms, in both skewed and non-skewed tangent girder bridges. This additional stiffness typically is not accounted for in line girder analyses. Fisher reported studies of seven simple-span and three

continuous-span steel girder bridges with skew offsets ranging from 0° to 62°. These studies focused primarily on dead load deflections due to slab placement and compared field measurements of these deflections to predictions based on traditional line girder analysis, line girder analysis with modifications proposed by Fisher, and 3D FEM analysis. For the simple-span structures, the traditional line girder analysis deflection predictions differed significantly. The researchers' more rigorous 3D FEM analysis models exhibited much better correlation with the field measurements due to the consideration of the stiffness of the cross-frame and the stay-in-place metal deck forms. The researchers concluded that the influence of cross frame stiffness was significant in both skewed and non-skewed bridges in relation to accurate prediction of the relative non-composite dead load deflections of individual girders, as well as being significant in skewed bridges in relation to the accurate prediction of total system non-composite dead load deflection. In other words, omitting consideration of cross frame stiffness as is typically done in line girder analyses may lead to erroneous deflection predictions, which can lead to problems with screed mispositioning, thus leading to under- or over-thickness decks.

One level above line girder analysis are the 2D or grid analysis techniques. In a grid analysis, cross-frames and diaphragms are included in the analysis model but with significant simplifying assumptions involved. In a grid analysis, cross-frames and diaphragms are modeled using a single line element, regardless of the structural configuration of the actual cross-frame or diaphragm, which may be a plate diaphragm, an X-, K-, or inverted K-type truss cross-frame, or another configuration. Several approaches are commonly taken to modeling the stiffness of a cross-frame or diaphragm in a grid analysis. The nature of the approach used can affect the results because different approaches require different simplifying assumptions, which include the use of different stiffness parameters.

Regardless of the type of modeling being performed (2D, 3D, others) most designers will omit refined consideration of the flexibility of connection details such as bolted gusset plate connections. Instead, for truss-type cross-frames, most designers assume that the chord and diagonals act as pin-ended truss members for analysis modeling as well as for design checks.

### **3.11.1 Impact of Connection Stiffness on Cross Frame Stiffness**

The stiffness of the end connections of cross frame members to connection plates and girders can potentially influence the overall stiffness of the cross frame members. There are a number of associated issues that may be worth considering in the analysis of steel girder bridges.

Battistini et al. (22) performed a range of studies of different X, K, and Z-type cross-frames composed of single-angle members. Their studies indicate physical cross frame stiffnesses ranging from 0.55 to 0.75 of the calculated stiffness based the modeling of the cross-frames using truss elements. These reduced stiffnesses are due to the bending eccentricities at the connections of the single-angle cross frame members.

The cross-frame's function is to provide stability bracing to girder compression flanges; therefore, overestimation of the stiffness of the cross frame members would be unconservative. In such cases, a conservative lower bound estimate of the cross frame stiffness may be appropriate.

However, from the perspective of the analysis of an indeterminate structural systems for the purpose of calculating member forces and displacements (i.e., for a 2D or 3D analysis of a bridge to determine force effects and displacements in cross-frames, girders, etc.), under- or over-estimation of the stiffness can produce either unconservative or conservative force results; however, generally there is no conservative or unconservative prediction of deflections when the aim is to estimate the deflected geometry of the structure.

The research by Battistini et al. (22) is an initial step in providing quantitative guidance in this matter, but providing comprehensive guidance will likely require further research. Readers of this guideline

document are encouraged to watch for further developments regarding this matter in the near future, and to watch for associated revisions in the AASHTO LRFD bridge design provisions. In the interim, in lieu of more refined analysis guidance, designers may want to consider using an approximate adjustment to cross-frame member stiffnesses. Battistini, et al. (22) recommended modeling the axial stiffness of single-angle members in truss-type cross-frames as  $0.65 EA/L$ , where  $A$  is the gross cross-sectional area of the single-angle members and  $L$  is the length of the members between the work points in the truss model. (where  $A$  is the gross cross-section area of the angles, and  $L$  is the length of the angles between the work points of the truss model). The behavior of flange-connected T-section members is similar, and thus  $0.65EA/L$  also is recommended for the truss modeling of these types of members. These approximations are also recommended (and should generally be expected to be conservative) for evaluating the adequacy of cross-frames functioning to provide stability bracing to girder compression flanges.

### 3.11.2 Simplified Euler-Bernoulli Approximations of the Stiffness of Truss-Type Cross-frames

Many designers and current commercial 2D analysis software packages use an Euler-Bernoulli beam approach to achieve a simplified approximation of the stiffness of truss-type cross-frames.

For example, some designers determine the equivalent stiffness of a cross frame modeled as a line element by calculating only the flexural stiffness. In this approach, the truss-type cross-frame is modeled separately and a unit force couple is applied to one end. Deflections in the direction of loading are calculated and used to determine an equivalent end rotation. The equivalent end rotation and unit force couple then are analyzed using an equivalent propped cantilever to back-calculate the associated equivalent moment of inertia of the cross-frame, which is then used as the primary stiffness property of the line element used in the grid analysis to model the cross frame stiffness (Figure 3.11.2-1) (69).

Meanwhile, other designers determine the equivalent stiffness of a cross-frame modeled as a line element based on shear stiffness. In this approach, the truss-type diaphragm is modeled separately and a unit vertical force is applied to one end. Vertical deflections are calculated and are used as an equivalent shear deformation to back-calculate the associated shear stiffness of the cross-frame, which is then used as the primary stiffness property of the line element used in the grid analysis to model the cross frame stiffness (Figure 3.11.2-2).

Note also that there are further variants on these two methods. The methods described above use pinned boundary conditions on the left end of the cross-frame, with translation restrained in the nonloaded direction. A variant on this is to assume that the nodes on the left end of the cross-frame are free to translate both vertically and horizontally. This now results in at least four different methods to determine the equivalent stiffness of a truss-type cross-frame, each of which typically produces very different values for the equivalent moment of inertia.

None of these approaches is wrong in and of itself, but each approach focuses only on one of several stiffness parameters while others are neglected. In an actual bridge, there is the potential that both stiffness parameters may have noticeable influence on the overall structural response of the bridge. Differential deflection of adjacent girders might primarily engage the shear stiffness of the cross-frames, while differential rotation (twisting) of adjacent girders might be more likely to engage the flexural stiffness of the cross-frames.

In general, this method is less accurate than the methods shown in Section 3.11.3 and Section 3.11.4.



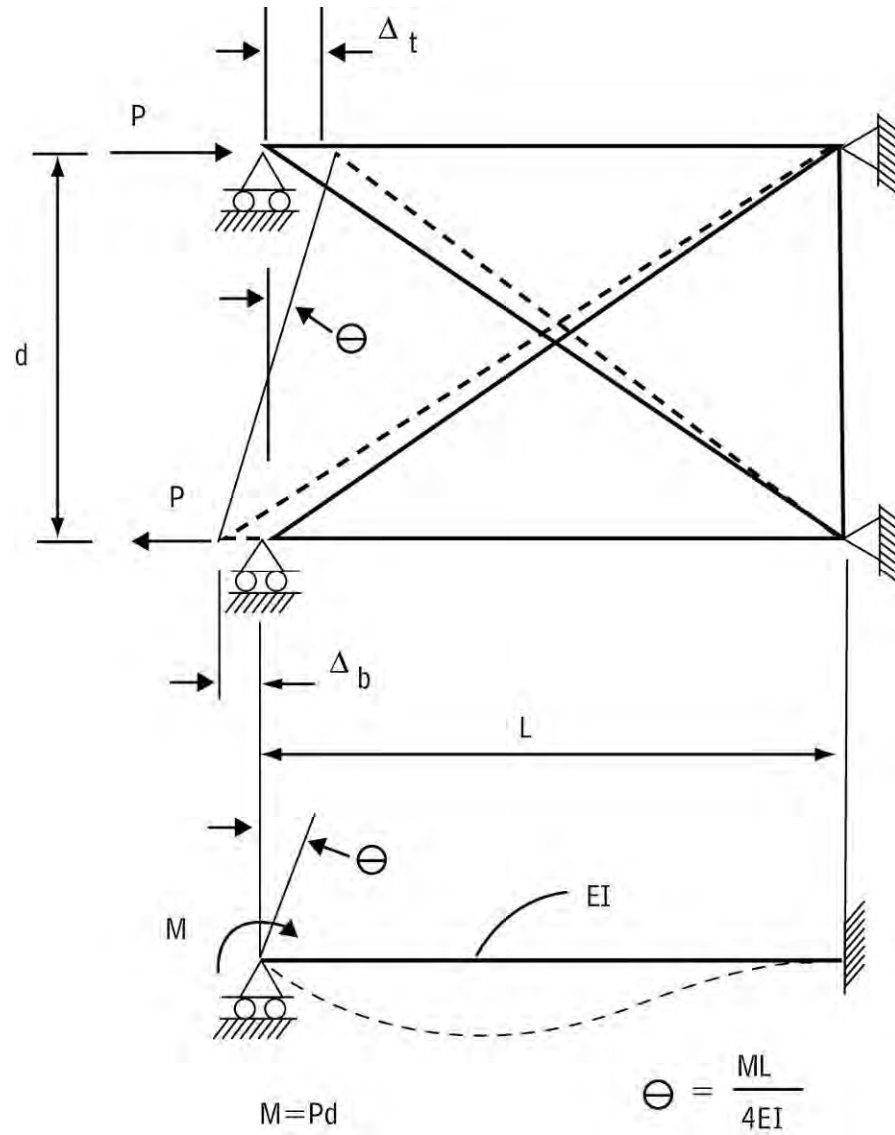
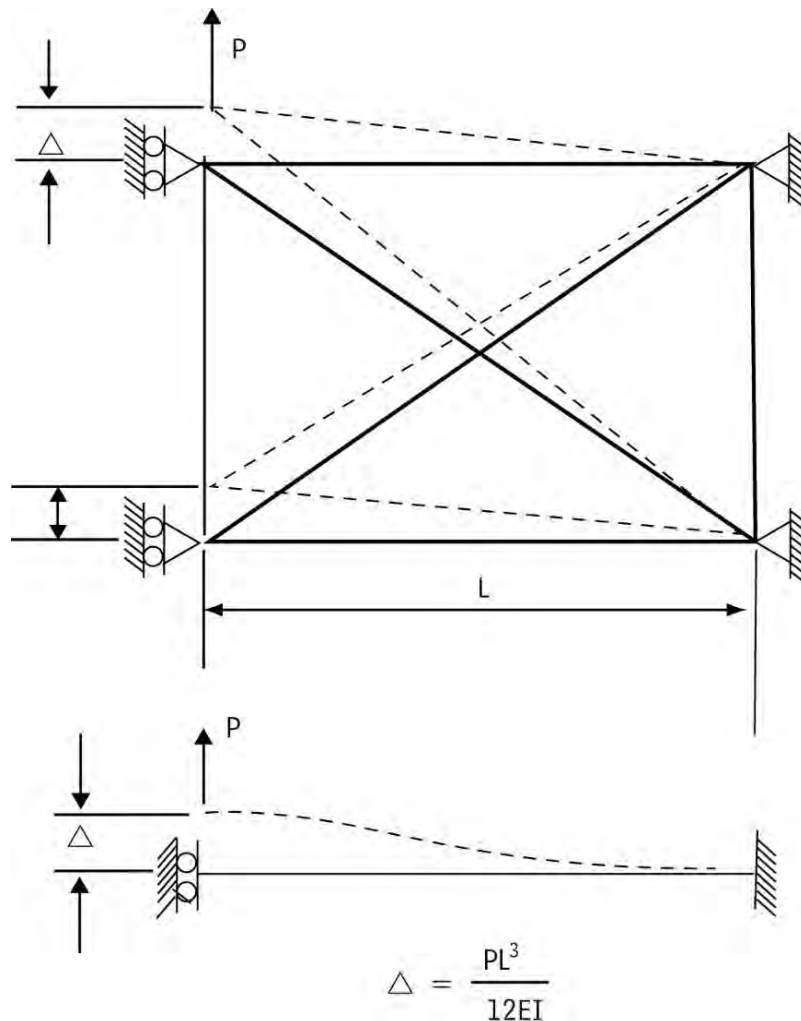


Figure 3.11.2-1. Cross frame model used to determine the equivalent stiffness of the line element used to model the flexural stiffness of the actual cross-frame.



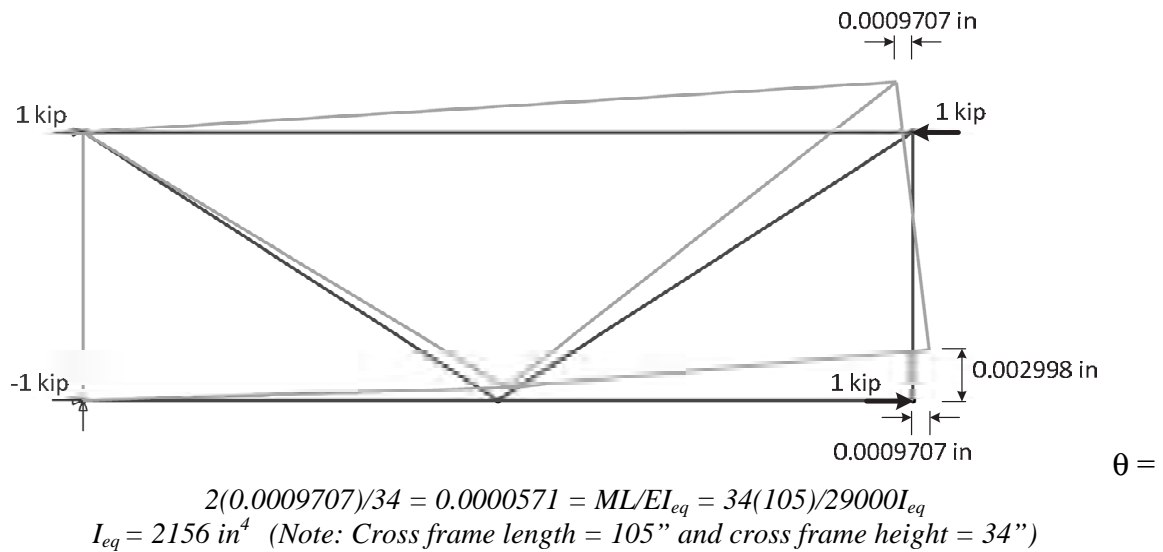
**Figure 3.11.2-2. Cross frame model used to determine the equivalent stiffness of the line element used to model the shear stiffness of the actual cross-frame.**

### 3.11.3 Shear-Deformable (Timoshenko) Beam Approach for Modeling the Stiffness of Truss-Type Cross-frames

Until recently, the significance of neglecting some of these stiffness parameters has not been conclusively determined for all possible cases of structure configuration although some research into this issue has been performed (28, 29, 31, 71). However, the research by White et al. (88) provides a much more comprehensive evaluation of this issue, and quantitatively demonstrates the inaccuracies that may be introduced when some of these simplifications are used. White et al. (88) also provide recommendations of improved methods for modeling the stiffness of truss-type cross-frames when 2-D analysis approaches are used.

Section 3.2.3 of White et al. (88) provide a complete discussion of this issue and recommends two alternate approaches that might be implemented to provide improved estimates of the stiffness of truss-type cross-frames in 2-D models: 1) an improved approximation using shear-deformable (Timoshenko) beam element representation of the cross-frame; and 2) an “exact” representation of the stiffness contributions of each member in a truss-type cross-frame via the implementation of user-defined finite elements.

The shear-deformable (Timoshenko) beam approach is described here first. Figure 3.11.3-1 illustrates the first step of a more accurate approach for the calculation of the cross frame equivalent beam stiffnesses. This approach simply involves the calculation of an equivalent moment of inertia ( $I_{eq}$ ) as well as an equivalent shear area ( $A_{seq}$ ) for a shear-deformable (Timoshenko) beam element representation of the cross-frame. In this approach, the equivalent moment of inertia is determined first based on pure flexural deformation of the cross-frame (zero shear). The cross-frame is supported as a cantilever at one end, and is subjected to a force couple applied at the corner joints at the other end, producing constant bending moment. The associated horizontal displacements are determined at the free end of the cantilever, and the corresponding end rotation is equated to the value from the beam pure flexure solution  $M/(EI_{eq}/L)$ . One can observe that this results in a substantially larger  $I_{eq}$ , and that this  $EI_{eq}$  represents the “true” flexural rigidity of the cross-frame.



**Figure 3.11.3-1. Calculation of equivalent moment of inertia based on pure bending.**

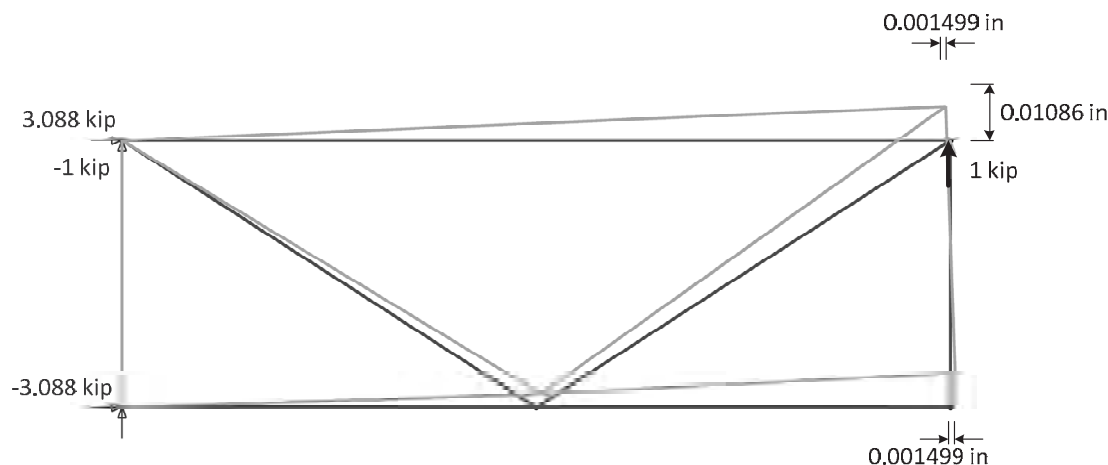
In the second step of the improved calculation, using an equivalent Timoshenko beam element rather than an Euler-Bernoulli element, the cross-frame is still supported as a cantilever but is subjected to a unit transverse shear at its tip. Figure 3.11.3-2 shows the corresponding displacements and reactions for this model, as well as the Timoshenko beam equation for the transverse displacement and the solution for the  $A_{seq}$  of the FHWA Test Bridge cross-frame.

It should be noted that the end rotation of the equivalent beam in Figure 3.11.3-2 is

$$\begin{aligned} \theta &= VL^2/2EI_{eq} - V/GA_{seq} \\ &= 1(105)^2/2(29000)(2156) - (1)(2.6)/(29000)(2.008) = 0.00004352 \text{ radians} \end{aligned}$$

However, from the deflected shape in Figure 3.11.3-2,  $\theta = 2(0.001499)/34 = 0.00008818$  radians. Therefore, it can be observed that the shear-deformable Timoshenko beam element is not able to match the “exact” kinematics of the cross-frame.

Figure 3.11.3-3 compares the cross-frame end shears and moments from an exact physical model to the nodal shears and moments for the equivalent Timoshenko beam for the case of a propped cantilever subjected to end moment. The Timoshenko beam comes reasonably close to fitting the force response of the cross-frame, compared to similar results from an Euler-Bernoulli beam element. See White et al. (88) for full presentation of this derivation and comparison.

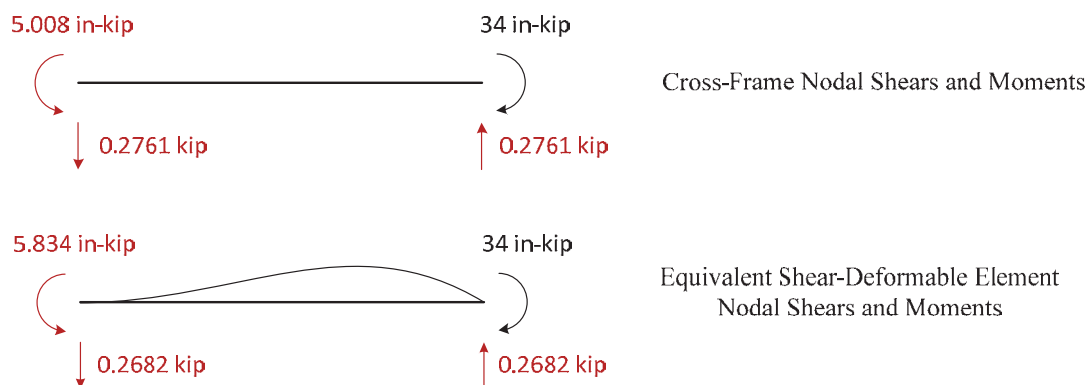


$$\Delta = 0.01086 \text{ in} = VL^3/3EI_{eq} + VL/GA_{seq}$$

$$= 1(105)^3/3(29000)(2156) + (1)(105)(2.6)/29000A_{seq}$$

$$A_{seq} = 2.008 \text{ in}^2$$

**Figure 3.11.3-2. Calculation of equivalent shear area based on tip loading of the cross-frame supported as a cantilever.**



**Figure 3.11.3-3. Cross frame nodal shears and moments and equivalent shear-deformable beam shears and moments.**

It can be shown that the Timoshenko beam element provides a closer approximation of the physical model cross frame behavior compared to the Euler-Bernoulli beam for all other types of cross-frames typically used in I-girder bridges as well, including X and inverted V cross-frames with top and bottom chords, as well as X and V cross-frames without top chords. However, similar to the above demonstrations, White et al. (88) show that the Timoshenko beam model is unable to provide an exact match for all cases.

### 3.11.4 “Exact” Modeling the Stiffness of Truss-Type Cross-frames

The next logical refinement is to develop generic X, V, inverted-V, X without top chord, and V without top chord models with variable width and height and variable cross-section area for the cross frame members (including different cross-section areas for the different members). Section 6.2.2 of Appendix B of White et al. (88) describes the development of one “exact” equivalent beam element

of this form as well as a rather easy implementation of this element as a user-defined element within one brand of commercial finite element analysis software systems. Sanchez (74) provides detailed developments of this form for all of the above cross frame types.

### 3.12 Modeling the Torsional Stiffness of I-Girders

Article 3.1.2.1 provides general discussion of the torsional response of I-girders; as noted there, I-girders respond to torsion by means of both St. Venant torsional shear flow and also by means of warping (a.k.a. flange lateral bending). White, et al. (88) show that conventional use of just the St. Venant term ( $GJ/L$ ) in characterizing the torsional stiffness of I-girders can result in a dramatic underestimation of the true girder torsional stiffness. This can lead to a number of problems with the analysis results, perhaps most importantly a significant error in the predictions of deflections. The underestimation of the true girder torsional stiffness is due to the neglect of the contributions from flange lateral bending, i.e., warping of the flanges, to the torsional properties. Even for intermediate steel erection stages where some of the cross-frames are not yet installed, the typical torsional contribution from the girder warping rigidity ( $EC_w$ ) is substantial compared to the contribution from the St. Venant torsional rigidity ( $GJ$ ). It is somewhat odd that structural engineers commonly would never check the lateral-torsional buckling capacity of a bridge I-girder by neglecting the term  $EC_w$  and using only the term  $GJ$ . Yet, it is common practice in conventional 2D-grid methods to neglect the warping torsion contribution coming from the lateral bending of the flanges.

The research conducted by White, et al. (88) shows that an equivalent torsion constant,  $J_{eq}$ , based on equating the stiffness  $GJ_{eq}/L_b$  with the analytical torsional stiffness associated with assuming warping fixity at the intermediate cross frame locations and warping free conditions at the simply-supported ends of a bridge girder, results in significant improvements to the accuracy of 2D-grid models for I-girder bridges. This observation was based in part on the prior research developments by Ahmed and Weisgerber (3), as well as the commercial implementation of this type of capability within some software packages. The term  $L_b$  in the stiffness  $GJ_{eq}/L_b$  is the unbraced length between the cross-frames.

When implementing this approach, a different value of the equivalent torsional constant  $J_{eq}$  must be calculated for each unbraced length having a different  $L_b$  or any difference in the girder cross-sectional properties. Furthermore, it is important to recognize that the use of a length less than  $L_b$  (e.g., the incorrect usage of a finite element length smaller than  $L_b$ ) typically will result in a substantial over-estimation of the torsional stiffness. Therefore, when a given unbraced length is modeled using multiple elements, it is essential that the unbraced length  $L_b$  be used in the equations for  $J_{eq}$ , not the individual element lengths.

By equating  $GJ_{eq}/L_b$  to the torsional stiffness ( $T/\phi$ ) for the open-section thin-walled beam associated with warping fixity at each end of a given unbraced length  $L_b$ , where  $T$  is the applied end torque and  $\phi$  is corresponding relative end rotation, the equivalent torsion constant is obtained as

$$J_{eq(fx-fx)} = J \left[ 1 - \frac{\sinh(pL_b)}{pL_b} + \frac{[\cosh(pL_b) - 1]^2}{pL_b \sinh(pL_b)} \right]^{-1} \quad (3.12-1)$$

Similarly, by equating  $GJ_{eq}/L_b$  to the torsional stiffness ( $T/\phi$ ) for the open-section thin-walled beam associated with warping fixity at one end and warping free boundary conditions at the opposite end of a given unbraced length, one obtains

$$J_{eq(s-fx)} = J \left[ 1 - \frac{\sinh(pL_b)}{pL_b \cosh(pL_b)} \right]^{-1} \quad (3.12-2)$$

Appendix C, Section 6.1.2 of White et al. (88) shows a complete derivation of these equivalent torsion constants.

The assumption of warping fixity at all of the intermediate cross frame locations is certainly a gross approximation. 3D-frame analysis generally shows that some flange warping (i.e., cross-bending) rotations occur at the cross frame locations. However, the assumption of warping fixity at the intermediate cross frame locations leads to a reasonably accurate characterization of the girder torsional stiffnesses pertaining to the overall deformations of a bridge unit as long as:

- There are at least two I-girders connected together, and
- They are connected by enough cross-frames such that the connectivity index  $I_C$  is less than 20 ( $I_C < 20$ ), where  $I_C$  is defined in Appendix B.

White et al. (88) provide examples of implementation of this methodology.

### 3.13 Deck Modeling (e.g., Deck Effective Width, Composite Action)

One of the complicating factors of steel bridge analysis is the need to adequately address the effects of the concrete deck.

Most modern steel girder bridges have a concrete deck that is typically made to act in a composite manner with the girders by means of shear connectors, typically headed steel studs welded to the top flanges of the girders. This deck may be fully cast-in-place, may feature a partial thickness precast concrete deck panel, or may even be a full-depth precast concrete deck.

Some older bridges are non-composite, with no physical shear connectors. These various types of construction need to be addressed appropriately. Note that in some cases, bridges that are detailed as non-composite may exhibit some degree of composite behavior. Aktan et al. (4) provide one discussion of this phenomenon; other authors have addressed this as well. Engineers are cautioned when counting on composite behavior in bridges without physical shear connectors (i.e., when counting on composite behavior based only on bond between the top flange and the deck); this type of composite behavior is not always reliable and, even if it is effective, once the bond strength is exceeded the composite connection between the flange and deck will slip with no warning and with no residual composite capacity. See Article 3.19 for more discussion of the analysis of older bridges.

If the deck is detailed and constructed to act in a composite manner with the girders (by means of shear connectors), then the analysis of those girders must consider the stages of construction, the section properties associated with each stage of construction, and which loads are applied to each stage.

Loads applied before the concrete deck is cast and hardened act only on the bare steel girder section (assuming typical, unshored construction; note that shored construction of steel girder bridges is rarely, if ever, undertaken and is generally discouraged). These are typically called the *non-composite loads* since they act on the non-composite steel girder section. These loads cause stresses in the steel girder section that are locked in and remain the same even though the deck may act in a composite manner for later loading. These stresses must be calculated using the section properties associated with the non-composite section.

Loads applied after the deck is cast and hardened act on the composite section (i.e., the section consisting of the steel girder and the concrete deck acting together). However, there are many variations of section properties that must be considered in this condition:

- In negative moment regions, typically the deck is considered to be cracked and the concrete section is considered to be ineffective. However, if shear connectors are provided throughout

the negative moment region, the shear connectors will still provide shear continuity between the deck and the girders, and the deck longitudinal reinforcing (which is bonded to the deck concrete even if the concrete is cracked) acts as part of the composite cross section (i.e., the section properties are based on the properties of the steel girder and the deck longitudinal reinforcing acting together). If shear connectors are not provided in the negative moment region, the girder is typically considered to act in a totally non-composite manner (i.e., neither the deck concrete nor the deck longitudinal reinforcing contribute to the section properties).

- In positive moment regions, transient loads such as live loads are considered to act on the short-term composite section. This section consists of the girders and the deck concrete acting together in a composite manner with an effective modular ratio of  $n$  (ratio of the modulus of elasticity of the steel and the concrete).
- In positive moment regions, permanent composite loads, such as the weight of barrier rails, future wearing surfaces, lights, and utilities, are considered to act on the long-term composite section. This section consists of the girders and the deck concrete acting together in a composite manner with an effective modular ratio of  $3n$  (3.0 times the ratio of the modulus of elasticity of the steel and the concrete). This reflects long-term creep effects in the concrete, effectively reducing the stiffness of the concrete over time.

In calculating the effects of the deck on girder section properties in the longitudinal direction, the effective width of the deck contributing to each girder's section properties must be determined. Historically, empirical rules were used to determine the effective width, including limitations such as 12 times the deck thickness or  $1/4$  the span length. However, recent research resulted in a change to the *AASHTO LRFD Bridge Design Specifications* (8) and now the entire deck can be considered effective. Typically, for an interior girder, this means that the effective width of the deck extends half of the girder spacing on each side of the girder; similarly, for an exterior girder, the effective width extends half of the girder spacing to the interior girder and to the limit of the overhang on the exterior side.

In analysis models, the effects of the deck are handled in various ways, depending on the type of analysis model being used. For line girder and 2D grid analysis models, the effects of the deck in the longitudinal direction are typically addressed by adjusting the section properties of the girder elements to reflect the composite section properties for composite load cases. The effects of the deck in the transverse direction are typically ignored in line girder models. In 2D grid models, the effects of the deck in the transverse direction are often handled by modeling effective strips of deck, particularly at cross frame locations, for composite load cases. In plate- and eccentric-beam models and 3D FEM models, the deck is typically explicitly modeled using plate, shell, or brick elements for the composite load cases, thus capturing the effects of the deck in both the longitudinal and transverse directions directly.

### 3.14 Bearings, Substructures, and Boundary Conditions for Models

#### 3.14.1 Rollers versus Pins

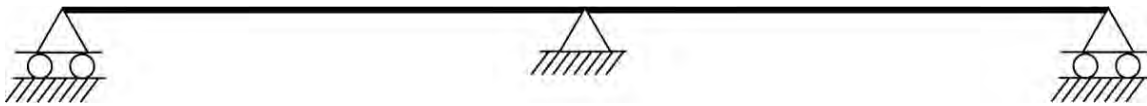
Proper definition of boundary conditions is essential to correct modeling of structural behavior. Consider first a simple-span beam (Figure 3.14.1-1). The left support is a pin support—a support that allows rotation but prohibits vertical or horizontal displacement. The right support is a roller—a support that allows both rotation and horizontal displacement but prohibits vertical displacement. In order for the beam to truly be considered statically determinate, the right support must be a roller so that the bottom flange is free to extend when subject to tension caused by positive moment (moment induced by vertical gravity loads). If both supports were pin supports, then the bottom flange extension would be restrained and some degree of indeterminacy would be introduced to the problem because there would be potential for development of fixed end moments.



**Figure 3.14.1-1. Simple-span beam with statically determinate boundary conditions.**

However, this indeterminacy is dependent on the location of the horizontal restraint relative to the neutral axis of the beam. If the beam is prismatic (constant cross section) and the horizontal restraints are located at the neutral axis of the beam, no moment restraint would exist at the ends of the beam under idealized conditions.

These basic concepts are not limited to a single simple-span structure. Multiple-span continuous beams are subject to the same provisions (Figure 3.14.1-2).

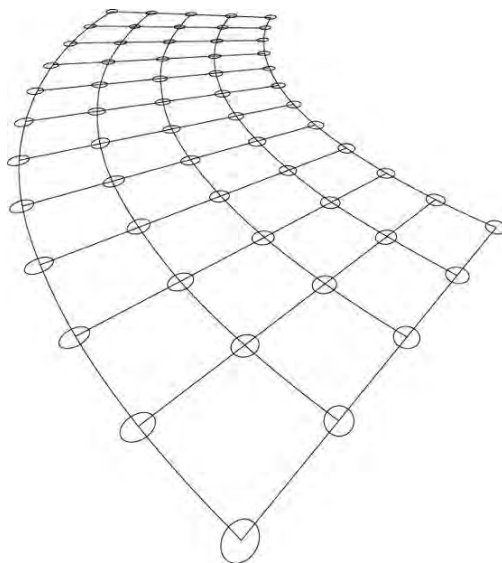


**Figure 3.14.1-2. Two-span continuous beam.**

When steel girders are modeled using a line girder approach and a single girder is modeled using line elements, engineers often neglect to pay close attention to the subtleties of this boundary condition question. The line element is modeled from node to node, with the neutral axis assumed to be coincident with the nodal geometry and thus located at the support point. Furthermore, in the real structure, typically some mechanism allowing lateral displacement is provided as part of provisions to allow thermal expansion and contraction. Designers can easily fall into a habit of complacency with regard to modeling of boundary conditions, allowing their computerized design program to use default boundary condition assumptions without properly scrutinizing those assumptions. In many 1D line girder analysis programs, the specific boundary condition assumptions will have little if any effect on the analysis results so this lack of scrutiny of the boundary conditions typically causes no problems.

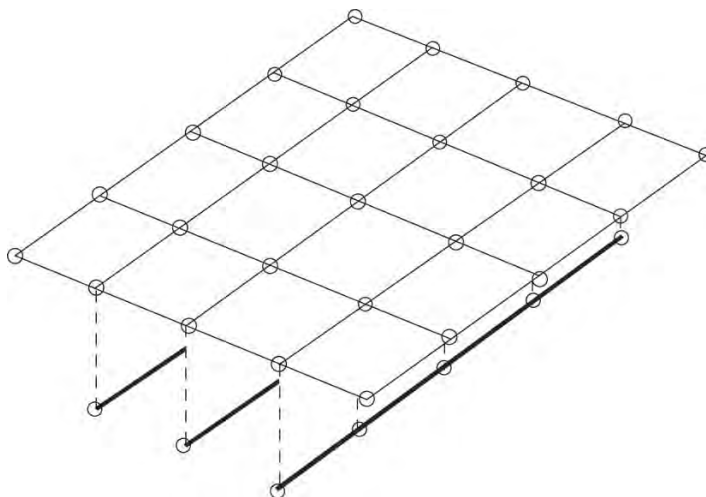
Consider next a 2D grid analysis. Initially, consider a traditional grid analysis model, in which the entire bridge superstructure is modeled using an array of nodes and line elements confined to a single, horizontal plane (Figure 3.14.1-3). For the purposes of this discussion, assume a relatively simple structure in which only primary girder flexure and shear effects are significant. Once again, for modeling of gravity loading effects, it is not necessarily critical to carefully differentiate which supports allow for horizontal translation or not.





**Figure 3.14.1-3. Traditional 2D grid analysis model.**

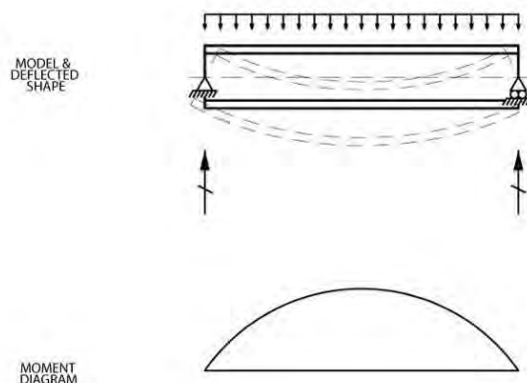
Now consider a modification to the traditional 2D grid analysis model—specifically the plate- and eccentric-beam model (Figure 3.14.1-4). In a plate- and eccentric-beam model, the deck is modeled using plate elements and the girders are modeled using line elements. The line elements used to model the girders are located at an offset from the deck plate elements equal to the distance from the deck to the neutral axis of the girder. In such a model, moment is resisted not only by the girder acting in flexure, but also by the force couple between the girder and the deck. In such a model, care must be taken to model the boundary conditions more accurately or the model will produce incorrect results. If, for instance, all of the supports were modeled as pins (resisting both vertical and horizontal translation), the model will behave as if there is some degree of moment fixity at the supports.



**Figure 3.14.1-4. A variant on the 2D grid analysis model—the plate- and eccentric-beam model.**

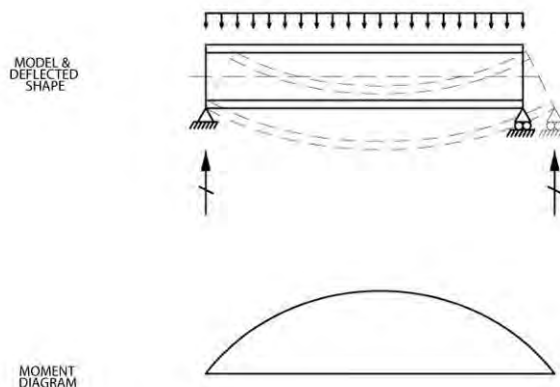
A more detailed presentation of the beams illustrated in Figure 3.14.1-1 and Figure 3.14.1-2 is provided in Figures 3.14.1-5 through 3.14.1-8. Figure 3.14.1-5 shows a simply supported beam with a pin support at one end and a roller at the other, with both supports located at the neutral axis of the beam. Given that the supports are at the neutral axis, there is no extension or contraction of the beam

length at that level due to vertical loads. The beam acts as would be intuitively expected for a simply supported beam.



**Figure 3.14.1-5. Simply supported beam with a pin support at one end and a roller at the other, with both supports located at the neutral axis of the beam.**

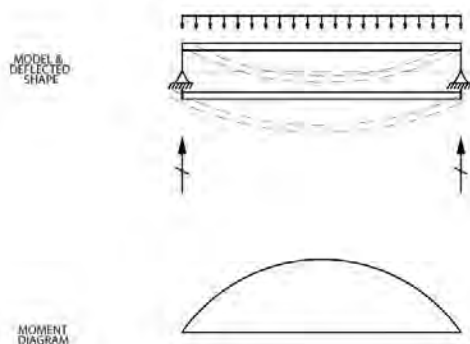
Figure 3.14.1-6 shows the same beam, with the same pin support at one end and roller support at the other end but with the supports at the bottom flange. In this model, under vertical loading, the beam undergoes positive moment along its entire length, and the bottom flange extends due to flexural tension stress. Because one of the supports is a roller, it offers no resistance to this flange extension and thus no restraint. The beam acts as would be intuitively expected for a simply supported beam.



**Figure 3.14.1-6. Simply supported beam with a pin support at one end and a roller at the other, with both supports located at the bottom flange.**

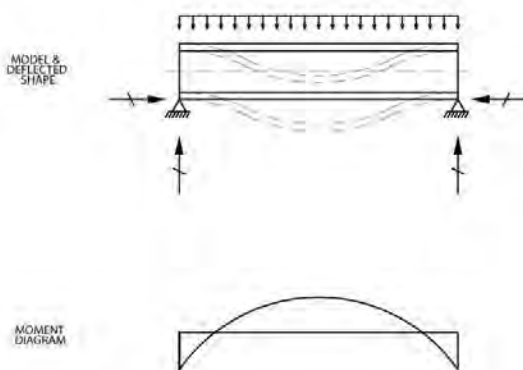
Figure 3.14.1-7 shows the same beam as Figure 3.14.1-5 but with pin supports at both ends. Again, as in Figure 3.14.1-4, the supports are at the neutral axis of the beam. Since the supports are at the neutral axis, there is no extension or contraction of the beam length at that level due to vertical loads. So, even though the pin supports at both ends of the beam would act to restrain any contraction or

extension displacement, because they are located at the neutral axis, there are no contraction or extension displacements to restrict. The beam acts as would be intuitively expected for a simply supported beam.



**Figure 3.14.1-7. Simply supported beam with a pin support at both ends, with both supports located at the neutral axis of the beam.**

Figure 3.14.1-8 shows the same beam as Figure 3.14.1-7 but with the supports at the bottom flange. In this case, because the supports are at the bottom flange and the supports are pins at both ends, the bottom flange is restrained from extending, given that the ends of the flange are restrained against translation. This restraint of the bottom flange extension results in horizontal reactions at the two pin supports. Because these horizontal reactions are located at the bottom flange, some distance from the neutral axis, this causes a net uniform negative moment to be applied to the beam (the negative moment is equal to the horizontal restraint force times the eccentricity between the bottom flange and the neutral axis). This net uniform negative moment is superpositioned with the positive moment caused by vertical loading on the span, resulting in a moment diagram much like that of a beam with some degree of moment restraint at the ends. The behavior of a beam like that shown in Figure 3.14.1-8 is analogous to the behavior of a rigid frame.



**Figure 3.14.1-8. Simply supported beam with a pin support at both ends but with both supports located at the bottom flange.**

### 3.14.2 Fixed versus Guided versus Free Bearings

The preceding discussion of roller versus pin boundary conditions is just one example of the importance of proper modeling of boundary conditions in a steel girder bridge analysis, considering just three degrees of freedom (primary girder rotation, vertical translation, and longitudinal translation).

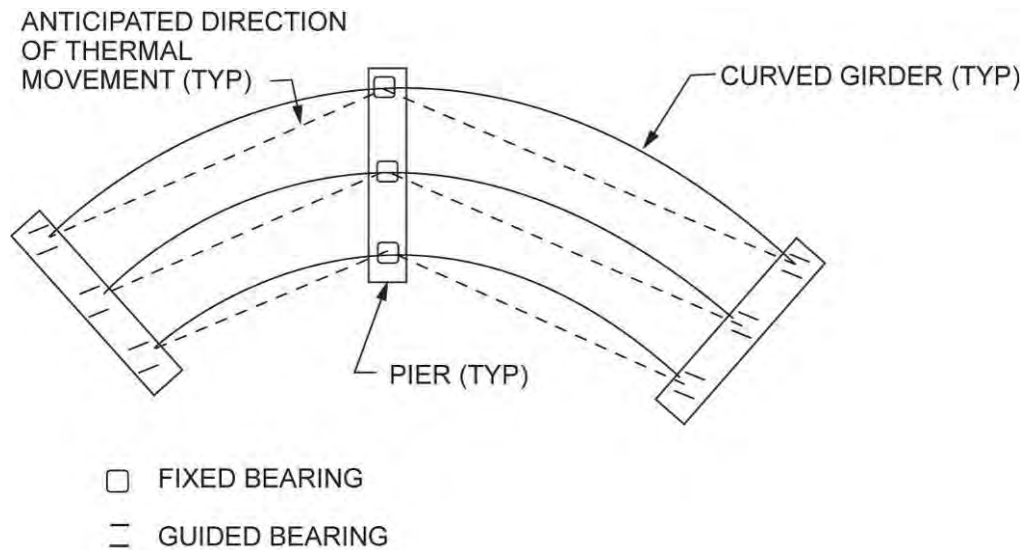
The importance of boundary conditions can be further illustrated by adding another degree of freedom—transverse translation. In many bridges, there are multiple types of bearings used, including:

- Fixed bearings that allow no translation in either the longitudinal or transverse direction,
- Guided bearings that allow translation in one direction (either transverse or longitudinal) but prevent translation in the associated orthogonal direction, and
- Free bearings that allow translation in both directions.

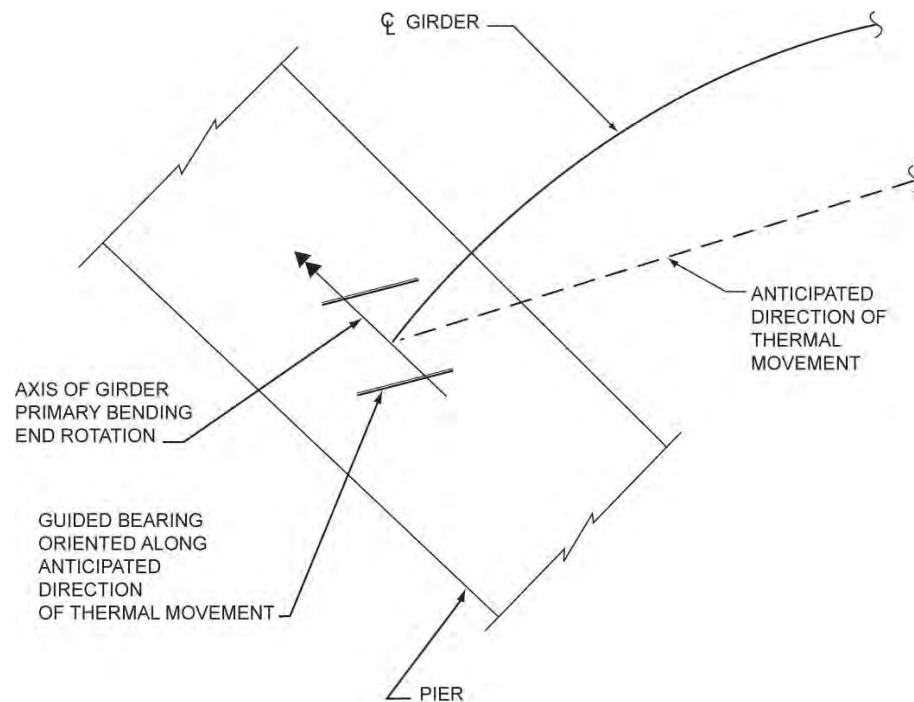
Bearing conditions must be carefully chosen to accommodate anticipated bridge movements in predictable and acceptable ways, and those bearing conditions must also be carefully modeled in the superstructure analysis model in order to accurately calculate the response of the structure to various loading conditions. Note also that during bridge erection, bearing points may be temporarily blocked (partially fixed), so the construction case may not have guided or nonguided (free) bearing points. This may be a consideration if significant thermal movements are anticipated at partially erected structural conditions.

For example, in a curved girder bridge, the bearings can be oriented so that the free directions of guided bearings are aligned with chord lines drawn between the supports in an effort to align translational freedom with the thermal expansion and contraction movements anticipated under uniform temperature changes (Figure 3.14.2-1). The bearing orientations must be reproduced in the analysis model, not only to correctly model the structure response under uniform temperature change but also to correctly model the structure response to other loading conditions such as dead load, live load, and centrifugal force. This generally requires a refined analysis that includes modeling of the depth of the structure, the effects of the deck acting as a diaphragm, the configuration of the cross-frames, and other structural modeling considerations. Girder primary bending rotation may be aligned more closely to the local radial line at the support, which may not be aligned with the anticipated thermal movement direction (Figure 3.14.2-2). Thus, the bearings may offer some degree of restraint to girder primary bending moment in a way similar to that described for the simple comparison of pinned versus roller supports above. The increasing use of integral abutments in steel bridges offers another example of complex restraint conditions that would require careful modeling as the horizontal resistance of the piles presents a case that is neither 100 percent fixed nor 100 percent free.

Depending on the specific configuration of the structure in question, improper modeling of bearing conditions (boundary conditions) could have a significant impact on the correctness of the analysis results. Boundary conditions should be carefully modeled and, in cases where the support stiffness is not known with certainty (such as with integral abutments), it may be advisable to run more than one analysis with different assumptions to assess the sensitivity of the structural response to the different boundary condition assumptions, with consideration given to designing for the resulting force and deflection envelopes.



**Figure 3.14.2-1.** The direction of anticipated thermal movement in a curved bridge is often not coincident with the centerline of the girders. Typically, the guided bearings are aligned with the direction of anticipated thermal movement.



**Figure 3.14.2-2.** Because the direction of anticipated thermal movement often is not aligned with the centerline of the girder in a curved girder bridge, while guided bearings typically are aligned with the direction of anticipated thermal movement, the bearings may not be aligned with the anticipated direction of the bottom flange extension under positive girder moment (i.e., the axis of primary girder rotation at the end of the girder is radial to local tangent to the girder centerline but not radial to the direction of anticipated thermal movement).

### 3.14.3 Unusual Substructures and the Effect of Variable Substructure Stiffness

Substructures are essential elements in bridge structures because they carry the superstructure and transmit the loads to the foundation. In general, bridge piers have different configurations, shapes, and sizes. Bridge piers can be in the form of hammerhead, multicolumn bent, pile bent, solid wall, single column, or integral, depending on the form of the superstructure present, clearance requirements, soil conditions, and aesthetics. Abutments (or end bents) have a similar variety of configurations. Traditionally, the bridge superstructure is usually supported on top of the pier or abutment cap by means of bearings.

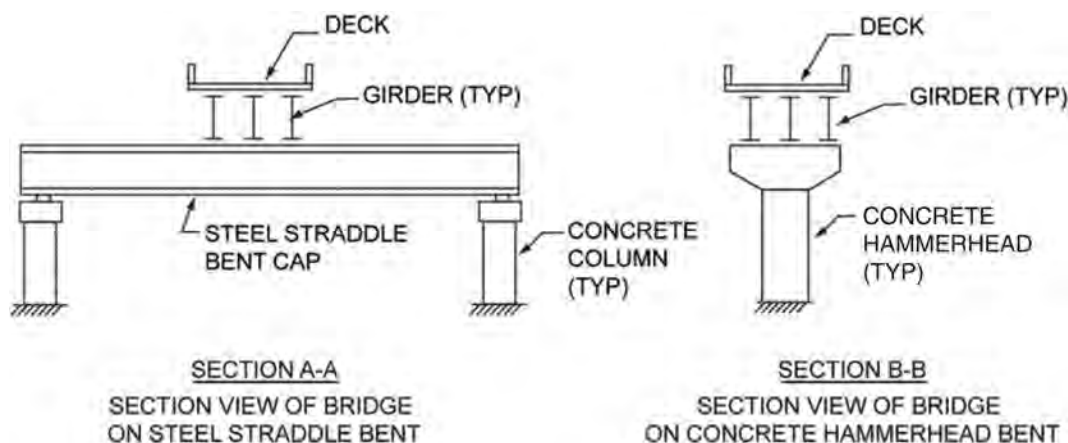
In many cases, the effects of the configuration and stiffness of substructures on the behavior of the superstructure are insignificant and can be safely neglected in the superstructure analysis. However, there are some cases where the effects of substructure stiffness on superstructure behavior are significant. Some of these cases are discussed below.

#### 3.14.3.1 *Straddle Bents and Other Variable Substructure Stiffness Effects*

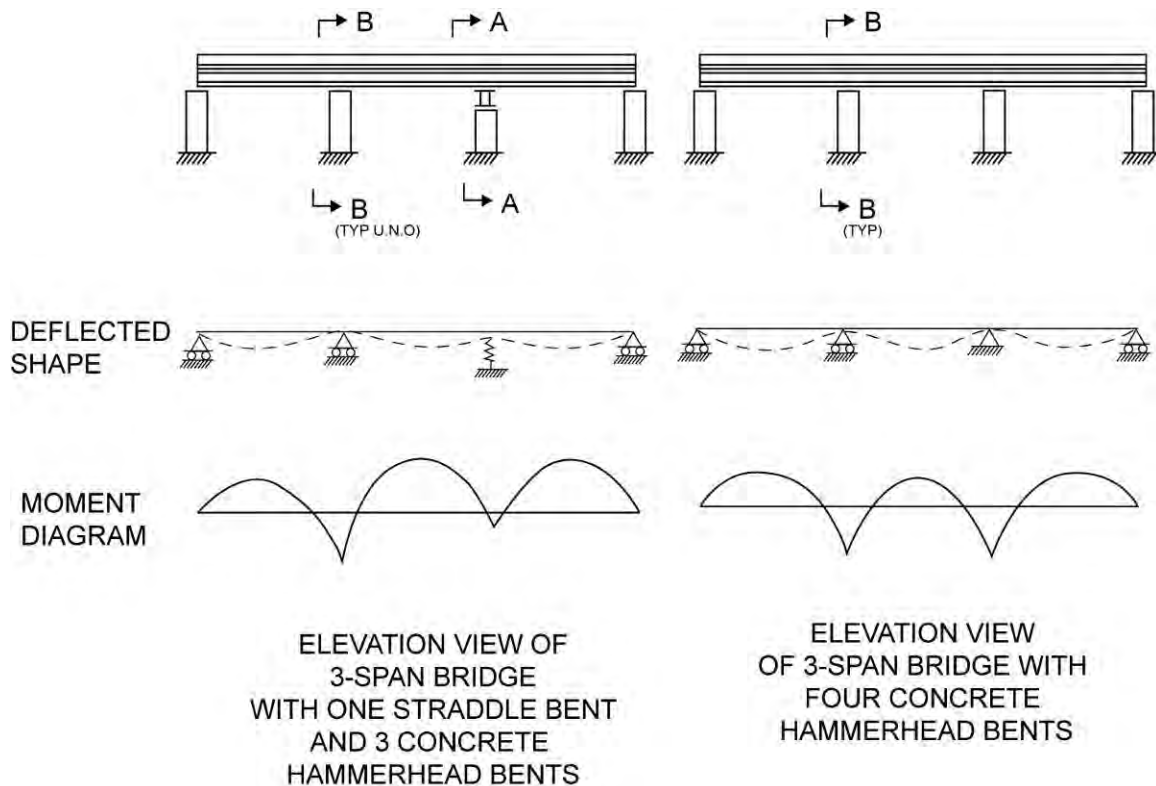
The prior discussions of boundary conditions focus primarily on correct modeling of absolute measures of fixity or freedom of the bearings. This assumes that bearings are either fixed or free in various directions, an all-or-nothing approach to boundary conditions for various degrees of freedom. For many structures, these types of simplifications are adequate. However, for some structures, the questions associated with modeling of boundary conditions are more complicated.

In the previous discussions, it is implicitly assumed that the vertical stiffness of all supports is infinite, or at least uniform (i.e., it is implicitly assumed that all bearings have the same vertical stiffness). In many cases, this is a reasonable enough assumption and will produce reasonably correct results. However, in some structures, the stiffness of various supports is not equal or uniform, and consideration of the vertical stiffness of various supports is necessary.

For example, consider the case of a long-span steel straddle bent versus a concrete single column hammerhead bent with short, stocky overhangs (Figure 3.14.3.1-1). The vertical stiffness offered by the long-span steel straddle bent will be less than that offered by the concrete hammerhead bent because the straddle bent cap possesses significant vertical flexibility, while the concrete hammerhead is essentially rigid in the vertical direction. If several supports of a multispan continuous steel girder bridge are concrete hammerhead bents, with one support being a long-span steel hammerhead bent, the response of the girders to vertical loading will be different than in a structure that is otherwise identical but has all concrete hammerhead bents (Figure 3.14.3.1-2).



**Figure 3.14.3.1-1. Section views of a bridge with girders sitting on a steel straddle bent versus girders sitting on a concrete hammerhead illustrates how the straddle bent may provide a much more flexible support than the concrete hammerhead bent.**



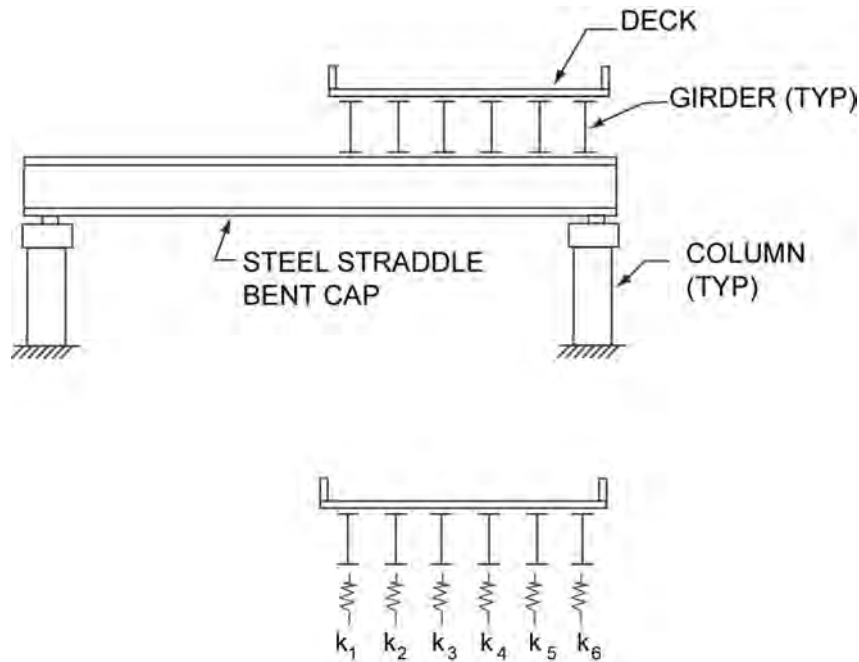
**Figure 3.14.3.1-2. Elevation views of a bridge with three concrete hammerhead supports and one flexible steel straddle bent support versus a bridge with four concrete hammerhead supports show how the superstructure response to loading may be different given different support point stiffnesses. See 0 for Sections A-A and B-B.**

As another example, consider a bridge with a relatively wide, multiple-girder cross section, again supported at one or more bents by a steel straddle bent (Figure 3.14.1-3). In this case, the vertical stiffness offered by the support for the leftmost girder in the cross section will be different than that offered by the support for the rightmost girder in the cross section.

These types of variations in vertical support stiffness can be significant in some cases and should be considered in the analysis model. In cases such as this, it may be prudent to model part or all of the substructure elements in order to address the effects of relative support stiffness.

While the immediately preceding discussion focuses primarily on the vertical stiffness of substructures, the lateral stiffness of substructures can have an influence on the superstructure behavior as well, (e.g., the superstructure's response to thermal movements, interaction with integral abutments, etc.). In general, if these types of effects on the superstructure's behavior are of significance or of concern to the designer, consideration should be given to including representations of the stiffness of the substructures in the superstructure analysis model.

It should be noted that it is difficult, if not impossible, to determine with absolute confidence the exact level of stiffness or flexibility offered by various substructure configurations. As a result, designers are encouraged to consider a range of relative stiffness assumptions and to design the structure to accommodate any behavior within that envelope of substructure stiffness assumptions. A simple range can typically be determined by considering both the assumption of fully rigid support and the assumption of a flexible support based on the actual structure configuration.



**Figure 3.14.3-3.** In some cases, individual girders at a given line of support may have different support stiffnesses, causing different load distribution among the girders than would be found if all girders had equally stiff supports. In this case  $k_6 > k_5 > k_4 > k_3 > k_2 > k_1$ .

#### 3.14.3.2 Integral Straddle Bents

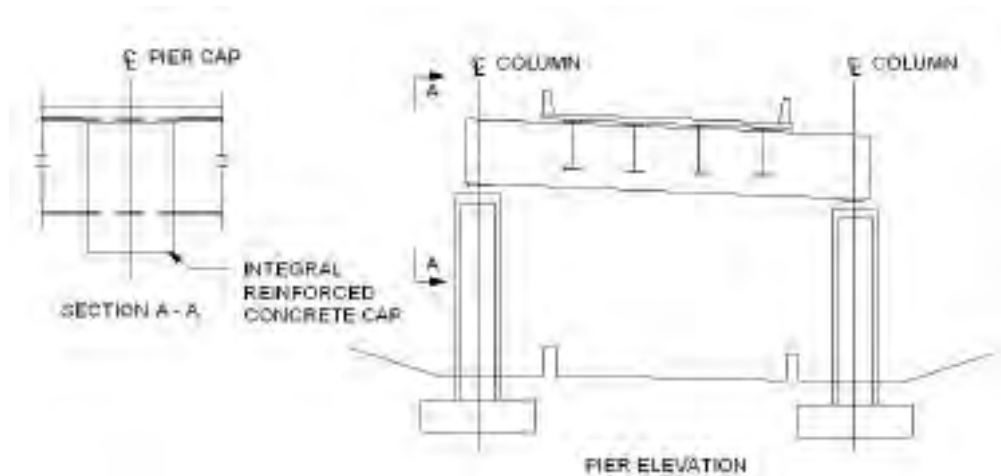
Integral straddle bents are utilized where inadequate vertical or horizontal clearances exist at roadway crossings. In general, these situations occur in bridge widening projects in urban areas or at complex multilevel interchanges where placement for bridge supports are not available. Therefore, straddle bents are used to span over these roadways with geometric constraints where cost is not prohibitive for relocation at a project site. Nevertheless, straddle bents should be the last resort for a bridge support in development of efficient span arrangement.

The integral straddle bent can be constructed with concrete or steel, in either instance where commonly the longitudinal girders of bridge are integrally framed into these transverse support beams.

In general, a concrete cap member must be post-tensioned to keep the member depth to a minimum or at least the same depth as longitudinal members. However, for a short bent cap, it can be constructed with a mildly reinforced concrete section. In both concrete schemes, the straddle bent would be maintenance-free as well as less likely to have fatigue-related issues during service life of the structure. The construction of a concrete cap will require involved construction sequencing effort and coordination during longitudinal girder erection, which will have an adverse impact on maintenance of traffic for the roadways below during construction stages. The framing connection of longitudinal steel girders to a concrete cap is much more detail-oriented and complex, therefore involving construction time longer than that for steel caps. For an example of this construction type (Type 1), see Figure 3.14.3-2-1.

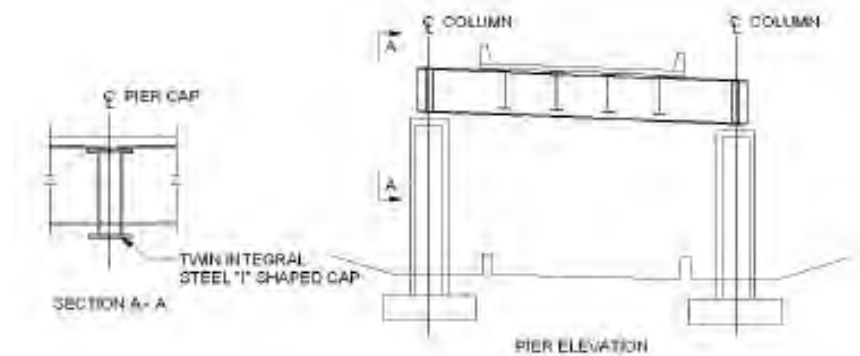
On the other hand, there are several integral steel cap options; in recent years, there has been a trend for use of twin I-beams (Type 2). The idea behind this concept is based on the redundancy implemented by the two independent I-beam members to provide a redundant load path. Another option is a welded box beam section (Type 3), which has performed very well for many years, but this concept has been sidelined due to inherent fatigue details and lack of redundancy.



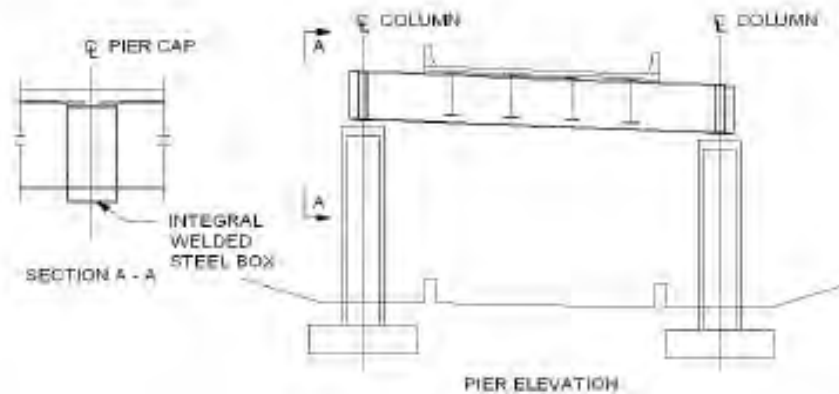


**Figure 3.14.3.2-1. Reinforced Concrete Integral Pier Cap, Type 1**

In integral steel straddle bents, the framing of longitudinal girders is easily achieved by typical bolted connections, similar to those at a field splice. Therefore, the integral steel straddle bent erection can minimize impact of a structure's work envelope required for construction, particularly in urban settings. This type of construction offers unquestionable advantages in this regard over the concrete bent cap, which may result in much higher superstructure cost. For examples of these construction types (Types 2 and 3), see Figure 3.14.3.1-2 and Figure 3.14.3.1-3.



**Figure 3.14.3.2-2. Twin I-Girder Integral Pier Cap, Type 2**



**Figure 3.14.3.2-3. Welded Steel Box Girder Cap Integral Straddle Bent Cap, Type 3**

All three types can be considered virtually redundant, with good resistance for torsion and out-of-plane bending forces. The welded steel box girder type (Type 3) may be considered fracture critical, so greater toughness is required. The benefit of this type is higher torsional resistance and easier inside access than the other two types. If traffic and construction time are concerns, steel girders are preferred due to no shoring.

### 3.14.3.3 *Integral Pier Caps*

Integral pier caps offer the opportunity to bury the pier cap in the superstructure and thus they are often used to solve vertical clearance problems. In addition to solving clearance problems, integral pier caps are more robust structurally and can be more aesthetically pleasing than conventional piers. Two types of integral pier caps for steel girder bridges, 1) integral post-tensioned pier cap type and 2) integral steel cap girder type, are discussed here.

#### *Post-Tensioned Concrete Integral Pier Caps*

In general, an integral post-tensioned cap on a single column will experience shear, flexure, and torsion under the effects of dead and live loads (including impact). In addition, post-tensioning bars or strands will induce axial forces in the cap. Since the steel girders are continuous through the cap, forces imposed on the cantilevers by the applied loading must be carried from one panel between the girders to adjacent ones by means of stirrups and shear connectors welded to the webs of the steel girders. Current bridge specifications are not directly applicable to post-tensioned integral pier caps supporting continuous composite steel girder bridges.

An example of the design of an integral cap in flexure is shown in Figure 3.14.3.3-1. Post-tensioning is specified at two stages to minimize stresses in the concrete. The shear and torsion design of the cap results in a large percentage of closed transverse reinforcement. Additional longitudinal mild steel is placed along the perimeter of the cap due to torsional demand and serviceability, as shown in Figure 3.14.3.3-2.

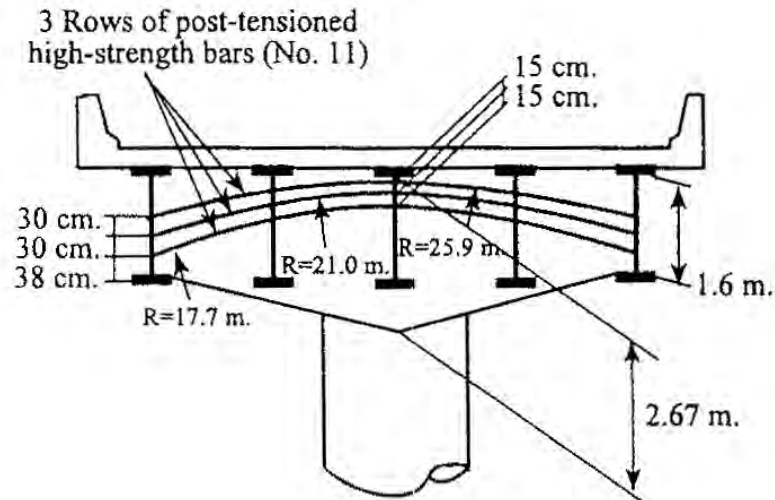


Figure 3.14.3.3-1. Location of post-tensioned bars in the pier cap.

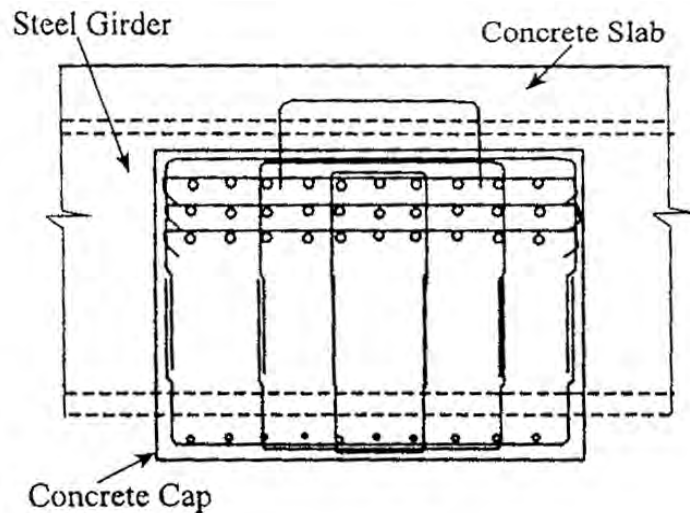


Figure 3.14.3.3-2. Detail of transverse reinforcement in the cap.

#### *Steel Integral Pier Caps*

Steel integral pier caps can take the form of I-shaped or box-shaped members but all share the same basic concept of integrating a steel pier cap directly with the steel superstructure by means of intersecting structural steel elements connected by bolting or welding. Rocker bearings have been used in Texas as part of the connection between a concrete pier and a steel cap girder supporting continuous steel bridge girders. A schematic of this type of connection is shown in Figure 3.14.3.3-3. Twin bearings at each concrete pier are designed to resist moments in the transverse direction caused by eccentric truck traffic, as shown in Figure 3.14.3.3-4. In the longitudinal direction (the direction parallel to traffic flow), the rocker bearing combined with long anchor bolts is designed to produce an ideal pin support so that the continuous longitudinal bridge girders are not restrained at the pier. The main reason for the free rotation concept is to avoid fatigue in the steel cap girder details caused by alternate span loading. This connection, which is essentially a fixed support in the transverse direction and a pinned support in the longitudinal direction, is fairly complex and costly.

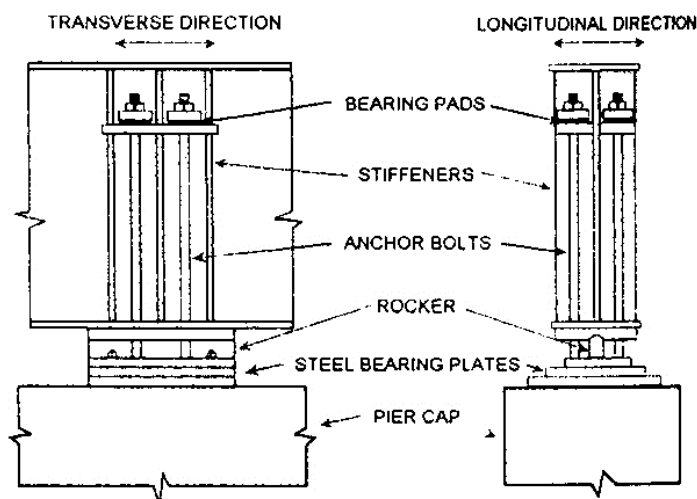


Figure 3.14.3.3-3. Schematic of typical connection of an integral steel cap girder.

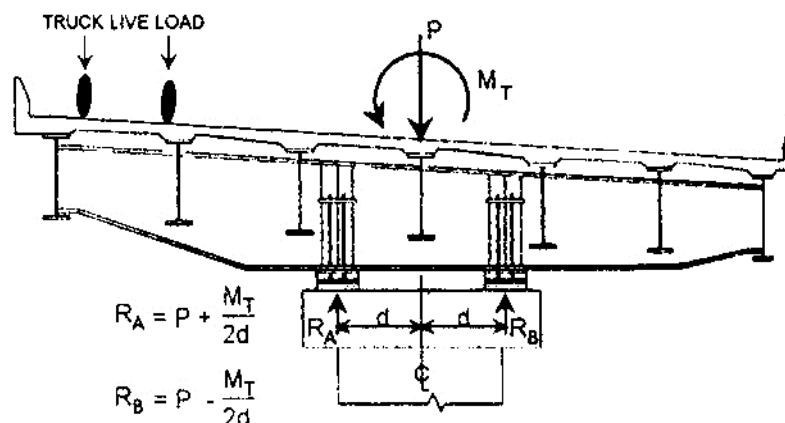


Figure 3.14.3.3-4. Vertical loads on bearings for an integral steel cap girder.

Based on the results of the testing conducted at the University of Texas at Austin (15), a detail for the integral steel cap was developed, as shown in Figure 3.14.3.3-5 and Figure 3.14.3.3-6. It incorporates two major changes from the typical connection; the first change is to replace the rocker element and sole plate with a rolled wide-flange section. Two major advantages of the wide-flange bearing detail are that it provides a positive connection between the cap girder and the pier cap (the previous detail requires anchor bolts) and needs none of the fabrication necessary for the rocker bearing. Also, the new detail replaces the threaded steel rod (and bearing pad) with high-strength threadbar, which is specifically designed for pretensioning and for which installation procedures are well established. The new detail is less complex than the previous detail and has the capability of resisting uplift.

Integral abutment bridges are designed without any expansion joints in the bridge deck. They are generally designed with the stiffness and flexibilities spread throughout the structure/soil system so that all supports accommodate the thermal and braking loads. They are single- or multiple-span bridges having their superstructure cast integrally with their substructure. Generally, these bridges include capped pile stub abutments. Piers for integral abutment bridges may be constructed either integrally with or independently of the superstructure.

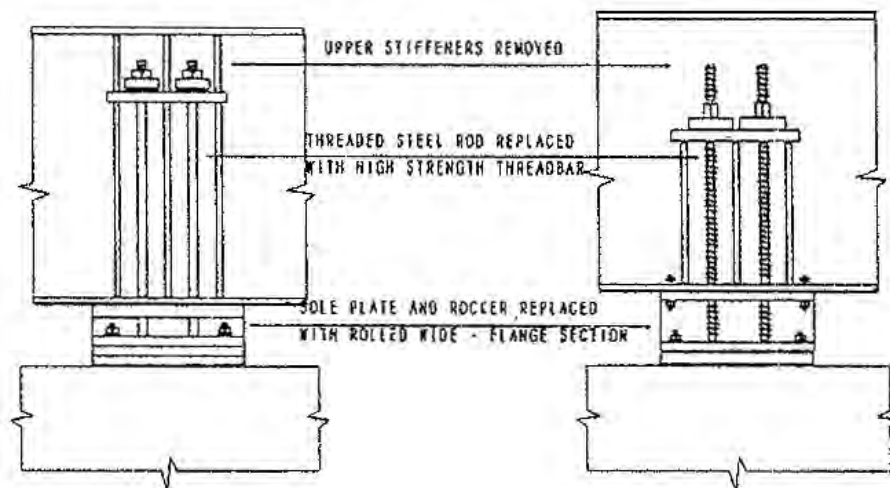


Figure 3.14.3.3-5. Transverse direction detail.

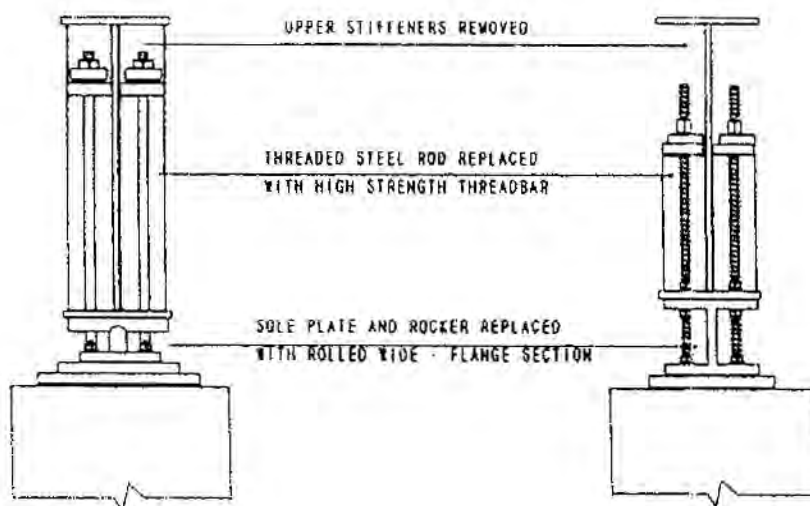


Figure 3.14.3.3-6. Longitudinal direction detail.

#### 3.14.3.4 Integral Abutments

Less than desired service life of bridges has been blamed on numerous factors but none as often as drainage problems. The use of continuous superstructures has eliminated many deck expansion joints at piers and therefore reduced the probability of steel corrosion and concrete deterioration caused by leaking joints. Eliminating deck expansion joints at abutments offers the same advantage but requires a significant alteration to conventional bridge design and construction called an integral abutment.

Essentially, the superstructure and the abutments are one unit. Full-depth concrete end diaphragms are connected to steel girders with shear studs or horizontal steel reinforcing bars through holes in the girder webs. The end diaphragms are connected with vertical steel reinforcing bars to concrete grade beams, which temporarily support the girders. The grade beams are each supported by a single line of piles. If H-piles are used, they are usually oriented so that longitudinal superstructure movement bends them about their weak axis. Prior to driving each pile, a hole several times the effective pile diameter is drilled to a predetermined depth and filled with pea gravel or another material that does

not compact. The pile is driven through the material and into the underlying strata to a depth that provides vertical and lateral support. Pile orientation and predrilled holes reduce the shear and moment produced by superstructure rotation and translation on the supporting piles.

Depending on the span length between integral abutment and adjacent pier or—if the bridge is a single span—another integral abutment, the ratio of superstructure to supporting pile flexural stiffness is between 50 and 100. Thus, the integral abutment offers little rotational resistance. In fact, the steel superstructure can be conservatively designed without rotational restraint at the abutments and the supporting piles can be conservatively designed assuming the moment at the abutment interface is at the plastic limit. This conservatism allows any axial force in the steel superstructure caused by the translational resistance of the piles to be left out of the design as is normally done with consecutively fixed piers. These design simplifications are also made possible because the full-depth concrete end diaphragm which provides continuity between the steel superstructure and the integral abutment is usually one of the last cast-in-place concrete elements constructed.

There are two main types of integral abutments:

- *Full integral abutment on piles* (Figure 3.14.3.3-1)—There is a full monolithic connection between the end of the superstructure and abutment with a single line of steel H-piles that flex to accommodate thermally induced bridge deck movements. This is the most efficient design in most situations and every effort should be made to achieve full integral construction.

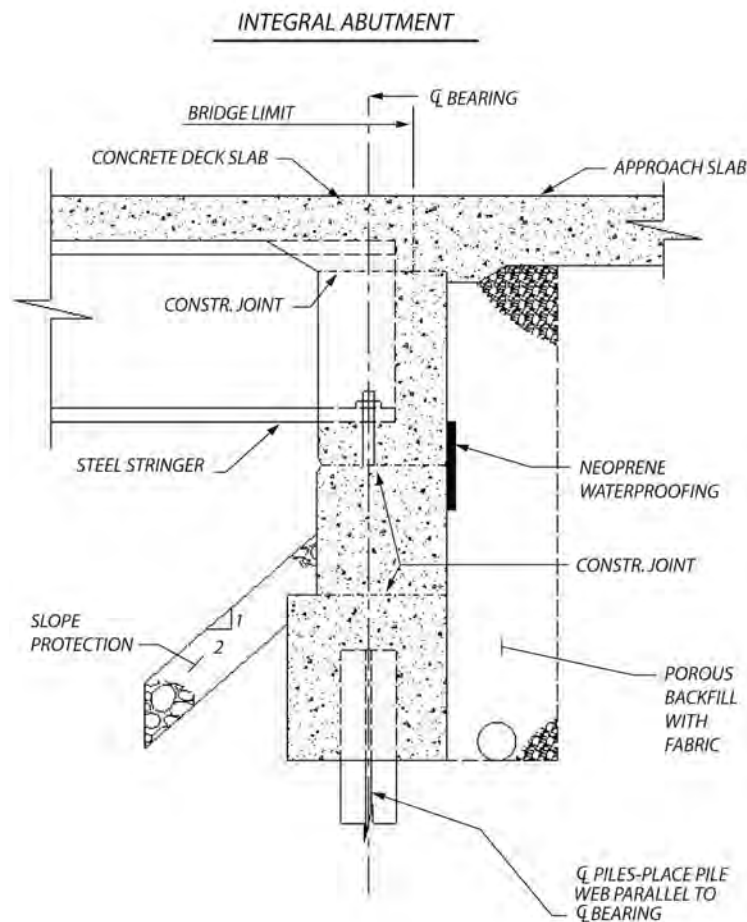
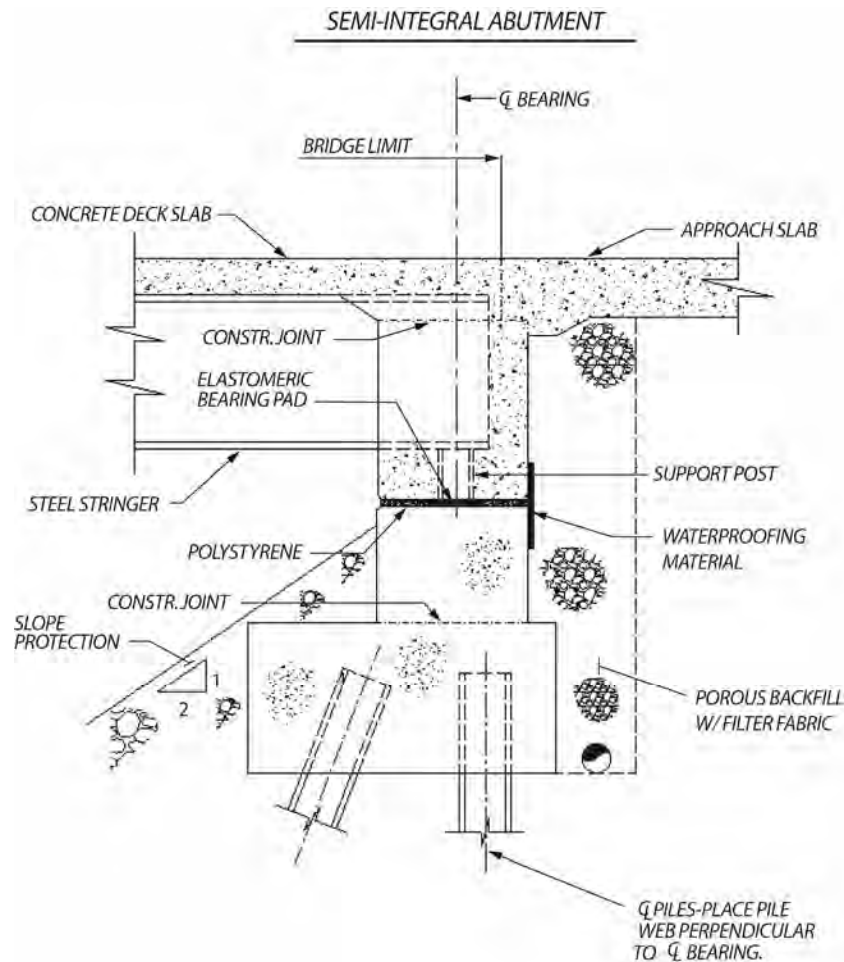


Figure 3.14.3.4-1. Integral abutment with attached approach slab.

- *Semi-integral abutment with sliding bearings* (Figure 3.14.3.4-2)—Semi-integral bridges are defined as single- or multiple-span continuous bridges with rigid, nonintegral foundations and movement systems primarily composed of integral end diaphragms, compressible backfill, and movable bearings in a horizontal joint at the superstructure-abutment interface, which is applicable where superstructure loads are too heavy for small flexible piles, expansion movements are large, or foundation conditions do not permit flexing of supporting piles.



**Figure 3.14.3.4-2. Semi-integral abutment with attached approach slab.**

Joints should be eliminated whenever possible. There have been a number of studies on behavior of integral abutments in recent years, so in establishment of span arrangements for jointless bridge, one should refer to local owner-agency policy for guidance. Although no general agreement regarding a maximum safe-length for integral abutment and jointless bridges exists among the state DOTs, the study has shown that design practices followed by most DOTs are conservative and longer jointless bridges could be constructed.

A number of good design guides exist that address integral abutment design (1, 2, 27, 52, 53, 64, 84, 85, 86).

While superstructures with deck-end joints still predominate, the trend appears to be moving toward integral abutments.

### 3.15 Roadway/Structure Geometry Considerations

The overall economy of a structural steel bridge is highly dependent on the constraints placed on the structure by the roadway geometry. It is important that the bridge designer understand the basic principles of roadway geometrics to be able to suggest alternates that can have a significant impact on structure economy. The bridge designer should work with the roadway designer to the greatest extent possible to improve and simplify roadway alignments within the vicinity of bridges. The geometric issues outlined below (e.g., skew, curvature, merges/splits, superelevation transitions) can lead to significantly more complicated structure design or detailing, or both. These complications often result in higher design costs and higher fabrication/construction costs.

- Geometric improvements that might result in the simplification of structural steel framing and achieve a more *Eliminate/Reduce Skews*—Skews on steel bridges present complexities in design, fabrication, and erection that translate into increased cost. Skew angles should be eliminated or reduced wherever possible through a combination of alignment adjustments or span length increases, or both. While the span length increases would imply increased bridge cost, the simplification of details and erection should be investigated to establish a more realistic cost of structure rather than using cost per square foot as a decision-making basis. Contractors and detailers should be consulted in making these critical configuration decisions. Where skews cannot be avoided, guidelines for simplifying cross frame geometry are available from NSBA.
- *Eliminate/Reduce Curvature or Spirals, or Both*—The differences between the design requirements for straight bridges and curved bridges can be significant. In cases where the roadway approaching a bridge is curved, investigate the possibility of adjusting the P.I. location or radius of the curve to eliminate or minimize the length of the bridge which is curved, such that a simplified approach to evaluating curvature effects can be utilized and so that detailing and fabrication are simplified. In particular, spiral geometry is exceedingly complex with regard to steel girder detailing and fabrication and should be removed from the bridge if at all possible.
- *Eliminate/Reduce Merging/Splitting Geometry*—Where bifurcations are required on bridge structures, every effort should be made to locate these bifurcations at bridge pier locations to simplify framing requirements. The addition or deletion of girders to accommodate these changes within a span adds considerable complexity to the design, fabrication, and erection of the steel framing and to the overall cost of the bridge structure.
- *Eliminate/Minimize Effects of Superelevation and Superelevation Transition*—Superelevation transitions within spans complicate steel fabrication and deck construction. The cross-frames within the area of superelevation also vary in geometry, eliminating repetition of elements, and may result in additional cost. Transitioning from crown sections to superelevated sections while on the structure makes deck screeding extremely difficult for the contractor. Eliminating or minimizing the effects of superelevation transitions can also prevent local ponding or other drainage problems.
- *Eliminate/Reduce Width Transitions*—Width transitions result in variable girder spacing, which complicates the analysis and also increases the number of different cross-frames or diaphragms, increasing fabrication costs.
- *Locate Piers and Set Span Arrangement for Structural Efficiency*—When possible, locate piers such that the resulting span arrangement is optimized for structural efficiency. This leads to more economical superstructure designs and avoids problems with uplift, adverse dead load deflection behavior, and difficult construction.



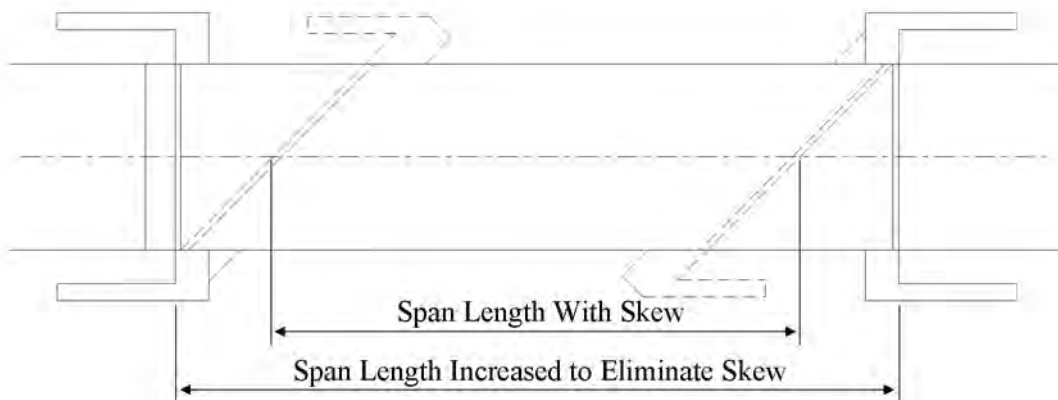
### 3.15.1 Suggestions to Simplify Structure Geometry in Curved and/or Skewed Bridges

There are situations where complex roadway geometry cannot be avoided. In those cases, the designer should make some effort to simplify the structural geometry. There is no comprehensive list of tips for simplifying geometry; every situation is unique and the possible scenarios are infinite. Some suggestions are provided below as examples.

**Use tangent girders for curved bridges:** In some cases where the horizontal roadway geometry includes a slight curve, it may be possible to use tangent girders in chorded framing arrangements, especially in simple-span situations. Check that the resulting minimum and maximum overhang dimensions are acceptable to the owner and are constructable. Using tangent girders can simplify the framing plan, potentially allowing for uniform girder lengths and uniform cross frame dimensions, and the elimination of curvature in the girders will reduce the fabrication cost of the girders.

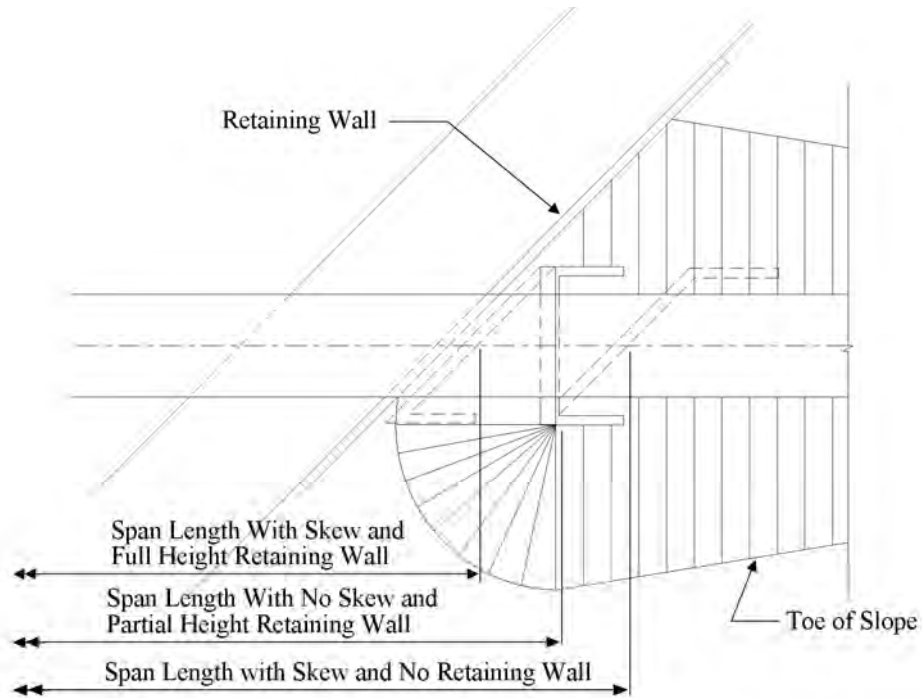
**Reduce or eliminate the effects of skew.** Suggestions include:

- **Lengthen Spans**—By increasing the span length of end spans, it may be possible to locate the abutments far enough from roadways below the structure to allow for the use of radial abutments while still maintaining adequate horizontal clearance. Designers should consider the cost of the longer span and increased total deck area of the bridge versus the cost premium associated with the complications of skew in the bridge (see Figure 3.15.1-1).

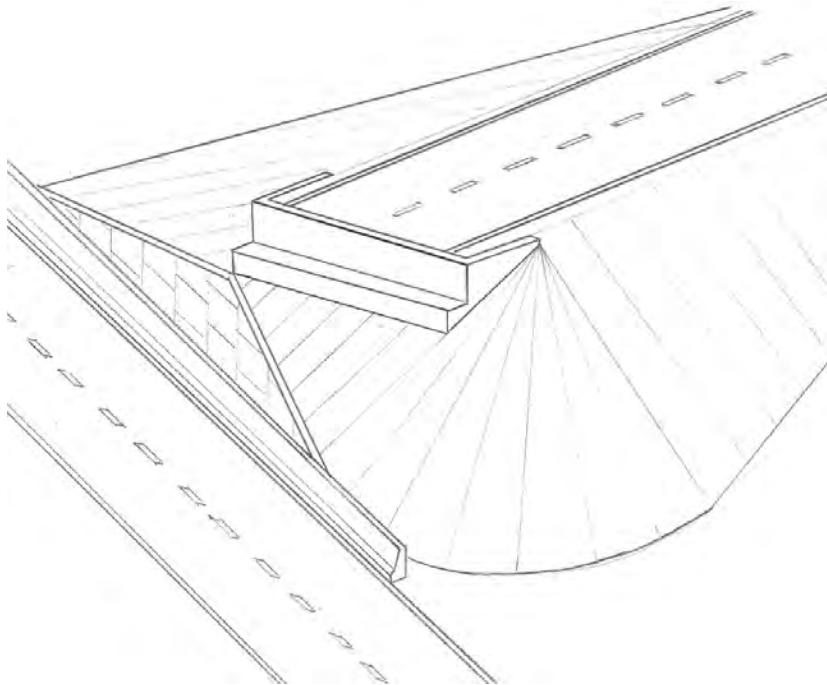


**Figure 3.15.1-1. Increasing span length can allow skew to be reduced at abutments while still maintaining adequate horizontal clearances.**

- **Use Retaining Walls**—In some cases, it may be feasible to use a retaining wall to allow the use of a radial abutment without the abutment header slope violating the horizontal clearance envelope of the roadway below. These walls typically are of variable height and require odd-shaped slope protection behind the wall. Designers should consider the cost of the wall versus the cost premium associated with the complications of skew in the bridge. In a project that already has extensive retaining walls, adding more wall to reduce skew in a bridge may cost very little compared to the cost savings in the bridge. But if the project otherwise uses no retaining walls, the cost of even a small retaining wall used to reduce bridge skew may be very high due to the need to mobilize a specialty subcontractor for a very small amount of work (see Figure 3.15.1-2, Figure 3.15.1-3, and Figure 3.15.1-4).



**Figure 3.15.1-2. Using a small retaining wall can allow skew to be reduced with little or no increase in span lengths.**



**Figure 3.15.1-3. Isometric sketch of the use of a small retaining wall to reduce skew without lengthening the span.**



**Figure 3.15.1-4.** A steel girder bridge where a small mechanically stabilized earth (MSE) retaining wall was used to reduce the severity of skew without significant increase in span length.

- *Use Integral Interior Bent Caps*—In cases where a traditional radial bent cap would have insufficient vertical clearance and where the vertical profile of the bridge cannot be raised, the typical solution is to skew the interior bent. However, it may be beneficial to use a radial bent cap and to maintain adequate vertical clearance by making the bent cap integral with the superstructure. See Figure 3.15.1-5 and Figure 3.15.1-6 for examples of steel and concrete integral caps with steel girders.



**Figure 3.15.1-5.** An integral steel pier cap used to avoid a conventional concrete rectangular cap (concrete drop cap). This allows the use of a radial cap while still maintaining adequate vertical clearances over traffic below the bridge. This is a single steel pier cap; some designers and some owner-agencies prefer using a double I-beam cap to address redundancy issues.



**Figure 3.15.1-6.** An integral post-tensioned concrete pier cap used in an interchange in Raleigh, NC to avoid a conventional concrete rectangular cap (concrete drop cap). This allows the use of a radial cap while still maintaining adequate vertical clearances over traffic below the bridge.

- *Use Dapped Girder Ends with Inverted-Tee Bent Caps*—At expansion joint locations where the girders are discontinuous, it may be feasible to utilize an inverted-tee bent with dapped end girders to allow the use of a radial bent cap that still maintains adequate vertical clearance. In some cases, the bottom soffit of the inverted-tee cap can be kept at the same depth as the bottom of the girders (see Figure 3.15.1-7). While the dapping of girder ends adds fabrication cost, the cost premium may be less than the cost of an integral bent cap.

If skew cannot be reduced or eliminated, then designers should consider the suggestions provided in other sections of this document for reducing the effects of skew and simplifying the design, detailing, and construction of the bridge, including:

- Using lean-on bracing or selectively omitting or softening diaphragms to reduce adverse transverse stiffness effects, or
- Using breakback or blister details to simplify deck, barrier rail, expansion joint, and abutment detailing (see 38 for descriptions of these details).



**Figure 3.15.1-7. Dapped ends on curved steel plate girders resting on an inverted-tee bent cap on a bridge in Texas. In this case, the dapped girder ends were used to match the depth of the precast girders in the next span; but this same concept could be used to allow use of a radial bent cap the same depth as the girders, providing an effect similar to that of an integral bent cap.**

### 3.15.2 Superelevation Effects on Loading

A truck or train load on a bridge structure with a horizontally curved roadway would experience a centrifugal radial force, which is a function of the vehicle weight and speed, and the roadway curvature, as prescribed by AASHTO or AREMA. Roadway superelevation, if present, would counteract the overturning effect of the radial force. No centrifugal force is applied to a highway lane load because of presumed large spacing between high-speed vehicles resulting in low traffic density.

The centrifugal force is expressed as a percentage of the live load without impact, as follows:

$$C = 0.00117S^2D = 6.68 \frac{S^2}{R} \quad (3.15.2-1)$$

where:

$C$  = centrifugal force as percent of live load with no impact

$S$  = vehicle speed mph

$D$  = degree of curvature

$R$  = radius of curvature in ft.

The centrifugal force is applied horizontally at 6 ft above the roadway surface for a highway bridge or at 8 ft above the top of rail for a railroad bridge. In either case, the vertical wheel loads near the outside of the bridge are increased, while those near the inside are decreased. Superelevation reduces the overturning effect of the centrifugal force. The mechanics of centrifugal force and superelevation are demonstrated through Figure 3.15.2-1 and corresponding equations:

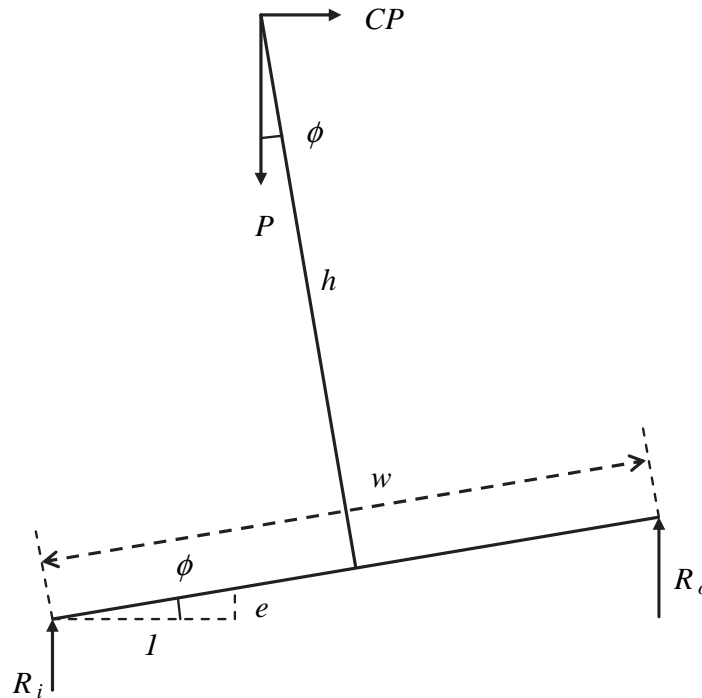


Figure 3.15.2-1. Centrifugal force and superelevation effects.

where:

$P$  = wheel load without impact

$C$  = centrifugal force factor

$CP$  = centrifugal force

$e$  = superelevation

$\phi$  = superelevation angle

$w$  = transverse distance between wheels, typically 6 ft for trucks or 5 ft for trains

$h$  = distance between centrifugal force application point and midpoint between wheels

$R_o$  = wheel load reaction near the outside of the roadway or its higher elevation

$R_i$  = wheel load reaction near the inside of the roadway or its lower elevation

Considering moment equilibrium about the inner wheel at the lower roadway elevation and re-arranging the various moment terms would result in the following expression for the outer wheel load:

$$R_o = P \left( \frac{1}{2} - \frac{h}{w} \tan \phi + C \frac{h}{w} + \frac{C}{2} \tan \phi \right) \quad (3.15.2-2)$$

Substitute  $e$  for  $\tan \phi$  for the typical case of a small angle:

$$R_o = P \left( \frac{1}{2} - \frac{h}{w} e + C \frac{h}{w} + \frac{C}{2} e \right) \quad (3.15.2-3)$$

Considering the equilibrium of vertical forces would result in the following expression for the inner wheel load:

$$R_i = P \left( \frac{1}{2} + \frac{h}{w} e - C \frac{h}{w} - \frac{C}{2} e \right) \quad (3.15.2-4)$$

These modified wheel loads are applied to the bridge deck and distributed to supporting bridge elements. The resulting force effects should be compared with the effects of unmodified wheel loads or with the load case of no centrifugal force.

### 3.16 Second-Order Effects

All methods of structural analysis are based on:

- 1) The fundamental satisfaction of equilibrium,
- 2) Compatibility of the structural deformations (or the kinematic assumptions on how the structure deflects), and
- 3) Constitutive relationships (force-deformation or stress-strain relations).

If the deflected position of the structure is considered in satisfying the equilibrium requirements, then the analysis is said to be a *second-order analysis*. The *AASHTO LRFD Bridge Design Specifications* (8) Articles 4.6.5 and 4.5.3.2 also use the phrase *large deflection theory* to refer to any method of analysis that considers second-order effects. Second-order effects are additional internal forces and additional deformations and deflections due to these forces, resulting from the applied loads acting through the deformations of the structure under the applied loads. It is common to refer to a structural analysis that considers second-order effects as a *stability analysis*, as in *AASHTO LRFD Bridge Design Specifications* (8) Article 4.5.3.2.

#### 3.16.1 Second-Order Analysis Methods

Generally, there are two types of second-order analyses that can be conducted: a buckling analysis or a second-order load-deflection analysis:

- 1) In a buckling analysis, the engineer solves for the load level at which the structure would *bifurcate* from its initial geometry into a buckled (bent) configuration. This type of analysis involves (either implicitly or explicitly) the solution for eigenvalues (buckling load levels) and eigenvectors (buckling modes). Effective length factors for stability design of columns, beams, and beam-columns are based on an eigenvalue buckling analysis.
- 2) The other type of second-order analysis is one in which the influence of second-order effects on the overall load-deflection response is inherently tracked at any given load level. That is, for any structural component subjected to an axial compression ( $P$ ), any bending deformations leading to relative transverse displacements ( $\delta$  or  $\Delta$ ) result in additional internal moments in the structure, equal to  $P\delta$  or  $P\Delta$ . These additional internal moments cause additional bending deformations of the structure, which in turn cause additional  $P\delta$  or  $P\Delta$  moments.

*AASHTO LRFD Bridge Design Specifications* (8) Articles C4.1, C4.5.3.2.1, and C6.10.1.6 refer to *second-order analysis* methods as a *geometric nonlinear analysis*. The term geometric nonlinear analysis comes from the fact that when second-order effects are included in the response, the structure depends on the changes in its geometry under load. Because of these changes in the geometry, the structural displacements and the internal forces generally are no longer proportional to the applied loads. That is, the load-displacement relationship is nonlinear.

There are two types of eigenvalue buckling analysis that can be conducted:

- 1) A linear buckling analysis and
- 2) A nonlinear buckling analysis.

In a linear buckling analysis, the influence of prebuckling displacements is neglected and the solution for the load at which the structure bifurcates from its initial undeflected geometry to a bent (buckled) geometry is sought. Conversely, in a nonlinear buckling analysis, the influence of prebuckling displacements is considered, and the solution for the bifurcation from the loaded geometry, at which point the structural stiffness matrix becomes singular, is sought. The reader is referred to Cook (41), Chapter 17, for a detailed summary of the concepts for this type of buckling analysis.

### 3.16.2 Analysis Assumptions versus Observed Behavior

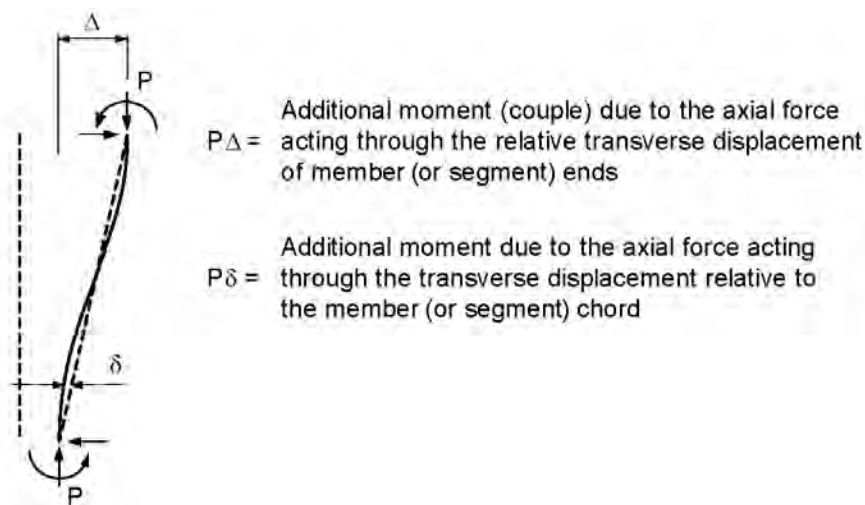
Strictly speaking, there are no structural components that actually bifurcate or buckle from a perfectly straight configuration into a bent configuration. Therefore, a buckling analysis is simply a theoretical solution applicable to columns, beams, or plates that are ideally straight or ideally flat in their initial unloaded geometry. Due to unavoidable geometric imperfections, these types of members will always exhibit a load-deflection response in which the component bends under increasing applied load. However, for components that are very close to being straight or flat in their initial unloaded geometry, the load-deflection response tends to be a close representation of the eigenvalue buckling response up until a stage in which the lateral deflections become sufficiently large that small angle approximations, such as  $\sin \theta \cong \theta$  (where  $\theta$  is a general bending rotation), are no longer valid.

### 3.16.3 Application

Second-order effects are typically significant in columns and beam-columns subject to large axial forces. As such, they are predominantly discussed in the context of these types of members. Figure 3.16.3-1 shows a common description of two types of second-order effects in column and beam-column members, the so called  $P$ -small delta ( $P$ - $\delta$ ) and  $P$ -large delta ( $P$ - $\Delta$ ) effects. The  $P$ - $\Delta$  effect refers to the couple caused by the member axial forces acting through the relative lateral displacement between the member ends. Conversely, the  $P$ - $\delta$  effect involves the effect of the member axial forces acting through the bending deflections of a member relative to a chord drawn between the member ends. The  $P$ - $\Delta$  effect occurs even in an ideally pin-ended member, although when a pinned-ended member is part of a triangulated truss framing system, the overall structural stiffness of the system can be large enough such that this effect is relatively minor. The  $P$ - $\delta$  effect occurs only when members are subjected to bending between their ends.

These second-order effects can lead to structural instability if the applied loads cause additional deformations—which cause additional second-order internal forces, leading to supplementary deformations—and the structure continues to deflect under increasing second-order internal forces. Broadly speaking, the structure is inherently stable if the additional internal moments and the corresponding additional transverse displacements are relatively small. Structural instability is generally defined as a state at which, due to the second-order effects, small changes in applied load or small variations from the ideal geometry (geometric imperfections) result in large changes to the overall equilibrium position due to the second-order effects.





**Figure 3.16.3-1.  $P\Delta$  and  $P\delta$  effects in beam-columns (17).**

The resistance equations for columns and beam-columns in the *AASHTO LRFD Bridge Design Specifications* (8) are generally based on inputting the maximum second-order internal axial force and moment determined from a second-order structural analysis of the elastic, ideally geometrically perfect, structure. Hence, consideration of geometric imperfections in the structural analysis is not required. This is contrary to the statement in *AASHTO LRFD Bridge Design Specifications* (8) Article 4.5.3.2.1 that “The effect of ... out-of-straightness of components shall be included in stability analyses and large deflection analyses.” For columns or beam-columns that are loaded predominantly in a single plane of bending, no out-of-plane moments are considered in the member strength checks. In this case, the design for overall stability of the members and the structural system includes (explicitly) only the in-plane second-order internal forces. Generally, member design also includes the use of effective length ( $K$ ) factors, which, when combined with member strength equations, accounts for geometric imperfection effects. The *AASHTO LRFD Bridge Design Specifications* (8) also use this approach for the design of arch ribs. In-depth discussions of these considerations are available from the references (50, 88).

Note that a first-order elastic analysis (also referred to as geometrically linear analysis or an analysis based on small-deflection theory) combined with moment amplifiers to capture the second-order effects is a second-order load-deflection analysis. The amplifier equations are simply an approximate way of solving for specific types of second-order load-deflection effects. The engineer generally must recognize when a given amplifier equation is applicable to capture the second-order response and when it is not.

### 3.16.3.1 Straight Steel Girders, Trusses, and Arches

When important, second-order effects for most steel girder, truss, and arch bridge designs, in their final constructed configuration, can be handled sufficiently via the calculation of member effective length factors (explicitly or implicitly from an eigenvalue buckling analysis) or via the second-order amplification of the internal moments via amplification factor equations. *AASHTO LRFD Bridge Design Specifications* (8) Articles 4.5.3.2.2b and 4.5.3.2.2c provide recommendations for calculation of moment amplification factors for beam-columns and arches.

### 3.16.3.2 Curved Steel Girders

When considering the influence of girder flange lateral bending due to horizontal curvature, torsional loads from overhang brackets, skew effects, wind effects, or a combination thereof, the *AASHTO*

*LRFD Bridge Design Specifications* (8) Chapter 6 resistance checks handle the flanges effectively as equivalent beam-columns. The flange major-axis bending stresses are analogous to the stresses due to beam-column axial load and the flange lateral bending stresses are analogous to the bending moments in a beam-column. As such, second-order amplification of the flange lateral bending moments in a girder compression flange generally must be considered. *AASHTO LRFD Bridge Design Specifications* (8) Article 6.10.1.6 provides an equation for approximating these amplification effects. Research studies have indicated that this equation tends to provide a conservative estimate of the true second-order amplification of the compression flange lateral bending stresses associated with lateral bending between the cross-frames. This equation does not address overall global second-order amplification effects, which can be important in some cases during construction.

*AASHTO LRFD Bridge Design Specifications* (8) Article C.4.6.1.2.1 indicates that:

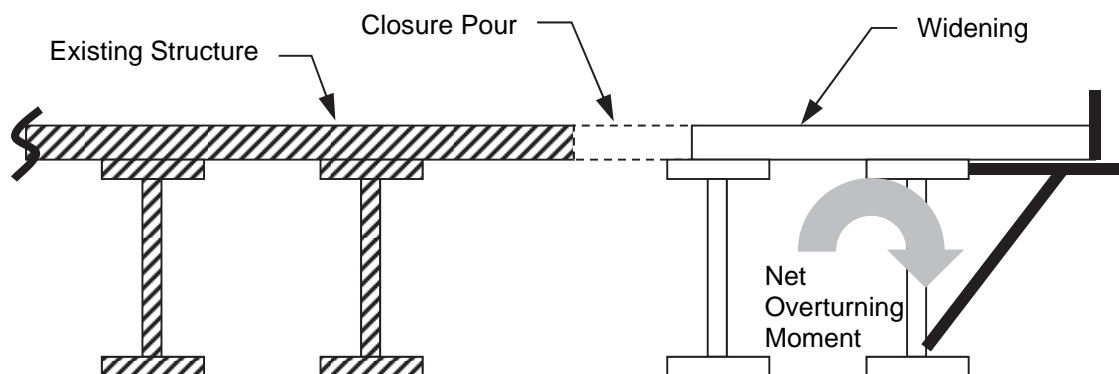
Small-deflection theory is adequate for the analysis of most curved-girder bridges. However, curved I-girders are prone to deflect laterally when the girders are insufficiently braced during erection. This behavior is not well recognized by small-deflection theory.

The corresponding errors are not necessarily due to large-deflection theory versus small-deflection theory. Rather, they are often due to the lack of representation of warping torsion in typical structural analysis models. However, much of the overall torsional resistance of curved girder bridges typically comes from the combination of the various member components as a structural system. As noted by the above statement (8), the predominant errors can occur during early stages of erection, when individual girders may not be sufficiently braced or tied together such that they function as an integral unit.

*AASHTO LRFD Bridge Design Specifications* (8) Article 4.6.1.2.4b states that “The effect of curvature on stability shall be considered for all curved I-girders.” The influence of curvature on the theoretical member bifurcation loads is generally small. However, second-order load deflection effects can be significant in some cases with curved I-girders, particularly during construction. The commentary to this article provides approximate equations for estimating the first-order flange lateral bending stresses in curved I-girders. These stresses must then generally be amplified prior to checking member design criteria.

### 3.16.3.3 *Narrow Steel Girder Systems*

In some cases involving relatively narrow bridge units, such as the erection of two- or three-girder cross sections for bridge widening, or when erecting or placing the deck concrete on narrow units during general intermediate stages of erection, it may be prudent to evaluate the overall global stability of the unit. Yura (95, 96) provides a number of tools for estimating the global eigenvalue buckling load of these types of I-girder systems. For curved I-girder systems, the designer may need to limit the load level to a small fraction of this buckling load to avoid excessive amplification of the lateral displacements within the bridge unit. Designers are cautioned to pay close attention to the specific loading applied to these narrow systems; eccentric loading resulting in global overturning moments can be a common intermediate loading condition and during the intermediate state the narrow two- or three-girder system may be subject to stability issues. For example, consider the two-girder widening in Figure 3.16.3.3-1. The deck overhang on the right represents an eccentric loading causing a global overturning moment. Meanwhile, due to the use of a closure pour, there is no counterbalancing overturning moment on the left side. Often in cases such as these, the intermediate cross-frames are detailed to be installed after deck placement (or with significantly oversized slotted connections) to allow for non-composite loading and deflection of the widening to occur independent of the existing structure. Such cases may be subject to stability problems; cases have been reported in the field where what appear to be simple bridge widenings undergo severe over-rotation during deck placement.



**Figure 3.16.3.3-1. Narrow steel girder system that may be subject to instability problems.**

#### 3.16.3.4 Global Second-Order Amplification

In certain situations, steel I-girder bridges can be vulnerable to overall (i.e., global) stability related failures during their construction. The non-composite dead loads must be resisted predominantly by the steel structure prior to hardening of the concrete deck. Yura, et al. (95) shows that relatively narrow I-girder bridge units (i.e., units with large span-to-width ratios) may be susceptible to global stability problems rather than cross-section or individual unbraced length strength limit states.

Furthermore, due to second-order lateral-torsional amplification of the displacements and stresses, the limit of the structural resistance may be reached well before the theoretical elastic buckling load. Therefore, in curved and/or skewed bridge structures sensitive to second-order effects, simply ensuring that the loads for a given configuration are below an estimated global elastic buckling load is not sufficient. Large displacement amplifications can make it difficult to predict and control the structure's geometry during construction well before the theoretical elastic buckling load is reached.

Possible situations with the above characteristics include widening projects on existing bridges, pedestrian bridges with twin girders, phased construction involving narrow units, and erection stages where only a few girders of a bridge unit are in place. In all of these cases, the problem unit is relatively long and narrow.

White et al. (88) recommend a simple method that can be used to alert the engineer to undesired response amplifications due to global second-order effects. The linear response prediction obtained from any of the first-order analyses can be multiplied by the following amplification factor:

$$AF_G = \frac{1}{1 - \frac{M_{\max G}}{M_{crG}}} \quad (3.16.3.4-1)$$

where  $M_{\max G}$  is the maximum total moment supported by the bridge unit for the loading under consideration, equal to the sum of all the girder moments, and

$$M_{crG} = C_b \frac{\pi^2 sE}{L_s^2} \sqrt{I_{ye} I_x} \quad (3.16.3.4-2)$$

is the elastic global buckling moment of the bridge unit, per Yura et al. (95). In the equation,  $C_b$  is the moment gradient modification factor applied to the full bridge cross-section moment diagram,  $s$  is the spacing between the two outside girders of the unit,  $E$  is the modulus of elasticity of steel, and

$$I_{ye} = I_{yc} + \frac{b}{c} I_{yt} \quad (3.16.3.4-3)$$

is the effective moment of inertia of the individual I-girders about their weak axis, where  $I_{yc}$  and  $I_{yt}$  are the moments of inertia of the compression and tension flanges about the weak-axis of the girder cross-section respectively,  $b$  and  $c$  are the distances from the mid-thickness of the tension and compression flanges to the centroidal axis of the cross-section, and  $I_x$  is the moment of inertia of the individual girders about their major-axis of bending (i.e., the moment of inertia of a single girder).

Yura et al. (95) developed this formulation considering multiple girder systems with up to four girders in the cross-section of the bridge unit. The individual girders were assumed to be prismatic and all the girders were assumed to have the same cross-section. The engineer must exercise judgment in applying this equation to general I-girder bridge units with stepped or other non-prismatic cross-sections, as well as cases where the different I-girders have different cross-sections.

In addition to providing an estimate of the second-order effects on the overall girder displacements, the amplification equation also can be used to predict potential increases in the girder stresses. Hence, to address potential second-order amplification concerns with narrow structural units, the results of an approximate 1D or 2D analysis should be amplified, using the amplification equation prior to conducting the constructability checks required by AASHTO LRFD Article 6.10.3 (8). The limit states in Article 6.10.3 are:

- Nominal initial yielding due to combined major-axis bending and flange lateral bending,
- Strength under combined major-axis and flange lateral bending,
- Bend buckling or shear buckling of the girder webs,
- Reaching a flange lateral bending stress of  $0.6F_y$ , and
- Reaching the factored tensile modulus of rupture of the concrete deck in regions not adequately reinforced to control the concrete crack size.

Section 2.9 of the NCHRP 725 Report, Appendix C (88), provides a detailed example showing the results of these calculations for an example narrow bridge unit that experienced construction difficulties (over-rotation of the bridge cross-section) during the deck placement.

Research by White et al. (88) suggests that the amplification equation should be used to detect possible large response amplifications during preliminary construction engineering. If the amplifier shows that a structure will exhibit significant nonlinear behavior during the deck placement, then in many cases, the scheme adopted for the construction should be revisited. In these cases, by conducting a detailed 3D FEA of the suspect stages, one may often find that the physical second-order amplification is somewhat smaller than predicted by the above simple estimate. If the second-order amplification is still relatively large in the more refined model, one should consider reducing the system response amplification by providing shoring or by bracing off of adjacent units. If  $AF_G$  from the amplification equation is less than approximately 1.1, it is recommended that the influence of global second-order effects may be neglected.

If it is found necessary to construct a structure that has potentially large response amplification during the deck placement, the engineer should perform a final detailed check of the suspect stages using a second-order (geometric nonlinear) 3D FEA. (It is recommended that this scenario with an  $AF_G$  larger than approximately 1.25 should be considered as requiring an accurate second-order 3D FEA.) In addition, it will be necessary to ensure that the deck placement does not deviate from the assumptions of the analysis in any way that would increase the second-order effects. Obviously, in most cases, it is best to stay away from these issues.

Substantial second-order effects during the steel erection may be a concern in some situations; however, particularly during the earliest stages of the steel erection, if the steel stresses are small and if the influence of the displacements on fit-up is not a factor, large second-order amplification of the deformations typically does not present a problem.

Steel tub girders generally have as much as 100 to more than 1000 times the torsional stiffness of a comparable I-girder section. Therefore, when steel tub girders are fabricated with proper internal cross-frames to restrain their cross-section distortions as well as a proper top flange lateral bracing (TFLB) system, which acts as an effective top flange plate creating a pseudo-closed cross-section with the commensurate large torsional stiffness, lateral-torsional buckling is rarely a concern. Furthermore, second-order amplification in bridge tub girders is rarely of any significance even during lifting operations and early stages of the steel erection. However, overturning stability of curved tub girders, or tub girder bridge units, can be a significant issue if it is not properly identified and addressed. White et al. (88) briefly discuss overturning stability considerations and provide a parameter that may be useful for estimating when overturning may be an issue. However, in any situation where there is potential for overturning instability, it should be explicitly checked in detail.

#### 3.16.3.5 *Second-Order Amplification of Flange Lateral Bending between Cross-Frames*

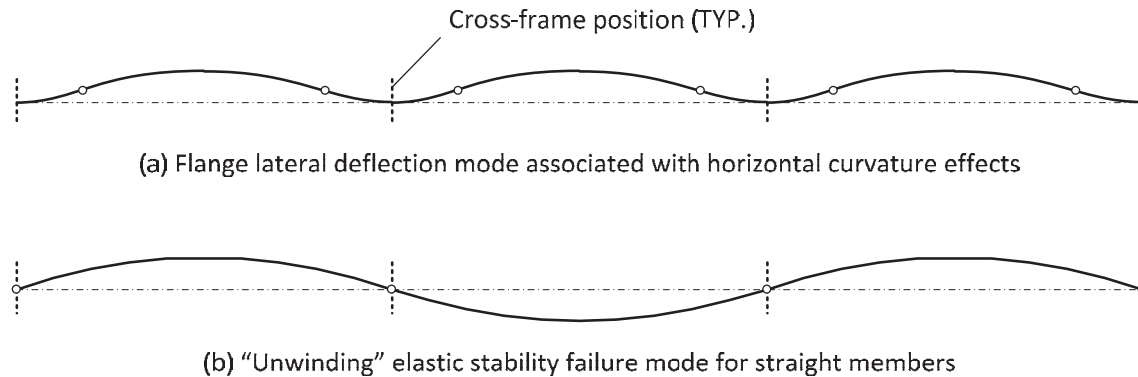
Design-analysis compression flange lateral bending estimates usually are based on a first-order analysis. They do not consider any potential amplification of the bending between cross frame locations due to second-order effects. That is, they do not consider equilibrium on the deflected geometry of the structure in the evaluation of the stresses. The corresponding “local” second-order flange lateral bending stresses (local to a given unbraced length between cross-frames) can be estimated by multiplying the first order  $f_c$  values by the following amplification factor discussed in Article 6.10.1.6 of the AASHTO LRFD Specifications:

$$AF = \frac{0.85}{1 - f_b / F_{cr}} \geq 1.0 \quad (3.16.3.5-1)$$

where  $F_{cr}$  is the elastic lateral-torsional buckling stress for the compression flange, based on the unbraced length  $L_b$  between the cross-frames, and  $f_b$  is the maximum major-axis bending stress in the compression flange within the targeted unbraced length. It should be noted that when the equation gives a value less than 1.0,  $AF$  must be taken equal to 1.0; in this case, the second-order amplification of the flange lateral bending is considered negligible.

When determining the amplification of  $f_c$  in horizontally curved I-girders, White et al. (91) indicate that for girders with  $L_b/R > 0.05$ ,  $F_{cr}$  may be determined using  $KL_b = 0.5L_b$ . For girders with  $L_b/R < 0.05$ , they recommend using the actual unsupported length  $L_b$ . The use of  $KL_b = 0.5L_b$  for  $L_b/R < 0.05$  gives a better estimate of the amplification of the bending deformations associated with the approximate symmetry boundary conditions for the flange lateral bending at the intermediate cross frame locations, and assumes that an unwinding stability failure of the compression flange is unlikely for this magnitude of the girder horizontal curvature. Figure 3.16.3.5-1 illustrates the flange lateral

deflections associated with the horizontal curvature effects as well as the unwinding stability failure mode for a straight elastic member.



**Figure 3.16.3.5-1. Second-order elastic deflection of a horizontally-curved flange versus the unwinding stability failure mode of the compression flange in a straight member.**

### 3.17 Phased Construction, Redecking, and Widening

Analysis of structures during phased construction, redecking, and widening has several components that should be considered to ensure that the structural behavior is consistent with the applied modeling approach. The constructibility of a structure should be considered in every design. In many cases, stresses or deflections can be controlled by construction loading. The sequence of construction may result in cumulative loading effects resulting in locked-in superstructure stresses. The designer should consider the effects of permanent dead load deflections, transient live load deflections, stability of the partial and completed structure, and cross frame/diaphragm detailing.

#### 3.17.1 Consideration of Permanent (Dead Load) Deflections

Permanent dead load deflections should be carefully considered during the analysis of a superstructure that will have a longitudinal joint in the deck slab. The analysis could represent a new structure or an existing structure that has a longitudinal construction joint due to the width of the structure, complex superelevation geometry, or the need to build the structure with a part-width method of construction. The analysis could also represent an existing structure that is being widened with minimal or no removal of the existing deck slab. Another possible scenario could be a new structure or an existing structure that has an open longitudinal joint in the deck slab due to the width of the structure but has steel superstructure framing that is connected with diaphragms or cross-frames across the full width of the structure. Regardless, consideration of the permanent dead load deflections is important to ensure the constructibility of the structure. The permanent dead load deflections will impact the girder camber diagram, deck slab haunch detailing, and the cross frame/diaphragm detailing.

The analysis of a structure with a longitudinal construction joint in the deck slab should first consider the number and sequence of transverse deck placements. The number and sequence of deck placement regions may be influenced by the maintenance and protection of traffic scheme, width of the structure, skew of the structure, number and location of superelevation break-overs, and other design or construction considerations. Typically, the use of two transverse deck placements is the most common approach if partial width construction (phased construction) is required. However, there may be benefits in providing a third transverse deck placement as a closure placement to minimize the impacts of differential deflections. The analysis will also need to consider the type of

cross-frames/diaphragms and the type of connections that are specified. For horizontally curved structures, the analysis will have to consider the transverse sequence of the deck placements to account for the load shedding. Stability of the framing system will be an important consideration in selecting the placement sequence.

The analysis of widened structures should consider the impact of girder deflections and girder rebound of the new and existing girders. These deflections can be a function of both the non-composite and long-term composite section properties. The wet concrete deflections are a function of the non-composite section modulus of the new superstructure components and may be a function of the long-term section modulus of existing girders. Widened structures will typically require the removal of a portion of the existing deck slab, barriers, railings, utilities, or a combination thereof. This necessitates the consideration of the rebound of the existing girders adjacent to the widening. Consideration should be given to which section properties are appropriate to estimate the rebound of the existing structure. The removal or addition of the tributary dead load can be a function of the non-composite section modulus, the long-term composite section modulus, or somewhere in between. It is the engineer's responsibility to evaluate the appropriate design approach to accurately estimate the displacements. The anticipated deflection or rebound, or both, of the existing girders and the camber of the new girders should be reported in the contract documents.

Depending on stage widths, girder spacing, temporary deck overhang dimensions, and other parameters that influence the behavior of the structure, the loads imparted by temporary barriers can potentially complicate the determination of girder deflections. Furthermore, these temporary barrier loads can be, in some cases, quite eccentric to the center of gravity of the stage bridge section. This eccentric loading effect needs to be accounted for when tabulating final deflections for the purposes of girder camber. It is agreed that existing girders can theoretically have the potential to rebound when deck and barrier loads are partially removed to facilitate a widening. However, span arrangement, degree of skew, girder spacing, diaphragm stiffness, and other characteristics play a role in whether this rebound can really take place. Designers are encouraged to obtain input from experienced engineers before assuming any rebound will occur.

### **3.17.2 Consideration of Transient (Live Load) Deflections during Phased Construction**

Transient live load deflections should be considered during the analysis of a superstructure that will have a longitudinal joint in the deck slab. This is primarily a concern when a structure is widened or when a deck slab is placed under part-width construction with live load in the completed portion of the structure. The transient live load deflections will impact the cross frame design and detailing. The live load deflections and vibrations may impact the quality of the deck slab finish during the deck placement and curing process. Allowing live loads on the structure during deck slab placement and curing can result in an uneven finish and cracking. However, when live loads must be maintained on a structure, the engineer should consider the design implications of this loading condition.

Analysis of the superstructure for live load during the interim phases of construction can be performed in a manner similar to that followed for the final analysis of the completed structure. The phased construction live load lane positions and temporary barriers should be considered to ensure that the temporary construction condition does not control the superstructure design. For typical straight multi-girder systems, the live loads will be supported completely by the girders that are composite with the deck slab. For significantly skewed or horizontally curved girder systems, the effects of live load may be present in the portion of the superstructure that is not yet composite with the deck slab due to load shedding. Consideration should be given to the cross frame/diaphragm detailing with respect to live load deflections. Cross frame/diaphragm connections can either be detailed with standard round holes or slotted holes in the closure pour bay. The connection detailing will determine whether girders adjacent to the composite superstructure will contribute to supporting the live loads and temporary barriers used during phased construction. For girder-floor beam

structures (two girders), the phased construction live load and the temporary barrier loads will be resisted by both girders. One girder will have a non-composite section modulus, while the other will have short-term composite section modulus. The floor beams must also be checked to verify that the non-composite floor beams have sufficient capacity to support the phased construction force effects.

### **3.17.3 Stability of Narrow Sections during Interim Phases of Construction**

During girder erection or deck placement, or both, there are commonly times when only part of the structural steel system is in place, resulting in a narrow section. The analysis of narrow sections should be evaluated with consideration to the overall stability of the superstructure framing system, uplift conditions at supports, wind overturning effects, phased construction loading, temporary traffic control loads, skew effects, and horizontal curvature effects on dead and live loads. For this discussion, a narrow typical section is defined as a two- or three-girder cross section in the final fully erected condition. Consideration for using one- or two-girder sections that may exist during construction phasing should be evaluated during the development of the erection plan for similar conditions discussed herein.

The stability of the structure should be verified for all phases of construction and the final fully completed structure. The stability is a function of the span arrangement, support skew, horizontal curvature, bearing restraints, and live loading conditions on the structure. Uplift at the supports should be investigated during all phases of construction and in the final condition. The different loading conditions may influence the type and guide orientation of the bearings. Consideration to the guide orientation of curved structures can greatly improve the performance of narrow sections with respect to allowing the structure to behave in a manner that does not lock in forces that cause the structure to twist and cause an uplift situation. Careful span arrangement selection is the best way to eliminate the concern with uplift. The stability of horizontally curved structures should be investigated. See also Article 4.1.3 for further discussion of stability analysis.

### **3.17.4 Considerations for Diaphragms/Cross-Frames between New and Existing Sections**

Diaphragms and cross-frames between new and existing sections of phased or widened typical sections require special consideration when designing and detailing the superstructure. The type and configuration of the diaphragms/cross-frames and the timing of when the bracing is fully connected directly influences the analysis of the girders and bracing system. On straight, minimally skewed structures, the diaphragms/cross-frames are typically considered secondary members that do not participate in the distribution of superstructure dead and live loads. They are provided to brace the compression flange of the main girders and resist wind loads. On horizontally curved members or straight members with significant skews, cross frame members are considered primary members that will participate in the distribution of dead and live loads in addition to bracing the main girders and resisting wind loads. During phased construction, the designer should acknowledge that the cross-frames for all structures provide a restraining force that opposes differential deflections resulting from differential loading of the respective portions of the construction phasing typical section. Regardless of horizontal curvature or magnitude of skew, differential loading and the resulting cross frame forces and structural integrity of the framing system should be considered.

The type of diaphragm/cross frame connections will directly influence the design of the existing and new girders. This includes the consideration of structure stresses and differential girder deflections, both of which will impact the constructibility of the structure. There are two common detailing strategies that will influence the analysis of the structure. The first approach is to detail all connections with standard round holes at all connections. This is appropriate for all horizontally curved structures or significantly skewed structures. This will require that 2D or 3D analyses be employed at each construction stage to calculate the magnitude of diaphragm/cross frame forces due to the differential loading.



The second approach is to allow the adjacent portions of the superstructure to act independently until the adjacent deck slabs for the new and existing portions of the superstructure have cured. This minimizes the impact of differential loading and may allow opportunities to simplify the analysis with a standard line girder analysis. Once the deck slab cure has been achieved, the cross-frames between the new and existing girders are connected. At this time, a closure pour may be required. This approach minimizes locked-in stresses and eliminates the majority of the differential loading stresses that would be introduced in a fully connected framing system. This can be achieved by detailing standard slotted holes on the diaphragm/cross frame connection between the new portion of the superstructure and the existing girder. The remainder of the connections would use standard round holes. Alternately the diaphragm/cross-frame can be installed between the new and existing superstructure after the deck has cured on the new superstructure. This requires that the diaphragm/cross-frame holes for the existing girder is field drilled to assure a proper fit-up.

Regardless of the structure type, consideration of the loading configurations, differential deflections, and the impacts that these items have on the diaphragm/cross-frames should be evaluated. These items can have a significant influence on the level of sophistication in the analysis and the constructibility of the structure. Note that many owner-agencies have established policies regarding their preferred methods for addressing these situations; designers should be aware of these preferences and approach these situations appropriately.

### **3.18 Temperature Effects**

#### **3.18.1 Uniform Temperature Changes**

Typically, steel girder bridges are designed and detailed such that uniform temperature changes do not cause any loading to the superstructure elements. In tangent girder bridges, this is most commonly accomplished by means of bearings designed, detailed, and oriented to allow for free longitudinal movement of the superstructure under uniform temperature changes. Most often in a multiple-span tangent girder bridge, one bent is chosen as the fixed bent and one or more of the bearings at that bent are longitudinally restrained by means of anchor bolts or guide blocks so that the superstructure is prevented from moving longitudinally at that support; meanwhile, the other bents are detailed as free or expansion bents and the bearings at those bents are detailed with sliding surfaces (e.g., polytetrafluoroethylene (PTFE) sliding surface bearings) or with elastomeric bearings with no longitudinal restraints, thus allowing the superstructure the freedom to move longitudinally at those bents.

In curved girder bridges, the problem of providing for free thermal expansion/contraction movement becomes more complicated. Most designers in this situation orient the bearings to allow for movements along lines from the fixed center of no movement. See Article 3.14.2 for more discussion on this concept.

When fixed bearings are utilized, particularly on multiple bents, a proper analysis accounting for the transverse or longitudinal fixities (including bent stiffness) should be conducted. In such cases, a more refined analysis, such as a plate- and eccentric-beam grid analysis or a 3D FEM analysis may be warranted, since the restraint offered by the bearings is located at the bottom flange. See Article 3.14 for more discussion of the importance of correctly modeling boundary conditions for situations such as this.

#### **3.18.2 Temperature Gradients**

The effects of temperature gradients (different temperatures at the top of the superstructure versus the bottom of the superstructure) have often been neglected in the design of most routine steel girder bridges since the concrete deck provides shading which prevents most solar radiation from affecting the girders and causing such gradients.

Some nonstandard structures such as steel box beam monorail structures for rail or automated people mover applications may not have a concrete deck, however. For such structures, designers are advised to consider evaluating the effects of temperature gradients on the steel superstructure. These situations are typically addressed on a case-by-case basis. *AASHTO LRFD Bridge Design Specifications* (8) provide some limited guidance on this issue in Article 3.12.3.

### 3.18.3 Temperature Effects during Construction

One area where the effects of thermal gradients and uneven solar heating have caused problems with steel girder bridges in the past is during construction, prior to the placement of the concrete deck. In isolated cases, uneven solar heating has caused differential deflections or unanticipated twisting of steel girders, or both, during construction, resulting in misalignment of girders, fit-up problems, and other construction issues. A particularly good write-up of one such incident (Clearwater Bridge, the first steel/concrete composite box girder bridge in Florida) is provided by USS (82).

### 3.18.4 Wide and/or Highly Skewed Bridges

The case of wide or highly skewed bridges warrants some special discussion with regard to thermal movements. Designers are reminded that thermal expansion and contraction occur in both the transverse as well as the longitudinal directions. For most bridges, the ratio of length to width is such that the predominant direction of thermal movement is longitudinal and the effects of thermal movements in the transverse direction can be reasonably neglected. However, in the case of particularly wide bridges, designers should investigate the effects of thermal expansion and contraction particularly with regard to the restraint provided by bearings on the exterior girders. In wide bridges, it may be appropriate to only provide transverse restraint to select bearings near the centerline of the structure's cross-section. Providing transverse restraint in bearings supporting exterior girders may result in thermally induced stresses being introduced into the cross-frames and other framing elements since the steel superstructure elements will expand and contract at a different rate than the concrete bent cap.

Also, designers should be aware that in severely skewed bridges, the effects of thermal expansion and contraction may be particularly problematic and worthy of study and analysis during design. In severely skewed bridges, the net thermal movements may not be aligned with the "normal" longitudinal orientation typically provided for guided bearings. In extreme cases, if these effects are not evaluated and properly accounted for, the results include 1) bearings that bind and are not free to move as intended, 2) unintended stresses in framing members or girders, or 3) deck cracking problems.

## 3.19 Analyzing Older Bridges

The analysis of existing bridges is addressed through bridge inspection, load rating, fatigue evaluation, and, where deemed necessary, material or load testing, or both. The procedures to be followed are prescribed in AASHTO's *The Manual for Bridge Evaluation* (12).

### 3.19.1 Bridge Inspections

In accordance with the National Bridge Inspection Standards (NBIS), routine bridge inspections are performed on a regular basis at prescribed intervals, usually every two years for typical bridges. Guidance for the various types of inspections is provided in AASHTO's *The Manual for Bridge Evaluation* (12). These inspections are visual in nature and do not include nondestructive testing of materials or other extraordinary measures to uncover latent defects. These visual inspections may provide the basis for further evaluation depending on the degree of degradation or distress noted in the report.

In-depth inspections are detailed, close-up inspections typically performed on specific members where degradation may not be easily detectable during the routine inspection and may include nondestructive field tests, material sampling, and load testing to provide the required information to assess the bridge's durability or capacity, or both. Guidance on material testing procedures is provided in AASHTO's *The Manual for Bridge Evaluation* (12). In-depth inspections are usually performed over longer intervals than routine inspections.

Special inspections are performed where a specific detail showing an increased degree of deterioration has come under increased scrutiny, either due to distress/failure of similar details or as a result of more recent research. These special inspections can be prescribed at any time and are usually utilized to monitor a specific deficiency or detail and may include testing or further analysis, or both, through the load rating process.

Damage inspections are a type of special inspection that is usually performed where structural damage has occurred due to environmental or human factors. These inspections document section degradation that is analyzed through the load rating process. Special tests may be required to determine the degree of material degradation that may have occurred due to the incident.

### 3.19.2 Load Rating

Load rating of existing structures is prescribed to provide an assessment of the load carrying capacity of the bridge. Guidance for load rating of existing bridges is provided in AASHTO's *The Manual for Bridge Evaluation* (12). Typically, approximate methods are first utilized to evaluate the structure capacity in order to keep the load rating effort at a minimum. If the load rating from the approximate methods does not equal or exceed the target capacity, refined methods of analysis are typically utilized to recognize more reasonable load distribution and other factors associated with defining greater accuracy of the analysis. The approximate and refined methods of analysis for each type of bridge superstructure are discussed elsewhere in this document. Where analysis indicates that a bridge requires posting, or is categorized as structurally deficient, nondestructive load testing of the bridge may be utilized to justify higher bridge capacity. Guidance on nondestructive load tests is provided in AASHTO's *The Manual for Bridge Evaluation* (12).

Also, designers should be careful to appropriately consider the actual remaining section properties of any damaged or deteriorated sections. See Article 3.2 for more discussion of section property modeling.

Some older bridges are non-composite, with no physical shear connectors. These various types of construction need to be addressed appropriately. Note that in some cases, bridges that were detailed as non-composite may exhibit some degree of composite behavior. Aktan, et al. (4) provide one discussion of this phenomenon; other authors have addressed this as well. Engineers are cautioned when counting on composite behavior in bridges without physical shear connectors (i.e., when counting on composite behavior based only on bond between the top flange and the deck); this type of composite behavior is not always reliable and, even if it is effective, once the bond strength is exceeded the composite connection between the flange and deck will slip with no warning and with no residual composite capacity.

If the construction history of the bridge is reliably known (i.e., if the construction history is documented via as-built drawings, construction records, first-hand knowledge, etc.), the load rating engineer may be able to give consideration to modeling the structure's construction history correctly in the load rating analysis. Typical items which might warrant consideration include:

- Deck placement sequence (versus placement of entire deck at once)
- Staged construction

- Widening
- Redeckings
- Rehabilitation that added additional structural elements or significantly reinforced existing elements

Each of these events might result in the establishment of various states of locked-in load or stress effects, which could potentially affect the remaining rated capacity of the structure to carry live load.

### 3.19.3 Fatigue Evaluation

Analysis of fatigue in existing bridges is not typically a component of the load rating of structures; however, fatigue can have a significant effect on the durability of a bridge. There are two types of fatigue that can affect the performance of an existing bridge: load-induced fatigue and distortion-induced fatigue. Guidance on these fatigue evaluations is provided in AASHTO's *The Manual for Bridge Evaluation* (12).

#### 3.19.3.1 Load-Induced Fatigue

Load-induced fatigue is the in-plane fatigue that occurs due to the repetitive variations of stress that may occur within a structure due to applied loads or environmental factors. Calculated stresses are compared to threshold limits for specific details to identify elements that may be subject to advanced deterioration. Typical threshold limits can be found in Section 6 of the *AASHTO LRFD Bridge Design Specifications* (8). The stresses utilized in the evaluation can be fine-tuned through actual field measurements, which allows reduction of the partial load factors utilized in the analysis, or through material testing, which determines toughness characteristics of undocumented steels for a better assessment of the susceptibility of a detail to crack initiation/propagation. Suspect details can either be remediated or be identified for evaluation during routine or special inspections.

#### 3.19.3.2 Distortion-Induced Fatigue

Distortion-induced fatigue is the fatigue that takes place due to out-of-plane distortion that occurs near unrestrained connection details. Identification of details susceptible to distortion-induced fatigue is performed through the evaluation of details shown in the structural steel shop drawings for the existing bridge. While distortion-induced fatigue usually occurs early in the life of structures, the suspect details, once identified, should be inspected for any signs of distortion or cracking no matter the age of the bridge. Suspect details can either be remediated or be identified for evaluation during routine or special inspections.

### 3.19.4 Bridges with Hinges

Many older bridges were designed and detailed to behave in a statically determinate manner in order to facilitate the use of more simplified analysis methods in the days before computer modeling was readily accessible to engineers. For example, many bridges used pin and hanger or other hinge details to force statically determinate behavior. These bridges can still be analyzed using simplified and often hand calculation-based methods, but only if the original assumptions regarding the behavior of the structure are still valid. A detailed field inspection and some level of field monitoring or measurements may be appropriate to determine if hinge details are still functioning properly or if they have seized up (frozen) due to corrosion, deterioration, accumulation of dirt and debris, or other condition problems. As always, the engineer should be sure that the analysis being performed truly does accurately represent the behavior of the structure.

### 3.19.5 Load Testing as Part of Analysis of Existing/Older Bridges

For significantly complex structures or for structures that have exhibited behavior not consistent with original design assumptions or expectations, it may be warranted to include a nondestructive load testing program as part of the overall analysis. These types of load testing programs typically consist of the application of known loads to the structure and careful measurement of deflections, strains, and other performance characteristics. The analysis model is then run with the same loads, and the calculated deflections, stresses, and other performance characteristics are compared to the field measured values. Good correlation between the field measured values and the values predicated by the analysis model generally suggests that the analysis model is providing a reasonably accurate representation of the structure's behavior. Poor correlation should be investigated to determine if it is the result of faulty field measurements, incorrect modeling assumptions, or both. Generally, a limited load testing program is sufficient to demonstrate the validity of the model, after which the model can be run for a number of other loads with greater confidence.

## 3.20 Discontinuities in Structures

### 3.20.1 Stress Concentrations—Need for Detailed Stress Analysis

When complex structural details or significant discontinuities occur in a steel structure, the need may arise for a detailed stress analysis. Detailed stress analysis may be defined as a finite element method analysis intended to quantify stress results that would not be captured with the conventional  $P/A$ ,  $M_c/I$ , or  $VQ/Ib$  type relations that rely on cross-sectional force resultants and geometric properties. Examples of motivations for pursuing detailed stress analysis include quantifying concentrations of stress at a discontinuity or documenting the resistance of a plate to local buckling concerns. One objective of new design should be the adoption of standard and proven details, obviating the need for such analyses. Fatigue-sensitive details represent examples of instances where complex stress distributions, critically important to structural performance, may occur. Through extensive laboratory testing, the behavior of common and preferred details has been categorized and codified in a way that these can be safely designed using basic stress results from global structural analysis, rather than requiring detailed stress analysis.

Still, the need to evaluate unique conditions in new structures, damaged or deteriorated conditions, adequacy of existing details, or other similar considerations does sometimes give rise to the need for an advanced linear or even a nonlinear stress analysis (e.g., Sivakumar, 78). It is important to recognize that limits on stresses given in codes will not always be compatible with the results of such analyses. For example, detailed analysis of a longitudinally loaded welded joint will reveal certain amplified hot spot stresses. These would not be directly comparable to, for example, the permissible fatigue stress ranges for such a detail. The permissible ranges relate to global, member-level stresses, with the local amplification effects having been embedded into the threshold permissible stress of a given detail category. Rather, a first-principles approach to analysis of stress demand will typically require a first-principles approach to material resistance. This is rarely included in the normal scope of work for most bridge engineering tasks.

When detailed stress analysis is undertaken, a software tool of more sophistication than a basic 3D FEM, or plate- and eccentric-beam analysis is normally warranted. Such analyses often require consideration of material nonlinearity, capturing local yielding and plastic flow, and mechanical phenomena such as constraint-induced prevention of yielding. Element types considerably more sophisticated than even the 8-noded brick can arise. The importance of boundary condition modeling, discussed in Article 3.14 and other parts of this document, applies to detailed stress analysis as well. The most advanced analysis tools will offer a substructuring capability, in which an entire span may be modeled with conventional plate and beam elements, with the subject detail of concern transitioned into elements and meshing appropriate to the desired stress analysis. The boundary

conditions of the stress analysis are thus satisfied implicitly via compatibility with the overall analysis of the span.

### 3.20.2 Access Openings

Openings are sometimes required to provide construction or inspection access, especially in the case of hollow closed members such as tub girders. Sizes and shapes can vary from small circular hand-holes affecting a relatively small fraction of member cross section to noncircular, human-scale hatches affecting a significant fraction of member cross section. Such an opening would typically not be manifested in the section properties of an analysis model used to obtain moments, shears, and axial forces on a member cross section. It is often necessary, however, to use those force resultants to estimate stresses based on the net section properties at such an opening.

In a section subjected to repetitive tensile stress, the discontinuity introduced by an access opening may be evaluated as a fatigue-sensitive detail. As a simple example, consider a circular opening cut from a bottom flange, tensile stress region of a tub girder. The fatigue provisions of the *AASHTO LRFD Bridge Design Specifications* (8) identify this as a Category D detail, to be evaluated using a net section stress range. In practice, this represents a relatively severe condition, and would likely lead to efforts to relocate such an opening to a region of permanent compressive stress (removing the fatigue concern) or to reinforce the cross section. Such reinforcement might entail additional steel area (lowering the net section stress range) and welded details (which with appropriate design can improve the detail category).

In sections subjected to compressive stress, access openings above a certain size can lead to concern for local stability. Consider again the example of an access hatch in the bottom flange of a tub girder, now in a compression region. If the opening consumes enough of the flange section, the material remaining at the edges can assume proportions similar to the outstanding leg of an I-girder flange. For such an opening extending longitudinally one or two times the width of the outstanding leg, it would be appropriate to evaluate the  $b/t$  ratio of the plate remaining and restrict it to a level expected to preclude local buckling concerns (e.g., the compactness criteria for I-girder flanges and the *AASHTO LRFD Bridge Design Specifications* (8) criteria for plate compression in general). Alternatively, as in the case of the tensile fatigue stress concern, local reinforcement of the edge by some arrangement of welded plates may be considered or required.

### 3.21 References to Benchmark Analysis Problems

There are a number of current and historical design examples and other benchmark analysis examples available. These offer many opportunities for learning and for independent verification of other analysis methods. Design examples offer comprehensive presentation of analysis and design calculations, often with significant accompanying commentary. A number of organizations publish design examples, including the National Steel Bridge Alliance (NSBA), which is currently completing a full rewrite of the *Steel Bridge Design Handbook*, including a number of design examples. The entire *Steel Bridge Design Handbook*, including the design examples, is available for download free of charge from the NSBA website (<http://www.aisc.org/contentNSBA.aspx?id=20074>). The National Highway Institute (NHI) offers a number of bridge design courses that usually include a full companion workbook with design examples and commentary (<http://www.nhi.fhwa.dot.gov/Home.aspx>). The National Cooperative Highway Research Program (NCHRP), a branch of the Transportation Research Board (TRB) of the National Academies, also publishes reports that include benchmark analysis problems or design examples, most of which are free for download from their website (<http://www.trb.org/nchrp/public/nchrp.aspx>). Key among these is the NCHRP Report 725 (88), which includes a number of benchmark analysis problems.

## 4 ANALYSIS GUIDELINES FOR SPECIFIC TYPES OF STEEL GIRDER BRIDGES

### 4.1 Plate Girders—General Issues

#### 4.1.1 Cross Frame Modeling (Shear Stiffness, Flexural Stiffness)

Even though cross-frames are classified as secondary members in straight bridges, they perform a vital role in steel framing systems and make it possible for other members to perform as intended. Cross-frames provide stability by bracing compression flanges, helping to keep the section plane and maintain geometric integrity of the structure; control differential deflections; and offer a mechanism for load sharing between girders. In curved systems, cross-frames are primary load carrying members and essential components of the load path. To accurately predict the true behavior of a structure, the participation of these elements should be accounted for in the analysis.

Cross frame modeling is vastly dependent on the type of analysis performed. Line girder analyses do not warrant consideration of these elements, as they are simply defined as brace points along the compression flange. When 2D and 3D methods of analysis are employed, careful consideration as to the definition of these secondary members is crucial. These types of investigations account for the distribution and load sharing characteristics of cross-frames and will be the focus of this section.

##### *4.1.1.1 Two-Dimensional Analysis Techniques*

As previously discussed in this guide, two-dimensional modeling techniques lump the geometric properties of the steel framing into a series of nodes and beam elements lying in one plane. While this conversion is fairly straight forward for plate girder sections, the determination of properties becomes complex for cross-frames. Given that cross-frames are typically arranged in an X, K, or inverted K configuration, the behavior of several discrete members at different orientations and spatial locations needs to be reduced to equivalent properties that can be assigned to a single beam element. Article 3.11 presents two common methods for modeling the stiffness of cross-frames in a grid analysis—accounting for flexural stiffness or accounting for shear stiffness. While both approaches are viable, the designer should rationally consider how the structure will behave, whether shear or flexural stiffness will dominate, and choose a modeling approach accordingly. For instance, if a curved girder is analyzed, modeling the flexural stiffness of the cross-frames may be beneficial due to the twisting effects of the girders. On the other hand, if a tangent structure is investigated, modeling the shear stiffness of the cross-frames may be more beneficial as differential girder deflection is the mechanism inducing load in the cross-frames.

When performing traditional 2D grid analysis, consideration should be given to modeling the effects of the deck (for stages of analysis where the deck is in place and hardened) in the transverse direction using effect transverse deck strips. This is typically not required in plate- and eccentric-beam analysis or 3D analysis.

Several computer applications allow users to input geometric data of the bracing system, simplifying the computational effort of the designer. These modules should be used with care, however, and the user should have a complete understanding of the program's methodology before proceeding. When in doubt, there is always the option to manually input these parameters, thus providing tangible backup and complete understanding of the values used.

See Article 3.10.5 for discussion of the modeling of truss-type cross-frames when using 2D analysis methods. See Article 3.12 for discussion of the modeling of the torsional stiffness of I-girders when using 2D analysis methods.

#### 4.1.1.2 Three-Dimensional Analysis Techniques

An advantage of a three-dimensional analysis is the ability to define a framing system in  $x$ ,  $y$ , and  $z$  coordinates. This means a girder's components—top flange, web, and bottom flange—are physically defined at their appropriate geometric locations. Since the model now has depth, individual members comprising the cross-frames can be defined, along with the structural properties of each. Though cross frame member forces are part of the direct output, designers should be aware of the code checks listed. These models list axial load only and do not account for moments induced due to the eccentricity of connections. Special attention should be paid when single angles are used, as these shapes have complicated behavior when used in these applications, mandating additional capacity checks. Special attention should also be paid to the working lines of the cross frame members, as connection locations and member orientations can dramatically alter the results.

Even when 3D analysis is performed, designers are cautioned to consider the actual effectiveness of individual cross frame members. Highly slender members in very light cross-frames may be subject to elastic buckling and might not actually be effective in carrying significant compression loads and thus might not actually be effective as bracing members. The use of highly complicated nonlinear or iterative analyses is not encouraged; instead an awareness of this possibility and prudent detailing, analysis, and design measures should be used to achieve a sound design in a simple manner.

Many commercially available computer applications report cross frame member forces directly in the reported output. Care should be exercised when using this information, as it does not account for several factors that could result in member overstress. Such details that need to be accounted for are connection eccentricity and behavior specific to certain member shapes that can drastically reduce capacity.

#### 4.1.2 Lateral Bracing

The use of lateral bracing systems is no longer common, though in older versions of the AASHTO specifications lateral bracing systems were required for spans other than those that are now considered short spans. It is now recognized that the role of lateral bracing systems in resisting wind loads is negligible in the completed bridge. They still have a role in curved girders as part of a primary force resisting system for non-composite loads in particular, and for long straight bridges where they might be employed to stiffen the non-composite bridge against wind loads during construction.

The traditional analysis and design approach for lateral bracing systems has been to include the members in an analytical model as truss-type elements, thus being subjected to axial loads (tension or compression) only. Design of the members and their connections should, however, include the effects of eccentricity. Given the propensity to use WT and L shapes as lateral bracing members, the connection of the elements to a connecting gusset plate (the common case) automatically induces eccentricities in the connection that must be considered.

In a true 3D model of a structure (one in which the depth of the girders is discretely modeled), it is possible to model the location of the lateral bracing members correctly in the analysis, either at the intersection of the web and top flange or at the intersection of the web and bottom flange (whichever is a correct representation of the intended design details). The attachment of lateral bracing to the top flange is more common, but both systems have been used. When the lateral bracing is attached to the flanges, shortening/lengthening of the flanges under the stresses created by bending moments results in compatible deformations of the lateral bracing system. This is the cause of the simple axial strain force in the brace. At the same time, the lateral bracing force causes lateral and axial forces in the girder flanges. These behaviors are all completely captured in a 3D model.



In a grid-type model, the degrees of freedom (vertical translation and twist about two planar axes) are not sufficient to engage the lateral bracing system and the forces in the bracing system would be reported as zero. There is no mechanism to capture the brace forces in this type of model.

If the purpose of the lateral bracing system is simply to provide stiffness/stress control during wind loading on the non-composite bridge, either a grid or full 3D model can capture the wind load effects since the forces are in plane with the required deformations. The grid model should consist of line elements representing the lateral bending stiffness of the main girders, which are then connected with cross frame elements having the axial stiffness of the intended cross-frame, and the grid is triangulated with the lateral bracing layout. Wind loads are applied to the model and the lateral bracing forces are obtained. Presumably in this type of analysis, the main girders were designed using a line girder analysis, and the purpose of the independent wind load analysis was purely for the lateral bracing effects.

#### 4.1.3 Narrow Systems, Stability Analysis

In simple terms, narrow systems can be defined as those framing arrangements that possess large span-to-width ratios. Particular to steel plate girder structures, these systems are typically exemplified by a twin girder arrangement or by a layout consisting of only two girders in the cross section, though the definition can be expanded to include systems with more than two girders if girder lines are closely spaced. Potential situations with these characteristics include widening projects of existing bridges, pedestrian bridges with twin girders, phased construction, and erection stages where only a few girders of the bridge are in place.

Global stability is not explicitly addressed in the AASHTO specifications, which only consider buckling between cross-frames of individual girders; the global buckling of the girders acting as a system is not addressed. Hence, even if all code checks are compliant, stability of the overall structure can still be a concern. Global stability of these types of structures is primarily a concern during construction particularly during deck placement and before composite action has been achieved. In narrow curved and/or skewed I-girder bridge units, and/or narrow units that are subjected to torsional loads (for example, due to the deck placement), the global stability problem manifests itself in the amplification of the overall twisting of the unit. This results in increases in the girder layovers, as well as increases in the girder vertical displacements on the side of the unit where the vertical displacements—due to overall twisting of the unit—are additive with the girder major-axis bending displacements. The unit generally may exhibit excessive amplified displacements well before the structure reaches the global buckling load level. Therefore, it is important that these types of structures be kept well below their global buckling load level.

Global stability and second-order amplification in these types of structures can be evaluated by the use of amplification factors and, if necessary, second-order (nonlinear) 3D analysis. If stability is found to be an issue, the addition of upper and lower lateral bracing is an effective way to increase the torsional rigidity of the system. Full-length bracing may not be required and partial-length bracing has proved most economical in many cases, with bracing only required in bays closest to the supports. Increasing girder spacing is another way to enhance global stability, though it is not always a viable option due to geometry constraints.

In addition to buckling capacity, special attention must be paid to bearing arrangement. For structures with a small number of closely spaced girder lines, eccentric live loading and lateral loads can result in uplift at the bearings, leading to global instability of the structure. This condition should be checked and steps taken to resist uplift forces or, if possible, to increase girder spacing to eliminate uplift all together.

#### 4.1.4 Narrow Systems, Redundancy Analysis

Redundancy is the quality of a bridge to perform as designed in a damaged state due to the presence of multiple load paths. Conversely, nonredundancy is the lack of alternate load paths, meaning the failure of a single primary load carrying member would result in the failure of the entire structure. Related to nonredundancy and nonredundant members are fracture critical members. Fracture critical members are those in axial tension or tension components of bending members whose failure would result in the failure of the structure. These elements are labeled as such on the contract drawings and are subject to more stringent design, testing, and inspection criteria than those that are part of a redundant system.

The definition of a narrow plate girder system varies slightly from that used in stability discussions when focusing on redundancy. Whereas the system could contain any number of closely spaced girders in stability discussions, twin girder systems alone constitute a narrow system in the context of redundancy. This is due to the fact that only two primary elements exist to transfer load. If one of these fails, the second would be unable to support the entire weight of the structure, resulting in collapse. Other elements of the bridge, particularly the deck, could be able to carry additional loads encountered due to a nonredundant member failure and prevent collapse, which has been seen in the past. This built-in redundancy is difficult to predict, however, and is not explicitly recognized in design. As such, for typical plate girder bridges, a minimum of three girders are required to provide alternate load paths and be considered system redundant.

When possible, alternate load paths should be included in the design. Though this is not always an option, special consideration is warranted during the design of nonredundant structures. Due to the criticality of the primary load carrying members, attention should be paid to fatigue and effort should be made to eliminate detrimental details when possible. Sophisticated analyses have been performed in the past with some effectiveness to determine if two-girder systems are truly nonredundant or not, accounting for membrane action of the deck and load shedding properties of secondary members. These analyses are rather grueling and are not suggested as part of a typical design to avoid the penalties associated with the use of nonredundant and fracture critical members.

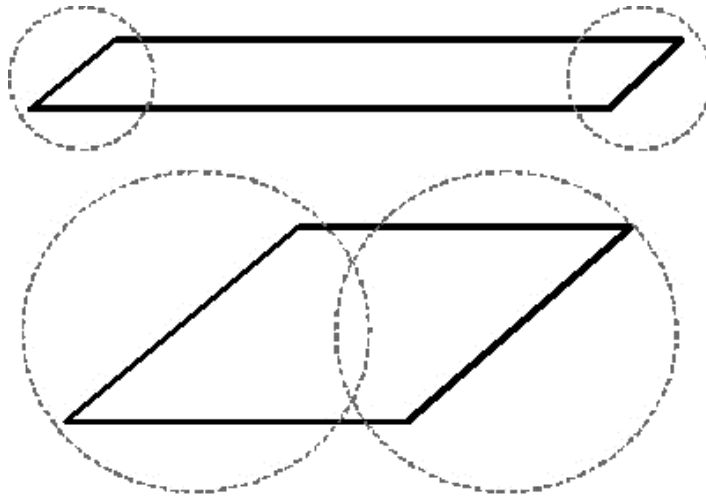
#### 4.1.5 Variable Depth Girders

Variable depth girders present some unique challenges to the analysis of steel girder bridges. In the global analysis model, the variation in stiffness of variable depth girders should be modeled carefully, considering all changes in moment of inertia, including the effects associated with the shifting of the neutral axis. If the stiffness of variable depth girders is not carefully modeled, dead load deflection predictions may be erroneous.

Variable depth girders are also subject to unique stress effects in the girder flanges and webs. These effects are addressed in the AASHTO specifications (8) and also in Blodgett (25).

#### 4.1.6 Width to Span Ratios and Influence on Secondary Effects

The ratio of bridge width to span length has an effect on the transverse stiffness response of steel girder bridges, particularly in skewed bridges. In general terms, secondary transverse load paths develop in skewed bridges as the cross-frames provide an alternate load path for gravity loads. The greater the ratio of bridge width to span length, the greater is the extent of these effects through the length of a span (see Figure 4.1.6-1).

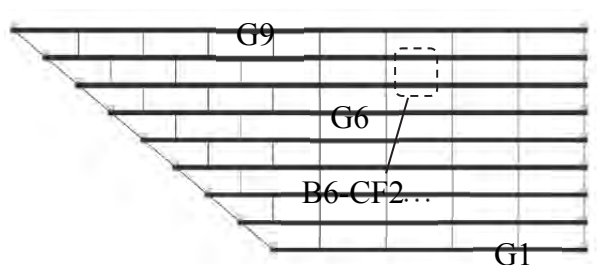


**Figure 4.1.6-1. Simplistic illustration of the concept of the effect of width to span ratio on transverse stiffness response in a skewed bridge. In this illustration, the circled areas will undergo more significant transverse stiffness response.**

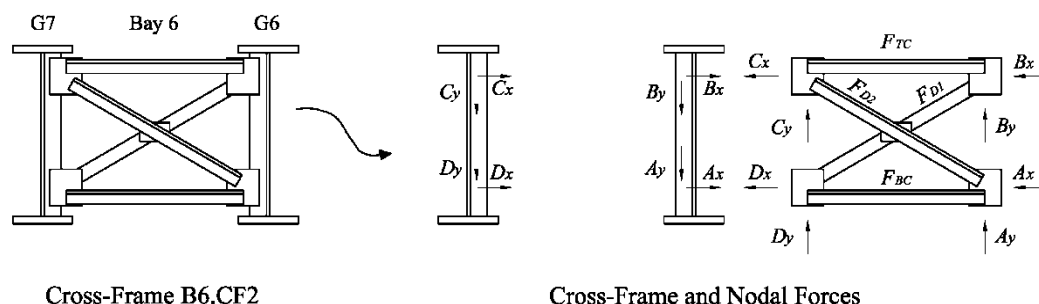
#### **4.1.7 Improved Calculation of I-Girder Flange Lateral Bending Stresses from 2D-Grid Analysis**

Even with the improvements proposed by White et al. (88), 2D analysis methods still fundamentally lack the ability to directly calculate flange lateral bending moments or stresses in I-girders. If the girders are curved and the bridge is radially supported, it is possible to estimate flange lateral bending moments using a simple formula associated with the V-Load method (73), which is also listed in the *AASHTO LRFD Bridge Design Specifications* as Eq. 4.6.1.2.4b-1 (8). However, this approach considers only the radius of curvature of the girder and thus is not applicable to straight girders and/or girders in bridges containing skewed bearing lines. Straight girders in skewed bridges also undergo flange lateral bending, and curved bridges with skewed supports have contributions to girder flange lateral bending both from horizontal curvature and from skew. Therefore, it is important to be able to estimate both effects. White et al. (88) propose a method to accomplish this, described below.

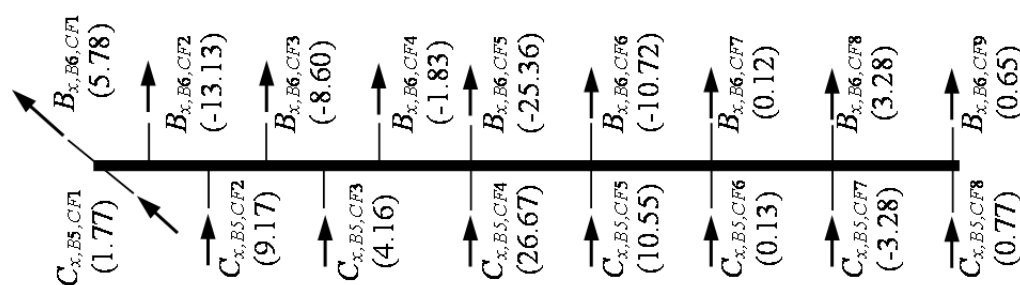
Figure 4.1.7-1a shows the plan of bridge NISS16 considered by White et al. (88). This is an 150 ft simple-span straight bridge with an 80 ft wide deck ( $w = 80$  ft.), a perpendicular distance between its fascia girders of 74 ft., and a skew of  $50^\circ$  at its left-hand abutment.



(a) Plan view of bridge NISS16



(b) Forces transferred from cross-frame B6-CF2 to girders G6 and G7



(c) Top flange of girder G6 subject to the horizontal components of the nodal forces

**Figure 4.1.7-1. Calculation of lateral bending stresses in the top flange of girder G6, in bridge NISS16 under total dead load (unfactored).**

Figure 4.1.7-1b illustrates the forces in cross-frame 2 (CF2) of Bay 6 in this structure, and the corresponding statically equivalent nodal horizontal and vertical forces transferred to the I-girders at the cross frame chord levels. These horizontal forces can be transformed to statically equivalent lateral forces applied at the flange levels of the I-girders by determining the couple associated with the horizontal forces, and then multiplying the chord-level couple forces by the ratio of the cross frame depth to the girder depth between the flange centroids,  $d_{CF}/h$ . In typical 2D-grid solutions,  $C_x = -D_x$  and  $B_x = -A_x$  in Figure 4.1.7-1b, and thus the forces shown in this figure are the couple forces.

Figure 4.1.7-1c shows the top flange forces applied to girder G6 in this bridge, determined from the improved 2D-grid method discussed in in Section 3.11.3, Section 3.11.4, and Section 3.12 of this document. The forces are still labeled “ $C_x$ ” and “ $B_x$ ,” for simplicity of the presentation. It should be noted that the chord level couple forces shown in Figure 4.1.7-1b are multiplied by  $(d_{CF}/h)$  to determine the flange level forces.

Given a general statical free-body diagram of a girder flange, such as the one shown for girder G6 in the figure, one would expect that the subsequent determination of the flange lateral bending stresses is an easy strength of materials calculation. In addition, if the girder is horizontally curved, the equivalent radial lateral loads corresponding to the horizontal curvature are also included in the free-body diagram. Furthermore, eccentric bracket loads from the overhangs can be included on fascia girders.

Unfortunately, the solution for the flange lateral bending stresses is not this simple. The problem is that the girder torsional stiffnesses, upon which the above calculation of the cross frame forces is based, include a contribution both from the girder warping torsion as well as the girder St. Venant torsion. As such, a portion of the above forces is transferred (by the interaction of the flange with the girder web) into the internal St. Venant torsion in the girders. More specifically, corresponding small but undetermined distributed lateral forces are transferred to the flange from the web in Figure 4.1.7-1c.

If the statical free-body diagram shown in Figure 4.1.7-1c is used to calculate the girder flange lateral bending stresses, slight errors accumulate as one moves along the girder length.

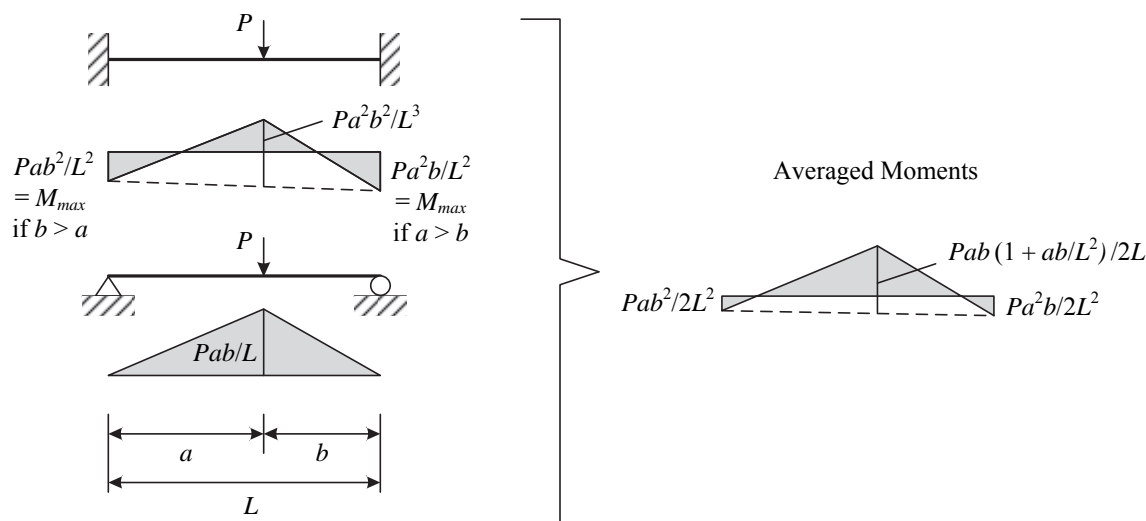
Solutions to this problem are as follows:

- Use the girder torsional rotations and displacements along with the detailed open-section thin-walled beam stiffness model associated with  $J_{eq}$  to directly determine the flange lateral bending stresses. This results in an imbalance in the flange lateral bending moments on each side of the intermediate cross-frames (since  $J_{eq}$  is based the assumption of warping fixity at the cross frame locations). This moment imbalance could be redistributed along the girder flange to determine accurate flange lateral bending moments. A procedure analogous to elastic moment distribution could be utilized for this calculation. Although this approach is a viable one, it is relatively complex.
- Focus on an approximate local calculation in the vicinity of each cross-frame, utilizing the forces delivered to the flanges from the cross-frames as shown in Figure 4.1.7-1c. Because of its relative simplicity, White et al. (88) proposed this approach, which is described further below.

Figure 4.17-2 illustrates the simplified approach adopted by White et al. (88) for calculating the I-girder flange lateral bending moments given the statically-equivalent lateral loads transferred at the flange level from the cross-frames. The calculation focuses on a given cross frame location and the unbraced lengths,  $a$  and  $b$ , on each side of this location. For simplicity of the discussion, only the force delivered from the cross frame under consideration is shown in the figure, and the cross-frame is assumed to be non-adjacent to a simply-supported end of the girder. In general, the equivalent lateral forces from horizontal curvature effects and/or the forces from eccentric bracket loads on fascia girders also would be included. Two flange lateral bending moment diagrams are calculated as shown in the figure, one based on simply-supported end conditions and one based on fixed end conditions at the opposite ends of the unbraced lengths. For unbraced lengths adjacent to simply-supported girder ends, similar moment diagrams are calculated; however, the boundary conditions are always pinned at the simply-supported end. The cross-frame under consideration is located at the position of the load  $P$  in the sketches. In many situations, the moments at the position of the load are the controlling ones in the procedure specified below.

Given the moment diagrams for the above cases, White et al. (88) determined that an accurate or conservative solution for the flange lateral bending moments and stresses is obtained generally by:

1. Averaging the above moment diagrams, and
2. Taking the largest averaged internal moment in each of the unbraced lengths (as explained below) as the flange lateral bending moment for that length.



**Figure 4.1.7-2. Lateral bending moment,  $M_l$ , in a flange segment under simply-supported and fixed-end conditions.**

This solution is repeated cross frame location by cross frame location along the length of the girders. The largest moment from the two solutions obtained for each unbraced length is taken as the estimate of the flange lateral bending moment in that unbraced length. (For the unbraced lengths at girder simply-supported ends, only one solution is performed.)

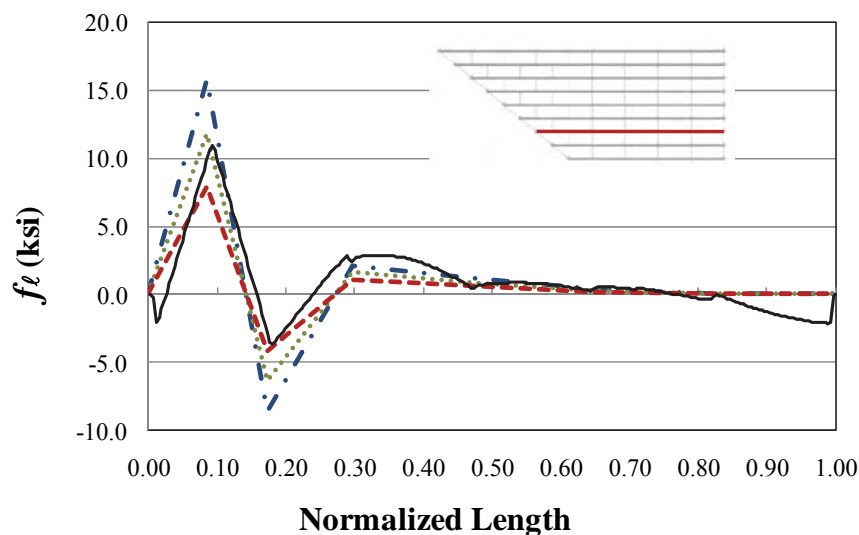
The above procedure recognizes that the true flange lateral bending moment is bounded by the “pinned-” and “fixed-ended” moment diagrams (neglecting the small St. Venant torsional contributions from the interaction with the web) and ensures that the flange lateral bending moments required for static equilibrium are never underestimated. For the girders in a curved radially-supported bridge without any deck overhang bracket loads, the procedure produces maximum flange lateral bending moments approximately equal to  $qL_b^2/12$ , where  $q$  is the radial equivalent flange load corresponding to the horizontal curvature (this is the same as the result estimated with the formula associated with the V-load method (73), also listed in the *AASHTO LRFD Bridge Design Specifications* as Eq. 4.6.1.2.4b-1 (8)). The V-Load method formula is considered to provide an applicable and acceptable approach for estimating girder flange lateral bending stresses in curved bridges with radial supports. For curved bridges, the above procedure effectively extends the V-load formula to include an estimate of the girder flange lateral bending effects from support skew.

White et al. (88) calculated girder flange lateral bending stresses in separate 3D FEA as well as open-section thin-walled beam solutions for comparison to the approximate solution proposed here. Figure 4.1.7-3 illustrates the accuracy associated with using the procedure from Figure 4.7.1-2 for the NISSS16 bridge. One can observe that the flange lateral bending stresses from the 3D FEA simulation model are predicted quite well. The above recommended approach tends to be somewhat conservative in extreme cases where the dimensions  $a$  and  $b$  (as shown in 0 4.7.1-2) are substantially different.

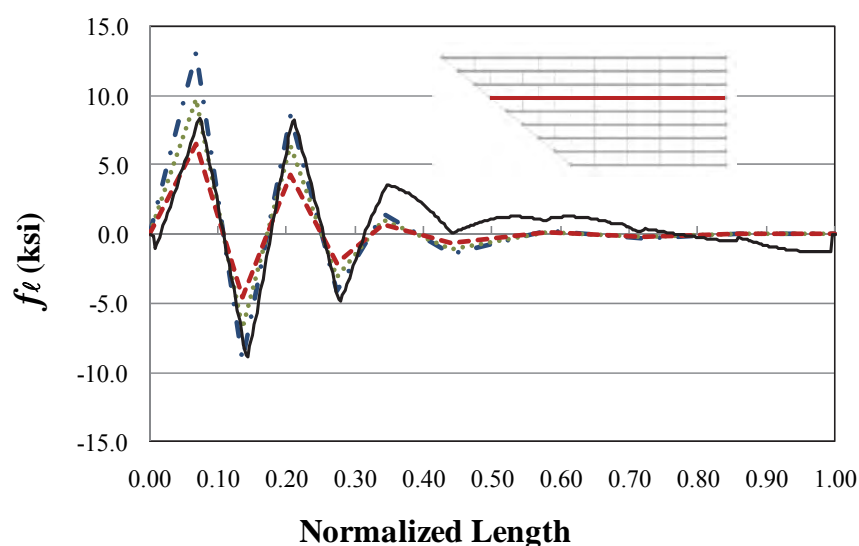
The above recommended procedure for calculating the girder flange lateral bending moments is applicable for estimating the girder bottom flange moments from both non-composite and composite

loadings using 2D grid and/or plate eccentric beam analysis models. It is essential that the 2D analysis include the improvements discussed in White et al. (88), and summarized in Section 3.11 and Section 3.12 of this document.

This procedure requires 1) the equivalent lateral forces from the cross-frames at the level of the bottom flange, 2) the equivalent flange-level lateral forces from eccentric overhang bracket loads on fascia girders, and/or 3) for curved girders, the distributed radial loads that are equivalent to the horizontal curvature. In addition, the above procedure is applicable for calculating the top flange lateral bending moments prior to the concrete deck becoming composite. (After the deck is composite, AASHTO does not require the checking of any flange lateral bending on the top flange, since it may be assumed that all of these lateral bending effects are taken by the deck.) In straight-skewed bridges, the flange level lateral forces from the cross-frames can be relatively large when the cross-frames are staggered. (This is the case near the skewed end of the bridge NISSS16 in Figure 4.1.7-1c and Figure 4.1.7-3.) However, where the cross-frames are framed in contiguous lines, the net lateral force on the internal girder flanges are typically much smaller, since the lateral force from the cross-frame on one side of the girder tends to be balanced by a lateral force from the cross-frame on the other side of the girder. (This is the case at the non-skewed end of the bridge NISSS16 in Figure 4.1.7-1c and Figure 4.1.7-3.)



(a) Girder G3



(b) Girder G6

**Figure 4.1.7-3. Bridge NISSS16 flange lateral bending stresses under total dead load (unfactored).**

## 4.2 Tangent Steel Plate Girders or Rolled Beams

### 4.2.1 No Skew or Limited Skew (<20°)

Tangent steel girder bridges are the simplest type of bridge to analyze, especially those with supports skewed less than or equal to 20°. For these bridges, cross-frames or diaphragms should be oriented parallel to the skew in order so that there will be very little differential deflection at the cross-frames to cause a shifting of the load paths. This type of bridge can generally be analyzed using the line



girder analysis method. There are a variety of commercial software packages that can quickly analyze this type of bridge with minimal user input and are effective in predicting girder stresses and dead load deflections. Hand calculations can also be used, but commercial software is usually faster and more cost-effective. If phased construction is required, then more rigorous analysis methods should be considered relating mostly to predicting dead load deflections and girder camber. Phased construction will require some type of closure pour due to cambered girders being erected adjacent to constructed girders that are in their final position. The differential position of the girders could be significant depending on the differing degrees of non-composite and composite dead load at different stages of construction. If this difference is large, then a grid analysis for dead load should be considered to determine dead load deflections with the typical line girder analysis for forces and stresses. At least two analyses may be required depending on the size of the closure pour; one for the typical girder and another for the phased girder. This staging effect can be minimized by using a smaller closure pour between construction stages.

There are relatively few problems associated with the analysis of most types of tangent girder bridges. Differential deflections in skewed and phased bridges should be evaluated to determine if the predicted actions of the bridge are reasonable. Cross frame design should be given special attention with bridges exhibiting large differential deflections. Some analysis methods can show excessive cross frame loading requiring stiffer cross-frames. Stiffer cross-frames can attract more load and, subsequently, the designer can get into an iterative cycle resulting in heavy cross-frames. Tangent bridges with small skews should not have any major design problems with the cross-frames. Bridges with major and multiple skews could be more problematic.

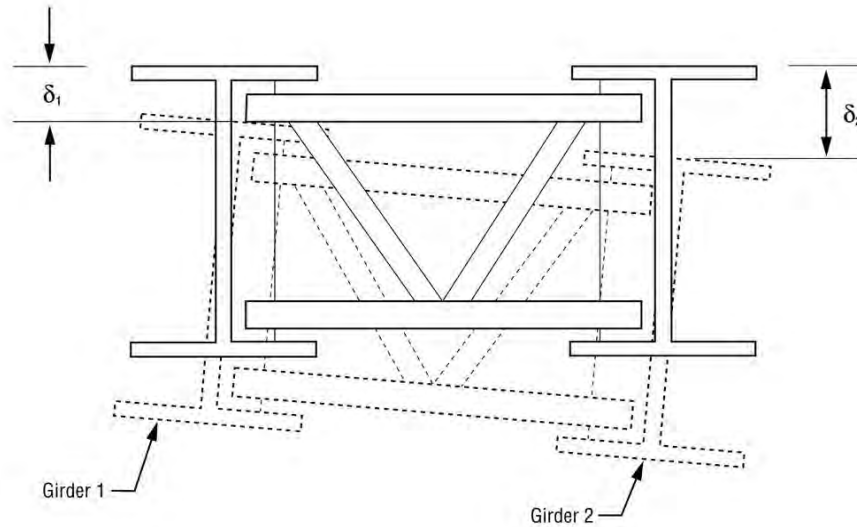
#### 4.2.2 Significantly Skewed

Tangent steel girder bridges with supports skewed more than 20° usually require that the cross-frames be oriented normal to the girders. This type of bridge will have shifting of the load paths through the cross-frames or diaphragms. Line girder analysis may not be sufficiently accurate in predicting deflections and forces in the girders. The degree of inaccuracy depends on the severity of the skew. For this type of bridge, the use of a refined analysis method that adequately addresses cross frame stiffness and also accounts for girder warping stiffness is recommended. There are several commercial software packages available for this type of analysis method.

As mentioned previously, skewed bridges exhibit many of the same behaviors as curved girders. For example, in a bridge with straight girders, an overall skew, and right cross-frames, the cross-frames will cause flange lateral bending.

For tangent bridges skewed more than 20°, staggered cross-frames are sometimes used to reduce the effects of differential deflection across the overall bridge width along the perpendicular cross-frames. Using staggered cross-frames reduces the transverse stiffness of the structure and typically results in reduced cross-frame forces. However, since the cross-frames are not in line across the bridge width, any lateral loads induced through the cross-frames (e.g., due to differential deflection, wind loads, or other lateral loading effects) cause additional lateral bending of the girder flanges as they transfer the lateral loads from cross-frame to cross-frame. The effects of the modified load paths in this type of bridge need to be evaluated using one of the analysis methods listed in this document for significantly skewed bridges.

Right (non-skewed) cross-frames in skewed bridges connect adjacent girders at different positions on the length of each girder, with each girder experiencing different displacement at the point of connection. As a result, these cross-frames are subject to forced racking displacements, which cause internal loads in the cross-frames (Figure 4.2.2-1). The cross frame loads include horizontal components that induce flange lateral bending effects, very much analogous to the effects that are the basis of the V-load method of curved girder analysis discussed later.

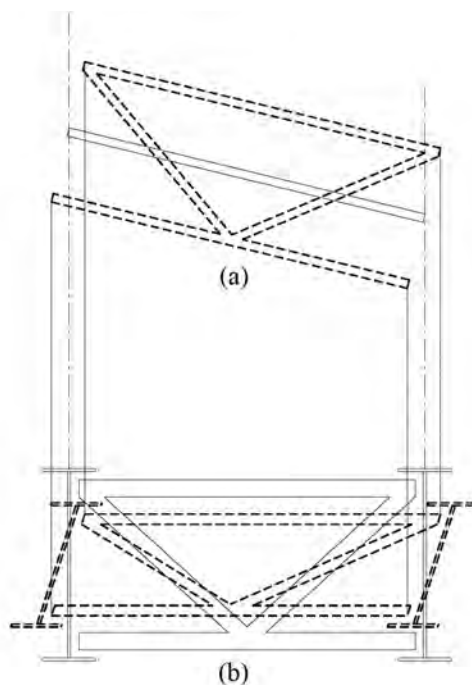


**Figure 4.2.2-1. Right (non-skewed) cross-frames in skewed bridges connect girders at different points along their span length. As a result, the cross-frames are subject to differential displacements. Due to their high in-plane stiffness, they undergo an in-plane rotation rather than racking. The top corners of the cross-frames move horizontally, causing flange lateral bending in the girders.**

Furthermore, near the ends of the girders in skewed bridges, cross-frames begin to act as alternate load paths as their stiffness approaches or exceeds that of the girders. Even if select cross-frames are oriented on the skew or if select cross-frames are omitted as suggested by Krupicka and Poellot (67), the remainder of the cross-frames still undergo this type of behavior and cause the skewed girder system to exhibit many of the same characteristics as a curved girder system (e.g., high cross frame forces, flange lateral bending stresses), even if the girders themselves are straight.

It may seem that these effects can be avoided by skewing the intermediate cross-frames (Reference 8 suggests this is an option up to 20° of skew) so that girders are not connected at points of differential deflection but this does not completely eliminate the introduction of cross frame-induced flange lateral bending. AASHTO/NSBA guideline document G12.1 (15) and Beckman et al. (22, 24) touch on this topic, but further discussion is warranted here. Bending rotations (rotations about the transverse axis of the girder) are associated with vertical deflections of the girders caused by major axis bending. These are primary bending rotations well known to all structural engineers. Assuming the same bending moment diagram in all girders in the bridge cross section, skewed cross-frames would connect the girders at points of identical deflection and rotation.

As the cross-frames rotate to match the primary girder rotations, they also try to rack because they are trying to rotate about the transverse axis of the girders, which is not coincident with the centerline axis of the cross-frames since they are skewed. However, due to their high in-plane stiffness, the cross-frames experience an in-plane rotation rather than racking. So as the top and bottom corners of the cross-frames move forward and backward to follow the primary girder rotation, they also move outward and inward within the plane of the cross-frame (Figure 4.2.2-2), inducing flange lateral bending in the girder flanges.



**Figure 4.2.2-2. Cross-frame skewed to match the bridge skew also induces flange lateral bending. Girders undergo primary bending rotation as well as deflection and cross-frames must rotate with the girders. However, since the axis of cross frame rotation is not perpendicular to the plane of the girder webs, the cross-frames try to rack. But again, due to their high in-plane stiffness, they instead experience an in-plane rotation, causing flange lateral bending.**

The examples above are just samples of how a straight bridge with a skew exhibits similar behavior to a curved girder bridge and why it must be designed using many of the same approaches. Numerous references offer good discussions of the effects of curvature and skew in steel girder bridges (15, 23, 24, 38, 39, 40, 49, 55, 70, 71, and 73). It should be noted that in some cases (for example, a skewed bridge with tangent girders), the approaches taken to detailing, erection, and deck placement may result in conditions where the bridge is forced into an out-of-plumb condition prior to deck placement, followed by the application of the deck self-weight, which effectively causes some counterbalancing internal loading leading to a final condition with girders plumb and theoretically zero (or nearly zero) dead load flange lateral bending stresses and cross frame forces. See Beckman and Medlock (24) and similar references for more discussion). The reader is also referred to a comprehensive quantitative presentation of this topic in NCHRP Report 725 (88).

### 4.2.3 Multiple Different Skews

Tangent steel girder bridges with supports skewed at differing degrees can exhibit large variations in girder forces, deflections, and load paths between the girders. The amount of variation is related to the complexity in the geometry and the severity of the differing skews, span lengths, and related cross frame or diaphragm configuration. This type of bridge could result in girder uplift at supports depending on the layout. This type of bridge can have actions similar to curved bridges due to the potential for twisting of the girders resulting from differential deflections. Depending on the complexity, an appropriate refined analysis method that adequately addresses cross frame stiffness and also accounts for girder warping stiffness is recommended. Line girder analysis should only be considered if all skews are less than 20° and differential deflections are minimal. There are several commercial software packages available for this type of analysis method.

#### 4.2.4 Through Girder Bridges

The through girder bridges could be analyzed by the line girder analysis method, but the typical software packages do not handle this type of bridge since the deck loads are transmitted through floor beams to the longitudinal girders at the edge of the structure. Hand calculations could be used due to the simple framing concept of the two-girder system. However, if the speed of a computer application is desired, the grid analysis method using a general analysis software package could be considered. The modeling can be relatively simple. The longitudinal girders in short- to medium-span bridges typically have little torsional restraint and are modeled with a series of point loads at these attachment points.

Designers should be cautious when studying the floor beam-to-girder connection. Depending on the loads in the girders, these connections sometimes need to take the form of knee-brace type connections and some degree of rigidity needs to be provided by these connections in order to provide some measure of bracing to the top (compression) flange of the through girders. Brockenbrough (26) provides a good discussion of this issue.

### 4.3 Curved Steel Plate Girders or Rolled Beams

In 1969, a research project sponsored by 25 state highway departments and under the direction of the Federal Highway Administration was started in order to develop requirements for curved girders. Even though steel curved girders were designed and constructed prior to this time, design procedures were based on extrapolating available information developed for straight girder bridges. In November 1976, the working stress method for steel horizontally curved highway bridges was successfully balloted by the AASHTO Highways Subcommittee on Bridges and Structures. The 1980 guide specifications included both the working stress and load factor design methods, the latter developed under Project 190. Subsequent projects led to later revisions to the guide specifications and ultimately into the *AASHTO LRFD Bridge Design Specifications* under NCHRP Project 12-52. Interested readers are encouraged to consult Linzell et al. (68) for a good summary of the history of curved girder design.

#### 4.3.1 Methods of Analysis

Two of the primary differences between curved girders and straight girders are the presence of torsion and the interaction between adjacent girders. When deciding the type of analysis to perform, the degree of curvature is one of the primary variables to consider. Previous guide specifications have allowed the designer to ignore curvature effects on the primary moments for given subtended angles. However, this did not mean all curvature effects could be ignored; designers still needed to address non-uniform torsion. *AASHTO LRFD Bridge Design Specifications* (8) now states several criteria for which the effects of curvature may be ignored in determining the major-axis (primary) bending moments and shears for curved I-girder bridges. However, non-uniform torsion (i.e., flange lateral bending) still needs to be evaluated using an appropriate method. When curvature does affect the primary moments, there are several types of methods currently used to design curved I-shape steel girder bridges such as the V-load, grillage and 3D finite element methods. For both the V-LOAD and grillage analysis, small deflection theory is assumed. Specifically, any second-order elastic effects of the flange lateral bending stresses are not considered. The *AASHTO LRFD Bridge Design Specifications* (8) provide an approximate method that the designer can use to evaluate second-order effects and determine if another refined analysis needs to be used.

Appendix B of this document provides guidance on selecting the appropriate level of analysis for dead load non-composite modeling, given the structure's geometric complexity.

#### 4.3.1.1 V-Load Analysis

One of the first methods used for curved steel girder design was the V-load analysis. In 1963, USS published a structural report that presented a simplified approximate analysis technique for open framed (i.e., no horizontal lateral bracing at or near the plane of the bottom flanges) curved I-girder bridges. The method was later modified and simplified for multi-girder systems. At first, this method had been proven valid only for non-composite I-girder bridges with radial supports. Subsequently, studies were conducted in order to support that the method is applicable to composite I-girders with any general support configuration (i.e., skew angles). The V-load analyses were compared with the corresponding 3D finite-element curved bridge models. These models consisted of configurations with different combinations of radial and skewed supports. The dead load V-load results were extremely accurate. The live load V-load results were strongly influenced by the lateral distribution factors that were used.

The V-load method can be summarized in a three-step process. First, the curved girder is analyzed as a straight girder (line girder analysis) using the length of the developed spans. The second step is to apply fictitious V-load forces to the developed girder, in order to calculate the V-LOAD moments. The third step is to compute the flange lateral bending using the M/R approximation. The flange lateral bending (resulting from non-uniform torsion) is evaluated by assuming the girder flange acts as an equivalent beam supported at the diaphragm locations.

The analysis uses live load effects that are based on assumed girder distribution factors. Specification distribution factors give acceptable V-load results for exterior girders and sometimes conservative results for interior girders. The analysis also assumes that the internal torsional load is resisted by self-equilibrating sets of diaphragms or cross-frames. It is important to note that the cross frame properties are not directly considered in the distribution of forces. Cross frame loads are developed by static equilibrium using the diaphragm panel moments and shears. Lastly, the analysis assumes linear distribution of girder shears across the bridge section.

The V-load method should be used with caution for bridges containing variable girder spacing as it may invalidate the assumption of the linear distribution of girder shears across the bridge section. The V-load method should not be used for framing systems containing lateral bracing (closed framing) as this affects the distribution of loads. The V-load method is not easily adapted to evaluate a slab placement sequence and, if this is required, the user may want to use grillage or 3D finite element methods. As stated previously, studies have shown reasonable correlations between the V-load method and FEM analysis when considering supports at various skew angles. However, the *AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges (10)* recommends that the use of the V-load method be limited to bridges with skews less than approximately 10°. In conclusion, there have been extensive studies to support that the V-load method can be used for the design of curved I-shape open framing steel bridges that feature routine span lengths and cross frame spacing, good span balance, gentle curvature, and limited skew.

See also Article 1.1.3 for more discussion of the V-load method.

#### 4.3.1.2 Grid or Grillage Analysis

With the advent of the computer age, a more refined analysis using grillage was developed and subsequently incorporated into commercial computer programs. Using either a general analysis program or a commercial computer program, a grillage analysis can be developed for a horizontally curved steel bridge. Various girder horizontal alignments and support arrangements can be directly modeled.

In both general analysis and commercial computer programs, the model can be either a plane grid or a space frame. In a plane grid, the bridge structure is modeled as a 2D grid in a stiffness format with

three degrees of freedom at each nodal point (corresponding to torsion, shear, and bending moment). Loadings can only be applied perpendicular to the grid.

In either type of program, the girders can either be non-composite or composite with a concrete deck using beam elements. The cross-frames are usually modeled by using an equivalent beam. Supports are modeled using pins or rollers. Most programs internally calculate member self-weights, however additional weight due to beam details (e.g., connections, shear connectors) must be added as either a percentage increase of the self-weight or as an additional uniform line load applied to each girder. Other non-composite and composite dead loads can be applied to the structure as uniform line loads to specific girders.

Live loads are a more complicated issue. In general analysis programs, the user must load the structure or develop a post processing program to calculate live load effects. Commercial programs typically generate influence surfaces and the live load effects are calculated either by determining the governing positions of trucks and lane loading on the influence surfaces or by the application of distribution factors to slices of the influence surfaces that are, in effect, influence lines but are those which are based on the refined analysis. Note, however, that distribution factors that are appropriate for influence lines based on refined analysis differ from those that would be appropriate to be applied on influence lines determined from line girder analysis, i.e., continuous beam analysis.

In most of the current grid/grillage analysis computer programs, there are only three node parameters at each node of the finite grid element (see Articles 1.2.2 and 1.2.3). The warping stress cannot be directly obtained from the grid analysis. The maximum warping stress at the tip of the I-girder flange can be determined as:

$$\sigma_w = \frac{M \ell^2}{12 R D S_f} \quad (4.3.1.2-1)$$

where:

$M$  = girder vertical bending moment (k-in.)

$\ell$  = unbraced length of the girder (in.)

$R$  = girder radius (in.)

$D$  = web depth (in.)

$S_f$  = section modulus of the flange (56, 73)

Note that if there are four node parameters at each node of the grid element (see Article 1.2.4), the warping stress can be directly generated by the grid analysis.

Non-uniform torsion can be determined in several different ways and is dependent on the program used. One is using the V-load method to determine the flange lateral bending stress. Another method uses empirical warping stress equations given in engineering textbooks. Both methods assume the girder flange is supported at the cross frame locations with assumed boundary conditions (usually fixed-end conditions). Other modeling techniques can be employed in which the warping stress is directly determined by the analysis (i.e., introducing a kinematic degree of freedom associated with the warping in the element stiffness matrix).

Cross-frames are generally modeled with beam properties in which different approximates have been addressed previously. Most commercial programs then use a post-processor to determine the individual member forces of each cross-frame (i.e., X-frame or K-frame) using the moments and shears developed in the beam element. The most common shapes used are angles and WT's. Consideration of the eccentricity of the axial load between the point of load introduction (connection to gusset plate or stiffener) and the centroid of the section should be included in the design calculations.

In regards to evaluating the slab placement sequence, the grillage method is easily adapted to consider placement sequences. Most commercial programs only require defining the limits of the slab placement along each girder line. Different modular ratios can be input for girder lines in order to compute the section properties of the previously placed section.

For more discussion, see also Article 1.2.2 for 2D grid analysis methods, Article 1.2.3 for plate- and eccentric-beam grid analysis methods, and Article 1.2.4 for generalized grid analysis methods.

#### *4.3.1.3 3D Finite Elements Analysis*

Typically, the most analytically complicated level of analysis is 3D analysis. Similar to grid analyses, a 3D analysis is a finite element modeling technique. However, instead of limiting the model to a 2D grid of nodes and line elements, a 3D analysis models the main structural elements of the bridge in three dimensions. Girder flanges are modeled using beam or plate elements, webs are modeled using plate elements, and cross-frames and bracing are modeled using truss or plate elements (as appropriate for the given cross frame or bracing configuration). The deck can be modeled using brick elements or plate elements.

How certain features are modeled can greatly affect the results of the analysis and inappropriate decisions (decisions that are inconsistent with the true behavior of the structure in these areas) can result in erroneous results and an inadequate design. For this reason, there are many critical decisions when building a 3D analysis model:

- How to model the bearings (e.g., which bearings are fixed and which are free, which directions of movement are guided and which are restrained, what the direction of guided movements is, and other critical boundary condition assumptions),
- Whether it is necessary to model the substructures (in certain cases, substructure stiffness/flexibility can have significant effects on the behavior of the superstructure),
- How to model structural connections (e.g., cross frame connections to the girders, lateral bracing connections),
- How to account for offsets between girder flanges to the neutral axes of cross-frame and bracing members,
- How to account for offsets between the girder flanges to the deck elements,
- How to model connectivity of the girders and the deck,
- How to model the moving live loads,
- How to model staged deck placement,
- How to model staged girder erection,
- How to account for centrifugal force effects and effects of deck super-elevation, and
- How to account for girder out-of-plumbness.

Since the major structural components are directly modeled, a 3D analysis has the advantage of being a very rigorous analysis. A 3D analysis directly models all significant stiffness characteristics of the bridge and element results are directly available for all modeled elements. Complex structural configurations are modeled in detail, rather than approximating the overall stiffness parameters with estimated single values. For example, to model a tub girder in a 3D analysis, the bottom flange is modeled with separate elements, as are the two webs, the two top flanges, the internal cross-frames, and the top flange lateral bracing. In contrast, in a grid analysis, the entire tub girder (flanges, webs, internal cross-frames, and top flange lateral bracing) is modeled as a beam with the stiffness of much

of the internal framing approximated using simplified calculations or empirical estimates. This greater detail and rigor in a 3D model theoretically leads to more accurate analysis results.

However, 3D analysis involves much greater modeling effort than grid analysis. The resulting model is significantly more complex and, as a result, there is an increased chance that errors may inadvertently be introduced to the analysis. Furthermore, the greater detail and greater volume of direct results for each and every element of the structure can be a two-edged sword. While there is value in having direct results for all elements of the structure, the sheer volume of the results can become overwhelming in terms of the required post-processing effort. Many designers find 3D analysis results much less intuitive and harder to visualize and understand. In the end, a 3D analysis is only as accurate as the assumptions made in building the model (discussed above). As a result, there is greater risk that mistakes in the analysis will be missed or that analysis results will be misinterpreted. Therefore, it is imperative that the 3D model is compared to other analysis tools (e.g., grid model, benchmark solutions) in order to confirm its accuracy. In summary, there is a very real question associated with 3D analysis: Are the greater accuracy and detail worth the effort?

While there are obvious disadvantages to 3D analysis, there are virtually no limitations to the type or complexity of structures that can be modeled using this technique. The limitations come down to the time and money available to perform the analysis and the experience and comfort the designer has with more complicated analysis models.

A number of commercial programs are available to perform part or all of a 3D analysis. In addition, a 3D analysis can be performed using any general finite element analysis program. Be aware, though, that a significant amount of effort might be involved in building and post-processing the analysis model and performing AASHTO code checks on the girders, cross-frames, and other elements—particularly considering 3D models provide girder results in terms of flange and web stresses or forces, while many AASHTO design equations are written in terms of overall girder moments and shears.

#### **4.3.2 Skewed and Curved I-Girder Bridges**

The behavior of curved girder bridges is relatively well understood and reasonably predictable if all supports for the bridge are radial. However, if the supports are skewed, the behavior of a curved girder bridge becomes much more complex and difficult to predict. The reader is directed to the discussion of skewed tangent girder bridges in this document (e.g., Articles 4.2.1, 4.2.2, 4.2.3) for discussion on many of the issues associated with skew of steel girder bridges. The reader may also find helpful guidance in Coletti et al. (38).

### **4.4 Tub Girders—General Issues**

Steel tub girders as primary structural members, when seen from outside, give the impression of solid section members with a simple resistance path to loads that are typical in bridge structures. From that perspective, it would seem that it would be easy to model these members as a line that represents the set of centers of gravity of all sections along the length of the structure. It would be easy then to perform structural analysis of these primary members and thus obtain loading distribution along these members either by line girder analysis, grid analysis, or even frame analysis. As shown so far in this guide, this approach is commonly used for typical steel framings, such as simple plate girder structures, or for rolled shapes.

However, steel tub girders represent integral members that are made out of a series of simpler members attached together that, when assembled correctly, will produce a solid and stiff section that will resist basic loads in an integral fashion. Such members are:



- *Steel plates.* These are welded together to make the basic tub section that will resist primary basic loadings.
- *Internal cross-frames consisting of rolled shapes such as angles or structural T-sections.* In some instances, internal diaphragms made of steel plates and stiffeners are used instead of the cross-frames. Either of these components is used to ensure the integrity of the trapezoidal shape of the tub section while the comprising plates described above resist the primary loadings.
- *Upper lateral bracing, also consisting of rolled shapes such as angles or structural T-sections.* These members ensure the integrity of the tub girders resisting distortional loads between cross frame sections resulting from St. Venant torsion, as well as warping loads resulting from torsional loads. These members play a significant role in maintaining the integrity of the tub girder cross section while resisting loading during construction, especially until the deck slab has been placed and cured. In addition, while they are usually neglected in the design of steel tub girders, these members also contribute to resisting longitudinal flexure, and a portion of their section could be considered as a supplemental section to the top flange either in tension or compression.
- *Connection plates and stiffeners of different types.* These include transverse stiffeners or connection plates, or both, for the cross-frames, bottom flange longitudinal stiffeners used in the negative moment region, and bottom flange transverse strut used to stiffen the bottom flange from lateral flexural stresses.
- *The deck slab.* Once it is cured, the deck slab contributes to the resistance of all types of loading by the steel tub girders in a composite fashion.

As a result, the complexity of the behavior of steel tub girders could be compared more to a 3D truss rather than a simple beam behavior.

The behavior of these members could be further complicated by the layout of the different structures. Some of the features that could complicate the behavior of this type of structure include:

- *Curvature.* In general, the smaller the radii of the curvature, the more complex the behavior. Specifically, the sweep of the curve between the two adjacent piers is a key parameter. As the span length decreases, the effect of tighter curvature diminishes.
- *Support skew.* The higher the degree of the skew of any of the piers supporting the superstructure, the more complex the stress distribution in the structure.
- *Framing irregularity.* Steel tub girders represent very stiff structural elements. As such, they are better suited to be used on regular alignment. Complications in structural behavior due to irregularity in framing, such as flaring or converging alignments, are exacerbated by the greater stiffness of steel tub girder structures.
- *Variable section.* Often a variable depth tub section is used, especially in long spans. Deeper sections are used at the supports to resist larger moments and the section is made shallower at the midspan to allow for more clearance underneath the bridge span.
- *Span length.* It could have an adverse effect if combined with one of the features listed in the previous bullet points. Longer spans combined with tight curve radii would give a larger sweep and, therefore, a more complex behavior. On the other hand, a longer span combined with skewed support could diminish the adverse effect of the support skew.
- *Construction phase (specifically during erection and placement of the concrete).* This is usually the most fragile stage of the construction of steel tub girder structures. Historically,

the only cases of structural collapse of these structures have occurred during this particular phase. Once the concrete cures, the stiffness and strength of this type of structure improves by an order of magnitude.

Typically, in straight structures with regular framing members such as bracing elements and intermediate cross-frames, support diaphragms are considered secondary members. Their main use is to provide stability for the primary members due to vertical or lateral loads. While these members do help with the distribution of the vertical loads, their contribution is usually neglected. Thus, the primary members can be modeled and analyzed as separate line girders. It is common that the engineer will run line girder analysis for one interior girder and one exterior girder. Secondary members in these instances would be selected from local design standards. It is generally good practice to check these members for resistance to the wind and incidental eccentricity, especially during construction (when the deck is still not in place) but also during service life.

In the case of structures with complex framing layouts, the secondary members become very important elements in distributing and resisting vertical and horizontal loads. These members do not just provide stability for the primary members; they are an integral part of the structural path that takes all the loads being applied to the superstructure and transfers them into the substructure. Hence, in all steel structures that have an irregular framing, every member in the bridge needs to be analyzed and designed as a primary structural element.

The level of analysis required for the steel tub girders will be dictated by the following:

- *Analysis purpose.* Whether this is preliminary design, final design, or a subsequent redundancy analysis and
- *Degree of complexity of the framing.* The more complex, the more detailed an analysis would be required.

In addition, an analysis performed to evaluate available redundancy may require nonlinear material and geometry capabilities.

#### **4.4.1 Analysis of Internal Framing (Internal Intermediate Diaphragms, Top Flange Lateral Bracing)—Grid Analysis and Hand Calculations versus 3D**

Without internal intermediate diaphragms and top flange lateral bracing, a tub girder is an unstable open section with very little torsional stability. A 3D analysis directly models these members and provides design forces for them. 2D models and line-girder analyses treat each tub girder as a complete member and provide no direct way of determining design forces for these internal members.

Internal intermediate diaphragms are utilized to control the cross-sectional distortion of the tub girder due to global twisting and local distortion due to applied loads. There are a number of empirical guidelines for minimum sizes of the members that have been around for some time; see Heins (56), among others. Recently, Fan and Helwig (48) have provided a more analytical approach to internal diaphragm sizing and maximum spacing to control distortion.

Top flange lateral bracing plays a more important role in overall girder stability. Top flange lateral bracing provides the fourth side to the box to create a closed and torsionally rigid shape. The force in the brace can be directly correlated to the torsion in the girder. Furthermore, the lateral bracing participates in the major axis bending of the cross section particularly during the application of wet concrete load and the total force in the braces is the sum of these effects. This is further discussed below in Article 4.5.1.

#### 4.4.2 Consideration of External Intermediate Diaphragms

Unlike steel I-girders, due to the significant torsional stiffness inherent in box girders, the external intermediate diaphragms are not critical to the load path in a curved structure. The external intermediate diaphragms in a box girder system do provide a very important role in the constructability of the system; however, external intermediate diaphragms in box girders serve to limit the relative twist and deflection between adjacent girders, thereby providing a uniform slab thickness in the completed unit. Once the bridge deck is placed and cured, these diaphragms have little effect on the behavior of the girder system and often are removed from the completed structure. The potential removal of the intermediate external diaphragms presents another stage in the construction history of the completed unit that must be investigated. The final analysis model of the completed structure should not include the external diaphragms if they are to be removed after the bridge deck is placed.

In addition to the analysis of the completed structure, the designer needs to consider the effects of the slab placement and determine what is required for intermediate external diaphragms. There are a few tools to choose from. A 3D finite-element analysis will provide direct forces for the intermediate diaphragms and allow the designer to place diaphragms to minimize relative twist of the adjacent girders. Another option for designers is a computational approach; Helwig et al. (58) present formulas and methodology for calculating required external diaphragm spacing as well as calculating force in external bracing members.

If external intermediate diaphragms are to be left in place, it is important that they be detailed appropriately to handle the live load stress reversals. Although the forces are relatively small, the typical detail of an intermediate K-frame diaphragm is a highly fatigue-prone detail and should be avoided if the diaphragms are to remain in place. Full-depth plate diaphragms with bolted connections to the girders are typically used for diaphragms that are to remain in place. These diaphragms have substantial stiffness in the completed structure and should be included in the analysis model.

#### 4.4.3 Cross Frame Modeling (Shear Stiffness, Flexural Stiffness)

When investigating the construction sequencing of tub girders or in instances where the intermediate cross-frames are to remain in place in the final structure, it is important to model the cross frame properties accordingly.

A 3D FEM analysis provides the most straightforward approach to modeling diaphragms. It is important to model not only the geometry and properties of the external diaphragms, but also to accurately model the internal diaphragm within the box at the same location. As with all 3D FEM output, the results are only as good as the model. If the model is accurate, the designer should be able to determine member forces directly from the analysis.

It is in the modeling of external diaphragms that a grid analysis must be very closely scrutinized. Because a grid analysis typically models each tub girder using a single beam element at the centerline of girder, there is a discrepancy between the modeled geometry and the actual geometry of external diaphragms. The modeled geometry has a working point at the centerline of the girder when the actual geometry is detailed to the face of the girder web. Thus in the model, the length of the external diaphragm is effectively much longer than the length of the external diaphragm in the actual bridge.

The guidance provided in Article 3.11 for modeling equivalent stiffness of external diaphragms is applicable for box girders as well, with a few modifications. In the previous discussion of I-girders, the length of the external diaphragms, for stiffness calculations, is taken as the girder spacing. With tub girders, the cross-frame has two components: the external diaphragms and the internal diaphragm. The consideration of the internal diaphragm stiffness, assuming that the girder cross section is distorting, is complicated and typically not considered in a 2D analysis. The designer has two

practical choices for modeling the external diaphragms: 1) simply model the external diaphragms between the girder centerlines or 2) treat the internal diaphragm as rigid and model the external diaphragms with an effective length between the webs of two adjacent girders.

2D grid analysis software packages typically allow the designer either to 1) input member sizes and diaphragms geometry and let the program calculate the equivalent stiffness or 2) input equivalent flexure or shear section properties, or both, directly into the analysis. Although inputting the cross frame members and geometry would appear to be a simple task, the process of accurately modeling the stiffness of cross-frames in 2D models is actually quite complex, as discussed in Article 3.11. Most grid analysis programs are modeling actual member sizes and generic cross frame geometry (either X- or K-brace) but are modeling completely erroneous geometry by taking the working points at centerline of girder. It is recommended that the designer calculate equivalent diaphragm stiffness by hand and then input that stiffness directly into the analysis. The correct methodology for the stiffness calculation can be debated but at least by calculating it outside the program the designer can control and document the assumptions.

When calculating the external diaphragm stiffness by hand, the designer has control of how to treat the internal diaphragm and can determine the effective length of external diaphragms. If taking the effective external diaphragm length as between the webs of adjacent girders, it is important to convert that external diaphragm stiffness into a center-to-center equivalent before entering the stiffness into the analysis program.

The diaphragms' output from the analysis software should be scrutinized as well. Given the geometric discrepancies in modeling the diaphragms, as discussed above, diaphragms' individual member forces should not be taken directly from a grid analysis unless the designer can verify the methodology for determining those forces used by the analysis software. Designers should note that reported forces are likely reported at the centerline of the girder, not at the external diaphragms to girder connection.

There is no correct answer for modeling external diaphragms of tub girders using a grid analysis; it is always going to be an approximation. The important thing is for designers to understand the issues and limitations of the software and to make consistent and informed decisions about their analysis.

#### **4.4.4 Narrow Systems, Stability Analysis**

For trapezoidal steel box girder bridges, a narrow system typically refers to a bridge with a superstructure consisting of a single box girder. Single box girder superstructures are finding favor in recent years due to the inherent torsional rigidity of the box and what many would consider enhanced aesthetics that can be achieved with a structure of this type. However, the use of this type of superstructure requires special design considerations. Structures consisting of multiple box girders can also have many of the same design considerations as a single box girder if any stage of construction involves erection of a single box without any form of external bracing.

The two primary considerations of narrow box girder systems are the stability of the box during construction and structural redundancy. For many years, it was felt that if the out-of-plane moment of inertia was greater than the in-plane moment of inertia, there would be no problem with buckling since the girder was bending about the weaker axis. However, failures of single steel box girders during construction are causing the industry to rethink that notion.

One such failure, a pedestrian bridge in Marcy, NY, documented by Weidlinger Associates (87), showed that this notion may be flawed. The structure, consisting of a 170-ft single box girder, collapsed when the deck pour reached about 60 percent completion. The pour was occurring continuously from one end of the structure. Since the bridge was tangent, global lateral torsional buckling of the section was not considered; instead, the designers apparently assumed that since the primary bending due to gravity loading was acting about the weaker axis of the girder, top flange

lateral bracing was not required. As a result, the single tub girder behaved as an open section with very low torsional stiffness and collapsed during deck placement.

More recent investigations by Yura and Widiyanto (96) have determined that the assumption that lateral torsional buckling will not occur if the out-of-plane moment of inertia is greater than the in-plane moment of inertia (based on the assumption that lateral torsional buckling would not occur since the girder was bending about its weaker axis) is only true in very specific circumstances with a doubly symmetric cross section of an initially straight beam loaded through its centroid or with a uniform moment.

Steel box girders are singly symmetric sections that are often cambered with uncertain loading, particularly during construction. Steel box girders gain significant torsional stiffness when they consist of a closed or quasi-closed cross section. Closed box section girders can have a stiffness that is as much as 100 times greater than open sections. As discussed in Article 4.4.1, a tub girder can be made to behave in a quasi-closed manner through the use of top flange lateral bracing. Since global buckling of the girder involves a torsional rotation of the entire section, increasing the torsional rigidity of the section also significantly increases the global buckling resistance. Often, only a minimal amount of top flange lateral bracing is required to increase the stiffness enough to resist global lateral torsional buckling. Yura and Widiyanto (96) found that adding top flange lateral bracing to only the last three bays on each end of the Marcy Bridge would have prevented its collapse.

The investigations by Yura and Widiyanto (96) also found that increasing the outward slope of the webs also reduces the buckling capacity of the girder (in cases where no top flange lateral bracing is provided and the tub girder is functioning as an open-section member). In most situations, where a single box girder is used on a tangent bridge of moderate span length, approximate solutions using a 2D grid analysis in conjunction with the approximation of top flange lateral bracing forces provided by the equations developed by Fan and Helwig (47, 48), as discussed in Article 4.4.1, are adequate. In addition, an equation for the approximate critical buckling moment can be found in Yura and Widiyanto (96). For extremely long spans or single box structures with horizontal curvature, it is highly recommended that a full 3D FEM analysis be performed.

#### 4.4.5 Narrow Systems, Redundancy Analysis

Another consideration of narrow box girder systems involves the redundancy of the structural system. Redundancy is dependent on the ability of the structural system to provide an alternate load path should any one major structural element fail. A narrow box girder system, such as a single box girder bridge, would be considered nonredundant due to the single bottom flange, thereby becoming fracture critical and requiring more frequent inspections. Fracture critical structural inspections vary from every two to five years, with two years being the more typical and inspections increasing in frequency if a problem is found. Fracture critical inspections are very costly, up to five times more than a conventional inspection. Inspections of closed members, such as box girders, are particularly expensive since inspectors must go inside the box for the inspection. This typically involves providing a snooper for access to the box girder and traffic control to close down the supported roadway structure while the inspection is occurring. In many cases, the life cycle costs of the inspections could outweigh the additional expense of designing a non-fracture critical structure. These issues should be discussed with the structure owner to ensure that they completely understand the implications of a single box structure.

Consideration should be given to designing either a two- or three-box system. At this time, a two-box system is still considered a fracture critical bridge structure. However, there has been recent research into the redundancy of a two-box system, e.g., Williamson and Frank (92). In many cases, a two-girder system can be shown to be redundant through analysis and consideration of other structural

elements, special design considerations, and careful detailing of secondary framing/bracing members and the concrete slab.

When evaluating redundancy of a two-girder system, several mechanisms can be considered to demonstrate redundant behavior in the event of fracture of one girder. These include considering the truss action between the concrete slab and external intermediate diaphragms, the torsional stiffness of the external intermediate diaphragms, the flexural capacity of the concrete slab, and the contribution to the overall stiffness of the system of the concrete barrier rails.

From about 2005 to 2010, research was conducted at the University of Texas investigating the redundancy of two-box girder systems. Tests were performed on a two-box girder system consisting of a 120-ft span and 23-ft roadway supporting a simulated HS20 truck load. The bottom flange was dynamically fractured using linear shape charges. From this work by Williamson and Frank (92), the investigators were able to recommend design procedures and details that enable such a system to withstand fracture of one girder without collapse. As of this writing, it is anticipated that this research may eventually lead to simplified design methods or detailing provisions, or both, to allow two-box girder systems to be considered non-fracture critical.

#### **4.4.6 Variable Depth Girders**

The issues related to variable depth I-girders are essentially the same as for variable depth tub girders. See Article 4.1.5.

#### **4.4.7 Recent Improvements to Simplified Analysis Methods for Tub Girders**

White et al. (88) recommend several improvements for the simplified analysis of curved and skewed tub girder bridges, including:

- Development of an improved method for estimating the influence of skew on tub girder internal torques using basic 1D analysis procedures,
- Investigation of the influence of skew (and torsion due to skew) on the cross-section distortion of box girders, and
- Calculation of local effects from the longitudinal components of the axial forces in the diagonals of the top flange lateral bracing (TFLB) system, which result in “saw-tooth” type local spikes in the longitudinal normal stresses in tub girder top flanges.

These recommendations complement a large volume of recommendations provided by Helwig et al. (58) on approximate analysis techniques for tub girder bridges. The reader is directed to Helwig et al. (58) and to White et al. (88) for further discussion of these recommendations.

### **4.5 Tangent Steel Tub or Box Girders**

#### **4.5.1 No Skew or Limited Skew (<10°)**

Tangent box girders are the simplest of box girders and are very similar in analysis to tangent I-girders. There are, however, a few special considerations that are unique to box girders. Similar to I-girders, there are many options for analysis procedures: line girder analysis, 2D grid analysis, and 3D finite element analysis.

Typical tangent box girders are well suited for a line girder analysis. This analysis method requires the distribution of dead loads based on tributary spacing and the calculation of live load distribution factors according to *AASHTO LRFD Bridge Design Specifications* (8). Moments and shears along the girder are calculated either by hand or by using a simple line girder computer model. There are several commercial software packages available that can perform this analysis as well as perform code checks on the girder section.

A 2D grid analysis can be a little bit more involved. This procedure typically models all girders in the system in either a commercial software package or general finite element analysis program. The model consists of line elements for the girders connected with shell elements or a grillage of beam elements to represent the deck. This analysis method has advantages over a line girder analysis in that the dead loads can be applied directly and distributed to the girders through the analysis. Live loads also are applied to an influence surface that maximizes effects on each girder without the calculation of distribution factors. Most bridge design software has the ability to quickly develop these live load influence surfaces.

A 3D FEM may not be necessary for tangent box girder bridges. There are cases where it can be advantageous, though. Most 3D models are customized for each structure so more complicated framing can easily be accommodated. 3D FEM analyses are also helpful for complex loadings, such as light rail vehicle derailment. One of the primary advantages of a 3D FEM is the direct modeling of secondary members, such as top flange lateral bracing and internal diaphragms. This analysis provides forces for designing these members without the use of approximations.

Appendix B of this document provides guidance on selecting the appropriate level of analysis for dead load non-composite modeling, given the structure's geometric complexity.

The need for top flange lateral bracing is well recognized after the collapse of the Marcy Bridge during construction. This bracing provides torsional stiffness to the girder prior to the hardening of the concrete deck. Although braces for tangent girders are typically sized to satisfy slenderness requirements only, the braces do carry significant axial forces due to participation in major axis bending. Fan and Helwig (47, 48) have presented equations for approximating the forces in top flange lateral bracing members due to major axis bending for both X-brace configurations and Warren truss configurations. There are no approximate equations for Pratt truss configurations and the Fan and Helwig (47, 48) formulas should not be used. As previously mentioned, these forces can be pulled directly from a 3D analysis.

When using a line girder analysis or grid analysis, it is important to consider flange lateral bending. Although the girder is tangent, it is still subject to torsion caused by differential loading, skew, or irregular framing, and there are still two sources of flange lateral bending that should be considered: slab overhang forms and top lateral flange bracing. Slab overhang forms are typically attached directly to the top flanges and support the wet concrete as well as the deck screed rail and traveling screed. These loads are applied with significant eccentricity and the resulting moments can create significant flange lateral bending moments that need to be considered in design. The other major source of flange lateral bending is the top flange lateral bracing. These braces typically frame directly into the top flanges and the axial forces, due to major axis bending, induce lateral forces and moments in the flanges. Again, the equations presented by Fan and Helwig (47, 48) offer a good approximation for these lateral bending moments.

Other miscellaneous design elements include internal diaphragms, intermediate external diaphragms, and external pier diaphragms. The internal diaphragms are used to control cross section geometry and typically consist of K-frames. For tangent girders, these K-frames typically consist of minimum size angles to meet slenderness requirements; forces from a 3D analysis can be used if available. Intermediate external diaphragms are used to control relative twist of adjacent girders during the deck placement. In tangent girders, these intermediate external diaphragms are typically not required although they may be desired to improve the global stability of the girder system. External pier diaphragms are required at the ends of the girder units to resolve torsion in the system into vertical reactions at the bearings. The design of external pier diaphragms is addressed by Coletti et al. (37).

#### 4.5.2 Significantly Skewed

Many of the same analysis recommendations provided for tangent tub girders with little or no skew apply to significantly skewed tangent tub girders, with some exceptions. Designers are cautioned that the magnitude of loads in internal and external bracing of skewed tub girders is extremely sensitive to the effects of the skew. Care should be taken to adequately address the determination of these internal loading effects. Appendix B of this document provides guidance on selecting the appropriate level of analysis for dead load non-composite modeling, given the structure's geometric complexity.

#### 4.5.3 Multiple Different Skews

The introduction of multiple different skews in steel tub girder bridges complicates the determination of loads in the internal and external bracing even more than in significantly skewed steel tub girders with parallel skewed supports. See Article 4.5.2 for more discussion.

### 4.6 Curved Steel Tub or Box Girders

Generally, due to their large torsional stiffness, steel tub girders are most suited for use in bridges whose alignments are on tight curves. However, in many cases, the alignment curvature is not significant and, therefore, the primary bending controls the structural behavior of the girders. In such instances, as specified in the *AASHTO Guide Specifications for Horizontally Curved Steel Girder Highway Bridges (11)*, primary bending can be analyzed by neglecting the effect of the curvature and assuming a straight alignment. Span lengths to be used in these analyses should be the lengths of the spans along the curve. In any instance, torsional effects should be evaluated carefully as, even in straight or slightly curved alignment, incidental torsion is present both during construction and service life.

In most cases, steel tub girders are used to carry traffic in tight curve layouts. In such cases, torsional effects are usually coupled with primary bending to produce a more complex stress distribution in all members of the structure. While this distribution could be approximately investigated using straight line girder analysis combined with simplified hand calculation methods that estimate the effect of the curve, currently it is as easy and cost effective to do such preliminary approximations by using either a grid or frame type analysis. If there is a relative extra effort involved to input the transverse framing data, this will usually pay off at the end by minimizing the number of iterations to design the most efficient structure.

Grid analysis represents another level of accuracy that falls between the line girder analysis and 3D FEM. The advantage compared to line girder analysis is that these methods are able to model the behavior and contribution of some transverse elements, such as intermediate and support diaphragms and that of the slab. Thus, these types of analyses are able to capture the interaction and contribution of all girders in sharing the resistance to the vertical loads. However, they have their limitations and shortcomings when compared to the 3D FEM methods, including:

- Since girders and bracing are typically modeled using standard beam elements in 2D grid models, cross-sectional distortion (warping) is not typically represented in grid modeling.
- For most of the practical curved steel box girder bridges, designed according to current AASHTO specifications, the warping stress will be less than 10 percent of the normal stress due to vertical bending and can be estimated by BEF Method (93). Test and analytical results by Huang (61) show that the grid model with about 20 elements in longitudinal direction per span can well predict the internal forces of the curved multi-box girder bridges with external bracings located only over supports under live loads. The stiffness of the transverse grid element can be determined based on the whole deck width of the element. The entire deck width of bridge can be safely treated as effective concrete area in bridge design and analysis.



For the most complex structures, the ones that involve many of the features mentioned above, 3D finite element analyses are recommended to carry out final design. Their advantages are as follows:

- Steel tub girders are made up of steel plates and can be modeled with shell elements that more accurately represent the behavior of the steel plates both in bending and axial loading.
- Bracing members that mainly resist loads in tension/compression can be modeled by either axial elements or by beam elements.
- Intermediate and support diaphragms are commonly designed either as a truss cross-frame or as a plate diaphragm. Truss cross-frames can be modeled using either axial or beam elements, similar to the bracing elements. Plate diaphragms would be modeled either as beam elements, when their depth to length ratio indicates beam behavior, or as an assembly of built-up shell elements.
- Deck slabs can be modeled using either shell elements or 3D volumetric brick elements. It should be mentioned that not all structural 3D FEM analysis packages oriented toward bridge design include the brick element feature.
- Currently, most commercial 3D FEM programs include mathematical offset features. This means that the structural behavior of the composite/integral section can be modeled as closely as possible.
- Pre- and post-processing of the data has become more user-friendly. Some 3D FEM packages include features that automatically calculate integral effects of any section cut in the structure; however, it is important that the engineer understand the basis of these calculations to be able to verify that they are being appropriately compiled. This allows for the designer to obtain moments ( $M$ ), shears ( $V$ ), and axial loads ( $P$ ), which are more useful to the designer in applying code requirements.
- Many available programs employ some sort of influence surfaces or series of influence lines that help automate the calculation of the envelope of critical live loading.
- Some 3D FEM programs are capable of modeling nonlinear behavior, both material and geometry nonlinearity.

#### 4.6.1 Skewed and Curved Steel Tub or Box Girder Bridges

The behavior of curved girder bridges is relatively well understood and reasonably predictable if all supports for the bridge are radial. However, if the supports are skewed, the behavior of a curved girder bridge becomes much more complex and difficult to predict. The reader is directed to the discussion of skewed tangent girder bridges in this document (e.g., Articles 4.5.1, 4.5.2, 4.5.3) for discussion of many of the issues associated with skew of steel girder bridges. The reader may also find helpful guidance in Coletti et al. (38).

Many of the same effects described for steel I-girder bridges also occur in steel tub girder bridges, but steel tub girder bridges also exhibit unique characteristics in terms of their behavior when skewed. Most of these are associated with the additional structural elements that are unique to tub girder bridges and with the fact that tub girders, unlike I-girders, exhibit very high levels of torsional stiffness.

For instance, end diaphragms in tub girder bridges are typically oriented parallel to the skew, just as in I-girder bridges. These end diaphragms are typically full-depth plate sections, both within individual tub girders and between adjacent tub girders, and have high levels of in-plane shear (or racking) stiffness. As described above for intermediate diaphragms oriented parallel to the skew in I-girder bridges, these skewed end diaphragms are subject to high in-plane racking shear stresses, as

they resist racking deformations that result from the significant girder primary bending rotations occurring at the ends of the girders. Again, as for I-girder bridges, these effects occur even if the girders themselves are tangent. Moreover, in tub girders, these effects can be quite pronounced due to the large size and stiffness of typical tub girder end diaphragms.

In addition, the interior and exterior pier and end diaphragms anchor the ends of a tub girder, preventing any layover of the tub girders at their ends. As a result, significant twisting effects can occur along the length of each girder in a skewed tub girder bridge. Since tub girders, as closed-cell or quasi-closed-cell structures, inherently exhibit high levels of torsional stiffness, noticeable torsional loading can occur. This torsion causes a St. Venant torsional shear flow around the perimeter of the girder, resulting in shears in the webs and the bottom flange. In the non-composite condition, this shear flow also adds to loading in the top flange lateral bracing system (the truss-type structural elements which form the fourth side of the box in a tub girder prior to deck placement), while in the composite condition this shear flow causes an in-plane shear in the deck.

Similarly, skew will affect the loading of the internal intermediate diaphragms in a tub girder. These diaphragms (typically X-frames or inverted K-frames spaced at intervals inside each tub girder) are provided to help control cross-sectional distortion of the tub girder caused by torsion.

Thus, even though the tub girder itself may be straight, the fact that it is skewed and has skewed end diaphragms will cause torsional loading, leading to increased loads in many different elements of the structure.

## **4.7 Bridges with Significantly Complex Framing**

### **4.7.1 Variable Girder Spacing**

Although variable girder spacing should be avoided if possible, in some instances required horizontal alignments and roadway geometries force the bridge engineer to establish a framing plan that implements variable girder spacing. The most common roadway alignment that results in variable or flared girder spacing is associated with a ramp alignment merging with or splitting off from a mainline alignment. In this situation, flaring some or all of the girders cannot be avoided.

The analysis of flared girders is similar to the analysis of parallel girders except that the following items vary across the girder length as a function of the varying girder spacing:

- Magnitudes of dead and some superimposed loads,
- Live load distribution, and
- Composite section properties.

At the end of the span with a narrower spacing, clearly the tributary width supported by that girder is less than the tributary width at the wide-spacing end. Due to the varying magnitude of load, often narrower or thinner plates, or both, will be used for the flanges at the narrow-spacing end of the girder while heavier flange plates are required toward the wide-spacing end. The result is an asymmetric girder with increasing stiffness from the narrower-spacing end to the wider-spacing end. Similar to other plate-girder analyses, this variation in stiffness must be accounted for when estimating cambers/deflections and when evaluating the distribution of live loads.

Variable girder spacing somewhat complicates live load distribution. The AASHTO *Standard Specifications for Highway Bridges (14)* prescribe simple live load distribution factors based on the girder spacing. So, at any position along the girder length, a unique live load distribution factor would apply based on the girder spacing at that discrete position. It is common for designers to use an average girder spacing to study mid-span bending moments when using the standard specifications. For evaluation of maximum live load shear, however, the wide-spacing end could be the basis for

maximum girder shear. The latest version of the *AASHTO LRFD Bridge Design Specifications* (8) imposes a more rigorous set of formulas and techniques for determining the live load distribution factors for moment and shear. The girder spacing is one parameter included in many of the simplified formulas. As such, variable girder spacing results in a constantly changing live load distribution factor along the girder length. The designer should exercise judgment when establishing the design live load distribution factors for moment and shear using the LRFD live load distribution factor formulas. Simply using the average girder spacing may not be sufficient since girder stiffness and other parameters are inputs to these formulas. It would be reasonable to check multiple positions along the span (or spans) to ascertain representative live load distribution factors for design or to perform a more refined analysis in which live load effects in each girder are directly calculated based on relative stiffness, rather than relying on empirical live load distribution factors.

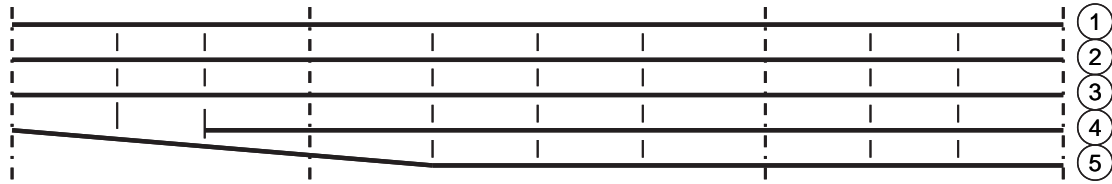
Not only do the loads differ on girders with varying spacing, but the composite section properties also differ according to the effective flange width and tributary width. These section properties must be carefully determined in order to reasonably estimate the cambers and deflections at discrete points along the girder.

Given these complications to the loading, stiffness modeling, prediction of constructed geometry, and calculation of strength and service capacities, designers might want to consider using more refined analysis methods when designing bridges with variable girder spacing. For example, some line girder analysis methods may be incapable of modeling the variations in dead loads and live loads found in variable spacing girder designs. Similarly, some grid analysis methods, if they rely on the AASHTO simplified live load distribution factors rather than refined analysis of live load distribution, may not provide enough refinement in the prediction of live load effects and a plate- and eccentric-beam approach or a 3D FEM approach may be more appropriate. Each project should be considered on a case-by-case basis to determine the need and value of using a particular level of analysis, based on the significance of any possible inaccuracies in the predictions of deflections, moments, shears, and other structural elements.

#### 4.7.2 Discontinuous Girders

Similar to variable girder spacing, the introduction or termination of girder lines at intermediate locations within a framing unit is a complication that is avoided when possible. In practice, the consideration of variable spacing and discontinuous girder lines often go hand in hand, as both can be used to address the support of variable width roadways. Attempts can be made to avoid compounding these complexities; where variable spacing of an otherwise regular framing plan is practical, it may provide the simpler analysis, design, and construction option. When variable spacing cannot reasonably accommodate the requirements, discontinuing girders within a more regular framing plan may be considered.

For purposes of illustration, a framing plan of a three-span steel stringer unit with a terminated girder line is shown in Figure 4.7.2-1. In this example, girder lines 3, 4, and 5 will all be subject to the types of live load distribution and composite section property concerns addressed in the discussion on variable spacing. In addition, it is apparent that girder line 4 will have a vertical support of some flexibility at the left end and that girder lines 3 and 5 will receive concentrated loads from the diaphragm into which line 4 is terminated.



**Figure 4.7.2-1. A three-span framing plan of 5 girders with bearing centerlines and intermediate diaphragm locations shown. Line 4 is terminated at a diaphragm location as line 5 tapers inward.**

Grid analysis methods are suited to such conditions, in this case, provided they include the ability to introduce a node within a diaphragm element and terminate a girder line into it. It is instructive, however, to consider what can be done in such cases with simpler line girder tools. With line girder models of 3, 4, and 5, the responsibility for satisfying both equilibrium and compatibility becomes a bookkeeping task on the designer. As a hypothetical example:

- The spring stiffness of the line 4 support at the termination point can be evaluated from the line 3 and 5 models. (For this purpose, line 5 can be developed into a straight line, removing the kink.)
- Line 4 can then be analyzed, with the effect of the tapering girder spacing on loading and section properties tracked or not, at the designer's discretion.
- The reactions at the flexible support can be transferred to line 3 and 5 models, perhaps considering the envelope of simple beam and propped cantilever (fixed at 3) assumptions for behavior of the transferring diaphragm.
- Proceed with analysis of lines 3 and (developed) 5, again considering or neglecting variable spacing effects as may be practical and appropriate.

Such an approach has the benefit of relying on the more basic and widely understood line girder analysis tools. It also presses the designer to contemplate the mechanics of the framing behavior at the girder termination point, drawing attention to the loads and connection forces that will be required for the special diaphragm design. At a minimum, this approach would be a useable check of a more sophisticated grid or system analysis. (In this example, restoring the kink of line 5, after analyzing a developed straight line girder model, would present another important exercise in ensuring that the final design and details reflect the requirements of equilibrium and compatibility.)

### 4.7.3 Transfer Girders

Examples of transfer girders in bridge applications arise in conjunction with discontinuous girder lines (see Article 4.7.2) and in the straddle bents and integral pier caps (discussed in Article 3.14.3). These are distinguished from the typical girder condition addressed by the *AASHTO LRFD Bridge Design Specifications* (8), which to some degree assume and specialize their requirements to the case of longitudinal primary members of a regular framing plan. Live loads on transfer girders are not established by distribution factors. The span-to-depth requirements and recommended deflection limits of the specification are not tuned to the conditions of transfer girders. In contrast, transfer girder conditions present a purer structural beam problem of a span of some length and number of supports, controlled by one or more large point load reactions (the loads to be transferred).

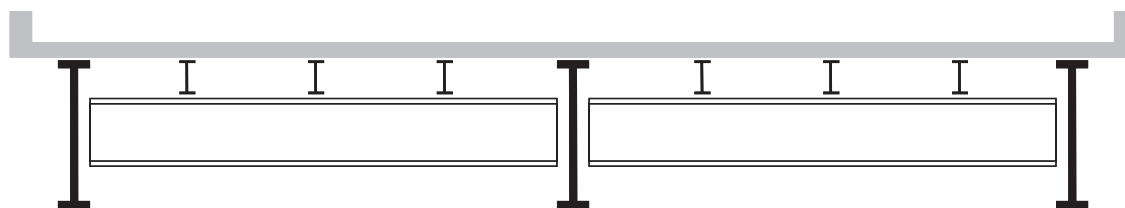
At the same time, there is nothing that exempts transfer girders in a bridge from all of the usual specification requirements on flexural and shear response to strength, serviceability, and fatigue limit states. The analysis, however, is not amenable to line-girder methods, and code-checking software of any type is not typically geared to transfer girder configurations. Thus, analysis of transfer girders often becomes an offline exercise executed in an independent model, loaded with point loads that

may arise as the reaction data from a line girder or grid analyses. The sophistication of a transfer girder analysis model can vary, from a simple line of beam elements connecting load and support points, to a detailed mesh of plate elements reflecting webs, flanges, stiffeners, and other structural elements depending on the complexity of the situation and the needs for design.

Judgment is often required in selecting a manageable number of live load conditions that adequately represent feasible truck locations above and potentially controlling demands on the girder itself. Unfavorable live load positions and lower-limit load factors on dead load should be used to explore the possibility of uplift in transfer girders with multiple supports or long overhangs. Unusual connection details that can arise in transfer girders should be evaluated for potential exposure to distortion-induced fatigue.

#### 4.7.4 Girder-Substringer Systems

A typical girder-substringer configuration is illustrated in Figure 4.7.4-1. In this case, three main girders are shown. At intervals associated with typical brace points of the main girders, floor beams span between them. Within each girder bay, a series of smaller, more closely spaced substringers are carried on the floor beams and support the deck. Notably, this cross-section configuration does not appear in the *AASHTO LRFD Bridge Design Specifications* (8) gallery of types for which live load distribution factors are provided (Section 3 and 4).



**Figure 4.7.4-1. Cross-section view of a girder-substringer system.**

Basic 2D and advanced grid analysis methods can readily capture the distributions of self-weight and non-composite dead load in such a system. They can also address the question of live load distribution, while capturing demands on all major elements (stringers, floor beams, and girders) simultaneously in a single model. The tradeoff is the increased complexity in live load application, the proliferation of load placement options that come with consideration of transverse location in addition to longitudinal, the proliferation of output, and potentially the post-processing demands of assembling composite section forces from disparate model elements.

An analysis approach that may be considered is the use of a basic 2D grid model to explore only the load distribution properties of the system. A manageable regime of unit line load placements; unit area load placements of one lane in width; and full-deck area load can provide moment, shear, and reaction results for the stringers and girders. By comparison of such results to cases in which similar loadings are applied directly to isolated models of a stringer or girder, effective live load distribution factors can be extracted. These can then be used in line girder type analyses, provided the capacity for override of typical AASHTO distribution factors is available. The stringers in such models would be supported at floor beam locations and girders at pier locations. The flexibility of the stringer supports at floor beam locations is arguably built in by virtue of the method used to construct distribution factors. The reaction results from the stringer model would provide input to the floor beam analyses.

The analysis of deflections is one subtlety that can arise in girder-substringer systems. If deflections are taken from a grid or other type of system analysis, it must be recognized that floor beam and stringer results are a composite of elastic deflection of the particular member plus deflection of its supports. Alternatively, if the analysis is broken out into independent analyses with floor beams and stringers isolated on idealized supports, it must be recognized that deflection of the support points is

manifested in other analysis models. Careful record keeping and accurate super positioning of results can avert errors in establishing girder camber or deck casting elevations, or both.

#### 4.7.5 Elevated T-Intersections

Elevated T-intersection bridges represent a unique combination of simple bridge structures into a complex system. Typically, the framing plan for an elevated T-intersection steel girder bridge features a fairly routine-looking steel girder bridge where along one exterior girder a series of steel corbels are installed (see Figure 4.7.5-1). These typically take the form of a series of partial depth stiffeners capped by a seat plate. Atop the seat plates will be bearings that support the ends of the girders of the intersecting bridge that is framing into the main bridge.



**Figure 4.7.5-1. Framing of an elevated T-intersection steel girder bridge in McAllen, TX. The intersecting span is supported on a series of steel corbels formed of partial depth stiffeners with a cap plate.**

The intersecting span may attach to the main bridge at or near a support or may attach at some other point in the span away from the supports. The further the point of attachment is from the supports of the main bridge, the more significant are the effects of the intersecting span's loads.

There are numerous analysis, design, detailing, fabrication, and erection considerations for elevated T-intersection bridges. Designers are cautioned to thoroughly evaluate the structure and its anticipated behavior.

For simplicity, the intersecting bridge may be evaluated separately from the main bridge. In this case, the intersecting bridge can be analyzed and designed much like a regular bridge, with the exception that at one end it will have more flexible supports, something like the case of a bridge supported on a long-span straddle bent (see Article 3.14.3). The main bridge, however, presents a more complicated analysis and design problem. The main bridge can be analyzed much like a regular steel girder bridge but with additional loads; the support reactions from the intersecting bridge must be applied as point

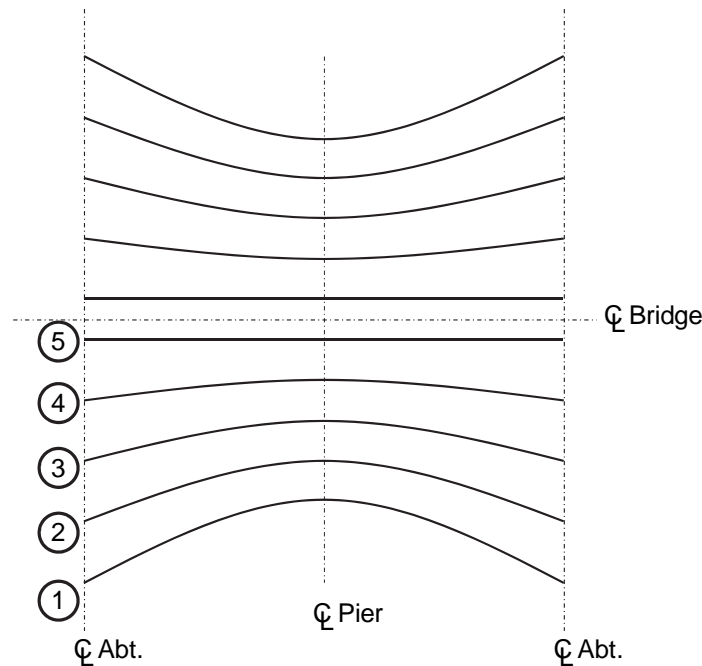
loads on the exterior girder of the main bridge. Note that the bearings are located at some offset from the web of the main bridge exterior girder, so in addition to applying the intersecting bridge reactions as vertical loads, consideration should be given to applying a concentrated torque as well. The case of dead load modeling is fairly simple as all loads are constant and static. However, the case of live load modeling is much more complex; for the main bridge, the designer must consider not only the moving live loads of traffic on the main bridge itself but also variations in the live load reactions of the intersecting bridge. Simplifying assumptions regarding the combination of main bridge and intersecting bridge live load effects can help to simplify the design, as long as the total live load envelope (maximum positive moment, negative moment, and shear effects) is adequately addressed. Care should also be taken to watch for uplift in bearings opposite to the location where the intersecting bridge frames into the main bridge.

Detailed design of the connection details, such as the corbel stiffeners, is required and should be undertaken with care. The partial depth stiffeners can be analyzed in a manner similar to bearing stiffeners, checking bearing stress, local buckling capacity, strength of welded connections, and other critical design parameters. The main bridge exterior girder should be locally checked for all applicable loads. Providing backup stiffeners on the opposite side of the web from the partial depth corbel stiffeners is advisable. Providing additional cross-frames or diaphragms in the main bridge in the vicinity of the intersecting bridge connection to the main bridge may also be prudent to help distribute the concentrated loads to adjacent girders and to help control twist of the exterior girder. Supplemental deck supports may also be required if the deck flares locally in the vicinity of the intersecting bridge's connection to the main bridge.

#### 4.7.6 Single Point Urban Interchanges

Single point urban interchanges (SPUIs) provide a compact solution for maintaining all-way access between two major roadways. A bridge structure separates the through movement on each mainline. A single, three-phase traffic signal on one of the routes controls the through movement and two sets of opposing left-turn movements. The signalized intersection is sometimes located at grade, in which case the bridge element is a relatively straightforward, though long, span. In other cases, it is the signalized intersection that gets located on the bridge structure, above the other mainline. The resulting geometry can depart severely from conventional bridge shapes and conditions can vary so much that conventional framing solutions may never emerge. As a result, SPUI structures are examples of some of the most complex challenges in bridge analysis and single structures can encompass many of the special conditions discussed in this report.

Chang and James (30) provide a detailed description of one framing solution, the basic concept of which is illustrated in Figure 4.7.6-1. The bridge unit spans between parallel closed abutment walls with a center pier. The framing plan has a bilateral symmetry about the centerline of the bridge and the centerline of the pier, and comprises lines of tub girders with variable radii. At a glance, many complicating factors are revealed, including sharp curvatures, variable girder spacing, and variably skewed support conditions. Advanced grid methods are employed in the analyses.



**Figure 4.7.6-1. Variable radii, concentric curved tub girder approach to SPUI framing.**

Figure 4.7.6-2 is a photo taken from below of one quadrant of the framing in another SPUI design. The analysis for this structure was performed using a plate- and eccentric-beam type method. In the more regular areas of the framing plan, this model was verified against conventional line girder results. Figure 4.7.6-3 is an edited version of the photo, keyed to the following illustrations of complex framing analysis features. More detail is as follows:

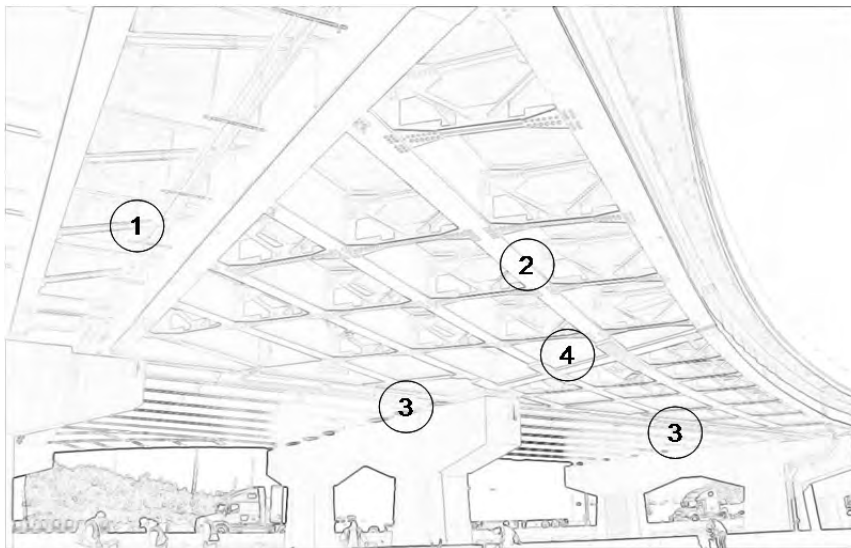
1. This is the first interior bay of 19 total bays formed by tangent parallel girders forming the core of the mainline-over bridge. The ramp and curved infill areas are framed into the fascia girder defining this bay. Because the ramp girders frame into and brace the fascia, the triangular cross-frames within this bay are designed to share vertical load but not to attempt restraint of small fascia girder rotations. The eccentric beam approach allowed faithful stick modeling and extraction of design forces for these frames. They were connected via rigid links to the girders, which were modeled as line elements at their respective neutral axes.
2. The ramp flare infill area is framed with curved girders. The cross-frames here are thus primary load carrying elements as well, with members and connections designed to analyzed forces. As is often the case, when attempts are made to faithfully model the forces in these elements, the results lead to designs and connections that can look very heavy. In comparison, straight girder framing often utilizes much lighter transverse framing from standardized details adopted without formal analysis.
3. Several discontinued girder lines are visible. The diaphragms against which these lines terminated were modeled using two-noded beam elements.
4. Traffic clearance and vertical stiffness requirements combined to warrant the use of an integral crosshead extension (transfer girder) off of the concrete pier. The ramp flare girders were interrupted but moment connected across the transfer girder. In this case, the welded I-shaped transfer girder was modeled along with the rest of the grid and the necessary data for design was extracted directly from the overall model. This integration facilitated investigation



of live load effects on the transfer girder and investigation of uplift potential under deck casting and live loads (the girder is supported on the concrete pier cap via multiple bearings).



**Figure 4.7.6-2. View of curved infill-framing area between two orthogonal regions of conventional parallel stringer framing in one of the ramp flare areas of a SPUI bridge structure.**



**Figure 4.7.6-3. Legend view; refer to enumerated discussion points in the text.**

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**GLOSSARY**

*2D Analysis Methods*—See Articles 1.2.2, 1.2.3, and 1.2.4.

*3D Analysis Methods*—See Article 1.2.5.

*3D Grid Analysis Methods*—See Article 1.2.4.

*Aspect Ratio*—Ratio of the length to the width of a rectangle.

*Beam Element*—See Article 1.2.6.

*Bearings*—A structural device that transmits loads while facilitating translation or rotation, or both.

*Bifurcating Girders*—Bifurcating girders are transversely-framed longitudinal girders where a single framing system splits into two separate framing systems at a longitudinal point, creating a fork or a Y. There is usually a transverse member (or multiple members) at the point where the framing splits. This can also be the point where a portion of the framing terminates and the other portion continues.

*Box Girder*—A steel girder with two or more webs and a single bottom flange, with either multiple separate top flanges (one at the top of each web) or a single top flange. The specific case of two webs and two separate top flanges is typically called a tub girder. See Tub Girder.

*Brick Element*—See Article 1.2.6.

*Bridge Temporary Works*—Structural members that are used for temporary support of permanent structural elements. Examples include temporary shoring towers, pier brackets, overhang falsework brackets, and top flange stiffening trusses.

*Boundary Condition*—The supports for a structural analysis model. The boundary conditions are the points where the model is grounded such that certain degrees of freedom (DOFs) are restrained.

*Composite Girder*—A steel I-girder or tub girder connected to a concrete slab with shear continuity, typically achieved via shear connectors, so that the steel girder and the slab or the longitudinal reinforcement within the slab respond to bending as a unit.

*Cross-frame*—A vertically oriented transverse truss framework connecting adjacent bending members. See also diaphragm.

*Curved Girder*—An I- or tub girder that is curved in a horizontal plane for any portion of its length.

*Dead Loads*—Permanent, nonmoving loads on a structure (e.g., self-weight of the member, the weight of a concrete deck, the weight of permanent utility attachments).

*Deck Placement*—The placement of wet concrete deck onto the structural steel girders of a steel girder bridge.

*Deck Placement Sequence*—The sequence in which various areas of the deck are placed.

*Diaphragm*—A vertically oriented solid transverse member connecting adjacent longitudinal girders or inside a tub girder. See also Cross-Frame.

*Differential Deflection*—Non-uniform deflection of adjacent girders; i.e., when one girder deflects a greater amount than an adjacent girder.

*Elevated T-Intersections*—A bridge that supports roadways approaching from three or more distinct directions, such that each roadway and its supporting structure are oriented orthogonally or nearly so to the other roadways and their supporting structures.



*FEM Analysis*—Finite element method analysis; a numerical technique for finding solutions of partial differential or integral equations, or both. FEM analysis is often used to solve structurally indeterminate problems.

*Finite Element*—See Article 1.2.6.

*Geometric Nonlinearity*—A second-order effect where structural deformations are not linearly proportional to the applied loads, as a result of changes in the structure geometry.

*Global Stability*—The stability of an entire structural system in terms of resistance to buckling (as opposed to the stability of an individual girder or other element within a structural system in terms of resistance to local buckling of that girder or element).

*Grid Analysis Methods*—A method of analysis in which all or part of the superstructure is discretized into components in a horizontal plane that represents the characteristics of the structure. See also Article 1.2.2.

*Hand Analysis Methods*—See Article 1.1.

*Horizontally Curved Bridge*—A bridge where the roadway horizontal alignment follows a curve or spiral on the structure. A horizontally curved bridge may use horizontally curved girder or may use straight or chorded girders as its primary structural framing system.

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

*Line Girder Analysis Methods*—See Article 1.1.2.

*Live Loads*—Transient vertical loads on a bridge, typically representing the weight of vehicles or pedestrians on the bridge.

*Material Nonlinearity*—A material response where stress and strain are not linearly related.

*Multiple Skew Bridge*—A bridge that is supported on bents or end bents that have different skew angles.

*No Load Fit (NLF)*—A practice where the structural steel in a bridge is detailed to fit under the idealized case of no loads on the structure (no dead load, no live load, including no self-weight). NLF conditions can be approximated in the field by the use of temporary shoring or holding cranes.

*Non-Uniform Torsion*—An internal resisting torsion in thin-walled sections that consists of St. Venant and warping torsion, in which plane sections do not remain plane. St. Venant torsion produces shear stresses and warping torsion produces normal and shear stresses.

*Non-composite Dead Loads*—Dead loads that are applied to the bare structural steel system prior to the hardening of the concrete deck (which would make the structural system composite). Examples of non-composite dead loads include self-weight of the structural steel and weight of the wet concrete deck for unshored construction.

*Nonlinear Analysis*—An analysis that directly accounts for geometric nonlinearity effects, material nonlinearity effects, or both. Also referred to as a second-order analysis.

*Normal Stress*—A stress acting perpendicular to the cross section of a structural member.

*Plate and Eccentric Beam Grid Analysis Methods*—See Article 1.2.3.

*Plate Element*—See Article 1.2.6.

*Plate Girder*—An I-shaped structural steel member made up of two flanges connected to a vertical web. In a welded plate girder, the connections between the flanges and the web are accomplished by welding.

*Primary Member*—A member that carries primary structural loads, especially gravity loads, and that is an essential part of the structural system for carrying primary structural loads. For example, structural steel girders, floor beams, and cross-frames in curved girder bridges are primary members.

*Pure Torsional Stresses*—In-plane shear stresses that vary linearly across the thickness of an element of the cross section and act in a direction parallel to the edge of the element.

*Rolled Shape*—Any steel section (angles, channels, I-shaped or H-shaped, T-shaped), which is formed by rolling mills while the steel is in a semi-molten state.

*Shell Element*—See Article 1.2.6.

*Single Point Urban Interchanges (SPUI)*—A type of roadway geometry where multiple roadways converge at a single point. A SPUI is similar to a diamond interchange but is compressed in such a way as to allow multiple simultaneous left hand turning movements to facilitate moving large volumes of traffic through an intersection in a confined space. SPUIs can occur on top of or under a bridge. When a SPUI is situated on top of a bridge it is called a structurally supported SPUI.

*Skew Angle*—The angle between the centerline of a support and a line normal to the roadway centerline. Note that different owner-agencies follow different protocols for measuring and denoting skew angles.

*Steel Dead Load Fit (SDLF)*—A practice where the structural steel in a bridge is detailed to fit under the application of the self-weight of the structural steel only.

*Steel Erection*—The process of lifting, placing, and connecting structural steel elements in their intended final position in a bridge.

*Torsion*—The twisting of a structural element due to an applied torque (twisting moment).

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

*Total Dead Load Fit (TDLF)*—A practice where the structural steel in a bridge is detailed to fit under the application of all dead load (including self-weight of the steel as well as the weight of the wet concrete deck).

*Tub Girder*—A steel girder with two webs and a single bottom flange, with two separate top flanges (one at the top of each web). There is generally some type of bracing between the top flanges and internal diaphragms or cross-frames within the box section. See also box girder.

*V-Load Method*—An analysis in which curved I-girders are represented by equivalent straight girders and the effects of curvature are represented by vertical and lateral forces applied at the cross frame locations. Cross frame properties are not considered in the analysis. See also Article 1.1.3.

*Warping Normal Stresses*—Stresses that act perpendicular to the surface of the cross section and are induced by warping torsion. They are constant across the thickness of an element of the cross section but vary along the length of the element.

*Warping Shear Stresses*—In-plane shear stresses that are constant across the thickness of an element of the cross section, but vary in magnitude along the length of the element. They act in a direction parallel to the edge of the element.

## APPENDIX A SURVEY OF CURRENT PRACTICE

### A.1 Background

One of Task Group 13's first objectives was to survey the current practices of steel girder design throughout the US. The Task Group wanted to see how the various owner-agencies and other organizations are designing various types of steel girder bridges with the following goals in mind:

- Assessing the current state of practice in steel girder design,
- Noting trends in steel girder design,
- Collecting guidelines, methods, and tools, and
- Collating a set of best practices, guidelines, and suggestions for publication in an AASHTO/NSBA Steel Bridge Collaboration steel girder design document.

The survey addressed a wide variety of steel girder bridge types. If there was a particular type of steel girder bridge that a given organization did not typically design, they were encouraged either to skip that section of the survey or to offer opinions as to how they would approach that type of design if they were to perform it.

### A.2 Participation in the Survey

The survey was conducted primarily in the summer of 2007, but responses were received through the summer of 2008. A total of 37 state DOTs responded to the survey. These surveys were grouped and collectively titled the "AASHTO responses." Most AASHTO surveys were filled out in a relatively complete manner in terms of answering the check box questions, but expanded commentary by the survey respondents was sporadic.

A total of six responses were received from railroad bridge engineers, representing either railroad owner-agencies or consulting engineers who specialize in railroad bridge engineering. These surveys were grouped and collectively titled "AREMA responses." In general, the AREMA surveys were not completely filled out; the respondents typically limited their comments to the tangent (straight) plate girder/rolled beam questions.

A total of 21 responses were received from others. Eighteen of these were from consulting engineers, two from owner-agencies other than U.S. state DOTs, and one from a fabricator/erector. These surveys were grouped and collectively titled "At large responses." Most at large surveys were filled out in a relatively complete manner in terms of answering the check box questions but expanded commentary by the survey respondents was sporadic.

Due to the limited number of AREMA and at large responses, the bulk of this summary will be devoted to description of the AASHTO responses. Comments on the AREMA and at large responses will be limited and will mostly describe any significant differences from the AASHTO responses.

### A.3 AASHTO Responses

#### A.3.1 Overall Trends and Comments

As might be expected, there were wide variations in the responses to various survey questions from the state DOTs. Each state is somewhat unique in terms of how much of their bridge design and construction volume consists of steel girder bridges, as well as being unique in the specific nature of their steel girder bridges. Some states do no steel girder design or construction at all, while others use steel girders extensively in a wide range of simple and complex bridge applications. Some states use steel girders primarily in simple applications (e.g., tangent, non-skewed rolled beams and plate girders), while others use steel girders primarily in complex applications (e.g., longer spans, curved

girder bridges) where prestressed concrete girders or other types of bridges are not feasible. As a result, there are understandable substantive differences in the responses to many of the questions in the survey.

However, many general trends were also apparent. There appears to be a growing interest in some of the more subtle nuances of steel girder behavior and analysis as designers face more challenging steel bridge projects featuring longer spans and more severe curvature and skew. At the same time, there is still a great reliance on, and confidence in, simple, tried and true analysis methods.

The following general points can be drawn from this survey:

- Some states are realizing a need for more careful analysis of some of the more complex steel girder bridges they are increasingly facing.
- Some states are content with their current tools and methods. This is often linked to trends in those states that are not leading them to face more complex steel bridge design and construction projects.
- Among all states, even among the subset of states that are facing more complex steel girder bridges and are considering the need for more rigorous analysis methods, there is a wide variety of practices being used for steel girder bridge design.

More detailed summaries of the answers to the various survey questions are presented below, grouped to match the titled sections of the survey.

### **A.3.2 Responses to General Questions**

In terms of identified needs for better guidance and understanding in the area of analysis of steel girder bridges:

- Behavior and analysis of skewed bridges (numerous references).
- Behavior and analysis of curved girder bridges (several references).
- Constructability of steel girders, stability of steel girders during construction, and behavior of steel girders through all stages/phases of construction.

In terms of types or classes of steel girder bridges where there have been more than occasional problems during construction, where the problems can be traced back to issues with the original analysis and design, there were several references to issues with:

- Phased construction.
- Dead load deflections.
- Skewed bridges.

One state responded with a very interesting story of a curved bridge that exhibited a deflected shape upon erection was significantly different from the deflected shape predicted by the originally used grid analysis. The structure subsequently needed to be reanalyzed during construction using a nonlinear 3D analysis to address second-order effects. The state believes this to be an example where a grid analysis approach “was not sufficient to capture the behavior of the structure and [we] are looking to establish some possible guidelines for curved structures regarding the level of analysis that needs to be performed depending upon geometry and complexity.”

### A.3.3 Responses to Specific Questions

#### A.3.3.1 *Responses to Questions about Tangent (Straight) Steel Plate Girders or Rolled Beams with No Skew or Limited Skew (Skewed Less than 20° from Non-Skewed)*

Approximately 60 percent of the responding states listed these types of bridges as being commonly used. They are used in both integral end bent and conventional end bent applications. They are typically used in both simple-span and continuous applications, on both narrow and wide bridges. They are used in a wide range of span ranges, with medium span ranges (100 ft to 250 ft) being most prevalent.

Some form of line girder analysis, often by hand calculations but more often by in-house or commercial software, is by far the most prevalent design method being used. A wide range of software packages (10 or more different commercial packages and probably an equal or greater number of in-house software packages) is being used.

A similarly wide range of reference documents is used as guidelines for these designs.

A wide variety of comments were received regarding widening or phased construction, or both, including a few problems, but there did not appear to be a significant, direct link from any problems to the analysis methods used.

#### A.3.3.2 *Responses to Questions about Tangent (Straight) Steel Plate Girders or Rolled Beams, Significantly Skewed (Skewed More than 20° from Non-Skewed)*

As might be expected, these types of bridges were less often listed as being routinely used but still nearly half of the responding states listed them as such.

The use of integral end bents with skewed tangent girder bridges is less frequent than with non-skewed tangent girder bridges and even the states that routinely use integral end bents limit the use of integral end bents to bridges with comparatively small skew angles.

These bridge types are typically used in both simple-span and continuous applications, on both narrow and wide bridges. They are used in a wide range of span ranges, with medium span ranges (100 ft to 250 ft) being most prevalent.

Again, as for non-skewed tangent girder bridges, some form of line girder analysis, often by hand calculations but more often by in-house or commercial software, is by far the most prevalent design method being used. Likewise, a wide range of software packages is being used.

However, for skewed tangent girder bridges, the use of either a grid analysis or a 3D analysis is more prevalent, being mentioned only eight times for non-skewed bridges but 16 times for skewed bridges.

Special considerations and limitations mentioned for skewed bridges were more specific than for non-skewed bridges, with several responses mentioning special consideration to cross frame analysis, design, and detailing.

Most of the issues associated with widening or phased construction, or both, for skewed tangent girder bridges seem to be similar to those listed for non-skewed tangent girder bridges.

#### A.3.3.3 *Responses to Questions about Curved Steel Plate Girders or Rolled Beams with No Skew or Limited Skew (Skewed Less than 20° from Non-Skewed)*

These types of bridges were reported as being less frequently used than tangent (straight) girder bridges but still more than half of the responding states reported using them either routinely or occasionally, with some mentioning that steel girders are often selected for curved bridges, if there is no way to avoid curvature on the bridge.

The use of integral end bents with curved steel girder bridges appears to be much less common than for tangent (straight) girder bridges.

Curved steel girder bridges appeared to be more commonly used in continuous rather than simple-span applications, although simple-span applications were certainly not rare. Curved steel girder bridges also appeared to be more commonly used in narrow bridge applications rather than wider bridge applications. They are used in a wide range of span ranges, once again with medium span ranges (100 ft to 250 ft) being most prevalent.

In terms of analysis methods used, the V-load method, either by hand or via a computer program, was listed as being used or recommended by 18 of the responding states, although two of those respondents described DESCUS or MDX as their “V-load software package.” Meanwhile, 23 of the responding states said that they used or recommended grid analysis for curved, non-skewed bridges. Seven of the responding states said that they used or recommended 3D analysis.

Note that for this question, the respondents were directed to indicate all methods that apply, hence the number of responses sums to more than the 37 responding states.

In summary, the V-load method appeared to still be popular and commonly used, although grid analysis methods appeared to be most prevalent for these types of structures, with 3D analysis methods being the least prevalent.

In terms of limitations, none of the responding states specifically indicated limits on span length or degree of curvature for any given analysis approach (V-load, grid, or 3D), although one state hinted that limits on when to move from grid to 3D were being evaluated.

There was much less comment on either widening or phased construction, or both, of curved, non-skewed bridges than was provided for tangent (straight) girder bridges. Of perhaps most significance were a few comments indicating that curved steel bridges were seldom if ever widened, at least in some states.

#### *A.3.3.4 Responses to Questions about Curved Steel Plate Girders or Rolled Beams, Significantly Skewed (Skewed More than 20° from Non-Skewed)*

The trend for less application with more complexity continued; curved steel girder bridges with significant skew were listed as less frequently used than curved non-skewed steel girder bridges.

The use of integral end bents for curved steel girder bridges with significant skew was even less than for curved, non-skewed steel girder bridges.

Curved steel girder bridges with significant skew appeared to be more commonly used in continuous rather than simple-span applications, although simple-span applications were certainly not rare. Curved steel girder bridges with significant skew also appeared to be more commonly used in narrow bridge applications rather than wider bridge applications. They are used in a wide collection of span ranges, once again with medium span ranges (100 ft to 250 ft) being most prevalent.

As opposed to curved, non-skewed steel girder bridges, curved steel girder bridges with significant skew were less likely to be analyzed by the V-load method, although the V-load method was still listed as used or recommended by 13 of the responding states. Meanwhile, the use of grid or 3D analysis methods was more likely for curved steel girder bridges with significant skew, with grid analysis methods being used or recommended by 20 of the responding states, and 3D analysis methods being used or recommended by 10 of the responding states. Note that for this question, the respondents were directed to indicate all methods that apply, hence the number of responses sums to more than the 37 responding states. In summary:

- The V-load method appeared to still be popular and commonly used but less so than for curved, non-skewed bridges, with grid analysis methods appearing to still be most prevalent for these types of structures.
- 3D analysis methods are more popular for curved, significantly skewed bridges than for curved, non-skewed bridges.

Few comments were received regarding limitations or special considerations, or for wider bridges or phased construction, or both.

#### A.3.3.5 *Responses to Questions about Tub Girders or Box Girders*

Fewer responses were received regarding tub or box girders than were received regarding rolled beams and plate girders. In general, it appeared that tub or box girders are much less common among the states.

In terms of analysis methods, the trends for steel tub or box girders seemed to match the trends for rolled beams and plate girders:

- Line girder analysis methods are more prevalent than grid or 3D analysis methods for tangent (straight) girder bridges.
- Grid analysis methods, and to a lesser extent 3D analysis methods, are more prevalent for curved girder bridges.
- There was, however, a noticeable shift toward more refined methods being preferred for tub or box girders versus rolled beams or plate girders, i.e., for tangent tub or box girders there was a greater percentage of votes for grid analysis than was seen for tangent rolled beams or plate girders.
- Likewise, for curved tub or box girders there were a greater percentage of votes for 3D analysis than was seen for curved rolled beams or plate girders.

#### A.3.3.6 *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)*

There was little response to these questions and few, if any, trends were observed. Single point urban interchanges (SPUIs) were mentioned as becoming more prevalent. Line girder, grid, and 3D analysis methods were all listed as being recommended by nearly equal numbers but not much could be drawn from that voting given the wide range of structures covered by this question and the lack of much in the way of expanded responses from the survey respondents.

### A.4 AREMA Responses

There were only six AREMA responses and, for the most part, these responses were limited solely to answering questions about tangent (straight) rolled beams and plate girders, which is not surprising as steel girder railroad bridges rarely if ever use curved girders, tub, or box girders. In addition, continuous span applications were reported as being rare among railroad bridges. Most applications were for narrower bridges, most often with shorter spans (100 ft or less) but occasionally for medium spans (100 ft to 250 ft).

Given the much simpler nature of railroad bridges, it was not surprising to see that line girder analysis methods predominated, particularly for non-skewed bridges. Grid analysis and 3D analysis were mentioned as being used or recommended to some extent for skewed bridges.

Very little discussion was provided regarding widenings or phased construction, or both, for railroad bridges.

### **A.5 Other Responses**

The other responses were fewer than the AASHTO responses (21 total) and were from a wide variety of respondents (18 consulting engineers, two non-state DOT owner agencies, and one fabricator-erector).

In general, the other responses appeared to track with the AASHTO responses. There was perhaps greater preference for more refined analysis approaches (e.g., grid versus line girder, 3D versus grid) for each structure type. There also appeared to be less reliance on in-house programs and more reliance on commercial software. However, overall, with such a small pool of respondents with such divergent backgrounds, not many trends could be determined from these survey responses.



## B.1 Recommendations on Methods of Analysis

A substantial number of studies were conducted by White et al. (88) to determine the ability of approximate 1D and 2D methods of analysis to capture the behavior predicted by refined 3D finite element models. To evaluate 1D methods, a commonly available commercial line-girder analysis program, STLBRIDGE, was used to analyze the behavior of straight skewed I- and tub girder bridges. The 1D analysis of curved and curved- skewed I-girder bridges was based on the V-load method (73) using the software VANCK. The 1D analysis of curved and curved- skewed tub girder bridges was based on a line-girder analysis coupled with additional calculations based on the M/R method (81). To evaluate 2D methods two commercially available software programs, typically employed by bridge designers, were used to investigate the behavior of these same bridges: the software MDX for analysis using a conventional 2D-grid approach and the capabilities of LARSA-4D for analysis using a conventional 2D-frame approach. To evaluate linear elastic 3D finite element analysis methods, the software program ABAQUS was used to investigate the behavior of these same bridges. The 1D, 2D, and linear elastic 3D analysis results were compared to benchmark nonlinear “simulation” 3D finite element analysis solutions, and also prepared using the software program ABAQUS including the modeling of second-order effects (geometric nonlinearity). Where possible, extant bridges were evaluated and if those bridges had been instrumented, the nonlinear simulation benchmark analysis results were validated against measured responses.

For clarity, it should be emphasized that the research conducted by White et al. (88) focused predominantly on non-composite behavior and non-composite modeling. The findings regarding the accuracy of the various methods presented in Tables B.2-1, B.3-1, and B.3-2 are only valid for evaluating the accuracy of non-composite, dead load models.

It should be noted that the findings regarding the accuracy of the various methods presented here reflect the state of the industry at the time of the writing of the 2nd Edition of G13.1, and the state of the industry at the time White et al. (88) performed their research. White et al. (88) also recommended several enhancements to 2-D analysis methods, which would significantly improve their accuracy; the results presented here in Tables B.2-1, B.3-1, and B.3-2 include consideration of the anticipated implementation of those recommendations. Summaries of each of the two primary proposed improvements to 2D analysis methods are provided in Section 3.11, Section 3.11.3, Section 3.11.4, and Section 3.12 of this document.

## B.2 I-Girder Bridges

A quantitative assessment of the analysis accuracy was obtained by identifying error measures that compared the approximate (1D and 2D methods) solutions to the 3D FEA benchmark solutions. Using the quantitative assessments the various methods of analysis were ranked based on a scoring system developed to provide a comparative evaluation of each analysis method with regard to the accuracy of its analysis predictions for various structural responses.

Tables B-1, B-2, and B-3 summarize the scoring system for the various methods and behaviors monitored. The scoring criterion is as follows:

- A grade of A is assigned when the normalized mean error is less than or equal to 6 percent, reflecting excellent accuracy of the analysis predictions.
- A grade of B is assigned when the normalized mean error is between 7 percent and 12 percent, reflecting a case where the analysis predictions are in “reasonable agreement” with the benchmark analysis results.

- A grade of C is assigned when the normalized mean error is between 13 percent and 20 percent, reflecting a case where the analysis predictions start to deviate “significantly” from the benchmark analysis results.
- A grade of D is assigned when the normalized mean error is between 21 percent and 30 percent indicating a case where the analysis predictions are poor, but may be considered acceptable in some situations.
- A score of F is assigned if the normalized mean errors are above the 30 percent limit. At this level of deviation from the benchmark analysis results, the subject approximate analysis method is considered unreliable and inadequate for design.

**Table B.2-1. Matrix for Recommended Level of Analysis—I-Girder Bridges, Non-Composite Dead Load Analysis Models**

Response	Geometry	Worst-Case Scores			Mode of Scores		
		Traditional 2D-Grid	1D-Line Girder	Improved 2D-Grid <sup>g</sup>	Traditional 2D-Grid	1D-Line Girder	Improved 2D-Grid <sup>g</sup>
Major-Axis Bending Stresses	$C (I_c \leq 1)$	B	B	A	A	B	A
	$C (I_c > 1)$	D	C	A	B	C	A
	$S (I_s < 0.30)$	B	B	A	A	A	A
	$S (0.30 \leq I_s < 0.65)$	B	C	A	B	B	A
	$S (I_s > 0.65)$	D	D	A	C	C	A
	$C\&S (I_c > 0.5 \& I_s > 0.1)$	D	F	A	B	C	A
Vertical Displacements	$C (I_c \leq 1)$	B	C	A	A	B	A
	$C (I_c > 1)$	F	D	A	F	C	A
	$S (I_s < 0.30)$	B	A	A	A	A	A
	$S (0.30 \leq I_s < 0.65)$	B	B	A	A	B	A
	$S (I_s > 0.65)$	D	D	A	C	C	A
	$C\&S (I_c > 0.5 \& I_s > 0.1)$	F	F	A	F	C	A
Cross-Frame Forces	$C (I_c \leq 1)$	C	C	B	B	B	A
	$C (I_c > 1)$	F	D	B	C	C	A
	$S (I_s < 0.30)$	NA <sup>a</sup>	NA <sup>a</sup>	B	NA <sup>a</sup>	NA <sup>a</sup>	A
	$S (0.30 \leq I_s < 0.65)$	F <sup>b</sup>	NA <sup>c</sup>	B	F <sup>b</sup>	NA <sup>c</sup>	A
	$S (I_s > 0.65)$	F <sup>b</sup>	NA <sup>c</sup>	B	F <sup>b</sup>	NA <sup>c</sup>	A
	$C\&S (I_c > 0.5 \& I_s > 0.1)$	F <sup>b</sup>	NA <sup>c</sup>	B	F <sup>b</sup>	NA <sup>c</sup>	A
Flange Lateral Bending Stresses	$C (I_c \leq 1)$	C	C	C	B	B	B
	$C (I_c > 1)$	F	D	C	C	C	B
	$S (I_s < 0.30)$	NA <sup>d</sup>	NA <sup>d</sup>	NA <sup>d</sup>	NA <sup>d</sup>	NA <sup>d</sup>	NA <sup>d</sup>
	$S (0.30 \leq I_s < 0.65)$	F <sup>b</sup>	NA <sup>e</sup>	C	F <sup>b</sup>	NA <sup>e</sup>	B
	$S (I_s > 0.65)$	F <sup>b</sup>	NA <sup>e</sup>	C	F <sup>b</sup>	NA <sup>e</sup>	B
	$C\&S (I_c > 0.5 \& I_s > 0.1)$	F <sup>b</sup>	NA <sup>e</sup>	C	F <sup>b</sup>	NA <sup>e</sup>	B
Girder Layover at Bearings	$C (I_c \leq 1)$	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>
	$C (I_c > 1)$	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>	NA <sup>f</sup>
	$S (I_s < 0.30)$	B	A	A	A	A	A
	$S (0.30 \leq I_s < 0.65)$	B	B	A	A	B	A
	$S (I_s \geq 0.65)$	D	D	B	C	C	A
	$C\&S (I_c > 0.5 \& I_s > 0.1)$	F	F	B	F	C	A

<sup>a</sup> Magnitudes should be negligible for bridges that are properly designed & detailed. The cross-frame design is likely to be controlled by considerations other than gravity-load forces.

<sup>b</sup> Results are highly inaccurate due to modeling deficiencies addressed in Ch. 6 of the NCHRP 12-79 Task 8 report. The improved 2D-grid method discussed in this Ch. 6 provides an accurate estimate of these forces.

<sup>c</sup> Line-girder analysis provides no estimate of cross-frame forces associated with skew.

<sup>d</sup> The flange lateral bending stresses tend to be small. AASHTO Article C6.10.1 may be used as a conservative estimate of the flange lateral bending stresses due to skew.

<sup>e</sup> Line-girder analysis provides no estimate of girder flange lateral bending stresses associated with skew.

<sup>f</sup> Magnitudes should be negligible for bridges that are properly designed & detailed.

<sup>g</sup> The improved 2D-grid method requires the usage of an equivalent St. Venant torsion constant, which estimates the influence of the girder warping response on the torsional stiffness, as well as a Timoshenko beam cross-frame model that accounts for both the shear and bending flexibility of the cross-frames. See Articles 3.11 and 3.12 of this document for detailed discussions of these improvements. In addition, the improved 2D-grid method is limited to the analysis of systems with at least two girders connected by enough cross-frames such that  $I_c$  is less than or equal to 20.

The normalized mean error is calculated as

$$\mu_e = \frac{1}{N \cdot R_{FEAmax}} \sum_{i=1}^N e_i \quad (B2-1)$$

where  $N$  is the total number of sampling points in the approximate model,  $R_{FEAmax}$  is the absolute value of the maximum response obtained from the FEA, and  $e_i$  is the absolute value of the error relative to the 3D FEA benchmark solution evaluated at point  $i$ .

$$e_i = |R_{approx} - R_{FEA}| \quad (B2-2)$$

The summation in the above is computed for each girder line along the full length of the bridge, and the largest resulting value is reported as the normalized mean error for the bridge. The error measure  $\mu_e$  is useful for the overall assessment of the analysis accuracy since this measure is insensitive to isolated discrepancies, which can be due to minor shifting of the response predictions, etc. The normalized local maximum errors,  $e_i / R_{FEAmax}$ , are generally somewhat larger than the normalized mean error. Also, in many situations, unconservative error at one location in the bridge leads to comparable conservative error at another location. Hence, it is simpler to not consider the sign of the error as part of the overall assessment of the analysis accuracy.

In Table B.2-1, the scoring for the various measured responses is subdivided into six categories based on the bridge geometry. These bridge categories are defined as follows:

- Curved bridges with no skew are identified in the Geometry column by the letter “C.”
- The curved bridges are further divided into two subcategories, based on the connectivity index,  $I_C$ , defined as:

$$I_C = \frac{15000}{R(n_{cf} + 1)m} \quad (B2-3)$$

where  $R$  is the minimum radius of curvature,  $n_{cf}$  is the number of intermediate cross-frames in the span, and  $m$  is a constant taken equal to 1 for simple-span bridges and 2 for continuous-span bridges. In bridges with multiple spans,  $I_C$  is taken as the largest value obtained from any of the spans.

- Straight-skewed bridges with no curvature are identified in the Geometry column by the letter “S.”
- The straight-skewed bridges are further divided into three sub-categories, based on the skew index,  $I_S$  where  $I_S$  is taken as:

$$I_S = \frac{w \tan \theta}{L_s} \quad (B2-4)$$

where  $w$  is the width of the bridge measured between fascia girders,  $\theta$  is the skew angle measured from a line perpendicular to the tangent of the bridge centerline, and  $L_s$  is the span length at the bridge centerline. In bridges with unequal skew of at the bearing lines,  $\theta$  is taken as the angle of the bearing line with the largest skew.

- Bridges that are both curved and skewed are identified in the Geometry column by the letters “C&S.”

Two letter grades are indicated for each of the cells in Table B.2-1. The first letter grade corresponds to the worst-case results encountered from either of the two 2D-grid solutions considered by White et al. (88), or from the 1D-line girder calculations, within each of the specified categories. The second letter grade indicates the mode of the letter grades for that category, i.e., the letter grade encountered most often for that category.

Table B-1 can be used to determine when a certain analysis method can be reasonably expected to produce acceptable results. The following two examples illustrate how Table B.2-1 is to be used.

### B.2.1 I-Girder Bridge Level of Analysis Example 1

Consider a horizontally curved steel I-girder bridge with radial supports, “regular” geometry (constant girder spacing, constant deck width, relatively uniform cross frame spacing, etc.), and  $I_C \leq 1$ , for which the engineer wants to perform a traditional 2D-grid analysis to determine the forces and displacements during critical stages of the erection sequence. (It should be noted that if  $I_C$  is calculated for an intermediate stage of the steel erection in which some of the cross-frames have not yet been placed, the number of intermediate cross-frames  $n_{cf}$  in Eq. B2-3 should be taken as the number installed in the erection stage that is being checked. In addition, the radius of curvature  $R$  and the constant  $m$  should correspond to the specific intermediate stage of construction being evaluated, not the bridge in its final erected configuration.)

For the girder major-axis bending stresses and vertical displacements ( $f_b$  and  $\Delta$ ), the results are expected to deviate somewhat from those of a 3D analysis in general, because a worst-case score of B is assigned in Table B.2-1 for all of these response quantities. The worst-case normalized mean error in these results from the 2D-grid analysis will typically range from 7 to 12 percent, as compared to the results from a refined geometric nonlinear 3D FEA. However, one can expect that for most bridges, the errors will be less than or equal to 6 percent, based on the mode score of A for both of these responses.

Therefore, in this example, if the major-axis bending stress results and vertical displacement results are of prime interest, a 2D-grid model should be sufficient if worst-case errors of approximately 12 percent are acceptable. Given that the bridge has very “regular” geometry, it is likely that the  $f_b$  and  $\Delta$  errors are less than or equal to 6 percent. (The worst-case score is considered as the appropriate one to consider when designing a bridge with complicating features such as a poor span balance, or other “less regular” geometry characteristics.)

It is important to note that the engineer can “compensate” for potential unconservative major-axis bending stress errors in the design by adjusting the target performance ratios desired for the construction engineering analysis. For example, with the above bridge, the engineer may require that the performance ratio be less than or equal to  $1/1.12 = 0.89$  or  $1/1.06 = 0.94$  for the girder flexural resistance checks to gain some further confidence in the adequacy of the analysis. Conversely, over-prediction and under-prediction of the vertical displacements can be equally bad. Nevertheless, 12 percent or 6 percent displacement error may be of little consequence if the magnitude of the displacements is relatively small, or if the deflections are being calculated at an early stage of the steel erection and it is expected that any resulting displacement incompatibilities or loss of geometric control can be subsequently resolved. However, if the magnitude of the displacements is large or if it is expected that the resulting errors or displacement incompatibilities may be difficult to resolve, then the engineer should consider conducting a 3D FEA of the subject construction stage to gain further confidence in the calculated displacements. This step in the application of Table B.2-1 is where the

bridge span length enters as an important factor, since longer-span bridges tend to have larger displacements.

It should be noted that compared to the creation of 3D FEA models for overall bridge design, including calculation of live load effects, the development of a 3D FEA model for several specific construction stages that may be of concern involves a relatively small amount of effort. This is particularly the case with many of the modern software interfaces that facilitate the definition of the overall bridge geometry.

For calculation of the girder flange lateral bending stresses and the cross frame forces in the above example bridge, the worst-case errors are expected to be larger, on the order of 13 to 20 percent (corresponding to a grade of C for both of these responses). However, the mode score is B, and since the bridge has very regular geometry, it is likely that the normalized mean error in the flange lateral bending stresses and cross frame forces is less than 12 percent. If these errors are acceptable in the engineer's judgment, then the 2D-grid analysis should be acceptable for the construction engineering calculations. As noted above, the engineer can compensate for these potential errors by reducing the target performance indices. With respect to the flange lateral bending stress, it should be noted that the  $f_c$  values are multiplied by  $1/3$  in the AASHTO  $1/3$  rule equations. Therefore, the errors in  $f_c$  have less of an influence on the performance ratio errors than errors in  $f_b$ . When checking the AASHTO flange-yielding limit for constructability, both  $f_c$  and  $f_b$  have equal weights. Based on these considerations, the best way to compensate for different potential unconservative errors in the  $f_c$  and  $f_b$  values is to multiply the calculated stresses from the 2D-grid analysis by 1.20 and 1.12 (or 1.12 and 1.06) respectively prior to checking the performance ratios.

### B.2.2 I-Girder Bridge Level of Analysis Example 2

Consider a straight steel I-girder bridge with skewed supports and a skew index,  $I_s = 0.35$  (corresponding to the intermediate erection stage being evaluated) for which the engineer wants to perform a traditional 2D-grid analysis to determine the forces and displacements during critical stages of the erection sequence.

After reviewing Table B.2-1, it is observed that for major axis bending stresses and vertical deflections, a worst-case score of B is shown for straight skewed I-girder bridges with  $0.30 < I_s \leq 0.65$ . Furthermore, it can be observed that the mode of the scores for these bridge types is a B for the major-axis bending stresses and an A for the vertical displacements. Therefore, a properly prepared conventional 2D-grid analysis would be expected to produce major-axis bending stress and vertical deflection results that compare reasonably well with the results of a second-order elastic 3D FEA, such that the normalized mean error would be expected to be less than or equal to 12 percent.

If the layout of the cross-frames in the skewed bridge is such that overly stiff (nuisance) transverse load paths are alleviated, the engineer may expect that the error in the displacement calculations may be close to 6 percent or less. In this case, the engineer should be reasonably confident in the 2D-grid results for the calculation of these responses. As noted in the previous example, the potential unconservative errors in the stresses can be compensated for in the construction engineering design checks; however, positive or negative displacement errors are equally bad.

The girder layover at the skewed bearing lines is often of key interest in skewed I-girder bridges. Table B.2-1 shows that the girder layover calculations have essentially the same magnitude of errors and resulting grades as the girder vertical displacements. This is because the skewed bearing line cross-frames are generally relatively rigid in their own planes compared to the stiffness of the girders.

Hence, the girder layovers are essentially proportional to the girder major-axis bending rotations at the skewed bearing lines.

For the calculation of the cross frame forces and/or the girder flange lateral bending stresses in the above example, one can observe that the conventional 2D-grid procedures are entirely unreliable. That is, the scores in Table B.2-1 are uniformly an F. The reason for this poor performance of the traditional 2D-grid methods is the ordinary modeling of the girder torsional properties using only the St. Venant torsional stiffness  $GJ/L$ . The physical girder torsional stiffnesses are generally much larger due to restraint of warping, i.e., flange lateral bending and effects. In addition, for wide skewed bridges and/or for skewed bridges containing specific overly-stiff (nuisance) transverse load paths, the limited accuracy of the cross frame equivalent beam stiffness models used in conventional 2D-grid methods may lead to a dramatic loss of accuracy in the cross frame forces.

Lastly, conventional 2D-grid methods generally do not include any calculations of the girder flange lateral bending stresses due to skew. Hence, the score for the calculation of the flange lateral bending stresses is also an F in Table B.2-1.

White et al. (88) recommends several important modifications to conventional 2D-grid procedures that are relatively simple for software providers to implement yet provide substantial improvements in the analysis accuracy (see also Section 3.11, Section 3.11.3, Section 3.11.4, and Section 3.12 of this document). To realize the benefits of these improvements in typical bridge design practice it will be necessary for commercial 2D-grid software providers to implement these types of improvements, since manual implementation of the improvements tends to be cumbersome and time consuming for the engineer. Therefore, this document focuses solely on the accuracy of conventional 2D-grid and 1D line-girder procedures.

### B.2.3 I-Girder Bridge Level of Analysis Example 3

Consider a curved girder bridge with radial supports and a connectivity index  $IC > 1$ . This would be a bridge with relatively tight radius and relatively few cross-frames connecting the girders. A bridge like this would be expected to exhibit relatively less system behavior and relatively more individual girder behavior in response to curvature. Upon initial inspection, the scores for both 1D and 2D methods appear to be relatively poor, consisting of Cs, Ds, and Fs. However, it is worthwhile to take time to understand these scores more thoroughly.

Consider the major axis bending stress scores first. While there is a D for 2D models, note that the D is a worst case score. The mode of scores is a B, which suggests for most cases that a 2D analysis will predict major axis bending stresses reasonably well. The worst case score may reflect a bridge with particularly extreme geometry. For 1D models, both the worst case score and the mode of scores is a C, indicating the method had a normalized mean error of 13 to 20 percent—not great, but not terrible. Keep in mind that when looking at stress response categories, the engineer can always choose to accept the inaccuracy and deal with it by being conservative, in this case perhaps by limiting the allowable performance ratio.

Next, consider the vertical displacement scores. Things look a little worse here. For the 2D methods, both the worst case score and the mode of scores are F, indicating the method had a normalized mean error greater than 30 percent, which is pretty poor. In fact, the scores for the 2D method are worse than those for the 1D method, which almost seems counterintuitive. However, remember that this bridge has a high connectivity index, meaning it has relatively few cross-frames and the girders are behaving more independently and less like a system. Note that one of the key findings of the NCHRP Report 725 research was that current 2D methods neglect warping stiffness. Individual I-girders in a curved girder bridge carry torsion by means of warping, and neglecting the warping stiffness can

make a significant difference in the results, especially when the girders are behaving relatively independently rather than as a system of girders. So the F grade makes sense.

Keep in mind that when looking at displacements there is no way to be conservative. Over-predicting deflections is just as bad as under-predicting deflections when trying to predict the constructed geometry to facilitate fit up.

However, these grades should not be construed as requiring 2D methods to be completely discounted for a bridge of this type. Looking at these results, and remembering that one can be conservative when evaluating stresses, there are some options. The designer may choose to use a 2D method to size the girders of this bridge, considering both composite and non-composite loads, and applying some conservatism when doing so. Then the designer may choose to build a 3D model of just the non-composite case, to obtain dead load deflections to evaluate fit up, develop girder camber diagrams, etc. Building a 3D model of both the non-composite and composite condition of the bridge, using it to evaluate moving live loads, and to pull stresses and forces for design can be a big and challenging effort, but building a 3D non-composite dead load model just to get deflections may be relatively easy.

### **B.3 Tub Girder Bridges**

Similar to the I-girder bridges, a quantitative assessment of the analysis accuracy of tub girder bridges was obtained by focusing first on the normalized mean errors in the approximate (1D and 2D method) solutions for the girder major-axis bending stresses, internal torques, and vertical displacements compared to benchmark 3D FEA results. Using the quantitative assessments, the various methods of analysis were assigned scores in the same manner as the scoring discussed in Section 3.1.1 for the I-girder bridge responses. Table B.3-1 summarizes the scores for the above responses in tub girder bridges.

It is interesting that the Table B.3-1 scores for the major-axis bending stresses and vertical displacements are relatively good. However, the worst-case scores for the internal torques are generally quite low. These low scores are largely due to the fact that the internal torques in tub girder bridges can be sensitive to various details of the framing, such as the use and location of external intermediate cross-frames or diaphragms, the relative flexibility of these diaphragms as well as the adjacent internal cross-frames within the tub girders, skewed interior piers without external cross-frames between the piers at the corresponding bearing line, incidental torques introduced into the girders due to the specific orientation of the top flange lateral bracing system members (particularly for Pratt-type TFLB systems), etc. White et al. (88) provides a detailed evaluation and assessment of the causes for the errors in the girder internal torques for the tub girder bridges considered in the NCHRP Report 725 research.

**Table B.3-1. Matrix for Recommended Level of Analysis—Tub Girder Bridges, Non-Composite Dead Load Analysis Models**

Response	Geometry	Worst-Case Scores		Mode of Scores	
		Traditional 2D-Grid	1D-Line Girder	Traditional 2D-Grid	1D-Line Girder
Major-Axis Bending Stresses	S	B	B	A	B
	C	B	C	A	B
	C&S	B	C	B	B
Girder Torques	S	F	F	D	F
	C	D	D	A	B
	C&S	F	F	A	B
Vertical Displacements	S	B	B	A	A
	C	A	B	A	A
	C&S	B	B	A	A
Girder Layover at Bearing Lines	S	B	B	A	A
	C	NA <sup>a</sup>	NA <sup>a</sup>	NA <sup>a</sup>	NA <sup>a</sup>
	C&S	B	B	A	A

<sup>a</sup> Magnitudes should be negligible where properly designed and detailed diaphragms or

Similar to the considerations for I-girder bridges, the external diaphragms and/or cross-frames typically respond rigidly in their own plane compared to the torsional stiffness of the girders. Therefore, the girder layovers at skewed bearing lines tend to be proportional to the major-axis bending rotation of the girders at these locations. As a result, the errors in the girder layover calculations obtained from the approximate methods tend to be similar to the errors in the major-axis bending displacements.

The connectivity index,  $I_C$ , does not apply to tub girder bridges, since this index is primarily a measure of the loss of accuracy in I-girder bridges due to the poor modeling of the girder torsion properties. For tub girder bridges, the conventional St. Venant torsion model general works well as a characterization of the torsional response of the pseudo-closed section tub girders. Hence,  $I_C$  is not used for characterization of tub girder bridges in the table. Furthermore, there is only a weak correlation between the accuracy of the simplified analysis calculations and the skew index  $I_S$  for tub girder bridges. Therefore, the skew index is not used to characterize tub girder bridges in Table B.3-1 either. Important differences in the simplified analysis predictions do exist, however, as a function of whether the bridge is curved, “C,” straight and skewed, “S,” or curved and skewed “C&S.” Therefore, these characterizations are shown in the table.

In addition, to the above quantitative assessments, the calculation of bracing component forces in tub girder bridges is assessed separately in Table B.3-2. It is useful to address the accuracy of these response calculations separately from those shown in Table B.3-2 since the simplified bracing component force calculations take the girder major-axis bending moments, torques, and applied transverse loads as inputs and then apply various useful mechanics of materials approximations to obtain the force estimates. That is, there are two distinct sources of error in the bracing component forces relative to the 3D FEA benchmark solutions:



- (1) The error in the calculation of the input quantities obtained from the 1D line-girder or the 2D-grid analysis, and
- (2) The error introduced by approximations in the component force equations.

White et al. (88) provides an overview of the most commonly employed bracing component force equations evaluated here. It should be noted that the calculation of the top flange lateral bending stresses in tub girders is included as one of the bracing component force calculations. This is because these stresses are influenced significantly by the interaction of the top flanges with the tub girder bracing systems.

White et al. (88) observed that in many situations, the bracing component force estimates are conservative relative to the 3D FEA benchmark solutions. Therefore, it is useful to consider a signed error measure for the bracing component force calculations. In addition, the bracing component dimensions and section sizes often are repeated to a substantial degree throughout a tub girder bridge for the different types of components. Hence, it is useful to quantify the analysis error as the difference between the maximum of the component forces determined by the approximate analysis minus the corresponding estimate from the 3D FEA benchmark, i.e.:

$$e_{max} = (R_{approx,max} - R_{FEA,max}) / R_{FEA,max} \quad (B.3-1)$$

for a given type of component. The grades for these responses were then assigned based on the same scoring system as that used for the assessments based on normalized mean error with one exception: Separate grades were assigned for the positive (conservative) errors and for the negative (unconservative) errors in Table B.2-3. In situations where no negative (unconservative) errors were observed in all of the bridges considered in a given category, the symbol “--” is shown in the cells of the matrix and the cells are unshaded.

The mode of the grades is shown only for the top flange diagonal bracing forces in Table B.2-3. The mode of the grades for the other component force types are not shown because of substantial positive and negative errors in the calculations that were encountered in general for the tub girder bridges, and because, in cases where a clear mode for the grades existed, the mode of the grades was the same as the worst-case grade.

In addition to the above considerations, it should be noted that current simplified estimates of the tub girder bridge bracing component forces are generally less accurate for bridges with Pratt-type top flange lateral bracing (TFLB) systems compared to Warren and X-type systems. A small number of tub girder bridges with Pratt-type TFLB systems are considered by White et al. (88). Therefore, the composite scores for these bridges are reported separately in Table B.2-3.

**Table B.3-2. Matrix for Recommended Level of Analysis—Tub Girder Bridges, Non-Composite Dead Load Analysis Models**

Response	Sign of Error	Geometry	Worst-Case Scores		Mode of Scores	
			Traditional 2D-Grid	1D-Line Girder	Traditional 2D-Grid	1D-Line Girder
TFLB Diagonal Force	Positive (Conservative)	S	D	D	D	C
		C	D	F	B	F
		C&S	D <sup>a</sup>	F	B	F
		Pratt TFLB System	C	F	A	F
	Negative (Unconservative)	S	F <sup>b</sup>	C		
		C	-- <sup>c</sup>	--		
		C&S	--	--		
		Pratt TFLB System	--	--		
TFLB & Top Internal CF Strut Force	Positive (Conservative)	S	C	C		
		C	F	F		
		C&S	F	F <sup>d</sup>		
		Pratt TFLB System	F	F		
	Negative (Unconservative)	S	C	C		
		C	--	A		
		C&S	--	C		
		Pratt TFLB System	D	D		
Internal CF Diagonal Force	Positive (Conservative)	S	NA <sup>e</sup>	NA <sup>e</sup>		
		C	F	F		
		C&S	F	F		
		Pratt TFLB System	--	F <sup>f</sup>		
	Negative (Unconservative)	S	NA <sup>e</sup>	NA <sup>e</sup>		
		C	--	--		
		C&S	--	D		
		Pratt TFLB System	B	--		
Top Flange Lateral Bending Stress (Warren TFLB Systems)	Positive (Conservative)	S	C	C		
		C	F	F		
		C&S	F	F <sup>d</sup>		
	Negative (Unconservative)	S	C	C		
		C	--	A		
		C&S	--	C		

<sup>a</sup> Modified from a C to a D considering the grade for the C and the S bridges.

<sup>b</sup> Large unconservative error obtained for bridge ETSSS2 due to complex framing. If this bridge is considered as an exceptional case, the next worst case unconservative error is -15 % for NTSSS2 (grade = C).

<sup>c</sup> The symbol "--" indicates that no cases were encountered with this score.

<sup>d</sup> Modified from a B to an F considering the grade for the C bridges.

<sup>e</sup> For straight-skewed bridges, the internal intermediate cross-frame diagonal forces tend to be negligible.

<sup>f</sup> Modified from an A to an F considering the grade for the C and C&S bridges.

### B.3.1 Tub Girder Bridge Level of Analysis Example

Consider a horizontally curved steel tub girder bridge with a Warren top flange lateral bracing system and skewed supports for which the engineer wants to perform a traditional 2D-grid analysis to determine the forces and displacements during critical stages of the erection sequence. The bridge has “regular” geometry (constant girder spacing, constant deck width, a relatively uniform top flange lateral bracing (TFLB) system and internal cross frame spacing, solid plate end diaphragms, single bearings for each girder, etc.).

A properly prepared 2D-grid analysis would be expected to produce major axis bending stresses and vertical deflections with mean errors less than 12 percent relative to a rigorous 3D FEA solution, since the worst-case score assigned for both of these quantities is a B for the subject “C&S” category. Furthermore, it can be observed that the mode of the scores for the vertical displacements is an A, and hence, given the “regular” geometry of the above bridge, it is expected that the vertical displacements most likely would be accurate to within 6 percent.

Unfortunately, the worst-case score is an F for the 2D-grid estimates of the internal torques in the “C&S” bridges. As noted previously, this low score is due to the fact that the internal torques in tub girder bridges can be very sensitive to various details of the framing, such as the use and location of external intermediate cross-frames or diaphragms, the relative flexibility of these diaphragms as well as the adjacent internal cross-frames within the tub girders, skewed interior piers without external cross-frames between the piers at the corresponding bearing line, incidental torques induced in the girders due to the specific orientation of the top flange lateral bracing system members (particularly for Pratt-type TFLB systems), etc. Fortunately though, the web and bottom flange shear forces due to the internal torques are often relatively small compared to the normal stresses due to the major-axis bending response of the girders. Furthermore, the mode of the scores for the internal torques is an A from Table B.3-1. Therefore, the engineer must exercise substantial judgment in estimating what the expected error may be for the internal torque from a 2D-grid analysis, and in assessing the impact of this error on the bridge design. As indicated by White et al. (88) for I-girder bridges, one can compensate for any anticipated potential unconservative error in the internal force or stress response quantities by scaling up the corresponding responses by the anticipated error, or by adjusting the target values of the performance ratios.

Based on Table B.3-2, the worst-case score for the positive (conservative) error in the calculation of the TFLB diagonal forces in the above example bridge is a D whereas the mode of the scores is a B. The table shows that no unconservative errors were encountered in this calculation for the tub girder bridges studied by White et al. (88). Since the example bridge is “very regular,” the engineer may assume that the TFLB diagonal force calculations are conservative, but reasonably accurate, relative to the refined 3D FEA benchmark values.

For both the TFLB and top internal cross frame strut forces and the internal cross frame diagonal forces in “C&S” bridges, Table B.3-2 shows a grade of F for the conservative error. Also, the table shows that no unconservative errors were encountered by White et al. (88) for these responses. Therefore, the engineer can assume that the forces for these components, as determined from a 2D-grid analysis plus the bracing component force equations, are highly conservative. It should be noted that the forces in the top struts of the internal cross-frames at exterior diaphragm or exterior cross frame locations can be very sensitive to the interaction of the external diaphragm or cross-frame with the girders. These forces should be determined based on consideration of statics at these locations given the forces transmitted to the girders from the external diaphragm or cross frame components. White et al. (88) did not consider these component forces in their error assessments.

Lastly, Table B.3-2 shows that the tub girder top flange lateral bending stresses tend to be estimated with a high degree of conservatism by 2D-grid methods combined with the bracing component force equations. In addition, no unconservative errors were encountered in the tub girder bridges studied by White et al. (88) for the top flange lateral bending stresses. Therefore, the engineer can also assume that these stress estimates are highly conservative.