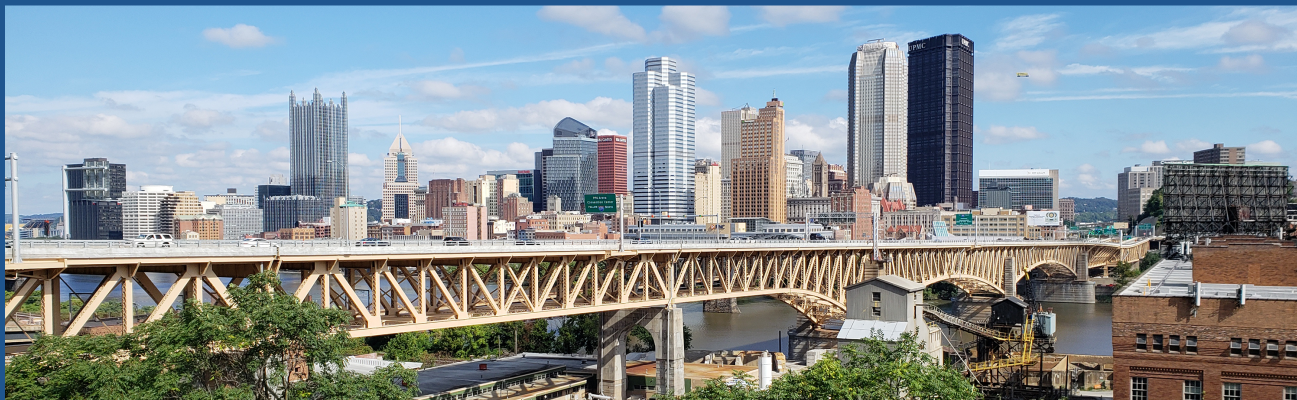

Guidelines for Steel Truss Bridge Analysis

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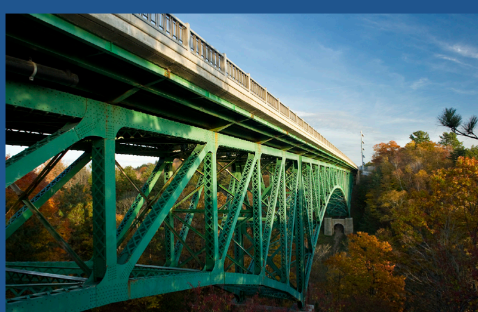

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AASHTO/NSBA STEEL BRIDGE COLLABORATION

American Association of State Highway and Transportation Officials

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Preface

This document is a guideline 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, construction, inspection, and long-term maintenance. Each standard represents the consensus of a diverse group of professionals.

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Foreword

This document was prepared by the AASHTO/NSBA Steel Bridge Collaboration, Task Group 13, Analysis of Steel Bridges, and provides Engineers with guidance on methods of analysis for steel trusses and can be used for design, erection, rehabilitation, load rating, or demolition analysis of trusses. Trussed arches are not covered in this document. This document is a guideline document, and offers suggestions, insights, and recommendations, but no rules. Content is presented regarding:

- Definition and description of truss types and members.
- Different analysis types for steel truss bridges.
- Typical analysis steps for steel truss bridges.
- Analysis assumptions for support conditions for steel truss bridges.

The authors of this document are all members of Task Group 13 and represent a broad cross-section of the bridge industry, including bridge Owners, bridge construction professionals, bridge Engineers, and academic researchers. The authors approached the writing of this document as a consensus effort; as such, many different opinions are represented, and explanation and commentary are provided when more than one acceptable solution is presented. The primary intended audience for this document is any Engineer who is tasked with analyzing a steel truss bridge.

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SECTION 1—TRUSS COMPONENTS, CONFIGURATIONS, TYPES, AND BEHAVIOR

Descriptions of truss configurations, components, and types are provided in this Section along with a discussion on truss behavior. Refer to Section 2 for a detailed discussion of various truss analysis methods and Section 3 for step-by-step procedures.

1.1—TRUSS CONFIGURATIONS

There are a wide variety of truss bridge configurations. Configurations are based upon where the deck is located and on the span type (refer to Article 1.2 for descriptions of the truss components). The three most basic configuration definitions are based on the location of the deck and include:

Through Truss: The deck is located within the truss structure. The truss structure itself has four sides: two (or more) vertical truss panels, an overhead lateral bracing panel, and a system of transverse floorbeams (and possibly longitudinal stringers) below. Generally, the floor system provides the bottom plane of lateral stiffness via a concrete deck made composite with the floorbeams and/or stringers, but occasionally a lateral bracing panel or steel decking provides the bottom plane of lateral stiffness. See Figure 1.1-1 and Figure 1.1-5 for illustrations of through-truss configurations.

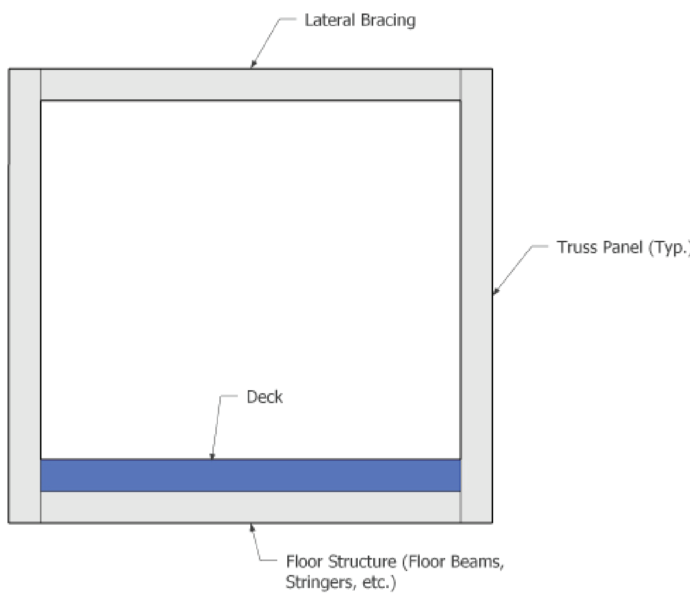


Figure 1.1-1—Through-truss section view.

Deck Truss: The deck is located above the truss structure. The truss structure typically consists of two (or more) vertical truss panels, bottom lateral bracing, and a system of transverse floorbeams (and possibly longitudinal stringers) above, often with a concrete deck made composite with the floorbeams and/or stringers. Occasionally steel decking or a top lateral bracing system are used. See Figure 1.1-2 for an illustration of a deck truss configuration.

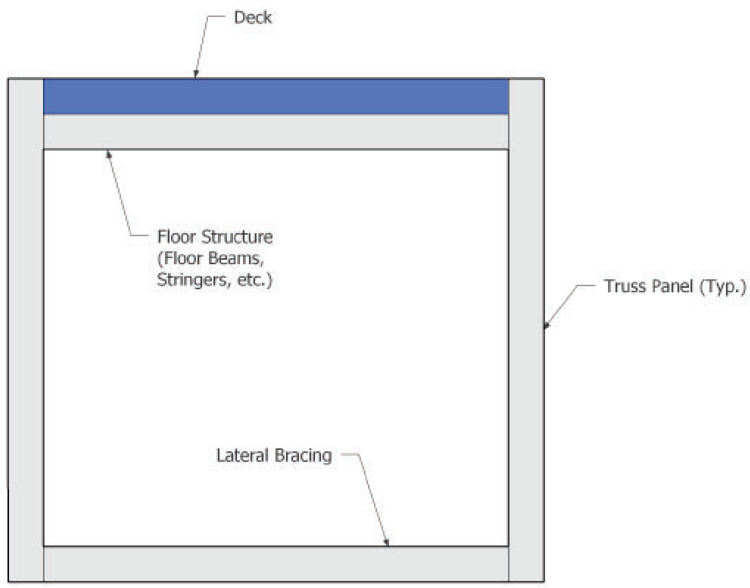


Figure 1.1-2—Deck truss section view.

Pony Truss: Similar to a through-truss bridge in that the roadway is located within the truss structure, but a pony truss bridge has no top lateral bracing system. See Figure 1.1-3 and Figure 1.1-4 for illustrations of pony truss configurations.

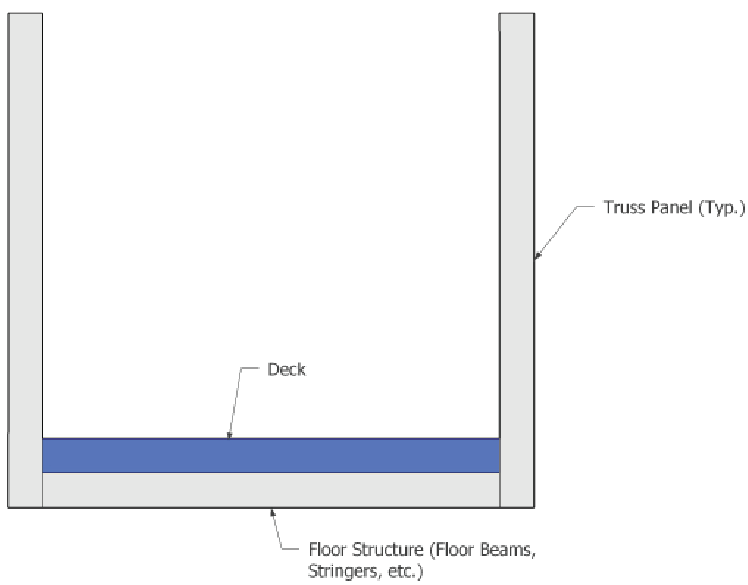


Figure 1.1-3—Pony truss section view.

There are many variants of these three basic riding surface location truss configurations, such as half-through trusses, where the vertical truss panels are partly above and partly below the roadway.

The configuration of a truss bridge can also be defined by span configuration. For example:

Simple-Span Truss Bridges: These consist of one or more simply supported truss spans. See Figure 1.1-4.

Figure 1.1-4—Simple-span pony truss.

Continuous Truss Bridges: These consist of multiple spans in which the truss structure is continuous over the internal supports. See Figure 1.1-5.

Figure 1.1-5—Continuous through-truss bridge.

Cantilever Truss Bridges: The designation "cantilever truss" can refer both to the construction method and to the final configuration of a multi-span truss bridge. In terms of construction methods, a cantilever truss is built by progressively cantilevering the structure from the supports. Anchor spans provide support for the cantilevers. In the main span, two cantilevers will extend from the anchor spans and the gap between the two cantilevers receives a "suspended span."

In terms of final configuration, a cantilever truss bridge is typically a multiple-span bridge with anchor spans, cantilevers, and one or more suspended spans (see Figure 1.1-6). In this type of structure, the suspended span is often simply supported at the cantilever ends, rendering the overall bridge structurally determinate from an analysis standpoint. If the suspended span is made continuous with the cantilevers, the final configuration of the bridge is more typically referred to as a "continuous truss," even though it was constructed using cantilever truss construction methods.

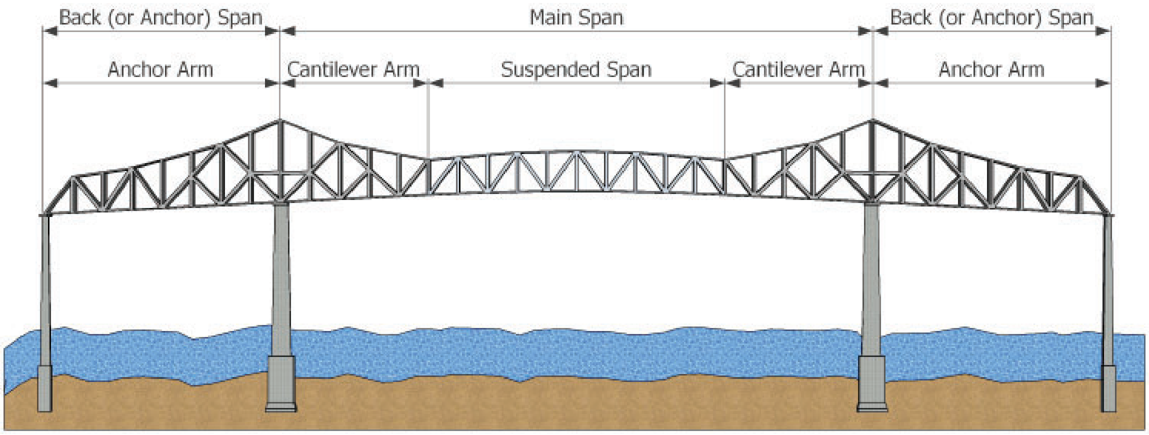


Figure 1.1-6—Cantilever truss bridge.

1.2—DESCRIPTIONS OF TRUSS COMPONENTS

A truss bridge typically consists of two vertically oriented truss planes that are connected with lateral members to create a three-dimensional structure. A floor system is used to support the bridge deck. Figure 1.2-1 depicts a typical through truss with a floorbeam–stringer floor system.

Truss planes are made up of straight members typically arranged in a triangular pattern and intersecting at joints known as panel points. The primary load-carrying members of a truss plane are the bottom (lower) chords, top (upper) chords, verticals, and diagonals. Lateral member types include lateral bracing, portal frames, sway frames, and struts. Floor systems consist of floorbeams, stringers, and decks.

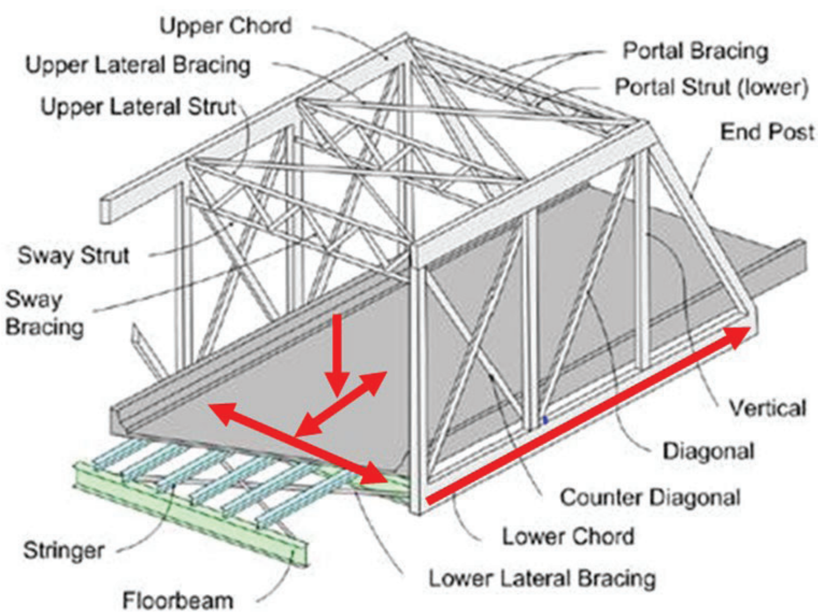


Figure 1.2-1—Truss members, floor systems, and bracing and load path (red arrows).

Truss planes can also be arranged in rectangular pattern, known as Vierendeel trusses; however, their behavior is different than triangular trusses. Article 1.3.2 describes Vierendeel trusses.

1.2.1—Upper and Lower Chords

The top and bottom members, known as chords, are oriented to act like the flanges of a beam. The chords carry axial loads. For simple-span trusses, the lower chord carries the tension forces, and the upper chord carries the compression forces. The diagonally sloped end post is a chord member which can carry compression or tension forces depending on whether the truss is supported respectively at the bottom or at the top chords.

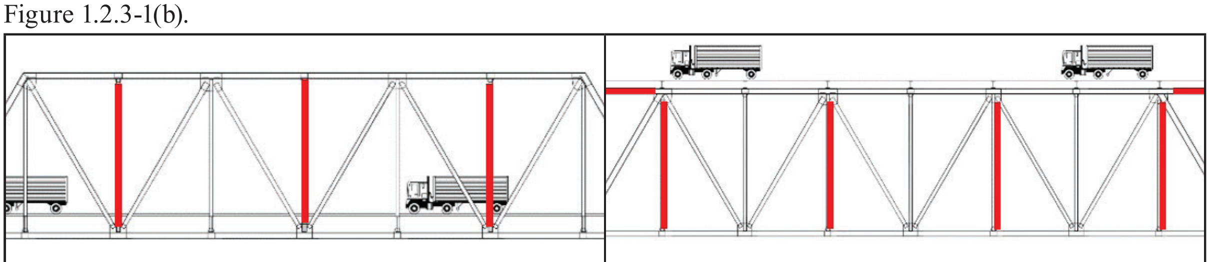
1.2.2—Diagonals

The upper and lower chords are connected by diagonals and/or vertical members (see Article 1.2.3). Diagonals function in a manner similar to the web of a beam. Diagonals generally provide the necessary shear capacity for the truss plane, though the primary loading is still axial.

1.2.3—Verticals

Vertical members function in a manner similar to the web of a beam. They also serve as a support to limit the dead load bending stresses in the chord members by reducing the unsupported member length. Verticals can be in either tension or compression. The tension verticals are commonly called hangers, and compression verticals are often called posts. Verticals provide additional panel points through which deck and vehicle loads can be applied to the truss.

A "zero-force member" in a truss structure is a member that, under classical truss theory, does not have a tension or compression force from any traffic loading configuration. While a zero-force member may carry minor axial loads from its own self-weight or the weight of connecting elements, such as chords or secondary members, it does not have any axial forces from the dead load of the floor system or from live loads. A common zero-force member is a truss vertical that forms a right angle with a truss chord, as long as a truss diagonal or a floorbeam for the deck does not also frame into that node (see Figure 1.2.3-1(a)). Zero-force vertical members do not carry any live loads and they have a very small dead load due to supporting the chords. Zero-force verticals are often important members for increasing stability by bracing the compression chord and for improving the global stiffness in the transverse direction when they are part of the sway bracing system. A chord may also be a zero-force member, as shown in Figure 1.2.3-1(b).



(a) Through Truss | (b) Deck Truss

Figure 1.2.3-1—Zero force member examples (highlighted red).

1.2.4—Lateral Bracing

Lateral bracing provides stability between the two truss planes by connecting the upper and/or lower chords to resist wind load and sidesway caused by moving vehicular traffic. These can consist of X-braces, which run from panel point to panel point; K-braces, which run from a panel point to a midpoint of a cross-member, such as a floorbeam or upper chord cross-member; or Warren truss lateral bracing framing patterns.

1.2.5—Portal Frames and Sway Frames

The portal frame is a rigid frame or short truss member that runs transversely between the upper chords of the two truss planes. Two portal frames are typically used in a two-truss system at the first and last panel points of the truss. The portal frame carries the transverse load from the top lateral bracing system down to the bearings.

A sway frame provides transverse stiffness and bracing for the top compression chord and sometimes, at the vertical members along the length of the bridge. Figure 1.2.5-1 identifies portal and sway bracing in a truss.

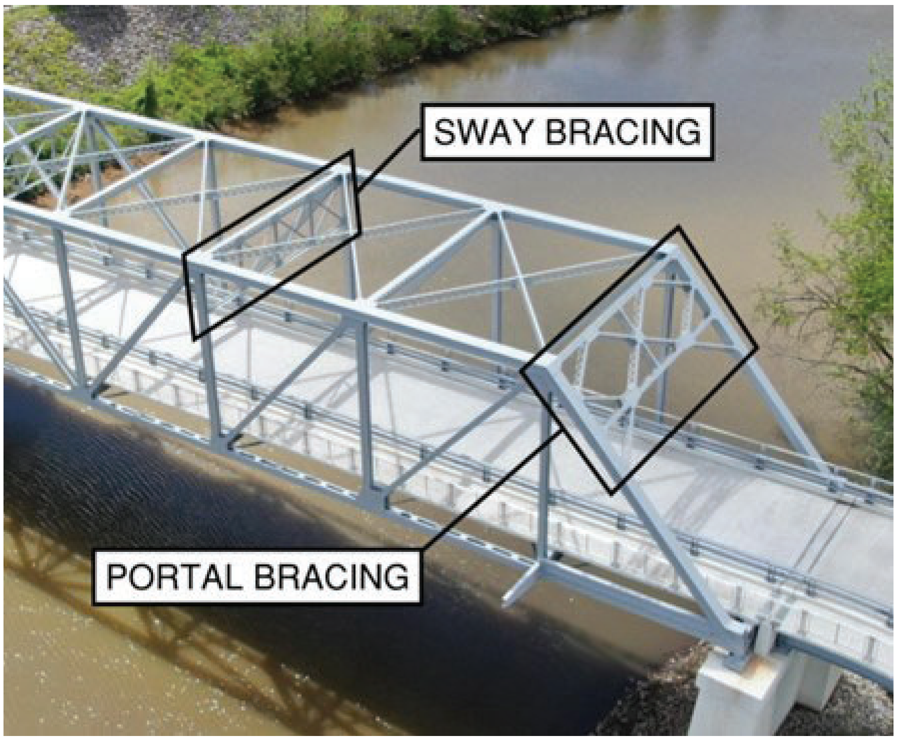


Figure 1.2.5-1—Sway bracing and portal bracing.

1.2.6—Floor Systems

Floor systems, consisting of floorbeams, stringers, and decks, are designed to provide a riding surface for the live load and to transfer the load to truss panels. The floorbeam runs transversely between the two planar trusses and the stringers run longitudinally along the length of the bridge.

Figure 1.2.6-1 depicts a truss floor system where the stringers bear on the top of the floorbeam (also known as a stacked system). Typically, this system has stringers running continuously over a floorbeam to create a two-span continuous stringer, but simple-span stringers and stringers running continuously over two floorbeams creating a three-span continuous stringer are also found. This floor system type is often referred to as a "floating floor system" because the stringers sit on top of the floorbeams. Figure 1.2.6-2 shows a floor system where the stringers frame into the floorbeams directly. This floor system type is often referred to as an "integral floor system" because the stringer and floorbeams work integrally to support the deck. Figure 1.2.6-3 depicts a floor system with only a deck and floorbeams. This is only common for smaller trusses that have short panel spacings. This floor system type is often referred to as a "ladder floor system" because the framing plan of floorbeams looks like a ladder.

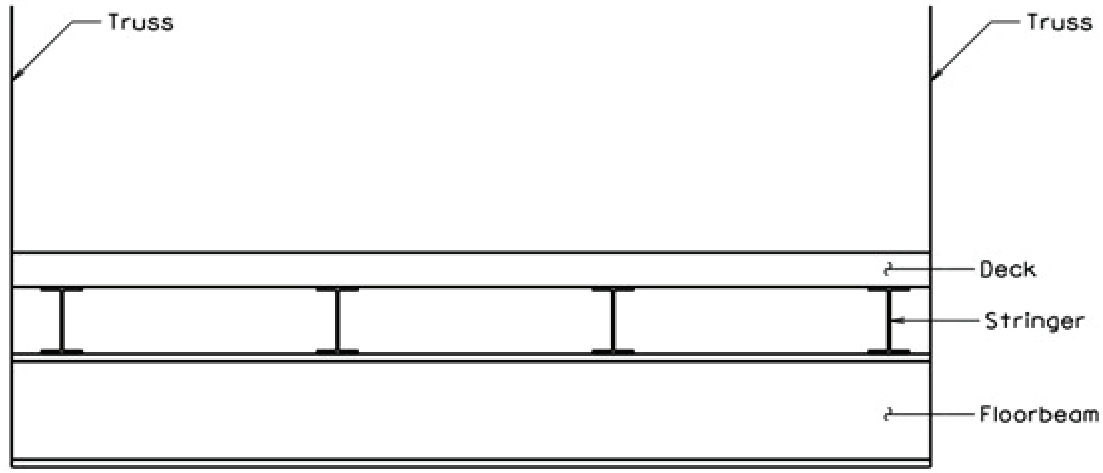


Figure 1.2.6-1—Truss floor system with stringers bearing on the floorbeams.

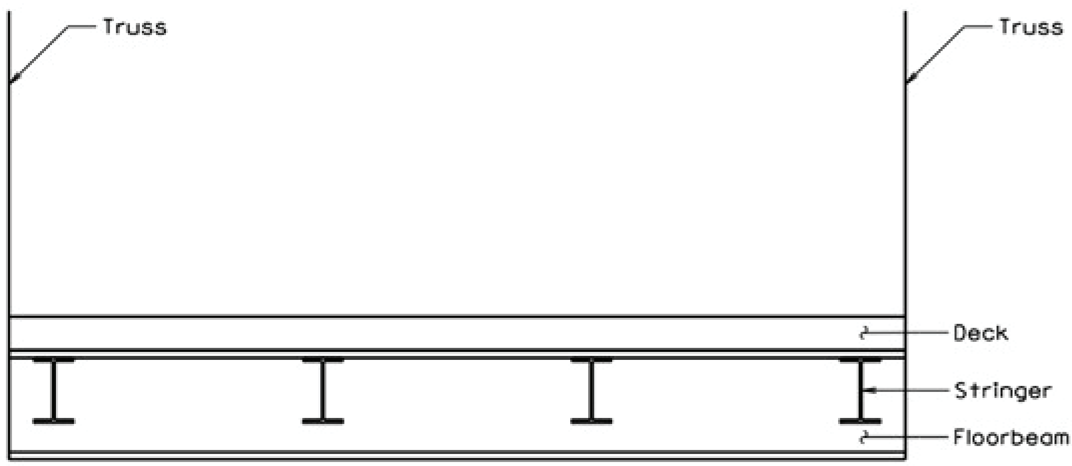


Figure 1.2.6-2—Truss floor system with stringers framing into the floorbeams.

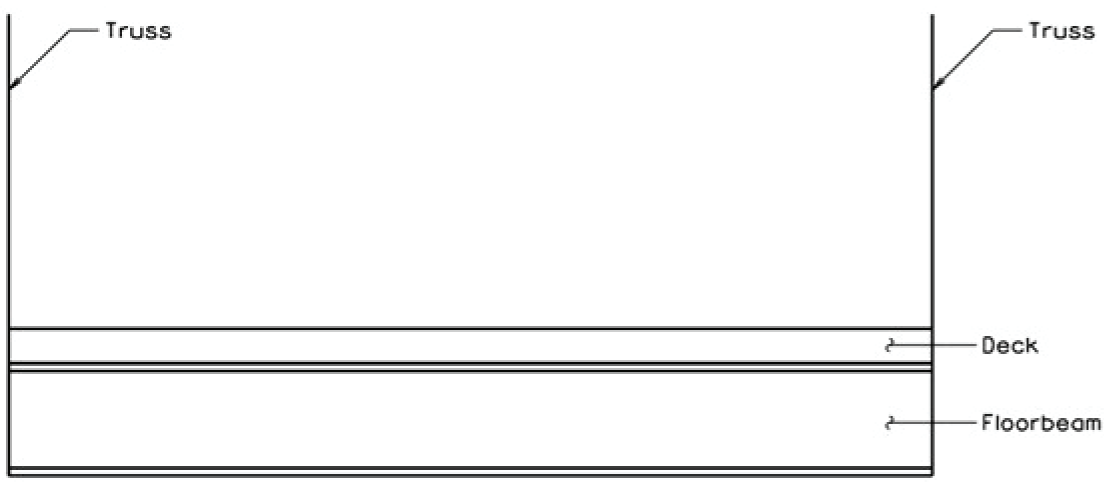


Figure 1.2.6-3—Truss floor system with only floorbeams.

1.2.7—Panel Points

Panel points are joints where truss members intersect. These panel point connections typically consist of gusset plates with members bolted, riveted, or welded together, or with members connected with a single large-diameter pin. Prior to approximately the middle of the 20th century, truss bridges were traditionally constructed with riveted connections; however, that practice is generally obsolete as of the 2020s, as are pin-connected trusses.

1.2.8—Connections

1.2.8.1—Primary Member Panel Point Connections

Connections between truss chords, verticals, and diagonals can be detailed as pinned/hinged, fixed, or partially restrained connections. Pinned/hinged connections typically consist of a single large-diameter (three inches or more) steel pin that passes through multiple intersecting members. While not commonly used today, pinned/hinged connections were commonly used in the past because they minimize secondary stresses compared to fixed joints. Corrosion of the pin may lead to partial fixity of the pin.

Fixed connections consist of a direct moment connection between connected members. Fixed connections are commonly used in some truss applications, such as prefabricated pedestrian trusses, but are uncommon in modern highway structures. Prefabricated pedestrian bridges generally use rectangular hollow structural sections (HSS) with fully welded connections. Fatigue design for live load is not required for pedestrian bridges, which makes fully welded connections a viable option.

Partially restrained connections consist of indirect moment connections between members, using connecting elements such as gusset plates. Most modern highway structures use bolted gusset plate connections between primary members. Rivets were often used in the past. The partially restrained connections may include multiple layers of gusset plates and secondary splice plates. Highway bridges with partially restrained connections are commonly analyzed assuming pinned connections, especially for older existing truss bridges. Refer to Articles 3.5.1, 3.5.8, 3.5.10, and 3.5.11 for detailed information on how to model the different connection types. Figure 1.2.8.1-1 depicts a primary member gusset plate connection.

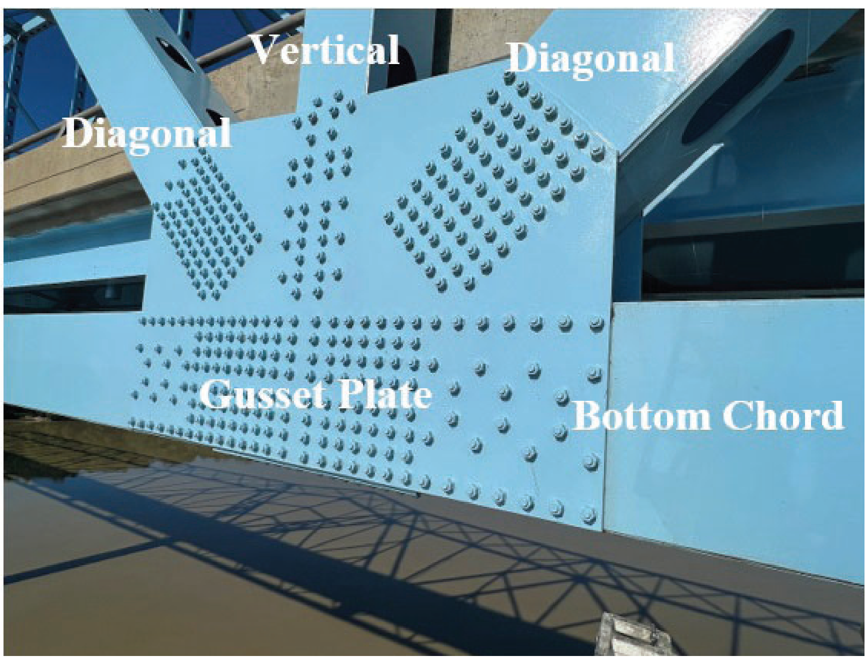


Figure 1.2.8.1-1—Panel point gusset plate connection.

1.2.8.2—Floorbeam Connections

Truss floorbeam ends are generally connected to truss verticals or truss chords using either a fixed or partially restrained connection. Fixed connections (refer to Figure 1.2.8.2-1) engage both the flanges and the web of the floorbeam to provide full moment continuity. Partially restrained connections (refer to Figure 1.2.8.2-2) engage the floorbeam web only, providing full shear transfer and partial bending restraint. Moment continuity is required only if the trusses are dependent upon frame action with the floorbeams to provide stability and resistance to lateral loads.

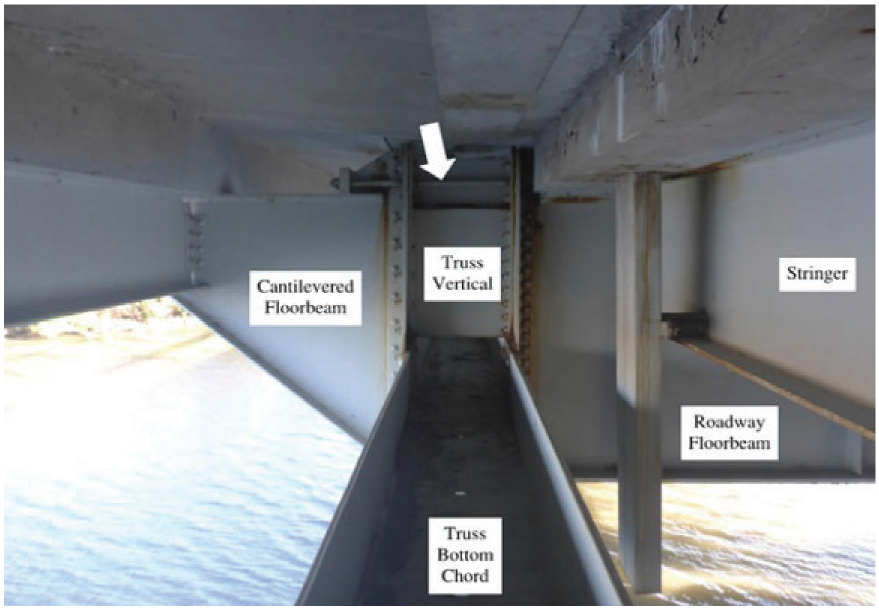


Figure 1.2.8.2-1—Fixed floorbeam connection to support a cantilevered sidewalk, which uses top flange tie rods (arrow) and bottom flange continuity plates (not shown). See also Figure 3.5.2.8-2.

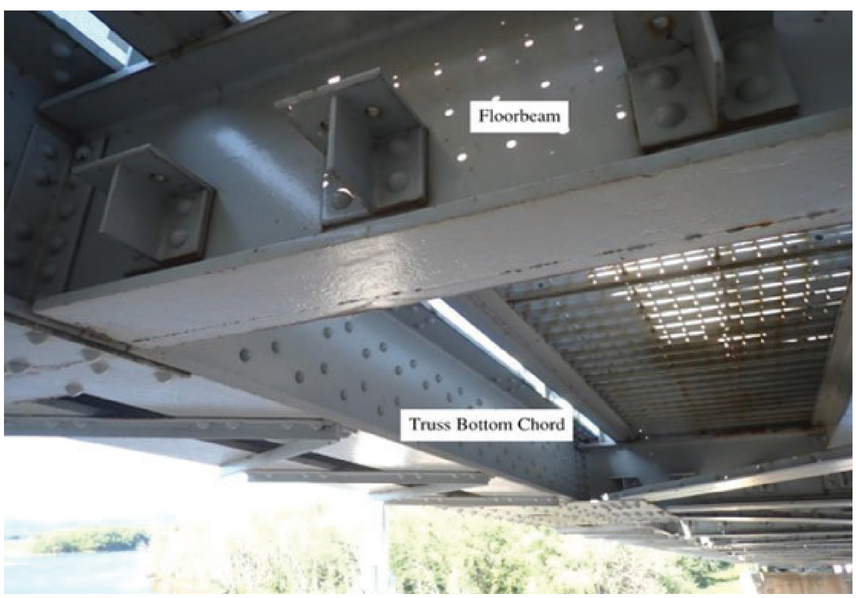


Figure 1.2.8.2-2—Partially restrained floorbeam connection to the truss bottom chord.

1.2.8.3—Lateral Bracing Connections

Lateral bracing members are generally connected to the truss chords using pinned, bolted, riveted, or welded connections. These connections often incorporate gusset plates, as this allows more geometric flexibility than a direct member-to-member connection. Moment continuity is generally not required for lateral bracing connections, as bracing members are normally designed to carry axial load only. Figure 1.2.8.3-1 depicts a lateral bracing connection.

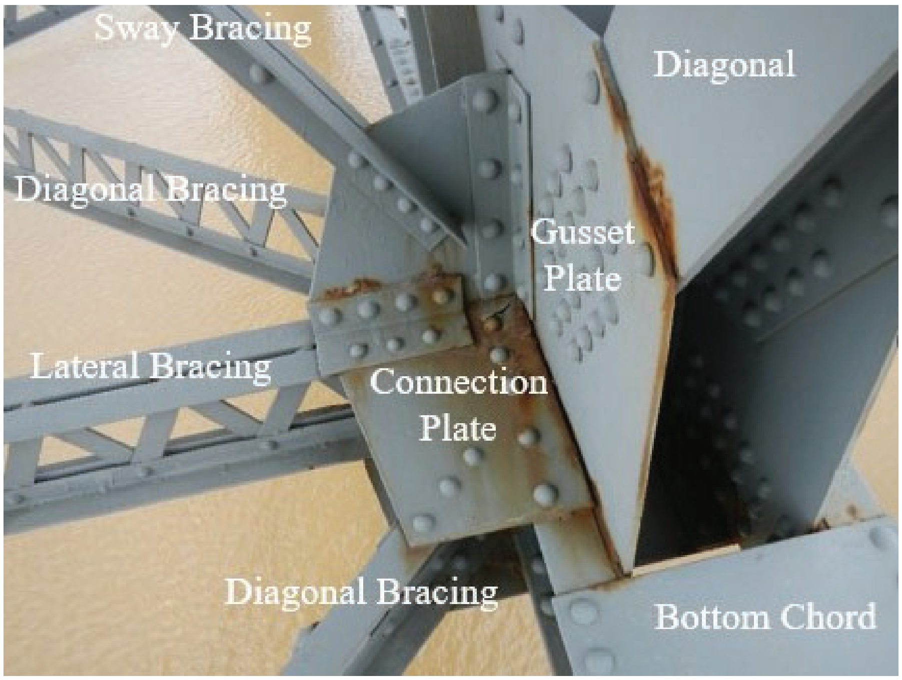


Figure 1.2.8.3-1—Lateral bracing connection.

1.2.9—Supports

1.2.9.1—Typical Truss Supports

Proper definition of boundary conditions is essential to correct modeling of structure behavior. Consider the behavior of a simple-span truss (see Figure 1.2.9.1-1). 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. For the truss to be considered statically determinate, the right support must be a roller so that the lower chord is free to extend when subject to tension. If both supports were pin supports, the lower chord would be unable to extend and develop tension, the distribution of load in the truss would be altered, and the truss would become statically indeterminate.

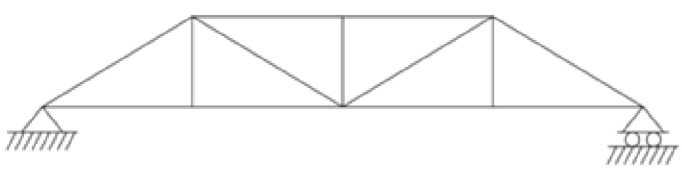


Figure 1.2.9.1-1—Simple-span truss with statically determinate boundary conditions.

These concepts are not limited to single-span trusses. Multiple-span continuous trusses are subject to the same provisions (see Figure 1.2.9.1-2).

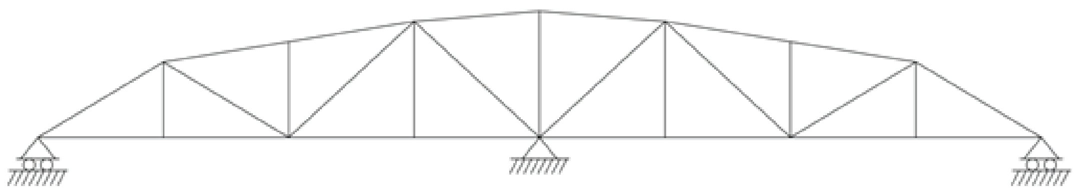


Figure 1.2.9.1-2—Two-span continuous truss.

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 to accurately calculate the response of the structure to various loading conditions.

Most bridge bearings (steel-laminated elastomeric bearings, pot bearings, disc bearings, etc.) are designed to accommodate rotations. In the context of this discussion, the terms “fixed,” “free,” etc. are referring to restraint of horizontal translation. Be sure to correctly represent the constraints provided by the bearings in all degrees of freedom in the analysis model.

Common bearing types for steel truss bridges include pin, rocker, roller, steel-laminated elastomeric, pot, and disc bearings. When examined in detail, the behavior of these bearings is often complex, due to factors such as rotational friction, partial fixity, and nonlinear material behavior. These factors may be captured by highly detailed models, but this is generally not practical or necessary in a global bridge model. Bearing behavior needs to be modeled only to the extent needed to represent its effect on the rest of the bridge. In most truss models, conventional bearing types can be represented using simple boundary conditions, without a need to consider the more complex aspects of bearing behavior. Refer to Article 3.5 for detailed information regarding modeling members and connections. Refer to AASHTO/NSBA G9.1, *Steel Bridge Bearing Guidelines* (2022), for additional information.

1.3—TRUSS TYPES

1.3.1—Triangulated Truss Types

A large variety of truss types have been used in truss bridges; there is probably no single, completely comprehensive catalog of all truss types. However, the Historic American Engineering Record (HAER) has produced a one-page catalog illustrating 30 different types of trusses which have been used in bridges in the United States. This collection, while not exhaustive, features the most cited and used truss types (see Figure 1.3.1-1).

The various truss types offer different advantages and disadvantages in terms of structural function, structural efficiency, constructability, and ease of fabrication. In very simplistic terms, older bridges often use a given type of truss in order to achieve some measure of material economy (least weight), often by sacrificing simplicity, ease of fabrication, and ease of construction. This represents an older design philosophy where least-weight designs were considered more desirable, due to higher material costs versus lower fabrication and erection costs. This often results in bridges that were complex to fabricate and construct. In more modern designs, simpler truss types are often used, reflecting trends in the industry that have led to lower material costs versus higher fabrication and erection costs. Trusses offer advantages in terms of greater strength-to-weight ratios than typical girder bridge types such as steel plate girder bridges. For this reason, truss bridges are often considered for long-span bridges or for bridges carrying particularly heavy live loads (such as railroad bridges). The disadvantages of trusses include greater fabrication costs and the potential for higher life-cycle costs due to more challenging long-term maintenance and more complex future widening considerations compared to typical girder bridge types. When used in the right applications for the right reasons, trusses represent a valuable and effective tool in the bridge Engineer's toolbox.

Different truss types may also offer advantages in terms of structural function. For example, consider the simple case of choosing between a Warren truss with verticals versus a Warren truss without verticals. If there is reason to limit the spacing of floorbeams, it may be more advantageous to use a Warren truss with verticals, which would accommodate more floorbeams at tighter spacing and supported at points where their reactions can be carried by axial forces in truss members. However, if a wider floorbeam spacing is feasible, it may be more economical to use a Warren truss without verticals, eliminating a large number of truss members and providing a cleaner, less cluttered, more aesthetically appealing appearance.

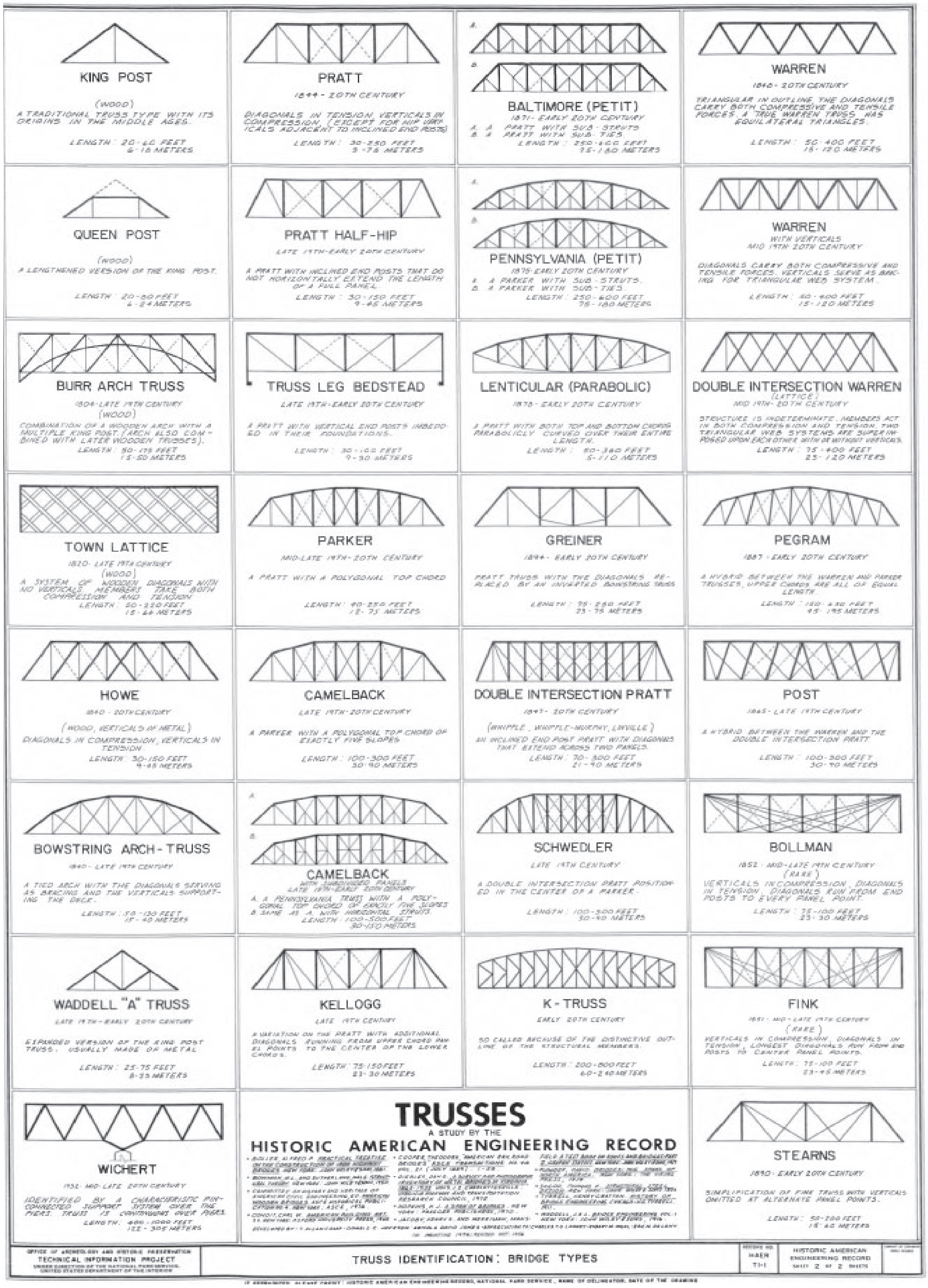


Figure 1.3.1-1—Catalog of truss types compiled by HAER.

1.3.2—Vierendeel Trusses

Vierendeel trusses (or Vierendeel frames) differ from typical trusses in that they are designed to carry global shear forces through flexure of the chords and verticals. As such, they require no diagonal members (see Figure 1.3.2-1). They consist of upper and lower chords with a series of verticals rigidly connected at the joints. Due to the moment continuity at the joints, the structure behaves like a rigid frame. All members are subjected to bending and shear forces in addition to the axial forces common in all trusses. Vierendeel trusses were developed after secondary stresses (flexure and shear) were observed in riveted connections at truss joints, which are normally idealized as pin connections in analysis. In Vierendeel trusses, the secondary bending and shear forces are not neglected. The rigidity of the connections is recognized, and the truss members are designed for axial load, shear, and moment. Analysis of Vierendeel trusses can be completed using hand calculation methods for analyzing indeterminate structures (such as the moment distribution method), but such analyses are extremely laborious. Simplifying assumptions such as assuming the presence of hinges at inflection points may make hand calculations more viable. However, given the high degree of indeterminacy of Vierendeel trusses, using computer analysis software is recommended.

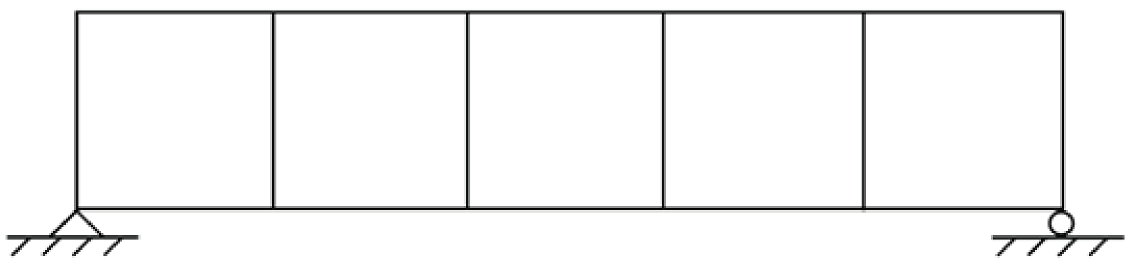


Figure 1.3.2-1—Vierendeel truss diagram.

Vierendeel trusses were popular in Europe in the early 1900s. The first types of these structures had riveted joints, but switched to fully welded joints as welding became a more established practice. Unfortunately, since there were many unknowns surrounding the long-term effects of welding and inadequate quality control procedures, Vierendeel trusses had numerous issues, including a few catastrophic collapses. In the United States, the use of Vierendeel trusses has not been very prevalent as a primary means of support in bridge spans, but they have been used for tower structures of long-span bridges and bracing systems for arches. The stability of the truss without diagonal members results in large openings between verticals, providing opportunities for incorporating architectural details.

1.3.3—Type Selection

There are primarily three types of trusses: deck trusses, through trusses, and pony trusses (see Article 1.1).

Deck trusses (see Figure 1.1-2) are typically used when there is sufficient clearance under the bridge to accommodate the depth of the truss structure. Deck trusses offer the advantage of unlimited overhead clearance for vehicles on the bridge.
Through trusses (see Figure 1.1-1) are often used where vertical clearance below the bridge is limited since the depth of structure below the deck (typically just floorbeams and stringers) is very shallow compared to the overall depth of the truss structure.
Pony trusses (see Figure 1.1-3) are a variant of the through-truss type, but without lateral bracing for the truss upper chord. Pony trusses are typically used for shorter span lengths (less than 200 feet), particularly in prefabricated bridge applications as discussed below.

Prefabricated steel truss bridges can represent a cost-effective solution for highway bridges for span lengths up to 200 feet using prefabricated steel pony trusses, and up to 300 feet using prefabricated steel through trusses, especially in applications where:

- the vertical clearance under the bridge is limited,
- minimizing field work and the need for temporary shoring is desirable, or
- minimizing field construction time is desirable (i.e., accelerated bridge construction).

Stick-built steel truss bridges are generally more cost-effective for highway bridges with span lengths between 450 feet and 900 feet. The cost of fabrication and erection of a stick-built truss bridge is typically more than that of a steel plate girder bridge at comparable span lengths, but material costs may be less due to the inherent structural efficiency (strength-to-weight ratio) of trusses compared to steel girder bridges. Generally, the net cost of a truss will be less than that of a plate girder bridge at span lengths above 450 feet. The practical upper limit for simple-span trusses is approximately 750 feet. Continuous or cantilever trusses can span considerably longer distances than simple-span trusses; continuous trusses begin to be cost-effective when the span length exceeds 550 feet. The cost of truss spans increases rapidly as the span length exceeds 900 feet. In recent years, as cable-stayed bridges have become more common and their technology has been refined, very few trusses with spans longer than 900 feet have been constructed in the United States.

1.4—TRUSS BEHAVIOR

Trusses behave essentially as large beams and can be either simple-span or continuous. Bearings located at the support points must be capable of allowing the appropriate rotations and thermal movements. Trusses are generally assumed to be pinned at their connections and are idealized to behave as axial compression or tension members. For a simple span, the upper chord members are designed as compression members, while the lower chord members are designed as tension members. However, in continuous spans, some members will experience a stress reversal and be subject to both tension and compression. Where truss members frame into other truss members and not at pinned connections, the connections can induce bending forces into those members and must be designed as combination axial and flexure members. Connections of the vertical and diagonal members to the chord members are typically achieved with bolted connections using gusset plates. In addition to the upper chord compression bracing, adequate bracing of the truss planes to each other is necessary for global stability. (The floorbeams provide significant bracing.)

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SECTION 2—APPLICATION AND ALTERNATIVES

This Section discusses the basic concepts of analyzing trusses as well as hand calculation and finite element modeling methods for analyzing trusses.

2.1—BASIC CONCEPTS

2.1.1—Statically Determinate vs. Statically Indeterminate

A truss is statically determinate if all support reactions and member forces can be calculated using the equations of equilibrium ($\sum F_x = 0$ and $\sum F_y = 0$ for a two-dimensional planar truss). The total number of equilibrium equations available is two times the number of joints, $2j$, for a two-dimensional planar truss. The total number of unknowns in the structure is member forces, m, plus reactions, r. The equation for determining if a two-dimensional planar truss structure is determinate is shown below.

Statically determinate if $m + r = 2j$, where:

m = number of members
r = number of support reactions
j = number of joints
(Note: The truss is unstable if $m + r = 2j$)

Example:

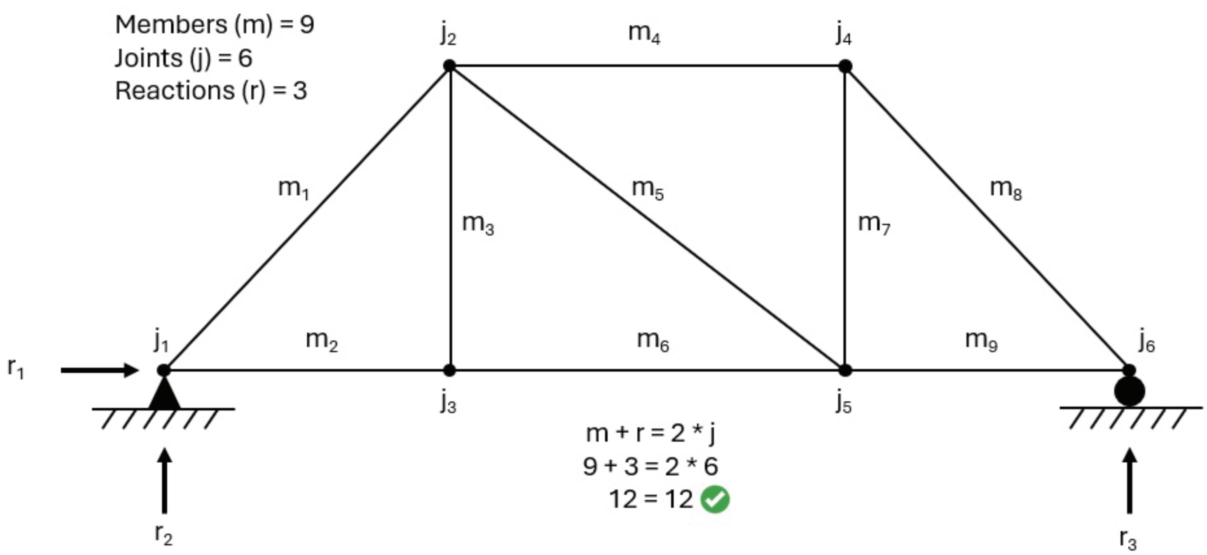

Figure 2.1.1-1—Statically determinate truss.

Since $m + r = 2j$, the truss above is statically determinate.

If the structure is statically indeterminate, ($m + r > 2j$), the degree of indeterminacy, i, may be calculated as $i = (m + r) - 2j$.

Example:

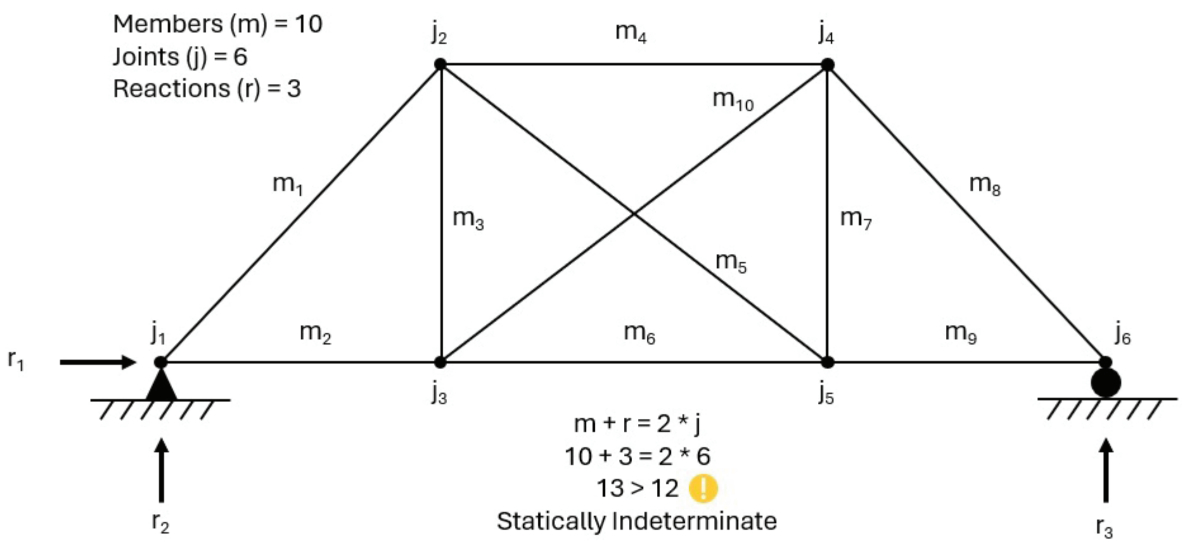

Figure 2.1.1-2—Statically indeterminate truss.

Since $m + r > 2j$, the truss above is statically indeterminate.
$(m + r) - 2j = 1$. Therefore, the degree of indeterminacy is 1.
The main advantage of indeterminate trusses is additional system redundancy provided by either additional members or supports. Disadvantages include additional restraint in the truss which could induce stresses due to support settlements, temperature changes, or fabrication errors, as well as the added complexity of analyzing an indeterminate truss.

2.1.2—Redundancy

Historically, steel trusses have been proven to be highly redundant structures. They may in fact possess multiple modes of redundancy, which are defined in Title 23 Code of Federal Regulations (CFR) Part 650.305 Subpart C*, National Bridge Inspection Standards* (NBIS) as:

1. Load Path Redundancy
2. Internal Redundancy
3. System Redundancy

Load path redundancy requires no analysis beyond engineering judgment. It is simply the presence of three or more primary load-carrying members. The majority of steel truss bridges will not possess load path redundancy due to having only two truss lines.

Internal redundancy can be evaluated for members in tension that are fabricated of built-up steel members where the member components are mechanically fastened using rivets or bolts. Many older steel trusses were constructed using built-up members with mechanically fastened components. Guidance for the evaluation of internal member redundancy is provided in the AASHTO *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members*. These *Guide Specifications* provide the Engineer with the load and performance criteria for the member assuming a tensile component of the member is failed. The National Steel Bridge Alliance (NSBA) has developed spreadsheets to perform this analysis. The Internally Redundant Member (IRM) analysis tools can be downloaded for free from the NSBA Design Resources website at https://www.aisc.org/nsba/design-resources/.

System redundancy requires refined, three-dimensional finite element analysis capable of modeling nonlinear geometric and nonlinear material properties and behavior. Guidance for the analysis of system-level redundancy is provided in the AASHTO *Guide Specifications for Analysis and Identification of Fracture Critical Members and System Redundant Members*. These Guide Specifications provide the Engineer with the load and performance criteria for the structure assuming an entire primary tensile load-carrying member is failed.

When evaluating an existing steel truss bridge for redundancy, the complexity, level of effort, and therefore engineering cost, increases going from the first mode listed above, load path redundancy, to the last mode, system redundancy. Thus, it is recommended that when analyzing an existing steel truss for redundancy, the Engineer should first consider load path redundancy, then internal member redundancy. If the truss tension members do not meet the requisite criteria for either of these, then system redundancy may be analyzed. For new structures, the Owner and Engineer should evaluate the following and select the appropriate design approach:

- The high cost of frequent in-depth inspections for a truss with nonredundant steel tension members (NSTM) versus the lower initial fabrication cost.
- The increased initial cost of a truss using mechanically fastened internally redundant tension members versus the lower cost of less frequent, and less rigorous, inspections.
- The significant increase in design complexity and potentially increased initial cost of a truss using system redundant members versus the lower cost of less frequent, and less rigorous, inspections.

2.2—HAND CALCULATIONS AND BASIC ANALYSIS METHODS

There are two classic hand analysis methods for solving for the forces in truss components.

- Method of joints, which considers free body diagrams of individual joints,
- Method of sections, which considers free body diagrams of portions of a truss.

The method of joints is useful in learning to visualize the action of members in tension or compression while the method of sections is useful for investigating the forces in a critical member of a truss.

Both methods are based on the following assumptions which make it easy to analyze the truss by hand by reducing the number of unknown forces:

- Members are pin-connected.
- The assembly is loaded at joints.
- Each member is an axial two-force member with the equal end forces, opposite and colinear.
- The members act either in pure tension or pure compression, with no internal shears or moments.

2.2.1—Method of Joints

The method of joints uses the assertion that the sum of forces in two orthogonal directions at any pinned joint must be equal to zero. In other words, the sum of component forces of any member axial forces, reactions, and externally applied loads at a joint must be in equilibrium. For simple (determinate) trusses, a set of simultaneous equations may be created for all joints and solved for all unknown forces. A simplified, joint-by-joint approach may be used by selecting a logical order of joints and the most convenient coordinate system at each joint. By selecting a joint with at least one known and only two unknowns to begin with (e.g., a vertical bearing reaction with only two members attached to the joint), member forces can be determined successively from joint to joint. Additionally, by recognizing joints with only two unknowns, tension/compression-only members, and zero-force members, it may be possible to determine a certain number of member forces in indeterminate truss structures.

The method of joints is straightforward to perform with hand calculations but is usually not the most efficient way to perform the analysis, except in the cases of the smallest and most simplistic trusses. Modern software packages can conveniently and quickly determine truss member forces, especially for two-dimensional analysis. However, the method of joints is still useful to determine the general behavior of a truss and check the results of software packages. A detailed discussion of the method of joints is available in many entry-level university statics textbooks, including *Vector Mechanics for Engineers: Statics* by Beer et al. (2019). Additionally, there are open-source textbooks and courses available on the internet such as *Engineering Statics: Open and Interactive* by Baker and Haynes (2020), as well as free instructional videos.

2.2.2—Method of Sections

Similar to the method of joints, the method of sections is a hand calculation method to determine member forces in trusses, typically a simple determinate truss. The method is accomplished by "slicing" through the truss to create two separate sections of the truss. The slice must pass through only three members. The unknown member forces can then be determined by solving for the three equations of equilibrium (two orthogonal directions and moment) about a convenient point, accounting for reaction forces and externally applied loads.

The method of sections is useful for determining forces quickly and verifying analysis results but is generally not a practical method for the complete design or load rating of a truss. See Article 2.2.1 for suggested references which also discuss the method of sections in more detail.

2.2.3—Influence Lines and Surfaces

An influence line is a graphical representation of a response in a truss member as a unit load moves along the two-dimensional structure, typically at the bridge deck level. The influence line captures the response from live load moving longitudinally along the bridge and does not capture the effects of the live load's transverse placement across the width of the bridge. An influence surface is a graphical representation of a response in a truss member as a unit load moves on a defined surface (bridge deck) on a three-dimensional structure. The influence surface captures the response from live load moving both transversely and longitudinally on the bridge deck.

An influence line for a statically determinate truss is an assemblage of straight lines along the roadway structure with data points occurring at panel points (usually chord joint locations). A chord member influence line is triangular in shape while diagonal and vertical member influence lines are usually three lines—jumping from positive to negative at the panel point coinciding with their location. An individual influence line is only applicable to a specific member. For axial load, the total potential number of influence diagrams is equal to the number of members in the truss. If the truss is statically indeterminate, the influence lines will have a more complex shape, such as a series of chord lines approximating a curve for chord members of continuous span trusses. Refer to structural analysis text books for additional information on influence lines.

Influence lines can be obtained by static or kinematic methods. In the static method, the ordinate values of the influence line for a member are obtained by loading each joint along the deck with a unit load and finding the member force due to this unit load. In the kinematic method, the influence lines are obtained by the application of the Muller–Breslau Principle, which involves removing the constraint to the specific force and determining the deflected shape of the resulting structure. Regardless of the method used, the influence results are then multiplied by wheel or axle loads of a specific vehicle placed at the optimal location to obtain the maximum or minimum total live-load force in the member due to the vehicle as it traverses the structure. The member force can potentially be found using the method of joints or method of sections, which are discussed in Articles 2.2.1 and 2.2.2. The "influence" data may be prepared in tabular form and plotted for a specific truss member response, where columns represent the truss panel points and rows correspond to individual members.

In addition to the hand methods, there are multiple computational ways to handle live-load modeling for truss structures. One method (less common due to the advancements of computer software) is 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 can produce 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 on the structure. For each live load case generated, the analysis model is fully executed and force effects for all truss members are calculated. The brute force method generates a large number of responses used to develop the force envelopes for various members in the structure.

An alternative to the brute force method is the influence line or influence surface methods (static or kinematic) described previously accomplished with a computer. In these approaches to live-load modeling, the response at a given point in the model (e.g., a point on a truss chord, hanger, floorbeam, stringer, etc.) is calculated for a defined number of 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. The influence surface approach to modeling live load effects allows the Engineer to quickly determine the controlling responses of the structure at given locations. This approach minimizes the amount of output and allows the Engineer to focus on the critical loading effects rather than spending substantial time collating a large set of data to determine envelope results.

2.3—COMPUTER ANALYSIS USING THE FINITE ELEMENT METHOD

2.3.1—Element Types

The following provides information on element types commonly used when performing finite element analyses of steel trusses. Elements are typically classified by geometric complexity as follows. Line elements are represented by a line in the analysis model. The stiffness of a line element is determined by assigning material and cross-sectional properties to the element, such as cross-sectional area, moment of inertia, or shear area. Surface elements are represented by a surface in the analysis model, having length and width. Stiffness of a surface element is determined by assigning material properties and a thickness to the element. Volume elements are represented by a volume in the analysis model, having a length, width, and height. Only material properties are needed to determine the stiffness of the volume element since its geometry is fully defined by its dimensions. Commonly used line, surface, and volume elements are described further below:

- Line Elements
  - **Truss element**—An element in which the responses are solely axial tension/compression along the length of the component. Truss elements have one local translational degree of freedom (DOF) at each node along the local axis of the member (corresponding to axial deformation of the member). These elements are typically used to represent truss and bracing members in two-dimensional or three-dimensional truss analyses. These elements should not be used to represent members which are required to carry flexure or shear (like Vierendeel truss members or floor system members) as they are not formulated to carry these forces.
  - **Beam or frame element**—A two-dimensional beam element has three DOFs at each node, two translational DOFs and one rotational DOF, corresponding to the axial force, shear, and bending moment. These elements are typically used to represent truss members in two-dimensional truss analyses, or the lateral bracing members when analyzing the two-dimensional truss behavior in the horizontal planes.

    A three-dimensional beam element has six DOFs at each node: three translational DOFs, and three rotational DOFs (corresponding to axial deformation, shear deformations in the orthogonal transverse axes, moments about the orthogonal transverse axes, and torsion). These elements are typically used for representing truss members, bracing members, and floor system members within three-dimensional truss analysis models. Depending on the element formulation, elements may or may not capture torsion of asymmetric sections correctly.
- Surface Elements
  - **Plate element**—An element that typically consists of three to nine nodes in a plane. The internal element responses generally consist of moments and shears and do not include axial forces in the plane of the element. The result values are usually per unit length of the plate. Plate elements can have various combinations of nodal degrees of freedom; refer to the documentation of the analysis package for available formulations. These elements are not commonly used in truss analysis but may be used to represent noncomposite decks within three-dimensional truss analyses.
  - **Shell element**—An element that combines the effects of plate bending, shear, and axial force within the plane of the element. The result values are usually per unit length of the plate. Shell elements can be either flat or curved out-of-plane. Small flat shell elements can be used to form curved surfaces. Shell elements can have various combinations of nodal degrees of freedom; refer to the documentation of the analysis

package for available formulations. These elements are typically used to represent composite decks within three-dimensional truss analyses and may be used to represent member elements or gusset plates in more advanced local analyses of truss connections.

- Volume Elements
  - **Brick or solid element**—An element supporting three translational degrees of freedom per node, where nodes are usually provided in tetrahedral or hexahedral arrangement. The number of nodes typically can range from a minimum of four, forming a tetrahedral, to eight, forming a hexahedral. Many more nodes can be added, depending on the complexity of the element.These elements are used to represent various components of trusses in more advanced local stress analyses and are not commonly used in general truss analysis.

2.3.2—Two-Dimensional Analysis Description

As the primary behavior of conventional truss bridges in response to gravity loads (dead load and live load) is essentially planar, trusses are often modeled as two-dimensional plane structures. In most truss analysis models, members are assumed to be connected by pin joints, and loads are applied only at joint locations, so that members experience axial loads only, without consideration of shears or bending moments. Member self-weight is generally divided equally between the member end nodes. Live loads are distributed to the trusses using the lever rule and applied at nodes that coincide with transverse floorbeam locations. This general analysis approach is described in the AASHTO *LRFD Bridge Design Specifications* (*AASHTO LRFD BDS*) (2024), Article 4.6.2.4.

The assumption of pin-joint end conditions is appropriate for most truss structures, where the bending stiffness of the truss members is relatively small relative to the overall stiffness of the truss. However, member bending can become a significant secondary effect and may need to be included in some cases. This is discussed in detail in Article 3.5.10.

When a two-dimensional model is used to evaluate vertical loads, lateral loads must be evaluated using a separate analysis. For trusses with upper and/or lower lateral bracing systems, independent two-dimensional models may be needed to calculate design forces for wind and other lateral loads. Note that some members (such as upper and lower truss chord members) may be included in both the vertical and lateral load analysis, and that the design of these members should consider force results from both models.

2.3.3—Three-Dimensional Analysis Description

Three-dimensional models (Figure 2.3.3-1 and Figure 2.3.3-2) generally contain all structural components of the truss bridge, including chords, diagonals, verticals, sway bracing, lateral bracing, floor system, and deck. Loads may be applied in any direction, so three-dimensional analyses are often performed for wind or seismic analyses where both vertical and transverse loads are significant. Three-dimensional analyses are also performed when the distribution of load from one truss line to another may be significant, such as in system redundancy analyses.

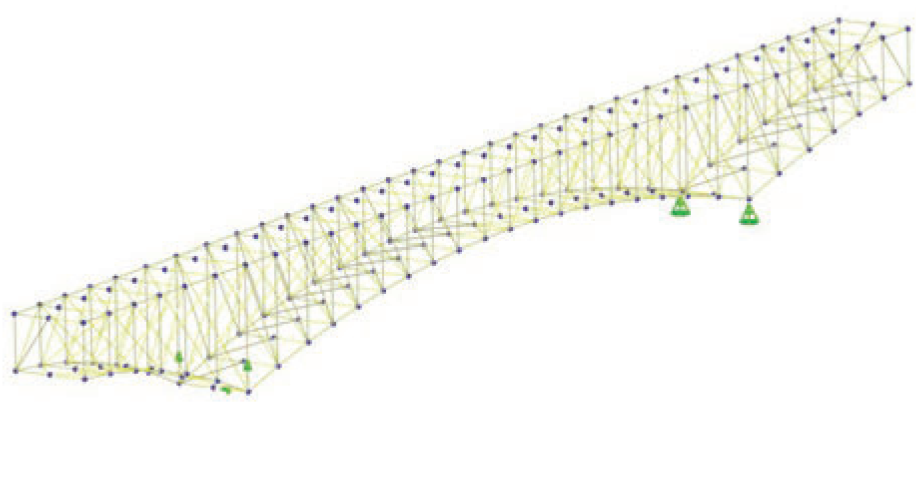

Figure 2.3.3-1—Vehicular deck truss with the deck not included in the model.

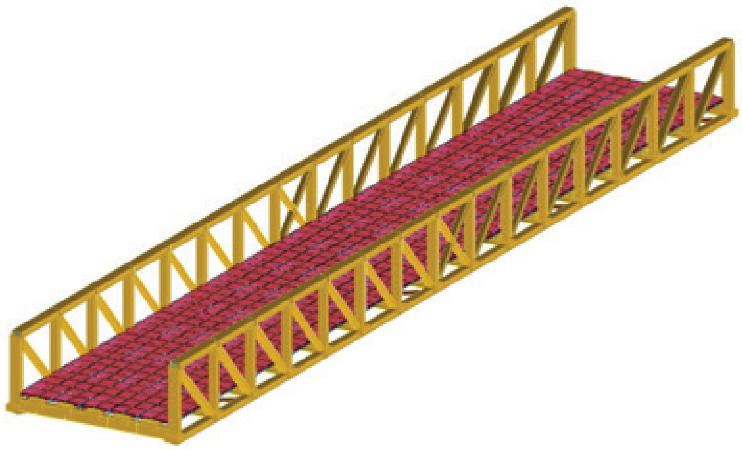

Figure 2.3.3-2—Pedestrian pony truss with deck included in the model.

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SECTION 3—ANALYSIS PROCEDURES

This Section discusses how to select an analysis method for trusses. A detailed description of the steps to create an analysis model of a truss is also provided in the order an Engineer would typically follow to analyze a truss.

3.1—SELECT ANALYSIS METHOD

On the most basic level, two types of analyses of trusses may be performed: two-dimensional/planar analysis or three-dimensional analysis (refer to Articles 2.3.2 and 2.3.3). A two-dimensional analysis of a truss contains all members of a single truss line in a single plane. Two-dimensional analyses are typically reserved for cases when only in-plane loadings are considered and in analyses where the three-dimensional behavior of the truss is not important or can be neglected. For example, a truss analysis to complete a load rating might use a two-dimensional analysis, with distribution of loading determined by the lever rule. Refer to *AASHTO LRFD BDS* (2024), Article 3.2, for the definition of the lever rule. In such a case, the exact distribution of loads through three-dimensional behavior of the truss including the lateral and sway systems does not need to be considered. It should also be noted the sway or lateral bracing systems may also be analyzed using a separate two-dimensional analysis for the in-plane demands they accommodate.

Three-dimensional analyses generally contain all structural components of the truss bridge, including chords, diagonals, verticals, sway bracing, lateral bracing, and the floor system. Three-dimensional analyses are often performed for wind or seismic analyses where both vertical and transverse loads are significant since loads can be applied in any direction. Three-dimensional analyses are also performed when the distribution of load from one truss line to another may be significant, such as in system redundancy analyses.

Ultimately, the Engineer must decide which analysis type applies better to their situation. This includes considering the goal of the analysis, the time allotted to perform and check the analysis, and the level of complexity desired by the client or required by the situation being analyzed. For example, the Warren through truss shown in Figure 3.1-1 was analyzed for wind and construction loads using a three-dimensional model, and hand-checked for connections via a planar two-dimensional analysis. Two-dimensional analyses are often easier to complete and faster to analyze and check compared with three-dimensional analyses. If loads in multiple directions need to be considered, multiple two-dimensional models need to be created for each portion of the truss that consider any interaction between the models. While a three-dimensional analysis may be used anywhere a two-dimensional analysis can be used, three-dimensional analyses are often more complex and take longer to analyze and check than a two-dimensional analysis.

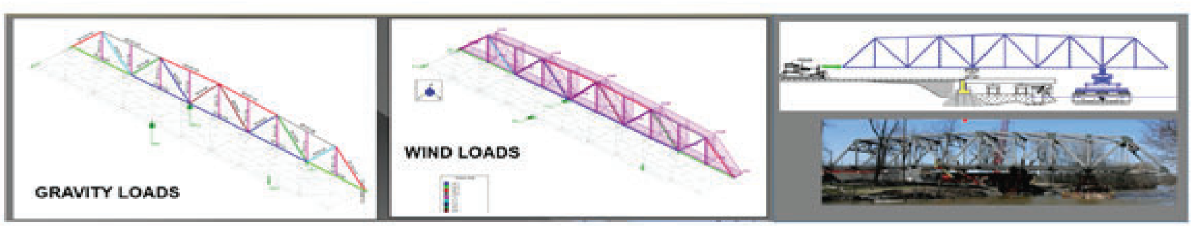


Figure 3.1-1—Three-dimensional truss modeling for construction and wind loads.

Table 3.1-1 lists the typical components that are included in two-dimensional and three-dimensional models of trusses. By definition, a two-dimensional analysis does not include members outside of the plane created by the truss. The floor system (stringers and floorbeams) loads are calculated by hand or using a separate analysis and then added to the two-dimensional truss model. Table 3.1-2 lists the attributes of two-dimensional and three-dimensional models.

Table 3.1-1—Typical Components Included in Analysis Models of Trusses.

Components	Two-Dimensional	Three-Dimensional
Upper and Lower Chord	Yes	Yes
Verticals	Yes	Yes
Diagonals	Yes	Yes
Stringers	No*	Yes
Floorbeams	No*	Yes
Upper and Lower Lateral Bracing	No**	Yes
Fixed Connections	No	Yes
Gusset Plates	No	No***

* Independent model of stringers and floorbeams can be created.

** Independent two-dimensional models can be utilized to obtain the lateral force effects. Refer to Article 3.5.3.

*** Refer to Article 3.5.11 on submodeling.

Table 3.1-2—Two-Dimensional vs. Three-Dimensional Attribute Comparison.

	Two-Dimensional	Three-Dimensional
AASHTO calculations for live load distribution factors (LLDFs)	Yes	No
Deck included in model to distribute live loads (transverse load sharing between trusses)	No	Yes*
Asymmetric damage	No	Yes
Force effects due to lateral wind load	No**	Yes
Force effects due to seismic load	No	Yes
Erection forces and sequence	Yes	Yes
Out-of-plane behavior	No	Yes
Pinned connections (only axial member forces)	Yes	No and Yes***
Obtain truss bending moments	No	No and Yes***
System redundancy analysis	No	Yes
Research	No	Yes

* Refer to Article 3.5.5 for a further description on when/how to model the deck in a three-dimensional analysis.

** Independent two-dimensional models can be utilized to obtain the lateral force effects. Refer to Article 3.5.3.

*** Refer to Article 3.5.10 for a detailed explanation.

3.2—TYPICAL ANALYSIS STEPS FOR TRUSSES

The analysis steps for trusses depend on the type of truss and the goals of the analysis. This Section provides generalized steps for analyzing trusses. Two example truss bridges defined below and shown in Figure 3.2-1 are used to describe the typical truss analysis steps:

Example Truss 1: Analysis of a bridge built circa 1930 consisting of nine simply supported Pratt through-truss spans needs to be performed to complete a load rating of the primary components of the bridge.

Example Truss 2: Analysis of a large three-span continuous through truss with five simply supported Warren through-truss approach spans on each side of the main three-span unit needs to be performed to aid in the design of seismic retrofits. The approach spans are similar, but not identical. The structure is supported on shared concrete bents at the joints between superstructure units. The concrete deck on the bridge is fully composite with the floor system.

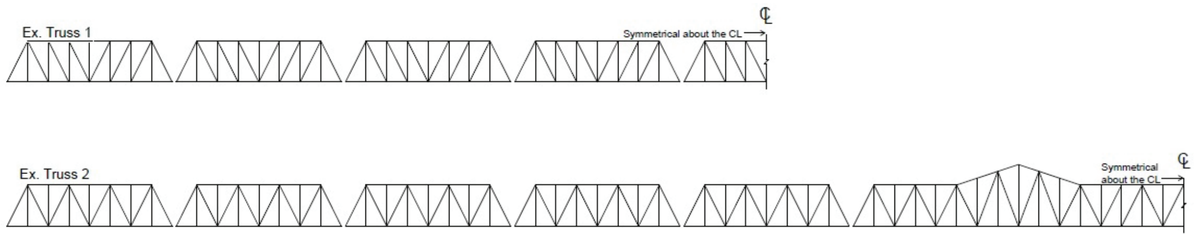


Figure 3.2-1—Example trusses.

The typical analysis steps are as follows:

1. **Examine and gather truss information.** The first step in the analysis is to examine existing contract and/or shop drawings and inspection reports to establish the truss type, configuration, and components (refer to Section 1).
   a. **Example 1:** The design drawings are examined; however, shop drawings and detailed information are not available. The nine spans are found to be identical.
   b. **Example 2:** Both the design and shop drawings are examined.
2. **Determine the goals of the analysis.** Depending on the work being performed and the features or arrangement of the truss being analyzed, different goals may be warranted:
   a. **Example 1 (Load Rating Project) Goals**: Determine the dead loads and live load for all load rating vehicles in the primary members.
   b. **Example 2 (Seismic Retrofit Project) Goals**: Determine how the bridge behaves under dead load and seismic loads, including deformations and member forces.
3. **Choose a truss analysis type.** A two-dimensional analysis of the members in a single truss line may be sufficient to achieve the goals of the analysis. In other cases, a more sophisticated three-dimensional analysis may be required. Refer to Section 2. Element types used in the analysis must be selected. For most analyses of trusses, beam or truss elements are sufficient. However, some analyses including member connections or gusset plates may require more detailed modeling utilizing shell or solid elements. Refer to Section 2 for element definitions and two-dimensional and three-dimensional analysis descriptions.
   a. **Example 1**: The primary members are subjected to in-plane gravity loading only, so a two-dimensional analysis is sufficient. No detailed connection analysis is required, so beam or bar elements can be used to represent the truss members.
   b. **Example 2**: The bridge is subjected to seismic demands, which will induce lateral and vertical loading on the truss members. Therefore, a three-dimensional analysis is required. Beam or bar elements can be used to model the truss members because the adequacy of the connections will be reviewed as part of the seismic retrofit design, but detailed analysis of the connections is not required.
4. **Choose analysis software.** The software needs to have the capabilities to perform the analysis type selected in Step 3 and achieve the analysis goals. For simple truss analyses, hand calculations using the method of joints or method of sections may be more efficient than using software programs. The choice of software package also depends on what is available within the Engineer's organization.
   a. **Example 1:** A software package with an automated live load placement capability is chosen because several different vehicles must be analyzed for the load rating. This analysis could also be done with influence lines and spreadsheet calculations. Refer to Article 2.2.3.

   b. **Example 2:** The seismic demands on the trusses will be determined using a response spectrum analysis. A software package which can perform modal analysis and combine modal responses to the design spectrum is chosen because the seismic demands on the truss must be determined.

5. **Create geometry and mesh.** Create the geometry and mesh of the model utilizing existing bridge information and incorporating the decisions made in previous steps. Some analysis packages use directly defined or drawn meshes, while others may require a defined geometry and a mesh later applied to the geometry. For a beam or bar element model, the geometry is typically straightforward and consists of placing points/nodes at the connection locations, and then connecting these points/nodes together with lines/elements. Meshing of axially loaded members is usually done with bar or beam elements. For some software packages, member end-releases may need to be considered and assigned during this step. See Article 3.3 for further information.

   a. **Example 1:** The centerline geometry of a single truss line is created within the software and meshed using beam elements. Since all spans are identical, only a single span model is created.

   b. **Example 2:** The centerline geometry of all truss members is created within the software. Since the spans are not necessarily identical and adjacent spans share foundations, the entire bridge is modeled in the software, including the concrete bents. The floor system is modeled using beam elements which support a shell-element concrete deck, thereby simulating the fully composite behavior between the two.

6. **Apply section and material properties.** Each software package typically has many ways to specify the geometric properties of a member. In some cases, section properties can be computed directly by the program, where in others they may need to be calculated outside the software. If bar elements are being used and bending moments are neglected, only the cross-sectional area of the members are required. Similarly, material properties may be selected from the software's library, or entered manually. See Article 3.4 for further information.

   a. **Example 1:** The members are built-up riveted steel sections. The software's built-in section property calculator is used to calculate the section properties of each member and apply them to the members within the model. The software's library of steel material properties match the steel properties used in the bridge, so it is applied to the members within the model.

   b. **Example 2:** The members are built-up riveted steel sections. The software's built-in section property calculator is used to calculate the section properties of each member and apply them to the members within the model. The software's library of steel material properties matches the steel properties used in the bridge, so it is applied to the members within the model. The concrete bents and concrete deck are also assigned section properties calculated by the software package and material properties stored in the software package's library.

7. **Apply boundary conditions and other constraints.** Examine the drawings and bridge information to determine the boundary conditions. Model boundary conditions should mimic the real structure boundary conditions. If modeling both superstructure and substructure within the same model, other constraints may be needed to tie the superstructure to the substructure with the proper behavior. Depending on the software package, member end-releases may need to be considered and assigned during this step if not done so previously.

   a. **Example 1:** At one end of the truss, a pinned boundary condition is provided, simulating the pinned bearing in the real structure. At the other end, a roller boundary condition (allowing translation along the truss length) is provided, simulating the rocker bearing in the real structure.

   b. **Example 2:** Boundary conditions are applied at the bases of the concrete bents, simulating the foundations. Between the truss units and the concrete bents, joint elements with appropriate bearing stiffness are used to provide the proper expansion behavior between the superstructure and substructure.

8. **Apply loading.** Some packages compute the self-weight of members internally and apply it automatically, while some packages require external calculation and entry as an applied load. Weights of connection components, such as gusset plates, are not typically included in the software's automatic loading and must

therefore be computed by hand. Any nonstructural components must also be accounted for using manually entered loadings. For seismic analyses, the distribution of mass throughout the structure must be properly accounted for. The computation of live load must also be considered. Analysis for live loading can range from applying a single point or distributed load to the use of complex autoloaders which determine the most critical loadings for the structure. Refer to the software package's documentation for more information on autoloaders. Other loads may be present, such as wind or thermal loadings. See Article 3.6 for further information.

a. **Example 1:** The software computes and applies the self-weight of the truss members automatically. Since a two-dimensional analysis is being performed, the self-weight of the deck and floor system members are manually calculated and applied. The weight of the gusset plates is applied as point loads at the nodes. Additionally, the application of live load to the truss must be considered, as the live load is not applied directly to the lower chord; it passes through the floor system and is applied only at the nodes of the truss. To incorporate this behavior, massless beam elements with negligible stiffness are provided coincident with the elements representing the lower chord of the truss members. These members properly distribute the live load to the truss nodes.

b. **Example 2:** Since conducting a seismic analysis, all self-weight must be entered as mass. The program automatically calculates the self-weight of the superstructure and substructure members, including the concrete deck. Self-weight of other permanent bridge components is calculated and applied manually to the model as point masses.

9. **Set analysis parameters and run analysis.** This step largely depends on the desired outcome of the analysis and the software package. For more specific information, refer to the documentation with the software package. See Article 3.7 for further information.

a. **Example 1:** The autoloader within the software is set up to move the required rating vehicles across the structure and provide the maximum axial force in each member due to each vehicle.

b. **Example 2:** The software is set up to conduct a modal analysis of the structure, and to compute enough modes to activate 90 percent of the structure's mass in each of the horizontal axes and the vertical axis. The response spectra are also entered, so that the program can calculate the seismic demands on each component.

10. **Review basic results.** Typically, the first thing to examine after running the analysis is the deflected shapes for the various load cases. The deflected shapes may reveal problems with the analysis that need to be corrected. Additionally, reactions are examined and compared to the total load applied to the structure. Comparison to "back-of-the-envelope" analyses can be extremely useful in verifying that the results are reasonable or solving issues that arise in the results. Often, this step leads to revisions to the model to correct mistakes or to adjust methodology. See Article 3.7 for further information.

a. **Example 1:** The deformed shape and reactions for applied self-weight was not as expected, and a mistake in the entry of the floor system loading was discovered. This was corrected, and the revised results seemed reasonable.

b. **Example 2:** The results of the modal analysis showed reasonable deformed shapes and reactions for the structure. The demands on the structural members seemed extremely large, so the input was checked and an error in the response spectra input was found. The error was corrected, and the revised results seemed more reasonable.

11. **Extract results.** In a truss analysis, the axial force within the truss members is often the desired result. If bending moments within the truss members are being considered, these may be extracted as well. Typically, the bending moments acting concurrently with the maximum axial forces are considered, but the maximum bending moment with concurrent axial force may rarely control. Conservatively, the maximum axial force and maximum bending moment may be considered to be acting together, even though they are not concurrent. See Article 3.7 for further information.

a. **Example 1:** The axial forces for the truss members are extracted for dead load and for live load under each of the rating vehicles. These results are used to rate the structure.

b. **Example 2:** The axial forces and bending moments for the truss members resulting from the response spectra analysis are extracted and used to determine where seismic retrofits are required.

3.3—GEOMETRY

When constructing truss models, whether a simple two-dimensional model or a more complicated three-dimensional model, it is typically advisable to establish the nodal geometry of the truss in the model in a simplified manner. Generally, locating the nodes at the intersections of the centerlines of the truss members, or along the centers of gravity of the truss member cross-sections, is an effective modeling strategy. The choice of using centerline geometry versus center of gravity geometry can be based on the shape of the cross-sections of the truss members. If the truss members are doubly symmetric, or nearly so, such that the center of gravity of the cross-section is at or near the centerline of the members, the centerline geometry can be used. If the truss members are singly symmetric and the center of gravity is a significant distance from the centerline of the members (such as might be the case for T or angle shaped members), then using center of gravity geometry may be more advisable. In either case, nodes along the upper and lower chords of the truss should be located in line with each other.

Eccentricities representing differences between the actual center of gravity of the truss member sections and the nodal geometry (see example in Figure 3.3-1) can typically be accounted for in most structural analysis software using member end offsets. Whether to even account for such offsets is typically a decision which involves some level of engineering judgment. These offsets represent eccentricities which can induce bending moments in the truss members. If these bending moments are significant, they probably should be accounted for in the model. If the eccentricities and resulting bending moments are minor and not significant, the value associated with simplifying the analysis model probably will outweigh the perception of "additional refinement." Adding too many unnecessary complications to a truss bridge analysis model typically leads to inefficient use of budget and schedule resources and models which are difficult to debug and which can produce misleading results.

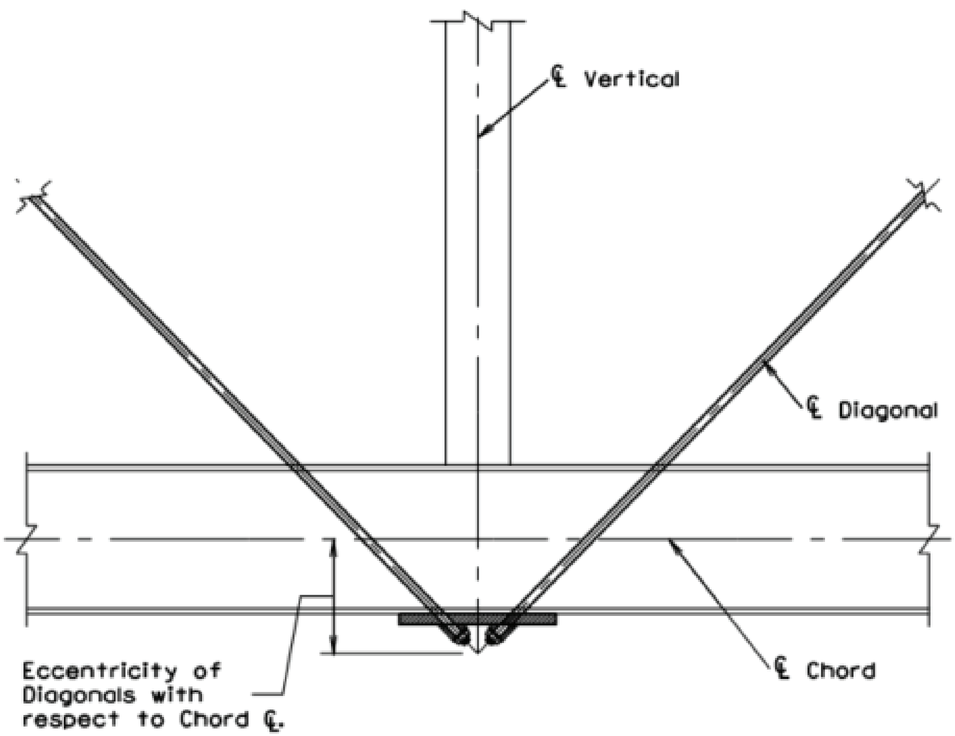


Figure 3.3-1—Example of eccentricity induced by misalignment of truss member centerlines.

Another geometric consideration for truss analysis is skew. Skewed trusses have supports with bearing centerlines that are not perpendicular to the truss lines but are parallel to the substructure supports. As a result, at any point along the bridge centerline, the deflections and member forces of one truss (e.g., the left) are not the same as the other truss (e.g., the right) as these two points are not the same distance into the span from the adjacent supports along their respective truss lines. The general behavior of skewed trusses can be considered analogous to skewed girder bridge behavior, which is well covered in other literature. While there are differences due to framing geometry and relative stiffness of lateral elements (cross frames or floor systems and sway bracing) to longitudinal elements (girder or trusses), the analogy works from a qualitative sense.

In addition to the location of nodal geometry, the selection of two-dimensional or three-dimensional analysis and load application are affected by the skew angle and how the floor system and bracing are oriented. Below is a list of items the Engineer should consider when developing an analysis model for a skewed truss bridge:

- **Two-Dimensional vs. Three-Dimensional Analysis:** The following items in this list affect the Engineer's decision to model the structure as two- or three-dimensional. Increasing complexity or the necessity for refined results generally increases the need to model the structure in three dimensions. If the truss is modeled in three dimensions, the following items can be modeled implicitly. However, depending on the severity of these items and proper selection of geometry and loading, a two-dimensional analysis model may be adequate to accomplish the project goals. In fact, many skewed trusses were successfully designed and constructed prior to the advent of computer-aided design.
- **Skew Angle:** Like multi-girder bridges, skewed trusses behave similar to square (not skewed) trusses up to a certain level of skew. In multi-girder bridges, this limit is often considered to be approximately 20 degrees from perpendicular. There is not a similar accepted rule for trusses, so the Engineer must use their judgment. In conjunction with the level of skew, the floor system and bracing orientation, discussed below, also must be considered in that decision.
- **Floor System Orientation:** Another geometric consideration is the orientation of the floor system. Specifically, are the transverse floor beams parallel to the skewed supports (connected to the same panel points on opposing truss lines) or perpendicular to the bridge centerline (connected to different panel points on opposing truss lines)? Like multi-girder bridges with a skew of 20 degrees or less, where it is recommended that cross frames are parallel to the skewed supports, skewed floor beams attached to equivalent points on each truss will result in similar behavior as a square bridge. A floor system that is square to the bridge centerline introduces more noticeable three-dimensional effects into the structure, including a "twisting" of the overall cross-section since the connection of the floorbeam to the truss is at a different location along the span of the left truss, compared to the connection location on the right truss. Additionally, floor beams and stringers near the end supports will have varying lengths that need to be considered in their analysis.
- **Lateral Bracing and Sway Frame Orientation:** Like floor systems, the orientation of lateral bracing (top and bottom chord) and sway frames can affect how a skewed truss behaves compared to a square one. Counterintuitively, bracing systems and sway frames that are parallel to the skewed supports result in more "square" behavior.
- **Analysis Intent:** When considering if the skew necessitates the added complexity of a three-dimensional analysis, the intent of the analysis must be considered. Is the analysis for a load rating, where only the primary member forces (chords, diagonals, and verticals) are necessary which can be obtained from a two-dimensional analysis, or is it a new design where more precise secondary member forces which can be obtained from a three-dimensional analysis (bracing and floor system) are desired? If secondary member forces are required, is the skew small enough that they can be modeled or hand-calculated separately using classical approximate methods to eliminate the need for a more complex three-dimensional analysis?
- **Load Determination and Placement:** If a three-dimensional analysis is selected, the placement of loads is self-evident. However, for a skewed truss modeled in two dimensions, bracing and floor system loads are generally applied as point loads to truss nodes. The orientation of the floor system and bracing (parallel to the skewed supports or perpendicular to the bridge centerline) will affect how dead and live loads are applied to the truss. This is particularly true for the perpendicular configuration at truss panel points near the end supports. Often, the intermediate floor beams at these locations will be attached on one end to the truss at the acute corner and the other end will rest on the substructure or frame into a skewed end floor beam.

3.4—SECTION PROPERTIES

The section properties required to analyze a steel truss vary depending upon the analysis method.

- In conventional two-dimensional models where members are assumed to be connected by pin joints, only the cross-sectional area is needed, as the members will carry only axial loads (however, it is generally prudent to also include some value for the shear area, the moment of inertia, and torsional constant to minimize the risk of mathematical problems occurring when the stiffness matrix is inverted).
- In conventional two-dimensional models where members are assumed to be restrained against rotation at the joints (such that secondary bending moments can develop), the cross-sectional area, the shear area, and the moment of inertia in the plane of bending are all required (however, it is generally prudent to also include some value for the torsional constant to minimize the risk of mathematical problems occurring when the stiffness matrix is inverted).
- In three-dimensional models, depending upon the configuration of the structure and the assumptions made about connection fixity, members may develop axial loads, bending along both axes, and torsion. In this situation, the cross-sectional area, the shear area, the moment of inertia for both bending axes, and the torsional constant are all required.

In general, gross section properties should be used in the analysis model. The section properties used in the model are intended to represent the effective stiffness of the member, rather than the critical section used to determine the member stresses and member capacity. In most cases, local discontinuities such as section loss or fastener holes do not significantly affect member stiffness, and thus should not be included in the model. Special consideration is necessary for the effective stiffness of latticed members, discussed in Article 3.4.3.

The moment of inertia determines the bending stiffness of a member in the strong and weak axes. Typically, the moment of inertia is determined using the gross section. This may be affected by the size of openings in the member. At a bolted splice location, the section properties can typically be determined using the gross section, provided the connection is detailed as a full moment connection. Similarly, a large opening in the web of an I-girder shape can often be neglected if its presence will have a small change in the moment of inertia of the member, particularly if it is located at or near the neutral axis. Conversely, however, if there is a large opening in the flange of an I-girder or box member, it will have a significant impact and a reduced moment of inertia should be determined that reflects the net section.

The considerations for modeling the torsion constant are similar to those for modeling the moment of inertia. The torsion constant of an I-shaped section is generally relatively small; I-sections (as open cross-sectional shapes) typically carry torsion primarily by means of warping torsion (flange lateral bending) rather than by St. Venant torsion. The torsion constant of a box girder can typically be based on gross section properties unless there is a large opening in the web or flange such that the section behaves more like an open section rather than a closed section; the torsional stiffness and resistance of an open section are significantly less than that of a comparably sized closed section. It is recommended that for members with large openings and significant torsion, a localized finite element method analysis be utilized to determine how the load will transfer around the opening.

In all cases, the designer should consider the nature of how loads are introduced to a member. Singly symmetric or asymmetric sections that are not loaded through their shear center can be subjected to significant torsion and members that are not loaded through their centroid may be subjected to bending, which should be considered in their design.

3.4.1—Rolled Shapes

Dimensions and section properties for current standard rolled shapes are tabulated in the American Institute for Steel Construction (AISC) *Steel Construction Manual*. Dimensional information for older, non-standard shapes can be more difficult to determine. In the past, the various mills were not standardized. Steel manufacturers historically rolled their own shapes and published catalogs with tabulated dimensions and section properties.

Many of these catalogs are available for download on the AISC website. Another source for information on historical shapes is AISC's *Design Guide 15: Rehabilitation and Retrofit*. For existing bridges for which tabulated data for a historical section cannot be located, field measurement of the section may be required.

3.4.2—Built-Up Members

Built-up members consist of interconnected components joined together with welds or fasteners to act as a single element. Components can be connected directly with welds or mechanical fasteners or connected indirectly using battens or lacing. Members connected with battens or lacing are referred to as "latticed members" and are discussed separately in Article 3.4.3.

Limitations on the longitudinal spacing of connectors between components are set in the *Specification for Structural Steel Buildings* (AISC, 2022) in sections D4 (tension members) and E6 (compression members). Connectivity limitations are also discussed in Article 6.9.4.3 of the *AASHTO LRFD BDS* (2024). Section properties for directly connected (non-latticed) built-up members that satisfy applicable connectivity limitations can be calculated assuming that the member components act as a single composite unit.

3.4.3—Latticed Members

Latticed members have largely been replaced by perforated cover plates but may be found as tension and compression members in older trusses. Latticed members are built up from primary members, usually angles or channels, connected by lacing or battens. Stay plates, or tie plates, are placed at the ends of the members and where the lacing or battens are interrupted. Lacing or battens usually consist of flat bars, though angles were sometimes used. Figure 3.4.3-1 shows a through-truss bridge with latticed members with lacing used for compression and tension diagonal members and latticed members with battens used for vertical members of the main trusses in the foreground. Latticed compression members with battens are not often found on bridges as early bridge engineering books discouraged their use. Design of latticed compression members in old bridges were often based on rules-of thumb for proportioning along with the requirement that the slenderness ratio, *KL*/*r*, of the primary members between lacing connections not exceed 75 percent of the overall member *KL*/*r*.

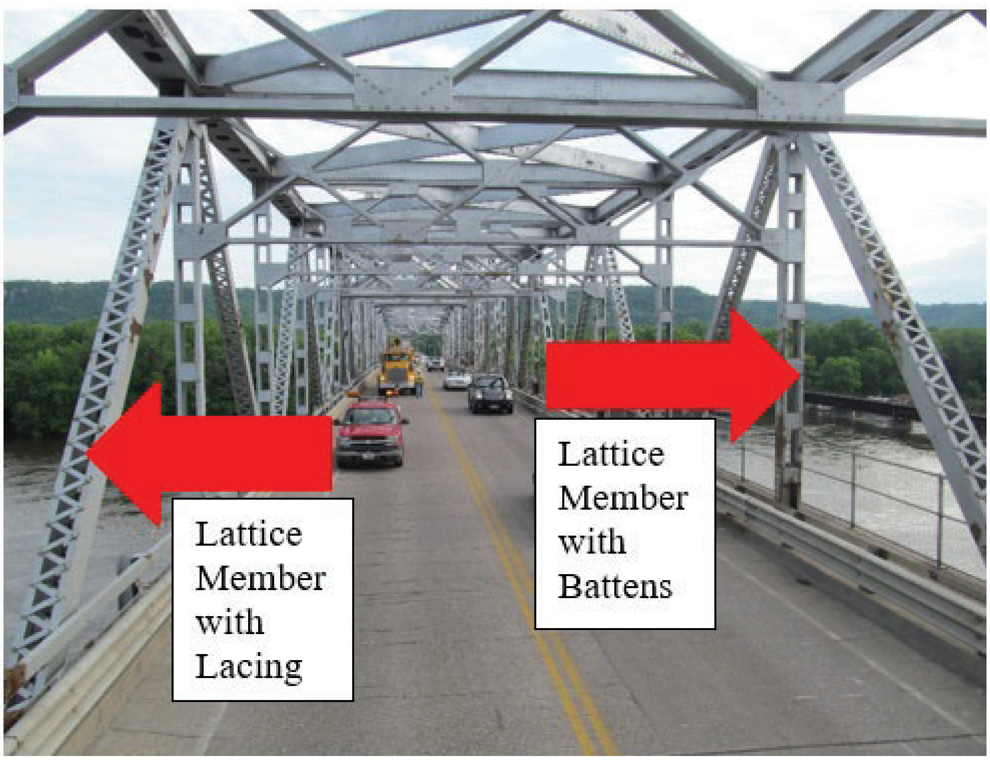


Figure 3.4.3-1—Typical latticed and batten member configurations.

Where the analysis of latticed or battened members requires calculation of section properties other than those provided in *AASHTO LRFD BDS* (2024) Article 6.9.4.3, as may be needed in seismic analysis, the procedures contained in work by Duan, Reno, and Lynch (2000) may be used. Their procedure considers actual section integrity for a latticed member by the reduction factors, b_m for moment of inertia and b_t for torsional constant, to account for the shear flow transferring capacity of lacing bars or battens and their connections. Refer to Duan, Reno, and Uang (2002) for procedures for localized component buckling modes (sometimes termed compound buckling), for latticed or battened members. For the exact solution to compound buckling, see Timoshenko and Gere (1963) Section 2.18.

3.5—MODELING MEMBERS AND CONNECTIONS

3.5.1—Analysis of Truss Panel Members

The vast majority of two-dimensional and three-dimensional truss analyses will be performed using one-dimensional elements, such as truss or bar elements, to represent primary truss members. As such, modeling of connections between these primary members is as simple as connecting the one-dimensional elements together within the software. Most analyses do not require the explicit consideration of the stresses within the connections, as these types of checks are traditionally completed outside of the software program using the member end forces as inputs. However, the end-fixity of the members must be considered to obtain the proper behavior of the truss and a reasonable distribution of stresses for connection analysis. If the member is physically pin-connected (often seen in older trusses), modeling the member with pinned ends is appropriate. If the member is connected to the other members using bolted, riveted, or welded connections, modeling the member with pinned ends can provide a reasonable approximation of the forces in the members. Modeling with fixed ends may be warranted in some instances to obtain more accurate forces in the members. For more details on when analysis using fixed-end connections is warranted, refer to Article 3.5.10.

For localized analyses of connections, higher-order elements such as shell or solid elements may be used. Truss member ends and connection/gusset plates in these types of analyses can be modeled using shell or solid elements, with beam elements representing the truss members between the connection regions. In these types of analyses, member end-fixity is explicitly considered. However, this type of connection analysis is generally reserved for research-type activities, or when concerns with a particular connection arise in practice (e.g., inadequate ratings or visible distress) where such analysis may be warranted.

3.5.2—Analysis of Truss Floor Systems

Truss floor systems typically consist of floorbeams, stringers, and the bridge deck. Floorbeams span transversely between the joints of the main trusses at panel point locations. For through trusses, each end of a floorbeam is typically connected to a truss at or near the joint between the lower chord, vertical, and (if present at the connection) diagonal truss members, as shown in Figure 3.5.2-1. For deck trusses, the floorbeam connection occurs at or near the joint between the upper chord, vertical, and diagonal members. Steel stringers span longitudinally between or on top of the floorbeams, parallel to the trusses, and support the bridge deck.

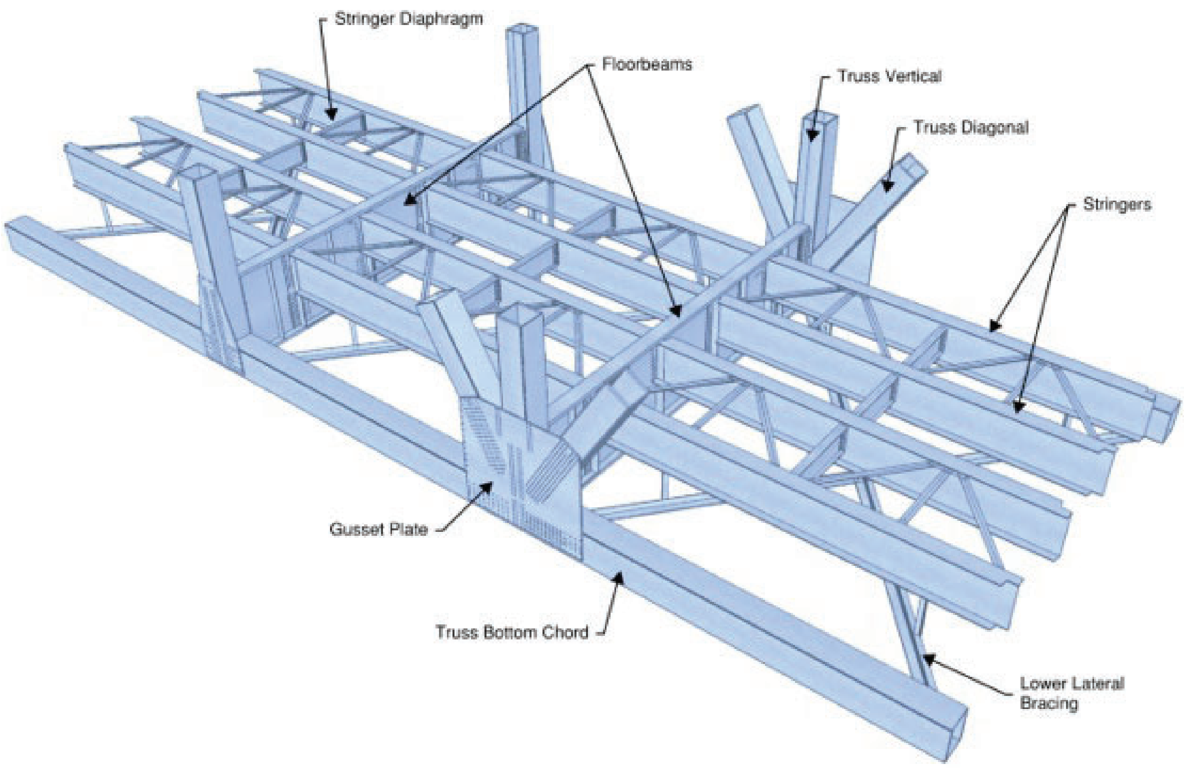


Figure 3.5.2-1—Typical through-truss floor framing members.

Stringers typically consist of rolled steel I-shapes, and floorbeams typically consist of rolled steel I-shapes or built-up (welded, bolted, or riveted) steel I-shaped members. A typical cross-section of a through-truss floor system is shown in Figure 3.5.2-2. In some cases, floor trusses that span between the main trusses are used instead of floorbeams to support a wide deck, as shown in Figure 3.5.2-3.

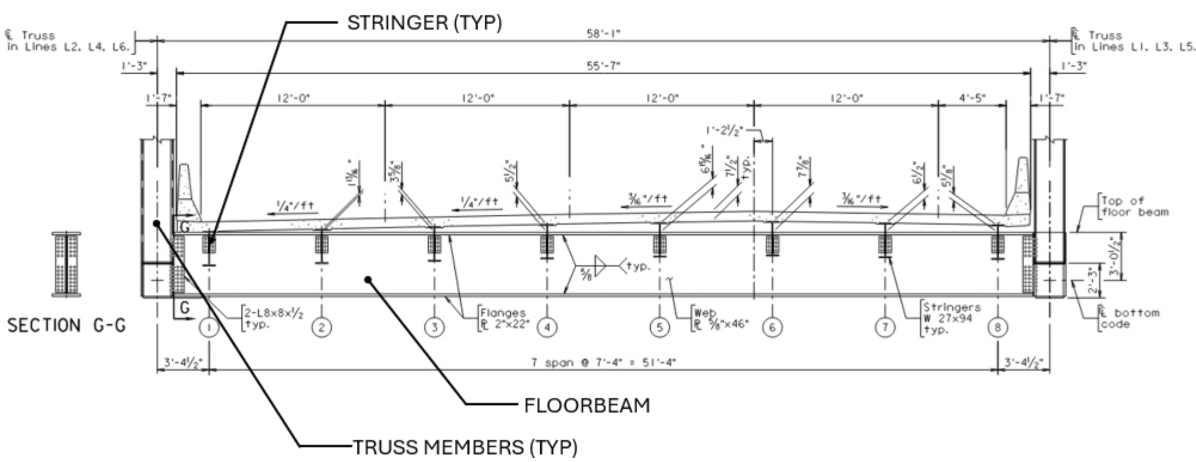


Figure 3.5.2-2—Typical cross-section of the floor system framing of a through truss showing the truss members, floorbeams, and stringers.

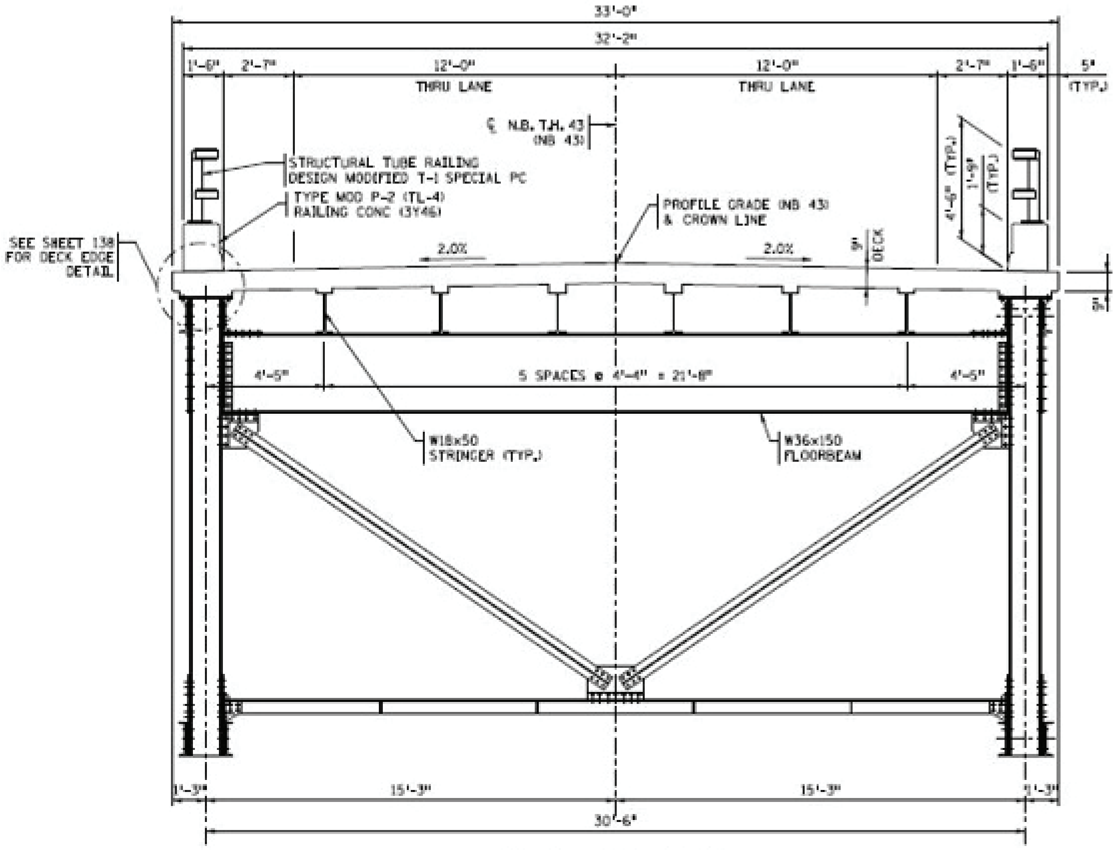


Figure 3.5.2-3—Floor truss spanning between the main trusses.

3.5.2.1—Stringer Analysis

For most trusses, line girder analysis methods are appropriate for analyzing stringers subject to in-plane flexure and shear. The analysis is similar to a multi-beam bridge where the line girder model utilizes beam elements to represent one stringer and the model's boundary conditions are adjusted to represent the support conditions for the stringer. Dead loads are applied based on the weight of the deck and superimposed dead loads falling within the tributary width for the stringer. Live loads (vehicles and lane loads) are applied as moving loads along the length of the stringer and live load distribution factors from Article 4.6.2.2.1 of the *AASHTO LRFD BDS* (2024) can be used to calculate the demands based on the number of lanes supported by each stringer. Moment and shear demand from the line girder model are then used to design the stringers or to perform a load rating analysis. Additional considerations related to the stringer boundary conditions and composite action are discussed in the following sections.

3.5.2.2—Stringer Continuity

Stringers can be supported by the floorbeams in various ways, as shown in Figure 3.5.2.2-1. Often each end of the stringer is attached to the floorbeam using a bolted connection between the stringer web and a floorbeam stiffener/connection plate, or using a clip angle connection to attach the stringer web directly to the floorbeam web (Figure 3.5.2.2-1(a) and (b), respectively). In both cases, the stringer behaves approximately as simply supported between the floorbeams, provided the stringer flanges are not directly attached to the floorbeam.

Stringer positive moments may be reduced by providing continuous stringers that can develop positive and negative moments. Stringer continuity is often achieved by having stringers that are continuous over the length of several truss bays, which are supported by the top flange of the floorbeams (Figure 3.5.2.2-1(c)). Stringer splices (including both flange and web plates) are then typically located near inflection points, similar to beam bridges, in order to provide manageable stringer lengths for fabrication and erection. Stringer continuity may also be provided with the use of flange continuity plates to splice the ends of the stringers at each floorbeam.

(a) (b) (c) (d)

Figure 3.5.2.2-1—Typical stringer connections, including (a) a clip angle connection between the stringer web and the floorbeam (FB) web, (b) stringer web connected to a floorbeam stiffener, (c) stringer that is continuous over the floorbeam, and (d) seated connection at an expansion joint.

Whether the stringers are simply supported between floorbeams or continuous over the intermediate floorbeams, the stringer connections described above can develop axial forces due to their longitudinal stiffness and load sharing with the adjacent truss chord under live load or due to thermal effects. In long-span trusses, expansion joints may be needed in the floor system to limit the build-up of axial forces in the stringers and deck due to live load sharing and/or thermal effects. Expansion/contraction joints in the truss floor system may consist of a seated beam connection to support the stringer on the floorbeam (Figure 3.5.2.2-2 and Figure 3.5.2.2-3) or a pin-and-hanger connection which are typically found on older trusses (see Figure 3.5.2.2-4). A line of diaphragms is often provided at stringer expansion joints to provide lateral stability for the stringer ends. In some cases, clip angle connections with slotted holes can also provide lateral stability for the stringer web (Figure 3.5.2.2-1(d)). Seated beam connections and pin-and-hanger joints are analyzed with axial and moment releases in the stringer at these locations.

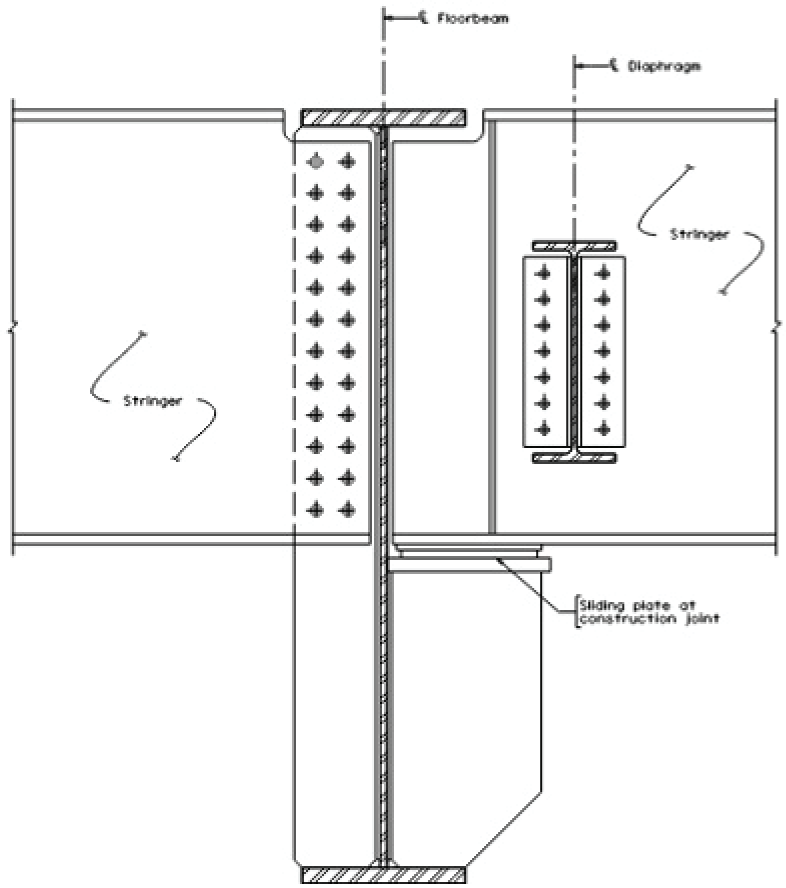


Figure 3.5.2.2-2—Example of a seated beam connection for a stringer expansion/contraction joint.

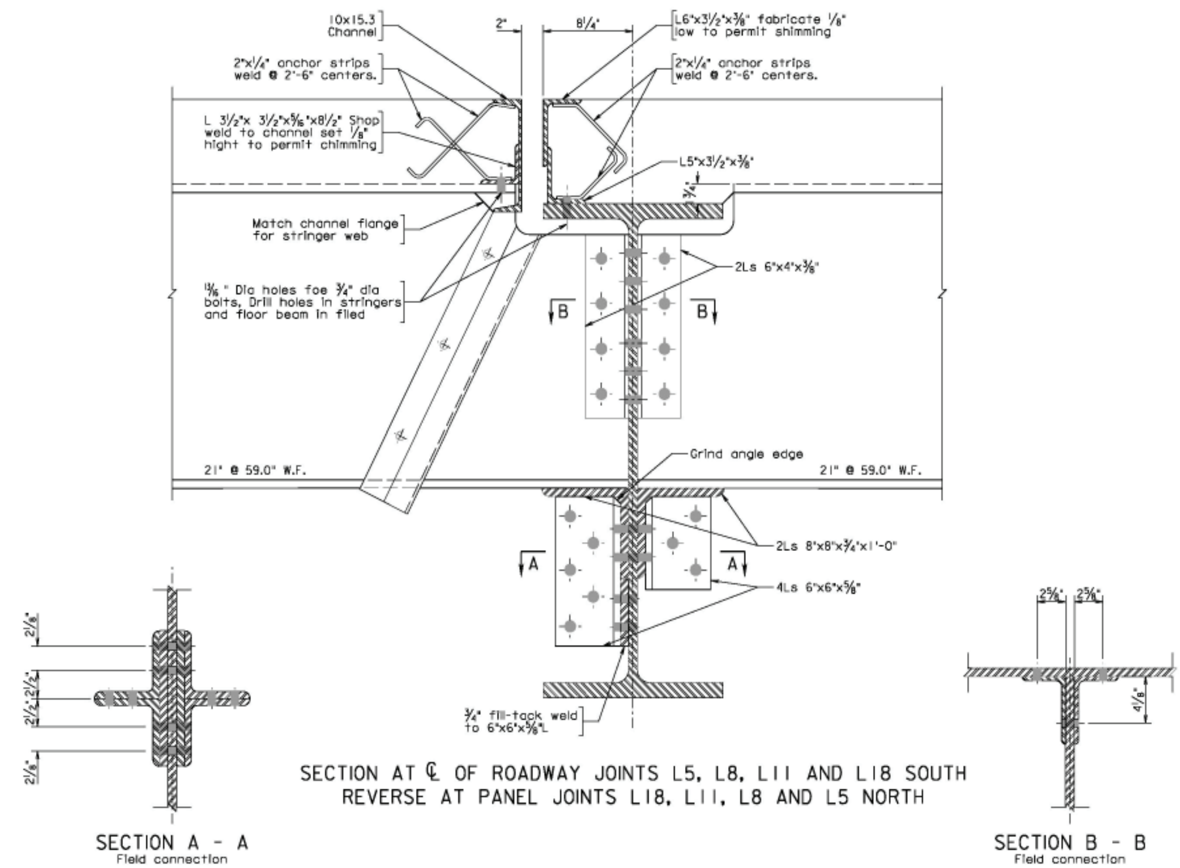


Figure 3.5.2.2-3—Sliding stringer to floorbeam connection detail at deflection joints.

Figure 3.5.2.2-4—Example of a pin-and-hanger connection for a stringer expansion/contraction joint.

3.5.2.3—Stringers—Composite vs. Noncomposite

For stringer analysis, the dead loads of the stringer self-weight, deck slab, and haunches (*DC*1 loadings) are applied to the noncomposite stringer cross-section. The stringer analysis procedure for dead loads applied to the hardened deck slab (*DC*2 loadings) and live loads will depend on whether or not the stringers are made to be composite with the deck slab with the use of shear studs. For composite stringers, the composite section of each stringer (including a tributary width of the deck slab) will resist superimposed dead loads applied to the composite section as well as live loads. The live load distribution factors of Article 4.6.2.2.1 of *AASHTO LRFD BDS* typically may be used to determine the number of lanes supported by each stringer. For noncomposite sections, *DC*2 loads and live loads would instead be resisted by the noncomposite steel stringer cross-section.

For both noncomposite and composite stringers, self-weight, haunch, and concrete deck slab loads will be resisted by a discretely braced or unbraced compression flange, depending on if intermediate diaphragms connect the stringers. For loads applied to the hardened deck slab, the top flange is often considered to be continuously braced regardless of whether shear studs are used for loads applied to the hardened deck slab. Refer to *AASHTO LRFD BDS* (2024), Article C6.10.1.5.

New truss bridges and existing truss bridges that have undergone prior rehabilitation will typically use shear studs on the stringers, and therefore resist DC2 loadings and live loads with a composite cross-section. Older truss bridges that have not had a prior deck rehabilitation may have noncomposite stringers. Review of the original truss bridge design plans and the design plans of prior rehabilitations is typically needed to determine whether existing stringers are composite with the deck slab.

3.5.2.4—Stringers Coped Ends

When stringers are attached to the web of the floorbeams, the ends of the stringers are usually coped to clear the floorbeam top flange. Coping the stringer end permits the top flange of the stringer to be elevated above the floorbeam top flange to support the bridge deck. An example of a coped stringer end is shown in Figure 3.5.2.4-1. The reduced section at the coped end of the stringer should be carefully considered in the stringer design and may control the moment or shear capacity, particularly if the cope results in a reduction in the web depth or occurs over a significant length. However, this reduced section is not typically considered in the section properties used to define the stiffness matrix of a finite element model; it is only used for computing the strength of the member at its end. In addition, the capacity may be controlled by local buckling of the stringer web. For the evaluation of existing bridges, the coped ends of stringers may be prone to deterioration if they are located near expansion joints. Section loss of the web and bottom flange at the dapped end should be considered when performing the stringer evaluation. Refer to AISC *Steel Construction Manual* (AISC, 2023) Section 9-6 for additional information.

Figure 3.5.2.4-1—Example of a coped stringer end (red circle).

3.5.2.5—Stringers in Parallel with Deck Truss Upper Chords

Some newer deck trusses use a structural configuration where the stringer top flange and truss upper chord are at the same elevation and are both made composite with the concrete deck slab. These stringers may be analyzed with the same procedures used for other composite stringers, which is similar to an analysis for multi-beam bridges. When this configuration is used, the stringers are typically continuous over several intermediate floorbeams and may require consideration of cumulative deflections, as discussed below. The upper chords of deck trusses with this configuration must be designed for primary moments applied by the deck dead loads and live loads between panel points in addition to axial forces. The new deck truss approach spans to the existing Winona Bridge through-truss main river crossing are one such example of this structural configuration, as shown in Figure 3.5.2.5-1. The Winona Bridge includes a historic cantilever through-truss bridge with a main span of 450 feet that spans over the Mississippi River in Winona, MN, and was opened to traffic in 1942.

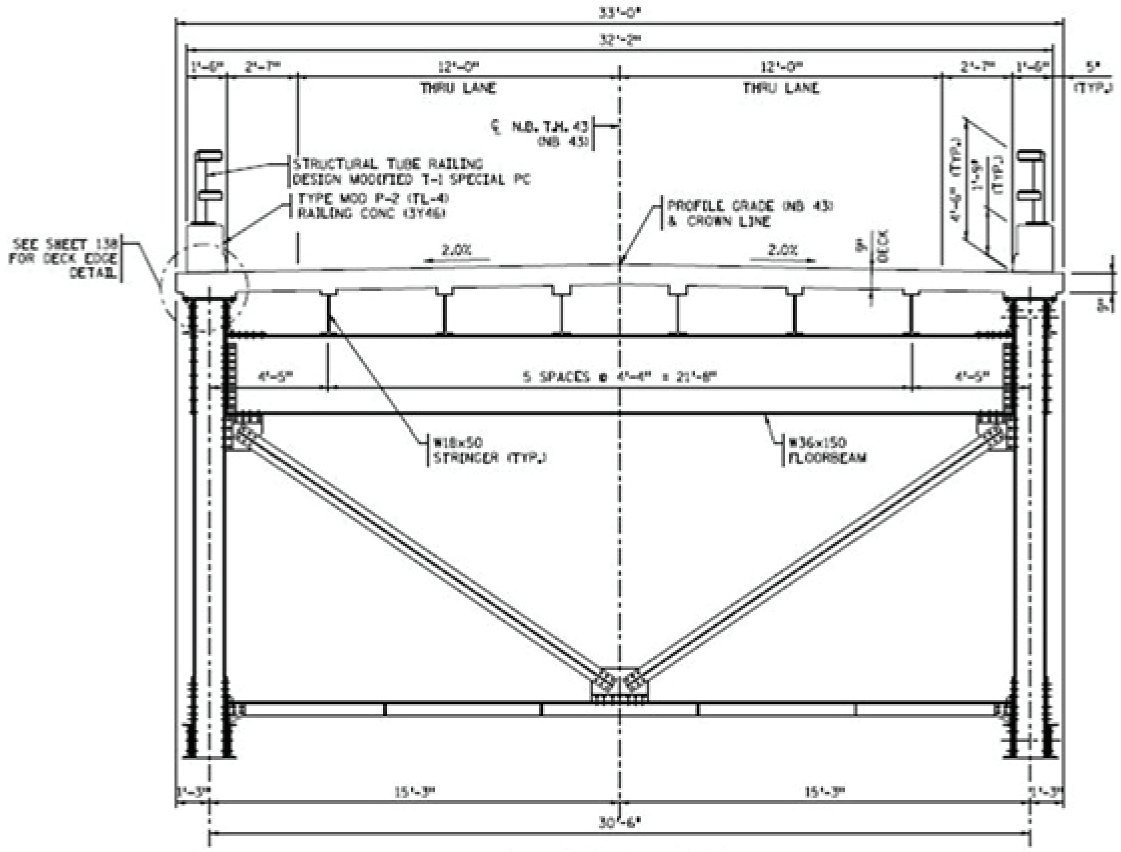


Figure 3.5.2.5-1—Winona Bridge new deck truss approach span transverse section, showing deck slab composite with upper truss chords.

3.5.2.6—Cumulative Deflections

When floorbeams span a long distance or have a relatively low stiffness to resist deflections, deflection of the floorbeam may influence the behavior of the supported stringers. For example, when placing a concrete deck, the deflected shape of a stringer will depend on its own stiffness as well as the stiffness (and deflection) of the floorbeams supporting the stringers. Another example is if the screed machine is supported near the truss lines, then as the floorbeam deflects from concrete placement it will deflect while the screed does not, increasing the concrete deck thickness during the construction activity when compared to the screed machine dry-run. Therefore, haunch heights may need to be adjusted to achieve the correct deck profile. Similarly, when stringers are continuous over intermediate floorbeams, the live load moment and shear diagrams for the stringer may be influenced by the differential deflections of the floorbeams and global deflections of the truss. In other words, the stringer "support" can deflect due to local floorbeam flexibility and overall truss flexibility. The deflection of the stringer supports can change the moment and shear magnitudes and distribution as compared to a simple line girder analysis with rigid supports. In many cases, a check of floorbeam deflections (local) and truss deflections (global) from dead and live loads will often reveal that combined, or cumulative, deflections of the stringer support are negligible.

If needed, floorbeam stiffness values may be included in a line girder analysis model for the stringers by deriving the appropriate spring constants at each floorbeam support. The forces developed in continuous stringers due to differential floorbeam deflections may alternatively be calculated by including the continuous stringers, floorbeams or floor trusses, and main trusses in a three-dimensional analysis model. A three-dimensional analysis model may be more appropriate when the flexibility of the truss is considered significant and the moments and shears in the continuous stringers may be affected by global deflections of the truss. A three-dimensional analysis approach was taken for the analysis of the Innerbelt Bridge due to the structure's width and horizontal curvature. The three-dimensional analysis model of the Innerbelt Bridge is shown in Figure 3.5.2.6-1.

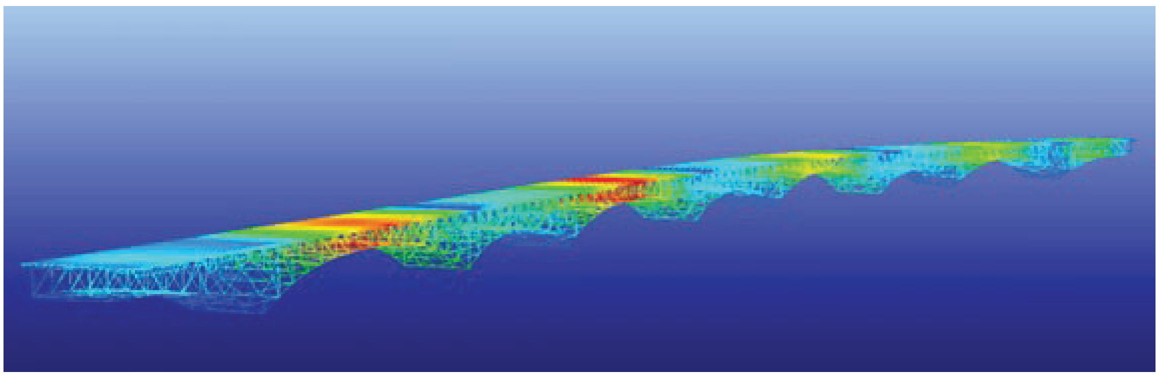


Figure 3.5.2.6-1—Three-dimensional analysis model of the Innerbelt Bridge.

3.5.2.7—Floorbeam Analysis

Floorbeams are also typically analyzed by line girder models to calculate in-plane moments and shears. The lever rule may be used to efficiently determine live loads on floorbeams. The lever rule analysis includes a line girder analysis of the stringers to determine the longitudinal distribution of live load for a given design vehicle between adjacent floorbeams of the truss. Influence lines for the shear where the stringer is supported by the floorbeam may be derived, and the design vehicle is placed on the influence line to maximize the stringer end shear and hence the loading on the floorbeam where each stringer is supported. In other words, a line of wheel loads, parallel to the stringers, is positioned on the bridge deck to obtain the maximum reaction at the floorbeam.

For the purposes of the discussion that follows, R_{WL} is defined as the reaction that occurs at the transverse floorbeam being analyzed from a line of longitudinal wheel loads, with due consideration of stringer continuity and support conditions. This reaction is then applied to the bridge deck, which is considered to consist of simply supported segments spanning between adjacent stringers. The wheel load reactions, R_{WL}, are placed on the deck segments in different configurations to maximize live load moments and shears in the floorbeams, as shown in Figure 3.5.2.7-1 and Figure 3.5.2.7-2, respectively. The wheel load reactions are applied to the floorbeam at the stringer locations. When performing this analysis using the lever rule, the multiple presence factors of *AASHTO LRFD BDS* Article 3.6.1.1.2 are manually applied to the wheel loads.

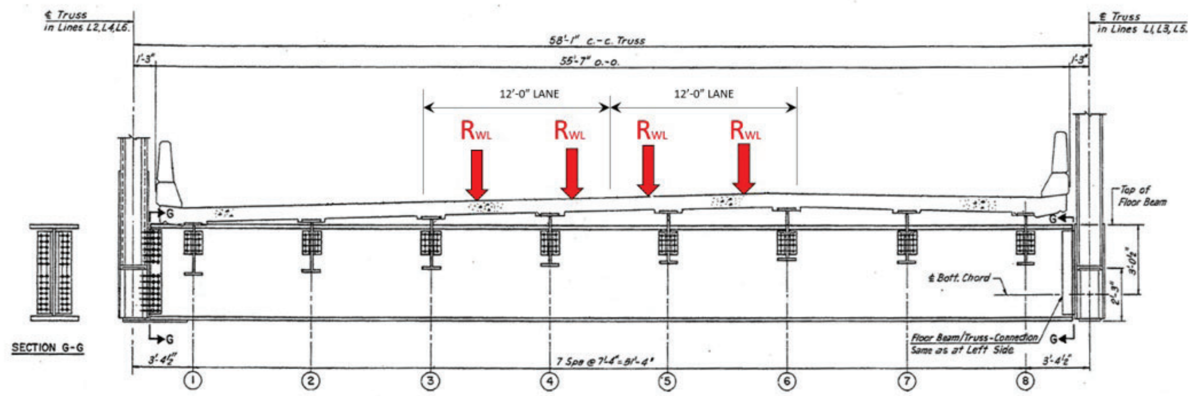


Figure 3.5.2.7-1—Positioning of the wheel line reactions, R_{WL}, for maximum floorbeam positive moment (two lanes loaded shown).

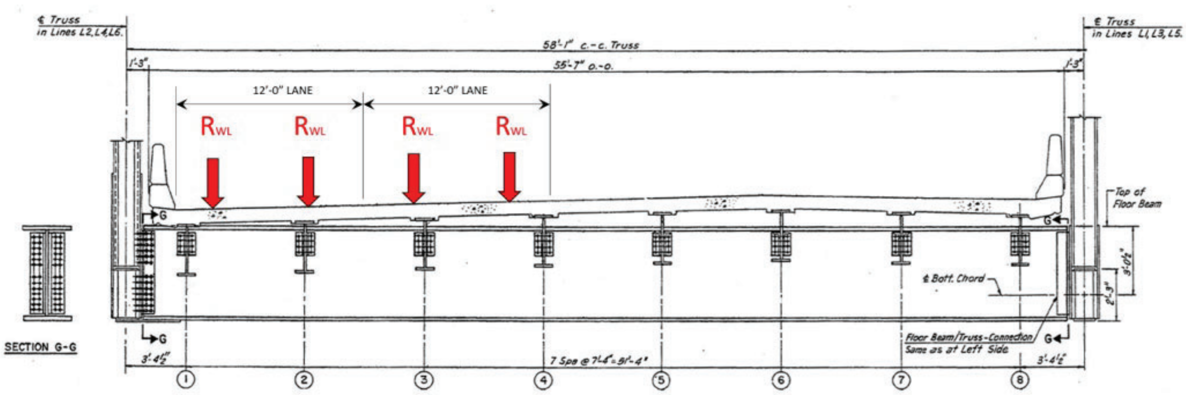


Figure 3.5.2.7-2—Positioning of the wheel line reactions, R_{WL}, for maximum floorbeam end shear (two lanes loaded shown).

Stringers typically act as discrete bracing points for a floorbeam when the attached stringers prevent lateral displacement of the floorbeam top (compression) flange. When stringers are rigidly connected to floorbeam webs or stiffeners, the floorbeam top flange will be discretely braced for loads applied before the concrete deck slab has hardened and continuously braced if the hardened concrete deck is composite with the floorbeam top flange. When the floorbeams are not braced by the concrete deck or if stringers are continuous over the floorbeam top flange (resting on top of the floorbeam), the floorbeam top flange will be discretely braced at each stringer if the stringer connection prevents lateral displacement of the floorbeam flange. Stringers that are free to expand/contract longitudinally, relative to the floorbeam, should not be relied upon for discrete bracing of the floorbeam. The unbraced length of the floorbeam is typically accounted for in the capacity calculations.

3.5.2.8—Floorbeam End Connection Fixity

Floorbeams of truss systems are traditionally considered to be connected to the main truss members with pinned (shear) connections at their ends. In these cases, the floorbeam webs are typically connected to the main truss member joints, or gusset plates, with double-angle connections. The detail of the Winona Bridge floorbeam to lower chord double-angle connection, for example, is shown in Figure 3.5.2.8-1. In this case, two sets of double-angles are used to fit the angles on the built-up truss lower chord.

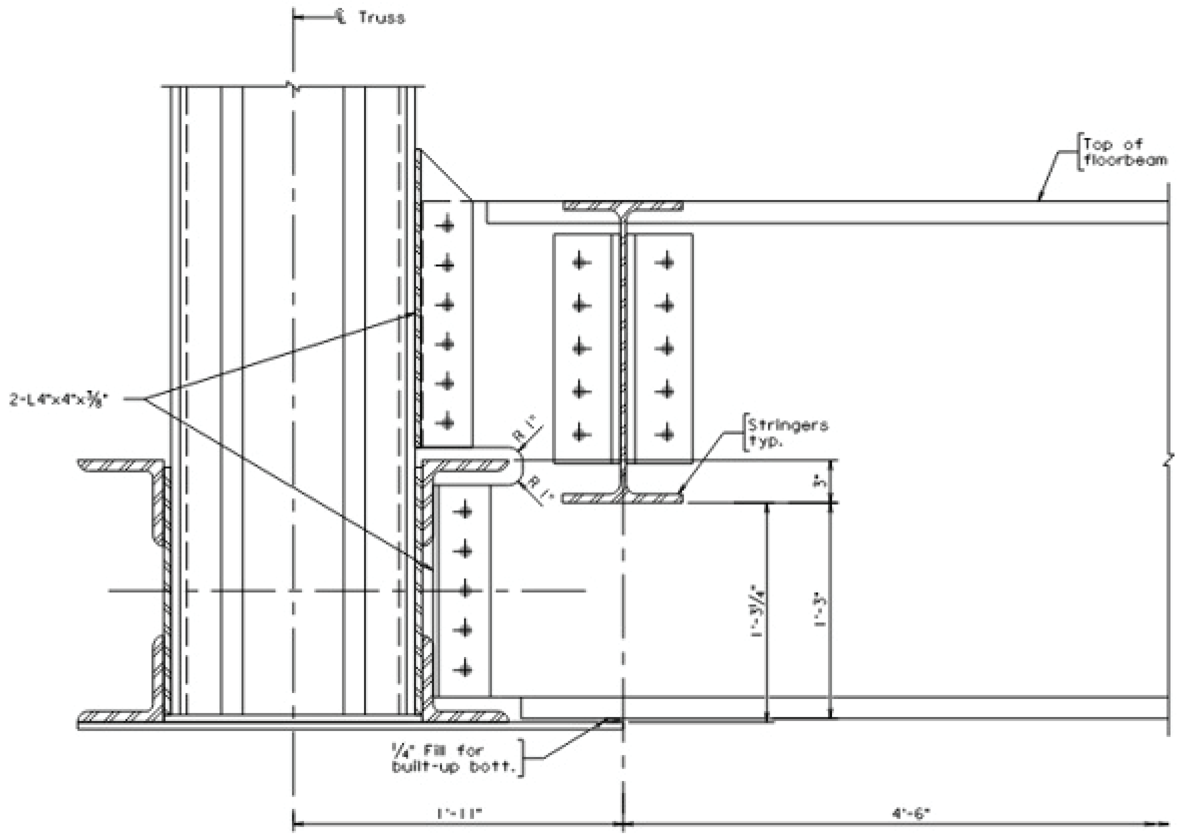


Figure 3.5.2.8-1—Winona Bridge floorbeam to lower chord double-angle connection.

Some flexural restraint can develop in these connections depending on their size and layout and whether the floorbeam top or bottom flanges are connected as well. When analyzing floorbeams, Engineers often neglect the flexural restraint at the floorbeam ends in superstructures consisting of two trusses because the end moment is transferred to weak axis bending in the main truss diagonals and verticals, which are typically flexible enough to relieve the floorbeam end restraint. Additionally, an analysis of the fastener configuration used in these connections will often show that the flexural capacity of the connection is very limited. A floorbeam analysis with pinned-end connections is also usually conservative since it will maximize floorbeam positive moment; however, negative moment in floorbeam end connections will often need to be considered when the floorbeam's flanges are connected to the truss panel point (such as gusset plates for lateral bracing), the truss diagonal and vertical members have a high bending stiffness (such as box sections), or the floorbeam is continuous across the truss (such as to support a cantilevered sidewalk). The assumption of a pin-connected end is a reasonable and conservative assumption for strength level loading; however, if the behavior of a truss at service-level loadings is needed, a fixed connection will likely provide a better representation of the truss's initial behavior at these lower level loads.

In systems with three or more trusses, flange continuity plates are often provided so that the floorbeams are continuous across the interior trusses. In these cases, negative moments are developed in the floorbeam due to the continuity. Similarly, floorbeam flange continuity plates are sometimes provided in truss systems with a supported sidewalk on the outboard side of the truss. When an outboard sidewalk is present, the negative moment at the end of the main floorbeam is typically limited to the negative moment developed from the sidewalk dead load on the floorbeam cantilever. Examples of floorbeam flange continuity plates at a cantilevered sidewalk and at an interior truss are shown in Figure 3.5.2.8-2.

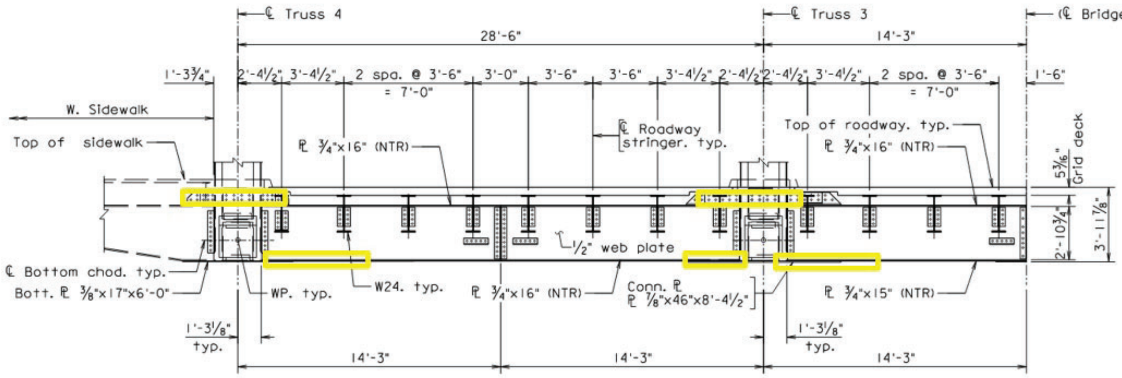


Figure 3.5.2.8-2—Floorbeam flange continuity plates (highlighted in yellow) for a cantilevered sidewalk (left) and for floorbeam continuity across an interior truss (right).

3.5.2.9—Two-Dimensional vs. Three-Dimensional Analyses

As discussed previously, line models are typically adequate for analyzing stringers and floorbeams used in truss floor systems. In rare cases, a three-dimensional model may be needed to evaluate stringers and floorbeams. For example, when performing a load rating on the stringer and floorbeam elements, the use of the lever rule (see *AASHTO LRFD BDS* [2020], Article C4.6.2.2.1) and AASHTO code-specified live load distribution factors are often conservative. The use of a three-dimensional model for the stringers and floorbeams, which may or may not include the truss members, will likely result in a more refined, potentially less conservative, prediction of the distribution of a truck's wheel loads and reduce the calculated demands for the stringers and floorbeams, but may redistribute negative moment in the stringer to positive moment regions in superstructures with flexible floorbeams or trusses. Shell elements or grid systems can be used to represent the deck for application of live loads; however multiple models may be necessary to account for composite and noncomposite loading and behavior. Local or global three-dimensional models may also be needed to calculate cumulative deflections and moment demands for stringers supported on long-span floorbeams.

Another common use of three-dimensional models is to evaluate longer-span trusses to determine the need for intermediate expansion joints in the floor system. These three-dimensional models typically show the longitudinal elements of the floor system (stringers, deck, and bracing in the plane of the chord) will carry a portion of the dead and live load axial forces in the adjacent truss chord. Axial forces in the stringers from load-sharing chord forces and/or thermal effects may generate weak axis moments (and associated fatigue stresses) in the floorbeam elements, particularly in floorbeams with weak axis restraint at the end connections due to flange continuity plates or gusset connections for bracing elements. A three-dimensional model will quantify these secondary effects and help determine the need for expansion joints (axial releases) in the stringers and floor system.

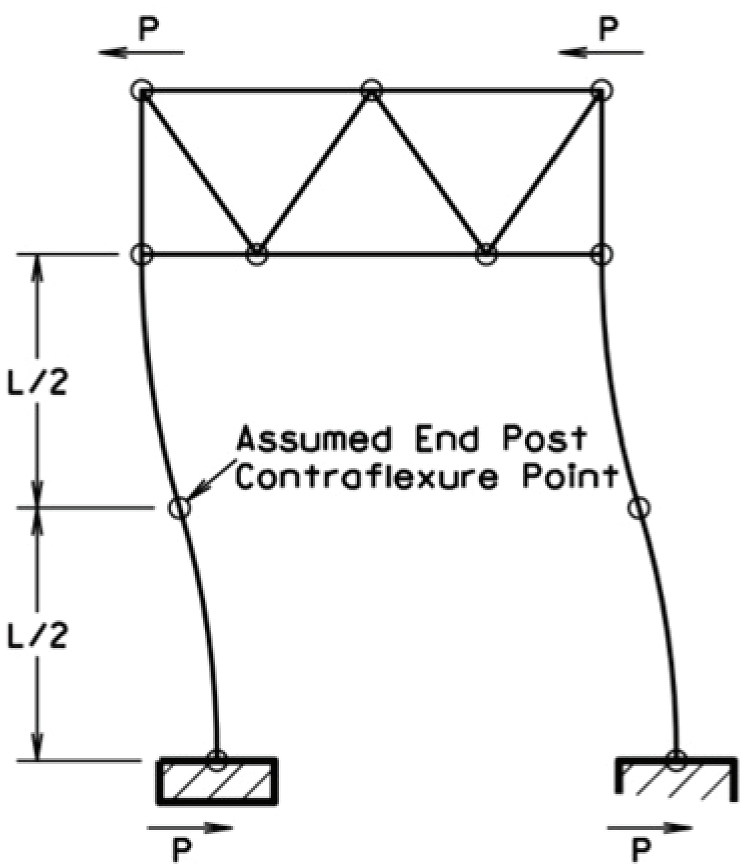
3.5.3—Analysis of Lateral Systems Using Two-Dimensional Analysis

Lateral force systems in trusses are generally composed of four components that function together to transfer lateral forces from the upper and lower chord to the truss supports: top lateral system, sway frames, portal frames, and bottom lateral system. These systems function to resist lateral loads (wind and seismic loads) and brace compression chords. The top lateral system functions as a horizontal truss distributing lateral forces acting on the upper chord and tributary area of diagonals/verticals to the end portal frames. The portal frames then transfer forces from vertical loads and upper chord lateral wind loads into the truss supports. The bottom lateral system functions as a horizontal truss distributing lateral forces acting on the lower chord and tributary area of the diagonals/verticals directly into the truss supports. And, finally, truss sway frames function to limit lateral distortion of the truss, specifically verticals and diagonals, and increase overall torsional rigidity—which increases the effectiveness of the top lateral system by minimizing overall deflections along the truss. When a concrete deck is composite with the floor system, either at the lower chord or upper chord (e.g., deck trusses), the lateral bracing system is most effective during construction prior to deck curing. After construction is complete, the concrete deck acts as a diaphragm to transfer lateral loads back to supports.

For traditional truss designs, those typically utilizing a sway frame throughout the truss, a two-dimensional analysis is an efficient method to proportion the bracing members. The top and bottom lateral systems are often initially proportioned to meet compression member slenderness ratios. Then a basic truss analysis with the lateral forces based on wind loads calculated at each panel point of the lateral system is performed to confirm or refine the member sizes. The method of sections serves as an efficient way to directly calculate the forces in the diagonal components of the lateral system. Forces in horizontal struts can then be calculated from the diagonal member forces.

A sway frame's primary purpose is to prevent distortion of the cross-section and increase torsional rigidity of the system. The frames are generally proportioned for the minimum design slenderness ratio of compression members. An approximate two-dimensional frame analysis is then appropriate using panel shears from the top lateral system. This approximate check is made to ensure that stresses are within allowable limits. There are many historical documents providing hand calculation methods for determining these forces. Alternatively, wind loads can be directly applied to a three-dimensional truss analysis to determine the forces.

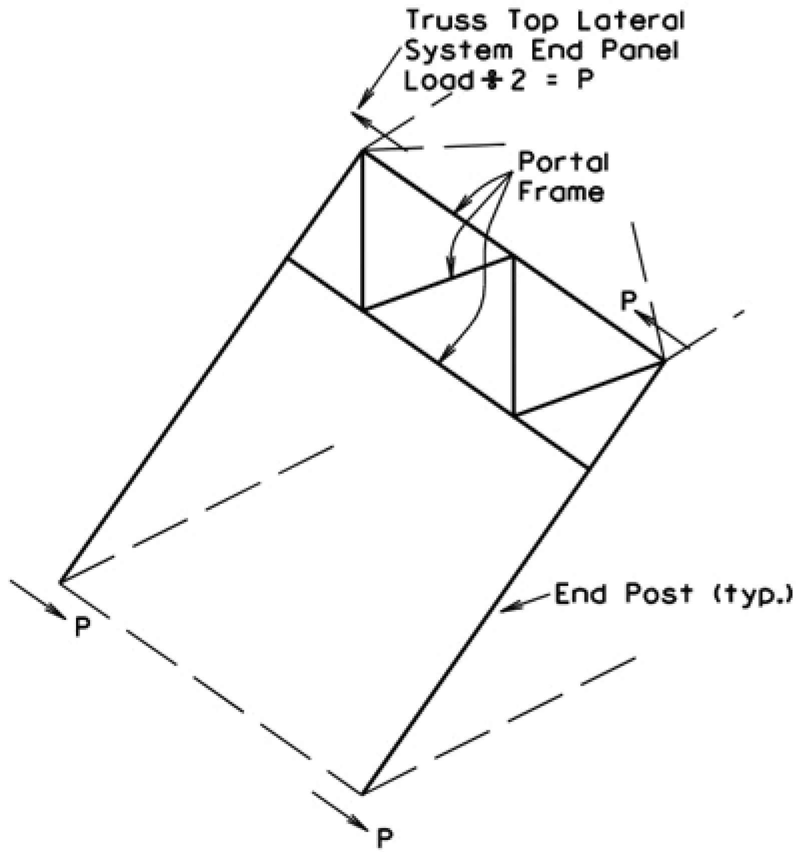
The portal frame design resolves the upper chord end panel point "reactions" from the top lateral system through the end posts to bearing reactions. A two-dimensional pin-connected analysis of the portal frame is sufficient if it is a truss system (as opposed to a Vierendeel strut). The portal is normally a statically indeterminate frame, with the end post resisting lateral loads in bending between the portal frame and supports. Due to the approximate nature of the design loadings, an approximate analysis approach is satisfactory. One approach is to assume a point of contraflexure in the end post halfway between the bottom of the portal brace and the bottom of the end post (or support). The shear on the plane is then assumed to be divided equally between the two end posts. Refer to Figure 3.5.3-1.



(a) Equivalent Model

(continued on next page)

Figure 3.5.3-1—Portal frame forces.



(b) Isometric view of the structure

Figure 3.5.3-1 (continued)—Portal frame forces.

One important consideration for the applicability of a two-dimensional analysis for lateral bracing systems is the overall lateral framing system selection. Modern highway truss designs often feature a Warren truss diagonal member configuration that eliminates intermediate sway frames to reduce the number of members and simplify gusseted connections, reducing overall structure cost and increasing aesthetic appeal by providing a more open feel and less visual clutter in the truss. Without a sway frame to prevent distortion of the cross-section through truss action, as described above, the top lateral bracing strut, truss diagonals, and floorbeam and their connections must be framed together and act as a moment frame to resist distortion from lateral loads. This approach typically warrants a three-dimensional analysis of the structure to properly design the lateral system, particularly the connections. The lateral bending moments in the portal frame necessitate the consideration of second-order moment amplification in these compression members, especially for larger truss structures and deck truss bridges. Second-order effects can be determined by a geometric nonlinear analysis or estimated using approximate methods (Refer to *AASHTO LRFD BDS* [2020], Article 4.5.3.2.2).

3.5.4—Analysis of Portal and Lateral Bracing Members within Three-Dimensional Truss Models

When three-dimensional analysis of a truss bridge is performed, the portal and lateral bracing members are incorporated directly between the structural nodes where they are connected to primary members within the real structure. Similar to the primary truss members, most portal and lateral bracing members are modeled using truss or three-dimensional beam elements. When modeling portal framing or lateral bracing, the analyst must consider the most appropriate representation of moment continuity at the member end connections. The choice of modeling the connections as "pinned" (i.e., no moment continuity) or "fixed" (i.e., full moment continuity) should reflect the nature of the structural framing, the connection details, and the expected behavior of the structure. Other considerations when making decisions on the modeling of moment continuity at member end connections include:

- The overall depth of the connection perpendicular to the axis of rotation (e.g., deeper connections may act with more fixity)
- Whether only the webs of sections or both the webs and flanges of sections are connected to the adjacent members (e.g., web-only connections tend to behave more like pinned connections, whereas connections in which both webs and flanges are connected tend to behave more like fixed connections)
- Whether the proposed framing arrangement would be expected to involve transfer of moments or not (e.g., Vierendeel portal bracing is intended and expected to exhibit moment continuity around the frame of the portal bracing, where portal bracing with diagonal members is intended and expected to exhibit truss behavior with the framing members subject primarily to axial loading only).

3.5.5—Analysis of Deck within Three-Dimensional Models

Deciding to model the bridge in three dimensions necessitates that the Engineer determine how to model the deck. The most common case is of a deck consisting of a reinforced concrete slab supported by a floor system (e.g., stringers and floorbeams supporting the deck). Assuming the floor system members are discretely modeled, the Engineer must first decide whether to:

Option 1: Represent the deck using section property modifications to the floor system members, as appropriate, and discretely apply loads that account for deck self-weight, superimposed dead loads, and other loads of importance, such as vehicular live loads. The section property modifications must correctly represent whether the deck is acting in a composite manner with each floor system member or not;
Option 2: Explicitly model the deck using either (a) two-dimensional plate or shell elements or (b) three-dimensional solid elements (not the most efficient or preferred option for most truss analyses) and apply appropriate loads to those elements. Consideration must be given to connecting the deck elements to the floor system elements using rigid links, direct coupling between deck and floor system nodes, or other means to represent any composite connections. as appropriate. This should include due consideration of application of the self-weight of the deck as a noncomposite load on the structural steel members prior to the deck concrete hardening (i.e., prior to the deck acting as part of the structural system and prior to activating the elements used to model the deck as part of the stiffness model) and modeling the distribution of the mass of the deck as necessary for a seismic analysis, as discussed in Article 3.5.5.3.

Additional important composite floor system modeling information is found in Article 3.5.2.3.

Other types of decks, such as open steel grating decks or corrugated steel forms filled and overlaid with an asphalt topping, will warrant similar consideration by the Engineer to determine what, if any, contribution the deck makes to the stiffness of the overall floor system and how to properly represent that in the model. In unusual situations such as this, consultation with an experienced senior bridge Engineer familiar with three-dimensional modeling is encouraged.

3.5.5.1—Considerations when Representing a Deck Using Modifications to Floor System Member Section Properties

While Option 1 in Article 3.5.5 is generally not encouraged as ramifications that result could counteract benefits that justified modeling the truss in three dimensions, an Engineer could determine that the approach has merit based on the focus of the analyses. If that is the case, the following items should be carefully considered:

a. Appropriately representing deck stiffness;

b. Appropriately distributing stationary loads, which include deck self-weight and other superimposed dead loads, to the floor system; and

c. Appropriately distributing and moving live loads across the bridge.

If Option 1 is used, adequately representing deck stiffness necessitates modifying the section properties of members provided with shear connectors to develop composite action between the deck and floor system. These modifications account for out-of-plane flexural stiffness and transformed section properties should be calculated to correctly represent member continuity, member spacings, effective deck widths, deck overhang dimensions, and, when appropriate, the age of the deck (i.e., consideration of early strength and stiffness gain of the deck concrete). The section properties of the steel and concrete elements should be transformed so that the composite section (representing the steel member and effective concrete deck width) is correctly represented in the model as a single, "homogenous," steel section.

Care should be taken to not over-stiffen the model by developing longitudinal (i.e., stringer, parallel to span) and transverse (i.e., floorbeam, orthogonal to span) transformed sections to represent the composite deck and floor system unless the floor system is truly designed and detailed that way; this is a rare situation that typically involves framing the stringers directly into the floor beam webs. If this is the case and both the stringers and the floorbeams are in physical contact with the deck and the Engineer is confident that composite action exists for both types of members (by means of shear connectors), then it may be appropriate to account for the contribution of the deck to the section properties of both the stringers and the floor beams, but use caution when developing the transformed section properties to avoid overlapping effective widths.

More typically, the floor system is designed and detailed with a "stacked" framing arrangement (e.g., the stringers are positioned above the floor beams). In such a system, only the elements in physical contact and acting compositely with the deck via shear connectors (e.g., the stringers in a typical stacked framing arrangement) should include a contribution of an effective deck width in their section properties.

Additional information describing how composite section properties are calculated and how steel bridge decks can be modeled can generally be found in steel design textbooks. Alternately, Grubb et al. (2015) provide a detailed discussion of composite behavior in steel girder bridges. When shear connectors are not present between the deck and floor system composite action should not be assumed.

If Option 1 is used, accurate representation of the effects of dead and other permanent loads (e.g., wearing surface) on the bridge requires distributing those loads to select floor system members. If both longitudinal member (e.g., stringers) and transverse members (e.g., floorbeams) are in contact with the deck, loads should be placed onto the longitudinal members in accordance with the design load path and it is recommended those members be subjected to distributed loads whose magnitudes are calculated based on center-to-center spacings and, for fascia members, include the deck overhangs. Depending on the software being used, care may need to be taken to utilize correct load distribution factors for the analyses being performed.

Choosing to use Option 1 and not explicitly modeling the deck generally necessitates using empirically based distribution factors and/or the lever rule for transversely distributing live loads to the floor system. Factors should be selected from applicable specifications (such as the empirical live load distribution factors in Section 4 of the *AASHTO LFRD BDS* [2020]) and, along with the lever rule, should conservatively address the effects of vehicular loading for various vehicle locations within design lanes. If a more refined calculation of the transverse distribution of live loads is desired, simplified models of the floor system can be created separate from the main bridge model and the results evaluated to calculate equivalent live load distribution factors. Once loads are distributed to the floor system, they can be "moved" across the truss using moving load algorithms embedded in the software or via use of several static load steps that increment each distributed axle load across the floor system at predefined intervals. For practicality of modeling and of post-processing of model results, a reasonable step size should be chosen for the incremental movement of these live load effects. Generally, an increment of one foot is reasonable for shorter spans (up to 100 feet); for longer spans, a larger increment may be appropriate. As an alternate to this "incremental moving load" analysis approach, the use of influence line or influence surface approaches can often provide a more computationally efficient solution to modeling the effects of moving live loads by providing explicit, critical load placement information. See Article 2.2.3 for further discussion of influence line and influence surface modeling approaches.

3.5.5.2—Considerations when Explicitly Modeling the Deck

When explicitly representing the deck in a three-dimensional truss bridge model, the Engineer can choose to use two-dimensional plate, two-dimensional shell, or three-dimensional solid (i.e., "brick") elements. While the use of two-dimensional plate or shell elements is preferred due to reduced computational demand coupled with acceptable accuracy, three-dimensional elements can be selected by the Engineer and, as a result, information will be provided for both two-dimensional and three-dimensional elements.

3.5.5.3—Plate/Shell Elements

As stated previously, an Engineer can represent the deck using one of two types of two-dimensional elements: plates or shells. As stated in Article 2.3, effectively formulated shell elements capture in- and out-of-plane flexural, shear, and axial stiffness. Plate elements ignore in-plane membrane effects and instead provide element and nodal effects that are used to estimate bending moments and shears with respect to the element's in-plane axes. Article 2.3 indicates that, in addition to modeling plate bending, shell elements are capable of capturing membrane effects through the element thickness and, as a result, more effectively predict additional tension or compression effects in the element under large, out-of-plane, deformations. While either type of element can be used to represent the deck, plate elements are generally sufficient to accurately capture the stiffness of a deck in a truss bridge model since bridge decks rarely experience large out-of-plane deformations.

Both types of elements should be formulated assuming loads are applied normal to their undeformed geometry (i.e., out-of-plane) and the number of degrees of freedom per element is a function of the number of nodes associated with the specific element; the Engineer typically can choose the shape (triangular or quadrilateral) and the number of nodes along element edges. Since shell elements address membrane effects, they generally have more degrees of freedom than comparable plate elements. While most modern computer processors can handle large numbers of degrees of freedom when completing a finite element analysis, Engineers should be cognizant that selecting higher DOF elements (e.g., shell in lieu of plate) or using many elements to represent the deck may provide limited benefits with respect to model accuracy in exchange for increased model complexity and analysis time. The introduction of additional degrees of freedom increases the chances that the model will produce spurious analytical results, suggesting the presence of secondary loads that are likely not present in the actual structure and complicating the debugging and validation of the bridge model. Engineers should select element nodal line locations to coincide with the locations of floor system longitudinal and transverse members and also to provide element aspect ratios less than 2:1 (in plan).

Care must be taken to correctly position deck two-dimensional elements relative to the rest of the floor system using appropriate geometric offsets to avoid excessive member "overlapping" and inaccurate representation of floor system stiffness. Techniques to effectively utilize a combination of "line" (e.g., truss, beam) and two-dimensional elements to model bridge floor systems have been extensively studied and recommended approaches are summarized elsewhere (AASHTO/NSBA Collaboration, 2019).

Since the primary purpose of explicitly modeling the deck is to provide a more refined distribution of dead and live loads to the floor system, explicitly modeling deck reinforcement is strongly discouraged in truss bridge models. Including reinforcement will have no bearing on load distribution if deck material properties are correctly defined.

The loading associated with the self-weight of the deck is typically modeled by applying the deck self-weight as line loads on structural steel elements that directly support the deck (typically either stringers or floor beams, depending on the framing system being used); this correctly captures the loading locked into the structure in the noncomposite condition. The deck elements are turned on after this load application without applying the self-weight of these elements. This is accomplished in various ways depending on the software used.

Less commonly (generally only for seismic analysis), it may be appropriate to model the self-weight of the deck by accounting for the unit weight of the deck as a material property in the plate/membrane elements used to model the stiffness of the deck. This allows for correct representation of the distribution of the mass of the deck in a dynamic analysis (as part of a seismic analysis of the bridge). In this situation, care must be taken to correctly capture the locked-in stress effects of the noncomposite loading of the deck without double-counting the loading caused by the weight of the deck when that deck self-weight (mass) is later accounted for in the self-weight of the deck plate/membrane elements themselves. Many software packages have specific methods for accomplishing this, including specifying deck line loads should be considered as inertial mass for dynamic analyses or the use of mass elements.

Any additional permanent loads acting on the deck itself (such as barrier rails, future wearing surfaces, lights, or other appurtenances) are typically applied to the top surfaces of the plate/membrane elements used to represent the deck.

Using two-dimensional (plate or shell) elements generally allows for direct application of moving loads to their top surface via selection of the software's moving load application tool or, when the tool is not available, (1) direct application of wheel loads to the element at prescribed coordinate locations across the bridge deck or, when out-of-plane elemental loads cannot be accommodated, (2) direct application of point loads to the deck nodes. Loads should be positioned in a manner that captures the critical member force effects for the member or members being examined. If direct nodal loads are used (i.e., if the loads are only applied directly to nodes), a large number of closely spaced nodal lines may be required to adequately capture the full envelope of live load internal member force effects.

For additional discussion of plate and shell elements, see FHWA's *Manual for Refined Analysis* (2019).

3.5.5.4—Solid Elements

The use of three-dimensional solid elements to model the deck of a truss bridge is inappropriate and over-complicated for most bridge design and rating exercises. The use of three-dimensional solid elements to model the deck of a truss bridge is typically only appropriate for academic research.

While two-dimensional plate and shell elements focus on plate bending, membrane effects caused by out-of-plane loads when subject to large deformations, and the utilization of numerical techniques to estimate other effects within them, three-dimensional solid elements (i.e., "brick" elements) are formulated to explicitly address all potential actions, including bending about three local Cartesian axes, axial effects along the three axes, and effects that produces out-of-plane behavior along all element surfaces. As a result, each node in the element can have up to six degrees of freedom; however, solid elements typically used in bridge design (and the associated software) only have stiffness in the three orthogonal directions of translation. While the use of brick elements to represent the deck may seem appealing to the Engineer given the ability to explicitly model all structural components, care must be taken when they are being considered as (1) degrees of freedom and, subsequently, model solution size can increase dramatically; (2) deciding to explicitly define all elements with the deck can increase the likelihood that errors will be introduced; and (3) decks are largely loaded out-of-plane and most plate and shell elements accurately estimate all effects needed to assess composite and noncomposite deck, floor system, and bridge response in the linear elastic range.

3.5.5.5—Other Considerations

When the modeled truss bridge has a composite floor system, depending on the goals of the analysis, a noncomposite and two composite (i.e., short- and long-term) cross-sections may need to be considered. If the modified floor system approach outlined in Article 3.5.5.1 is used, the Engineer must calculate composite section properties and multiple analyses must be performed to assess response. If two-dimensional or three-dimensional elements are used to explicitly represent the deck, multiple analyses also may need to be completed with material properties and, when three-dimensional elements are selected, modeling techniques potentially need to be altered to address differing levels of composite action.

When two-dimensional or three-dimensional elements are used to model the composite floor system, effective coupling, or "linking," of the deck and floor system behavior must occur to correctly represent the actual level of composite action. This is commonly accomplished by direct nodal coupling (i.e., the degrees of freedom of floor system and coincident deck nodes are the same) or via the use of rigid "links" or "offsets." Should the selected software not have the capability of automatically assigning direct coupling or offsets when directed by the Engineer, rigid links between floor system and deck centroidal axes should be created at coincident nodes located at those centroidal locations. These rigid links should be modeled using beam elements with negligible mass and effective stiffness significantly larger than other elements within the model. As noted in Articles 3.5.5 and 3.5.5.3, care should be taken to model the application of the self-weight of the wet concrete deck in a manner that both correctly captures the locked-in loading effects of applying the deck self-weight as a noncomposite load and the stiffness provided by the deck in the composite state. Additionally, if thermal loads are applied to the model, careful consideration should be given to how they are applied to the rigid links to avoid artificial restraints and inaccurate thermal forces.

As stated earlier, for floor systems whose longitudinal members continuously span supporting transverse members, composite action should not be assumed in negative bending regions where shear studs are not provided. For situations where minimal shear studs do exist between the member and deck, deck tensile stresses may need to be checked in negative bending for appropriate composite cross-sections (i.e., short- or long-term). Similar checks may need to occur in the elements in areas of deck positive bending between floor system members, such as in the middle of a deck panel, although the likelihood that excessive tensile strains will be encountered is generally quite small. If transformed sections are used to represent the deck, the Engineer should take care to calculate stresses using the correct element material, geometry, or both.

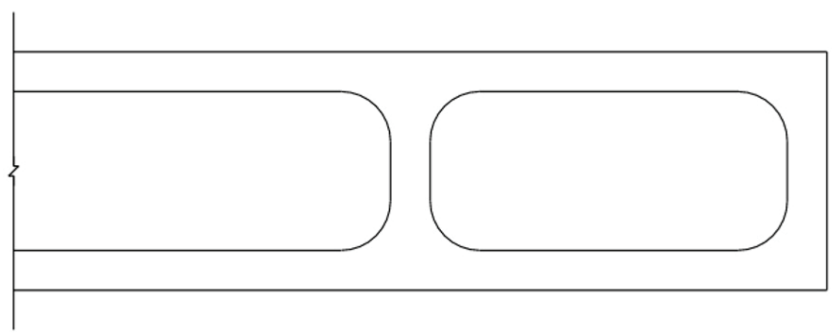
Should dynamic analyses need to be performed, it is important to accurately locate the center of mass within elements used to represent the deck using either the modified floor system discussed in Article 3.5.5.1 or the explicitly modeled deck from Article 3.5.5.2. Transformed sections used to represent the floor system in Article 3.5.5.1 must have their centers of mass located by the Engineer, while the software will be able to locate them should plate/shell or solid elements be used to represent the deck. Regardless, care must be taken to correctly assign member densities/unit weights.

When construction or rehabilitation activities necessitate tracking truss behavior at various points in time the Engineer must carefully consider the effects of deck stiffness and location for composite systems. Situations where this type of modeling approach could be considered are infrequent. Examples would largely be limited to considerations of deck pour sequencing for appreciably skewed trusses or for partial width deck replacement projects. If response during a deck pour needs to be evaluated, either for full- or partial-width construction, the model may need to differentiate between "fresh" concrete locations and locations where the concrete has begun to set and appreciable gains in strength and stiffness are anticipated. Resulting models would need to have zones that contain noncomposite floor system sections and potentially sections experiencing varying levels of composite action.

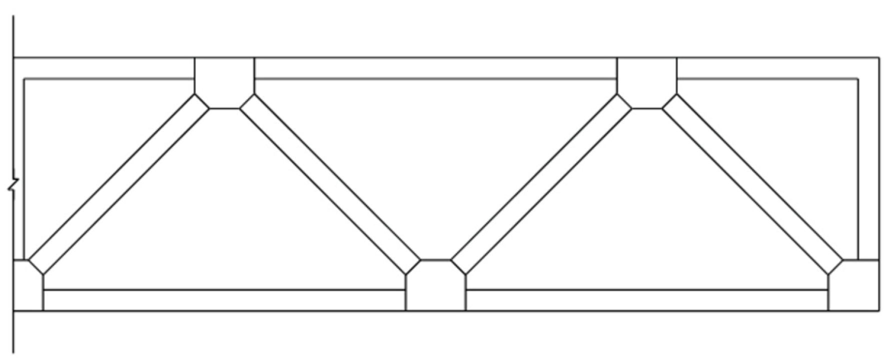
Refer to Article 4.1.2 for a discussion on when to include camber in the analysis model.

3.5.6—Analysis of Vierendeel Trusses

Instead of treating the truss members as pin-ended members capable only of carrying axial load (the typical simplifying assumption for truss analysis) as is done in a traditional truss, in a Vierendeel truss the members are treated as beam members and moments are carried around the entire frame, including through the joints where the members are connected. See Figure 3.5.6-1.



(a)



(b)

Figure 3.5.6-1—Vierendeel (a) vs. Warren (b) planar truss.

Although considered structurally inefficient by many Engineers, Vierendeel trusses have been used in the past and are still used today. Vierendeel trusses sometimes appear in building and industrial construction, where the open web is attractive for allowing the passage of utilities or the positioning of windows, etc. In these applications they sometimes are used as deep beams or "transfer" beams. Vierendeel trusses have been used as the primary load-carrying members in bridges, although only rarely, not recently, and more commonly in Europe. They mostly appear in modern bridges as part of the lateral bracing system for arches (more commonly) and for truss bridges (rarely). In these applications, Vierendeel trusses are valued for their clean, uncluttered appearance.

A Vierendeel truss is a highly indeterminate structure, as are many rigid frame structures. Analysis of a Vierendeel truss can be approached in several different ways, including:

- finite element modeling;
- classical methods of analyzing indeterminate frames; and
- making significant simplifying assumptions, the most common being assuming hinges at the mid-height point of the vertical members and sometimes even at mid-bay points of the horizontal members. No pin is assumed at mid-height of the end posts, since assuming such would result in a collapse mechanism.

Manuscripts addressing the analysis and design of Vierendeel trusses are rare, and often quite old (dating back to times when analysis by classical methods was more common). Pearson (1959) discussed several methods of analysis of Vierendeel trusses, including the method of consistent deformations, the method of least work (Castigliano's second theorem), the slope-deflection method, and the moment distribution method.

When analyzing a Vierendeel truss, the truss members should be modeled as beam members (able to carry axial force as well as moments and shears about both axes) and the connections should be modeled as full moment connections. As mentioned above, a Vierendeel truss is actually a rigid frame, and as such should be modeled so that moments can be carried through the full length of all members and their connections. The connections should be designed to carry the end moments of the connected members, similar to the connections in a rigid frame.

3.5.7—Modeling Substructures

Substructures are essential elements in bridge structures because they support the superstructure and transmit the loads to the foundation. In general, bridge piers have different configurations, shapes, and sizes. Bridge piers can take a number of forms, including hammerhead, multicolumn bent, pile bent, solid wall, or single column. Abutments (or end bents) have a similar variety of configurations. 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. These cases are discussed in detail with respect to steel girder bridges in AASHTO/NSBA G13.1, *Guidelines for Steel Girder Bridge Analysis*, Article 3.14.3. The information presented in G13.1 is applicable to steel truss bridges as well, and the Engineer should consult G13.1 for guidance.

3.5.8—Gusset Plate Analysis

In addition to analyzing the truss members to determine the forces due to various loads, an analysis of the gusset plates may be required. Refer to the latest provisions for truss gusset plate design in the *AASHTO LRFD BDS* (2024). Additional information on the development and implementation of these provisions can be found in the supporting research document by Ocel (2013).

However, it is generally neither necessary nor recommended that the gusset plates themselves be explicitly represented (using plate or shell elements) in a truss analysis model; that level of modeling detail is generally only warranted for academic research. The influence of the truss gusset plates on the overall stiffness of the model and on load distribution within the structure is not significant and capturing the precise stress distribution within the gusset plates using a highly refined model is unwarranted and inconsistent with the truss gusset plate design provisions in the *AASHTO LRFD BDS* (2024).

3.5.9—Guidance for Analysis with Tension-Only Members

In truss bridge terminology, a "counter" is a diagonal member of a truss, typically proportioned to be very slender, such that it would be expected to elastically buckle when subject to compression under dead load (i.e., such that it would behave as a "tension-only" member). These members often have turnbuckles to allow them to be tightened during or after construction. They are used in pin-connected trusses to account for the possibility of live load completely unloading the dead load tension of the opposing diagonal in the same bay. In this circumstance, without the counter the tension diagonal would be subjected to compression. In a pin-connected truss, the members are typically tension-only (eyebars) and have little to no compression capacity. If the live load is substantial enough to completely unload the dead load tension of one of these diagonals, a counter is used to carry the remaining portion of the load in this bay. This change in load path essentially makes the behavior nonlinear, which complicates the analysis. Refer to Figure 3.5.9-1.

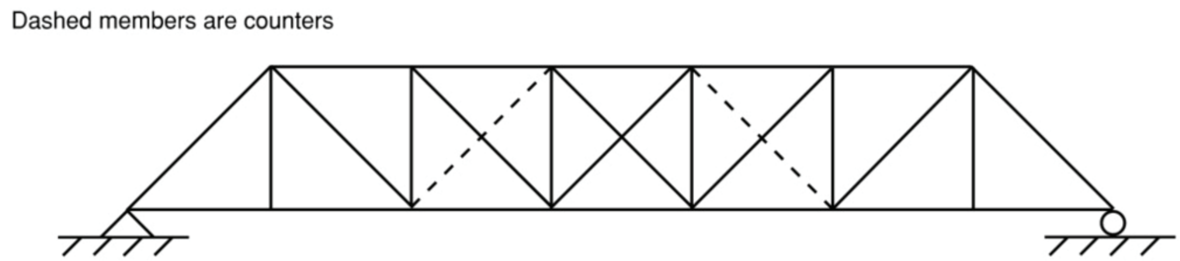


Figure 3.5.9-1—Pratt truss with counters depicted.

The tension in counters is dependent on construction sequencing and the amount of initial preloading, if any. This makes approximating the dead load in a counter very difficult as this construction information is most likely unknown. The design intent is that these members are only loaded by live load. After dead load, they are tightened just to the point of removing slack, but not yet resisting dead load. This essentially creates a redundant load path for distortion of this truss bay. The same loading that would put tension in the counter would reduce the locked-in tension of the opposite diagonal.

Assuming no information is available about the in-situ tension of the counters, the analysis guidance for this type of bridge is to exclude the counter elements from the dead and live load analysis. Then, when evaluating the opposite diagonal, if its dead load tension is totally unloaded by live load, the balance of the live load will be carried by the counter. This type of load redistribution requires the tension diagonal to have a negligible compression stiffness, which is typically the case for eyebars. Essentially, this analysis assumes the eyebars experience some elastic (Euler) buckling, causing additional load to be carried by the counter. It is important to consider the minimum dead load factors for this evaluation since the dead and live loads are of opposite sign.

Tension-only members, such as counters, can be directly modeled in many three-dimensional finite element method analysis programs. However, it is important that the Engineer fully understand the specific features of the members and how they work in a given analysis program, and that they are properly used and the model is correctly run to allow the tension-only members to function properly and give correct results. For example, it is important to understand that finite element method programs recognize such "tension-only" members only if a nonlinear analysis is performed. The program will run an initial analysis assuming all members are fully effective and identify which tension-only elements are in compression. Then the program will run a second analysis with those members inactivated and identify any additional tension-only members are now in compression in the second iteration of the analysis. Then the program will run a third iteration, and so on until convergence is achieved and no additional tension-only members are found to be in compression. There may be other software-specific features or nuances associated with the use of tension-only members in a given three-dimensional finite element method program.

3.5.10—Effects of Connection Modeling and Behavior on Global Modeling and Behavior

Truss members of older bridges may be connected with pins at their ends. More commonly, gusset plate connections which include some flexural rigidity are utilized to connect truss members. The flexural rigidity of gusset plates will induce bending moments in truss members when the bridge deforms under load. The resulting first-order bending moments in truss members are referred to as "secondary moments." Secondary moments are different than primary moments, which are first-order moments in truss members that are required to maintain equilibrium. Primary moments typically develop when loads are applied to truss members away from truss joints, making it statically impossible for the truss member to carry load through axial force alone.

When truss members are connected with gusset plates, the Engineer should consider whether the structural analysis should be performed using elements with pinned or fixed-end boundary conditions at the joints. If pinned-end boundary conditions are used secondary moments will be neglected in the analysis, but secondary moments will be recognized in the analysis if fixed-end boundary conditions are used.

For over 100 years, Engineers have debated the importance and significance of secondary moments in bridge trusses. Grimm in 1908 posited that analysis of secondary moments in bridge trusses was necessary to ensure an adequate factor of safety, especially in older railroad trusses that were subject to heavier loadings than those for which they were originally designed. Later, Parcel (1934), Bleich (1952), Bolton (1955), and others concluded that in most cases, secondary moments had little effect on the ultimate strength of truss bridges. They reasoned that under ultimate loads, as the main members and connections approach their ultimate strength, yielding and plastic flow begins that releases the rotational fixity from the connections. As the joints lose rigidity, secondary moments dissipate, and as long as the truss members can support axial loads, the truss remains in equilibrium because a load path exists that satisfies equilibrium. This reasoning provides a justification for analyzing truss bridges using pin-connected members, even when the actual member connections consist of gusset plates with some flexural rigidity.

The Guide to Stability Design Criteria for Metal Structures provides a succinct discussion of why secondary moments at truss member ends can typically be neglected at the strength limit state:

> "In triangulated frameworks (trusses), the loads are usually applied at the joints. … Deflections of the joints and the truss as a whole are caused by the axial deformations of the members. The angles between members meeting at a joint also change because of these deformations. If the members are connected at the joints by welds or bolts, the angle changes produce secondary bending stresses. These have little effect on the buckling strength (and tensile strength) of truss members. Because of local yielding of extreme fibers of the members near the joints, the secondary moments gradually dissipate as the truss is loaded to its ultimate strength. They can therefore be neglected in the buckling analysis" (Ziemian, 2010).

Nair provides a paper on truss secondary moment analysis that discusses the appropriateness of using pinned and fixed-end conditions for truss member analysis. For typical cases where service-level secondary stresses are below 4,000 psi, Nair recommends that secondary stresses (the stresses caused by secondary moments) can be neglected, and a pin-connected member analysis can be utilized. However, "in trusses with very large gusset plates or unusually stubby members, flexural stresses might be much higher than the recommended [4,000 psi] secondary stress limit and should be checked by analysis" (Nair, 1988). Nair further notes that "if flexural stresses are found to be excessive, the 'truss' should be regarded as a 'frame' and the members should be designed for axial force, flexure, and shear" (1988). This approach is intended to prevent "local buckling, connection distress, or other possible problems" that could be caused by high secondary moments.

Nair's 400-psi limit on secondary stresses was ultimately incorporated in the AASHTO *Standard Specifications for Highway Bridges* (2002) in Article 10.16.3 for the consideration of secondary moments within tension members. A modified limit of 3,000 psi is provided for compression members in the same Article. Per the AASHTO *Standard Specifications for Highway Bridges*, stresses in excess of these limits should be treated as a primary stress. In the same section, the AASHTO *Standard Specifications for Highway Bridges* (2002) also state that "secondary stresses due to truss distortion or floorbeam deflection usually need not be considered in any member, the width of which, measured parallel to the plane of distortion, is less than one-tenth of its length" (2002). The current *AASHTO LRFD BDS* (2024) recommendations for secondary moment analysis in truss members follow an approach similar to the AASHTO *Standard Specifications for Highway Bridges* (2002). As noted in the *AASHTO LRFD BDS* (2024), Article 6.14.2.3, "secondary stresses due to truss distortion or floorbeam deflection need not be considered in any member whose width measured parallel to the plane of distortion is less than one-tenth of its length."

In most trusses, these secondary stresses due to distortion of the truss are small and can be safely ignored in design or rating calculations. However, truss members with large depth-to-length ratios can potentially develop significant secondary stresses which should be considered. As such, it may be beneficial to include this behavior in analysis models by providing fixed-end connections at each end of the truss members within the analysis. These distortional stresses can then be checked and included or neglected based on their magnitude. When the distortional stresses are indeed small enough to be neglected, there will be an insignificant difference in the axial forces within the truss members between an analysis where the truss members have pinned ends and an analysis where the same members have fixed ends. Secondary stresses due to member eccentricity and self-weight, which *are* required to maintain force equilibrium, should be included in both the two-dimensional and three-dimensional models.

Ultimately, the recommended approach for the analysis of truss members is to follow the guidance of the *AASHTO LRFD BDS* (2024) or the AASHTO *Standard Specifications for Highway Bridges* (2002), depending on which specifications are being utilized for the project. For projects which use the *AASHTO LRFD BDS* (2024), members meeting the depth-to-length requirement may be modeled with pin-ended connections, therefore safely neglecting secondary moments. For projects using the AASHTO *Standard Specifications for Highway Bridges* (2002), the secondary stresses should be calculated and checked against the secondary stress limits, so an initial analysis with fixed-end members may be required. For these *Standard Specification* projects, the limiting depth-to-length ratio may be used as an initial screening tool for determining whether or not the secondary stresses will need to be considered; however, the language of the AASHTO *Standard Specifications for Highway Bridges* (2002) indicates that the secondary stresses must ultimately be checked against the provided numerical limits. Primary moments due to loads applied to truss members away from joints, member eccentricity, and self-weight, which *are* required to maintain force equilibrium, should be included. Primary moments due to member eccentricity and self-weight are typically more significant for longer-span trusses.

For cases where secondary moments are included by using fixed-end conditions for truss members in the analysis model, consideration should be given to also including the forces induced by the truss fabrication procedure, if any. Refer to Articles 3.6.1.7 and 4.1. Beginning in 1917 with the construction of the Sciotoville Bridge over the Ohio River, some trusses have been constructed with forced alignment of truss connections to induce stresses opposite in sense to truss secondary moments (Griggs, 2007).

For analysis of fatigue or service-level loading, secondary moments may produce undesirable structural performance. Secondary stresses from transient loads that develop at the ends of members due to connection restraint have the potential to cause fatigue problems, even if they do not reduce the ultimate strength of the structure. Therefore, secondary moments may need to be considered for these types of analyses even in cases where they are not considered for strength analyses.

In cases where assessment is being conducted for an existing bridge, considerations should be given to the field conditions of the gusset plate connection. Relatively thin gusset plates, deteriorated gusset plates, or both might experience large levels of deformation or local buckling when the connected members are excessively loaded, causing large local deformations in the gusset plate. These large deformations could result in large out-of-plane deformations of the gusset plate, which could compromise the ultimate strength of the connected members. In such cases, it is recommended that three-dimensional models are developed for proper assessment of local as well as global demand.

3.5.11—Submodeling of Connections

Detailed modeling of connections or other details in truss bridges is rarely needed, and is not warranted for typical bridge design, load rating, or rehabilitation projects. Situations in which detailed modeling (typically using a fine mesh of shell elements) can be beneficial include research studies, detailed forensic investigations, analysis of unusual structures or details, analysis for emergency repair of fractured or damaged structures, analysis of connections with localized deterioration or distortion, and internal redundancy studies.

If detailed modeling is warranted, the Engineer should begin developing the model with a clear understanding of the intended goals and extent of the modeling. Some general considerations are noted below:

The development time (including debugging) and run-time of detailed modeling can be substantially longer than models in which typical truss and beam elements are used. Carefully consider the development and run-time requirements before embarking on a detailed modeling analysis.

Using the correct boundary conditions and applying correct loadings at the boundaries of the detailed modeling is crucial for achieving accurate results. It may be beneficial to integrate the detailed modeling into a larger existing model of the structure to ensure loadings and displacements at the boundaries of the detailed modeling are accurate.

Consider whether material or geometric nonlinear analysis is needed in the detailed modeling investigation. Investigations of structures in which a fracture has occurred (such as forensic, emergency repair, or internal redundancy analyses) may require material nonlinear analysis to accurately model the post-fracture behavior of the structure. Investigations of connections with localized deterioration and distortion may require material nonlinear analysis to capture the effects of the deterioration and geometric nonlinear analysis to include the impacts of out-of-plane distortion of connection plates in compression.

The design team should work with the bridge Owner to develop acceptance criteria, ideally before embarking on the detailed modeling. It is not unusual for detailed modeling to show that some yielding has occurred in the regions of interest, and acceptance criteria based on maximum levels of plastic strain may be warranted.

One example of detailed modeling for an internal redundancy study is the analysis that was undertaken for the Winona Bridge rehabilitation project. The Winona Bridge includes a historic cantilever through-truss bridge with a main span of 450 feet that spans over the Mississippi River in Winona, Minnesota, and was opened to traffic in 1942. The rehabilitation design criteria included that all tension members would be retrofit to be internally redundant. Retrofits were only needed if analysis demonstrated that the structure's built-up tension members did not already satisfy internal redundancy criteria. For tension members that did require retrofit, one retrofit strategy that was utilized was the addition of high-strength steel bars inside the existing built-up box sections.

The project's Engineers used detailed modeling to verify that the anchorages of the high-strength bars at the top and bottom faces of the truss chords and the existing gusset plate connection plates and rivets would not become distressed due to the fracture of one plate in an existing built-up box section. Selected anchorage plates, gusset plates, and built-up tension members were modeled with a relatively fine mesh of shell elements and integrated into the existing three-dimensional analysis model of the through-truss spans. Rivets in the gusset plate connections were modeled with multilinear link elements to model their force versus displacement behavior. The detailed modeling was connected to adjacent truss elements of the three-dimensional model using rigid links, and these connections were made away from the area of interest for the internal redundancy investigation. Through use of detailed modeling, the project's Engineers were able to verify that the high-strength bar and existing gusset plate connections would perform as intended if a fracture were to occur in an existing tension member retrofit with high-strength bars. Figure 3.5.11-1 shows an image of a portion of the detailed modeling for the Winona Bridge.

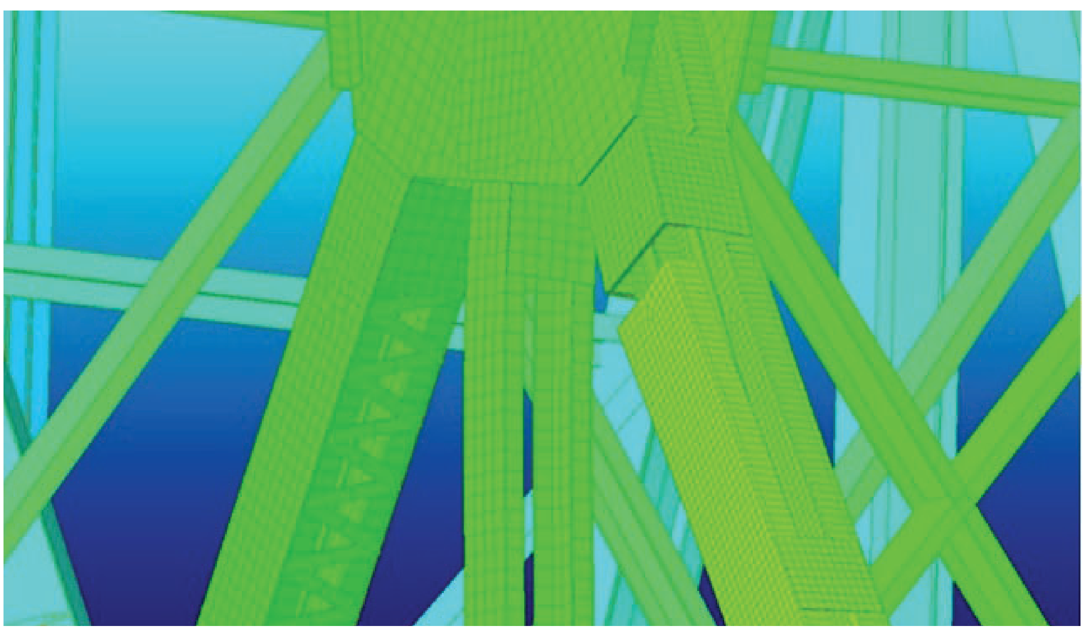


Figure 3.5.11-1—Detailed modeling of a region of the Winona Bridge for internal redundancy.

3.6—LOADS ON THE PERMANENT STRUCTURE

Numerous loads are applied during the analysis of steel truss bridges including permanent, variable, or transient loads. Depending on the configuration of the truss, some loads are applied to the noncomposite section while others are applied after the deck is hardened and acting in a composite manner with the deck framing system. The most common truss superstructure loads are listed in Table 3.6-1.

Table 3.6-1—Summary of Common Loads Acting on Trusses

Load	***AASHTO LRFD BDS*** **Load Category Abbreviation**	**Permanent or Transient?**	**Applied to Noncomposite or Composite Structure? (See Note 1)**	**Primary Direction of Action? (See Note 2)**
Dead Load—Self-Weight of Truss Members	*DC*	Permanent	Noncomposite	Vertical
Dead Load—Weight of Deck Forming System	*DC*	Permanent (stay-in-place forms) or Transient (removable forms)	Noncomposite	Vertical
Dead Load—Weight of Deck	*DC*	Permanent	Noncomposite	Vertical
Dead Load—Barriers, Sidewalks, and Fencing	*DC*	Permanent	Composite	Vertical
Dead Load—Future Wearing Surface	*DW*	Permanent (variable)	Composite	Vertical
Dead Load—Utilities and Other Appurtenances	*DW*	Permanent	Composite (sometimes noncomposite)	Vertical
Construction Loads	See Note 3	Transient	Noncomposite and Composite	Horizontal and Vertical
Live Load	*LL*	Transient	Composite	Vertical
Dynamic Load Allowance (Impact)	*IM*	Transient	Composite	Vertical
Centrifugal Force	*CE* (See Note 4)	Transient	Composite	Horizontal and Vertical
Braking	*BR*	Transient	Composite	Horizontal and Vertical
Wind on Superstructure and Substructure	*WS*	Transient	During Construction: Noncomposite Final Condition: Noncomposite or Composite	Horizontal and Vertical
Wind on Live Load	*WL*	Transient	Composite	Horizontal and Vertical

(continued on next page)

Table 3.6-1 (continued)—Summary of Common Loads Acting on Trusses

Load	***AASHTO LRFD BDS*** **Load Category Abbreviation**	**Permanent or Transient?**	**Applied to Noncomposite or Composite Structure? (See Note 1)**	**Primary Direction of Action? (See Note 2)**
Uniform Thermal Contraction or Expansion	*TU*	Transient	During Construction: Noncomposite Final Condition: Noncomposite or Composite	N/A
Thermal Gradient	*TG*	Transient	During Construction: Noncomposite Final Condition: Noncomposite or Composite	N/A

Note 1: The application of load differs, depending on whether the model is two-dimensional (without floor system stiffness) or three-dimensional (with floor system stiffness). If the model is two-dimensional, all loads are typically converted to panel point loads on a noncomposite model. If the model is three-dimensional, the model can be noncomposite or composite (depending on whether or not shear connectors are present), and many loads are applied directly to the modeled floor system.

Note 2: The application of horizontal loads is only applicable to three-dimensional truss models. For two-dimensional models, their effects on the truss must be determined either through hand calculations or two-dimensional models of the lateral bracing systems in the horizontal plane.

Note 3: See the *AASHTO LRFD BDS* (2024), Section 3, for more discussion of current load factors and other considerations for treatment of construction loads.

Note 4: Centrifugal Forces, *CE*, are specific to curved alignments for which truss bridges are not commonly utilized.

3.6.1—Dead Loads

The following dead load descriptions describe the application of dead loads on a three-dimensional model that considers composite action with the deck. For a two-dimensional truss model, the following loads are determined, based on tributary areas, and applied as point loads at panel (or hanger) points. Article 4.6.2.4 of the *AASHTO LRFD BDS* (2024) discusses the distribution of loads to planar frame (two-dimensional) and space frame (three-dimensional) analyses.

3.6.1.1—Weight of Structural Steel

The self-weight of structural steel should be applied to the noncomposite deck framing system (floorbeams, stringers, etc.) as well as the primary truss elements. Include a weight percentage (10–15 is typically used) increase for details not explicitly modeled (gussets, stiffeners, bolts, rivets, etc.). For basic truss 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.

3.6.1.2—Weight of Deck Forming System

In most cases it is appropriate to assume that the weight of the deck forming system is applied to the completed, noncomposite, structural steel deck framing system and can be applied as a simple, uniformly distributed line load on each floorbeam or stringer. The type of forming system (permanent, stay-in-place forms versus removable forms) will determine whether the loading needs to be considered as a permanent load or only as a temporary condition. When permanent forming is used, typically the effects of its weight are approximated as a uniformly distributed line load or are indirectly considered as an approximate percentage of the weight of the deck. Many Owner agencies have prescribed design criteria for this calculation.

In cases where the longitudinal edge of the deck is not supported directly over a stringer, truss chord, or tie member, the twisting effect of overhang formwork should be analyzed locally on the exterior support member (typically a longitudinal stringer) according to Article C6.10.3.4 of the *AASHTO LRFD BDS* (2024). Additional discussion related to this topic can be found in Chapter 11 of the NSBA *Steel Bridge Design Handbook* (Wright and Grubb, 2022). In the absence of Owner agency construction loading criteria, Article 2.3.3 of the AASHTO *Guide Design Specifications for Bridge Temporary Works* (2017a) should be followed.

3.6.1.3—Weight of Deck

The application of concrete to a truss bridge is typically applied as a simple, uniformly distributed line load on each floorbeam or stringer. When initially placed on the deck framing system, the wet concrete offers no structural capacity or stiffness to the system and represents nothing more than a gravity load. Concrete deck slabs are often made composite with the underlying structural framing by use of headed shear studs on either the stringers, floorbeams or both. The effect of placing the deck in stages needs to be considered on a stage-by-stage basis. See Article 4.1.3 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.

3.6.1.4—Barriers, Sidewalks, and Fencing

The weight of barrier rails, median barriers, sidewalks, and fencing typically represent simple dead loads applied to the deck framing system. If the deck system is composite, these loads are to be applied to the long-term composite section of the stringer or floorbeam. Article 4.6.2.2.1 of the *AASHTO LRFD BDS* (2024) provides 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 three-dimensional analysis, the analysis model may be used to determine the distribution of these loads based on the stiffness of the modeled structural deck framing elements by placing the barrier load at its actual location. 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.

3.6.1.5—Future Wearing Surface

The weight of any provisional future wearing surfaces represent simple dead loads applied to the deck framing system. If the deck system is composite, the future wearing surface weight should be applied to the long-term composite section of the stringer or floorbeam. Most Owner agencies specify an allowance for a future wearing surface for new bridges. These are typically based on an assumption for a net increase in deck slab thickness due to a future overlay and may vary from 20 psf to 50 psf depending on the Owner agency. Unless required for unique geometries, the weight of the future wearing surface is typically assumed to be distributed equally to the stringer–floorbeam system.

3.6.1.6—Utilities

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 resulting loads should be considered in the analysis of the bridge. In extreme cases, these attachments can have dramatic effects on steel deck framing systems or even the supporting truss members, but in most cases the effects are minor. Engineers are encouraged to consider 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, these loads can be distributed similarly to that of barriers and sidewalks in the analysis and will be applied in the composite or noncomposite state, depending on the time of the utility's attachment during construction.

3.6.1.7—Locked-In Forces

If the construction staging of the truss erection (and/or floor system) produces locked-in force effects such as changing a pinned connection to a fixed connection during the erection sequence, these locked-in forces must be determined through the analysis, most likely by construction staging functions in the modeling software. Refer to Article 4.1 for additional information.

3.6.1.8—Creep and Shrinkage

Traditional truss structures are generally not analyzed for creep and shrinkage. However, for post-tensioned concrete decks (which are not commonly used in trusses), creep and shrinkage should be considered, per Article 5.4.2.3 of the *AASHTO LRFD BDS* (2024). The application of these loads would necessitate a three-dimensional analysis. Post-tensioning is applied by modeling the elements explicitly or applying equivalent external loads to the deck. Some software packages will calculate and apply creep and shrinkage forces internally given specified parameters; otherwise, equivalent strains may be applied to the deck elements.

3.6.2—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 if the deck slab is poured composite with the underlying floorbeam/stringer system. Live loads are transient loads which need to be applied in various patterns moving both longitudinally and transversely over the bridge. Article 3.6.1.2.1 of the *AASHTO LRFD BDS* (2024) prescribes a notional HL-93 live load that includes both a lane load component (to be applied in patterns over the bridge to determine the most critical loading conditions) and vehicular load (point load) component. Additionally, owners may require the analysis of various legal and permit loads for design and/or ratings.

3.6.2.1—Live Load Distribution

In a preliminary or simplified analysis approach, it is commonplace to apply live loads, or influence line, directly to the truss lower chord member (assuming the roadway is supported by a floor system located at the lower chord) in a two-dimensional analysis that does not include the deck framing system. In this case, a simplified transverse live load distribution should be considered to determine the maximum number of truck or lanes that can influence a single truss. It should be noted that moments and shears in lower chord members that are not pin-connected will not be accurate based on this analysis (i.e., the live load in the actual structure is not directly applied to the lower chord, only at joints). Article 4.6.2.4 of the *AASHTO LRFD BDS* (2024) specifies that the lever rule be used to distribute gravity loads to the truss when using a planar (two-dimensional) analysis. In cases where a refined (three-dimensional) analysis is desired, the loading influence surface can be applied directly to the deck framing system and distributed by the model with consideration of the section and material properties.

Software packages may offer influence line and surface analyses which automatically determine the critical live load locations; otherwise, multiple load cases placing the vehicle at discrete locations along the structure may be used. Using influence lines or surfaces with structures that contain tension-only elements (e.g., counterforts or eye bars) should be done with caution. Structures with tension-only elements are nonlinear, while influence analyses are generally linear. If a linear analysis is performed, it should be verified that the compression forces due to live load do not "unload" the tensile forces in tension-only members due to the permanent loads. If the tension-only members are unloaded, the results of the linear analysis are invalid and alternative loading methods should be considered and employed. Refer to Article 3.5.9.

Similarly, when distributing live load to the truss, the Engineer may elect to use either a simplified or refined analysis to distribute live loads to the floorbeam/stringer system. The *AASHTO LRFD BDS* (2024) do not provide explicit guidance on how to distribute these loads when performing a simplified analysis so judgment should be applied by the Engineer when considering the configuration of the system they are analyzing.

Generally speaking, stringers transmit loads to the floorbeams which carry the load to the panel points of the trusses. In this scenario, the stringers behave similarly to multi-beam bridges, and it may be appropriate to distribute the live loads using either the lever rule or the empirical distribution factors outlined in Article 4.6.2.2 of the *AASHTO LRFD BDS* (2024). Distribution to the floorbeams may be considered by simply applying the maximum live load end reactions from the stringers or by using the lever rule longitudinally to distribute the live loads directly to the transverse floorbeams.

In cases where a refined analysis is desired, the loading can be applied directly to the deck framing system and distributed by the model with consideration of the section and material properties. Generally, stringers are either stacked (continuous over top of floorbeams) or framed (at the same level as and bolted to floorbeams). In a refined analysis, the difference in these systems can have an effect on the resulting forces.

3.6.2.2—Multiple Presence Factor

Multiple presence factors represent modifications to the live loads to reflect the probability that multiple lanes will be fully loaded simultaneously. Engineers should be aware that these factors need not be applied when using empirical live load distribution factors which already include the multiple presence factor. Conversely, multiple presence factors must be added when distributing live loads using the lever rule or when applying live loads directly for refined analyses.

3.6.2.3—Dynamic Load Allowance

Moving vehicles produce a larger effect on bridges than a static load of equal magnitude. Factors for “dynamic load allowance” specified in Article 3.6.2.1 of the *AASHTO LRFD BDS* (2024) or “impact factor” specified in AASHTO *Standard Specifications for Highway Bridges* (2002) are used to approximate the effect of moving vehicle induced dynamic loading. However, the *AASHTO LRFD BDS* (2024) allow that the dynamic load allowances can be reduced if justified by sufficient evidence based on a dynamic analysis of vehicle and bridge interaction and/or test results. The AASHTO LRFD dynamic load allowance factor (33 percent) is applicable to all truss superstructure elements except the deck joints, which have a higher factor. In a linear static analysis, the dynamic load allowance can be applied to the wheel loads in the model or to the resulting forces from the unfactored live loads.

3.6.2.4—Fatigue Loads

Similar to design live loads, fatigue loads are applied to truss structures for new designs or fatigue evaluations of existing structures. Article 3.6.1.4 of the *AASHTO LRFD BDS* (2024) defines this load. For fatigue, only one lane of traffic is applied.

Multiple presence factors are not applied in the fatigue limit state. In cases where the approximate, empirical single-lane distribution factors are used for the design of stringers per Article 4.6.2.2 of the *AASHTO LRFD BDS* (other than the lever rule), the force effects need to be divided by the single-lane multiple presence factor of 1.20.

3.6.2.5—Standard Specification Design Live Load

Based on Owner policies and software requirements, existing truss analysis may need to be performed using loading from the AASHTO *Standard Specifications for Highway Bridges* (2002). The AASHTO *Standard Specifications for Highway Bridges* (2002) define the HS-20 design load as a truck load and a concentrated load plus uniform lane load. What is not discussed in these Specifications is how the concentrated load provisions apply in conditions that are not purely flexural—for example, on a truss bridge. Implementing these concentrated and distributed loads on a truss bridge requires some interpretation of the Specifications, discussed in the following Articles.

3.6.2.5.1—Concentrated Load

The specified concentrated load is applied as a concentrated 18-kip load for moment and 26 kips for shear. This provision is easily interpretable for girder bridges. In truss bridges however, this specification requires some engineering judgment. For axial and combined-action members of a truss, some determination needs to be made about whether the member performs a more flexure-like or shear-like action in the global behavior of the truss. The general guidance here is that upper and lower chord members can be considered as flexural members and should be evaluated using the moment magnitude of concentrated load and the diagonals and verticals can be considered as shear members and should be evaluated using the shear magnitude of concentrated load. This also applies for the requirement of an additional concentrated load in an adjacent span of continuous span bridges to maximize negative moment effects. For chord members near interior supports, the axial force demand envelope of the members can be used to determine if the member is ever a part of a negative moment type behavior (compression in lower chord or tension in the upper chord). If so, a second concentrated load in an adjacent span needs to be considered.

3.6.2.5.2—Distributed Load

The distributed load represents a series of closely spaced 15- and 2-ton trucks dating back to the 1935 AASHO *Specifications for Highway Bridges*. The intent of this loading is to maximize the load effect on members in longer-span bridges. In this sense the distributed load is considered notional; it should only be applied where its effects are additive to the action being evaluated. That is, it cannot be used to reduce the magnitude of the total member demand. For a truss, there is an important consideration, particularly for diagonals. Maximizing the load effect in truss diagonals most likely requires patterning of the uniformly distributed load. An influence line (or influence surface) solver can identify which regions contribute to which actions and produce the worst-case loading. Any other solver would require the user to specifically designate the loading patterns required to produce the worst case for a given member and action. It is recommended to use an influence surface approach to at least identify the patterning for maximizing a specific output quantity, which then may be implemented in another solver approach. One option for primary truss members is to generate a set of influence surfaces for multiple members, output quantities, and then extract some general trends from these patterns, which could be used to produce results similar to an influence solver with fewer overall load cases. For example, a simply supported truss may be loaded in patterns corresponding to individual truss bays, then the results from each bay can be combined if they're additive to the member action under consideration.

Members subject to the combined effects of axial load and flexure present an additional complication. The same loading pattern may not maximize both effects simultaneously, so separate loading configurations may be required for each output quantity, then interaction would be checked separately for the combined effects under each separate loading pattern.

3.6.3—Wind Loads

The effects of wind load can be a significant factor in the design of truss bridges. Truss bridges are deeper than multi-beam bridges of comparable length and are often used for longer-span applications in locales with more pronounced wind exposure conditions. To resist wind load and limit deflections, lateral bracing in the horizontal or near-horizontal planes of the bottom and upper chords (except pony trusses) is often provided. Additionally, for typical truss configurations, sway frames along the structure and portal frames at supports are provided to transfer wind loads from the upper chord to the lower chord and into the bearings and substructure units. A more detailed discussion is provided in Article 3.5.4.

3.6.3.1—Wind-on-Structure

Like multi-beam structures, the effects of wind on a truss superstructure should be considered during construction and in its permanent condition. However, the effects of wind loads may also need to be considered under future coating maintenance operations where containment systems can create a large surface area for wind application.

Wind direction is random and hence the direction of wind load on a bridge, theoretically, should be varied to obtain the maximum forces in the bridge component under consideration. However, for the design of superstructure elements of a typical truss bridge it has been the tradition to consider the wind-on-structure loads in the two orthogonal directions (transverse and longitudinal) of the bridge. Wind load is applied simultaneously in the transverse and longitudinal directions to obtain the maximum design loads in the structure. Refer to Articles 3.8.1.2.2 and 3.8.1.2.3 of the *AASHTO LRFD BDS* (2024) for combinations of longitudinal and transverse wind loads.

In the analysis of truss superstructures, wind-on-structure load is considered a moving load and needs to be applied on that portion of the bridge that produces maximum effects in the member under consideration. For the case of a simple-span truss bridge, the wind-on-structure load is typically applied to the full length of the bridge to determine the maximum force effects.

Wind pressure is exerted on all the truss members that are exposed to wind in any given direction, and the load is equal to the wind pressure intensity multiplied by the projected area of the exposed members in a plane normal to the wind direction. The members that are on the upstream (relative to the direction of the wind gust) side of the truss are called "windward trusses/members," and those on the downstream are termed "leeward trusses/members." The open nature of a truss bridge superstructure means that wind load is applied to both the windward and leeward trusses simultaneously. This is in contrast to a closed multi-beam bridge where wind pressure is assumed to be applied only to the windward face. Due to the shielding effect, the wind load on leeward members is smaller compared to that on the windward members. However, it may be prudent to assume the wind pressure on leeward trusses is equal to that on the windward members in the case of very wide bridges. It should be noted that AASHTO and most Owner agencies do not have requirements for such considerations. Therefore, the Engineer should apply engineering judgment on a case-by-case basis. For larger scale structures, wind tunnel testing and/or wind-induced vibration may also need to be evaluated.

3.6.3.2—Wind on Live Load

Wind on live load should be considered on the permanent structure and may need to be evaluated under future maintenance operations. Unlike wind-on-structure, wind-on-live load need not be considered during construction except under staged construction scenarios. The interruptible distributed patch loading used to represent wind on live load should be applied on to the tributary areas that produce the largest force effects. Most truss bridge superstructures are simple-span structures, making the controlling tributary areas very intuitive to identify. For continuous truss superstructures, multiple patch loading configurations may need to be evaluated. Wind on live load is transferred through the deck, into the floor system and into the lower chord framing system (for typical configurations), which can be readily modeled in three-dimensional analyses. Its effects on two-dimensional analysis will need to be calculated by alternate means.

3.6.3.3—Effects of Wind Overturning Loads

Truss bridges are generally much deeper than multi-beam bridges which creates more eccentricity from the wind-on-structure loads (e.g., eccentricity of wind loads acting on upper chord measured to bearing elevation). The overturning forces associated with wind-on-structure loads need to be evaluated along with the overturning from the wind on live load (i.e., transverse loading at six feet above deck level). While overturning forces are often inconsequential on multi-beam bridges, these same loads can have significant effects for truss bridges, which have a much more pronounced height-to-width ratio. In addition to the overturning effects due to eccentric lateral wind loads, Article 3.8.2 of the *AASHTO LRFD BDS* (2024) requires the consideration of a vertical, or uplift, wind coincident with the transverse wind. This load is applied upwards at the quarter point of the deck. For two-dimensional planar analyses, the equivalent eccentric load acting at the truss line can be calculated and applied.

3.6.3.4—Bearing and Restraint Forces Due to Wind Loads

Most commonly, truss bridge configurations use two primary bearings at each support. In these instances, the overturning effects from wind loading are transferred to the support through a force couple that increases the reaction at the leeward bearing and reduces the reaction at the windward bearing. This minimum reaction should be evaluated in the bearing design, particularly during construction before the weight of the concrete deck is applied.

The lateral deflection of the superstructure under wind load should also be evaluated to verify that the bearings have adequate lateral rotational capacity. In cases where the bearings are over restrained in this direction, stress concentrations can be introduced into the framing system. Similar to the effects of overturning forces, this condition may be amplified during construction before the deck is in place to increase the lateral stiffness of the superstructure for structures that do not include a wind bracing system.

3.6.3.5—Wind Load Paths

Wind load on the main truss members and attachments is transferred to the top and bottom lateral bracing system through the truss panel points. From the panel points, the lateral bracing system transmits the wind loads to the end portal/sway braces which carries the load to the support bearings. Since all the joints in a truss are typically considered to be "pin connections," the wind load on individual members is transferred to the truss panel points in proportion to the tributary area and applied as joint loads as opposed to member loads in the analysis.

It is common practice to assume that the wind load on the main truss web members (verticals and diagonals) is distributed equally between the top and bottom lateral bracing systems applied at panel points. The load on the barrier, railings, and other miscellaneous attachments is applied at the panel points of the chord under whose tributary area these elements (barriers, etc.) fall. For example, in the case of a through truss, the wind load on the flooring system, barriers, and railings is applied at the lower chord panel points whereas for a deck truss bridge it is applied at the upper chord panel points. The wind load on the main truss chord members is applied at the panel points of the respective chords. The local effect of wind loads on the individual truss members is generally negligible due to the modest depth of any individual member. However, long-span truss bridges may require member sizes that necessitate evaluation of the lateral bending due to the locally applied wind loads.

3.6.4—Thermal Loads

Thermal forces in a simple-span truss are minimal where one end is free to move longitudinally. For a continuous truss, thermal loads will develop between fixed supports. Additionally, for analyses of existing trusses, inspection reports should be reviewed for evidence of "frozen" expansion bearings, which will develop reactions due to thermal loading.

For wide structures, consideration should be given to the bearing fixity in the transverse direction. Thermal effects will cause a lateral force in the end floorbeams if both bearings are fixed transversely. Special care can be taken in detailing the bearing to allow for temperature movements in the transverse direction to alleviate the transverse forces. Alternatively, the flexibility of the bearing and substructure can be considered to eliminate or reduce these results.

The AASHTO LRFD BDS (2024) specify two types of temperature changes to be considered during the analysis or design of bridge superstructures, uniform and gradient. For the uniform temperature change, Article 3.12.2 of the AASHTO LRFD BDS (2024) requires an analysis of uniform temperature decrease and increase from the setting temperature to determine the force effects on the truss members. Software programs often allow the desired temperature change to be directly defined for each member. For three-dimensional analyses, all elements of the model should be subjected to the same thermal change and have the correct coefficients of thermal expansion defined. If the uniform change is not applied to some elements, the results will be erroneous. Additionally for trusses with multiple fixed supports (or frozen bearings), the substructure elements or equivalent foundation springs should be included to obtain the correct forces and movements. In two-dimensional analyses with multiple fixed supports, the resulting forces and deflections based solely on thermal loading of the main truss members will be inaccurate. This is because the analysis does not account for the thermal movements and resistance contributions of the bracing members and floor system.

The AASHTO LRFD BDS (2024) also specify a prescribed multilinear temperature gradient to model non-uniform heating effects in bridge superstructure across its depth, primarily due to concrete decks shielding the structural steel below from solar radiation. The temperature gradient, as specified in Article 3.12.3 of AASHTO LRFD BDS (2024), is considered in addition to uniform temperature changes. Traditionally, temperature gradient is not modeled for conventional truss analysis. Where deemed necessary (such as a forensic investigation of an existing structure), a three-dimensional analysis with more refined modeling of the floor system to appropriately apply temperature changes is required. The gradient would typically be applied to the deck and floor system below (including bracing) with the assumption that the main truss members are all of a uniform temperature. Depending on the analysis software, gradients may be directly applied to the model, or, for indeterminate structures, equivalent external restraining forces must be used. Article C4.6.6 of *AASHTO LRFD BDS* (2024) provides additional discussion. The complexity and variability of thermal gradient loading is beyond the scope of this document. Where required, the Engineer is urged to consult references specific to thermal gradient analysis.

3.6.5—Seismic Loads

Trusses located in Seismic Zones 2 to 4 may require response spectrum or time history analyses to determine seismic movements and forces per Article 3.10 of the *AASHTO LRFD BDS* (2024). These types of analysis are described in Article 4.7.4 of the *AASHTO LRFD BDS* (2024). For seismic analysis, loads are not applied to the structure. Instead, ground displacements or their equivalent accelerations are applied to the structure, resulting in structure displacements and loads. These results are dependent on the stiffness, mass, and damping of the structure. As a result, a three-dimensional analysis with proper modeling of the structural elements, including the substructure, is necessary to obtain the correct distribution of mass and stiffness. Various software packages handle mass modeling and damping in different ways. A thorough understanding of the software's methods and capabilities should be achieved before modeling begins. For damping, Article C4.7.1.4 of *AASHTO LRFD BDS* (2024) suggests one percent for steel structures unless more refined data is available. Refer to the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* (2023) for additional seismic modeling details.

Typically, trusses are assumed or designed to remain elastic (no yielding) during seismic events. The lateral inertial forces induced in the superstructure are resisted by lateral bracing, which transmits the loads to the bearings and substructure units. The basic approach to determining seismic loads for truss bridges is similar to that for other bridge types. In terms of analysis, the Engineer needs to be cognizant of the arrangement of the lateral load resisting system of the bridge. Some truss bridges only have lateral bracing in one plane, resulting in a torsionally flexible structure which must be accounted for in the analysis. Trusses also tend to have the center of mass, driven by the deck, eccentric from the lateral center of stiffness, usually controlled by the lateral bracing system, which also induces torsional motions.

In terms of seismic force resistant systems, trusses can usually take advantage of the same substructure-based systems as other bridge types with the inelastic behavior confined to the substructure. This is preferred for new design, as well as retrofit when possible. There are also methods of evaluating inelastic behavior in the truss members themselves, and these are dealt with in the FHWA *Seismic Retrofitting Guidelines for Complex Steel Truss Highway Bridges*.

3.7—RUN ANALYSIS; VERIFY AND INTERPRET RESULTS

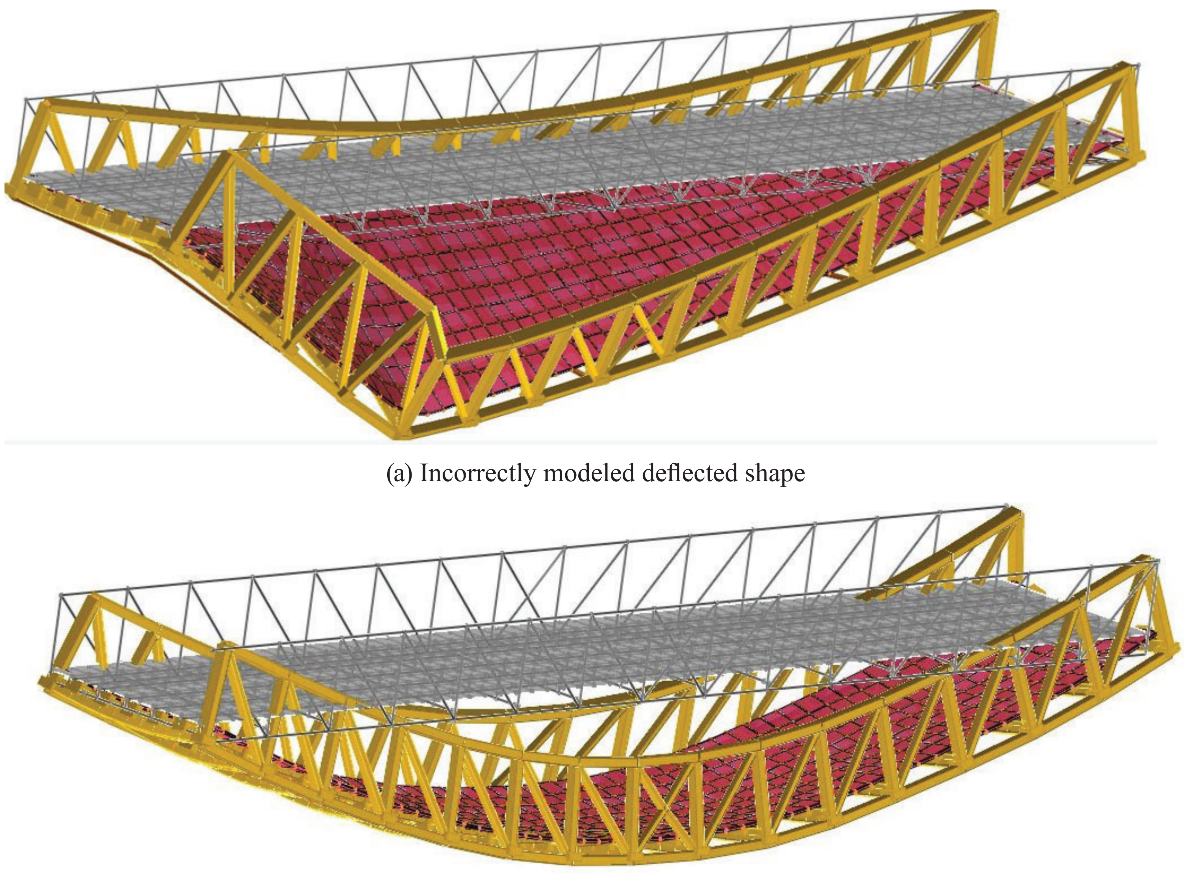
Prior to using an analysis program for evaluating any structure type, the Engineer should be comfortable with the program's analysis method and the results. A benchmark analysis solution can be utilized as it provides Engineers with a result set that can be used to compare the performance of their particular analysis program to a known benchmark result. An ideal benchmark analysis solution will often include analysis results from a specific program compared to and in agreement with either experimental or field measured results. Alternatively, a benchmark solution may be simply determined via hand calculations or other theoretical solutions.

In the case of a truss superstructure analysis, an ideal benchmark solution will provide results for member forces, reactions, and displacements. The Engineer can recreate a model of the benchmark solution truss in their desired analysis program and compare results to determine the viability of the analysis program. In general, differences of 10% in the results between the Engineer's analysis program and the benchmark solution are reasonable to show the validity of the desired analysis program.

Unfortunately, in the published literature there is very little in the way of full benchmark solutions for truss structures that provide results for all member forces, reactions, and displacements. There are published studies that focus on several members and compare and supply analysis results for only those members with field measured or experimental data. These published results may be sufficient in some cases but may not give the Engineer full comfort with their analysis program. Alternatively, a model comparative investigation of a truss solved by hand methods may be reasonable to validate a certain analysis program. Hand calculation solutions and similar examples are provided in multiple textbooks for structural analysis.

It is important to validate the results of the model before using these results for design or rating purposes. There are various ways to validate the results of a truss model. The following is a list of potential, but not all-inclusive, validation methods. Use as many as are necessary to have confidence in the model and its results. If any of the checks invalidate the model, the model inputs and assumptions should be re-evaluated.

1. **Examining the deflected shape**—After the model is run, examine the exaggerated deflected shape under loading. By intuitively knowing what the deformed structure should look like, the Engineer can identify obvious modeling errors. Refer to the two exaggerated deflected shapes in Figure 3.7-1. Both models run and give results. However, the model on the top (which erroneously has a chord member that has axial load released), obviously is not giving the correct result and should be re-evaluated.



(a) Incorrectly modeled deflected shape

(b) Correctly modeled deflected shape

Figure 3.7-1—Exaggerated deflected shape of an incorrectly and correctly modeled structure.

2. **Reactions**—Reactions can be estimated by hand calculations. It is important to verify that the independent hand calculations match the model results within a reasonable percentage.

3. **Member force check—**Hand calculations can also estimate the upper and lower chord force at midspan within a reasonable percentage, to assure that the model is behaving as expected. See the following example for midspan of a simple-span truss subjected to a distributed load.

$$\frac{\left(\dfrac{wL^2}{8}\right)}{(\text{Truss depth})} = \text{approximate midspan chord force} \quad (3.7\text{-}1)$$

4. **Thermal movements**—Thermal movements can be estimated through hand calculations and compared to the movements from the model.
5. **Comparing to previous results (in addition to Methods 2, 3, and 4 above)—**If this is an existing bridge, there are likely member force tables, reaction tables, and expansion joint movement tables in the plans that can be compared to your model results. Note that there may be differences between the structure now and the original as-built structure, such as deterioration, retrofits, new deck, new barriers, new wearing surface, etc. The original analysis may have also been limited to two-dimensional analysis, due to computational limitations, whereas the updated model may be a three-dimensional analysis. Regardless, the comparison between original and new results should be relatively close and give confidence in the model results. If there are significant differences, the differences should be justified.
6. **Comparing to simplified model—**If necessary to have confidence in a three-dimensional truss model, consider making a simplified two-dimensional model of the same structure and comparing truss member forces between the two models.
7. **Using different software—**If time and budget allow, and the complexity of the structure warrants such an analysis, it may be prudent to analyze the structure in two different software packages and compare the results.
8. **Parametric study**—If one input variable is particularly important or uncertain (such as member end-fixity or boundary condition stiffness), a parametric study may be warranted. This allows Engineers to evaluate how sensitive the model is to that variable and apply engineering judgment in selecting an appropriate value.

3.7.1—Benchmark References

Laurendeau (2011) describes the nondestructive live load testing and subsequent finite element modeling of a cantilevered deck arched Pratt truss bridge. The bridge was instrumented with 151 strain gauges on various floor and truss members along with eight displacement gauges strategically placed along the truss. The recorded gauge readings were used to determine bridge behavior and the calibration of a working finite element model.

Hickey et al. (2009) report on live load tests and modeling for a steel deck truss bridge. Analytical results were compared with field measured results for 14 members observed during controlled live loading of the bridge. Data from live loads were obtained by loading two trucks to 25 tons each. Trucks were positioned at eight locations on the bridge in four different relative truck positions, and data was recorded continuously and reduced to member forces for model validation comparisons. Deflections at selected truss nodes were also recorded for model validation purposes.

SECTION 4—SPECIAL ANALYSIS TYPES

4.1—ANALYSIS FOR CONSTRUCTION

This Section discusses special analysis types for trusses and discusses how to perform each of the analysis types.

4.1.1—Steel Erection

It is often necessary to evaluate stress and stability of truss members through varying stages of steel erection. For analysis of the construction sequence, it is recommended to use a program that allows for incremental stages of construction and calculates member forces at each stage. This requires a staged construction analysis where, within each stage, appropriate elements are activated/deactivated, member properties modified, boundary conditions changed, or loadings applied/removed to mimic the planned erection sequence. Careful thought should be put into the level of refinement and detail for the staged construction analysis to achieve the desired goal. The truss design Engineer is primarily concerned with a few critical stages of erection to verify that critical truss members are not overstressed by the conceptual erection scheme. The erection Engineer, when it is required on the contract plans, will prepare a much more refined model that analyzes each stage of erection to obtain the anticipated forces and deflections in the structure, as well as to ensure stability as described in Article 4.1.7. If the final member forces from a staged construction analysis are different than those from a linear static analysis, it may be necessary to evaluate the effects of locked-in forces generated by the selected erection sequence.

If difficulties are encountered in performing a forward analysis of the construction sequence, the analysis may be performed in reverse starting from the completed structure and removing members in reverse order from the construction sequence. This approach ensures that the structure "ends" at the final desired geometry.

4.1.2—Camber Effects

Truss bridges, when fabricated and erected, typically include some level of built-in camber. The intent of this camber is to (1) provide geometric control of the structure so that the truss has the desired profile after the anticipated deflections occur, and (2) help counteract secondary effects in the members under the applied loadings. For most of the life of the structure, the truss members are subjected to dead loads with the occasional change from live, wind, or other transient loads; consequently, it is most common to camber truss spans for the total dead load condition. This allows the structure, under dead loads, to maintain the desired plan geometry and eliminates secondary dead load moments in the members.

When loads are applied to the truss, the structure deflects, and individual members are subjected to either axial compressive or tensile forces. These compressive or tensile forces cause either shortening or elongation in the members, respectively. To create the camber in the truss, fabricators increase the length of compression members by the anticipated shortening under total dead load and decrease the length of tension members by the anticipated elongation under total dead load. The total dead load is taken from the components that are in place at the completion of construction and doesn't typically include any future wearing surface. The required change to the fabricated length is given by the equation $\Delta = PL/AE$, where:

Δ = displacement along the centerline of the member
P = axial force in tension or compression
L = length of the member
A = area of the member
E = modulus of elasticity of the member

While truss connections are often idealized as pinned, most gusseted connections have some level of flexural rigidity. As the truss deflects, these rigidities induce secondary bending moments into the members. Often, when truss members with axial camber adjustments are assembled, these cambers induce moments that are opposite of those due to dead load. Hence, after dead load is fully applied to the truss, these moments are essentially zero.

While cambering may affect secondary moments developed under dead load, it will not have a significant impact on the axial loads in the members (the axial loads from models with pinned ends or fixed ends are reasonably similar). Therefore, most analysis models do not incorporate the anticipated camber, especially for cases where member end connections are pinned. The Engineer should be cautioned that not incorporating camber effects directly in a model that contains member end fixities could yield erroneous results for dead load deflections, introduce unrealistic secondary dead load moments in the members, or cause unrealistic dead load forces in transverse bracing members of three-dimensional models. The level of error depends on the level of connection fixity assumed compared to the true behavior, the ratio of member length-to-width, and the related flexibility of the members.

The expected dead load camber may be incorporated into the truss analysis for the final in-service condition and the staged erection analysis to track the truss geometry and assist in post-processing of member results by accurately capturing dead load moments in the members. This can be done by assigning a unique strain ($\varepsilon = \Delta L$) to each member to match the fabricated length. An initial dead load analysis is performed to calculate the compressive and tensile forces in each main truss member, which are utilized to calculate the strain for the cambered dead load analysis. The cambered load case can then be generated and analyzed with the dead load case. The cambered dead load analysis results in secondary moments at the ends of members and deflections at truss panel points that are zero or nearly zero.

4.1.3—Deck Placement Sequence

When the concrete deck may be too large for a Contractor to place the entire deck in a single operation, the Engineer develops a deck placement sequence, which specifies the order to place different portions of the deck. By casting the deck in different stages, portions of the structure act compositely while other portions do not, impacting the force and stress distribution through the system as subsequent deck segments are placed.

The deck placement sequence is typically included for the floor system stringer analysis. The Engineer should consider the influence of the floor system framing before incorporating the deck placement sequence into the analyses of the floorbeam and truss members. The deck placement sequence is rarely required for floating floor systems analyses, where the floor system acts independently from the truss members but may be considered for cases where the deck is composite with floorbeams or truss chord members as the creep and shrinkage of the deck can cause challenges with the weak axis bending of the end floorbeams

The deck placement sequence can be evaluated through staged construction analyses. When the deck is explicitly modeled, deck elements and their connectivity to the stringers can be activated within the various deck placement stages to capture the behavior. When the deck is included in the model using composite section properties for the stringers, the stringer member properties can be modified in the different construction stages to reflect composite sections when the deck has cured in specific locations.

4.1.4—Wind Load During Construction

Wind loads should be considered when performing an erection analysis. Refer to AASHTO *Guide Specification for Wind Loads on Bridges During Construction* (2017b) for information on how to calculate the wind load during erection. Temporary bracing, in addition to permanent bracing, may be required to provide stability between the planes of the truss. Additional support (erection tower, tie-downs) may be required at critical stages of erection.

Include temporary bracing and/or supports in the analysis model as elements or boundary conditions that are activated for the corresponding stage during construction. Refer to Article 3.6.3 for information on how to apply the wind loads in the analysis model.

4.1.5—Live Loads During Construction, Rehabilitation, or Demolition

Construction analyses performed during the design phase should include a construction live load that accounts for personnel, the weight of the screed, and other equipment necessary to pour a concrete deck. The magnitude of the construction live load should be prescribed in the bridge design criteria and the assumed values should be listed on the plans. The application of the construction live load during design is usually limited to the deck area and is not applied to other surfaces of the bridge. The final erection analysis by the Contractor's Engineer should incorporate the exact temporary loads placed at corresponding locations along the bridge that align with the Contractor's selected methods and equipment to ensure the truss capacity is greater than the applied construction loads.

4.1.6—Asymmetric Deck Replacement

If replacement of the concrete deck is required to be staged asymmetrically in the transverse direction, an erection analysis should be performed to determine loads in the main members, as well as the bracing members. Typical design for trusses assumes symmetric placement of the concrete deck, unless noted on the plans. With the unsymmetric loading, additional forces will be imparted to the bracing members and floor system due to the differential deflections when one plane is unloaded. The erection analysis should also consider potential effects on the floor system.

4.1.7—Stability Analysis During Construction

Complex structures, such as trusses, are designed and analyzed for stability in the final condition, with a high-level check done for the erection sequence shown in the design documents. The actual intermediate steps used by the Contractor should each be evaluated for stability as part of the erection engineering analysis, as some truss members may require temporary bracing during various stages. In some cases, temporary cross bracing intended to control the geometry and provide lateral stability in stages may be required when permanent bracing is not installed. Detailed buckling analyses for every stage of erection are often not necessary but should be performed at critical erection stages. More details on stability analyses are discussed in Articles 4.2 and 4.3.

4.1.8—Bridge Demolition Analysis

When a truss bridge is removed from service and local conditions dictate, it may be necessary to incrementally "deconstruct" the bridge. In this case, an analysis will need to be performed to analyze the bridge at different stages of demolition. This will be similar to a staged construction erection analysis except members will be removed in each stage, rather than added. Smaller truss bridges can often be removed using cranes without deconstruction; however, the truss typically has different support points when rigged and hung from a crane hook. As a result, the truss members may have a very different state of stress, including load in zero-force members or tension members that are now in compression. In each of these cases, the demolition analysis is used to determine member forces, obtain anticipated deflections, verify stability, and determine temporary bracing and support requirements for each stage of demolition.

4.2—SECOND-ORDER ANALYSIS

If the analysis considers the deflected position of the structure to satisfy equilibrium requirements, then the analysis is said to be a second-order analysis. As stated in the *AASHTO LRFD BDS* (2024), Article C4.5.3.2.1, "The second-order effect arises from the translation of applied load creating increased eccentricity. It is considered as geometric nonlinearity and is typically addressed by iteratively solving the equilibrium equations or by using geometric stiffness terms in the elastic range." Articles 4.6.5 and 4.5.3.2 of the *AASHTO LRFD BDS* (2024) also use the phrase "large deflection theory" to refer to any method of analysis that considers second-order effects. Articles C4.1 and C4.5.3.2.1 refer to second-order analysis methods as a "geometric nonlinear analysis."

Generally, there are two types of analyses that consider second-order effects: eigenvalue buckling analysis and geometric nonlinear second-order load-deflection analysis. In an eigenvalue buckling analysis, the Engineer solves for the load level at which the structure, or an individual member, would bifurcate from its initial geometry into a buckled configuration. This type of analysis involves the determination of eigenvalues (buckling load levels) and eigenvectors (buckling modes). Effective length factors for stability design of columns, beams, and beam-columns can be calculated using eigenvalue buckling analyses of individual members. Global system buckling analysis, also referred to as stability analysis, is discussed in Article 4.3.

In a geometric nonlinear second-order load-deflection analysis, the influence of second-order effects on the overall load-deflection response is calculated for a given load condition. For most steel truss bridge designs, second-order effects are minor and can be handled sufficiently without conducting a second-order load-deflection analysis. When using conventional truss analysis methods, second-order effects on compression members are considered through effective length factors. *AASHTO LRFD BDS* (2024), Article 4.6.2.5 provides guidance for the selection of appropriate effective length factors. For frame-type structures where member end moments are considered, second-order effects can be approximated through the adjustment of the internal forces using amplification factor equations. *AASHTO LRFD BDS* (2024), Article 4.5.3.2.2b provides recommendations for calculation of moment amplification factors for beam-columns. Additional evaluation of second-order effects should be considered for structures with non-standard configurations or structures that exhibit large deflections under normal loading conditions. Second-order analysis should be considered for truss bridges that experience large lateral deflections due to wind loads, such as those using Vierendeel bracing, which does not use a triangulated system to resist lateral loads.

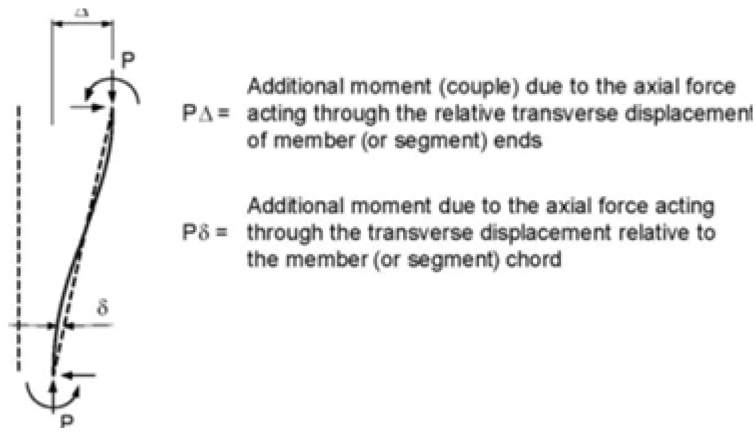


Figure 4.2-1—Two types of second-order effects in column and beam-column members.

For a more detailed discussion of second-order analysis, the Engineer is referred to the FHWA *Manual for Refined Analysis in Bridge Design and Evaluation* (2019).

4.3—STABILITY ANALYSIS

Stability must be considered in the evaluation of trusses because they contain compression members. A local stability check is required to ensure the compression elements (compression chord, compression diagonals and verticals, bracing members) will not buckle between lateral brace points by selecting an effective length factor as given in the *AASHTO LRFD BDS* (2024) based on end connection type. A conservative value of 1.0 could be assumed.

Historically, lateral stability for trusses was provided through the use of truss style lateral bracing in the planes of both the upper and lower chords. For a through truss, a portal frame is frequently used to restrict sway of the upper chords. In addition, intermediate sway bracing is often used in transverse sections, between trusses, to prevent distortion of the overall cross-section (also known as sidesway)

Global stability of a through truss depends primarily on the portal bracing, as practically any form of truss lateral bracing system will provide sufficient stiffness within the plane of the top chord members. Portal frames are typically designed to carry the forces in the upper lateral system down to the bearings, and this, combined with the compression design of the portal members, typically results in sufficient stiffness to preclude global buckling.

For deck trusses, it is typical to carry the truss bracing directly to the lateral bearings, and no portal frame is needed. Trussed arches are not considered in this document.

Evaluation of global stability is particularly important for trusses with unbraced compression chords (half-through or "pony" trusses), trusses with long spans, and trusses with non-traditional bracing systems. An approximate method for stability analysis of half-through trusses is provided in the *AASHTO LRFD BDS* (2024), Article 6.14.2.9, and the *Guide to Stability Design Criteria for Metal Structures* (Ziemian, 2010). For long-span trusses and trusses with non-traditional bracing, global stability can be evaluated using a buckling analysis.

There are two types of buckling analyses that can be conducted: a linear elastic buckling analysis, commonly called an eigenvalue analysis, and a more complex nonlinear analysis. In an eigenvalue analysis, buckling of an initially perfect system under the action of specified loads is predicted. Members are assumed without deformations and the analysis computes a factor called the eigenvalue, a multiple of applied loads at which buckling is predicted. A nonlinear analysis is a stepwise analysis where the same loads are applied but in steps from zero to their full intensity. After each analysis stage, the model is altered to reflect the deflected shape, loads act through displaced geometries, and the model eventually converges or becomes unstable. Amplified forces from the analysis can be used for member design and the deflected shape can be examined to determine if the behavior meets the design objectives. Nonconvergence of this model indicates instability under the applied loads.

Nonlinear analysis requires some form of initial deformation to initiate the second-order effects associated with buckling behavior. In cases where loading is purely axial, an initial imperfection in the model geometry may be needed to initiate buckling deformations. If the loading includes an orthogonal force or moment that will produce lateral deformation, introduction of an imperfection is generally not required.

Eigenvalue analysis predicts the theoretical elastic buckling strength of an idealized structure exhibiting linear elastic behavior. However, initial imperfections and nonlinear behavior often prevent actual structures from reaching their full theoretical elastic buckling strength. As a result, eigenvalue analysis can produce unconservative results, over-predicting the buckling strength. Eigenvalue analysis does have several advantages and uses. It is computationally efficient and is useful as a preliminary estimate of the critical buckling load. Eigenvalue analysis is often used to determine possible buckling modes or as a baseline for a nonlinear buckling analysis. It is also an efficient way to evaluate the comparative performance of alternatives.

A detailed discussion of the application of eigenvalue and nonlinear buckling analysis is provided in the FHWA's *Manual for Refined Analysis in Bridge Design and Evaluation*.

Arch structures, including trussed arches, have unique stability concerns not encountered in traditional truss bridges, and are not covered in this document.

4.4—LOAD RATING ANALYSIS

Load rating analysis provides an assessment of the live load capacity of a bridge, determines if posting is required, and helps identify repairs that are needed for the bridge. AASHTO's *Manual for Bridge Evaluation* (*MBE*, 2018) provides guidance on load rating bridges. Some bridge owners have additional load rating guidance which supplements the AASHTO guidance.

Typically for routine truss bridges, approximate analysis methods are used to evaluate member demand. Refined methods of analysis are typically used for complex structure types or where the controlling rating factors are less than 1.0 and a refined analysis could result in rating factors greater than 1.0. Refinements include creating a three-dimensional finite element model, including the barrier stiffness in the model, and/or performing a load test on the structure.

Load rating a truss bridge involves analyzing the stringers, floorbeams, main truss members (chords, diagonals, and verticals), and gusset plates to determine the forces acting on each member in response to various live load vehicles. The calculated capacity of each element is used in conjunction with the dead and live load demands to determine the load rating factors.

4.4.1—As-Designed and As-Built Load Rating Analysis

Many Owners require load rating factors to be shown on construction drawings for new bridge designs. For new bridge designs, the truss load rating analysis will be completed in conjunction with the design analysis. An as-designed load rating analysis may be required by the Owner during the design process.

An as-built load rating analysis needs to be performed after the bridge has been constructed and includes any differences between the as-designed bridge and current as-built condition. For example, a truss bridge may have undergone maintenance work such as member repairs or a bridge deck overlay.

4.4.2—Consideration of Dead Load Sequencing in Load Rating Analysis

Often the erection sequencing is shown on the plans and the load rating Engineer should consider the sequencing when performing the load rating analysis which may require a staged construction analysis as discussed in Article 4.1.

4.4.3—Load Rating Analysis of Deteriorated Structures

Deterioration and section loss are typically accounted for in element capacity calculations, not the load rating analysis. Deterioration in gusset plates requires careful documentation and consideration during a load rating analysis due to the number of controlling limit states and failure planes in gusset plates. In extreme cases where the deterioration is significant and changes the distribution of forces through the truss or its members, the new distribution of forces should be accounted for in the load rating analysis. For example, if severe deterioration exists on one gusset plate of a truss vertical, a greater portion of the loads may act through the other gusset plate, which results in an eccentricity of the member force.

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APPENDIX A—TRUSS ANALYSIS EXAMPLES

A1—TWO-DIMENSIONAL MODEL EXAMPLES

A1.1—KY-644 BRIDGE

KY-644 Bridge is a single-lane bridge crossing KY-644 over Levisa Fork of the Big Sandy River (Bridge No. 064B00038N) in Lawrence County, Kentucky (Figure A1.1-1). The bridge is 475 feet in length and consists of four spans, two of which are Pratt through-truss spans, a Warren through-truss span with verticals, and a dual steel girder span. The bridge, which was constructed in 1905 and rehabilitated in 1970, is considered the longest pin-connected bridge in Kentucky. The maximum span length is 170.9 ft. to the center line of the pins at the ends of the trusses. The existing bridge has two 9-inch-wide curbs. The curb-to-curb roadway width is 12 feet, 2 inches, and the out-to-out width of the superstructure is 13.75 feet. The truss members are made of riveted built-up tubular sections, eyebars, and counters made of circular rods. The deck is supported by stringers which bear on transverse floor beams. The bridge has been determined eligible for listing in the National Register of Historic Places.

Figure A1.1-1—KY-644 Bridge over Levisa Fork of the Big Sandy River, KY.

A1.1.1—Analysis Method Selection

A two-dimensional model was created for each truss span to update the load rating for the existing structure and to develop options for strengthening the bridge. Two-dimensional analysis was deemed appropriate for performing the load rating since it focuses on the in-plane force effects from gravitational and traffic loads.

A1.1.2—Model Geometry

Nodes were located at the truss connections. The model included all components of the truss. Each truss span is modeled as a two-dimensional simply supported truss. The bearings of each truss span are modeled as hinge at one end and a roller at the other end (Refer to Figure A1.1.2-1 for model schematics).

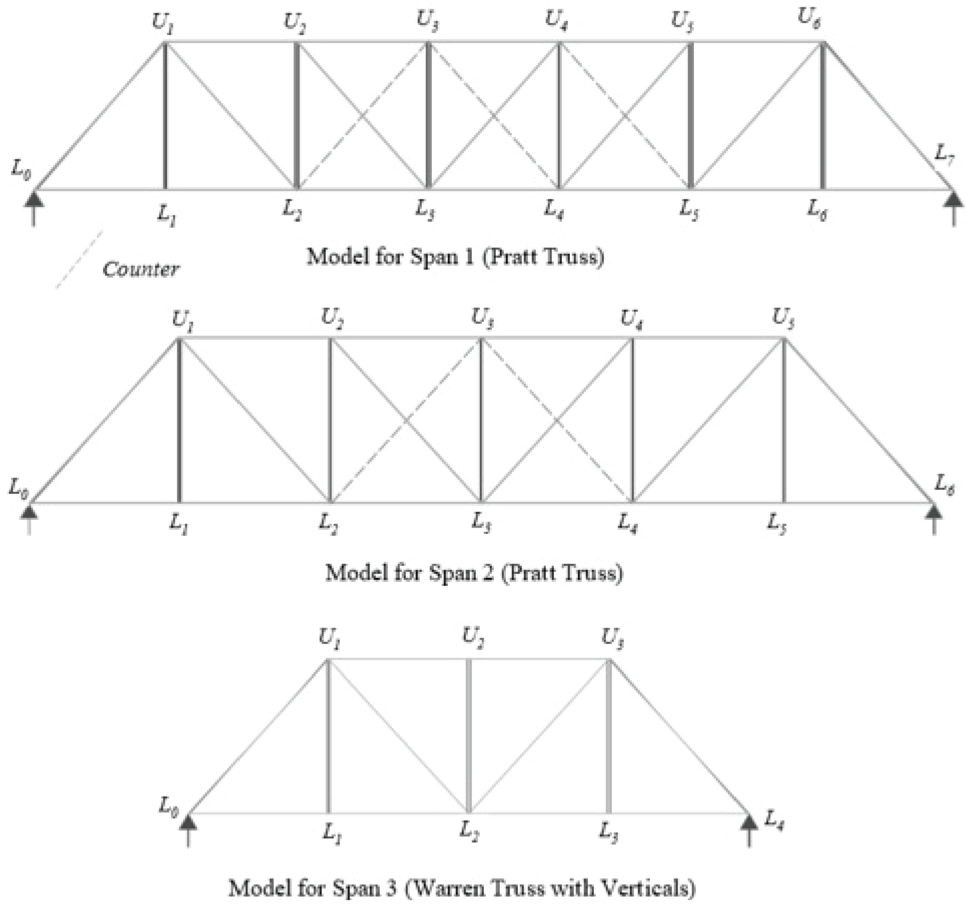

Figure A1.1.2-1—Two-dimensional truss model.

A1.1.3—Model Elements and Section Properties

The truss members are modeled using truss elements (tension/compression members). Counters are modeled as tension-only members. Each pin connection was analyzed separately. Stringers and floorbeams were analyzed separately using beam elements. The upper and lower lateral bracing were not modeled explicitly. The weights of the bracing elements and the deck were estimated and were applied as point loads at the truss joints.

A1.1.4—Model Boundary Conditions

The truss boundary conditions were applied at the truss ends. Vertical and lateral displacements are restrained at both ends. In the longitudinal direction, one end of the truss is restrained at the hinge support, while the other end is free to move at the roller support. For the two-dimensional model, the floorbeams and stringers were analyzed separately as simply supported beams. The floorbeam-to-stringer connections were not modeled.

A1.1.5—Model Loads

The dead load was estimated based on the field measurements of the members' cross-sections, and the thickness of the concrete deck.

The moving loads applied to the model consisted of HS-20 loading, the AASHTO legal loads, emergency vehicles, and the Kentucky overload vehicles.

A1.1.6—Model Analysis

An investigation was undertaken to determine the Inventory and Operating load rating capacity for the truss bridge structure.

A1.2—VIRGINIA ROUTE 743 OVER NORTH FORK RIVANNA RIVER

This bridge is a two-span pony truss located in Albemarle County, Virginia. The bridge was built in 2010 and carries Advanced Mills Road over the North Fork Rivanna River. The bridge is approximately 235 feet in length and owned by the Virginia Department of Transportation (VDOT). Span 1 is approximately 80 feet and Span 2 is 155 feet. The centerline to centerline of truss distance is 33 feet, 6 inches. The roadway clear width is 30 feet. Figure A1.2-1 provides an overall view of the bridge.

Figure A1.2-1—Virginia Albemarle County pony truss.

A1.2.1—Analysis Method Selection

A two-dimensional model was created to perform a load rating of the structure. A three-dimensional model was deemed unnecessary. The two-dimensional model provided a reasonable approximation of the dead and live load forces in each of the truss components.

A1.2.2—Model Geometry

The steel truss is composed of top and bottom chords, verticals, and diagonals. The floor system consists of transverse floorbeams and simply supported longitudinal stringers which frame into the floorbeams. The stringers support a noncomposite concrete deck sitting atop corrugated metal flooring. Nodes are located at the truss connection points where lines of action of each truss member intersect. Figure A1.2.2-1 shows an elevation view of both truss span configurations.

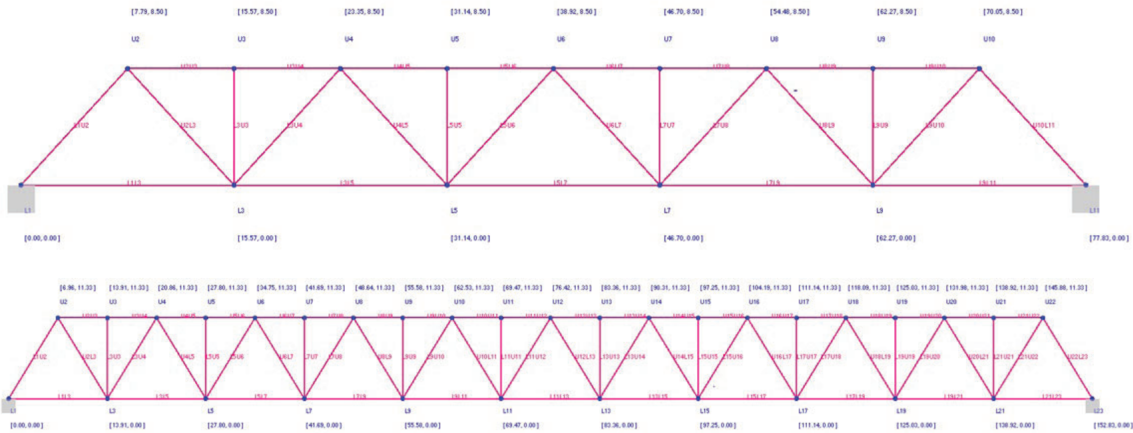

Figure A1.2.2-1—Two-dimensional truss model elevation view

A1.2.3—Model Elements and Section Properties

The upper chords, and vertical and diagonal members are rolled W-shapes. The W-shape was defined in the model. The bottom chords are composed of two channel members. The section properties (area, moment of inertia, and section modulus) were calculated and input into the model. The reduced areas of the truss members with bolt holes were calculated and input into the model. The floorbeams and stringers are also rolled W-shapes which were defined in the model.

An equivalent concrete deck thickness was calculated by combining the weight of the concrete deck and the steel corrugated flooring and converting it to an equivalent concrete thickness so that the model is correctly accounting for the weight of the steel corrugated flooring.

The live load distribution factor for each truss member was calculated using the lever rule. Distribution factors for the stringers were calculated using the AASHTO *Standard Specifications for Highway Bridges* (2002) distribution factors for steel corrugated decks and the lever rule.

A deteriorated model of the structure was also created to account for deterioration in the truss members, stringers, and floorbeams. The section properties were modified to account for any deterioration listed in the latest bridge inspection report.

The gusset plates were not modeled. The truss member forces were pulled from the analysis and input into a Mathcad spreadsheet to rate each gusset plate.

A1.2.4—Model Boundary Conditions

The truss boundary conditions were modeled with a pin at one support and a roller at the other support for each span. The floorbeams and stringers were modeled as simply supported. The floorbeam-to-stringer connections were not modeled.

A1.2.5—Model Loads

The analysis program calculated and applied the self-weight of the truss. An additional dead load was applied to account for the gusset plates, bolts, welds, etc. This load was calculated by analyzing the truss under self-weight and then subtracting the total dead load reaction obtained from the model from the truss dead load shown on the plans. The dead load of the stringers and floorbeams was calculated by the model. The dead load of the deck was calculated by the program using the equivalent concrete deck thickness.

The moving loads applied to the model consisted of HS-20 loading, VDOT legal and permit trucks, specialized hauling vehicles (SHVs), and emergency vehicles (EV2 and EV3).

A1.2.6—Model Analysis

A linear elastic analysis was run to determine the load rating for the truss members and floor system members. The software package utilized for this analysis performs a two-dimensional analysis on the truss and the truss floor system. The stability of the unbraced top chord was checked following the guidance on this topic in the *Guide to Stability Design Criteria for Metal Structures* (Ziemian, 2010).

A1.3—WABASH MEMORIAL BRIDGE LOAD RATING

This bridge is composed of 48 approach spans and a three-span tied arch through-truss bridge located in Mt. Vernon, Indiana. The bridge was built in 1956 and rehabilitated in 1987, 1993, 2008/2009, and 2012. The bridge is approximately 4932 feet in length and owned by the Indiana Department of Transportation (INDOT). Refer to Figure A1.3-1 for an overall photo of the bridge. The truss members consist of built-up box or I-shapes. The floor system consists of built-up floorbeams and stringers framing into floorbeams along the truss spans. The approach spans consist of multiple girder systems with variable depth plate girders or steel rolled beams.

Figure A1.3-1—Overall view of Wabash Memorial Bridge.

A1.3.1—Analysis Method Selection

A two-dimensional model was initially created to examine the truss since it was felt an increased refinement of the demand that could be provided by a three-dimension model was not needed for an improved load rating. The simplicity of the truss span geometry also allowed for a two-dimensional model to be used.

A1.3.2—Model Geometry

Nodes were located at the panel points. The model only presented all primary members of the truss and additional elements to facilitate live load lane placement. Camber was incorporated in the model geometry based on the original plans. Figure A1.3.2-1 depicts the truss model and identifies the different truss components.

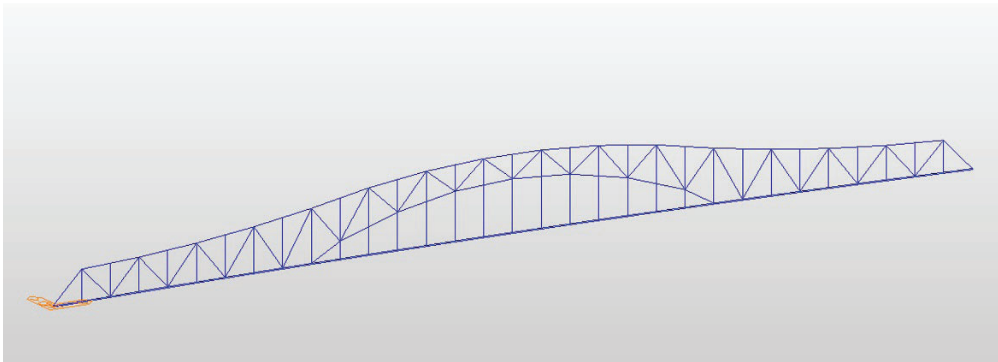

Figure A1.3.2-1—Two-dimensional model.

A1.3.3—Model Elements and Section Properties

A cross-section of each primary truss member was drawn in MicroStation and then imported into the MIDAS section builder program. Within the section builder, a MIDAS SEC file was created for each cross-section and section properties were calculated by the program based on the cross-section sketch. The section properties were then inserted into the MIDAS model as SEC files. The upper and lower lateral bracing, floor system, portal/sway bracing, and miscellaneous connections were incorporated as dead load point loads applied at the panel points.

A1.3.4—Model Boundary Conditions

Pinned or roller supports were placed at the pier and abutment locations. The panel points were modeled to allow rotation since connection at the panel points is achieved through rivets. As a result, the truss members were treated as axial load-only members.

A1.3.5—Model Loads

The dead load consisted of the truss member self-weight, deck slab, deck overlay, haunch, stay-in-place forms, stringers, diaphragms, floorbeams, parapet, top and bottom lateral bracing, and portal and sway frames. Each dead load component was modeled as a point load and applied at the panel points. To avoid putting the truss members in bending due to live load, longitudinal and vertical "dummy" elements were created. The lane definition was applied along the longitudinal dummy members so that the axial live load would be carried to the vertical dummy members, which would then be applied to the truss panel points (see Figure A1.3.5-1). Beam end-releases were also placed at the ends of the longitudinal dummy elements, so the vertical dummy members remain loaded only axially.

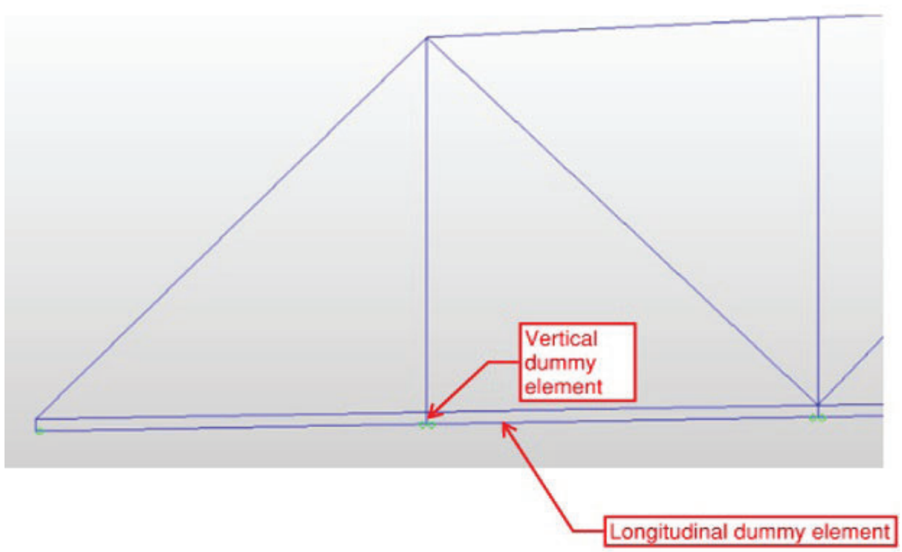

Figure A1.3.5-1—Vertical and longitudinal dummy elements connected to truss frame.

A1.3.6—Model Analysis

As stated previously, the truss is being analyzed for a load rating using load factor resistance (LFR) methodology. The truss demand produced by the model was copied into post-processing spreadsheets for the load rating of the truss members and gusset plates. Multiple-lane loading was considered by applying a live load distribution factor to the truss demand outside of the model. If low load ratings are produced for the truss members or gusset plates, a three-dimensional model will be considered for refined load distribution.

A1.4—THE MILLARD E. TYDINGS MEMORIAL BRIDGE PROJECT

The Millard E. Tydings Memorial Bridge is a steel continuous truss bridge that spans over the Susquehanna River, around 45 miles northeast of Baltimore, MD. It was built in 1963. It was designed with ASTM A36 steel. The bridge is 5,061 feet long divided into 14 spans. The Warren truss spans follow a pattern that repeats throughout the length of the bridge. This pattern consists of a suspended span, a cantilever arm, an anchored span, and again, a cantilever arm. The suspended span is 245 feet, the cantilever arm is 122.5 feet, and the anchored span has a length of 428.75 feet. The suspended and the cantilever arms constitute spans of 490 feet. The deck was made with lightweight concrete with a unit weight of 115 pcf, and it is supported by steel elements. The deck has a total width of 87.33 feet with a total of six traffic lanes. Refer to Figure A1.4-1 for an overall photo of the bridge.

Figure A1.4-1—Perspective view of Tydings Bridge, MD.

A1.4.1—Analysis Method Selection

Originally, the sliding plate system was designed to replace the finger joint system by assuming only horizontal slide, but a further review indicated that the system was affected by the complex movement of the whole structure. The model intended to provide results that could prove that besides horizontal movement, the sliding plates also were affected by an angular movement. Consequently, gaps were formed between the sliding plates, and their size depended on the temperature range considered. The commercial software, CSiBridge, as well as the BEST Center software, TRAP (Truss bridge Rating and Analysis Program), were used to model the trusses of the Tydings Bridge and to conduct the thermal analysis.

A1.4.2—Model Geometry

Once plans were collected, the first step consisted of creating a two-dimensional finite element method model, so the different elements could be placed. Note that a two-dimensional model shown in Figure A1.4.2-1 works for the purpose of this section due to its length and symmetry. Starting from the station with coordinates (0, 0, 75) feet, and the end station located 2,082.5 feet away in the longitudinal direction, the typical layout of the bridge was done. The layout is made of four cantilever arms, two suspended spans, and two anchored spans (see Figure A1.4.2-2). As the layout is symmetrical, half of the bridge elements were modeled manually, and the rest were replicated.

Figure A1.4.2-1—Two-dimensional model of the five-span thermal analysis model.

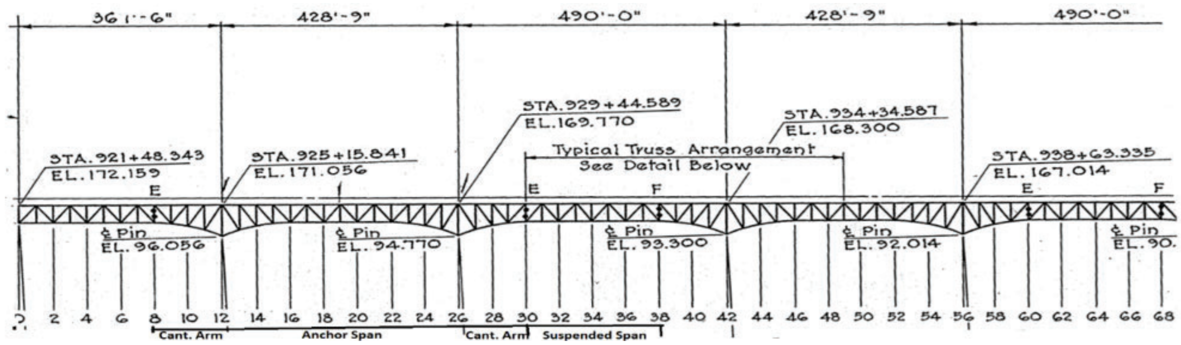

Figure A1.4.2-2—Truss panel configuration of the Tydings Bridge.

A1.4.3—Model Elements and Section Properties

Supported by the trusses, the floor system includes just over 300 floorbeams, of three types. The beams are designated F1, F2, and F3. Beams F1 and F3 are plate girders with one 60-inch × $^{5}/_{16}$-inch web, two 14-inch × $^{3}/_{4}$-inch × 56-foot cover plates, and four 8-inch × 6-inch × $^{3}/_{4}$-inch angles. F2 beams are composed of one 60-inch × $^{5}/_{16}$-inch web, two 13-inch × $^{1}/_{2}$-inch × 56-foot-10½-inch cover plates, and four 6-inch × 6-inch × $^{9}/_{16}$-inch angles. F3 beams are located at all floor breaks (transverse floor joints dividing the deck floor into panels for thermal purposes), F2 beams are located at each expansion joint, and F1 beams are at all the remaining panel point locations. Carried by the floorbeams, seven different stringers were used, designated A through G. Each stringer spans between the floorbeams, stiffening the entire structure. The sections used in the design are shown in Table A1.4.3-1, while the spacing is shown in Table A1.4.3-2.

Table A1.4.3-1—Stringer Sections for Tydings Bridge.

Stringer	End Spans	Intermediate Spans
A, B, C, D, F	W24×76	W24×76
E, G	W24×84	W24×76

Table A1.4.3-2—Stringer spacing for Tydings Bridge.

Spacing	Distance
A—B, B—C, C—D, F—G	6 ft 9 in.
D—E	6 ft $^{3}/_{16}$ in.
E—F	4 ft 4 in.

A1.4.4—Model Boundary Conditions

Following the modeling of the elements, the next step consisted of the introduction of the boundary conditions. Two rollers at both ends of the bridge, and four pins at the remainder of the piers, represented the connection of the bridge with the substructure. On top of that, some frames were released of axial loading to simulate the expansion joints (see Figure A1.4.4-1), and the degrees of freedom were set to only translational movement (U1, U2, and U3) to convert the model into a truss model.

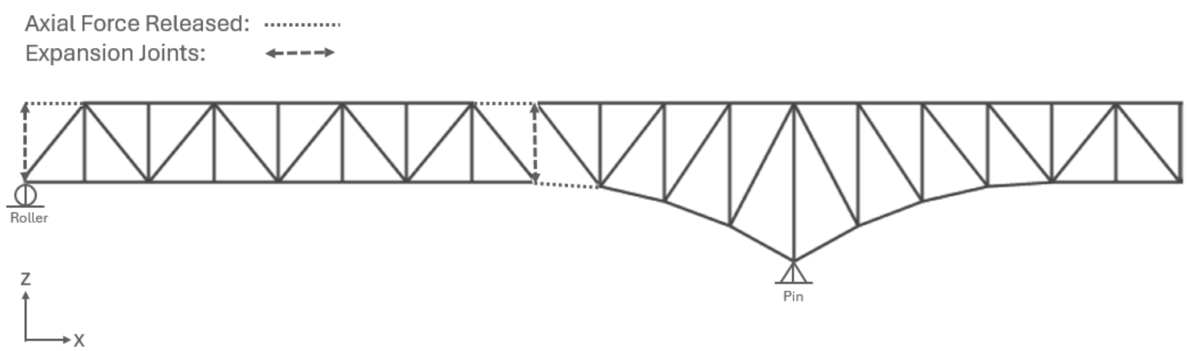

Figure A1.4.4-1—Two-dimensional model of the Tydings Bridge with boundary conditions (dotted lines indicate "released" elements; dashed lines indicate location of expansion joints).

A1.4.5—Model Loads

The reactions at the piers under dead load have been checked to prove the stability of the model. Consequently, hand calculations were done to compute the total weight of the structure. The design of this bridge was for 6-lane HS-20 truck loading and the design temperature range is from –10° F to 120° F.

A1.4.6—Model Analysis

In order to check the thermal movements, the horizontal and vertical movements of each chord were plotted. In the case of horizontal movement, the graphics show the same results (but mirrored) from the symmetrical axis of the bridge, due to a combination of dead loads and uniform temperature. Horizontal movements in the graphic of the bottom chord show maximum and minimum displacements at the two ends of the model due to the boundary condition established (two rollers). On the other hand, movements are restrained at the location of the intermediate piers (points 13, 27, 43, and 57 in the Figure A1.4.6-1). Figure A1.4.6-2 depicts the horizontal deflection of the upper chord.

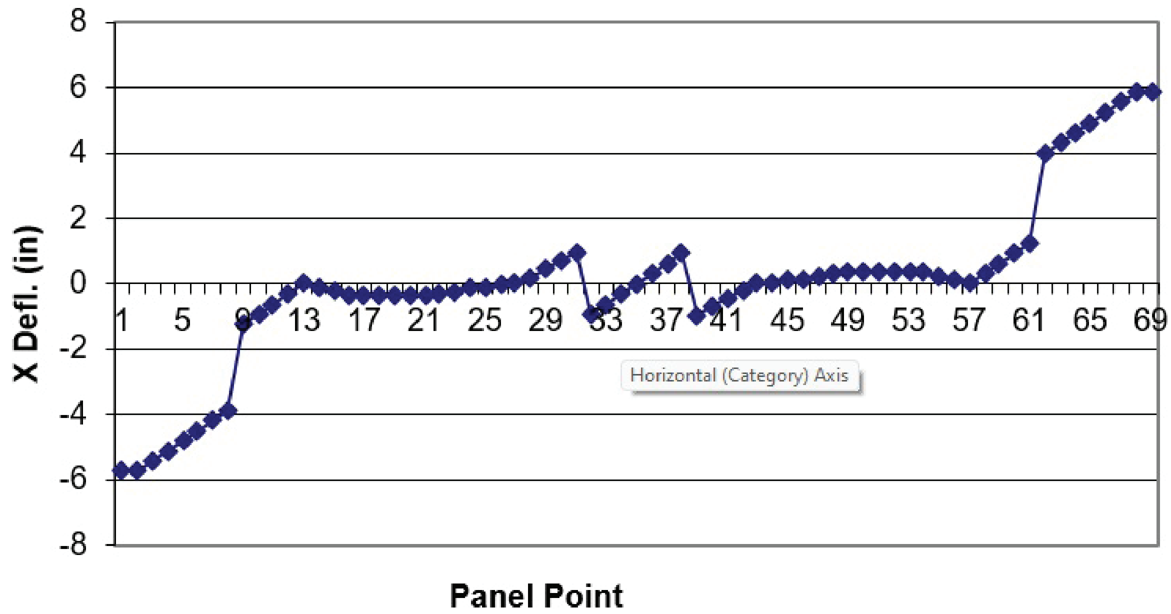

Figure A1.4.6-1—Horizontal deflection of the lower chord.

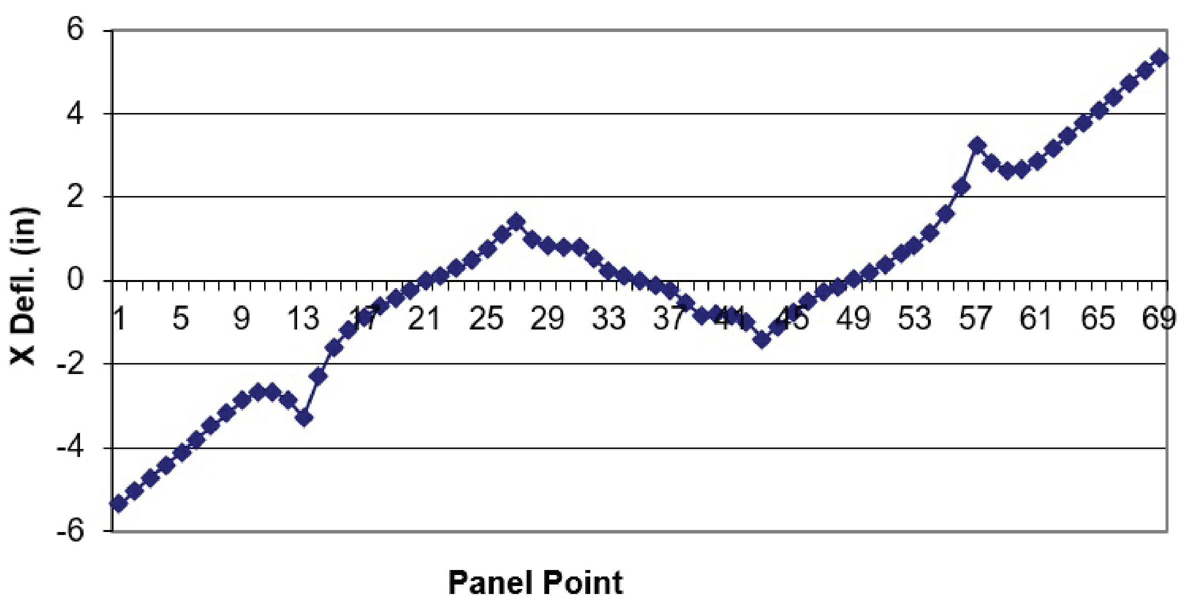

Figure A1.4.6-2—Horizontal deflection of the upper chord.

In addition, vertical movements due to a combination of dead load and uniform temperature are also symmetrical. Nonetheless, discontinuities occur at the location of the expansion joints, leading to the conclusion that angular movement is expected when temperature changes. Therefore, the assumption that sliding plates only move horizontally is not correct; the analysis of the complex movement of the bridge and the design of the sliding plates were not performed thoroughly. The following graphics show the vertical movements and discontinuities, that can be found at points 9 (245 feet) and 61 (1,837.5 feet) in Figure A1.4.6-3 and Figure A1.4.6-4, for instance.

Additionally, the vertical deflections have a minimum at the expansion joints of the first and fifth spans, and a maximum at the middle point of the anchored spans, as the pinned connections between the piers and the superstructure restrain the movement and there are not any expansion joints along the spans. Lastly, as the vertical movements in the lower chord are restrained at the piers, deflection is zero.

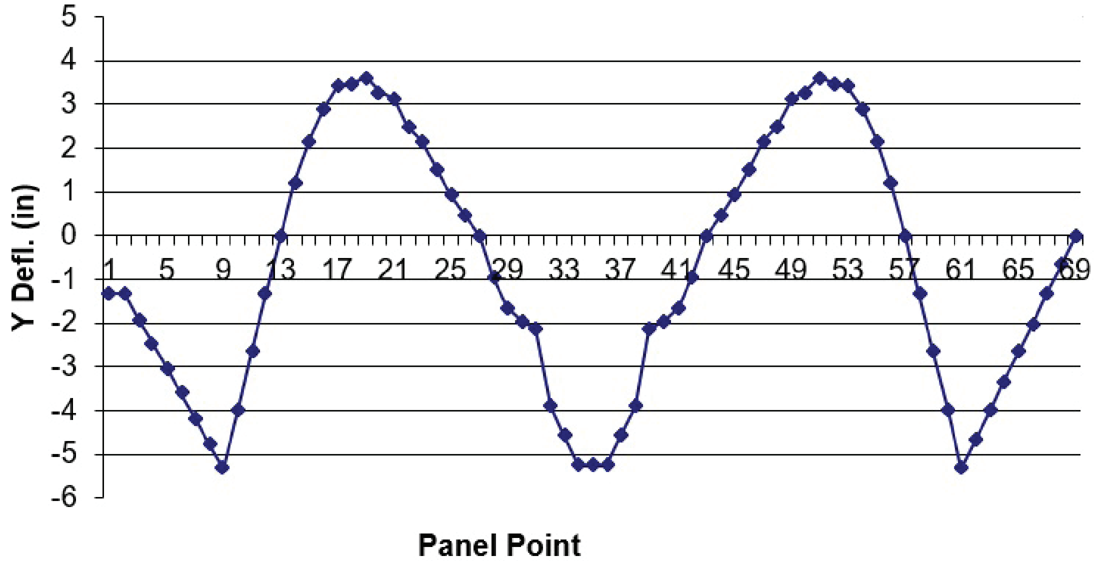

Figure A1.4.6-3—Vertical deflection of the lower chord.

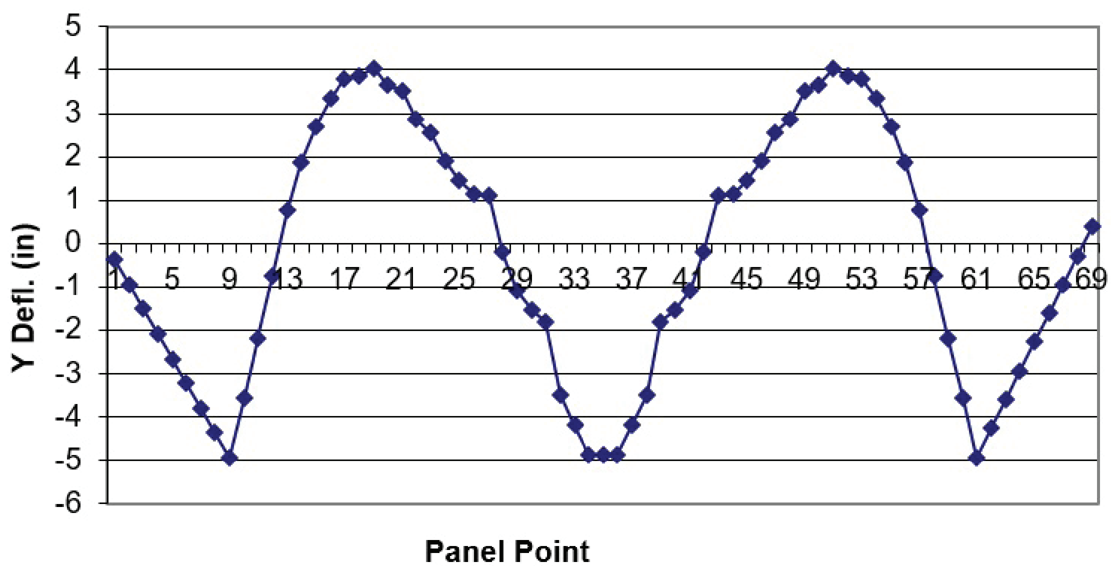

Figure A1.4.6-4—Vertical deflection of the upper chord.

In the field, the angular movement caused the formation of gaps between the top of sliding plate and the bottom of the fixed plate. Thus, the sliding plate did not totally bear against the bottom plate. The size of the gaps was affected by temperature changes, expanding when the change was positive and contracting when negative. The sliding plate, therefore, worked as a cantilever system at certain moments, generating large stresses across the longitudinal (perpendicular to the traffic) weld.

When the angular movement is combined with traffic loading and temperature changes, the expansion joint system is affected by unexpected stresses that can cause welds to crack. In the case of the Tydings Bridge, the two-dimensional finite element method model used for the thermal analysis proves the existence of the angular movement beyond the slide movement.

A2—TWO- AND THREE-DIMENSIONAL MODEL EXAMPLES

A2.1—MD 355 OVER THE MONOCACY RIVER EMERGENCY REPAIRS

This bridge is a two-span Parker through-truss steel truss bridge located in Frederick, Maryland. The bridge was built circa 1930 and previously rehabilitated in 1980. The bridge is approximately 315 feet in length and owned by the Maryland Department of Transportation State Highway Administration (MDOT-SHA). Refer to Figure A2.1-1 for an overall photo of the bridge.

A garbage truck struck the overhead sway bracing, which subsequently damaged four vertical members. The end portal was damaged and the impact to the overhead members severed the first two verticals on the east truss and irreparably damaged the first two verticals on the west truss.

The scope of the repairs was to remove and replace the damaged vertical members. The objective of the modeling was to determine if the bridge was stable (in its damaged state), and what effects the repair operation would have on the bridge. The plan was to lift the upper chord with hydraulic jacks and replace the vertical members. The jacks were mounted atop a jacking frame which was anchored into the existing bridge deck. "Sister beams" were installed to temporarily stabilize the vertical members prior to the repair operation.

Figure A2.1-1—Elevation view of the Parker Truss Bridge.

A2.1.1—Analysis Method Selection

A two-dimensional model was used as a starting point to determine conservative design loads for the four temporary sister beams. It was not used to model the overall jacking operation because the design team wanted to capture the load sharing effects of the floor system and remaining overhead bracing.

A three-dimensional model was developed in parallel to better capture any transverse load sharing as well as to evaluate the impacts of the asymmetric damage and jacking operations. Additionally, the three-dimensional model was able to more accurately model the behavior of the bottom chord, which had been partially encased in a concrete parapet during a rehabilitation in the 1980s.

The three-dimensional model was checked to verify model assumptions. A comparison of the modeling results (two-dimensional vs. three-dimensional) showed strong correlation for the as-built state, with the two-dimensional model being slightly more conservative. The deformed shape of the damaged three-dimensional model closely compared to the deformed shape as measured in the field.

A2.1.2—Model Geometry

The steel truss consisted of top and bottom chords, with transverse floorbeams and longitudinal stringers. Stringers framed into floorbeams and supported a composite reinforced concrete deck. The truss bridge also consisted of end portals, sway bracing, and overhead lateral bracing. Nodes were located at each gusset plate connection and top/bottom chord splice locations for the truss members, and at bolted angle connections for the stringer–floorbeam system. Figure A2.1.2-1 shows an elevation view of the typical truss span configuration while Figure A2.1.2-2 and Figure A2.1.2-3 demonstrate the three-dimensional deflected shape of the as-built and damaged state truss under dead load.

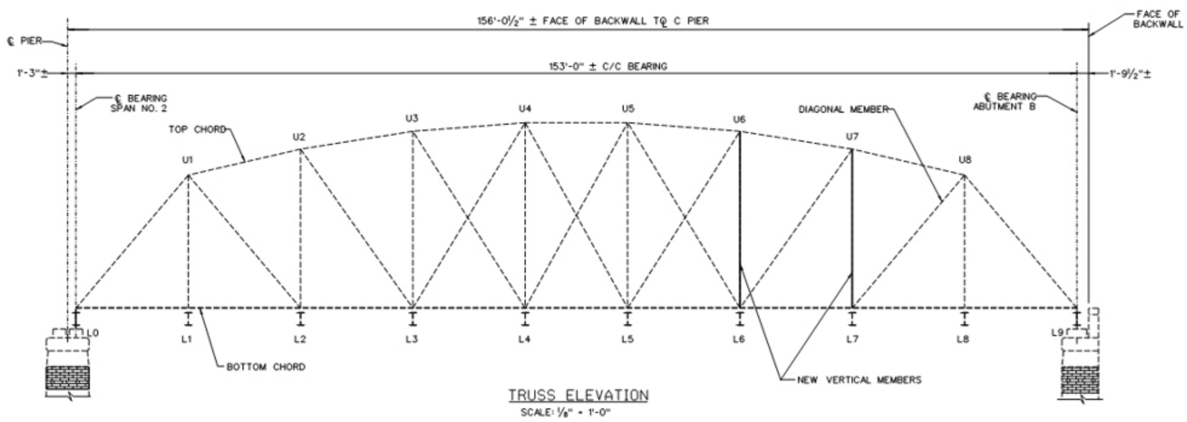

Figure A2.1.2-1—Truss elevation, Span 2.

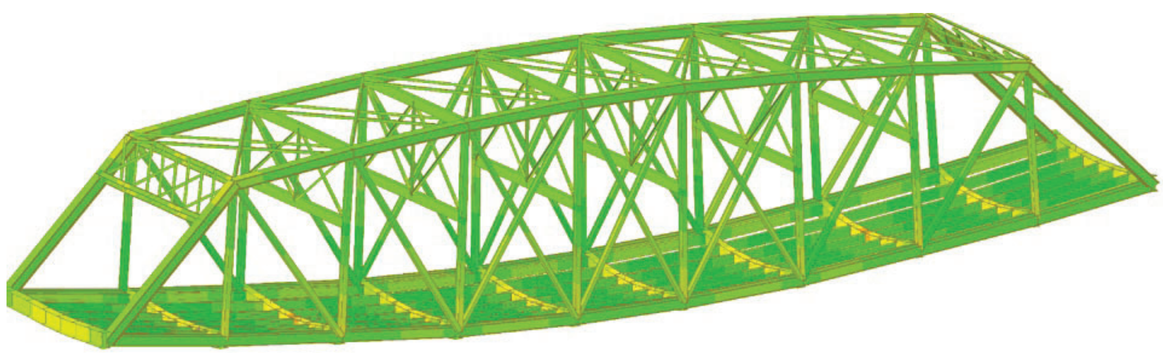

Figure A2.1.2-2—Three-dimensional truss model (as-built condition).

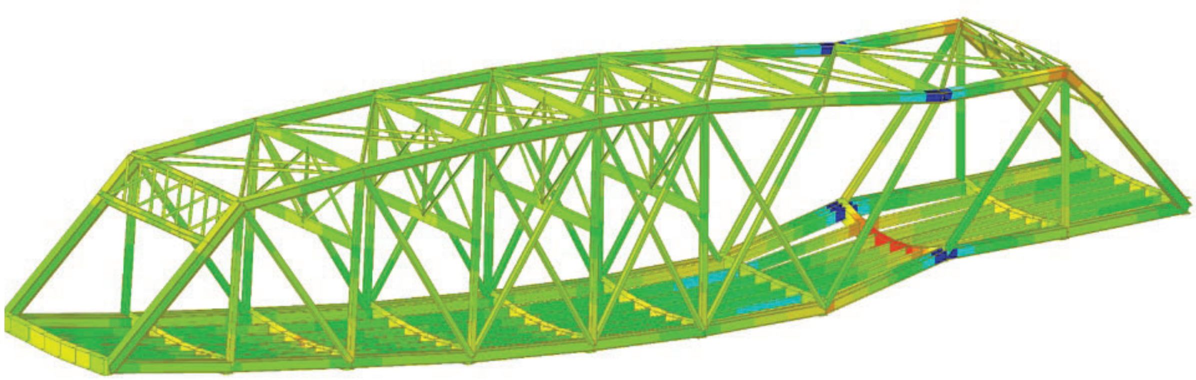

Figure A2.1.2-3—Three-dimensional truss model (damaged condition).

A2.1.3—Model Elements and Section Properties

Two-Dimensional Model (STAAD):

The two-dimensional model used all beam elements. The upper chord was input as a user-defined "prismatic section" to account for the top cover plate. All other steel sections were either double channels or Carnegie beam sections, which were selected from the program's historical shape database. The deck system (i.e., concrete, stringers, floorbeams) was not included in the model.

Three-Dimensional Model (MIDAS):

The three-dimensional model used all beam elements with composite longitudinal stringers. The floorbeams were conservatively modeled as noncomposite. Due to time constraints, the deck was not input directly as a plate element but was modeled by adding effective deck width to the stringers.

Steel sections were taken from the as-built drawings and were mostly user-defined in the model. The truss verticals, diagonals, stringers, and floorbeams were all historical steel Carnegie beam sections. The upper chord consisted of a double channel with a top cover plate and bottom batten plates, and the bottom chord consisted of double channels with top and bottom batten plates.

A2.1.4—Model Boundary Conditions

Two-Dimensional Model (STAAD):

For the two-dimensional model, the bearings were input as a pinned support at one end and a roller support at the other. All gusset plate connections were conservatively modeled as pinned connections.

Three-Dimensional Model (MIDAS):

For the three-dimensional model, the bearings were input as pinned supports at one end and roller supports at the other. There were a total of four bearings, two at each support, and it was assumed that one side of the truss was restrained from lateral movement. All other connections (stringer to floorbeam connections, gusset plate connections, top/bottom chord splices, etc.) were modeled as fixed connections.

A2.1.5—Model Loads

Two-Dimensional Model (STAAD):

For the two-dimensional model, all loads were input as point loads at each node based on tributary areas and/or the lever rule. Dead loads were based off the self-weight of all the steel members and superimposed dead load from the concrete deck and barrier. A uniform construction live load was applied along with moving concentrated loads to represent the known weights of manlifts used to install the sister beams.

Three-Dimensional Model (MIDAS):

The initial three-dimensional model was set up for the as-built case (dead load only). The self-weight of the steel members was applied in the model, and the concrete weight was applied as line loads to the stringers.

To model the repair operation, additional loads were applied to the "damaged" condition. The jacking forces were applied as point loads on the upper chord and resulting base reactions (from the jacking frame) were applied as point loads to the stringer–floorbeam system. A uniform construction live load was also applied to account for the repair crews.

For comparison with a physical live load test, two distinct maintenance vehicles were weighed and measured and modeled as moving point loads using the three different transverse vehicle placements.

A2.1.6—Model Analysis

The truss was analyzed to simulate the forces on the sister beams as well as the sequential jacking and vertical member replacement operations. The sister beams carried only construction loads before being sequentially removed and replaced by new, permanent vertical members. Jacking forces were applied eccentrically to the upper chord members with the jacking frame reactions applied to the floorbeam–stringer system. Strain gauges were installed on the bridge during jacking and replacement operations and repositioned for the live load test. Results were compared in real time to the model predictions.

A2.2—POINT MARION BRIDGE

This bridge is a 412-foot, 6-inch simple-span park-through truss located in Fayette and Green Counties, Pennsylvania. The structure was built in 2010 for the Pennsylvania Department of Transportation. The floor system is composed of floorbeams with fully bolted end connections, stringers framed into the floorbeams, and a concrete-filled grid deck. Main truss and bracing members are a combination of welded box and I-shapes except for the bottom chord, which is a bolted built-up box shape to provide internal redundancy for the tension member. Portal frames are located at the end posts and a sway frame is provided at the midspan vertical. Refer to Figure A2.2-1 for an overall photo of the bridge.

Figure A2.2-1—Point Marion Bridge, PA.

A2.2.1—Analysis Method Selection

A three-dimensional model was created of the truss to design members for dead, live, thermal, and wind loads. A three-dimensional model was chosen to accurately determine the participatory loads in the top and bottom lateral bracing due to applied vertical dead and live loads as well as lateral wind forces. To ensure compliance with the Owner's load rating software for trusses, a two-dimensional model was also created to determine main member dead and live load forces, which may be higher than those obtained from a three-dimensional model, which gains the benefit of bracing, floor system, and deck elements partially resisting primary loads in addition to the top and bottom chord members.

A2.2.2—Model Geometry

Nodes were located at the intersection of main truss member work lines. Floor system and bottom lateral bracing members were offset from these work points to their correct neutral axis locations. All main members, bracing members, floor system members, and the deck were included in the model. Figure A2.2.2-1 provides a screenshot of the three-dimensional model without the deck elements shown. The substructure units were not included in the model.

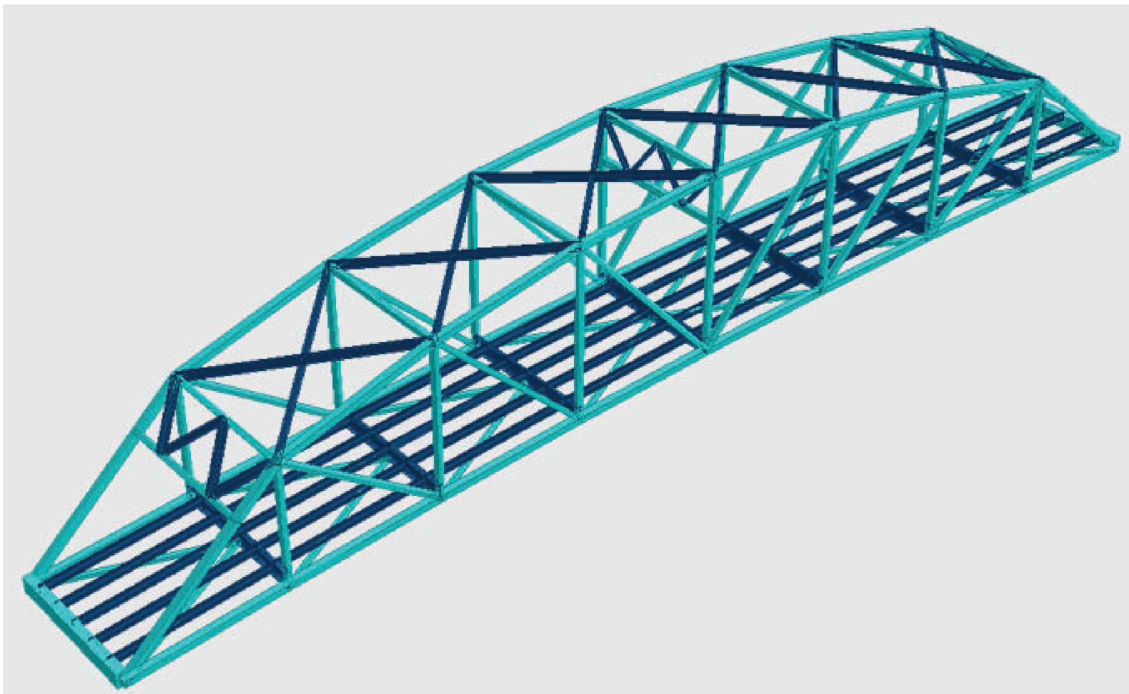

Figure A2.2.2-1—Three-dimensional truss model.

A2.2.3—Model Elements and Section Properties

Truss, bracing, and floor system members were modeled as beam elements. The deck was modeled with shell elements. Intermittent manholes and access holes were considered for member adjusted gross and net section properties to determine member capacities; however, member section properties specified in the analysis model were based on full gross section properties to obtain accurate forces and camber values. For the built-up bottom chord, comprised of rectangular plates for the flanges and webs and structural angles at the corners, the total area of the beam element in the model was based on the total area including the plates and angles. For the concrete-filled grid deck, an equivalent thickness and material properties were applied to the shell elements to obtain the proper stiffness.

A2.2.4—Model Boundary Conditions

All main, bracing, and floor system members were modeled as beam elements, except for the bottom lateral bracing, which were defined as truss members due to end-fixity and slenderness. Appropriate end-releases were applied to stringer member ends at their connections to the floorbeams. All other member ends were restrained for moment based on the connections as detailed. Gusset and connection elements were not explicitly modeled.

The member end-fixity in the three-dimensional model captured the frame action of the floorbeam connection to the truss vertical, the connection of the end (box) floorbeam to the bottom/top truss chord knuckle, and the portal and sway frame bottom strut connection to the upper chord end post and vertical, respectively. It could be seen that the resulting moment fixity between the floorbeam and vertical members varied along the span, reducing towards midspan where the torsional restraint of the bottom chord and moment restraint of the longer truss verticals was reduced. For all other connection locations, even though the members were defined as beam elements with moment fixity, the resulting member moments were negligible as expected due to member slenderness. The two-dimensional model using the Owner's rating software employed truss (axial-only) elements for all truss members.

Bearing fixity was based on the articulation specified in the design plans. Because the structure is simple span with a fixed and expansion support, the bearings were attached to rigid ground instead of modeling the piers (or their equivalent stiffness) explicitly in the model. Figure A2.2.4-1 shows a schematic of lateral wind load bending moments and bearing fixity at the expansion end of the truss.

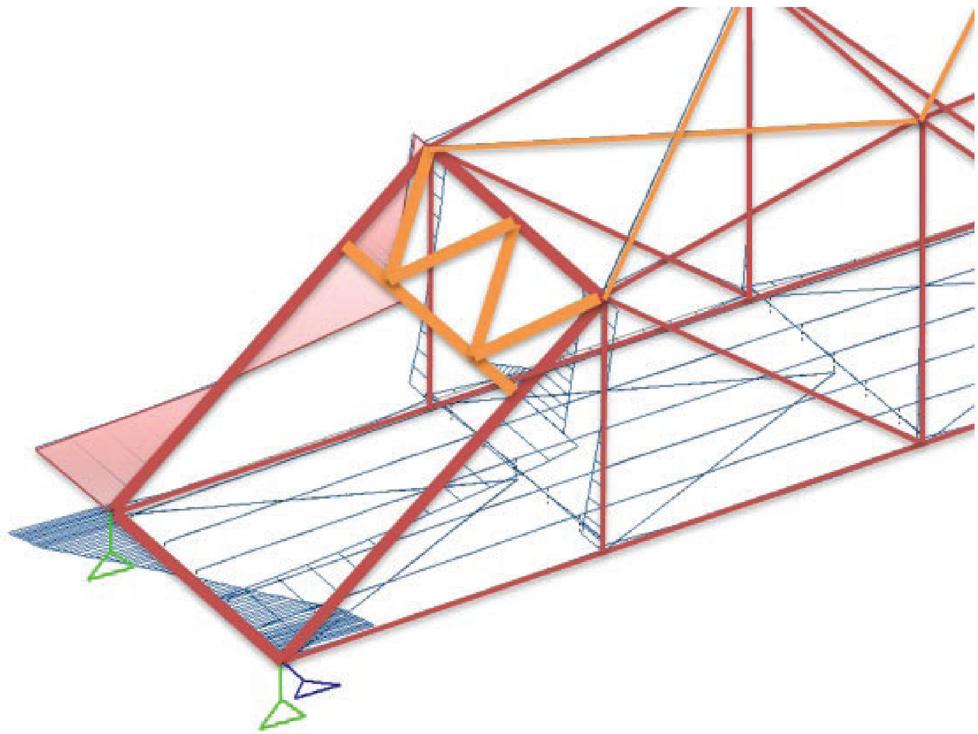

Figure A2.2.4-1—Member moments due to wind loading.

A2.2.5—Model Loads

Dead, pedestrian, live, thermal, and wind loads were applied to the three-dimensional truss. Steel dead load was applied using the self-weight of the member in the model with panel point loads to account for detail weight (gusset plates, etc.). The weight of the deck was applied as a line load to the stringers based on tributary area. Barrier and future wearing surface loads were equally distributed to all stringers and applied as line loads as well. Live loads (HS-20 and permit truck) were applied as equivalent wheel loads to the stringers along the length of the structure. Equivalent pedestrian loads were applied at floorbeam locations and enveloped. Wind loads—lateral, longitudinal, and upward—were applied as line loads to the appropriate truss and floor system members. Only dead and live load forces were applied to the two-dimensional model for computing ratings. Loads were applied as concentrated loads to panel points.

A2.2.6—Model Analysis

The main three-dimensional model was analyzed as a linear static analysis. Secondary effects in compression members were accounted for using AASHTO's approximate method for moment magnification. In addition to the main design and two-dimensional rating analysis, a modified version of the three-dimensional model was created to analyze the proposed erection method shown in the design plans. The example erection scheme assumed a single falsework tower located in the river two panel points away from one pier (approximately 100 feet). The truss would be stick-built using cantilever construction from this pier, over the falsework tower, to the opposite pier. The erection model was used to check member erection forces, which can be higher and of opposite sign than final design forces. The uplift forces at the first pier due to the cantilever construction were also determined, ensuring the end floorbeam and pier could resist these forces using the schematic tie-downs shown in the plans. The actual erection scheme used by the Contractor was very similar. A photo of the cantilevered truss during erection is shown in Figure A2.2.6-1.

Figure A2.2.6-1—Cantilever truss during erection.

A3—THREE-DIMENSIONAL MODEL EXAMPLES

A3.1—HURRICANE BRIDGE PROJECT

This bridge is a four-span Warren deck truss located in DeKalb County, Tennessee. It was built in 1949 by the Army Corps of Engineers and rehabilitated in 1977 and 2011. The bridge is approximately 1,800 feet in length and owned by the Tennessee Department of Transportation (TDOT). Refer to Figure A3.1-1 for an overall photo of the bridge.

Figure A3.1-1—Hurricane Bridge photograph.

A3.1.1—Analysis Method Selection

A three-dimensional model was created of the entire bridge (superstructure and substructure) as part of a study for the National Science Foundation. The research explored a novel temperature-driven (TD) approach for detailed evaluation and monitoring of a structural system. At the heart of the TD methodology is the idea that temperature variations can be treated as a measurable "loading" of the structure and thus be used to obtain a complete input-output relationship. This is achieved through instrumentation of both the critical members and movement mechanisms (expansion bearings, joints, etc.).

For the Hurricane Bridge, each instrumentation location included sensing for temperature measurement (input) along with measurement of mechanical strains, displacements, and rotations (output). These measured relationships associated with member forces and movement mechanisms allowed for the identification of a unique signature (or baseline) of the structure. The signature was used within the structural identification process, which included model calibration using the field measured data. This allowed for the determination of various structural parameters (e.g., behavior of the bearings and the pin-and-hanger connections) and evaluation of the structural performance under different scenarios.

A two-dimensional model was not used because of the comprehensive nature of the research study. The out-of-plane behavior of the truss due to local and global thermal gradients was explored.

A3.1.2—Model Geometry

The original as-built plans and rehabilitation drawings were used to model the geometry of the structure. For the truss system, the nodal locations were placed at the intersection of the lines of action between the truss members. The nodal locations for the stringers and floorbeams were placed at their center of gravity and member offsets were used when necessary. In addition, the nodes for the deck were placed at mid-height. Figure A3.1.2-1 provides an illustration of the three-dimensional model.

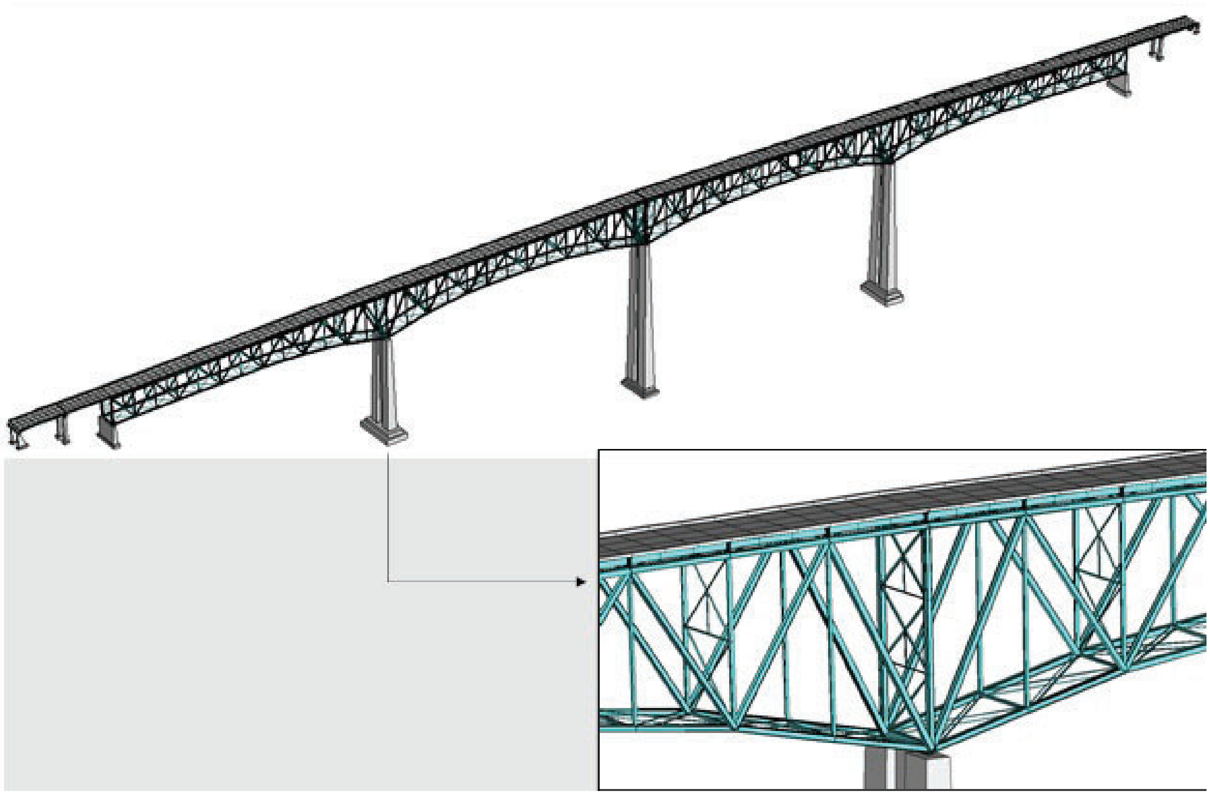

Figure A3.1.2-1—Three-dimensional truss model for the Hurricane Bridge.

A3.1.3—Model Elements and Section Properties

The primary steel members (truss, lateral bracing, stringer, floorbeam, etc.) were modeled using beam elements. The section properties were automatically calculated by the finite element software program. To ensure proper element orientation, a three-dimensional graphical review of all members was performed. The deck was modeled with shell elements at the appropriate offset location. The bearings and other bridge components that allowed movement were modeled with connection elements. This allowed for modification of their stiffness during the model calibration process.

Note that the substructures were also modeled. Two approaches were explored, which included three-dimensional solid elements and beam elements (using a tapered feature). The results were relatively consistent. Consequently, the beam elements were selected due to substantially less processing time, which is important when the model was analyzed thousands of times during the model calibration process.

A3.1.4—Model Boundary and Continuity Conditions

The boundary conditions of the model included connection elements at the abutment bearings and fixed connections at the base of the piers. It was decided that the piers would be modeled because of their flexibility and influence over the thermal behavior of the model. As mentioned earlier, beam elements were used for the pier sections.

The continuity conditions were mainly modeled with connection elements. This was to allow for easier modification (or "tuning") of the model during the calibration process. Some of the locations included the bearings at the top of the piers and the connection between the truss and floor system. The pin-and-hanger detail used member end-releases for the beam elements. The software had the capability to simulate partial fixity end restraints in all degrees of freedom.

A3.1.5—Model Loads

The *a priori* model was first analyzed under self-weight for general quality control checks and later used for general assessment of the structural behavior. To fully capture the self-weight, the components not included were either modeled as a concentrated mass (e.g., gusset plate connections) or modeled with element mass modifiers.

Since the primary component of the study was to research the TD structural evaluation concept, temperature changes were applied to the model. The magnitudes of these thermal "loads" were those measured from the field experiment.

After the model was calibrated, additional analysis was performed that included general live-load modeling of conventional AASHTO vehicles (as defined in Section 3 of *AASHTO LRFD BDS*).

A3.1.5.1—Model Analysis

The model was mostly analyzed using linear static analysis to determine the dead load, live load, and thermal demands. Natural frequency analysis was also performed to identify the fundamental frequencies for the first few vibration modes. In the end, the calibrated three-dimensional model was analyzed to perform conventional load ratings and evaluate the performance of the movement systems (bearings and pin-and-hanger assemblies).

A3.2—RHODE ISLAND AVENUE PEDESTRIAN BRIDGE PROJECT

This pedestrian bridge is a tubular span truss located at the Rhode Island Avenue Metro Station in Washington, D.C. The structure spans CSX Railroad and connects the Metropolitan Branch Trail with the Rhode Island Avenue Metro Station. The bridge was built in 2015 with a length of 200 feet. It is owned by the District Department of Transportation (DDOT). Refer to Figure A3.2-1 for an overall photo of the bridge.

Figure A3.2-1—Rhode Island Avenue Pedestrian Bridge, Washington, D.C.

A3.2.1—Analysis Method Selection

A three-dimensional model was created to design the pedestrian bridge. The truss member demands from dead load, pedestrian live load, maintenance vehicle live load, wind load, thermal load, and snow load were obtained from the model. A two-dimensional model was considered. However, the increased accuracy of the results justified the minimal additional modeling effort. In addition, the three-dimensional model provides the truss member demands for the lateral bracing members.

A3.2.2—Model Geometry

For the truss system, the nodal locations were placed at the intersection of the lines of action between the truss members. The nodal locations for the stringers were placed at their center of gravity and member offsets were used when necessary. Figure A3.2.2-1 provides an illustration of the three-dimensional model.

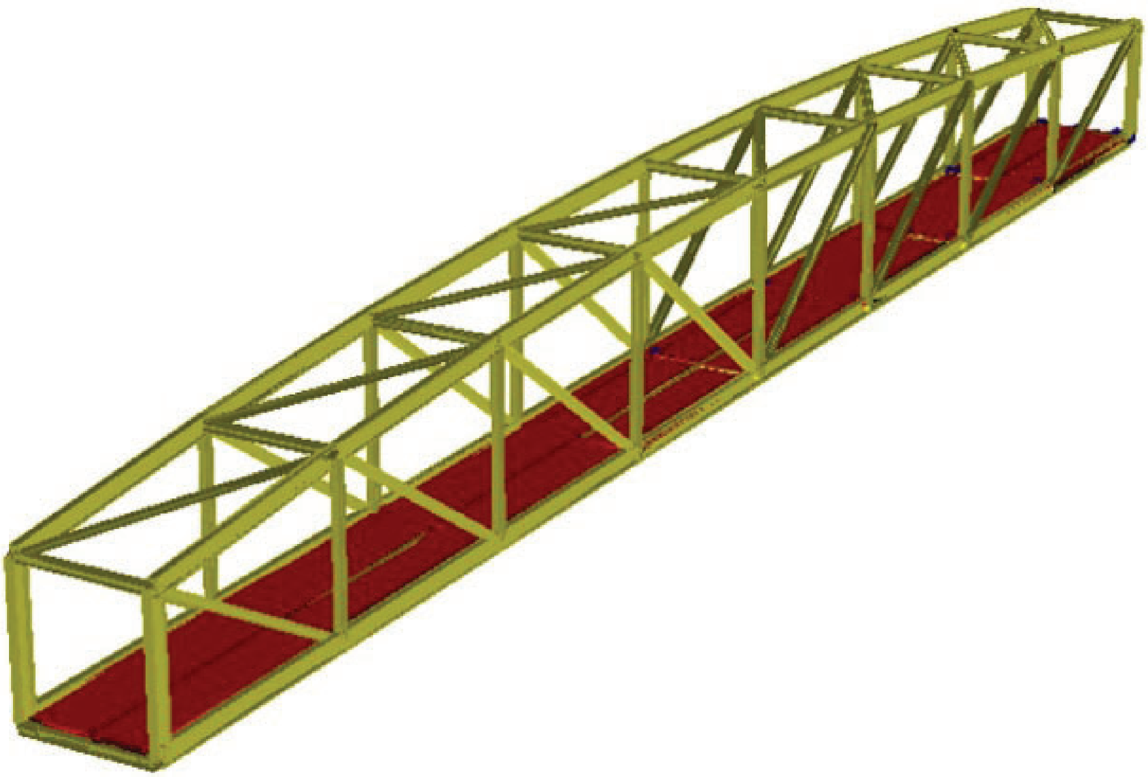

Figure A3.2.2-1—Three-dimensional truss model for the Rhode Island Avenue Pedestrian Bridge.

A3.2.2.1—Model Elements and Section Properties

The primary steel members (truss, lateral bracing, and stringers) were modeled using beam elements. The section properties were available within the finite element software program (all HSS sections were used). Plate elements were used to model the deck and were offset at the appropriate location.

A3.2.2.2—Model Boundary and Continuity Conditions

Rigid links were used to connect the corners of the truss lower chords to the bearing locations. Then the appropriate nodal restraints were used to simulate the bearings. The truss member connections were all fully welded so everything was modeled as rigid (no member end-releases).

A3.2.2.3—Model Loads

The model was analyzed under dead load, pedestrian live load, maintenance vehicle live load, wind load, thermal load, and snow load. Additional dead load was applied for miscellaneous nonstructural components (e.g., railing).

A3.2.2.4—Model Analysis

The truss was analyzed using linear static analysis to determine all the truss member forces. These forces were used to iteratively design the truss members. The truss forces were used for the connection designs. In addition, the reactions were used for the bearing and pier designs.

A3.3—MILLARD TYDINGS BRIDGE PROJECT

This bridge is a cantilever steel truss located in Perryville, Maryland, and spans the Susquehanna River. The bridge was originally finished in 1963 with an approximate length of 5,000 feet (maximum span of 490 feet). It is owned by the Maryland Transportation Authority (MDTA). Refer to Figure A3.3-1 for an overall photo of the bridge.

Figure A3.3-1—Photo of Millard Tydings Bridge, Perryville, MD.

A3.3.1—Analysis Method Selection

A three-dimensional model was created for a detailed evaluation of the structural system. This evaluation would later include model calibration with field data from a load test. The primary objectives of the study were to:

- Identify the floor system and truss force distribution,
- Evaluate the boundary and continuity conditions,
- Establish the nominal stresses at fatigue prone details,
- Update the live load ratings, gusset plate analysis, and fatigue life estimate.

A two-dimensional model was not used because of the complexity of the structure.

A3.3.2—Model Geometry

The original as-built plans were used to model the geometry of the structure. For the truss system, the nodal locations were placed at the intersection of the lines of action between the truss members. The nodal locations for the stringers and floorbeams were placed at their center of gravity, and member offsets were used when necessary. In addition, the nodes for the deck were placed at mid-height. Figure A3.3.2-1 provides an illustration of the three-dimensional model.

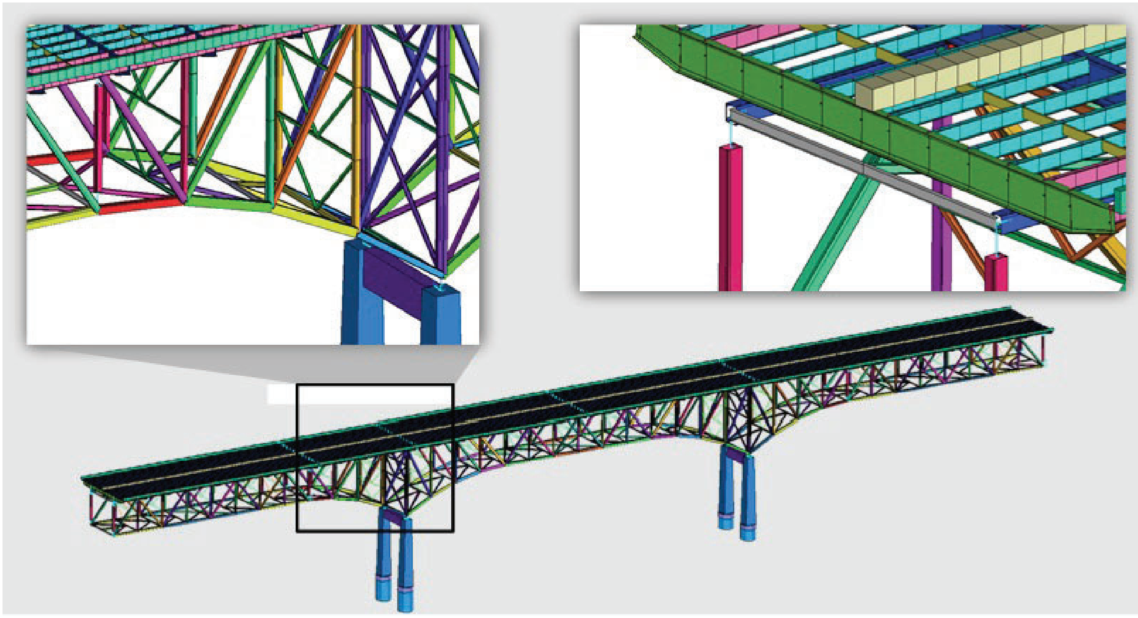

Figure A3.3.2-1—Three-dimensional truss model for the Millard Tydings Bridge.

A3.3.3—Model Elements and Section Properties

The primary steel members (truss, lateral bracing, stringer, floorbeam, etc.) were modeled using beam elements. The section properties were automatically calculated by the finite element software program. In order to accurately capture the truss member mass and stiffness due to the member cut-outs, a refined three-dimensional solid model was created (see Figure A3.3.3-1). This model was used to obtain mass and stiffness modifiers that were input into the overall model.

Figure A3.3.3-1—High-resolution three-dimensional solid model of a single truss member with cut-outs.

The deck was modeled with shell elements at the appropriate offset location. The bearings and other bridge components that allowed movement were modeled with connection elements. This allowed for modification of their stiffness during the model calibration process. Note that the substructures were also modeled with beam elements.

A3.3.4—Model Boundary and Continuity Conditions

The boundaries of the model were the limits of the superstructure and the base of the piers. The ends of the superstructure were modeled with spring elements to simulate the adjoining spans. The bases of the piers were assigned fixed connections. It was decided to model the piers due to their flexibility and influence over the global structural behavior. As mentioned earlier, beam elements were used for the pier sections.

The continuity conditions were mainly modeled with connection elements. This was to allow for easier modification (or “tuning”) of the model during the calibration process. Some of the locations included the bearings at the top of the piers and the connection between the truss and floor system. The pin-and-hanger detail used member end-releases for the beam elements. The software had the capability to simulate partial fixity end restraints in all degrees of freedom.

A3.3.5—Model Loads

The model was analyzed under self-weight and live loading. To fully capture the self-weight, the components not included were either modeled as a concentrated mass (e.g., gusset plate connections), or mass modifiers were applied to the elements. The live loads from the truck load test were input into the model. After the model was calibrated, additional analysis was performed that included general live load modeling of conventional AASHTO vehicles.

A3.3.6—Model Analysis

The truss was primarily analyzed using linear static analysis to determine the dead load and live load demands. The calibrated three-dimensional model was used to run different simulations that determined:

- Nominal stress levels at fatigue prone details
- Floorbeam shear stresses (originally a concern by the Owner)
- Stringer composite action with the deck
- Truss expansion bearings movement under live loading
- “Dummy” truss members contribution to truss
- Floor system to truss level of connectivity.

A3.4—PEDESTRIAN BRIDGE THROUGH TRUSS

The example bridge (Figure A3.4-1) was based on a pedestrian bridge on the A38 near Weeford in the UK, constructed in 2008. U.S. dimensions, sections, and materials were adopted for a study in 2018–2019, which included comparison of member resistance calculations using the Eurocode (2005) and AASHTO (2017c).

The example bridge is a simply supported single span of 144 feet, with two parallel 11-foot-deep Warren trusses of 12 equal bays, positioned at 10-foot centers. Top and bottom booms are rectangular HSS, truss diagonals are round HSS, floorbeams are I-sections and roof bracing members round HSS.

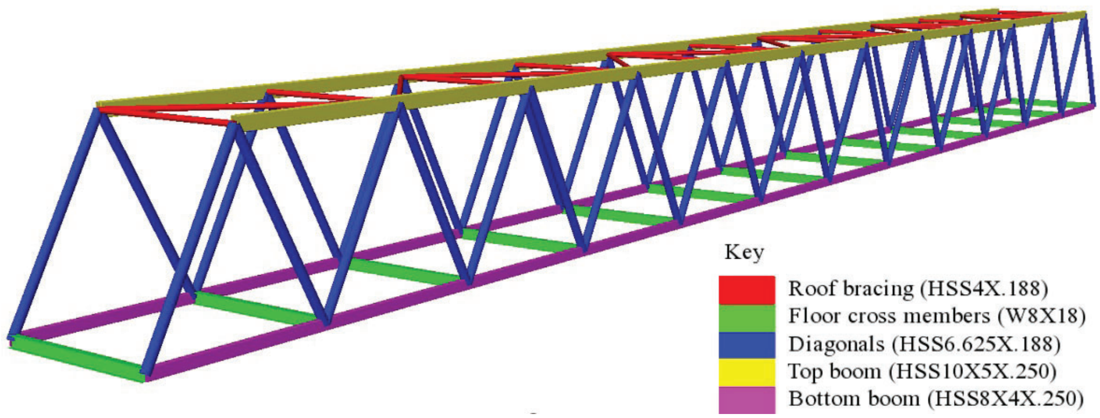

Figure A3.4-1—Pedestrian bridge model of bridge in Weeford, UK.

A3.4.1—Analysis Method Selection

A three-dimensional model was created of the truss to examine load effects (and thereby design requirements) under simple load cases. A two-dimensional model was not used for several reasons:

a. The design load effects should include second-order (geometrically nonlinear or "large displacement") effects. While AASHTO (2017c) offers some simple moment magnification formulae, the Eurocode (2005) does not and so a second-order analysis is really required, including imperfections. This can only be carried out using a three-dimensional model.

b. Floor member (W-section) analysis was required. While a simple two-dimensional line beam could have been used to model one such beam, it was of interest to investigate the possible treatment these cross-members as having a moment-carrying connection to the main truss members (see Article A3.4.4).

c. Design checks were being carried out by the software and omitting members such as roof members would mean that they were excluded from the design checking. Hence a model which included all members was needed.

A3.4.2—Model Geometry

Nodes were located at the intersection of member centerlines. The model included all components of the truss frame (there are no gusset plates) but did not include the deck (precast concrete components).

See Figure A3.4-1 for the components that were included in the model.

Load effects are obtained at 11 internal locations along each beam element in LUSAS, and design check calculations are carried out at each of these locations.

A3.4.3—Model Elements and Section Properties

The bridge was modeled using thick (Timoshenko) beam elements using LUSAS v18.0, since the use of truss (bar) elements excludes member effects such as flexure and torsion.

Top and bottom booms are rectangular HSS, HSS10×5×¼ and HSS8×4×¼, respectively. Floorbeams are I-sections, W8×18. All other members are round HSS (main diagonals HSS6.625×0.188 and roof bracing members HSS4×0.188).

Steel grade is ASTM A500 Grade C for all hollow sections and AASHTO M 270M/M 270 Grade 50 for the W-beams.

A3.4.4—Model Boundary Conditions

The example truss was initially modeled with simply supported articulation: pinned at one bearing and guided/free so as to allow free expansion and contraction.

Errors in support conditions are some of the most common idealization problems identified in bridge models received by the LUSAS Support Team. A common error in a bridge such as this is to define all bearings as pinned (fixed in all translations).

Even considering just a first-order analysis, such a change to the support conditions alters behavior of the structure considerably. The tension in the bottom boom is significantly modified and instead of utilizations around 80 percent near midspan and 10 percent near the supports; midspan utilizations dropped to about 20 percent and end utilizations were pushed up to greater than 90 percent.

Regarding member-to-member connections, the "continuous" top and bottom booms were idealized with moment-carrying connection from one bay to the next. All truss diagonals, bracing members, and floorbeams were initially assumed to be pinned at their connections to the booms. If these connections are modified to be moment-carrying, utilizations are considerably affected.

Floorbeams W8×18 (I-section) would see a *decrease* in utilization from 50 percent to 38 percent. Roof members would also see a *decrease* in utilization on the order of ¼ to ½. End diagonals (round HSS, HSS6.625×0.188) would see an *increase* in utilization from 66 percent to 77 percent.

It was assumed that the precast deck components provide no lateral restraint to the floorbeams or bottom boom of the truss. While in reality any decking system will provide some restraint in at least one horizontal direction, this assumption retained lateral torsional buckling as a design criterion which suited the purposes of the study in comparing Eurocode (EN1993-2) and AASHTO (2017c) design criteria.

A3.4.5—Model Loads

In addition to the self-weight of the truss members, a uniform dead load of magnitude 1.8 kip/ft was applied to the floorbeams, representing 0.05 ksf superimposed dead load and 0.1 ksf live load. Load factors, which differ from code to code, were omitted in this study, which focused on the differences in member resistance between AASHTO (2017c) and Eurocode (EN1993-2).

A3.4.6—Model Analysis

There was no consideration of staged construction required for this structure.

A linear elastic (first-order) analysis was used with AASHTO (2017c) for which amplification factors are available (Article 4.5.3.2.2b). The Eurocode offers no such approximations, leading the Engineer towards a second-order analysis. The co-rotational formulation for geometrically nonlinear analysis is available in LUSAS for the beam elements used in this truss, and so this was the approach taken when considering Eurocode member utilizations.

A3.4.7—Selected Conclusions of Comparative Study (AASHTO vs. Eurocode)

The AASHTO (2017c) calculations were found to be considerably more concise than those of the Eurocode. The study highlighted a range of differences but the principal issues which might be worthy of further consideration for those using AASHTO (2017c) for truss checks would be:

1. Compression check.
    a. AASHTO (2017c) compression member utilizations were *unconservative* by comparison to the Eurocode. They are based on a single buckling curve with an assumed out of straightness of span/1,500. The Eurocode approach uses an imperfection factor selected according to section shape, limiting thicknesses and grade of steel.
    b. AASHTO (2017c) limiting slenderness ratios or proportion limits preclude the use of some sections that are allowed in the Eurocode and found (by Eurocode checks) to have low utilizations. Can such slenderness limits be overlooked in certain cases? They are similar to those given in ANSI/AISC 360-16 wherein the commentary indicates that they may be related to construction economics, ease of handling, and minimizing inadvertent damage. Hence, in an existing truss, perhaps they can be deemed acceptable and checked for strength in the usual way.

2. Shear check. The more nuanced approach to the determination of shear areas in the Eurocode may produce a more realistic (*less* conservative) value for shear resistance.

3. Interaction checks.

    a. Eurocode provides rules for interaction of bending with shear (EN1993-1-1 clause 6.2.8), and bending, shear, and axial force (EN1993-1-1 clause 6.2.10), for which there are no comparable Articles in AASHTO.

    b. Bending and axial compression (buckling) checks are more straightforward in AASHTO but may be more prone to conservatism as compared to the Eurocode (EN1993-1-1 clause 6.3.3).

A3.5—WINONA BRIDGE REHABILITATION

The Winona Bridge (Minnesota Bridge 5900) carries Trunk Highway 43 across the Mississippi River in Winona, MN (see Figure A3.5-1). The main spans are a three-span historic cantilever through-truss completed in 1942, with a maximum span length of 450 feet. Having served Winona since just after the entry of the United States into World War II, the Winona Bridge had become a beloved historic landmark to the city. Since a prior rehabilitation in the mid-1980s, it had also become significantly deteriorated, with pack rust accumulating on the portions of the truss below the level of the roadway. Maximum truck weights on the bridge had to be restricted through load posting due to increasing deterioration of the deck truss approaches. In consultation with the community, the Minnesota Department of Transportation (MnDOT) considered both rehabilitation and replacement of the bridge, and ultimately elected to construct a new parallel bridge for inbound traffic from Wisconsin, while rehabilitating the Winona Bridge for two lanes of outbound traffic.

The bridge rehabilitation design criteria included the removal of the load posting, and the structure having the capacity to support the same design, legal, and permit loadings as a new bridge. Additionally, each truss tension member would be analyzed for internal redundancy, and any tension member determined not to be internally redundant would be retrofitted to become internally redundant.

Figure A3.5-1—Winona Bridge 5900 through-truss bridge in Winona, MN.

A3.5.1—Analysis Method Selection

To analyze the Winona Bridge through truss, two-dimensional and three-dimensional finite element models were built using MIDAS Civil software. The design team started by developing a two-dimensional finite element model of the truss members because it was assumed that the two-dimensional model would produce controlling forces. The lever rule was used to calculate distribution factors for various live load positions in order to determine how much of the loading would be distributed to each truss. Additionally, a two-dimensional model was desirable because it allows for the truss to be easily load-rated in the future.

A three-dimensional model was then developed in order to analyze the floor system of the bridge, which included the floorbeams, stringers, and connections (Figure A3.5.1-1). Removal of the intermediate expansion joints during the 1985 deck replacement caused the stringers to act in parallel with the truss bottom chord members, causing additional forces in the stringers and in floorbeams near expansion joints. A three-dimensional model helped to capture this behavior.

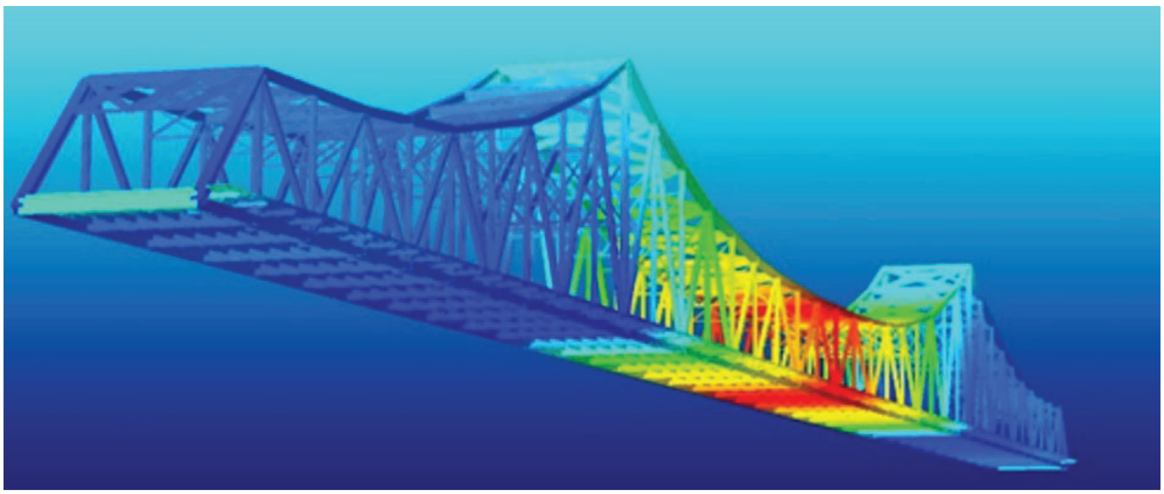

Figure A3.5.1-1—Three-dimensional finite element analysis model of the Winona Bridge through truss.

Forces from the two-dimensional and three-dimensional models for the truss members were compared, and it was confirmed that the two-dimensional results were controlling for the main truss members and would be used in the load rating. To perform the load rating of the Winona Bridge truss members, a Microsoft Excel spreadsheet was developed that incorporated checks of noncomposite built-up box sections for in-plane bending, out-of-plane bending, and axial force. When evaluating the truss members at the strength limit state, the members were modeled with pinned ends, allowing rotation but not translation. An external spreadsheet was used to account for additional primary moments caused by connection eccentricity, or the truss working line not coinciding with the member centroids. These primary moments are necessary to allow the structure to remain in equilibrium while carrying load. It has been shown that secondary moments typically do not reduce truss member capacity at the strength limit state, making it acceptable to model the truss members with pinned ends. However, at the fatigue limit state, the truss members were modeled with rigid, fixed ends to capture both primary and secondary moments, which contribute to the total stress range experienced by each member.

A3.5.2—Design Criteria

The design criteria included specifying that each member of the through truss have design LRFR inventory ratings above 1.00 for the HL-93 loading, including a 110-percent HL-93 double-truck loading for regions of the truss in which the upper chord is in tension. (These regions are analogous to negative moment regions of a girder bridge). The rehabilitation design followed LRFR methodology, and through consultation with the Owner, the selected system factors were 0.95 for truss members and gusset plates, 0.85 for floorbeams, and 1.00 for stringers. Selected condition factors were 0.85 for inspected members with heavy pack rust, 0.95 for inspected members with moderate or light pack rust, and 1.00 for inspected members with measured section loss or no deterioration.

Truss member capacity was determined using *AASHTO LRFD BDS* provisions for axial tension and compression. Following *AASHTO LRFD BDS,* Article C6.9.4.3.1-1, the slenderness ratio was modified for compression members with lacing to incorporate the shearing effect. For compression members with lacing or batten plates, compound buckling was considered when slenderness ratios of the individual built-up components exceeded 75 percent of the global slenderness ratio, following *AASHTO LRFD BDS*, Article C6.9.4.3.1. Additionally, local buckling was considered for slender member elements following *AASHTO LRFD BDS*, Article 6.9.4.2.2. The impacts of the shearing effect and compound buckling considerations to overall member resistance were typically minimal except for cases of significant member deterioration and for internal redundancy design described as follows.

A3.5.3—Design for Internal Redundancy

One of the most critical aspects of the rehabilitation design was to address Minnesota legal requirements concerning fracture critical steel tension members. These so-called Chapter 152 requirements were instituted by the Minnesota state government in the aftermath of the 2007 collapse of the I-35W Mississippi River Bridge. The design team quickly determined that providing load path redundancy to the truss was not feasible. Adding an additional truss line, or any other independent structural system to supplement the existing through trusses, would be cost-prohibitive and would irreparably mar the historic character of the bridge. Instead, the design team took advantage of the cross-sectional dimensions of the existing through-truss tension members to add internal redundancy.

At the time of analysis, the Federal Highway Administration considered tension members with internal redundancy to be fracture critical. A member with internal redundancy is designed to remain intact if any one component of the cross-section fractures. Following this definition, a tension member consisting of a single rolled shape, for example, cannot be internally redundant. A member consisting of two or more steel pieces that are connected together *can* be internally redundant, provided that the section possesses sufficient resistance to carry load in the unlikely event of the fracture of any single component of the cross-section. For the Winona through truss, providing internal redundancy to the tension members would satisfy Chapter 152 requirements by preventing collapse in the event of a fracture, together with documenting why providing load path redundancy was not feasible.

The existing deck trusses were found to be beyond repair. To maintain the historic character of the river crossing as a whole, MnDOT elected to replace the deck trusses in kind with new deck trusses. One important aspect in which the new deck trusses differ from the originals is that all tension members were designed for internal redundancy (see Figure A3.5.3-1).

Figure A3.5.3-1—Rendering of the new deck truss approach span.

The existing through trusses are the most prominent portion of the bridge, and replacement of the through truss in kind was not a desirable option from both historic and financial perspectives. Nearly all of the existing through truss members are built-up box sections, with the upper chords consisting of closed boxes, and the diagonals, verticals, and bottom chords consisting of double channel sections connected with either lacing bars or batten plates. (The exceptions are the hangers between the cantilever and suspended spans, which are rolled I-sections).

To create internally redundant tension chord members, the design team proposed adding plating to the through truss tension member top and bottom chords as required. For the verticals and diagonals, internal redundancy would be provided by adding high-strength steel bars inside the existing built-up box sections (see Figure A3.5.3-2). The diagonals and verticals are most visible to motorists, and concealing the bars inside the existing box sections was anticipated to result in minimal changes to the appearance of these prominent components of the bridge. The proposed changes to the appearance of the bridge were minor enough that after evaluating the proposed retrofits, the State Historic Preservation Office concluded that there would be no significant impact to the historic character of the structure. This was an important finding that fulfilled one of the project's goals and allowed the final design of these retrofit concepts to proceed.

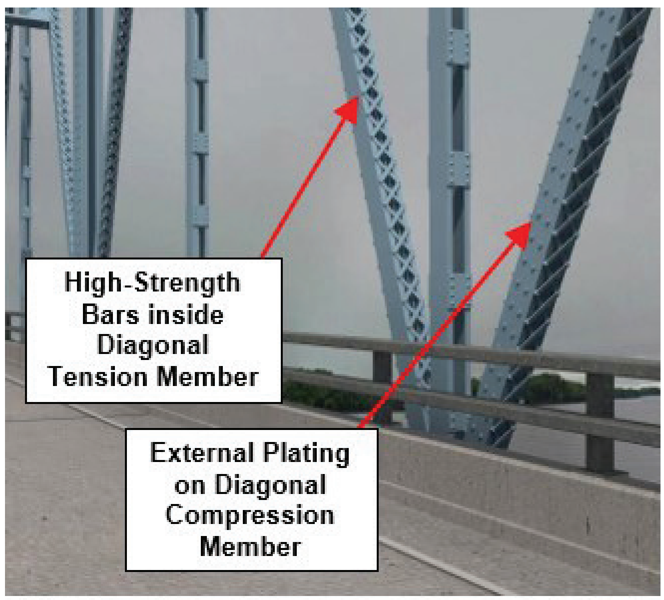

Figure A3.5.3-2—Rendering showing proposed through truss retrofits at deck level.

To evaluate the strengthening required to achieve internal redundancy, the truss tension members were evaluated for two conditions. The first was the loss of any one component of the built-up cross-section, without consideration of dynamic force effects resulting from a fracture. The second condition was the fracture of one component of the built-up cross-section, with consideration of dynamic effects from fracture, but with lower overall force effects. The resistance of the tension member with any one component missing was determined using the provisions of *AASHTO LRFD BDS*, Section 6. Since this project was completed, AASHTO has released the *Guide Specifications for Internal Redundancy of Mechanically-Fastened Built-Up Steel Members*. It is recommended that Designers follow the recommendations of these Guide Specifications for future internal redundancy investigations and retrofits.

For the Winona Bridge internal redundancy analysis, members resistance was compared to the demand from Extreme Event III load combinations, $1.25DC + 1.50DW + 1.30(LL+IM)$, for the loss of one component without dynamic effects, and the load combination, $1.10DC + 1.35DW + 0.75(LL+IM) + 1.1$(Dynamic Forces), for fracture of one component with dynamic impact. These load combinations were obtained from the FHWA publication *Design Guidelines for Arch and Cable-Supported Signature Bridges* and are typically used to evaluate the loss of a cable or hanger in cable-stayed or tied arch structures.

Plating and high-strength bars were specified as required to provide sufficient resistance to satisfy the demands from these load combinations. Not all tension members required retrofit, as some tension chords possess sufficient resistance to satisfy these load combinations with any one existing plate removed.

To provide internal redundancy, the additional plating and high-strength bars need to engage to carry load if one component of an existing tension member fractures. The plating is bolted to the existing tension chords and is therefore expected to engage evenly with the remaining intact original steel. The high-strength bars, by contrast, are attached to the through truss only at anchorages on the exterior faces of the chords at each end of the retrofitted member. The high 120 ksi yield strength of the bars allows sufficient resistance to be added to the verticals and diagonals by adding comparatively little additional cross-sectional area to these members, whose original steel has a much lower yield point of 33 ksi.

The design team investigated several issues to ensure that the high-strength bars would function satisfactorily to provide internal redundancy after a fracture. These analyses included determining the amount of load shedding to adjacent members and additional deformations as a result of a reduction in axial stiffness after fracture, investigating whether existing rivets in gusset plates would fail due to differential displacements between the fractured member and connection, investigating the effects of eccentricity of the remaining steel from the member centerline, and whether the high-strength bars would effectively engage to carry load.

To answer these questions, the design team undertook a detailed refined three-dimensional finite element analysis that was integrated into the conventional three-dimensional finite element model of the entire through truss. The analysis focused on the theoretical fracture of a tension diagonal member retrofitted with high-strength bars near the quarter point of the main span over the river. This member, U13-L14, consists of two steel channel sections that are connected with batten plates, and was chosen because of the comparatively large area of each channel section (see Figure A3.5.3-3).

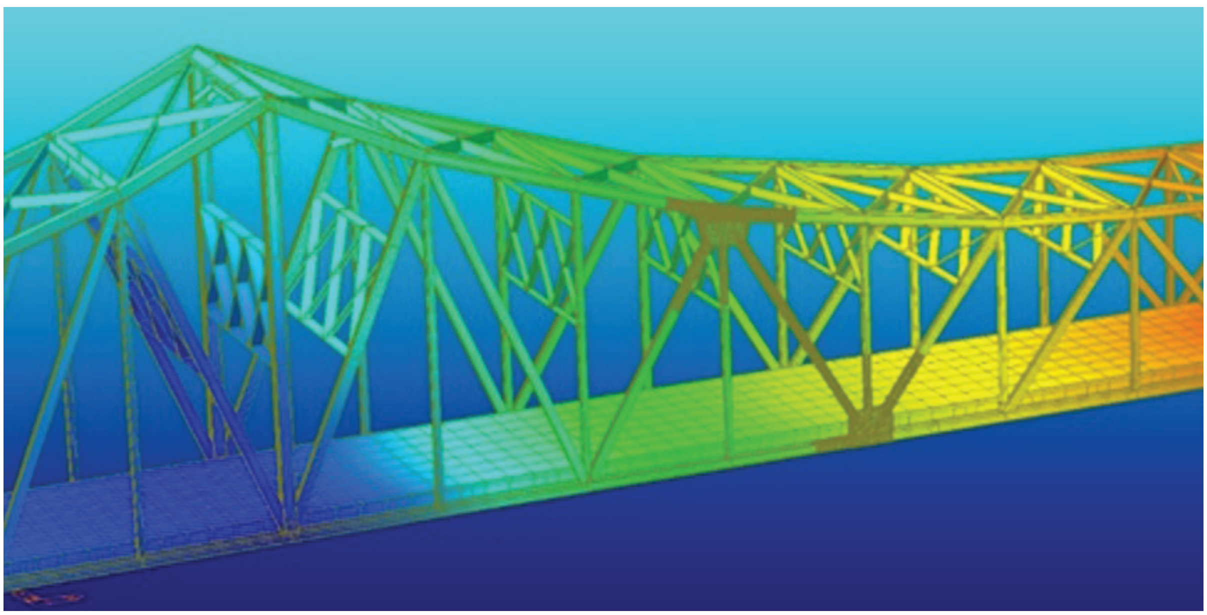

Figure A3.5.3-3—Integration of detailed refined analysis of U13-L14 into three-dimensional through truss model.

The analysis considered the fracture of one of the two channel sections near the U13 gusset plate, under a factored load corresponding to the loss of one component. The remaining intact channel at the location of the fracture was modeled with an element with material nonlinear properties, which ensures that the principal stress in the remaining steel cannot exceed the yield stress. The rivets connecting the fractured member and the adjacent truss members to U13 and L14 were modeled with multilinear links with the variable displacement versus shear characteristics of the actual rivets used on the bridge. The four high-strength bars, located inside the two channel sections of U13-L14, were modeled with truss elements connected to anchorage and wedge plates on the top face of U13 and the bottom face of L14 (see Figure A3.5.3-4).

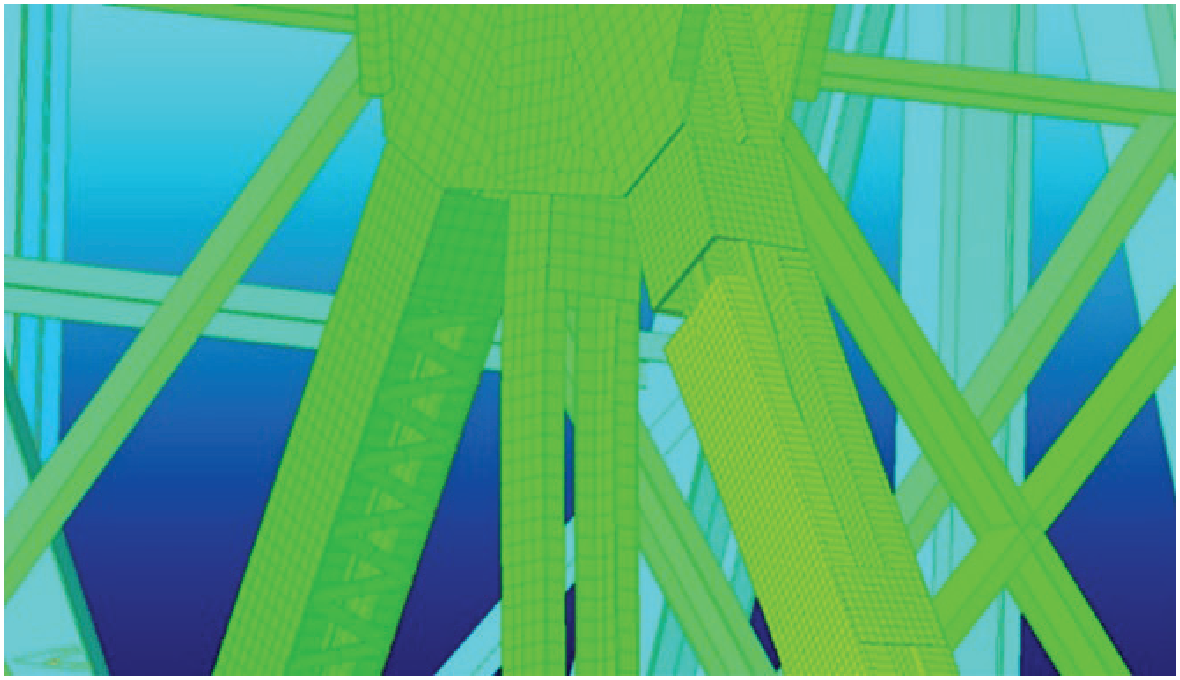

Figure A3.5.3-4—Detailed modeling of U13 gusset plate region showing theoretical fracture below U13 gusset plate.

The conclusions of the analysis gave the design team confidence that the high-strength bars would function as intended to provide internal redundancy in the event of a fracture. Forces in the members adjacent to U13-L14 were observed in the model to increase around five percent compared to pre-fracture values, an increased demand which these members have ample capacity to resist. The most heavily loaded rivets in the U13 and L14 gusset plates remained far from their maximum limits of shear and deformation. The remaining unfractured channel in the model did reach yield as anticipated, but this did not overstress the U13 gusset plate adjacent to the theoretical fracture location. With one of the U13-L14 channels fractured and the second channel at yield, loads in the model were distributed to the high-strength bars in proportion to the ratio of the bar area to the total area of the bars plus remaining steel.

Vertical global truss displacements computed by the model at panel points 13 and 14 increased less than half an inch compared to pre-fracture values, an increase that was not excessive and did not distress the adjacent truss members or connections. An additional conclusion of the analysis was that designing the high-strength bars with sufficient area to carry the total member force after fracture and neglecting the contribution of the members' remaining unfractured original steel, provides sufficient axial stiffness to prevent post-fracture deformations from becoming excessive. All of the high-strength bar retrofits are accordingly designed to carry the total member forces from the Extreme Event III load combinations.

A3.5.4—Construction Manager/General Contractor (CMGC) Process Benefits

The design team took advantage of the close collaboration with the Contractor afforded by the CMGC process to ensure that the through truss could be repaired and retrofitted for internal redundancy while remaining within the project budget. As retrofit design development was underway the construction company's subcontractor for through truss steel retrofits built a plywood mockup of a bottom chord gusset plate connection with the high-strength bars and anchorages included to ensure that the proposed retrofits would fit inside the existing steel sections as intended and were constructible (see Figure A3.5.4-1). These efforts also reduced the uncertainty and risk to the Contractor prior to bid, which likely resulted in a lower overall bid price. Contractor cost estimates made using milestone plan submissions helped guide the design team to focus project resources on preserving the historic through truss and led to simplifications to the design of the replacement concrete approach spans.

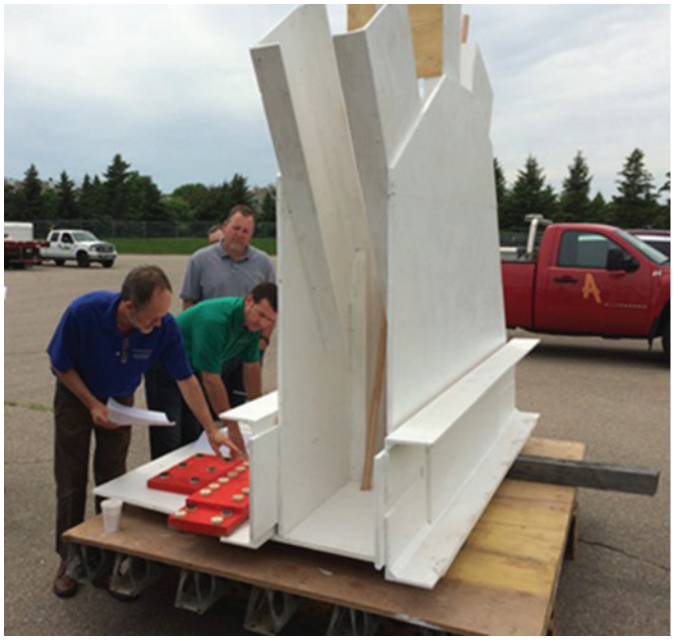

Figure A3.5.4-1—Plywood mockup of retrofitted Winona Bridge through-truss joint.

The final $38.9M bid for the rehabilitation of the through truss and removal of the deck truss and concrete I-girder approach spans was consequently within the financial resources of the project, allowing construction to move forward. Given the uncommon nature of the retrofit work being undertaken, it would have been challenging to achieve a similar result using either a design-bid-build or design-build approach, where the same close coordination between Owner, Designer, and Contractor prior to bid would have been difficult to realize.

A3.5.5—Additional Analysis During Construction

As part of the rehabilitation of the though truss spans, the members were blast cleaned and repainted. The blast cleaning removed accumulated pack rust, and together with removal of the existing deck, allowed for more detailed section loss measurements to be taken than prior to the start of construction. Detailed loss measurements were taken of each member during construction, and in a few cases additional strengthening plates were added to deteriorated floorbeam ends and main truss members. The project design documents had included provisions for additional strengthening plates to address this situation. The Designers provided MnDOT with a final rehabilitated condition load rating that included the impacts of all section loss measurements taken during construction and demonstrated that the rehabilitated structure satisfied the project design criteria. The Winona Bridge rehabilitation project was completed and opened to traffic in July 2019.