Project No. <u>12-113</u>

Proposed Modifications to AASHTO Cross-Frame Analysis and Design

APPENDIX D
PHASE I SUMMARY

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CHAPTER D1

Introduction

The final report of NCHRP Project 12-113 succinctly summarizes the key outcomes of the research for practicing engineers. As such, many of the detailed results were not included in the body of the main document for clarity. The intent of this appendix is to provide a more comprehensive overview of the work completed in Phase I of the project. Similarly, Appendix E and F expand on Phase II and III of the project, respectively. For reference, Phase I is comprised of the following tasks, which were previously identified in main body of the final report as well as the project Request for Proposals (RFP):

- Task 1: Conduct a literature review of relevant domestic and international research, guidelines, and specifications including fatigue loading for transverse members. Document current design practices and how design software incorporates these design practices.
- Task 2: Synthesize the results of the literature review to identify the knowledge gaps related to the research objectives. These gaps should be addressed in the final product or in the recommended future research as budget permits.
- Task 3: Propose an analytical program to be executed in two parts as follows. Part 1, to be executed in Phase II, includes modeling and validation of three bridges as described in the field experiment in Task 4. Part 2, to be executed in Phase III, conducts comprehensive parametric studies to achieve the research objectives using validated models in Part 1 of the analytical program. At a minimum, the analytical program should consider the following:
    - Analytical and loading studies (finite element analysis) to investigate appropriate fatigue stress ranges for evaluation of cross-frames for right, skewed, and curved bridges, and Fatigue I and II;
    - The influence of girder spacing, cross-frame stiffness and spacing (including staggered), and deck thickness;
    - Development of stability bracing requirements for steel I-girders during construction and in-service extending available solutions to include bottom flanges in compression in multi-span continuous bridges with non-prismatic girders. The analytical studies should include an evaluation of how to combine stability bracing strength requirements with consideration of other loads such as wind, construction, etc.; and
    - Parametric modeling studies to investigate the effective stiffness of cross-frames, including issues such as the effect of connection details and connection plate stiffness on cross-frame member stiffness reduction.
- Task 4: Propose a field experiment, to be executed in Phase II, to achieve the project objectives. At a minimum, the field experiment should consider experimental verification of analytical models by instrumenting cross-frames including one right bridge, one skewed bridge, and one horizontally-curved bridge to measure cross-frame member fatigue force ranges under controlled application of live load. The same field experiment shall be repeated for in-service effective and maximum stress ranges.

- Task 5: Identify existing articles of the American Association of State Highway and Transportation Officials LRFD Bridge Design Specifications (henceforth referred to as AASHTO LRFD) that require modification.
- Task 6: Prepare Interim Report No. 1 that documents Tasks 1 through 5 and provides an updated and refined work plan for the remainder of the research no later than 4 months after contract award. The updated plan must describe the process and rationale for the work proposed for Phases II though IV.

Tasks 3 and 4 largely served as an opportunity for the research team (RT) to outline its preliminary Phase II plans to the research panel. Given that Appendix E covers this material more conclusively and in greater depth, the major outcomes of Tasks 3 and 4 are not covered here. An exception to this is in Chapter D5, for which a detailed overview of the potential bridges to instrument is provided. Ultimately, three bridges were selected for field monitoring, as is further discussed in Appendix E.

Additionally, Tasks 2 and 5 (identifying gaps in knowledge and AASHTO articles to modify) is covered extensively in the main body of the report, as it directly relates to the motivation and scope of the research project. As such, this topic is not covered in this appendix.

Thus, this document primarily covers the general background and literature review, an industry survey, and a review of commercial software packages. The appendix is organized in a traditional report format; it is divided into seven distinct chapters. Following this introductory chapter, Chapter D2 provides an overview of the current AASHTO LRFD code provisions pertaining to cross-frames, as well as a summary of pertinent literature. Chapter D3 discusses the development and results of an industry survey distributed to Departments of Transportation (DOTs) and consulting design firms. An overview of the fatigue design approach currently utilized by commercial software packages is provided in Chapter D4. Then, a summary of the bridges considered for instrumentation in Phase II is provided in Chapter D5. Finally, two supplementary chapters are included at the end to provide the reader with the additional reference material. In Chapter D6, the survey submitted to bridge owners and consultants is provided for reference. In Chapter D7, the typical cross-frame details submitted by various bridge owners are compiled and tabulated.

CHAPTER D2

Background

AASHTO LRFD (2017) defines a cross-frame as "…a transverse truss framework connecting adjacent longitudinal flexural components or inside a tub section or closed box used to transfer and distribute vertical and lateral loads and to provide stability to the compression flanges. Sometimes synonymous with the term diaphragm." Cross-frames serve many roles throughout the construction and service life of a steel bridge. They primarily function as stability braces to enhance the lateral-torsional buckling (LTB) resistance of the bridge girders. Provided that they are properly designed and detailed, cross-frames restrain the twist of a girder cross-section at discrete locations along the length; hence, there are aptly referred to as torsional braces. From a stability perspective, the critical stage for bracing often occurs during construction of the concrete bridge deck when girders are noncomposite. In multi-span continuous bridges where the composite deck provides continuous lateral and torsional restraint to the top flange, cross-frames also provide additional stability to bottom compression flanges in the negative moment regions, despite girder instability generally being much less impactful at this stage.

Aside from their primary role as stability braces, cross-frames also resist a variety of lateral and gravity loads throughout the life of a bridge and tie together the girders across the width. Cross-frames resist the applied torque on fascia girders due to typical deck overhang construction and distribute lateral loads across the structure (e.g. wind). They also restrain differential movement in girders (i.e., vertical deflection and rotation) caused by dead and construction loads on the noncomposite system (e.g. externally applied loads and locked-in fit-up forces) and live loads on the composite system. In the completed structure, the passage of heavy truck traffic subjects cross-frames to cyclic loading conditions. Under repeated loading, cross-frames can theoretically become susceptible to load-induced fatigue cracking even for stress magnitudes well below the yield strength of the material.

In horizontally curved bridges, cross-frames are considered primary structural elements and engage the girders across the bridge width to behave as a unified structural system and to resist the torsion developed from the curved geometry. The respective roles of a cross-frame system during construction and in service are illustrated schematically in Figure D2-1 and Figure D2-2.

To expand on these features of a cross-frame system, this chapter presents several sections. Past and present AASHTO code provisions related to cross-frame analysis and design are first outlined in Section D2.1. Then, Sections D2.2, D2.3, and D2.4 discuss the pertinent research conducted over the last few decades related to fatigue, stability bracing requirements, and cross-frame stiffness, respectively.

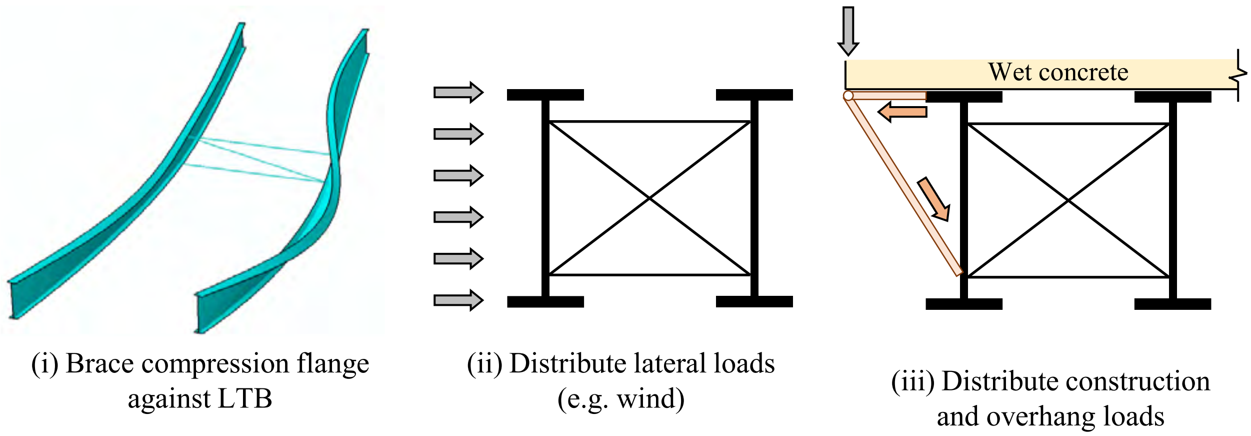


Figure D2-1: Primary functions of cross-frames during deck construction

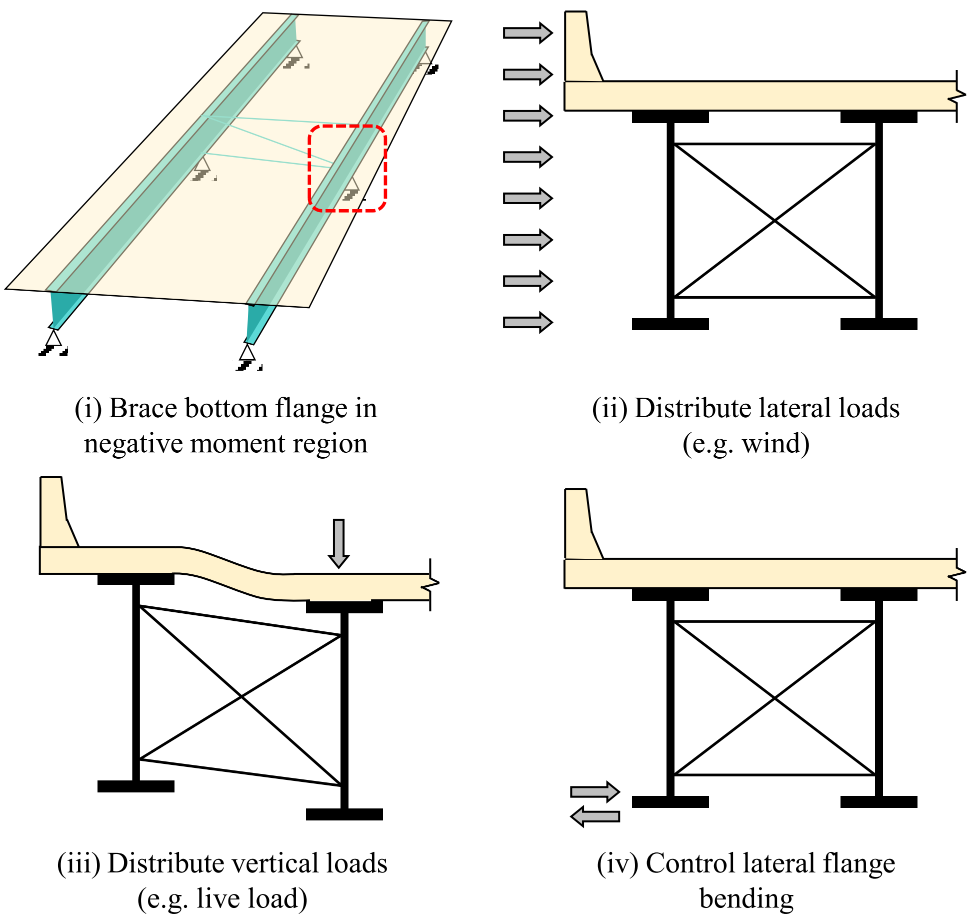


Figure D2-2: Primary functions of cross-frames in composite bridges in service

D2.1 Review of Code Provisions

The following subsections provide a brief discussion of the legacy code provisions in AASHTO LRFD pertaining to cross-frames in steel I-girder bridge systems. This is followed by an outline of the modern code provisions pertaining to cross-frames in the current 9th Edition of AASHTO LRFD. Note that only those provisions that directly relate to the outcomes of this research project are discussed in detail.

D2.1.1 Legacy Code Provisions

In 1949, AASHTO Specifications introduced a maximum cross-frame spacing limit of 25 feet in steel girder bridges. The 25-foot spacing limit initially targeted shorter-span bridges, which were more common in that era. Similar to modern practices, most states also utilized standard details and member sizes such that cross-frames were seldom engineered or designed elements. As bridge engineering and construction modernized over the following decades, longer spans became more prevalent. Additionally, stress demands on steel components generally increased given the advent and use of higher strength materials.

While the 25-foot requirement generally resulted in satisfactory performance of steel bridge superstructures, it was essentially an arbitrary limit based on experience and general rules-of-thumb that existed at the time. In many situations, though, designing steel superstructures for the 25-foot maximum spacing produced overly conservative and uneconomical cross-frame layouts. Because (i) cross-frames represent an expensive component of bridge fabrication and erection and (ii) regions around cross-frame panels have historically been susceptible to distortion-induced fatigue cracking, requiring excessive cross-frames was undesirable.

AASHTO Specifications maintained the 25-foot spacing limit until the 1st Edition of AASHTO LRFD (1994), at which point it was removed and replaced with a requirement for sizing and spacing based on rational analysis and design. Rather than use a prescriptive and standard design approach, the need for cross-frames was to be evaluated for all stages of construction and the final condition of the bridge. Based upon this requirement, the cross-frame spacing is permitted to exceed 25 feet, assuming it is justified by rational analysis. In those instances, caution must be exercised when extending the spacing significantly beyond 25 feet since the demand on the cross-frames will increase and standard details may not be suitable.

AASHTO LRFD 3th Edition (2005) resulted in a unification of the design provisions for straight and horizontally curved girders within AASHTO specifications. For curved systems, an upper cross-frame spacing limit was established to limit flange lateral bending stresses resulting from torsion and to theoretically preclude elastic lateral torsional buckling of the compression flange in curved I-girders. Additional discussion is provided below in Section D2.1.2.1.2.

All modern bridges generally make use of composite action between the steel girders and concrete bridge deck. As a result, the top flange of the girder in the finished bridge is continuously braced in the positive moment regions of the composite bridge, and the critical stage for stability generally occurs during construction. The critical stages for stability can occur during erection when partial bracing is provided or during placement of the bridge deck when the fresh concrete does not provide restraint to the girder. Though the cured concrete deck can provide torsional restraint in the negative moment regions with proper shear stud detailing, this deck restraint is not considered in most designs. The cross-frame spacing in the negative moment regions is usually based on the resistance of the girder alone and the factored design moment in the final condition at the strength limit state. Although the approach in the positive and negative moment regions is "rational" with respect to the girder buckling resistance, such an approach does not address the required size of the cross-frames from the perspective of the minimum required stiffness or strength for adequate bracing.

Historically, cross-frame locations were often regions of cracking in the girder webs as a result of distortion-induced fatigue. While cross-frame connections to main elements are evaluated for load induced fatigue, the wide-spread tendencies for distortion-induced fatigue cracking in the girder webs around the cross-frames were alleviated in the 1980s with the requirement to positively attach the connection plates (i.e., transverse web stiffeners that connect the cross-frames to the girders) by welding or bolting to the girder flanges. An exception is permitted to this requirement where cross-frames are used on rolled beams in straight bridges with composite concrete decks whose support cross-frames are normal or not skewed more than 10 degrees from normal and with the intermediate cross-frames placed in contiguous lines parallel to the supports (Article 6.6.1.3.1). Note that the term "Article" in this appendix refers to 9th Edition AASHTO LRFD code provisions, unless noted otherwise.

This relaxation appears to be completely based on good performance observed with this detail and girder arrangement. However, cross-frame and diaphragm locations still pose a fatigue concern in steel bridges, particularly in systems with significant support skew or horizontal curvature. The primary concerns in these systems are related to the fatigue behavior of the cross-frame members and their connections, which typically experience larger loads in these systems.

In recent years, there have been a number of advances in the body of knowledge related to the stability of bridge components as well as the fatigue performance related to cross-frame systems. Some of these improvements in understanding include the recognition of system buckling modes (Yura et al. 2008, Han and Helwig 2016), issues with detailing and fit-up of cross-frames in skewed and curved I-girder bridges (Chavel and Earls 2006, Chavel et al., 2016, White et al. 2015), lean-on bracing concepts for straight skewed I-girder bridges (Helwig and Wang 2003; Romage 2008), corrections in the stiffness modelling of the cross-frames (Wang 2013, Battistini et al. 2016, White et. al. 2012), and establishing proper fatigue categories for the members that comprise the cross-frames (McDonald and Frank 2009, Battistini et al. 2013).

Improvements in computational resources over the last few decades allow engineers to carry out sophisticated analyses on bridge systems that can produce efficient and reliable structural systems satisfying both construction and in-service design requirements. Though the computational resources and analytical programs permit relatively sophisticated analyses on bridge systems, the accuracy of any analysis is limited by the modeling assumptions and level of understanding of the fundamental behavior of the structure. While some commercial software packages may provide an evaluation of the fatigue performance of the girders and cross-frames, many analytical models that are used for design may consist of either line-girder models or grillage models (in which the girders and braces are modeled using 2D line/beam elements). These models are often not capable of accurately evaluating the stability bracing behavior of the cross-frames or diaphragms and the accuracy of a fatigue evaluation is questionable. Even the most detailed "three-dimensional" finite element (FEA) models generally represent the cross-frame members as axially-loaded truss elements and do not reflect the impact of eccentric connections. Recent research has shown this can significantly reduce the cross-frame stiffness (Battistini et al 2016). Neglecting the reduction in stiffness due to eccentric connections will generally be unconservative from a stability or torsional behavior perspective and overly-conservative from a fatigue standpoint, since modeling stiffer cross-frames will result in larger live-load induced forces than occur in reality.

D2.1.2 AASHTO LRFD Bridge Design Specifications (9th Edition)

The following subsections provide an overview of the current design requirements for cross-frames according to the 9th Edition of AASHTO LRFD (2020).

D2.1.2.1 General Cross-Frame Design Requirements

D2.1.2.1.1 Analysis of Cross-Frames

Article 4.6.3.3.4 of AASHTO LRFD states that when performing a static analysis of cross-frames using a grillage type model (i.e., a model that converts cross-frames to equivalent, single line beam elements), both cross-frame flexure and shear deformation must be considered when calculating the equivalent beam stiffness. Neglecting to account for shear deformations can lead to significant error in the calculation of cross-frame stiffness.

This article also states that the influence of end connection eccentricities must be considered when calculating the equivalent axial stiffness to be used when a cross-frame is composed of single-angle or tee-section members. The stiffness of cross-frames composed of these members can be significantly reduced because of end connection eccentricity (Wang 2013; Battistini et. al. 2016). The commentary currently recommends applying a reduction factor of 0.65 to the axial stiffness of equal leg angles, unequal leg angles connected to the long leg, and flange-connected tee-section members. This recommendation was based upon early results from the work of Wang and Battistini. Equations (presented in Section D2.4.2 of this appendix) were developed for the reduction in stiffness of single-angle cross-frames that consider the geometry and member sizes of the specific cross-frame. These equations provide improved accuracy over the current commentary language that uses the fixed factor of 0.65.

D2.1.2.1.2 General Requirements for Cross-Frames in Steel I-Girder Bridges

Article 6.7.4.1 requires the need for cross-frames to be investigated at both the construction stage and during the in-service condition of a steel I-girder bridge. Permanent cross-frames must be designed for all applicable limit states. AASHTO LRFD defines a primary member as:

> "…a member designed to carry the loads applied to the structure as determined from an analysis,"

and therefore cross-frames in horizontally curved girders are considered primary members. In the 9th Edition of AASHTO LRFD, the definition of a primary member was changed to:

> "…a steel member or component that transmits gravity loads through a necessary as-designed load path. These members are therefore subjected to more stringent fabrication and testing requirements; considered synonymous with the term main member."

At a minimum, the cross-frames in straight steel I-girder bridges must be designed to transfer wind loads in the finished condition of the bridge. However, because the cross-frames in horizontally curved bridges are considered primary members, they must be designed for all limit states, including fatigue.

Article C6.7.4.1 states that previous versions of AASHTO LRFD required cross-frames to not be spaced at a distance greater than 25 feet. This provision was replaced by the requirement for a rational analysis. However, for horizontally curved bridges, Article 6.7.4.2 requires that the cross-frame spacing not exceed the spacing calculated by the following equation, or 30 feet, whichever is less:

$$L_b < L_r < \frac{R}{10} \qquad \textit{D2.1}$$

where: L_b = spacing of the cross-frames; L_r = limiting unbraced length determined from AASHTO LRFD Eq. 6.10.8.2.3-5; and R = minimum girder radius within the panel. The limit of $R/10$ in Eq. D2.1 is consistent with past practice. Limiting the unbraced length to L_r theoretically precludes elastic lateral torsional buckling of the compression flange and helps to limit flange lateral bending stresses resulting from torsion.

Article 6.7.4.2 requires that cross-frames used in systems with rolled beams be a minimum of 0.5 of the depth of the girders, and cross-frames used in systems with plate girders be a minimum of 0.75 of the depth of the girders.

Where the supports of a steel I-girder bridge are not skewed, the intermediate cross-frames should be placed in contiguous lines perpendicular to the girders. If the supports of a steel I-girder bridge are skewed at an angle less than 20 degrees from normal, then intermediate cross-frames may be placed in continuous lines parallel to the supports, as shown in Figure D2-3(a). For small skew angles, this arrangement permits the cross-frames to be attached to the girders at points of nearly similar length along the girders (i.e., points of nearly equal stiffness), thus reducing the relative deflection between the cross-frame ends and the restoring forces in these members. If the supports of a steel I-girder bridge are skewed at an angle greater than 20 degrees from normal, intermediate cross-frames must be oriented perpendicular to the girders, as shown in Figure D2-3(b). This requirement is consistent with past practice and is likely related, in part, to fabrication difficulties and problems encountered when braces were oriented parallel to the skew angles at larger support skews. The fabrication difficulties are related to issues with welding access to the acute corner between the connection plate and the web. As a result, for braces at the support locations that are typically oriented parallel to the skew, a common detail is to use a perpendicular connection plate (web stiffener connected the cross-frame to the girder), combined with a bent plate to account for the skew. Bent plate details are discussed more in Section D2.4.1 of this appendix. Another reason for requiring perpendicular braces for larger skew angles is likely the result of observed problems when typical cross-frame sizes were used with parallel braces. The parallel orientation results in substantial reductions in the brace stiffness due to the longer brace and skewed orientation.

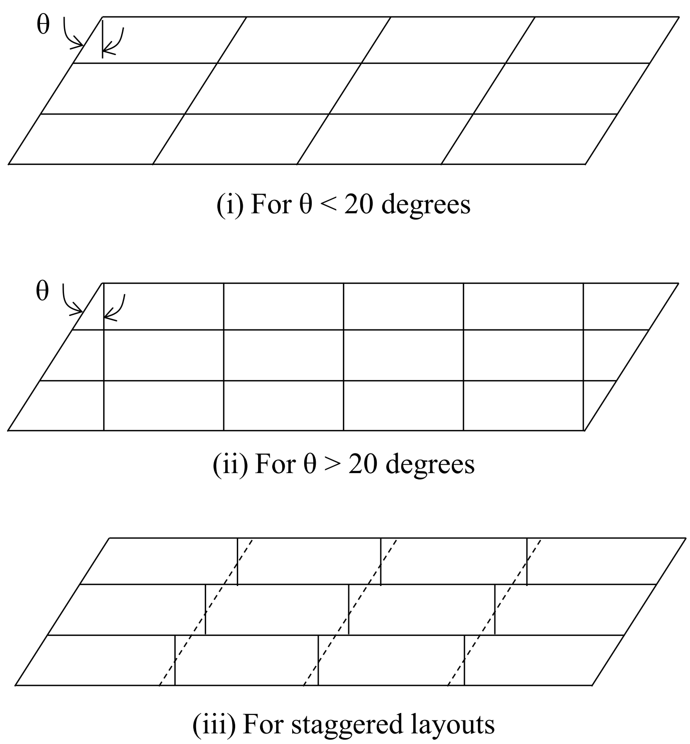


Figure D2-3: Cross-frame layout as a function of skew angle, θ

Where support lines are skewed more than 20 degrees from normal, it may be advantageous to orient the intermediate diaphragms or cross-frames normal to the girders in discontinuous lines, to selectively omit certain diaphragms or cross-frames, and/or to stagger the diaphragms or cross-frames in adjacent bays between the girders Figure D2-3(c). In highly-skewed bridge systems, a perpendicular line of braces will result in very significant differences in the girder displacements at the two ends of the bracing line. For example, with skews greater than 45 degrees, one end of the bracing line may frame into one fascia girder near midspan while the other end of the bracing line may frame into the other fascia girder near the support, thereby resulting in very large forces induced in the braces. One particularly problematic situation can occur when a bracing line frames into the support with a highly-skewed girder system, as shown in Figure D2-3(b). Improved behavior with the bracing line near the support can be achieved with the omission of highly-stressed diaphragms or cross-frames near the obtuse corners of a span, provided the potentially larger unbraced length in this region does not compromise the buckling behavior. This is demonstrated schematically in Figure D2-4.

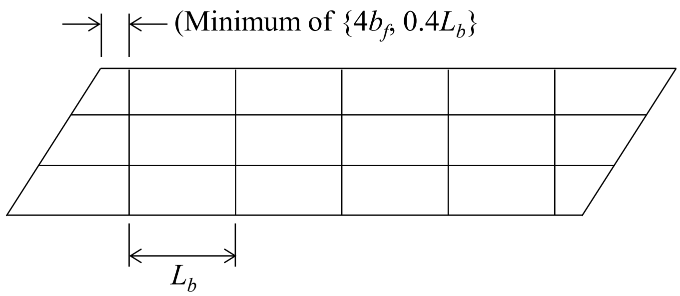


Figure D2-4: Recommended offset dimension at skewed supports to alleviate cross-frame force effects

In the 9th Edition of AASHTO LRFD, additional language is provided in Article C6.7.4.2 discussing potential framing arrangements to both reduce the number of cross-frames or diaphragms within the bridge as well as to reduce the overall transverse stiffness effects in skewed I-girder bridges. In addition, a recommended offset of the first intermediate cross-frames or diaphragms placed normal to the girders adjacent to a skewed support is provided to alleviate the introduction of a stiff load path that will attract and transfer large transverse forces to the skewed support, particularly at the obtuse corners of a skewed span. At skewed interior piers in continuous-span bridges, transverse stiffness effects are alleviated most effectively by placing diaphragms or cross-frames along the skewed bearing line, and locating normal intermediate diaphragms or cross-frames at distances greater than or equal to the minimum offset from the bearing lines discussed above. Framing of a normal intermediate cross-frame into or near a bearing location along a skewed support line is strongly discouraged unless the cross-frame diagonals are omitted.

D2.1.2.2 Design for Fatigue

D2.1.2.2.1 Limit States

Article 1.3.2.3 of AASHTO LRFD states that the fatigue limit states shall limit the stress range that results from the passing of a single design truck occurring over a given number of cycles. Articles 3.4.1 and C3.4.1 describe two limit states for load-induced fatigue design: Fatigue I and Fatigue II, for the respective cases of infinite fatigue life and finite fatigue life. Because fatigue behavior is a function of the cyclic stress range, only live loads, the dynamic load allowance, and centrifugal forces are considered in both limit states.

According to the 9th Edition of AASHTO LRFD, the Fatigue I limit state is related to infinite load-induced fatigue life. If a member has infinite fatigue life, then it will theoretically be able to withstand an infinite number of cycles, provided the applied effective stress range amplified by the Fatigue I load factor does not exceed the specified constant amplitude fatigue limit (CAFL) of the specific detail. The load factor to be used with the Fatigue I limit state is 1.75. This load factor, when applied to the effective fatigue design truck (i.e., the HS-20 with a load factor of 0.75) discussed in the next section, corresponds to a truck or stress range with a return period of about 1 in 10,000. Variable amplitude fatigue testing has shown that if the CAFL is exceeded at a frequency of less than 1 in 10,000 infinite life can be ensured.

The Fatigue II limit state is related to finite load-induced fatigue life. If a member has a finite fatigue life, then it will theoretically fail due to fatigue when a given number of cycles at a given stress range are completed. The AASHTO fatigue design curves correspond to a probability of cracking of only about 2.5% or in other words, a 97.5% probability of survival. The load factor to be used with the Fatigue II limit state is 0.8. This value is representative of the effective stress range produced by the general truck population.

Note that the load factors reported above are based on current 9th Edition Specifications. These load factors were recently modified (increased) based on Strategic Highway Research Program 2 (SHRP 2) Project R19B, which was funded by the Transportation Research Board. Modjeski and Masters led the research efforts for the project (Kulicki et al. 2014, Modjeski & Masters 2015). These load factors were calibrated to recent WIM records in the context of girder moments and shears. One-dimensional line-girder analyses were performed such that cross-frame force effects were not considered in the development of the load factors.

The increase in load factors reflects the fact that truck populations across the country are generally getting heavier. Historically, the ratio between the load factors for the Fatigue I and Fatigue II limit states has been 2:1, but considering the findings of the Project R19B research team, this ratio is now closer to 2.2:1.

D2.1.2.2.2 Fatigue Loading

Article 3.6.1.4.1 states that one design truck, as specified in Article 3.6.1.2.2, is to be used to calculate the fatigue design stress range. The use of a single truck in a single design lane was confirmed in the SHRP 2 R19B project for the case of fatigue in the main bridge girders. It is also specified that the rear axles of the truck shall have a constant spacing of 30 feet. The dynamic load allowance specified in Article 3.6.2 is applied to the fatigue load. The fatigue design truck is shown in Figure D2-5.

Article 3.6.1.4.3a states that if using a refined analysis method, one must consider a single design truck positioned both longitudinally and transversely on the bridge to produce the maximum stress range on a given component. The position of design lanes is ignored because it is often difficult to predict any future changes that might result in the shifting of design lanes on the bridge deck.

The dynamic load allowance, IM, to be applied to the design truck is specified in Article 3.6.2.1. When evaluating fatigue (in a girder or other "global" components such as a cross-frame), a value of 1.15 is to be used for the dynamic load allowance per AASHTO LRFD Table D3.6.2.1-1. The dynamic load allowance is used to account for "average" wheel load impact from moving vehicles. Peak impacts, such as 1.30, are not appropriate for fatigue since fatigue is intended to represent stress ranges produced during normal in-service conditions. This dynamic response may be caused by either hammering effects resulting from moving wheel loads and/or from the dynamic response of the bridge. Interestingly, field studies consistently show that even 1.15 is not frequently exceeded.

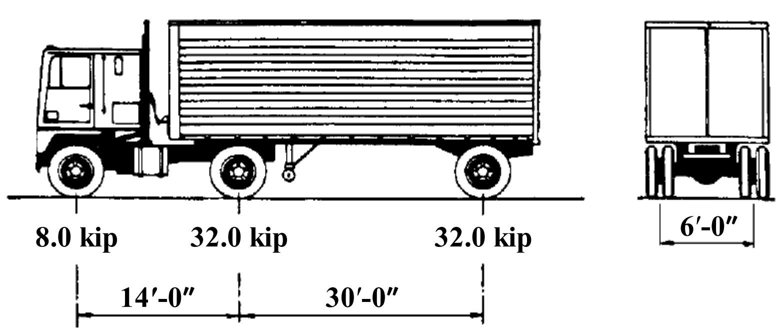


Figure D2-5: AASHTO fatigue truck (adapted from Article 3.6.1.4.1)

Article 3.6.3 discusses the provisions for centrifugal force, CE. CE loads account for the overturning effect due to lateral forces created when a truck is rounding a horizontally curved bridge. This will generally apply only to horizontally curved bridges since straight bridges will likely not experience significant lateral forces from vehicular traffic.

Article 3.6.1.4.2 discusses the methodology for determining the frequency of the fatigue load if the information is not available from other sources. The frequency of the fatigue load is to be taken as the single-lane average daily truck traffic, $ADTT_{SL}$. The $ADTT_{SL}$ is the expected number of trucks per day in a single lane averaged over the life of the bridge, which is best determined in consultation with traffic engineers. This frequency is to be applied to all components of the bridge being designed for fatigue. In lieu of more accurate information, the single-lane average daily truck traffic may be calculated as follows:

$$ADTT_{SL} = p * ADTT \tag{D2.2}$$

where: p = fraction of truck traffic in a single lane; and $ADTT$ = total number of trucks in one direction per day over the life of the bridge. As the number of lanes increase, the value of p decreases. As mentioned in Article C3.6.1.4.2, consultation with traffic engineers regarding any directionality of truck traffic may lead to the conclusion that one direction carries more than one-half of the bidirectional $ADTT$. If such data are not available, designing for 55 percent of the bidirectional $ADTT$ is suggested. Guidance is also given in Article C3.6.1.4.2 regarding the extrapolation of available traffic growth data for the fatigue design life of the bridge (taken as 75 years in AASHTO LRFD).

D2.1.2.2.3 Design for Load-Induced Fatigue

Article 6.6.1 discusses how to check the limit states of fatigue and fracture. Fatigue is categorized as either load-induced fatigue or distortion-induced fatigue. Load-induced fatigue is characterized by the application of repeated in-plane tensile stresses to an element. Distortion-induced fatigue represents fatigue effects due to out-of-plane secondary stresses not quantified in the analysis and is often the result of improper detailing. Historically, cracking in the webs in the vicinity of cross-frame locations is one of the most common cases of distortion-induced fatigue. Prior to the 1980s, distortion-induced cracks in the webs were relatively common and caused by the lack of a rigid load or stress path to transmit the force in the cross-frame members from the web to the flange. Distortion-induced fatigue is not a topic of this research and is not discussed further.

Articles 6.6.1.2.1 and C6.6.1.2.1 state that only the live load stress range is to be considered for fatigue design, and that residual stresses caused by fabrication are not to be considered when investigating fatigue. Permanent loads do not contribute to the stress range. Residual stresses are not included explicitly because they are included implicitly through the specification of the stress range as the sole dominant parameter for fatigue design. Growth in the fatigue cracks is caused by cyclic tensile stresses. However, for cases with a stress reversal (i.e., stress ranges including both tensile and compressive components) the complete live load stress range, consisting of the full range including both the tensile and compressive stress components, is used to check fatigue. The reason the compression component is included is because the residual stresses locked in during fabrication may cause the entire stress range cycle to be shifted into the tensile stress region. Even if there is only a small component of tension in the stress range, the crack can propagate. In the case of stress reversal, even if the compression component of the stress range is much larger than the tensile portion, the fatigue limit states must be considered.

If the live load tensile stress calculated using the Fatigue I limit state load factor is smaller than the compressive stress due to the unfactored compressive permanent loads, then there is no net tensile stress, and there is no need to further consider fatigue. The Fatigue I limit state is used for this check because it is associated with the upper bound (i.e., the 1 in 10,000 return period) stress range an element may experience.

In calculating section properties for the design of composite steel girders, the concrete deck is converted to an equivalent amount of steel based on the modular ratio, $n = E_s/E_c$, where E_s and E_c are the respective elastic moduli of the steel and concrete. Depending on the nature of the calculations, either "short-term" or "long-term" composite section properties may be used. In calculations utilizing "short-term" composite properties, the deck is transformed into an equivalent steel section using the modular ratio, n. For "long-term" composite section properties, the effects of creep and shrinkage are approximated and the concrete is transformed using $3n$. According to Article 6.6.1.2.1, dead and live load stresses and live load stress ranges for fatigue design at all sections in the member due to loads applied to the composite section may be computed using the long-term composite section for dead loads and the short-term composite section for the live loads, assuming the concrete deck is effective for both positive and negative flexure. Shear connectors must be provided throughout the entire length of the member, and the longitudinal reinforcement must satisfy the provisions of Article 6.10.1.7 in order for the concrete to be considered effective for negative flexure. Properly reinforced concrete can provide significant resistance to tensile stress at service load levels. Recognizing this behavior will have a significantly beneficial effect on the computation of fatigue stress ranges for details located on or near the top of the girder in regions of stress reversal and in regions of negative flexure.

The primary guidance for the application of the fatigue loads in cross-frames can be found in Article C6.6.1.2.1; however, the guidance has changed in recent years. Prior to 2015, AASHTO LRFD Article C6.6.1.2.1 described a possible fatigue loading condition for cross-frames when these effects are determined from a refined analysis. Stresses are created in cross-frames when one girder deflects with respect to an adjacent girder. The proposed loading condition involves the passing of two trucks simultaneously, with one truck traveling along one girder and the other truck traveling along an adjacent girder (slightly behind the first truck). It was further suggested that a factor of 0.75 be applied to the resulting stress range to account for the low probability of occurrence of two vehicles located in these critical relative positions. In no case was the calculated stress range to be less than the stress range caused by the loading of only one lane.

While this loading condition creates the worst possible fatigue stress range in a cross-frame, a 2015 interim revision noted that it is highly unlikely that this loading condition is a common occurrence throughout the service life of the bridge. As a result, the 2015 interim revisions recommended positioning a single fatigue truck in one transverse position for each longitudinal position evaluated. This was determined solely on the consensus of AASHTO T-14 members and advisors to this subcommittee based on experiences and an informal review of available data. The group subsequently agreed that this loading more accurately represents the typical loading condition that will be experienced by the cross-frames throughout the design service life of the bridge. One of the primary goals of this research is to determine if that is in fact the case, or if perhaps a different loading condition is necessary to more accurately reflect the design stress ranges experienced by the cross-frame members. This was documented in the main body of the final report, as well as Appendix F.

All details being evaluated for load-induced fatigue are to satisfy the following equation from Article 6.6.1.2.2 of AASHTO LRFD:

$$\gamma(\Delta f) \leq (\Delta F)_n \qquad D2.3$$

where: γ = load factor pertaining to either the Fatigue I limit state or the Fatigue II limit state; (Δf) =calculated stress range experienced by the detail under consideration; and $(\Delta F)_n$ = nominal fatigue resistance of the detail determined as specified in Article 6.6.1.2.5.

The nominal fatigue resistance, $(\Delta F)_n$, can be calculated using one of two equations, depending on which limit state is being checked. The limit state and equation to check depends on the value of the calculated $(ADTT)_{SL}$ relative to the value specified in Table D6.6.1.2.3-2 for the component or detail under consideration. If the component or detail is to be checked for infinite life using the Fatigue I limit state, the nominal fatigue resistance is to be calculated as follows:

$$(\Delta F)_n = (\Delta F)_{TH} \qquad \textit{D2.4}$$

where: $(\Delta F)_{TH}$ = constant amplitude fatigue threshold (CAFT). If it is determined that the stress range in a given detail is lower than the CAFT, then the detail is considered to have an infinite fatigue life. Article 6.6.1.2.3 further recommends that components and details on fracture-critical members (FCMs) should always be designed for infinite life.

If the component or details is to be checked for finite life using the Fatigue II limit state, the nominal fatigue resistance is to be calculated as follows:

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{\frac{1}{3}} \qquad \textit{D2.5}$$

where: A = a detail-category constant specified in Table D6.6.1.2.5-1 representing the y-axis intercept of the *S-N* curve (Figure DC6.6.1.2.5-1) for each detail category; and N = estimate of the total number of cycles the detail can expect to experience over the 75-year fatigue design life based on the single-lane average daily traffic value. The value of N for cross-frames depends on the number of stress cycles per truck passage, n, which for transverse members is affected by the spacing of the cross-frames (Table D6.6.1.2.5-2). The 75-year $(ADTT)_{SL}$ values above which the infinite life check governs given in Table D6.6.1.2.3-2 assume one stress range cycle per truck passage (i.e., $n = 1.0$). For other values of n, the values in the table must be divided by n. If a fatigue life other than 75 years is sought, the table values must be multiplied by the ratio of 75 divided by the fatigue life sought in years.

All details are categorized according to Table D6.6.1.2.3-1 of AASHTO LRFD. There are eight detail categories ranging from A to E′. Detail Category A is associated with base metal and is considered the best fatigue detail category. Detail Category E′ is considered the worst detail category.

One detail in Table D6.6.1.2.3-1 that is important to this research project is Condition 7.2, which deals with single angles and tee-section members welded to gusset plates by longitudinal fillet welds along both sides of the connected element of the member. In the 7th Edition of AASHTO LRFD, this detail was listed as Category E. However, research performed by McDonald and Frank (2009) and Battistini (2014) showed that this detail is actually a Category E′ detail due to the effects of connection eccentricity. This detail category was revised accordingly in the 2016 interim revisions.

D2.1.2.3 Stability Bracing Requirements

AASHTO LRFD does not currently provide minimum strength or stiffness requirements that are specific to cross-frames. The current American Institute of Steel Construction (AISC) stability provisions are discussed in Section D2.3.4.4. Developing minimum strength and stiffness requirements for cross-frames is an objective of this research project.

D2.2 Fatigue Loading for Cross-Frames

Fatigue is an important consideration in the design of cross-frames. This section of the appendix provides a summary of selected past research pertinent to determining fatigue loading demands on cross-frames.

D2.2.1 Field Load Testing of Cross-Frames

Several researchers have conducted field load tests on cross-frame systems over the last few decades. In these investigations, cross-frame members are usually instrumented with strain gages, and then trucks with a known weight are placed at various locations of the bridge. For the given truck locations, truck axle configurations, and truck axle weights, the forces in the cross-frame members are measured. These types of studies provide data that can be compared to finite element bridge models, to assess capabilities and limitations of various modeling approaches and assumptions. Several studies have investigated cross-frame behavior during the erection and deck construction stages (Romage 2008, Fasl, Romage, et al. 2009, Rowles 2014), but the primary focus is on those that studied the behavior in composite systems (Keating et al. n 1997, McConnell et al. 2016). These two studies are summarized herein.

Through the instrumentation of three steel I-girder bridges in Texas and additional analytical studies, Keating et al. (1997) primarily evaluated two distinct issues: (i) does the presence of cross-frames impact the simplified live load distribution factors used for girders in AASHTO LRFD?; (ii) are there methods to determine cross-frame forces based on simple variables (e.g. relative stiffness of the concrete deck and cross-frames)? The research team installed strain gages at select cross-frame members and conducted a controlled live load test (i.e., placed trucks of known weight and axle configurations at specified locations along the deck and measured the response). Note that only one strain gage per cross-frame cross-section was installed, such that axial and bending stresses could not be properly diagnosed. The analytical studies, although exploring a variety of support skews and other bridge parameters, were limited to simple, straight spans with two design lanes.

From these studies, the research team concluded that the AASHTO LRFD distribution factors cannot be uniformly reduced due to the presence of cross-frames. Note that, in the original development of these factors, the presence of cross-frames or diaphragms were not explicitly considered. Additionally, several plots were developed that illustrated the correlation of key bridge parameters and cross-frame force magnitudes. For instance, it was observed that, as cross-frame stiffness increased relative to the deck stiffness, cross-frame force effects also subsequently increased. From this data, the researchers offered these simple plots as tools to effectively circumvent the need for a refined analysis. In other words, rather than performing a 2D or 3D analysis, these graphs could be potentially used to estimate cross-frame design forces as a function of cross-frame stiffness and support skew. The work preferred as part of NCHRP 12-113 largely expands on these concepts by exploring a wider range of bridges, including multiple-span and curved systems.

McConnell et al. (2016) compared field-measured results with FEA solutions of two different skewed bridges in Delaware, one with contiguous lines of cross-frames (65-degree skew angle) and one with a staggered layout (32-degree skew angle). Similar to Keating et al., the researchers installed strain gages at select cross-frames and girder cross-sections. A dump truck of known weight and axle configuration was slowly driven across the bridge length (i.e., pseudo-static load conditions) at different transverse lane positions. Among other results, the researchers measured and evaluated the lateral bending stresses in the instrumented girder flanges, as well as axial and bending stresses in the instrumented cross-frame members.

In general, it was observed that the lateral bending stresses were more significant in the bridge with a staggered cross-frame layout, despite having less support skew. This statement is consistent with the commentary provided in Article C6.7.4.2 regarding lateral flange stresses and discontinuous cross-frames. In terms of cross-frame stresses, it was observed that the bridge with contiguous bracing lines produced higher cross-frame forces for comparable loading conditions. These results were attributed to the fact that (i) contiguous cross-frames increase the overall transverse stiffness of the superstructure compared to staggered layouts and (ii) the 65-degree support skew results in more substantial differential girder displacements and therefore increased cross-frame forces. Additionally, load-induced force effects were generally maximized in the end-bay cross-frame panels near the intermediate skewed supports.

Most importantly, when compared to the measured results, it was found that 3D FEA models, for which cross-frames were modeled as shell elements, produced the most accurate analytical results. McConnell et al. also utilized the simple tools developed by Keating et al. (1997) to estimate force effects but found significant discrepancies. The simple tools predicted cross-frame force effects in excess of 50% conservative (i.e., smaller) compared to the measured responses.

NCHRP 12-113 investigated similar cross-frame behaviors in skewed bridge systems but for a wider range of steel I-girder systems. The effects of staggered cross-frame layouts and skew angle, in particular, are examined extensively. It should also be noted that both Keating et al. (1997) and McConnell et al. (2016) investigated only controlled live loads in their field experiments (i.e., specified trucks of known axle weights and configurations). As presented in the final report, the present study monitored cross-frame stress ranges under (i) controlled live loads and (ii) under real in-service truck traffic for an extended period of time. To the knowledge of the authors, cross-frame stress ranges under truck traffic on a composite bridge deck had never been extensively measured and evaluated prior to NCHRP 12-113.

D2.2.2 Multiple Presence Factors

Throughout the service life of the bridge, multiple trucks will be present in a single bridge span simultaneously. Therefore, it is important to consider the effects of multiple truck presence when designing a bridge. The multiple presence factor in the 9th Edition of AASHTO LRFD is not supported by WIM data. Thus, the research performed by Bowman (2012) and Fu (2013) derived a multiple presence factor (MPF) based upon WIM truck data. The data was collected for 436 months from 43 sites with approximately 68 million trucks. According to the research, the following three factors have a major role in the MPF: span length, ADTT, and number of lanes. The equations for calculating the MPF per Fu (2013) is given in the following expressions:

$$\begin{aligned} MPF &= \frac{N-lane\ load\ effect\ in\ component}{One-lane\ load\ effect\ in\ component} \times \frac{1}{\#\ of\ lanes} \\ &= \frac{LE_{total}}{LE_{onelane}} \times \frac{1}{\#\ of\ lanes} \end{aligned} \tag{D2.6}$$

$$\frac{LE_{total}}{LE_{onelane}} = \frac{\sqrt[3]{\sum f_i LE^3_{i\_total}}}{\sqrt[3]{\sum f_j LE^3_{j\_onelane}}}\ (i,j = 1,2,3,\dots) \tag{D2.7}$$

In the equation above, LE is the fatigue load effect measured from WIM data. From the test results, the research proposes the MPF for strength and fatigue limit states.

$$MPF_{fatigue} = \frac{0.988 + 6.87 \times 10^{-5}\ span\ length + 4.01 \times 10^{-6}\ ADTT + 1.07 \times 10^{-2}/N}{N} > \frac{1}{N} \tag{D2.8}$$

$$MPF_{Strength} = \frac{-0.081 + 1.08 \times 10^{-3}\ span\ length + 1.33 \times 10^{-4}\ ADTT + 2.10/N}{N} > \frac{1}{N} \tag{D2.9}$$

Fu found that the current code-specified MPFs can be overly conservative for the strength limit state by as much as 400%. Although the AASHTO LRFD currently ignores multiple presence for fatigue design, the MPF developed by Fu (2013) provides some guidance on this issue.

D2.2.3 Proposed AASHTO Updates by Modjeski and Masters (2015)

As discussed earlier in this chapter, recommendations from the SHRP 2 Project R19B research conducted by Modjeski and Masters (2015) led to Fatigue I and Fatigue II load factor changes in the 8th Edition of AASHTO LRFD. The research by Modjeski and Masters was focused on providing guidance for 100-year design life by developing design and detailing guidelines, as well as calibrated service limit states (SLSs) using reliability theory. Based on a survey of bridge owners and a review of national and international literature, the following SLSs were developed for calibration: foundation deformations, reinforced concrete component cracking, live load deflections, permanent deflections, prestressed concrete component cracking, reinforced concrete component fatigue, and steel fatigue. The researchers for this work used WIM data from 32 bridge sites across the country that included over 35 million useful records (after filtering) to form their final recommendations.

Three outcomes of the Modjeski and Masters research are of particular interest to this research. The first outcome is the aforementioned Fatigue I and Fatigue II load factor changes. In the 7th Edition of AASHTO LRFD, the load factors for Fatigue I and Fatigue II limit states were 1.5 and 0.75, respectively. One of the objectives of the Modjeski and Masters research was the development of statistical parameters (i.e. bias and coefficient of variation) of fatigue loading by using WIM data. Based on the findings from the WIM data, truck traffic simulation, rainflow cycle counting, and Monte Carlo simulation, Modjeski and Masters proposed to update the load factors for the fatigue limit states to 2.0 and 0.8 for Fatigue I and II, respectively, to account for current and project truck loads.

Further analysis of the statistical parameters determined that a value of 1.75 for the Fatigue I limit state was more appropriate. Accordingly, the current 9th Edition AASHTO LRFD specifies load factors of 1.75 for the Fatigue I limit state (per the additional analysis of the WIM data), and 0.8 for the Fatigue II limit state, based on the original recommendation made by Modjeski and Masters. These recommendations are summarized in Table D2-1.

Table D2-1: Summary of recommended updates to AASHTO LRFD load factors based on Modjeski and Masters (2015)

Fatigue Limit State Load Combination	Live Load Factor (Modjeski and Masters Recommendation)	AASHTO LRFD 9th Edition
Fatigue I	~~1.50~~ → **2.0**	**1.75**
Fatigue II	~~0.75~~ → **0.8**	**0.8**

The second outcome was the design and detailing recommendations to change the constant A for categories D, E, and E′. The report also suggests changing the CAFT values for categories B′, D, and E′. These changes were due to the fact that when the proposed load factor changes were applied to the statistical data, the reliability indices were too large (exceeding +/- 0.2 on the reliability index). Instead of changing the resistance factor for select detail categories, Modjeski and Masters recommended altering the constant A and CAFT appropriately. It should be noted that these recommended changes were not made in the 8th Edition of AASHTO LRFD. These recommendations are summarized in Table D2-2.

The third outcome was the validation of the single truck, single lane placement for fatigue limit state design. While the research indicated this placement is appropriate (even with the rare occurrence of passing trucks), the WIM data suggested the cycles per passage approach currently used in AASHTO LRFD could be simplified.

Table D2-2: Summary of recommended updates to AASHTO LRFD fatigue categories based on Modjeski and Masters (2015)

Detail Category	Constant A times 10^8 (ksi^3)
D	~~22.0~~ 21.0
E	~~11.0~~ 12.0
E′	~~3.9~~ 3.5

Detail Category	Threshold (ksi)
B′	~~12.0~~ 13.0
D	~~7.0~~ 8.0
E′	~~2.6~~ 3.1

D2.3 Stability Bracing Strength and Stiffness Requirements

Ss noted previously, cross-frames primarily serve as discrete, torsional braces to enhance the LTB resistance of girders, particularly during erection and deck construction. As such, this section provides background information on lateral-torsional buckling of doubly-symmetric I-girders as well as a summary of work related to the stability bracing requirements torsional bracing systems.

D2.3.1 Lateral Torsional Buckling of Doubly-Symmetric I-Girders

Timoshenko derived the expression for the critical moment for lateral-torsional buckling in a doubly-symmetric section. (Timoshenko 1961). Timoshenko's original derivation is applicable to uniform moment on a beam free to warp at the ends of the unbraced length. The expression is given in the following equation:

$$M_{cr} = \frac{\pi}{L_b}\sqrt{EI_yGJ + \frac{\pi^2E^2C_wI_y}{L_b^2}} \qquad D2.10$$

where: M_{cr} = buckling moment; L_b is the unbraced length of the beam; E = modulus of elasticity; I_y = weak-axis moment of inertia; G = shear modulus of elasticity; J = torsional constant; and C_w = torsional warping constant.

There are two terms under the radical of the equation with the first term related to the St. Venant torsional stiffness and the second related to the warping torsional stiffness. One of the primary roles of cross-frames in steel girder bridges is to provide bracing for lateral torsional buckling. Provided the bracing is properly sized, the unbraced length, L_b in Eq. D2.10 is the spacing between the cross-frames. The sizing of the cross-frame should consider the fundamental requirements of effective stability bracing, which is discussed in the following subsections.

D2.3.2 Stability Brace Requirements

Winter (1960) was the first to demonstrate that effective stability bracing must satisfy both stiffness and strength criteria. The initial work of Winter was focused on lateral bracing for columns; however, he also extended the application to lateral bracing for beams. His work consisted of modeling the member to braced as a series of rigid links with hinges at the lateral brace points. Winter's model allowed the "ideal brace stiffness" to be determined, which is the stiffness necessary to just force a perfectly straight member to buckle between the brace points. The model also allowed the impact of imperfections to be simply evaluated. Winter's work demonstrated that a stiffness larger than the "ideal" stiffness needed to be provided to control out-of-plane deformations and brace forces.

Although Winter's work on lateral bracing was extended to beam bracing, there are many factors that impact the bracing behavior of beam systems. Therefore, there have been a number of studies over the years to study the bracing behavior and requirements for effective beam bracing. In general, beam bracing can be achieved by restraining lateral deformation (lateral bracing) or by restraining twist of the section (torsional bracing). Cross-frames fit into the category of torsional bracing since the braces restrain twist of the section. The following subsection provides a summary of previous investigations that have focused on stability bracing for beams.

D2.3.3 Fundamentals of Beam Bracing

While effective beam bracing can be achieved by providing either lateral bracing or torsional bracing, this appendix focuses primarily on torsional bracing since cross-frames fit into this category. There have been a number of studies on torsional bracing systems, however, the fundamental work that is most significant to torsional bracing of beams if from Taylor and Ojalvo (1966) and Yura (1992, 2001).

Taylor and Ojalvo (1966) studied the effectiveness of both continuous and discrete torsional restraints. The research produced the following expression that could be used to estimate the buckling capacity of a doubly-symmetric beam with continuous torsional restraint and uniform moment:

$$M_{cr} = \sqrt{M_0^2 + \bar{\beta}_b E I_y} \qquad D2.11$$

where: M_0 = buckling capacity of the beam if it were unbraced; and $\bar{\beta}_b$ = continuous torsional brace stiffness.

Yura (1992, 2001) carried out detailed studies on stability bracing of both lateral and torsional bracing of doubly- and singly-symmetric I-sections. The investigations included both experimental and parametric finite element studies that provided an in-depth understanding of the many factors that impact the behavior of beam bracing. As part of the work summarized in the 1992 research report by Yura, 76 experimental tests were performed. The tests demonstrated the impact of several key parameters including brace stiffness, brace location, stiffener size, initial imperfections, moment gradient, and location of the load impacts the behavior of torsional braces.

Yura (1992, 2001) modified the expression by Taylor and Ojalvo to include the impact of the many variables the impact beam bracing. The modified equation is given in the following expression:

$$M_{cr} = \sqrt{C_{bu}^2 M_o^2 + \frac{C_{bb}^2 n \beta_T E I_y}{2 C_T L}} < M_y \ or \ M_{bp} \qquad D2.12$$

where: M_o = buckling capacity of the beam if it were unbraced; C_{bu} and C_{bb} = two limiting factors corresponding to an unbraced beam and an effectively braced beam respectively; n = number of intermediate braces; β_T = torsional stiffness of a single brace or cross-frame (the calculation of this parameter is discussed later in this section); I_y = moment of inertia of the section about an axis through the web (weak axis moment of inertia); C_T = top flange loading modification factor ($C_T = 1.2$ for top flange loading; $C_T = 1.0$ for all other loading conditions); and L = length of the span.

The terms M_y or M_{bp} on the right of the inequality represent the upper limit on the applicability of the equation. M_y is the yield moment (this could be replaced with the plastic moment capacity if desired), while M_{bp} represents the buckling capacity of the girder corresponding the buckling between the brace points.

D2.3.4 Total System Stiffness of a Discrete Torsional Brace

For a given maximum design moment, Eq. D2.12 can be used to solve for the required stiffness of the torsional bracing system (i.e. the stiffness of the cross-frame system). There are generally three components of the bracing system that impact the bracing behavior for torsional bracing systems: i) stiffness of the brace (cross-frame), ii) effective resistance of cross sectional distortion, and iii) the in-plane stiffness of the brace. From a stiffness perspective, many bracing systems follow the equation for springs in series. Considering the three stiffness components of a torsional bracing system, the total system stiffness is given by the following expression (Yura et. al., 1992):

$$\frac{1}{\beta_{Ts}} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} \qquad D2.13$$

where: β_{Ts} = total torsional stiffness of the bracing system; β_b = stiffness of the brace; β_{sec} = stiffness of the cross section considering cross-sectional distortion; and β_g = function of the in-plane girder stiffness.

From a design perspecitve, the Eq. D2.13 can be used to solve for the required stiffness of the torsional bracing system, $\beta_{T\,req'd}$. For a safe design, the total torsional brace stiffness $\beta_{Ts} \geq \beta_{T\,req'd}$. Mathematically, β_{Ts} will be less than the smallest of the three stiffness components. Therefore, it is important to consider all three components. The three stiffness components are discussed in detail in the following subsections.

D2.3.4.1 Torsional Brace Stiffness, β_b

The brace stiffness is a function of the geometry of the cross-frame. The most common cross-frame geometries in practice consist of either "X-frames" or "K-frames" as depicted in Figure D2-6. A third type of cross-frame, the "Z-frame," is also used in practice, but is not common.

X-frame systems can be idealized in a number of different ways to develop stiffness equations. Stiffness equations for the different cross-frame systems are provided in the literature [Yura (1992, 2001), Helwig and Yura (2015)] and subsequently presented in this chapter. To understand the force distribution in the cross-frame as well as the stiffness derivations, an understanding of the demand that is placed on the cross section from a bracing perspective is necessary. Twisting in adjacent girders will typically cause cross-frames to deform as shown in Figure D2-6(a). The deformation that is shown results in one diagonal in compression while the other is in tension. Cross-frames are often comprised of single angle members that have a relatively low buckling strength. As a result, the compression diagonal is sometimes conservatively neglected and the tension diagonal is sized to so that the cross-frame has sufficient stiffness. Such a system is often referred to as a "tension-only" diagonal system. For modeling purposes, the cross-frame is often represented with a single and labeled a "Z-frame."

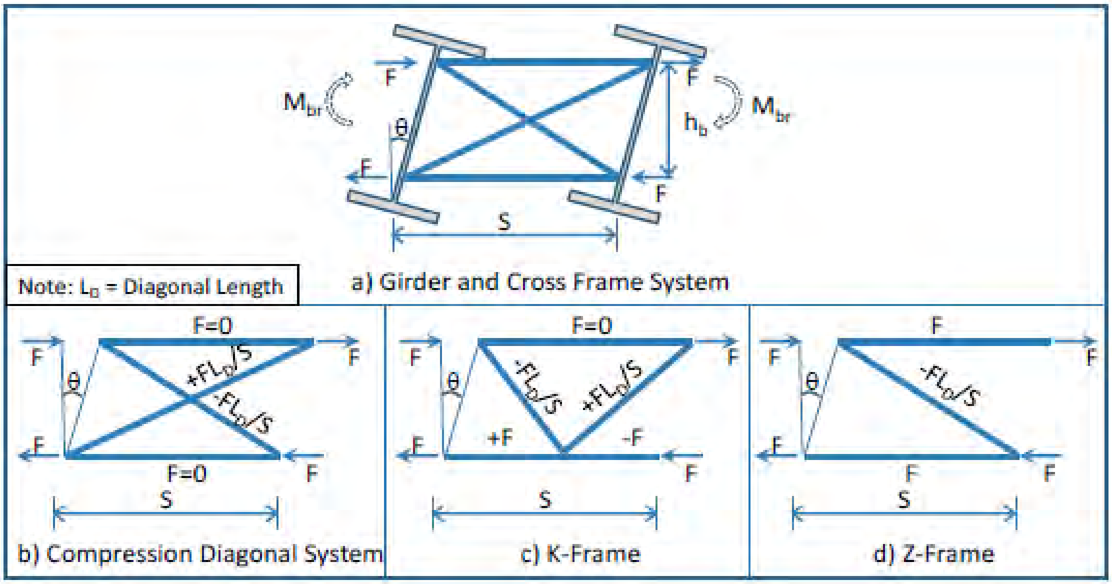


Figure D2-6: Common cross-frame analytical models

Figure D2-7 shows the force distribution in a compression X-frame. In design, the brace moment, M_{br}, for a given girder and cross-frame layout is determined using Eq. D2.22. The brace moment is idealized as a force couple as indicated Figure D2-7, with $M_{br} = Fh_b$. The free body diagram in Figure D2-7(a) shows the forces on the cross-frame as well as equal and opposite forces on the girders on either side of the cross-frame. The top and bottom struts in the compression X-frame are zero force members; however, these members are usually included since they provide some additional redundancy in the cross-frame if the compression diagonal buckles.

The axial deformations in the truss structure comprised by the cross-frame members can be determined as a function of the force couple "F" and the cross-frame geometry. The girder rotation, θ (depicted in Figure D2-6), can be determined from the relative lateral deformation at the top and bottom of the cross-frame. The stiffness of the cross-frame can then be determined from the expression: $\beta = M_{br}/\theta$ with $M_{br} = Fh_b$

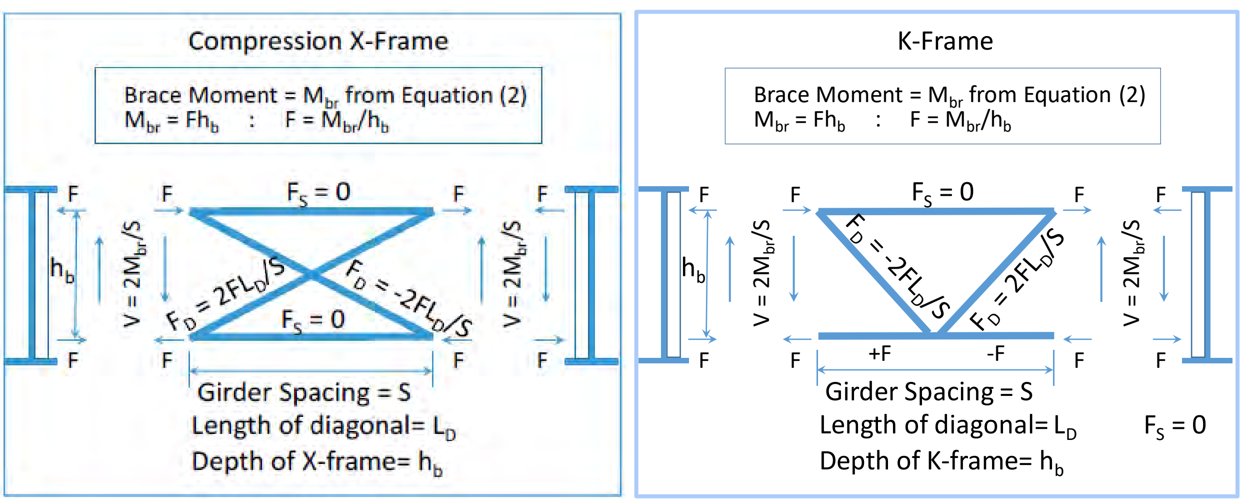


Figure D2-7: Compression X-frame and K-frame force distribution diagrams

The following equation can be used to calculate the stiffness of an X-frame represented by a compression system as shown in Figure D2-7 (Yura 1991, Yura 2001, AISC 2016):

$$\beta_b = \frac{A_D E S^2 h_b^2}{L_D^3} \qquad D2.14$$

where: A_D = cross-sectional area of the diagonals; S = girder spacing; h_b = depth of the cross-frame; and L_D = length of the diagonal members.

The following equation can be used to calculate the stiffness of an X-frame or Z-frame represented by a tension-only diagonal system as shown in Figure D2-6(d). (Yura 1991, Yura 2001, AISC 2010a):

$$\beta_b = \frac{E S^2 h_b^2}{\frac{2L_D^3}{A_D} + \frac{S^3}{A_S}} \qquad D2.15$$

where: A_S = cross-sectional area of the struts; and other parameters are defined in Eq. D2.14.

The K-frame system can be considered in a similar fashion as shown in Figure D2-7. The top strut in the K-frame is a zero-force member; however, the strut is important since it prevents a buckling mode in which the top flanges of the girders move in opposite directions. There have been some problems that have occurred when the top strut is not provided.

The following equation can be used to calculate the stiffness of a K-frame as shown in Figure D2-7 (Yura 1991, Yura 2001, AISC 2010a):

$$\beta_b = \frac{2E S^2 h_b^2}{\frac{8L_D^3}{A_D} + \frac{S^3}{A_S}} \qquad D2.16$$

The parameters are as defined in Eq. D2.14.

D2.3.4.2 Cross-Sectional Distortion, β_{sec}

Cross-sectional distortion can have a significant impact on the stiffness of the brace if the braces are shallow compared to the depth of the girder. The following equation was derived by Yura (1992, 2001) for calculating the resistance to cross-sectional distortion when full-depth web stiffeners are used:

$$\beta_{sec} = 3.3\frac{E}{h}\left(\frac{(N + 1.5h)t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \qquad D2.17$$

where: h = distance between flange centroids; N = contact length of the torsional brace (refer to Yura (2001) for further explanation); t_w = thickness of the web; t_s = thickness of the stiffener; and b_s = width of the stiffener. Since many bracing systems may not have a “contact length”, N, the stiffness expression can be rewritten as follows (Helwig and Yura (2015)):

$$\beta_{sec} = 3.3\frac{E}{h}\left(\frac{(1.5h)t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \qquad D2.18$$

The first term in the parenthesis is the effective moment of inertia of the portion of the web assumed to participate in the distortion while the second term is the moment of inertia of the stiffener about the middle of the web. Only the region outside the depth of the brace contributes to cross-sectional distortion. Although controlling web flexibility/distortion is extremely important for effective torsional bracing, because most cross-frames are relatively deep, the impact of cross sectional distortion is usually not of significant concern.

D2.3.4.3 In-Plane Girder Stiffness, β_g

Another factor that can have a dramatic impact on the effectiveness of torsional braces is the in-plane stiffness of the girders. The impact of the in-plane stiffness was first documented in Helwig et. al (1993) which demonstrated the effect for twin girder systems. The forces that develop in a cross-frame were depicted in Figure D2-7. The force components include shears that act upward on one girder and downward on the other girder. These shears cause a rigid body rotation of the girder system that reduces the effectiveness of the brace. For a twin girder, the following solution was presented to account for this effect by Helwig et. al (1993) as:

$$\beta_g = \frac{12S^2EI_x}{L^3} \qquad \textit{D2.19}$$

Where: S = girder spacing; I_x = in-plane moment of inertia of the girder; and L = span length. Yura (2001) developed an expression for systems containing more than two girders (Helwig and Yura, 2015):

$$\beta_g = \frac{24\left(n_g - 1\right)^2 S^2EI_x}{n_gL^3} \qquad \textit{D2.20}$$

where: n_g = number of girders in the system.

The in-plane girder effect is primarily important in relatively narrow girder systems such as 2- and 3-girder systems. As is discussed later in this background section, the in-plane girder stiffness effect is closely tied to the system buckling mode, for which a solution has been recently incorporated into AASHTO LRFD (Yura et. al., 2008; Han and Helwig, 2016). Because the impact of the in-plane stiffness is not a major issue with relatively wide systems, the expression is not included in the AISC (2010a).

D2.3.4.4 Current AISC Provisions

The studies conducted by Yura (1992, 2001) resulted in the foundation of the torsional bracing stiffness and strength requirements that were incorporated into the 3rd Edition of the AISC LRFD Design Specification for Structural Steel Buildings (1999). While AISC is primarily considered a specification for steel buildings, the majority of the work leading to the AISC provisions was funded by the Texas Department of Transportation (TxDOT) and therefore the work is directly applicable to bridge systems.

The stiffness equation that is in the AISC specification was developed from Eq. D2.12. The first term in the radical that represents the contribution of the girder with no intermediate (between the supports) bracing was conservatively neglected. The left hand side of the equation (M_{cr}) was then set equal to the maximum design moment (M_r) and the expression was solved for the required stiffness (assuming top flange loading so $C_T = 1.2$). Including a resistance factor, results in the stiffness equation that is in the AISC specification (2010) and given below as Eq. D2.21. The stiffness is based upon providing twice the ideal stiffness to control brace forces. Winter's work on column bracing showed that providing twice the ideal stiffness resulted in a deformation at the brace that was equal to the initial imperfection so that the brace force was the brace stiffness times the magnitude of the initial imperfection. Whereas lateral column bracing is based upon a lateral sweep of $L_b/500$, which comes from the AISC Code of Standard Practice (2010b) on erection practices, imperfections for beams should include some amount of twist. Work by Wang and Helwig (2005) showed that the critical imperfection shape for torsional bracing consists of a lateral sweep of the compression flange while the tension flange remains straight. The resulting twist is $L_b/500h_o$. With twice the ideal stiffness, the brace moment is $\beta_T\theta_o$ as given by Eq. D2.22:

$$\beta_T = \frac{2.4LM_r^2}{\phi nEI_{yeff}C_b^2} \tag{D2.21}$$

$$M_{br} = \beta_T\theta_o = \frac{2.4LM_r^2}{\phi nEI_{yeff}C_b^2} * \frac{L_b}{500h_o} \tag{D2.22}$$

where: β_T = system torsional brace stiffness; θ_o = initial twist from the girder imperfection; L = span length; M_r = maximum factored moment within the span; ϕ = resistance factor taken as 0.75; n = number of intermediate (between the supports) cross-frames within the span; and E = modulus of elasticity. The effective weak-axis moment of inertia is taken by the following expression $I_{yeff} = I_{yc} + \left(\frac{t}{c}\right)I_{yt}$, where I_{yc} = weak axis moment of inertia of compression flange; I_{yt} = weak axis moment of inertia of tension flange; t = centroidal distance from tension flange; c = centroidal distance from compression flange; C_b = moment gradient factor in region of maximum moment in span; L_b = spacing between cross-frames; and h_o = distance between flange centroids.

In discussing the bracing provisions that are in the AISC Specification, references are made below to the 14th Edition of the specification (AISC 2010a) as well as the 15th Edition that recently was published (AISC 2016). Eq. D2.21, which defines the torsional brace stiffness required was moved from the Commentary of the 14th Edition of AISC to Appendix 6 of the Specification in the 15th Edition. The difference between the 14th Edition expression and Eq. D2.21 is inclusion of the term I_{yeff} , that makes the expression applicable for both singly- and doubly-symmetric I-shaped sections. There was a change in the torsional brace moment M_{br} , in the 15th Edition based upon recommendations from Prado and White (2015).

The studies by Prado and White focused on relatively short unbraced lengths and the necessary brace forces to withstand significant inelasticity. Recently some cases were identified where the required torsional brace moment in the 15th Edition may be unconservative. This is a major component of the present research study. For bridge applications, Eq. D2.22 will provide accurate estimates of the design forces. As noted earlier, the bracing demand in the negative moment region may be critical in the finished bridge. However, although some inelasticity may occur in the vicinity of the brace, typical design procedures conservatively neglect the torsional restraint from the concrete deck and Eq. D2.22 should provide conservative but reasonable estimates of the corresponding brace forces.

D2.3.5 System Buckling of Narrow Girder Systems

The bracing provisions discussed thus far have focused on the necessary stiffness and strength of the bracing to reduce the unbraced length and thereby improve the lateral-torsional buckling resistance. While, for many problems, reducing the spacing between the cross-frames enhances the buckling resistance of the girders, in recent years the profession has become aware of a system mode of buckling that is relatively insensitive to the spacing between the braces. Relatively narrow systems such as two- and three-girder bridges are the most susceptible to the mode. The mode of buckling was originally discovered while considering the Marcy Pedestrian Bridge collapse in 2002; however, shortly after the discovery problematic bridges were identified such as the twin girder bridge widening shown in Figure D2-8. During placement of the concrete deck, the girder experienced significant twist causing a lateral movement of more than 10 inches of the bottom flange relative to the top flange.

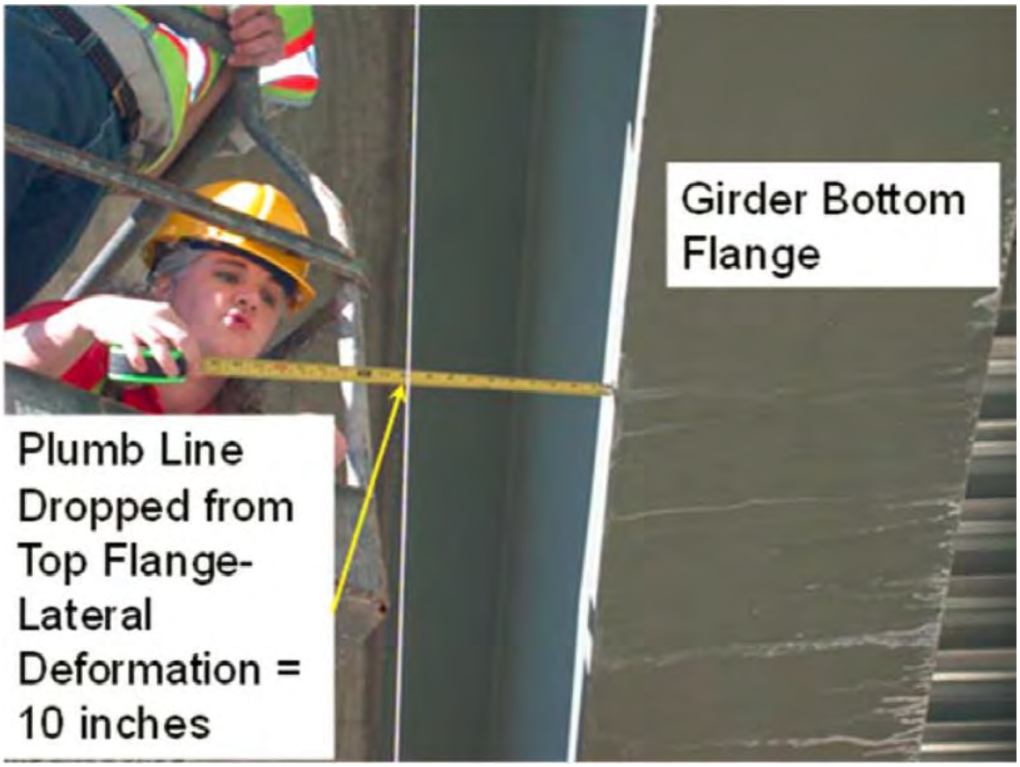


Figure D2-8: System buckling of twin I-girders systems

A solution for predicting the critical system buckling mode of the girder system was derived and published in Yura et al. (2008) and is given in the following expression:

$$M_{gs} = \frac{\pi^2 SE}{L^2}\sqrt{I_{eff} I_x} \qquad D2.23$$

where: M_{gs}= total moment resistance of the girder system; I_x = strong axis moment of an individual girder; L = span of the girder system under consideration; and the other terms have been previously defined.

Eq. D2.23 was derived analytically and found to have good agreement with critical buckling loads determined from an eigenvalue buckling analysis. Subsequent studies by Sanchez and White (2012) that included large displacement analyses on imperfect girder systems found that significant second-order amplification is possible in the system buckling mode. As a result, when Eq. D2.23 was originally incorporated into AASHTO LRFD, an upper limit of 50% of the predicted capacity was included to avoid large second order effects. More recently, additional large displacement analyses have been carried out looking at the impact of moment gradient on the system buckling of continuous girders as well as the propensity for second order effects (Han and Helwig, 2016). The studies found a number of mitigating factors that limit second-order amplification. The studies identified that the "critical shape" imperfection is often not likely to occur since the cross-frames will usually cause nearly a "pure sweep" imperfection with very little twist. As a result, the limit on the critical buckling equation was raised from 50% to 70% in the 8th Edition of AASHTO LRFD. In addition, the new provisions incorporate moment gradient factors of 1.10 for simple spans and partially erected continuous girders and 2.0 for fully erected continuous spans.

The system mode of buckling is interrelated with the torsional brace stiffness requirements outlined earlier. From a behavioral perspective, the system mode of buckling generally controls when the in-plane girder stiffness component (β_g) given in Eq. D2.20 is less than the total required torsional stiffness predicted by Eq. D2.22 (or Eq. D2.11 if the full torsional buckling equation is utilized).

D2.3.6 Stability of Curved Girder Bridges

Sanchez and White (2012) described the implications of structural stability of curved steel I-girders during construction using a case study as an example. In the case study, a state of incipient instability caused work to halt during the construction of a three-girder bridge unit. The authors discussed a common misunderstanding of the existing AASHTO LRFD provisions in considering the stability-critical conditions during I-girder bridge construction. Namely, the simple amplification factor provided by AASHTO LRFD does not consider global second-order amplification (which must be checked for all limit states). Without consideration of these second-order effects, the system stability of certain slender bridges can be at risk.

Sanchez and White (2012) compared one-dimensional (1D) and two-dimensional (2D) approximate analyses (line-girder and grillage models, respectively) of the case study bridge based on a refined three-dimensional (3D) analysis. The primary conclusions of the study were that significant nonlinear effects in slender, curved bridge units may begin at load levels well below the global buckling load, and that this behavior is not identified in first-order analyses. As noted above, the work by Sanchez and White (2012) led to the limit on the elastic critical buckling load to control second order effects.

D2.4 Influence of Cross-Frame Member End Connection Eccentricity on Cross-Frame Stiffness

As mentioned previously, the stiffness of a cross-frame system follows the behavior of springs in series. The effective stiffness of cross-frames can be calculated using Eq. D2.13, which is repeated here for convenience:

$$\frac{1}{\beta_{Ts}} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} \qquad D2.13$$

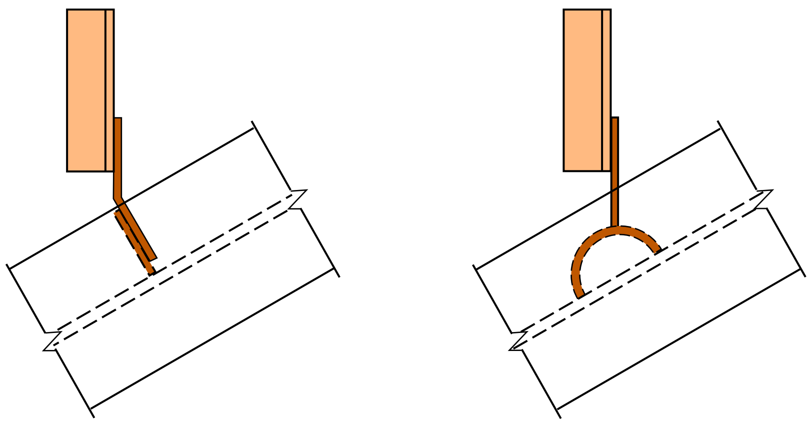
This expression shows that the effective stiffness of a cross-frame will be less than the least stiff component in the system. If the stiffness related to cross sectional distortion, β_{sec}, and the in-plane stiffness of the girder, β_g, are sufficient, but if the stiffness of the brace, β_b, is low, then the effective stiffness will suffer due to the flexible brace. This could potentially render the cross-frame inadequate from a stiffness standpoint. This example illustrates the importance of each component of the total system stiffness.

If the total system stiffness of a cross-frame is underestimated (i.e., the actual cross-frame stiffness is greater than assumed in the analysis), the result will be conservative from a stability bracing standpoint. However, this same situation may be unconservative from a fatigue standpoint. If the cross-frame is stiffer than considered in analysis, then the actual stress ranges experienced by the cross-frame will be higher than those computed in the analysis. This could potentially result in fatigue issues at some point in during the life of a bridge. The opposite is also true. If the total system stiffness is overestimated, the actual cross-frame that gets constructed will have less stiffness than assumed in the analysis. This is unconservative from a stability standpoint, and conservative from a fatigue standpoint. The overestimate can lead to the prediction of stress ranges that the actual cross-frame will not experience, potentially leading a designer to believe the cross-frame will not pass the fatigue checks. Thus, it is important that the actual stiffness of a cross-frame be represented with reasonable accuracy in the analysis of steel I-girder bridge systems.

D2.4.1 The Use of Bent Plate Connections in Skewed Bridges

The primary objective of the research performed by Quadrato (2010, 2014) was to find a way to reduce girder end twist in skewed steel I-girder bridges so that the first row of cross-frames could be located further from the abutment. The reason for wanting to move the first row of cross-frames further from the abutment is because they experience significant live-load induced forces due to differential deflections of the girders. By moving the first row of cross-frames away from the abutment, the research team suggested that the load-induced fatigue forces experienced by the cross-frames might be reduced.

The most common detail being used for end cross-frame to girder connections at the time the research was performed was a bent plate connection. One objective of the project was to investigate the behavior of the bent plate connection, and another objective was to propose an alternate connection that would improve upon the bent plate connection. The research team proposed a split pipe connection as an alternative to the bent plate. The research program included field instrumentation, two small scale laboratory testing programs, large scale laboratory testing, and parametric studies (Quadrato 2010, 2014).



(i) Bent plate connection (ii) Split-pipe connection

Figure D2-9: Typical bent plate stiffener used for skewed cross-frame layouts and split-pipe detail proposed by Quadrato (2010)

The first small scale test was performed to compare the connection stiffness of the bent plate connection and the split pipe connection. The primary concern with the bent plate connection detail is that it introduces eccentricity into the connection, which can significantly reduce the connection stiffness. Each specimen was tested under uniaxial tension and deflections were measured in both the horizontal and vertical directions. The tests demonstrated that when using the bent plate detail, deflections of the connection plates in both the vertical and horizontal directions increase as bridge skew increases. It was also concluded that increasing the stiffness of the bent plate does not have a large impact on the connection stiffness, and that as the bend radius is increased, the stiffness of the connection is reduced. The split pipe detail showed much better stiffness performance in both the horizontal and vertical directions than the bent plate stiffener. Because the split pipe detail is stiffer than the bent plate connection detail, it was concluded that the split pipe connection could provide significantly stiffer cross-frames for applications in skewed steel bridges (Battistini 2009). A second small scale test was performed to investigate the fatigue performance of the split pipe stiffener but is not discussed further.

The large-scale testing program was performed to validate the parametric studies and to compare the performance of the split pipe and bent plate connection details with multiple support and bracing conditions. The testing program was performed using one, two, and three girder setups. After performing the large-scale tests and the parametric studies, the research team confirmed that using the concentric split pipe connection detail instead of the eccentric bent plate detail would allow the first row of cross-frames to be moved further from the supports. Moving the cross-frames away from the support reduces the forces in the cross-frames because they will be in an area of smaller differential deflections. Lastly, Quadrato (2010, 2014) recommended updating the initial cross-frame effective stiffness equation (Eq. D2.13) to include the contribution to the stiffness of the connection. This is done to clearly separate the stiffness contribution of the cross-frame members and the connections. The updated equation is as follows:

$$\frac{1}{\beta_{Ts}} = \frac{1}{\beta_b} + \frac{1}{\beta_{sec}} + \frac{1}{\beta_g} + \frac{1}{\beta_{conn}} \qquad D2.24$$

D2.4.2 The Impact of Single-Angle Members on Cross-Frame Stiffness

The research of McDonald and Frank (2009) was focused on testing single-angle members that were welded to connection plates. The goals of the study were to confirm that the detail was in the appropriate fatigue category in AASHTO LRFD and to compare the performance of balanced welds and unbalanced welds.

The testing program was intended to evaluate the performance of single-angle members with gusset plates welded to each side of the angle. Each specimen was to be uniaxially loaded under tension. However, upon loading the first specimen, large amounts of bending were observed, even though the angle was loaded in tension. To avoid damaging the testing machine, further tests were performed by placing angles back-to-back. This created a system that acted more like a double-angle member, which may have improved the performance of the angles. While this research was not performed to study the impacts of eccentricity on connection stiffness, it was apparent from the testing that the use of single-angle members resulted in large amounts of out-of-plane bending (McDonald and Frank 2009).

Figure D2-10: Bending of single-angle member subjected to tension caused by eccentric connections (McDonald and Frank 2009)

The research outlined by Wang (2013) and Battistini, et. al. (2016) investigated the stiffness and fatigue behavior of three types of cross-frames: X-type frames, K-type frames, and Z-type frames. Z-type frames are not commonly used, but were being investigated as a possible alternative to X-type and K-type frames. Further, the researchers tested a variety of cross-frame member types, including single angles, double angles, and HSS sections.

The brace stiffness, β_b, contains two parts: the stiffness of the cross-frame members, such as angles, tubes, or WT sections, and the stiffness of the connections joining the cross-frame members and the girders. If either component lacks sufficient stiffness then the effective stiffness of the brace will suffer, which will cause the system stiffness of the cross-frame to suffer. In Eq. D2.13 the connection stiffness is implied to be part of the brace stiffness. To clearly separate the contributions to stiffness from the members and connections, Eq. D2.24 was recommended by Quadrato (2010, 2014), as described previously.

As part of the research program outlined by Wang (2013) and Battistini, et. al. (2016), large scale tests were performed on the three types of cross-frame configurations performed on six different cross-frame configurations: X-type cross-frames composed of equal leg, single-angle members, X-type cross-frames composed of unequal leg, single-angle members, K-type cross-frames composed of single angle members, Z-type cross-frames with a double-angle diagonal member and single angle struts, Z-type cross-frames composed of double-angle members, and Z-type cross-frames with HSS section members for both the diagonal and struts.

Each of the cross-frame specimens was tested in a setup that replicated the torsion-induced forces that would be experienced by a cross-frame in a steel bridge. Each cross-frame was instrumented with strain gauges to measure axial strains in each member and linear potentiometers to measure the rotation of the cross-frame. The axial forces experienced by each member were calculated from the strain data using the numerical regression technique outlined in (Helwig and Fan 2000). The rotations were used to calculate the brace stiffness provided by each cross-frame using the following equation (Wang 2013; Battistini et. al. 2016):

$$\beta_b = \frac{M}{\theta} \qquad D2.25$$

where: M = moment created by the force couples applied to the cross-frame by actuators; and θ = measured rotation of the cross-frame.

The data for brace stiffness values that was collected from the laboratory experiments was compared to stiffness estimates made using analytical solutions and estimates made using computer models. Two types of models were used to make stiffness predictions: a line element model and a shell element model. The analytical and line element models overestimated the stiffness by 52% to as much as 82% in cases where a single angle was used for any member of the cross-frame. In cross-frames made up concentric members such as double-angles or HSS sections, the error between the analytical solution and experimental data was very small. In all cases, the stiffness prediction made using shell elements agreed with the test data.

From the comparison of test results, analytical solutions, and models, it is apparent that the reason for the discrepancy in the stiffness values was the use of single-angle members. The eccentric connection of single-angle members led to large reductions in stiffness that were not captured by the analytical or line element solutions but were captured very accurately in by the 3D shell element models.

Following the comparison of the analytical, line element, and shell element solutions to the experimental data, a parametric FEA analysis was performed to study the impact of different connection variables. The shell element models that were compared to the experimental data were used as a basis for the parametric studies because of the strong agreement with the experimental data. The variables included in the parametric study to include the brace height, girder spacing, angle leg width, and angle leg thickness. Table D2-3 summarizes the parameters evaluated in the parametric studies. In this table, h_b is the cross-frame height, S is the girder spacing, b is the angle leg width, and t is the angle leg thickness. The thickness of the connection plate was not considered because standard details call for ½-inch plates to be used. A thicker connection plate (or gusset plate) will increase the eccentricity (a detriment to the stiffness) and also increases the plate bending resistance (a benefit to the stiffness). The research team did not consider that the thickness of the connection plate or the gusset plate had a major impact on the stiffness of the cross-frame; however, this will be confirmed as part of the current research project.

Table D2-3: Variables in the FEA parametric study for X-frames and K-frames

$\mathbf{h_b}$ (in)	$\mathbf{S}$ (in)	Range of $\mathbf{S/h_b}$	$\mathbf{b}$ (in)	$\mathbf{t}$ (in)
48	96, 108, 120, 132, 144	2-3	3, 4	1/4, 3/8, 1/2, 5/8
60	96, 108, 120, 132, 144	1.6-2.4	3, 4	1/4, 3/8, 1/2, 5/8
72	96, 108, 120, 132, 144	1.3-2	4, 5	1/4, 3/8, 1/2, 5/8
84	96, 108, 120, 132, 144	1.1-1.7	4, 5	1/4, 3/8, 1/2, 5/8
96	96, 108, 120, 132, 144	1-1.5	5, 6	1/4, 3/8, 1/2, 5/8

The goal of the parametric analysis was to determine which of these variables had the most significant impact on the stiffness of the cross-frame. It was determined that the three most important variables were the girder spacing to cross-frame height ratio, the angle member eccentricity, and the thickness of the angle member. The data collected from the parametric study was used to create the following stiffness reduction factors that can be used for single-angle cross-frames (Wang 2013; Battistini, et. al. 2016):

$$R_{XFrame} = 1.062 - 0.087\left(\frac{S}{h_b}\right) - 0.159\bar{y} - 0.403t \qquad D2.26$$

$$R_{KFrame} = 0.943 - 0.042\left(\frac{S}{h_b}\right) - 0.048\bar{y} - 0.420t \qquad D2.27$$

These correction factors can be used to modify the stiffness estimates made by analytical solutions or line element solutions. Once a designer has calculated the reduction coefficient using the equation for the appropriate cross-frame type, the coefficient can be multiplied by the area of the cross-frame members to account for the reduction in stiffness caused by the eccentricity of the connection. It should also be noted that a version of these correction factors can be found in Article C4.6.3.3.4 of AASHTO LRFD. The article currently recommends applying a reduction factor of 0.65 to the axial stiffness of equal leg angles, unequal leg angles connected to the long leg, and flange-connected tee-section members.

CHAPTER D3

Industry Survey

A survey was developed and distributed by the RT to gather information from Departments of Transportation and consultants with respect to (i) commonly used software packages, (ii) fatigue issues with cross-frames encountered during the design process, and (iii) load-induced fatigue issues with cross-frames encountered in existing bridges. The survey provided valuable information that helped direct the RT and the scope throughout the project. The following sections outline the development of the survey, and a summary of the data collected.

D3.1 Development and Distribution

The survey was developed electronically using the survey platform Qualtrics, which allows survey recipients to respond electronically using a single link. The purpose of the survey was to get input on cross-frame fatigue design from the Departments of Transportation and consultants that have experience with steel I-girder bridge design. The survey can be found in Chapter D6. It should be noted that the survey found in Chapter D6 is not the electronic survey that was distributed to DOTs and consultants, but the content is the same. In total, eight questions were asked, as outlined below.

Questions one and two were intended to gather information about the organization and people participating in the survey. This information included the organization name and location, as well information on the respondent including name, position within the organization, and contact information. It was noted in the survey that providing personal information was optional.

The third question was intended to gather information about which software packages are most commonly used for steel I-girder bridge design and for which types of bridges (i.e. straight with normal supports, straight with skewed supports, or horizontally curved).

Question four was intended to gather information on the use of software to perform load-induced fatigue checks for cross-frames in steel I-girder bridges and to gather information on any concerns users might have about how the software checks fatigue of cross-frames. Lastly, if a user indicated that they did not use a specific software package to check fatigue in cross-frames, they were asked how they evaluated fatigue in the braces (if at all).

Question five was intended to determine whether organizations primarily targeted infinite fatigue life, or infinite fatigue life and finite life where applicable. One of the goals of this question was to determine how organizations evaluate cross-frames that usually have E′ connection details.

Question six was intended to collect information about any load-induced fatigue issues organizations encountered in cross-frames of existing steel I-girder bridges.

Question seven was used to collect various details for straight bridges with normal supports or skewed supports and horizontally curved bridges. Understanding the various typical details each DOT uses is important to the RT. With this understanding, a more meaningful parametric study could be performed.

Finally, question eight provided a comment box that allowed respondents to provide additional input related to load-induced fatigue. The survey was distributed to members of all 50 DOTs. The RT requested that the DOTs also send the survey to consultants to diversify the responses.

D3.2 Survey Results

There were 57 responses to the survey distributed by the RT. Of the 57 responses, 36 were responses from DOTs. The Texas DOT contributed three responses from different locations, and the New Hampshire DOT contributed three responses from the same location. In total, 64% of the states responded for a total of 32 unique state responses. 17 of the responses were from consultants, and four of the responses were from other sources including universities, erectors, and anonymous respondents. Figure D3-1 shows a chart of the survey response distribution.

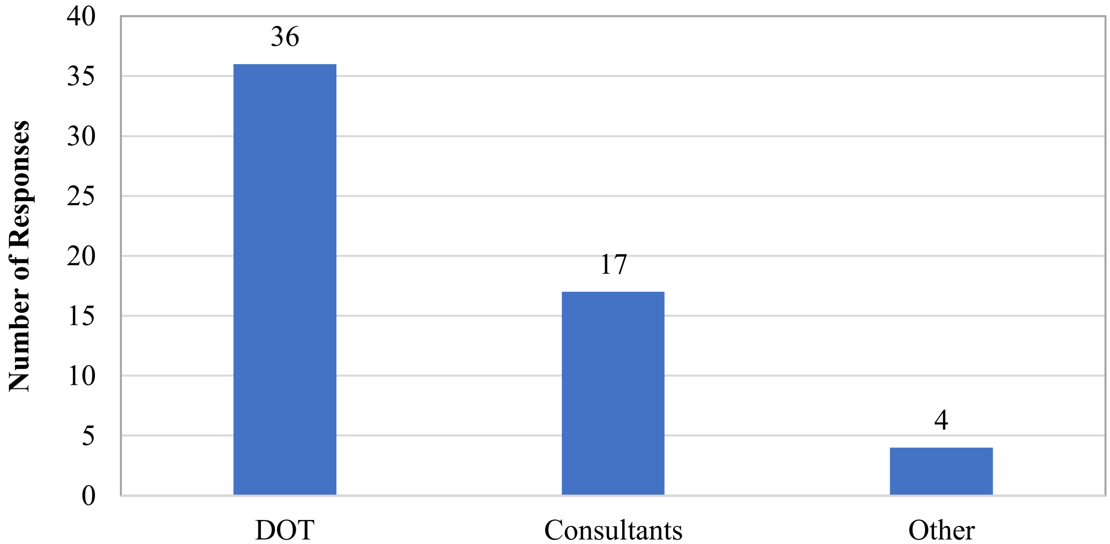


Figure D3-1: Survey response distribution

Because of the high response rate, especially from DOTs, the RT believes the results of the survey was a reliable tool with respect to focusing research efforts in the remaining phases of the project. Figure D3-2 shows the distribution of DOT responses across the United States.

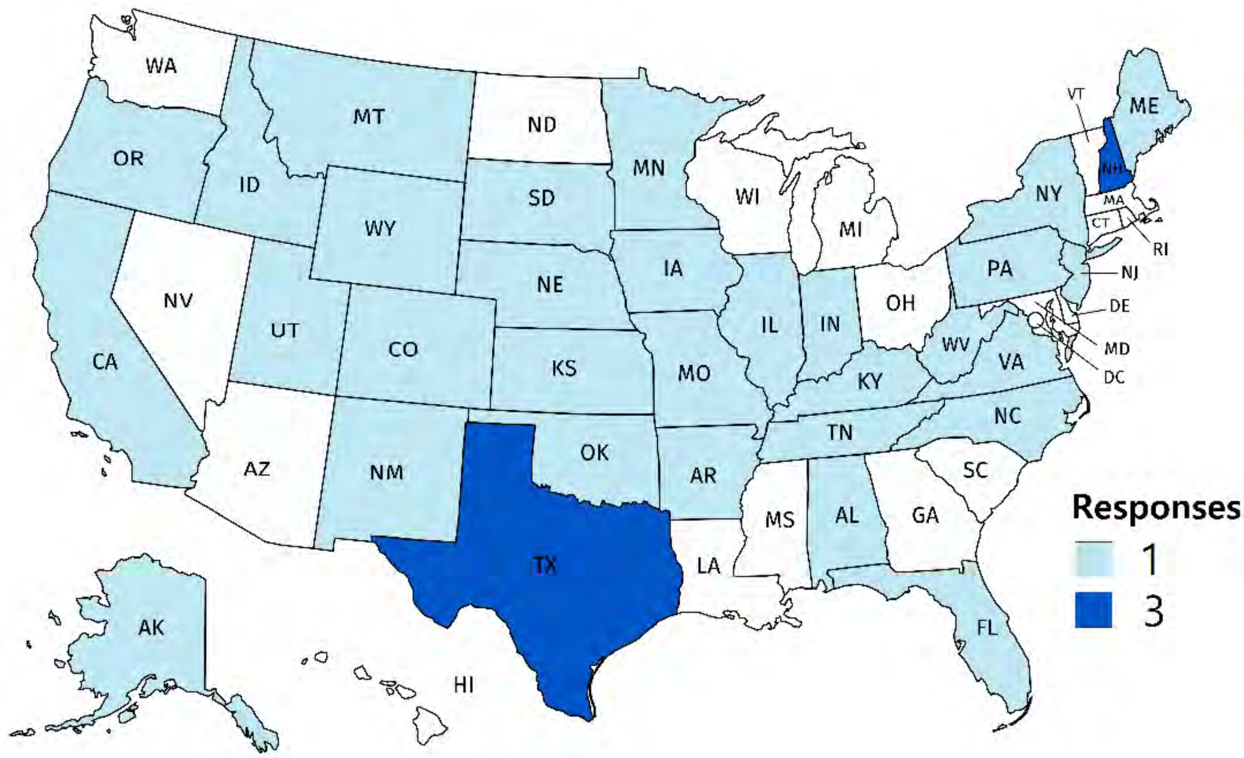


Figure D3-2: Distribution of DOT responses across the United States

D3.2.1 Software and Fatigue Checks

The results of the industry survey indicate that a wide range of software packages are used for bridge design in the United States. Only software packages that received more than three votes are shown in the tables below, however, all software packages (twenty-three software packages named by the survey participants) are included when "total" values are discussed.

To avoid creating "advertising" for or against a given software package, the names of the programs are not included in this appendix. As such, the commercial software packages have been identified arbitrarily with a letter such as "Software A." Table D3-1 through Table D3-4 summarize the results of question three, which was intended to collect information on which software packages are most commonly used for the design of steel I-girder bridges. Note that Software A and B listed in the subsequent tables are consistent with the programs discussed extensively in the main report.

Table D3-1: Software packages used for both straight and horizontally curved I-girder bridge design

Software Package	DOTs	Consultants	Other	Total
Software B	15	11	1	27
Software C	11	5	1	17
Software A	8	4	1	13
Software D	6	4	1	11
Software E	4	6	1	11
Software F	7	2	0	9
Software G	8	0	0	8
Software H	5	5	0	10
Software I	4	5	0	9
Software J	2	6	0	8
Software K	2	2	0	4

Table D3-2: Software packages used for design of straight I-girder bridges with normal supports

Software Package	DOTs	Consultants	Other	Total
Software B	15	8	1	24
Software C	11	5	1	17
Software A	4	2	1	7
Software D	2	4	0	6
Software E	2	2	1	5
Software F	6	2	0	8
Software G	6	0	0	6
Software H	4	1	0	5
Software I	3	5	0	8
Software J	2	5	0	7
Software K	0	1	0	1

Table D3-3: Software packages used for design of straight I-girder bridges with skewed supports

Software Package	DOTs	Consultants	Other	Total
Software B	14	8	1	23
Software C	9	2	0	11
Software A	7	4	1	12
Software D	2	3	1	6
Software E	4	6	1	11
Software F	7	2	0	9
Software G	5	0	0	5
Software H	4	5	0	9
Software I	3	5	0	8
Software J	2	2	0	4
Software K	2	2	0	4

Table D3-4: Software packages used for design of horizontally curved I-girder bridges

Software Package	DOTs	Consultants	Other	Total
Software B	14	8	1	23
Software C	0	1	0	1
Software A	8	4	1	13
Software D	4	4	1	9
Software E	4	6	1	11
Software F	5	2	0	7
Software G	4	0	0	4
Software H	5	5	0	10
Software I	1	1	0	2
Software J	0	0	0	0
Software K	2	2	0	4

Referring to Table D3-1, Software B received the most votes overall, with a total of 27 votes. It also received the most votes for each type of bridge configuration. Software C received 17 votes overall. It is popular for the design of straight bridges with normal or skewed supports, but it only received one vote for the design of horizontally curved bridges.

Question four of the survey was intended to gather information on the use of software to perform load-induced fatigue checks in cross-frames of steel I-girder bridges. The responses to question four are summarized in Table D3-5.

Table D3-5: Responses to: "To your knowledge, can the analysis software your organization has used perform fatigue design checks of cross-frames in steel I-girder bridges?"

Software Package	Yes	No	Not Sure
Software B	14	6	7
Software C	2	12	3
Software A	4	4	5
Software D	4	4	3
Software E	3	5	3
Software F	0	3	6
Software G	0	2	4
Software H	4	2	4
Software I	0	6	2
Software J	0	6	3
Software K	1	1	2

Respondents indicated 36 times that the software used by their organization for the design of steel I-girder bridges has fatigue design check capabilities. Respondents indicated 104 times that the software could not perform fatigue design checks or that they were unsure if it had such capabilities. All responses were considered, however, only the responses for Software A through K are shown in the tables. A response of "no" or "not sure," indicates that the respondent does not use a particular software for fatigue design checks.

When referring to Table D3-5, it should be noted that some respondents indicated "no" to this question; meaning the software does not have fatigue design check capabilities. The RT investigated the capabilities and limitations of some of the listed software packages and found that certain programs (e.g. Software B) do have fatigue design check capabilities. In the instances where respondents indicated that such a software (with fatigue design check capabilities) does not have fatigue design check capabilities, the RT interpreted the response to mean that the respondent is not aware of the capabilities, and therefore, does not use them.

If a respondent indicated that the software used by their organization could perform fatigue design checks, they were asked a follow-up question asking whether they used those features. The responses to the question are summarized in Table D3-6. Only software packages that received votes are displayed in the table.

Table D3-6: Responses to: "Do you use the software to check fatigue of cross-frames in steel I-girder bridges?"

Software Package	Yes	No
Software B	14	0
Software C	0	2
Software A	3	1
Software D	3	1
Software E	3	0
Software H	3	1
Software K	1	0

Of the 36 responses indicating that a software package can perform fatigue design checks, respondents indicated 29 times that their organization uses the software to perform fatigue design checks of cross-frames in steel I-girder bridges. These numbers suggest that, in general, if designers are aware of the software fatigue capabilities, they use them.

Next, respondents were asked if any difficulties or concerns arose when using the software to perform fatigue checks in cross-frames. Table D3-7 summarizes the responses. Only software packages that received votes are displayed in the table.

Table D3-7: Responses to "have you had any difficulties or concerns related to cross-frame fatigue checks made by the software?"

Software Package	Yes	No
Software B	8	6
Software A	0	3
Software D	2	1
Software E	0	3
Software H	1	2
Software K	0	1

Respondents indicated 12 times that they did encounter issues or had concerns related to the fatigue checks made by the software. If a respondent indicated they did have an issue or concern with the way a software package checks fatigue of cross-frames, they were then asked to elaborate on the issue. Table D3-8 summarizes the responses to the question. Only software packages that received votes are displayed in the table.

Table D3-8: Reported issues or concerns with fatigue checks performed by commercial software

Software Package	Fatigue check unclear	Software reported problem that required modification	Stress range determination is unclear	Overly conservative results
Software B	5	2	6	5
Software D	1	0	2	1

Six respondents indicated that they were unclear how their specified software program checks the fatigue limit state in cross-frames. When elaborating on this issue, respondents stated that because the software does not show the steps it takes to check fatigue, it is difficult to confirm whether the results are accurate. In fact, two respondents said that Software B reported a problem with the cross-frame fatigue check that required a design modification to correct the issue.

Similarly, eight respondents reported that the manner in which their specified software determines the stress range for fatigue design was unclear. One respondent commented that the way Software B computes cross-frame forces results in excessively conservative estimates based upon their independent comparison with a 3D FEA of a problematic bridge. This occasionally results in cross-frame fatigue controlling the design, and further analysis using a software capable of performing 3D FEA is necessary. Another respondent stated that Software B appears to be using too many lanes in calculating the fatigue stress range. Another respondent noted that their organization uses a different fatigue truck when using Software C and Software D to calculate fatigue stress ranges.

Six respondents indicated that the software produces overly conservative results for fatigue design checks. One respondent noted that this was not necessarily the fault of the software, but rather the recent change of fatigue category from E to E′ for single-angles used in cross-frames. Additionally, one respondent indicated that issues were experienced with Software M. This data was excluded because it is the opinion of the RT that this software is a general-purpose software and not specifically programmed for bridge design.

If a respondent indicated that they did not use their software to perform fatigue checks, they were asked what method was used instead. Figure D3-3 summarizes the responses to that question.

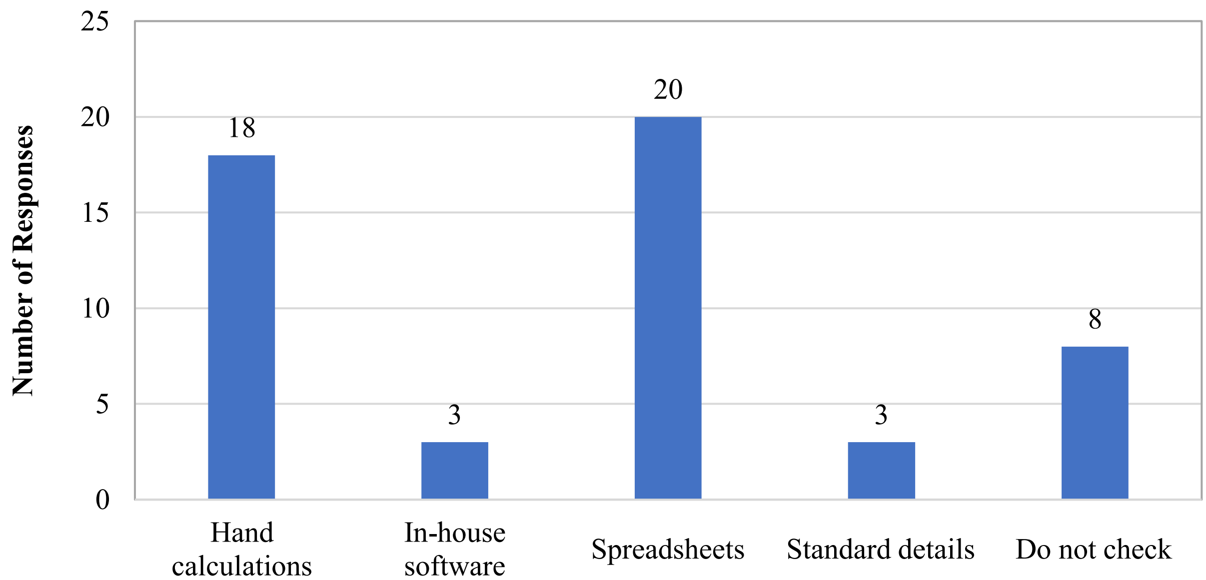


Figure D3-3: Alternate methods of performing fatigue checks if software is not used

18 respondents said they use hand calculations to perform fatigue design checks in cross-frames, three said they use in-house software, and 20 said they used spreadsheets. Some respondents indicated that they utilize some combination of spreadsheets, hand calculations, and software.

To determine the fatigue stress range to be used in the spreadsheets or hand calculations, 17 respondents indicated that they determine the stress range using one of the previously mentioned commercial software packages. Four respondents said they used a method such as the V-load method for curved bridges to determine the fatigue stress range.

Three respondents said that standard details are utilized for cross-frames. Eight indicated that they do not check fatigue in cross-frames. Of these eight, some noted that this applies only to straight bridges, and that fatigue checks would be performed for horizontally curved bridges. Others noted that their organization does not check fatigue of cross-frames, but consultants would be hired to perform the checks.

Question five asked recipients to indicate whether they design cross-frames for infinite fatigue life, finite fatigue life, or if both were considered when applicable.

Ten respondents said that they only design cross-frames for infinite fatigue life. 33 responded that they design cross-frames for infinite fatigue life and finite fatigue life, where applicable. Based on the feedback received during the industry survey, it is presumed that these 33 respondents base the appropriate limit state on AASHTO guidance, but that was not clarified. One respondent indicated they only checked cross-frames for finite fatigue life. 11 respondents responded “N/A”, which is probably because they make use of typical details or hire consultants.

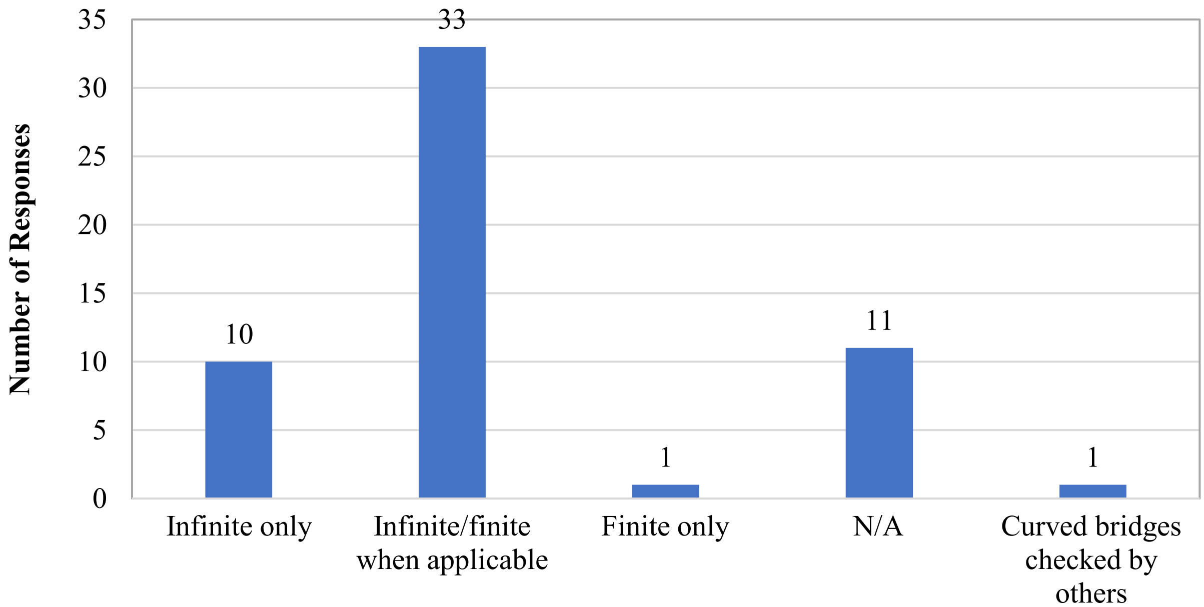


Figure D3-4: Responses to: "When checking fatigue of cross-frames in steel I-girder bridges - which of the following does your organization consider?"

D3.2.2 Load-Induced Fatigue Cracking

Survey recipients were also asked if their organization had experienced any issues due to load-induced fatigue cracking in existing bridges. 15 respondents indicated that they did have issues related to load-induced fatigue cracking, and 39 said they have not had any such issues.

The Maine DOT stated that they do not typically have the traffic volume to cause load-induced fatigue issues, but they do have heavy trucks. The heavy trucks were not described in detail, but it was stated that the design live load caused by the HL-93 design truck is increased by 25% (HS25) for design purposes. However, it was noted that load-induced fatigue often works in tandem with corrosion and distortion-induced fatigue. The New Mexico DOT stated that they have experienced load-induced fatigue issues in cross-frames primarily on widened bridges and bridges that were constructed in the 1960s and 1970s that have thinner decks.

The California DOT noted that the load-induced fatigue problems they have experienced do not appear to be due to in-plane stresses, but rather out-of- plane bending due to differential deflection. It was noted that this often occurs in the slow truck lane located on the outer edge of the bridge. An example was provided where two girders (girders 1 and 2) support the outer truck lane on a bridge. The cross-frames in between these girders as well as the cross-frame adjacent to these girders will experience load-induced fatigue issues. The fatigue cracks are often located at the diaphragm connection stiffeners, not the intermediate ones.

The Arkansas DOT said that their Heavy Bridge Maintenance crews have observed cracking near the top and bottom of the cross-frame to I-girder connection, but the cracks are primarily seen at the top connection. To repair this, a portion of the connection plate is removed and holes are drilled in the I-girder web. After these repairs, fatigue cracking is still sometimes experienced at the connection plates.

Multiple responses pertaining to distortion-induced fatigue were recorded. The typical solutions for this type of fatigue are crack arrest holes or proper detailing where the stiffener is welded to the top and bottom flange. These responses are not discussed further as they are not pertinent to the current study.

D3.2.3 Typical Cross-Frame Details

Typical intermediate cross-frame details that are used in steel I-girder bridges were requested in the survey. Of the 32 unique DOTs that responded, 20 provided typical cross-frame details. Since many states had multiple details, a total of 58 details were recorded and reviewed. The following items were of interest when reviewing typical cross-frame details:

- Cross-frame configuration/layout (X-frame, K-frame, or Z-frame),
- Member sections,
- Member end connections (configuration; bolted or welded), and
- Use of gusset plates and gusset plate thickness

Only the information provided to the RT through the survey responses is included in the summary of the typical cross-frame detail. A table summarizing the properties of the cross-frames can also be found in Chapter D7.

D3.2.3.1 Cross-Frame Geometry and Member Sections

In terms of configurations, X-type and K-type cross-frames (or simply X-frames and K-frames) are the most commonly used in practice. In total, the standard DOT details compiled include seven different configuration types. Three of those seven are related to X-type cross-frames, three to K-type, and one to Z-type. These are represented schematically in Figure D3-5.

X-Frame 1 (19 total details obtained) consists of both struts, X-Frame 2 (11 details) neglects the top strut, and X-Frame 3 (3 details) neglects both struts. K-Frame 1 (16 details) includes both struts, K-Frame 2 (16 details) neglects the top struts, and K-Frame 3 (16 details) inverts the orientation of the diagonals such that they meet along the mid-length of the top strut. Additionally, only one Z-type configuration was obtained from one state DOT. Consequently, it is labelled as Z-Frame 1. Table D3-9 summarizes specific information that was provided for these individual structural members from the different details provided. Both the configuration types and member sections commonly used are detailed.

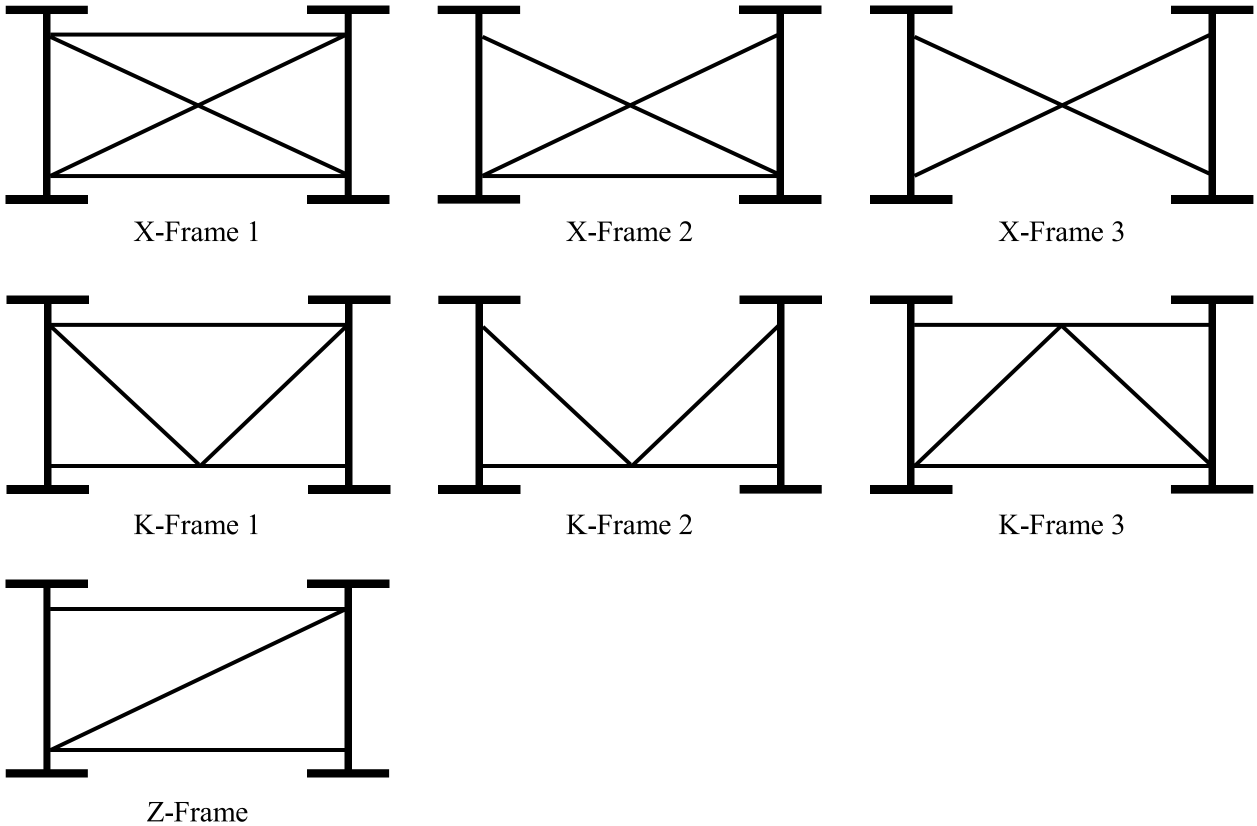


Figure D3-5: Various cross-frame configurations used across the US

Table D3-9: Summary of cross-frame geometry and member sizes by state

DOT	#	Frame Type	Diagonals	Bot Chord	Top Chord
Alaska	1	K-Frame 3	L_x_x_	L_x_x_	C_x_
Alaska	2	X-Frame 3	L_x_x_	None	None
Arkansas	1	X-Frame 1	L3-1/2x3-1/2x5/16	L6x6x5/16	L6x6x5/16
California	1	K-Frame 1	L152x152x19 (mm)	2L152x152x19 (mm)	L152x152x19 (mm)
California	2	K-Frame 1	2L152x152x19 (mm)	2L152x152x19 (mm)	L152x152x19 (mm)
Colorado	1	K-Frame 3	2L4x4x3/8	2L4x4x3/8	2L4x4x3/8
Florida	1	K-Frame 1	WT5x15	WT5x15	WT5x15
Illinois	1	X-Frame 2	L4x4x3/8	L4x4x3/8	None
Indiana	1	X-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None

Table D3-9: Summary of cross-frame geometry and member sizes by state

DOT	#	Frame Type	Diagonals	Bot Chord	Top Chord
Indiana	2	X-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None
Indiana	3	K-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None
Indiana	4	K-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None
Iowa	1	X-Frame 1	L4x3x5/16	WT4x10.5	WT4x10.5
Iowa	2	X-Frame 1	L4x3x5/16	WT4x10.5	WT4x10.5
Maine	1	X-Frame 3	L3x3x5/16	None	None
Maine	2	X-Frame 2	L3x3x5/16	WT4x9	None
Maine	3	X-Frame 3	WT4x9	None	None
Maine	4	X-Frame 2	WT4x9	WT5x11	None
Maine	5	X-Frame 2	WT4x9	WT5x11	None
Maine	6	X-Frame 1	WT5x11	WT5x11	WT5x11
Minnesota	1	X-Frame 2	L_x_x_	L_x_x_	None
Minnesota	2	X-Frame 1	L_x_x_	L_x_x_	L_x_x_
Missouri	1	X-Frame 1	L3x3x5/16	L3-1/2x3-1/2x5/16	L3-1/2x3-1/2x5/16
Missouri	2	X-Frame 1	L3x3x5/16	L4x4x5/16	L4x4x5/16
Montana	1	K-Frame 1	L6x6x5/8	L6x6x5/8	L6x6x5/8
Montana	2	K-Frame 1	L5x5x5/8	L5x5x5/8	L5x5x5/8
Montana	3	X-Frame 2	L4x4x3/8	L4x4x3/8	None
New Jersey	1	X-Frame 1	L6x6x3/8	L6x6x3/8	L6x6x3/8
New Jersey	2	K-Frame 2	L3.5x3.5x3/8	L3.5x3.5x3/8	None
New Jersey	3	K-Frame 1	L5x5x1/2	MC6x18	MC6x18
New Jersey	4	K-Frame 1	L3.5x3.5x3/8	L3.5x3.5x3/8	L3.5x3.5x3/8
New York	1	K-Frame 1	L_x_x_	L_x_x_	L_x_x_
New York	2	X-Frame 1	L_x_x_	L_x_x_	L_x_x_
New York	3	K-Frame 1	L_x_x_	L_x_x_	L_x_x_

Table D3-9: Summary of cross-frame geometry and member sizes by state

DOT	#	Frame Type	Diagonals	Bot Chord	Top Chord
New York	4	X-Frame 1	L_x_x_	L_x_x_	L_x_x_
North Carolina	1	X-Frame 2	L_x_x_	L_x_x_	None
North Carolina	2	X-Frame 1	L_x_x_	L_x_x_	L_x_x_
North Carolina	3	X-Frame 1	L_x_x_	L_x_x_	L_x_x_
North Carolina	4	K-Frame 1	L_x_x_	L_x_x_	L_x_x_
North Carolina	5	K-Frame 3	WT_x_	WT_x_	C_x_
North Carolina	6	K-Frame 3	WT_x_	WT_x_	C_x_
Tennessee	1	K-Frame 3	WT_x_ or 2WT_x_	WT_x_ or 2WT_x_	L_x_x_ or 2L_x_x_
Tennessee	2	Z-Frame	WT_x_	WT_x_	WT_x_
Texas	1	X-Frame 1	L4x4x3/8	L4x4x3/8	L4x4x3/8
Texas	2	X-Frame 1	L5x5x1/2	L5x5x1/2	L5x5x1/2
Texas	3	X-Frame 1	L6x6x9/16	L6x6x9/16	L6x6x9/16
Texas	4	K-Frame 1	L4x4x3/8	L4x4x3/8	L4x4x3/8
Texas	5	K-Frame 1	L5x5x1/2	L5x5x1/2	L5x5x1/2
Texas	6	K-Frame 1	L6x6x9/16	L6x6x9/16	L6x6x9/16
Virginia	1	K-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8
Virginia	2	K-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8
Virginia	3	X-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8
Virginia	4	X-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8
West Virginia	1	K-Frame 1	L5x5x1/2	L5x5x1/2	L5x5x1/2
Wisconsin	1	X-Frame 2	L3-1/2x3-1/2x5/16 to L5x5x5/16	L5x5x5/16 to L6x6x3/8	None
Wisconsin	2	X-Frame 2	L3-1/2x3-1/2x5/16 to L5x5x5/16	T Section 7x6.5x1/2 to 8.5x6.5x1/2	None
Wisconsin	3	X-Frame 1	L3-1/2x3-1/2x5/16 to L5x5x5/16	L5x5x5/16 to L6x6x3/8	L5x5x5/16 to L6x6x3/8
Wisconsin	4	X-Frame 1	L3-1/2x3-1/2x5/16 to L5x5x5/16	T Section 7x6.5x1/2 to 8.5x6.5x1/2	T Section 7x6.5x1/2 to 8.5x6.5x1/2

The data in the table shows that single angle sections are the most common structural shape used in the cross-frames. Not all typical details provide typical cross-frame cross-section sizes. In these instances, the cross-frame geometry is indicated and the size is omitted in the table (e.g. single angles without a given size are listed as “L_x_x_”). In these instances, it is the responsibility of the designer to conduct a rational analysis and determine the required size.

The sizes of single angles that are used ranges from L3x3x5/16 (Maine; Missouri) to L6x6x5/8 (Montana). Double angles are used as components in typical cross-frame details by two states – California and Colorado. Alaska, New Jersey, and North Carolina both make use of channel members for the top chord of some of their typical cross-frame details. In many of these cases, these channel sections are used for cross-frames at end supports, for which the top strut is composite with the deck to stiffen the unrestrained deck edge at joints. As noted previously, only intermediate cross-frames (i.e., cross-frames in between span supports) are studied in this project.

WT sections are also used for cross-frame members in some states including Florida, Maine, North Carolina, and Tennessee. WT section sizes range from WT4x9 to WT5x15. Built up tee sections are used for some cross-frame details in Wisconsin. Refer to Table D3-10 for a summary of cross-frame section sizes. As noted, some states do not have predetermined member sizes, which implies that the angle sizes need to be determined on a case-by-case basis.

Table D3-10: Summary of cross-frame sections utilized

Section Type	No. of Different Cross-Frames with this Section Type	Minimum Observed	Maximum Observed
Single Angle	49	L3x3x5/16	L6x6x5/8
Double Angle	4	2L4x4x3/8	2L152x152x19 (2L6x6x3/4)
Channel	4	Not Specified	Not Specified
WT	12	WT4x9	WT5x15
2WT	1	Not Specified	Not Specified
Built-up Tee	2	7x6.5x1/2	8.5x6.5x1/2

D3.2.3.2 Connection Details

The purpose of this section is to discuss the member end connections for various cross-frame types observed from the survey results. Of the 58 details reviewed by the RT, 37 make use of gusset plates (GP) to connect members to the connection plate (CP) which is welded to the I-girder. A total of 21 details do not use gusset plates. Figure D3-6, which is a Colorado DOT typical detail, illustrates a cross-frame with gusset plate connections. Figure D3-7, which is an Arkansas DOT typical detail, illustrates a cross-frame with no gusset plate connections.

In almost all instances where gusset plates are used to attach cross-frame members to the connection plates, the gusset plates are bolted to the connection plates. The only exception to this is the Texas DOT cross-frame details, where the gusset plates are welded to the connection plates with 5/16″ fillet welds. Gusset plate thickness values range from 3/8″ to 3/4″. 16 details show gusset plates with a thickness or minimum thickness of 3/8″, seven use a thickness of 1/2″, four show a thickness of 5/8″, and two show a thickness of

3/4″. Some states do not specify the minimum thickness of the gusset plates. Typically, when gusset plates are used, the top chord, bottom chord, and diagonals are fillet welded to the gusset plate. Of the respondents, only California and Indiana connect cross-frame members to gusset plates with bolts.

In all instances where gusset plates are not used to connect members to the connection plates, the top and bottom chords are directly bolted to the connection plates. The diagonals are connected in one of two ways. They are either bolted directly to the connection plate or they are fillet welded to the top and bottom chords (such as is done in Virginia and West Virginia typical details).

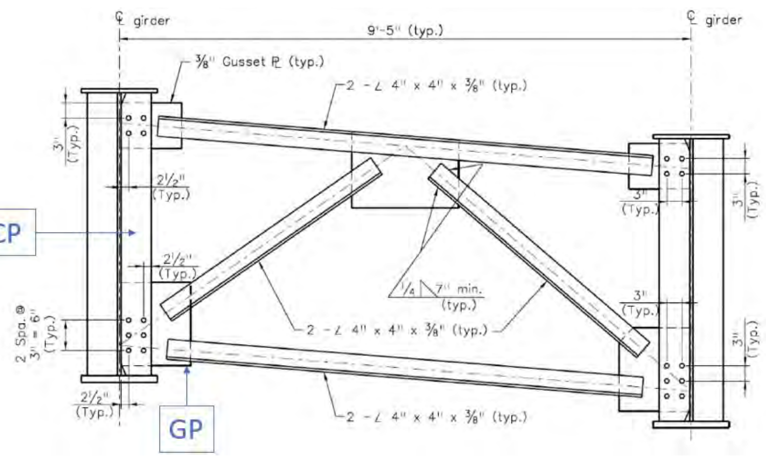


Figure D3-6: Typical cross-frame connection that includes gusset plates (detail from Colorado DOT)

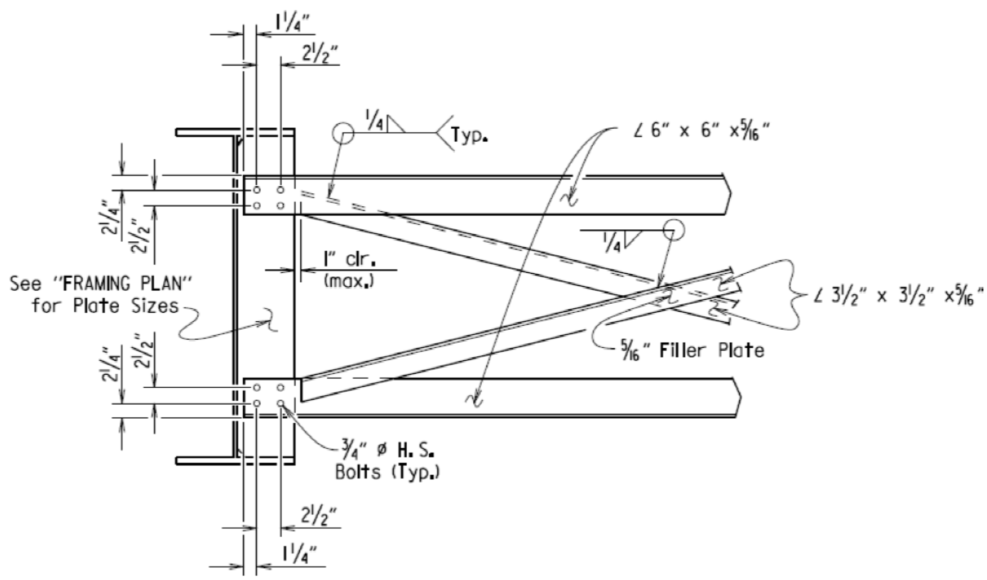


Figure D3-7: Typical cross-frame connection that does not include gusset plates (detail from Arkansas DOT)

CHAPTER D4

Commercial Software Review

Gaining an initial understanding of the methods employed by common commercial software packages in modeling cross-frames and how the software checks fatigue was an objective in Phase I of this project. Researchers were able to get feedback on this topic from five of the software packages that were listed by respondents to the survey discussed in Chapter D3. While some of the information was obtained directly from the software vendor, in other cases feedback was obtained from design engineers experienced with the specific software. The software in which feedback was obtained from a designer familiar with the modeling techniques are indicated in the subsequent sections dealing with the specific software. The software packages for which the RT were able to obtain specific modeling information are as follows:

- Software A
- Software B
- Software H
- Software K
- Software V

The RT gathered information on each software package from the literature available on the websites of each software company. In addition to reviewing the information available online, the RT contacted representatives from the above companies and inquired about how the software approaches the modeling of cross-frames and how the software handles fatigue design. Although five different software packages were evaluated at this stage of the project, only Software A and B are discussed herein. Note that these two programs were predominantly used throughout Phase III of NCHRP 12-113. Thus, the focus of the discussion herein is on those two particular programs. The following sections discuss the methods Software A and B use to model cross-frames and how fatigue design is handled by each software package.

D4.1 Software A

Software A is a general-use 3D analysis program that is specifically programmed for bridge analysis and design. It can analyze a variety of bridge systems including suspension bridges, cable-stay bridges, and steel I-girder bridges. The information on this software was provided by a designer familiar with the software. The software is flexible in that it allows designers to create a 3D bridge model through a series of user inputs, as well as develop simplified 2D models manually.

For 3D analysis, cross-frame elements are typically represented as pin-ended truss elements, connected to at a meshed node on the web-to-flange juncture. Software A does not automatically assign an axial stiffness modifier or spring per the recommendation in AASHTO LRFD Article C4.6.3.3.4. The user has the option to manually assign a stiffness modifier by artificially adjusting the cross-sectional area of the member or the elastic modulus of the steel material.

Software A uses an influence-surface approach to determine the stress range in the cross-frames. Fatigue design checks are possible with Software A; however, the program does have limitations. Based on the experience of the designer the program itself is not able to check fatigue directly. Instead, one must analyze the structure and check that the cross-frame forces are within code limits outside of the software.

D4.2 Software B

Software B is specifically marketed and programmed as a 2D analysis program. It can be used to design a wide variety of geometries including straight bridges with normal supports, curved bridges, and bridges with skewed supports. Three different types of 2D models can be used when designing bridges using Software B. The first type is a grillage model, where all elements are laid out in a two-dimensional plane and have three degrees of freedom at each node location. The second type is a plate and eccentric beam model. In this model type, the concrete slab is modeled as a plate element, and the girders are represented as beam elements offset from the slab. The offset is utilized to include the effects of composite action between the concrete slab and steel I-girders. The last type of model that can be used is a line-girder model, which cannot evaluate cross-frame forces.

In both the grillage and plate and eccentric beam finite element models, the cross-frames are converted to equivalent beam elements. As a first step in the process, the cross-frame is modeled as a truss. One end of the cross-frame is released, and a roller is placed under the released end, allowing it to rotate. A torsional moment is created by applying a horizontal force couple at the top and bottom of the truss. The torsional rotation of the released end is determined by finding horizontal displacements of the top and bottom nodes and then dividing by the depth of the truss. The moment of inertia of the equivalent beam is determined to provide the same torsional stiffness as seen in the truss model of the cross-frame. The torsional stiffness of the equivalent beam is the sum of the torsional stiffness of all the components in the cross-frame. After the analysis is complete, the equivalent beam representations of the cross-frames are converted back to the original truss geometry and the force in each cross-frame member is determined.

When evaluating fatigue in cross-frames, Software B creates an influence surface for a given cross-frame. For grillage models, for which the deck is not explcitrly modeled, load is distributed to nearby girder elements via the lever rule. The software then evaluates the maximum stress range created in the cross-frame by a single truck passing along a traffic lane per the 2015 interim revisions to AASHTO LRFD. It was also noted that when evaluating single angle cross-frames, Software B also treats welded connections as fatigue category E′ per the 2015 interim revisions to AASHTO LRFD.

D4.3 Conclusions and Software Recommendation

The following conclusions can be drawn from the above summary of two common software packages:

- The way cross-frames are modeled varies from equivalent beam modeling (e.g. Software B) to finite element modeling as line or shell elements (e.g. Software A).
- Both software packages described use an influence-surface approach to calculate the stress range in cross-frames.

To compare with a general-purpose program such as Abaqus, Software A (3D) and B (2D) were selected for further investigation in Phase II and III of the project. These two programs are among the most-used software packages by the individuals surveyed, which is a good indication of the software's widespread use throughout the industry.

CHAPTER D5

Evaluation of Potential Bridges

In accordance with the Task 4 requirements, the RT compiled a list of ten candidate bridges in Texas for potential instrumentation. The candidate bridges were selected based on a combination of reconnaissance efforts by the RT and assistance from TxDOT. This chapter is intended to provide the reader additional context on the field experimental studies outlined extensively in Appendix E. The pros and cons of each candidate bridge with respect to anticipated cross-frame behavior are examined. Ultimately, only three bridges were selected to perform field monitoring studies in Phase II of the project.

Table D5-1 summarizes the geographic location of each candidate bridge, as well as general geometric parameters (e.g. support skew) and access issues. Based on those tabulated parameters, the pros and cons of each bridge are assessed herein.

Table D5-1: Candidate bridges considered for experimental testing

Bridge Number	Bridge Description	Location	Description	Span Length (ft.)	Radius or Skew[1]	Lanes/ Width	Access Issues
1	IH 45 Frontage over FM 2854 & BNSF RR	Conroe	• Straight/normal supports • Prestressed concrete beam (Spans 1-5; 9-13) and continuous steel plate girder (Spans 6-8) • 5 steel girders/span	194-240-194 (plate girder only)	0°	3/ 41 ft.	• FM 2854 under Span 9 • Santa Fe St (2 lanes) and BNSF RR under Span 10 • Median under Span 11
2	St. Francis Ave over IH 30	Dallas	• Straight/normal supports • Rolled I-beams (W33x130s) • 4 steel girders/span	30-60-70-70-60-30	0°	2/ 36 ft.	• Embankments under Spans 1 and 6 • IH 30 Frontage Roads under Spans 2 and 5 • IH 30 under Spans 3/6
3	US 87 over UPRR & Beals Creek	Big Spring	• Straight/normal supports • Continuous steel plate girder (Spans 1-3) and rolled I-beams (Spans 4-6) • 10 steel girders/span	111-185-115 (plate girder only)	0°	4/ 68 ft.	• Accessible space under Span 1 • UPRR/Beals Creek under Spans 2 and 3
4	IH 45 over SH 105	Conroe	• Straight/skewed supports • Continuous steel plate girder • 12 steel girders/span	125-215-125	48°	5/ 96 ft.	• SH 105 (10 lanes) under Span 2 • U-turns under Spans 1 and 3 (with additional paved area)
5	Skillman over Loop 12	Dallas	• Straight/skewed supports • Rolled I-beams (W30x108s and W24x100s) • 6 steel girders/span	39-52-52-39	44°	6/ 41 ft.	• Embankments under Spans 1 and 4 • Loop 12 under Spans 2 and 3
6	Louisiana over IH 35E	Dallas	• Straight/skewed supports • Rolled I-beams (W30x135s and W30x150s) • 6 steel girders/span	37-75-87-36	13°	2/ 44 ft.	• Embankments under Spans 1 and 4, Loop 12 under Spans 2 and 3

[1]Skew is measured from the transverse direction of the bridge

Table D5-1 (con't): Candidate bridges considered for experimental testing

Bridge Number	Bridge Description	Location	Description	Span Length (ft.)	Radius or Skew[1]	Lanes/ Width	Access Issues
7	IH 45 NB Feeder to SH 242 WB	The Woodlands	• Horizontally curved • Prestressed concrete (Spans 1-13; 20-25) and 2 units of cont. steel girders (Spans 14-19) • 4 steel girders/span	252-188-157-255-311-235 (plate girders only).	960 ft.	1/28 ft.	• IH 45 NB under Span 14 • IH 45 SB under Span 15 • Service Rd under Span 16 • SH 242 WB under Span 17 • SH 242 EB under Span 18 • Median under Span 19 and portions of Spans 15-18
8	IH 635 & IH 35E Connection A	Dallas	• Horizontally curved • Prestressed concrete beams (Spans 1-6; 11-23) and continuous steel plate girder (Spans 7-10) • 4 steel girders/span	103-155-131-98 (plate girders only)	819 ft.	1/ 26 ft.	• IH 35E SB Frontage Rd under Span 7 • IH 20 EB and IH 35E to IH 20 WB under Span 8 • IH 20 to IH 35E NB and IH 20 WB under Span 9 • Median under Span 10
9	IH 635 & IH 35E Connection B	Dallas	• Horizontally curved • Prestressed concrete beams (Spans 1-9; 14-26) and continuous steel plate girder (Spans 10-13) • 4 steel girders/span	98-131-155-103 (plate girders only)	790 ft.	1/ 26 ft.	• Median under Span 10 • IH 20 EB and IH 35E to IH 20 WB under Span 11 • IH 20 to IH 35E NB and IH 20 WB under Span 12 • IH 35E NB Frontage Rd under Span 13
10	SH 146 over Port Drive & UPRR	Seabrook	• Horizontally curved • Prestressed concrete beams (Spans 1-3; 8-10) and continuous steel plate girder (Spans 4-7) • 4 steel girders/span	210-285-285-210 (plate girders only)	600 ft.	1/ 28 ft.	• SH 146 Feeder under Span 4 • UPRR under Span 5 • Port Rd under Span 6 • Median under Span 7, half of Span 6

[1]Skew is measured from the transverse direction of the bridge

Per the Task 4 requirements, the field experiments were to include a straight bridge with normal supports, a straight bridge with skewed supports, and a horizontally curved bridge. The candidate bridges that were under consideration were all located in Texas, which was ideal with regards to travel, cost efficiency, and proximity to the research lab. The pros and cons recognized by the RT are listed in Table D5-2 through Table D5-4. Table D5-2 examines straight bridges with normal supports, Table D5-3 examines straight bridges with skewed supports, and Table D5-4 examines horizontally curved bridges.

The three bridges that were ultimately selected for field instrumentation and testing (Bridge 1, 4, and 10) are identified with an asterisk in each of the corresponding tables. Note that the discussion on these three structures is expanded in Appendix E, the Phase II summary.

Table D5-2: Pros and cons for straight and normal candidate bridges

Bridge Number	Bridge	Pros	Cons
1*	IH 45 Frontage over FM 2854 & BNSF RR	• Deep plate girders • Long spans (span of choice for instrumentation is 194 ft.) • Few girders/span (5) • Lanes for load placement options (3) • Little to no traffic control needed during instrumentation (1 span) • Interstate bridge with likely high ADT/ADTT for rain-flow data • Relatively close proximity to research team (167 miles from Austin, 39 miles from Houston)	• Traffic control difficult on IH 45 during controlled live loading • BNSF RR eliminates one span for possible instrumentation
2	St. Francis Ave over IH 30	• Few girders/span (4) • Lanes for load placement options (2) • Little to no traffic control needed for instrumentation at embankments (2 spans) • Likely possible to close bridge during controlled live loading	• Rolled I-beams • Short spans (30-70 ft.) • Traffic control difficult on IH 30 during instrumentation • City street with likely low ADT/ADTT for rain-flow data • Not in close proximity to research team (195 miles from Austin, 244 miles from Houston)
3	US 87 over UPRR & Beals Creek	• Deep plate girders • Medium spans (span of choice for instrumentation is 111 ft.) • Lanes for load placement options (2 NB and 2 SB) • Little to no traffic control needed during instrumentation (1 span) • Lane closures likely possible during controlled live loading • Highway bridge with moderate ADT/ADTT for rain-flow data	• Numerous girders/span (10) • UPRR switching yard eliminates all but one span for possible instrumentation • Not in close proximity to research team (294 miles from Austin, 452 miles from Houston)

Table D5-3: Pros and cons for straight and skewed candidate bridges

Bridge Number	Bridge	Pros	Cons
4*	IH 45 over SH 105	• Deep plate girders • Moderate to long spans (span of choice for instrumentation is 125 ft.) • Lanes for load placement options (5) • Little to no traffic control needed during instrumentation (2 non-consecutive spans) • Interstate bridge with likely high ADT/ADTT for rain-flow data • Relatively close proximity to research team (167 miles from Austin, 40 miles from Houston)	• Numerous girders/span (12) • Wide bridge (96 ft.) • Traffic control difficult on IH 45 during controlled live loading
5	Skillman over Loop 12	• Few girders/span (6) • Lanes for load placement options (3) • Little to no traffic control needed during instrumentation (2 non-consecutive spans)	• Rolled I-beams • Short spans (40-50 ft.) • City street with likely low ADT/ADTT for rain-flow data • Not in close proximity to research team (197 miles from Austin, 247 miles from Houston)
6	Louisiana over IH 35E	• Few girders/span (6) • Lanes for load placement options (2) • Little to no traffic control needed during instrumentation (2 non-consecutive spans)	• Rolled I-beams • Short spans (40-90 ft.) • City street with likely low ADT/ADTT for rain-flow data • Not in close proximity to research team (183 miles from Austin, 241 miles from Houston)

Table D5-4: Pros and cons for horizontally curved candidate bridges

Bridge Number	Bridge	Pros	Cons
7	IH 45 NB Feeder to SH 242 WB	• Deep plate girders • Long spans (span of choice for instrumentation is 235 ft.) • Few girders/span (4) • Little to no traffic control needed during instrumentation (2 non-consecutive spans) • State highway bridge with likely high ADT/ADTT for rain-flow data • Relatively close proximity to research team (162 miles from Austin, 34 miles from Houston)	• Only one lane (plus shoulders) for live load placement • Lane closure may be difficult on IH 45 during controlled live loading
8	IH 635 & IH 35E Connection A	• Deep plate girders • Moderate spans (span of choice for instrumentation is 98 ft.) • Few girders/span (4) • Interstate highway bridge with likely high ADT/ADTT for rain-flow data	• Potentially only one span with cross-frames • Only one lane (plus shoulders) for live load placement • Lane closure may be difficult on interstate highways during controlled live loading and instrumentation • Not in close proximity to research team (179 miles from Austin, 235 miles from Houston)
9	IH 635 & IH 35E Connection B	• Deep plate girders • Moderate spans (span of choice for instrumentation is 98 ft.) • Few girders/span (4) • Interstate highway bridge with likely high ADT/ADTT for rain-flow data	• Potentially only one span with cross-frames • Only one lane (plus shoulders) for live load placement • Lane closure may be difficult on interstate highways during controlled live loading and instrumentation • Not in close proximity to research team (179 miles from Austin, 235 miles from Houston)
10*	SH 146 over Port Drive & UPRR	• Deep plate girders • Long spans (span of choice for instrumentation is 210 ft.) • Few girders/span (4) • Little to no traffic control needed during instrumentation (1.5 consecutive spans) • State highway bridge with likely high ADT/ADTT for rain-flow data • Relatively close proximity to research team (191 miles from Austin, 30 miles from Houston)	• Only one lane (plus shoulders) for live load placement • Lane closure may be difficult on SH 146 during controlled live loading

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CHAPTER D6

Industry Survey Outline

The industry survey distributed to state DOTs, bridge owners, and consultants in Phase I of the project is provided in the following pages for reference. Recall that the responses to this survey are summarized in Chapter D3.

National Cooperative Highway Research Program (NCHRP) Project 12-113 - Proposed Modifications to AASHTO Cross-Frame Analysis and Design recently began at Ferguson Structural Engineering Laboratory at the University of Texas at Austin. Key elements of this project are to evaluate the load-induced fatigue performance of cross-frames in straight, curved, and skewed steel I-girder bridges, and to provide quantitatively based guidance on the calculation of the fatigue design forces (i.e. the force/stress ranges) in the cross-frame members of curved and/or severely skewed I-girder bridges analyzed using refined analysis methods. Another key element of the study is to investigate the influence of connection end details and connection plate stiffness on the cross-frame member stiffness assumed in the analysis. As part of this investigation, the research team is conducting a survey related to steel I-girder bridge design software frequently used by bridge owners and consulting engineers, as well as commonly used details for cross-frames. Your participation in this survey can provide valuable direction to the research team.

This survey will likely take 10-15 minutes to complete. While we value as much information that you can provide, feel free to complete only those sections that you wish to participate in.

1. Organization Information:

    Organization: ____________________

    City: ____________________

    State: ____________________

2. Personal Information (OPTIONAL):

    Name: ____________________

    Position / Title: ____________________

    Phone Number: ____________________

    Email Address: ____________________

3. Select the steel bridge design software you / your organization most frequently use for steel I-girder bridges from the list below. Also, indicate which software is used for each steel I-girder bridge condition listed below. (Choose all that apply)

	Software	Normal Supports	Skewed Supports	Curved Bridges
☐	LEAP Bridge Steel (Bentley)	☐	☐	☐
☐	RM Bridge (Bentley)	☐	☐	☐
☐	LARS Bridge (Bentley)	☐	☐	☐
☐	CSiBridge (Computers & Structures, Inc.)	☐	☐	☐
☐	LARSA 4D (LARSA)	☐	☐	☐
☐	LUSAS Bridge (LUSAS)	☐	☐	☐
☐	MDX (MDX Software)	☐	☐	☐
☐	midas Civil (MIDAS Information Technology Co.)	☐	☐	☐
☐	midas FEA (MIDAS Information Technology Co.)	☐	☐	☐
☐	LRFD Simon Steel Bridge Design Suite (NSBA)	☐	☐	☐
☐	DESCUS I (University of Maryland/OPTI-MATE)	☐	☐	☐
☐	Merlin-DASH (University of Maryland/OPTI-MATE)	☐	☐	☐
☐	Other: ____________________	☐	☐	☐
☐	Other: ____________________	☐	☐	☐
☐	Other: ____________________	☐	☐	☐

4. To your knowledge, can the analysis software your organization has used perform fatigue design checks of cross-frames in steel I-girder bridges?
    - ☐ Yes
    - ☐ No
    - ☐ Not sure

    a. *If yes to question 4*; Do you use the software to check fatigue of cross-frames in steel I-girder bridges?
    - ☐ Yes
    - ☐ No

    b. *If yes to question 4a*; Regarding the steel bridge analysis software your organization has used, have you had any difficulties or concerns related to cross-frame fatigue checks made by the software?
    - ☐ Yes
    - ☐ No

    c. *If yes to question 4b*; Elaborate on the issue(s) with cross-frame fatigue checks, if possible. If issues have been encountered for multiple software packages, specify which software package each comment applies to. (please select and comment on all that apply)
    - ☐ The way the software checks cross-frame fatigue is unclear: [comment box]
    - ☐ Analysis software reported a problem with cross-frame fatigue that required design modification: [comment box]
    - ☐ The way the software obtains force/stress ranges in the cross-frame members is unclear: [comment box]
    - ☐ Software results appear to be overly conservative for cross-frame fatigue: [comment box]
    - ☐ Software results appear to be unconservative for cross-frame fatigue: [comment box]
    - ☐ Other: [comment box]

    d. *If no/not sure to question 4 or 4a*; how do you check fatigue of cross-frames in steel I-girder bridges? (please select all that apply)
    - ☐ Hand calculations
    - ☐ In-house software
    - ☐ Spreadsheets
    - ☐ Other: [comment box]

    e. If "In-house-software" or "Spreadsheets" is selected for 4d; How does the in-house software or spreadsheet compute force/stress ranges in cross-frame members? For example, is an influence surface used or is some other method used?
    - ☐ [comment box]

5. When checking fatigue of cross-frames in steel I-girder bridges – do you / your organization consider only infinite fatigue life, or is finite fatigue life considered when applicable?
   - ☐ Infinite fatigue life
   - ☐ Infinite and finite fatigue life when applicable
   - ☐ Other: [comment box]
   - ☐ N/A

6. Regarding actual behavior of steel I-girder bridges in service, have you / your organization encountered **load-induced** fatigue cracking at cross-frames or cross-frame to I-girder connections? If yes, please use the space provided to comment on the location of cracking, repairs made, and the effectiveness of the repairs.

   To clarify, only check "yes" if fatigue cracking due to in-plane, repetitive stresses has been observed at cross-frame connection locations. Do not check "yes" if observed cracking resulted from distortion-induced fatigue issues related to improper detailing. (see image below)

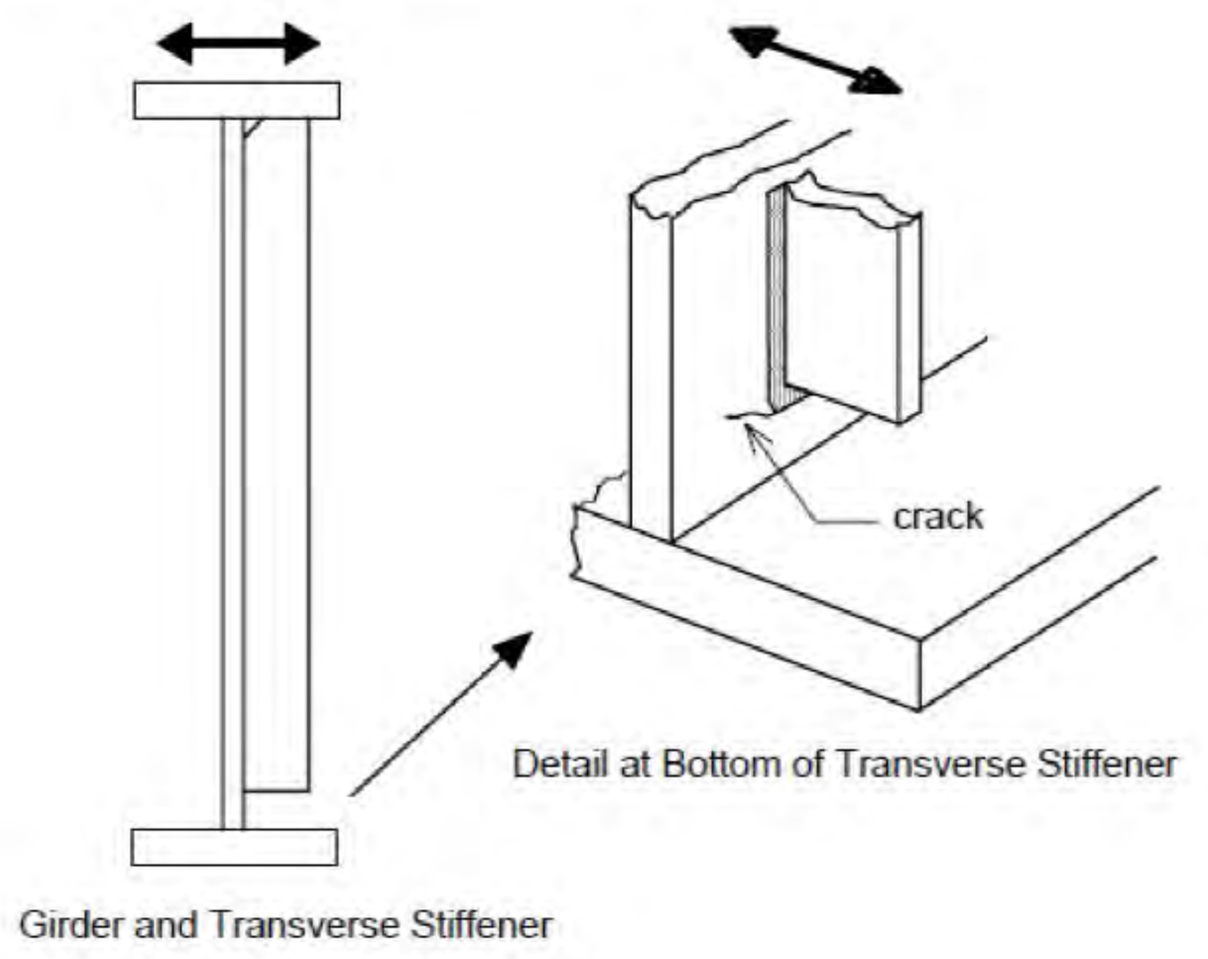

Image from: Mertz, Dennis, Ph.D., P.E. Steel Bridge Design Handbook Design for Fatigue. Publication no. FHWA-HIF-16-002 - Vol. 12. Vol. 12. N.p.: U.S. Department of Transportation / Federal Highway Administration, 2015. Print.

- ☐ Yes: [comment box]
- ☐ No

7. To assist in defining the range of parameters that will be considered in this study, the research team is seeking commonly-used cross-frame details from around the United States or any other pertinent information on cross-frames. If you are able, please provide a link or file upload containing typical cross-frame details you / your organization frequently use with steel I-girder bridges. If different details are used for straight bridges than for curved or severely skewed bridges please provide examples of each: [File upload] or [url link]

   Alternatively, feel free to forward (by mail or email) any information on the details to the following contact:

   Professor Todd Helwig
   Ferguson Structural Engineering Laboratory
   10100 Burnet Road, Bldg. #177
   The University of Texas at Austin
   Austin, Texas 78759
   Email: thelwig@mail.utexas.edu
   Phone: 512 924-5903

8. Please provide any additional comments you believe will be useful. For example, you could expand on the fatigue loading used for cross-frames: [comment box]

CHAPTER D7

Typical Cross-Frame Details

As part of the industry survey, the RT compiled standard cross-frame details from various state DOTs across the country. In this chapter, specific characteristics and parameters of those details are summarized in tabulated form. Recall that Section D3.2.3 also provides an overview of these details.

Table D7-1: List of pertinent cross-frame details used by state DOTs

DOT	#	Frame Type	Diagonals	Bot. Chord	Top Chord	Top GP?[1]	Bot. GP?[2]	Center PL?[3]	PL Thk.	GP Conn.[4]	Top Chord End Conn.[5]	Bot Chord End Conn.	Diagonal End Conn.[6]
Alaska	1	K-Frame 3	L_x_x_	L_x_x_	C_x_	No	Yes	No	N.S.	Bolted to CP	Bolted to CP	Welded to GP	Welded to GP
Alaska	2	X-Frame 3	L_x_x_	None	None	Yes	Yes	No	N.S.	Bolted to CP	N/A	N/A	Welded to GP
Arkansas	1	X-Frame 1	L3-1/2x3-1/2x5/16	L6x6x5/16	L6x6x5/16	No	No	Yes	5/16"	N/A	Bolted to CP (3/4" Bolts)	Bolted to CP (3/4" Bolts)	Welded to T/B (1/4" Fillet)
California	1	K-Frame 1	L152x152x19	2L152x152x19	L152x152x19	Yes	Yes	Yes	19mm	Bolted to CP (22mm Bolts)	Bolted to GP (22mm Bolts)	Bolted to GP (22mm Bolts)	Bolted to GP (22mm Bolts)
California	2	K-Frame 1	2L152x152x19	2L152x152x19	L152x152x19	Yes	Yes	Yes	19mm	Bolted to CP (22mm Bolts)	Bolted to GP (22mm Bolts)	Bolted to GP (22mm Bolts)	Bolted to GP (22mm Bolts)
Colorado	1	K-Frame 3	2L4x4x3/8	2L4x4x3/8	2L4x4x3/8	Yes	Yes	Yes	3/8"	Bolted to CP	Welded to GP (1/4" Fillet min.)	Welded to GP (1/4" Fillet min.)	Welded to GP (1/4" Fillet min.)
Florida	1	K-Frame 1	WT5x15	WT5x15	WT5x15	Yes	Yes	Yes	5/8"	Bolted to CP	Welded to GP (5/16" Fillet min.)	Welded to GP (3/8" Fillet min.)	Welded to GP (5/16" Fillet min.)
Illinois	1	X-Frame 2	L4x4x3/8	L4x4x3/8	None	Yes	Yes	Yes	3/8"	Bolted to CP	N/A	Welded to GP (1/4" Fillet)	Welded to GP (1/4" Fillet)
Indiana	1	X-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None	Yes	Yes	Yes	3/8"	Bolted to CP	N/A	Welded to GP (Fillet)	Welded to GP (Fillet)
Indiana	2	X-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None	No	No	Yes	N/A	N/A	N/A	Bolted to CP	Bolted to CP
Indiana	3	K-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None	Yes	Yes	Yes	3/8"	Bolted to CP	N/A	Welded to GP	Bolted to GP
Indiana	4	K-Frame 2	L3-1/2x3-1/2x3/8	L3-1/2x3-1/2x3/8	None	No	No	Yes	3/8"	Bolted to CP	N/A	Bolted to CP	Bolted to CP
Iowa	1	X-Frame 1	L4x3x5/16	WT4x10.5	WT4x10.5	No	No	Yes	N/A	N/A	Bolted to CP	Bolted to CP	Bolted to CP
Iowa	2	X-Frame 1	L4x3x5/16	WT4x10.5	WT4x10.5	Yes	Yes	Yes	3/8"	Bolted to CP	Welded to GP (1/4" Fillet)	Welded to GP (1/4" Fillet)	Welded to GP (1/4" Fillet)
Maine	1	X-Frame 3	L3x3x5/16	None	None	No	No	Yes	N/A	N/A	N/A	N/A	Bolted to CP
Maine	2	X-Frame 2	L3x3x5/16	WT4x9	None	No	No	Yes	N/A	N/A	N/A	Bolted to CP	Bolted to CP
Maine	3	X-Frame 3	WT4x9	None	None	No	No	Yes	N/A	N/A	N/A	N/A	Bolted to CP
Maine	4	X-Frame 2	WT4x9	WT5x11	None	No	Yes	Yes	N.S.	Bolted to CP	N/A	N.S.	Bolted to CP
Maine	5	X-Frame 2	WT4x9	WT5x11	None	No	No	Yes	N/A	N/A	N/A	Bolted to CP	Bolted to CP
Maine	6	X-Frame 1	WT5x11	WT5x11	WT5x11	No	No	Yes	N/A	N/A	Bolted to CP	Bolted to CP	Bolted to CP
Minnesota	1	X-Frame 2	L_x_x_	L_x_x_	None	Yes	Yes	Yes	N.S.	Bolted to CP	N/A	Welded to GP (Fillet)	Welded to GP (Fillet)
Minnesota	2	X-Frame 1	L_x_x_	L_x_x_	L_x_x_	Yes	Yes	Yes	N.S.	Bolted to CP	Welded to GP (Fillet)	Welded to GP (Fillet)	Welded to GP (Fillet)
Missouri	1	X-Frame 1	L3x3x5/16	L3-1/2x3-1/2x5/16	L3-1/2x3-1/2x5/16	Yes	Yes	Yes	N.S.	Bolted to CP	Welded to GP (Fillet)	Welded to GP (Fillet)	Welded to GP (Fillet)
Missouri	2	X-Frame 1	L3x3x5/16	L4x4x5/16	L4x4x5/16	Yes	Yes	Yes	N.S.	Bolted to CP	Welded to GP (Fillet)	Welded to GP (Fillet)	Welded to GP (Fillet)
Montana	1	K-Frame 1	L6x6x5/8	L6x6x5/8	L6x6x5/8	Yes	Yes	Yes	5/8"	Bolted to CP (7/8" Bolts)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
Montana	2	K-Frame 1	L5x5x5/8	L5x5x5/8	L5x5x5/8	Yes	Yes	Yes	5/8"	Bolted to CP (7/8" Bolts)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)

Table D7-1 (con't): List of pertinent cross-frame details used by state DOTs

DOT	#	Frame Type	Diagonals	Bot. Chord	Top Chord	Top GP?[1]	Bot. GP?[2]	Center PL?[3]	PL Thk.	GP Conn.[4]	Top Chord End Conn.[5]	Bot Chord End Conn.	Diagonal End Conn.[6]
Montana	3	X-Frame 2	L4x4x3/8	L4x4x3/8	None	Yes	Yes	Yes	1/2"	Bolted to CP (7/8" Bolts)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
New Jersey	1	X-Frame 1	L6x6x3/8	L6x6x3/8	L6x6x3/8	Yes	Yes	Yes	5/8"	Bolted to CP	Welded to GP (3/8" Fillet)	Welded to GP (3/8" Fillet)	Welded to GP (3/8" Fillet)
New Jersey	2	K-Frame 2	L3.5x3.5x3/8	L3.5x3.5x3/8	None	Yes	Yes	Yes	3/8"	Bolted to CP	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
New Jersey	3	K-Frame 1	L5x5x1/2	MC6x18	MC6x18	Yes	Yes	Yes	N.S.	Bolted to CP	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
New Jersey	4	K-Frame 1	L3.5x3.5x3/8	L3.5x3.5x3/8	L3.5x3.5x3/8	Yes	Yes	Yes	3/8"	Bolted to CP	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
New York	1	K-Frame 1	L_x_x_	L_x_x_	L_x_x_	Yes	Yes	Yes	3/8" min.	Bolted to CP	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
New York	2	X-Frame 1	L_x_x_	L_x_x_	L_x_x_	Yes	Yes	Yes	3/8" min.	Bolted to CP	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
New York	3	K-Frame 1	L_x_x_	L_x_x_	L_x_x_	Yes	Yes	Yes	3/8" min.	Bolted to CP	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
New York	4	X-Frame 1	L_x_x_	L_x_x_	L_x_x_	Yes	Yes	Yes	3/8" min.	Bolted to CP	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
North Carolina	1	X-Frame 2	L_x_x_	L_x_x_	None	No	No	No	N/A	N/A	N/A	Bolted to CP	Bolted to CP
North Carolina	2	X-Frame 1	L_x_x_	L_x_x_	L_x_x_	No	No	No	N/A	N/A	Bolted to CP	Bolted to CP	Bolted to CP
North Carolina	3	X-Frame 1	L_x_x_	L_x_x_	L_x_x_	Yes	Yes	No	N.S.	Bolted to CP	Welded to GP (Fillet)	Welded to GP (Fillet)	Welded to GP (Fillet)
North Carolina	4	K-Frame 1	L_x_x_	L_x_x_	L_x_x_	No	No	Yes	N.S.	N/A	Bolted to CP (7/8" Bolts)	Bolted to CP (7/8" Bolts)	Bolted to CP (7/8" Bolts)
North Carolina	5	K-Frame 3	WT_x_	WT_x_	C_x_	No	No	Yes	N.S.	N/A	Bolted to CP	Bolted to CP	Bolted to CP
North Carolina	6	K-Frame 3	WT_x_	WT_x_	C_x_	No	No	Yes	N.S.	N/A	Bolted to CP	Bolted to CP	Bolted to CP
Tennessee	1	K-Frame 3	WT_x_ or 2WT_x_	WT_x_ or 2WT_x_	L_x_x_ or 2L_x_x_	No	No	Yes	N.S.	N/A	Bolted to CP	Bolted to CP	Bolted to CP
Tennessee	2	Z-Frame	WT_x_	WT_x_	WT_x_	No	No	No	N/A	N/A	Bolted to CP	Bolted to CP	Bolted to CP
Texas	1	X-Frame 1	L4x4x3/8	L4x4x3/8	L4x4x3/8	Yes	Yes	Yes	1/2"	Welded to CP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
Texas	2	X-Frame 1	L5x5x1/2	L5x5x1/2	L5x5x1/2	Yes	Yes	Yes	1/2"	Welded to CP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
Texas	3	X-Frame 1	L6x6x9/16	L6x6x9/16	L6x6x9/16	Yes	Yes	Yes	1/2"	Welded to CP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
Texas	4	K-Frame 1	L4x4x3/8	L4x4x3/8	L4x4x3/8	Yes	Yes	Yes	1/2"	Welded to CP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
Texas	5	K-Frame 1	L5x5x1/2	L5x5x1/2	L5x5x1/2	Yes	Yes	Yes	1/2"	Welded to CP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)
Texas	6	K-Frame 1	L6x6x9/16	L6x6x9/16	L6x6x9/16	Yes	Yes	Yes	1/2"	Welded to CP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)	Welded to GP (5/16" Fillet)

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DOT	#	Frame Type	Diagonals	Bot. Chord	Top Chord	Top GP?[1]	Bot. GP?[2]	Center PL?[3]	PL Thk.	GP Conn.[4]	Top Chord End Conn.[5]	Bot Chord End Conn.	Diagonal End Conn.[6]
Virginia	1	K-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8	No	No	No	N/A	N/A	Bolted to CP	Bolted to CP	Welded to T/B (5/16" Fillet)
Virginia	2	K-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8	No	No	No	N/A	N/A	Bolted to CP	Bolted to CP	Welded to T/B (5/16" Fillet)
Virginia	3	X-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8	No	No	Yes	3/8"	N/A	Bolted to CP	Bolted to CP	Welded to T/B (5/16" Fillet)
Virginia	4	X-Frame 1	L5x5x3/8	L6x6x3/8	L6x6x3/8	No	No	Yes	3/8"	N/A	Bolted to CP	Bolted to CP	Welded to T/B (5/16" Fillet)
West Virginia	1	K-Frame 1	L5x5x1/2	L5x5x1/2	L5x5x1/2	No	No	No	N/A	N/A	Bolted to CP (7/8" Bolts)	Bolted to CP (7/8" Bolts)	Welded to T/B (1/4" Fillet min.)
Wisconsin	1	X-Frame 2	L3-1/2x3-1/2x5/16 to L5x5x5/16	L5x5x5/16 to L6x6x3/8	None	Yes	Yes	No	3/8"	Bolted to CP	N/A	Welded to GP (Fillet)	Welded to GP (3/16" Fillet)
Wisconsin	2	X-Frame 2	L3-1/2x3-1/2x5/16 to L5x5x5/16	T Section 7x6.5x1/2 to 8.5x6.5x1/2	None	Yes	Yes	No	3/8"	Bolted to CP	N/A	Welded to GP (Fillet)	Welded to GP (3/16" Fillet)
Wisconsin	3	X-Frame 1	L3-1/2x3-1/2x5/16 to L5x5x5/16	L5x5x5/16 to L6x6x3/8	L5x5x5/16 to L6x6x3/8	Yes	Yes	No	3/8"	Bolted to CP	N/A	Welded to GP (Fillet)	Welded to GP (3/16" Fillet)
Wisconsin	4	X-Frame 1	L3-1/2x3-1/2x5/16 to L5x5x5/16	T Section 7x6.5x1/2 to 8.5x6.5x1/2	T Section 7x6.5x1/2 to 8.5x6.5x1/2	Yes	Yes	No	3/8"	Bolted to CP	N/A	Welded to GP (Fillet)	Welded to GP (3/16" Fillet)

Notes:

[1] This column states whether gusset plates (GP) are used to connect cross-frame members to connection plates (CP) at the top of a cross-frame for a given detail.

[2] This column states whether gusset plates (GP) are used to connect cross-frame members to connection plates (CP) at the bottom of a cross-frame for a given detail.

[3] This column states whether a center plate is used to connect cross-frame members. For X-frame configurations, this will likely be a filler plate used to connect diagonals in their centers. For K-frame configurations, this means a gusset plate used to connect diagonal ends to either the top or bottom chord.

[4] The gusset connection column describes how the gusset plates (GP) are attached to the connection plate (CP), if applicable.

[5] The end connection columns describe how the top and bottom chords are connected. They may be bolted or welded and they may be connection to either a gusset plate (GP) or directly to the connection plate (CP).

[6] The diagonal end connection column describes how the diagonals are connected to the girders. They may be bolted or welded and they may be connection to either a gusset plate (GP) or directly to the connection plate (CP). In some instances, they are welded to the top and bottom chords (T/B).