APPENDIX J. PROPOSED CHANGES TO AASHTO SPECIFICATIONS

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM:

SUBJECT: The Manual for Bridge Evaluation: Section 1, Article 1.6; Section 6, Articles C6A.4 and C6A.6; Part B, Articles C6B.5.2.1 and C6B.5.3.1; and Appendix L6B

TECHNICAL COMMITTEE: T-18 Bridge Management, Evaluation, and Rehabilitation/T-14 Steel

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☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☐ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☒ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/11/2013
DATE REVISED:

AGENDA ITEM:

<u>Item #1</u>

Make the revisions to the indicated articles of the Manual for Bridge Evaluation shown in Attachment A.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Following the I-35W Bridge collapse investigation, the National Transportation Safety Board (NTSB) made five recommendations to the Federal Highway Administration (FHWA) and AASHTO. One of these recommendations was to require bridge owners to include main truss member gusset plates as part of the load rating process for these bridges.

To assist the states with this process, FHWA issued a Guidance document in February 2009. This document required, at a minimum, for main truss member gusset plates to be evaluated for five limit states using either the Load Factor Rating (LFR) or Load and Resistance Factor Rating (LRFR) philosophies.

The Guidance document was based on existing provisions in the AASHTO LRFD Bridge Design Specifications and the older AASHTO Standard Specifications for Highway Bridges along with engineering judgment. The FHWA Guidance document was thought to yield conservative gusset plate ratings. As States began to evaluate their inventory with the Guidance document, a need for more direction on some checks was identified, while some facets of other checks were thought to be too conservative. This was the case particularly for the shear reduction factor (Ω) associated with the shear yielding check, and the K-factor selection for use in the column analogy compressive buckling resistance check.

To address these concerns, FHWA initiated a research project collaboratively with the AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) to evaluate the shear, tensile and compressive resistance of gusset plates at the strength limit state (NCHRP Project 12-84). The project tested 12 full-scale experimental gusset plate connections, and used finite element analysis to explore a variety of geometric parameters that could not be experimentally investigated. The outcome of this project resulted in these proposed revisions to the AASHTO Manual of Bridge Evaluation. A companion item proposes similar revisions to the AASHTO LRFD Bridge Design Specifications based on the results of this research. It should be noted that a decision was made to ensure that the LRFR and LFR gusset plate rating specifications in the MBE are reasonably self-sufficient and do not refer back to the LRFD Bridge Design Specifications and the Standard Specifications to a significant extent for determining the factored resistance of the gusset plate and its connections.

A second companion item proposes an example for inclusion in the MBE Appendix A: Illustrative Examples that will illustrate LRFR and LFR of main truss member gusset plates according to these proposed revisions. It is envisioned that the 2009 FHWA Guidance document for the load rating of these gusset plates will not be maintained in the future, and that the proposed provisions contained herein would supersede the Guidance document.

ANTICIPATED EFFECT ON BRIDGES:

Assuming that most bridge owners have rated their gusset plate inventory using the existing FHWA Guidance document, the following summarizes some of the more important differences between the proposed MBE LRFR provisions and the LRFR provisions provided in the FHWA Guidance document:

1. In rating for shear yielding, the Ω factor is 0.88 and ϕ_{vy}=1.00 in the proposed provisions for a total reduction factor of 0.88 applied to the average shear stress. In the FHWA Guidance document, the Engineer had the ability to choose Ω=0.74 or Ω=1.00 in conjunction with a ϕ_{vy}=0.95 for a total reduction factor of 0.70 or 0.95 applied to the average shear stress. Therefore, if Ω=0.74 was assumed originally, the proposed specifications will result in an ~25% increase in the rating for shear yielding. If Ω=1.00 was assumed originally, the proposed specifications will result in an ~8% decrease in the rating for shear yielding. No changes are made to the rating procedures for shear rupture in the proposed provisions.
2. In the calculation of the rating for compression resistance, higher or lower ratings will be obtained using the proposed provisions over those obtained using the FHWA Guidance document depending on the assumptions that were made when rating with the FHWA Guidance document. The FHWA Guidance document recommended using an equivalent column length, which was the average of three different lengths along the Whitmore plane, referred to as L_{avg}. This length and an assumed column length factor were used to calculate a column slenderness parameter, λ_{avg}, which in turn was used to calculate the critical buckling stress of the idealized column. The new rating provisions will certainly produce less favorable ratings over the FHWA Guidance document when $\lambda_{avg} < \sim 1.0$. This is because the partial plane shear yield criterion that is instituted in the proposed provisions will control for these very compact gusset plates, and this criterion was not checked in the FHWA Guidance document. On the contrary, if $\lambda_{avg} > \sim 1.5$ the new provisions will produce more favorable ratings over the FHWA Guidance document because of the new effective column length factor of 0.5, which is much lower than the K-factor that was likely employed when using the FHWA Guidance document. The ratings with the new provisions and the FHWA Guidance document are expected to be similar when $1.0 < \lambda_{avg} < 1.5$. No changes are made to the rating procedures for tension yielding and net section fracture on the Whitmore plane.
3. The new proposed rating specifications use a resistance factor of 1.00 for block shear rupture, whereas in the FHWA Guidance document (and AASHTO LRFD Bridge Design Specifications), this factor is 0.80. The factor of 0.80 was found to provide a reliability index of 4.5 whereas the decision was made to use a reliability index of 3.5 in the MBE for Inventory level assessments; therefore, the resistance factor was increased accordingly. This should result in a 25% increase in block shear rupture ratings over those obtained using the FHWA Guidance document.
4. The FHWA Guidance document recommended using a Whitmore section analysis in the rating of tension and compression chord splices. The real stress patterns in the analysis models did not correlate well with this assumption and this method is no longer recommended. Therefore, if this particular check controlled using the FHWA Guidance document, it will no longer apply under the proposed provisions. A new chord splice check is introduced within the proposed specifications that better accounts for the variability of gusset plate geometries versus the Whitmore section approach. The effect of this new approach on the load rating for these splices is difficult to ascertain as its effect will be specific to each joint, which typically has a unique geometry.
5. Overall, the proposed MBE rating specifications reflect a better understanding of gusset plate behavior than the provisions provided in the 2009 FHWA Guidance document and should result in a more uniform reliability of gusset plate ratings.

NCHRP Project 12-84 primarily focused on the development of an LRFR approach to gusset plate rating. This required the derivation of resistance factors to provide a target reliability index of 3.5 for Inventory level assessments. The translation of these resistance factors to an LFR philosophy is difficult because the live-load models are different (HS20 versus HL93), and the project did not perform a comprehensive live-load study for both short and long span trusses. As a result, the LRFR resistance factors cannot merely be carried over to LFR. If a nominal resistance equation utilizes a reduction factor specific to that resistance behavior (for instance, the reduction factor Ω for shear yielding), that factor was carried over, but most of the resistance factors were made unity for LFR. If no better information could be derived from the research, the same resistance factor published in the FHWA Guidance document was repeated in the proposed specifications. As a result, some resistance factors will be different between LRFR and LFR in the proposed provisions. The following summarizes some of the more important differences between the proposed MBE LFR provisions and the LFR provisions provided in the FHWA Guidance document:

6. In the rating for shear yielding, the Ω factor is 0.88. In the existing FHWA Guidance document, the Engineer had the ability to choose Ω=0.74 or Ω=1.00. Therefore if Ω=0.74 was assumed originally, the proposed specifications will result in an ~19% increase in the rating for shear yielding. If Ω=1.00 was assumed originally, the proposed specifications will result in an ~12% decrease in the rating for shear yielding. No changes are made to the rating procedures for shear rupture in the proposed provisions.
7. In the calculation of the rating for compression resistance, higher or lower ratings will be obtained using the proposed provisions over those obtained using the FHWA Guidance document depending on the assumptions that were made when rating with the FHWA Guidance document. The FHWA Guidance document recommended using an equivalent column length, which was the average of three different lengths along the Whitmore plane, referred to as L_{avg}. This length and an assumed column length factor were used to calculate a column slenderness parameter, λ_{avg}, which in turn was used to calculate the critical buckling stress of the idealized column. The new rating provisions will certainly produce less favorable ratings over the FHWA Guidance document when $\lambda_{avg} < \sim1.0$. This is because the partial plane shear yield criterion that is instituted in the proposed provisions will control for these very compact gusset plates, and this criterion was not checked in the FHWA Guidance document. On the contrary, if $\lambda_{avg} > \sim1.5$ the new provisions will produce more favorable ratings over the FHWA Guidance document because of the new effective column length factor of 0.5, which is much lower than the K-factor that was likely employed when using the FHWA Guidance document. The ratings with the new provisions and the FHWA Guidance document are expected to be similar when $1.0 < \lambda_{avg} < 1.5$. No changes are made to the rating procedures for tension yielding and net section fracture on the Whitmore plane.
8. For block shear rupture, the resistance equation was updated to reflect the revision made to this equation in the Fifth Edition AASHTO LRFD Bridge Design Specification. A resistance factor of 0.85 is specified for the block shear rupture check, which is the same as the factor specified in the FHWA Guidance document. Thus, it is unlikely there will be a significant difference between the block shear rupture rating determined using the proposed LFR specification and the FHWA Guidance document.
9. The FHWA Guidance document recommended using a Whitmore section analysis in the rating of tension and compression chord splices. The real stress patterns in the analysis models did not correlate well with this assumption and this method is no longer recommended. Therefore, if this particular check controlled using the FHWA Guidance document, it will no longer apply under the proposed provisions. A new chord splice check is introduced within the proposed specifications that better accounts for the variability of gusset plate geometries versus the Whitmore section approach. The effect of this new approach on the load rating for these splices is difficult to ascertain as its effect will be specific to each joint, which typically has a unique geometry.

REFERENCES:

FHWA. 2009. Load Rating Guidance and Examples For Bolted and Riveted Gusset Plates In Truss Bridges, FHWA-IF-09-014, U.S. Department of Transportation, Washington, DC.

See also the revised MBE Article 1.6 in Attachment A.

OTHER:

None

ATTACHMENT A – 2013 AGENDA ITEM -- T-18/T-14

Make the following revisions to Articles 1.6, 6A.4.2.4, 6A.6, C6B.5.2.1, C6B.5.3.1 & Appendix L6B of the Manual for Bridge Evaluation:

1.6—REFERENCES

Add the following references:

Brown, J. D., D. J. Lubitz, Y. C. Cekov, and K. H. Frank. 2007. *Evaluation of Influence of Hole Making Upon the Performance of Structural Steel Plates and Connections*, Report No. FHWA/TX-07/0-4624-1. University of Texas at Austin, Austin, TX.

Kulak, G. L., J. W. Fisher, and J. H. A. Struik. 1987. *Guide to Design Criteria for Bolted and Riveted Joints*, Second Edition. John Wiley and Sons, Inc. New York, NY.

NCHRP. 2013. *Guidelines for the Load and Resistance Factor Design and Rating of Welded, Riveted and Bolted Gusset-Plate Connections for Steel Bridges*, NCHRP Report 7XX, Transportation Research Board, National Research Council, Washington D.C (to be published).

Sheikh-Ibrahim, F. I. 2002. "Design Method for the Bolts in Bearing-Type Connections with Fillers," *AISC Engineering Journal*, American Institute of Steel Construction, Chicago, IL, Vol. 39, No. 4, pp. 189-195.

Yura, J. A., K. H. Frank, and D. Polyzois. 1987. *High-Strength Bolts for Bridges*, PMFSEL Report No. 87-3. University of Texas, Austin, TX, May 1987.

Yura, J. A., M. A. Hansen, and K.H. Frank. 1982. "Bolted Splice Connections with Undeveloped Fillers," *Journal of the Structural Division*. American Society of Civil Engineers, New York, NY, Vol. 108, No. ST12, December, pp. 2837-2849.

6A.4—LOAD-RATING PROCEDURES

6A.4.2.4—System Factor: φ_s

System factors are multipliers applied to the nominal resistance to reflect the level of redundancy of the complete superstructure system. Bridges that are less redundant will have their factored member capacities reduced, and, accordingly, will have lower ratings.

System factors that correspond to the load factor modifiers in the *AASHTO LRFD Bridge Design Specifications* should be used. The system factors in Table 6A.4.2.4-1 are more conservative than the LRFD design values and may be used at the discretion of the evaluator until they are modified in the *AASHTO LRFD Bridge Design Specifications*.

The system factor for riveted and bolted gusset plates for all force effects shall be taken as 0.90.

C6A.4.2.4

Structural members of a bridge do not behave independently, but interact with other members to form one structural system. Bridge redundancy is the capability of a bridge structural system to carry loads after damage to or the failure of one or more of its members. Internal redundancy and structural redundancy that exists as a result of continuity are neglected when classifying a member as nonredundant.

6A.6—STEEL STRUCTURES

6A.6.3—Resistance Factors

Except as specified herein, resistance factors, φ, for steel members, for the strength limit state, shall be taken as specified in LRFD Design Article 6.5.4.2.

For load rating of main truss member gusset plates, the resistance factors shall be taken as follows:

- For gusset plate compression $\varphi_{cg} = 0.95$
- For gusset plate chord splices $\varphi_{cs} = 0.85$
- For gusset plate shear yielding $\varphi_{vy} = 1.00$
- For gusset plate block shear rupture $\varphi_{bs} = 1.00$

C6A.6.3

For service limit states, φ = 1.0.

6A.6.5—Effects of Deterioration on Load Rating

A deteriorated structure may behave differently than the structure as originally designed and different failure modes may govern its load capacity. Corrosion is the major cause of deterioration in steel bridges. Effects of corrosion include section loss, unintended fixities, movements and pressures, and reduced fatigue resistance.

C6A.6.5

Tension Members with Section Losses Due to Corrosion

Corrosion loss of metals can be uniform and evenly distributed or it can be localized. Uniform reduction in the cross-sectional area of a tension member causes a proportional reduction in the capacity of the member. Since localized corrosion results in irregular localized reductions in area, a simplified approach to evaluating the effects of localized corrosion is to consider the yielding of the reduced net area as the governing limit state. Due to their self-stabilizing nature, stress concentrations and eccentricities induced by asymmetrical deterioration may be neglected when estimating the tension strength of members with moderate deterioration.

For eyebars and pin plates, the critical section is located at the pin hole normal to the applied stress. In evaluating eyebars with significant section loss in the head, the yielding of the reduced net section in the head should be checked as it may be a governing limit state.

Deterioration of lacing bars and batten plates in built-up tension members may affect the load sharing among the main tension elements at service loads. At ultimate load, yielding will result in load redistribution among the tension elements and the effect on capacity is less significant.

Compression Members with Section Losses Due to Corrosion

Uniform Corrosion

Local Effects—The susceptibility of members with reduced plate thickness to local buckling should be evaluated with respect to the limiting width/thickness ratios specified in LRFD Design Article 6.9.4.2. If these values are exceeded, AISC *LRFD Manual of Steel*

Construction may be used to evaluate the local residual compressive capacity.

Overall Effects—Most compression members encountered in bridges are in the intermediate length range and have a box-shape or H-shape cross section. Moderate uniform corrosion of these sections has very little effect on the radius of gyration. The reduction of compressive resistance for short and intermediate length members, for moderate deterioration, is proportional to the reduction in cross-sectional area.

Localized Corrosion

Deterioration at the ends of fixed-end compression members may result in a change in the end restraint conditions and reduce its buckling strength. Localized corrosion along the member can cause changes in the moment of inertia. Asymmetric deterioration can induce load eccentricities. The effects of eccentricities can be estimated using the eccentricity ratio ec/r^2, where e is the load eccentricity in the member caused by localized section loss, c is the distance from the neutral axis to the extreme fiber in compression of the original section, and r is the radius of gyration of the original section. Effects of eccentricity may be neglected for eccentricity ratios under 0.25.

Built-Up Members with Deteriorated Lacing Bars/Batten Plates

The main function of lacing bars and batten plates is to resist the shear forces that result from buckling of the member about an axis perpendicular to the open web. They also provide lateral bracing for the main components of the built-up member. Localized buckling of a main component can result because of loss of lateral bracing from the deterioration of the lacing bars. The slenderness ratio of each component shape between connectors and the nominal ~~nomina~~ compressive resistance of built-up members should be evaluated as specified in LRFD Design Article 6.9.4.3.

Corrosion of lacing bars and batten plates reduces the shear resistance of the built-up member and, therefore, a reduction in its overall buckling strength may result. Approximate analytical solutions for the buckling resistance of built-up members with deteriorated lacing and batten plates can be formulated using a reduced effective modulus of elasticity of the member, given in NCHRP Report 333. It has been determined that moderate deterioration of up to about 25 percent loss of the original cross-section of lacing bars and batten plates has very little effect on the overall member capacity, as long as the resistance to local failure is satisfactory.

Flexural Members with Section Losses Due to Corrosion

Uniform Corrosion

The reduction in bending resistance of laterally supported beams with stiff webs will be proportional to the reduction in section modulus of the corroded cross-section compared to the original cross-section. Either the elastic or plastic section modulus shall be used, as appropriate. Local and overall beam stability may be affected by corrosion losses in the compression flange.

The reduction in web thickness will reduce shear resistance and bearing capacity due to both section loss and web buckling. When evaluating the effects of web losses, failure modes due to buckling and out-of-plane movement that did not control their original design may govern. The loss in shear resistance and bearing capacity is linear up to the point where ~~there~~ buckling occurs.

Localized Corrosion

Small web holes due to localized losses not near a bearing or concentrated load may be neglected. All other web holes should be analytically investigated to assess their effect.

A conservative approach to the evaluation of tension and compression flanges with highly localized losses is to assume the flange is an independent member loaded in tension or compression. When the beam is evaluated with respect to its plastic moment capacity, the plastic section modulus for the deteriorated beam may be used for both localized and uniform losses.

Main Truss Member Gusset Plates

The resistance of gusset plates may be reduced if section loss due to corrosion is present at certain locations coinciding with the failure planes assumed in applying the resistance equations specified in Article 6A.6.12.6.

For evaluating the tension resistance, only the section loss that intersects the Whitmore plane must be accounted for when calculating the resistance. The section loss may be smeared uniformly over the entire Whitmore plane.

For evaluating the shear resistance, the use of the remaining area across a failure plane is sufficient for determining the resistance regardless of whether or not multi-layered gusset plates are present or the corrosion is localized, is asymmetric about the connection work point, or affects only one gusset plate.

For evaluating the compressive resistance, the actual area remaining in the partial shear plane defined in Article 6A.6.12.6.6 is to be considered. When evaluating the compressive resistance according to Article 6A.6.12.6.7, an equivalent plate thickness should be defined for the Whitmore width based on a projection upon the Whitmore width of all section loss occurring between the Whitmore width and the adjoining members in the direction of the member, as shown in Figure C6A.6.5-1. In this case, a smeared uniform plate thickness must be derived for L_{total} considering the isolated section loss occurring over $L_{corrosion}$. These methods were found to be conservative as reported in NCHRP (2013).

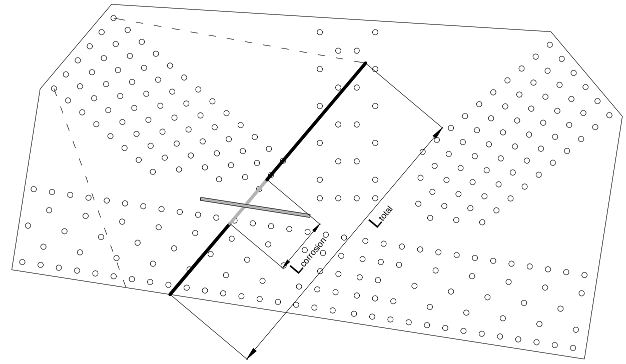


Figure C6A.6.5-1---Section-loss band projected upon the Whitmore section to determine an equivalent average plate thickness for the compressive resistance evaluation

6A.6.12.1—General

External connections of nonredundant members shall be evaluated during a load rating analysis in situations where the evaluator has reason to believe that their capacity may govern the load rating of the entire bridge. Evaluation of critical connections shall be performed in accordance with the provisions of these articles.

C6A.6.12.1

External connections are connections that transfer calculated load effects at support points of a member. Nonredundant members are members without alternate load paths whose failure is expected to cause the collapse of the bridge.

It is common practice to assume that connections and splices are of equal or greater capacity than the members they adjoin. With the introduction of more accurate evaluation procedures to identify and use increased member load capacities, it becomes increasingly important to also closely scrutinize the capacity of connections and splices to ensure that they do not govern the load rating.

~~Specifically, truss gusset plate connection analysis has been summarized in *FHWA Gusset Guidance- Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges*, FHWA-IF-09-014, February 2009. A good deal of engineering judgment is required to apply this guidance as connection geometry is variable and to account for effects of measurable corrosion if present. Other references as follows may also be helpful in order to use the guidance:~~

~~Cheng, J. J.R. and G. Y. Grondin. 2001. *Design and Behavior of Gusset Plate Connections.*~~

~~Galambos, T. V. 1998. *Guide to Stability Design Criteria for Metal Structures*, Fifth Edition. John Wiley and Sons, New York, NY.~~

~~Yamamoto, et al. 1998. “Buckling Strengths of Gusseted Truss Joints,” *ASCE Journal of Structural Engineering*, Vol. 114.~~

~~Analysis of gusset connections of truss bridges should be preceded by a field investigation of gusset plates at all truss joints. Field inspections of gusset plates~~

~~need to focus on corrosion, distortion, and connections. Section losses can occur along gusset plate areas that trap debris or hold water, usually along the top of the bottom chord. Distortion in the gusset plate can be from original construction, or can be caused by overstressing of the plate due to overloads, inadequate thickness/bracing, forces associated with pack rust between plates, or traffic impact. Gusset plate member connections should be inspected closely according to the provisions of Article 4.8.3.10.~~

6A.6.12.6—Gusset Plates

Main truss member gusset plates shall be load rated for shear, compression, and/or tension, as applicable, occurring in the vicinity of each connected member. Except as specified herein, a load rating analysis of main truss member gusset plates shall be conducted according to the provisions of Articles 6A.6.12.6.1 through 6A.6.12.6.9. Alternatively, a load rating analysis may be performed according to the provisions of Article 6A.6.12.6.11. The system factor, φ_s, for riveted and bolted gusset plates specified in Article 6A.4.2.4 shall be applied to the load rating of the gusset plates. The load rating provisions specified herein may be used for the evaluation of gusset plates for design loads, legal loads or permit loads, and shall utilize the appropriate live load factors provided in these Specifications for the load rating of primary members.

C6A.6.12.6

A load rating analysis of the main truss gusset connections of truss bridges should be preceded by a field investigation of the gusset plates at all truss joints. Field inspections of the gusset plates need to focus on corrosion, distortion, and the connections. Section losses can occur along gusset plate areas that trap debris or hold water, usually along the top of the bottom chord. Distortion in the gusset plate can occur during the original construction, or can be caused by overstressing of the plate due to overloads, inadequate thickness/bracing, forces associated with the development of pack rust between the plates, or traffic impact. Gusset plate member connections should be inspected closely according to the provisions of Article 4.8.3.10. Effects of deterioration on the resistance of the gusset plate should be accounted for as discussed in Article C6A.6.5.

The resistance equations provided herein were developed and calibrated to a target reliability index of 3.5 at the Strength I Inventory level at a dead-to-live load ratio, DL/LL, of 1.0. For larger values of DL/LL, calculated resistances are to be reduced as specified in Article 6A.6.12.6.1. In situations where DL/LL is less than 1.0, an increase in the calculated resistances could be justified by backward interpolation according to the provisions of Article 6A.6.12.6.1, although the gains would be anticipated to be marginal.

The provisions provided in this article are intended for the load rating of double gusset-plate connections used in trusses that may each be made from multiple layers of plates. The validity of the requirements for application to single gusset-plate connections has not been verified.

These provisions are based on the findings from NCHRP Project 12-84 (NCHRP, 2013), and supersede the 2009 FHWA Guidelines for gusset-plate load ratings. Example calculations illustrating the application of the resistance equations for gusset-plate connections contained herein are provided in NCHRP (2013) and in Appendix A.

6A.6.12.6.1 – Resistance Reduction for DL/LL Ratio

If the dead-to-live load ratio, DL/LL, as determined by the member forces on the gusset plate connection is greater than 1.0, the resistances determined in Articles 6A.6.12.6.2 through 6A.6.12.6.11 shall be reduced as specified herein. The resistance reduction shall decrease linearly from 1.00 to 0.90 as DL/LL increases from 1.0 to 6.0. The resistance reduction shall not be taken as less than 0.90.

C6A.6.12.6.1

To maintain a constant reliability index, the required resistance factor decreases as the dead-to-live ratio, DL/LL, increases. Since resistance factors were developed and calibrated for a DL/LL of 1.0, the reduction factor on the resistances specified herein accounts for the necessary decrease in the resistance factor for DL/LL greater than 1.0.

6A.6.12.6.2 – Fastener Shear Resistance

C6A.6.12.6.2

The factored shear resistance of rivets, $\varphi_s F_{uv}$, at the strength limit state shall be determined as specified in Article 6A.6.12.5.1.

The factored shear resistance, φR_n, of a high-strength bolt (ASTM A325 or ASTM A490) or an ASTM A307 bolt (Grade A or B) at the strength limit state in joints whose length between extreme bolts measured parallel to the line of action of the force is less than 50.0 in. shall be taken as:

- Where threads are excluded from the shear plane:

$$\varphi R_n = \varphi_s 0.48 A_b F_{ub} N_s \qquad (6A.6.12.6.2\text{-}1)$$

- Where threads are included in the shear plane:

$$\varphi R_n = \varphi_s 0.38 A_b F_{ub} N_s \qquad (6A.6.12.6.2\text{-}2)$$

where:

φ_s = resistance factor bolts in shear specified in LRFD Design Article 6.5.4.2

A_b = area of the bolt corresponding to the nominal diameter (in.^{2})

F_{ub} = specified minimum tensile strength of the bolt specified in Table 6A.6.12.6.2-1

N_s = number of shear planes per bolt

The factored shear resistance of a bolt in connections greater than 50.0 in. in length shall be taken as 0.80 times the value given by Eq. 6A.6.12.6.2-1 or 6A.6.12.6.2-2.

For ASTM A307 bolts, shear design shall be based on Eq. 6A.6.12.6.2-2. When the grip length of an ASTM A307 bolt exceeds 5.0 diameters, the factored resistance shall be lowered one percent for each 1/16 in. of grip in excess of 5.0 diameters.

Table 6A.6.12.6.2-1 – Specified Minimum Tensile Strength of Bolts

	F_u (ksi)
A307 Grade A or B	*60*
A325 for diameters 0.5 through 1.0 in.	*120*
A325 for diameters greater than 1.0	*105*
A490	*150*

When bolts carrying loads pass through undeveloped fillers 0.25 in. or more in thickness in axially loaded connections, the factored shear resistance of the bolt shall be reduced by the following factor:

The nominal resistance of a high-strength bolt in shear, R_n, is based upon the observation that the shear strength of a single high-strength bolt is about 0.60 times the tensile strength of that bolt (*Kulak et al., 1987*). However, in shear connections with more than two bolts in the line of force, deformation of the connected material causes a nonuniform bolt shear force distribution so that the resistance of the connection in terms of the average bolt resistance decreases as the joint length increases. Rather than provide a function that reflects this decrease in average bolt resistance with joint length, a single reduction factor of 0.80 was applied to the 0.60 multiplier. This accommodates bolts in joints up to 50.0 in. in length without seriously affecting the economy of very short joints. The nominal shear resistance of bolts in joints longer than 50.0 in. must be further reduced by an additional 20 percent. Studies have shown that the allowable stress factor of safety against shear failure ranges from 3.3 for compact, i.e., short, joints to approximately 2.0 for joints with an overall length in excess of 50.0 in. It is of interest to note that the longest and often the most important joints had the lowest factor, indicating that a factor of safety of 2.0 has proven satisfactory in service (*Kulak et al., 1987*).

The average value of the nominal resistance for bolts with threads in the shear plane has been determined by a series of tests to be 0.833 ($0.6F_{ub}$), with a standard deviation of 0.03 (*Yura et al., 1987*). A value of about 0.80 was selected for the formula based upon the area corresponding to the nominal body area of the bolt.

The shear resistance of bolts is not affected by pretension in the bolts, provided that the connected material is in contact at the faying surfaces.

The threaded length of an ASTM A307 bolt is not as predictable as that of a high-strength bolt. The requirement to use Eq. 6A.6.12.6.2-2 reflects that uncertainty.

ASTM A307 bolts with a long grip tend to bend, thus reducing their resistance.

Fillers must be secured by means of additional bolts so that the fillers are, in effect, an integral part of a shear-connected component at the strength limit state. The integral connection results in well-defined shear planes and no reduction in the factored shear resistance of the bolts. For undeveloped fillers 0.25 in. or more in thickness, the reduction factor given by Eq. 6A.6.12.6.2-3 is to be applied to the factored resistance of the bolts in shear. This factor compensates for the reduction in the nominal shear resistance of a bolt caused by bending in

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)} \right] \quad (6A.6.12.6.2\text{-}3)$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on the top and bottom of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in.^{2})

6A.6.12.6.3 – Bolt Slip Resistance

The nominal slip resistance of a high-strength bolt in a slip-critical connection at the service limit state shall be taken as:

$$R_n = K_h K_s N_s P_t \quad (6A.6.12.6.3\text{-}1)$$

where:

N_s = number of slip planes per bolt

P_t = minimum required bolt tension specified in Table 6A.6.12.6.3-1 (kip)

K_h = hole size factor taken as 1.0 for standard holes, or as specified in LRFD Design Table 6.13.2.8-2 for oversize or slotted holes

K_s = surface condition factor specified in Table 6A.6.12.6.3-2

Table 6A.6.12.6.3-1 – Minimum Required Bolt Tension

Bolt Diameter, in.	*Required Tension – P_t (kip)*	
	A325	*A490*
5/8	*19*	*24*
¾	*28*	*35*
7/8	*39*	*49*
1	*51*	*64*
1-1/8	*56*	*80*
1-1/4	*71*	*102*
1-3/8	*85*	*121*
1-1/2	*103*	*148*

Table 6A.6.12.6.3-2 – Values of K_s

For Class A surface conditions	0.33

the bolt. The reduction factor is only to be applied on the side of the connection with the fillers. The factor was developed mathematically (Sheikh-Ibrahim, 2002), and verified by comparison to the results from an experimental program on axially loaded bolted splice connections with undeveloped fillers (Yura et al., 1982). Alternatively, if fillers are extended beyond the connected parts and connected with enough bolts to develop the force in the fillers, the fillers may be considered developed.

For slip-critical high-strength bolted connections, the factored slip resistance of a bolt need not be adjusted for the effect of the fillers. The resistance to slip between the fillers and either connected part is comparable to that which would exist between the connected parts if the fillers were not present

C6A.6.12.6.3

Extensive data developed through research has been statistically analyzed to provide improved information on slip probability of high-strength bolted connections in which the bolts have been preloaded to the requirements of Table 6A.6.12.6.3-1. Two principal variables, bolt pretension and coefficient of friction, i.e., the surface condition factor of the faying surfaces, were found to have the greatest effect on the slip resistance of connections.

Hole size factors less than 1.0 are provided in LRFD Design Table 6.13.2.8-2 for bolts in oversize and slotted holes because of their effects on the induced tension in bolts using any of the specified installation methods. In the case of bolts in long-slotted holes, even though the slip load is the same for bolts loaded transverse or parallel to the axis of the slot, the values for bolts loaded parallel to the axis have been further reduced, based upon judgment, because of the greater consequences of slip.

The minimum bolt tension values given in Table 6A.6.12.6.3-1 are equal to 70 percent of the minimum tensile strength of the bolts. The same percentage of the tensile strength has been traditionally used for the required tension of the bolts.

Further information on the surface condition factors provided in Table 6A.6.12.6.3-2 may be found in LRFD Design Article C6.13.2.8.

For Class B surface conditions	0.50
For Class C surface conditions	0.33

The following descriptions of surface condition shall apply to Table 6A.6.12.6.3-2:

- Class A Surface: unpainted clean mill scale, and blast-cleaned surfaces with Class A coatings,
- Class B Surface: unpainted blast-cleaned surfaces and blast-cleaned surfaces with Class B coatings, and
- Class C Surface: hot-dip galvanized surfaces roughened by wire brushing after galvanizing.

6A.6.12.6.4 –Bearing Resistance at Fastener Holes

The effective bearing area of a fastener shall be taken as its diameter multiplied by the thickness of the gusset plate on which it bears.

For standard holes, oversize holes, short-slotted holes, and long-slotted holes parallel to the applied bearing force, the factored resistance of interior and end fastener holes at the strength limit state, φR_n, shall be taken as:

- With fasteners spaced at a clear distance between holes not less than 2.0*d* and with a clear end distance not less than 2.0*d*:

$$\varphi R_n = \varphi_{bb} 2.4 dtF_u \qquad (6A.6.12.6.4\text{-}1)$$

- If either the clear distance between holes is less than 2.0*d*, or the clear end distance is less than 2.0*d*:

$$\varphi R_n = \varphi_{bb} 1.2 L_c tF_u \qquad (6A.6.12.6.4\text{-}2)$$

where:

ϕ_{bb} = resistance factor for fasteners bearing on material specified in LRFD Design Article 6.5.4.2

d = nominal diameter of the fastener (in.)

t = thickness of the connected material (in.)

F_u = tensile strength of the connected material (ksi)

C6A.6.12.6.4

The term fastener in this article is meant to encompass both rivets and high-strength bolts.

Bearing stress produced by a fastener pressing against the side of the hole in a connected part is important only as an index to behavior of the connected part. Thus, the same bearing resistance applies regardless of fastener shear strength or the presence or absence of threads in the bearing area. The critical value can be derived from the case of a single fastener at the end of a tension member.

It has been shown that a connected plate will not fail by tearing through the free edge of the material if the distance L, measured parallel to the line of applied force from a single fastener to the free edge of the member toward which the force is directed, is not less than the diameter of the fastener multiplied by the ratio of the bearing stress to the tensile strength of the connected part (*Kulak et al., 1987*).

The criterion for nominal bearing strength is:

$$\frac{L}{d} \geq \frac{r_n}{F_u} \qquad (C6A.6.12.6.4\text{-}1)$$

where:

r_n = nominal bearing pressure (ksi)

F_u = specified minimum tensile strength of the connected part (ksi)

In these Specifications, the nominal bearing resistance

L_c = clear distance between holes or between the hole and the end of the member in the direction of the applied bearing force (in.)

of an interior hole is based on the clear distance between the hole and the adjacent hole in the direction of the bearing force. The nominal bearing resistance of an end hole is based on the clear distance between the hole and the end of the member. The nominal bearing resistance of the connected member may be taken as the sum of the resistances of the individual holes.

Holes may be spaced at clear distances less than the specified values, as long as the lower value specified by Eq. *6A.6.12.6.4*-2 is used for the nominal bearing resistance.

For determining the factored bearing resistance of long-slotted holes loaded perpendicular to the applied bearing force, refer to LRFD Design Article 6.13.2.9.

6A.6.12.6.5 – Multi-Layered Gusset and Splice Plates

Where multi-layered gusset and splice plates are used, the resistances of the individual plates may be added together when determining the factored resistances specified in Articles 6A.6.12.6.6 through 6A.6.12.6.9 *provided that enough fasteners are present to develop the force in the layered gusset and splice plates.*

C6A.6.12.6.5

Kulak et al.(1987) contains additional guidance on determining the number of fasteners required to develop the force in layered gusset and splice plates.

6A.6.12.6.6 Gusset Plate Shear Resistance

Gusset plates shall be load rated for shear yielding and shear rupture at the strength limit state.

For shear yielding, the factored shear resistance shall be taken as:

$$V_r = \varphi_{vy} 0.58 F_y A_{vg} \Omega \qquad (6A.6.12.6.6\text{-}1)$$

where:

Ω = shear reduction factor for gusset plates taken as 0.88

φ_{vy} = resistance factor for gusset plate shear yielding specified in Article 6A.6.3

A_{vg} = gross area of the shear plane (in.^{2})

F_y = specified minimum yield strength of the gusset plate (ksi)

For shear rupture, the factored shear resistance shall be taken as:

$$V_r = \varphi_{vu} 0.58 F_u A_{vn} \qquad (6A.6.12.6.6\text{-}2)$$

where:

φ_{vu} = resistance factor for gusset plate shear rupture specified in LRFD Design Article 6.5.4.2

C6A.6.12.6.6

The Ω shear reduction factor is used only in the evaluation of truss gusset plates for shear yielding. This factor accounts for the nonlinear distribution of shear stresses that form along a failure plane as compared to an idealized plastic shear stress distribution. The nonlinearity primarily develops due to shear loads not being uniformly distributed on the plane and also due to strain hardening and stability effects. The Ω factor was developed using shear yield data generated in NCHRP Project 12-84 (NCHRP, 2013). On average, Ω was 1.02 for a variety of gusset-plate geometries; however, the data were scattered due to proportioning of load between members, and variations in plate thickness and joint configuration. The specified Ω and resistance factors have been calibrated to account for shear plane length-to-thickness ratios varying from 85 to 325.

Failure of a full width shear plane requires relative mobilization between two zones of the plate, typically along chords. Mobilization cannot occur when a shear plane passes through a continuous member; for instance, a plane passing through a continuous chord member that would require shearing of the member itself.

Research has shown that the buckling of connections with tightly spaced members is correlated with shear yielding around the compression members. This is important because the buckling criteria used in Article 6A.6.12.6.7 would overestimate the compressive buckling resistance of these types of connections. Once a plane yields in shear, the reduction in the plate modulus

$\underline{A_{vn}}$ = net area of the shear plane (in.^{2})

$\underline{F_u}$ = specified minimum tensile strength of the gusset plate (ksi)

Shear shall be checked on relevant partial and full failure plane widths. Partial shear planes shall only be checked around compression members and only Eq. 6A.6.12.6.6-1 shall apply to partial shear planes. The partial shear plane length shall be taken along adjoining member fastener lines between plate edges and other fastener lines. The following partial shear planes, as applicable, shall be evaluated to determine which shear plane controls:

- *The plane that parallels the chamfered end of the compression member, as shown in Figure 6A.6.12.6.6-1;*
- *The plane on the side of the compression member that has the smaller framing angle between the that member and the other adjoining members, as shown in Figure 6A.6.12.6.6 -2; and*
- *The plane with the least cross-sectional shear area if the member end is not chamfered and the framing angle is equal on both sides of the compression member.*

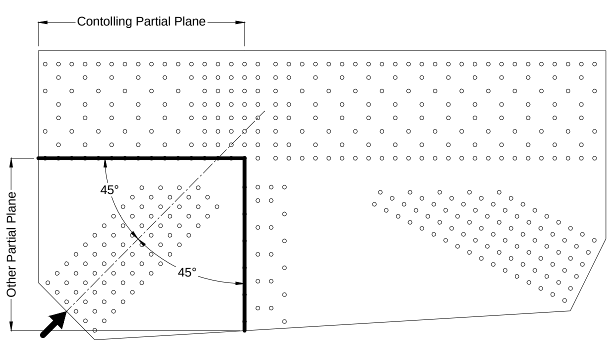


Figure 6A.6.12.6.6-1 – Example of a controlling partial shear plane that parallels the chamfered end of the compression member since that member frames in at an angle of 45 degrees to both the chord and the vertical

reduces the out-of-plane stiffness such that the stability of the plate is affected. Generally, truss verticals and chord members are not subject to the partial plane shear yielding check because there is no adjoining member fastener line that can yield in shear and cause the compression member to become unstable. For example, the two compression members shown in Figure C6A.6.12.6.6-1 would not be subject to a partial plane shear check.

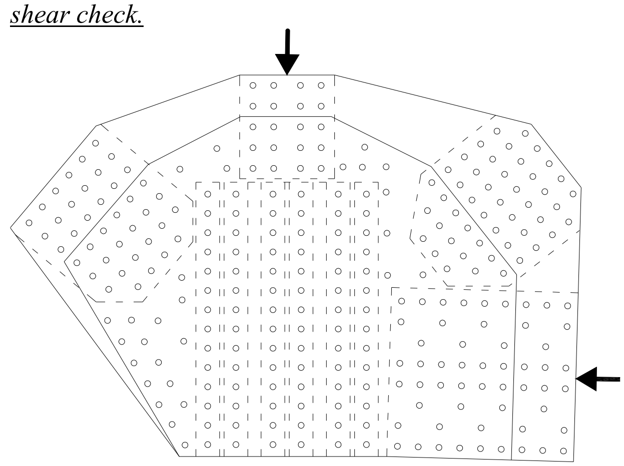


Figure C6A.6.12.6.6-1 – Example showing truss vertical and chord members in compression that do not have admissible partial shear planes that must be checked

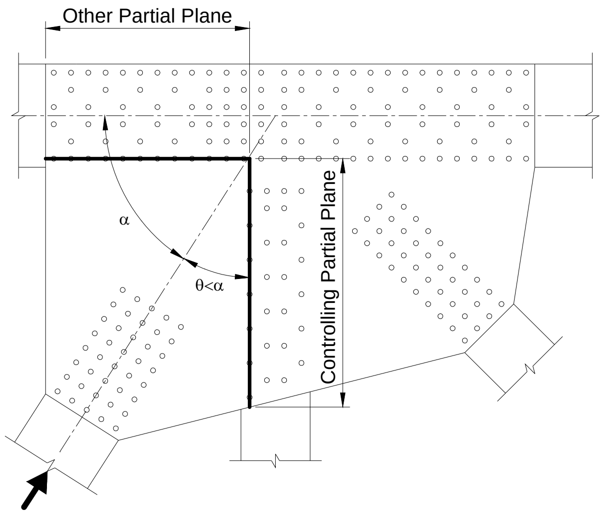


Figure 6A.6.12.6.6-2 – Example of a controlling partial shear plane on the side of a *compression member without a chamfered end that has the smaller framing angle between that member and the other adjoining members (i.e. $\theta < \alpha$)*

6A.6.12.6.7 Gusset Plate Compression Resistance

Gusset plate zones in the vicinity of compression members shall be load rated for plate stability at the strength limit state. The factored compressive resistance, P_r, may be taken as the compressive resistance of an idealized Whitmore plate.

The factored compressive resistance of an idealized Whitmore plate shall be taken as:

$$P_r = \varphi_{cg} P_n \qquad (6A.6.12.6.7\text{-}1)$$

in which:

P_n = nominal compressive resistance of an idealized Whitmore plate determined from Eq. 6A.6.12.6.7-2 or 6A.6.12.6.7-3, as applicable:

- If $\dfrac{P_e}{P_o} \geq 0.44$, then:

$$P_n = \left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o \qquad (6A.6.12.6.7\text{-}2)$$

C6A.6.12.6.7

Experimental testing and finite element simulations performed as part of NCHRP Project 12-84 (NCHRP, 2013) have found that truss gusset plates subject to compression always buckle in a sidesway mode in which the end of the compression member framing into the gusset plate moves out-of-plane. The buckling resistance is dependent upon the chamfering of the member, the framing angles of the members entering the gusset, and the standoff distance of the compression member relative to the surrounding members. The research found that the compressive resistance of gusset plates with large standoff distances was best predicted with modified column buckling equations and Whitmore section analysis. When the members were heavily chamfered reducing their standoff distance, the buckling of the plate was initiated by shear yielding on the partial shear plane adjoining the compression member causing a destabilizing effect, as discussed in Article C6A.6.12.6.6.

Eq. 6A.6.12.6.7-4 is derived by substituting plate properties into column buckling formulas along with an effective length factor of 0.5 that was found to be relevant for a wide variety of gusset-plate geometries (NCHRP, 2013).

- If $\frac{P_e}{P_o} < 0.44$, then:

$$P_n = 0.877 P_e \qquad (6A.6.12.6.7\text{-}3)$$

P_e = elastic critical buckling resistance (kips)

$$= \frac{3.29E}{\left(\frac{L_{mid}}{t_g}\right)^2} A_g \qquad (6A.6.12.6.7\text{-}4)$$

where:

φ_{cg} = resistance factor for gusset plate compression specified in Article 6A.6.3

A_g = gross cross-sectional area of the effective Whitmore plate determined based on 30 degree dispersion angles, as shown in Figure 6A.6.12.6.7-1 (in.^{2}). The Whitmore width shall not be reduced if the width intersects adjoining member bolt lines.

E = modulus of elasticity (ksi).

F_y = specified minimum yield strength (ksi)

L_{mid} = distance from the middle of the Whitmore width to the nearest member fastener line in the direction of the member, as shown in Figure 6A.6.12.6.7-1 (in.).

P_o = equivalent nominal yield resistance = F_yA_g (kips)

t_g = gusset plate thickness (in.)

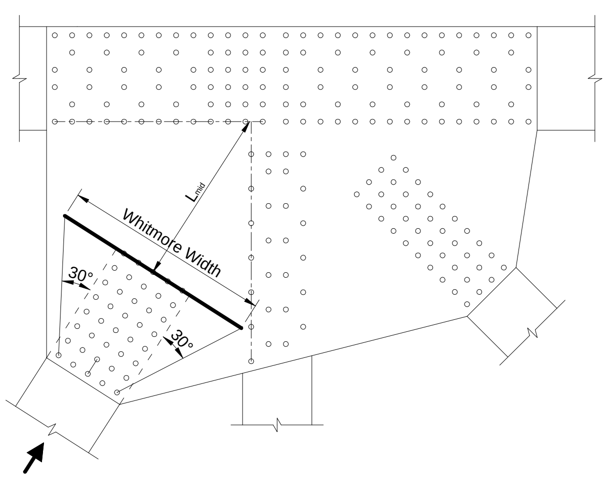


Figure 6A.6.12.6.7-1 – Example connection showing

C6A.6.12.6.8

A conservative model has been adopted to predict the block shear rupture resistance in which the resistance to rupture along the shear plane is added to the resistance to rupture on the tensile plane. Block shear is a rupture or tearing phenomenon and not a yielding phenomenon. However, gross yielding along the shear plane can occur when tearing on the tensile plane commences if $0.58F_uA_{vn}$ exceeds $0.58F_yA_{vg}$. Therefore, Eq. *6A.6.12.6.8-1* limits the term $0.58F_uA_{vn}$ to not exceed $0.58F_yA_{vg}$. Eq. *6A.6.12.6.8-1* is consistent with the philosophy for

the Whitmore width for a compression member derived from 30 degree dispersion angles and the distance L_{mid}

6A.6.12.6.8 Gusset Plate Tension Resistance

Gusset plate zones in the vicinity of tension members shall be load rated for block shear rupture, yielding on the Whitmore plane, and net section fracture on the Whitmore plane at the strength limit state.

The factored block shear rupture resistance shall be taken as:

$$P_r = \varphi_{bs} R_p (0.58 F_u A_{vn} + F_u A_{tn}) \leq \varphi_{bs} R_p (0.58 F_y A_{vg} + F_u A_{tn}) \qquad (6A.6.12.6.8\text{-}1)$$

where:

φ_{bs} = resistance factor for gusset plate block shear rupture specified in Article 6A.6.3

A_{vg} = gross area along the plane resisting shear stress (in.^{2})

A_{vn} = net area along the plane resisting shear stress (in.^{2})

A_{tn} = net area along the plane resisting tension stress (in.^{2})

F_y = specified minimum yield strength of the connected material (ksi)

F_u = specified minimum tensile strength of the connected material (ksi)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size

The factored tensile resistance, P_r, shall be taken as the lesser of the values given by Eqs. *6A.6.12.6.8-2* and *6A.6.12.6.8-3*.

$$P_r = \varphi_y F_y A_g \qquad (6A.6.12.6.8\text{-}2)$$

$$P_r = \varphi_u F_u A_n R_p U \qquad (6A.6.12.6.8\text{-}3)$$

where:

φ_y = resistance factor for yielding of tension members specified in LRFD Design Article 6.5.4.2

tension members where the gross area is used for yielding and the net area is used for rupture.

The reduction factor, R_p, conservatively accounts for the reduced rupture resistance in the vicinity of holes that are punched full size (Brown et al., 2007). No reduction in the net section fracture resistance is required for holes that are drilled full size or subpunched and reamed to size.

The net area, A_n, is the product of the plate thickness and its smallest net width. The width of each standard hole shall be taken as the nominal diameter of the hole. The width of oversize and slotted holes, where permitted, shall be taken as the nominal diameter or width of the hole. The net width shall be determined for each chain of holes extending across the member or element along any transverse, diagonal, or zigzag line.

The net width for each chain shall be determined by subtracting from the width of the element the sum of the widths of all holes in the chain and adding the quantity $s^2/4g$ for each space between consecutive holes in the chain, where:

s = pitch of any two consecutive holes (in.)

g = gage of the same two holes (in.)

φ_u = resistance factor for fracture of tension members specified in LRFD Design Article 6.5.4.2

A_g = gross cross-sectional area of the effective Whitmore plate determined based on 30 degree dispersion angles, as shown in Figure 6A.6.12.6.8-1 ($in.^2$). The Whitmore width shall not be reduced if the width intersects adjoining member bolt lines.

A_n = net cross-sectional area of the effective Whitmore plate determined based on 30 degree dispersion angles, as shown in Figure 6A.6.12.6.8-1 ($in.^2$). The Whitmore width shall not be reduced if the width intersects adjoining member bolt lines.

F_u = specified minimum tensile strength of the gusset plate (ksi)

F_y = specified minimum yield strength of the gusset plate (ksi)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size

U = reduction factor to account for shear lag; taken as 1.0 for gusset plates

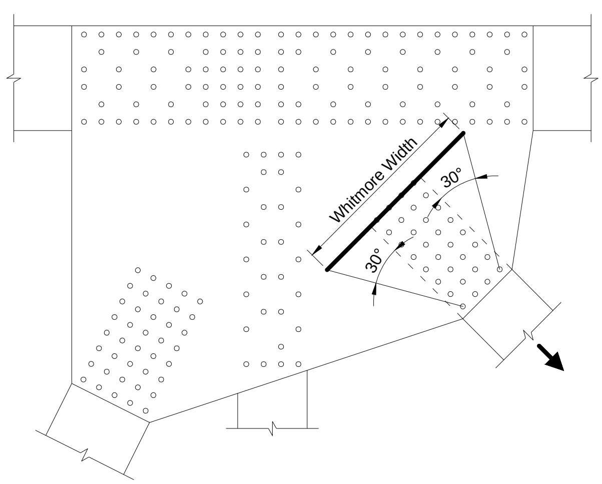


Figure 6A.6.12.6.8-1 – Example connection showing the Whitmore width for a tension member derived from 30 degree dispersion angles

6A.6.12.6.9 Chord Splices

Gusset plates that splice two chord sections together

C6A.6.12.6.9

The resistance equations in this article assume the gusset and splice plates behave as one section to resist the applied axial load and eccentric bending that occurs due to the fact that the resultant forces on the section are offset from the centroid of the section. The spliced section is treated as a beam and the factored resistance is typically determined assuming the stress in the spliced section at the resistance limit is equal to the specified minimum yield strength of the gusset plate.

The application of the idealized Whitmore plate check specified in Article 6A.6.12.6.7 should not be shall be checked using a section analysis considering the relative eccentricities between all plates crossing the splice and the loads on the spliced plane.

For compression chord splices, the factored compressive resistance, P_r, of the spliced section at the strength limit state shall be taken as:

$$P_r = \varphi_{cs} F_{cr} \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \tag{6A.6.12.6.9-1}$$

in which:

F_{cr} = stress in the spliced section at the resistance limit (ksi). F_{cr} shall be taken as the specified minimum yield strength of the gusset plate when the following equation is satisfied:

$$\frac{Kl\sqrt{12}}{t_g} < 25 \tag{6A.6.12.6.9-2}$$

where:

φ_{cs} = resistance factor for gusset plate chord splices specified in Article 6A.6.3

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

K = effective column length factor taken as 0.50 for chord splices

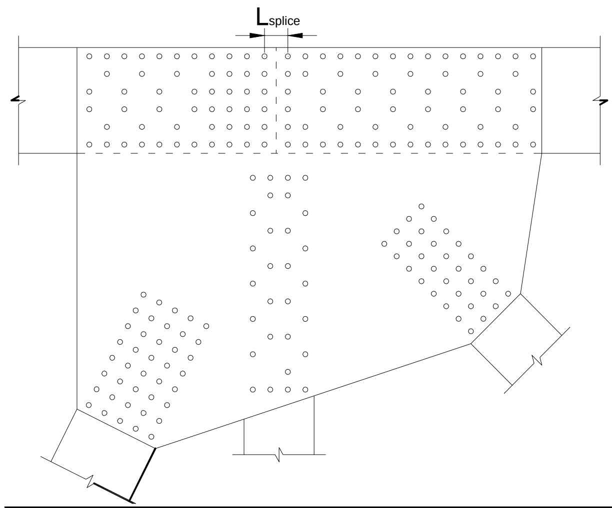
l = *center-to-center distance between the first lines of fasteners in the adjoining chords shown as L_{splice} in Figure 6A.6.12.6.9-1 (in.)*

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

t_g = gusset plate thickness (in.)

applied to members of a compression chord splice.

The slenderness limit for the spliced section given by Eq. 6A.6.12.6.9-2 will normally be met. If not, the Engineer will need to derive a reduced value of F_{cr} to account for possible elastic buckling of the gusset plate within the splice.



(Figure 6A.6.12.6.9-1 – Example connection showing the chord splice parameter, L_{splice}

For tension chord splices, the factored tensile resistance, P_r, of the spliced section at the strength limit state shall be taken as the lesser of the values given by Eqs. *6A.6.12.6.9-3* and *6A.6.12.6.9-4*.

$$P_r = \varphi_{cs} F_y \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \quad \text{(6A.6.12.6.9-3)}$$

$$P_r = \varphi_{cs} F_u \left(\frac{S_n A_n}{S_n + e_p A_n} \right) \quad \text{(6A.6.12.6.9-4)}$$

where:

φ_{cs} = resistance factor for gusset plate chord splices specified in Article 6A.6.3

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

A_n = net area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

F_y = specified minimum yield strength of the gusset plate (ksi)

F_u = specified minimum tensile strength of the gusset plate (ksi)

The yielding and net section fracture checks on the Whitmore plane specified in Article 6A.6.12.6.8 are not considered applicable for checking tension chord splices; however, block shear rupture should be checked for tension chord splice members.

C6A.6.12.6.10

NCHRP Project 12-84 (NCHRP, 2013) found no direct correlation between the buckling resistance of the gusset plate and the free edge slenderness. However, properly stiffening the free edge, as discussed below, could suppress plate buckling.

Since gusset plate buckling was always observed to occur in a sway mode, merely adding stiffeners to just the free edges will not provide any appreciable increase in the compressive resistance of the plate. Either a diaphragm must be added between the two gussets to stiffen against sway, or else stiffening elements must be placed along the free edges such that their full out-of-

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

S_n = net section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

6A.6.12.6.10 Edge Slenderness

Gusset plates shall not be load rated on the basis of edge slenderness.

6A.6.12.6.11 - Refined Analysis

A refined simulation analysis using the finite element method may be employed to determine the nominal

plane yield moment resistance can be developed at the planes that would bend if sway occurs. These requirements do not apply if the free edge is merely being stiffened without relying on an increase in buckling resistance.

The effect of edge stiffening on the compressive resistance of the gusset plate was examined experimentally and analytically in NCHRP Project 12-84 (NCHRP, 2013). The increase in compressive resistance was highly dependent upon the configuration of the connection and was found to vary from 6% to 45%. Generally, connections using chamfered members that allowed for very closely spaced member arrangements experienced little increase in compressive resistance. Connections that had large spans of free plate between the compression members and the surrounding members experienced the largest increase in compressive resistance.

A refined simulation analysis, which is permitted according to the provisions of Article 6A.6.12.6.11, may be used to better quantify the increase in compressive resistance offered by stiffened free edges if utilized for that purpose. However, if the moment of inertia of the edge stiffening element about the surface of the plate is 300 or more times that of the plate itself, the idealized Whitmore plate buckling check from Article 6A.6.12.6.7 need not be applied. The partial shear plane yield requirements of Article 6A.6.12.6.6 should still be applied though in this case.

C6A.14.2.8.11

A refined simulation analysis does not consider the variability of material properties and fabrication tolerances assumed in the AASHTO LRFR calibration. As a result, to be consistent with the philosophy of the AASHTO LRFR specifications, the 0.90 reduction factor was developed as a partial φ factor accounting for these two issues. This value assumes the simulation analysis is accurate enough such that there is no variation in the professional factor and was calibrated to provide a target reliability index of 3.5.

The necessary fidelity of the model is dependent upon the failure mode under investigation. For instance, simple planar shell finite element models of single gusset plates have been successfully used to identify the nominal shear resistance of gusset-plate connections. These models included nonlinear material properties with strain hardening, and member loads were applied as surface tractions at fastener locations. However, additional modeling effort is required to predict the nominal compressive buckling resistance of a gusset plate.

Considering the following list of model attributes, NCHRP Project 12-84 researchers were able to attain *resistance of a gusset-plate connection at the strength limit state in lieu of satisfying the requirements specified in Articles 6A.6.12.6.6 through 6A.6.12.6.9. The nominal resistance obtained from the refined simulation analysis shall be multiplied by 0.90 in order to obtain the factored resistance of the connection.*

model predictions within 9% of experimental values for a *3-dimensional two-panel truss system isolated out of an entire bridge where the connection of interest was located in the center between two panels* (NCHRP, 2013). Model symmetry was not used because the sway buckling mode would not be captured. The following list summarizes other important attributes of the preceding model:

- *The gusset plate, splice plates, and the members for a distance of two member depths away from the gusset-plate edge were modeled with shell elements. The truss was represented with beam elements at all other locations;*

- *The shell elements were able to capture nonlinear geometric and material effects. Nonlinear material properties considered strain hardening;*

- *Each fastener was represented with a line element with deformable, nonlinear material properties;*

- *The mesh contained initial imperfections on all compression members with a maximum out-of-plane magnitude limited by the smaller of: 1) the longest free edge length divided by 150; 2) 0.1 times the gap between the end of the compression member and the next adjoining member; or 3) 100% of the gusset-plate thickness;*

- *The model was proportionally loaded until failure. Typically, buckling can be identified when the analysis no longer converges to a solution. Shear failures are more difficult to identify, but typically occur when the plate exhibits load/displacement softening or when a strain threshold is exceeded after which the analysis predictions become unrealistic.*

Part B—Allowable Stress Rating and Load Factor Rating

6B.5.2—Allowable Stress Method

In the Allowable Stress method, the capacity of a member is based on the rating level evaluated: Inventory level-Allowable Stress, or Operating level-Allowable Stress. The properties to be used for determining the allowable stress capacity for different materials follow. For convenience, the tables provide, where appropriate, the Inventory, Operating, and yield stress values. Allowable stress and strength formulas should be those provided herein or those contained in the AASHTO Standard Specifications. When situations arise that are not covered by these specifications, then rational strength of material formulae should be used consistent with data and plans verified in the field investigation. Deviations from the AASHTO Standard Specifications should be fully documented.

When the bridge materials or construction are unknown, the allowable stresses should be fixed by the Engineer, based on field investigations and/or material testing conducted in accordance with Section 5, and should be substituted for the basic stresses given herein.

6B.5.2.1—Structural Steel

The allowable unit stresses used for determining safe load capacity depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine a nominal yield point. When information on specifications of the steel is not available, allowable stresses should be taken from the applicable "Date Built" column of Tables 6B.5.2.1-1 and 6B.5.2.1-2.

Table 6B.5.2.1-1 gives allowable Inventory stresses and Table 6B.5.2.1-2 gives the allowable Operating stresses for structural steel. The nominal yield stress, F_y, is also shown in Tables 6B.5.2.1-1 and 6B.5.2.1-2. Tables 6B.5.2.1-3 and 6B.5.2.1-4 give the allowable Inventory and Operating Stresses for bolts and rivets. For compression members, the effective length, KL, may be determined in accordance with the AASHTO Standard Specifications or taken as follows:

KL = 75 percent of the total length of a column having riveted end connections

= 87.5 percent of the total length of a column having pinned end connections

The modulus of elasticity, E, for steel should be 29,000,000 lb/in.^{2}

If the investigation of shear and stiffener spacing is desirable, such investigation may be based on the AASHTO Standard Specifications.

C6B.5.2.1

When nonspecification materials are encountered, standard coupon testing procedures may be used to establish the nominal yield point. To provide a 95 percent confidence limit, the nominal yield point would typically be the mean coupon test value minus 1.65 standard deviations.

Mechanical properties of eyebars, high-strength eyebars, forged eyebars, and cables vary depending on manufacturer and year of construction. In the absence of material tests, the Engineer should carefully investigate the material properties using manufacturer's data and compilations of older steel properties before establishing the yield and allowable stresses to be used in load rating the bridge.

The formulas for the allowable bending stress in partially supported or unsupported compression flanges of beams and girders, given in Tables 6B.5.2.1-1 and 6B.5.2.1-2 are the corresponding formula based on given in Table 10.32.1A of the Allowable Stress Design portion of the AASHTO Standard Specifications. The equation in Table 6B.5.2.1-1 is to be used for an Inventory Rating and the equation in Table 6B.5.2.1-2 is to be used for an Operating Rating.

The previously used formulas are inelastic parabolic formulas which treat the lateral torsional buckling of a beam as flexural buckling of the compression flange. This is a very conservative approach for beams with short unbraced lengths. The flexural capacity is reduced for any unbraced length greater than zero. This does not reflect the true behavior of a beam. A beam may reach M_p with unbraced lengths much greater than zero. In addition, the formula neglects the St. Venant torsional stiffness of the cross-sections. This is a significant contribution to the latera1 torsional buckling resistance of rolled shapes, particularly older "I" shapes. The previous formulas must also be limited to the values of l/b listed. This limit is the slenderness ratio when the estimated buckling stress is equal to half the yield strength or 0.275 F_y in terms of an allowable stress. Many floor stringers will have unbraced lengths beyond this limit. If the formulas are used beyond these limits, negative values of the allowable stress can result.

The new formulas have no upper limit which allows the determination of allowable stresses for all unbraced lengths. In addition, the influence of the moment gradient upon buckling capacity is considered using the modifier C_b in the new formulas.

The specification formulas are based on the exact formulations of the lateral torsional buckling of beams. They are currently used in the AISC LRFD Specifications and other specifications throughout the world. They are also being used to design and rate steel bridges by the Load Factor method. Figures 6B.5.2.1-1 and 6B.5.2.1-2 given below show a comparison between the specification formulas and the previous specification formulas for two sections. Figure 6B.5.2.1-1 compares results for a $W18 \times 46$ rolled section. The new specification gives a much higher capacity than the previous specification. The difference is due to the inclusion of the St. Venant torsional stiffness, J, in the proposed specification. Figure 6B.5.2.1-2 shows a similar comparison for a plate-girder section. The section, labeled section 3, has 1.5×16 in. flanges and a $^{5}/_{16} \times 94$ in. web. The previous specification equation gives higher values than the new specification for large unbraced lengths. The previous specification is unconservative in this range. Both graphs show that, for small unsupported lengths, the new specification gives higher allowable stress values. The higher values result from the fact that there is an immediate reduction in capacity versus unsupported length in the previous specification.

Tables 6B.5.2.1-3 and 6B.5.2.1-4 contain the allowable inventory and operating stresses for low-carbon steel bolts, rivets, and high-strength bolts. For high-strength bolts (Table 6B.5.2.1-4), the values for inventory rating correspond to the Allowable Stress design values in the AASHTO Standard Specifications (Tables 10.32.3B and 10.32.3C). The values for the operating rating correspond to the inventory rating values multiplied by the ratio 0.75/0.55. The corresponding values for low-carbon steel bolts (ASTM A307) in Table 6B.5.2.1-3 are based on the values given in Table 10.32.3A of the Standard Specifications.

~~Guidance on the treatment of gusset plates can be found in Article C6A.6.12.1.~~

Specifications and guidance for determining the capacity of gusset plates can be found in Appendix L6B – Formulas for the Capacity, *C*, of Typical Bridge Components Based on the Load Factor Method. Allowable Inventory and Operating stresses for fasteners used in gusset plates can be found in Tables 6B.5.2.1-3 and 6B.5.2.1-4.

Guidance on considering the effects of deterioration on load rating of steel structures can be found in Article C6A.6.5.

6B.5.3—Load Factor Method

Nominal capacity of structural steel, reinforced concrete and prestressed concrete should be the same as specified in the load factor sections of the AASHTO Standard Specifications. Nominal strength calculations should take into consideration the observable effects of deterioration, such as loss of concrete or steel-sectional area, loss of composite action or corrosion.

Allowable fatigue strength should be checked based on the AASHTO Standard Specifications. Special structural or operational conditions and policies of the Bridge Owner may also influence the determination of fatigue strength.

C6B.5.3

Nominal capacities for members in the proposed guidelines are based on AASHTO's Standard Specifications contained in the load factor section. This resistance depends on both the current dimensions of the section and the nominal material strength.

Different methods for considering the observable effects of deterioration were studied. The most reliable method available still appears to be a reduction in the nominal resistance based on measured or estimated losses in cross-sectional area and/or material strengths.

At the present time, load factor methods for determining the capacity of timber and masonry structural elements are not available.

6B.5.3.1—Structural Steel

The yield stresses used for determining ratings should depend on the type of steel used in the structural members. When nonspecification metals are encountered, coupon testing may be used to determine yield characteristics. The nominal yield value should be substituted in strength formulas and is typically taken as the mean test value minus 1.65 standard deviations. When specifications of the steel are not available, yield strengths should be taken from the applicable "date built" column of Tables 6B.5.2.1-1 to 6B.5.2.1-4.

The capacity of structural steel members should be based on the load factor requirements stated in the AASHTO Standard Specifications. The capacity, *C*, for typical steel bridge members is summarized in Appendix L6B. For beams, the overload limitations of Article 10.57 of the AASHTO Standard Specifications should also be considered.

The Operating rating for welds, bolts, and rivets should be determined using the maximum strengths from Table 10.56A in the AASHTO Standard Specifications.

The Operating rating for friction joint fasteners (ASTM A325 bolts) should be determined using a stress of 21 ksi. A_1 and A_2 should be taken as 1.0 in the basic rating equation.

C6B.5.3.1

Guidance on considering the effects of deterioration on load rating of steel structures can be found in Article C6A.6.5.

~~Guidance on the treatment of gusset plates can be found in Article C6A.6.12.1.~~ Specifications and guidance for determining the capacity of gusset plates can be found in Appendix L6B – Formulas for the Capacity, *C*, of Typical Bridge Components Based on the Load Factor Method.

APPENDIX L6B—FORMULAS FOR THE CAPACITY, *C*, OF TYPICAL BRIDGE COMPONENTS BASED ON THE LOAD FACTOR METHOD

Add the following paragraphs to Appendix L6B – Formulas for the Capacity, *C*, of Typical Bridge Components Based on the Load Factor Method − at the end of Section L6B.2:

L6B.2.6—Gusset Plates

Main truss member gusset plates shall be load rated for shear, compression, and/or tension occurring in the vicinity of each connected member. The following sections below outline the necessary checks for performing a Load Factor rating for these gusset plates, which are based on the research performed under NCHRP Project 12-84 (NCHRP, 2013) that only considered LRFR. These provisions supersede the 2009 FHWA Guidelines for gusset-plate load ratings. All of the resistance factors used in this section have not been rigorously determined considering the base HS-20 live load model used for Load Factor rating. Future research looking into the live load variability for truss systems may justify the use of lower resistance factors. An example gusset plate rating in both LRFR and LFR can be found in Appendix A.

L6B.2.6.1 – Fasteners

Fasteners in bolted and riveted gusset plate connections shall be evaluated to prevent fastener shear and plate bearing failures.

The shear capacity of one fastener shall be taken as:

$$C = (\phi F)mA$$

where:

ϕF = shear capacity per fastener area of one fastener specified in Table L6B.2.6.1-1(ksi)

m = number of shear planes

A = cross-sectional area of one fastener (in.^{2}). For rivets, use the undriven diameter to calculate the area.

When bolts carrying loads pass through undeveloped fillers 0.25 in. or more in thickness in axially loaded connections, the bolt shear capacity shall be reduced by:

$$R = \left[\frac{(1+\gamma)}{(1+2\gamma)}\right]$$

where:

γ = A_f/A_p

A_f = sum of the area of the fillers on the top and bottom of the connected plate (in.^{2})

A_p = smaller of either the connected plate area or the sum of the splice plate areas on the top and bottom of the connected plate (in.^{2})

Alternatively, if fillers are extended beyond the connected parts and connected with enough bolts to develop the force in the fillers, the fillers may be considered developed. For rivets, the Undeveloped Filler Plate Reduction Factor, R_3, shall be considered as specified in Article 6A.6.12.5.1.

Table L6B.2.6.1-1

	ϕF (ksi) [a]
A307	18
A325 - threads included in shear plane	35
A325 – threads excluded from shear plane	43
A490 - threads included in shear plane	43
A490 – threads excluded from shear plane	53
Rivets	See Table 6A.6.12.5.1-1
[a] – Tabulated values shall be reduced by 20 percent in bearing-type connections whose length between extreme fasteners in each of the spliced parts measured parallel to the line of axial force exceeds 50 inches.	

The bearing capacity of the connected material at standard, oversize, short-slotted holes or long-slotted holes parallel to the applied force shall be taken as:

$$C = 0.9L_c tF_u \leq 1.8dtF_u \qquad (10\text{-}166b)$$

where:

d = nominal diameter of the fastener (in.)

t = thickness of the gusset plate (in.)

F_u = specified minimum tensile strength of the gusset plate given in Table 10.2A (ksi)

L_c = clear distance between the holes or between the hole and the edge of the material in the direction of the applied bearing force (in.)

For determining the bearing capacity of long-slotted holes loaded perpendicular to the applied force, refer to Article 10.56.1.3.

The Operating rating for friction joint high-strength bolts should be determined according to the provisions of Article 6B.5.3.1.

L6B.2.6.2 – Multi-Layered Gusset and Splice Plates

Where multi-layered gusset and splice plates are used, the resistances of the individual plates may be added together in determining the overall resistance provided that enough fasteners are present to develop the force in the layered gusset and splice plates.

L6B.2.6.3 – Gusset Plate Shear Resistance

Gusset plates shall be load rated for shear yielding and shear rupture on relevant partial and full shear failure plane widths.

Yielding

$\underline{C = \varphi_{vy}(0.58)F_yA_g\Omega}$

Rupture

$\underline{C = \varphi_{vu}(0.58)F_uA_n}$

where:

φ_{vy} = resistance factor for gusset plate shear yielding taken as 1.00

φ_{vu} = resistance factor for gusset plate shear rupture taken as 0.85

Ω = shear reduction factor for gusset plates taken as 0.88

A_g = gross area of the plate resisting shear (in.^{2})

A_n = net area of the plate resisting shear (in.^{2})

F_y = specified minimum yield strength of the gusset plate given in AASHTO Table 10.2A (ksi)

F_u = specified minimum tensile strength of the gusset plate given in AASHTO Table 10.2A (ksi)

Partial shear planes shall only be checked around compression members and only shear yielding on partial shear planes shall be checked. The partial shear plane length shall be taken along adjoining member fastener lines between plate edges and other fastener lines. The following partial shear planes, as applicable, shall be evaluated to determine which shear plane controls:

- *The plane that parallels the chamfered end of the compression member, as shown in Figure L6B.2.6.3-1;*
- *The plane on the side of the compression member that has the smaller framing angle between the that member and the other adjoining members, as shown in Figure L6B.2.6.3-2; and*
- *The plane with the least cross-sectional shear area if the member end is not chamfered and the framing angle is equal on both sides of the compression member.*

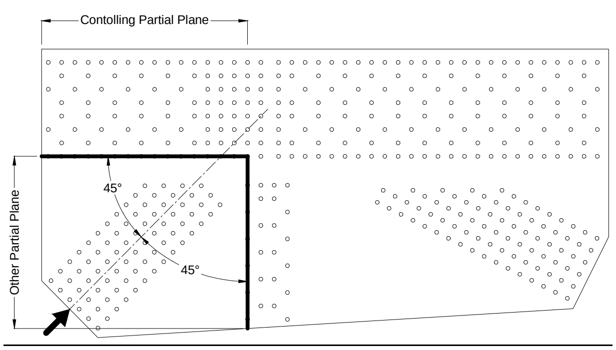


Figure L6B.2.6.3-1 – Example of a controlling partial shear plane that parallels the chamfered end of the compression member since that member frames in at an angle of 45 degrees to both the chord and the vertical

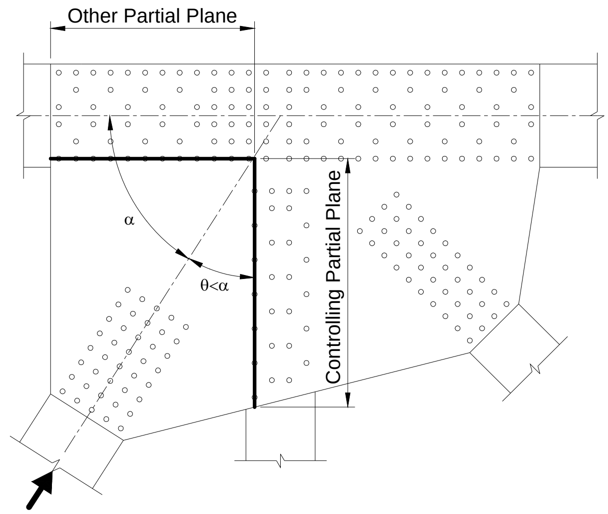


Figure L6B.2.6.3-2 – Example of a controlling partial shear plane on the side of a *compression member without a chamfered end that has the smaller framing angle between that member and the other adjoining members (i.e. $\theta < \alpha$)*.

L6B.2.6.4 – Gusset Plate Compression Resistance

Gusset plate zones in the vicinity of compression members shall be load rated for plate stability, but is not applicable to the rating of compression chord splices. The compressive capacity may be taken as the compressive capacity of an idealized Whitmore plate.

The compressive capacity of an idealized Whitmore plate shall be taken as:

$C = \varphi_{cg}\,(0.85)A_sF_{cr}$

in which:

- If $\dfrac{KL_c\sqrt{12}}{t} \leq \sqrt{\dfrac{2\pi^2 E}{F_y}}$, then:

$$F_{cr} = F_y\left[1 - \frac{F_y}{4\pi^2 E}12\left(\frac{KL_c}{t}\right)^2\right]$$

- If $\dfrac{KL_c\sqrt{12}}{t} > \sqrt{\dfrac{2\pi^2 E}{F_y}}$, then:

$$F_{cr} = \frac{\pi^2 E}{12\left(\dfrac{KL_c}{t}\right)^2}$$

where:

φ_{cg} = resistance factor for gusset plate compression taken as 1.00

A_s = gross cross-sectional area of the effective Whitmore plate determined based on 30 degree dispersion angles, as shown in Figure L6B.2.6.4-1 (in.^{2}). The Whitmore width shall not be reduced if the width intersects adjoining member bolt lines.

E = modulus of elasticity of gusset plate (ksi)

F_y = specified minimum yield strength of the gusset plate (ksi)

K = effective length factor in the plane of buckling taken as 0.50 for gusset plates

L_c = distance from the middle of the Whitmore width to the nearest member fastener line in the direction of the member, as shown in Figure L6B.2.6.4-1 (in.)

t = gusset plate thickness (in.)

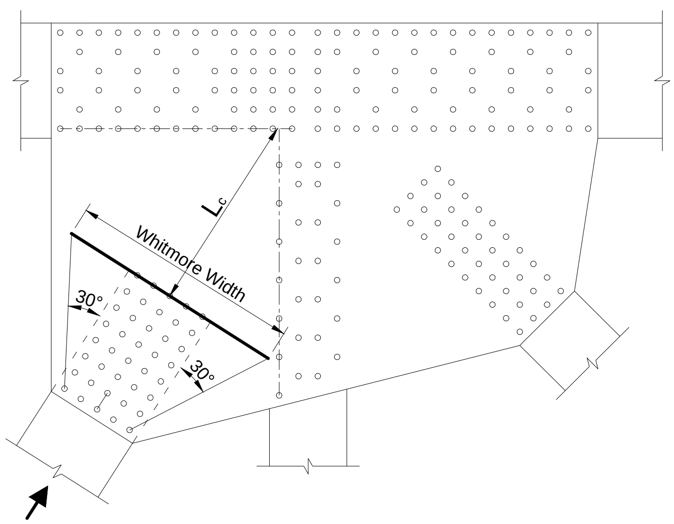


Figure L6B.2.6.4-1 – Example connection showing the Whitmore width for a compression member derived from 30 degree dispersion angles and the distance L_c

L6B.2.6.5 – Gusset Plate Tension Resistance

Gusset plate zones in the vicinity of tension members shall be rated for yielding on the effective area of the Whitmore plane, though not applicable to the rating of tension chord splices.

Yielding

$C = \varphi_y F_y A_e$

in which:

A_e = effective cross-sectional area of the effective Whitmore plate determined based on 30 degree dispersion angles, as shown in Figure L6B.2.6.5-1 (in.^{2}). The Whitmore width shall not be reduced if the width intersects adjoining member bolt lines.

$$= A_n + \beta A_g \leq A_g \tag{10-4w}$$

where:

φ_y = resistance factor for yielding of tension members taken as 1.00

A_n = net section of the member (in.^{2})

β = 0.0 for AASHTO M270 Grade 100/100W steels, or when holes exceed 1-1/4 inch in diameter

= 0.15 for all other steels and when holes are less than or equal to 1-1/4 inch in diameter

F_y = yield strength of the plate specified in AASHTO Table 10.2A (ksi)

Block Shear

The fastener pattern shall also be load rated for the block shear rupture capacity. The block shear rupture capacity shall be taken as:

$C = \varphi_{bs} R_p (0.58 F_u A_{vn} + F_u A_{tn}) \leq \phi_{bs} R_p (0.58 F_y A_{vg} + F_u A_{tn})$

where:

φ_{bs} = resistance factor for block shear rupture taken as 0.85

A_{vg} = gross area along the plane resisting shear stress (in.^{2})

A_{vn} = net area along the plane resisting shear stress (in.^{2})

A_{tn} = net area along the plane resisting tension stress (in.^{2})

F_y = yield strength of the plate specified in AASHTO Table 10.2A (ksi)

F_u = tensile strength of the plate specified in AASHTO Table 10.2A (ksi)

R_p = reduction factor for holes taken equal to 0.90 for bolt holes punched full size and 1.0 for bolt holes drilled full size or subpunched and reamed to size

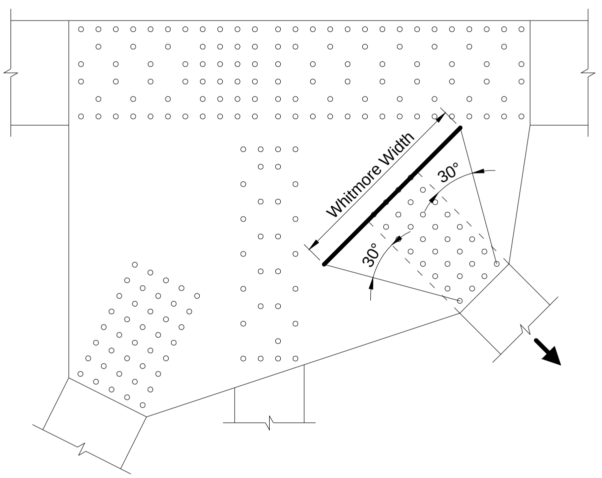


Figure L6B.2.6.5-1 – Example connection showing the Whitmore width for a tension member derived from 30 degree dispersion angles

L6B.2.6.6 - Chord Splices

Gusset plates that splice two chord sections together shall be checked using a section analysis considering the relative eccentricities between all plates crossing the splice and the loads on the spliced plane. The application of the idealized Whitmore plate check should not be applied to members of a compression chord splice.

For compression chord splices, the compressive capacity of the spliced section shall be taken as:

$$C = \phi_{cs} F_{cr} \left(\frac{S_g A_g}{S_g + e A_g} \right)$$

in which:

F_{cr} = stress in the spliced section at the capacity limit (ksi). F_{cr} shall be taken as the specified minimum yield strength of the gusset plate when the following equation is satisfied:

$$\frac{Kl\sqrt{12}}{t_g} < 25$$

(Note: if the preceding equation is not satisfied, the Engineer will need to derive a reduced value of F_{cr} to account for possible elastic buckling of the gusset plate within the splice.)

where:

φ_{cs} = resistance factor for gusset plate chord splices taken as 1.00

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

K = effective column length factor taken as 0.50 for chord splices

l = *center-to-center distance between the first lines of fasteners in the adjoining chords shown as L_{splice} in Figure L6B.2.6.6-1 (in.)*

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

t_g = gusset plate thickness (in.)

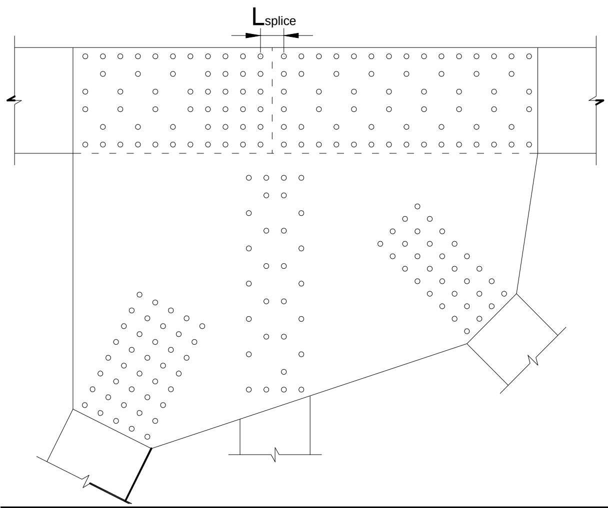


Figure L6B.2.6.6-1 - Example connection showing chord splice parameter, L_{splice}

For tension chord splices, the tensile capacity of the spliced section shall be taken as the lesser of the values given by the following equations:

$$C = \phi_{cs} F_y \left(\frac{S_g A_g}{S_g + eA_g} \right)$$

$$C = \phi_{cs} F_u \left(\frac{S_n A_n}{S_n + eA_n} \right)$$

where:

φ_{cs} = resistance factor for gusset plate chord splices taken as 1.00

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

A_n = net area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

F_y = specified minimum yield strength of the gusset plate (ksi)

F_u = specified minimum tensile strength of the gusset plate (ksi)

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

S_n = net section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

The yielding check on the effective area of the Whitmore plane specified in Article L6B.2.6.5 is not considered applicable for checking tension chord splices; however, block shear rupture should be checked for tension chord splice members.

L6B.2.6.7 - Edge Slenderness

Gusset plates should not be load rated based on any edge slenderness criteria.

L6B.2.6.8 – Refined Analysis

A refined simulation analysis using the finite element method may be employed to determine the nominal resistance of a gusset-plate connection at the strength limit state in lieu of satisfying the requirements specified in Articles L6B.2.6.3 through L6B.2.6.6. See Article 6A.6.12.6.11 for further guidance on the suggested model attributes.

2013 AASHTO BRIDGE COMMITTEE AGENDA ITEM:

SUBJECT: LRFD Bridge Design Specifications: Section 6, Articles 6.3, 6.5.4.2, 6.7.3, 6.14.2.8 & 6.17

TECHNICAL COMMITTEE: T-14 Steel

☒ **REVISION** ☒ **ADDITION** ☐ **NEW DOCUMENT**

☒ **DESIGN SPEC** ☐ **CONSTRUCTION SPEC** ☐ **MOVABLE SPEC**
☐ **MANUAL FOR BRIDGE EVALUATION** ☐ **SEISMIC GUIDE SPEC** ☐ **COASTAL GUIDE SPEC**
☐ **OTHER**

DATE PREPARED: 1/11/2013
DATE REVISED:

AGENDA ITEM:

Item #1

Make the revisions to the indicated articles in Section 6 shown in Attachment A.

OTHER AFFECTED ARTICLES:

None

BACKGROUND:

Following the I-35W Bridge collapse investigation, the National Transportation Safety Board (NTSB) made five recommendations to the Federal Highway Administration (FHWA) and AASHTO. One of these recommendations was to require bridge owners to include main truss member gusset plates as part of the load rating process for these bridges.

To assist the states with this process, FHWA issued a Guidance document in February 2009. This document required, at a minimum, for main truss member gusset plates to be evaluated for five limit states using either the Load Factor Rating (LFR) or Load and Resistance Factor Rating (LRFR) philosophies.

The Guidance document was based on existing provisions in the AASHTO LRFD Bridge Design Specifications and the older AASHTO Standard Specifications for Highway Bridges along with engineering judgment. The FHWA Guidance document was thought to yield conservative gusset plate ratings. As States began to evaluate their inventory with the Guidance document, a need for more direction on some checks was identified, while some facets of other checks were thought to be too conservative. This was the case particularly for the shear reduction factor (Ω) associated with the shear yielding check, and the K-factor selection for use in the column analogy compressive buckling resistance check.

To address these concerns, FHWA initiated a research project collaboratively with the AASHTO-sponsored National Cooperative Highway Research Program (NCHRP) to evaluate the shear, tensile and compressive resistance of gusset plates at the strength limit state (NCHRP Project 12-84). The project tested 12 full-scale experimental gusset plate connections, and used finite element analysis to explore a variety of geometric parameters that could not be experimentally investigated. Primarily, the goal of NCHRP Project 12-84 was to derive new load rating provisions for inclusion in the MBE to satisfy NTSB Recommendation H-08-23, "*When the findings of the Federal Highway Administration–American Association of State Highway and Transportation Officials joint study on gusset plates become available, update the Manual for Bridge Evaluation accordingly*." A separate companion Agenda item is taking care of addressing these recommendations with significant proposed additional content to the MBE. A decision was made to ensure that the LRFR and LFR gusset plate rating specifications in the MBE are reasonably self-sufficient and do not refer back to the LRFD Bridge Design Specifications or the Standard Specifications to a significant extent for determining the factored resistance of the gusset plate and its connections. Therefore, it is not imperative that the gusset plate design provisions derived from the research findings be included in the LRFD Bridge Design Specifications, although it makes sense to unify the two specifications for consistency, and to ensure that gusset plates on new truss bridges are designed based on the latest state-of-the-art knowledge in order to provide a more uniform reliability.

ANTICIPATED EFFECT ON BRIDGES:

The current specification provisions in LRFD Bridge Design Specification Article 6.14.2.8 allow the Engineer significant discretion in the design of truss member gusset plates. The new provisions are much more comprehensive and should result in a more unified design approach and a more uniform reliability for these gusset plate designs. The new provisions may result in thicker gusset plates than would be required using the current specifications, but the cost associated with thickening gusset plates is relatively marginal, and should be the only significant perceived difference when the proposed design specifications are employed.

REFERENCES:

See the revised Article 6.17 in Attachment A.

OTHER:

None

ATTACHMENT A – 2013 AGENDA ITEM -- T-14

Make the following revisions to Articles 6.3, 6.5.4.2, 6.7.3, 6.14.2.8 & 6.17 of the LRFD Bridge Design Specifications:

6.3—NOTATION

A_g = gross area of a member (in.^{2}); gross cross-section area of a compression member (in.^{2}); gross area of a flange (in.^{2}); gross cross-sectional area of the member (in.^{2}); gross area of the section based on the design wall thickness (in.) (6.6.1.2.3) (6.8.2.1) (6.9.4.1.1) (6.9.4.1.2) (6.9.4.1.3) (6.10.1.8) (6.12.1.2.3c) (6.14.2.8.4) (6.14.2.8.6)

A_n = net cross-section area of a tension member (in.^{2}); net area of a flange (in.^{2}); net area of gusset and splice plates (in.^{2}) (6.6.1.2.3) (6.8.2.1) (6.10.1.8) (6.14.2.8.6)

A_{vg} = gross area along the cut carrying shear stress in block shear (in.^{2}); gross area of the connection element subject to shear (in.^{2}); gross area of gusset plate subject to shear (in.^{2}) (6.13.4) (6.13.5.3) (6.14.2.8.3)

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane in gusset plates (in.) (6.14.2.8.6)

F_{cr} = critical buckling stress for plates (ksi); elastic lateral torsional buckling stress (ksi); shear buckling resistance (ksi); elastic local buckling stress (ksi) (C6.9.4.2) (6.10.1.6) (6.12.1.2.3c) (6.12.2.2.3) (6.12.2.2.5) (6.14.2.8.6)

L_{mid} = in a gusset plate connections, the distance from the last row of fasteners in the compression member under consideration to the first row of fasteners in the closest adjacent connected member, measured along the line of action of the compressive axial force (6.14.2.8.4)

P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, ~~and~~ as specified in Article 6.9.4.1.3 for torsional bucking or flexural-torsional buckling, as applicable (kips) (6.9.4.1.1), and as specified in Article 6.14.2.8.4 for gusset plate buckling (6.14.2.8.4)

P_n = nominal bearing resistance on pin plates (kip); nominal axial compressive resistance (kip); total longitudinal force in the concrete deck over an interior support for the design of the shear connectors at the strength limit state, taken as the lesser of either P_{1n} or P_{2n} (kip) (6.8.7.2) (6.9.2.1) (6.10.10.4.2) (6.14.2.8.4)

P_e = elastic critical buckling resistance determined as specified in Article 6.9.4.1.2 for flexural buckling, and as specified in Article 6.9.4.1.3 for torsional bucking or flexural-torsional buckling, as applicable (kips) (6.9.4.1.1) (6.14.2.8.4)

S_g = elastic gross section modulus of gusset plates and splice plates (in.^{3}) (6.14.2.8.6)

S_n = elastic net section modulus of gusset plates and splice plates (in.^{3}) (6.14.2.8.6)

t_g = gusset plate thickness (6.14.2.8.4)

ϕ_{vy} = resistance factor for truss gusset plate shear yielding (6.5.4.2) (6.14.2.8.3)

ϕ_{cg} = resistance factor for truss gusset plate compression (6.5.4.2) (6.14.2.8.4)

ϕ_{cs} = resistance factor for truss gusset chord splice (6.5.4.2) (6.14.2.8.6)

θ = angle of inclination of the bottom flange of a variable web depth member (degrees); angle of inclination of the web plate of a box section to the vertical (degrees) (C6.10.1.4) (6.11.9); framing angle of compression member relative to an adjoining member in a gusset-plate connection (6.14.2.8.4)

Ω = shear yield reduction factor for gusset plates (6.14.2.8.3)

6.5.4—Strength Limit State

6.5.4.1—General

Strength and stability shall be considered using the applicable strength load combinations specified in Table 3.4.1-1.

6.5.4.2—Resistance Factors

Resistance factors, ϕ, for the strength limit state shall be taken as follows:

- For flexure $\phi_f = 1.00$
- For shear $\phi_v = 1.00$
- For axial compression, steel only $\phi_c = 0.90$
- For axial compression, composite $\phi_c = 0.90$
- For tension, fracture in net section $\phi_u = 0.80$
- For tension, yielding in gross section $\phi_y = 0.95$
- For bearing on pins in reamed, drilled or bored holes and on milled surfaces $\phi_b = 1.00$
- For bolts bearing on material $\phi_{bb} = 0.80$
- For shear connectors $\phi_{sc} = 0.85$
- For A 325 and A 490 bolts in tension $\phi_t = 0.80$
- For A 307 bolts in tension $\phi_t = 0.80$
- For F 1554 bolts in tension $\phi_t = 0.80$
- For A 307 bolts in shear $\phi_s = 0.75$
- For F 1554 bolts in shear $\phi_s = 0.75$
- For A 325 and A 490 bolts in shear $\phi_s = 0.80$
- For block shear $\phi_{bs} = 0.80$
- For shear, rupture in connection element $\phi_{vu} = 0.80$
- For truss gusset plate compression $\phi_{cg} = 0.75$
- For truss gusset plate chord splices $\phi_{cs} = 0.65$
- For truss gusset plate shear yielding $\phi_{vy} = 0.80$
- For web crippling $\phi_w = 0.80$
- For weld metal in complete penetration welds:
  - o shear on effective area $\phi_{e1} = 0.85$
  - o tension or compression normal to effective area same as base metal
  - o tension or compression parallel to axis of the weld same as base metal
- For weld metal in partial penetration welds:
  - o shear parallel to axis of weld $\phi_{e2} = 0.80$
  - o tension or compression parallel to axis of weld same as base metal
  - o compression normal to the effective area same as base metal
  - o tension normal to the effective area $\phi_{e1} = 0.80$
- For weld metal in fillet welds:

C6.5.4.2

Base metal ϕ as appropriate for resistance under consideration.

The resistance factors for truss gusset plates were developed and calibrated to a target reliability index of 4.5 for the Strength I load combination at a dead-to-live ratio, DL/LL, of 6.0. More liberal ϕ factors could be justified at a DL/LL less than 6.0.

- tension or compression parallel to axis of the weld — same as base metal
- shear in throat of weld metal — $\phi_{e2} = 0.80$

- For resistance during pile driving — $\phi = 1.00$
- For axial resistance of piles in compression and subject to damage due to severe driving conditions where use of a pile tip is necessary:
  - H-piles — $\phi_c = 0.50$
  - pipe piles — $\phi_c = 0.60$
- For axial resistance of piles in compression under good driving conditions where use of a pile tip is not necessary:
  - H-piles — $\phi_c = 0.60$
  - pipe piles — $\phi_c = 0.70$
- For combined axial and flexural resistance of undamaged piles:
  - axial resistance for H-piles — $\phi_c = 0.70$
  - axial resistance for pipe piles — $\phi_c = 0.80$
  - flexural resistance — $\phi_f = 1.00$

The basis for the resistance factors for driven steel piles is described in Article 6.15.2. Further limitations on usable resistance during driving are specified in Article 10.7.8.

Indicated values of ϕ_c and ϕ_f for combined axial and flexural resistance are for use in interaction equations in Article 6.9.2.2.

6.7.3−Minimum Thickness of Steel

Structural steel, including bracing, cross-frames, and all types of gusset plates, except for gusset plates used in trusses, webs of rolled shapes, closed ribs in orthotropic decks, fillers, and in railings, shall not be less than 0.3125 in. in thickness. The thickness of gusset plates used in trusses shall not be less than 0.375 in.

For orthotropic decks, the web thickness of rolled beams or channels and of closed ribs in orthotropic decks shall not be less than 0.25 in., the deck plate thickness shall not be less than 0.625 in. or four percent of the larger spacing of the ribs, and the thickness of closed ribs shall not be less than 0.1875 in.

Where the metal is expected to be exposed to severe corrosive influences, it shall be specially protected against corrosion or sacrificial metal thickness shall be specified.

6.14—PROVISIONS FOR STRUCTURE TYPES

6.14.2.8—Gusset Plates

6.14.2.8.1−General

~~The provisions of Articles 6.13.4 and 6.13.5 shall apply, as applicable.~~

Gusset or connection plates should be used for connecting ~~main~~ truss members, except where the members are pin-connected. The fasteners connecting each member shall be symmetrical with the axis of the member, so far as practicable, and the full development of the elements of the member should be given consideration.

Re-entrant cuts, except curves made for appearance, should be avoided as far as practicable.

~~The maximum stress from combined factored flexural and axial loads shall not exceed $\phi_f F_y$ based on the gross area.~~

~~The maximum shear stress on a section due to the factored loads shall be $\phi_v F_y/\sqrt{3}$ for uniform shear and $\phi_v 0.74 F_y/\sqrt{3}$ for flexural shear computed as the factored shear force divided by the shear area.~~

~~If the length of the unsupported edge of a gusset plate exceeds $2.06(E/F_y)^{1/2}$ times its thickness, the edge shall be stiffened. Stiffened and unstiffened gusset edges shall be investigated as idealized column sections.~~

Gusset plates shall satisfy the minimum plate thickness requirement for gusset plates used in trusses specified in Article 6.7.3. Except as specified herein, gusset plates shall be designed for shear, compression, and/or tension occurring in the vicinity of each connected member, as applicable, according to the requirements specified in Articles 6.14.2.8.3 through 6.14.2.8.5. Gusset plates serving as a chord splice shall also be independently designed as a splice according to the provisions of Article 6.14.2.8.6. The edge slenderness requirement specified in Article 6.14.2.8.7 shall be considered.

6.14.2.8.2 – Multi-Layered Gusset and Splice Plates

Where multi-layered gusset and splice plates are used, the resistances of the individual plates may be added together when determining the factored resistances specified in Articles 6.14.2.8.3 through 6.14.2.8.6 *provided that enough fasteners are present to develop the force in the layered gusset and splice plates.*

~~C6.14.2.8~~

~~Following the 2007 collapse of the I-35W bridge in Minneapolis, the traditional procedures for designing gusset plates, including the provisions of this Article, have been under extensive review. As of Spring 2008, new design procedures have not been codified. Guidance from FHWA is expected shortly. Designers are advised to obtain the latest approved recommendations from Owners.~~

C6.14.2.8.1

The provisions provided in this article are intended for the design of double gusset-plate connections used in trusses. The validity of the requirements for application to single gusset-plate connections has not been verified.

These provisions are based on the findings from NCHRP Project 12-84 (NCHRP, 2013). Example calculations illustrating the application of the resistance equations for gusset-plate connections contained herein are provided in NCHRP (2013).

C6.14.2.8.2

Kulak et al.(1987) contains additional guidance on determining the number of fasteners required to develop the force in layered gusset and splice plates.

6.14.2.8.3 Shear Resistance

Gusset plates shall be designed for shear yielding and shear rupture.

For shear yielding, the factored shear resistance shall be taken as:

$$V_r = \phi_{vy} 0.58 F_y A_{vg} \Omega \quad (6.14.2.8.3\text{-}1)$$

where:

ϕ_{vy} = resistance factor for truss gusset plate shear yielding specified in Article 6.5.4.2

Ω = shear reduction factor for gusset plates taken as 0.88

A_{vg} = gross area of the shear plane (in.^{2})

F_y = specified minimum yield strength of the gusset plate (ksi)

For shear rupture, the factored shear resistance shall be determined from Eq. 6.13.5.3-2.

Shear shall be checked on relevant partial and full failure plane widths. Partial shear planes shall only be checked around compression members and only Eq. 6.14.2.8.3-1 shall apply to partial shear planes. The partial shear plane length shall be taken along adjoining member fastener lines between plate edges and other fastener lines. The following partial shear planes, as applicable, shall be evaluated to determine which shear plane controls:

- *The plane that parallels the chamfered end of the compression member, as shown in Figure 6.14.2.8.3-1;*
- *The plane on the side of the compression member that has the smaller framing angle between the that member and the other adjoining members, as shown in Figure 6.14.2.8.3-2; and*
- *The plane with the least cross-sectional shear area if the member end is not chamfered and the framing*

C6.14.2.8.3

The Ω shear reduction factor is used only in the evaluation of truss gusset plates for shear yielding. This factor accounts for the nonlinear distribution of shear stresses that form along a failure plane as compared to an idealized plastic shear stress distribution. The nonlinearity primarily develops due to shear loads not being uniformly distributed on the plane and also due to strain hardening and stability effects. The Ω-factor was developed using shear yield data generated in NCHRP Project 12-84 (NCHRP, 2013). On average, Ω was 1.02 for a variety of gusset-plate geometries; however, there was significant scatter in the data due to proportioning of load between members, and variations in plate thickness and joint configuration. The specified Ω-factor has been calibrated to account for shear plane length-to-thickness ratios varying from 85 to 325.

Failure of a full width shear plane requires relative mobilization between two zones of the plate, typically along chords. Mobilization cannot occur when a shear plane passes through a continuous member; for instance, a plane passing through a continuous chord member that would require shearing of the member itself.

Research has shown that the buckling of connections with tightly spaced members is correlated with shear yielding around the compression members. This is important because the buckling criteria used in Article 6.14.2.8.4 would overestimate the compressive buckling resistance of these types of connections. Once a plane yields in shear, the reduction in the plate modulus reduces the out-of-plane stiffness such that the stability of the plate is affected. Generally, truss verticals and chord members are not subject to the partial plane shear yielding check because there is no adjoining member fastener line that can yield in shear and cause the compression member to become unstable. For example, the two compression members shown in Figure C6.14.2.8.3-1 would not be subject to a partial plane shear check.

angle is equal on both sides of the compression member.

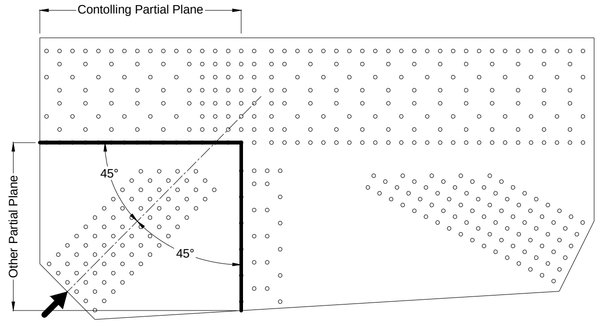


Figure 6.14.2.8.3-1 – Example of a controlling partial shear plane that parallels the chamfered end of the compression member since that member frames in at an angle of 45 degrees to both the chord and the vertical

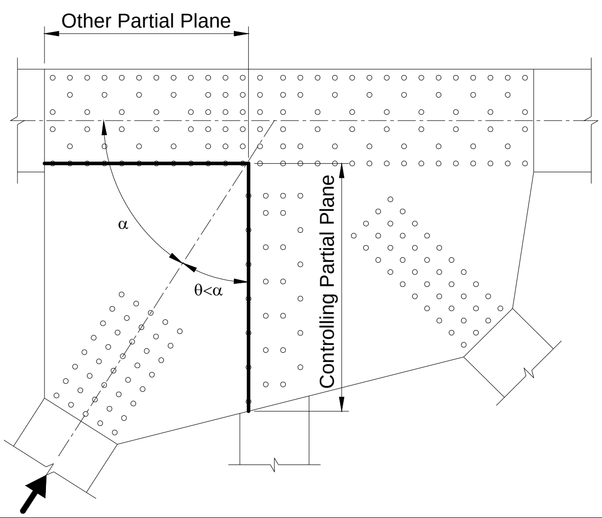


Figure 6.14.2.8.3-2 – Example of a controlling partial shear plane on the side of a *compression member without a chamfered end that has the smaller framing angle between that member and the other adjoining members (i.e. θ < α)*

6.14.2.8.4 Compresion Resistance

Gusset plate zones in the vicinity of compression members shall be designed for plate stability. The factored compressive resistance, P_r, may be taken as the compressive resistance of an idealized Whitmore plate.

The factored compressive resistance of an idealized Whitmore plate shall be taken as:

$$P_r = \phi_{cg} P_n \qquad (6.14.2.8.4\text{-}1)$$

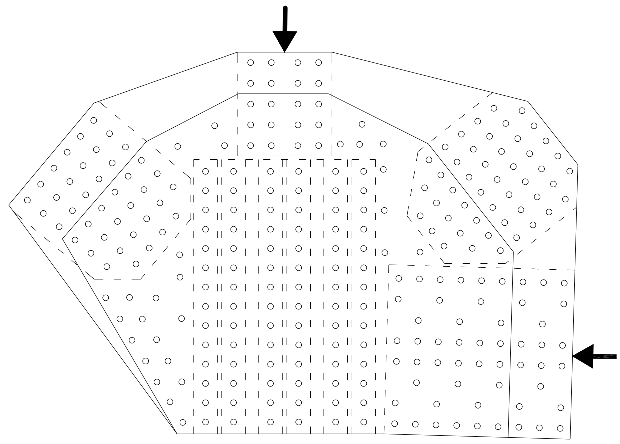


Figure C6.14.2.8.3-1 – Example showing truss vertical and chord members in compression that do not have admissible partial shear planes that must be checked

C6.14.2.8.4

Experimental testing and finite element simulations performed as part of NCHRP Project 12-84 (NCHRP, 2013) have found that truss gusset plates subject to compression always buckle in a sidesway mode in where:

ϕ_{cg} = resistance factor for truss gusset plate compression specified in Article 6.5.4.2

P_n = nominal compressive resistance of an idealized Whitmore plate determined from Eq. 6.9.4.1.1-1 or 6.9.4.1.1-2, as applicable.

In the calculation of P_n, the slender element reduction factor, Q, shall be taken as 1.0, and the elastic critical buckling resistance, P_e shall be taken as:

$$P_e = \frac{3.29E}{\left(\frac{L_{mid}}{t_g}\right)^2} A_g \qquad (6.14.2.8.4\text{-}2)$$

where:

A_g = gross cross-sectional area of the effective Whitmore plate determined based on 30 degree dispersion angles, as shown in Figure 6.14.2.8.4-1 (in.^{2}). The Whitmore width shall not be reduced if the with intersects adjoining member bolt lines.

L_{mid} = distance from the middle of the Whitmore width to the nearest member fastener line in the direction of the member, as shown in Figure 6.14.2.8.4-1 (in.)

t_g = gusset-plate thickness (in.)

which the end of the compression member framing into the gusset plate moves out-of-plane. The buckling resistance is dependent upon the chamfering of the member, the framing angles of the members entering the gusset, and the standoff distance of the compression member relative to the surrounding members. The research found that the compressive resistance of gusset plates with large standoff distances was best predicted with modified column buckling equations and Whitmore section analysis. When the members were heavily chamfered reducing their standoff distance, the buckling of the plate was initiated by shear yielding on the partial shear plane adjoining the compression member causing a destabilizing effect, as discussed in Article C6.14.2.8.3.

Eq. 6.14.2.8.4-2 is derived by substituting plate properties into Eq. 6.9.4.1.2-1 along with an effective length factor of 0.5 that was found to be relevant for a wide variety of gusset-plate geometries (NCHRP, 2013).

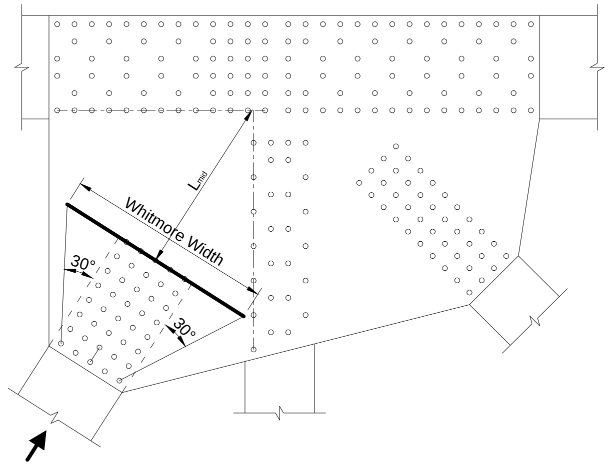


Figure 6.14.2.8.4-1 – Example connection showing the Whitmore width for a compression member derived from 30 degree dispersion angles and the distance L_{mid}

6.14.2.8.5 Tension Resistance

Gusset plate zones in the vicinity of tension members shall be designed for yielding, fracture and block shear rupture according to the provisions of Article 6.13.5.2. When checking Eqs. 6.8.2.1-1 and 6.8.2.1-2, the Whitmore width defined in Figure 6.14.2.8.5-1 shall be used to define the effective area. The Whitmore width shall not be reduced if the width intersects adjoining member bolt lines.

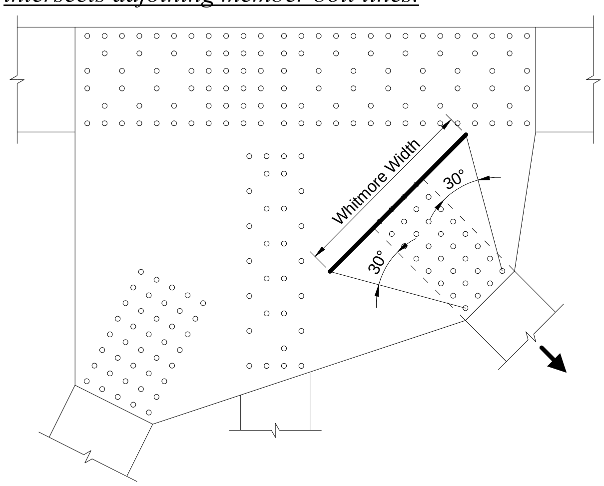


Figure 6.14.2.8.5-1 – Example connection showing the Whitmore width for a tension member derived from 30 degree dispersion angles

6.14.2.8.6 Chord Splices

Gusset plates that splice two chord sections together shall be checked using a section analysis

C6.14.2.8.6

The resistance equations in this article assume the gusset and splice plates behave as one section to resist the applied axial load and eccentric bending that occurs considering the relative eccentricities between all plates crossing the splice and the loads on the spliced plane.

For compression chord splices, the factored compressive resistance, P_r, of the spliced section shall be taken as:

$$P_r = \phi_{cs} F_{cr} \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \quad (6.14.2.8.6\text{-}1)$$

in which:

F_{cr} = stress in the spliced section at the resistance limit (ksi). F_{cr} shall be taken as the specified minimum yield strength of the gusset plate when the following equation is satisfied:

$$\frac{Kl\sqrt{12}}{t_g} < 25 \quad (6.14.2.8.6\text{-}2)$$

where:

ϕ_{cs} = resistance factor for truss gusset plate chord splices specified in Article 6.5.4.2

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to the spliced plane (in.)

K = effective column length factor taken as 0.50 for chord splices

l = *center-to-center distance between the first lines of fasteners in the adjoining chords shown as L_{splice} in Figure 6.14.2.8.6-1 (in.)*

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

due to the fact that the resultant forces on the section are offset from the centroid of the section. The spliced section is treated as a beam and the factored resistance is typically determined assuming the stress in the spliced section at the resistance limit is equal to the specified minimum yield strength of the gusset plate.

The application of the idealized Whitmore plate check specified in Article 6.14.2.8.4 should not be applied to members of a compression chord splice.

The slenderness limit for the spliced section given by Eq. 6.14.2.8.6-2 will normally be met. If not, the Engineer will need to derive a reduced value of F_{cr} to account for possible elastic buckling of the gusset plate within the splice.

t_g = gusset plate thickness (in.)

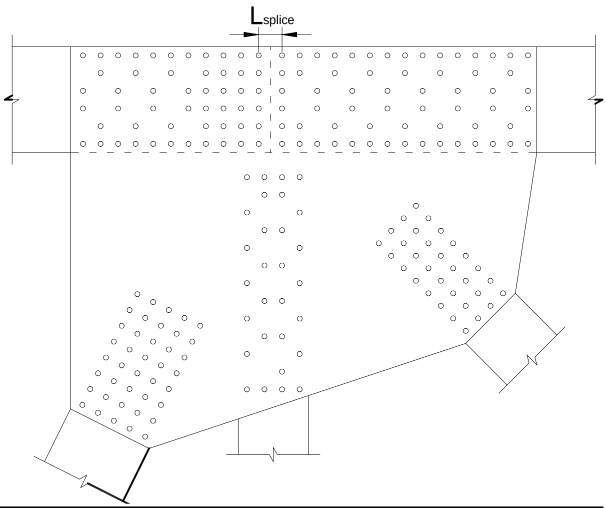


Figure 6.14.2.8.6-1 – Example connection showing chord splice parameter, L_{splice}

For tension chord splices, the factored tensile resistance, P_r, shall be taken as the lesser of the values given by Eqs. 6.14.2.8.6-3 and 6.14.2.8.6-4.

$$P_r = \phi_{cs} F_y \left(\frac{S_g A_g}{S_g + e_p A_g} \right) \qquad (6.14.2.8.6\text{-}3)$$

$$P_r = \phi_{cs} F_u \left(\frac{S_n A_n}{S_n + e_p A_n} \right) \qquad (6.14.2.8.6\text{-}4)$$

where:

ϕ_{cs} = resistance factor for truss gusset plate chord splices specified in Article 6.5.4.2

A_g = gross area of all plates in the cross-section intersecting the spliced plane (in.^{2})

A_n = net area of all plates in the cross-section intersecting the spliced plane (in.^{2})

e_p = distance between the centroid of the cross-section and the resultant force perpendicular to

The yielding and net section fracture checks on the Whitmore plane specified in Article 6.14.2.8.5 are not considered applicable for checking tension chord splices; however, block shear rupture should be checked for tension chord splice members

C6.14.2.8.7

This article is intended to provide good detailing practice to reduce deformations of free edges during fabrication, erection, and service. NCHRP Project 12-84 (NCHRP, 2013) found no direct correlation between the buckling resistance of the gusset plate and the free the spliced plane (in.)

F_y = specified minimum yield strength of the gusset plate (ksi)

F_u = specified minimum tensile strength of the gusset plate (ksi)

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

S_n = net section modulus of all plates in the cross-section intersecting the spliced plane (in.^{3})

6.14.2.8.7 Edge Slenderness

If the length of the unsupported edge of a gusset plate exceeds $2.06(E/Fy)^{1/2}$ times its thickness, the edge should be stiffened.

edge slenderness. Gusset plate buckling was observed to occur in a sway mode, Thus, *a*dding stiffeners to just the free edges will not provide any appreciable increase in the compressive resistance of the plate unless a diaphragm is added between the two gussets to stiffen against sway, or the stiffening elements are placed along the free edges such that their full out-of-plane yield moment resistance is developed at the planes that would bend if sway occurs. Therefore, simply adding edge stiffeners alone will not provide the desired minimum compressive buckling resistance at the strength limit state.

6.17−REFERENCES

Add the following reference:

NCHRP. 2013. *Guidelines for the Load and Resistance Factor Design and Rating of Welded, Riveted and Bolted Gusset-Plate Connections for Steel Bridges*, Transportation Research Board, National Research Council, Washington D.C (to be published).