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

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

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

In which:

P_e = elastic critical buckling resistance (kips)

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

where:

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

P_o = equivalent nominal yield resistance = $F_y A_g$ (kips), in which:

F_y = specified minimum yield strength (ksi)

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

E = modulus of elasticity (ksi)

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

t_g = gusset plate thickness (in.)

reasonably predicted using modified column buckling equations and Whitmore section analysis. When the members were heavily chamfered, reducing the L_{mid} 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 (Ocel, 2013).

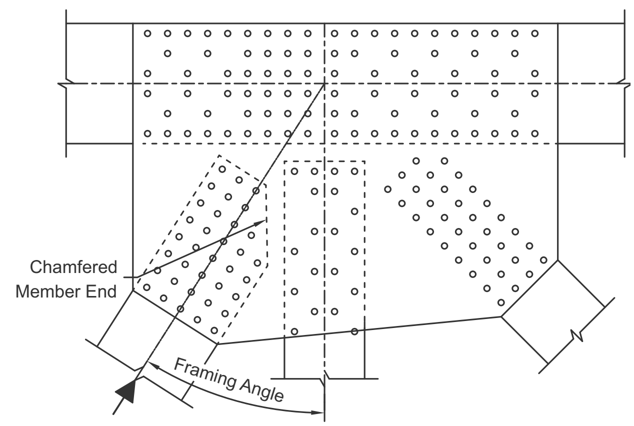

Figure C6A.6.12.6.7-1—Example Connection Showing a Typical Chamfered Member End and Member Framing Angle

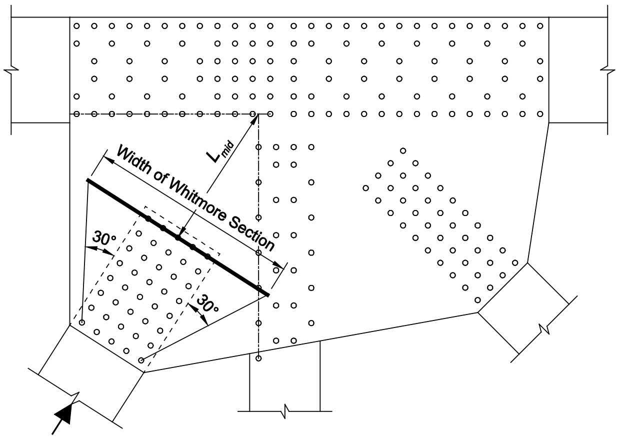

Figure 6A.6.12.6.7-1—Example Connection Showing the Whitmore Section for a Compression Member Derived From 30 Degree Dispersion Angles and the Distance L_{mid}

The provisions of this Article shall not be applied to compression chord splices.

6A.6.12.6.8—Gusset Plate Tensile Resistance

The factored tensile resistance, P_r, of gusset plates at the strength limit state shall be taken as the smallest factored resistance in tension based on block shear rupture, yielding on the Whitmore section, and net section fracture on the Whitmore section.

The factored block shear rupture resistance shall be taken as:

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

where:

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

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

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

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)

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

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 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 is to be taken as the nominal diameter of the hole. The width of oversize and slotted holes, where permitted, is to be taken as the nominal diameter or width of the hole. The net width is to 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 is to 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 = gauge of the same two holes (in.)

The factored resistances for yielding on the Whitmore section and net section fracture on the Whitmore section shall be determined from Eqs. 6A.6.12.6.8-2 and 6A.6.12.6.8-3, respectively.

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

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

where:

φ_y = resistance factor for yielding of tension members specified in Article 6A.6.3

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

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

φ_u = resistance factor for fracture of tension members specified in Article 6A.6.3

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

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

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

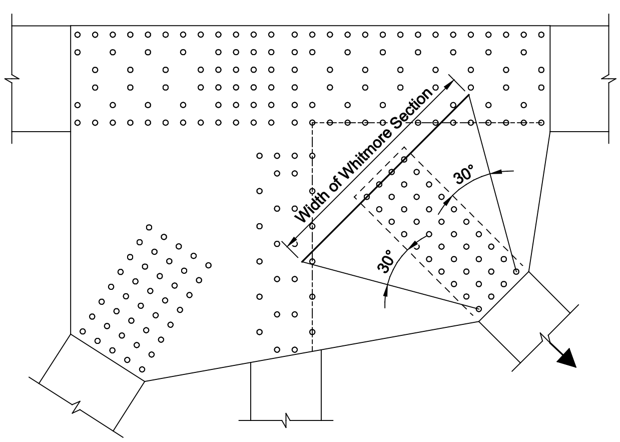

Figure 6A.6.12.6.8-1—Example Connection Showing the Whitmore Section for a Tension Member Derived from 30 Degree Dispersion Angles

The provisions of this Article shall not be applied to tension chord splices.

6A.6.12.6.9—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.

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 limit of usable resistance (ksi); shall be taken as the specified minimum yield strength of the gusset plate when the following equation is satisfied:

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

where:

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

S_g = gross section modulus of all plates in the cross-section intersecting the spliced plane (in.^{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_{splice} = center-to-center distance between the first lines of fasteners in the adjoining chords as shown in Figure 6A.6.12.6.9-1 (in.)

t_g = gusset plate thickness (in.)

C6A.6.12.6.9

This Article is only intended to cover the load rating of chord splices that occur within the gusset plates. For gusset plates also serving the role of a chord splice, the forces from all members framing into the connection must be considered. The chord splice forces are the resolved axial forces acting on each side of the spliced section, as illustrated in Figure C6A.6.12.6.9-1. Generally, a difference in the resolved forces on the two sides of a chord splice arises due to the use of envelope forces, i.e. forces due to nonconcurrent loads. Where envelope forces are used, the resolved forces should include consideration of the concurrence or nonconcurrence of forces to avoid potentially unconservative reductions in the connection force. The chord splice should be investigated for the larger of the two resolved forces on either side of the splice.

For chord splices that are in full compression under the loads being considered and that are detailed with milled ends in full contact bearing at the splice, resolved forces may be adjusted to account for the transfer of a portion of the compressive load though end bearing as long as the capacity in bearing is verified. The portion of the compressive load that may be transferred in bearing in such cases is specified in LRFD Design Article 6.13.6.1.3.

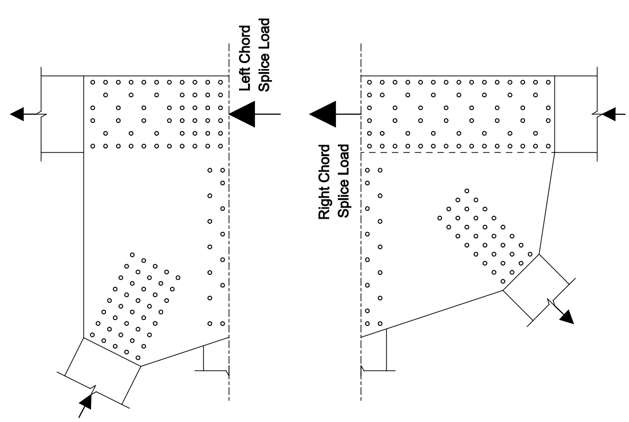

Figure C6A.6.12.6.9-1—Example Connection Showing the Resolution of the Member Forces into Forces Acting on Each Side of a Chord Splice

The resistance equations in this Article assume the gusset and splice plates behave as one combined spliced

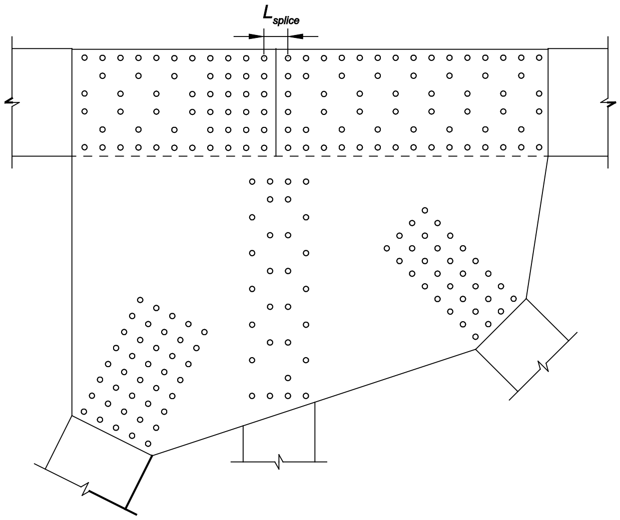

Figure 6A.6.12.6.9-1—Example Connection Showing the Chord Splice Parameter, L splice

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.

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

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

where:

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

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

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

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

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 combined section, as illustrated in Figure C6A.6.12.6.9-2. The combined spliced section is treated as a beam and the factored resistance at the strength limit state is determined assuming the stress in the combined section at the limit of usable resistance is equal to the specified minimum yield strength of the gusset plate if the slenderness limit for the spliced section given by Eq. 6A.6.12.6.9-2 is met, which will typically be the case. 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.

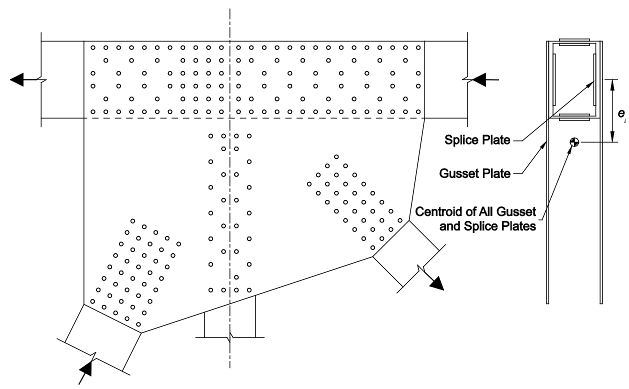

Figure C6A.6.12.6.9-2—Illustration of the Combined Spliced Section at a Chord Splice

The Whitmore section check specified in Article 6A.6.12.6.7 is not considered applicable for the load rating of a compression chord splice.

The yielding and net section fracture checks on the Whitmore section specified in Article 6A.6.12.6.8 are not considered applicable for the load rating of a tension chord splice.

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

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

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.)

Tension chord splice members shall also be checked for block shear rupture as specified in Article 6A.6.12.6.8.

6A.6.12.6.10—Edge Slenderness

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

C6A.6.12.6.10

NCHRP Project 12-84 (Ocel, 2013) found no direct correlation between the buckling resistance of the gusset plate and the free edge slenderness. In addition, merely adding stiffeners to just the free edges will not provide any appreciable increase in the compressive resistance of the plate. However, properly stiffening the free edges, as discussed below, could suppress plate buckling.

Since gusset plate buckling was always observed to occur in a sway mode, either a diaphragm must be added between the two gussets, preferably also connected to the chord, to stiffen against sway; or else stiffening elements must be placed along the free edges such that their full out-of-plane yield moment resistance can be developed at the planes that would bend if sway occurs. These requirements do not apply if the free edges are merely being stiffened without relying on an increase in buckling resistance. In this case, there are no criteria specified for sizing of the edge stiffeners, but the traditional practice of using angles with leg thicknesses of 0.50 in. has generally proven adequate to reduce deformations of the free edges during fabrication, erection, and service.

The effect of proper edge stiffening on the compressive resistance of the gusset plate was examined experimentally and analytically in NCHRP Project 12-84 (Ocel, 2013). The increase in compressive resistance was highly dependent upon the configuration of the connection and was found to vary from 6 percent to 45 percent. 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. That is, properly stiffened free edges tend to suppress buckling as predicted by the Whitmore section analysis specified in Article 6A.6.12.6.7 in gusset plates with large L_{mid} distances. However, proper edge stiffening will likely not suppress the buckling resulting from partial plane shear yielding in cases with small L_{mid} distances. Therefore, in such cases, the resistance calculated according to the provisions of Article 6A.6.12.6.6 would be considered to represent the upper bound of compressive buckling resistance with properly stiffened free edges, unless a refined simulation analysis indicates otherwise. 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 properly stiffened free edges.

6A.6.12.6.11—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 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.

If a load rating conducted in accordance with Articles 6A.6.12.6.6 through 6A.6.12.6.9 indicates an unacceptable load rating and the limiting capacity is based on compression (e.g., Partial Shear, Whitmore) or a deteriorated condition, then a more refined analysis may be performed. Any more rigorous analysis must be consistent with a rational application of established engineering principles.

C6A.6.12.6.11

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 reduction factor specified in Article 6A.6.12.6.1 is also to be considered.

The necessary fidelity of the model is dependent on 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 model predictions within 9 percent of experimental values for a three-dimensional, two-panel truss system isolated out of an entire bridge where the connection of interest was located in the center between two panels (Ocel, 2013). Model symmetry was not used because the sway buckling mode would not be captured. The following list, which is not considered exhaustive, summarizes other important attributes of this 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 percent of the gusset plate thickness; and

- 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.

Because the basic compression checks comprise empirical fit of a wide range of conditions, significant improvements in accuracy can be provided by explicitly considering the flow of forces through the plate and the capacities of the sections resisting those forces. Refined modeling approaches based on a first principles analytical approach utilizing fundamental steel design theory may be used. Examples of other approaches are illustrated in Figures C6A.6.12.6.11-1 and C6A.6.12.6.11-2. The 0.90 reduction factor required for the application toward the results of a finite element analysis should be applied to the basic corner check of Figure C6A.6.12.6.11-1, but not to the truncated Whitmore section of Figure C6A.6.12.6.11-2 as it was shown to be comparable in accuracy to the full Whitmore section for which there is no reduction.

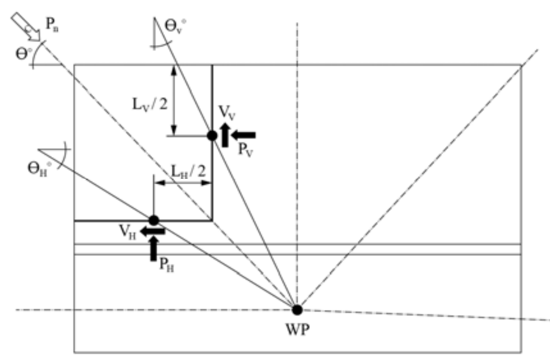

Figure C6A.6.12.6.11-1—Basic Corner Check

In this approach the following assumptions and constraints are made:

- Failure surfaces represent minimum section that includes all member fasteners.
- Forces act at centroid of respective section surfaces.
- Surfaces can carry no moment.
- Combination and normal and shear forces limited by von Mises stress criterion.
- Resultant of each section forces pass through nodal work point.
- Resultant of all section forces must align with member.

Subject to the limitations of other checks, this approach provides a more accurate estimate of capacity when compared to the partial shear check. Since this method is generally conservative, it can be further refined by removing certain constraints. For example, it is not essential for the resultants of the section forces to pass through the work point, nor is it necessary for the failure sections to carry no moment. Provided that there is adequate capacity in other areas of the gusset plate, these constraints can be eliminated. If they are eliminated, the other sections of the plate must be evaluated for the corresponding demands. All other checks, i.e., horizontal shear, block shear, etc. still apply. Refer to the WJE reference for examples demonstrating this approach.

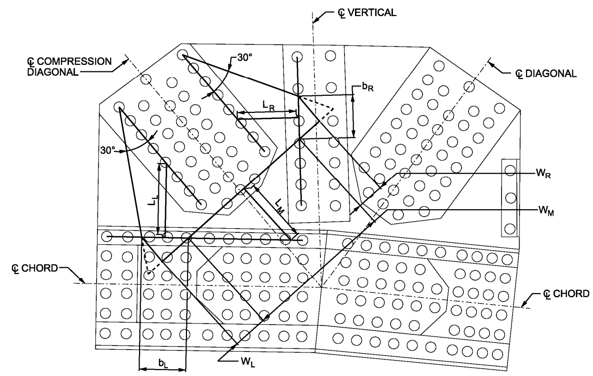

Figure C6A.12.6.11-2—Truncated Whitmore Section

- The Truncated Whitmore Section Method was developed at the Georgia Institute of Technology by Dr. Donald White, et al., as a part of the NCHRP 12-84 Project. Illustrative examples of its application are found in Appendix I, Section 5 of the Final Report, and designated Method 2.
- In utilizing the Truncated Whitmore Section Method, the equations found in Article 6A.6.12.6.7 are applicable, except that:

  a) L_{mid} is replaced by the lengths L_M, L_L, and L_R in calculating the nominal resistance of the widths W_M, W_L, and W_R.

  b) The constant value 3.29 in Eq. 6A.6.12.6.7-4 is to be replaced with the value 6.71 in Warren trusses with a vertical framing into the joint in addition to the diagonal members and 4.25 for all other gusset and joint geometries for calculation of the nominal resistance of the width, W_M, and by 6.71 for calculation of the nominal resistance of the widths W_L and W_R. These coefficients are due to calibration differences between Method 1 (Partial Shear Plane Method) and Method 2.

  c) When computing the nominal compression resistance, P_n, the tributary portions of the gusset gross cross-sectional area within the base dimensional widths W_L and W_R are to be reduced 10 percent.

d) The total gusset plate compressive resistance is taken as the sum of the contributions from W_M, W_L, and W_R.

- In cases where the Whitmore section is not truncated by the adjacent fastener lines, as shown in Figure C6A.6.12.6.11-2, L_L and L_R are zero and the Truncated Whitmore Section Method reduces to the traditional Whitmore section method with W_M equal to the Whitmore Section width defined in Article 6A6.12.6.7 and $L_M = L_{mid}$.
- When using the Truncated Whitmore Section Method (Method 2), no Partial Plane Shear strength check is required.

6A.7—WOOD STRUCTURES

6A.7.1—Scope

The provisions of Article 6A.7 apply to the evaluation of wood bridges constructed of sawn lumber or glued laminated timber.

6A.7.2—Materials

The reference design values for existing timber bridge components in satisfactory condition may be taken as given in LRFD Design Articles 8.4.1.1.4 and 8.4.1.2.3 and adjusted for actual conditions of use in accordance with LRFD Design Article 8.4.4. To obtain values for species and grades not included in the LRFD articles, a direct conversion of Allowable Stress Design Values in the *National Design Specification for Wood Construction*, 2005 Edition may be performed.

C6A.7.2

The material and member properties based on as-built information may need to be adjusted for field conditions such as weathering or decay. The Engineer's judgment and experience are required in assessing actual member resistance.

Southern Pine and Douglas Fir are the more common types of timber used in bridge construction. Plans and other relevant contract documents should be reviewed to determine the species and grade of wood. When the type of timber is unknown, field identification and grading may be done based on visual appearance, grade marks, local experience, and grade description requirements. Sampling for testing may be done where more exact information is required.

6A.7.3—Resistance Factors

Resistance factors (φ) for the strength limit state shall be taken as given in LRFD Design Article 8.5.2.2.

C6A.7.3

Some older timber bridges may not have the roadway deck continuously attached to the beams. The resistance of beams not continuously braced in the lateral direction should be reduced in accordance with LRFD provisions (LRFD Design Article 8.6.2).

6A.7.4—Limit States

The applicable limit states for the evaluation of wood bridges shall be taken as specified in Table 6A.4.2.2-1 and in these Articles.

C6A.7.4

Deflection control on timber components as specified in LRFD Design Article 2.5.2.6.2 may be applied to evaluation if the bridge superstructure was observed to exhibit excessive flexing under normal traffic. This is an optional requirement.

6A.7.4.1—Design-Load Rating

Rating factors for the design-load rating shall be based on the Strength I load combination.