Evaluation and Repair of Bridge Truss Gusset Plates

HOWARD HILL, JONATHAN C. McGORMLEY, JONATHAN LEWIS, WADE CLARKE
and THOMAS NAGLE

Abstract

Gusset plates used to connect members in large steel trusses are important elements in many existing bridge structures. As such, their capacities can influence bridge structural load ratings, especially when the effects of deterioration and/or damage have become significant. In order to provide accurate load ratings, avoid unnecessary repairs and, when necessary, design appropriate repairs, gusset plate conditions and characteristics must be properly incorporated in the responsible engineer's evaluation. Because the cost of being conservative is far greater for existing structures than for new designs, engineers evaluating existing gusset plates should not rely too heavily on design-based methods when making final load rating and repair decisions. The purpose of this paper is to provide some practical guidance to the process of gusset plate evaluation and repair in order to promote efficient use of limited bridge maintenance resources.

Keywords: gusset plates, shear, compression, deterioration, and repairs.

In February 2009, the Federal Highway Administration (FHWA) published the document *Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges*, Publication No. FHWA-IF-09-014 (Ibrahim, 2009), hereinafter referred to as the *Guide*. Its purpose is to provide engineering guidance in the structural evaluation of existing steel gusset plates. The *Guide* presents a straightforward methodology for evaluating the strengths of gusset plates and gusset plate fasteners that is based on current American Association of State Highway and Transportation Officials (AASHTO) design and load rating methods. Because they are relatively simple and based on provisions that are familiar to practicing engineers, the *Guide* provisions are easy to implement, and gusset plates that satisfy them are expected to provide reliable service.

Like most design-based provisions that must be relatively simple and yet applicable to a wide variety of cases, the *Guide* methods can be quite conservative under certain circumstances. Therefore, gusset plate elements that do not satisfy the basic *Guide* evaluation provisions should not be considered inadequate on this basis alone. The author of the *Guide* clearly recognized this fact, as demonstrated by the occasional reference to performing "more rigorous analysis" (e.g., in Sections 3.2 and 3.3). Given the cost of implementing modifications to in-service structures, it is usually worthwhile to take a more rigorous look at structural elements that do not satisfy basic design provisions. As an example, consider the situation in which a conservative design procedure indicates a 1-in.-thick gusset plate is needed for a new truss joint (i.e., one that is still on paper), while a more rigorous approach would have shown a ¾-in.-thick plate to be sufficient. The cost to the project is limited to an extra ¼ in. thickness of plate material, which would likely be less than the cost of pursuing the more rigorous design approach. In contrast, if a ¾-in.-thick gusset plate on an in-service bridge is deemed inadequate using a conservative evaluation approach, when a more rigorous approach would have shown it to be adequate, the cost to the project is far more than the price of a little extra plate material.

To put the cost of conservatism in perspective, consider a load rating process for a bridge where 70% of the service load is dead load and 30% of the service load is live load. In this case, underestimating actual gusset plate strength by only 10% can lower the live load rating by 33%. If the conservative approach suggests a 15% deficiency (while a more rigorous analysis approach would indicate that the bridge is actually sufficient), the cost of the conservatism would include unnecessary restrictions on bridge use—and unnecessary repairs to remove those restrictions.

Where significant deterioration has occurred, design-based evaluation procedures are typically unable to accurately estimate member capacities. This is due to the fact that such procedures have no way of reasonably quantifying the effects of localized or generally nonuniform degradation or damage. As a result, attempts to evaluate deteriorated or

Howard Hill, Ph.D., P.E., S.E., Director of Project Operations and Principal, Wiss, Janney, Elstner Associates Inc., Northbrook, IL. E-mail: hhill@wje.com

Jonathan C. McGormley, P.E., S.E., Principal, Wiss, Janney, Elstner Associates Inc., Northbrook, IL (corresponding). E-mail: jmcgormley@wje.com

Jonathan Lewis, S.E., Associate Principal, Wiss, Janney, Elstner Associates Inc., Northbrook, IL. E-mail: jlewis@wje.com

Wade Clarke, P.E., S.E., Senior Associate, Wiss, Janney, Elstner Associates Inc., Northbrook, IL. E-mail: wclarke@wje.com

Thomas Nagle, P.E., S.E., Operations Director, Nagle Signs Inc., Waterloo, IA. E-mail: tjnagle@naglesigns.com

damaged elements using standard design procedures often lead to extremely conservative conclusions. Again, the cost of conservatism when dealing with in-service structures can be very high.

When a deteriorated plate requires repair or reinforcing, it is important to give proper credit to its existing capacity. If this is not done, repair/retrofit efforts can become excessive and unnecessarily costly. In many cases, localized deterioration can be effectively addressed with localized repairs. It is also important to note that, unless the bridge in question has been shored or otherwise externally supported for some time, even its most heavily deteriorated plates have proven capable of carrying full dead load demands and heavy trucks (albeit with unacceptable factors of safety, thus necessitating repair).

The purpose of this paper is to provide some practical guidance in the application of additional evaluation rigor that can help increase the efficiency and effectiveness of our limited bridge maintenance resources. Rational methods to account for and properly address common forms of gusset plate deterioration are also explored.

CONSERVATISM IN FHWA GUIDE STRENGTH PROVISIONS

Global Shear Strength

The *Guide* procedures for evaluating gusset plate shear yield strength are especially prone to excessive conservatism. This is due to the fact that the shear yield strength equation [*Guide* Equation 6: $V_r = \phi_{vy} V_n = \phi_{vy}(0.58) F_y A_g \Omega$] includes a factor ($\Omega$) that is either 0.74 or 1.0 per the *Guide*, while little guidance is provided for making the appropriate selection in any particular case. In our opinion, given this lack of guidance, engineers will tend to take the conservative approach and use 0.74, even though most situations would warrant a much higher factor.

The current AASHTO (2007) design specifications for truss gusset plates (Section 6.14.2.8) include a strength reduction factor of 0.74 for situations involving *flexural shear*. Unfortunately, the term *flexural shear* is not defined. Because the presence of normal stress (e.g., stress created by flexure) reduces shear strength, it may well be prudent to reduce shear strength where significant normal stress exists. Many potentially critical sections in common gusset plates must carry both shear and normal stresses. However, rare are the cases where normal stresses would be high enough to warrant a 0.74 reduction factor on shear strength. In most practical situations, applying the 0.74 reduction factor will lead to substantial underestimation of actual shear strength. According to Drucker (1956), the interaction of shear and moment acting on a rectangular cross-section can be represented as follows:

$$\frac{M}{M_0} = 1 - \left(\frac{V}{V_0}\right)^4$$

where

M_0 = plastic moment strength

V_0 = plastic shear strength

Rearranging this equation to solve for V gives the following:

$$V = V_0 \left[1 - \left(\frac{M}{M_0}\right)\right]^{0.25}$$

where

$\left[1 - \left(M/M_0\right)\right]^{0.25}$ = plastic strength reduction factor

For a rectangular plate with M equal to the moment at first yield, according to Drucker (1956), the reduction factor becomes 0.76. Therefore, the AASHTO and *Guide* 0.74 value appears to be a reasonable reduction factor for a steel plate cross-section that is also carrying normal stresses resulting from yield-level moments. Also apparent is that lesser normal stresses will have less effect on shear strength. While the universal application of a lower bound strength reduction factor is suitable for new designs—where the cost of the associated conservatism is small—it is not appropriate in situations where the cost of being conservative can be very high. When evaluating existing, in-service gusset plates, it is appropriate to use a moment/shear interaction relationship to establish shear strength estimates based on actual conditions, rather than a single reduction value for all cases.

Consider the gusset plate shown in Figure 1. Section A-A comprises a potentially critical shear plane that represents the only load path for horizontal forces in the web members to be resolved in the chord. Section A-A also carries substantial normal stresses resulting from the vertical components of the web member forces. In order for the gusset plate to be in equilibrium and for the connected members to carry only axial loads as likely assumed, the free-body diagram shown in Figure 2 must be satisfied. As shown, the moment acting on Section A-A must equal the horizontal force in the chord multiplied by the distance, e. This moment can be used in conjunction with the interaction equation shown previously to establish a corresponding shear strength.

Figure 3 provides a schematic representation of the node U10 gusset plates whose failure led to the 2007 collapse of the I-35W Bridge in Minneapolis, Minnesota. At the time

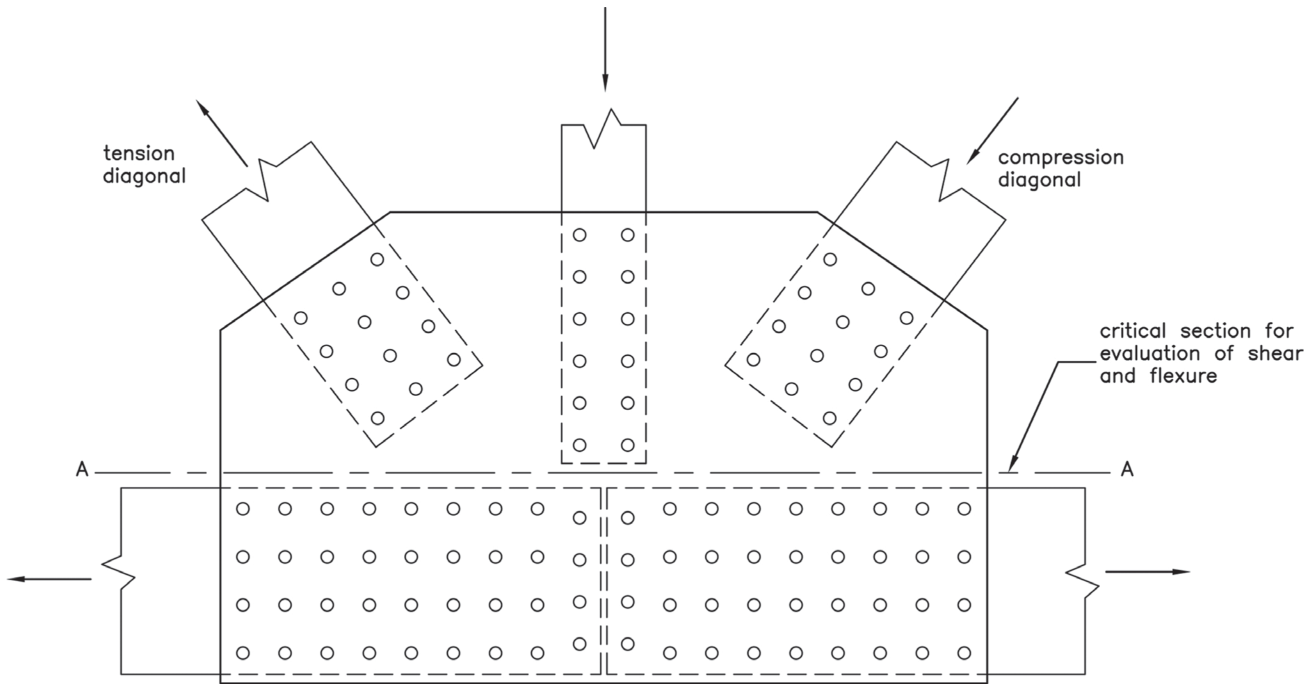

Fig 1. Typical gusset plate high-stress section.

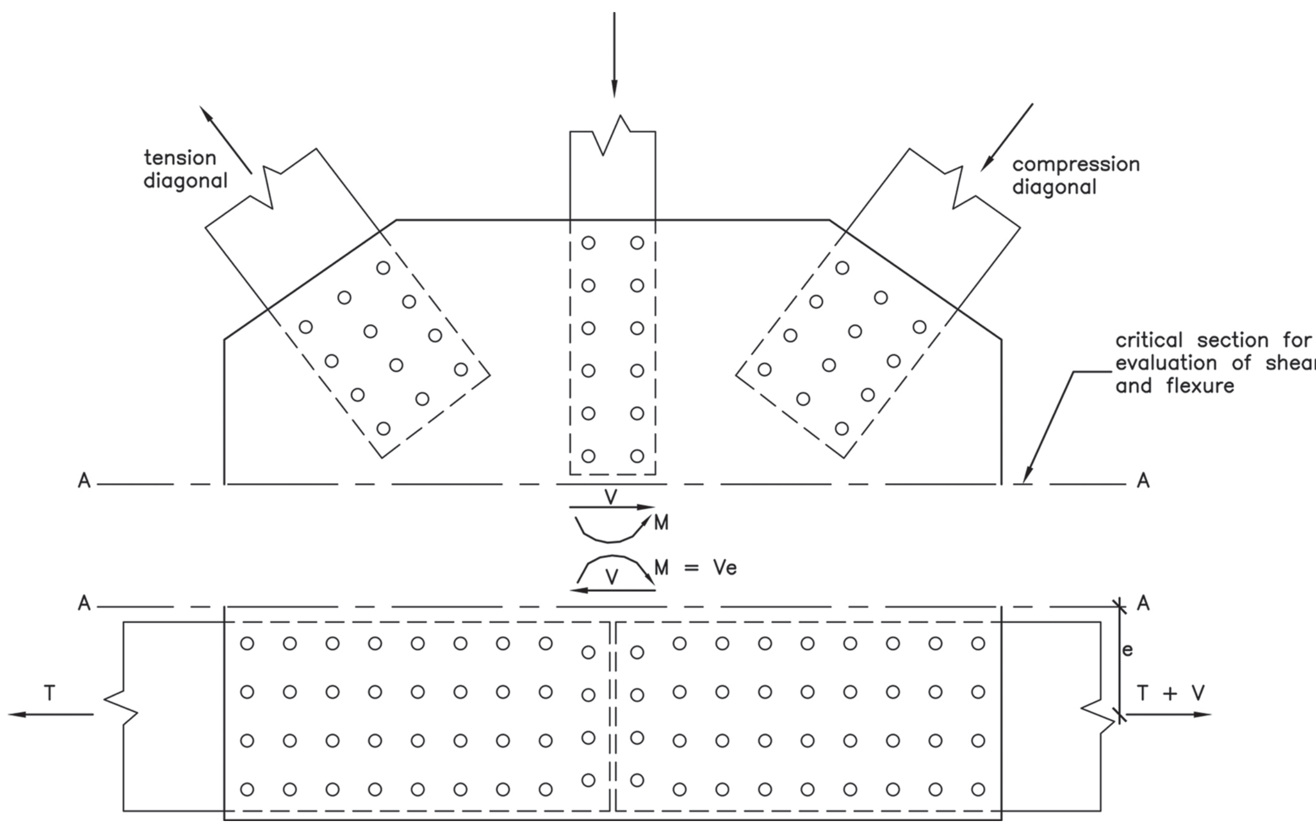

Fig 2. Horizontal section free-body diagram.

of their failure, the most critical of the U10 gusset plates were carrying about 92% of their respective plastic shear strengths (about 0.92 × 0.6 × tensile yield strength × area) along Section A-A. Using the moment acting on Section A-A at the time of collapse, Drucker's (1956) interaction equation predicts a shear strength reduction factor of 0.89. While this appears slightly conservative based upon the evidence, it provides a far better estimate than the AASHTO factor of 0.74, which would underestimate shear strength by an additional 17%. As indicated earlier, the cost of such a discrepancy can be substantial.

The *Guide* text accompanying Equation 6, which is used to calculate the factored shear yield resistance, mentions "stiffness to prevent buckling and develop the plastic shear force of the plates." There should be little concern of shear buckling affecting gusset plate strength in most practical situations. If we apply standard AASHTO plate girder shear buckling provisions to a range of gusset plate dimensions, where the gusset plate width is analogous to plate girder height, we would find that the transverse stiffener spacing needed to develop full plastic shear strength would typically be measured in feet. Because the horizontal sections of high shear in most gusset plates are bounded by stiffening elements that are separated by inches rather than feet, shear buckling is rarely a controlling limit state.

Gusset Plate Compression

The *Guide* provisions for evaluating gusset plate compression suggest idealizing the plate as a column with a width equal to the Whitmore section and determining the capacity by selecting an appropriate effective length. The process outlined in the *Guide* is similar to that which has been proposed by Dowswell (2006). However, the *Guide*'s application of the process, most notably in the selection of an effective length factor (K), appears much more conservative. For example, Dowswell recommends use of full yield strength in situations where the *Guide* sample problems use a K value of 1.2. In fact, K values greater than 1.0 are not recommended in any of the situations covered by Dowswell.

The treatment of gusset plate compression in the *Guide* sample problem is quite conservative. When a gusset plate joint is detailed so that the edges of the compression diagonal fall within a few inches of the edges of the adjoining chord and web elements, sidesway buckling of the type envisioned in the *Guide* samples is effectively restrained. To put it another way, for sidesway buckling to occur in such a situation, essentially all of the plate material surrounding the end of the compression member would have to reach yield-level principal stresses. Even where buckling may preclude development of full yield strength, the *Guide* procedures may be overly conservative.

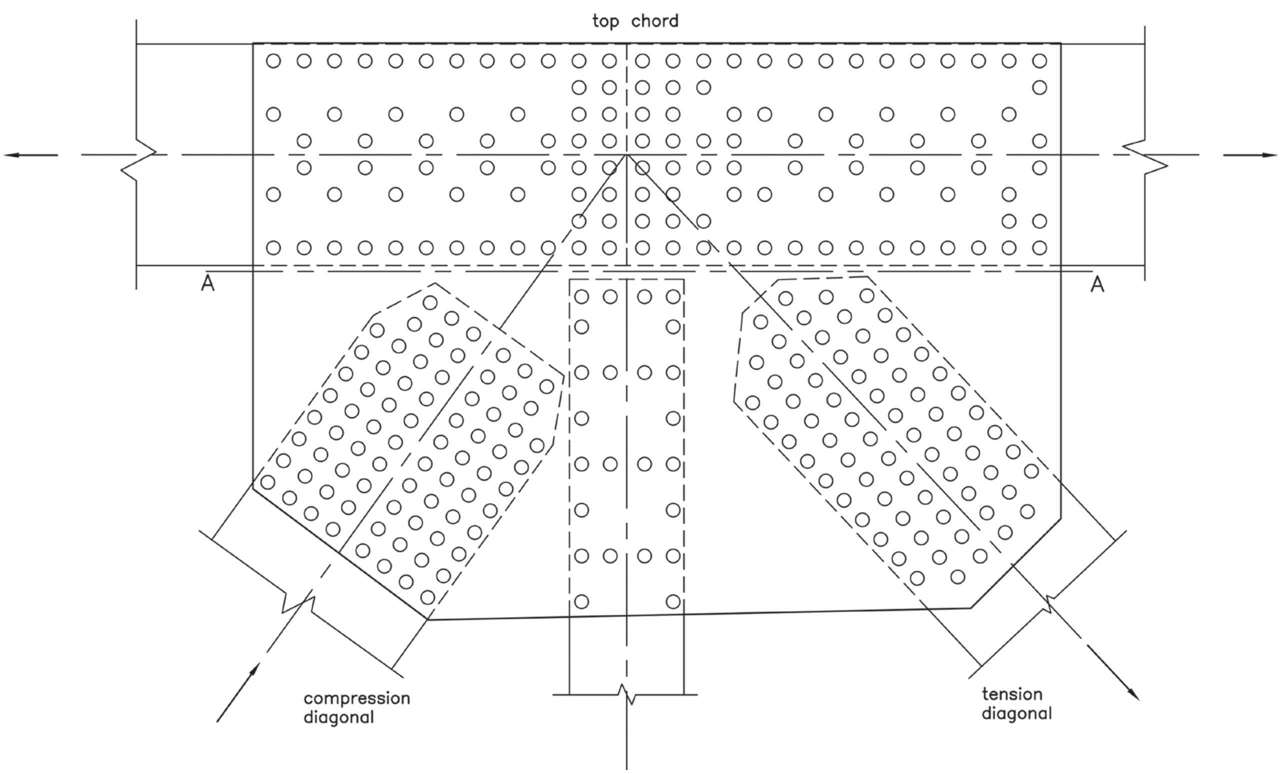

Fig. 3. I-35W U10 gusset plate detail.

In most common situations, the approach used to assess compression capacity in the *Guide* examples may not have a significant impact. This is due to the fact that even when using very conservative K factors, effective lengths often remain so small that the calculated critical compressive stress is not much less than the corresponding yield strength of the plate material. However, the *Guide* procedure, and most forms of equivalent column procedures can be very conservative under certain circumstances. For example, consider the situation shown in Figure 4 where the sloped bottom chord segment and vertical member form a relatively small angle in which the compression diagonal fits. In this case, the compression diagonal must terminate a considerable distance from the work point, even though the edges of this member nearly contact the edges of the neighboring members. As a result, the length of the compression region measured along the compression member centerline is quite long, leading to a significant average-length value as defined by the *Guide*. When used in conjunction with the *Guide* compression zone evaluation procedure, this average length leads to a significant reduction in strength. However, because this compression zone is stiffened along both sides by the adjoining web and chord elements, the equivalent column approach does not represent actual behavior. For the case shown in Figure 4, the compression zone acts more like the compression flange of a box beam (i.e., it does not act like a column). If we look at the compression flange of a box beam in a region of constant moment (i.e., where it is under the equivalent of a constant axial load), its buckling strength is unrelated to its length. This is because column-type buckling is restrained by the webs that are located along each of its edges. In this case, local buckling is the only buckling of concern, and local buckling depends on the width of the element, its thickness and the degree of restraint along each edge.

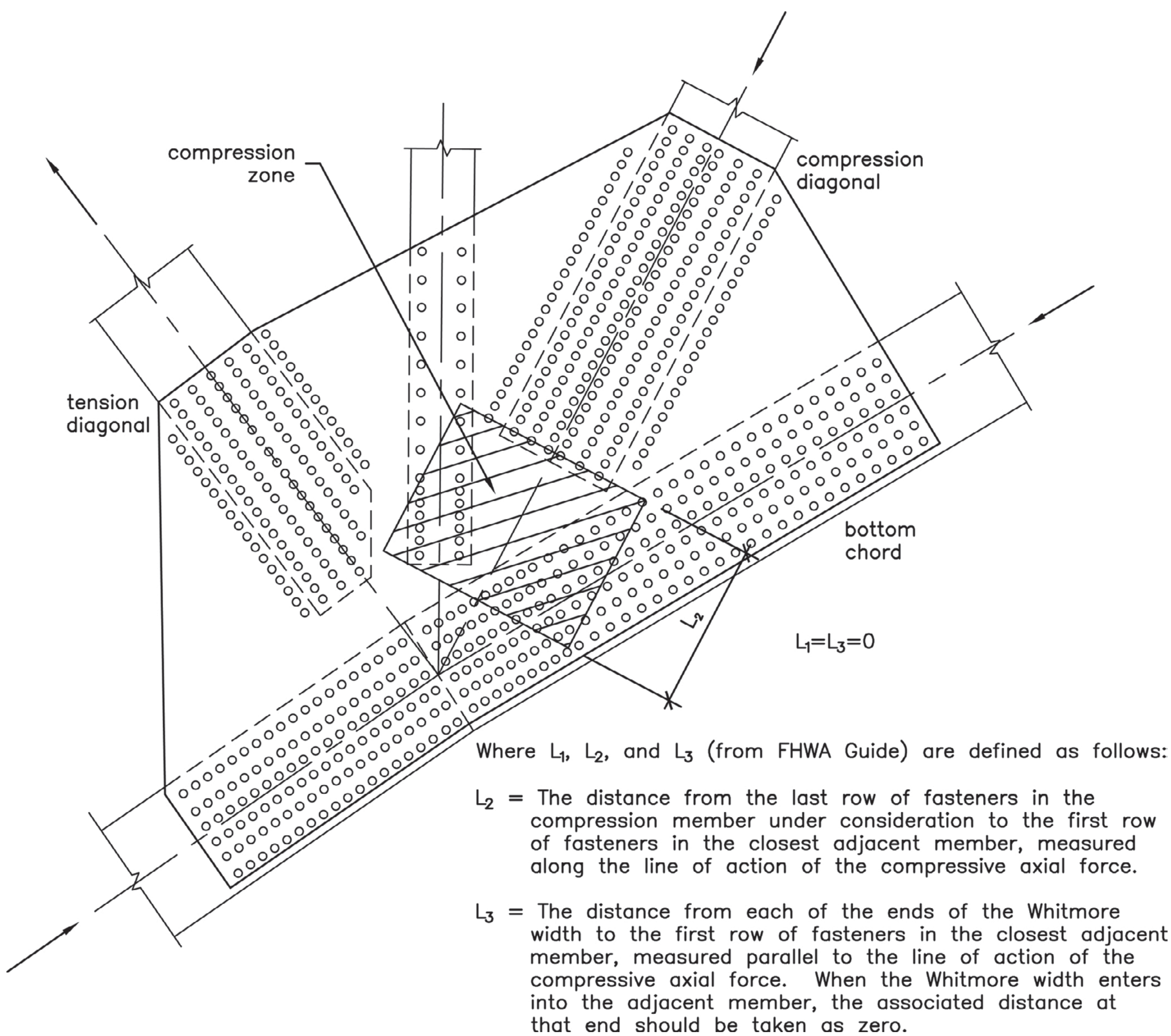

Fig. 4. Long "compression zone" example.

Figure 5 shows the compression zone from Figure 4 with some equivalent plate boundaries and associated dimensions. These dimensions and the associated edge conditions can be used with standard plate buckling references (Galambos, 1998; Salmon and Johnson, 1996) to determine critical buckling stress. In cases such as that shown in Figure 4, the plate buckling approach can be used to mitigate the conservatism associated with the equivalent column approach.

EVALUATING GUSSET PLATE DETERIORATION

Properly accounting for section loss or other forms of damage is of prime importance in gusset plate evaluation. Because the effects of deterioration (e.g., corrosion) are highly variable and often highly localized, a general, formulaic evaluation approach will rarely be able to accurately capture their effect on strength. The evaluator must resist the temptation to make broad, overly conservative assumptions in an attempt to keep the evaluation simple. For example, if a portion of a gusset plate has sustained up to 50% section loss in one area, applying the basic *Guide* formulas using half of the original plate thickness would likely lead to erroneous conclusions and unnecessary repairs. Instead, analytical methods that account for the degree of deterioration and its location should be employed.

Documenting Existing Conditions

A proper evaluation requires good documentation of the gusset plate condition. Simply knowing the maximum depth of pitting or the total area of perforation is not sufficient. A reasonable estimate of plate strength needs to be based on reasonable estimates of both the magnitude and location of significant section loss. The gusset plate shown in Figure 6 has sustained significant section loss at and around the perforation. At a minimum, documentation of such deterioration should include an outline of the hole perimeter and an outline of the surrounding, full thickness perimeter (i.e., the line around the hole where measureable section loss begins).

If the area of section loss around a hole is large or if the degree of section loss does not vary in a consistent fashion between the edge of the hole and the full thickness perimeter, then intermediate thickness measurements may be necessary to describe the loss.

Figure 6 also shows an area of linear pitting. In this context, linear pitting is section loss that occurs along a generally straight path that is much narrower than it is long. Linear pitting is one of the most common forms of significant gusset plate deterioration. It often occurs where debris accumulates along the intersections of gusset plates with horizontal and low sloping surfaces of chord and web members and is the result of moisture retention on abutting steel surfaces. The fact that areas of debris accumulation often coincide with areas of high gusset plate stress (e.g., in the critical block shear perimeter of a web member or in the critical web/chord shear transfer zone) can make linear pitting especially significant.

When documenting linear pitting, it is important to measure the maximum depth of pitting at regular and closely

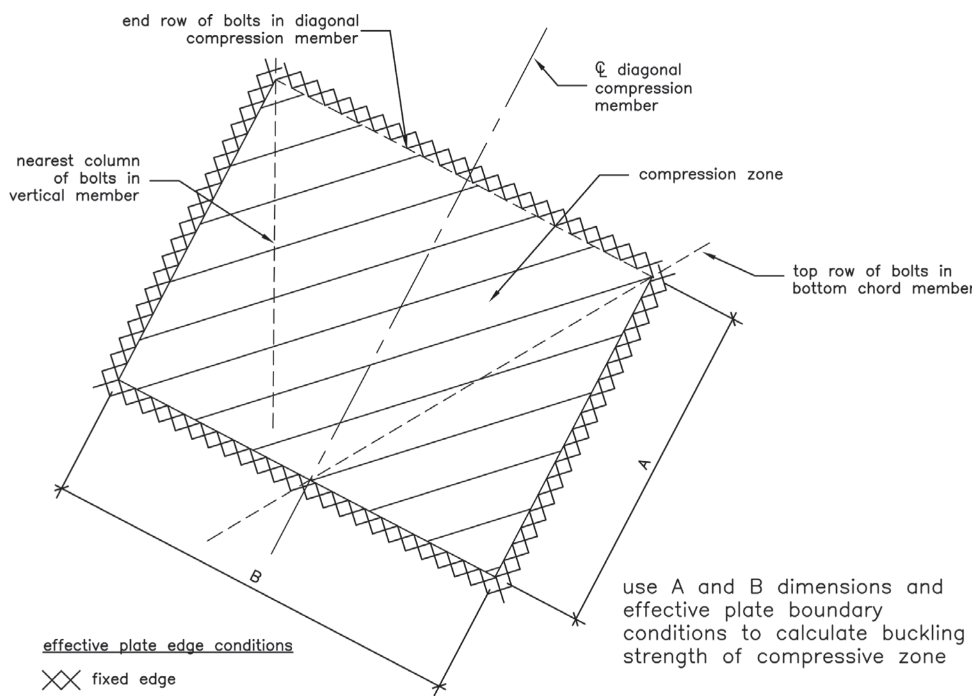

Fig. 5. Equivalent rectangular plate.

spaced intervals (say, every few inches) along the pitted zone and record the total width of measureable pitting at each location. This will enable the engineer to make reasonable estimates of section loss when evaluating potential failure surfaces that include all or just part of a line of pitting. If the width of pitting is generally less than the diameter of a common structural bolt (say, 1½ inches or less), a single measurement approximating the maximum depth usually provides sufficient information at each location along the length of the pitted area. As the width of a zone of linear pitting increases, it becomes increasingly important to document the extent of section loss transverse to the long dimension (e.g., take several measurements in the short direction at each location selected along the long dimension).

Evaluating Gusset Plates with Linear Pitting

As is the case with any type of deterioration, linear pitting can occur in an infinite variety of forms, defined by different combinations of severity, extent and orientation. This paper addresses the most common forms of linear pitting, although the principles involved would be applicable to other forms as well.

Bottom Chord Narrow (BCN) Linear Pitting

One of the most common forms of significant gusset plate deterioration on bridges is a narrow band of linear pitting located parallel to and slightly above the top of the bottom chord, an example of which is shown in Figure 6. Pitting of this type can reduce the ability of a plate to transfer forces between the chord and the web members.

To be considered narrow, a band of linear pitting should be no wider than the diameter of the largest common structural bolts (say, 1½ inches). In this case, as load builds up in the plate, yielding would likely begin in the thinnest sections of the pitted zone. This initial yielding would occur at a load significantly less than the load required to cause initial yielding in the original, unpitted plate. However, this does not mean the plate shear strength is proportionally reduced. First, local areas of deep pitting (i.e., where the plate is very thin) can experience yield-level stresses under small shear loads and then strain further as needed to allow mobilization of the strength of the rest of the plate. In this case, the ductility of the steel makes it unnecessary to limit the strength to the load that causes first yield, and the actual yield strength through the pitted zone is more accurately represented by the average thickness rather than the minimum thickness.

Strain hardening is another factor that should be taken into consideration when evaluating the effect of highly localized section loss. In the Figure 6 example, the shear yield strength through the pitted zone is clearly less than the shear yield strength through a horizontal plane immediately above or below the pitted zone. Therefore, as shear loading increases, the steel in the pitted zone will yield first. However, as shear strains in the pitted zone increase, this steel will strain harden. If the available rupture strength of the pitted zone exceeds the available yield strength of the

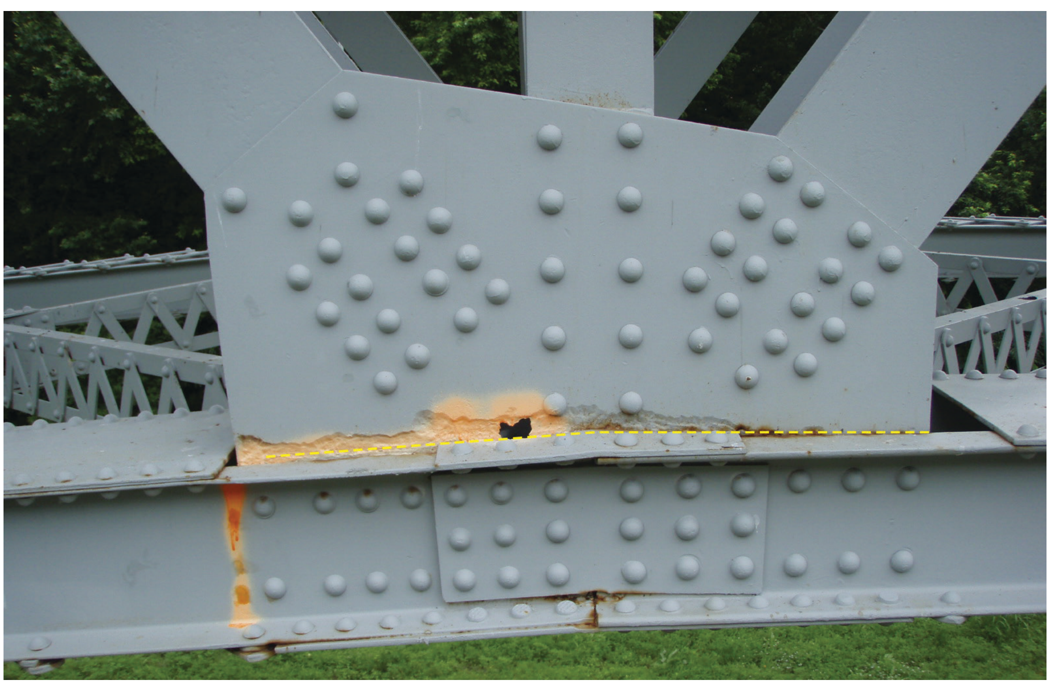

Fig. 6. Narrow band of section loss in gusset plate just above chord member.

surrounding unpitted areas, the yielding of the unpitted plate will be mobilized, just as it would in an nondeteriorated plate. In such cases, the localized pitting would not reduce the available strength of the gusset plate.

The mechanism just described is what justifies checking shear or tensile rupture on net sections through rows of bolt holes, while checking shear or tensile yield on gross sections. The strain needed to mobilize strain hardening over the small dimension affected by the bolt holes is achievable without excessive deformation. Because the same can be said for a narrow band of pitting, the same mechanism would develop. In most gusset plates that have pitting of the type shown in Figure 6, there is a horizontal row of chord fasteners (i.e., bolts or rivets) located parallel to and a short distance below the pitted zone. The section loss through this fastener row due to hole drilling is considerable, yet it does not result in a proportional reduction in the available shear strength of the plate and may not affect available strength at all since the shear or tensile yield strength of the gross section may be less than the corresponding rupture strength of the net section. To put it another way, if the section loss due to pitting is less severe than the section loss through an adjacent horizontal plane caused by hole drilling, the pitting will not reduce the shear strength of the plate any more than the holes themselves. Therefore, when evaluating the effects of BCN pitting, one of the first steps should be the calculation of the net area in the pitted zone. If the available rupture strength of the pitted net area is greater than the available yield strength of the gross area of the undeteriorated plate, or if the pitted net area is greater than the net area through a nearby row of fasteners, the pitting will not significantly reduce the shear strength of the plate.

Conversely, if the available rupture strength of the net area of a BCN pitting zone is less than the available yield strength of the adjacent nondeteriorated areas, and if it is also less than the available rupture strength of the net sections at nearby parallel rows of fasteners, the BCN pitting can reduce the strength of the plate. Of primary concern in this regard are (1) the transfer of forces between the web members and the chord at the node and (2) the transfer of forces between a single web member and the rest of the members at the node. Methods for evaluating the effects of critical BCN pitting are discussed later.

The horizontal line shown in Figure 6 goes through a BCN zone of pitting that sustains high horizontal shear stresses and high vertical normal stresses. Therefore, it represents a potential critical gusset plate failure plane. A method for determining the capacity of this area in a nondeteriorated plate was provided earlier. The same method can be used to evaluate a plate with BCN linear pitting. Further, if the net area of the pitted zone is greater than the net area through the adjacent row of fasteners, the pitting can effectively be ignored. Otherwise, the pitted net area should be treated like any other net area. The shear/moment interaction approach outlined earlier—substituting ultimate strength for yield strength of the material—is one possible technique.

In a general sense, forces are transferred between the end of a web member and the rest of the node via the gusset plate material that overlaps and surrounds the end of the member. An example of a web member and the associated areas of gusset plate force transfer are shown in Figure 7. Also shown in Figure 7 are a zone of BCN linear pitting and two gusset plate corner sections. The smaller, yellow corner section represents the smallest corner of the gusset plate that must be capable of transferring the total web member load to/from the node, which means it is the most critical corner section for this member in a nondeteriorated gusset plate. The capacity of the corner could be evaluated by calculating the forces that could be sustained on each leg. However, in nondeteriorated plates, other calculations such as the Whitmore check provide reasonable representations of the plate's ability to transfer forces between a single member and the rest of the node.

In plates exhibiting significant BCN linear pitting, Whitmore and other design-based checks might not capture or otherwise appropriately represent the effects of the critical failure mechanism. For this reason, design-based evaluation methods need to be supplemented with something more rigorous. In this case, corner checks that include the BCN linear pitting zone can be used to determine whether the pitting has created a more critical failure mechanism. The larger red corner section in Figure 7 was drawn so that one side includes the BCN linear pitting zone. A relatively simple, and conservative, corner check of this situation is illustrated in Figure 8. In this check, the vertical leg of the corner is located to create the minimum perimeter that encompasses all fasteners. The sides of the corner are assumed to carry only normal (green arrow) and shear (yellow arrow) forces, which act at the center of each plane. In addition, the overall resultant of the forces on the horizontal and vertical planes is constrained as follows:

1. It must act through the truss node work point (point A).
2. It must be aligned with the centerline of the web member.
3. The resultant moment in the web member must be zero.

Because the various members framing into the node and the member in question typically can sustain some degree of flexure, these constraints are conservative. The maximum combination of normal and shear force acting on the pitted leg of the corner is typically established first. This is done by:

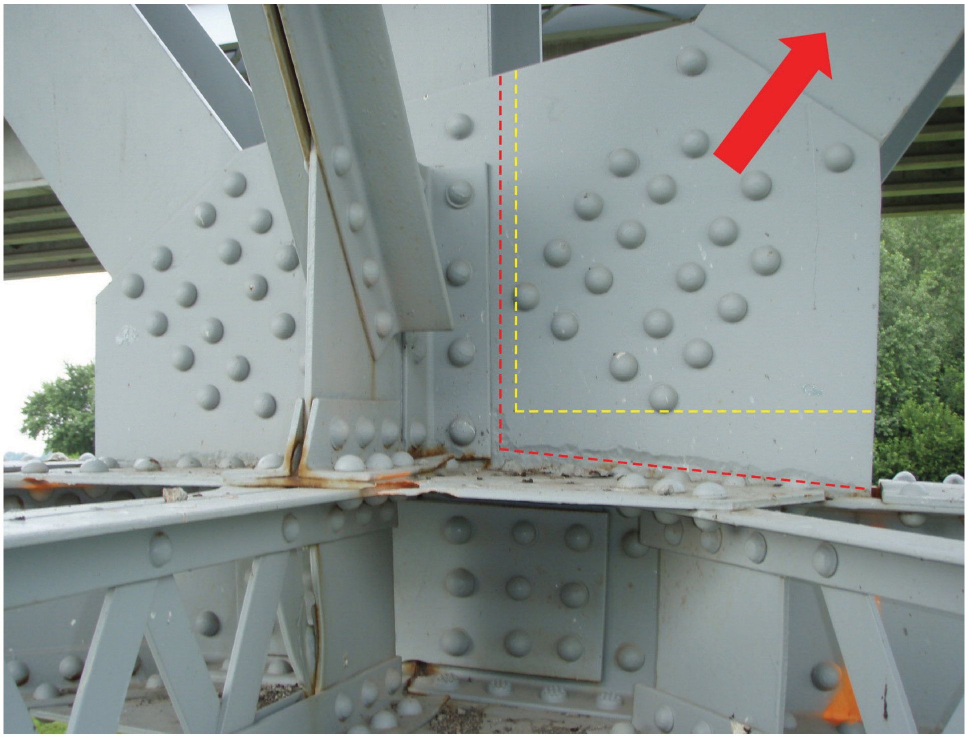

Fig. 7. Gusset plate corner resisting tension generated by diagonal web member.

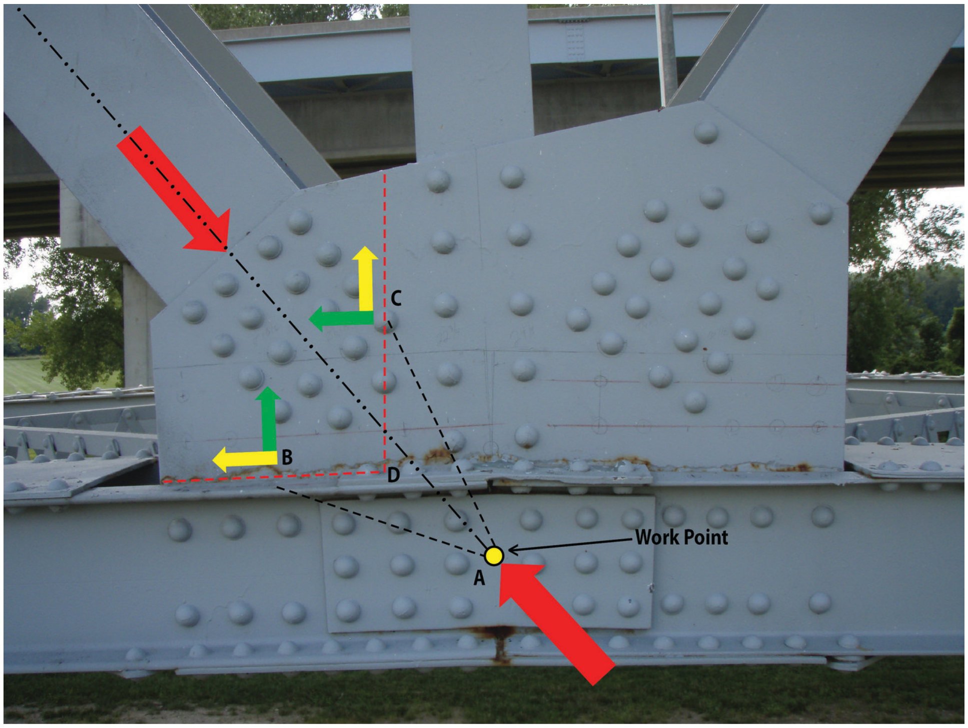

Fig. 8. Equilibrium check for a representative corner section of a gusset plate.

1. Assuming the resultant of the two forces follows Line AB, which connects the truss node work point with a point located in the middle of the horizontal leg. This assumption relates the magnitude of the shear force to the normal force or vice versa.

2. Finding the magnitude of the forces that satisfies von Mises yield criteria on the horizontal leg.

Once the axial and shear forces on the horizontal leg are determined, equilibrium is used to determine the corresponding shear and normal forces on the vertical leg. Again, the relationship between the shear and normal forces on the vertical leg is constrained such that their resultant passes through the work point of the gusset plate. Once the forces acting on the vertical leg are determined, the von Mises yield criterion is checked on the vertical surface to verify that it does not control (e.g., the horizontal leg yields before the vertical leg). If the vertical leg is found to control, the forces corresponding to von Mises yielding could be applied to the vertical leg and equilibrium used to determine the corresponding forces on the horizontal leg. However, if the vertical, nondeteriorated leg governs the strength of the corner, then the pitting has not significantly affected the ability of the plate to transfer forces between the member in question and the rest of the node.

With the axial and shear forces on the vertical and horizontal legs known, they are combined to produce an equivalent resultant force that, because of the constraints described earlier, acts along the axis of the member. This resultant force represents the level at which von Mises yielding will initiate on one of the legs of the gusset plate corner and can be treated as a lower bound on the strength of the postulated corner section. It is important to note that significant additional capacity could in most cases be mobilized by considering strain hardening on the pitted section (if appropriate) and/or by taking advantage of the ability of truss members and plate sections to carry flexure. Indeed, if the conservative assumptions outlined in the preceding paragraph result in a calculated overstress condition, they should be relaxed. Sample calculations for a typical, basic corner check are included in the Appendix to this paper.

It should also be noted that if the pitting is not very uniform, the centroid of the remaining material will not necessarily coincide with the midpoint of the horizontal leg (point B). The resulting eccentricity of the normal force on the horizontal section should be considered in these cases.

When determining the resistance of the entire node, it is simple and conservative to use twice the capacity of the most deteriorated gusset plate at the node. Taking the lower capacity of the two plates and multiplying by 2 eliminates the need to consider the out-of-plane eccentricity that would develop from assuming two different capacities for the inside and outside gusset plates. This simplifying assumption can obviously be applied to other gusset plate limit states (e.g., fastener strength and block shear) as well.

Of course, if the members framing into the node have more capacity than the deteriorated gusset plates, they would be able to sustain some flexure before failure of the gusset plates. Therefore, the constraints described earlier related to the concentricity of the forces and the prohibition of moment in the members could be relaxed, resulting in higher capacities.

Bottom Chord Wide (BCW) Linear Pitting

Occasionally, bottom chord linear pitting can occur over an area that is much wider than the diameter of common bridge fasteners (see Figure 9). In such cases, treating the minimum net area in the pitted zone like the net area through a row of fasteners may not be appropriate. In situations where wide pitting is also consistently deep, the strains required to mobilize rupture strength across the entire pitted zone may lead to excessive or unrealistic plate deformations. To avoid inappropriate use of rupture strengths in such cases, evaluations that involve BCW linear pitting should include the following:

- A check of the rupture strength based on the thinnest section along the zone of interest (as described earlier for BCN linear pitting).
- A check of the yield strength based on the overall average thickness in the zone of interest. Such a check requires multiple thickness measurements in the transverse direction at each point along the pitted zone.

Figure 9 shows two recommended checks for gusset plates with areas of BCW pitting. The check shown at the left is of the rupture strength through the thinnest section in the plate. If the pitting is deep enough, rupture strength will control over the yield strength on the gross section. However, because the zone of pitting is so wide, it may be unrealistic to assume that the rupture strength can be developed in the thinnest area before strains in the less pitted areas lead to excessive deformations. In this case, using the average thicknesses at the three transverse points (t_1, t_2 and t_3 in Figure 5) at each longitudinal interval along the pitted zone in conjunction with the material yield strength may be more critical than using the minimum thickness/rupture strength check.

In any case, it is important to carefully review the spatial orientation and extent of the deterioration to determine if the rupture strength of the critical section can be mobilized without excessive plate deformation. As mentioned earlier, a good rule of thumb is to compare the narrow dimension of the deteriorated area to the fastener diameter. If the width is less than the fastener diameter, rupture strengths can likely be mobilized with acceptable plate deformation.

Web Member Narrow (WMN) Linear Pitting

Another form of linear pitting common to bridge trusses is that which occurs parallel to and just above the top flanges of sloped web members. As is the case with BCN linear pitting, WMN linear pitting is usually associated with accumulated debris that holds moisture against abutting steel surfaces. A case of WMN linear pitting is shown in Figure 10.

WMN linear pitting can be incorporated into an evaluation in much the same way BCN linear pitting is addressed. Typically, the net area should be evaluated to see if there will be any effect on block shear capacity.

Web Member Wide (WMW) Linear Pitting

WMW linear pitting is similar to BCW linear pitting and should be addressed in a similar manner.

GUSSET PLATE STRENGTHENING REPAIRS

Responsible use of available resources demands that repairs be installed only where they are needed or where it is more efficient to install repairs than it is to perform more rigorous analysis that *may* indicate repairs are not needed. Accordingly, conclusions regarding the need for repairs should not be based on overly conservative evaluation procedures. Furthermore, when it is clear that repairs are needed, they should be efficiently designed to address the actual deficiency.

When considering options for addressing localized gusset plate deterioration, the responsible engineer should note that the existing plate has substantial capacity, typically enough to carry all of the service dead load and a significant portion of the design live load without failing. If this were not true, the connection would have failed. Accordingly, most deficient elements only need to be supplemented to restore appropriate safety factors—and not replaced—and often by a relatively small amount.

It is also important for designers of gusset plate repairs to recognize that steel plates and steel structural fasteners are very ductile elements. In fact, plate and fastener ductility are relied on implicitly when standard design methods are employed (e.g., by not superimposing residual stresses with load-induced stresses, by assuming uniform—or nearly so—distribution of shear load in bolt groups, by using plastic shear and flexural strengths). If the benefits of ductility are overlooked, a repair designer may make excessively conservative assumptions that could lead to unnecessarily high repair costs. For example, given the inherent ductility of gusset plate connections, it would rarely be necessary to design repair or strengthening elements to resist the full applied loads. The existing plates can be relied upon to carry their share of the applied loads, even if they must undergo significant plastic deformation in the process. Figure 11 shows an efficient repair addressing the type of BCN deterioration shown previously in Figure 6. In this repair, a new steel angle connected to the bottom chord flange and existing plate/diagonals was added to supplement the reduced strength of the zone affected by substantial BCN deterioration. Such repairs can be proportioned using common splice and connection load transfer mechanics.

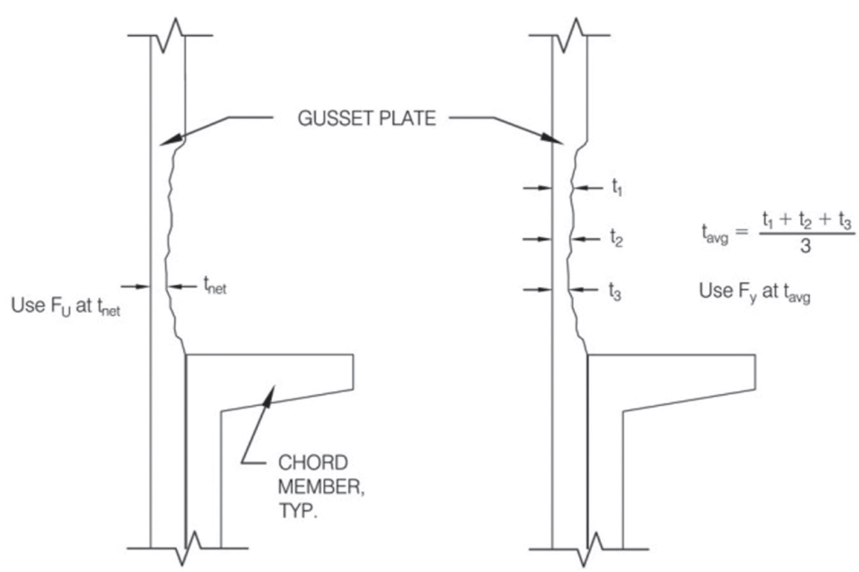

Fig. 9. Section through gusset plate showing two recommended checks for areas of BCW pitting.

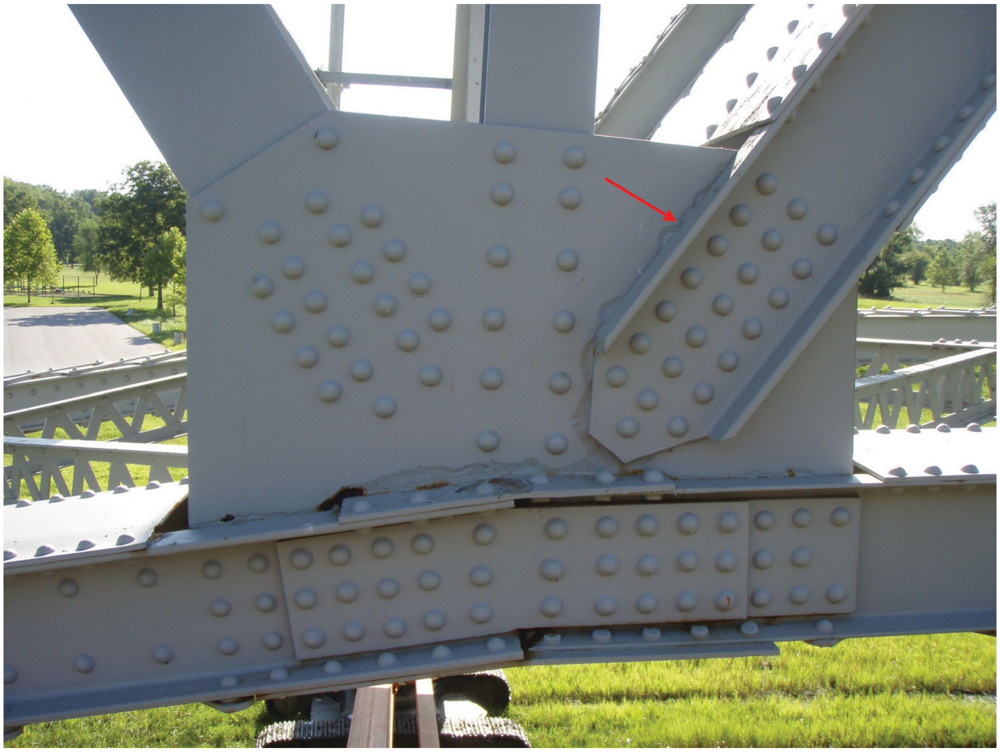

Fig. 10. Pitting a long top flange of diagonal member (WMN pitting).

Fig. 11. Efficient repair addressing BCN deterioration.

SUMMARY

The FHWA *Guide* provides a simple, straightforward, design-based approach for evaluating truss connection gusset plates. As such, it is a useful tool for quickly screening bridge truss connection plates and providing conservative load ratings. However, given its necessarily conservative nature, using it to define when repairs are needed can lead to unnecessary expenditures of limited bridge maintenance resources. In cases where the *Guide* approach indicates deficiencies, more rigorous evaluation methods should usually be employed.

When a gusset plate has experienced significant corrosion or other form of damage, the *Guide* approach is ill-suited to accurately quantify the effects, if any, on plate strength. Such cases should be evaluated using alternative methods. For example, free-body diagrams that incorporate deteriorated areas at their boundaries can be used to supplement the free-body checks that are considered sufficient for undamaged plates. Example free-body diagrams for deteriorated plates are presented herein, and sample calculations are provided in the Appendix. Given the cost of implementing changes to in-service gusset plates, finite-element modeling may also be practical.

When designing repairs to truly deficient plates, efficiency demands that the strength and ductility of the existing plate be taken into consideration. Otherwise, repairs and the associated costs can quickly become excessive. Robust and economical gusset plate repairs need not be mutually exclusive.

REFERENCES

AASHTO (2007), *AASHTO LRFD Bridge Design Specifications*, 4th edition, American Association of State Highway and Transportation Officials.

Dowswell, B. (2006), "Effective Length Factors for Gusset Plate Buckling," *Engineering Journal*, AISC, Second Quarter, pp. 91–101.

Drucker, D. (1956), "The Effect of Shear on the Plastic Bending of Beams," *Journal of Applied Mechanics,* ASME, Vol. 23, No. 12, pp. 509–514.

Galambos, T. (1998), *Guide to Stability Design Criteria for Metal Structures*, 5th edition, John Wiley & Sons, New York, NY.

Ibrahim, F. (2009), *Load Rating Guidance and Examples for Bolted and Riveted Gusset Plates in Truss Bridges*, Publication No. FHWA-IF-09-014, FHWA, U.S. Department of Transportation.

Salmon, C.G. and Johnson, J.E. (1996), *Steel Structures: Design and Behavior*, 4th edition, Harper Collins, New York, NY.

APPENDIX

Example

This check is a simple and conservative way to evaluate equilibrium of the gusset plate corner that resists the force imparted by the diagonal. The diagonal member force is assumed to be resisted by a combination of shear and axial forces acting on the vertical and horizontal legs that bound the corner. For the deteriorated state, the corner is chosen such that the legs extend through the deterioration. See Figure A-1.

Leg 1 is defined as the horizontal leg of the corner, leg 2 is the vertical leg, and θ_{member} is the angle between the horizontal and vertical planes.

$\theta_{member} = 52.37°$
$t_{gusset} = 7/16$ in.

Leg 1 properties *(horizontal)*

$L_1 = 13.31$ in.
$t_{loss1} = 3/16$ in. (average section loss across 2-in.-wide band)
$A_1 = L_1 (t_{gusset} - t_{loss1}) = 3.33$ in.^{2}
$\theta_1 = 26.55°$

Leg 2 properties *(vertical)*

$L_2 = 17.75$ in.
$t_{loss2} = 0$ in. (no section loss)
$A_2 = L_2 (t_{gusset} - t_{loss2}) = 7.77$ in.^{2}
$\theta_2 = 23.00°$

Design Steps

1. Assume the horizontal leg controls. Pick V such that the von Mises stress equals F_y on horizontal leg.
2. Determine V (and, by extension, P) on vertical leg such that the resultant force acting on the entire corner is along the axis of the diagonal. In other words, the resultant of all forces acts at angle θ_{member}.
3. Compute the resultant axial force. This is the axial force in the diagonal that the gusset can sustain without substantial yielding and without generating any significant moment at the node.
4. If the band of section loss is narrow (e.g., less than 1.5 in. wide), F_u can be used on the reduced section instead of

F_y. Section loss considered in this example is assumed to be wide, thus F_y is used.

Leg 1 (horizontal)

Trial V: $V_1 = 60.9$ kips

P is determined such that the resultant force on horizontal leg acts through the gusset work point.

$P_1 = V_1 \tan(\theta_1) = 30.4$ kips

Iterate on trial V until von Mises yield criteria is met along the leg:

$$\sigma_1 = \frac{P_1/A_1}{2} + \sqrt{\left(\frac{P_1/A_1}{2}\right)^2 + \left(\frac{V_1}{A_1}\right)^2} = 23.4 \text{ ksi}$$

$$\sigma_2 = \frac{P_1/A_1}{2} - \sqrt{\left(\frac{P_1/A_1}{2}\right)^2 + \left(\frac{V_1}{A_1}\right)^2} = -14.3 \text{ ksi}$$

$$\sigma_{vm} = \sqrt{\sigma_1^{\ 2} + \sigma_2^{\ 2} - \sigma_1\sigma_1} = 33.0 \text{ ksi (onset of von Mises yielding)}$$

Leg 2 (vertical)

Trial V: $V_2 = 108.1$ kips

P is constrained as in leg 1.

$P_2 = V_2 \tan(\theta_2) = 45.9$ kips

After determining the trial V for leg 1, determine the forces on vertical leg that will result in no net moment acting about the axis of the member. Using iteration or spreadsheet solver functions, the Von Mises stresses may be calculated:

$$\sigma_1 = \frac{P_2/A_2}{2} + \sqrt{\left(\frac{P_2/A_2}{2}\right)^2 + \left(\frac{V_2}{A_2}\right)^2} = 17.2 \text{ ksi}$$

$$\sigma_2 = \frac{P_2/A_2}{2} - \sqrt{\left(\frac{P_2/A_2}{2}\right)^2 + \left(\frac{V_2}{A_2}\right)^2} = -11.3 \text{ ksi}$$

$$\sigma_{vm} = \sqrt{\sigma_1^{\ 2} + \sigma_2^{\ 2} - \sigma_1\sigma_2} = 24.8 \text{ ksi}$$

This is less than the yield stress, F_y, of 33 ksi, so the assumption that the horizontal leg controls is correct.

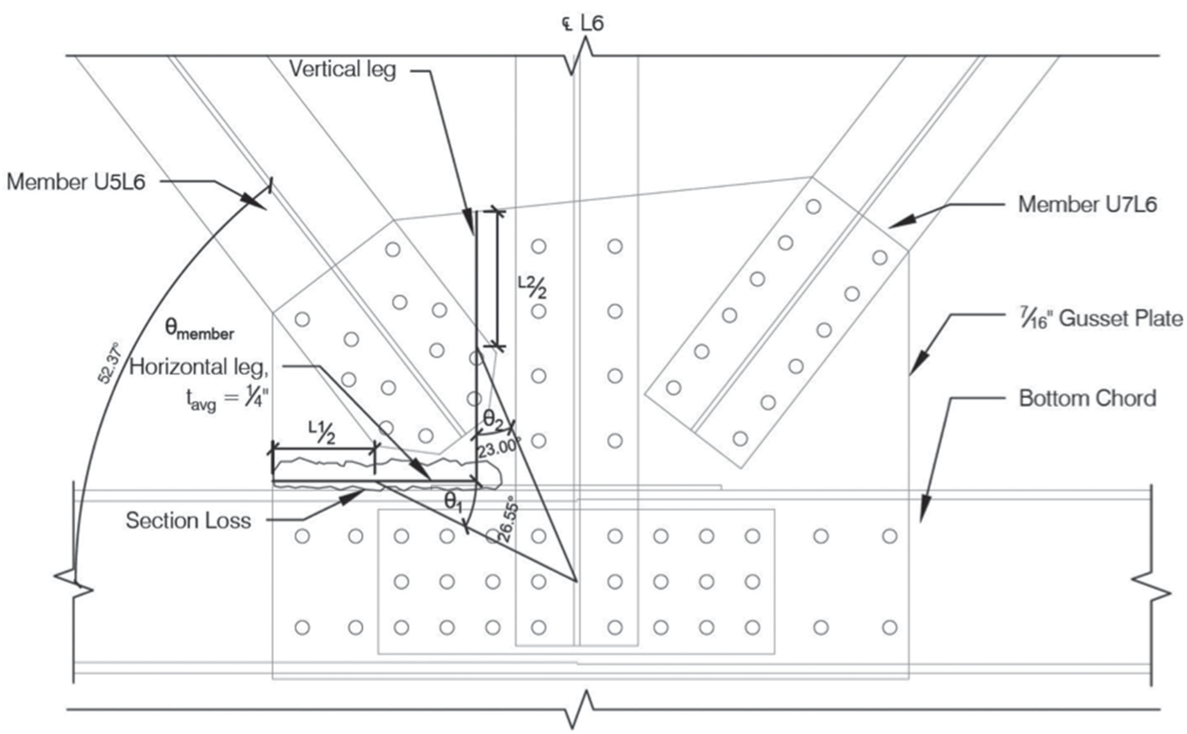

Fig. A-1. Example configuration.

Because the trial forces act through gusset work point by definition, we must simply check that they align with the member axis to ensure equilibrium of the corner:

$$\text{atan}\left(\frac{V_2 + P_1}{V_1 + P_2}\right) = 52.37° = \theta_{member} = 52.37°$$

o.k.—solution found that meets all established conditions

$\phi_y = 0.9$
$\phi_v = 0.9$

Strength reduction factors vary based on governing code. For simplicity, we assume 0.9 for the yielding limit state.

Compute the total resultant force acting on the corner:

$$\phi R = \sqrt{(\phi_v V_2 + \phi_y P_1)^2 + (\phi_v V_1 + \phi_y P_2)^2} = 157 \text{ kips}$$

Because there are two gusset plates per node, compare this computed capacity to one-half of the design load in the diagonal.

Notes

1. The computed capacity is a lower bound in this case. The constraint that only shear and axial forces act on the legs could be relaxed, as could the requirement that resultant member and nodal forces are concentric. Also, if the band of deterioration is narrow, it is conservative to use F_y on the net section. Items like this should be explored if the methods described previously result in insufficent capacity.
2. In cases where the deterioration is not uniform along the leg (e.g., a large hole at one end), it may be necessary to refine the equilibrium check to account for the resulting eccentricty. In such a case, the resultant force will not act through the midpoint of the corner leg as assumed.