Block shear failure of steel gusset plates

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ABSTRACT

Following the catastrophic failure of the I-35W bridge in Minnesota in 2007, the Federal Highway Administration issued guidelines for the load rating of bolted and riveted gusset plates in truss bridges (FHWA, 2009). This paper develops finite-element models capable of predicting the behavior of gusset plates in tension, resulting in possible failure by block shear. Nonlinear finite-element analysis validated by experimental data confirms the safety and validity of the FHWA load rating formulas for the block shear strength of riveted and bolted gusset plates. For a variety of geometries, the Load and Resistance Factor Rating (LRFR) value produces factors of safety between 2.1 and 2.7. This study also provides guidance on the mesh density required around the holes, the application of bolt loads, and the approximation involved in modeling the perimeter holes only.

Keywords: block shear; FHWA Guidance; finite-element analysis; gusset plate; I-35W; load rating; load distribution; tension failure.

1 INTRODUCTION

Following the catastrophic failure of the I-35W bridge in Minnesota in 2007 and its investigation (NTSB 2008a, b), the Federal Highway Administration issued guidelines for the load rating of bolted and riveted gusset plates in truss bridges (FHWA, 2009). The guidelines combine safety with simplicity by providing bridge owners simple hand formulas based on laboratory tests, but do not discuss the formulas in a commentary. This paper develops finiteelement models capable of predicting the behavior of gusset plates in tension, resulting in possible failure by block shear. The intent is to supply bridge owners and managers with a more complete set of tools, not only for load rating but also for structural analysis. A companion paper addresses the compression behavior of gusset plates (Crosti and Duthinh, 2010).

2 CURRENT GUIDELINES

Block shear failure is a limit state that combines tension failure on one plane and shear failure on a perpendicular plane (Fig. 1). The guidelines assume that, when one plane reaches ultimate strength, the other plane develops full yield. Therefore, two possible failure modes can develop: in the first, rupture occurs along the net tension plane and full yield develops along the gross shear plane. The second failure mechanism assumes that rupture occurs along the net shear plane while full yield develops along the gross tension plane.



Figure 1. Example of potential block shear rupture planes for gusset plates in tension (FHWA, 2009).

If
$$A_{tn} \ge 0.58 A_{vn}$$
, then
 $P_r = \varphi(0.58F_yA_{vg} + F_uA_{tn})$ Eq.1
Otherwise: $P_r = \varphi(0.58F_uA_{vn} + F_yA_{tg})$ Eq.2

where:

 φ = resistance factor = 0.80 for Load and Resistance Factor Rating (LRFR), or

 $\varphi = 0.85$ for Load Factor Rating (LFR);

 A_{tg} = gross area along plane resisting tension;

 A_{tn} = net area along plane resisting tension;

 A_{vg} = gross area along plane resisting shear;

 A_{vn} = net area along plane resisting shear;

 F_u = minimum tensile strength of gusset plate; and

 F_y = minimum yield strength of gusset plate.

Next, we compare these formulas with numerical and experimental results.

3 FINITE-ELEMENT MODEL

This study used shell elements with four-nodes, available in commercial software STRAND, to perform nonlinear finite-element analyses of gusset plates under tension. For validation, we first simulated a test (Fig. 2) performed at the University of Alberta (Nast, Grondin and Cheng, 1999).



Figure 2. University of Alberta gusset plate test (Nast, Grondin and Cheng, 1999).

Plate 1 has the dimensions shown in Fig. 3, a thickness of 9.61 mm and ten bolt holes of diameter 24.3 mm (bolt failure is not part of this study). The model is fixed along the two perpendicular edges at the bottom and left. The analysis accounts for the nonlinearity of the material and large displacements. The material is bilinear elasto-plastic, with Young's modulus of 215 GPa, yield strength of 410 MPa and tangent modulus of 2.15 GPa. The analysis uses true stress and true strain.



Figure 3. Finite-element model meshed with 9520 elements.

The tension load is applied by three point loads on the upper half of each bolt hole (Fig. 4).



Figure 4. Application of bolt loads.

From Eqs. (1) and (2), and as detailed in Table 1, (net area calculations conform with AASHTO, 1994) the gusset plate fails in block shear at an unfactored load ($\varphi = 1.0$) of 1512 kN.

Table 1. Calculation of block shear capacity.

Tension	Eq. 1	Shear	Eq. 2
L_t	73.8 mm	L_{v}	306 mm
t	9.61 mm	t	9.61 mm
Ø	24.3 mm	Ø	24.3 mm
	27.5 mm		27.5 mm
A_{tg}	710 mm ²	A_{vg}	5880 mm ²
f_y	410 MPa	f_y	410 MPa
f_u	600 MPa	f_u	600 MPa
A_{tn}	446 mm ²	A_{vn}	3510 mm ²
		$0.58 A_{vn}$	2030 mm ²
P_u	1818.64 kN	P_u	1511.60 kN

3.1 Finite-element mesh

Two levels of mesh refinement were used to model the regions of stress concentration around the bolt holes (Fig. 5) where failure would likely initiate.



Figure 5. Mesh refinement around holes.

In Fig. 6 the load-displacement response of the node in the middle of the oblique edge is used to

compare the test and numerical results from the University of Alberta with the unfactored FHWA value P_u , the LRFR value 0.80 P_u , and the finite-element results for the coarse and the fine mesh.



Figure 6. Results of mesh study.

Results from the present two STRAND meshes agree well with the ABAQUS model used by the University of Alberta. All three finite-element results slightly underestimate the test results, especially at the onset of yielding. The LRFR value falls on the limit of the elastic range, whereas the unfactored FHWA value produces a small amount of yielding. The coarse mesh is used in the rest of this study.

3.2 Bolt load application

The previous finite-element results predicted premature yielding compared to test results. One possible reason is the bolt load, which is applied in too concentrated a fashion at three nodal points for each bolt hole. In the next step, we distributed the bolt load to nine points over a 90° arc (Fig. 7), following a sinusoidal distribution, $P = P_0 \cos \alpha$, where $-45^\circ \le \alpha \le 45^\circ$.



Figure 7. Coarse and fine bolt load application.

Figure 8 compares the FHWA values, the test results and the results for the coarse and fine bolt load applications. There is not much difference between the results for the two bolt load applications, even though the trend is opposite to what is expected.



Figure 8. Results for coarse and fine bolt load application.

3.3 Number and arrangement of holes

Next we investigate the influence of the number and arrangement of the perimeter bolt holes on the maximum load and displacement at failure. Previous finite-element studies include Topkaya, 2004. We show that, although the FHWA loads are safe, the degree of safety and the ductility of the connection vary with geometry. In the following analyses, our criterion for structural failure is the attainment of a maximum strain of ± 100 %, as justified in Huns, Grondin and Driver, 2002.

In the next three cases, the number of holes is increased to 16, but their diameter and all the other plate dimensions are the same as before. As well, the displacements of interest continue to be at the middle of the oblique side.

Plate 2:

Plate 2 has a tension length L_t of 0.139 m, a shear length L_v of 0.358 m and bolt holes arranged as shown in Fig. 9. Since $A_{tn} < 0.58 A_{vn}$, failure is by block shear. Figs. 9 and 10 show the maximum principal strain profile and the load-displacement curve of this gusset plate at various load steps.

In Fig. 9, elastic regions are in color whereas plastic regions and the holes are in white. As expected, straining is most intense between the holes, yielding initiates near the holes, then spreads to the connecting regions. The strain profile is compatible with eventual rupture along the shear plane while full yield develops across the tension plane between the two end holes.

Figure 10 shows an ultimate load of 2720 kN at a displacement of 14 mm, and thus the LRFR value provides a factor of safety of ultimate load/LRFR value = 2720/1267 = 2.15.

Plate 3:

The bolts in Plate 3, shown in Fig. 11, are distributed over a narrower width. This gusset plate has a tension length L_t of 0.0462 m and a shear length L_v of 0.358 m. Again, since $A_{tn} < 0.58 A_{vn}$, failure is by block shear.

Similar observations about the maximum principal strain profile and the load-displacement curve can be made for plate 3 as for plate 2.



- Ey<E<Ey





Figure 9. Strain profiles of Plate 2 ($A_{tn} < 0.58A_{vn}$).



Figure 10. Load displacement for Plate 2.

Fig. 12 shows an ultimate load of 2088 kN at a displacement of 14 mm, and thus the LRFR value provides a factor of safety of ultimate load/LRFR value = 2088/974.4 = 2.14.







Figure 11. Strain profiles of Plate 3 ($A_{tn} < 0.58A_{vn}$).



Figure 12. Load-displacement curve for Plate 3.

Plate 4:

Figure 13 shows the results for Plate 4 with tension length L_t of 0.150 m and shear length L_v of 0.273 m. In this case, $A_m > 0.58 A_{vn}$ and failure is by tension. Fig.14 shows an ultimate load of 2190 kN at a displacement of 7.8 mm, and thus the LRFR value provides a factor of safety of ultimate load/LRFR value = 2190/918.2 = 2.38.

Thus, Plates 2 and 3, which failed by block shear, even though they had rather different hole arrangement, behaved very similarly. Plate 4, which failed by tension, had a slightly higher factor of safety but less ductility than Plates 2 and 3. In all cases, the FHWA values are safe and adequate.







Figure 14. Load-displacement curve for Plate 4.



1500

1000

500

0

0.0

Ther 34

lncr.26

Incr.21

Plate 5

Plate 2

 $5.0 \quad 10.0 \quad 15.0 \quad 20.0 \quad 25.0 \quad 30.0 \quad 35.0 \quad 40.0 \quad 45.0 \quad 50.0$ D (mm) Figure 16. Load-displacement curve for Plate 5.

Theoretical Value (Pu)

Theoretical Value (0.8*Pu)

Plate 5:

Compared to Plate 2, Plate 5 has a third row of bolts. The strain profiles shown in Fig. 15 include the strains that exceed yield at the last load step, when the highest strains reach 100 % (at a few localized points). The profile has better resolution and is more instructive when the plot is limited to strains of 50 %. In the last plot of Fig. 15, the elastic regions and the holes are in white. Fig.16 shows an ultimate load of 3468 kN at a displacement of 47.3 mm, and thus the LRFR value provides a factor of safety of ultimate load/LRFR value = 3468/1267 = 2.73. Fig. 16 also shows the beneficial effect of adding internal bolts, which increase the strength and ductility of the gusset plate. A simplified analysis that only accounts for the perimeter bolts would underestimate the strength and ductility of the gusset plate.

Plate 6:

We now analyze one of the example plates from the guidance document (FHWA, 2009). The gusset plate considered is shown in Fig. 17 and its steel is elasto-plastic, with Young's modulus of 200 GPa, a yield strength of 248 MPa, and a tangent modulus equal to 1% of the elastic modulus. Failure is by block shear. Fig. 18 shows an ultimate load of 10750 kN at a displacement of 43.2 mm, and thus the LRFR value provides a factor of safety of ultimate load/LRFR value = 10750/4704 = 2.28.



Figure 18. Load-displacement curve for Plate 6.

4 CONCLUSION

Nonlinear finite-element analysis validated by experimental data confirms the safety and validity of the FHWA load rating formulas for the block shear strength of riveted and bolted gusset plates. For a variety of geometries, the LRFR value produces factors of safety between 2.1 and 2.7. This study also provides guidance on the mesh density required around the holes, the application of bolt loads, the effects of geometry, and the approximation involved in modeling the perimeter holes only.

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