Rationale Behind and Proper Application of the Ductility Factor for Bracing Connections Subjected to Shear and Transverse Loading

CHRISTOPHER M. HEWITT and WILLIAM A. THORNTON

In 1990, AISC and ASCE jointly commissioned a task group to develop a design philosophy for predicting the strength of bracing connections. The design methods in place prior to that time were only applicable to connections made to the webs of columns, as a proper methodology for brace-to-column flange connections had not been agreed upon. The overall goal was to create a complete design philosophy that could be used to accurately predict the strength of bracing connections and ensure ductile structural response.

The task group devised a study of five bracing connection design methods. Among the five design methods evaluated were the Uniform Force Method, developed by Thornton (1991), and a modified version of a design method developed by Richard (1986). The study included a comparison of the predicted response of each model to the actual response found by testing in both isolated, Bjorhovde and Chakrabarti (1985), and complete frame action cases, Gross (1990), for each bracing connection. After extensive study and comparison the committee recommended the use of the Uniform Force Method as the most accurate predictor of the true structural response of bracing connections.

Although it was found that the Uniform Force Method was the most accurate predictor of true connection performance, the virtues of each design method were considered for application to an overall design philosophy. Richard's design method was based on empirical results derived from 54 finite element models. Richard's studies showed that frame action significantly affects the gusset-to-frame fastener force distributions and that the gusset connection force distributions primarily depend on the plate aspect ratio and the brace angle. Richard's data suggested that uneven stress distributions are produced across a gusset edge in a bracing connection, the peak stress of such a distribution being approximately 1.4 times the average predicted stress value at ultimate load (see Figure 1). As explained by Thornton (1984), the Uniform Force Method captures the effect of frame action, but obviously the method does not capture the uneven stress distribution.

In response to Richard's findings, the ability of a gusset connection to redistribute forces and achieve a uniform distribution was considered. In the case of a gusset welded to the web of a W-shape member, the flexibility of the web was assumed to be sufficient to redistribute an uneven force distribution. However, in the case of a bracing connection welded to the flange of a member, which has significantly more rigidity than a connection to the web, the potential inability of the system to accommodate force redistribution was considered. The development of a peak stress induced at some point across the welded connection might cause the weld to fail at the point where the stress is concentrated, causing an unzipping of the weld and a progressive failure of the welded connection. In this case, the uniform force distribution used in the Uniform Force Method might be violated, resulting in failure below the theoretically predicted strength of the connection.

An example of this scenario can be seen in the 3rd Edition *Load and Resistance Factor Design Manual of Steel*

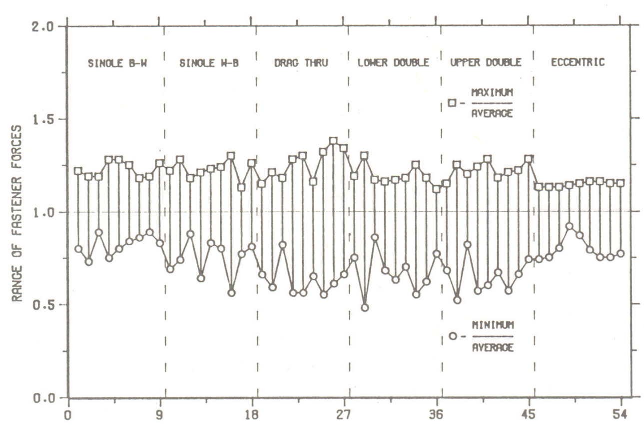

Fig. 1. Summary of finite element model results from Richard's work displaying the ratio of peak vs. average yield stress.

Christopher M. Hewitt is staff engineer, American Institute of Steel Construction, Inc., Chicago, IL.

William A. Thornton is president, Cives Engineering Corporation, Roswell, GA.

Construction (*LRFD Manual*), Chapter 13, Example 13.1 (AISC, 2001). Because of the proximity of the brace claw angle to the gusset to beam connection, a peak stress can be expected at point A in Figure 2.

To maintain ductility in the connection for the case of a welded joint subjected to both shear and transverse loading, such as the attachment of a gusset to the flange of a member, the *LRFD Manual* states, on page 13-28:

> From Richard (1986) it is recommended that the design factored force be increased by 40 percent to ensure adequate force redistribution in the weld group and the validity of the Uniform Force Method. Thus,
>
> $$D_{req} = \frac{1.4P_{ub}}{1.392l}$$
>
> (Note that if a moment existed on this interface the connection would be designed for the larger of the peak stress and 1.4 times the average stress.)

This 40 percent increase in the design force for the welded joint was adopted to account for uneven distributions in a directly welded gusset edge connection to a member flange, predicted by a ratio of peak versus average stress in the joint, and has been the basis for the recommendations of the *Manual*. By providing over strength in the connection, this factor maintains the validity of the design assumptions in the Uniform Force Method. Looking back at Richard's test results, the upper bound value of a statistical 90 percent confidence interval of the graphed data points suggests a value of 1.25 as an appropriate design value, assuming a normal distribution. The 90 percent confidence interval upper bound of 1.25 should replace the value of 1.4 currently recommended in the *Manual*, particularly since a resistance factor is already applied in the design procedure to account for variations in material properties. Ultimately, the actual weld size need not exceed that required to develop the strength of the thickness of plate used.

PROPER APPLICATION OF THE DUCTILITY FACTOR

The ductility factor should be applied to ensure adequate strength across the weld under any loading condition, and its application should not be limited to connections to resist seismic loads. The most critical element to the proper application of this factor is a calculation of the "average stress" that is compatible with Richard's work. In order to comply with the conditions of Richard's analysis, the "average stress" is taken as the average scalar value of the combined axial, shear, and bending stress across the connection. Figure 3 shows typical distributions of shear and bending stresses. The bending stress distribution is taken as rectangular because the forces considered in Figure 1 are at the ultimate capacities of the 54 connections considered in Richard's studies.

The "average stress" considering a rectangular stress distribution can be calculated in the following manner:

Let $f_b = f_b$ for $x = 0 \rightarrow l/2$

$f_b = -f_b$ for $x = -l/2 \rightarrow 0$

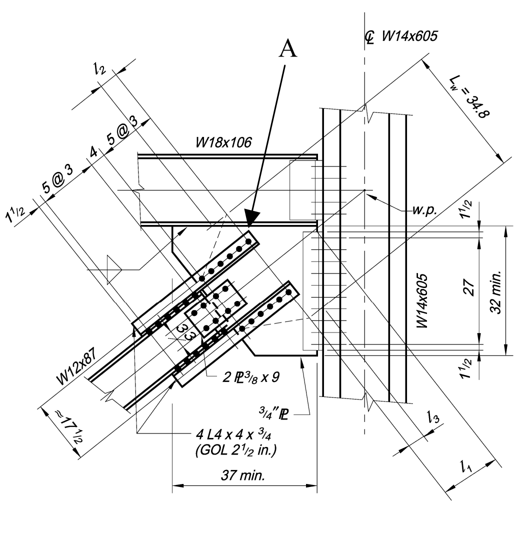

Fig. 2. Welded bracing connection from 3rd Edition LRFD Manual.

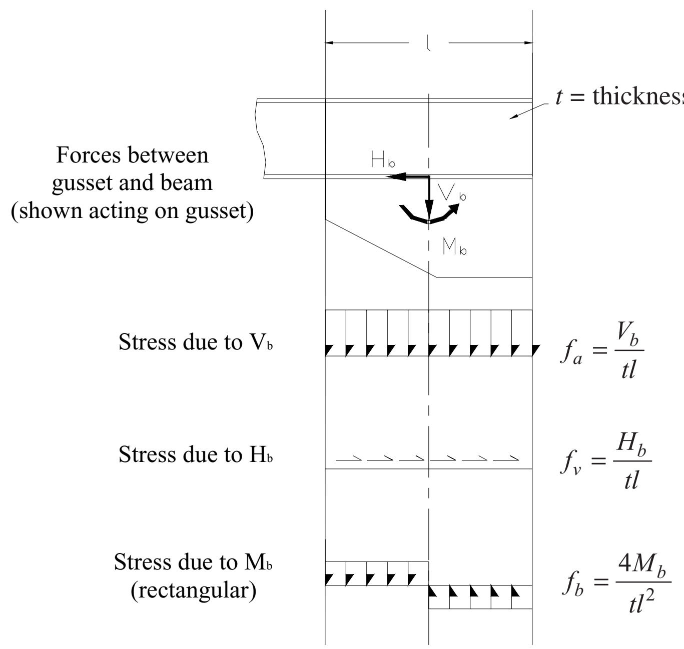

Fig. 3. Gusset-to-beam connection stress distribution.

$$f_{avg} = \frac{1}{l}\left[\int_{-l/2}^{0} \sqrt{(f_a - f_b)^2 + f_v^2}\, dx + \int_{0}^{l/2} \sqrt{(f_a + f_b)^2 + f_v^2}\, dx\right]$$

From this integration, we arrive at the following calculation for the average stress across the connection:

$$f_{avg} = \frac{1}{2}\left[\sqrt{(f_a - f_b)^2 + f_v^2} + \sqrt{(f_a + f_b)^2 + f_v^2}\right]$$

Likewise, the peak stress value is then taken as:

$$f_{peak} = \sqrt{(f_a + f_b)^2 + f_v^2}$$

The connection design stress is then taken as the greater of f_{peak} or $1.25 f_{avg}$, and the weld is sized accordingly. By completing this calculation, the designer ensures that a design peak stress larger than the weld strength is not induced across the joint and ductility (in other words, load redistribution capability) is maintained under any loading condition.

Figures 4 and 5 show a real application of the appropriate use of this factor.

In this example, ΔV_b is taken as 132 kips, to reduce the beam shear to:

$$221 + 369 - 132 = 448 \text{ kips}$$

which is just under the shear capacity of the beam. This is done in order to avoid the need for a doubler plate for shear and is covered as special case 2 in the *LRFD Manual*, page 13-36. Note from Figure 4 that a doubler was still required, but this is for axial force block shear and is not required to be welded in the k-area.

On the gusset to beam connections interface, the resultant forces are:

H_b = 643 kips shear
$V_b - \Delta V_b$ = 369 − 132 = 237 kips axial
M_b = 38.5 × 132 = 5094 kip-in. moment

The length of the connected edge is 36.5 in. (the set back from the column face is 1 in.).

The gusset is 1¾ in. thick and is made of A992 (Grade 50) steel. The gusset stresses are:

$$f_v = \frac{643}{1.75 \times 36.5} = 10.1 \text{ ksi} < 27 \text{ ksi} \quad \text{o.k.}$$

$$f_a = \frac{237}{1.75 \times 36.5} = 3.71 \text{ ksi}$$

$$f_b = \frac{5090 \times 4}{1.75 \times 36.5^2} = 8.73 \text{ ksi}$$

$$f_a + f_b = 3.71 + 8.73 = 12.4 < 45 \text{ ksi} \quad \text{o.k.}$$

Note that the thickness of the gusset plate was determined by the limit state of Whitmore buckling. The required weld size for the gusset to beam flange is calculated as follows:

$$f_{peak} = \sqrt{(3.71 + 8.73)^2 + 10.1^2} = 16.0 \text{ ksi}$$

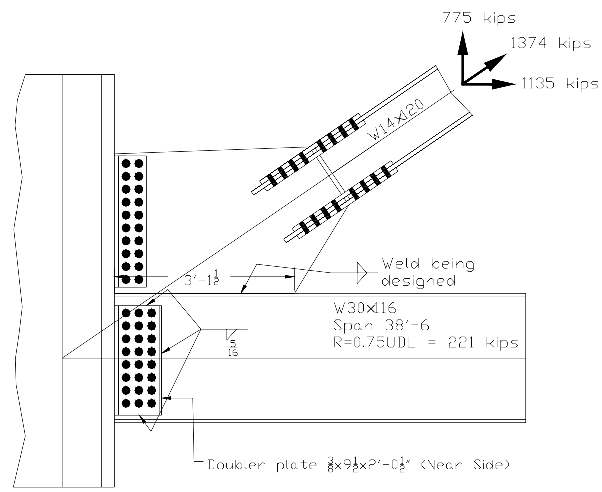

Fig. 4. Sample bracing connection.

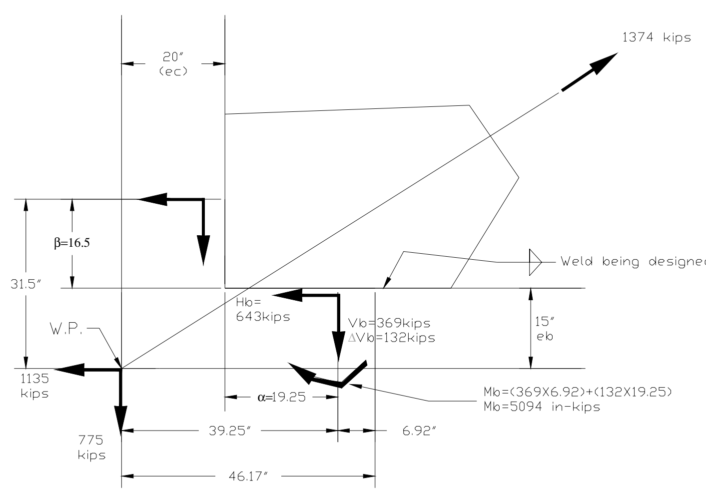

Fig. 5. Free body diagram of bracing connection.

$$f_{avg} = \frac{1}{2}\left[\sqrt{(3.71-8.73)^2+10.1^2}+\sqrt{(3.71+8.73)^2+10.1^2}\right]$$
$$= 13.7 \text{ ksi}$$

The stress for weld design is:
$f_{weld} = \max(f_{peak}, 1.25 f_{avg})$
$f_{weld} = \max(16.0, 1.25 \times 13.7) = 17.1$ ksi

and the weld size required is (in 1/16 in.)

$$D = f_{weld}\left(\frac{t}{2}\right)\times\left(\frac{1}{1.392}\right) = \frac{17.1\times1.75}{2\times1.392} = 10.7$$

The required weld size is thus 11/16 in.

CONCLUSIONS AND RECOMMENDATIONS

A re-evaluation of the statistical data used to derive this factor suggests that the 40 percent increase recommended in the *LRFD Manual of Steel Construction* is overly conservative. It is recommended that this factor be revised to 25 percent, applied consistently with the procedure outlined here and in Richard's work. A 25 percent stress increase is sufficient to ensure that the weld has sufficient strength to resist a peak stress in the weld with statistical confidence comparable to that used in the design of other connection types.

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