

# Experimental Study of Gusseted Connections

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## INTRODUCTION

Gusset plate connections are common in lateral bracing systems in steel construction. Recent analytical studies<sup>12,4</sup> have demonstrated that the gusset plate produces a rigid beam-to-column connection, even if the beam flanges are not connected to the column. The importance of this rigid beam-to-column connection is that the gusset plate carries a significant load from “frame action” in addition to that from the diagonal bracing member. The influence of framing members on gusset plate behavior has not yet been studied experimentally and the effect of the stresses produced by frame action on the gusset plate capacity is not known. In addition, tests to date<sup>11,2,10</sup> have all used configurations in which the centroidal axes of the members framing into the connection have all intersected at a single point. The loads introduced into a gusseted connection when the centroidal axes of the framing members do not intersect at a point have not been experimentally determined. Finally, the column in a lateral bracing system may be oriented so that the gusset is connected either to the flange or to the web. Unless the web is stiffened, a connection to the column web is more flexible than one in which the gusset is connected to the flange. No experimental work to date has addressed the difference in performance between these two types of column connection and no data is available to indicate whether stiffening of the web is required.

At the suggestion of the AISC Task Group on Bracing for Heavy Construction, an experimental program was undertaken at the National Institute of Standards and Technology\* (NIST) to determine:

- The influence of the members framing into the connection on the behavior and strength of the connection;
- The effect of connection eccentricity on gusset plate capacity and on the distribution of forces to the framing members;
- The difference in performance between a connection made to the column flange and one made to the column web and whether stiffening is required for strength.

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Three nearly full-scale braced frame subassemblages were tested, each representing a different connection configuration. This paper summarizes the findings of the NIST tests and compares the results obtained with design practice and proposed analytical procedures. A detailed report on this experimental investigation may be found in Ref. 5.

## CURRENT DESIGN PRACTICE

The size and shape of a gusseted connection is generally dictated by fastener requirements. The thickness of the gusset plate is selected on the basis of one or more of the following:

- maximum stress calculated using beam formulas,
- maximum stress on the Whitmore section,
- block shear failure mode,
- buckling of the gusset plate.

Beam formulas continue to be used, despite their apparent failings, largely because their use has produced designs that perform satisfactorily. Whitmore<sup>11</sup> proposed a procedure to estimate the maximum stress in the gusset plate, and this procedure is in widespread use today. Recently, the block shear concept has been developed for the prediction of the ultimate strength of a bolted gusset plate in tension.<sup>12,6,8</sup> Kulak, et al.<sup>7</sup> recommend that both the maximum stress on the Whitmore section and the block shear failure mode be evaluated and the more severe condition be used for selecting the plate thickness. When the diagonal brace acts in compression, plate buckling capacity can be checked using the procedure described in Ref. 3.

Design of the gusset plate attachments is dictated by the loads which must be transferred to the beam and column. However, the distribution of diagonal bracing loads to the framing members is not well understood. Generally, two options are available to the designer. One is to detail the connection so that the axes of all members framing into the connection pass through a common point (Fig. 1a). This alternative insures that the force in the diagonal brace does not produce a moment in the connection and is termed a concentric connection. However, the loads along the gusset plate edges produced by this arrangement (Fig. 1b) are difficult to assess. In addition, this arrangement generally produces a long gusset plate which has a lower buckling capacity than has a more compact gusset plate.

A second option is to select a “working point” which permits a simplified approach to determining the loads on the edges of the gusset plate. For example, if the line of force in the diagonal brace passes through the corner of the gus-

set plate, a common and simple procedure<sup>3</sup> is to assume that the vertical component of the brace force is carried by the gusset-to-column connection and the horizontal component is carried by the gusset-to-beam connection (Fig. 2b). In addition to simplifying the design of the gusset attachments, this approach generally produces a compact gusset plate which has a greater buckling capacity than a long gusset plate. However, if the line of action of the force in the diagonal member does not pass through the intersection of the beam and column centroidal axes, the force in the diagonal member will produce a moment in the connection (Fig. 2a). Due to the moment produced with this arrangement, such a connection is termed eccentric. The moment, if significant, must be accounted for in the selection of the beam and column members.

Richard<sup>9</sup> proposed a method for determining the distribution of the force in the diagonal brace to the beam and column which is independent of the choice of the working point. In this method, the distribution of the diagonal brace force depends on the angle the diagonal brace makes with respect to the beam and the aspect ratio of the gusset plate. Richard's method was used as the basis for design of the gusset plate attachments in this study.

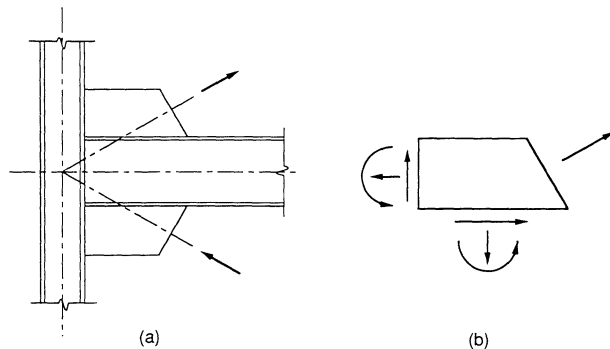


Fig. 1. Axes of all framing members pass through a single point.

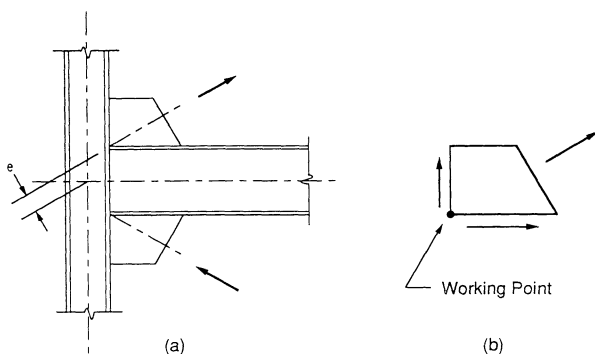


Fig. 2. Axes of diagonal bracing members pass through the corners of the gusset plates.

## EXPERIMENTAL PROGRAM

### Braced Frame Subassembly

The braced frame configuration selected for study in this project is one having diagonals alternating in direction from story to story. A schematic of the braced core of a multistory steel building having such an arrangement of diagonals is shown in Fig. 3a. The subassembly selected for this experimental program and the assumed boundary conditions and loading are shown schematically in Fig. 3b. The height of the subassembly was 9 ft 0 in. between column pin supports. The width from column centerline to beam pin support was 7 ft 6 in. This subassembly represents approximately a  $\frac{3}{4}$ -scale structure. The geometry of the subassembly and the pin fixtures used to represent the boundary conditions are shown in Fig. 4. Each specimen was loaded in the plane of the frame as shown in Fig. 3b. Lateral load was applied so as to produce tension in the upper diagonal. The specimen was loaded to failure which was defined as the point at which the subassembly was unable to carry additional load.

### Test Specimens

Three specimens, each with a different connection detail, were tested. The two principal variables were: (1) gusset geometry (concentric vs. eccentric connection detail), and (2) column orientation (strong axis vs. weak axis column connection). The three specimens were as follows:

- *Specimen No. 1*—Concentric, strong-axis column connection where the line of force in the diagonal passes through the intersection of the beam and column centroidal axes producing a non-compact gusset plate (Fig. 5).
- *Specimen No. 2*—Eccentric, strong-axis column connection where the line of force in the diagonal passes through the corner of the gusset plate producing a compact gusset plate (Fig. 6).

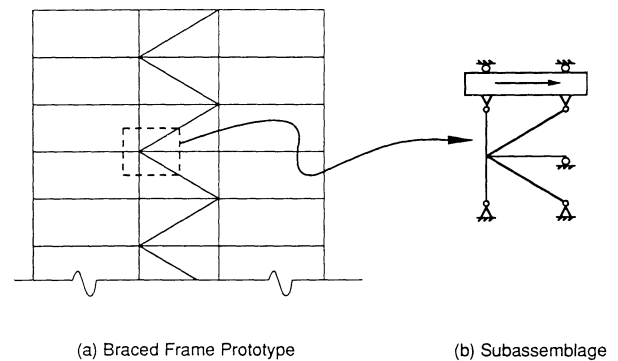


Fig. 3. Braced frame prototype and subassembly selected for study.

- *Specimen No. 3*—Eccentric, weak-axis column connection where the line of force in the diagonal passes through the corner of the gusset plate producing a compact gusset plate (Fig. 7).

The framing members for each test specimen were the same. The column was a W10×49, the beam was a W16×40, and the diagonal braces were W8×21s. The gussets were attached to the beam flanges with fillet welds and were connected to the column by clip angles welded to the gusset and bolted to either the column flange or web. The gusset plates were ¼-in. thick and the clip angles were L3×3½×¼. The beam flanges were coped to allow the clip angles to run the entire depth of the connection as a single piece thereby simplifying fabrication and fit-up. The diagonal braces were bolted to the gusset using WT5×11 sections. The gusset plates and clip angles were fabricated from 36 ksi material and all other members were fabricated from 50 ksi steel. E70 electrodes were used for all weldments and A325 high strength bolts were used for all bolted attachments.

### Instrumentation

The test specimens were instrumented with resistance-type strain gages to record strains and with linear variable differential transformers (LVDT) to record displacements or rotations. Strain gages were applied to determine the axial force, shear force and bending moments in all members and the distribution of stress along the connected edges of the gusset plates.

### EXPERIMENTAL RESULTS

The lateral load on the subassembly and the lateral deflection of the top of the subassembly were recorded. Load

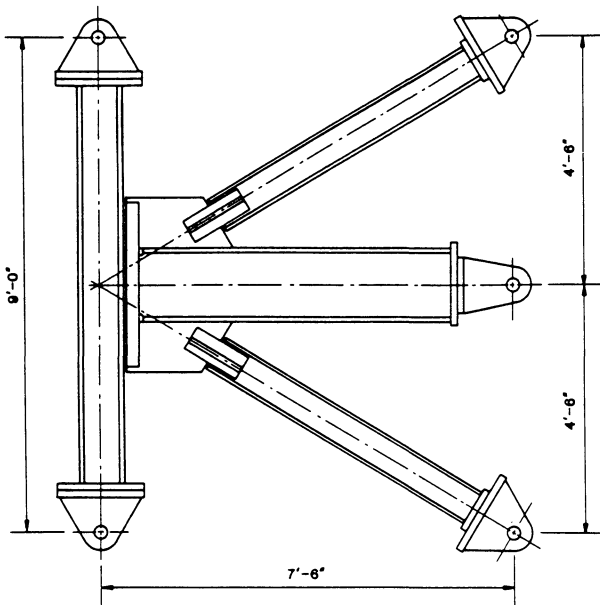


Fig. 4. Geometry of subassembly.

|                                     | Specimen No. |          |         |
|-------------------------------------|--------------|----------|---------|
|                                     | 1            | 2        | 3       |
| Frame Stiffness (kip/in.)           | 288          | 258      | 240     |
| Mode of Failure                     | buckling     | buckling | tearing |
| Ultimate Lateral Load (kip)         | 107          | 123      | 122     |
| Force in Tension Diagonal (kip)     | 100          | 113      | 125     |
| Force in Compression Diagonal (kip) | 116          | 138      | 139     |

vs. deflection plots for the three specimens are shown in Figs. 8–10. Lateral stiffness of the subassembly, specimen failure mode, ultimate subassembly lateral load and forces in the diagonal bracing members at failure are summarized in Table 1.

### Observations for Specimen No. 1

The load vs. deflection plot for Specimen No. 1 is shown in Fig. 8. A frame stiffness of 288 kip/in. was obtained by linear regression analysis. Slip of the lower splice between the gusset and the diagonal occurred at a lateral load of 90 kips as evidenced by a sudden drop in lateral load. Buckling of the lower gusset occurred suddenly at a lateral load of 107 kips. The lateral frame load dropped from 107 kips to 96 kips as shown in Fig. 8. When the gusset buckled, the force in the compression diagonal was 116 kips and the force in the tension diagonal was 100 kips. The corresponding frame displacement was 0.50 in.

### Observations for Specimen No. 2

The load vs. deflection plot for Specimen No. 2 is shown in Fig. 9. The frame stiffness was 258 kip/in. Slip of one of the splices occurred at a lateral load of approximately 124 kips with an attendant drop in lateral load. As the lateral

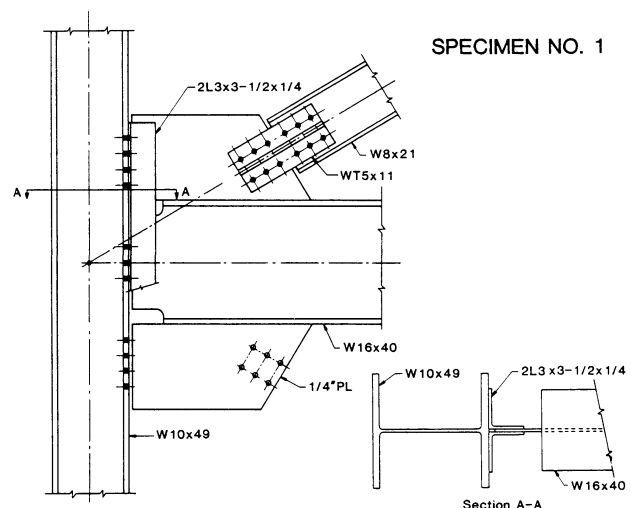


Fig. 5. Specimen No. 1.

load increased, the lower gusset buckled before the load at which the slip occurred was reached. The lateral displacement corresponding to the buckling load was 0.80 in. The load in the compression diagonal at buckling was 138 kips and the load in the tension diagonal was 113 kips. Buckling of the gusset was accompanied by a sudden drop in lateral load as in the test of Specimen No. 1.

### Observations for Specimen No. 3

As in the previous tests, slip of the splices occurred in Specimen No. 3 with the earliest occurring at a lateral load of 80 kips. The load vs. deflection plot is shown in Fig. 10. The initial slope of load vs. deflection plot was 240 kip/in. The lateral load reached a peak value of 122 kips and then dropped suddenly to approximately 83 kips. The lateral load continued to drop without attaining a stable value as the frame was displaced further. At the maximum load obtained, the lateral frame displacement was 1.00 in., the load in the compression diagonal was 139 kips, and the load in the tension diagonal was 125 kips. No buckling of the compression gusset plate was observed. However, noticeable distortion of the tension gusset and a widening of the gap between the upper diagonal and gusset were noted. Removal of the splice Ts revealed that the tension gusset had torn along the lower bolt line and there were obvious signs of yielding.

### Material Properties

After testing of all the specimens, coupons were cut from each of the gusset plates and tension tests were performed in accordance with ASTM Standard A370.<sup>1</sup> The coupons from the gusset were obtained from material which had experienced little or no stress during the tests. The average results from six tensile tests are:

$$F_y = 46.7 \text{ ksi}$$

$$F_u = 73.5 \text{ ksi}$$

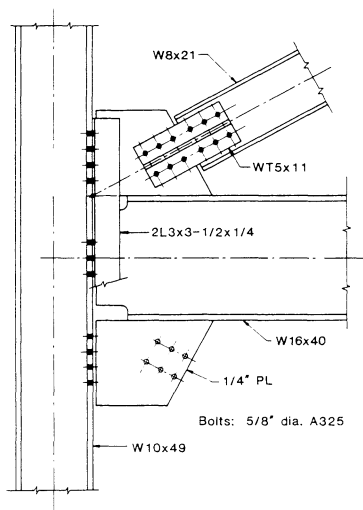


Fig. 6. Specimen No. 2.

These values will be used in calculations of nominal resistance for the failure modes observed.

## COMPARISONS BETWEEN TESTS

### Concentric vs. Eccentric Bracing Connection

The connection with eccentric brace loading and compact gusset plate (Specimen No. 2) had a lateral load capacity of 123 kips as compared to the connection without eccentric loading (Specimen No. 1) which had a capacity of 107 kips. This is a 15% increase in buckling load capacity. The stiffness of Specimen No. 1 was 288 kip/in. which was greater than that of Specimen No. 2 which was 258 kip/in.

The moment diagrams for Specimens Nos. 1 and 2 are shown in Figs. 11 and 12, respectively. The moments in these figures correspond to a lateral load of 70 kips. The moments in the diagonals are not included because they are negligible in comparison with the moments in the column and beam. Two observations can be made regarding the moments in the framing members. The first is that the beam carries a significant moment (the beam was assumed for analysis purposes to be pinned). This is because the gusset plates produce a nearly fixed condition for the beam-to-column connection. The second observation concerns the moment produced by the force in the diagonal bracing members acting eccentric to the intersection of the framing member centroidal axes. It was found that this moment was distributed to the beam and column roughly in proportion to the ratio of moment of inertia to length ( $I/L$ ) of the members framing into the connection.

### Strong-Axis vs. Weak-Axis Column Connection

While the ultimate lateral load for Specimen No. 2 was practically the same as that for Specimen No. 3, the failure mode of the gusset was different for the two tests. The gusset attaching the compression diagonal buckled in Specimen

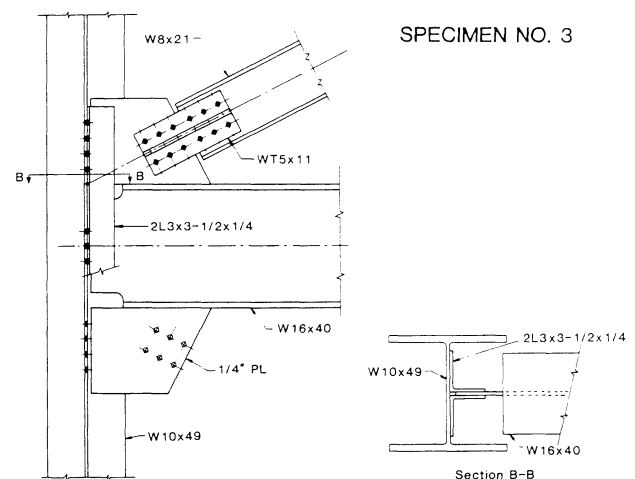


Fig. 7. Specimen No. 3.

No. 2 while the gusset attaching the tension diagonal tore in Specimen No. 3. The tearing of the gusset in Specimen No. 3 was likely a result of the rotation allowed by the connection due to the flexibility of the column web which increased the load in the tension diagonal. The loads in the compression diagonals were 138 kips and 139 kips for Specimen Nos. 2 and 3, respectively, while the loads in the tension diagonals were 113 kips and 125 kips, respectively. The stiffness of Specimen No. 2 was 258 kips/in. compared with 240 kip/in. for Specimen No. 3.

The moments corresponding to a lateral load of 70 kips for Specimen No. 3 are shown in Fig. 13. As can be seen, the moment produced by frame action as well as the moment produced by the eccentricity of the bracing loads was carried almost exclusively by the beam. This suggests that the thin column web was not sufficiently stiff to mobilize the column flanges in resisting the moments.

### COMPARISON BETWEEN COMPUTED CAPACITIES AND EXPERIMENTAL CAPACITIES

A comparison is made between the nominal capacities computed using various design approaches and the experimental results reported herein. Only the capacities corresponding to the following failure modes, which produced the four lowest capacities, are compared:

- capacity of gusset-to-column connection
- yielding of the Whitmore section
- buckling of the gusset plate
- tearout of the gusset plate

The capacities presented are expressed in terms of the force in the diagonal member.

#### Capacity of Gusset-to-Column Connection

The bolt capacity in the clip angles-to-column connection was computed using the procedure described in the AISI

LRFD Specification.<sup>8</sup> The maximum allowable bolt tension was found to be 0.92 kips/bolt using a yield stress for the clip angles of 36 ksi. Two methods were used to compute, on the basis of this value, the nominal capacity of the gusseted connection in terms of the force in the diagonal bracing member. Both methods are described in *Engineering for Steel Construction*<sup>3</sup> and are referred to as Case I and Case II. The only difference between the two methods is the way in which the location of the neutral axis is computed. Case I produced a nominal capacity of 44.9 kips while Case II, the more conservative approach, produced a nominal capacity of 31.3 kips. A reasonable value to use for comparison purposes is the average of these two values, or 38 kips.

Compared to measured forces in the diagonal bracing members of 116 kips and 138 kips for Specimen Nos. 1 and 2, respectively, which failed by gusset buckling, and 125 kips for Specimen No. 3 which failed by gusset tearing, the nominal capacity of the gusset-to-column connection of 38 kips is extremely conservative. This finding is further reinforced by the lack of visible signs of distress or prying of the clip angles, even after extensive buckling of the gusset. One possible explanation for this lack of agreement is that forces on the gusset-to-column attachment resulting from frame action tended to oppose the forces resulting from the diagonal bracing. That is, the top brace was in tension which produced tension on the upper gusset-to-column connection. However, the beam-to-column connection, being very rigid, produced compression in the upper gusset at the gusset-to-column attachment as frame action contributed to resisting the applied lateral load. These two forces tended to cancel.

#### Yielding of the Whitmore Section

Application of the Whitmore criterion involves constructing what will be referred to here as the "Whitmore section" ( $l_w$  in Fig. 14). This section is defined as the length of the line

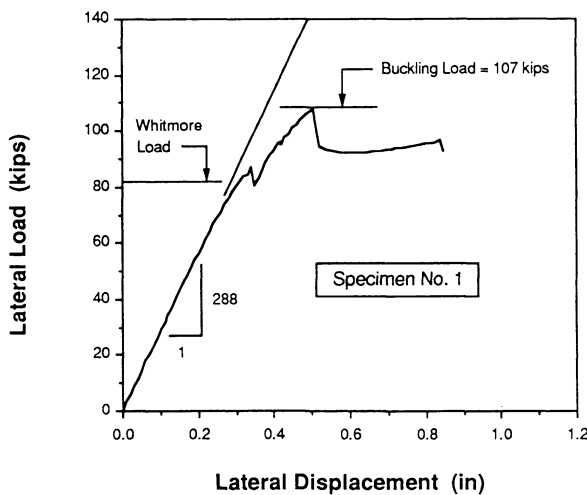


Fig. 8. Load vs. displacement for Specimen No. 1.

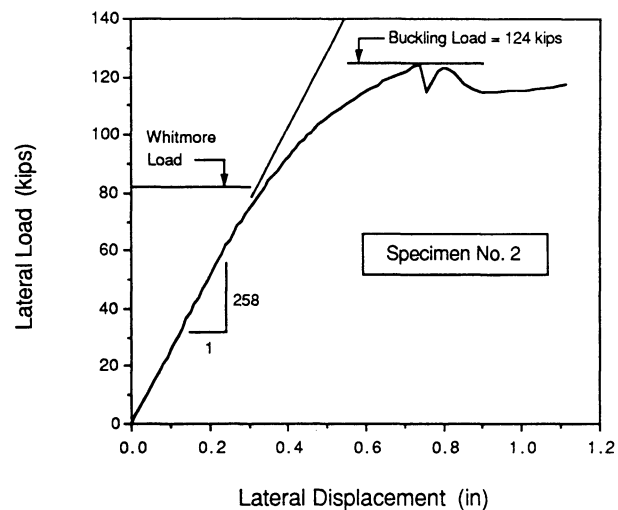


Fig. 9. Load vs. displacement for Specimen No. 2.

passing through the last row of fasteners and extending to the intersection of the lines drawn from the first fasteners at a 30° angle from the line of the fasteners. Yielding occurs when the direct stress on the Whitmore section exceeds the tension yield stress of the gusset plate material. This yield criterion may be expressed as,

$$R_n = A_w F_y$$

where

- $R_n$  = nominal gusset capacity
- $A_w = \ell_w t$
- $F_y$  = yield stress of gusset plate
- $\ell_w$  = Whitmore section (see Fig. 14)
- $t$  = gusset plate thickness

The nominal Whitmore yield capacity, computed using the measured yield stress of the gusset, is found to be 89 kips. The lateral load on the subassembly corresponding to the nominal Whitmore yield capacity in the more heavily loaded diagonal brace, or Whitmore load, is approximately 82 kips for all three tests. From Figs. 8, 9 and 10, it is observed that the Whitmore load is well above the load at which yielding first occurred as evidenced by departure from linearity of the load vs. deflection curve. Thus, the Whitmore criterion appears to be a good indicator of general yielding of the gusset plate.

### Buckling of the Gusset Plate

The gusset plate buckling capacity is computed on the basis of the procedure described in *Engineering for Steel Construction*.<sup>3</sup> The buckling strength is based on the stress on the Whitmore section acting on a unit strip of the gusset plate treated as a column. A unit column strip is assumed to have

a length equal to the average of  $\ell_1$ ,  $\ell_2$  and  $\ell_3$  (see Fig. 15). Thus, from the AISC LRFD Specification,<sup>8</sup> Sect. E2, upon substitution of nomenclature (the nominal capacity  $R_n$  is substituted for the nominal axial strength  $P_n$  and the area of the Whitmore section  $A_w$  is substituted for the gross area of the member  $A_g$ ) we have,

$$R_n = A_w (0.658^{\lambda_c^2}) F_y \quad \text{for } \lambda_c \leq 1.5$$

where

- $\lambda_c = \frac{K\ell}{r\pi} \sqrt{\frac{F_y}{E}}$
- $A_w$  = area of the Whitmore section =  $\ell_w t$
- $K$  = effective length of member
- $\ell$  = unbraced length of member =  $\ell_{avg} = (\ell_1 + \ell_2 + \ell_3)/3$
- $r$  = radius of gyration about plane of buckling =  $t/\sqrt{12}$
- $E$  = modulus of elasticity

An effective length factor of  $K = 0.5$  is used rather than the suggested value of  $K = 0.65$ , because it is believed that the “fixed-fixed” boundary conditions are closely approximated. In addition, the measured average yield stress of the gusset plate is used to calculate the nominal buckling capacity. Also, the “strength ratio” is calculated where the strength ratio is equal to the experimental capacity divided by the computed nominal capacity. The strength ratio is a measure of how well the design equation (nominal capacity) predicts the observed behavior. A strength ratio less than 1.0 means that the capacity is over-predicted (unconservative) and a strength ratio greater than 1.0 means that the capacity is under-predicted (conservative).

The strength ratios for Specimens No. 1 and No. 2, which failed by gusset buckling, are contained in Table 2. For Specimen No. 1,  $\ell_{avg} = 8.08$  in. and the resulting nominal buck-

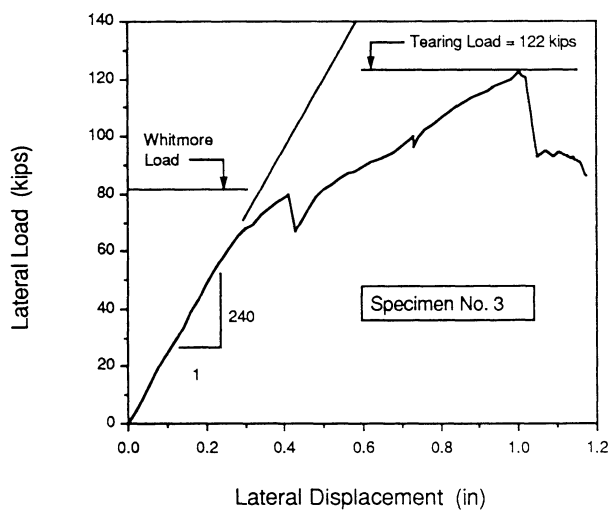


Fig. 10. Load vs. displacement for Specimen No. 3.

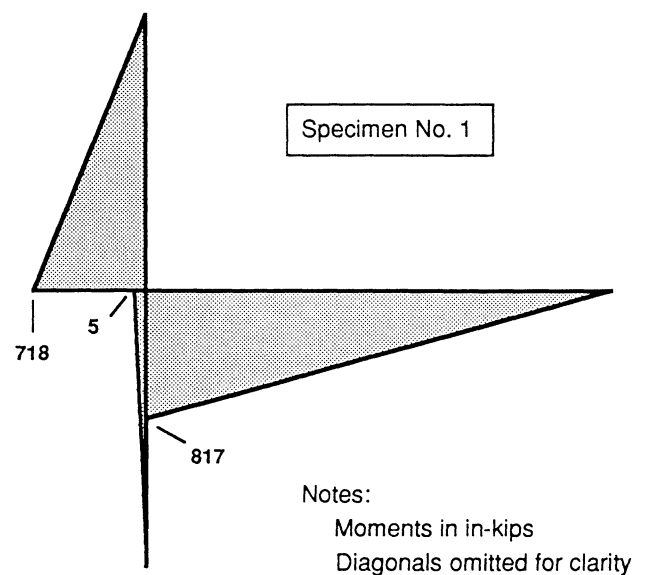


Fig. 11. Moment diagram for Specimen No. 1.

| Specimen No. | Nominal Buckling Capacity (kip) | Measured Ultimate Capacity (kip) | Strength Ratio |
|--------------|---------------------------------|----------------------------------|----------------|
| 1            | 71.8                            | 116                              | 1.6            |
| 2            | 80.1                            | 138                              | 1.7            |

ling capacity is found to be 71.8 kips and the strength ratio is 1.6. For Specimen No. 2,  $\ell_{avg} = 5.64$  in giving a nominal buckling capacity of 80.1 kips and a strength ratio of 1.7.

### Tearout of the Gusset Plate

Specimen No. 3 failed by tearout of the gusset plate. In this section, experimental results for Specimen No. 3 will be compared with several methods for computing tearout capacity. The parameters used in the following equations are illustrated in Fig. 14. Results are summarized in Table 3.

**Hardash and Bjorhovde**—The design of a gusset plate based on the block shear concept is a recent development. Block shear failure became a concern when the allowable bearing stress for bolted connections was increased from  $1.35 F_y$  to  $1.50 F_u$ . Hardash and Bjorhovde<sup>6</sup> conducted a series of tests at the University of Arizona to determine the ultimate capacity of a tension connection. Based on their work and on results of similar studies conducted at the University of Alberta and the University of Illinois, Hardash and Bjorhovde proposed the following equation for the block shear capacity:

$$R_n = F_u A_{nt} + 1.15 F_{eff} \ell t$$

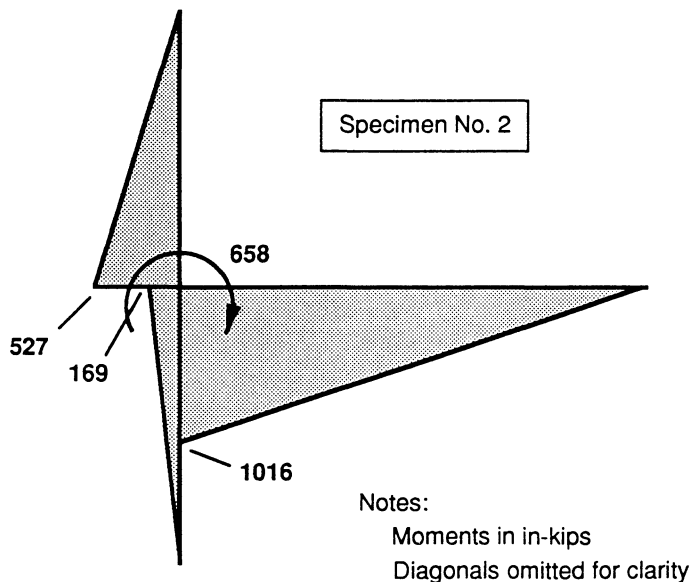


Fig. 12. Moment diagram for Specimen No. 2.

where

$$A_{nt} = \text{net area subjected to tension} = (s - D)t \text{ (see Fig. 14)}$$

$$F_{eff} = (1 - C_\ell) F_y + C_\ell F_u$$

$$C_\ell = 0.95 - 0.047\ell$$

$\ell$  = connection length

$F_u$  = tensile strength of gusset

**Williams and Richard**—Another equation for determining the ultimate block shear load was presented by Williams and Richard.<sup>12</sup> Note that Williams and Richard's definition of connection length does not include the edge distance. Hence, their definition for net area subjected to shear is denoted by an asterisk, “\*”. For connections with fewer than six bolts in a row, the following equation is reported:

$$R_n = 0.58 F_y A_{ns}^* + F_u A_{gt}$$

where

$$A_{ns}^* = \text{net area subjected to shear} = 2(\ell - a - 2D)t \text{ (see Fig. 14)}$$

For connections with more than six bolts per row, Williams and Richard recommend that the gross area subjected to shear,  $A_{gs}^*$ , rather than the net area subjected to shear be used in the above equation, where they define  $A_{gs}^* = 2(\ell - a)t$ .

**AISC Load and Resistance Factor Design**—The nominal block shear strength by the AISC Load and Resistance Factor Design<sup>8</sup> approach is the *larger* value of  $R_n$  given by the following two equations:

$$R_n = 0.6 F_y A_{gs} + F_u A_{nt}$$

or

$$R_n = 0.6 F_u A_{ns} + F_y A_{gt}$$

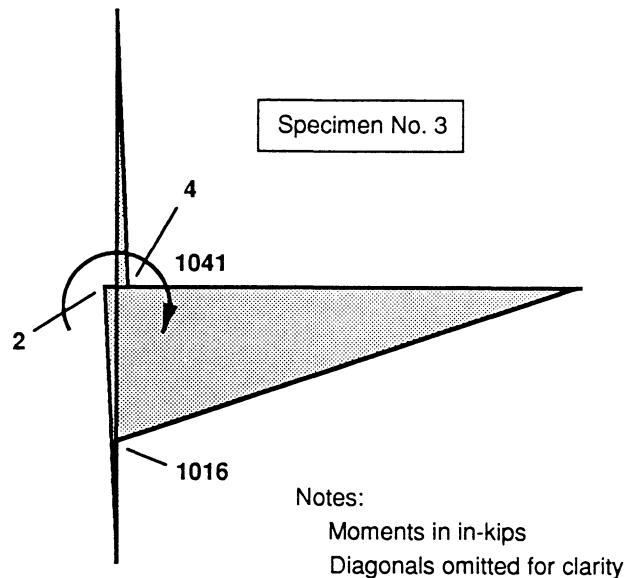


Fig. 13. Moment diagram for Specimen No. 3.

| Method                             | Nominal Tearout Capacity (kip) | Measured Ultimate Capacity (kip) | Strength Ratio |
|------------------------------------|--------------------------------|----------------------------------|----------------|
| Hardash and Bjorhovde <sup>6</sup> | 137                            | 125                              | 0.9            |
| Williams and Richard <sup>12</sup> | 91                             | 125                              | 1.4            |
| AISC LRFD <sup>8</sup>             | 111                            | 125                              | 1.1            |

where

$A_{gs}$  = gross area subjected to shear =  $2\ell t$  (see Fig. 14)

$A_{ns}$  = net area subjected to shear =  $2(\ell - 2\frac{1}{2}D)t$

$A_{gt}$  = gross area subjected to tension =  $st$

Note that the LRFD Specification (Sect. B2) recommends that the nominal bolt hole diameter  $D$  be increased by  $\frac{1}{16}$  in. for computing the net area in tension  $A_{nt}$ .

The three methods for computing nominal tearout capacity are seen to give maximum diagonal brace loads ranging from 91 to 137 kips as shown in Table 3. These results translate to strength ratios which range from 0.9 to 1.4. The nominal tearout capacity as predicted by the AISC LRFD<sup>8</sup> approach is seen to give satisfactory results.

### SUMMARY AND CONCLUSION

#### Summary

The behavior of three diagonally braced steel subassemblies was studied experimentally. The parameters which were varied included the gusset geometry and column orientation. Specimen No. 1 was a strong-axis column connection in which the axes of the bracing members, beam and column

all intersected at a single point. This connection was referred to as having no eccentricity. Specimen No. 2 was a strong-axis column connection in which the axes of the bracing members did not coincide with the intersection of the beam and column axes. Because forces in the diagonal bracing introduce a moment into the connection, this configuration was referred to as eccentric. Specimen No. 3 was similar to Specimen No. 2 except that the column was oriented so that its weak axis acted in the plane of the bracing. The specimens were loaded to failure in their plane and load-deformation, as well as strain data, were recorded.

The failure mode for both strong-axis column connections (Specimen Nos. 1 and 2) was buckling of the gusset connecting the diagonal bracing member in compression. The weak-axis column connection (Specimen No. 3) failed by tearing of the gusset plate connecting the diagonal bracing member in tension. Specimen No. 2, with a compact gusset, proved to be a stronger connection than Specimen No. 1. Although the mode of failure differed for Specimen No. 2 and Specimen No. 3, the failure load for the two specimens was essentially equal. The moment introduced by the eccentricity in the bracing was distributed to the beam and column in the strong-axis column connection. This moment was carried almost entirely by the beam in the weak-axis connection due to the flexibility in the web connection.

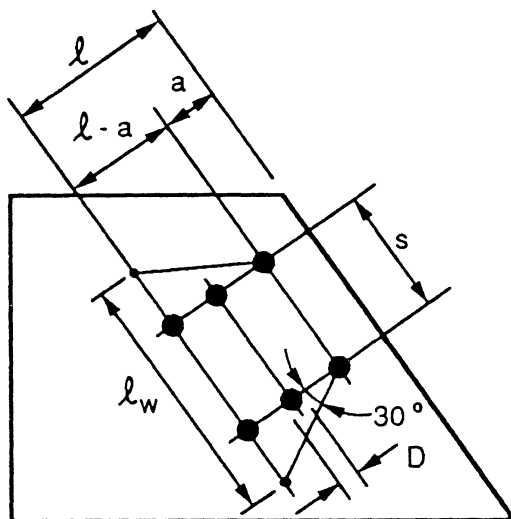


Fig. 14. Nomenclature for tearout equations.

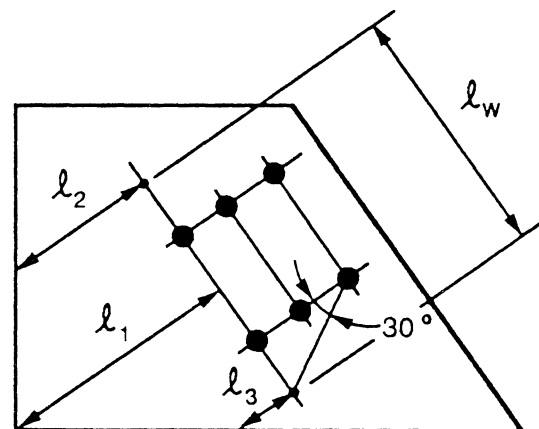


Fig. 15. Nomenclature for gusset buckling equation.

The connections were designed in accordance with AISC using Richard's guidelines<sup>9</sup> for distributing the bracing loads to the main framing members. In general, all three subassemblages performed well. The failure loads for all three specimens were found to be above the capacities predicted using a variety of methods. Generally, comparisons of nominal capacities with the experimental values resulted in a strength ratio in excess of 1.0; that is, the design equations were in most cases conservative. The current method of computing gusset buckling capacity appears to predict a capacity of 60% to 70% of that obtained experimentally. The capacity of the clip angles was computed to be very low using the AISC Specification, yet no distress in the clip angles was observed. One possible explanation for this is that frame action introduced a moment which loaded the gusset-to-column bolts in the opposite sense to the load produced by the bracing.

### Conclusions

The following conclusions were drawn from the results of the NIST tests:

- Connections designed using the AISC LRFD Specification<sup>8</sup> and Richard's equations<sup>9</sup> for distributing the bracing loads to the framing members had a strength ratio in excess of 1.0.
- Failure of the clip angles, as predicted per the AISC LRFD Specification<sup>8</sup> and Reference 3, was not observed and nominal capacity based on this criterion produced an unrealistically low connection capacity.
- The current method of computing gusset buckling,<sup>3</sup> using a value of  $K = 0.5$ , appears to be conservative predicting a buckling capacity of approximately 65% of the capacity obtained experimentally.
- Gusset tearout capacity is predicted very closely using the AISC LRFD approach.
- The weak-axis column connection had a capacity which was comparable to that of the strong-axis column connection suggesting that web stiffening is not required for strength.
- The moment introduced by the eccentric bracing in the weak-axis column connection (Specimen No. 3) was resisted mainly by the beam.
- The eccentric connection which had a compact gusset plate (Specimen No. 2) had a higher buckling capacity than the concentric connection with the long gusset plate.
- The moment introduced by the eccentricity of the bracing (Specimen No. 2) was resisted by the beam and column roughly in proportion to  $I/L$ .
- The moments introduced into the beam and column by the eccentric brace loading were considerably less than the moments resulting from the rigid beam-to-column connection.

### ACKNOWLEDGMENTS

The author would like to express his appreciation to the many individuals and organizations that contributed to this project. First, the assistance of the AISC Task Group on Bracing for Heavy Construction is gratefully acknowledged. The author would especially like to express his thanks to Dr. William Thornton for providing the specimen design capacity calculations and to Prof. Ralph Richard for providing the computer simulations of the NIST specimens. Finally, the author would like to thank Cives Steel Company for providing and fabricating the test specimens and fixtures.

### REFERENCES

1. ASTM A 370, Standard Methods and Definitions for Mechanical Testing of Steel Products, *1984 Annual of ASTM Standards—Section 1, Iron and Steel Products*, American Society for Testing and Materials, Philadelphia, 1984.
2. Bjorhovde, Reidar, and Chakrabarti, S. K., "Tests of Full-Size Gusset Plate Connections," *Journal of Structural Engineering*, ASCE, Vol. 111, No. 3, March 1985, pp. 667–684.
3. *Engineering for Steel Construction*, American Institute of Steel Construction, Chicago, 1984.
4. Fung, John Y., and Richard, Ralph M., "Inelastic Finite Element Analysis of Gusset Plates," The University of Arizona, 1987.
5. Gross, John L., and Cheok, Geraldine, "Experimental Study of Gusseted Connections for Laterally Braced Steel Buildings, NISTIR 88-3849, U.S. Department of Commerce, National Institute of Standards and Technology, November 1988.
6. Hardash, Steve G., and Bjorhovde, Reidar, "New Design Criteria for Gusset Plates in Tension," *Engineering Journal*, Vol. 22, No. 2, AISC, 1985, pp. 77–94.
7. Kulak, Geoffrey L., Fisher, John W. and Struik, John H. A., *Guide to Design Criteria for Bolted and Riveted Joints*, Second Edition, John Wiley & Sons, New York, 1987.
8. *Manual of Steel Construction—Load & Resistance Factor Design*, First Edition, American Institute of Steel Construction, Inc., Chicago, 1986.
9. Richard, Ralph, "Analysis of Large Bracing Connection Designs for Heavy Construction," Proceedings of the 1986 National Engineering Conference, AISC, Nashville, June 12–14, 1986.
10. Vasarhelyi, Desi D., "Tests of Gusset Plate Models," *Journal of the Structural Division*, ASCE, Vol. 97, No. ST2, February 1971, pp. 665–678.
11. Whitmore, R. E., "Experimental Investigation of Stresses in Gusset Plates," *Bulletin No. 16*, Engineering Experiment Station, University of Tennessee, May 1952.
12. Williams, George C., and Richard, Ralph M., "Steel Connection Designs Based on Inelastic Finite Element Analysis," The University of Arizona, 1986.