Dr. Ralph M. Richard

Analysis of Large Bracing Connection Designs for Heavy Construction

Author

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Richard has lectured and authored papers and reports on the design of steel connections and lateral bracing systems, including steel-plate shear walls. He has also published papers and lectured on the design of support systems for vibration and shock isolation of high-resolution optical components and electronic systems.

His major consulting work has been with General Dynamics, Welton Becket Associate Architects, the Ballistics Research Laboratory, City Investing Corporation, Mountain States Engineering, the Duval Corporation, IBM, the Ministry of Communications of Saudi Arabia and the National Optical Astronomy Observatories.

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Dr. Richard received the 1986 AISC TR. Higgins Lectureship Award for his research and papers on single-plate framing connections.

Summary

Gusset-plate connections for diagonal bracing systems usually comprise a combination of shop and field attachments. Typically, these connectors consist of weldments, bolted/welded, double-framing angles, and/or single plates which are shop-welded and field-bolted. Since gusset-plate connections are made either to the column flange or web and to the beam flange for diagonal bracing, a wide variation in frame stiffnesses and connection strengths result.

Using an inelastic finite element program which incorporates force-deformation relationships for welds, bolts and double-framing angles determined from laboratory tests, analyses of numerous braced-frame designs were made for diagonal bracing configurations. These studies have shown that diagonal gusset plate designs result in rigid (AISC Type 1) beam-to-column connections, even if the beam flanges are not connected to the column flange or web. Thus, the braced frame is actually a rigid-braced frame with nominal moments in both beams and columns due to rigidity of the gusset connection.

These analytical studies show a significant reduction in gusset plate size may result in not using the working point concept and designing the plate on the basis of the required gusset-to-brace connection length, even though a moment results from the brace force eccentricity. Moreover, this design results in a compact gusset plate which has a significantly higher buckling strength.

Equations for the design of the gusset plate and the connectors comprising bolts, weldments, single plates, and double framing angles are proposed.

INTRODUCTION

Gusset plate connections for diagonal bracing systems usually comprise a combination of shop and field attachments. Typically, these connectors consist of weldments, bolted/welded double framing angles, and/or single plates which are shop-welded and field-bolted. Since gusset plate connections are made either to the column flange or web and to the beam flange for diagonal bracing, and a large range in the bay width to story height ratio is possible, a wide variation in frame stiffnesses and connection strengths result.

Early gusset plate research was directed towards determining the stress distributions in truss connections. Rust (1, 2), Perna (3), Sandel (4), and Vasarhelyi (5) performed photoelastic stress analyses on model gusset plates. Whitmore (6, 7) developed a criterion, based on an experimental test, to estimate the maximum normal stress in a gusset plate. Additional experimental work was conducted on truss gussets by Irvan (8), Hardin (9), Chesson and Munse (10, 11), Birkemoe, Eubanks, and Munse (12), and Vasarhelyi (5). Recently, Richard (16) proposed applying the block shear concept to gusset plate design.

Previous to this study, gusset plate analyses did not include the nonlinear behavior of the fasteners; furthermore, the frame to which the gusset is attached was excluded from the model. Davis (13) simulated Whitmore's test using an elastic finite element model. Additional elastic analyses were made by Desai (14) and Vasarhlyi (5). Struik (15) made a nonlinear finite element analysis of Whitmore's gusset model which verified the location of the maximum gusset plate normal stresses. Detailed analyses have not been made to determine the fastener force distributions in diagonal bracing connections.

Using an inelastic finite element program which incorporates force-deformation relationships for welds, bolts, and double framing angles determined from laboratory tests as illustrated for double framing angles in Figure 1, the analyses of numerous braced frame designs were made for diagonal bracing configurations as shown in Figure 2. These studies have shown that diagonal gusset plate designs result in rigid (AISC Type I) beam to column connections, even if the beam flanges are not connected to the column flange or web. Thus, the "braced" frame is actually a "rigid-braced" frame with nominal moments in both the beams and the columns due to the rigidity of the gusset connection.

These analytical studies shown that significant reduction in gusset plate size may result by not using the working point concept and designing the plate on the basis of the required gusset-to-brace connection length, even though a moment results from the brace force eccentricity. Moreover, this eccentric design results in a compact gusset plate which has a significantly higher buckling strength.

STRUCTURAL MODELS

To determine the gusset plate stresses and the gusset-to-frame fastener force distributions, 60 different finite element models were generated and analyzed. To study the effect of various parameters, the following connection and frame properties were varied:

- bracing configurations: single, drag-through and double
- brace angles: 30, 45, and 60 degrees
- brace loads: working, yield and ultimate
- column-to-beam stiffness ratios: from 0.03 to 1.73
- gusset-to-frame fastener modes
- plate dimensions: from 24" x 46" to 58" x 24"
- brace eccentricities: from -8" to + 14"

Shown in Figure 3 is a typical subassembly model of the single gusset configuration illustrated in Figure 2 where only one gusset exists at the beam-column-brace intersection. The finite element model for this subassembly is shown in Figure 4. The model shown in Figure 5 is for the case wherein alternate bays are braced and is called the drag-through configuration. The double gusset configuration is shown in Figure 6. Summarized in Table 1 are the models generated to determine the force distributions in diagonal bracing connections.

Since the presence of the gusset plate results in a rigid (AISC Type I) beam-to-column connection, a portal frame analysis model wherein inflection points occur at the mid-story height of the columns is valid. Thus, pin supports can be used to provide the boundary conditions for the subassemblies.

Bracing members were connected to the gusset plates with splice plates bolted to the gusset with A325 bolts in double shear. The block shear yield load was calculated as shown in Figure 7 by using the gross shear area along the bolt lines and the gross tensile area across the end of the bolt lines. Typically, for a double row of eight bolts with a pitch of 2.25 inches and a spacing of 5 inches, the block shear yield load for a 3/8" A36 gusset plate is

$$P_y = [(2)(8)(2.25") / \sqrt{3} + 5."] (0.375")(36 \text{ ksi}) = 348 \text{ kips}$$

To insure that the gusset plate would be subjected to brace loads approximately equal to the ultimate brace load, a lateral load approximately 50% greater than the block shear yield load was applied to the model as shown in Figure 3. This load represents the lateral loads originating in the stories above of the frame, and was applied in six increments of 30%, 10%, 10%, 10%, 20%, and 20%. A typical gusset plate load versus the frame lateral deflection is given in Figure 8.

The brace angle, as shown in Figure 3 is defined as the angle that the bracing member makes with the beam. Brace angles of 30, 45 and 60 degrees were considered.

Three column sections (a weak axis W12x65, a strong axis W12x65, and a strong axis W24x100) were used to study the effect of the column-to-beam bending stiffness ratios. The length (or story height) of all columns was kept constant at 16', resulting in beam lengths (or bay widths) of approximately 28', 16', and 9' for the 30, 45, and 60 degree brace angles respectively. Thus, the column-to-beam stiffness ratio, $(I_c / L_c / (I_b / L_b)$, varied from 0.03 to 1.73.

A number of fastener modes are available to connect the gusset plate to the framing members. These include combinations of bolts, welds, double angles, single plates (shear tabs) , and structural tees. Of these possibilities, bolted double angles are the most flexible, whereas welds are the stiffest. Therefore, to envelop all possible combinations, a bolted-welded design was compared to a welded-bolted design, as shown in Figure 9. This comparison was made using the single gusset configuration. The difference in the gusset plate stresses and fastener force distributions for these two fastening designs was not significant from a design viewpoint; therefore, a welded-welded design was used for all other bracing configurations including the eccentric connection models. To assure an even transfer of load it is recommended that the gusset always be connected along the entire length of the plate edges with a uniform spacing of the fasteners.

Listed in Table 1 are the gusset plate dimensions studied. These dimensions are required to provide sufficient space to attach the bracing member to each plate. In addition, if the brace, beam, and column axes are required to intersect at a common working point, the plate dimensions generally are increased significantly. However, if the working point requirement is eliminated, then the plate dimensions decrease and the connection becomes eccentric, as shown in Figure 10.

ANALYTICAL RESULTS

These studies shown that frame action significantly affects the gusset-to-frame fastener force distributions. Deformed finite element meshes (exaggerated) are given in Figures 11 through 13 which show that the framing members pinch the gusset plate because the angle between the column and beam is reduced; and, as a result, the beam and column load the gusset significantly. To realistically simulate the behavior of diagonal bracing connections, the frame must be incorporated in the finite element model.

As a result of the finite element analyses conducted, it was established that the gusset force distributions primarily depend on the plate aspect ratio and the brace angle.

Shown in Figures 14 through 16 are variations of the fastener force distributions at the gusset plate working, yield, and ultimate loads. As the brace load increases, the force distributions become more uniform. The nonuniformity of these fastener forces is of the same order as those of typical splice connections. Therefore, designing gusset plate connections by assuming the gusset forces are resisted uniformly by the fasteners is consistent with current professional practice. Furthermore, in the compact gusset plates, in which the minimum required plate dimensions (those needed to accommodate the brace-to-gusset connection) are used, the force distributions are more uniform than for the noncompact plates. This explains why the range of fastener forces is narrowed for the eccentric connections. As illustrated in Figure 17, the fastener forces tend to become aligned with the brace as the brace load increases.

GUSSET PLATE FASTENER FORCE DESIGN EQUATIONS

To design the gusset-to-frame fasteners, an estimate of the force distribution is required. Previous research on this subject does not exist. One major reason for this is the difficulty of experimentally determining the distribution of loads in the bolts, welds, and angles (fastener elements) in these complex connections. Current methods for predicting the fastener force distribution assume that the horizontal and vertical components of the brace load are transferred to the beam and column respectively. Therefore, fasteners are designed to resist the shear forces as shown in Figure 18.

Based on the parametric studies involving the 60 analytical models, the design equations given in Figures 19 and 20 were developed to predict the gusset-to-frame fastener force distributions.

In these equations,

R_B = force resultant on the beam

R_C = force resultant on the column

θ_B = angle of beam resultant

a = horizontal plate dimension

b = vertical plate dimension

P = brace load

θ = brace angle

P_{HB} = horizontal force component on beam

P_{VB} = vertical force component on beam

P_{HC} = horizontal force component on column

P_{VC} = vertical force component on column

These equations are based on the fastener force distributions which occur when the gusset plate is subjected to the block shear yield load.

These gusset force distributions design equations were developed from the data shown in Figures 21 and 22. The brace load distributed to the beam (R_B) depends only on the magnitude of the brace load (P) and the plate aspect ratio (a/b). The orientation of the force transferred to the beam (θ_B) is a function of the brace angle (θ) and the plate aspect ration (a/b) . Once the beam force components are calculated, the column force components can be computed using the equations of equilibrium.

Plotted in Figures 23 through 26 are the analytical and design forces for the gusset plates considered in this study. The correlation between the design equation and the finite element results is excellent.

For double framing angles with a single A325 or A490 bolt in double shear, prying forces to do not control the bolt designs in the outstanding legs. Instead, the strength of the double angle connection is either the bolt double shear value or the bolt-bearing value of the bolt in the gusset plate. Therefore, if bolted double angles are used, the required number of A325 or A490 bolts can be simply determined by dividing the design equation value of R_B and R_C by the bolt shear or bearing value. The current design procedure shown in Figure 18 conservatively estimates the number of required bolts along the beam and column for most cases. The number of bolts required by the proposed design formulas given in Figures 19 and 20 are accurate to within one bolt for all but one of the gusset plates analyzed. A savings of 5% to 20% of the number of bolts required results when these design equations are used. The above observations explain why the current bolt and weld procedures for gusset plate designs are valid but generally conservative. It is noted that the proposed design method predicts *a* different distribution of forces from the current design method that is shown in Figure 18.

When designing a diagonal bracing connection, it is recommended that the entire length of the gusset edges should be fastened to the frame. The required number of bolts should be spaced uniformly to fill the complete length of the gusset. If the plate is welded to the framing member, the weld should extend along the entire length of the plate. This allows for a more even transfer of load along the fasteners from the gusset to the frame.

ADDITIONAL DESIGN CONSIDERATIONS

When the line of action of the bracing member does not go through the beam-column working point, a moment is introduced into the connection which must be resisted by the frame. If the frame is being analyzed with a computer program, a short beam link can be inserted into the frame model at each eccentric connection as shown in Figure 27. An alternative method can be used where the bracing member axis is assumed to intersect with the beam and column axes at a common point. This frame model can be analyzed to determine the primary member forces. As shown in Figure 27, a moment distribution can then be performed to determine the secondary member forces resulting from the moment caused by the eccentric connection. The frame members are then designed using the combined primary and secondary member forces.

Illustrated in Figure 28 are the von Mises stress contours for a typical gusset plate (0.375" thick) and the adjacent beam web

(0.468" thick) and column web (0.390" thick). These contours indicate that the brace load significantly contributes to the beam and column web stresses. Therefore, if the web thickness of an adjacent framing member is significantly less than the gusset plate thickness, then web stiffeners should be provided for that member.

When single plates are used instead of welds or double framing angles, the bolt diameter to plate thickness ratio, (D/t), should be greater than two (2) for A325 bolts or greater than 1.5 for A490 bolts to provide for a ductile connection (17).

ACKNOWLEDGMENT

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Table 1. Gusset Plate Dimensions and Eccentricities

Brace Angle	Column Sections*	Working Point Models (e=0) a	Working Point Models (e=0) b	Eccentric Models a	Eccentric Models b	Eccentric Models e
30°	W12 x 65 (WA)	58"	24"	30"	24"	14"
30°	W12 x 65 (SA)	52"	24"	30"	24"	11"
30°	W24 x 100 (SA)	46"	24"	30"	24"	8"
45°	W12 x 65 (WA)	39"	27"	27"	27"	8"
45°	W12 x 65 (SA)	33"	27"	27"	27"	4"
45°	W24 x 100 (SA)	27"	27"	27"	27"	0"
60°	W12 x 65 (WA)	27"	30"	24"	30"	3"
60°	W12 x 65 (SA)	24"	35"	24"	30"	-3"
60°	W24 x 100 (SA)	24"	46"	24"	30"	-8"

*WA - Weak axis column
SA - Strong axis column

a = horizontal dimension of gusset plate
b = vertical dimension of gusset plate
e = connection eccentricity

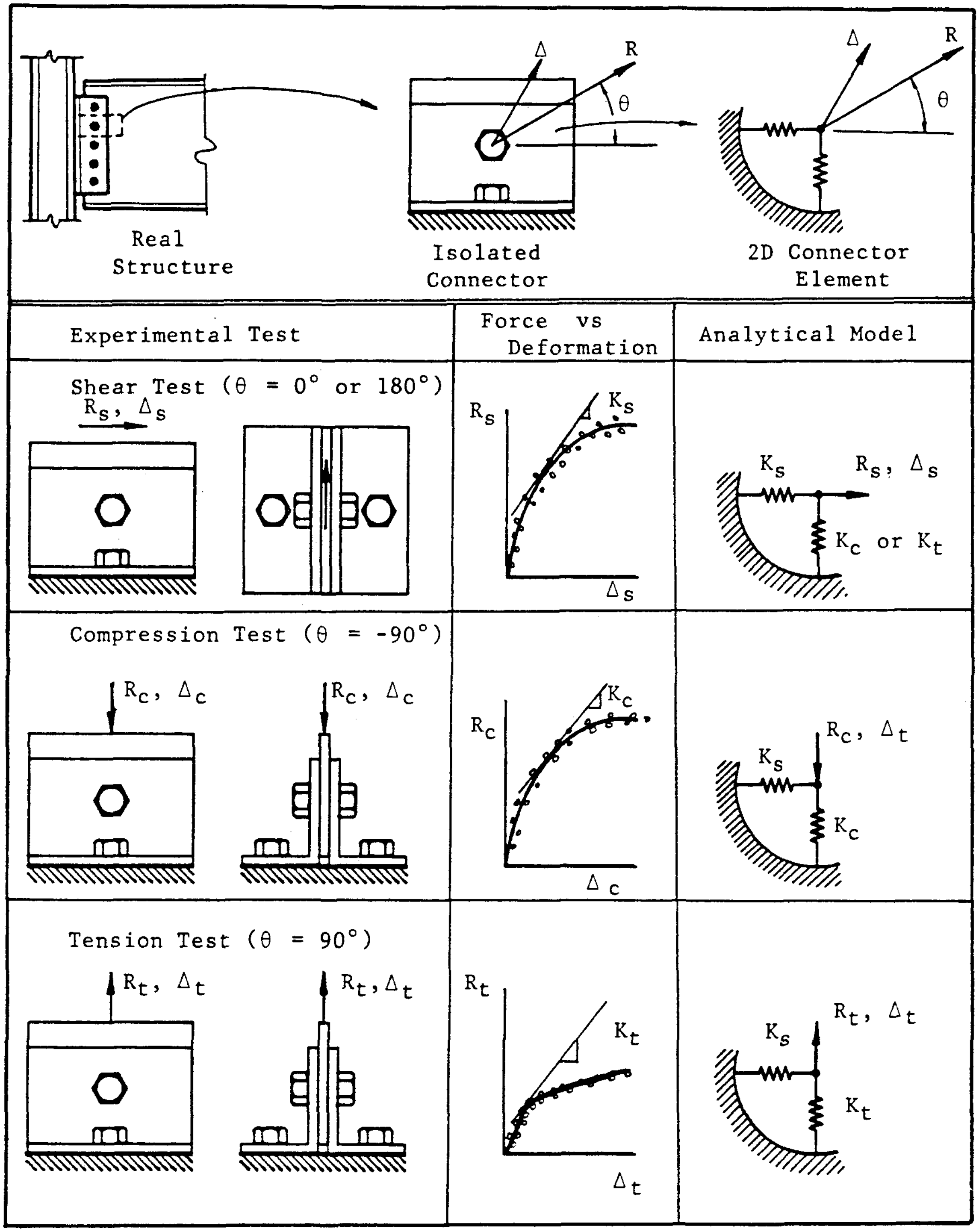


Figure 1. Formulation of Double Framing Angle Element

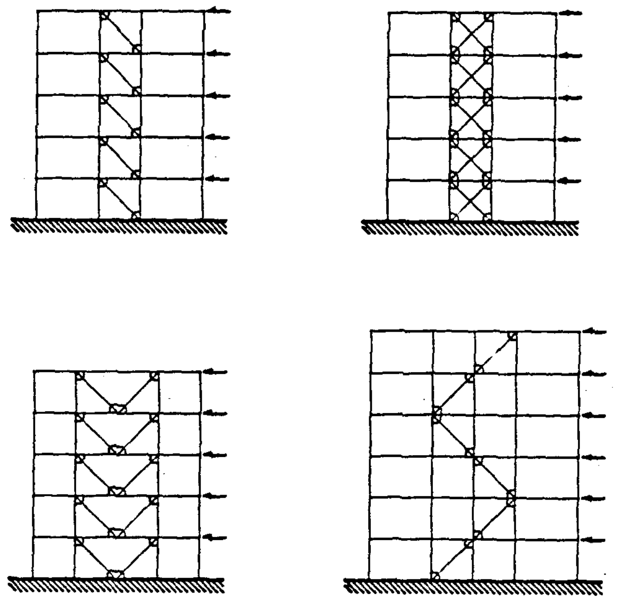


Figure 2. Braced Frames

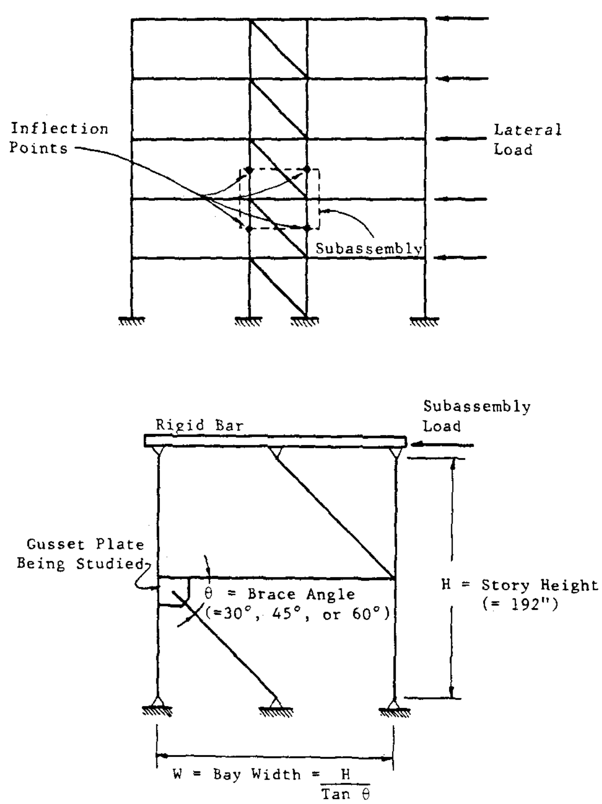


Figure 3. Isolation of Subassembly for Analysis

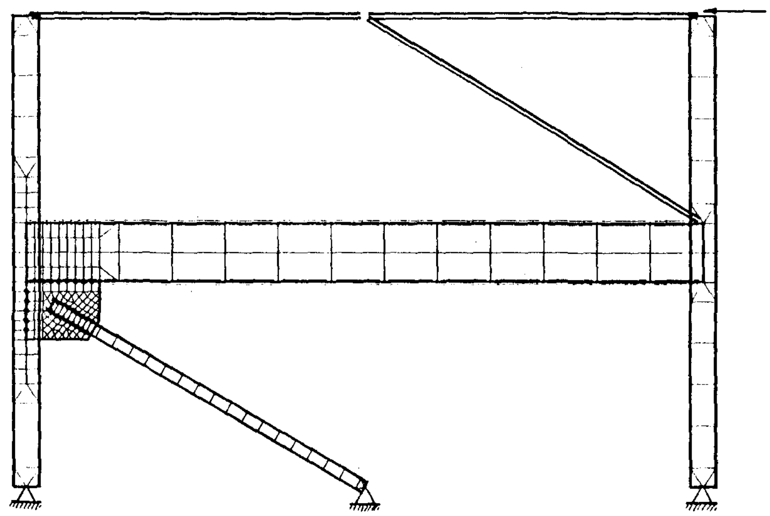


Figure 4. 30° Single Gusset Eccentric Model
(Weak Axis W12)

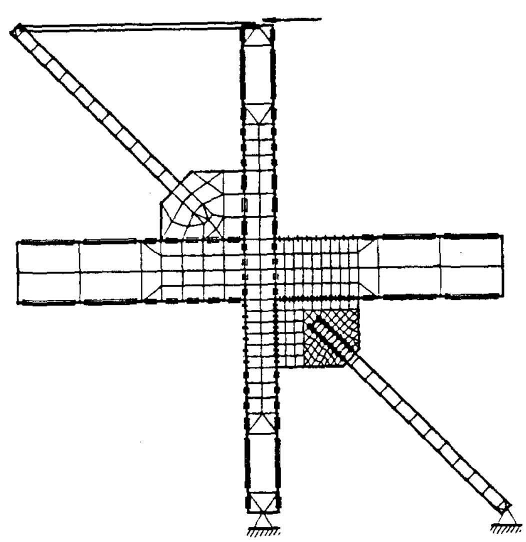


Figure 5. 45° Drag-Through Gusset Model
(Strong Axis W12)

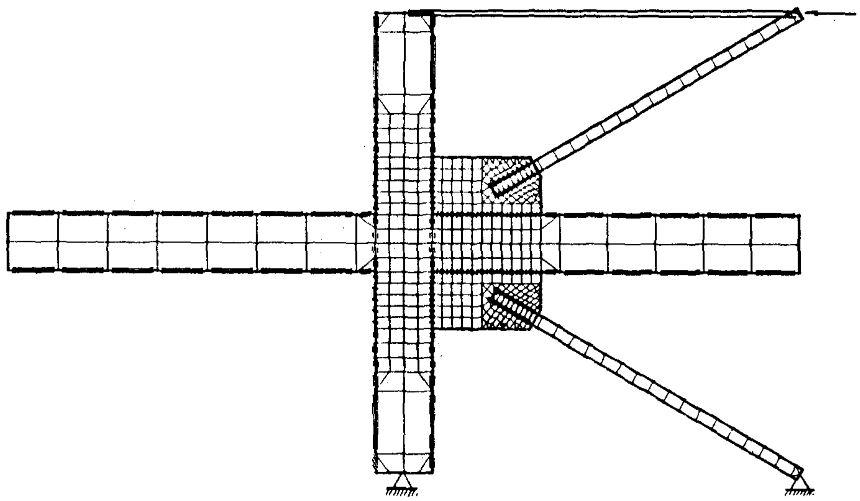


Figure 6. 30° Double Gusset Model (Strong Axis W24)

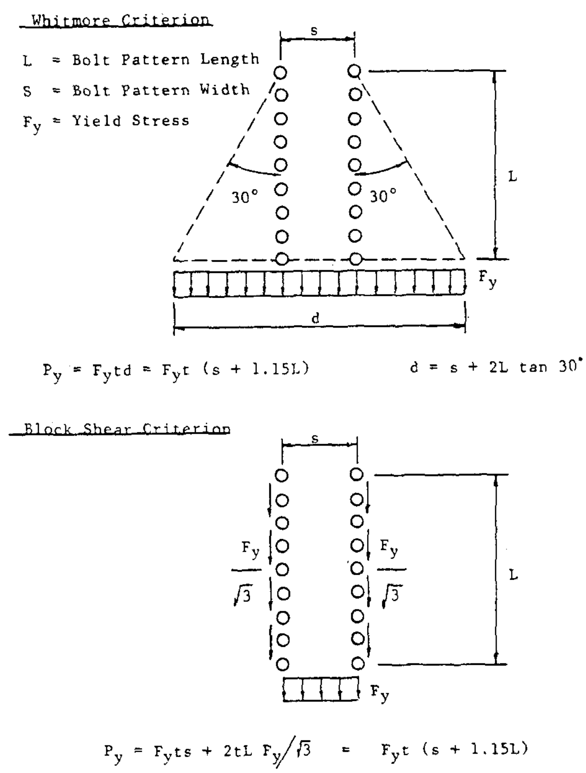


Figure 7. Comparison of Block Shear and Whitmore Criteria

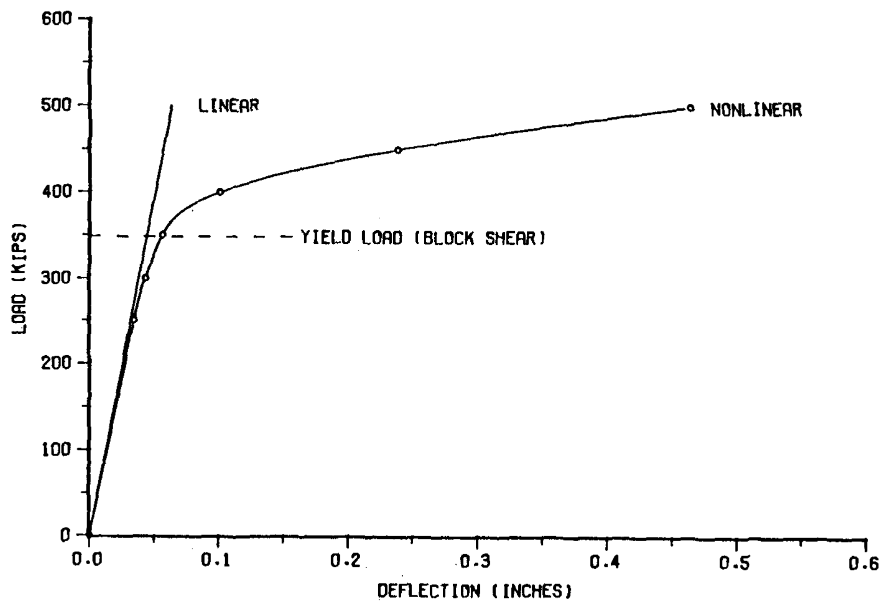


Figure 8. Gusset Plate Load vs. Deformation Response

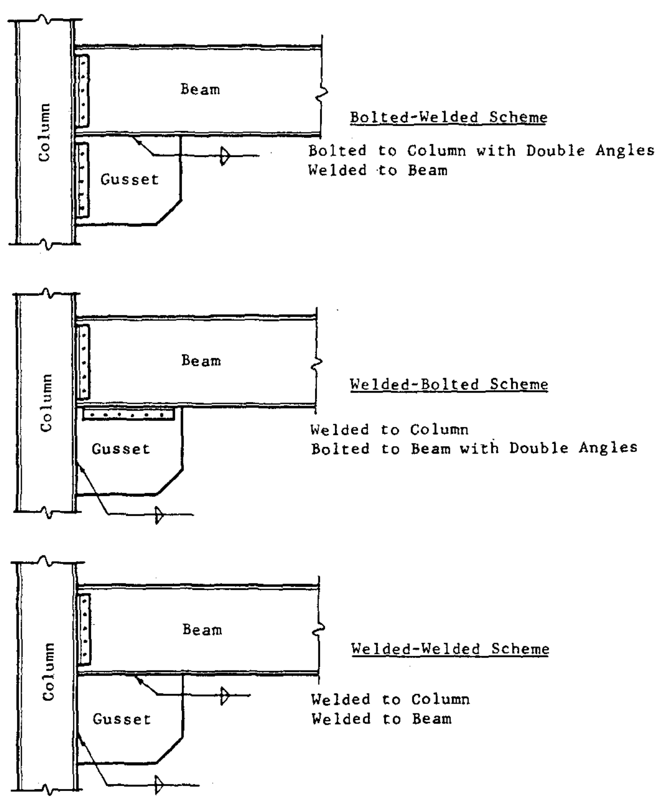
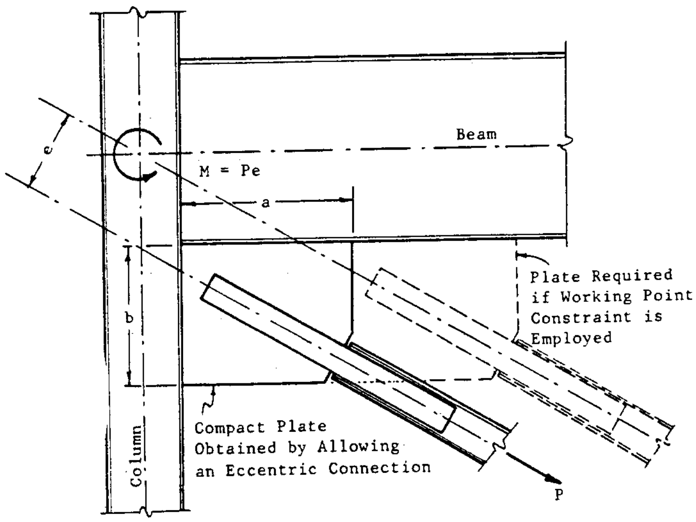


Figure 9. Gusset-to-Frame Connection Schemes



P = Brace Load

e = Eccentricity

M = Moment Caused by Eccentricity

Figure 10. Eccentric Connection

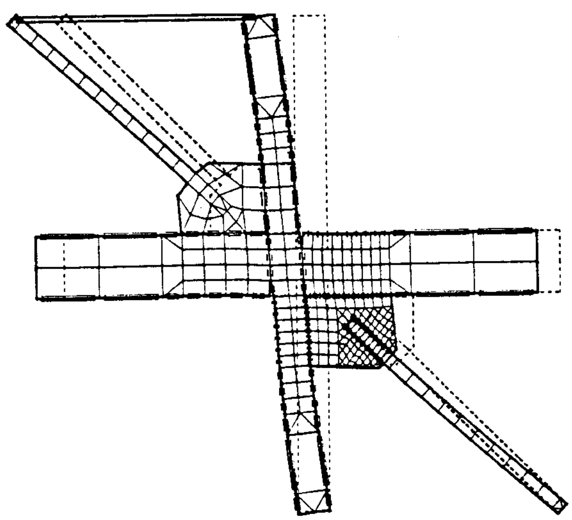


Figure 11. Deformed Mesh for 45° Drag-Through Gusset Model (Strong Axis W12)

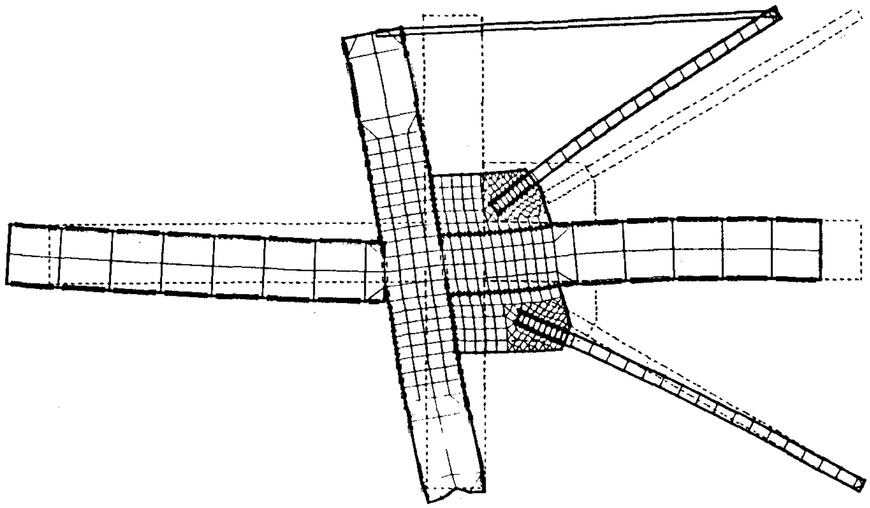


Figure 12. Deformed Mesh for 30° Double Gusset Model
(Strong Axis W24)

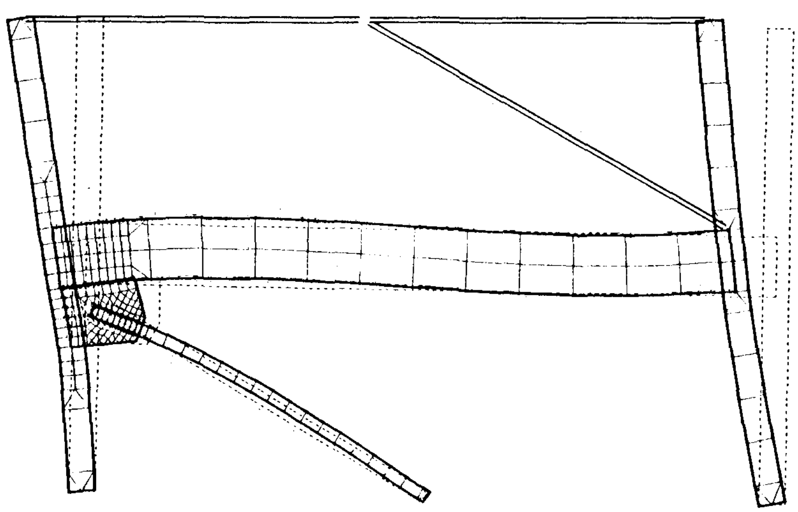


Figure 13. Deformed Mesh for 30° Single Gusset Eccentric Model
(Weak Axis W12)

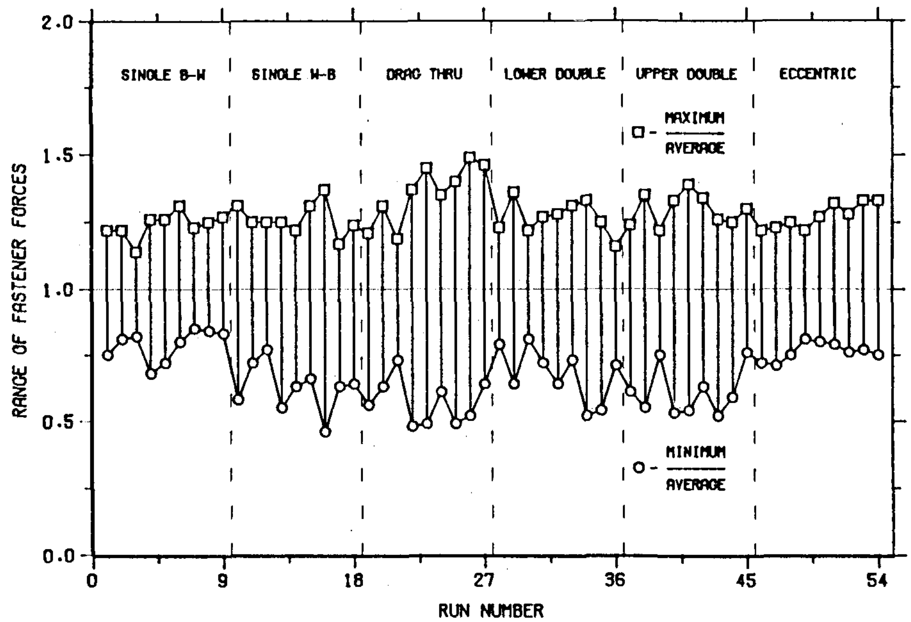


Figure 14. Fastener Force Distributions at Working Load

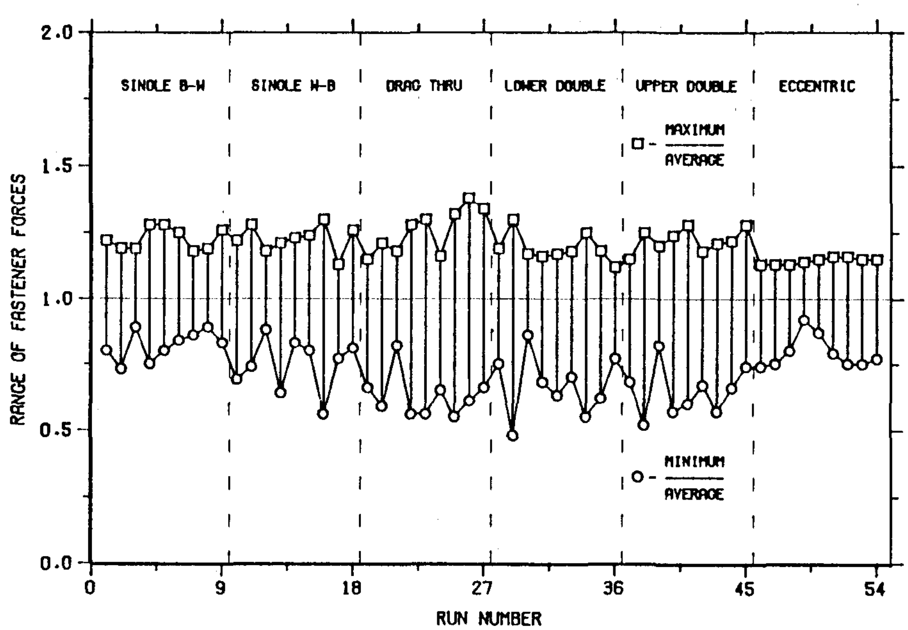


Figure 15. Fastener Force Distributions at Yield Load

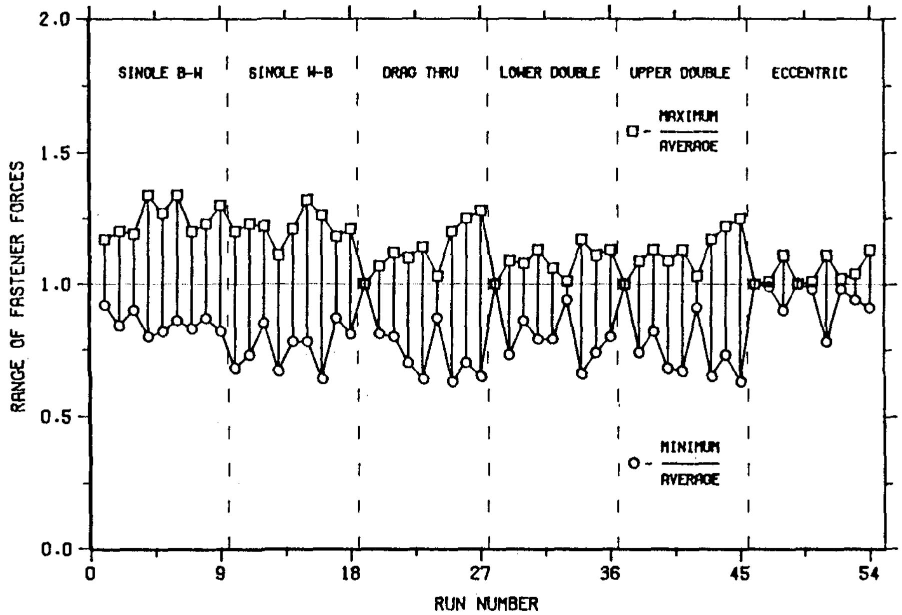


Figure 16. Fastener Force Distributions at Ultimate Load

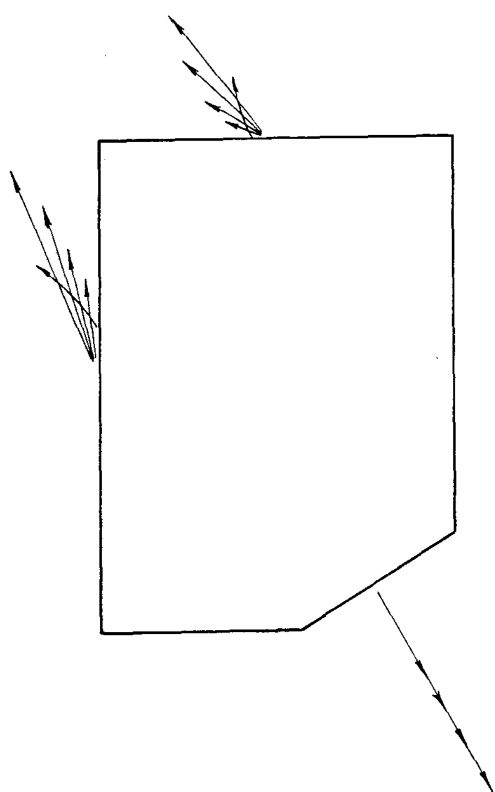


Figure 17. Alignment of Forces as Brace Load Increases

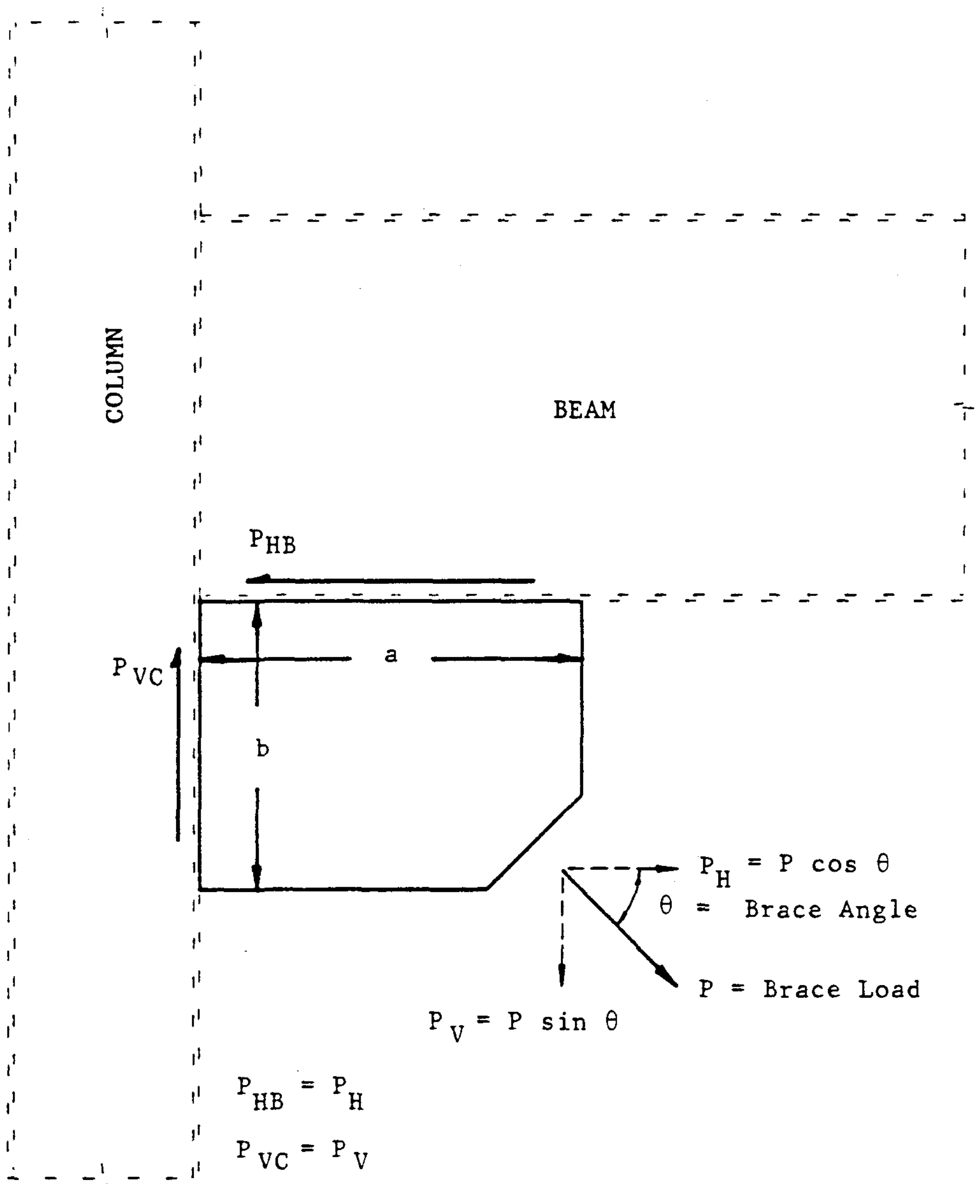
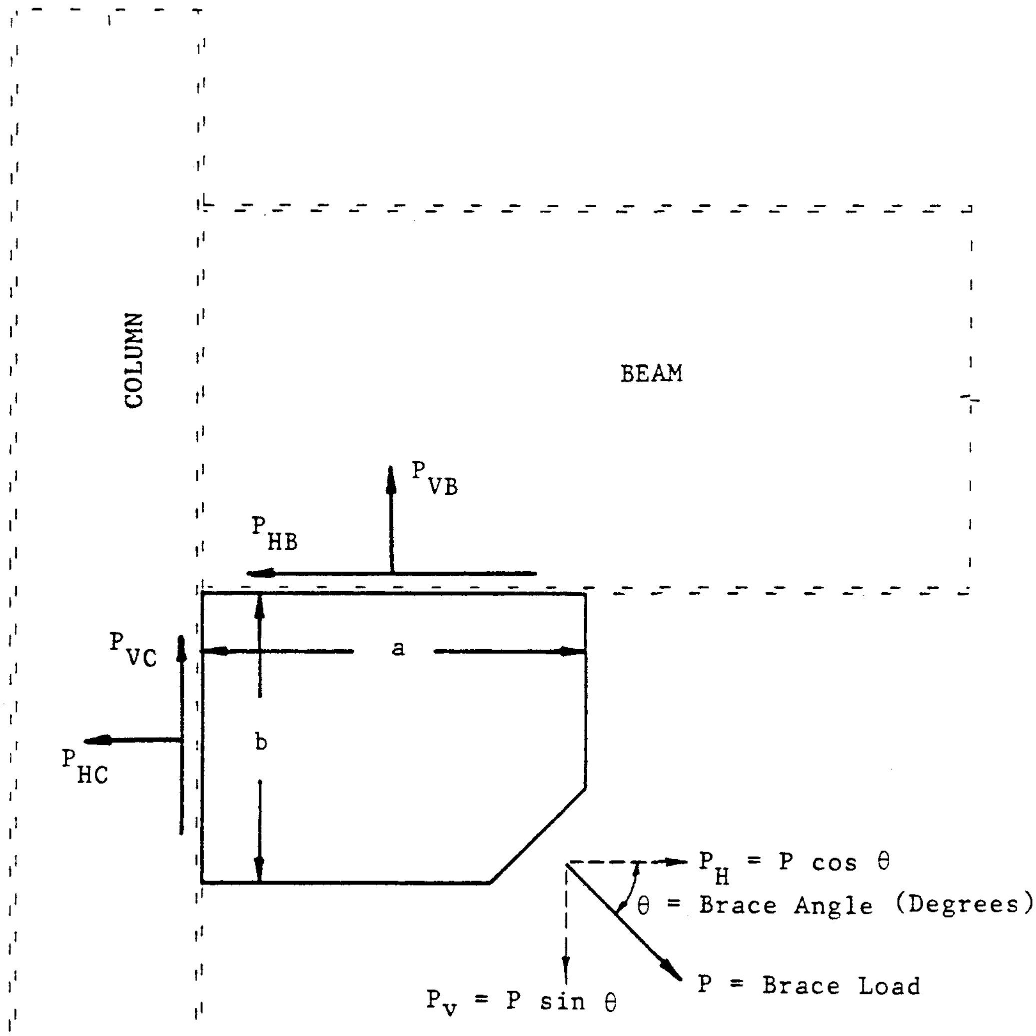


Figure 18. Current Method for Predicting Force Distributions



$$R_B = P\left(1.4\frac{a}{a+b} - 0.1\right)$$

$$\theta_B = 0.6\ \theta \quad (\theta \leq 45°)$$

$$\theta_B = 27 + \left(8.5 - 20\frac{a}{a+b}\right)(45 - \theta) \quad (\theta > 45°)$$

$$P_{HB} = R_B \cos\theta_B \qquad P_{HC} = P_H - P_{HB}$$

$$P_{VB} = R_B \sin\theta_B \qquad P_{VC} = P_V - P_{VB}$$

Figure 19. Force Components for Proposed Design Method

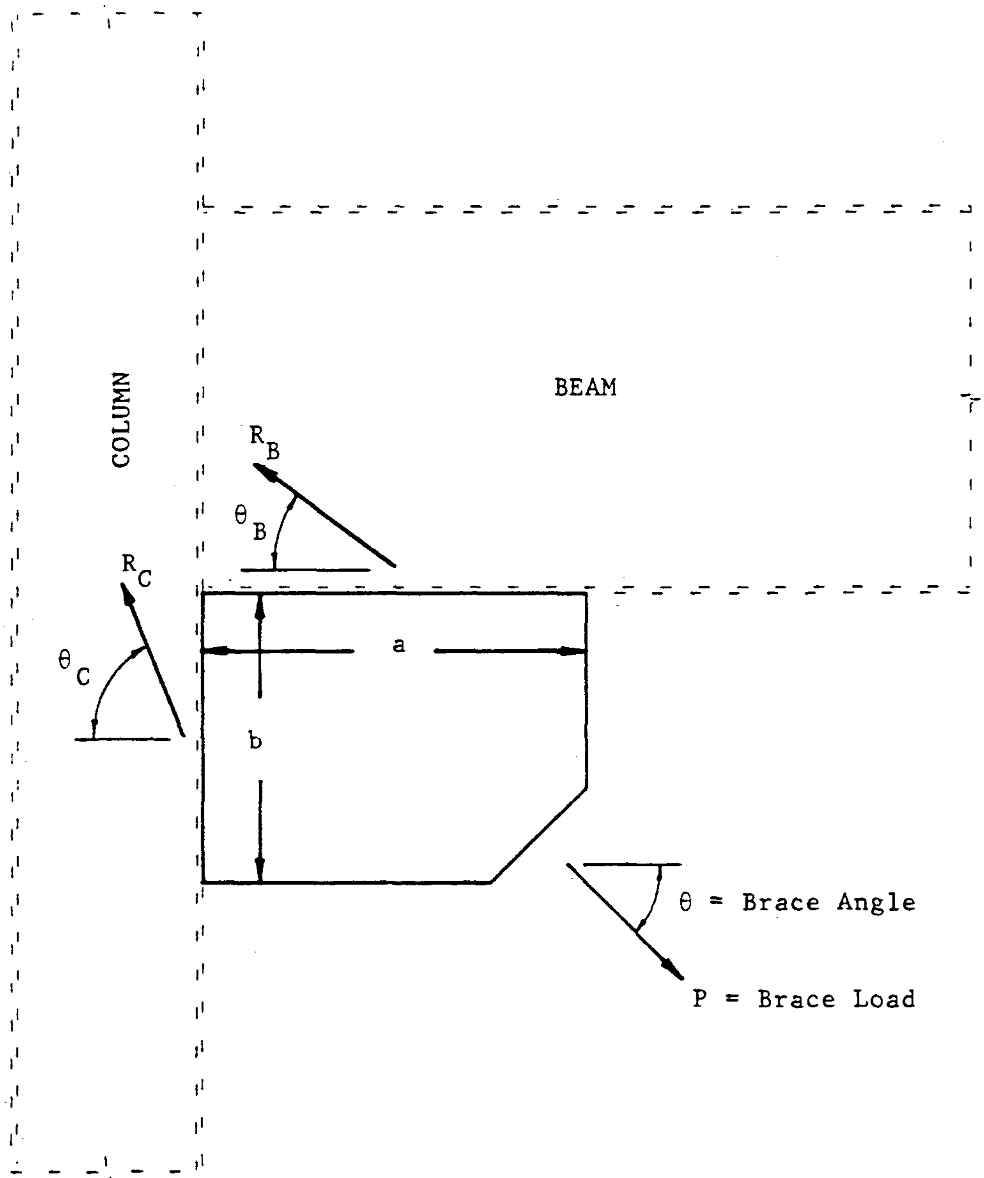


Figure 20. Force Resultants for Proposed Design Method

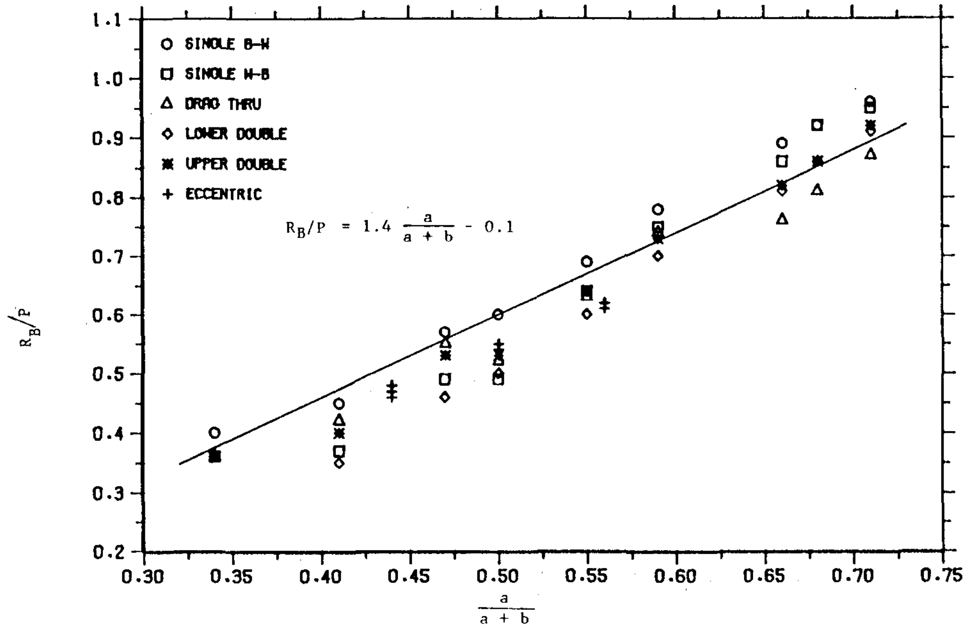


Figure 21. Origin of Design Equation for Resultant Force on Beam

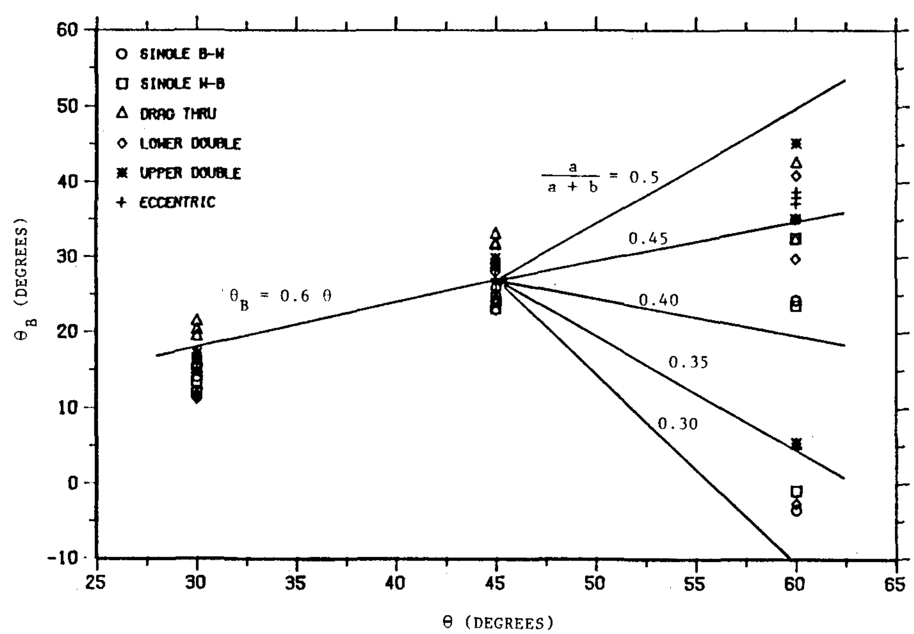


Figure 22. Origin of Design Equation for Angle of Resultant Force on Beam

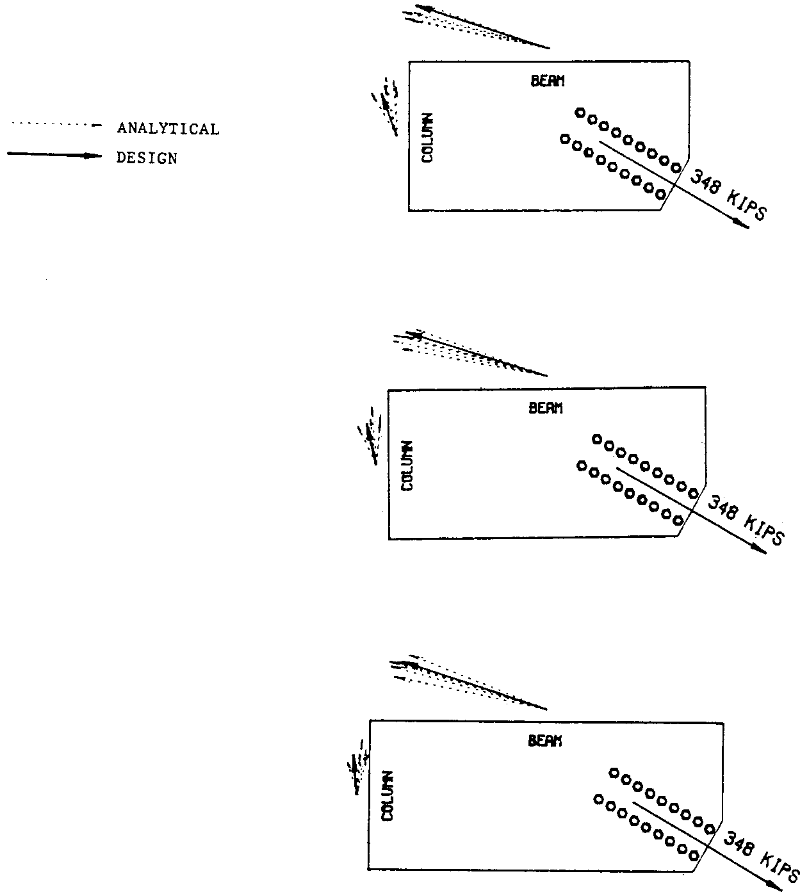


Figure 23. Analytical and Design Force Results for 30° Working Point Models

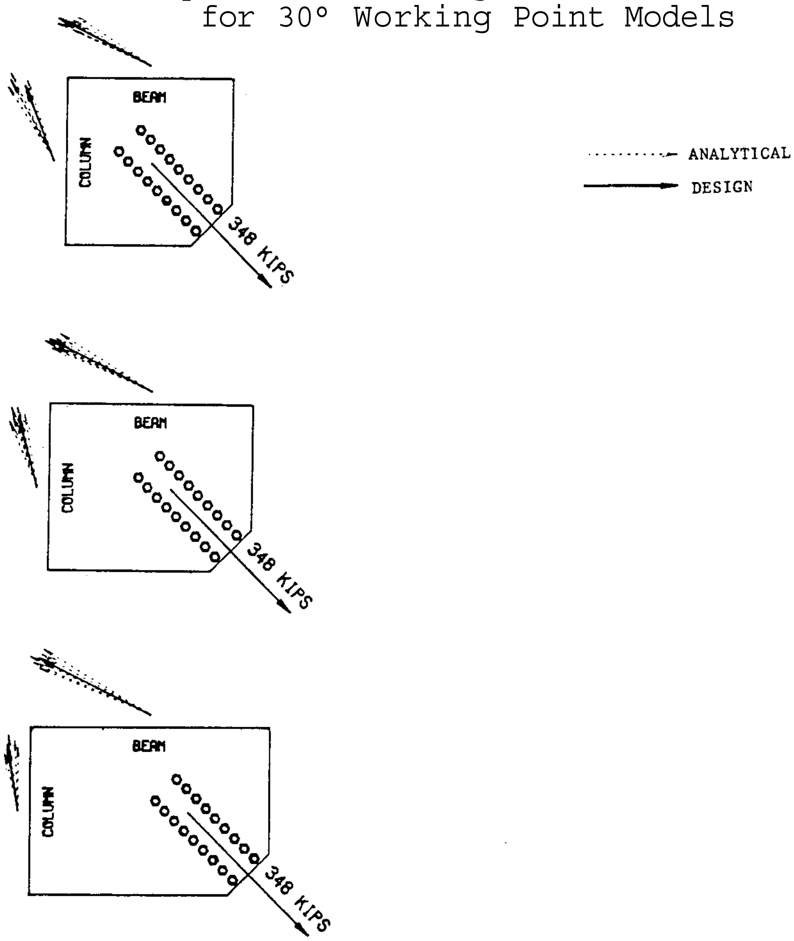


Figure 24. Analytical and Design Force Resultants for 45° Working Point Models

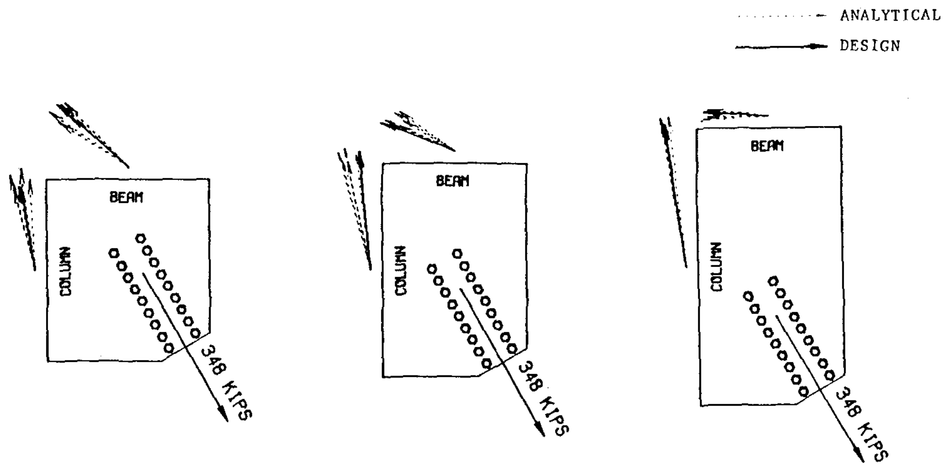


Figure 25. Analytical and Design Force Resultants for 60° Working Point Models

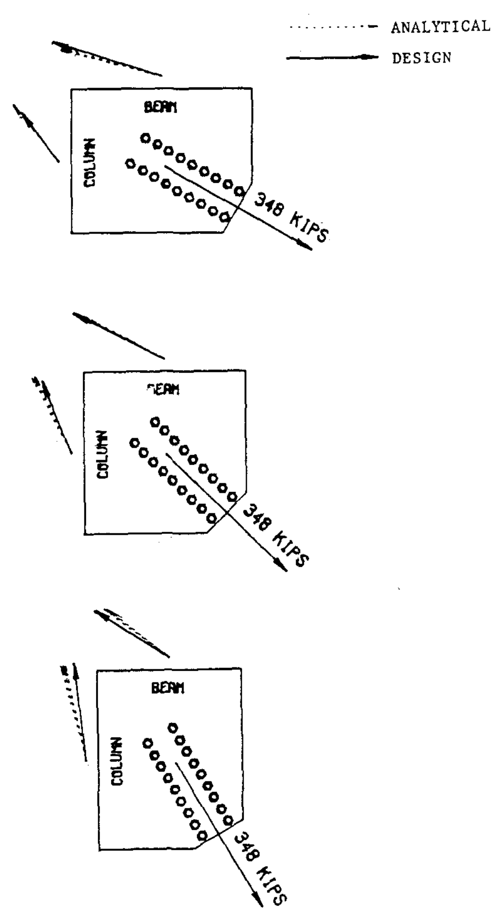


Figure 26. Analytical and Design Force Resultants for Eccentric Models

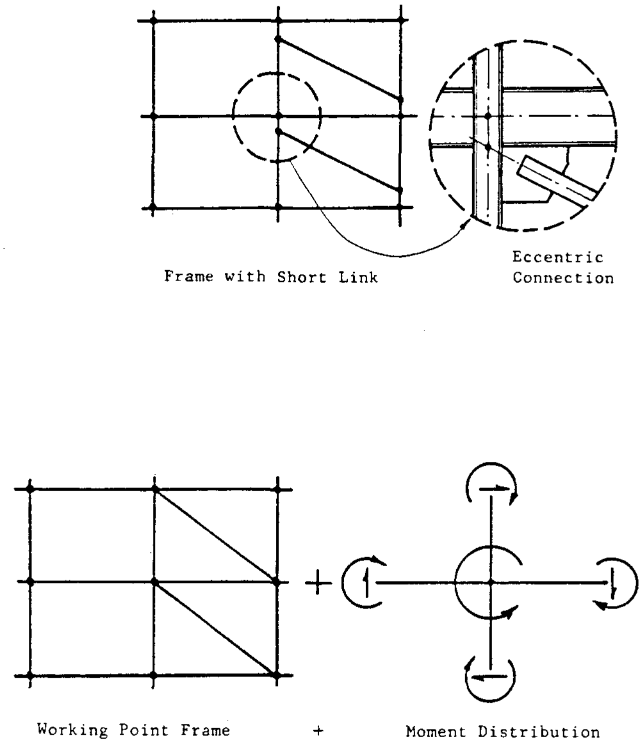


Figure 27. Design of Framing Members for Moment Caused by Connection Eccentricity

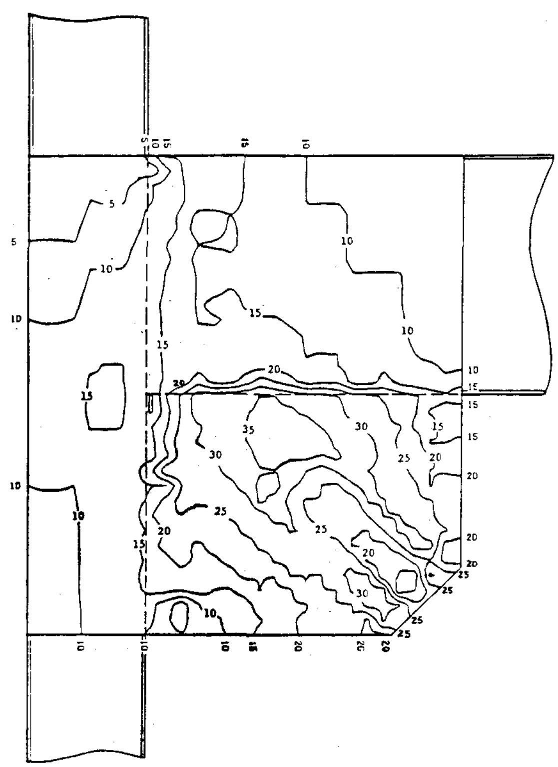


Figure 28. Typical Effective Stress Contours (ksi) in Gusset Plate, Beam Web, and Column Web