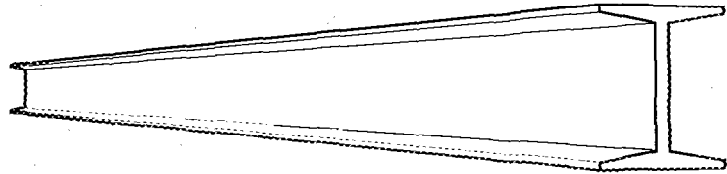


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DECEMBER 1998

Seismic Behavior and Design of Gusset Plates

by

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This report presents information and tips on seismic behavior and design of gusset plates used in steel concentrically braced frames. Gusset plates are used in steel building structures to connect the bracing members to the beams or columns. Gusset plates are also used in connections of steel trusses. In this report, first a summary of behavior of steel gusset plates is presented. This summary is based on the information obtained from laboratory tests, investigation of performance during actual earthquakes and analytical studies. After presenting the summary of behavior, a chapter is devoted to discussion of provisions in the current design codes that are relevant to seismic design of gusset plates. Then, seismic design of gusset plates is presented.

First Printing, December 1998

Figures and photos by Abolhassan Astaneh-Asl unless otherwise indicated.

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Acknowledgments

The publication of this report was made possible in part by the support of the Structural Steel Educational Council (SSEC). The author would like to thank all SSEC members for their support and comments. Particularly extensive written comments received from Rudy Hofer, James J. Putkey and Lanny Flynn of the Structural Steel Educational Council and Professor Subhash C. Goel of the University of Michigan were very valuable and appreciated. Publication of this report was made possible by financial support of the California Iron Workers Administrative Trust. The support was very valuable and sincerely appreciated.

The support provided by the American Iron and Steel Institute to the author's research on the subject, while he was a doctoral student at the University of Michigan (1979-1982) is acknowledged. The research conducted under Professors Subhash C. Goel and Robert D. Hanson's supervision, was essential in collecting and developing many technologies presented and used in this report. In addition, the later support of the University of California, Berkeley made it possible to conduct the cyclic tests of gusset plates in V-braced frames presented in this report. The tests were conducted by Albert Suen, formerly an undergraduate student at University of California at Berkeley. His efforts are acknowledged and appreciated. Many thanks are due to Ted Winneberger and other engineers of the W&W Steel Company of Oklahoma City for their continued support of author's work on the gusset plates since 1983.

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The opinions expressed in this report are those of the author and do not necessarily reflect the views of the University of California, Berkeley, the Structural Steel Educational Council or other agencies and individuals whose names appear in this report.

SEISMIC BEHAVIOR AND DESIGN OF GUSSET PLATES

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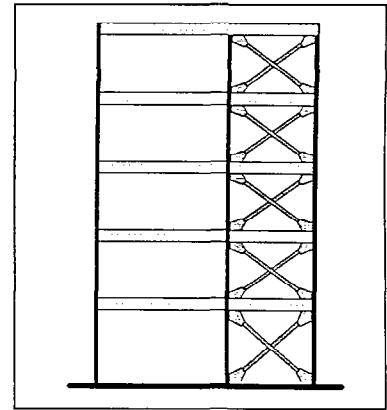
Notations

In preparing the following notations, whenever possible, the definitions are taken with kind permission of the American Institute of Steel Construction (AISC), from the "Seismic Provisions for Structural Steel Buildings" (AISC, 1997). Such definitions are identified by "(AISC, 1997)" at the end of the definition.

A	Area of cross section
A_g	Gross area. (AISC, 1997)
A_{gt}	Gross area subject to tension (in checking block shear failure)
A_{gv}	Gross area subject to shear (in checking block shear failure)
A_{gw}	Gross area of gusset plate as per Whitmore's 30-degree lines.
A_n	Net area. . (AISC, 1997)
A_{np}	Net area of plate
A_{nw}	Net area of gusset plate along Whitmore's section
A_{nv}	Net area subject to shear (in checking block shear failure)
C	Distance of extreme fiber from neutral axis
D	Outside diameter of round HSS tubes
E	Modulus of elasticity.
F_{cr}	Critical compressive stress
F_y	Specified minimum yield stress of the type of steel to be used, ksi. As used in the LRFD Specification, "yield stress" denotes either the minimum specified yield point (for those steels that have a yield point) or the specified yield strength (for those steels that do not have yield point). (AISC, 1997)
F_{ye}	Expected Yield Strength of steel to be used,(AISC, 1997)
F_u	Specified minimum tensile strength, (AISC, 1997)
I	Moment of inertia
K	Effective length factor for prismatic member. (AISC, 1997)
L	Unbraced length of compression or bracing member, (AISC, 1997)
L_{fg}	Length of free edge of gusset plate
M	Bending moment
M_p	Plastic moment capacity
N	Axial force
N_y	Capacity of cross section in yielding under axial load = $A F_y$
P_{bs}	Capacity based on block shear failure mode
P_n	Nominal axial strength of a member
P_u	Required axial strength on a member
P_{cr}	Critical axial strength on a member in compression
P_y	Nominal axial yield strength of a member, which is equal to $F_y A_g$, (AISC, 1997)
R	Seismic force reduction factor used in (ICBO, 1997)

R_y	Ratio of the Expected Yield Strength F_y to the minimum specified yield strength F_y . (AISC, 1997)
$T_{Element}$	Tension yield capacity of one element in built-up braces
V	Shear force
$V_{First\ Stitch}$	Shear force to be transferred by the first stitch in built-up braces
V_{Stitch}	Shear force to be transferred by a stitch in built-up braces
V_y	Capacity of cross section in shear yielding = $A(0.6 F_y)$
b	Width of compression element as defined in LRFD Specification (AISC, 1997)
l	unbraced length between stitches of built-up bracing members, (AISC, 1997)
r	Governing radius of gyration, (AISC, 1997)
r_i	Radius of gyration of one element
t	Thickness of plate or member elements
Ω_s	Horizontal seismic overstrength factor, (AISC, 1997)
λ	Slenderness parameter. (AISC, 1997)
λ_p	Limiting slenderness parameter for compact element. (AISC, 1997)
λ_r	Limiting slenderness parameter for non-compact element. (AISC, 1997)
ϕ	Resistance factor for yield failure modes = 0.90
ϕ_b	Resistance factor for bending=0.90 (AISC, 1997)
ϕ_c	Resistance factor for compression=0.85, (AISC, 1997)
ϕ_n	Resistance factor for fracture=0.75, (AISC, 1997)
σ	Normal stress due to bending
τ	Shear stress

1. INTRODUCTION



1.1. Introduction

Gusset plates have been used in riveted steel trusses of early bridge, buildings and industrial structures. Today, gusset plates are frequently used in steel braced frames to connect bracing members to columns and beams as shown in Figure 1.1. In addition, gusset plates are still used in steel trusses to connect truss members. Since 1960's, bolts and welds have almost totally replaced rivets in these connections. The focus of this report is on seismic behavior and design of gusset plates in steel braced frames. Most of the information presented herein, with proper judgment of design engineer, can also be applied to gusset plates in trusses and other applications. Figure 1.2 shows a number of typical gusset plate connections in braced frames.

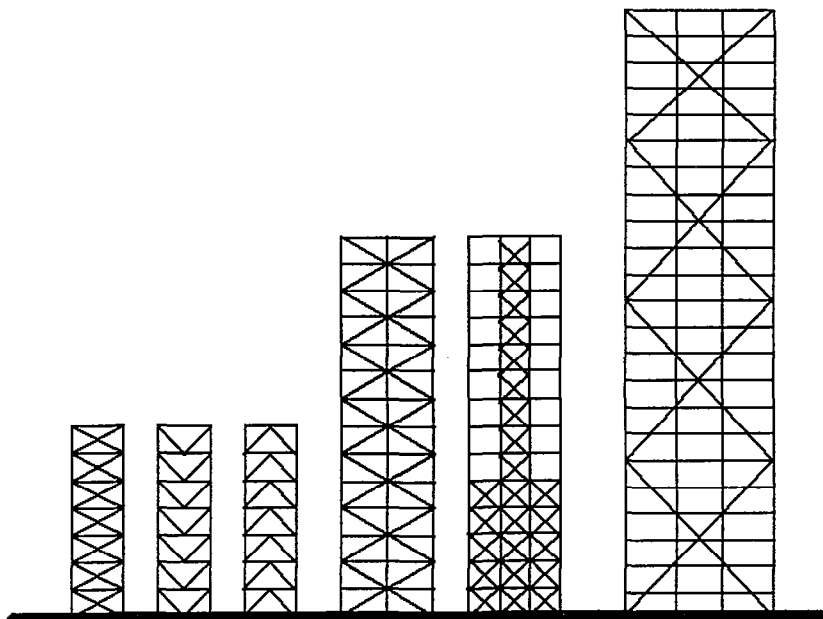
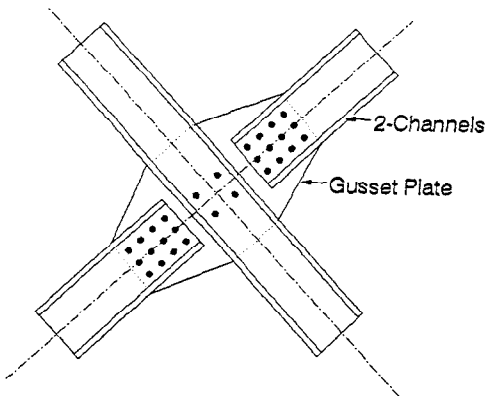
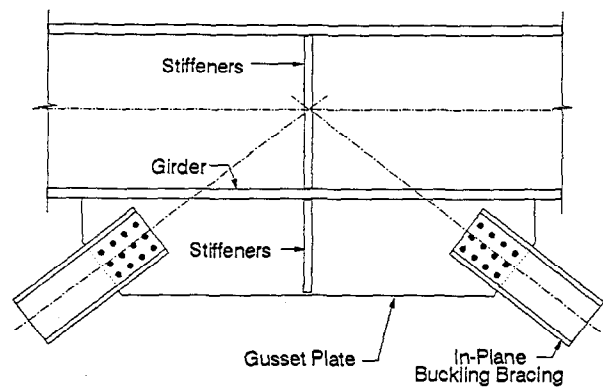
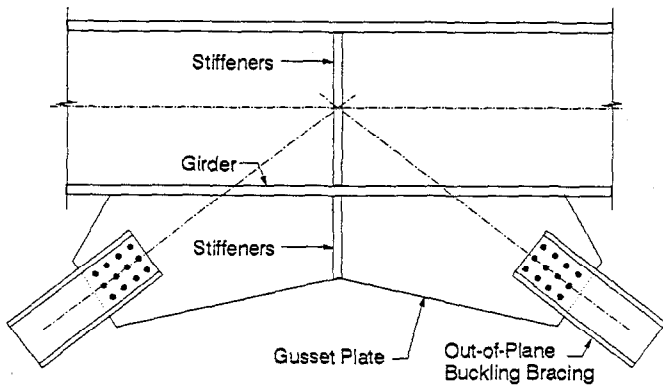
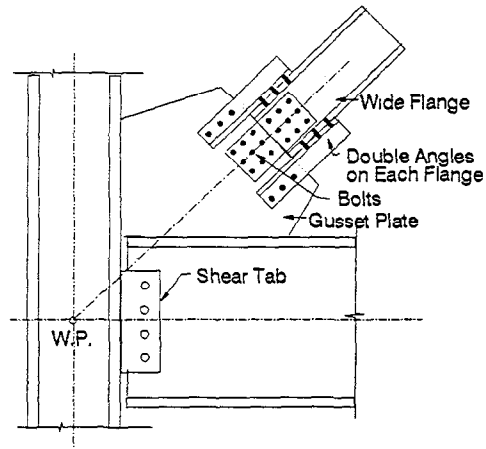
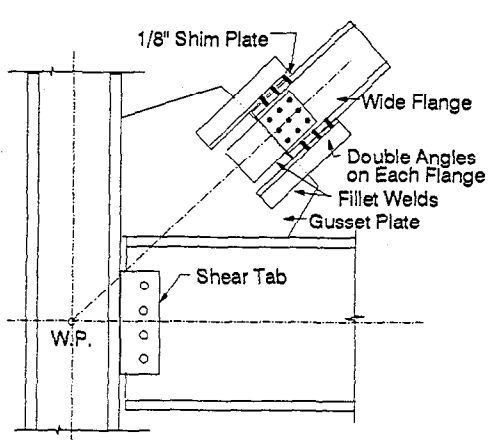
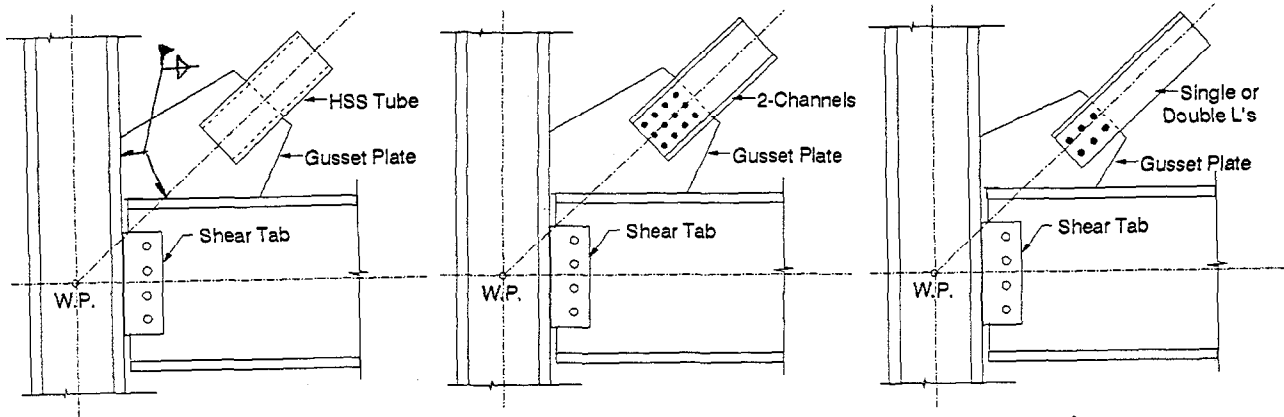


Figure 1.1. Typical Concentrically Braced Frames

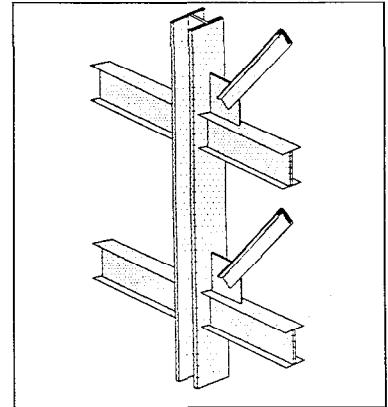
The main objectives of this report are:

- To discuss seismic behavior of common gusset plate connections.
- To present recommendations and tips on seismic design of typical gusset plates.



Note: The connection of brace to gusset, shown as bolted in most of these details, can also be welded instead of bolted.

2. SEISMIC BEHAVIOR OF GUSSET PLATE CONNECTIONS



2.1. Performance of Gusset Plates during Past Earthquakes

Gusset plates, in general, have performed satisfactorily during the past earthquakes. However, a few cases of failure of gusset plates have been reported in the aftermath of the past earthquakes including in the 1985 Mexico (Astaneh-Asl, 1986), the 1994 Northridge (Astaneh-Asl et al. 1994), and the 1995 Kobe-Japan earthquakes (Kanada and Astaneh-Asl, 1995). The observed failure modes have been in the form of fracture of welds, buckling of the gusset plates and fracture of net section of gusset plate or the bracing member, Figure 2.1. Most of these failures, especially brittle fracture of the net area can be related to non-ductile design and poor detailing of the gusset plate connections.

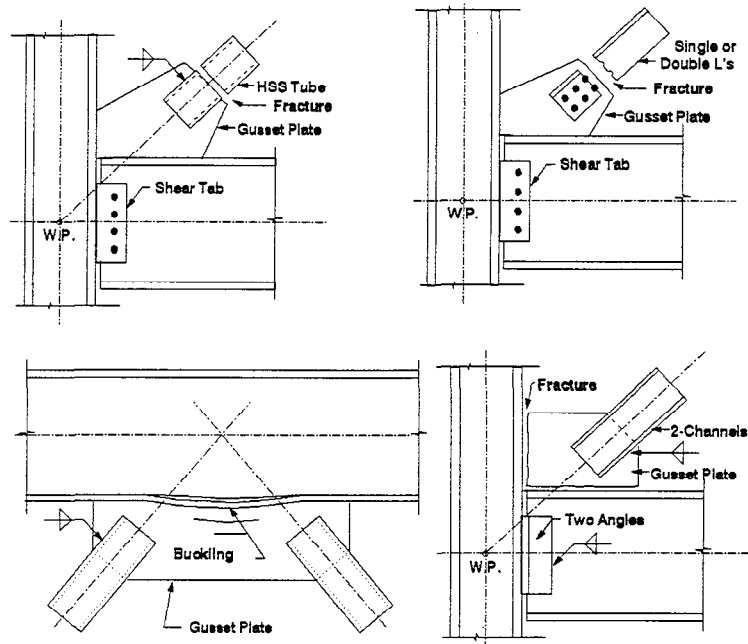


Figure 2.1. Observed Failures of Gusset Plates in the Past Earthquakes

Since gusset plate connections are one of the most critical elements of the braced frames, failure of these connections can result in significant loss of strength and stiffness of the lateral load resisting braced frame. Such losses, in turn, can result in formation of soft stories in the structure. In Chapter 3,

failure modes of gusset plates are discussed. In addition, design and detailing tips are provided that can result in ductile and desirable behavior of gusset plates and prevent observed brittle failures.

2.2. Seismic Behavior of Gusset Plates in the Laboratories

Actual behavior of gusset plates have been studied in the laboratories by a number of researchers. In the early tests of gusset plates (Rust, 1938), before the strain gages were used in structural engineering tests, instead of steel plates, gusset plates made of polished Bakelite were used. In these tests, by subjecting the Bakelite plate to polarized light, the researchers at the time were able to see and record the iso-stress lines in the gusset plates. These tests showed that forces applied to a gusset plate, by members attached to it, are distributed over an effective area of gusset near the connected members. Of course, due to elastic nature of Bakelite material, the stresses were elastic stresses. Rust's paper (Rust, 1938) not only had the results of gusset plate tests and perhaps first pictures of actual elastic stress distribution in the gusset plates, but, the paper also discussed many aspects of design of gusset plates.

The "Discussions" that were submitted for Rust's paper also had many useful information. For example, Russell C. Brinker of University of Hawaii, in his discussion referred to a series of presumably Bakelite gusset plate tests conducted by Theophil Wyss (Wyss, not dated). According to Brinker, Wyss concludes from his tests that the stress distribution within the gusset plate is along a 30-degree line. Almost 15 years later as discussed in the following, Whitmore proposed an effective area for gusset plate based on 30-degree distribution of stress within the gusset plate. The use of 30-degree lines is now known in design community as the Whitmore's method.

In 1952, R. E. Whitmore of University of Tennessee reported results of testing of a gusset plate connection (Whitmore, 1952). The material of gusset plates in the specimen was high strength aluminum with yield strength of about 39 ksi and modulus of elasticity of 10 ksi. The test was conducted on a specimen representing a truss joint with double gusset plates. The specimen was a 1/4-scale representative of actual connection of a 295 feet span truss. Whitmore produced the iso-stress lines obtained by the strain gages mounted on the tested gusset plate. The plots clearly confirmed stress trajectories to be along approximately 30-degree lines with the connected member.

Whitmore (1952) also presented distribution of normal and shear stresses along the "critical sections" of the tested gusset plate. His results indicated that the experimental stress diagrams were not similar to those predicted by the beam formulas, (i.e. $\sigma = MC/I$ and $\tau = VQ/It$). However, it is interesting to note that the maximum values of the normal and shear stresses measured in the test and predicted by the beam formulas were very close. This observation might explain why gusset plates designed using beam formulas, for almost two centuries, have performed satisfactorily. In other words, almost any test of gusset plate has indicated that stress distribution in the gusset plates cannot be predicted using beam formulas. However, the beam formulas have been able to predict the *maximum value* of the normal and shear stresses quite reasonably but in wrong location of the gusset plate. Since design of gusset plates, even today, is based on the maximum stress, the application of beam formulas have resulted in safe gusset plates.

Whitmore's test re-affirmed that in order to obtain rational values for "direct" stresses in a gusset plate in the areas at the end of the members, the concept of distribution of force along 30-degree lines could be used. In this simple concept, an effective area of gusset plate through the last line of connectors is established by drawing 30-degree lines from the first connector as shown in Figure 2.2. The "direct" stress in the gusset plate, in the areas just beyond the diagonal members, is calculated by dividing the axial force in the member by the area of this effective cross section. In addition to proposing the distribution of forces within the gusset plate to be under 30-degree angles, Whitmore (1952) endorsed the use of beam formulas to establish normal and shear stresses in "critical sections" of gusset plates.

It should be noted that Whitmore tested only one specimen and the specimen was made of high-strength aluminum. His test was followed by tests of two gusset plate connections by W. G. Irvan Jr. (1957) and B. O. Hardin (1958) of the University of Kentucky. Like Whitmore's test, the two specimens in these studies were also representative of bridge joints. The high-strength aluminum gusset plates used in these tests were densely strain gauged to measure state of stress in the gusset plate. The researchers reached the same conclusion as Whitmore that the beam formulas cannot establish stress distribution in gusset plates accurately. As Irvan (1958) stated: "The assumption that all of the beam formulas apply in calculating primary stress distribution on any cross section (either vertical or horizontal) is considerably in error".

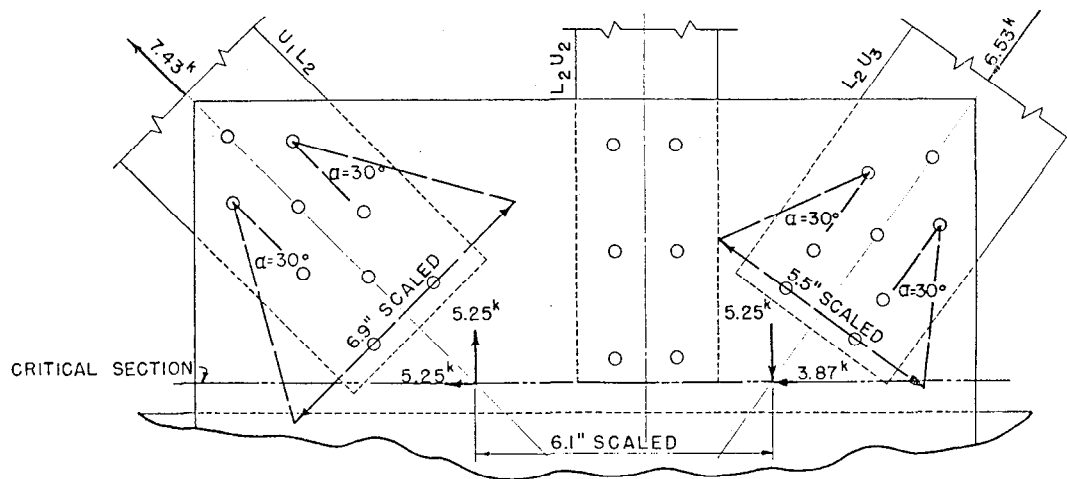


Figure 2.2. 30-degree Line Distribution of Stress Within a Gusset Plate (from Whitmore, 1952)

In 1971, D. D. Vasarhelyi of University of Washington published the results of testing of a steel gusset plate connection (Vasarhelyi, 1971). This appears to be the first test of an actual *steel* gusset plate. In this test, two diagonals were connected to a horizontal bottom chord member by a pair of steel gusset plates. Again, the specimen represented gusset plate connections of bridge trusses. The gusset plates were densely strain-gaged. In addition to testing the gusset plate connection specimen, Vasarhelyi conducted analytical studies to establish state of stress in the gusset plates. He used the then emerging computer based stress analyses and finite element methods. Vasarhelyi (1971) concluded, "The present elementary analysis appears to be adequate for most cases." Perhaps the most important finding of this study was that "The maximum stress in the gusset plate found by various simplified methods are only slightly different; the major deviations are in the location of those maximums"

All above tests were done monotonically by pushing or pulling the gusset plates in one direction. The first cyclic tests of realistic and full-size gusset plates were reported by Astaneh-Asl, Goel and Hanson of the University of Michigan (Astaneh-Asl et al., 1981-1986). The test program consisted of subjecting 17 full-scale double angle bracing members with end gusset plates to cyclic loading. A typical specimen inside the test set-up is shown in Figure 2.3. In the following, a brief summary of the results is provided.

The research indicated that cyclic behavior of gusset plates in braced frames strongly depends on the direction of buckling of the bracing member. When the bracing member buckles in-plane of the braced frame, Figure 2.4(a), three plastic hinges form in the member; one at mid-length and one at each end of the member just outside the gusset plate. Therefore, gusset plates in this case remain almost entirely elastic. However, when bracing member buckles out-of-plane of the braced frame, Figure 2.4(b),

one of the three plastic hinges still forms at mid-length of the member but two other hinges form inside the gusset plate. Formation of plastic hinge in the gusset plate means that the gusset plate should be designed such that it can provide for the relatively large rotation demands of the plastic hinge.

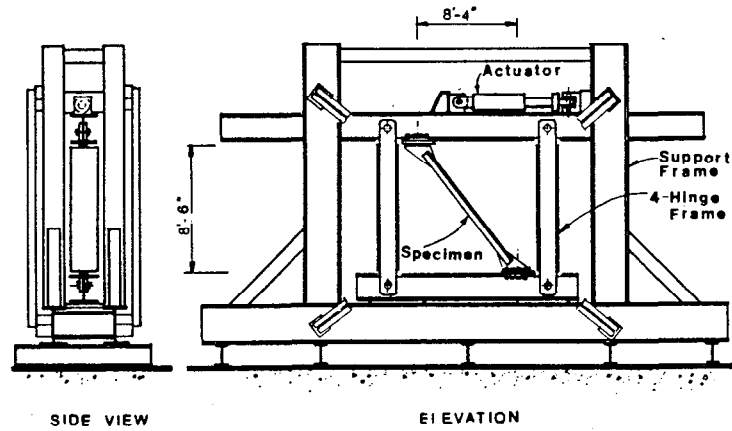
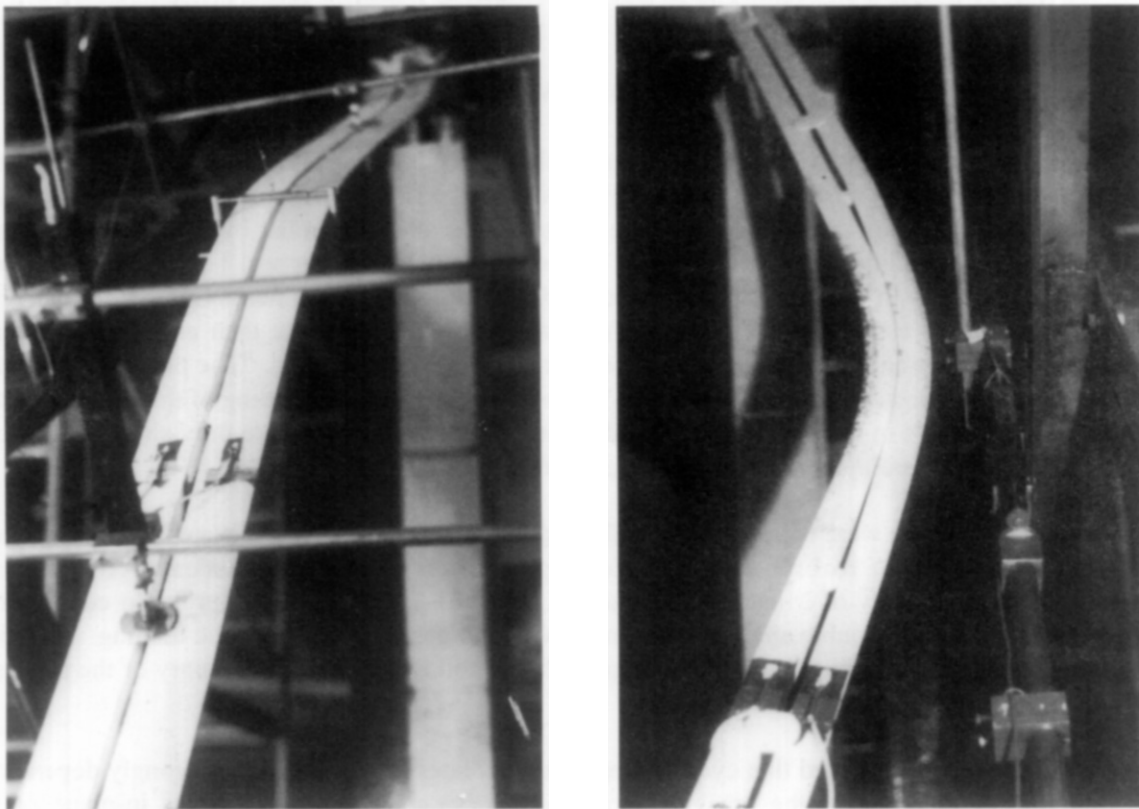


Figure 2.3. Typical Specimen Inside the Test Set-up (Astaneh-Asl et al., 1981)



(a)

(b)

Figure 2.4 (a) In-Plane and (b); Out-of-Plane Buckling of Bracing Members

When a bracing member buckles out-of-plane of the braced frame, plastic hinges form within the gusset plate. These plastic hinges need to be free to rotate; otherwise, the gusset plates can fracture in a few cycles. To ensure that gusset plate can freely rotate, the end of bracing member should be terminated a distance of at least $2t$ away from the re-entrant corner of the gusset plate (Astaneh-Asl et al., 1986). Figure 2.5 shows a poorly detailed gusset plate where the distance of $2t$ is not provided and the gusset plate has fractured in a few cycles. On the other hand, Figure 2.6 shows a properly detailed test specimen where the distance of $2t$ is provided. This specimen was able to tolerate more than 30 large inelastic cycles of axial load without fracture. Notice that the dark areas on the whitewashed specimens are indicators of yielded areas.

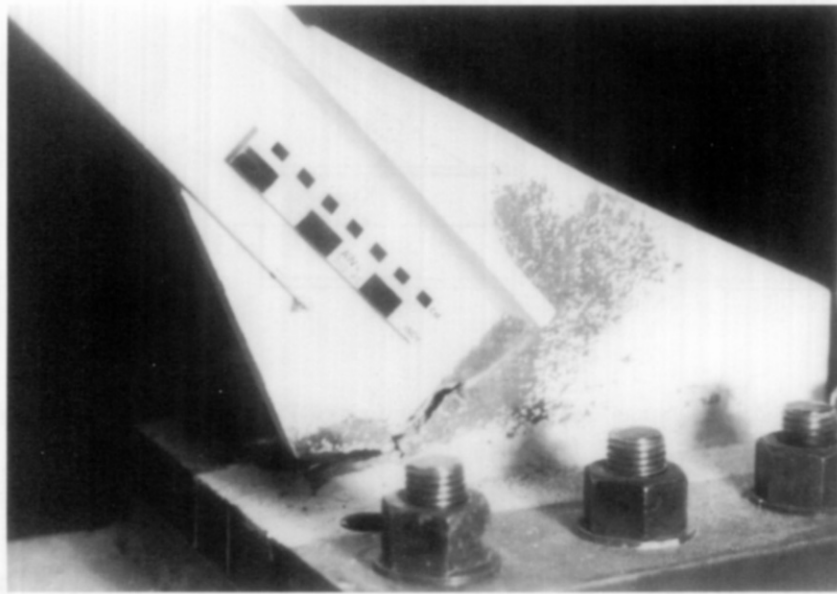


Figure 2.5 Fracture of Poorly Detailed Gusset Plate in Out-of-Plane Buckling

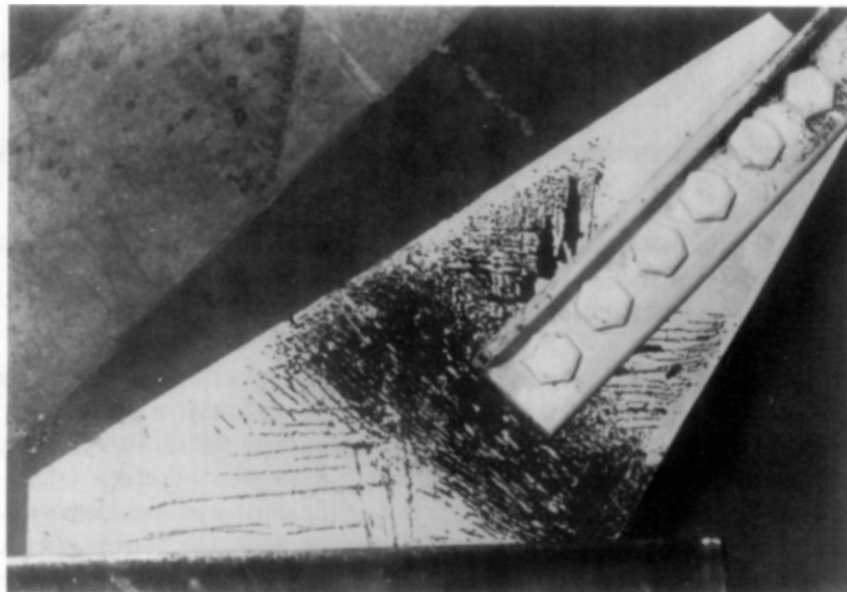


Figure 2.6. Ductile Behavior of Properly Designed Gusset Plate

Another finding of this study was to confirm validity of Whitmore's 30-degree stress distribution lines within the gusset plate under cyclic loading, Figure 2.8(a). It should be mentioned that Whitmore's concept of effective area of gusset plate, bound by 30-degree lines, was developed based on riveted specimens and later was used for bolted gusset connections as well. Astaneh et al. (1982) extended Whitmore's 30-degree stress distribution concept to welded gusset plate connection as shown in Figure 2.8(b).

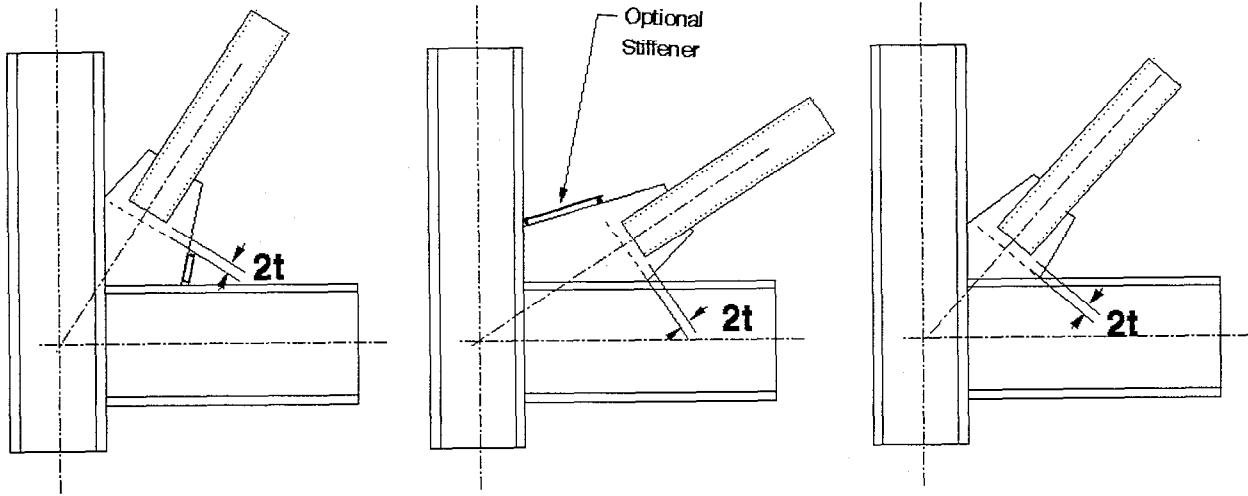


Figure 2.7. Distance "2t" for Gussets in Out-of-Plane Buckling Bracings (Astaneh-Asl et al., 1982)

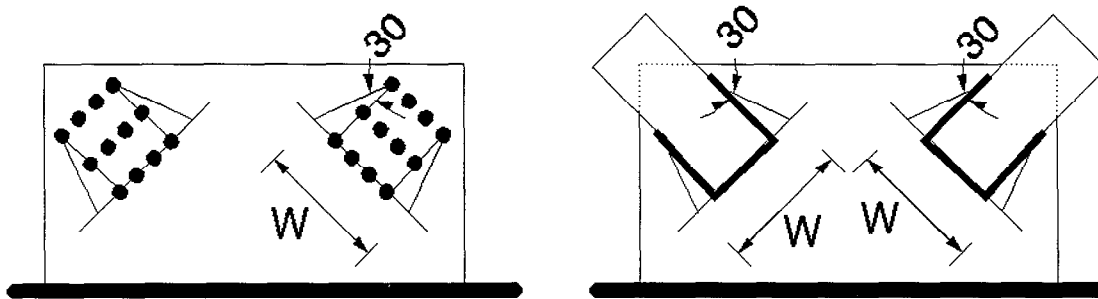


Figure 2.8. (a) Whitmore's Concept of Stress Distribution Within Gusset Plate; and (b) Extension of Whitmore's Concept to Welded Gusset Plates (Proposed by Astaneh-Asl et al., 1982)

In 1981, G. De Martino (1981) reported a summary of results of a number of tests of x-bracings with end gusset plates. The tests actually were done by L. Sanpaolesi of University of Pisa and his associates. In this program, bolted as well as welded specimens were tested. Based on the test results, De Martino concluded that: "The behavior of bolted joint is controlled by creep (slip) which does not however appear by chance, and has a positive effect; in fact it contributes considerably to the ductility of the joint and therefore improves dynamic response of the system." De Martino emphasized the extra ductility of bolted connections that results from slip and hole ovalization which is not available in welded connection. It is interesting to note that De Martino attributes many positive aspects to bolted gusset plate connections including high energy dissipation and desirable dynamic behavior of structural system.

However, in his view, the only positive aspect of welded connection is that compared to bolted connections, the behavior of welded connections is easy to understand, (De Martino, 1981).

During 1980's, more than 190 tests of bracing members and their connections were conducted in Japan. A summary of these studies is provided in Reference (Kato et al., 1992). Although most of the tests were focussed on the behavior of bracing itself, the tests provided invaluable data on cyclic behavior of concentric bracings and their gusset plates. Using the data and statistical methods, empirical rules for design of gusset plates were proposed. The study emphasized that the strength of gusset connection should be greater than yield strength of bracing member.

Bjorhovde and Chakrabarti (1984) and Bjorhovde (1988) reported results of monotonic tests of six steel gusset plates. The gusset plates in this program were representative of diagonal bracing connections in a braced frame. All specimens had a short bracing member bolted to the gusset. In the tests, the angle of bracing member with the horizontal beam was 30° , 45° or 60° . The gusset plates were either 1/8 or 3/8 inch steel plates. During the tests, the short bracing member was subjected to tension until gusset plate failed or capacity of the set-up reached. Unfortunately, due to limitations of the jack capacity, the 3/8" gusset plate specimens could not be loaded to failure. For 1/8" gusset plates, the failure mode of specimens with 30° and 45° bracing member, was tearing of the gusset plate along the last bolt hole line. They stated that: "This is in agreement with extrapolation of predictions based on Whitmore's criterion." (Bjorhovde and Chakrabarti, 1984). The specimen with 60-degree bracing member failed by tearing of net area of gusset through the bolt line connecting the gusset plate to the horizontal angles.

In 1985, Hardash and Bjorhovde (1985) reported results of 28 gusset plate tests where bolted gusset plates were subjected to tension load. In addition to these 28 tests, the researchers had added the results of another 14 similar tests conducted earlier at the University of Illinois and the University of Alberta. It appears that the main aim of this work was to study block shear failure mode in bolted gusset plates. The researchers using the accumulated test data developed and proposed a specific empirical equation to predict block shear failure capacity of the gusset plates in tension. They stated that the block shear failure mode in all specimens was due to tension tearing of net section of gusset along the last bolt lines perpendicular to the bracing member and shear yielding of gusset along the exterior bolt line parallel to the member.

R. M. Richard (1986) studied behavior of gusset plates- bracing member assemblies using inelastic finite element analyses. Sixty cases were analyzed. The author stated that to study behavior of gusset plates more realistically, the gusset plates and connected bracing members; beams and columns should be included. One of the design-oriented suggestions of this study was that: "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 brace eccentricity".

In 1988, Yamamoto et al. (1988) published the results of six monotonic tests of gusset plate connections. The specimens were double gussets and represented bridge truss joints. Using test results, the researchers developed and proposed empirical equations to be used in calculating gusset plate thickness.

Hu and Cheng (1987) reported results of 14 compression tests done on six gusset plate specimens. In some specimens there was a built-in eccentricity between the applied brace force and the working point. The failure mode of concentric specimen was buckling of gusset plate while for eccentric specimens, the failure mode was bending yielding of the splice plates connecting the gusset to the bracing member. The researchers concluded that: "The effective width concept which is currently used in designing gusset plates was found to be very unconservative if the primary failure mode is the plate buckling" (Hu and Cheng, 1987).

As indicated earlier in this chapter, the reader should notice that the effective width concept (also known as Whitmore's Method) was mentioned in 1930's in bridge design books. It appears that the concept was used to predict capacity of gusset plates in tension only. It is not surprising that Hu and Cheng tests show that the buckling capacity of gusset plates is much less than the tension capacity predicted by the Whitmore's method. The author believes that one should use Whitmore's 30° line effective width concept only to establish the effective width of the gusset plates. Whitmore's effective area can be calculated by multiplying effective width by the thickness of gusset plate. Finally, to obtain axial load capacity, the effective area of gusset should be multiplied by F_y to obtain tension yield capacity and by F_{cr} to obtain compression buckling capacity of gusset plate.

Astaneh-Asl (1989 and 1991) reported results of cyclic load tests of three gusset plate specimens representing the V-braced connections. Figure 2.9 shows the three specimens. The main parameter of study was to investigate ductility of the connection which was thought to be influenced by the location of the point of intersection of the bracing members and the girder. As shown in Figure 2.9(a), in Specimen 1, the point of intersection of bracing members was 2 inches from the weld line connecting the gusset to the girder. In Specimen 2, the point of intersection of braces was on the weld line. Specimen 3 was designed following the current practice where the centerline of bracing members and girder coincide at the "work point". Figure 2.10 shows test set-up used in the program. During the tests, the diagonal bracing members were subjected to tension and compression resembling the condition that develops in an actual Chevron and V-braced frame during seismic event. A brief summary of cyclic behavior of each specimen follows.

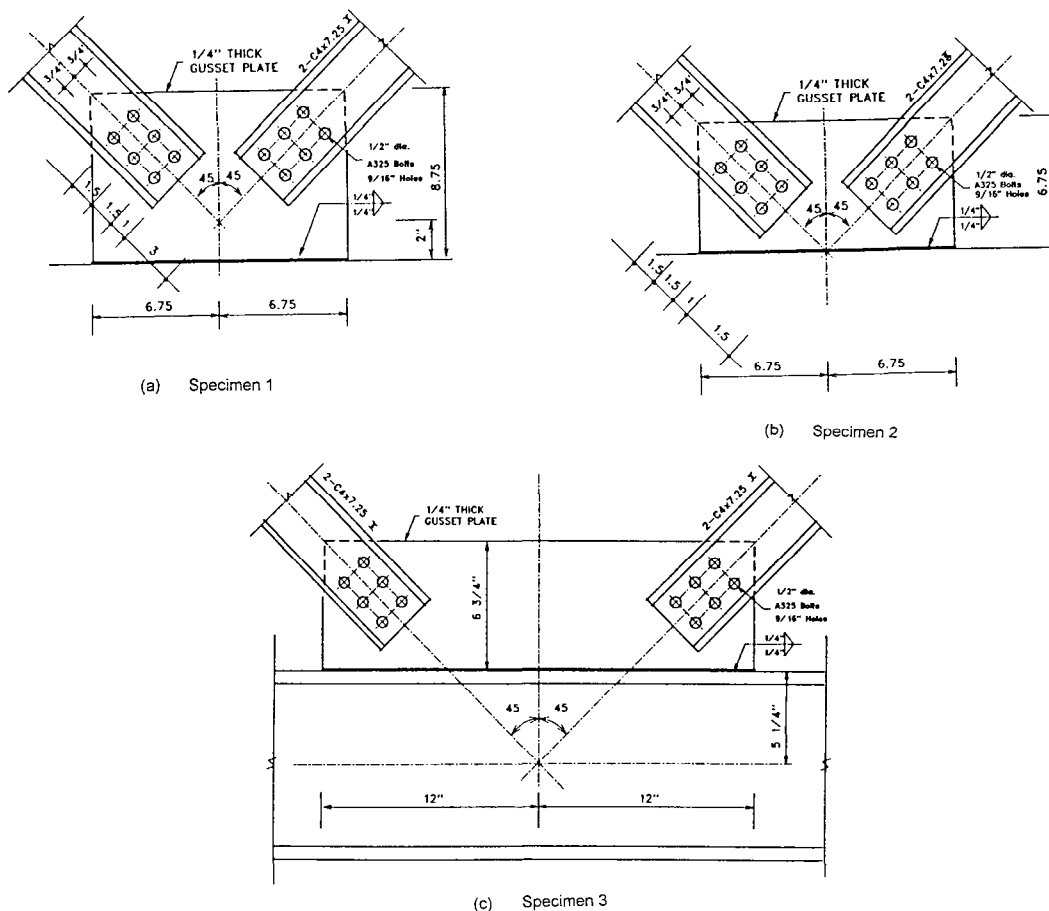


Figure 2.9 Test Specimens Subjected to Cyclic Loading (Astaneh-Asl, 1991)

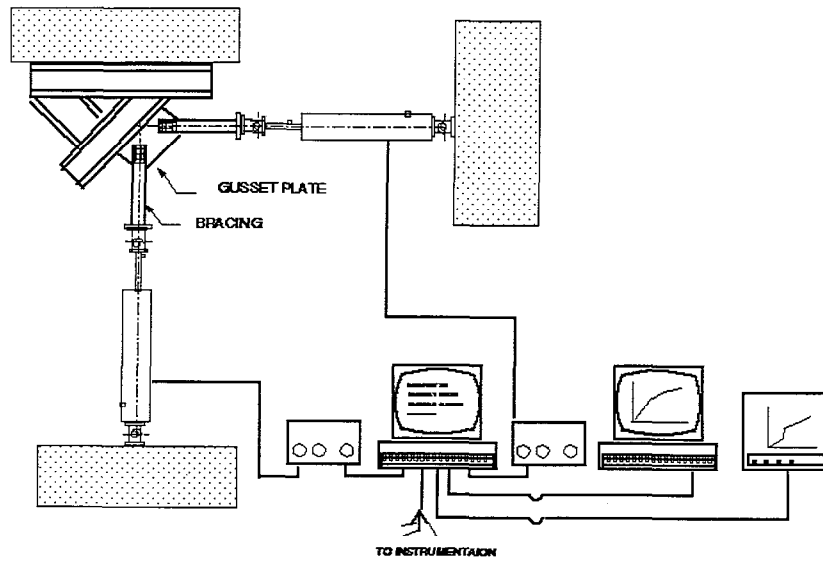


Figure 2.10. Test Set-up Used in Testing Gusset Plates

Specimen 1 had two inches of eccentricity of brace point relative to weld line, as shown in Figure 2.9(a). The gusset plate in this specimen generally behaved in a very ductile manner. The main inelasticity in this specimen was yielding of the free area of the gusset plate between the end of braces and the weld line. In addition, the bolted connection of bracing members to gusset plate were experiencing cyclic slip under combined effects of axial and bending deformations. Figure 2.11 shows appearance of Specimen 1 at the end of the test. Figure 2.12 shows shear force-shear displacement behavior of this specimen. The gusset plate in this specimen did not show any sign of fracture in tension or buckling in compression and tolerated more than six large inelastic load reversals. The test was stopped to avoid damage to the equipment when during the last cycle, minor out-of-plane buckling of the gusset plate was observed.

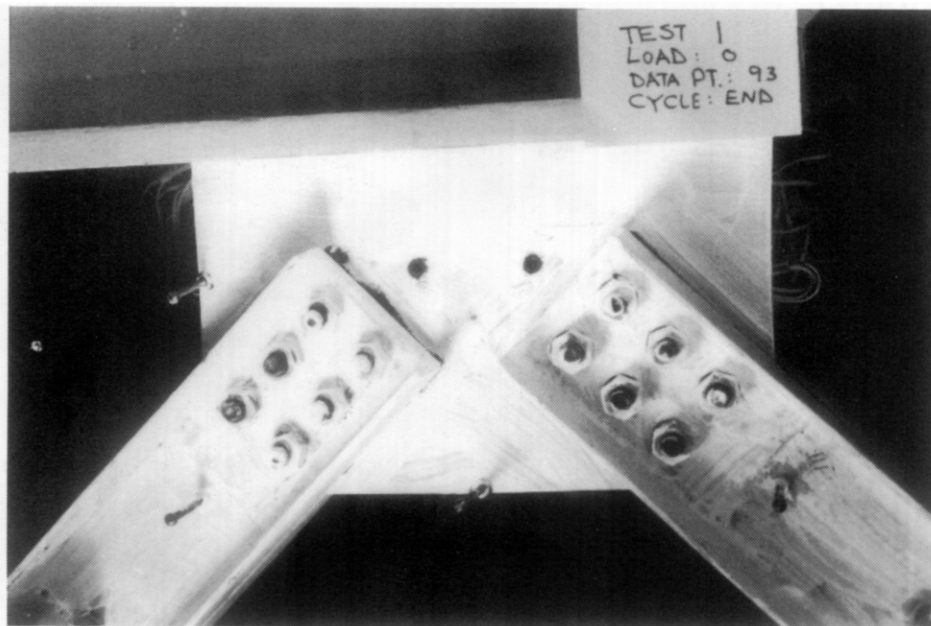


Figure 2.11. Specimen One at the End of Cyclic Tests

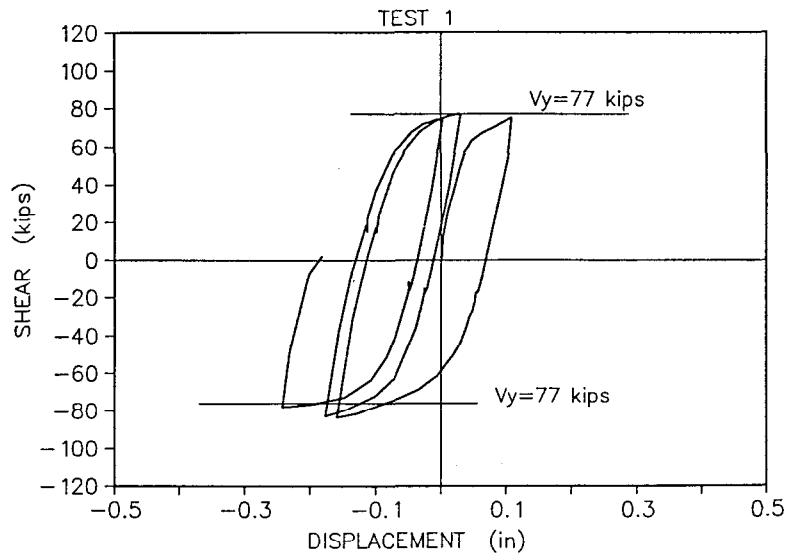


Figure 2.12. Shear Force-Shear Displacement Behavior of Specimen One

In Specimen 2, the point of intersection of braces was on the weld line connecting the gusset plate to the girder, Figure 2.9(b). This specimen behaved in a ductile manner although not as ductile as Specimen 1. The main inelastic activity in this specimen was shear yielding of the free area of the gusset plate between the end of the bracing members and the weld line. Cyclic slippage of the bolted connection of the braces to the gusset plate was also observed. Figure 2.13 shows Specimen Two at the end of the cyclic tests. Figure 2.14 shows shear force-shear displacement behavior of this specimen. The specimen tolerated five large inelastic cyclic deformations without showing signs of fracture in tension or buckling in compression. At this time, in order to observe governing failure mode, the bracing members were subjected to ever-increasing monotonic force until block shear failure of gusset plate occurred on the tension zone, Figure 2.13.

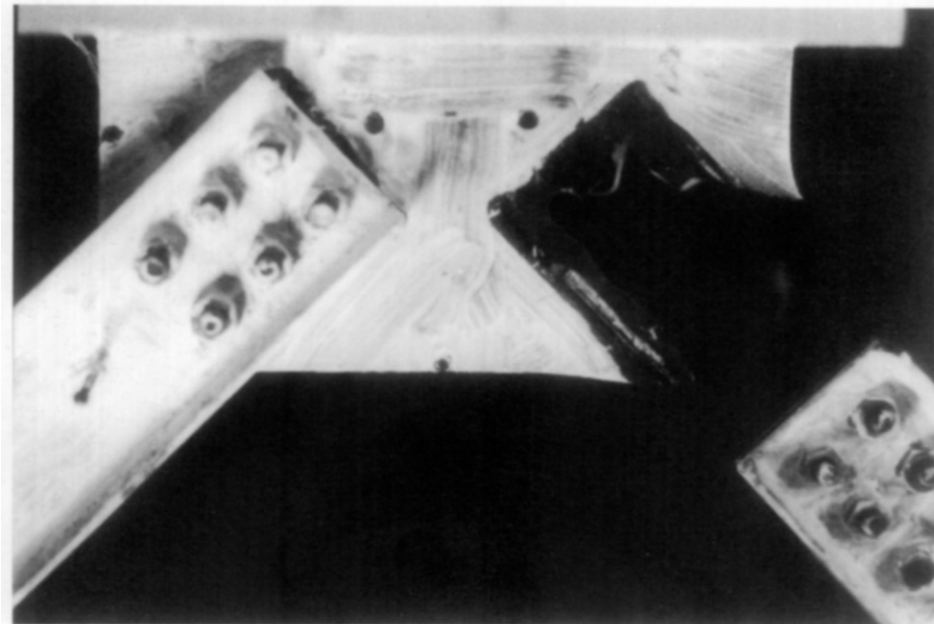


Figure 2.13. Specimen Two at the End of Cyclic Tests

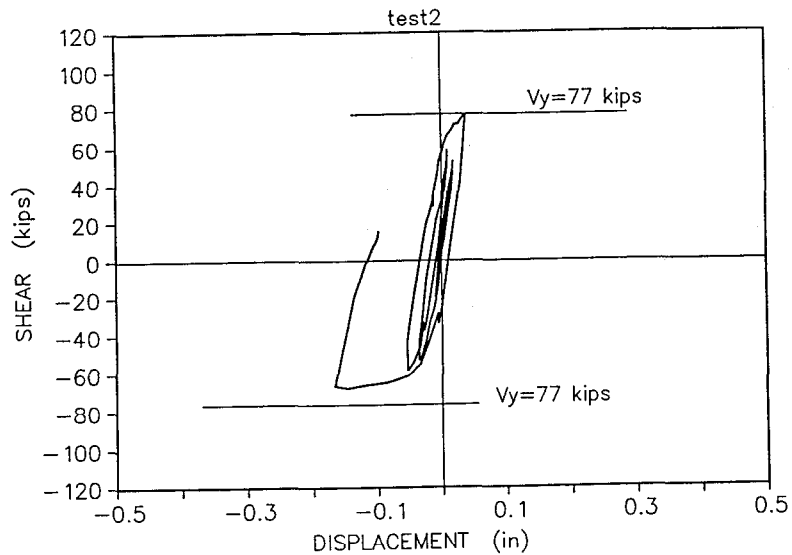


Figure 2.14. Shear Force-Shear Displacement Behavior of Specimen Two

In Specimen Three the point of intersection of the braces was on the centerline of the girder making this connection a concentric joint. Currently, almost all gusset plate connections in Chevron or V-braced frames are designed in this manner where centerline of all members connected by the gusset plate pass through the "work point". This specimen, representing current design practice, behaved in a relatively brittle and undesirable manner. The main inelastic activity in this specimen was buckling of the gusset in the areas near the edge of gusset plate as well as buckling of gusset near the end of compression bracing member. The strain gage measurements indicated that, although this was the largest of three gusset plates, the forces applied to gusset plate, were distributed in a very small area near the end of bracing member. Figure 2.15 shows Specimen Three at the end of the cyclic tests. Figure 2.16 shows shear force-shear displacement behavior of this specimen. Like previous two specimens, the bolts slipped in this specimen as well. The gusset plate tolerated only two large inelastic cycles, buckled significantly on compression side and fractured in a block shear failure mode on tension side as shown in Figure 2.15.

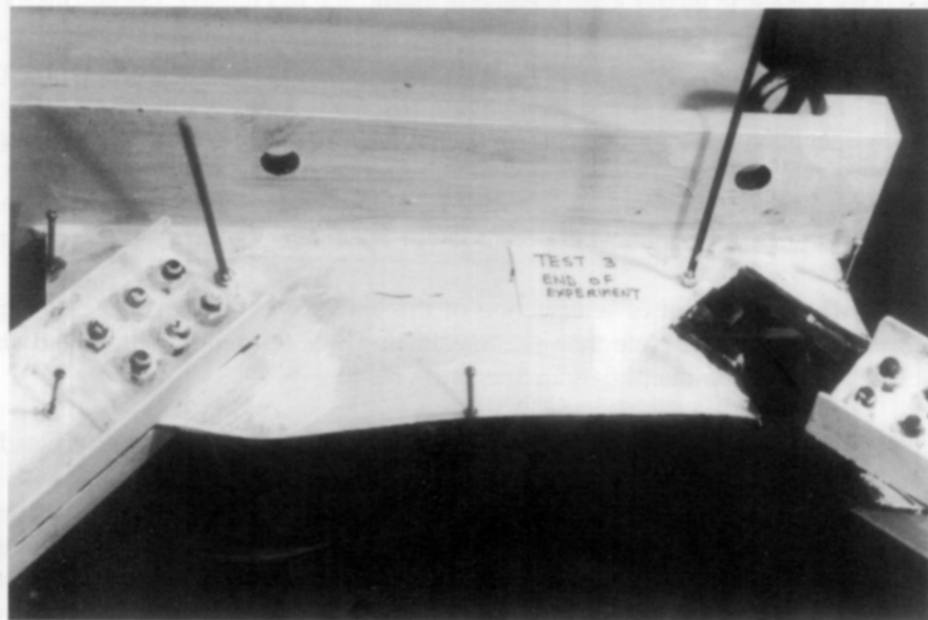


Figure 2.15. Specimen Three at the End of Cyclic Tests

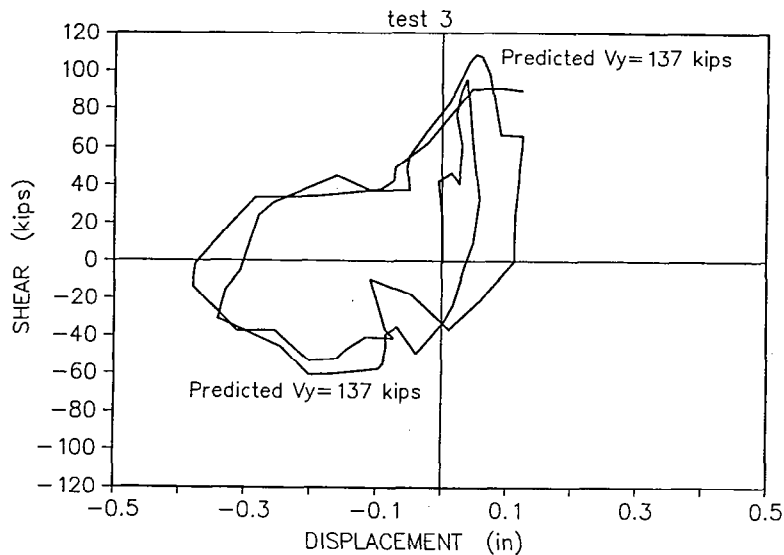


Figure 2.16 Shear Force-Shear Displacement Behavior of Specimen 3.

Based on the test data, the following conclusions were reached:

- Specimen 1, Figure 2.9(a), with largest eccentricity in the gusset plate, showed the best behavior and highest cyclic ductility. In this specimen, the point of intersection of the bracing members was 2 inches away from the flange of the beam. Consequently, there was a two inches of free length of gusset plate that was cyclically yielding in shear. This inelastic area was dissipating energy and acting as a ductile seismic fuse. The fuse could control the magnitude of force developed in bracing members and the braced frame. The use of this type of eccentric gusset plate is strongly recommended. Of course, if eccentric gusset plates are used, one might think that due to eccentricity in the connection, there would be extra moment in the joint that has to be carried by the girder. This statement is only true during the elastic phase of the behavior and if the geometry of the frame does not change noticeably. During inelastic phase of behavior, when the free area of the eccentric gusset plate yielded in shear, due to yielding, the stiffness of the gusset was lost and the moment present in the gusset was released to members. In other words, during inelastic phase of behavior, due to yielding and change of stiffness, the forces in the members and connections are redistributed while maintaining the static equilibrium under inelastic state of behavior. In Specimen 3, as the free zone of gusset plate yielded in shear, the bending moment in this region was released into the connected members. Figure 2.17 shows moment-rotation curves for all three specimens. The moment is measured on the centerline of the girder. The maximum moment for Specimen 1 was almost the same as for Specimen 3 which had no eccentricity..
- Specimen 2, where the point of intersection of the braces was on the flange of the girder, had much more compact gusset plate than the Specimen Three (current practice) and showed much better behavior and higher ductility than Specimen Three.
- Specimen 3, where the centerline of all members passed through one work-point did not show ductile and desirable behavior. The gusset plate buckled and the specimen could only tolerate two cycles of large inelastic deformations. This specimen was designed following the current practice in design of gusset plates where centerline of all members attached to the gusset plate pass through one point creating a concentric joint. In order to achieve better seismic behavior, higher ductility and smaller gusset plates, the use of "eccentric" gusset connections, shown in Figure 2.9(a) and 2.9(b), is recommended.

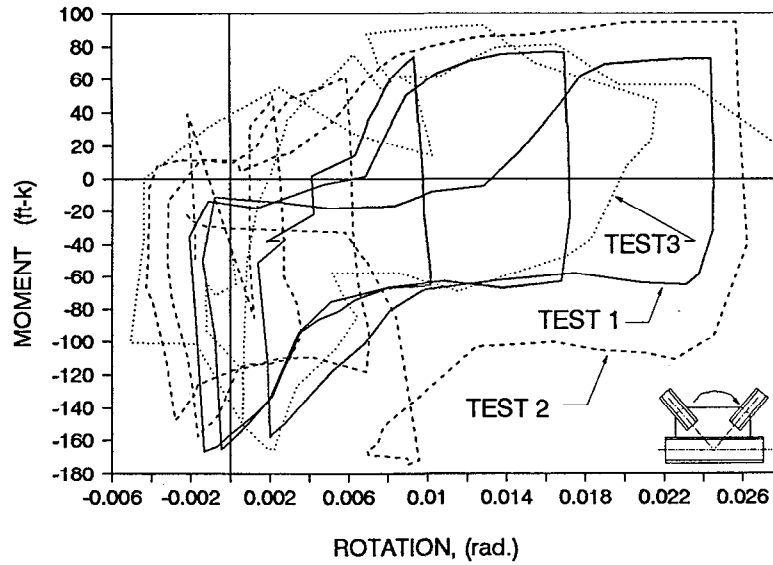


Figure 2.17. Moment-Rotation Curves for Gusset Plates in Three Specimens

As part of above research (Astaneh-Asl, 1991), the edge buckling of gusset plates was also studied. The problem of edge buckling of gusset plates has been known from the early days of design of steel bridge trusses. In fact, the bridge design codes such as AASHTO (1997) have had criterion for years to check this phenomenon and to prevent it. The current AASHTO criterion can be expressed as:

$$\frac{L_{fg}}{t} \leq 2.0 \sqrt{\frac{E}{F_y}} \quad (2.1)$$

V. L. S. Brown (1988) reported results of compressive gusset plate tests and analyses of edge buckling. The study recommended a formula to prevent edge buckling prior to gusset yielding. The formula, written in the above format, can be expressed as:

$$\frac{L_{fg}}{t} \leq 0.83 \sqrt{\frac{E}{F_y}} \quad (2.2)$$

Based on experience with performance of gusset plates in bridges, the above criterion appears to be adequate to prevent elastic buckling of free edge of relatively thin gusset plates subjected to monotonic loads. However, for gusset plates under large cyclic push-pull load, the edge buckling has occurred even when the above criterion was satisfied. The available test data on edge buckling of gusset plates are very limited. These test results are plotted in Figure 2.18. As figure indicates, for values of L_{fg}/t greater than $0.75\sqrt{E/F_y}$, the critical buckling stress, F_{cr}/F_{max} , is reduced significantly. Using these results, the author proposed the following criterion (Astaneh-Asl, 1991). The criterion is formulated to prevent cyclic buckling of the free edge of the gusset plates prior to the gusset plates reaching their maximum compression capacity.

$$\frac{L_{fg}}{t} \leq 0.75 \sqrt{\frac{E}{F_y}} \quad (2.3)$$

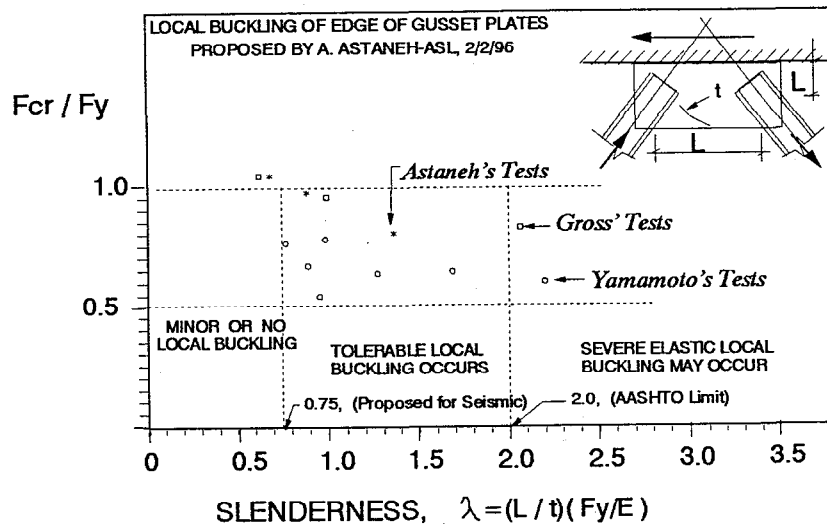


Figure 2.18. Reduction of Capacity Due to Edge Buckling of Gusset Plates

In 1990, J. Gross reported results of three monotonic tests of gusset plate bracing assemblies (Gross, 1990). The main parameters of study were the gusset geometry, eccentricity of forces in the connection and orientation of the column. Three nearly full-scale braced frame sub-assemblages were tested. For Specimens 1 and 2, the failure mode was buckling of gusset plate and for Specimen 3, tearing of gusset plate in tension. According to the study, Whitmore's procedure to calculate yield capacity of gusset plates predicted the significant yield point for these three specimens well.

Cheng et al. (1994) reported results of eight tests on four gusset plate specimens subjected to compression. Once again, as in their earlier tests, the researchers compared the buckling strength of gusset plates to the strength predicted by Whitmore's procedure and concluded that the capacities reached during the tests were less than Whitmore's prediction, thus, "...Whitmore's predictions produce unconservative estimates of the strength of the thin gusset plate loaded in compression." (Cheng et al., 1994). As discussed earlier, this finding is quite expected since Whitmore's procedure was developed for gusset plates in tension where material can reach its yield stress under tension. When gusset plate is in compression, due to elastic buckling of relatively thin gusset plates, the stress in the gusset plate material cannot reach yielding.

In 1998, Walbridge et al. (1998) reported results of three monotonically loaded gusset plates tested by Yam and Cheng (1993) and four cyclically loaded gusset plates tested by Rabinovitch and Cheng (1993). The hysteresis loops resulting from cyclic tests indicate quite ductile behavior for gusset plates. The researchers concluded that using finite element methods, cyclic behavior of gusset plates could be predicted well.

2.3. Summary of Behavior of Gusset Plates

2.3.a. Strength Failure Modes

The following failure modes have been observed in the field after earthquakes or in the laboratories:

- Fracture of gusset plate along the Whitmore's 30-degree effective width area.
- Block shear failure of gusset plate in tension.

- Buckling of gusset plate in compression.
- Buckling of free edge of gusset plate.
- Failure of critical section of a gusset plate due to combination of axial load, bending and shear.
- Failure of welds, bolts and angles connecting the gusset plate to the members

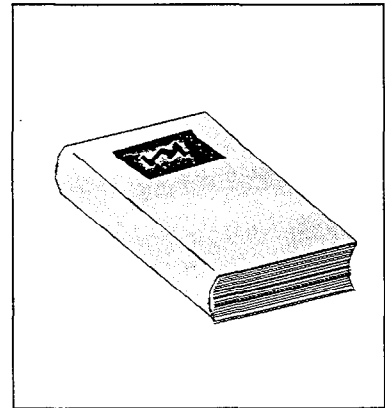
2.3.b. Ductility of Gusset Plate Connections

Studies of behavior of gusset plates with regard to their ductility have indicated that:

- Bolted gusset plates, if net section fracture is prevented, are more ductile than the similar welded gusset plates. This is due to additional ductility resulting from slippage of bolts.
- Ductility of a gusset plate strongly depends on direction of buckling of bracing member attached to it. If bracing member buckles out-of-plane of gusset plate, the gusset plate needs to be detailed to accommodate the end rotation demand of the bracing member in a ductile manner. This can be accomplished by ending the bracing member a distance of $2t$ from the re-entrant corner of the gusset (Astaneh-Asl, 1982), (AISC, 1997). However, if bracing member buckles in plane of gusset plate, the gusset remains almost elastic and there is no need for such ductile detailing.
- Buckling of gusset plate not only results in reduced compressive capacity but if buckling is elastic, it can lead to brittle and undesirable behavior. Therefore, in cases where buckling capacity of gusset is less than say 50% of tension capacity, the gusset plate should be stiffened or thickened to develop larger buckling capacity.
- Edge buckling of gusset plate is not desirable. The criterion proposed by Astaneh-Asl (1991), and given as Equation 2.3 in previous section may be used to check edge buckling under cyclic loading. If the above criterion is not satisfied, then it is likely that the edge of gusset plate will buckle under large cyclic inelastic stress reversals. To prevent edge buckling, the edge of gusset plate can be stiffened by adding a relatively small angle or plate as commonly done in bridge gusset plates. Also, one can increase the thickness of gusset or reduce the length of free edge to satisfy the above criterion.

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3. RELEVANT CODE PROVISIONS



3.1. Introduction

In this chapter, code provisions relevant to seismic design of gusset plates for concentrically braced frames are discussed. Currently, concentrically braced frames are divided into two categories by seismic codes such as AISC (1997) and UBC (ICBO, 1997). These two categories are “Special” and “Ordinary”. Special concentrically braced frames (SCBF’s) are designed and detailed to have higher ductility. The higher ductility enables SCBF’s to tolerate relatively large number of inelastic cyclic deformations during earthquakes without fracture or significant reduction in their strength and stiffness. In SCBF’s, the bulk of inelastic deformations occur in the bracing members or their connections. Such inelastic ductile deformations increases damping and reduces stiffness of the structure resulting in energy dissipation as well as smaller seismic forces generated in the structure. The Ordinary Concentrically Braced Frames (OCBF’s) for less seismically active regions do not need to be as ductile as SCBF’s.

Current seismic design codes, such as AISC Seismic Specifications (AISC, 1997) and Uniform Building Code (ICBO, 1997), allow special concentrically braced frames (SCBF’s) to be designed for smaller seismic forces than similar but ordinary concentrically braced frames. This is a direct result of higher ductility of special concentrically braced frames.

One of the most important components of concentrically braced frames, Special or Ordinary, is the connection of the bracing members to the beams and columns. These connections are typically gusset plate connections. Seismic design codes have a number of provisions directly related to seismic design of steel gusset plates. In the following, the provisions of recently released AISC Seismic Provisions for Structural Steel Buildings (AISC, 1997) are discussed. The provisions discussed in the following are those deemed directly related to the design of gusset plates. For actual code provisions on seismic design of gusset plates the reader needs to refer to all applicable provisions of the AISC document (AISC, 1997).

3.2 Relevant Provisions in the “AISC Seismic Provisions for Structural Steel Buildings”

The AISC Seismic Provisions for Structural Steel Buildings (AISC, 1997), has a number of provisions that are applicable to design of gusset plates in concentric braced frames. The provisions can be discussed in five categories: (a) Material, (b) Bolted Joints, (c) Welded Joints, (d) Connections in Concentrically Braced Frames and; (e) Bracing Members in Concentrically Braced Frames. In the following, these provisions are summarized for reader's information. For actual and proper application of these and other relevant provisions, the AISC (1997) should be used.

3.2.a. Material

According to the AISC seismic provisions (AISC, 1997), "When required in these provisions, the required strength of a connection or related member shall be determined from the Expected Yield Strength F_{ye} of the connected member, where

$$F_{ye} = R_y F_y \quad (3.1)$$

F_y is the specified minimum yield strength of steel and the values of R_y as given by the AISC (1997) are listed in Table 3.1.

Table 3.1. Values of R_y Given by the AISC-97 [a]

Material	R_y
ASTM A36 Steel	1.5
ASTM A572 Grade 42	1.3
For rolled shaped and bars of other grades of steel and for plates	1.1

Note:

[a] Other values of R_y are permitted to be used if the value of F_{ye} is determined by testing that is conducted in accordance with the requirements for the specified grade of steel.

AISC (1997) has provisions regarding notch-toughness of steel used as members in Seismic Force Resisting System. According to these provisions, rolled steel shapes with elements thicker than 1.5 inches as well as plates 1.5 inches thick or thicker in built-up cross sections, shall have a minimum Charpy V-Notch,(CVN) toughness of 20 ft-lbs at 70 degrees F. The AISC (1997) does not explicitly require gusset plates 1.5 inch thick or thicker to have the above CVN. However, it is suggested herein that the above CVN requirement is applied in design of thick gusset plates (thicker than 1.5 inches) in highly seismic areas (i.e. Zones 3 and 4) and particularly to welded gusset plates.

3.2.b. Bolted Joints

The bolts used in the joints of Seismic Force Resisting System are required by AISC (1997) to be fully tensioned high strength bolts with faying surfaces prepared as required for Class A or better slip critical joints. However, the bolts to resist shear can be designed based on their bearing strength as long as the faying surface is prepared to provide coefficient of friction of at least 0.33. The bearing strength of bolts cannot be taken greater than $2.4dtF_u$. The bolted joints are not permitted to share the load in combination with welds on the same faying surface. The bolt holes can be standard or slotted-holes where the length of slot is perpendicular to the direction of force. Other types of holes are also permitted if their use is justified as part of a tested assembly.

The AISC(1997) now specifies that: "The bolted connections for members that are part of the Seismic Force Resisting System shall be configured such that a ductile limit-state either in the connection or in the member controls the design". This is perhaps the most important issue in seismic design and was first explicitly introduced to steel design and incorporated into the AISC Manual of Steel Construction while developing the design guidelines for shear tabs (Astaneh-Asl et al., 1988). Later, this hierarchy of having ductile failure modes such as yielding precede more brittle failures was applied to seismic design of other connections such as bolted moment frame connections (Astaneh-Asl, 1995) and column-tree moment frames (Astaneh-Asl, 1997). In Chapter 4 of this report, the procedures suggested for seismic design of gusset plates, are entirely based on this philosophy of ensuring ductile failure modes to occur prior to the more brittle failure modes.

3.2.c. Welded Joints

The AISC(1997) has specific provisions regarding welding particularly with regard to full-penetration groove welds. In gusset plate connections seldom full-penetration welds are used. Instead, fillet welds are used to connect bracing members to the gusset plates and gusset plates to the girders and columns. Laboratory tests and actual seismic events have indicated that properly designed and executed fillet welds perform well under cyclic loading and have sufficient ductility. AISC (1997) has a provision that indicates: "For members and connections that are part of the Seismic Force Resisting System, discontinuities created by errors or by fabrication or erection operations, such as tack welds, erection aids, air-arc gauging, and flame cutting, shall be repaired as required by the engineer of Record". This provision is an important one and is fully applicable to design of gusset plates and should be followed, particularly for cases of relatively large welds used with thick (thicker than 1.5 inches) gusset plates, member flanges and webs. The issue of welding in seismic applications is still under study and development. Therefore, the reader is strongly urged to refer to the latest information on this subject.

3.2.d. Connections in Concentrically Braced Frames

The relevant provisions of the AISC (1997) regarding design of gusset plate connections are summarized in the following Table 3.2 for discussion purposes. In actual design, the designer needs to use the AISC (1994), AISC(1997) and other applicable codes.

Table 3.2. A Summary of the AISC-97 Seismic Provisions on Connections of Concentrically Braced Frames

Item	Value for Special Concentrically Braced Frame	Value for Ordinary Concentrically Braced Frame
Required Strength of Bracing Connections (gusset plates, and including beam-to-column connections if part of the bracing system)	Lesser of the following: <ul style="list-style-type: none"> • The nominal axial tensile strength of the bracing member, determined as $R_y F_y A_g$ • The maximum force, indicated by analysis, that can be transferred to the brace by the system. 	Least of the following: <ul style="list-style-type: none"> • The nominal axial tensile strength of the bracing member, determined as $R_y F_y A_g$ • The force in the brace that results from load combinations (4-1) and (4-2) given in AISC (1997) • The maximum force, indicated by analysis, that can be transferred to the brace by the system.
Tensile Strength: Based upon the limit states of rupture of effective net section and block shear rupture	Equal to or greater than the Required Strength as determined above.	Equal to or greater than the Required Strength as determined above
Flexural Strength: In the direction that analysis indicates the brace will buckle	Equal to or greater than $1.1 R_y M_p$ of the brace about the critical buckling axis [a]	Equal to or greater than $1.1 R_y M_p$ of the brace about the critical buckling axis [b]
Design of gusset plates	The design of gusset plates shall include consideration of buckling	The design of gusset plates shall include consideration of buckling

Notes:

- [a] "Exception: Brace connections that meet the requirements in Section 13.3b (2nd row in the above table), can accommodate the inelastic rotations associated with brace post-buckling deformations, and have a design strength that is at least equal to the nominal compressive strength $F_{cr} A_g$ of the brace are permitted." (AISC, 1997).
- [b] "Exception: Brace connections that meet the requirements in Section 14.3b (2nd row in the above table), can accommodate the inelastic rotations associated with brace post-buckling deformations, and have a design strength that is at least equal to the nominal compressive strength $F_{cr} A_g$ of the brace are permitted." (AISC, 1997).

3.2.e. Bracing Members in Concentrically Braced Frames

The AISC (1997) has two sets of provisions for Ordinary and Special concentrically braced frames. The major provisions are summarized in the following Table 3.2. For complete provisions and commentary see AISC (1997).

Table 3.1. A Summary of the AISC-97 Provisions on Concentrically Braced Frames and Their Members

Item	Value for Special Concentrically Braced Frames	Value for Ordinary Concentrically Braced Frames
Reduction Factor	$R = 6.4$ [a]	$R = 5.6$ [a]
Seismic Over-strength Factor	$\Omega_0 = 2.2$ [a]	$\Omega_0 = 2.2$ [a]
Required compressive strength of bracing members	$P_u \leq \phi_c P_n$	$P_u \leq 0.80 \phi_c P_n$
Overall slenderness of bracing members	$KL/r \leq 1000/\sqrt{F_y}$	$KL/r \leq 720/\sqrt{F_y}$
Slenderness of an element between stitches	$K l/r_i \leq 0.40(KL/r)$ [b]	$l/r_i \leq 0.40(KL/r)$
Width-thickness ratios of unstiffened elements of braces	$b/t \leq 65/\sqrt{F_y}$ [c]	$b/t \leq 141/\sqrt{F_y - 10 \text{ ksi}}$
Width-thickness ratios of stiffened elements of braces	$b/t \leq 253/\sqrt{F_y}$	$b/t \leq (970/\sqrt{F_y})(1 - 0.74 P_u / \phi_b P_y)$
Width-thickness ratio of stems of tees	$b/t \leq 65/\sqrt{F_y}$ [c], [d]	$b/t \leq 127/\sqrt{F_y}$
Width-thickness ratio of angles	$b/t \leq 52/\sqrt{F_y}$	$b/t \leq 52/\sqrt{F_y}$
Outside diameter to thickness ratio of round HSS (unless round HSS wall is stiffened).	$D/t \leq 1300/F_y$	$D/t \leq 1300/F_y$
Flat width to wall thickness ratio of rectangular HSS (unless rectangular HSS wall is stiffened).	$b/t \leq 110\sqrt{F_y}$	$b/t \leq 110\sqrt{F_y}$
Spacing and number of stitches	Uniform spacing and not less than 2 stitches used.	Not less than 2 stitches shall be equally spaced about the member centerline.
Bolted stitches	Not to be located within the middle $\frac{1}{4}$ of the clear brace length	-
Design shear strength of stitches	$V_{\text{Stitch}} \geq T_{\text{Element}}$	$V_{\text{First Stitch}} \leq 0.5(T_{\text{Element}})$

Notes:

[a] Values are from Uniform Building Code (1997).

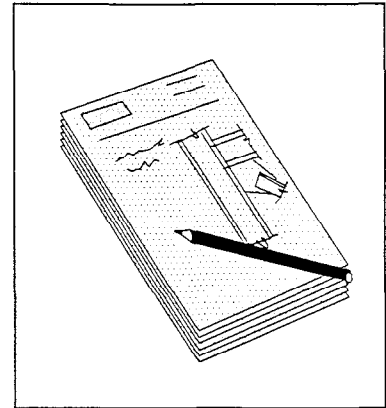
[b] "Exception: where it can be shown that the braces will buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio l/r_i of the individual elements between the stitches does not exceed $0.75(KL/r)$ of the built-up member" (AISC, 1997).

[c] There is no value of λ_p for the stem of tees in the AISC-LRFD Specification (AISC, 1994). The value given here is based on the Author's judgment.

[d] "Assumes an inelastic rotation capacity of 3. For structures in zones of high seismicity, a greater rotation capacity may be required" (AISC, 1994). It is suggested herein that for zones of high seismicity and to achieve larger rotation capacity, the limit of b/t of $52/\sqrt{F_y}$ as given in Table I-9-1 of the AISC (1997) be used.

The AISC (1997) has provisions on "Special Bracing Configuration Special Requirements" that are applicable to Chevron, V and K braced frames. For these special provisions, the reader is referred to AISC (1997). In addition, there are provisions in the AISC (1997) that apply to design of columns of concentrically braced frames. These provisions, not being directly relevant to design of gusset plates, are not discussed here and again, the reader is referred to AISC (1997).

4. SEISMIC DESIGN OF GUSSET PLATES FOR DUCTILE PERFORMANCE



Gusset plate should have sufficient strength to transfer the applied forces. In cases where the bracing member connected to gusset plate buckles out-of-plane, the gusset plate should have sufficient ductility to deform and provide the end rotation demand of the member. Fracture of a gusset plate most likely will result in considerable loss of strength and stiffness of the bracing member and the braced frame. Such losses can in turn result in undesirable and brittle performance of the braced frame. Therefore, to avoid such brittle behavior, the governing failure mode of the gusset plate connections in seismic applications should be a yielding failure mode and not a fracture mode.

The first step in seismic design of any connection, including gusset plates, is to identify failure modes (or limit states). Then, the failure modes should be arranged in a hierarchical order such that ductile and more desirable failure modes, such as yielding, occur prior to the brittle and less desirable failure modes; such as fracture. The concept of hierarchical ordering of failure mode has been suggested and successfully used by the author in the past in developing design procedures for shear connections (Astaneh-Asl, 1989), (Astaneh-Asl and Nader, 1989) (Astaneh-Asl, et al., 1989) and bolted moment frame connections (Astaneh-Asl, 1995).

During the earthquakes, bracing members and sometimes gusset plates, are the most active elements and experience yielding, buckling and other failure modes. Let us consider a bracing member with its end gusset plates as shown in Figure 4.1. Four important zones can be identified for this system. These, shown in Figure 4.1, are:

1. Bracing member,
2. Connection of bracing member to the gusset plate,
3. Gusset plate, and;
4. Connection of gusset plate to supporting beams and columns.

Each of the above four zones has its own failure modes. To have a desirable and ductile seismic performance, the governing failure mode within each of the above zones should be a ductile failure mode, such as yielding and not a brittle failure mode such as fracture. Since the above four zones are in series, like a chain, during an earthquake, the weakest of the four zones is expected to become more active and yield in tension and buckle in compression. In order to increase global ductility of a braced frame, the yield strength of the above-mentioned four zones should be in a hierarchical order. The order is shown in Figure 4.2. In this hierarchical order, the governing failure mode for each of the four zones should be yielding. Yielding of bracing member results in large axial plastic deformations, which in turn result in large ductility for the braced frame. Therefore, in the hierarchical order of yielding, shown in Figure 4.2,

yielding of member is designed to occur first followed by yielding of connection elements and components. On the other hand, yielding of relatively short elements of the system, such as welds connecting the gusset plate to its support cannot provide large global ductility. Therefore, such yielding is placed near the end of the hierarchical order of yielding shown in Figure 4.2. If a bracing system is designed following this concept, brittle fracture of elements of the system will be avoided and desirable and ductile seismic behavior will result.

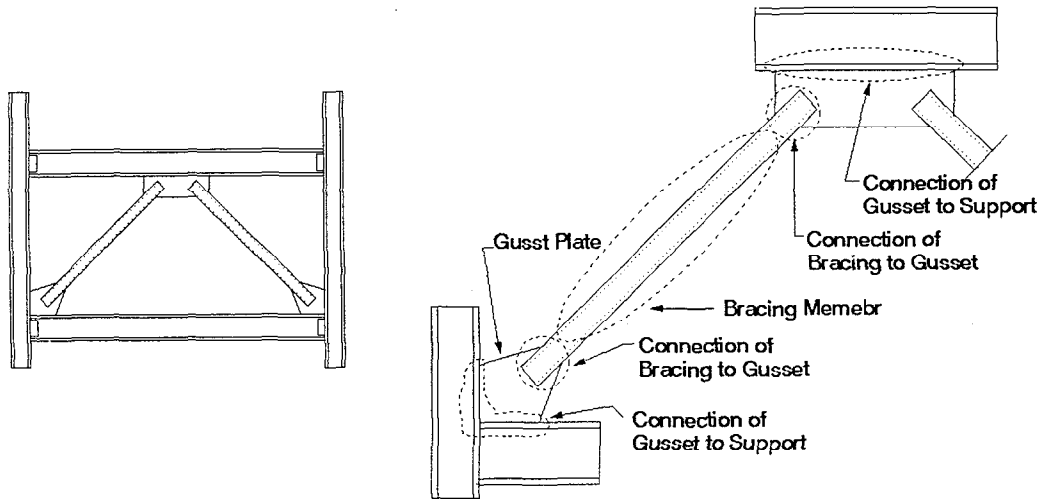


Figure 4.1. Typical X-bracing and Four Zones of Importance



Figure 4.2. Hierarchical Order of Yielding in Elements of a Braced Frame

In the following sections, issues related to seismic design of each of the above-mentioned four zones are discussed. The emphasis is on the failure modes and seismic design of gusset plate.

4.1. Seismic Design of Bracing Member

The current seismic design codes, such as AISC (1997) and UBC (ICBO, 1997), allow the use of "Ordinary" as well as "Special" concentrically braced frames. Table 3.2 in Chapter 3 of this report, provides a summary of the AISC seismic provisions (AISC, 1997) relevant to seismic design of bracing members in "Ordinary" and "Special" concentrically braced frames. The main difference between the bracing members in Ordinary and Special concentrically braced frames, is in more stringent requirements for special braced frames with regard to b/t and Kl/r of bracing member. This is done to make Special braced frames more ductile. It is suggested herein that in high seismic zones (Zones 3 and 4) only Special concentrically braced frames be used. The use of special concentric braced frames not only results in ductile and desirable performance but in many cases it can be more economical than the ordinary braced

frames.

The failure modes of a typical bracing member, in the order of their desirability, are:

- a. Yielding of gross area of member when subjected to tension force,
- b. Overall buckling of bracing member when subjected to compression force,
- c. Bearing failure of bolt holes in bolted built-up bracing members,
- d. Yielding of stitches and batten plates in built-up bracing members,
- e. Buckling of individual elements in built-up members,
- f. Local buckling of bracing member cross section,
- g. Slippage of bolts connecting the elements of member in built-up members,
- h. Fracture of bolt edge distance and bolt spacing,
- i. Fracture of stitches or batten plates in built-up members,
- j. Fracture of effective net area of member,
- k. Block shear failure of member; and,
- l. Fracture of bolts or welds in built-up members.

The above failure modes are listed in their hierarchical order of desirability and shown in Figure 4.3. Yielding of the bracing member is the most desirable and fracture of net area of brace is the least desirable. All of above failure modes are well known and well covered in the literature and design specifications (AISC, 1994, 1997). Since the emphasis of this report is on gusset plates, for design of bracing member, the reader is referred to the ASIC Specifications (AISC, 1994 and 1997).

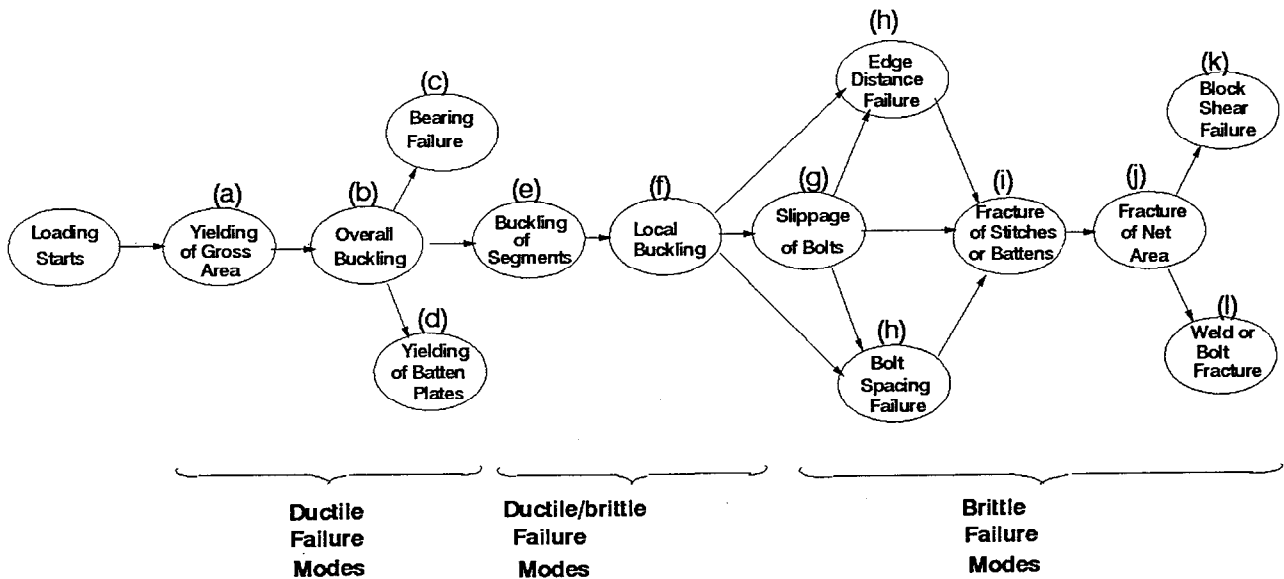


Figure 4.3. Failure Modes of Typical Bracing Members in a Hierarchical Order of Desirability

One of the important issues in today's seismic design of steel structures is to pay attention to variability of yield point of steel and how it might adversely affect seismic performance. Ductile performance of steel structures depends on timely yielding of steel and development of significant inelastic deformations prior to fracture. If actual yield point of steel is much higher than the specified value used in design, it is quite possible that during earthquakes, more brittle failure modes precede yielding resulting in non-ductile behavior of the structure.

Current design specifications such as AISC (1997), have provisions to increase specified yield point of steel to a more realistic level by multiplying it by R_y . Values of R_y as per AISC (1997) are given in Table 3.1, Chapter 3 of this report.

To obtain a ductile and desirable seismic performance in concentrically braced frames, where yield failure modes should be designed to occur prior to more brittle fracture modes, the following procedure for design of bracing members is suggested:

1. Use "special" braced frames in high seismic areas.
2. Apply load combinations prescribed by the governing code, analyze the structure and establish maximum axial tension and compression forces in the bracing members.
3. Use LRFD methods of the governing code and design the bracing members to carry axial tension and compression forces in the member such that yielding of gross area under tension and overall buckling of the member under compression govern over other failure modes listed above. For this step, use minimum specified yield stress, F_y of steel. The effective length factor of compression bracing member, K , can be taken as 1.0 for out of plane buckling and 0.65 for in plane buckling bracing members respectively.
4. Check other failure modes listed above and insure that they will not occur before axial load in the member reaches yield capacity of gross area in tension and overall buckling capacity of member in compression. To calculate tension yield and compression buckling capacities of the member to be used in this step, use expected realistic yield stress of steel equal to $R_y F_y$.
5. Design connections of the bracing member as explained in the following sections.

4.2. Seismic Design of Connection of Bracing Member to the Gusset Plate

Bracing members frequently are connected directly to the gusset plates using bolts or welds, Figure 4.4(a). However, when bracing member is a wide flange shape, often angles and plates are used to connect the wide flange bracing member to the gusset plate, Figure 4.4(b).

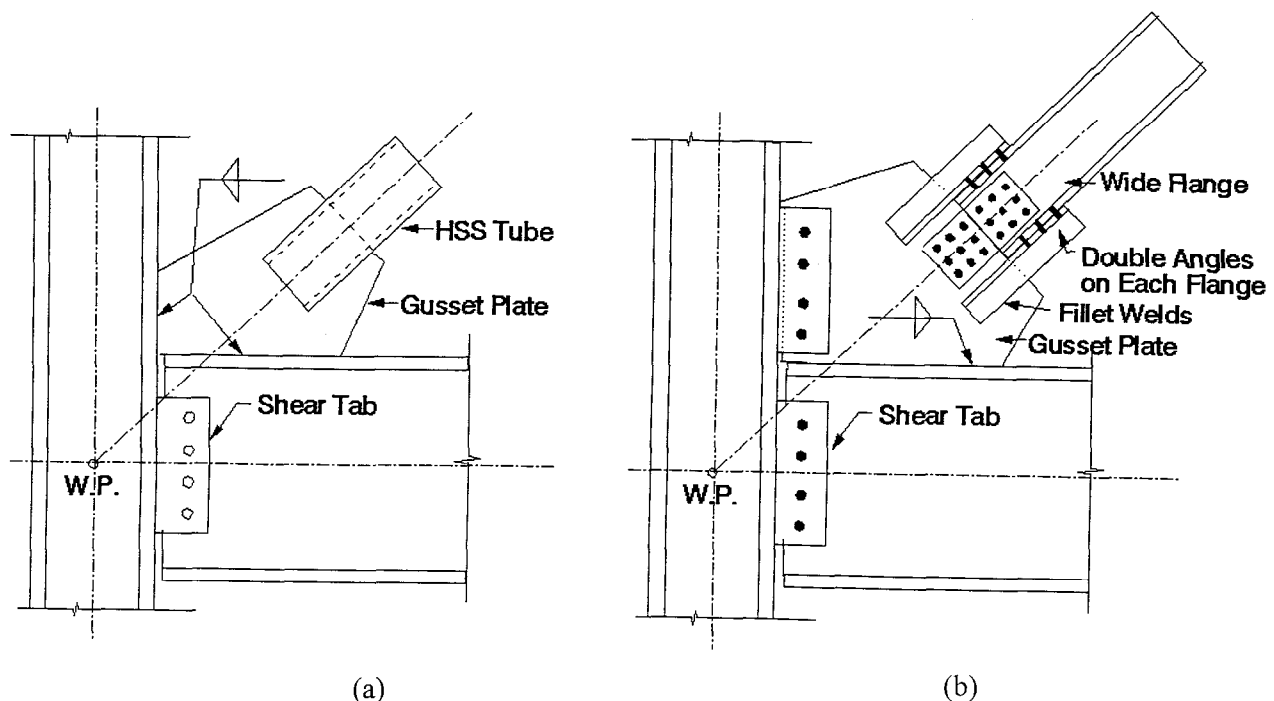


Figure 4.4. Two Examples of Connections of Bracing Members to Gusset Plates

Failure modes of typical connection of a bracing member to the gusset plate, in the order of desirability, are:

- a. Slippage of bolts in bolted connections,
- b. Yielding of gross area of angles and plates used in the connections, see Figure 4.4(b),
- c. Bearing failure of bolt holes,
- d. Local buckling of angles and plates used in the connection,
- e. Edge distance fracture and bolt spacing failure in bolted connections,
- f. Fracture of effective net area of angles and plates in the connection; and
- g. Block shear failure
- h. Fracture of bolts and welds.

The above failure modes are shown in Figure 4.5 in their hierarchical order of desirability from left to right. All failure modes should be checked to ensure that they will not occur before axial load in the member reaches yield capacity of the member gross area in tension and overall buckling capacity of member in compression. To calculate tension yield and compression buckling capacities of the member it is suggested herein to use conservatively expected yield stress of steel equal to $1.1 R_y F_y$.

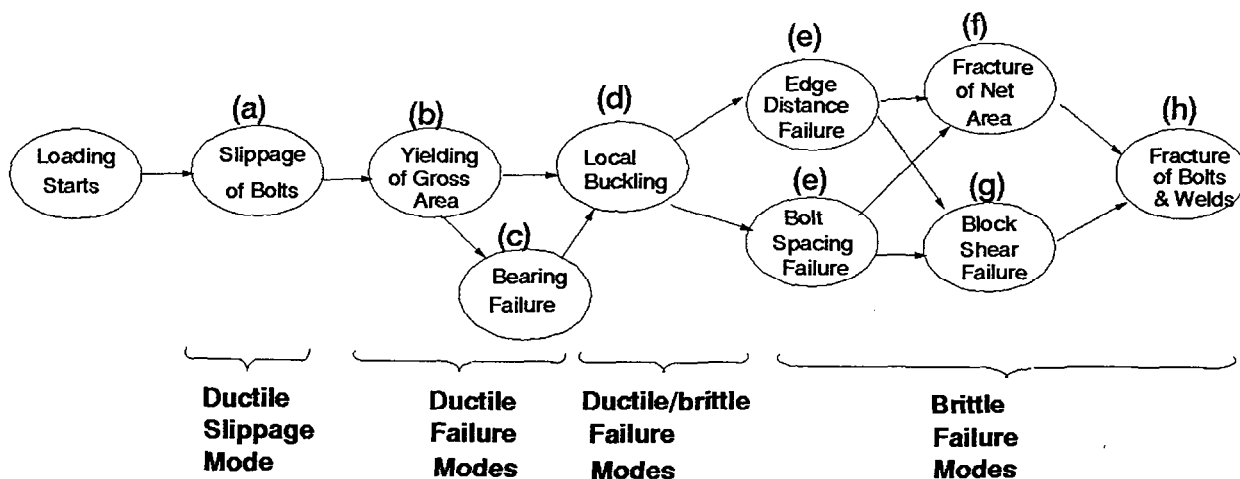


Figure 4.5. Failure Modes of Connection of Bracing Members to Gusset plates in a Hierarchical Order of Desirability

It is recommended that in bolted connections, the bolt groups be designed such that the bolt slippage does not occur until the axial force in the member reaches $0.80 R_y F_y$. This will help to prevent slippage of the bolted connections under service gravity and earthquake loads. However, it will not prevent the slippage of bolts under severe earthquakes. During severe earthquakes, bolt slippage can have many beneficial effects including increasing damping, decreasing stiffness, increasing ductility and elongating the period of vibration, all beneficial in improving the seismic behavior.

To avoid pre-mature local buckling of elements during severe seismic events, the b/t of these elements should not be greater than $52/\sqrt{F_y}$, (AISC, 1997).

To achieve a ductile behavior, it is suggested herein that the yield and bearing capacities of the connections of bracing member to gusset plate be greater than the axial yield capacity of member calculated using expected realistic yield stress of $R_y F_y$. To avoid brittle failure of connections, the capacity of the connection for net section fracture, block shear failure, bolt and weld failure modes should be at least equal or greater than axial tension yield capacity of the bracing member calculated using conservative expected realistic yield stress of $1.1 R_y F_y$.

4.3. Seismic Design of Gusset Plate

Failure modes of a typical gusset plate, in the order of their desirability, are:

- a. Yielding of Whitmore's area of gusset plate,
- b. Yielding of critical sections of gusset plate under combined stresses,
- c. Buckling of gusset plate,
- d. Buckling of edges of gusset plate,
- e. Block shear failure, and;
- f. Fracture of net area of gusset plate.

Figure 4.6 shows failure modes in their hierarchical order of desirability from left to right. In the following sections, design procedures and equations to address these failure modes are provided.

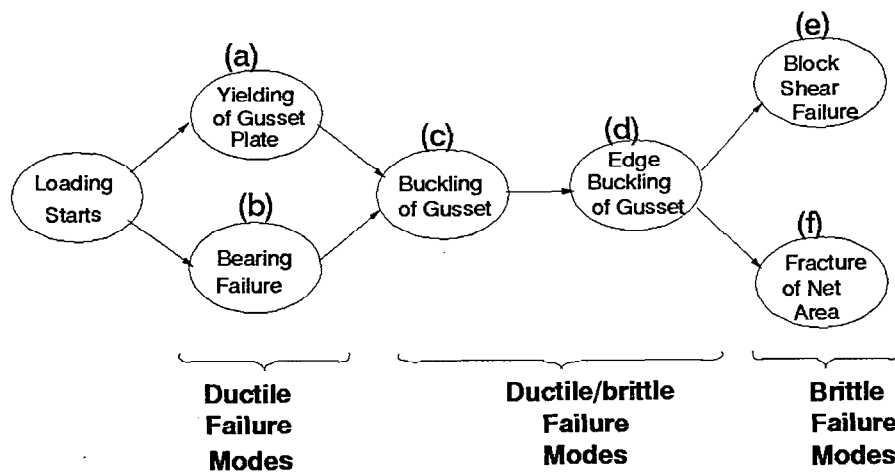


Figure 4.6. Hierarchical Order of Failure Modes of a Gusset Plate

4.3.a. Yielding of Whitmore's area of gusset plate

This is the most desirable failure mode of a gusset plate. Yielding of gusset plates can occur due to direct tension or compression, bending moment, shear and their combinations. Yielding due to direct tension or compression can occur within the Whitmore's effective width area due to "direct" stresses on Whitmore's area. The following equation can be used to establish yield capacity of a gusset plate due to direct axial load.

$$P_y = A_{gw} F_y \tag{4.1}$$

For definition of terms in all equations, see "Notations" at the beginning of this report.

4.3.b. Yielding of gusset plate under combined stresses

Critical sections of gusset plates can yield under a combination of axial load, bending and shear. To establish capacity of gusset plate subjected to combined loads, the following interaction equation is suggested. Since cross sections of gusset plates are normally rectangular, the curve for interaction of axial load, N , and bending moment, M , is a parabola. However, the interaction of shear V , and bending moment, M , is closer to a 4th degree polynomial. Based on these interactions, for interaction of axial force, N , Bending moment, M , and shear, V , the following equation is suggested:

$$(N/\phi N_y)^2 + M/\phi M_p + (V/\phi V_y)^4 \leq 1.0 \quad (4.2)$$

Occasionally, gusset plates do not have regular geometry shown in previous sections of this report. To check the above interaction equation on critical sections of irregular gusset plates, one can resort to free-body diagrams of various parts of gusset plate and establish N , M , and V on the boundaries of these free bodies and check the above equation using N , M and V on the critical sections.

4.3.c. Buckling of gusset plate

Due to direct compression, a gusset plate can buckle in the areas just beyond the end of bracing member as shown in Figure 4.7. To establish buckling capacity of a gusset plate subjected to direct compression, Whitmore's effective width can be used, Figure 4.7.

The following equation can be used to establish buckling capacity of a gusset plate subjected to direct compression:

$$P_{cr} = A_{gw} F_{cr} \quad (4.3)$$

where, F_{cr} is the critical stress acting on 1-inch wide gusset strips within the Whitmore's effective width as shown in Figure 4.7. These 1-inch strips are treated as columns and AISC-LRFD column equations are used to establish F_{cr} . The K , effective length factor for gusset plate is suggested to be taken as 1.2 (Brown, 1988). The relatively conservative value of 1.2 for K can be justified based on test results indicating that there is a possibility of end of bracing member moving out of plane.

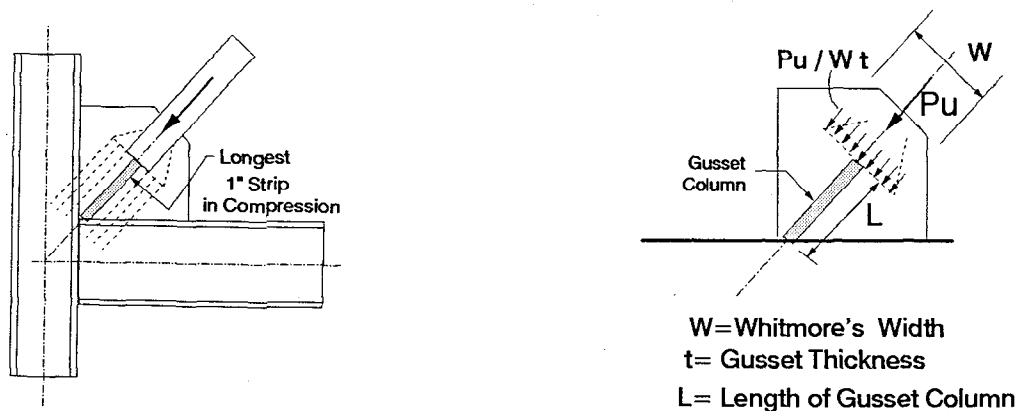


Figure 4.7. Buckling of Gusset Plates and Model to Calculate Buckling Capacity

4.3.d. Buckling of edges of gusset plate

As discussed in Chapter 2, in gusset plates with free edge, the long free edges can buckle. The edge buckling is an undesirable failure mode. Particularly under cyclic load reversals, as shown by Astanteh-Asl (1991), the edge buckling limits the cyclic ductility of the gusset plate. To prevent edge buckling under severe cyclic loading, the following equation, (given as Equation 2.3 in Chapter 2) is proposed by Astanteh-Asl (1991):

$$\frac{L_{fg}}{t} \leq 0.75 \sqrt{\frac{E}{F_y}} \quad (4.4)$$

4.3.e. Block shear failure

Block shear failure is a relatively brittle failure mode and undesirable. To design a ductile gusset plate, this failure mode should not govern over yield failure mode. To ensure that capacity of gusset plate in block shear failure is greater than its capacity in yielding, the following criteria is suggested:

$$\phi_n P_{bs} \geq \phi(1.1R_y P_y) \quad (4.5)$$

where, R_y is the ratio of expected yield strength to specified yield strength. This factor has been introduced into steel design in the aftermath of Northridge earthquake, when it was realized that actual yield point of steel produced today could be higher than the specified value. The AISC Specification (1997) provides a value of $R_y = 1.1$ for plates. In the above equation, P_y is tension yield capacity of gusset given by Equation 4.1., and P_{bs} is the nominal capacity of gusset plate in block shear failure calculated using the AISC Specification (1994) equations:

$$P_{bs} = 0.6R_y F_y A_{gv} + F_u A_{nt} \quad (\text{when } F_u A_{nt} \geq 0.6F_u A_{nv}) \quad (4.6)$$

$$P_{bs} = 0.6F_u A_{nv} + R_y F_y A_{gt} \quad (\text{when } F_u A_{nt} < 0.6F_u A_{nv}) \quad (4.7)$$

The above equations are similar to the equations given by the AISC Specification (1994) for the block shear failure but with one difference that the specified yield stress F_y is multiplied by material factor R_y . It should be mentioned that in some cases where gusset plate is relatively narrow and long, "incomplete" block shear failure could occur. This has been observed in laboratory tests (Astanteh-Asl, 1991) and shown in Figure 2.13 of this report.

4.3.f. Fracture of net area of gusset plate

Fracture of net area of gusset plate is the least desirable failure mode. To ensure that this relatively brittle failure mode does not occur prior to yielding of gusset plate, the following criterion is suggested:

$$\phi_n P_n \geq \phi(1.1R_y P_y) \quad (4.8)$$

Tests of gusset plates, as summarized in Chapter 2, have indicated that the net section fracture in gusset plates occurs within the Whitmore's "effective" area. Therefore, P_n should be calculated using the following equation:

$$P_n = A_{nw} F_u \quad (4.9)$$

4.4. Seismic Design of Connection of Gusset Plate to Its Supports

The first step in design of these connections is to establish forces applied to these connections. Currently, a number of simple methods, all satisfying static equilibrium, are used to establish gusset plate support forces. Figure 4.8 shows four of these methods currently used in design offices. Methods shown in Figure 4.8(a) and (b) are based on resolving the force of bracing member into two concentric or parallel resultants. These two methods have been used in design of gusset plates for decades. Method shown in Figure 4.8(c) was proposed by A. Astaneh-Asl based on his concept of using "Truss Analogy" to establish realistic stresses in the gusset plates (Astaneh-Asl, 1989, 1991). The method shown in Figure 4.8(d) was proposed by W. Thornton.(1991).

After establishing the reaction forces following one of the above methods, the connection of gusset plate to supports can be designed to resist these forces. It should be emphasized that in order to obtain a ductile connection with desirable seismic behavior, design of these connections should be done such that yielding failure modes become the governing failure modes over the fracture modes.

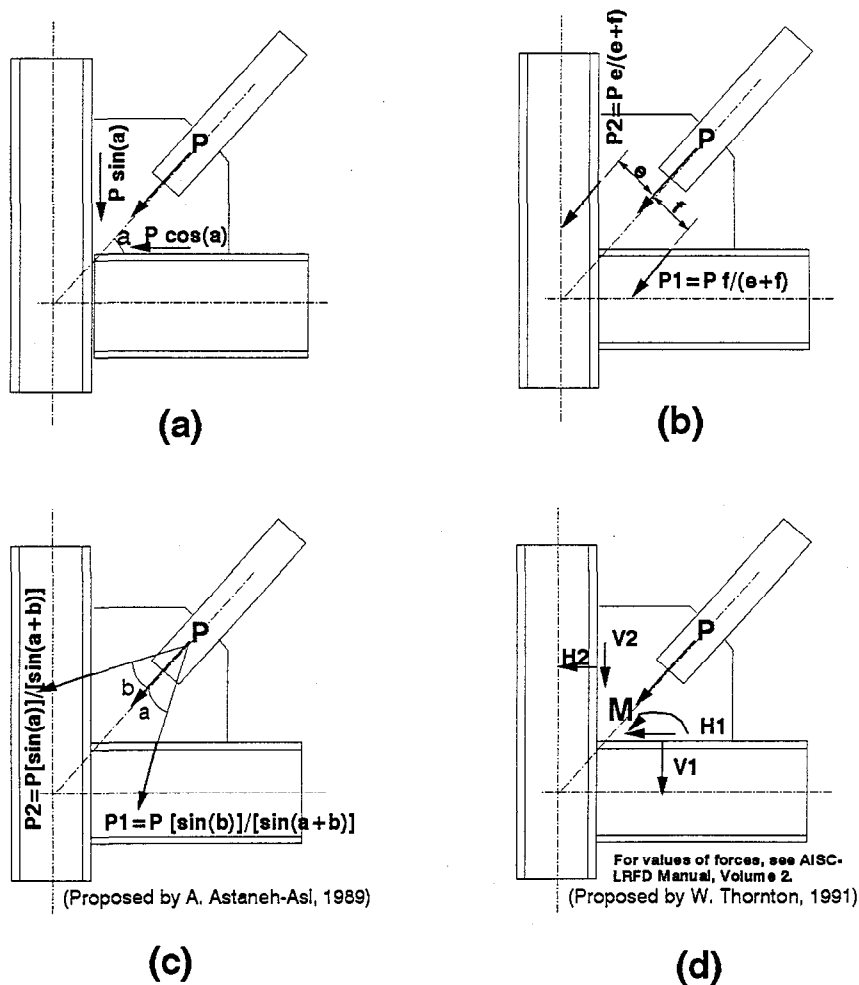


Figure 4.8. Methods Used to Establish Gusset Plate Support Forces

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Funding for this publication provided by the California Iron Workers Administrative Trust.