DESIGN AND BEHAVIOR OF GUSSET PLATE CONNECTIONS

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ABSTRACT

The behavior and design of gusset plate connections are reviewed and summarized. The paper first reviews the existing design methods for proportioning gusset plates under monotonic loading as well as under seismic loading. A summary of recent research at the University of Alberta on the monotonic and cyclic behavior of gusset plate connection is then presented. Based on the monotonic test series, a modified design method is proposed for proportioning the gusset plate to support compressive forces from brace members. The cyclic behavior of gusset plate connections is described using the results of experimental and analytical studies. The interaction between the gusset plate and the brace member is considered in the study. Current research programs are presented regarding the effect of various parameters on the cyclic behavior of gusset plate connections and the potential of a bracing system where the bracing member is designed as the strong element and the gusset plate is designed as the weak element.

INTRODUCTION

Because of the complex behavior of the gusset plate in concentrically braced frames (CBF's), the design of gusset plate connections has traditionally involved highly simplified methods (1-3). Although these methods have proven to be adequate, it is believed that the factor of safety associated with their usage is highly variable (4). Up until recently, the majority of the research on gusset plates has focused on elastic stress distributions or the inelastic behavior of gusset plates loaded monotonically in tension. Relatively little attention has been given to compressive or cyclic behavior. Typically, concentrically braced frames are designed to dissipate energy through yielding or buckling of the brace members. The remaining members and connections are designed to carry the forces that are present in the structure at the load level that causes the brace member to yield or buckle. This design approach embodies the philosophy of capacity design (5).

A series of tests and analytical studies were conducted recently at the University of Alberta (6-10) to investigate the compressive and cyclic behavior of gusset plate connections. In the compression test series (6-7), it was found that when a gusset plate is loaded in monotonic compression, normally significant yielding of the plate takes place before plate buckling. However, if a thin plate is used, the gusset plate can buckle at a load much lower than the yield load. As for the cyclic test series (8), it showed that the tensile capacity of the gusset plate remains stable under cyclic loading. The study also showed that the post-buckling capacity of the gusset plate, although less than the load required to buckle the gusset plate initially, tends to be stabilized after a few cycles. Based on these observations, a design approach that would take advantage of the energy dissipation potential of the gusset plate was proposed (8). This approach consists of designing the gusset plate as the weak element rather than the brace member. The behavior of gusset plate connections under cyclic loading was further verified and studied by a numerical investigation (9) using the finite element program ABAQUS (11) and full-scale tests (10). The analysis considered the effect of gusset plate support condition (rigid support versus flexible support provided by a beam and column assembly), initial imperfections in the plate, material yielding, slip of the bolted connection, bracing member – gusset plate interaction, and load history. The testing program included interaction between bracing member and gusset plate, and effect of edge stiffeners on the cyclic behavior of gusset plates.

The following presents a summary of the behavior of gusset plate connections, in both monotonic and cyclic behavior. The design method for gusset plate connection is reviewed and presented. The weak gusset plate – strong brace member concept is also examined as an alternative for concentrically braced frames (CFB's) under cyclic loading.

BEHAVIOR UNDER MONOTONIC LOADING

An experimental program by Whitmore in the early 1950's (1) studied the stress distribution in a gusset plate connection, a detail commonly found in Warren truss type bridges. The main objective of Whitmore's investigation was to determine the location and magnitude of the peak stress in the gusset plate. Based on the results of his investigation, Whitmore proposed a method for predicting the peak stress in a gusset plate for a given brace load. It was proposed that the peak stress could be estimated by taking the brace load and dividing it by an area equal to the plate thickness times what later became known as the "Whitmore effective width". The Whitmore effective width is defined as the distance between two lines radiating outward at 30° angles from the first row of bolts in the gusset-to-brace connection along a line running through the last row of bolts as shown in Fig. 1.

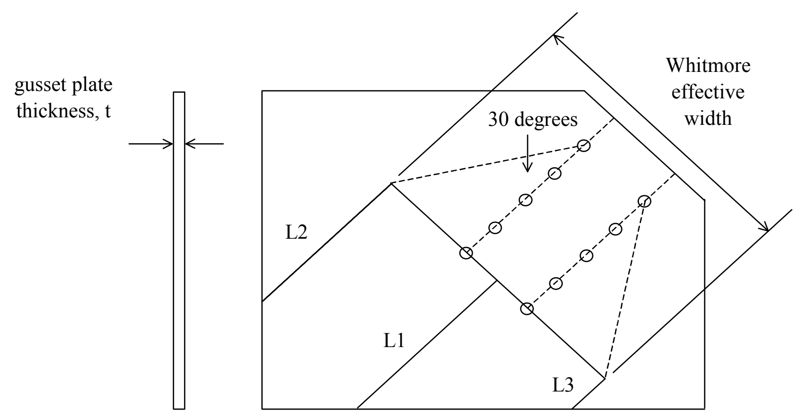


Figure 1 – Whitmore Effective Width

In 1957 Irvan (12) carried out a similar investigation with a model of a double gusset plate Pratt truss connection detail. Irvan's investigation showed that stress distributions computed with the beam method did not match well with test results. Irvan proposed a method of determining the peak stress that was similar to the Whitmore method. Hardin (13), Davis (14), and Varsarelyi (15) investigated the stresses in gusset plates loaded in the elastic range. Hardin's experimental investigation confirmed Irvan's conclusions regarding the beam method and supported Irvan's method for determining the magnitude of the peak stress in the gusset plate. Davis and Varsarelyi carried out finite element investigations of the elastic stresses in gusset plates. In general, these investigations confirmed the findings of the Whitmore's experimental investigations regarding the stresses in gusset plates loaded in the elastic range.

The behavior of gusset plate connections in the inelastic range also received some attention. Chakrabarti and Bjorhovde (16) and Hardash and Bjorhovde (2) looked at the inelastic behavior of gusset plate connections in tension. From their tests and those of other investigators, a block shear model was proposed to predict the ultimate capacity of gusset plate connections in tension. They proposed that the ultimate strength of the gusset plate is the sum of the tensile strength of the net area between the last row of bolts and the shear strength along the connection length.

Thornton (3), investigating the compressive strength of steel gusset plates, proposed an intuitive and lower bound approach whereby the compressive force in the steel gusset plate is carried by an equivalent column between the end of the bracing member and the beam to column joint. The method proposed by Thornton for calculating the elastic buckling load was expanded to include inelastic effect (17). The technique proposed by Thornton is based on the buckling capacity of unit strips of length L1, L2, and L3 (see Fig. 1) below the Whitmore effective width. The critical length of the column strip is taken as the maximum of L1, L2, or L3. Once the length of the column strip has been established, the compressive resistance of the column strip can be evaluated according to the column formulas in the design standards. The effective length factor was recommended to be 0.65. The gusset plate will not buckle if the buckling stress of the critical unit strip is greater than the normal stress on the Whitmore effective area.

Hu and Cheng (6) conducted an experimental and analytical investigation of the buckling behavior of gusset plate connections loaded monotonically in compression. Their test program focused on the effects of plate thickness, geometry, boundary conditions, eccentricity and reinforcement. The work of Hu and Cheng showed that thin gusset plates tend to buckle at a load much lower than the yield load predicted using the Whitmore effective width. In general, either sway or local buckling modes were observed depending on the out-of-plane brace restraint conditions. Further analytical work (18) showed that an increase in the stiffness of the gusset-to-brace splice plate of two to four times the gusset plate thickness should result in an increase in the buckling strength of the gusset plate. It was recommended that the distance between the end of the splice plate and the gusset-to-frame boundaries be kept to a minimum.

To investigate the compressive behavior of gusset plate connections, a total of thirteen full-scale tests were conducted by Yam and Cheng (7). One of the test setups is shown schematically in Fig. 2. The test parameters included gusset plate thickness, size, brace angle, out-of-plane brace restraint conditions, and moments in the framing members. The specimen dimensions and designations are shown in Table 1. The test specimens used in this investigation were stockier than those of Hu and Cheng (6), and, as a consequence, displayed more inelastic behavior. The compressive capacity of the gusset plate specimens was almost directly proportional to their thickness. The effects of beam and column moments and brace angle on the capacity of the test specimens in compression were found to be small.

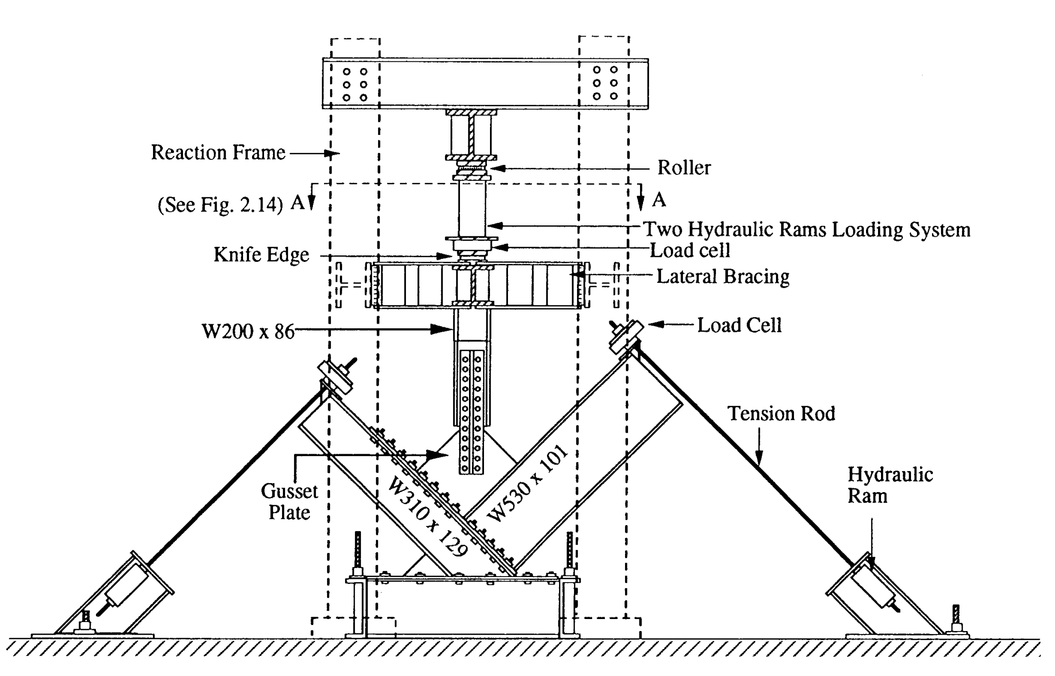


Figure 2 – Typical Test Setup

Table 1 Compressive Specimens Description and Summary of Test Results

Specimen Designation	Plate Size (mm x mm)	Plate Thickness (mm)	Brace Angle	Beam Moment (kN·m)	Column Moment (kN·m)	Ultimate Load (kN)
GP1	500 x 400	13.3	45°	–	–	1956
GP2	500 x 400	9.8	45°	–	–	1356
GP3	500 x 400	6.5	45°	–	–	742
SP1	850 x 700	13.3	45°	–	–	1606
SP2	850 x 700	9.8	45°	–	–	1010
AP1	500 x 400	13.3	30°	–	–	1720
AP2	500 x 400	9.8	30°	–	–	1210
AP3	500 x 400	6.5	30°	–	–	728
MP1	500 x 400	13.3	45°	250	125	1933
MP2	500 x 400	9.8	45°	250	125	1316
MP3	500 x 400	6.5	45°	250	125	721
MP3A	500 x 400	6.5	45°	375	187.5	819
MP3B	500 x 400	6.5	45°	0	0	821

A numerical analysis of the test specimens was subsequently performed (19) by using the finite element program ABAQUS (11). A three dimensional mesh was used to model the connection with the splice member placed on both sides of the gusset plate. The beam and column boundaries were fully restrained to simulate a rigidly welded connection. At the junction between the bracing member and the gusset plate, infinite rotational restraint was imposed on the top of the splice member. Point loads were applied on the splice member to simulate the load transferred from the bracing member. The bolted connection was simulated by rigid beam elements at the bolt locations. To account for the influence of bolt clamping force on the gusset plate, it was assumed that a 30 mm square surface was rigidly connected at the bolt locations by the rigid beam elements. For the MP type specimens, the beam and column were included in the model and allowed the application of beam and column moments. The analytical ultimate loads of the specimens based on the load-deflection analysis are shown together with the test results in Table 2. The analytical predictions are in good agreement with the test results.

To evaluate the validity of current design methods, the Whitmore load (P_W) based on the material static yield strength and the Thornton load (P_T) based on the effective length factor of 0.65 are included in Table 2.

Table 2 Comparison of Test Loads with Analytical and Design Loads

Specimen Designation	Ultimate Load, P_u (kN)	P_{ABAQUS} (kN)	$\frac{P_u}{P_{ABAQUS}}$	Whitmore Load, P_W (kN)	Thornton Load, P_T (kN)	Modified Thornton, P_T' (kN)	$\frac{P_u}{P_W}$	$\frac{P_u}{P_T}$	$\frac{P_u}{P_T'}$
GP1	1956	1987	0.98	1216	1142	1792	1.61	1.71	1.09
GP2	1356	1301	1.04	930	828	1300	1.46	1.64	1.04
GP3	742	710	1.04	555	439	689	1.37	1.69	1.08
SP1	1606	1608	1.00	1852	1072	1744	0.87	1.50	0.92
SP2	1010	993	1.02	1416	592	963	0.71	1.70	1.05
AP1	1720	1680	1.02	1216	1119	1757	1.56	1.54	0.98
AP2	1210	1177	1.03	930	801	1257	1.55	1.51	0.96
AP3	728	732	0.99	555	404	634	1.31	1.80	1.15
MP1	1933	1901	1.02	1216	1142	1792	1.59	1.69	1.08
MP2	1316	1348	0.98	930	828	1300	1.42	1.59	1.01
MP3	721	700	1.03	555	439	689	1.30	1.64	1.05
MP3A	819	805	1.02	555	439	689	1.48	1.87	1.19
MP3B	821	725	1.13	555	439	689	1.48	1.87	1.19

As can be seen from the table, the Whitmore method provides a conservative estimate of the design load for the specimens, except for the SP type specimens since the SP type specimens are more susceptible to the stability failure. The Thornton method, however, provides conservative estimates for all the specimens. The test to predicted ratios varied from 0.71 to 1.61 for the Whitmore method and from 1.31 to 1.87 for Thornton's method. The reason for the conservatism observed with Thornton's method is due to the extensive yielding in most of the specimens. Yielding allowed load redistribution in the specimens. In order to account for this load redistribution, it is proposed that a 45° dispersion angle be used to evaluate the effective width, instead of 30°. This modification of Thornton's method is then used to calculate the buckling strength (P_T') of the specimens using extended effective width and the appropriate column curves. The values of P_T' are listed in Table 2. The ratio of test loads to the modified Thornton loads varies from 0.92 to 1.19.

BEHAVIOR UNDER CYCLIC LOADING

Compared to the information available on the cyclic behavior of bracing members, the amount of information on the cyclic behavior of gusset plates is quite small. Jain et al. (20) studied the effect of gusset plate bending stiffness and bracing member length on the cyclic behavior of bracing members. Although the bracing member was the main subject of the investigation, three different gusset plates were used and the length of the brace member was varied. From a test program comprising 18 test specimens, none were designed to have the yield strength of the gusset plate lower than the bracing member. It was concluded that there is no advantage in making the flexural stiffness of the gusset plate greater than the flexural stiffness of the bracing member. An increase in flexural stiffness of the gusset plate was found to result in a decrease in the effective length of the bracing member. This, in turn, improves the cyclic behavior of the bracing member.

Astaneh-Asl et al. (21) studied the cyclic behavior of brace members composed of back-to-back double angles connected to gusset plates. The focus of their investigation was also the behavior of the bracing member. Both in-plane and out-of-plane buckling of the brace member was investigated. Single gusset plates connected only to a beam element were used in the investigation. Current code design procedures were found to be deficient. For brace members that buckle out-of-plane, Astaneh-Asl et al. stressed the importance of designing the gusset plates so that they can accommodate the formation of a plastic hinge, allowing brace buckling to take place without tearing of the connection.

Rabinovitch and Cheng (8) tested five full-scale specimens to study the behavior of gusset plate connections under cyclic loading. The effects of plate thickness, geometry, edge stiffness, and bolt slip were studied. With the exception of one test specimen, all the test specimens were rectangular and similar to the specimens tested in the compression series by Yam and Cheng (7). One test specimen, however, was designed to allow the free formation of a plastic hinge under compressive buckling deformation as per the recommendation of Astaneh-Asl et al. (21). The behavior of this latter test specimen was significantly different from that of the other specimens, it failed more rapidly than other more compact specimens and fracture was observed at the plastic hinge closed to beam and column. It also required much larger plate to accommodate the plastic hinge requirement. All other four specimens were governed by tensile fracture between last row of bolts in the gusset plate as was observed in the earlier tests by Chakrabarti and Bjorhovde (16). The tensile capacity of gusset plates under cyclic loading remained stable and the post-buckling compressive capacity tends to be stabilized after buckling. The addition of edge stiffeners was found to significantly improve the post buckling compressive strength and the energy dissipation characteristics of the gusset plate tested.

Based on the stable post-buckling strength of gusset plates under compression, a design approach that would take advantage of the energy dissipation potential of the gusset plate was proposed (8). This approach, referred to as the “weak gusset plate – strong brace member” concept, consists of designing the gusset plate as the weak element rather than the brace member. Nast et al. (10) tested four gusset plate – bracing member subassemblies, as shown in Table 3, to further investigate this concept. The first two test specimens were designed with the gusset plate as the weak element in compression; one with free edge stiffeners, and the second one without. The other two were designed with the brace member as the load-limiting element in compression; again one included free edge stiffeners, and the other did not.

Table 3 Summary of Test Specimens under Cyclic Loading

Specimen	Gusset Plate (mm x mm x mm)	Free Edge Stiffener Width x Thickness (mm x mm)	Bracing Member (mm)	Ultimate Tensile Load (kN)	Ultimate Compressive Load (kN)
T-1	450 x 550 x 9.5	50.7 x 9.5	1100	N/A	1951
T-2	450 x 550 x 9.6	N/A	1100	1819	1690
T-3	450 x 550 x 9.5	50.5 x 9.5	5250	1837	1350
T-4	550 x 450 x 9.5	N/A	5250	1841	1322

The axial load vs. axial deformation hysteresis plots for these four specimens are presented in Fig. 3. The ultimate tensile and compressive loads, based on the envelopes of the hysteresis curves, are listed in Table 3. Both tensile and compressive strengths were predicted accurately by the block shear model and modified Thornton’s method, respectively. The presence of free edge stiffeners does not have a significant effect on the gusset plate buckling strength, but increases the energy absorbed by the gusset plate – brace member assembly in the compression cycle. Little effect in the tension cycle by the stiffeners was observed.

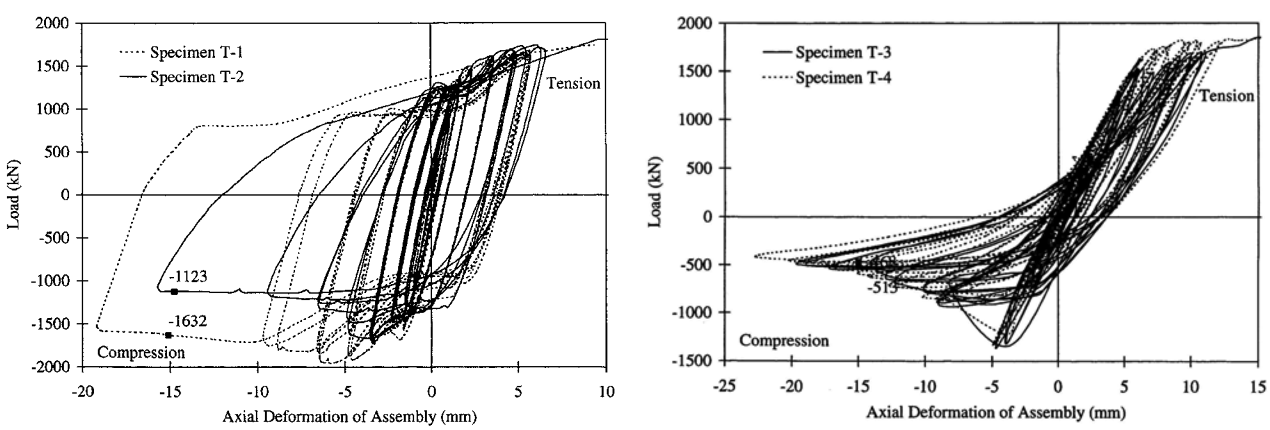


Figure 3 – Behaviour of Gusset Plate Assembly Under Cyclic Loads

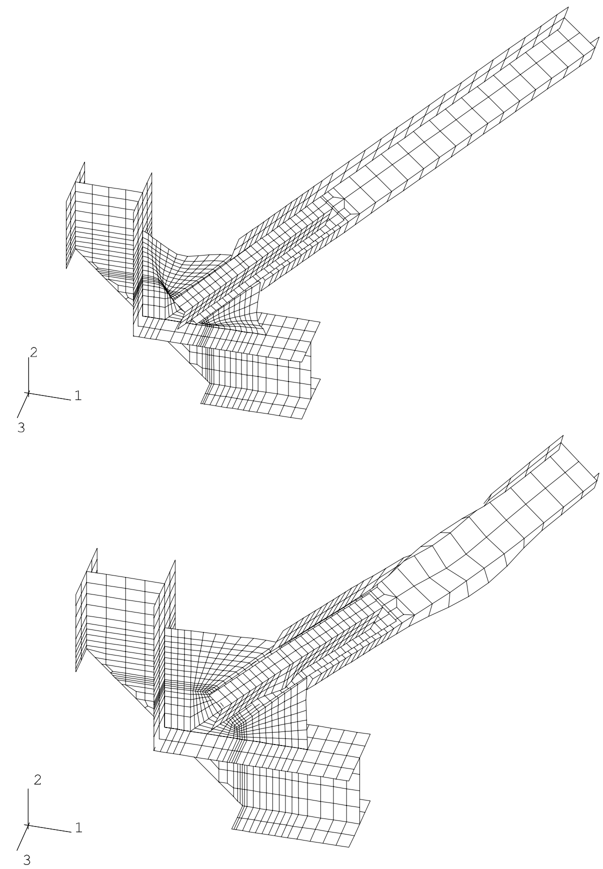
As shown in Fig. 3, the energy dissipated by Specimens T-1 and T-2 was much higher than Specimens T-3 and T-4. Yielding of the gusset plate in compression in the first two tests helped the connection to dissipate significantly more energy than buckling of the bracing member in the other two tests. However, all the connections failed in tension in the gusset plate with a relatively small deformation. This may limit the use of the "weak gusset plate – strong brace member" concept in seismic applications.

Walbridge et al. (9) investigated analytically the behavior of gusset plate – brace member assemblies for a number of parameters that were not investigated experimentally. A parametric study was conducted to study the effects of gusset plate – brace member interaction and load sequence on the behavior of steel gusset plate connections under cyclic loading. The various factors affecting the behavior and energy dissipation characteristics of gusset plate connections under cyclic loading conditions were considered.

Four types of behavior were studied for the gusset plate – brace member assemblies:
1) Effect of the bracing member yielding in tension before yielding of the gusset plate;
2) Effect of the gusset plate yielding in tension before yielding of the brace member;
3) Effect of buckling of the brace member before buckling of the gusset plate;
4) Effect of buckling of the gusset plate before buckling of the bracing member.

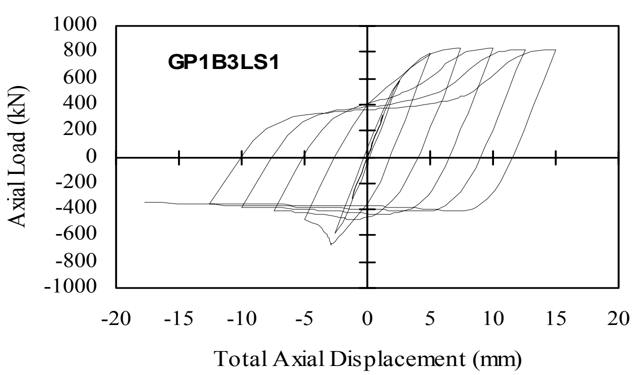
The models investigated were designed to investigate the above four cases for a 450 x 550 mm gusset plate of three different plate thicknesses, 6, 9, and 12 mm. The capacity in the tension cycle was either limited by yielding of the brace member or yielding of the gusset plate, depending on the design adopted for the specimen. In the compression cycle, the load carrying capacity was limited by either buckling of the gusset plate as shown in Fig. 4(a), or by buckling of the brace member about its weak axis, i.e. out of the plane of the gusset plate, as shown in Fig. 4(b). The specimens used in the analysis were designed so that each combination of tension and compression load limitation mechanisms would be exhibited.
The results of this investigation showed that there is very little effect on the behavior the assembly by different load sequences. As for the plate thickness, the analyses showed that the thicker gusset plate gave fuller hysteresis loop for the assemblies having a tension capacity limited by yielding of the gusset plate and a compression capacity limited by buckling of the gusset plate.

Fig. 5 shows the difference in behavior of the gusset plate – brace member assembly for different load limitation mechanisms for a 6 mm gusset plate (GP1). Fig. 5(a) represents the behavior when the load in tension is limited by yielding of the gusset plate and the load in compression is limited by buckling of the gusset plate. Fig. 15(b) represents the behavior when the limiting condition in tension is yielding of the gusset plate and the limiting condition in compression is buckling of the brace member. A comparison of Fig. 5(a) with Fig. 5(b) shows that buckling of the brace member as a limiting condition in the compression range results in a more significant reduction in compression capacity under cyclic loading and a deterioration of the load carrying capacity in tension. This reduction in tension stiffness at zero load can be quite significant when the compression capacity is limited by buckling of the bracing member, as shown in Fig. 5(b). This reduction in tension stiffness was not observed when buckling of the gusset plate limited the compression capacity. The same observation as Fig. 5(a) was made when the limiting condition in tension is yielding of the brace member and the limiting condition in compression is buckling of the gusset plate.

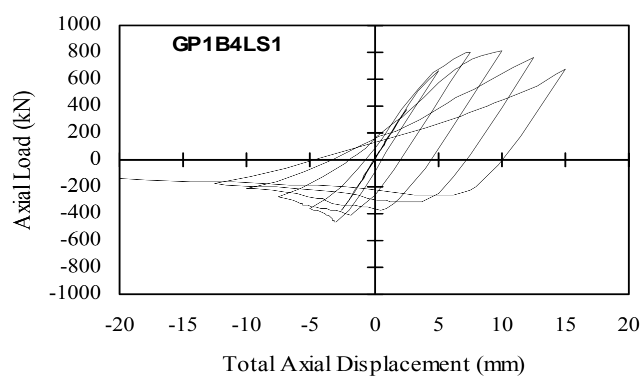


(a) Buckling of Gusset Plate (b) Buckling of Brace Member

Figure 4 – Failure Modes of Gusset Plate – Brace Member Assembly



(a) Gusset plate yielding – gusset plate buckling



(b) Gusset plate yielding – brace buckling

Figure 5 – Effect of Load Limitation

SUMMARY AND CONCLUSIONS

Recent developments in the behavior of gusset plate connections, both monotonic and cyclic behavior, were reviewed and summarized in the paper. Full-scale tests were conducted under both monotonic compression and cyclic loading. Numerical investigations were carried out using the finite element method, incorporating the effect of material and geometry non-linearity and initial imperfections.

The tensile strength of gusset plates can be predicted accurately by the block shear failure proposed by Chakrabarti and Bjorhovde (16). Based on the monotonic compression test results, a modified Thornton method has been proposed for proportioning the gusset plate to support compressive forces from brace members. In the modified Thornton method, a 45° dispersion angle is proposed to evaluate the effective width, instead of 30°, to account the load redistribution in the gusset plate. Good agreement is obtained with the test results by using the proposed method.

From the cyclic tests, the tensile capacity of gusset plates under cyclic loading remained stable and the post-buckling compressive capacity tends to be stabilized after buckling. The addition of edge stiffeners was found to have little effect on the gusset plate buckling strength but significantly improve the post buckling compressive performance and hence increase the energy absorbed by the gusset plate – brace member assembly in the compression cycle. This observation might prove the validity of the "weak gusset plate – strong brace member" concept, consists of designing the gusset plate as the weak element rather than the brace member. Further tests and analyses on the interaction between gusset plate and brace member also showed that yielding and stable post-buckling strength of the gusset plate in compression helped the connection to dissipate significantly more energy than buckling of the bracing member. In general, hysteresis plots for the weak gusset – strong brace member models exhibited less pinching and sustained higher post-buckling compressive loads than conventionally designed subassemblies. However, all the connections failed in tension in the gusset plate with a relatively small deformation. This may limit the use of the "weak gusset plate – strong brace member" concept in seismic applications.

Currently, a research effort is in progress to enhance the tensile performance of a gusset plate. Two approaches are used. One is to utilize the post-fracture behavior of the gusset plate. Leon and Swanson (22) showed that a well designed gusset plate could possess significant post-fracture capacity. The other approach is to provide an active fracture control mechanism in the gusset plate. The ultimate goal of the project is to assess the effectiveness of using gusset plates as the energy dissipators in seismically loaded braced structures.

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