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SEISMIC PERFORMANCE OF SCBF BRACED FRAME GUSSET PLATE CONNECTIONS

Charles W. Roeder[1], Dawn E. Lehman[2], Shawn Johnson[3], David Herman[3], and Jung Han Yoo[3]

ABSTRACT

Concentrically braced frames (CBFs) are desirable for seismic design. They are stiff and strong for preventing damage during small, frequent events, but Special CBFs (SCBFs) dissipate energy during severe, infrequent earthquakes through tensile yielding and post buckling inelastic deformation. Brace buckling places significant force and deformation demands on the brace-beam-column gusset plate connections. Variations of the AISC Uniform Force Method are used for the design of SCBF gusset plate connections, but their seismic behavior is not well understood and may not meet expectations. An analytical and experimental study into the seismic performance of CBFs and their gusset plate connections is described. The current connection design method, past research results, and results of the current research study are described. The results show improved SCBF performance may be achieved by permitting greater flexibility and inelastic deformation in the gusset plate and by designing the gusset plate welds based upon the plastic capacity of the gusset plate. This proposed design procedure balances the resistances predicted for the various yield mechanisms and failure modes of the brace and the connection.

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Keywords: Braced frames, Concentrically braced frames, Connections, Gusset plate, Welds

INTRODUCTION

Concentrically braced frames (CBFs) are commonly used for seismic design, because their strength and stiffness economically satisfies serviceability design limit states. Economical design for safety and collapse prevention requires that the structure tolerate large inelastic deformations and inelastic ductility during large infrequent earthquakes. CBFs develop cyclic inelastic deformation through axial yielding and post buckling deformation of the brace as illustrated in Figs. 1 and 2. The brace initially buckles in Zone 0-A shown in Fig. 1b, and plastic hinges form within the brace after buckling in Zone A-B, because of the P-δ moments. These hinges cause permanent plastic deformations and deterioration of brace resistance. Cyclic load reversals lead to the one-sided axial force-deflection behavior of the braced shown in Fig. 1a. Because of the one sided performance, SCBFs use braces in opposing pairs, and the resulting system has inelastic hysteretic behavior illustrated in Fig. 1c.

Braces are normally joined to the beams and columns of the braced frame through gusset plate connections such as illustrated in Fig. 3. Brace post buckling behavior of the brace places significant cyclic load and deformation demands on these connections. The connection must tolerate these end

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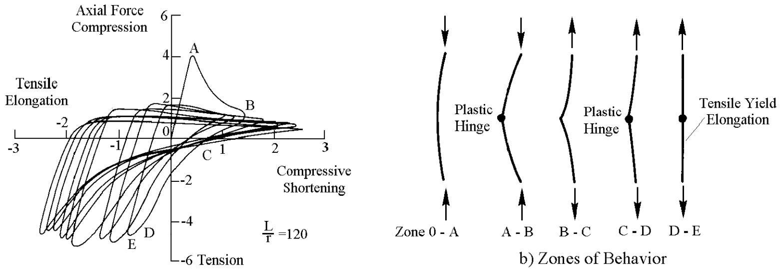
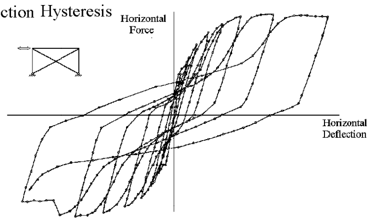
[1] Professor of Civil Engineering, University of Washington, Seattle, WA 98195-2700 USA, croeder@u.washington.edu
[2] Asst. Professor of Civil Engineering, University of Washington, Seattle, WA 98195-2700, delehman@u.washington.edu
[3] Former Graduate Assistant, University of Washington, Seattle, WA 98195-2700

rotations and the resulting brace bending moments, while developing the compressive buckling and tensile yield capacity of the brace.



a) Brace Axial Force-Deflection Hysteresis

b) Zones of Behavior



c) Resulting Frame Lateral Load-Deflection Hysteresis

Figure 1. Force-deflection behavior of CBFs

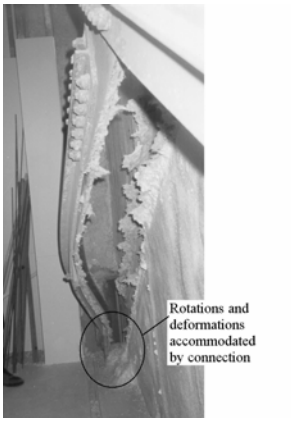


Figure 2. Photo of buckled brace

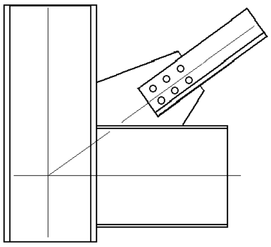


Figure 3. Gusset plate connection

CURRENT DESIGN REQUIREMENTS

Seismic design of gusset plate connections is performed to address these issues. The connections are designed by variations of the AISC Uniform Force Method (UFM) (AISC 2005a). The UFM was not originally developed for seismic design, but it has been adapted to seismic design through the AISC Seismic Design Provisions (AISC 2005b). For seismic design, the brace, beam and column are initially sized to provide the required seismic resistance (IBC 2003, ASCE 2005). With these member sizes, the expected resistances, P_{ut} and P_{uc}, of the brace are determined for tension and compression, respectively loading.

$$P_{ut} = R_y F_y A_g \tag{1a}$$

and

$$P_{uc} = R_y F_{cr} A_g \tag{1b}$$

The weld or bolt group joining the brace to the gusset plate are sized to resist these expected brace capacities, and the gusset plate (and brace if necessary) is checked for block shear failure through the bolt or weld group. These calculations establish a size, length and spacing for the bolts or welds, and they provide a minimum thickness for the gusset plate. The Whitmore width of the gusset plate is then established based upon the geometry of the bolt or weld group as depicted in Fig. 4. With welded high strength steel tubes (see Fig. 4a), the net section in the tube occurs at the end of the slot cut into the tube to slip over the gusset plate, and the net section of the gusset plate is effectively the area within the Whitmore width. For bolted connections (see Fig. 4b), the net section of the brace occurs at the first row of bolts in the brace, and the net section of the gusset plate is the area associated with the Whitmore width minus reduction for the bolt holes at the last row of bolts. The ultimate tensile capacity is checked at these net sections. SCBF frames are dominated by brace buckling, and a 2 t_p orthogonal clearance is used to assure that gusset plate will permit the end rotation necessary for brace buckling. This clearance results in a significant increase in the dimensions of the gusset plate as shown in Fig. 4. The Whitmore width is then used to establish the buckling capacity of the gusset plate. There are a several different methods for establishing this buckling capacity, but these methods are based upon the slenderness of the gusset plate and the compressive area associated with the Whitmore width. In some cases, the clear distance from the Whitmore width to the beam along the centroidal axis of the brace is used for the buckling length, and in other cases, an average of several lengths are used as illustrated in Fig. 4. Gusset plate buckling checks may increase the final gusset plate thickness. The forces on the welds between the gusset plate and the beam and column are then established by equilibrium with the maximum expected capacity of the brace, P_{ut}. The welds are sized for these forces and any eccentricity that may result. In each of these calculations, the ultimate capacity of brace, P_{ut} and P_{uc}, should satisfy the inequalities

$$P_{uc} < \phi R_n \tag{2a}$$

and

$$P_{ut} < \phi R_n \tag{2a}$$

Critical behaviors such as weld fracture require that the factored resistance be greater than 110% of the expected brace resistance to avoid premature brittle failure. These inequalities are particularly significant for the net section check, because the resistance factor, ϕ, for net section fracture is 0.75. As a consequence strict application of this limit often results in large, uneconomical brace reinforcement requirements, and a somewhat more generous net section limit check has evolved in the 2005 provisions (AISC 2005b).

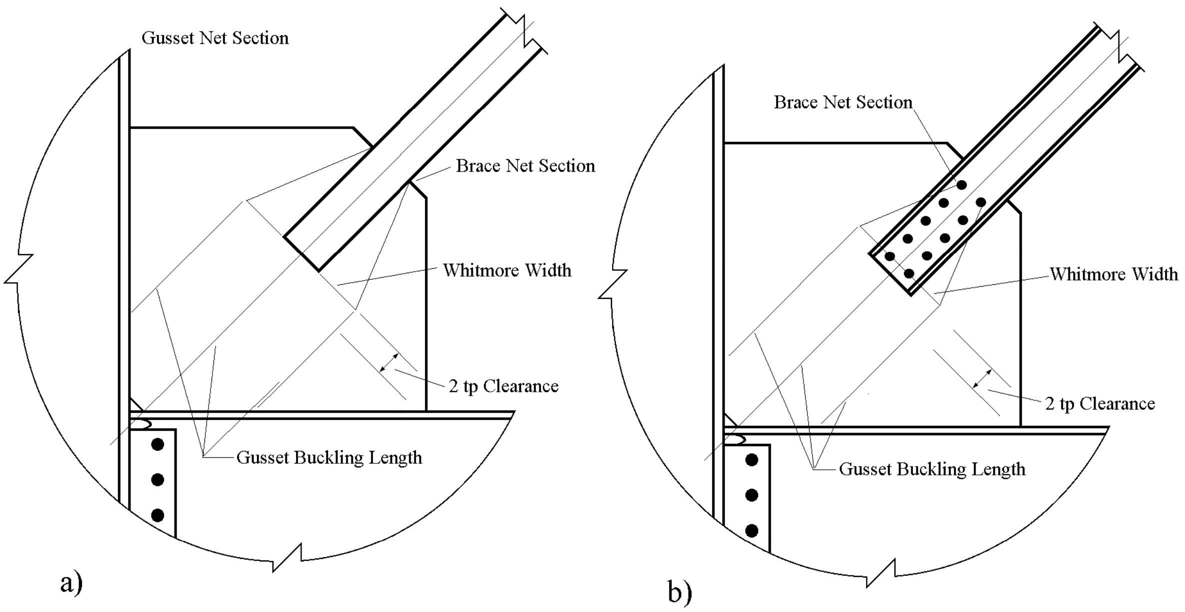
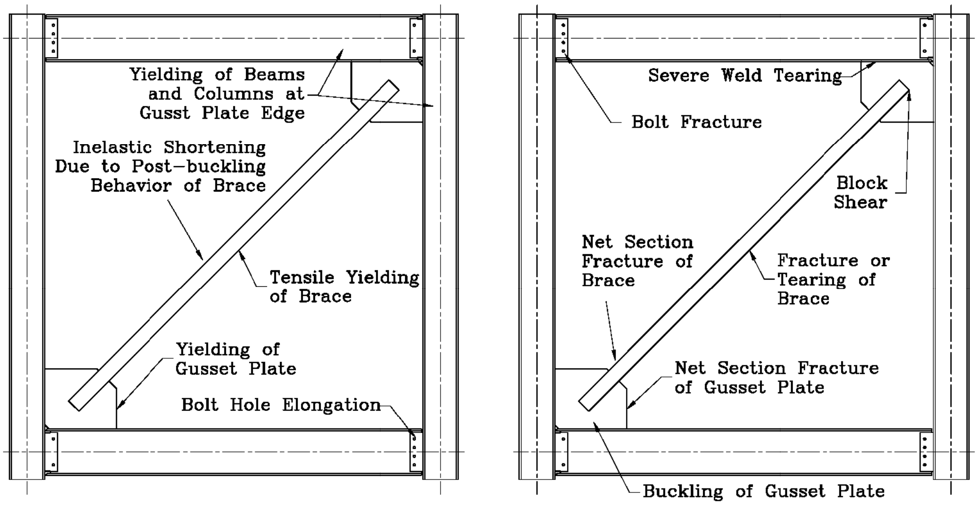


Figure 4. UFM gusset plate design parameters, a) HSS tubular brace welded to gusset, b) Channel brace bolted to gusset

The beam-column connection is then formed. Designer preference plays a major role in this part of the connection design, and wide variations in the beam-column connection design can be seen. A large component of force is transferred from the gusset plate to the beam flange, and this force normally must be transferred to the work point in the column. As a result, some engineers use complete joint penetration (CJP) welds to join the web and flanges of the beam to the column. These welded flange joints develop large axial load capacity in the beam members. Other engineers regard these beam flange welds as overly expensive and unnecessary. They may use a shear tab with fillet welds between the shear tab and the beam web with free flanges at the connection. In still other cases, large gusset plates are used which incorporate both the beam web and the gusset plate connection. The AISC specification limits the sharing of loads at a connection interface between bolts and welds, because of the different deformation characteristics of bolts and welds. However, some connections appear to fail this requirement. These diverse possibilities lead to a wide range of gusset plate connections in engineering practice. The performance of these connections is likely to be mixed.

PROPOSED DESIGN METHOD

To meet these diverse objectives, a seismic design methodology based on balancing the yield mechanisms and preventing undesirable failure modes is proposed. In traditional seismic design, elements initially are designed to meet the elastic force demands. The specific elements that are expected to yield (e.g., braces in CBFs) are designed to sustain inelastic deformation. Capacity design principles are used to design the adjacent elements and assure that the ductile element yields during extreme seismic events. The current design method is in effect a capacity design approach. This capacity design method may assure that the ductile element yields, but it does not guarantee adequate system ductility and inelastic deformation capacity, because it does not control the sequence of yielding or the extent of yield deformation.



a) b)

Figure 5. Typical behaviors for SCBFs; a) Yield mechanisms, b) Failure modes

The proposed balanced design approach is similar to traditional capacity design methods in that the framing elements are designed to meet the elastic force demands. The brace is designed to have tensile yield and post-buckling inelastic deformation, but it is also designed to achieve the desired plastic mechanism of the system. Greater ductility is achieved with the proposed method by assuring that multiple, desirable yield mechanisms are developed prior to initial fracture or failure. This process will easily satisfy serviceability design limits, since all members will have yield resistance greater than a specified seismic resistance, and CBFs are inherently stiff structures for elastic behavior. Collapse prevention and life safety design objectives require significant ductility of the structural system, and a sequence of yielding and duration of yield deformation are needed. This sequence of yielding assures that significant inelastic deformation can occur before undesirable failure modes are permitted. The balancing procedure further requires a sequencing of the failure modes to provide optimal ductility and inelastic deformation capacity. Further, proportional separation is provided between desirable and less desirable behaviors to reduce the probability of less acceptable yield mechanisms and failure modes occurring prematurely.

Figure 5 illustrates the possible yield mechanisms and failure modes for CBFs, and Eq. 3 expresses the proposed balance procedure as:

$$R_{yield\ mean} = R_y R_{yield} \leq \beta_{y1} R_y R_{yield,1} \leq \beta_{y\,2} R_y R_{yield,2} \ldots \leq \beta_{y\,i} R_y R_{yield,i} \qquad (3)$$

The nominal yield resistances, R_{yield}, for the various yield mechanisms are separated by a balance factor, β, to control the resistance of secondary yield mechanisms to achieve the balance state. Using this inequality for design, a yielding hierarchy is established. The primary yield mechanism is followed by secondary mechanism 1, which is followed by secondary mechanism 2, etc. The β values for the separation required to achieve ductile performance of the connection are based on the experimental performance and the separation required to achieve the performance goals in past research studies. These β have similar characteristics to the ϕ values commonly used load and resistance factor design, but they are fundamentally different, since they are selected to achieve ductility and inelastic deformation capacity rather than strength considerations.

Research shows that the formation of secondary (and subsequent) yield mechanisms increases the deformation capacity of the system and prevents premature failure of the connection (Roeder 2002). However, development of failure modes causes fracture, tearing, or deterioration of performance. Achieving a single failure mode does not necessarily imply collapse or total failure of the connection; multiple failure modes are usually required to achieve this extreme condition. However, a single failure mode results in significant, irrecoverable damage to the system. As a result, the balancing procedure shown in Eq. 4 is needed to separate and balance critical failure mode resistances.

$$R_{yield\ mean} = R_y R_{yield} < \beta_{fail,1} R_{fail,1} < \beta_{fail,2} R_{fail,2} \ \ldots \text{ and } \beta_{yield} < \beta_{fail} \qquad (4)$$

This balanced design approach requires that the resistance of all failure modes, R_{fail}, exceed the strength of the primary yield mechanism, and that less favorable failure modes have greater separation than more favorable failure behaviors. Each yield and failure balance parameter, β_{yield} and β_{fail}, is calculated as the ratio of the mean or expected value of the experimentally observed behavior, and the value is increased or decreased based upon the desirability of the outcome for that particular behavior combination. An earlier paper (Roeder, Lehman and Yoo 2005) has discussed this balance procedure and the rationale for establishing the β factors.

EXPERIMENTAL PROGRAM

This balance procedure requires a comprehensive understanding of CBF, brace and gusset plate behaviors. As a result, an experimental program has been initiated at the University of Washington to study is the seismic performance of gusset plate connections and the SCBF system. The experiments are performed on braced frame subassemblages, which are tested in a horizontal position as illustrated in Fig. 6. The frame assemblies are full scale simulations of a single braced bay typical of a full scale frame required for a 3 or 4 story building. The frames include beams above and below the brace, gusset plate connections at each end of the brace, and columns to complete the single frame assembly. The SCBF braces are HSS 125x125x9.5 mm tubes. The columns are W12x72, and the beams are usually W16x45 wide flange sections. The gusset plate connections are varied from specimen to specimen to evaluate:

- the current AISC and Uniform Force Method design procedures,
- the weld requirements between the gusset plate and the beam and column,
- the 2 t_p clearance requirements for brace end rotation,
- the thickness of the gusset plate,
- the relative stiffness of the brace, gusset plate and framing members, and

- the relative performance of tapered and rectangular gusset plates.

The frames are tested through a cyclic inelastic deformation history based upon the ATC24 testing (ATC 1992) protocol. This protocol requires a number of elastic cycles, which are used to establish elastic stiffness and resistance, and repeated cycles of inelastic deformation with increasing amplitude.

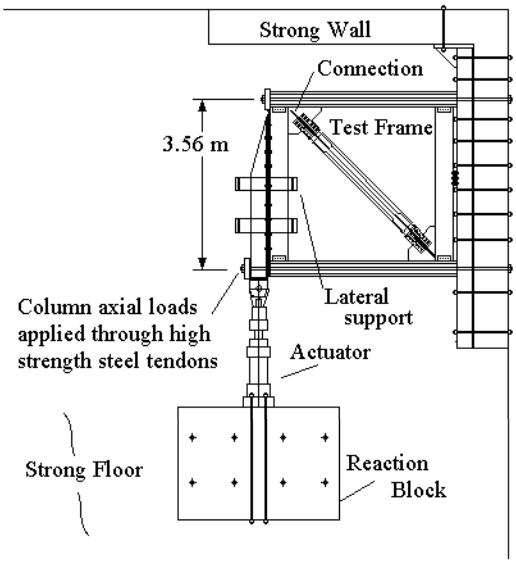
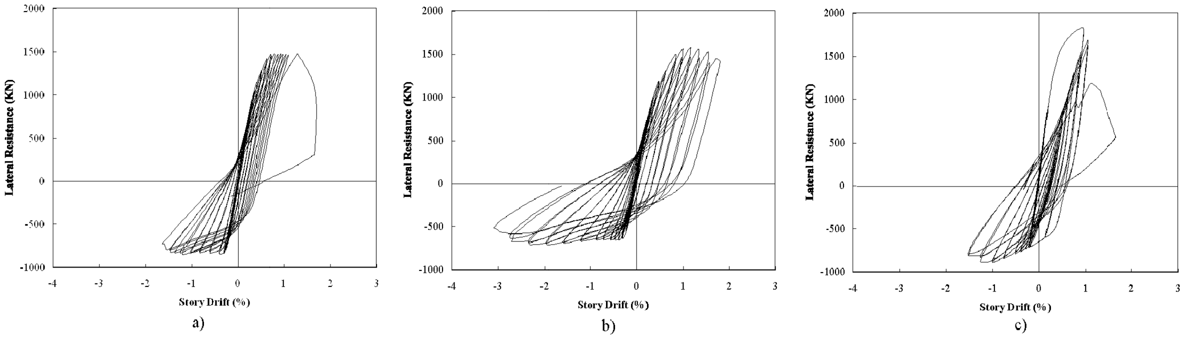


Figure 6. Schematic of test setup

As noted earlier, the goal of the research is to investigate the seismic performance and improve the design methods for SCBF systems and gusset plate connections. As a result, eleven SCBF frames and 5 buckling restrained braced frames have been tested to date. Table 1 summarizes these SCBF experiments. The experimental results are too lengthy for a comprehensive discussion in this paper. However, specific comparisons will be made to aid in drawing important initial conclusions from these research results. Figure 7 provides the force-deflection curves for 3 of the 11 test specimens, and Fig. 8 shows the specific details of the gusset plate connections for these specimens. Figures 7a and 8a shows the force-deflection behavior and connection details for specimen HSS1, respectively. HSS1 was designed by the current UFM method with the 2 t_p buckling clearance and with welds designed to the expected resistance of the brace, and the ductility of this specimen was very limited. Ductile weld tearing of the fillet welds joining the gusset plate to the beam and column was noted at relatively small inelastic deformations, and fracture of these welds occurred early in the test. Figure 9a shows a photograph of this weld fracture. This and other test and analysis results have shown that it is essential to design the gusset plate welds to develop the plastic capacity of the gusset plate rather than the plastic capacity of the brace.

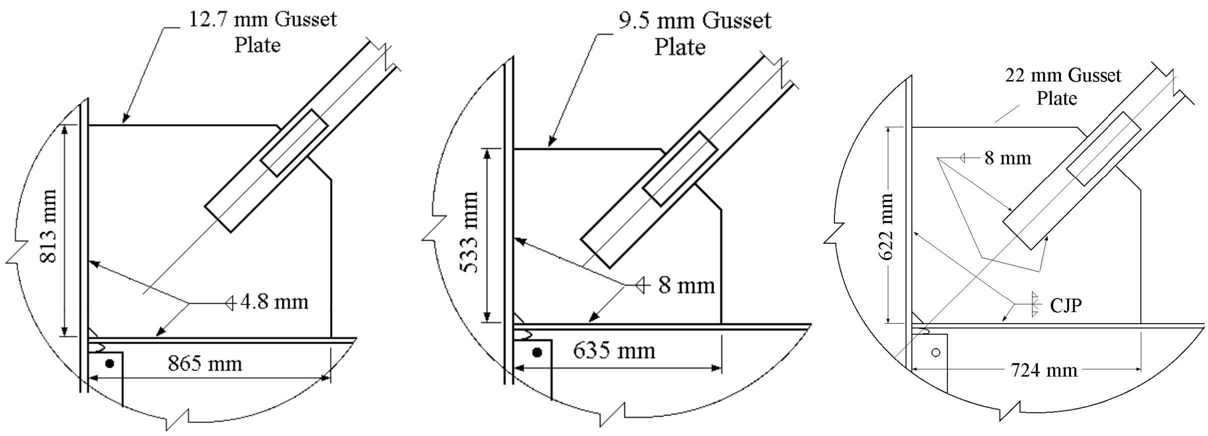


a)

b)

c)

Figure 7. Cyclic force-deflection behaviors; a) HSS1, b) HSS5, and c) HSS11



a) b) c)

Figure 8. Gusset plate design; a) HSS1, b) HSS5, and c) HSS11

Table 1. Summary of test program

Specimen Identification	Specimen Description	Failure	Maximum Deformation Range
HSS1	Baseline specimen designed by UFM with the 2 t_p linear clearance. Results in large gusset plate.	Brace buckling led to abrupt weld fracture	2.6%
HSS2	Weld sized to plastic capacity of plate and 6 t_p elliptical clearance model employed.	Brace fracture	4.0%
HSS3	Similar to HSS2 but thinner more flexible gusset plate	Brace fracture	4.6%
HSS4	Similar to HSS3 but with 8 t_p elliptical clearance	Brace fracture	4.6%
HSS5	Similar to HSS3 but slightly stronger fillet weld	Brace fracture	4.8%
HSS6	Similar to HSS5 but weld edges reinforced to delay initiation of fillet weld cracking	Brace fracture	4.7%
HSS7	Thicker gusset plate with fillet weld and 6 t_p elliptical clearance	Brace fracture	3.9%
HSS8	Similar to HSS3 but with 3 t_p elliptical clearance	Brace fracture	4.6%
HSS9	Slightly thicker gusset plate with CJP weld	Brace fracture	3.6%
HSS10	Tapered gusset plate	Brace fracture	4.4%
HSS11	Thick gusset plate with heavy beam	Brace fracture	2.4%

Specimen HSS1 employed a linear 2 t_p buckling clearance as required by the AISC seismic requirements (AISC 2005b) and illustrated in Fig. 4. This requirement results in relatively large gusset plates, which create a relatively large rigid zone in the connection region. This rigid zone induces significant local yield deformation in the beam and column adjacent to the gusset plate, and has a significant impact on frame performance. As a result, alternate clearance methods were evaluated in analysis and experiments. Figure 10 illustrates an elliptical clearance model that was used for specimens HSS5 and HSS11 and most other specimens considered in the test program. This elliptical clearance was developed based upon observed yielding the gusset plate during the experimental program and the results of nonlinear analyses (Yoo 2006) performed on the SCBF system. Specimen HSS5 used this elliptical clearance model and fillet welded gusset plates with welds designed to develop the full plastic capacity of the gusset plate. The gusset plate was designed to provide adequate resistance to develop the brace force, but there was little excess resistance and stiffness in the connection. Figs. 7b and 8b show the force-deflection behavior of this specimen and the connection details, respectively. This specimen attained significant ductility and inelastic deformation capacity. The brace sustained large out-of-plane deformation as illustrated in the photo of Fig. 9b, and ultimately the brace fractured at the center of the buckled region as illustrated in Fig. 9c. This test shows that gusset plate connections with weaker, more flexible gusset plates and the elliptical clearance model can achieve greater ductility with the high strength steel tubular braces commonly used in seismic design practice.


a) b) c)

Figure 9. Photographs of test results; a) Weld fracture of HSS1, b) Large out-of-plane buckling deformation of HSS5, c) Brace fracture

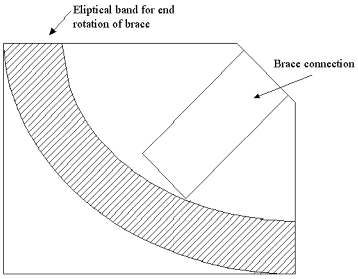


Figure 10. Elliptical buckling clearance

Since gusset plate strength and stiffness play a major role in the ductility achieved with SCBF frames, analyses were performed to examine the inelastic response of the SCBF frames and gusset plate when the beam and column size and gusset plate thickness were increased. These analyses showed a significant increase in stress and strain concentrations in the brace when these parameters were increased, and this implied that early fracture of the brace may be expected under these conditions. Specimen HSS11 was designed to test this hypothesis. The specimen had a thick gusset plate and the beam size was increased to a W16x89 to increase the connection stiffness as illustrated in Fig. 8c. Figure 7c shows the force deflection behavior observed in this test. Very limited ductility was achieved, because the connection stiffness provided by the thick gusset and heavier beam section resulted in concentration of plastic strains in the center of the buckled brace and dramatically reduced system ductility. The brace buckled out-of-plane but the out-of-plane deflection was much smaller than observed with HSS5 and other tests because of the early brace fracture. The stiffer gusset increased buckling resistance of the frame, because of the reduced effective length of the brace, but the reduction in inelastic deformation capacity has dramatic consequences to the total seismic performance of the system.

CONCLUSIONS

This research shows that SCBF frames and gusset plate connections can achieve significant ductility or provide relatively poor seismic performance depending upon the design of the gusset plate connection. The 2 t_p linear buckling clearance model results in relatively large gusset plates, and the large gusset plates are somewhat counterproductive because they also significantly increase the stiffness of the connection. The elliptical clearance model works equally well and produces smaller more flexible connections. Gusset plate welds should be sized to achieve the full plastic resistance of the plate rather than the expected resistance of the brace. The strength and stiffness of the gusset plate should be adequate to develop the expected resistance of the brace, but extra capacity is counter productive. Additional analytical and experimental work continues on this project, and further conclusions and observations are expected.

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