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SEISMIC PERFORMANCE AND DESIGNOF BRACED FRAME GUSSET PLATE CONNECTIONS

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ABSTRACT

Concentrically braced frames (CBFs) are stiff, strong systems for seismic design. Yielding and buckling of the brace dominate their inelastic deformation, and current design methods can lead to soft stories, unexpected failure modes and premature brace fracture. Gusset plate connections play a major role in CBF performance. A research program was initiated into the performance of CBF gusset plate connections, and this research is summarized. The work shows that gusset plate connection design strongly influences system performance. The connection must be stiff and strong enough to develop the full capacity of the brace, but extra strength and stiffness is detrimental to the performance and may reduce the inelastic deformation capacity of the frame. New methods for assuring that the gusset can develop end rotation of the brace and a design procedure for balancing the resistance of various failure modes are proposed.

Introduction

Steel concentrically braced frames (CBFs) are stiff, strong structures that are economical for seismic design. Their initial design employs a pinned-joint truss assumption, but the actual brace-beam-column connections are gusset plate (GP) connections as illustrated in Fig. 1. GP connections have considerable variability and are clearly not pinned joints. Both corner and midspan GP connections are illustrated in the figure. Corner GPs are restrained on two edges by a beam and column or by a column and base attachment. Midspan GPs are restrained on only one edge, and they occur with V-bracing or multi-story X-braced configurations. For seismic design, the brace is the ductile element and must sustain large inelastic deformations during extreme seismic events. The GP connection must develop the full brace resistance if the member ductility is to be achieved. Therefore, members are initially designed to achieve the factored design loads, but the GP connections are then designed to expected the expected yield resistance of the braces in tension and compression (AISC 2005). Seismic design places additional constraints on the GP connection design. The inelastic deformation of CBFs is dominated by post-buckling and tensile yield behavior of the braces, and this places large rotational demands

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on the GP connection. AISC Special CBF (SCBF) design rules address these design requirements (AISC 2005). SCBF rules require that the connection be strong enough to develop the expected resistance of the brace, and inelastic deformation demands require appropriate allowance for brace end rotation. This is usually accomplished with the $2t_p$ linear clearance model shown in Fig. 2a. These requirements sometimes lead engineers to produce heavy and massive connections based upon the assumption that bigger, stronger connections are better. The $2t_p$ linear clearance model often results in larger, thicker GPs, and the net effect has sometimes produced undesirable inelastic performance of the CBF system.

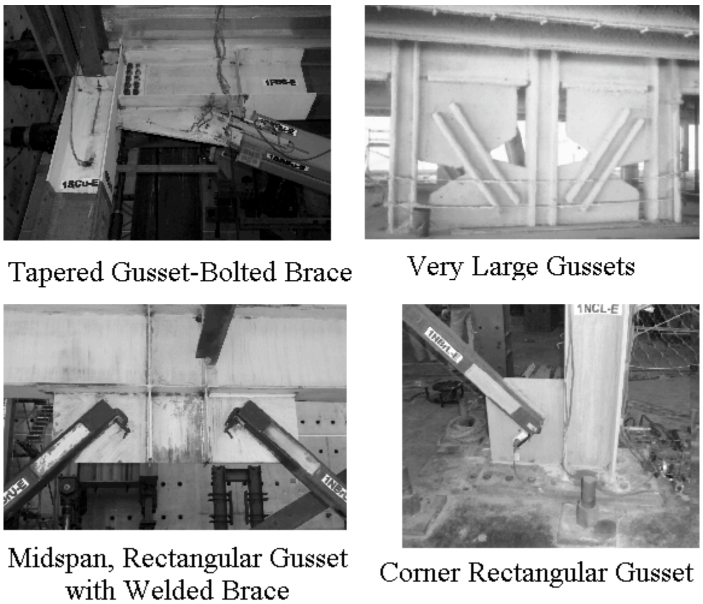


Figure 1. Typical CBF GP connections

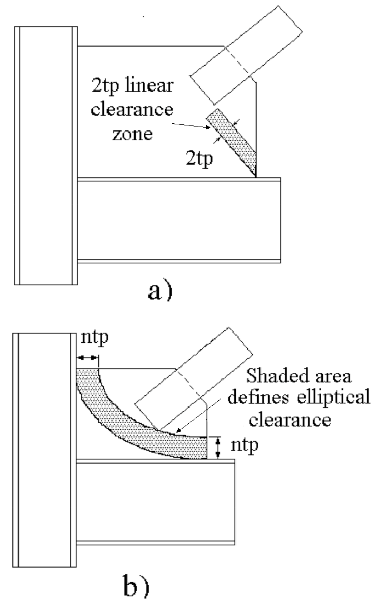


Figure 2. Gusset clearance models

Past Research on GP Performance and Current Design Method

Considerable experimental research has been performed on corner GP connections (Brown, 1988; Cheng et al., 1994; Grondin et al., 2000; Hu et al., 1987; Rabinovitch and Cheng, 1993; Yam 1994; Yam and Cheng 2002). These were typically component tests that did not include global frame or brace buckling behavior and employed monotonic rather than cyclic loading. This past research forms the basis of most current GP design procedures, and comparisons of design models with past results are made elsewhere (Roeder et al. 2004, 2005).

GP connections are designed to assure that the factored resistance, ϕR_n, for each connection design failure mode exceeds P_u. For nonseismic applications, P_u is the factored design load. For seismic design of SCBF connections (AISC 2005), P_u is taken as the expected tensile yield ($R_y A_g F_y$) and the expected compressive buckling force ($R_y A_g F_{cr}$), where R_y is the ratio of the mean or expected yield stress to the minimum specified yield stress, F_y, and A_g are the gross area of the brace. Multiple design checks such as depicted in Fig. 3a and b must be completed. Welds and bolts joining the steel brace to the GP are designed to resist the expected tensile capacity of the brace. A $2t_p$ clearance requirement (see Figs 2a, 3a and 3b) is used to assure end rotation of brace after brace buckling, and this requirement often leads very large GPs.

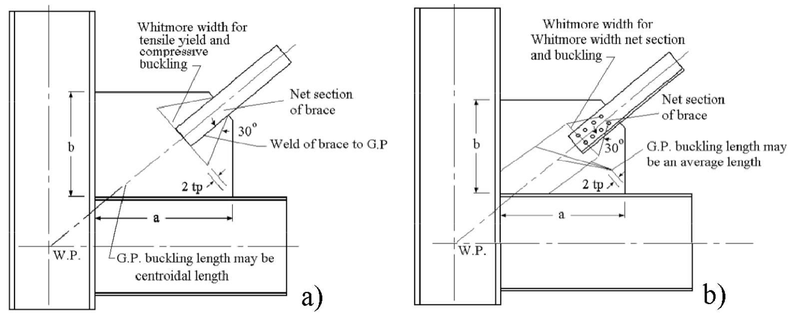


Figure 3. Typical GP design checks; a) Welded tube brace, b) Bolted angle brace

High strength tubular steel (HSS) brace members are commonly used. The tube is slotted to slip over the GP. Net section fracture of the brace at the tip of the slot (see Fig. 3a) may occur, and net section reinforcement is often required. Net section fracture may also occur at the first and last row of bolts for bolted brace connections as depicted in Fig. 3b. Block shear must be checked for welded or bolted braces as shown in Figs. 3a and 3b. The Whitmore width of the GP is defined by projecting a 30^{o} angle from the start to the end of the welded or bolted joint as shown in Fig. 3a and 3b. The area defined by the Whitmore width is used to check the GP for tensile yielding and buckling. In some cases, the effective GP length for buckling is an average length based upon key points across the Whitmore width (see Fig. 3b), and in others a centroidal length is used (see Fig. 3a). Edge buckling based on the free edge length is also sometimes checked, but comparison of edge buckling models with past research results show poor correlation between buckling predictions and experimental results (Roeder et al. 2004, 2005). After GP geometry is defined, the bolts or welds attaching the GP to the beam and column are sized with the Uniform Force Method (UFM) where design forces are defined by equilibrium with the expected tensile force of the brace. The beam-column connection is also part of the GP design.

Current Research Program

Current design methods for GP connections are variable and CBF performance has been mixed. An experimental program was initiated to better understand CBF and GP behavior and to improve current design methods, and Fig. 4 illustrates the test specimens and test setup. The test specimens include the brace, beams, columns, and GP connections and simulate a full-scale single-bay of the bottom story of a 3- or 4-story CBF or the upper story of a taller building. This setup models CBF performance better than past test programs. The braces, beams, and columns are typically HSS $5x5x^3/_8$ tubes, W12x72 and W16x45 sections of A992 steel, respectively, but other member sizes were occasionally used. The GP connections were varied from specimen to specimen to evaluate current GP design provisions and numerous variations in CBF and GP connection design. The specimens were subjected to a cyclic inelastic deformation history based upon the ATC-24 testing protocol (ATC, 1992). Table 1 summarizes the 27 SCBF frames tested to date. More complete information can be found elsewhere (Johnson, 2005; Herman, 2007; Kotulka 2007).

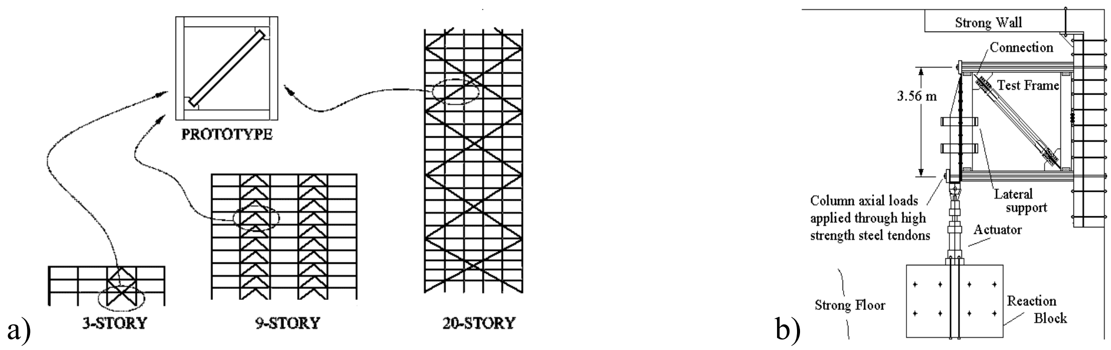
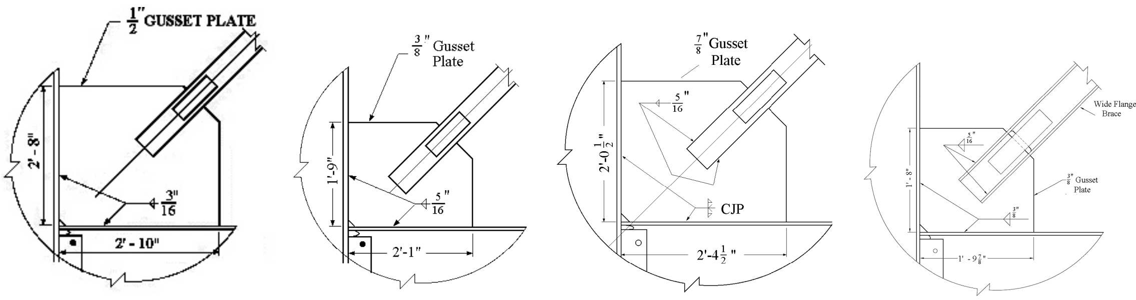


Figure 4. Test a) Specimen, b) Setup

Table 1. Summary of Current Test Results

Spec.	Specimen Description	Failure Mode	Drift Range
1	Baseline - UFM w/ 2 t_p linear clearance.	Weld fracture	2.6%
2	GP plastic capacity of weld & 5.8t_p elliptical clearance	Brace fracture	4.0%
3	#2 except thinner, more flexible gusset	Brace fracture	4.6%
4	#3 except 9.4t_p elliptical clearance	Brace fracture	4.6%
5	# 2 or 3 except 7.7t_p elliptical clearance	Brace fracture	4.8%
6	#5 except ends of fillet welds reinforced	Brace fracture	4.7%
7	Thick gusset with fillet & 6.4t_p elliptical	Brace fracture	3.9%
8	#3 except 3.3t_p elliptical clearance	Brace fracture	4.6%
9	Slightly thicker gusset w/ CJP weld & 5.7t_p elliptical	Brace fracture	3.6%
10	Tapered GP with 7t_p elliptical	Brace fracture w/ weld cracking	4.4%
11	Thick gusset w/ heavy beam & 6.4t_p elliptical	Brace fracture	2.4%
12	#1 (2t_p linear clearance) except CJP weld	Brace fracture	3.6%
13	#10 except CJP weld & 7t_p elliptical	Brace fracture	3.5%
14	#5 but 6t_p elliptical w/o net section reinforcement	Brace Fracture	4.0%
15	#14 but 6t_p elliptical & minimum for block shear	Brace fracture.	4.1%
16	One sided slip critical bolted brace connection with 2t_p linear clearance in extension plate	Net section fract below brace cap	5.8%
17	#10 but thinner gusset & 9.3 t_p elliptical	Brace fracture	4.8%
18	#5 w/ 8t_p elliptical & bolted shear tab beam conn	Brace fracture	3.8%
19	Double-T bolted brace connection	Conn fracture	1.5%
20	Bolted end plate gusset connection	Brace fracture	4.5%
21	Bolted end plate gusset connection	Bolt fracture	3.5%
22	Tapered gusset, Unwelded beam flanges as #18	Gusset tearing	3.9%
23	W6x25 wide flange brace, 8 t_p elliptical	Weld fracture	5.6%
24	3/8" gusset, 6 t_p elliptical	Brace fracture	4.6%
25	7/8" gusset w/ heavy beam, w/o net section reinf.	Brace fracture	3.8%
26	Thick (7/8") gusset with heavy beam and w/o net section reinforcement. Near fault cyclic deformation	Net section fracture	1.7%
27	3/8" gusset with elliptical clearance, without net section reinforcement and with near fault cyclic deformation	Net section fracture	2.5%

Figures 5 and 6 compare the results of a few tests (Specimens 1, 5, 11, and 23) to illustrate conclusions from this research. Specimen 1 was designed using the current AISC UFM with the $2t_p$ clearance requirement, and this specimen achieved little ductility because of early GP weld fracture as shown in Figs. 6a and 7a. While the UFM sizes the welds to achieve the expected resistance of the brace, brace and GP deformation places additional demands on those welds, which are not considered in the stress calculations. This test shows that the GP to frame welds must be designed to achieve the plastic capacity of the gusset rather than the expected capacity of the brace. The linear $2t_p$ buckling clearance results in relatively large GPs, which create a relatively large stiff, rigid zone, and force significant local yield deformation into the framing members. The elliptical clearance method shown in Fig. 2b leads to a thinner more compact GP connection and was developed from observed patterns of GP yielding in experiments and analysis. Specimen 5 utilized an $8t_p$ elliptical clearance and the GP welds were sized to develop the plastic capacity of the GP. It had thinner, more compact gussets, and comparison of Figs. 5a and 5b and 6a and 6b show that Specimen 5 attained much larger ductility and inelastic deformation capacity than Specimen 1. Its brace sustained large out-of-plane deformation (see Fig. 7b) and ultimately fractured in the buckled region (see Fig. 7c).



a) b) c) d)

Figure 5. GP design; a) Specimen 1, b) Specimen 5, c) Specimen 11, d) Specimen 23

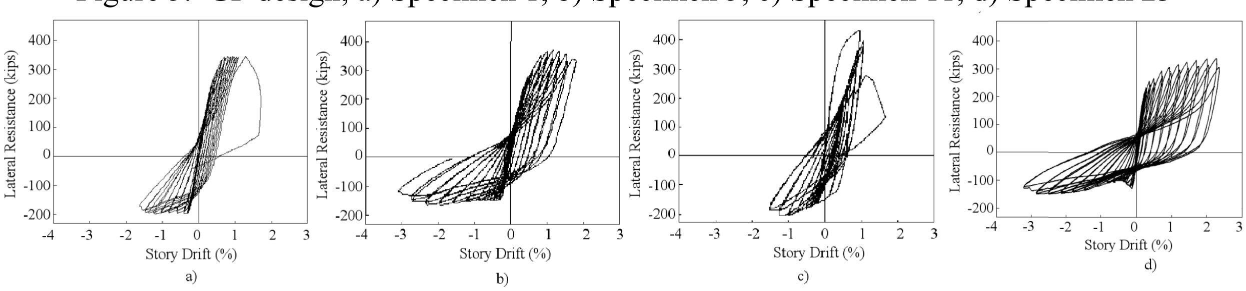
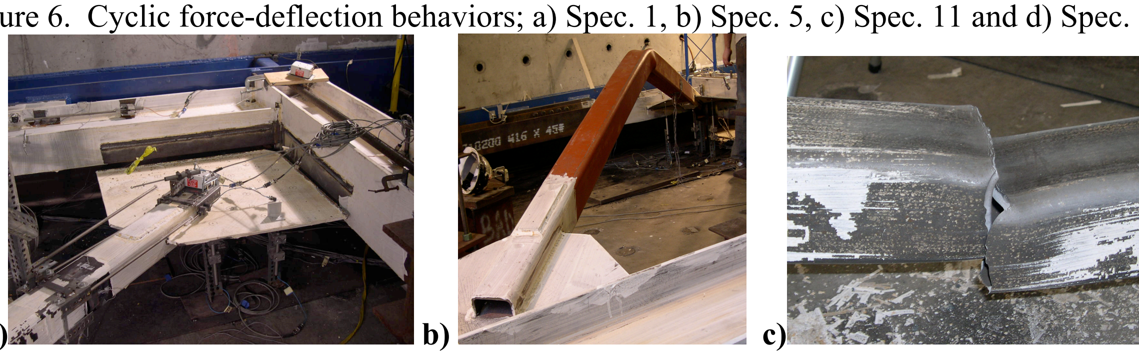


Figure 6. Cyclic force-deflection behaviors; a) Spec. 1, b) Spec. 5, c) Spec. 11 and d) Spec. 23



a) b) c)

Figure 7. Photographs; a) Weld fracture, b) Out-of-plane buckling, c) Brace fracture

Current AISC SCBF design requirements may lead one to believe that stiffer, stronger GPs may improve seismic performance. Specimen 11 (see Fig. 5c) evaluated this hypothesis, and Fig. 6c clearly shows that excess strength and stiffness in the GP lead to greater beam and column damage and reduced inelastic deformation capacity. This occurs because the stiffer, stronger connection provides greater resistance to brace rotation, and forces the brace into reverse curvature. Reverse curvature forces the plastic hinge of the brace into a shorter region causing larger strains in the brace and earlier brace fracture. HSS braces are generally thought to be less ductile than braces of some hot rolled sections. Specimen 23 was similar to Specimen 5 except that it used a comparable wide flange section for the brace. This specimen achieved greater ductility, but the greater brace ductility placed increased deformation demands on the GP causing gusset weld fracture. Tapered GP connections were also tested in this research program and generally produced good seismic performance, but the specific results of these tests are not shown here.

Midspan GP connections are different from corner GP connections, because they are restrained on one edge. This difference contributes positive benefits to midspan GPs, since it eliminates the very strong diagonal stress effect with extreme brace forces noted for corner gussets. However, midspan gussets are supported along only one edge, and this reduces their buckling restraint. Corner gussets can be designed with a relatively small effective length coefficient, K (K= 0.65), but a larger effective length coefficients (K= 1.2 to 1.4) are required for midspan GPs. Further, the rotational restraint of a midspan gusset is influenced by the lateral-torsional stability of the beam and floor system.

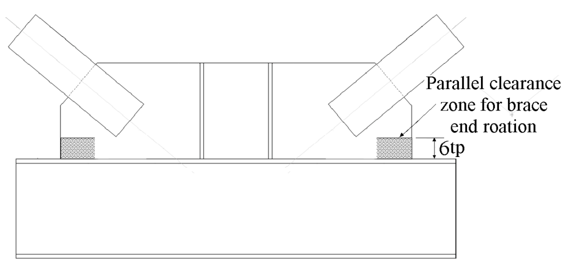


Figure 8. Parallel $6t_p$ clearance model for midspan gusset

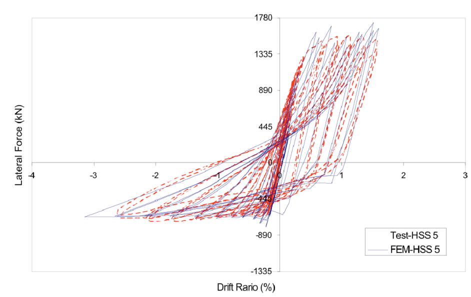


Figure 9. Comparison of measured and computed force-deflection behavior

Six large scale tests were performed on 2-story and 3-story frames at the NCREE Laboratory in Taiwan to evaluate midspan gussets and braced frame system performance. The 2-story frame utilized midspan GP connections, which were designed by the elliptical clearance method proposed for corner GP connections (Lehman et al. 2008). The 3-story frame utilized similar design methods with the $6t_p$ parallel clearance model shown in Fig. 8, which resulted in thin, compact gussets with no edge stiffeners. This linear clearance model was developed based upon an extensive nonlinear analysis program described later in this paper. In all cases, buckling of the midspan GPs was evaluated with an effective length coefficient, K, of 1.4 to account for the increased buckling potential. The resulting behavior was good and comparable to the corner GP results noted in Table 1.

Nonlinear finite element (FE) analyses were performed with the ANSYS computer program to investigate frame performance for single-story and multi-story frame tests (Yoo et al. 2008a). The model was constructed from quadrilateral shell elements using large-deflection formulations including geometric stiffness with bilinear kinematic plastic hardening of the steel. The cyclic inelastic behavior for the full load history of all test specimens was computed and compared to the test results with comparisons such as shown in Fig. 9. The comparison between experiments and analyses were very good at both the global performance and local deformation levels (Yoo et al, 2008b). These analyses also show good correlation between the computed equivalent plastic strain in the brace and GP and fracture of the brace and crack initiation in the GP. This equivalent plastic strain calculation was used as a guide to designing test specimens and predicting test results during the research program.

Proposed Design Method

Seismic design of braced frames requires a balance of strength, stiffness and ductility or inelastic deformation capacity. Current GP connection design methods insure adequate elastic strength and stiffness, but ductility and inelastic deformation capacity requires careful consideration of the yield mechanisms and failure modes of CBFs as illustrated in Fig. 10. A new seismic design methodology based on balancing yield mechanisms and preventing undesirable failure modes is proposed. The proposed balanced design approach builds upon traditional capacity design equations and methods. As with current methods, framing elements are designed to meet the force demands, and then the expected tensile yield ($T_{exp}=R_yF_yA_g$) and compressive buckling ($C_{exp}=R_yF_{cr}A_g$) capacities of the brace are used to design the connection. However, greater ductility is achieved with the balanced design method by assuring that multiple, desirable yield mechanisms are developed and that undesirable failure modes are delayed until significant inelastic deformation has occurred. The method satisfies serviceability requirements, since all members have resistance much greater than the factored design loads. The balance procedure targets a sequence of yielding so that the braced frame develops significant inelastic deformation before undesirable failure modes are permitted. This is accomplished by the balance conditions:

$$R_{yield\ mean} = R_yR_{yield} \leq \beta_{yield1}R_yR_{yield,1} \leq \beta_{yield\ 2}R_yR_{yield,2} \ldots \leq \beta_{yield\ i}R_yR_{yield,i} \qquad (1)$$

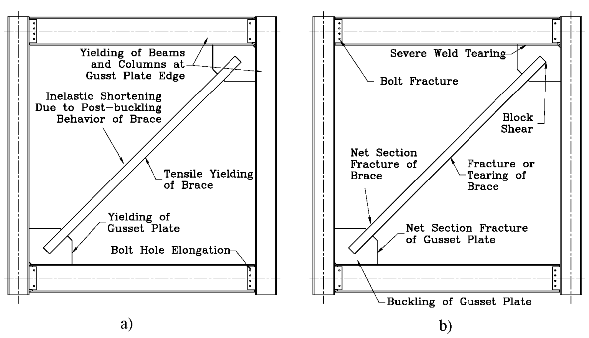


Figure 10. Typical behaviors for SCBFs; a) Yield mechanisms, b) Failure modes

The nominal yield resistances, R_{yield}, for the various yield mechanisms are separated by balance factors, β_{yield}, to control the resistance of secondary yield mechanisms. A yielding hierarchy or sequence of yielding is established by the magnitude of the β factors, which are larger or smaller as needed to assure the sequence of yielding desired for the connection behavior. A single failure modes cause fracture, tearing, deterioration of resistance, or irrecoverable damage to the system, but multiple failure modes are normally required to collapse the connection or system. The balance procedure also is used to balance failure modes:

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

This balanced design approach assures that the resistance of all failure modes, R_{fail}, exceed the strength of the primary and preferred secondary yield mechanisms, and it assures that less favorable failure modes have greater separation and smaller probability of occurrence than more favorable failure modes. The relative magnitudes of the β_{yield} and β_{fail} values assure the number of yield mechanisms to be expected and the separation between yielding and initial failure of the SCBF system. The β values are similar to resistance factors, ϕ, except that they are determined based on ductility and experimental performance rather than resistance.

Work continues on the design method, but some guidance on the design procedure and β values is possible. Welds and bolts joining the brace to the gusset are designed for the expected tensile resistance of the brace with a β factor equal to the ϕ factor used for these connections, because failure of these connectors has serious, detrimental consequences to system behavior. Net section fracture of the brace for these connections offers limited ductility prior to fracture, but it is predominantly a failure induced by near fault loading and stiff, strong GP connections. Hence, a β factor of approximately 0.9 is more appropriate than the current resistance factor of 0.75 for the design of these connections. Thin, compact GPs are encouraged, and tensile and flexural yielding of the GP enhances system ductility. As a result, a β factor of approximately 1.0 is proposed for the tensile yield resistance over the Whitmore width (or true width if smaller than the Whitmore width). Rectangular GPs should be designed with the $8t_p$ elliptical clearance model for corner GP connections and $6t_p$ parallel clearance model for midspan GP connections. The elliptical clearance model is similar to the $2t_p$ linear clearance model for tapered GPs with significant taper, but tests show that tapered gussets are a bit more prone to severe weld cracking. The elliptical clearance model result in thinner, more compact GPs with shorter buckling lengths, where the gusset buckling length is the average of the centroidal length and the two lengths at extreme points of the Whitmore width. The effective length coefficient, K, can be 0.65 for corner GPs and 1.4 for midspan GP connections. The β factor for this failure mode may also be taken as the AISC resistance factor for column buckling. Flexible GPs permit use of an effective length equal to the true length of the brace for out-of-plane brace buckling. Finally, welds and bolts joining the GP to the framing members should be sized to develop the full tensile and flexural yield resistance of the GP. These recommendations are tentative, because additional research is in progress, but these recommendations are expected to improve inelastic performance of CBFs.

Summary and Conclusions

Numerous experiments and analyses have been completed, and a number of important design recommendations can be noted. The experiments show that the welds joining the GP to the beam and column must be designed to achieve the plastic capacity of the GP rather than the expected resistance of the brace. The $6t_p$ to $8t_p$ elliptical clearance model leads to thinner, more compact gussets and provides equal or better performance for corner gussets than that achieved with the $2t_p$ linear clearance model. The 6tp horizontal clearance model provides similar benefits for the midspan gussets. Tapered GPs may provide good end rotational capacity for the brace but result in thicker gussets or greater inelastic demands on the GP and the welds. Yielding in the Whitmore width of the GP is desirable, but it should occur after initial yielding and buckling of the brace. The strength and stiffness of the GP must develop the expected resistance of the brace but should not be excessively large, because stiff, strong connections cause early brace fracture. The effective length of the brace may be taken as 1.0 based upon the true brace length when these rules are employed. Wide flange braces achieve larger inelastic deformations than HSS tubes but cause increased deformation demands on the GP connection.

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