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Improved Seismic Design of Steel Frame Connections

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Abstract

Reliable seismic performance requires structural systems capable of sustaining multiple inelastic displacement cycles at large drifts. Response of steel framing systems depends to a large part on the inelastic deformation capacity of the framing elements. Recent research into the seismic performance of moment resisting frames has shown that improved response, including increased ductility, is possible if limited yielding is also permitted in the connection in addition to the beams, and the resistances of these yield mechanisms are balanced. Adverse effects of premature failure are controlled by balancing the failure mode resistances with these yield mechanisms. However, current seismic design provisions do not permit these practices in braced frame gusset plate connections. In this paper, the design methodology to balance the yield mechanisms and failure modes is presented with the objective of improving the seismic response of steel framing systems. Initially, the strategy is demonstrated for special steel moment resisting frames with welded-flange-welded-web connections. Next, the design procedure is theoretically applied to special concentrically braced frames. Past experimental results are used to develop improved design models for braced frame gusset plate connections, and to provide preliminary estimates of the required design parameters. Additional research that is required to finalize the design procedure for braced frame systems is discussed.

Keywords: Moment resisting frames, braced frames, connections, inelastic response, seismic design

1. Introduction

Large, infrequent earthquakes can cause large internal force demands on building structures. The resulting internal forces may exceed maximum internal forces caused by the total gravity load. As such, under design seismic loading conditions structural systems generally must sustain large, cyclic drift demands. Common steel structural framing systems used for seismic design include moment resisting frames (MRFs) and concentrically braced frames (CBFs). MRFs are relatively flexible structures that primarily develop their inelastic deformations through beam flexure. CBFs are stiff, strong, and economical structural systems and their inelastic lateral response is dominated by inelastic deformation of the braces including brace buckling.

For both MRFs and CBFs, the inelastic deformation places large local force and deformation demands on the connection. However, current design provisions do not provide realistic estimates of these demands and may not result in reliable connection performance. This deficiency is largely because current design methods are based on linear elastic idealizations which do not prevent concentration of damage in the structure and do not control the sequence or occurrence of that damage. In addition, research indicates that the deformation capacity of the structural system is enhanced by permitting controlled inelastic action in the connection while ensuring that undesirable failure modes in both the framing elements and connections are prevented. To improve the performance of steel connections, a new design methodology based on this concept is proposed. The method balances the yield mechanisms and failure modes in the framing elements and connections to achieve the desired yielding hierarchy and prevent premature failure. As such, the method represents a significant extension of and improvement on the capacity design method. The balanced design methodology has been applied to MRFs and their connections; an example of the resulting design methods and parameters is provided. The methodology is then applied to special CBF (SCBF) systems; additional research needed for thorough evaluation of the design parameters is discussed.

2. Steel Frame Connection Behavior

In high seismic zones, design requires that structural systems possess large inelastic deformation capacities to meet the demands resulting from large, infrequent

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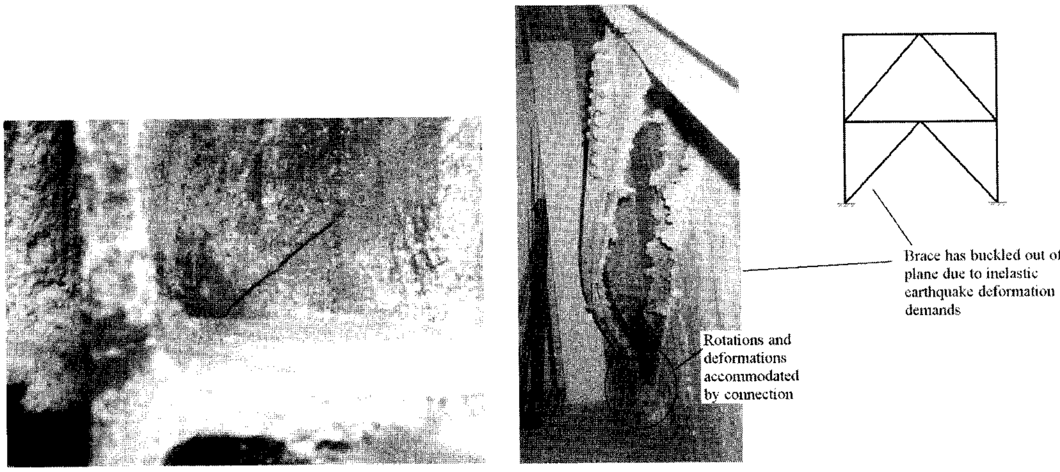


(a) MRF Connection Cracking (b) CBF Connection Rotation and Restraint

Figure 1. Seismic damage to steel frame connections.

earthquakes. The response of the system largely depends on the framing elements. However, the connections must resist member forces and accommodate deformations, and so connections influence the performance and may determine the deformation capacity of the system. Therefore, the design and response of the connections deserves careful attention.

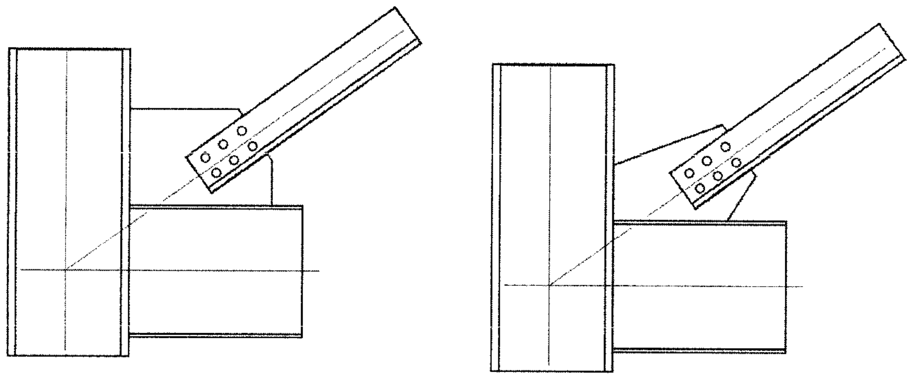
Special MRFs (SMRFs) are a category of steel MRF designed to achieve ductile performance during seismic loading. The deformation capacity of SMRFs is a result of the large plastic rotations sustained by the beam near the beam-column connections. Prior to 1994 the seismic performance of SMRFs was viewed as ideal. However, during the 1994 Northridge and 1995 Hyogoken-Nanbu Earthquakes cracking and fracture were noted in SMRF connections. Figure 1a shows one such fracture, where a crack initiated in the column at the beam flange weld and progressed into the column panel zone. This cracking was serious and widespread and it became evident that the large plastic deformations that occur adjacent to beam-column connections must be taken into account in the design of the connection. As such, improved SMRF connection design methods were developed (FEMA 350, 2000; Roeder, 2001) and resulted in improved connection performance.

The response of a CBF largely depends on the response of the brace, which contributes significantly to the lateral stiffness of the frame. Under lateral load, the brace may develop inelastic deformation through cyclic, tensile yield-compressive buckling response. Plastic hinges form within the brace after buckling as a result of the second-order moments. These actions cause permanent plastic deformations and deterioration of resistance. SCBF systems were developed to provide ductile cyclic, inelastic performance and they are the most commonly used braced frame system for design in regions of high seismicity (AISC, 2002). To achieve the desired performance, AISC seismic design requirements (AISC, 2002) provide limits on the cross-sectional geometry and local and global slenderness ratios of the brace.

As for SMRF systems, the seismic response of SCBF systems is greatly influenced by the connection response. In a SCBF, braces are normally joined to the beams and columns of the frame through gusset plate connections such as depicted in Fig. 2. In addition to the cyclic axial force demands, the post-buckling brace bending causes large end rotations that may place cyclic moment and rotation demands on the connection, as illustrated in Fig. 1b. The demand depends on the buckling mode of the brace (i.e., in-plane or out-of-plane). The rotation-deformation demands combined with the tensile force demands complicate connection design. With out-of-plane buckling, geometric limits for the gusset plate are established to accommodate the expected brace end rotation. Current seismic design specifications require that the connection be designed to be stronger than the brace, however the complex demands resulting from brace buckling are not considered. In addition, research indicates that yielding in the gusset plate connection, following brace yielding and buckling, results in increased ductility capacity and improved performance relative to connections that remain elastic (Grondin *et al.*, 2000). Improved ductility of SCBF systems may result if the brace and connection strengths are properly balanced and based on a realistic estimate of the connection demands.

3. Proposed Improved Design Procedure

Special steel framing systems have been developed to sustain large, cyclic inelastic drift demands resulting from maximum credible earthquakes. Previous research has focused on the primary framing elements of these systems (e.g., beams, columns, and braces) which has resulted in current seismic design requirements.



(a) Typical Rectangular Plate (b) Typical Tapered Gusset Plate

Figure 2. Illustration of typical braced frame gusset plate connections for seismic design.

However, the response of the connection greatly influences the response of these systems and recent earthquake reconnaissance indicates that the large stress and inelastic deformation demands on the steel frame connections frequently result in damage at these locations. To improve the frame performance, the response of the connection, which includes the connection strength, stiffness and ductility, must be directly considered in the frame design. Strength or resistance of the connection contributes to the frame resistance and must be designed to assure that the ductile elements of the structural system achieve their full resistance and ductility. In addition, the connection stiffness affects the dynamic response and therefore the deformation demands on the structural members and connections. For example, the connection stiffness factors into the effective length of the brace which significantly contributes to the system response.

Steel frames have multiple yield mechanisms where the resistance of each yield mechanism is the resistance at which yielding and stiffness changes are initiated. Controlled yielding of both connections and framing elements can contribute to the strength, stiffness, energy dissipation, and inelastic deformation capacity of steel framing systems, and development of multiple participating yield mechanisms may be desirable for good seismic performance. Proper balance of multiple yielding conditions and mitigation of undesirable failure modes can ensure this participation. Therefore, a well-balanced design which directly considers the connection properties and maximizes the inelastic deformation capacity is desired.

To meet these objectives, a seismic design methodology based on balancing the yield mechanisms and preventing undesirable failure modes has been developed for SMRF systems and is under development for SCBF systems. In traditional seismic design, plastic design principles are used and the elements are designed to meet the elastic force demands. The specific elements that are expected to yield (e.g., beams in a moment frame) are designed to sustain inelastic action. Capacity design principles are used to design the adjacent, and presumably non-yielding, elements such that they are stronger than the yielding elements.

The balanced design approach is similar to traditional seismic design methods in that the framing elements are designed to meet the elastic force demands and specific elements are designed to yield to achieve the desired plastic mechanism of the system. However, in contrast to traditional design, the balanced-design approach permits secondary yield mechanisms to develop in other elements (such as the connection) as well as the primary yielding element at large ductility demands. Furthermore, proportional separation is provided between desirable and less desirable behaviors to reduce the probability of less acceptable yield mechanisms and failure modes occurring prematurely. The design is balanced such that the yield capacity of the primary element is less than the yield capacity of the secondary yield mechanisms. However, with increased deformation demands and strain hardening in the primary member, the secondary yield mechanisms may develop and contribute to inelastic deformation capacity. Numerous secondary yield mechanisms are possible, such as shear yielding in the panel zone for MRF connections and inelastic deformation of the gusset-plate for SCBF connections.

The nominal yield resistances are designated R_{yield} in this paper. Relative to the strength of the primary yield mechanism, a reduction or balance factor, β, is used to increase the strength of the secondary yield mechanisms to achieve the balance state. This balanced approach can be expressed using the following inequality:

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

Using this inequality for design, the resulting yielding hierarchy would be the primary mechanism followed by secondary mechanism 1, followed by secondary mechanism 2, etc. The AISC (2002) seismic design provisions suggest that the mean yield resistance can be computed by multiplying the nominal yield resistance by a factor, R_y, which is defined in the subject specification.

Research shows that permitting the formation of secondary (and subsequent) yield mechanisms increased the deformation capacity of the system and can prevent premature failure of the connection (Roeder, 2002a). 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 will usually be required to achieve these extreme conditions. But a single failure mode results in significant, irrecoverable damage to the system. Therefore development of a single failure mode may be acceptable for some performance levels (e.g., collapse prevention or life safety), but may be unacceptable for performance levels that are determined by the system serviceability or functionality. In addition, specimens with a controlling yield mechanism resistance that is similar to (or larger than) the critical failure mode resistance achieve little or no ductility. Since ductile performance is necessary to assure good seismic performance for some performance levels (e.g., collapse prevention or life safety) and to meet current design provisions, balance between the controlling yield mechanism and critical failure mode resistances is required (as it is expected that the critical failure mode will occur prior to structural collapse). As such the balanced design approach also requires that the strength of all identified failure modes, R_{fail}, exceeds the strength of the primary yield mechanism, as shown in Eq. (2).

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

In addition or as an alternative to satisfying the inequalities in Equations (1) and (2), prescriptive details such as slenderness limits or weld type may be provided to limit the need to check one or more individual failure mode (FEMA 350, 2000).

The ductility and inelastic performance of the system is controlled by the combination of the controlling yield mechanism and the critical failure mode, and the proximity of the critical failure mode and controlling yield mechanism resistances. Some failure modes result in unpredictable performance or more serious consequences than others do, and significant separation between the controlling yield mechanism and these less desirable failure modes is warranted. Systems with a controlling yield mechanism resistance that is significantly smaller than the critical failure mode resistance develop large inelastic deformation and ductility. However, the deformation capacity of the controlling yield mechanism in combination with secondary mechanisms must be sufficient to meet the seismic deformation demand.

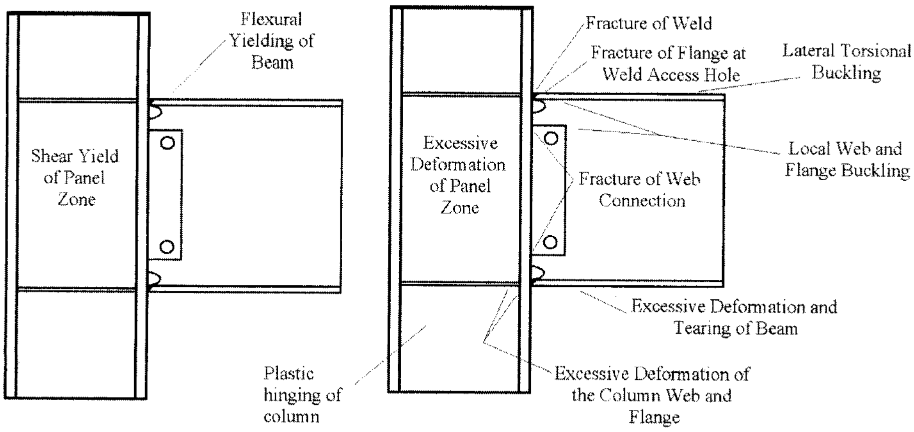
The β factors designed to balance the yield and failure modes are similar to the resistance factors, ϕ, in LRFD design (AISC, 2001) in that both are no greater than 1.0 and both are based upon the performance and variability of the response mechanism. However, the factors are fundamentally different in that ϕ factors are based upon strength, safety, and statistically extreme considerations, while β values are based entirely upon balancing the expected or average inelastic seismic behavior to meet the inelastic deformation requirements.

The required β factor is smaller when a given yield mechanism or failure mode is difficult to predict or has undesirable consequences. Larger β values are appropriate for ductile yield mechanisms or failure modes where the resistance is accurately predicted and/or the consequence of failure is less severe. Balance conditions are used to ensure the desired progression of yielding and to prevent premature and undesirable failure modes. As indicated in Eqs. (1) and (2), appropriate values for β_{yield} factors must be established for each yield mechanism to provide the desired progression of yielding. Appropriate values for β_{fail} factors must be established for each failure mode to assure that the most desirable failure mode occurs and is adequately separated from the initiation of yielding. Different inelastic deformation goals are needed for different types of connections, and as a result, connection-specific balance conditions are needed.

The proposed procedure can be developed for a wide variety of connections using the following steps.

1. Identify all yield mechanisms that may contribute to the energy dissipation and drift capacities of the frame
2. Identify all failure modes that may occur in or adjacent to the yielding elements.
3. Gather and/or develop experimental data to quantify the yield mechanisms and failure modes.
4. Using the experimental data, validate and improve existing models that provide estimates of the strength and/or deformation for each yield mechanism and failure mode.
5. Determine the primary yield mechanism. Develop equations to balance yield mechanisms and failure modes (similar to Equations (1) and (2)).
6. Using experimental data and design models develop balance parameters to match the yielding hierarchy and to prevent premature component failure.

Each yield balance parameter, β_{yield}, is calculated as the ratio of the mean value of the primary yield mechanism, $(R_y R_{yield})_{mean}$, to the mean value of the secondary yield mechanism of interest, $(R_{yield,i})_{mean}$. Each failure balance parameter, β_{fail}, is calculated as the ratio of the mean value of the primary yield mechanism, $(R_y R_{yield})_{mean}$ to the minimum value of the failure mode of interest, $(R_{fail})_{min}$. In both cases, the balance parameter must include statistical variation in material properties and element behavior. Note, in some cases the parameters to balance the failure modes may not be calculated directly and default prescriptive details may be provided. In the following sections, the design methodology is demonstrated for SMRF connections and proposed for SCBF connections.



(a) Yield mechanisms (b) Failure Modes

Figure 3. Yield Mechanisms and Failure Modes of WFWW Connection.

4. Demonstration Of Proposed Method

Damage to MRF connections was observed in the Northridge and Kobe earthquakes (Fig. 1a). To mitigate this type of damage in future events, the proposed balance design methodology was applied to SMRF beam-column connections and resulted in improved seismic performance. Experimental testing of the resulting welded-flange-welded-web (WFWW) connection demonstrates the validity of the proposed method.

Initially, using experiments performed on WFWW connections (Roeder, 2001; Ricles *et al.*, 2000), the potential yield mechanisms and failure modes were identified, as illustrated in Fig. 3. Figure 3a shows the permissible yield mechanisms, flexural yielding of the beam and shear yielding of the panel zone. Although local yielding may occur in the column flange and web due to local stress concentrations, the response mechanism does not contribute significant plastic deformation and was not classified as a secondary yield mechanism in the figure.

Figure 3b shows the wide range of failure modes that may occur. More ductile failure modes for WFWW connections include inelastic lateral-torsional buckling of the beam, local web and flange buckling, excessive deformation of column web and flange, excessive plastic deformation of beam, and excessive deformation of the panel zone. These more ductile failure modes may result in deterioration in resistance, but they can provide inelastic deformation before this deterioration becomes significant. Flexural yielding and plastic hinging of the column may be relatively ductile, but experimental research has shown that weak-column strong-beam MRFs can lead to unstable structural performance (Schneider *et al.*, 1993) and it is preferable that this response mechanism be avoided. Failure modes causing weld fracture, beam flange fracture at the weld access hole, and web connection fracture, can lead to brittle performance with dramatic loss of resistance and reduced inelastic deformation capacity.

Experimental results were used to establish the balance requirements (Ricles *et al.*, 2000; Roeder, 2001). Experiments show that connections for which both flexural yielding of the beam and shear yielding of the panel zone contribute to plastic deformation and for which flexural yielding is initiated before shear yielding have significantly larger ductility and plastic rotational capacity than other specimens (Roeder, 2001). To achieve this desired yielding hierarchy, two balance conditions were established relative to beam yielding (Eq. (3) and (4)). The minimum balance parameter, β_{min}, is provided to assure that shear yielding of the panel zone occurs (Eq. (4)); the maximum balance parameter, β_{max}, is designed to assure that flexural yielding of the beam occurs first (Eq. (3)).

$$M_{yield\ mean} = R_y M_{yield} < \beta_{max} M_{beam}(V_{panel\text{-}yield}) \quad (3)$$
$$M_{yield\ mean} = R_y M_{yield} > \beta_{min} M_{beam}(V_{panel\text{-}yield}) \quad (4)$$

The mean yield strength of the beam is estimated as $R_y M_{yield}$. The two inequalities can be used to determine the acceptable range for the shear strength of the panel zone, V_{panel}, which is calculated as a function of the beam flexural strength, M_{beam}. Values for the balance parameters are provided in Table 1.

A form of Equation (2) is used to balance the more ductile failure modes. Research indicates that a range of β factors from 0.7 to 0.9 will ensure shear panel yielding occurs and therefore may result in improved component ductility (Roeder, 2002b). To include the constraints of practical design, β factors that range from 0.6 to 0.9 are used (Table 1). This range of β factors is relatively large to accommodate, in part, the variation in material yield stress between adjacent structural components. Excessive deformation of the beam and local buckling are controlled by local slenderness limits as well as continuity plate requirements. Lateral-torsional buckling is limited by minimum unsupported length requirements.

Experiments show that the hierarchy in the occurrence

Table 1. Overview of WFWW Balance Conditions

	Mode or Mechanism	β Value	Notes on the Application
Yield Mechanisms	Flexural Yield of Beam	---	$M_y = S\ F_{yb}$ is primary yield mechanism. No β value required
	Shear Yield of Panel Zone	$0.6 < \beta < 0.9$	Maximum value of 0.9 to assure yielding hierarchy Minimum value of 0.6 to assure some yield in panel zone as strain hardening occurs due to beam flexure.
More Ductile Failure Modes	Local Web and Flange Buckling	Less than 0.9	Controlled by local slenderness and continuity plate requirements
	Lateral-Torsional Buckling of Beam	Less than 0.9	Controlled by minimum lateral support requirements
	Excessive Deformation of Panel Zone	Less than 0.65	Controlled by balance conditions
	Excessive Deformation of Beam	Less than 0.65	Controlled by local slenderness requirements
More Brittle Failure Modes	Column Flexure	Approx. 0.85	Controlled by required over strength of plastic moment capacity of column relative to the beam. Present limits may be larger than desirable.
	Weld Fracture	Less than 0.65	Assured by quality control measures. See Fig. 4.
	Flange Fracture at Weld Access Hole	Less than 0.65	Assured by quality control measures. See Fig. 4.
	Tearing of Beam at Hinge Location	Less than 0.65	Critical failure mode with quality control measures. Failure moment 35 to 50% larger than primary yield mechanism
	Fracture of Web Connection	Less than 0.65	If quality control measures are applied, this failure mode to occurs shortly after tearing at hinge location.

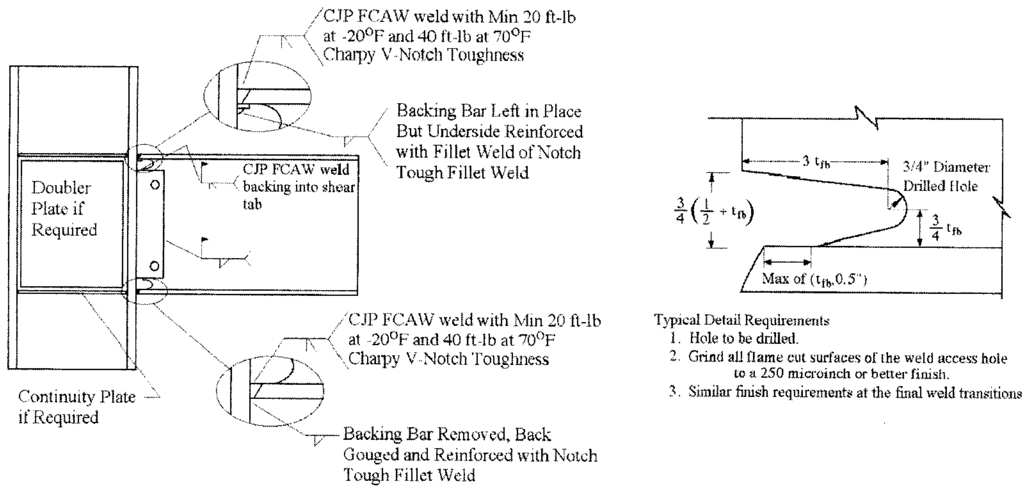
of more brittle failure modes also depends upon the quality control measures employed in design and construction. As such, quality control measures have been developed to achieve the desired hierarchy in these failure modes. If the flange weld does not possess adequate toughness or if root flaws are not properly removed, failure of weld will occur at very small inelastic deformations and usually precedes other failure modes. If the weld is improved, as described later and shown in Fig. 4, flange fracture at the weld access hole will occur at slightly larger deformation unless weld access hole geometry is improved. Finally, web attachment fracture is normally the last of these three brittle failure modes to occur. This balanced failure and sequence of failure modes is achieved by prescriptive details, which include:

- Complete joint penetration (CJP) weld between the beam web and the column and supplemental fillet weld around the shear tab.
- Required minimum CVN weld toughness and removed runoff tabs for both CJP flange and web welds.
- The bottom flange backing bar is removed, back gouged and reinforced with a fillet weld. The top flange backing bar is left in place and reinforced with a fillet weld.
- Weld access hole with the prescribed geometry and finish.

These requirements are necessary to assure the full connection ductility and are shown in Fig. 4 (Ricles *et al.*, 2000).

The beneficial effects of strain hardening combined with the large plastic deformations developed with the WFWW connection and the quality control measures described later assured that the β_{fail} value for the brittle failure modes was less than 0.65 (Table 1). This combined with the yield mechanism balance conditions was adequate to insure some panel zone shear yielding occurred in all test specimens. Flexural yielding and plastic hinging of the column is limited by assuring that the plastic bending capacity of the column is larger than the plastic bending capacity of the beam. This limit results in a β_{fail} of approximately 0.85. A smaller value of approximately 0.75 was recommended from the research results (Roeder, 2001), but economic constraints of engineering practice result in this somewhat higher value in the existing design specifications.

When the requirements summarized in Fig. 4 and Table 1 are satisfied, the critical failure mode for the WFWW connection will be excessive plastic deformation and tearing at the beam plastic hinge location. Local buckling initiates at this hinge location after a sufficient length of steel has yielded, and ductile tearing occurs after the large inelastic strains occur. Experiments show that tearing and cracking of the beam web-to-column weld initiates shortly after the critical



(a) (b)

Figure 4. Post-Northridge Welded-Flange-Welded-Web (WFWW) Connection (Ricles *et al.*, 2000).

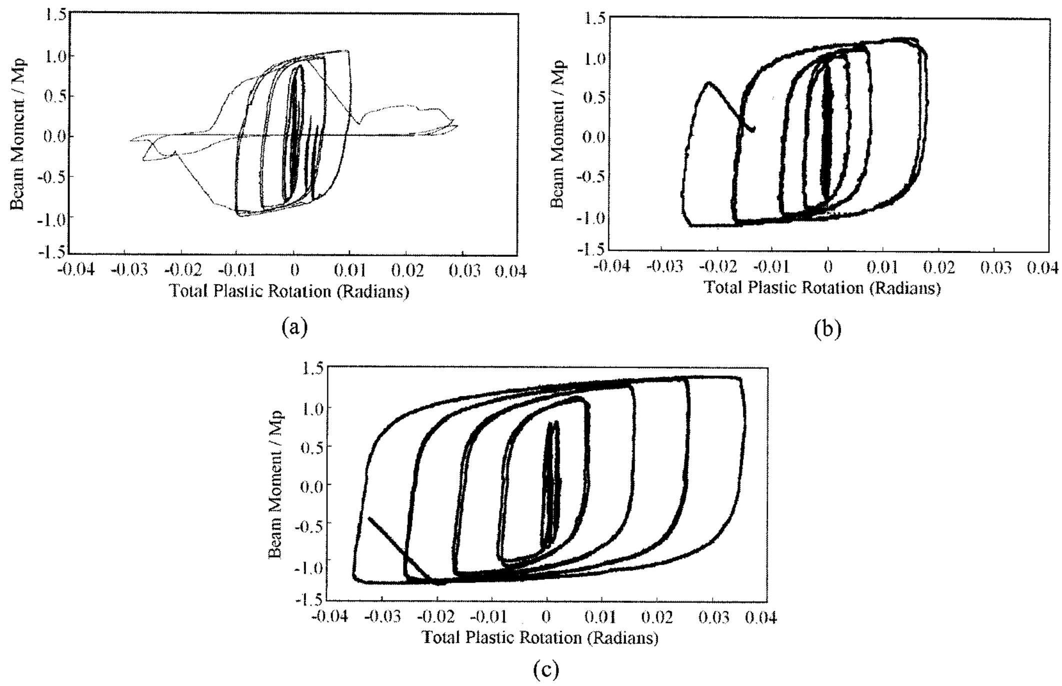
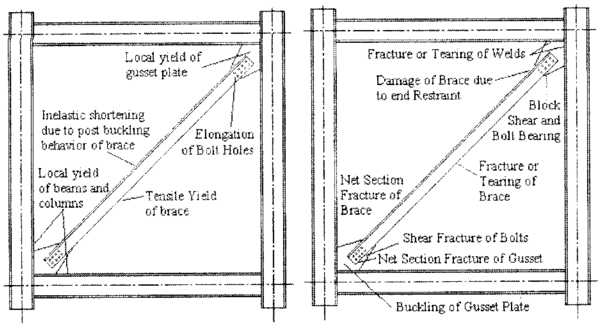


Figure 5. Response of pre and post-northridge MRF connections.

failure mode occurs and WFWW connections provide ductile and reliable behavior. To illustrate the benefits of the proposed design process for the welded-flange-welded-web SMRF connection, Figure 5 compares the response of connections with identical W36x150 beam and identical welding procedures. Figure 5a shows a typical moment-rotation curve for the proposed connection with a panel zone with β_y larger than 0.9 and without the benefits of the weld access hole geometry improvements and the web attachment requirements. This performance is significantly better than that achieved from the pre-Northridge SMRF connections, but ductility is limited. Figure 5b shows the benefit of the weld access hole geometry and finish and the strengthened panel zone on connection performance. These changes significantly improve the ductility of the connection over that shown in Fig. 5a. Figure 5c shows the behavior achieved when the proposed WFWW connection design procedure is employed. Comparison of this figure with Figs. 5a and 5b shows that the connection seismic performance is significantly improved by the method discussed previously.

5. Application of Procedure to SCBF Systems

As demonstrated in Fig. 5, implementation of the proposed design methodology for SMRF connections



a) Yield Mechanisms b) Failure Modes

Figure 6. Yield mechanisms and failure modes of CBF components.

resulted in improved seismic performance relative to traditional design procedures. Damage to CBFs in previous earthquakes and previous experimental research show that compressive buckling and tension yielding of the brace in a large part determine the seismic response of the SCBF system. The response of the brace is influenced by the connection properties, which must be directly accounted for in design. Equally important, yielding of the connection without premature failure may increase the deformation capacity of the framing system. However, current design does not provide this yielding hierarchy, and this suggests that the response of SCBF systems may also be improved by using a balance procedure to improve the performance of gusset plate connections. A recent research program has been undertaken to develop and validate design procedures based on balancing the yield mechanisms and failure modes observed for SCBF systems.

To achieve the desired yielding hierarchy, the yield mechanisms and failure modes for SCBF systems were identified from past experimental research and are illustrated in Fig. 6. For a SCBF, the controlling yield mechanisms should be inelastic shortening due to post-buckling deformation of the brace and tensile yielding of the brace. Limited local yielding of the gusset plate may be a tolerable secondary yield mechanism, since this may improve economy and increase the inelastic deformation capacity. Additional secondary yield mechanisms include yielding of the framing elements (beams and columns) and elongation of the bolt holes. Inelastic deformation of the framing elements has been noted in experimental studies, but its contribution to SCBF frame deformation capacity is not fully understood. More ductile failure modes include buckling of the gusset plate, bolt bearing or block shear and excessive plastic deformation of the brace, which may ultimately lead to fracture or tearing. Brittle failure modes include net section fracture of the brace or gusset plate, fracture of the gusset plate welds, and shear fracture of the bolts.

As for the MRF system, some brittle failure modes of the SCBF gusset plate connections may be prevented by means of prescriptive details. Procedures to determine the resistance of each of the yield mechanisms and failure modes are required for development of the balancing procedure, and existing experimental data was evaluated using established design models for SCBF yield mechanisms and failure modes.

The primary yield mechanisms include yielding and buckling of the brace. Prediction of these two yield mechanisms can be made by using AISC design methods to determine the primary yield resistances, as given in Equations (5a) and (5b) (AISC, 2002).

$$R_{\text{yield-buckling mean}} = R_y\ F_{cr}\ A_g \tag{5a}$$
$$R_{\text{yield-tension mean}} = R_y\ F_y\ A_g \tag{5b}$$

These yield resistances must be balanced with secondary yield mechanisms, and therefore accurate assessment of the tensile yield and compressive buckling forces is fundamental to the development of robust balance design procedures. The secondary yield mechanisms, such as tensile yield of the gusset plate, and failure modes must be balanced with respect to the primary tensile yield mechanism. Therefore:

$$R_{\text{yield-primary}} \leq \beta_{\text{yield-secondary}}\ R_{\text{yield-1}} \tag{6a}$$
$$R_{\text{yield-primary}} \leq \beta_{\text{fail}}\ R_{\text{fail}} \tag{6b}$$

A wide range of failure modes are possible, as shown in Figure 6, but most of these failure modes are based upon buckling, yielding or fracture of steel within a specific section. As a result, expressions to estimate the failure resistance values for net section fracture are expected to be of the form:

$$R_{\text{failure- net section fracture}} = C\ F_u\ A_{ns} \tag{7a}$$

where F_u is the ultimate tensile strength of the steel, A_{ns} is the area of the critical net section, and the constant C represents the variability of the tensile stress over the net section at the critical location. It should be noted that A_{ns} may occur at different locations, as shown in Figure 6, and be different values at these different locations. Failure resistance estimates for buckling are given in Equation (7b).

$$R_{\text{failure- buckling}} = F_{cr}\ A_{gs} \tag{7b}$$

where F_{cr} is the computed buckling stress for a critical failure mode and A_{gs} is the appropriate gross area for that failure mode. Finally, yielding may control some failure mode resistances, as will be shown in later discussion, and in this failure resistance may be estimated using Equation (7c).

$$R_{\text{failure- yielding}} = F_y\ A_{gs} \tag{7c}$$

Definition of A_{ns} for brace net section fracture is reasonably well established, but the appropriate A_{ns} for the gusset plate buckling and net section fracture must be determined.

6. Initial Proposals for SCBF Balance Procedures

Research has been conducted to develop balance procedures that are applicable to a range of braced frame connections. The initial expression are based on existing experimental research results. Further refinement of the balance procedures requires a focused experimental study, and this work is currently underway (Lehman *et al.*, 2004).

To date, the previous research has been used:

- to developed proposed equations that balance the yield mechanisms and failure modes for SCBF gusset plate connections,
- to determine the accuracy of various analytical predictions,
- to develop initial estimates of β values for the balancing procedure, and
- to establish issues of concern for applying the proposed balance strategy to SCBF systems.

Existing models of Equations (5) and (7) were evaluated with primary emphasis on predication of brace buckling, gusset plate buckling and more brittle failure modes of the gusset plate. Past experimental data on braced frame gusset plate connections (Hu and Cheng, 1987; Brown, 1988; Yam and Cheng, 2002; Rabinovitch and Cheng, 1993; Grondin *et al.*, 2000) and brace buckling tests (Astaneh-Asl *et al.*, 1982; El Tayeb, 1985) with gusset plate failures were examined.

Hundreds of brace buckling experiments have been performed. Relevant data were used to calculate the ratio of the ultimate measured compressive load of the brace to the yield-buckling resistance of Eq. (5a) as shown in Fig. 7. Test specimens included in figure experienced brace buckling, and the measured resistance is controlled by brace buckling. However, the ultimate failure modes may vary widely (as illustrated in Fig. 6) even though brace buckling was the dominate yield mechanism. As a result, different ultimate failures occur, and the legend in Fig. 7 identifies the ultimate failure mode of each test specimen. Only tests with adequate data to describe the buckling load, test specimen geometry, materials, and boundary conditions were included. The value of F_{cr} for

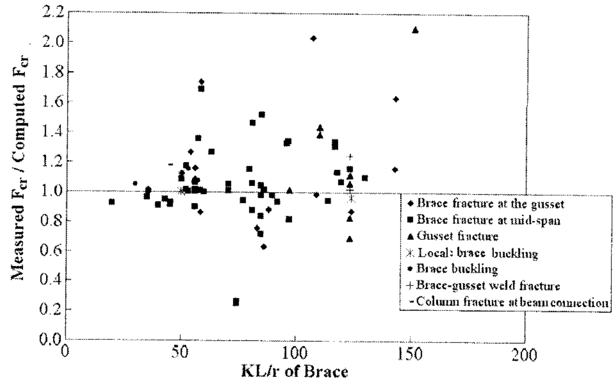
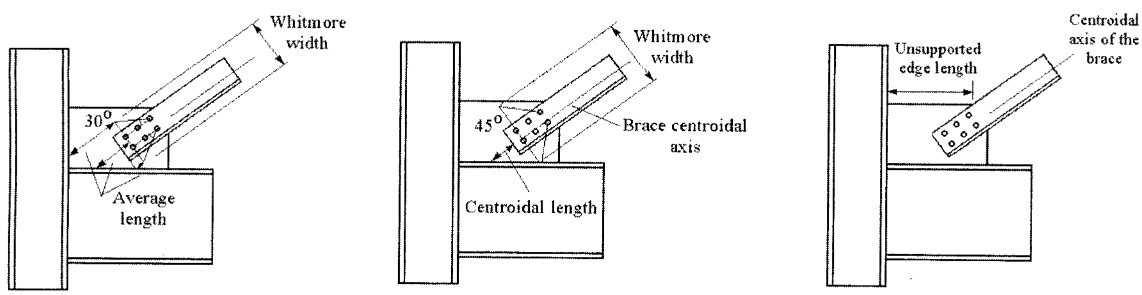


Figure 7. Comparison of measured brace buckling loads to predicted brace buckling loads.

Eq. (5a) was determined using AISC design methods, and therefore an average ratio somewhat greater than 1.0 was anticipated. Figure 7 shows that the scatter in the predicted buckling resistance prediction is large, and may be related to the failure mode occurring for that test. This inaccuracy is largely caused by uncertainty in the effective length coefficient, K, of the brace rather than uncertainty in the AISC design equations. The K-factor depends on an accurate assessment of the end restraint which depends on the gusset plate connection stiffness and restraint. A more accurate estimate of this buckling load is needed to establish the balance conditions required for ductile performance. Procedures to predict connection stiffness and its impact on brace buckling are needed.

Gusset plate buckling is another important failure mode. Most models for predicting gusset plate buckling use an equivalent Whitmore width (Whitmore, 1952) which is defined as the width over which an equivalent uniform stress acts. Traditionally, the Whitmore width is defined by a 30° distribution angle, as shown in Fig. 8a. Thornton (1991) used the Whitmore model in combination with an effective length factor to calculate a critical buckling load. For the buckling calculation, the gusset plate is treated as an imaginary strip column with the rectangular cross section defined by the Whitmore width. The length was the average of three measured



(a) Thornton Method (b) Modified Thornton Method (c) Unsupported Edge Length Method

Figure 8. Typical design models for gusset plate design.

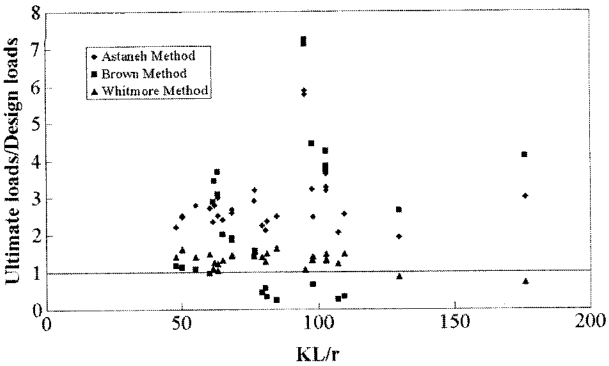
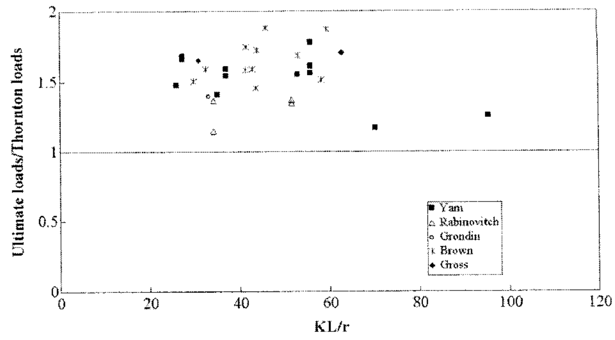
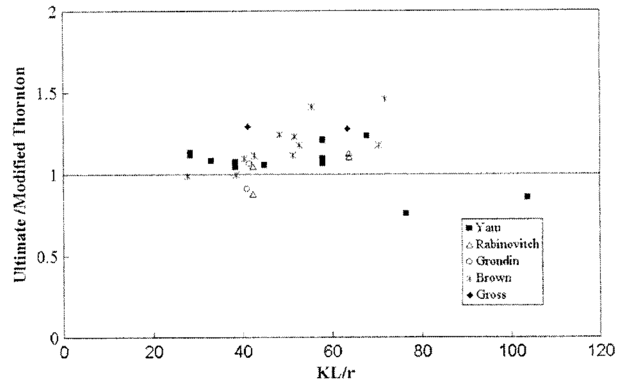


Figure 9. Accuracy of the whitmore, brown and astaneh-asl methods as a function of slenderness ratio.



(a) Thornton Model



(b) Modified Thornton Model

Figure 10. Accuracy of the thornton and modified thornton models as a function of slenderness ratio.

lengths from the centerline of the last row of bolts as shown in Fig. 8a. The effective length coefficient used is 0.65 which essentially assumes that the gusset plate is nearly fully restrained against end rotation and sidesway is prevented. The Modified Thornton method, shown in Fig. 8b, was proposed as an improvement to the Thornton method. It uses a Whitmore width defined by a 45^{o} projection angle, and an effective column strip length for buckling is the length along the centroidal axis. Brown (1988) and Astaneh-Asl (1989) proposed other gusset plate buckling models that use an unsupported edge length, as illustrated in Fig. 8c, combined with an effective width estimate.

Measured experimental results in which gusset plate buckling was reported were used to evaluate the models described above, and the results are presented in Figures 9 and 10. All of the test connections were concentric and therefore the impact of connection eccentricity was not investigated. In these figures, the ratio of the measured connection resistance to the resistance predicted by Eq. (7b). The ratio is plotted as a function of the gusset plate slenderness. The ratio is greater than 1.0 when a conservative prediction is provided. Figure 9 shows that the Brown and Astaneh-Asl models provide good comparisons for a few tests, but they show much more variability than the Whitmore based models and may be unconservative. The Whitmore model is has much less scatter, but it provides a very conservative resistance estimate except for more slender gusset plates.

The Thornton model is an adaptation of the Whitmore method to better address gusset plate buckling, and it is commonly used in engineering practice. The Thornton model results are shown in Fig. 10a. It conservatively estimates the gusset plate buckling capacity, since its average experimental gusset plate resistance is 1.54 times the predicted resistance, and the standard deviation of the ratio, σ, is 0.195. It provides improved estimates relative to the basic Whitmore and the Brown and Astaneh-Asl methods, but it is still quite conservative. Figure 10b compares the measured and predicted resistances using the Modified Thornton model. The Modified Thornton method is still on average conservative, since the average ratio is 1.11 and σ is 0.149, but the conservatism is less extreme. The prediction is not conservative in a few cases with slender gusset plates. The mean and standard deviation values suggest that 89% of the gusset plates designed by the Modified Thornton method will have larger resistance than the computed capacity. To develop a robust balanced equation, undue conservatism is not desirable because it leads to erroneous estimates of the failure mode for the structural system and erroneous predictions of the structural performance. Therefore the modified Thornton Model represents a good starting point for development of a balance procedure as described in this paper, but further improvements to this model may be possible.

Simple models for predicting the resistance and behavior of other failure modes are also required. Figure 11 compares the ratio of the measured and computed resistances obtained for several other failure modes as a function of brace slenderness. The experimental results show that fracture occurred at the end of the brace in a number of past experiments. Typically brace fracture occurred adjacent to the gusset plate connections for specimens where large plastic deformation in the brace resulted from brace buckling. The calculated resistance for this failure mode was the computed buckling capacity of the brace (AISC, 2002). The results, shown in Fig. 11,

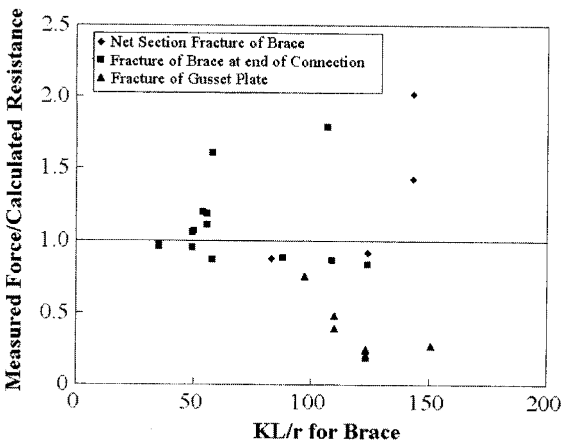


Figure 11. Model comparison for other failure modes.

illustrate that on average very slender braces did not meet the calculated resistance. Therefore, mitigation of this particular failure is likely to be aided by assuring that the SCBF brace meets the local and global slenderness requirements of AISC Seismic Design Specifications (AISC, 2002) and use of a β factor of 0.8.

The resistance values corresponding to net section failures of the brace were predicted using Equation 7a with C equal to 1.0 and A_{ns} determined by the AISC net section method. Figure 11 shows that these net section fractures occurred if the slenderness of the brace exceeded 75. Statistical analysis of the data shows that the mean resistance ratio and standard deviation for specimens with brace net section fractures was 0.925 and 0.271, respectively. This average resistance may be somewhat smaller than desired, and this suggests that a somewhat smaller β value would be appropriate. Fracture of the gusset plate may also be a net section fracture, and so the measured resistance for torn or fractured gusset plates was also normalized by Equation (7a) with a C value of 1.0 and A_{ns} is determined by the Whitmore method. Unfortunately, Figure 11 shows that this model is a poor indicator of the connection failure resistance. The statistical data shows that the mean and standard deviation of the measured to predicted resistance were 0.234 and 0.132, respectively. Further, the estimates were increasingly unconservative for more slender braces. As a result, this test data was examined more closely. It would normally be expected that many of these fractures would occur in the net section of the gusset at the last row of bolts, but most of the reported failures were not associated with these reduced sections. Many of these gusset fractures initiated near the weld of the gusset plate to the beam flange, and all of the fractures occurred in regions with large inelastic flexural strains required to achieve brace end rotations with tapered gusset plates, such as illustrated in Fig. 2b. None of these fractures were noted with rectangular gusset plates, such as illustrated in Fig. 2a. This plastic strain in the gusset is a fundamental requirement for many SCBF applications, because of a bending clearance requirement established in the AISC Seismic Design Specifications (AISC, 2002; Astaneh-Asl, 1982). As a result, it is expected that that much lower stress levels may be required at this critical section with tapered gusset plates than for the more standard rectangular gusset plates depicted in Fig. 2a. The stress limit for the tapered plate will probably need to be based upon yield stress rather than ultimate tensile stress with the tapered gusset plates, and the existing clearance requirements may need to be revisited. Clearly additional work is needed to address and better quantify these important failure modes.

Additional failure modes include bolt shear, bolt bearing, and block shear failures; all are clearly important. Analytical methods for predicting these failure mode resistances generally assume the brace force is uniformly distributed to all bolts, except for individual bolts with close edge distance or spacing. Past gusset plate connection experiments have seldom demonstrated these failure modes, but these failure modes (and the corresponding design assumptions) have been observed and evaluated in many other types of connection experiments (Roeder, 2001; Swanson *et al.*, 2000). Bolt shear is a relatively abrupt failure with severe consequences, but it is also predicted with reasonable accuracy and reliability, except for very large bolt groups. Block shear and bearing failures are accompanied by significant local inelastic deformation, and the measured capacity typically exceeds the computed resistance. Finally, weld fractures of the gusset plate have been noted in a very few tests which suggests that the weld process, electrode, and quality control contribute significantly to control of this failure mode.

Using the results of the simple analytical models to predict the failure modes, initial values for some of the balance parameters for SCBF systems are proposed. Table 2 provides a summary of the relevant primary yield mechanism, secondary yield mechanisms, more ductile and brittle failure modes. In addition, initial estimate of the balance parameters and rationale behind these proposed balance limits are provided. Although final β values for SCBF connections must be verified, the research discussed herein has provided the basis for further refinement of the design method, identified gaps in the available experimental data, and revealed shortcomings in current design models.

7. Summary, Conclusions and Recommendations for Future Work

A method for improving the seismic design performance of steel frame connections has been presented. The method employs balance conditions between controlling yield mechanisms and critical failure modes. This method was initially developed for and applied to steel SMRF connections. For those connections, the design procedure has been verified during past experimental

Table 2. Proposed balance equations and parameters for SCBF gusset plate connections

Mode or Mechanism		β Value	Notes on the Application
Primary Yield Mechanisms	Tensile Yield of Brace	Not Required	$P_y = A\ F_y$ controls for tensile failure modes.
	Brace Buckling	Not Required	$P_{cr} = A\ F_{cr}$ controls for compressive modes. Expressions that include effect of connection properties on K-factor are needed
Secondary Yield Mechanisms	Yield of Gusset Plate	Approx. 0.9	
	Yield of Framing Elements	Approx. 0.85	Requires accurate estimate of brace end moment and connection stiffness. Requires additional work.
	Gusset Plate Buckling	Approx.0.85	β times Modified Thornton Capacity > P_{cr}. Modified Thornton model provides reasonably accurate estimate.
More Ductile Failure Modes	Net Section of Brace	Approx. 0.8	Fracture conservatively estimated and expected with very slender braces.
	Excessive Deformation of Brace	Approx. 0.8	Controlled by AISC SCBF slenderness limits.
	Block Shear and Bolt Bearing	Approx. 0.8	Resistance estimates are conservative and failure mode is relatively ductile.
More Brittle Failure Modes	Fracture or Tearing of Brace	Less than 0.8	Existing SCBF design limits theoretically control.
	Gusset Weld Fracture	Less than 0.75	Weld fracture occurs at low stress levels with poor welds. Quality of weld and weld process must be controlled.
	Net Section Fracture of Gusset Plate	Uncertain but probably less that 0.7	Poor resistance estimate after brace buckling for slender braces. Significantly smaller for slender braces. May require different models for rectangular and tapered gusset plates. Work required here.
	Bolt Shear	Less than 0.8	Bolt resistance conservatively estimated but consequences severe. Existing models may be appropriate.

research, and variations of the method are employed in seismic design practice. A proposed application of this concept to SCBF gusset plate connections was described. Balance equations were proposed for the SCBF systems with gusset plate connection. To date, past experimental results have been used to develop initial estimates of the balance parameters. Additional research is needed to improve the proposed design models, further verify the balancing design concept, and validate the values of the balance parameters to achieve good seismic performance.

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