CHAPTER II

LITERATURE REVIEW

Hewitt, Sabelli and Bray (2009) recommend that Chevron bracing is avoided because it requires that the beam is designed to unbalanced forces caused by redistribution of internal forces when the bracing experience compression buckling (Figure 2.1).

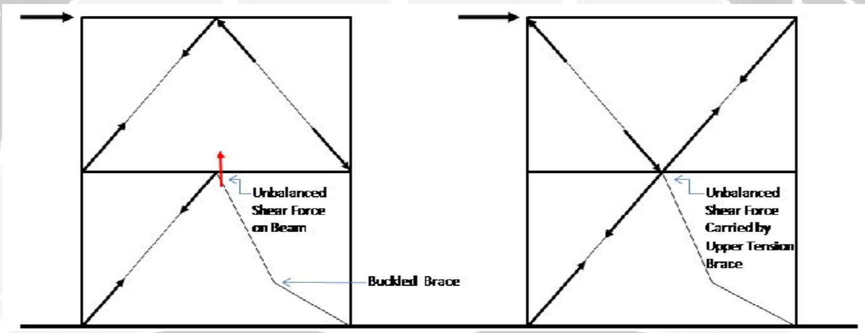


Figure 2.1 Comparison between *unbalanced vs balanced* on Chevron connection

A better alternative is to use two-story X bracing where the bracing at the top level which resist tensile force will resist the unbalanced forces on the beam so that the dimensions of the beam can becomes smaller. But the research results of Richards (2009) on SCBF with two-story X bracing shows that the axial forces in columns are sensitive to buckling in bracing so that the design of SCBF with two-story X bracing also require special attention such as the design of SCBF with Chevron type of bracing.

2.1 Buckling on Bracing

For frame systems that use type V bracing or inverted type V, if the bracing experience buckling compression force, connection on the middle beam which is intersect with the bracing will experience downward deflection as shown in the picture below. This deflection causes damage on the slab on top of that connection. Therefore, planners must anticipate the forces that are not balanced in the planning. Damage like this can not occur in X type of bracing because the bracing connections directly connected into the column.

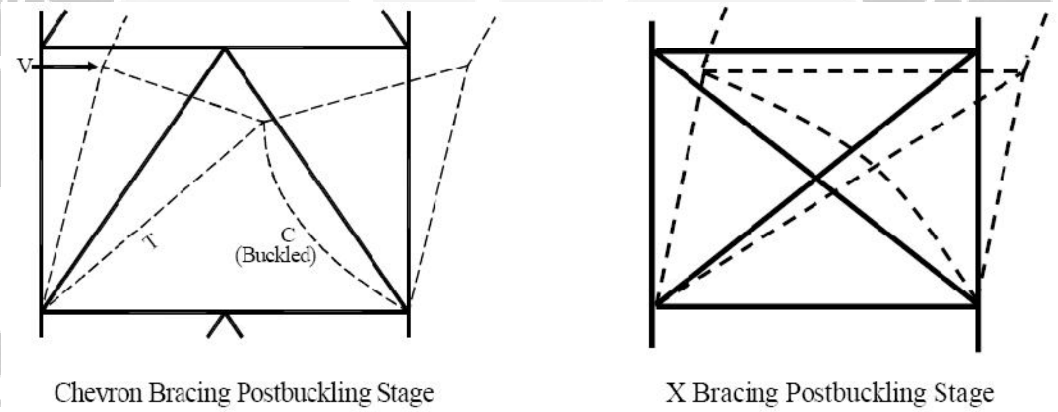
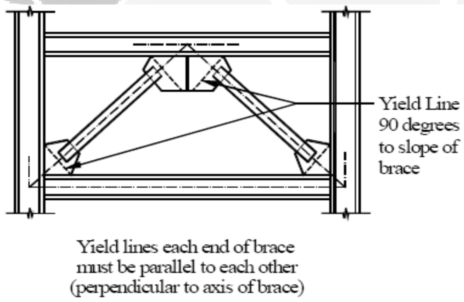


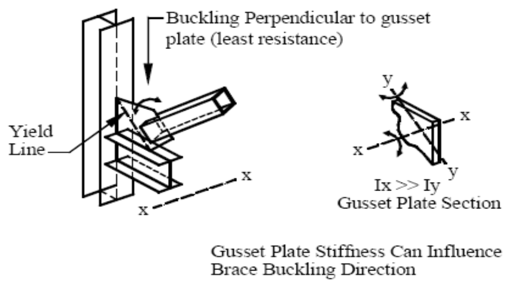
Figure 2.2 Chevron and X-Bracing Postbuckling Stage

Buckle in the bracing impact on the planning and detailing at the edge of bracing connection, so that on SCBF there are provisions that must be achieved to ensure a ductile response at the edge of bracing connection on SCBF when the bracing buckle because of strong earthquake. SCBF performance is depends on the buckling which is can be in-plane buckling of braces or out-of-plane buckling of braces. When bracing experience out-of-plane buckling of braces, this buckled bracing induces bending moments to the weak direction of the gusset plate. In order that the edge of bracing can rotate freely when buckled, two yield line in the gusset plate at the edge of bracing should be parallel like shown in the picture below. When these two yield lines in the gusset plate is not parallel then one of the edge experience greater rotational resistance from the other end. Gusset plate and edge of bracing is potentially experience failure or damage, the one which experience larger rotation.



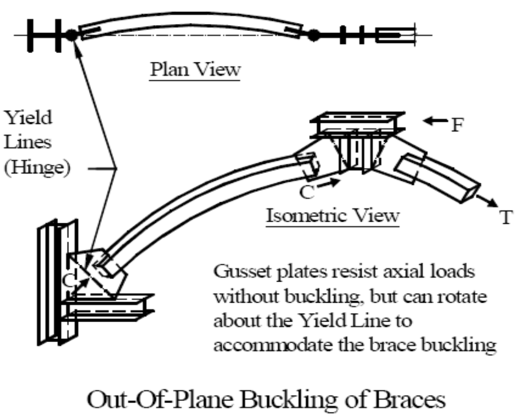
Out-Of-Plane Buckling of Braces

Figure 2-3A



In-Plane vs. Out-Of-Plane Buckling of Braces

Figure 2-3B



Out-Of-Plane Buckling of Braces

Figure 2.3 Out-of-Plane Buckling

2.2 Special Concentrically Braced Frame Design

SCBF designed such that so when the bracing resist a compressive force because of strong earthquake is experience buckling, SCBF capacity to resist the earthquake force was not reduced drastically. This post-buckling capacity is achieved through detailing and prevention of failures due to local buckling at the connection, also prevention of function failure even it have had bend. So if SCBF experience an earthquake that several times larger than the seismic design load, the integrity of SCBF stay good and SCBF still able to resist earthquake force without losing its capacity substantially.

AISC *Seismic Provisions for Structural Steel Buildings* (AISC 341-2005) were used to design steel structure with the needs of high ductility level (R > 3). In steel structures earthquake resistant, there are certain parts of the lateral load resisting system that is designed to withstand earthquake loads. As explained in the concept of *seismic fuse*, this task is to dissipate the earthquake energy with ductility through a stable cyclical yielding. Bracing work as a *seismic fuse* in CBF.

With a focus on ductility demand on the bracing then CBF behavior can be predicted. Braced is designed to remain ductile when experiencing cyclical yield.

The concepts of seismic fuse on CBF when applied to SCBF are:

1. Seismic energy dissipation is achieved through the tension yield and buckling on the bracing.
2. Bracing is designed to have a high level of ductility.
3. The connection of the bracing to the beams and columns should be designed to withstand loads due to yielding bracing and redistribution loads because of buckling in the bracing.
4. Bracing expected to experience buckling because of axial compressive force, and the gusset plates must be designed to resist flexural strength from the bracing. The gusset plates can also designed to accommodate the rotation of the buckling bracing.

2.2.1 The Bracing Design

Special Concentrically Braced Frame (SCBF) are expected to withstand significant inelastic deformation, so that SCBF shall meet some requirement. Terms of slenderness on SCBF are:

- $\frac{kL}{r} \leq 4\sqrt{\frac{E}{F_y}}$; where E = 29,000 Kips
- $4\sqrt{\frac{E}{F_y}} \leq \frac{kL}{r} \leq 200$; these term is permitted in frames which the available strength of the column is at least to the maximum load transferred to the column considering R_y.

The design strength of the bracing will follow the provision AISC 341-2005 13.2b which is the expected yield strength determined as $R_y\,F_y\,A_g$. R_y is the ratio of the expected yield stress to the spesified minimum yield stress F_y. The ratio is different depend on the type of the member. R_y is described on the table 2.1.

Table 2.1

R_y and R_t Values for Different Member Types

Application	R_y	R_t
Hot-rolled structural shapes and bars:		
• ASTM A36/A36M	1.5	1.2
• ASTM A572/572M Grade 42 (290)	1.3	1.1
• ASTM A572/572M Grade 50 (345) or 55 (380), ASTM A913M Grade 50 (345), 60 (415), or 65 (450), ASTM A588/A588M, ASTM A992/A992M, A1011 HSLAS Grade 55 (380)	1.1	1.1
• ASTM A529 Grade 50 (345)	1.2	1.2
• ASTM A529 Grade 55 (380)	1.1	1.2
Hollow structural section (HSS):		
• ASTM A500 (Grade B or C), ASTM A501	1.4	1.3
Pipe:		
• ASTM A53/A53M	1.6	1.2

Plates: • ASTM A36/A36M • ASTM A572/A572M Grade 50 (345), ASTM A588/A588M	1.3 1.1	1.2 1.2

2.3 Bracing Connections

AISC 341-2005 set the strength requirement of bracing connections (including beam-to-column connections which are part of the bracing system) should take the smallest value of the following things:

- The expected yield strength, in tension, of the bracing member, determined as $R_y F_y A_g$
- Maximum force, according to the analysis, which can be moved by the structure system to the bracing

But for this thesis, to design the connection just using the first point requirement, set as $R_y F_y A_g$.

2.3.1 Connection Design

It has been mentioned above that to design the connection it should refer to the strength of the bracing, that set as R_y F_y A_g. It means, the strength of the connection should be greater than the bracing connection. it can be written as,

$$P_{connection} > R_y F_y A_g$$

Which is if the connection is designed using bolted or welded, than it should be design to withstand the stress from the bracing.

2.4 Basic Theory

2.4.1 Loading analysis

Required Strength (U):

a. To resist dead load and live load:

$U = 1.2\, D + 1.6\, L$

b. To resist dead load, live load and earthquake load:

$U = 1.2\, D + 0.5\, L \pm 1.0\, E_x \pm 0.3\, E_y$

$U = 1.2\, D + 0.5\, L \pm 1.0\, E_y \pm 0.3\, E_x$

$U = 0.9\, D \pm 1.0\, E_x \pm 0.3\, E_y$

$U = 0.9\, D \pm 1.0\, E_y \pm 0.3\, E_x$

2.4.2 Connection

2.4.2.1 Bolted Connection using High Strength Bolt

The design shear strength of high strength bolt is $\phi\, R_n$, where the resistance factor ϕ is 0.75. The nominal shear strength of high strength bolts is given by the ultimate shearing stress times the nominal bolt area. And it is written as,

$$R_n = F_v\, A_b$$

and design strength as,

$$\phi\, R_n = 0.75\, F_v\, A_b$$

where A_b = area of the bolt

The strength of the bolt will be varies depend on the bolt that will be used. Table below is the nominal of F_v for bolt.

Table 2.2 Nominal Stress of Fasteners and Threaded Parts, ksi (MPa)		
Description of Fasteners	Nominal Tensile Stress, F_{nt} , ksi (MPa)	Nominal Shear Stress in Bearing-Type Connections, F_{nv}, ksi (MPa)
A307 bolts	45 (310)	24 (165)
A325 or A325M bolts, when threads are excluded from shear planes	90 (620)	48 (330)
A325 or A325M bolts, when threads are excluded from shear planes	90 (620)	60 (414)
A490 or A490M bolts, when threads are not excluded from shear planes	113 (780)	60 (414)
A490 or A490M bolts, when threads are excluded from shear planes	113 (780)	75 (520)

2.4.2.2 Welded Connection

The design and analysis of fillet welds is based on the assumption that the cross section of the weld is 45˚ right triangle, as shown in the Figure 3.1 any reinforcement (buildup outside the hypotenuse of the triangle) or penetration is neglected. The size of a fillet weld is denoted *w* and is the length of one of the two equal sides of this idealized cross section. Standard weld sizes are specified in increments of $1/16$ inch. Although a length of weld can be loaded in any direction in shear, compression, or tension, a fillet weld is weakest in shear and is always assumed to fail in this mode.

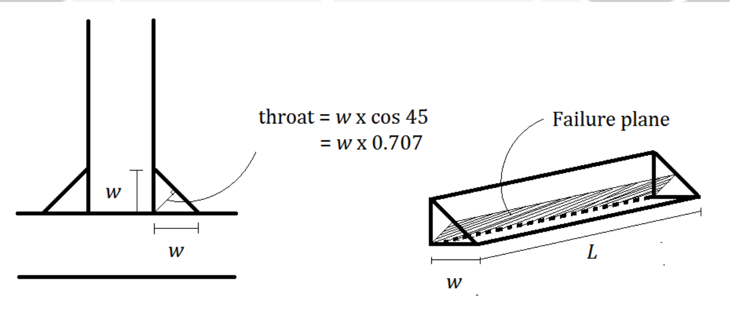


Figure 2.4 Throat and Failure Plane on Fillet Weld

Specifically, failure is assumed to occur in shear on a plane through the throat of the weld. For fillet welds made with the shielded metal arc process, the throat is the perpendicular distance from the corner, or root, of the weld to the hypotenuse and is equal to 0.707 times the size of the weld. Thus, for a given length of weld L subjected to a load P, the critical shearing stress is

$$f_v = \frac{P}{0.707\, w\, L}$$

where *w* is the weld size.

If the weld ultimate shearing stress, F_w, is used in this equation, the nominal load capacity of the weld can be written as

$$R_n = 0.707\, w\, L\, F_w$$

And the nominal design strength is,

$$\phi R_n = \phi\,(0.707\, w\, L\, F_w)$$

where $\phi = 0.75$

The strength of fillet weld depends on the weld metal used, it is a function of the type of electrode. The strength of electrode is defined as its ultimate tensile strength. The standard notation for specifying an eletrode is the letter E followed by two or three digits indicating the tensile strength in Kips per square inch and two digits specifying the type of coating. As strength is the property of primary concern to the design engineer, the last two digits are usually represented by XX, and a typical designation would be E70XX or just E70, indicating an electrode with an ultimate tensile strength of 70 Ksi.

The ultimate shearing stress F_w in a fillet weld is 0.6 times the tensile strength of the weld metal, denoted F_{EXX}. The nominal stress is therefore

$$F_w = 0.60\, F_{EXX}$$

In AISC 341-2005 there is a minimum size requirement of fillet welds that will be described on the table below.

Table 2.3 Minimum Size of Fillet Welds	
Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld, in. (mm)
To $^{1}/_{4}$ (6) inclusive	$^{1}/_{8}$ (3)
Over $^{1}/_{4}$ (6) to $^{1}/_{2}$ (13)	$^{3}/_{16}$ (5)
Over $^{1}/_{2}$ (13) to $^{3}/_{4}$ (19)	$^{1}/_{4}$ (6)
Over $^{3}/_{4}$ (19)	$^{5}/_{16}$ (8)

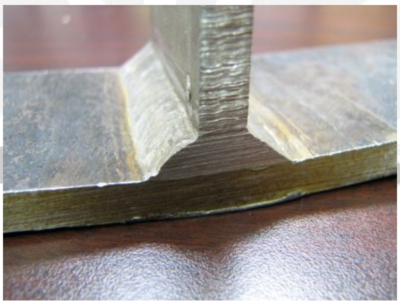


Figure 2.5 Fillet Weld

2.4.3 Tension Member

2.4.3.1 Tension Design

A tension member can fail by reaching one of two limit states, excessive deformation or fracture. To prevent excessive deformation, initiated by yielding, the load on the gross section must be small enough that the stress on the gross section is less than the yield stress F_y. To prevent fracture, the stress on the net section must be less than the tensile strength Fu. In each case, the stress *P/A* must be less than a limiting stress *F* or

$$\frac{P}{A} < F$$

Thus the load *P* must be less than *F A*, or

$$P < F A$$

The left side of this expression is the applied factored load, amd the right side is the strength. The nominal strength in yielding is,

$$P_n = F_y A_g$$

And the nominal strength in fracture is,

$$P_n = F_u A_e$$

Where A_e is the effective net area, which may be equal to either the net area or, in some cases, a smaller area.

The resistance factor $\phi = \phi_t$ is smaller for fracture than for yielding, reflectiong the more serious nature of reaching the limit states of fracture.

For yielding, $\phi_t = 0.90$

For fracture, $\phi_t = 0.75$

So from explation above it can be write as,

$$P_u \leq \phi_t R_n$$

or

$$P_u \leq \phi_t P_n$$

Because there are two limit states, both of the following condition must be satisfied:

$$P_u \leq 0.90\ F_y\ A_g$$

$$P_u \leq 0.75\ F_u\ A_e$$

2.4.3.2 Block Shear

For certain connection configuration a segment or "block" of material at the end of the member can tear out. For example, the connection of a plate tension member shown in Figure 2.3 is susceptible to this phenomenon, called block shear. For the case ilustrated, the shaded block would tend to fail by shear along the longitudinal section and by tension on the transverse section.

The procedure is based on the assumption that one of two failure surfaces fractures and the other yields. That is, fracture on shear surface isaccompanied by yielding on the tension surface, or fracture on the tension surface accompanies yielding on the shear surface. Both surfaces contribute to the total strength, and the resistance to block shear will be the sum of the strengths of two surfaces.

The nominal strength in tension is $F_u\ A_{nt}$ for fracture and $F_y\ A_{gt}$ for yielding, where A_{nt} and A_{gt} are the net and gross areas along the tension surface. Taking the shear yield stress and ultimate stress as 60% of the values for tension, the nominal strength for shear fracture is $0.6\ F_u\ A_{nv}$, and the strength for shear yielding is 0.6 $F_y\ A_{gv}$, where A_{nv} and A_{gv} are the net and gross areas along the shear surface.

There are two failure modes. For shear yield and tension fracture, the design strength is

$$\phi\ R_n = \phi\ [\ 0.6\ F_y\ A_{gv} + F_u\ A_{nt}\ ]$$

For shear fracture and tension yield,

$$\phi R_n = \phi [\ 0.6\ F_u\ A_{nv} + F_y\ A_{gt}\ ]$$

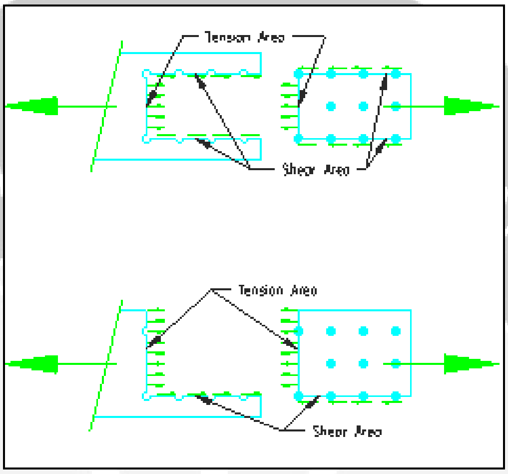


Figure 2.6 Block Shear

In both cases, ϕ = 0.75. Because the limit state is fracture, the controlling equation will be the one that has the larger fracture term. This means that if $F_u\ A_{nt} > 0.6\ F_u\ A_{nv}$, the first equation controls. Otherwise, the second equation control.

To avoid using a yield strength that is greater than the fracture strength along a surface, the AISC Spesification imposes an upper limit of $\phi [\ 0.6\ F_u\ A_{nv} + F_u\ A_{nt}\ ]$ on both equations. For some types of connection , block shear failure will actually involve fracture along both the shear and tension surfaces (Birkemoe and Gilmor, 1978). The block shear strength is therefore given in AISC J4.3 as follows:

a. When $F_u\ A_{nt} \geq 0.6\ F_u\ A_{nv}$

$$\phi R_n = \phi [\ 0.6\ F_y\ A_{gv} + F_u\ A_{nt}\ ] \leq \phi [\ 0.6\ F_u\ A_{nv} + F_u\ A_{nt}\ ]$$

b. When $F_u\ A_{nt} < 0.6\ F_u\ A_{nv}$

$$\phi R_n = \phi [\ 0.6\ F_u A_{nv} + F_y A_{gt}\ ] \le \phi [\ 0.6\ F_u A_{nv} + F_u A_{nt}\ ]$$

2.4.4 Compression Member

2.4.4.1 Compresion Design

The requirement for compression members are covered in Chapter E of the AISC Spesification. The relationship between loads and strength takes the form

$$P_u \le \phi_c P_n$$

where:

P_u = sum of factored loads

P_n = nominal compressive strength = $A_g F_{cr}$

F_{cr} = critical buckling stress

ϕ_c = resistance factor for compression member = 0.85

Instead of expressing the critical buckling stress F_{cr} as a function of the slenderness ratio $\frac{kL}{r}$, the pesification uses the slenderness parameter

$$\lambda_c = \frac{kL}{r\pi}\sqrt{\frac{F_y}{E}}$$

which incorporates the material properties but is nondimensional. For elasic members, it can written as

$$F_{cr} = \frac{\pi^2 E}{(kL/r)^2} = \frac{F_y}{\lambda_c{}^2}$$

For inelastic members, the tangent modulus equation is written as

$$F_{cr} = (0.658^{\lambda_c{}^2})\ F_y$$

Thus a direct a direct solution can be obtained, avoiding the trial and error approach inherent in the use of the tangent modulus equation. If the boundary between elastic and inelastic members is taken as $\lambda_c = 1.5$, the AISC equation for critical buckling stress can be summarized as follows.

For $\lambda_c \leq 1.5$,

$$F_{cr} = (0.658^{\lambda_c^2})\, F_y$$

For $\lambda_c \geq 1.5$,

$$F_{cr} = 0.877\, \frac{F_y}{\lambda_c^2}$$

2.5 Gusset Plate

The primary design steps for a gusset-plate connection, which take these failure modes into consideration, are as follows.

- The welds or bolts used to attach the brace to the gusset plate must be designed to provide the expected tensile yield resistance of the brace, and the weld length or bolt group must also be checked using the block shear design expression.
- The yield and buckling strengths of the plate are calculated using the Whitmore width and modified Thornton design expressions and compared with the tensile and compressive strengths of the brace, respectively. (Figure 2.7 shows these relevant variables for bolted and welded connections, respectively). The Whitmore width is defined by a 30° projected angle from the start to the end of the bolted or welded joint. In addition, an edge buckling check (Brown 1988; Astaneh-Asl 1998) is often employed, as suggested in Figure 2.7(c).

• The welds, which are fillet or complete joint penetration (CJP) welds, attaching the gusset plate to the beam and column are sized (the interface welds) for design forces determined using equilibrium methods with the expected tensile force in the brace.

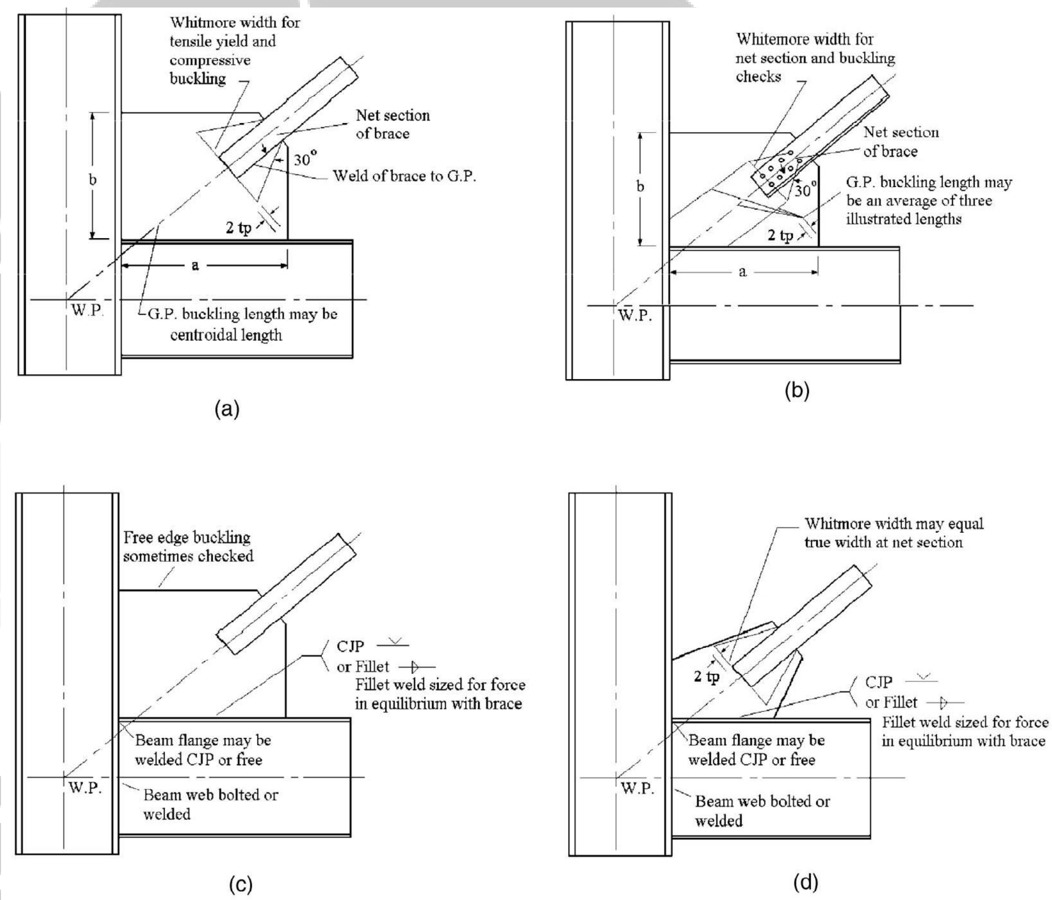


Figure 2.7 Schematic of SCBF gusset plate connections and design checks (a) welded tube brace; (b) bolted angle connection; (c) gusset plate welds; and (d) free edge buckling

The AISC seismic design provisions require allowances for the brace end rotation, and a 2*tp*-linear clearance at the end of and parallel to the axis of the brace is commonly employed (Figure 2.7), where ***tp*** (thickness of the gusset plate). This requirement can result in very large gusset plates.