

SEISMIC DESIGN AND STEEL CONNECTION DETAILING



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ABSTRACT

The seismic design of connections for both steel moment frame and braced frame members has become much more complicated since the original adoption of the 1997 UBC, current adoption of IBC2000 and pending adoption of IBC2003 in the next year of so by the various States and local Building Departments. Since the 1994 Northridge Earthquake, there have been significant changes in the AISC Seismic Provisions for Steel Buildings (1997 edition, 1997 Supplements #1 and #2, 2002 edition) impacting design of both steel moment frame and steel brace framed lateral resisting systems. The results from the SAC program, initiated after the Northridge Earthquake, lead to the development of the FEMA 350 - 354 documents which are being extensively used for steel moment frame design.

The intention of this paper is to provide some practical and useful information regarding connection design for both steel moment frames and steel braced frames. The emphasis is detailing for Seismic Design Categories (SDC) of D and higher which require Special Steel Moment Frames (SMF) and Special Concentric Braced Frames (SCBF). Detailing methods presented in this paper for SMF and SCBF may still be applicable to the steel Intermediate Moment Frame (IMF), steel Ordinary Moment Frame (OMF), and steel Ordinary Concentric Braced Frames (OCBF) in lower SDC A, B and C.

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MICHAEL L. COCHRAN S.E.

Introduction: Building Codes

The three typical model building codes used in the United States have now merged as a single model building code: the International Building Code, commonly referred to as the IBC. The first published code edition was the *2000 IBC* which has been adopted by some states and is in the adoption review process in other states. The *2003 IBC* edition is due to be published in the near future. It incorporates the changes found in the *2001 IBC* and *2002 IBC Supplements* that have impacts on the design of the lateral resisting systems for steel structures such as braced frames and moment resisting frames.

There is a second national building code being drafted by the National Fire Protection Agency (NFPA), which is gathering support from some governmental jurisdictions and agencies. Which model building code is selected by any individual states doesn't appear to matter much for new building design, since both model building codes reference seismic design of steel structures to the *1997 AISC Seismic Provisions for Structural Steel Buildings* (Yellow Book) and Supplements. The lateral resisting systems design restrictions in both model building codes are being referenced from the ASCE-7 document on building loadings.

Some states will continue to use the outdated 1997 Uniform Building Code (UBC) with some local amendments for the next couple of years. The continued use of the 1997 UBC by these states is due to individual state code adoption processes and ongoing political infighting between special interest groups, unrelated to structural engineering, that oppose code changes found in the family of IBC model codes. Such is the case in California which will likely continue with the 1997 UBC for another three years.

Engineers doing structural steel seismic design can reliably count on two documents as being state-of-the-art design; the *1997 AISC Seismic Provisions for Structural Steel Buildings* publication, including

Supplements Number 1 and Number 2, and the FEMA 350 - 354 document series for steel moment frame design. The *2002 AISC Seismic Provisions for Steel Buildings* edition is due to be published shortly, and design engineers should use this most recent edition whenever possible, or portions thereof, even if not yet adopted by the state and local building jurisdictions, as long as it is not in conflict with the legally adopted building code being enforced at the time. Many building departments will allow the use of more recent design provisions, especially if it can be shown that they have been adopted for the next three year building code cycle by means of the annual building code supplements (example: 2001 and 2002 IBC supplements).

If the design engineer chooses not to use the most recent state-of-the-art design information, since it has not yet been adopted by the local building department jurisdiction, the design engineer should remain aware of these design impact changes since they will likely be adopted in the next building code cycle of the IBC.

Steel Moment Frames:

The FEMA 350 – 354 publications should be used for the design of steel moment frame connections. The FEMA 350 documents are the result of the SAC program that was put together after the 1994 Northridge Earthquake due to the unexpected performance of steel moment frames. The *2002 AISC Seismic Provisions for Structural Steel Buildings* is not a replacement for the FEMA 350-354 documents, but should be used in conjunction with these documents. The design engineer should be aware of height restriction changes for steel moment frames shown in the 2002 IBC Supplement. In Seismic Design Categories (SDC) D, E, F the ordinary moment frame (OMF) is no longer permitted and the intermediate moment frame (IMF) is now restricted to 35 feet in height in SDC D, and not permitted in SDC's E and F except as allowed per the footnotes in IBC table 1617.6 "Design Coefficients and Factors For Basic Seismic-Force-Resisting Systems". Therefore, most three story and all taller buildings will have to use SMF's in SDC's D, E, F. Both the OMF and IMF will still be permitted in SDC's A, B and C with no Height Restrictions.

Single and two story buildings are a major portion of the overall building stock which is constructed each year in the United States. These buildings are often a combination of materials: concrete, masonry, and wood shear walls, wood and bare steel deck floor and

roof diaphragms, tilt-up construction, etc. Steel moment frames are often used in combination with some of these other lateral resisting systems or steel braced frames. Due to the lower R values associated with these type of building systems (Bearing Wall System and Building Frame System), typically the design engineer will be looking to use either an OMF or IMF instead of SMF's.

In the author's opinion, design engineers should use an IMF whenever possible instead of an OMF since the OMF is typically an untested connection. The IMF as defined in the AISC Seismic Provisions and the IBC is the same as the OMF in the FEMA 350 - 354 documents; they both have the same beam connection rotational requirements. Due to the uncertainty regarding reliability of an untested connection, the author would not recommend using the OMF in more than a single story building in any SDC, and only if for some reason an IMF could not be used. The use of the OMF will likely be limited to use in metal manufactured buildings (Butler type buildings).

Design of the moment frame connections has been fairly well addressed in various publications and engineering seminars. The most commonly used connection being the Reduced Beam Section (RBS), sometimes referred to as the Dog-bone connection. Two proprietary moment frame connections; "Sideplate" and "Slotted Beam Connections" have also been used in a number of steel framed buildings. Building owners have found in some cases these two proprietary connections to be an economical alternative to the non-proprietary moment frame connections such as the RBS.

Connection areas which have not been addressed typically are the moment frame roof beam connection and column foundation connection.

Steel Moment Frame Roof Connections:

This connection is often over-looked by the design engineer. The connection often involves a sloping beam or girder for roof drainage as oppose to the level framing members usually assumed with level floors within the building. Also this is the point where the column terminates.

Usually the beam connection to the column, is directly to the column flange. At the roof, a cap plate can be placed over the top of the column instead of using two fitted stiffener plates each side of the column web. Making the connection of the top beam flange to a column cap can be accomplished by

complete penetration welding of the beam top flange to the cap plate edge. The cap plate can be cantilevered beyond the face of the column flange to make for a cleaner connection as shown in the Figure 1.

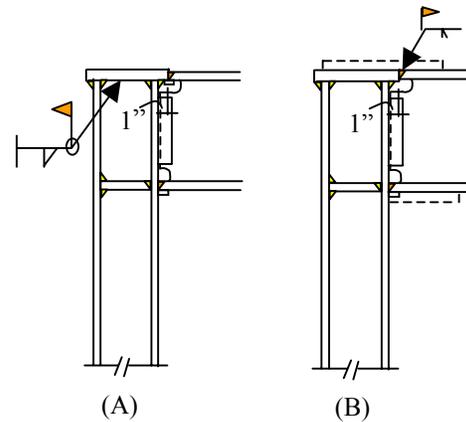


Figure 1: Column Cap Plate

The cap plate can cantilever out about one inch or so, which allows for a simpler complete penetration weld to the cap plate. You no longer have to worry about a potential stress riser that occurs when you have the back-up bar right up against the column flange, the predefined notch effect.

The cap plate can be fillet welded to the top of the column around the flanges and web, omitting the fillet welding in the k area of the column section. Since you can fillet weld across the face of the column flanges to the cap plate, you will likely be able to develop the shear force associated with yielding of the top beam flange cross sectional area.

Complete penetration welding of the beam flange directly to the cap plate edge also prevents potential laminar tearing that could occur in the column flange if the complete penetration weld was made directly to the face of the column flange. At the beam bottom flange you have no option but to weld to the column flange surface.

Figure 1B shows that flange cover plates could also be used over the cap plate to help develop the beam bending moments into the column.

The column continuity plate aligning with the lower beam flange may only require complete penetration welds to the column web and column flange welded to the moment frame beam flange. A fillet weld top and bottom of the continuity plate to the opposite column flange away from the moment frame beam may be all that is required. With the FEMA 350

design requirements to develop the continuity plate strength into the column web, considering the access holes to prevent welding in the column “k” area,

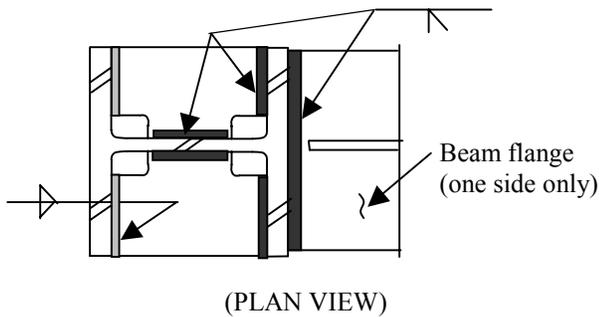


Figure 2: FEMA 350 FIGURE 3-6

Complete Penetration welds may be more practical than using very thick fillet welds to the column web.

There are also several other things the design engineer needs to consider when designing the connections shown in Figure 1:

1. The shear in the column just below the roof beam is equal to:
 $V_{col} = (\sum M_p \text{ beams}) / (\text{top floor story height} / 2)$
2. At the roof it is not necessary to satisfy Strong Column/Weak Beam which is very difficult to accomplish at interior columns of two bay or multi-bay frames since you have two beams framing only to the single column below the roof level (you have no column above the roof level to help redistribute beam moments)
3. The column panel zone shear check at the roof level, the shear resulting from summation of beam bending moments, is not reduced based upon distance to column inflection points (or half of average story height). All beam induced shear goes through the panel zone.

$$\text{Panel zone shear } V = C_y M_c / (d_b - t_{fb})$$

C_y and M_c are as defined in FEMA 350 sections 3.2.7 and section 3.2.6 respectively.

Moment Frame Base Connections:

Every building type is different, and the moment frame connection at the base is considered to be either pinned (example: base plate sitting on top of an elevated concrete slab) or fixed (base of column

encased by a concrete grade beam). In the author’s opinion, in order for the column base connection to be considered fixed, the base of the column needs to be attached to a moment frame beam or embedded in a concrete wall a sufficient depth. See Figure 3.

Using a base plate sitting on the surface of the concrete with anchor bolts to develop fixity into the foundation is not appropriate. The anchor bolts can/will develop more restraint than what is considered as being a true pinned connection, but due to elongation of the anchor bolts from tensile yielding, there will be more drift than if you had the fixity from a grade beam rigidly attached to the column base. You will be underestimating the amount of moment frame drift by trying to develop a moment frame connection just by anchor bolts.

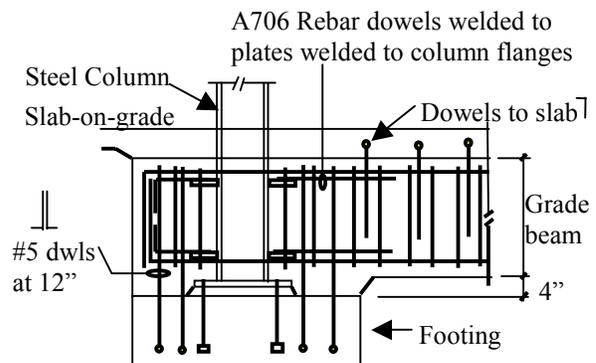
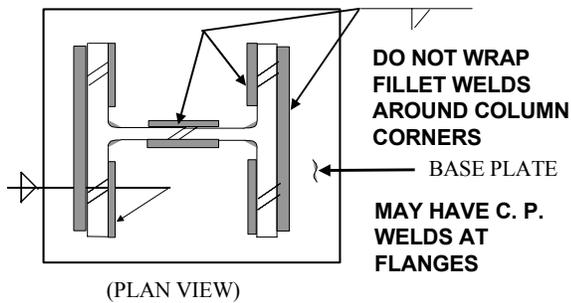


Figure 3: Concrete Grade Beam Around Steel Column Base

When you have high column uplift forces, use of the base plate in bending to transfer uplift forces from the column to the anchor rods is not appropriate. Steel shoes should be attached directly to the column sides to transfer the uplift force by shear and bearing from the column to the anchor rods (see page 4-131 of AISC Steel Construction manual, Ninth Edition for similar details). When the column is embedded in a concrete wall, then uplift can be transferred by shear into the concrete through Nelson studs welded to the flanges and webs of the steel column.

Rebars may need to be welded to the column to help develop the column bending moment into the concrete grade beam by means of a tension / compression coupling in the grade beam. As part of any good detailing, the grade beam and footing should be tied together as can be seen with the #5 dowels shown in Figure 3. Also you don’t want any welding in the “k” area of the steel column to the base plate as shown in Figure 4.



NO FILLET WELDING IN "K" AREA
Figure 4: Moment Frame Base Plate

Steel Braced Frame Systems

The design of steel brace frames has become much more complicated in high seismic regions, particularly those areas classified as seismic design categories (SDC) type D, E, F. Adoption of the the 2003 IBC will limit the use of Ordinary Concentric Braced Frames (OCBF's) in SDC D and E to 35 feet in height and not permitted in SDC F except as permitted by the footnotes (OCBF limited to roof top penthouse structures). Also, the OCBF will no longer be allowed to be used as part of a Dual System with steel moment frames as defined by the building code. Basically, all buildings three stories or taller will have to utilize Special Concentric Braced Frames (SCBF's) when using a braced frame system.

The stiffness and buckling behavior of a brace frame system is exactly the same for a SCBF and OCBF until the brace buckles. In a SCBF the selected brace size is generally smaller (larger permissible KL/r ratios than OCBF's), and is expected to buckle at a lower seismic axial load than for a comparably designed OCBF. *A brace frame doesn't know whether it is a SCBF or an OCBF until the brace buckles*, at which time buckling behavior is influenced by the brace gusset plate connections to the columns and beams.

The gusset plate design guidelines presented in this paper still require selection of a given model building code to follow and establish design criteria. Since buildings codes are basically living documents, the 1997 AISC Seismic provisions for Structural Steel Buildings, and its later supplements, are consider to represent the most current thinking on the state-of-the-art for seismic design of steel structures and shall be used for designing the SCBF connections as oppose to the 1997 UBC. There are subtle differences between these two particular design codes, and the design engineer should review both codes to determine the differences and

appropriateness of each. The IBC2000 references the 1997 AISC Seismic Provisions, but designers should still make themselves familiar with the 2002 AISC Seismic Provisions which will be adopted in the IBC2003.

The minimum prescriptive design requirements for steel special concentric braced frames (SCBF) are currently listed in the model building codes such as the 1997 Uniform Building Code and 1997 AISC Seismic Provisions for Structural Steel Buildings (Yellow Book). Additional commentary about the past performance and expected future performance of braced frame systems is also provided in the SEAOC Blue Book. Some design examples have been published by SEAOC in their Seismic Design Manual Series, but are currently based upon the 1997 UBC and are being updated to the IBC2000.

The IBC now adopts the AISC Seismic Provisions by reference, and has no specific brace frame design requirements as has been typically found in previous model building codes such as the UBC, BOCA, etc. The design engineers should therefore only have to check one source instead of two sources for prescriptive steel framing design requirements.

Information which typically has not provided in these model building codes or design provisions is practical design guidelines for brace frame gusset plate connections. The purpose of this paper again is to help fill this void and provide the practicing structural engineer with some practical and useful design information. Additional steel braced frame gusset plate design information can be found in seminar notes available from the Structural Engineers Association of California (SEAONC Section – April 2000 and SEAOSC Section – November 1999) and the Seismic Design Manual Vol III published by ICBO. An updated *Steel Tips* publication on SCBF design by the Structural Steel Educational Council in California may be available by the end of the year and would likely be available for download from the AISC website (www.AISC.org).

Brace Frame Types:

There are several typical configurations for Braced Frames which are shown in Figure 5. These braced frames can be designed as *either* OCBF's or SCBF's. The brace frame connection is typically concentric, that is the brace centerline intersects at the intersection of the column and beam centerlines.

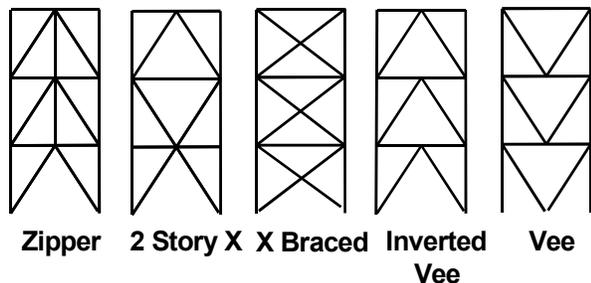


Figure 5: Types of Brace Frame Systems

Non-concentric connection bracing is permitted by the building code, and is often referred to as being an Eccentrically Brace Frame or EBF. Previous building codes have allowed non-concentric connection bracing without requiring the brace frame design to comply with EBF design requirements. The maximum connection eccentricity was limited to the depth of the beam or column member being intersected by the brace (example: 1997 UBC).

The 2002 AISC Seismic Provisions for Structural Steel Buildings have no written restrictions on the amount of eccentricity allowed in the brace frame connection before you must comply with EBF design requirements. In the authors opinion, some eccentricity in the SCBF's connection design should be permitted, which will help reduce the size of the required gusset plates (example: shifting the brace centerline such that it crosses the column flange at the intersection of the beam flange and column flange, see figure 6. The design engineer will have to account for all secondary moments in the connection due to the brace eccentricity from the column and beam centerlines. The SCBF or OCBF connection design eccentricity should probably not exceed the smaller of half of the beam depth or half of the column depth.

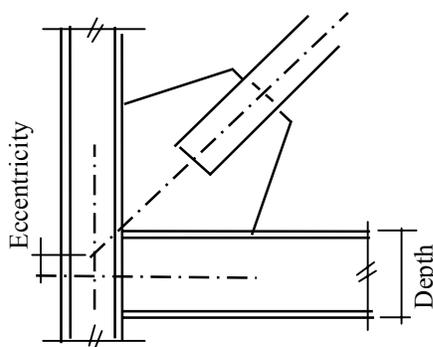


Figure 6: Eccentric Brace / Gusset Plate Connection

The connection design shown in this paper is based upon concentric connection design, but could easily be applied to a slight eccentricity as shown in Figure 6.

Brace Frame Behavior

Because of poor performance during past earthquakes of chevron bracing, only X bracing or chevron bracing with a zipper column is recommended in areas of high seismicity. Single diagonal braces are also permissible, but there are design penalties for such bracing systems unless you have multiple bays of bracing oriented such that the bracings are acting in tension and compression simultaneously along that line. Figure 7 depicts the problem with chevron bracing.

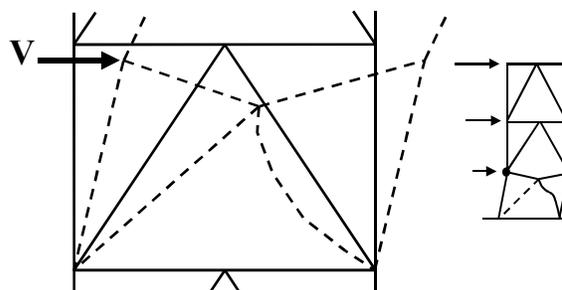


Figure 7: Chevron Brace Buckling

When the compression brace buckles, the tension brace force now doubles (before buckling have 50% of V in the tension brace and 50% of V in the compression brace). The vertical component of the tension brace axial force becomes a point load on the beam, pulling the beam down and possibly leading to hinging and buckling of the brace frame column.

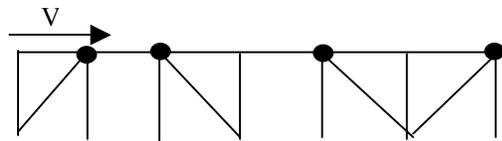
When chevron bracing is used, the beam must be designed for an unbalanced load when the compression brace buckles. Often the resulting brace frame beam design ends up weighing more than 200 pounds per foot. By comparison, when a two story X brace is used, when the compression brace buckles at the first floor, the braces at the second floor prevents the brace frame beam from buckling and you don't have to design the beam for an unbalanced loading.

When using the Zipper column in chevron bracing, you still have to design the beam for the unbalanced brace forces, but you can now spread this force over several floor levels. The question then is, how many of the compression braces buckle simultaneously, resulting in a cumulative vertical force that is to be transferred by the zipper columns over all of the floor

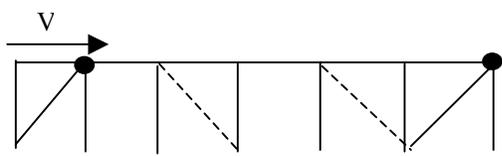
levels? There are no guidelines to determine the number of compression braces which may buckle. The number should be based upon engineering judgement and analysis, which may require a non-linear analysis of the building to determine the number of compression braces that buckle since not all braces may buckle, depending upon brace sizes selected.

The single story X or two story X braces are the preferred brace systems, since you have more truss behavior (axial loading) than flexural behavior (beam buckling) of the brace frame system. When the compression brace of the x brace system buckles, you still have truss action occurring between the tension braces and the columns, a result of the remaining triangular configuration of the brace frame bay. The engineer may want to check that tension yielding of the brace will not buckle the column in compression.

When using a series of alternating single diagonal braces along a common grid line, the engineer needs to verify that the drag collectors can transfer the forces to the remaining tension bays along that grid line when the compression braces buckle. The actual drag line becomes longer when the compression brace buckles. See Figure 8.



8A. Before compression braces buckle



8B. After compression braces buckle

● = Transfer point forces into braces

Figure 8: Alternating Single Diagonal Braces

As can be seen in Figure 8A, the drag lines are substantially shorter between force transfer points. After the compression braces buckle, the drag line becomes longer, as shown in Figure 8B. When you have X bracing or chevron bracing in a single bay, you don't have the same type of drag line condition to be concerned about when the compression brace buckles.

SCBF Brace Buckling

The SCBF is considered to be a better system than the OCBF due to the better ductility of the system achieved through individual brace member design and gusset plate design. The brace when axially loaded in compression will eventually buckle, the direction of brace buckling depends upon the brace shape orientation and the end restraints of the brace (the connections to the beams and columns).

The AISC Seismic Provisions provide an exception (Section 13.3c) that when met, allows for the brace to buckle out-of-plane and is further discussed in the AISC Seismic Provision Commentary. With the predominant use of gusset plates, this exception is probably used 95% of the time in brace design. The brace connection using a vertical gusset plate has a tendency to buckle out-of-plane due to the lack of stiffness in this direction.

As can be seen in the Figure 9 illustration, the gusset plate has significantly less stiffness in the out-of-plane direction. If the brace is a square tube, you have a 50-50 chance as to which way it will buckle. The end connections can then help influence which way the brace will buckle. Since we have significantly less stiffness in the out-of-plane direction, it is highly likely that the brace will buckle out-of-plane as opposed to in-plane.

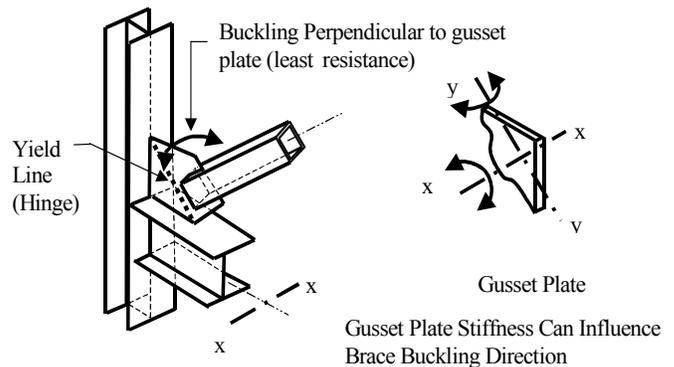


Figure 9: In-Plane vs Out-of-Plane Buckling Of Braces

When a brace buckles out-of-plane relative to the gusset plate, it attempts to form a hinge line in the gusset plate. In order for the brace to rotate and yield about this hinge line (act as a pin connection), the yield lines at each end of the brace must be perpendicular to the brace axis. This is illustrated in Figure 10 and Figure 11.

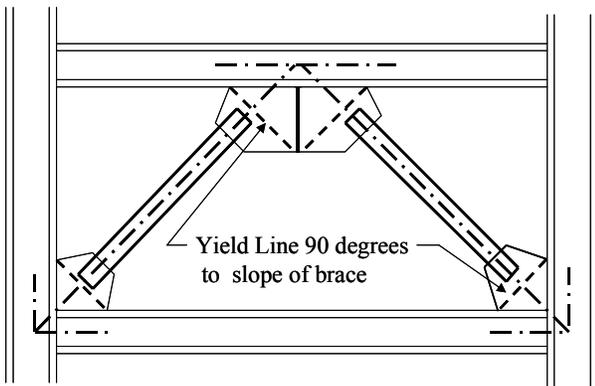


Figure 10: Bracing And Gusset Plate Yield Lines

to erection tolerances and other variables, it is strongly recommended that this design offset not be less than $3T$, that is three times the gusset plate thickness. The author recommends that the gusset plate design be for $4T$ while the detailing on the drawing shows $3T$. When the brace is mis-aligned in the field and you have $2T$ at one end and $4T$ at the other end, instead of $3T$ offset at each end, your design is still adequate.

There has been a misconception in previous interpretations of the yield line offset, that all you had to do was shape the end of the brace relative to the yield line so that they both were parallel to each other. Inherently what happens is that the yield lines at the opposite ends of the brace are not parallel (see Figure 10 for parallel yield line illustration), to each other and restraint builds up in the gusset plate as it attempts to buckle out-of-plane. The only way to relieve the stress is for the gusset plate to tear at one end of the brace, until the yield lines at each end of the brace are again parallel to each other.

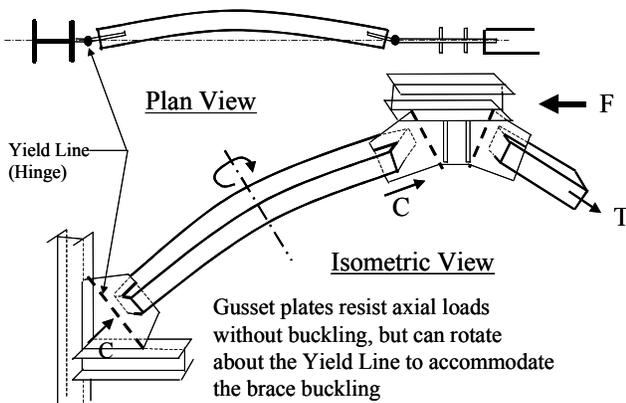


Figure 11: Brace Buckling

Another requirement, to allow for rotation about the yield line to occur, is a minimum offset from the end of the brace to the yield line, see figure 12. If this distance is too short, there is insufficient physical distance to accommodate yielding of the gusset plate

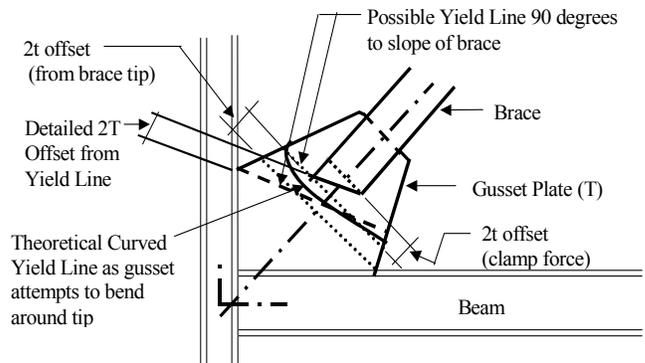
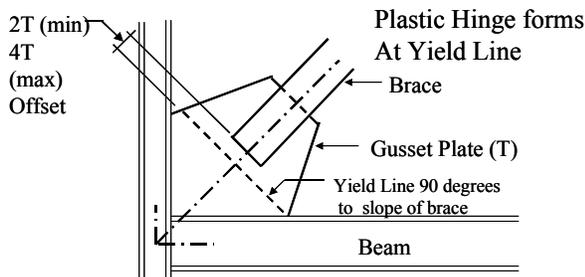


Figure 13: Gusset Plate Buckling And Yield Line (not a recommended design detail)

The illustration in Figure 13 depicts what happens when you try to shape the end of the brace to match the yield line slope. Due to the offset in the end of the brace, the yield line will attempt to bend around corner of the brace. This creates several problems, in that it is impossible to bend the plate about a longer curved line, since the curve creates more stiffness than a shorter straight line between two points which wants to be the hinge. The end tip of the brace along the upper edge is generally not stiff enough to cause a straight yield line to bend perpendicular to the brace axis about the brace tip end since there is only one side wall at this location to apply force to the gusset plate. The yield line is more likely measured from where the setback is a $2T$ distance from the two side walls which apply a clamping force on the gusset plate (the upper and lower brace edges) which creates two support points which the hinge line will attempt



$2T$ minimum required in order to develop a ductile hinge in the gusset plate (recommend design for $3T$ minimum)

Figure 12: Gusset Plate Yield Line

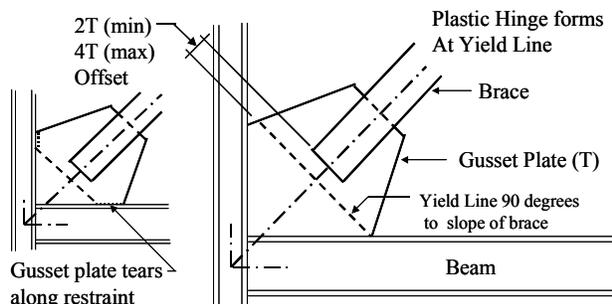
without fracture. Figure 12 depicts the minimum offset requirement of the building codes of $2T$. Due

to form away from. Depending upon the shape of the brace end, the yield line will likely occur near the end of the upper edge, but less than $2T$ from the upper edge end. The upper edge of the brace will most likely start to dig a hole in the gusset plate as the hinge line attempts to form perpendicular to the brace axis. If the upper edge brace point is significantly stiffer than the gusset plate (very thin gusset plate), it is possible that a yield line perpendicular to the brace axis could occur $2T$ from the end of the upper edge of the brace. Most likely, though, this scenario would not be expected to occur. The final point to remember is that when trying to force a yield line parallel to the shaped brace ends, the assumed gusset plate yield lines at the opposite ends of the brace are most likely not parallel to each other, developing additional restraint to out-of-plane buckling of the brace.

Gusset Plate Yield Line

The yield line is a rotational hinge which forms in the gusset plate a given distance away from the end of a brace buckling out-of-plane. When a brace buckles in-plane, there is no yield line in the gusset plate, since the bending deformation is occurring in the brace, and most likely not about the strong axis of the gusset plate which is supposed to be designed to be stronger than the flexural strength of the brace.

The yield line extends across the width of the gusset plate to both free edges of the gusset plate. Typically, though, the yield line intersects the junction of the beam or column flange with the end of the gusset plate. See Figure 14. The yield line cannot extend into the area of the gusset plate welding to the column flange or beam flange, as this will result in restraint to the gusset plate buckling out-of-plane.



Note: Recommend detail for $3T$ (check compression for $4T$)

Figure 14: Gusset Plate Out-of-Plane Buckling Yield Line

The gusset plate will have to tear along these edges to relieve the restraint to allow the yield line hinge to develop and brace to rotate about this hinge line.

A similar problem occurs with the concrete topping slabs over the steel deck. When you consider the metal deck and concrete topping slab around the gusset plate, this represents additional restraint to out-of-plane buckling and must be considered.

The gusset plate needs to be isolated from the topping slab if the yield line is to occur below the concrete topping slab surface. See Figure 15. If the yield line is below the concrete surface and the gusset plate is not isolated from the slab, there is the potential for gusset plate tearing. As shown in Figure 15, the gusset plate can be isolated from the slab by means of fire caulking or other compressible material.

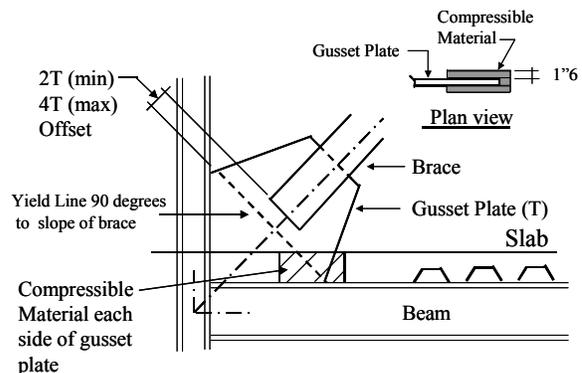


Figure 15: Yield Line Isolated From Slab

Figure 16 shows the larger gusset plate that occurs when the yield line intersection is at the slab surface. An additional stiffener plate is also shown to help support the free edge of the gusset plate. At the foundation the yield line typically needs to occur above the slab surface. See Figure 17.

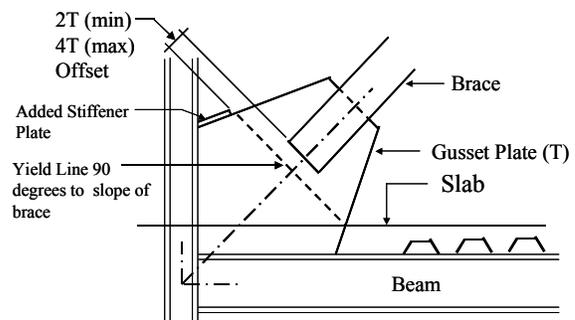


Figure 16: Yield Line Above The Slab Surface

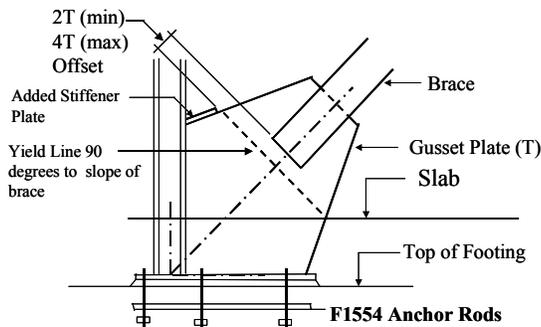


Figure 17: Gusset Plate At Foundation

The anchor rod connection design force to the foundation may not have to develop the brace yield strength vertical component depending upon the type of foundation. If the foundation is allowed to uplift or rock, as might occur in shallow foundations, then the foundation only needs to develop the weight associated with the foundation (spread footing, grade beams, slab-on-grade, soil wedge along side of footing, etc). A factor of safety should also be considered regarding the calculated foundation mass since you may not be accounting for all mass present. The author recommends a 1.3 factor of safety multiplier on the calculated mass since it is hard to predict precisely how the foundation will behave.

If the foundation is considered fixed, or not subject to uplift movement (pile foundation or large subterranean structure) then the brace tensile yield vertical component needs to be developed into the foundation. Due to overturning from floor levels above, the anchor rod design force may be more than the brace tensile yield capacity vertical component.

With potential high uplift loads, the design of the brace frame base plate may require special consideration, since bending in the base plate between the anchor rods and columns flanges/web may not be advisable. The anchor rods may need to be extended up the sides of the column and require bearing shoes welded to the sides of the column.

Gusset Plate Sizing (The Critical Angle Concept)

SCBF Gusset plate design can be very sensitive to changes in building bay sizes, unlike moment frames. Any change in bay width or height changes the slope of the brace which in turn changes the dimensioning of the gusset plate in order to maintain the 2T – 4T yield line offset from the end of the brace. See Figure 18. As a design engineer, you do not want to go back and redo your brace frame design just because the bay size increased in width by 2 inches.

If this was a moment frame, you would not worry about such a small change in bay size.

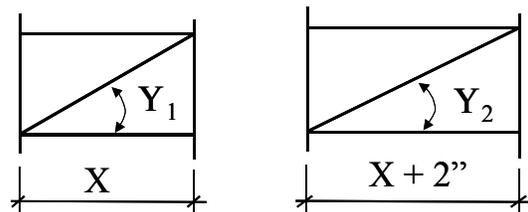
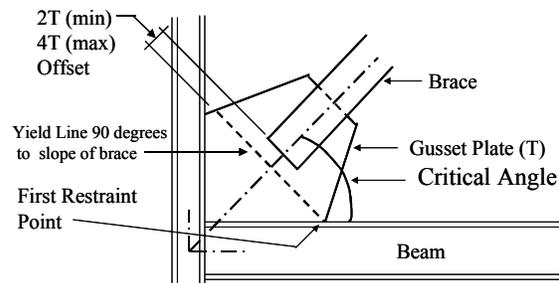


Figure 18: Changes In Bay Size

Sizing of the gusset plates does not have to be a difficult task, even with minor changes in the building brace frame bay size. The Critical Angle Concept was developed to help size the gusset plate. The information that is provided on the structural drawings is such that the steel detailer can complete the gusset plate geometry for the design engineer, even with small changes in building bay sizes. The Critical Angle Concept is nothing but geometry.

The Critical Angle Concept is based upon determining which side of the brace the yield line restraint occurs on. The yield line restraint is where the yield line intersects either the column or beam flange at one end, and the other end of the yield line intersects the free edge of the gusset plate. See Figure 19.



Critical Angle: The angle on the side of the brace which governs the gusset plate dimensions

Figure 19: Critical Angle Concept

As shown in Figure 19, the critical angle is on the side of the beam since the first restraint of the gusset plate yield line occurs at the beam flange. The opposite end of the yield line occurs at the free edge of the gusset plate.

The Critical Angle Concept is based upon the gusset plate being symmetrical about the end of the brace, and the gusset plate dimension can be easily calculated to determine the location of the yield line in the gusset plate. Certain design assumptions are required.

Recommend Design Geometry:

1. Use a 1" minimum offset from side of brace to edge of gusset plate (helps brace block shear check)
2. Assume a slope of 30 degrees at the edge of the gusset plate relative to the brace axis (helps brace block shear check)
3. Take the length of the brace welding to the gusset plate being equal to 2x the width of the brace (helps with shear lag development into gusset plate at $T = F_y R_y A$)
4. Assume a gusset plate thickness (between 0.5" to 1.5") to determine the 3T offset distance (F_y plate = 50 ksi to keep gusset plate thin).

With this information, you can now calculate the length of the yield line. You also know the physical size of your beam (beam depth) and columns (column depth). With the yield line length known, and the fact that the yield line is perpendicular to the brace axis, you can now just slide the brace gusset plate connection down along the slope of the brace and see where the end of the yield line hits first; the column flange or beam flange. Figure 20 illustrates a brace slope such that the critical angle is on the side of the beam.

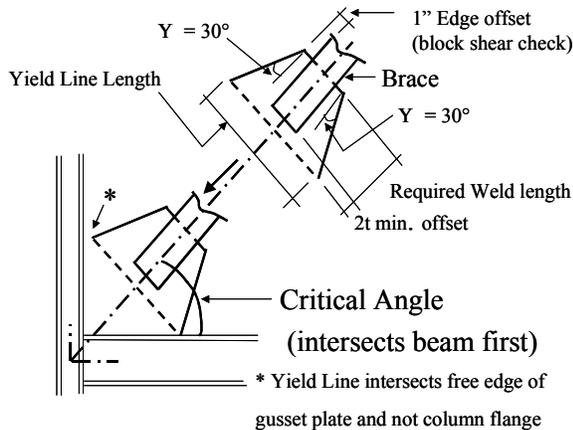


Figure 20: Critical Angle On Side Of Beam

Along the upper edge of the gusset plate, you can see that the yield line intersects the free edge of the gusset plate and not the column flange. If the column and beam were the same depth, and the brace slope was 45 degrees then each end of the yield line would hit the column flange and beam flange simultaneously. Since have this framing configuration this will rarely happen in real life, there will always be a gusset plate free edge intersection of the yield line.

Figure 21 illustrates a brace slope where the critical angle is on the side of the column. A computer program can be written to determine the actual

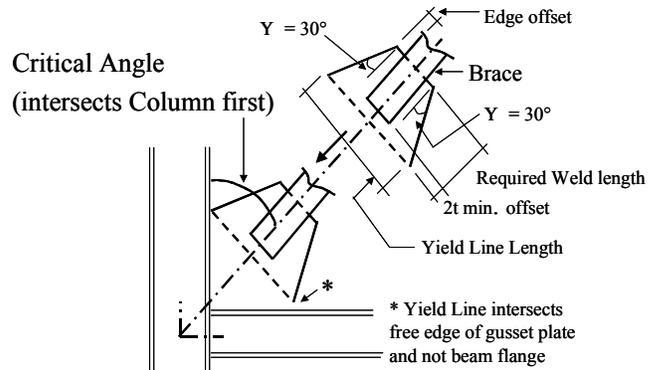


Figure 21: Critical Angle On Column Side

dimension of the gusset plate so as to maintain the 3T offset from the yield line and the length of the sides of the gusset plate.

The Uniform Force Method can be used to determine the weld sizes, but you must use Special Case #2 since you must determine the gusset plate dimensions prior to determining the weld sizes. Therefore, you will have some eccentricity in the connection design when designing the welds.

With the critical angle side determined, you can then check the compression buckling of the gusset plate and the required weld thickness to transfer the shears into the beam and column flanges.

Beam Midspan Connection

The brace midspan connection is similar to the beam /column brace connection. See Figure 22.

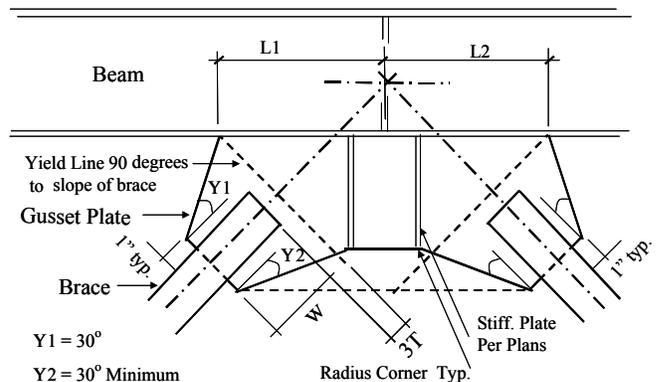


Figure 22: Brace Midspan Connection

The Critical Angle Concept can be applied to the beam/brace connection as well. The critical angle will always be on the side of the beam, see Figure 23, unless you have a zipper column. Then the critical angle may be on the side of the column.

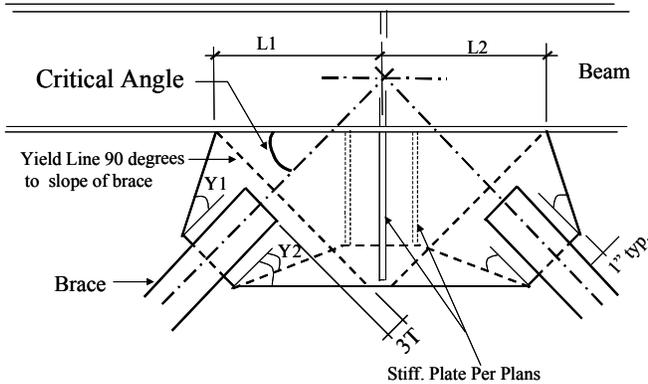
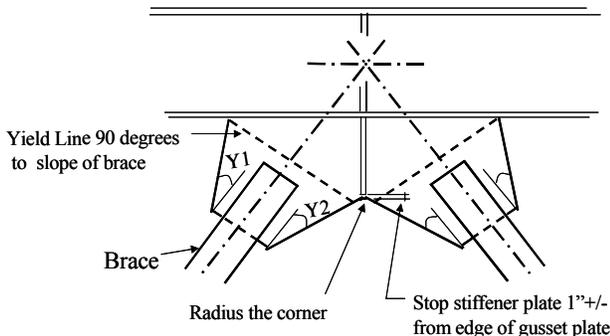


Figure 23: Beam / Brace Midspan Connection With Critical Angle

The bottom edge of the gusset plate should be tapered, to reduce the length of the free edge, instead of leaving it straight. Professor Astaneh-Asl of UC Berkeley has shown in a Steel Tips publication, authored by him, that a long unstiffened gusset plate free edge can buckle prematurely before reaching the compression load of the brace. Figure 24 shows the tapered edge of the gusset plate. It is important that the yield lines not overlap, and the gusset plate be trimmed back far enough to prevent such a yield line overlap from occurring.



- Taper gusset plate edges so yield lines do not overlap on the gusset plate
- Stop stiffener plates short so they do not intersect yield lines

Figure 24: Tapered Midspan Gusset Plate Connection

Out-of-plane stiffeners are required to brace the gusset plate to prevent out-of-plane buckling. If the gusset plate extends far enough below the bottom of the beam an out-of-plane diagonal brace attached

near the bottom of the gusset plate back to the floor/roof diaphragm should be provided. The out-of-plane brace is to stiffen the bottom edge of the gusset plate to help prevent out-of-plane buckling. The upper edge of the gusset plate is welded to the beam bottom flange and usually braced by an out-of-plane intersecting beam and full height shear plate to the brace frame beam web.

The intersection of the the two tapered edges of the gusset plate should be radiused to prevent any notches which might lead to fracture initiation due to hinge rotation of the legs of the gusset plate. The stiffener plate should also be held back to prevent welding near this hinging area.

Fabricator Notes:

As mentioned earlier, the structural drawings need to provide enough information regarding the gusset plate dimensions so the detailer can complete the design and make sure all of the dimensions close. Typically the dimension on one side of the gusset plate would be given, and the other dimension left up to the detailer to determine.

The reason again for showing these notes is so that when the architect changes the bay length or height by a few inches, you need not redo your design. Also, if you give all the dimensions for the edge of the gusset plates, and one is wrong, in trying to close the gusset plate dimensions the fabricator will simply circle up all the dimensions and ask you which one is correct. The answer may very well be none are correct. Being the design engineer you will have all of the gusset plate edge dimensions in your calculations, you just don't show them all on the structural drawings.

When shop drawings are submitted, you just compare the dimension shown with the dimensions you have in your calculations. This provides a check to help verify that the architect didn't make dimensional changes he hadn't informed you about. The important part of the shop drawing process is to make sure that all of the gusset plates are drawn to scale. If just the first gusset plate is drawn to scale and then copied for the other gusset plate details where the detailer only changes the dimensions, it will make checking the gusset plates by scale impossible. The design engineer needs to insist that the gusset plate details be drawn to scale.

Figure 25 illustrates the generic information that should be shown on the structural drawings to help

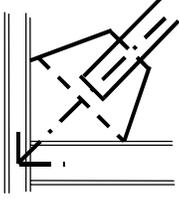
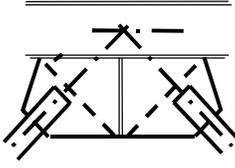
Example Detail	Provided Information	Fabricator Instructions	Structural Drawing
	<p>L = Given 3T = 3 x thickness of plate W = Required Weld length along brace A = To be determined γ = To be determined</p>	<ol style="list-style-type: none"> 1. Verify angle $\gamma < 30^\circ$. 2. Verify extension of yield line does not intersect the plate dimension "A". 3. Determine "A" dimension based upon plate information shown. 4. Notify the structural engineer if any of the above items can not be met. 5. Detailer may adjust the "A" and "L" dimensions (+/- 1/8"), as detailed, in order to facilitate the fabrication of the plate. 	
	<p>L = Given 3T = 3 x thickness of plate W = Required Weld length along brace A = To be determined γ_1 = To be determined γ_2 = To be determined</p>	<ol style="list-style-type: none"> 1. Determine "A" dimension based upon plate information shown. 2. Verify angle γ_1 and $\gamma_2 < 30^\circ$ 3. Verify yield lines do not intersect stiffener plates or each other on the gusset plate. 4. Notify the structural engineer if any of the above items can not be met. 5. Detailer may adjust the "A" and "L" dimensions (+/- 1/8"), as detailed, in order to facilitate the fabrication of the plate. 	

Figure 25: Typical Brace Frame Detail Sheet

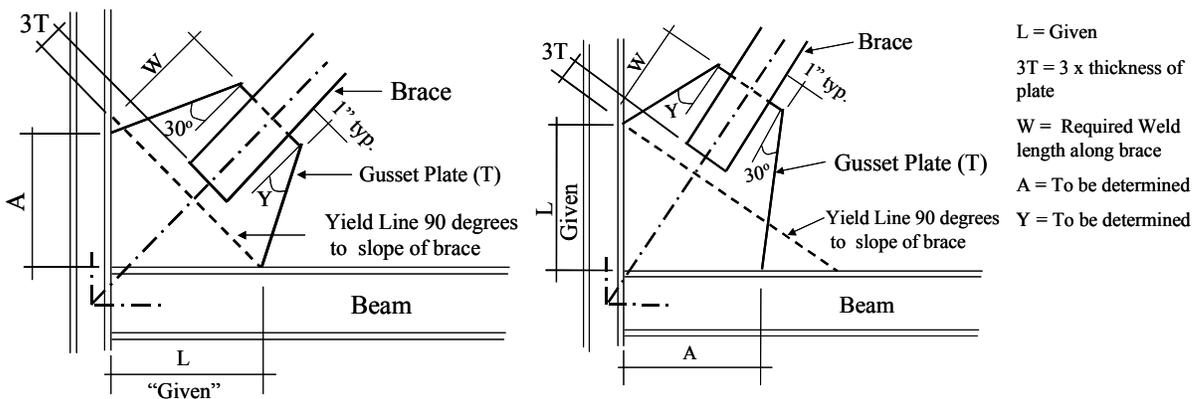


Figure 26: Brace to Column Detail

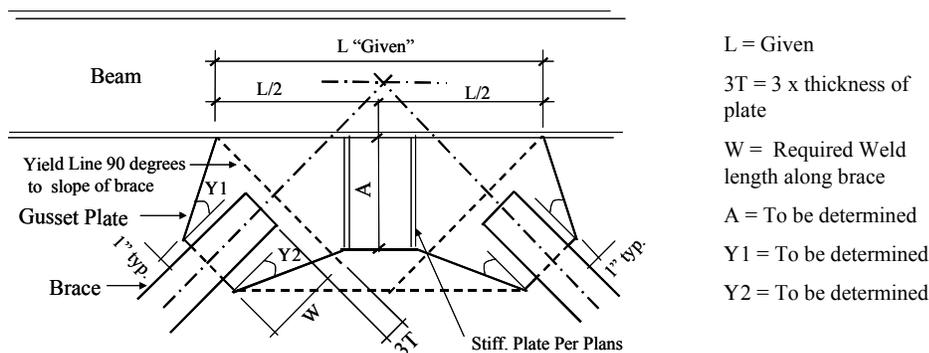


Figure 27: Brace to Beam Detail

the detailer complete the design of the gusset plate connection, especially when you consider that you will have sloping beams at the roof. Typically a table would be provided with the required starting dimensions for the individual gusset plates.

Figures 26 and 27 show the typical call outs on the generic detail and the specific provided information. The Fabricator Instructions shown in Figure 25 tell the detailer how to proceed with closing the gusset plate dimensions.

The compression design of the gusset plate to resist buckling can be checked using the Whitmore Method and checking the gusset plate as a short column.

Some Good Design Considerations

The following is a list of some good design consideration for brace frame gusset plate design.

1. Detail gusset plate for a 3T offset to allow for some field fit-up tolerance, but design for 4T offset from yield line for compression check of gusset plate.
2. Brace weld length to gusset plate not less than 1.5 times the width of the tube, pipe or angle to help address shear lag and block shear failure of gusset plate. Recommend use 2.0 times the width of the tube, pipe or angle instead of 1.5
3. Provide one inch offset from side of tube, pipes or angles braces to edge of gusset plate then start 30 degree slope of gusset plate edge to provide *Block Shear* resistance.
4. Gusset plate thickness will most likely be in the range of 5/8" to 1-1/4". Avoid thicker gusset plates for knife plate connections as they may significantly reduce the brace net cross sectional area to transfer forces (Use 50 ksi plate as required).
5. The gusset plate for the beam mid-span should be one piece. When the brace slope is rather flat, sometimes two separate gussets are used instead of a single individual plate. Separate gusset plates result in thicker gusset plates and heavier welds. A single

gusset plate also helps the fabricator with fit-up for welding to the beam.

6. Keep beam sizes the same depth and weight at each floor level to help minimize the number of variations in gusset plate sizes. This will not be possible at the roof due to the sloping beam connections to the columns.
7. The beam web thickness should not be less than 75 percent of the gusset plate thickness. The beam web may need to be reinforced to prevent web buckling and crippling due to the high axial forces in the connection.

Slotting of the end of the brace can result in a significant lost of brace net cross sectional area depending upon the thickness of the gusset plate. Since the slot in the tube or pipe brace is always longer than required in order to erect the brace over the gusset plate, the brace net section just in front of the gusset plate tip needs to be reinforced. See Figure 28. The reinforcement plate welding to the brace sides, as shown in Figure 28, must be sufficient to develop the yield capacity of the reinforcement plate each side of the brace net section.

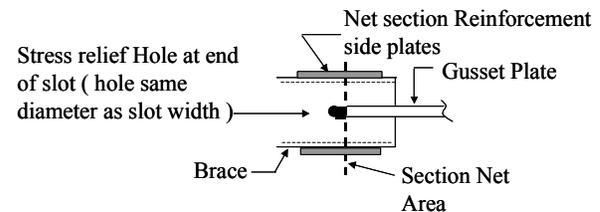


Figure 28: Reinforcement At Brace Net Section

The reinforcement plates can typically be added to the brace in the shop. Providing a drilled radius hole at the end of the slot is critical to help with stress redistribution around the edges of the brace slot when the brace is approaching tensile yielding. If the brace slot is only square cut, then you have a re-entrant corner condition in the steel plate that can lead to early fracture when highly stressed.

Field Verification

As part of the design process, the actual constructed brace / gusset plate connection should be field verified that it complies with the structural drawings. See Figure 29.

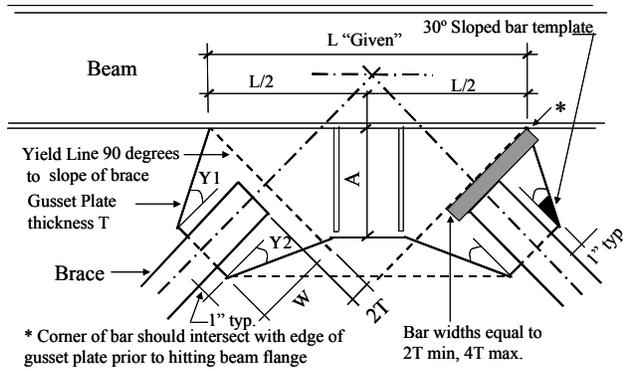


Figure 29: Gusset Plate Verification

Templates can be used to verify the slope of the gusset plate edges comply with the structural drawings. Bars can be utilized to determine that the offset from the yield line has been maintained. If the offset from the end of the brace is less than the required $2T$ offset, the end of the brace can be notched as shown in Figure 30. This notching of the brace end should allow the gusset plate yield line to still develop at the $2T$ minimum offset. The weld thickness can be increased if necessary if the length of the welds are shortened due to the notching of the end of the brace.

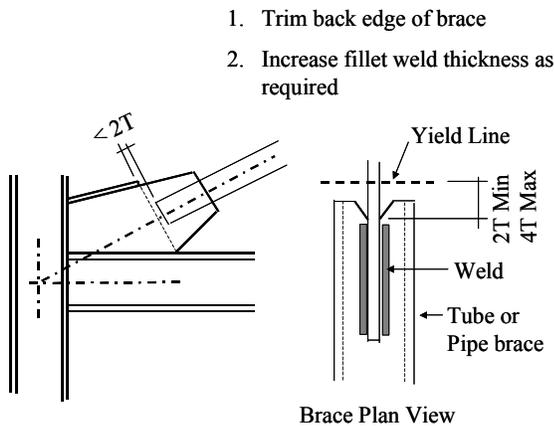


Figure 30: 2T Offset Field Fix

Information should be placed on the drawings indicating what needs to be field verified. The list should include the following:

Deputy inspector to verify:

1. That the $2T$ minimum ($4T$ max) offset is maintained each end of brace before welding of brace and after welding of brace (include diagram of field check method)

2. Verify 1" offset and angle slope along edge of gusset plate.
3. Isolation of gusset plate yield line from slab (if detailed) and stiffener plate locations
4. Deputy Inspector to discuss inspection criteria with structural engineer prior to welding of braces

Special Bracing Considerations

Other bracing configurations include the 2 story X and single story X bracing. The connection to the column is similar to the gusset plate details shown previously in this paper. As an alternative, a single large reinforced gusset plate can be used to receive the braces from above and below the beam. See Figure 31.

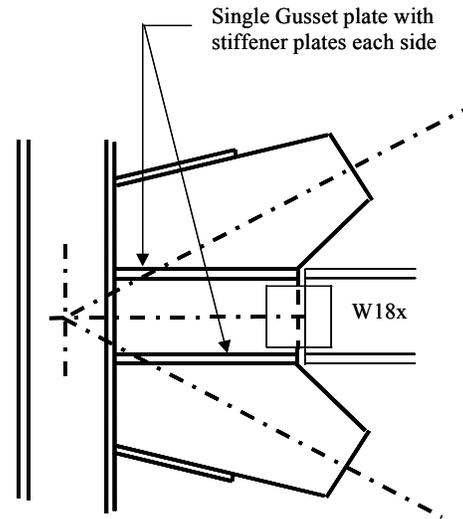


Figure 31: 2 Story X Brace Connection To Column

This detail would eliminate any concerns relative to the beam web being too thin when you weld a gusset plate above and below the beam that frames all the way into the column. Welding the beam flanges or lapping the gusset plate stiffeners over the beam flanges can also create a moment frame connection for some reserve strength if the braces buckle.

The Single story X brace also has the typical cross over connection between the braces that needs to be designed. Instead of using a single gusset knife plate through the continuous brace, the designer may want to consider using lap plates over the sides of the braces, especially if tubes are used as brace members. The advantage of the lap plates is since they are a

pair, one plate each side of the brace, they can be thinner than a single knife gusset plate and require less detailing and fit-up problems with the connection. See Figure 32.

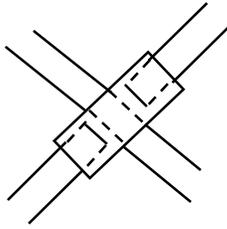


Figure 32: X Bracing Lap Plates

Future

A new bracing system is being developed called the Buckling Restrained Brace Frame. It is in the review process for the 2003 NEHRP Recommended Seismic Provisions, and will eventually be included in the AISC Seismic Provisions. As more competitors enter into the proprietary brace manufacturer business, these types of bracing may replace the SCBF as we currently know it.

Allowing for more eccentricity in SCBF gusset plate connections is likely to become more prevalent in order to reduce gusset plate sizes that typically occur using purely concentric brace connection designs.

Conclusions

The design of SMF's and SCBF's has become more complex in the recent codes. The design of steel moment frames has been well covered by FEMA 350 and the AISC 2002 Seismic Provisions For Steel Buildings. Design information has been provided for the top of column steel moment frame connection and detailing to provide fixity at the moment frame column base.

SCBF brace frame design and gusset plate design has been presented including the introduction of the "Critical Angle Concept" as a method to help in proper sizing of the gusset plates and the "Yield Line Concept" for proper detailing to promote hinging in the gusset plate resulting from the brace buckling out-of-plane. Gusset plate fabrication notes and field verification notes have been discussed and examples presented as to what might be shown on the structural drawings. Some good bracing gusset plate design recommendations have also been presented.

The design engineer must still refer to the AISC Seismic Provisions for Steel Buildings for overall design requirement for SCBF's and OCBF's

Acknowledgements

The author wishes to recognize the time and effort put in by the AISC Professional Members Committee - Los Angeles Chapter that spent many months in helping create the original brace frame presentations for SEAOC in 1999 and 2000 from which most of this paper has been based and written.