MINIMIZING THE STRENGTH OF BRACING CONNECTIONS

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Abstract: Steel bracing members and connections of Seismic Force Resisting Systems designed to meet North American codes are based on capacity design principals requiring that the connection be designed to allow the tensile brace member to achieve yield. Brace member selection is based on the compressive resistance of the element making the probable tensile resistance of the brace member significantly larger, necessitating excessively strong joints and increasing the cost of the structure. Capacity design principals also require that the columns supporting the braced members be designed to resist the probable tensile resistance of the brace, dictating increases in column size, especially for multiple storey structures, and further increasing the cost of the SFRS. This paper will describe a typical WF brace connection from a recent project and propose a method of limiting the tensile resistance for braced members in bolted connections while not reducing their compressive resistance.

INTRODUCTION:

Steel structures must be designed to resist lateral forces due to wind, seismic and crane lateral forces. The lateral force resisting systems employed to resist these forces include rigid frames, steel plate shear walls and by far the most prevalent, vertical bracing systems. Conventional bracing systems are simple to design and provide effective and economical lateral force resisting systems. Bracing systems can be constructed in many different configurations, often established by specific clearance constraints or to behave in predetermined fashion. Bracing configurations include tension only and tension compression cross braces, single tension compression diagonal braces and chevron or inverted chevron tension compression braces. These systems may be designed and detailed as concentrically or eccentrically braced frames. Sizing of the brace member is normally a simple task as the section is designed only to resist an axial tension or a compressive force. Braced members may be oversized in order to limit its axial deformation in order to control interstory drift.

The selection of the actual brace member is driven by economy and for smaller forces a single angle member is usually selected or possibly even a bar shape, either a rod or flat for a tension only system. As the design force increases, the member of choice becomes interconnected double angles for their ease of connection, fabrication and installation. As lateral forces continue to get larger, it becomes more effective to use hollow structural sections due to their high ratio of compressive resistance to mass.

Once the range and capacity of hollow sections is exceeded, wide flange sections are utilized. Forces beyond the capacity of wide flange shapes require custom built up sections. High rise buildings and heavy industrial plants typically fall into the category of structures where wide flange members are the common member choice to use as bracing members.

The Canadian design standard CAN/CSA-S16-05, "Limit States Design of Steel Structures", provides the requirements for the design of members and their connections in the seismic-force-resisting system (SFRS) for steel buildings. The code provisions are based on capacity design principals whereby specific elements or mechanisms are designed to dissipate energy and all other elements must be sufficiently strong for this energy dissipation to be achieved. In vertical bracing systems, the braces are the energy dissipating elements and the beams, columns and connections in the SFRS must be proportioned such that energy dissipation can occur. Steel used in the energy dissipating elements is limited to a maximum specified yield strength of 350 MPa (50 ksi). The United States design specification, ANSI/AISC 360-05 "Specification For Structural Steel Buildings", has very similar provisions for the design of members and their connections in the SFRS for steel buildings. The paper will concentrate on the technical problems associated with connection design for wide flange bracing in seismic areas in meeting the SFRS requirements and present a method to reduce the connection demand by introducing a fuse limiting the tension resistance of the brace, somewhat similar to the reduced beam section connection developed and tested for moment frames.

CONNECTION TYPES:

Connection design can truly be considered an art. Fabricators consider good connection designers to be worth their weight in gold. The cost related to shop fabrication is totally dependant on the man hours required to produce the connections linking individual members. No shop fabrication man hours are spent on the shaft portion of the member between connection points.

A good connection must of course resist the design forces but must also be economical and erectable in the field. The total cost of a connection includes the shop cost for the shop man hours required to manufacture the connection, the cost for the man hours to assemble and complete the connection in the field plus the cost of the connection materials. In many locations, the cost of a man hour, whether in the shop or in the field, is relatively the same and the least total number of man hours necessary to complete the connection, produces the best result. This however, is not the case in many of the large cities in the Northern cities of North America such as New York, Boston, Philadelphia, Chicago, etc. where field workers are unionized and the cost of a field man hour may easily be double than that of a shop man hour. The availability of skilled field workmen such as welders also influences the choice for the optimum connection. The time required to complete a connection in the field is also an important factor. Field welded connections increase the overall field time duration when compared to the time required to complete the equivalent field bolted connection. In the large cities cited above, field costs and labor shortages demand that bracing connections be bolted rather than welded.

One method of evaluating connection performance is to determine the "level" of the connection. This is a method that is used internally at Canam Group and Canam Steel Corporation to assess relative connection cost. Lower "level" ratings indicate more efficient connections. The connection "level" is established by counting the number of times the design force needs to be transferred. One can presume that the cost to make one full connection of the design force is relatively the same whether it is bolted or welded or made in the plant or in the field. Thus a "level" one connection only requires the member force to be transferred one single time. An example of a "level" one connection is a field welded moment connection of a beam to a column. The connection force is only transferred once, directly by welding of the beam flanges to the column flange. If the column flange requires stiffeners in order to locally reinforce the flange, then this becomes a "level" two connection as the design force must transferred through to the stiffeners. An example of a simple "level" two connection is a field bolted double angle tension brace connected to a gusset plate that is welded to a column base plate. This connection requires the force to be transferred twice, first by field bolting of the angles to the gusset plate and then by welding of the gusset plate to the column base. The "level" method is a fairly good predictor for assessing the relative costs when comparing connections.

Wide flange brace sections pose a problem in attaining simple efficient connections. Typically, column sections are used for the brace members. For all hot rolled North American wide flange column sections in the W10 (W250), W12 (W310) and W14 (W360) series of shapes, the area of each flange is relatively constant in proportion to the total area of the section, between 38% and 41%. This means the area of the web is between 18% and 24% of the total area of the section. Thus significant forces need to be connected occurring from both orthogonal planes. There are two possible orientations for a wide flange brace section, either with the web in a vertical plane or with the flanges in the vertical plane. The design of the connection of course depends on the orientation of the supporting column and strut girder. The floor beam/strut always has its web orientated in the vertical plane while the column may be turned along either axis. We shall now look at the two different brace orientations and the possible connections geometries.

Bolted WF Brace Connection With Web Vertical:

When the brace web is aligned with the floor beam/strut web, the logical choice is to have a vertical gusset plate aligned with both the strut and brace member webs. The horizontal portion of the gusset plate can be welded directly to the floor beam/strut flange while the vertical segment of the gusset plate can be welded to the beam end plate that is in turn bolted to the column flange or web. Figure 1 shows a perspective of a braced bay and Figure 2 shows an enlarged detail of this connection arrangement.

The bolts have been omitted for the sake of clarity. The brace web force is connected using two splice plates placed on each side of the brace web enabling the bolts to act in double shear. The flanges are connected to splice plates slotted to fit through the vertical gusset plate and attached to it by welding on both sides. The brace flanges can either be bolted or welded to the splice plates. This a "level" 3.5 connection for the brace web force, "level" 3 for the horizontal connection component and "level" 4 for the vertical component. This is also a "level" 3.5 connection for the flange force, "level" 3 for the horizontal connection component and "level" 4 for the vertical component. The bolted web splice plates allow the brace to be erected from one side between the flange splice plates. This arrangement allows the option of connecting the flange force by either field welding or field bolting.

Bolted WF Brace Connection With Web Horizontal:

When the brace web is horizontal, no simple connection arrangements become self evident. Connection geometry has to be based on the main member sizes selected and the corresponding widths of the elements. Most arrangements produce connection "levels" of 4 or more.

In order to make this configuration economical, planning of the connection geometry must be performed by the designer prior to member selection. When the vertical braced bay is only along one grid line axis, column orientation should be such that the column flanges line up with the brace flanges. Should vertical bracing be required along both perpendicular grid axes, box columns should be used and the depth of the box chosen so that the bracing gusset plates can be directly welded to the individual column flanges coordinating the depth so that the bracing member will fit with erection clearance inside the space between the pair of gusset plates. Figure 3 shows the basic concept of this connection.

Careful consideration should also be given to the design of the strut member. Two different choices can be made to facilitate connection design. The first is to have a boxed floor beam/strut that would be made the same width as the brace member, as shown in Figure 3. Both webs would be connected to the gusset plate that would serve to connect the vertical beam reaction and to transfer the horizontal component of the brace reaction. A hand hole is required at the top flange of the beam/strut to accommodate field bolting. A second option is to use a hot rolled wide flange section built up with two Tee sections on either side, as shown in Figure 4. The web of the beam is connected for vertical shear while the Tee flanges are designed to take horizontal brace force component. The Tee extends sufficiently to develop this force taking account shear lag effects. For clarity, bracing is only shown on only one axis in Figures 3 and 4, and the column illustrated is a box section. A wide flange section could be used with the web oriented perpendicular to the braced grid line for bracing in one axis only.

The first option described utilizing the box section is the less costly of the two even though it requires the fabrication of a built up section. Both joint configurations require additional fabrication cost being invested into the fabrication of the main members but the overall simplicity and effectiveness of the connection more than makes up for this cost with savings in the weight of connection elements and shop hours required for brace fabrication. Additionally, both these connections can be erected quickly and minimize overall field time. The connection using a boxed floor beam/strut is a "level 2" connection while the use of a wide flange shape with Tee's is a "level" 2.5 connection.

EXAMPLE CONNECTION - NEW YORK STADIUM PROJECT:

Canam is presently constructing four outdoor stadium projects in the greater New York City area. All four stadiums are designed as braced structures and some are mandated to be designed to meet the seismic provisions of the AISC Specification. Figure 1 is a perspective elevation of a typical bent created from the 3 Dimensional detailing model. Figure 2 is a close up of the connection at the column-beam joint. The diagonal brace is a W14x233 (W360x347), the beam/strut is a W30x108 (WF760x161), the column at the upper connection level is a W14x257 (W360x382) while the column receiving the brace at its base plate is a W30x326 (W760x484). Both 3D figures are drawn to scale. The gusset plates are detailed to provide for out of plane buckling for the brace with a fold line equal to 2t. The center to center distance between columns is 30.0 ft. (9.144 m) and the distance from under side of base plate to the centerline of the strut/floor beam is 26.33 ft. (8.026 m), resulting in a work point (WP) to work point brace length of 39.62 ft. (12.167 m).

The W14x233 brace has been designed with a k=1 and based on its WP to WP unsupported length has an axial resistance of 1,130 kips (5,030 kN). The tension resistance of this brace is 3,080 kips (13,700 kN) and the expected yield strength of the brace in tension, defined as $R_yF_yA_g$ is 3,780 kips (16,800 kN). The actual buckling length from gusset plate fold line to gusset plate fold line is actually 28.70 ft. (8.75 m). This actual unsupported length makes the true design axial resistance equal to 1,840 kips (8180 kN), indicating that the section has been significantly overdesigned. Having such large design over strengths is often the case in the design of compression braces.

Several engineers of large consulting firms were surveyed to discuss their current practice for the design of compression braces and all agreed that they design tension compression bracing using the WP to WP as the unsupported length of the brace and furthermore take an effective length factor k equal to 1.0.

The vertical component of the expected yield strength of the brace in this example is 64.9% of the brace force. Thus, according to capacity design principles, the column has to be designed to support an additional vertical load equal to 64.9% x (3,780-1,130) kips or 1,720 kips (7,640 kN). This extra force requires an additional 38.2 in^2 (24,600 mm^2) of area or 130 plf (193 kg/m) of steel for the column. In fact, this additional area may be somewhat smaller as it is possible that the column may have been originally sized for a different load combination case.

In order to reduce the significant additional costs required by capacity design of the bracing system, it is proposed to introduce a fuse that would limit the tension resistance of the brace while not affecting the compressive resistance of the brace, thereby greatly reducing the over strength required in both the connection and column.

In the New York City area, field labor costs dictate that the connection is field bolted. The bolt holes reduce the net section through the connection and the wide flange shape must be locally reinforced to preclude this failure mode and allow gross section yielding to occur at the brace midspan. The cover plates that replace the removed area must of course be developed beyond the extent of the bolted connection again adding further cost. These cover plates can be see in Figure 4.

PROPOSED WIDE FLANGE BRACE FUSE

The purpose of a fuse in tension compression braces is to predetermine the location and the load level at which tension yielding of the brace will occur. The fuse must of course allow the brace to yield in compression and therefore should be designed to remain elastic during the compressive load cycle. This requires that the fuse not buckle locally in compression while the brace goes through its inelastic cyclic deformation. The fuse proposed is shown in Figure 5 (a) & (b). The figure only shows a fuse located at the top flange but a similar cutout would be located at both the top and bottom flanges, symmetrically about the neutral axis of the beam. The fuse is created by removing a portion of the brace area. One method of achieving the desired result is to drill six holes at the corner points of the slotted area, four in the flanges and two in the web. The web is then cut horizontally between the lower horizontal tangents of the holes and vertically from the outer vertical tangents of the holes into the start of the k distance of the flange. Horizontal cuts are then performed from the top of the flange again joining the tangents of the holes. Finally, two transverse cuts are made from the tangents of the holes directly above the cuts previously made into the web. The result is a neat hole cut out of the wide flange section. Grinding may be necessary at the junction of the flange and web if the vertical web cut and the transverse flange cut are not perfectly aligned. The size of the fuse may be adjusted to achieve the desired load. Shop tests have been conducted for making this cut and the described procedure gives satisfactory results.

In tension compression bracing, inelastic buckling occurs at interstory drifts between 0.3 and 0.5 percent. In order to achieve sufficient ductility in the fuse, the brace should be designed to resist an interstory drift of 2%. The strain at initial strain hardening occurs at a strain rate of 0.015 in./in. for high-strength low-alloy steels such as the standard steel grade ASTM A992 utilized in building construction today. Limiting the strain in the fuse to this level would require a much too long fuse length. It is therefore proposed to increase the strain in the fuse section to 0.10 in./in.

In order to assess the expected yield stress of the fuse area, a factor $R_y = 1.1$ has to be applied to the specified yield stress to obtain the expected yield stress. At a strain rate of 10%, an additional R_{SH} factor needs to be applied on top of the R_y factor to take into account the effect of strain hardening. For A992 grade 50 steel, the minimum specified yield stress F_y is 50 ksi, the minimum tensile strength F_u is 65 ksi and the maximum ratio of F_y/F_u is 0.85. A conservative value for a typical ratio of F_y/F_u is 0.75. Thus the ultimate strength can be expected to be around 73.3 ksi. At 10% strain, the stress is very nearly half way between the yield strength and the ultimate strength indicating the stress would be in the range of 64.5 ksi. This would indicate that the expected stress due to hardening at a 10% strain should be approximately (73.3 ksi – 55 ksi)/2 + 55 ksi = 64.2 ksi. This result gives a value of 1.17 for R_{SH} and conservatively this value may be taken as 1.18, giving a combined effect of $R_y \cdot R_{SH} = 1.3$.

We will now look at the requirements for a fuse in the example of the New York stadium brace. The bay spacing is 30 ft. and the vertical height from base plate to neutral axis of the floor beam/strut is 26.33 ft. resulting in a brace length of 39.93 ft. At a 2% drift, the brace has an elongation of 0.453 ft. Thus, the design for a fuse length at a strain rate of 10% results in a minimum fuse length of 4.53 feet. This fuse length is quite long and to avoid the possibility of local buckling of a fuse component, it is proposed to use two fuses, one at each end of the brace, each having a length of 28 inches for a total overall length of 4.67 ft.

The proposed fuse will have a width of 5 inches (127.0 mm) across the top flange and a depth of 3.75 inches (95.25 mm) measured from the top of the flange to the horizontal cut along the web. This fuse removes 30.9% of the gross area of the brace, leaving a net area of 46.7 in^2 (30,100 mm^2). The expected tensile yield strength of the brace is given by the expression $R_y \cdot R_{SH} \cdot F_y \cdot A_{fuse}$ or 1.3 x 50 ksi x 46.7 in^2 = 3,040 kips, well above the design load of 1,130 kips and the estimated compressive resistance of the brace of R_y x 1,840 kips /0.9 = 2,250 kips.

The advantage of this fuse is that the section properties through the fuse in the axis of buckling are greater than the properties of the original section. For instance, r_{yy} increases from 4.09" to 4.80" while r_{xx} remains about the same from 6.61" to 6.52". The 4 remaining flange sections through the fuse are steel blocks having a width of 5.01" with a thickness of 1.72". The radii of gyration for these blocks are r_{xx} = 0.50" and r_{yy} = 1.46". The restraint conditions for the fuse blocks are fixed for local buckling about both principal axes, so that a k value of 0.5 can be assumed. The fuse length is 28" resulting in kL/r values of 28 and 9.6 for the fuse block. As the fuse is located within proximity of the end connection, both overall buckling and local buckling of the fuse are eliminated. This needs to be confirmed by a testing program. For smaller wide flange sections having thinner flanges, local buckling can be prevented by welding a small flat to the underside of the flange block creating a stronger Tee section that will prevent local buckling. The cost associated with the placement of the flats would still result in significant overall economies. This is illustrated in the subsequent example.

The introduction of the fuse results in designing the connection for 3,040 kips versus 3,780 kips, a reduce connection force of 19.6%. This is sufficient to allow the net section through the bolt holes to remain unreinforced.

Economy for a Tension Brace with Fuse:

The cost saving that can be achieved with an adequately designed tension brace fuse can be enormous. The actual shop fabrication drawings for the connections shown in Figures 1 and 2 are presented as Figures 6 through 9. Figure 6 shows the shop detail drawing for the floor beam/strut. Figure 7 is the shop drawing for the column that receives the brace at its base plate. Figure 8 is the shop detail drawing for the wide flange brace while Figure 9 is an enlarged detail of the brace connection.

The connection was redesigned for the reduced forces created by the introduction of the fuse. As a comparison, the connection components for the original design are listed and immediately followed in parenthesis by the components resulting from a fuse design.

1: 2 flange cover plates PL18" x 1" x 67.5" in length to restore the net section through the bolt holes, each attached by 70" of ½" fillet welds and 104.5" of 5/16" fillet welds. (Cover plates are completely eliminated)

2: 2 flange splice plates PL21" x 2 3/8" x 75.1" attached to the gusset plate with 8 - 11/16" fillet welds each 31.75" in length and requiring 44 - 1 1/8" diameter A490 bolts Slip Critical Class B bolts per plate. (Splice plates PL21" x 1 ¾" x 54.5" with 8 - 5/8" welds x 22" in length and 32 same type bolts per plate)

3: 2 web plates PL9" x 1" x 31.25" field bolted with 16 - 1 1/8" diameter A490 bolts Slip Critical Class B bolts. (No change)

4: 1 gusset plate PL 69" x 1 ¾" x 77" connected with 15/16" fillet weld for a total length of 260". (Gusset PL52" x 1 ½" x 60" with 7/8" fillets for length of 210")

5: 1 base plate extension PL54" x 1" x 57" welded to the base plate with a full penetration weld. (No change)

Total savings per brace end are as follows:

1: The number of bolts goes from 104 to 80 for a saving of 24 - 1 1/8" diameter A490 bolts Slip Critical Class B bolts. The estimated cost of an installed bolt in New York City is in excess of $20 per bolt that gives a saving in excess of $480.

2: The connection material weight goes from 6,479 pounds to 3,495 pounds resulting in a savings of 2,984 pounds. The estimated cost of plate material is approximately $1,200 per ton resulting in a saving of $1,790.

3: The weld volume is reduced by about 77 in^3 resulting in a reduction of 7 man hours of weld time or a cost of $420.

The total estimated savings per brace end are in the vicinity of $2,700 or a total of about $5,400 per brace. This particular project has more than 200 braces but not all of this size. A preliminary estimate would indicate that the fuse concept could save in the order of $500,000 for this project. This estimate excludes any weight savings in the columns due to lesser capacity design demand.

CONCLUSIONS:

Vertical bracing is by far the most economical method of providing a Seismic Force Resisting System to a building. Recent code requirements for capacity design in SFRS have greatly increased the cost for braced structures by significantly increasing the demand on the connection and the related cost of the connection in material weight, fabrication man hour content and the number of field hours necessary to complete the connection. Additionally, column and strut members at connection joints need to be increased in size to resist the greater connection force and to meet capacity design obligations.

The introduction of a fuse in the brace member can significantly reduce the connection demand while at the same time moderate the capacity demand on the column and struts. The proposed fuse meets these objectives while adding little additional cost to the manufacturing of the brace. Testing of the fuse will of course be necessary but the preliminary analysis conducted to date indicates that the fuse will allow the brace to yield cyclically in compression without affecting the overall brace resistance.

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