Designing for Ductile Performance of Bolted Seismic Connections to Axially Loaded Members

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Field bolted connections are often the safest and most economical option for structural steel erection. It is therefore desirable to maintain the use and economy of field-bolted connections as an option for seismically loaded structures, even within the seismic load resisting system. Some of the requirements of the AISC 2005 *Seismic Provisions for Structural Steel Buildings* (AISC, 2005a), hereafter referred to as the AISC Seismic Provisions, create additional challenges for the design of bolted connections. However, measures can be taken in the connection design and detailing to enhance ductility and improve the performance of the structure while maintaining economy.

This paper addresses two requirements of the Seismic Provisions: Section 7.2, Bolted Joints, and the requirement that vertical brace connections be designed for the expected tensile strength of the brace as stipulated in Sections 14.2 (ordinary concentrically braced frames or OCBF) and 13.3a(a) (special concentrically braced frames or SCBF). This paper does not address bolted moment connections. Guidelines for field bolted seismic moment connections are presented in FEMA 350 (FEMA, 2000).

REQUIREMENTS OF THE SEISMIC PROVISIONS

Since structures subjected to a seismic event could experience loads in excess of the design loads, inelastic response of the structure is expected. Therefore, structures must exhibit ductile behavior. Section 7.2 of the AISC Seismic Provisions gives the general requirements for bolted connections to be used in the seismic load resisting system. Bolts are required to be pretensioned and installed in standard holes or in short slots perpendicular to the force, and faying surfaces are to be prepared according to the requirements for a slip-critical Class A faying surface, even though the connection can be designed as a bearing joint. Oversized holes are also allowed in one ply at bracing connections. The nominal bearing strength of the material is limited to $2.4dtF_u$, instead of the more liberal $3.0dtF_u$, to limit deformation. Additionally, bolts are not permitted to share load with welds, due to load-deformation incompatibilities that can occur.

All of these requirements can be addressed simply during connection design; however, Section 7.1 states, "The design of connections for a member that is a part of the seismic load resisting system (SLRS) shall be configured such that a ductile limit state in either the connection or the member controls the design." This provision is intended to ensure ductile behavior of the system. Similarly, Sections 14.4 (OCBF) and 13.3a(a) (SCBF) address ductility for vertical braces. Sections 14.4 and 13.3a(a) define the expected yield strength of the brace as $R_yF_yA_g$ (for LRFD), where R_y accounts for the fact that the yield strength of the brace may exceed the specified minimum. Unless analysis indicates that the system cannot deliver a force equal to the expected yield strength of the brace, the brace connection must be designed for the expected yield strength to ensure ductility. It should be noted that while the requirements of Section 7.1 apply to all members that are part of the seismic load resisting system, including struts, collectors, diaphragms, trusses, and horizontal bracing, the requirements of Sections 14.4 and 13.3a(a) only apply to the vertical braces in a braced frame.

These ductility requirements are sometimes misinterpreted to mean that all seismically loaded members connected with bolted connections must be reinforced to ensure that net section fracture does not control the design. However, there are several other options available to the engineer that will allow reinforcing to either be eliminated or mitigated. This paper will examine some of these options.

DUCTILE AND NONDUCTILE LIMIT STATES

Examples of nonductile limit states are given in the AISC Seismic Provisions Commentary as tension or shear fracture, bolt shear and block shear rupture. Most engineers would also include weld fracture as a nonductile limit state. These

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limit states are undesirable in seismic design because they can lead to sudden separation of the connection, with little associated deformation to absorb energy from the seismic event.

Examples of ductile limit states are given as tension yielding and bearing deformation. Shear yielding can also be viewed as a ductile limit state. Although buckling might be interpreted as a nonductile limit state, given the possibility of a sudden failure, the AISC Seismic Provisions treat it as a ductile limit state, since the formation of plastic hinges in the buckled member can absorb seismic energy. Buckling can therefore be considered a ductile limit state as long as the connection can accommodate the inelastic rotations associated with the post-buckling deformations of the member. For gussets that connect vertical bracing, a $2t$ bend line is provided to accommodate these large rotations. Such practices are probably advisable for connections to other members in the seismic load resisting system where buckling is a consideration.

DESIGN OF SEISMIC CONNECTIONS NOT PART OF A MOMENT OR BRACED FRAME

As stated previously, members that are part of the seismic load resisting system, but not part of a braced or moment frame, must be proportioned so that a ductile limit state governs the design, but they are not required to be designed for the expected strength of the member. Often connections can be configured such that a ductile limit state in the connection rather than in the member controls the strength of the system. In this way the connection does not have to be designed to resist the full tensile strength of the member, as is sometimes believed necessary. As an example we will look at the simple case of a W14×68 strut designed for (± 300 kips) using load and resistance factor design. The connection shown in Figure 1 will be governed by a ductile limit state under both compression (301 kips buckling strength) and tension (337 kips plate yielding strength). Referenced sections in the example pertain to the 2005 AISC *Specification for Structural Steel Buildings* (AISC, 2005b).

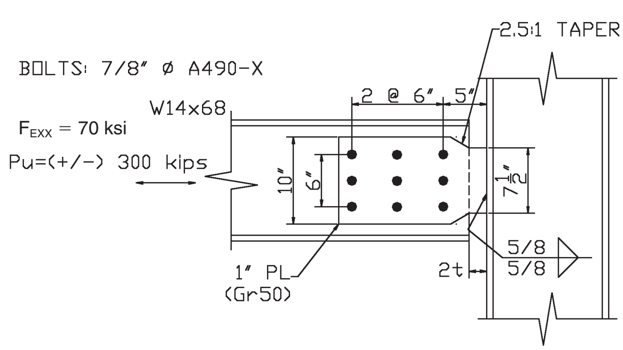


Fig. 1. Connection for W14×68 strut.

The limit states for this connection are:

Member Net Section Fracture Capacity (Section D2)

$$\phi P_{mf} = 0.75F_u U A_n = 0.75(65)(0.845)\left[20.0 - 3(1)(0.415)\right]$$
$$= 773 \text{ kips}$$

where

$$\bar{x} = \frac{\left(\frac{t_w}{2}\right)(d)\left(\frac{t_w}{4}\right) + 2\left(\frac{b_f - t_w}{2}\right)(t_f)\left(\frac{t_w}{2} + \frac{b_f - t_w}{4}\right)}{\left(\frac{t_w}{2}\right)(d) + 2\left(\frac{b_f - t_w}{2}\right)(t_f)}$$

$$= \frac{\left(\frac{0.415}{2}\right)(14)\left(\frac{0.415}{4}\right)}{\left(\frac{0.415}{2}\right)(14) + 2\left(\frac{10 - 0.415}{2}\right)(0.720)}$$

$$+ \frac{2\left(\frac{10 - 0.415}{2}\right)(0.720)\left(\frac{0.415}{2} + \frac{10 - 0.415}{4}\right)}{\left(\frac{0.415}{2}\right)(14) + 2\left(\frac{10 - 0.415}{2}\right)(0.720)}$$

$$= 1.86 \text{ in.}$$

$$U = 1 - \frac{\bar{x}}{l} = 1 - \frac{1.86}{12} = 0.845$$

Member Block Shear (Section J4.3)

$$A_{gv} = 2(15)(0.415) = 12.5$$

$$A_{nv} = 2[15 - 2.5(1)](0.415) = 10.4$$

$$A_{nt} = [6 - 2(1)](0.415) = 1.66$$

$$\phi P_{bsm} = 0.75\left[0.6F_u A_{nv} + U_{bs} F_u A_{nt}\right]$$
$$\le 0.75\left[0.6F_y A_{gv} + U_{bs} F_u A_{nt}\right]$$
$$= 0.75\left[0.6(65)(10.4) + (1.00)(65)(1.66)\right]$$
$$\le 0.75\left[0.6(50)(12.5) + (1.00)(65)(1.66)\right]$$
$$= 385 \le 362 \text{ kips}$$

Therefore, $\phi P_{bsm} = 362$ kips

Plate Yielding (Section D2)

$$\phi P_{py} = 0.9F_y d_p t_p = 0.9(50)(7.5)(1.0) = 337 \text{ kips}$$

Plate Net Section Fracture (Section D2)

$$\phi P_{pf} = 0.75F_u t_p\left(d_p - 3d_h\right) = 0.75(65)(1.0)[10 - 3(1.0)]$$
$$= 341 \text{ kips}$$

Plate Buckling (Section E3)

$$\frac{kl}{r} = \frac{1.2(5)\sqrt{12}}{1} = 20.8$$

$$\Rightarrow \phi F_{cr} = 43.6 \text{ ksi} \Rightarrow \phi P_{cr} = \phi F_{cr} A_g = 327 \text{ kips}$$

Weld Strength [AISC *Steel Construction Manual,* pg. 8-8 (AISC, 2005c)] (note that a 1.5 factor is applied to account for transverse loading of the weld)

$$\phi P_w = 2(1.5)(10)(1.392)(7.5) = 313 \text{ kips}$$

Bolt Strength (AISC *Steel Construction Manual,* Table 7-1, single shear)

$$\phi P_b = 9(33.8) = 304 \text{ kips}$$

In this example, the edge distance and horizontal bolt spacing used were such that the beam web block shear capacity did not control, and instead the ductile failure modes of the plate governed the capacity of the system. Other detailing practices were also incorporated in order to enhance ductile performance. The length of reduced plate width is set equal to twice the plate thickness. This provides for ductility in two ways. First, it provides a bend line to allow for post-buckling inelastic deformations of the main member. Second, it provides a tensile yield zone away from the weld. It has been shown that triaxial stresses develop local to the weld, which can lead to a brittle failure unless it is protected by an unrestrained yield zone (Blodgett, 1992, 1995). A length equal to the dimension across which plastic shear is to occur is sufficient to provide ductility in the connection. The 2t dimension is sufficient to allow ductile yielding to occur. Another advisable practice is to provide a gradual transition between the differing plate widths to minimize stress concentrations (Driscoll and Beedle, 1982). In the case shown, a 2.5:1 bevel is provided at the transition. This transition is consistent with the work of Driscoll and AWS D1.1. Another measure that has been shown to lessen stress concentrations and therefore increase ductility is to form a radius at the transition. In general, more gradual and smoother transitions will provide increased ductility. Until further guidance is provided by the governing specifications, the detailing requirements are a matter of engineering judgment. Similar measures can be applied to the design of horizontal bracing and trusses to cause the ductile failure to occur in the gusset plate.

DESIGN OF CONNECTIONS TO ASTM A992 BRACES WITHIN A SEISMICALLY LOADED BRACED FRAME

Connections to braces within a seismically loaded braced frame must be designed for the expected strength of the brace, $R_y F_y A_g$, and must also satisfy the ductility requirements of AISC Seismic Provisions Section 7.1. It may at first appear impossible to put holes in a member and still satisfy the $R_y F_y A_g$ requirements, since the expected strength of the member exceeds the nominal gross design strength of the brace. However, the AISC Seismic Provisions allow a similar increase in the tensile strength of the material, R_t. For ASTM A992 material, R_t = 1.1. From this, the ductility requirement becomes

$$0.75 R_t F_u (x) A_g \geq 0.9 R_y F_y A_g$$

where

x = ratio of the net area to the gross area

For A992 steel, where $R_y = R_t$, x is found to be [(0.90)(50)]/[(0.75)(65)] = 0.923. Therefore, in cases where the net-to-gross area ratio is greater than 0.923, no reinforcing of the main member will be required.

Using a reliability analysis, similar to that used as the basis of Load and Resistance Factor Design (LRFD), a slightly smaller value of x can be justified. The Commentary to the AISC *Specification for Structural Steel Buildings* (AISC, 2005b) indicates that the target reliability index, β, against fracture is 4.0.

The reliability index can be found as follows

$$\beta = \frac{\ln(R_m / Q_m)}{\sqrt{V_R^2 + V_Q^2}}$$

Since LRFD is a reliability method, based on statistical data, the determination of β can be based on statistical data collected by the University of Minnesota for the Structural Shape Producers Council. From these data we find values for the required strength, Q_m, the resistance, R_m, and the coefficient of variation, COV, as follows:

$$Q_m = F_y A_g$$

$$R_m = \frac{F_y}{0.77}(x) A_g$$

$$COV = 0.038$$

where x is the ratio of the net area to the gross area, and 0.77 and 0.038 are the mean and *COV* of the yield-to–tensile strength ratio for web only data obtained in the study.

Substituting these values and solving for β = 4.0, we obtain a value of 0.896 for x. If we were to use 0.76 and 0.040, the mean and *COV* of the yield-to-tensile strength ratio for flange-only data obtained in the University of Minnesota study, we obtain a value of 0.892 for x. Since 20,003 flange samples were tested and 4,925 web samples were tested, taking a weighted average gives a value of 0.893 for x. Therefore, in cases where the net-to-gross area ratio is greater than 0.893, no reinforcing of the main member will be required. This approach allows a greater spread between the net area and the gross area than the AISC Seismic Provisions. Given the large number of samples (24,928) and the nearly normal distribution of the data, the reliability analysis is well-suited to the problem.

We will use the case of an ASTM A992 W14×90 brace as an example:

The expected strength of the member for connection design is

$$P_u = R_y F_y A_g = 1.1(50)(26.5) = 1{,}460 \text{ kips}$$

The force taken through the flanges of the brace is calculated as

$$F_f = P_u\left(\frac{A_f}{A_g}\right) = 1{,}460\left[\frac{(14.5)(0.71)}{26.5}\right] = 567 \text{ k}$$

The force taken through the web of the brace is calculated as

$$F_w = P_u - 2F_f = 1{,}460 - (2)(567) = 326 \text{ kips}$$

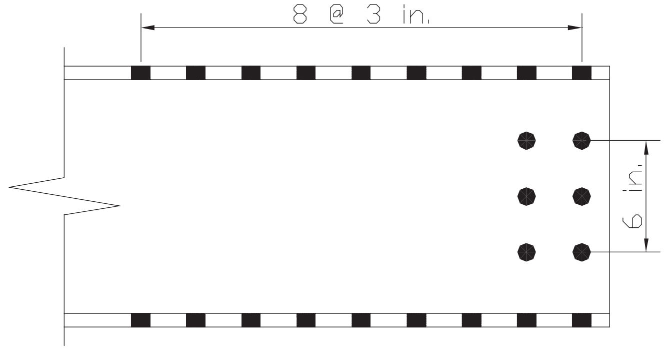
Assuming ⅞-in. A490-X bolts in single shear, the number of bolts required in the flange is

$$n = \frac{F_f}{\phi r_v} = \frac{567}{33.8} = 16.8 \text{ or 9 rows of 2 columns per flange}$$

Assuming bolts in double shear, the number of bolts required in the web is

$$n = \frac{F_w}{2\phi r_v} = \frac{326}{(2)(33.8)} = 4.82 \text{ or 2 columns of 3 rows}$$

The member is therefore configured as shown.



At the first set of holes (furthest from the support), the ratio of net-to-gross area is

$$x = \left(\frac{A_n}{A_g}\right) = 1 - \left(\frac{d_h t_f n}{A_g}\right) = 1 - \left[\frac{(1)(0.71)4}{26.5}\right] = 0.893$$

Therefore, the net section is okay, although it would not be okay per the AISC Seismic Provisions.

At the section with all the holes taken out, a portion of the load has been transferred out of the member and into the connection. Assuming each bolt resists an equal share of the load, the load remaining in the strut is

$$P_u = P_{tot}\left(\frac{n_s}{n_{tot}}\right) = 1{,}460\left(\frac{28}{48}\right) = 852 \text{ kips}$$

where

n_s = the number of bolts before the change in net section

n_{tot} = the number of bolts times the number of shear planes per bolt in the connection

The net section check then becomes

$$x = \frac{A_n}{A_g} = 1 - \frac{d_h t_f n + d_h t_w n}{A_g} = 1 - \frac{(1)(0.71)4 + (1)(0.44)3}{26.5}$$

$$= 0.843 > 0.893\left(\frac{28}{48}\right) = 0.521$$

Therefore, the net section is okay.

SUMMARY AND CONCLUSIONS

Good seismic performance depends on the dissipation of energy through inelastic deformations. Maintaining a satisfactory level of ductility is therefore required for good seismic performance. However, it can be shown analytically that it is not always necessary to add expensive reinforcing to the connected member in order to maintain the required ductility. By diligently observing design and detailing practices to preserve ductility in the connections, it is possible to utilize economic and safe field bolted connections in seismically loaded structures.

SYMBOLS

β = reliability index, the number of standard deviations that the mean of $\ln(R_m/Q_m)$ is from the origin

Q_m = mean value of the demand

R_m = mean value of the resistance

R_y = ratio of expected yield strength to the minimum specified yield strength; = 1.1 for A992 steel

V_R = coefficient of variance of the resistance

V_Q = coefficient of variance of the demand

t = thickness of plate, in.

x = the ratio of the net area to the gross area

All other variables are as defined in AISC (2005b).

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