SEISMIC Matters

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A closer look at seismic design with AISC's Steel Solutions Center.

HERE IN THE STEEL SOLUTIONS CENTER, we are periodically asked about the percentage of questions involving seismic design.

A quick analysis revealed that the answer is slightly more than one out of every 20 questions.

Even though seismic demand must be considered in the design of all structures, in many instances this demand is low enough that no special measures are required. Here, when we refer to "seismic design," we are really referring to buildings that must satisfy AISC's *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341, **www.aisc.org/standards**). Below is a handful of some of the more compelling and relatable seismic questions we've received recently. All of them are keyed to the newest (2016) edition of the *Seismic Provisions*.

A few notes before we dive into the questions and answers:

- We receive a fair number of questions related to the proper selection of the seismic force-resisting system (SFRS). Permitted systems are described in documents like the *International Building Code* and ASCE/SEI 7: *Minimum Design Loads for Buildings and Other Structures*. The organizations that develop and maintain these documents are the best source of information related to their intent.
- Structural steel is obviously the preferred choice for a structural system, but there are several steel options available. If permitted, a structural steel building "not specifically detailed for seismic resistance" (an R = 3 system) is generally the most economical choice. The February 2018 Steel Interchange column included three questions related to the selection of the SFRS. You can view it at **www.modernsteel.com**.
- In addition to the items presented here and in the February Steel Interchange, you can find other seismic-related Steel Interchange items at **www.modernsteel.com** (select Resources, then Steel Interchange). Items dating back to May 1997 are individually listed, making a search for the term "seismic" the easiest way to find seismic-related items.

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Slip-Critical Connections in the Seismic Force-Resisting System

Do all bolted connections in the SFRS have to be designed as slip-critical?

No. This is a relatively common question. With a few exceptions, Section D2.2 of the *Seismic Provisions* indicates that the available shear strength of bolted joints shall be calculated as bearing-type. However (again, with a few exceptions) the bolts shall be pretensioned and the faying surfaces shall have a Class A (or higher) slip coefficient.

In effect, the connections are designed as bearing connections and detailed, fabricated and erected similar to slip-critical connections.

Demand-Critical Welds

Are all welds in the SRFS required to be demand-critical?

No. The *Seismic Provisions* explicitly states every location where demand-critical welds are required. There are no requirements for demand-critical welds listed in Chapters A through D. Therefore, the easiest way to determine which welds in a particular system need to be demand-critical is to go to the section covering the system in question (E1 for ordinary moment frames/OMFs, F2 for special concentrically braced frames/SCBFs, etc.) and search for the words "demand critical." When designing special moment frames (SMFs) there may also be connection-specific requirements in AISC 358: *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (a free download from **www.aisc.org/seismic**).

Defining Elements in the Seismic Force-Resisting System

Is a particular element (a beam, a column, a horizontal brace, etc.) part of the SFRS?

Two answers are provided: one to the engineer and one to the fabricator.

Fabricator: Section A4.1 of the *Seismic Provisions* requires the engineer to designate the SFRS and identify members and connections that are part of the SFRS in the structural design drawings and specifications. If the intent is not clear relative to a particular member or connection, then you should seek clarification from the engineer.

You should not be expected to make this determination without the information required in the *Seismic Provisions*, and you should certainly not try to guess or assume the intent if it is not clear.

Engineer: Section A4.1 of the *Seismic Provisions* requires the engineer to designate the SFRS and identify members and connections that are part of the SFRS in the structural design drawings and specifications. As the engineer, you need to define and communicate your intent.

It is sometimes argued that the definition of the SFRS provided in the *Seismic Provisions* requires that all elements in the structure be considered part of the SFRS. This argument is often based on the assertion that all of the elements are included in the computer model, and therefore seismic demand "leaks" into every element in the model.

The *Seismic Provisions* requires the engineer of record (EOR) to define the SFRS in the contract documents. The intent of the *Seismic Provisions* is that the EOR will apply their own knowledge, judgement and experience when defining the SFRS. It is not the intent to have such items as gravity beams, conveyor supports, pipe hangers and grating clamps meet the requirements of the *Seismic Provisions*, though we have heard arguments made that we require such practices. Ultimately, the EOR must make this determination. The SFRS is not limited to the members in the named system (ordinary concentrically braced frame—OCBF—SMF, etc.) but neither must it extend to every element, or a majority of elements, in the structure. Elements that are sometimes overlooked are collectors and chords.

Per Section A1, the *Seismic Provisions* applies to buildings and other structures with moment frames, braced frames and shear walls. Other structures are defined as those structures designed, fabricated and erected in a manner similar to buildings, with building-like vertical and lateral force-resisting elements. As the structures being designed approach the limits of the scope of the *Seismic Provisions*, the engineer must apply significant judgment to its application to ensure the intent is satisfied. When it comes to industrial, nonbuilding structures, determining when and how the *Seismic Provisions* applies can also be tricky and requires some judgment.

Unlisted Materials in the Seismic Force-Resisting System

In order to reduce costs, the owner of a project I am working on would like to use material in the SRFS that is not listed in Section A3.1 of the *Seismic Provisions*. Is this permitted?

No. The AISC *Specification for Structural Steel Buildings* (ANSI/AISC 360, **www.aisc.org/specifications**) has a pretty liberal stance on materials. Section A3.1 provides a list of approved materials but does not prohibit the use of other materials. The Commentary provides further guidance.

However, the *Seismic Provisions* treats material selection differently. Section A3.1 states: "Structural steel used in the seismic force-resisting system (SFRS) shall satisfy the requirements of *Specification* Section A3.1, except as modified in these Provisions." It then provides requirements and exceptions related to the yield strength of the material. It goes on to state: "The structural steel used in the SFRS described in Chapters E, F, G and H shall meet one of the following ASTM Specifications…" and provides a list of permitted materials. There is an exception made for materials in buckling restrained braced frames (BRBFs), which are subject to cyclic qualification tests. The permitted materials have been selected to be consistent with tested seismic systems and to reflect desirable seismic performance characteristics (e.g., ductility or limited maximum tensile capacity) consistent with the requirements of the *Seismic Provisions*.

Even if other materials were not explicitly prohibited, their use in the SFRS could be difficult due to lack of expected material strengths established to be consistent with the *Seismic Provisions*. The lack of values for R_y and R_t effectively excludes the use of unlisted materials for yielding elements and makes correct implementation of some provisions virtually impossible.

Maximum Force that Can Be Delivered to the System

In a number of places, the *Seismic Provisions* allows the design force (required strength) for elements to be limited based on the maximum load effect that can be transferred to the element by the system. What does this mean and how is it implemented?

A basis of design is included for each system addressed in the *Seismic Provisions*. It describes the intended behavior of each system and often designates the expected source of inelastic deformation capacity (the yielding element) such as the moment beam in special and intermediate moment frames (IMFs) or the brace in BRBFs. In some cases, something other than the strength of the yielding (or buckling, etc.) member limits the force that can be delivered to an element. The exception you describe is intended to account for this possibility.

A good deal of judgment is required when exercising this option and to recognize that the exception rather than the rule is being applied. What may be acceptable behavior for one system may not be acceptable for another. The Commentary provides guidance.

For OMFs, the Commentary lists the following factors that can limit the forces transferred to the connection: column yielding, panel zone yielding, foundation uplift and the overstrength seismic load.

For SCBFs, the Commentary indicates that the maximum force that can be transferred to the connection can be determined by: performing a pushover analysis to determine the forces acting on the connections at the frame capacity, determining how much force can be resisted before causing uplift of a spread footing or performing a suite of inelastic time history analyses and enveloping the connection demands.

It must be recognized that limiting the force in this manner represents an exception and a relaxation relative to the behavior described in the basis of design. If the exception is employed, the emphasis should be on the qualifier "maximum." Again, the Commentary provides guidance and cautions against the use of lower-bound methods when determine the maximum load effect (force). Assumptions made relative to the effects of parameters like restraint and material strength should tend to overestimate the strength of the system. This is generally opposite to the approach taken in typical design.

It also needs to be recognized that the same uncertainty that makes the inelastic approach adopted for seismic design desirable or even necessary still exists when trying to calculate the maximum force that can be delivered by the system. When yielding cannot occur in some of the potential sources of inelastic deformation capacity, a more accurate determination of the demand is in order. However, as stated in the Commentary, "It is unrealistic to expect that such forces can be accurately calculated." Again, a cautious approach is warranted.

The resulting design must be consistent with the assumptions underlying the use of the exception (e.g., if anchor rods in the base plate are assumed to yield prior to buckling of the braces, the anchor rods must be designed and detailed so that they can yield and develop the ductility assumed by the designer).

Even after the maximum force has been determined, it is important to have a clear understanding of the underlying model. Engineers sometimes get lost in the maze of provisions when they approach the requirements as a lawyer might. It is better to take an engineering approach.

For example, if it has been determined that column panel zone shear limits the force (load effect, moment, etc.) that can be delivered by the system, then the best place to start your connection design is at the panel zone. Draw a free-body diagram of the panel zone at its ultimate strength. Then draw free-body diagrams of the stiffeners that transfer these forces. Finally, draw a free-body diagram of the beam-to-column connection to complete the design. Only after you have determined the maximum force, established a load path and completed all of the limit state checks should you put on your lawyer's hat to go back and cross the t's and dot the i's relative to any prescriptive *Seismic Provision* requirements. Remember the *Specification* and *Seismic Provisions* are written by engineers. There is usually some rational model underlying all of the words.

Erection Holes for Vertical Braces

The fabricator on our project has placed erection holes in hollow structural section (HSS) braces that are to be field welded during erection. Section F2.5c of the *Seismic Provisions* defines the protected zone as "the center one-quarter of the brace length and a zone adjacent to each connection equal to the brace depth in the plane of buckling." Figures C-F2.14 and C-F2.15 in the Commentary also indicate that the area near the end of the brace is a protected zone. Based on this, we believe the erection holes are prohibited. Please confirm that our understating is correct.

Your understanding is not correct. Users of the *Seismic Provisions* sometimes mistakenly assume that all work is prohibited in the protected zone. However, Section F2.5c also defines the "elements that connect braces to beams and columns" (essentially gusset plates) as protected zones. Gusset plates are commonly thermally cut, drilled and welded. Work is clearly permitted to be performed in the protected zone.

Section D1.3 prohibits "discontinuities specified in Section I2.1." Section I2.1 lists prohibited and permitted attachments and work. Some items are to be "repaired as required by the engineer of record." Among these items are erection holes. Section I2.1 also provides an exception: "Other attachments are permitted where designated or approved by the engineer of record." Therefore, the engineer of record is given some latitude relative to the treatment of erection holes. In my experience, it is common to provide erection holes and it is also common to place the erection holes such that they can be left in place after erection.

The commentary to Section D2.2 states: "The Provisions do not prohibit the use of erection bolts on a field-welded connection such as a shear tab in the web of a wide-flange beam moment connection." Figure C-D2.1 also indicates the use of erection bolts.

We have on a few occasions received inquiries about projects where erection holes were not provided because they were believed to be prohibited. In these cases, the braces had been misaligned during field welding due to the lack of erection holes. Correcting the mislocated braces presented a significant challenge and a significant cost.

Combining Forces at Column Bases

I am designing a column base per Section D2.6 of the *Seismic Provisions*. This section contains provisions for determining the required axial, shear and flexural strength of the column base. It is clear that when the loads are calculated using the overstrength seismic load, the shear, moment and axial loads can act simultaneously as indicated by the analysis. However, in some cases the loads determined from Section D2.6 are determined based on capacity of the column. In these cases, should the required axial, shear and flexural strengths be assumed to act simultaneously?

No. This can be inferred from the information provided in Section D2.6.

The capacity-based demands of Sections D2.6b(b)(1) [required shear strength] and D2.6c(b)(1) [required flexural strength] are derived from the same model, which is described in the Commentary as "a mechanism in which the column forms plastic hinges at the top and bottom of the first story." These load effects obviously must act simultaneously. However, once the hinge forms (as the moment reaches the expected flexural strength of the column) no further axial load can be delivered to the base plate through the column. Therefore, there is no need to consider a condition where the capacity-based moment and shear act and simultaneously with an axial load.

There is no axial load defined in Section D2.6a based on the expected strength of the column. In addition to calculating the demand "using the overstrength seismic load" Section D2.6a references Section D2.5, which in turn references Chapters E, F, G and H and Section D1.4a. Section D1.4a requires consideration of simultaneous inelasticity from intersecting frames when determining the required axial strength of the column and therefore the splice and base plate connection (with some exception). The "load effect resulting from the analysis requirements for the applicable system per Chapters E, F, G and H" can certainly involve a combination of shears, moment and axial load, but these are not based on the capacity of the column.

Though not explicitly stated in Section D2.6, it seems reasonable that the demand on the base plate need not exceed the expected strength of the column. The expected strength of the column should be calculated to produce the greatest load effect on the steel elements at the column base. It can be determined as: (1) the maximum moment and corresponding shear that could be delivered by the column (considering the expected column strength and strain hardening) in combination with the required axial load or (2) the maximum axial load that could be delivered by the column (considering the expected column strength) considering the required moment.

Prequalified Moment Connections

Do the beam-to-column moment connections for special and intermediate moment frames need to be prequalified per AISC 358?

No. The beam-to-column moment connections used in special and intermediate moment frames have to accommodate a specified story drift angle while maintaining a specified level of flexural strength. Conformance with these requirements must be demonstrated through physical tests. Sections E2.6c (IMF) and E3.6c (SMF) provide four options:

- Use of SMF prequalified connections designed in accordance with AISC 358
- Use of a connection prequalified for SMF in accordance with Section K1
- Use of tests reported in the research literature or documented tests performed for other projects
- Use of tests that are conducted specifically for the project

Though any of these methods are acceptable, the use of connections in AISC 358 is the most commonly used option. When designing buildings with SMF and IMF seismic force-resisting systems, the engineer should confirm that the design allows for use of conforming connections. It should be noted that AISC 358 contains prequalification limits for all included connection configurations and that the other methods of demonstrating conformance will also have limitations.

We also sometimes receive questions about connections to the weak axes of columns and connections involving HSS. There are no prequalified moment connections that can be made to the weak axes of columns, though AISC 358 does address boxed wide-flange columns, built-up box columns and flanged cruciform columns, which can be used to accommodate orthogonal frames. AISC 358 contains only one prequalified connection for use with HSS columns and it is limited to concrete-filled 16-in. square HSS columns. There is also only one prequalified connection for use with HSS beams, though this is limited to rectangular and square HSS. There are no prequalified connections using round HSS. ■