Design Tips for Steel in Low or Moderate Seismicity Regions

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National model building codes now include seismic design requirements that directly impact areas where seismicity is low or moderate.

This paper provides tips to structural engineers who design buildings in regions of low or moderate seismicity. These tips are based primarily on the authors' experiences designing, peer reviewing, and investigating steel-framed buildings. The topics covered include determining site class and seismic design category, selecting a steel seismic-force-resisting system, and applying detailing requirements.

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MOST OF THE UNITED STATES HAS LOW OR MODERATE SEISMICITY. Historically, buildings in these areas were not designed explicitly to resist earthquake forces. People living in these areas most likely have never experienced a damaging earthquake, and building designers and constructors in these regions do not have extensive experience with earthquake-resistant construction.

National model building codes now include seismic design requirements that directly impact areas where seismicity is low or moderate. Many state and local jurisdictions have adopted a newer national model code, forcing structural engineers throughout the United States to understand and apply these new code requirements and, in many cases, to explain them to architects, building owners, and contractors. The seismic design provisions are not simple. The consequence of misapplying these provisions could result in buildings that lack the intended strength and ductility or that are unnecessarily costly.

The core seismic design requirements in most modern United States building codes are found in *Minimum Design Loads for Buildings and Other Structures* (ASCE/SEI 7-05, 2005; ASCE/SEI 7-02, 2002). The latest version is ASCE/SEI 7-05, which recently replaced ASCE/SEI 7-02. The fundamental approach and requirements of the two versions are very similar; however, some important changes that are relevant to areas of low or moderate seismicity are noted throughout this paper.

The seismic design forces for a building depend on several factors: the ground motion characteristics of the geographic area (seismic maps); the geology of the site (site class); the occupancy of the building (seismic use group or occupancy category); the building's fundamental period of vibration (T); and the building's framing system and seismic force-resisting system (R Value).

ASCE/SEI 7-05 defines six seismic design categories (SDCs), A through F. SDC A and SDC B are for low seismic risk; SDC C is for moderate seismic risk; and SDCs D, E, and F are for high seismic risk. The only requirements for SDC A are a minimum lateral strength and complete load path; therefore, the tips included here apply primarily to SDC B and C buildings. Essential facilities and buildings that represent a substantial hazard to human life in the event of failure (Seismic Use Group III or Occupancy Category III) may be assigned to SDC D in regions of moderate seismicity.

Seismicity Maps

ASCE/SEI 7-05 includes the 2002 seismicity maps prepared by the Building Seismic Safety Council and the United States Geological Survey, while ASCE/SEI 7-02 includes the 1996 version of these maps. The 2002 maps include short period spectral response accelera-

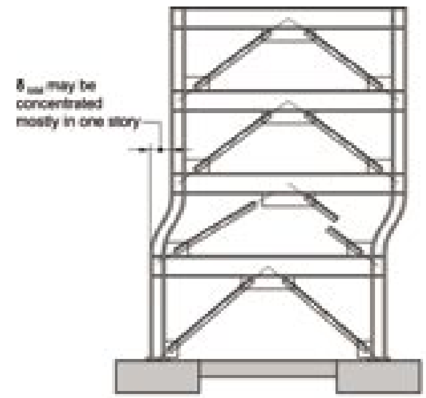

Figure 1. Drift mostly concentrated in one story.

tion parameters along much of the Atlantic seaboard that are about 10% lower than those from the 1996 maps, and include one-second spectral response acceleration parameters that are 20% lower than those from the 1996 maps (Frankel, et al., 2002). The lower spectral acceleration parameters embodied in the new maps will result in many buildings in the eastern United States falling into a seismic design category one lower than they would have based on the older maps.

The impact of the mapping changes is greatest on sites with underlying soils that significantly amplify bedrock accelerations. Some of the buildings on these sites that would have been SDC D based on the 1996 maps will fall into SDC C based on the 2002 maps. In specific regions such as New England, the new maps provide substantial reductions in the number of buildings that must be designed for SDC D; however, some buildings will still fall into this higher design category.

Even if the local building code does not yet incorporate the 2002 maps, designers should study the newer maps to determine if some reduction in spectral accelerations is shown in their area of practice. Building officials may accept the use of the newer maps prior to their being formally adopted into the local building code.

Site Classification

Site classification has a substantial impact on the seismic design category of a building. There are six site classes, A through F. Site Class A is hard rock and Site Class F is a site with poor soils requiring a site-specific response analysis. The site class impacts both the seismic design forces for the building and the seismic design category, which may limit the type of framing system that can be used, dictate how geometric irregularities are considered, and establish detailing requirements. Although the greatest impact occurs between SDC C and SDC D, differences between other seismic design categories have important design implications for buildings in regions of low or moderate seismicity.

Assigning the site classification is typically a geotechnical engineer's responsibility; however, structural engineers should become familiar with the site classification provisions so they can ask informed questions and make recommendations to assist the geotechnical engineer. The goal should be to obtain an accurate site classification, not a conservative one. When existing borings are available, the structural engineer can estimate what the site class will likely be before the geotechnical engineer classifies the site. In some cases, the structural engineer can assign the site classification.

There are three methods for determining the site class. Measured shear wave velocity is the most accurate of the methods. It is also likely the most expensive, but it is not prohibitively expensive. The average standard penetration resistance method is the most common method as it requires little or no extra field work and can be performed rapidly. The average undrained shear strength method is generally more accurate than the average standard penetration resistance (blow count) method when there are significant deposits of cohesive soil. However, the standard penetration resistance and the undrained shear strength methods are rather conservative because the correlation between site amplification and standard penetration resistance or undrained shear strength is more uncertain than the correlation between site amplification and shear wave velocity.

Four tests are available for measuring shear wave velocities: seismic cross-hole, seismic down-hole, seismic cone, and spectral analysis of surface waves. These tests are described in *Geotechnical Earthquake Engineering* (Kramer, 1996).

Seismic Force-Resisting Systems

For buildings in areas of low or moderate seismic activity, it often makes sense to use a structural steel system not specifically detailed for seismic resistance—an *R*-of-3 system. An *R*-of-3 system has the advantages of reduced design effort, reduced chance of errors, and of not restricting the type of connections and detailing used.

In the central and eastern United States, the choice of connection details has historically been left to the steel fabricator, allowing individual fabricators to select connections that are efficient for their shops to fabricate. Typically, structural engineers would provide connection forces for the fabricator to design the connections. When engineers provided specific details, it was common for fabricators to request a change. This practice is still common today. Unless a structural engineer is very clear in the design documents as to how a connection is to be fabricated, and firm in denying requested changes, the fabricator will often look to change the details. By selecting an *R*-of-3 system, the structural engineer allows the fabricator greater flexibility to provide connections that are efficient for the particular fabrication shop.

An *R*-of-3 system requires less design effort than other steel systems. The reduced design effort is the result of less time spent applying esoteric code provisions, developing project-specific connection details, and conveying the design intent. The offsetting consideration may be higher steel weight, as the design seismic forces will be greater than with a more ductile system. However, in some locations wind forces govern the design. Also, the added cost of fabricating ductile connections can easily make the cost of the steel framing for a ductile system greater than that for an *R*-of-3 building, offsetting any savings in the weight of steel.

Although providing ductile structures is desirable for buildings in regions of low or moderate seismicity, design and construction mistakes that reduce ductility are common in these buildings. Examples of such mistakes include inadequate bottom flange bracing of beams in moment frames, inadequate bracing of overhead beams in chevron braced bays, and inadequate connection strength. Mistakes such as these can greatly reduce the ductility of a structure. It is better to have the relative strength of the seismic force-resisting system raised by using a low value for an *R* factor than it is to provide a theoretically ductile system but not follow through with the detailing to achieve the ductility.

The basis for the *R*-of-3 system is the consensus opinion of the Building Seismic Safety Council's Provisions Update Committee (PUC) without strong backup research. Its use is restricted to SDCs A, B, and C in recognition that structures assigned to these SDCs do not require the same level of ductility to provide the required performance as buildings assigned to the high risk SDCs (FEMA, 2004). Some researchers and practitioners are now questioning the use of *R*-of-3 buildings based on the lack of research to demonstrate adequate performance. Although there is a strong desire to maintain such a system for low and moderate risk structures, results of ongoing research or future research could change the opinions of other committees involved in setting the requirements for systems. If so, greater restrictions could be placed on the use of this system, the value of *R* may be reduced, or some connection requirements may be added.

Other steel systems are also candidates for new design requirements. The provisions for ordinary concentrically braced frames changed with each edition of the *Seismic Provisions for Structural Steel Buildings* (AISC 1992; AISC 1997; AISC 341-02, 2002; AISC 341-05, 2005). Based on this history of changes to the provisions for ordinary concentrically braced frames, future changes are still possible. Recent testing has called into question some of the assumptions regarding the ductility of special concentrically braced frames. The assumptions regarding ductility of this system are now being studied. Further changes to the provisions for special concentrically braced frames, or more limiting restrictions on their use, are possible.

Based on the possibility of future restrictions or changes, there are situations for which the use of an *R*-of-3 system, ordinary concentrically braced frames, and special concentrically braced frames may not be the best choice. One of these situations is when a structure is designed for a future addition. In the last fifteen years, many engineers have had to tell clients that a planned vertical addition may not be constructed without strengthening the lower stories that were originally designed for the addition. When designing a building for a future vertical addition, it is prudent to use a system for which the requirements are well established and unlikely to change in the near future. Such systems include special steel moment resisting frames, which were closely scrutinized following the Northridge Earthquake, and eccentrically braced frames. Buckling restrained braced frames are also a good candidate, but the provisions are new and thus could change as more is learned about these systems.

An *R*-of-3 system or an ordinary concentrically braced frame system may not be the best choice for tall buildings. Currently, these systems have no height restrictions for SDC B and SDC C buildings. Failure of vertical elements of a seismic force-resisting system in a single story, such as the braces in a single story, tends to concentrate displacements in that story as shown in Figure 1. If the displacements are large enough, that story could become unstable. In regions of low or moderate seismicity, the drift demands are generally less than those for buildings in a region of high seismicity. Therefore, it is reasonable to use a less ductile system for taller buildings in regions of low or moderate seismicity than in regions of high seismicity.

Until further research is completed, the authors recommend limiting the use of the *R*-of-3 system and ordinary concentrically braced frames to buildings not exceeding a height of 110' for SDC B and 65' for SDC C. In considering whether to apply such an in-house limit, some consideration could be given to the inherent strength of the gravity system. For buildings with composite beams connected to columns with shear connections, the gravity system can provide substantial supplementary lateral strength and stiffness (Liu et al., 2000). The additional strength, stiffness, and column continuity help to distribute lateral deformations to multiple stories, even after failure of braces in a single story, as depicted in Figure 2. By comparison, a gravity system of steel joists adds little additional lateral strength or stiffness. However, the continuity of gravity only columns can substantially reduce the risk of forming a single story mechanism (Gupta et al., 1999), as depicted in Figure 3.

When a building is on deep foundations, the cost of the foundations can be a significant portion of the overall cost of the structure. The forces on the foundation can be reduced if a more ductile seismic force resisting system with a larger *R*-factor is used. Therefore an *R*-of-3 system or an ordinary concentrically braced frame may reduce the cost of the superstructure, but may not be the most economical choice for the project. Using systems that do not impose large design overturning moments on the foundation may result in the best overall economy. Special moment frames, eccentrically braced frames, and special concentrically braced frames all reduce the foundation design forces relative to those of an *R*-of-3 system or an ordinary concentrically braced frame, and may result in the lowest cost for the structure.

Ordinary concentrically braced frames are considered to be low ductility systems and are generally not permitted for buildings in SDC D. An important exception allows this popular and economical system to be used for SDC D buildings that do not exceed a height of 35'. This exception is applicable to many one-, two-, and three-story buildings.

When designing a SDC D building in a region of moderate seismicity, restrictions on allowed systems may require the selection of a seismic force resisting system that is not commonly used in the region. Complete details will be essential to prevent the fabricator from filling in the gaps with details that are not in compliance with the system. In these situations it may be most effective to use a system that impacts the structure locally, such as special concentrically braced frames or eccentrically braced frames, rather than a system such as a special moment frame, which will require a larger percentage of the structure to be different from the local historical practice.

Interconnection

ASCE/SEI 7-05 requires interconnection of all parts of a structure between separation joints. The smaller portion of the structure must be attached to the larger portion of the structure with

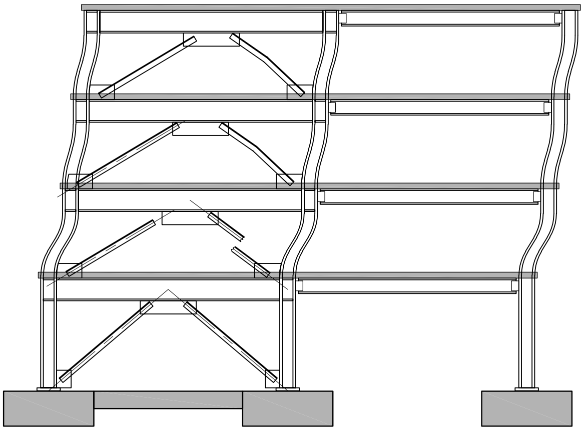

Figure 2. Gravity beams, columns and slab improve distribution of drift.

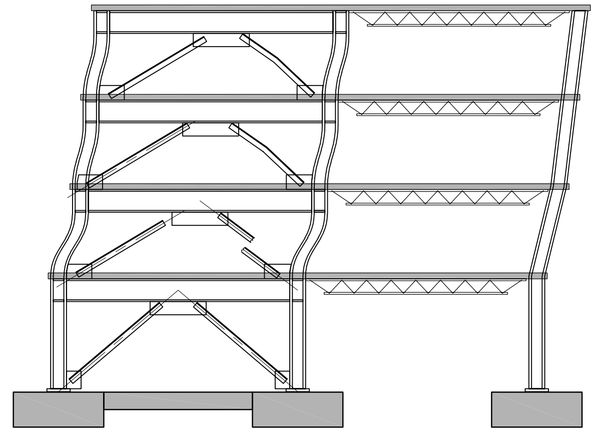

Figure 3. Gravity columns and slab improve distribution of drift.

elements having strength to resist a force of 0.133 times the short period design spectral response acceleration parameter times the weight of the smaller portion of the structure. In general, this requirement is easy to satisfy for structural steel buildings. A condition in which this requirement should be given greater consideration is at a change in elevation of a floor or roof diaphragm. Unless specific details are provided to transfer horizontal forces at the change in elevation, the columns attached to both diaphragm levels must be designed to transfer this force in column bending. Currently available commercial analysis and design software typically does not consider these interconnection forces, so the designer must take care to follow the forces through the structure and properly design all of the elements that transfer this force.

Buildings framed with open web steel joists and joist girders may require special attention to the interconnection requirement. Typically, the required continuity can be obtained through the connections of joists and joist girders located on column lines and attached to the columns. For buildings in which joists supported by joist girders must aid in connecting the smaller portion of a building to the larger portion, continuity can be established across the gap between the top chords of joists in adjoining bays. If the joists are aligned end to end, the required continuity can be achieved easily by adding a bar to connect the ends of joists in adjacent bays.

Connection Strength for Ordinary Concentrically Braced Frames

AISC 341-02 required brace connections of ordinary concentrically braced frames to develop the expected tensile strength of the brace. Bracing element sizes are controlled by stability, *KL/r*, requirements. Thus, designing for the expected tensile strength of the brace can result in forces that are many times greater than required for elastic behavior during an earthquake. AISC 341-05 addresses this situation by allowing the connections to be designed for the load combinations with seismic forces amplified by an overstrength factor, which can lead to substantially lower connection design forces.

Collector Elements

For SDCs C, D, E and F, collector elements must be designed for the effects of the seismic force amplified by an overstrength factor. In locations where the seismicity is high, collector element connections often consist of welded-flange connections. In SDC B and C buildings it is common to specify a horizontal force to account for the collector elements. In many cases a double angle shear connection is designed to resist the tension as well as the gravity shear. Although this may be acceptable in some circumstances, the flexibility of the connection may allow an objectionable crack to form in the slab prior to developing the required strength. An alternative that is often used is to include shear studs on the tops of the collector elements and to add slab reinforcement to develop the collector force where the collector element connects to columns and other elements.

Vertical elements of the seismic force resisting elements are often located where it is difficult to deliver the forces to the elements. A common situation, shown in Figure 4, is a braced frame located in an exterior wall at a floor opening for a stair or an elevator. In this situation, the diaphragm cannot transfer loads directly to the braced frame, so an effective collector is essential. A stiff connection between the collector and the column of the braced frame is important to effectively transfer the diaphragm force into the braced frame. In this case, simply specifying the horizontal connection force may not be enough. The connection could either be detailed on the design drawings or a stiffness limitation could be specified with the horizontal design force.

Another consideration for the braced bay being located in an exterior wall adjacent to an opening is the beam must be designed to transfer loads from the brace connection locations on top of the beam to the brace connection locations below the beam. Commonly used design software does not necessarily correctly model this condition, especially when a rigid diaphragm assumption is made. Even if brace connection locations are the same above and below the beam, this beam will have to transfer an axial force to other brace connections as the flow of forces through the braces are redistributed. This condition can often be addressed by attaching a channel to the top and bottom flange of the beam to provide the necessary stability to transfer the axial force in the beam.

The configuration of braces in an exterior wall adjacent to an opening can also affect the performance of a braced bay. For a chevron configuration, braces are connected to the underside of the beams near midspan. Bracing the bottom flange of the beam to resist out-of-plane movement or twist at this connection is essential. If bracing is not provided, the beam bottom flange can twist out-of-plane when a brace buckles. Thus, a chevron configuration should be avoided when the braced bay is located in an exterior wall adjacent to an opening. If a chevron configuration is used, the beam should be designed to resist the out-of-plane force of a buckled brace.

Simplified Design Procedure

ASCE/SEI 7-05 includes a new design procedure that has greater applicability to low-rise structures. ASCE/SEI 7-02 has a simplified analysis procedure that is applicable to many one- and two-story buildings in SDCs B and C, but this analysis procedure has not been widely used. The simplified design procedure in ASCE/SEI 7-05 is more complete and can be used for most simple buildings on firm soil and rock sites that are three stories or less in height. This new design procedure can be used to save design time for small simple buildings with little impact on the cost of the structure.

Flexible Diaphragms

ASCE/SEI 7-05 allows un-topped metal deck diaphragms to be treated as flexible diaphragms for buildings with shear walls and braced frames. ASCE/SEI 7-02 does not allow un-topped metal deck diaphragms to be treated as flexible diaphragms unless calculations demonstrate that the stiffness of the diaphragm relative to the stiffness of the vertical elements of the seismic force-resisting system meet certain criteria. In effect, ASCE/SEI 7-05 is now allowing a common practice among engineers to treat un-topped metal deck diaphragms as flexible without performing a stiffness check.

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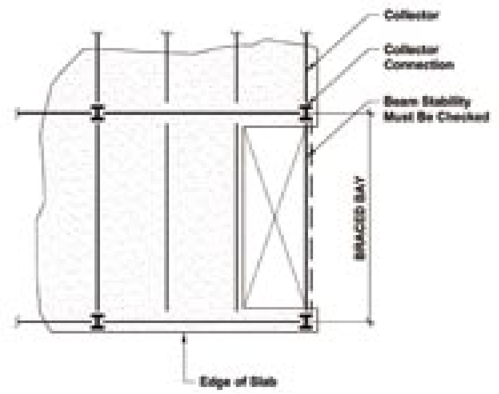

Figure 4. Collector connection for braced bay at floor opening.