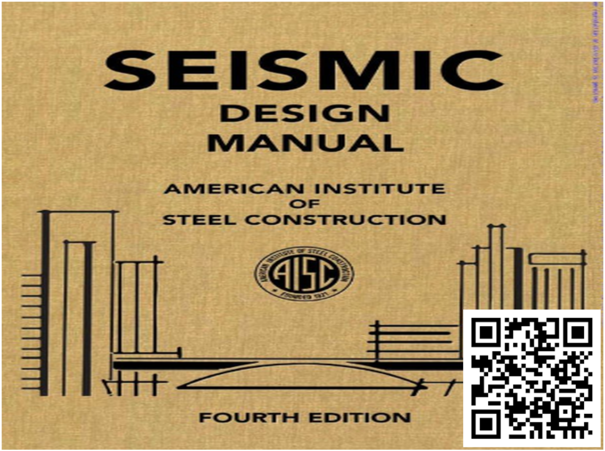
SEISMIC
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SEISMIC

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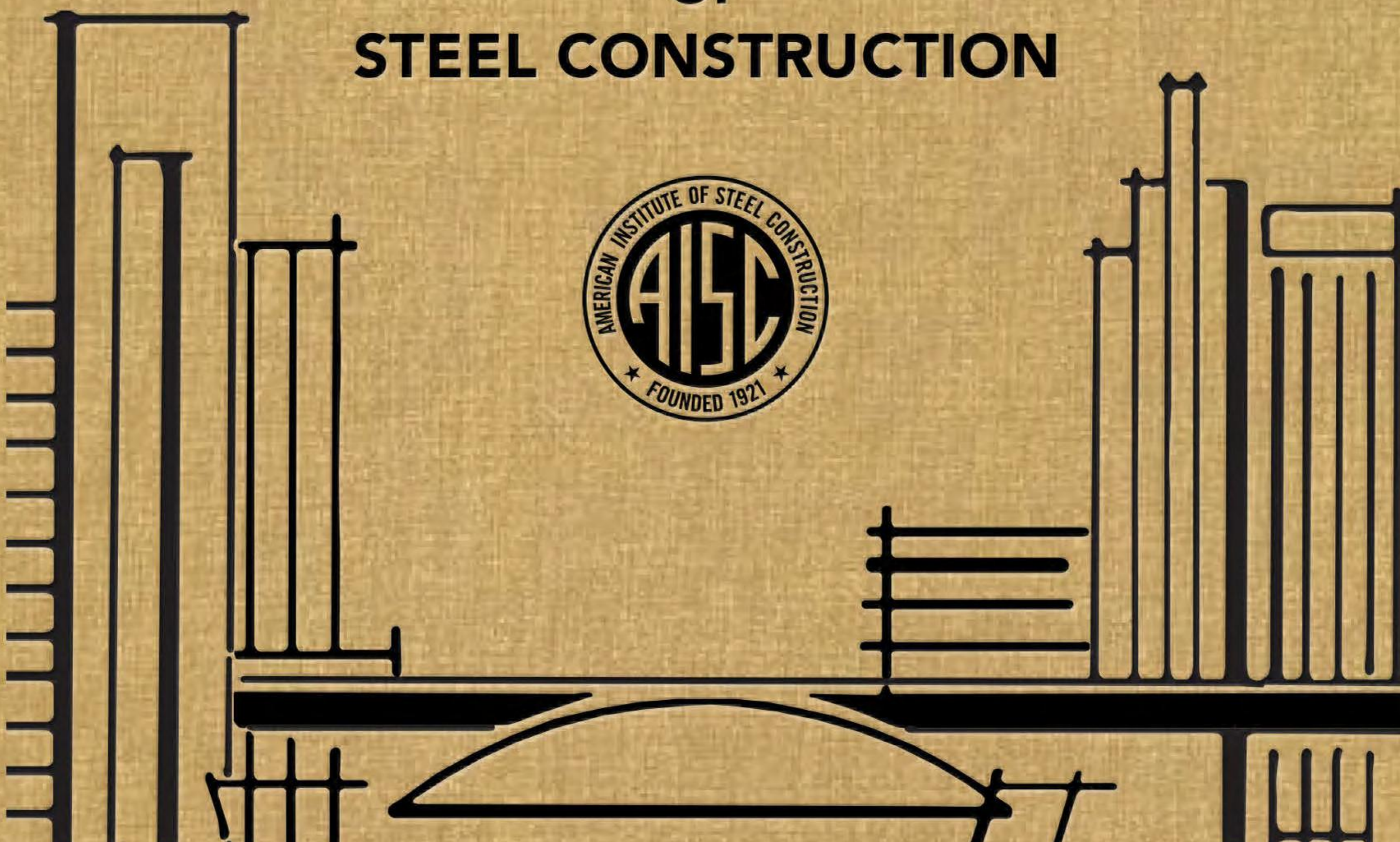

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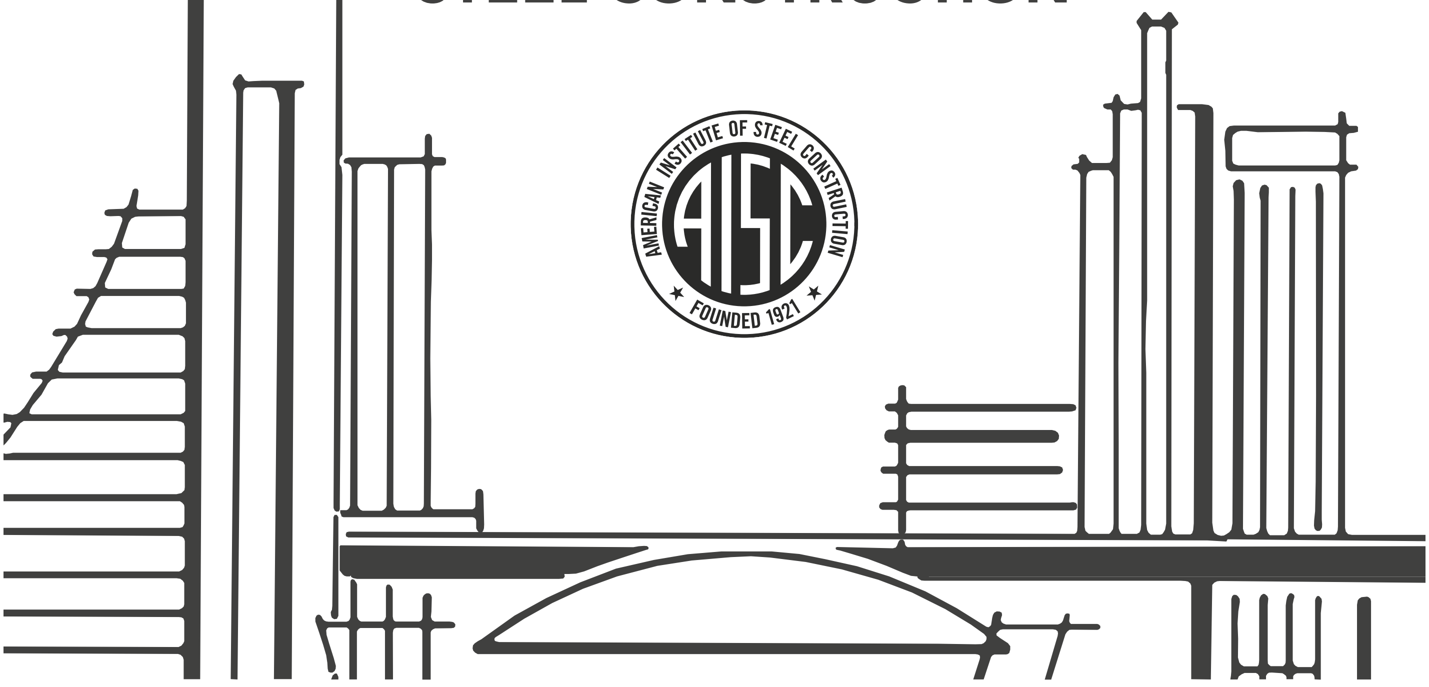

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FOREWORD

The American Institute of Steel Construction, founded in 1921, is the nonprofit technical specifying and trade organization for the fabricated structural steel industry in the United States. Executive and engineering headquarters of AISC are maintained in Chicago. The Institute is supported by four classes of membership: Full Members engaged in the fabrication, production, and sale of structural steel; Associate Members, who include Erectors, Detailers, Service Consultants, Software Developers, and Steel Product Manufacturers; Professional Members, who are individuals or firms engaged in the practice of architecture or engineering, including architectural and engineering educators; and Affiliate Members, who include Building Inspectors, Code Officials, General Contractors, and Construction Management Professionals. The continuing financial support and active participation of Members in the engineering, research, and development activities of the Institute make possible the publishing of this *Seismic Design Manual*.

The Institute's objective is to make structural steel the material of choice, by being the leader in structural-steel-related technical and market-building activities, including: specification and code development, research, education, technical assistance, quality certification, standardization, and market development.

To accomplish this objective, the Institute publishes manuals, design guides, and specifications. Best known and most widely used is the *Steel Construction Manual*, which holds a highly respected position in engineering literature. The Manual is based on the *Specification for Structural Steel Buildings* and the *Code of Standard Practice for Steel Buildings and Bridges*. Both standards are included in the *Steel Construction Manual* for easy reference.

The Institute also publishes technical information and timely articles in its *Engineering Journal*, Design Guide series, *Modern Steel Construction* magazine, and other design aids, research reports, and journal articles. Nearly all of the information AISC publishes is available for download from the AISC web site at **www.aisc.org**.

PREFACE

This is the fourth edition of the AISC *Seismic Design Manual*, intended to assist designers in properly applying AISC standards and provisions in the design of steel frames to resist high-seismic loadings. This Manual is intended for use in conjunction with the AISC *Steel Construction Manual*, 16th Edition.

The following consensus standards are printed in Part 9 of this Manual:

- 2022 *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341-22)
- 2022 *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications* (ANSI/AISC 358-22)

The design examples contained in this Manual demonstrate an approach to the design and are not intended to suggest that the approach presented is the only approach. The committee responsible for the development of these design examples recognizes that designers have alternate approaches that work best for them and their projects. Design approaches that differ from those presented in these examples are considered viable as long as the AISC *Specification* and AISC *Seismic Provisions*, sound engineering, and project specific requirements are satisfied.

The following major changes and improvements have been made in this revision:

- More thorough and comprehensive design examples, updated for the 2022 AISC *Seismic Provisions*, the 2022 AISC *Specification*, and new AISC Design Guides
- Addition of a table in Part 1 summarizing the applicable requirements of the AISC *Seismic Provisions* for the design of seismic force-resisting systems
- Addition of a table in Part 2 providing values for the second-order amplifier, B_2, for use with the approximate second-order analysis procedure
- Updated example for the column splice in a special moment frame to consider PJP groove welds at the flanges and web
- Addition of an example illustrating the connection design at the intersection of two braces in an X-configuration in an ordinary concentrically braced frame
- Addition of an example for determining the required strength at the base of a column in a special concentrically braced frame
- Addition of an example for determining the required strength in a column that is shared between a special moment frame and an orthogonal special concentrically braced frame
- Addition of multi-tiered buckling-restrained braced frame examples
- Updated discussion of diaphragms in Part 8 regarding load path, challenges in analysis, and common assumptions, along with guidance on diaphragm modeling

By the AISC Committee on Manuals,

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The committee gratefully acknowledges the contributions made to this Manual by Patrick J. Fortney and William T. Segui who passed away during this cycle, as well as the AISC Committee on Specifications, the Connection Prequalification Review Panel, and the following individuals: Michael Gannon, Keith Grubb, Eric Bolin, Michael Desch, and Matt Smith and Duff Zimmerman, who both provided AISC Board Oversight.

SCOPE

The specification requirements and other design recommendations and considerations summarized in this Manual apply in general to the design and construction of seismic force-resisting systems in steel buildings and other structures. The AISC *Seismic Design Manual* is intended to be applied in conjunction with the AISC *Steel Construction Manual* (AISC, 2023), which provides guidance on the use of the AISC *Specification for Structural Steel Buildings* (AISC, 2022a).

In addition to the requirements of the AISC *Specification*, the design of seismic force-resisting systems must meet the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* (AISC 2022b), except in the following cases for which use of the AISC *Seismic Provisions* is not required:

- Buildings and other structures in Seismic Design Category (SDC) A
- Buildings and other structures in SDC B or C with $R = 3$ systems [steel systems not specifically detailed for seismic resistance per ASCE/SEI 7, Table 12.2-1 (ASCE, 2022)]
- Nonbuilding structures similar to buildings with $R = 1½$ braced-frame systems or $R = 1$ moment-frame systems; see ASCE/SEI 7, Table 15.4-1
- Nonbuilding structures not similar to buildings (see ASCE/SEI 7, Table 15.4-2), which are designed to meet the requirements in other standards entirely

Conversely, use of the AISC *Seismic Provisions* is required in the following cases:

- Buildings and other structures in SDC B or C when one of the exemptions for steel seismic force-resisting systems above does not apply
- Buildings and other structures in SDC B or C that use cantilever column systems
- Buildings and other structures in SDC B or C that use composite seismic force-resisting systems (those containing composite steel-and-concrete members and those composed of steel members in combination with reinforced concrete members)
- Buildings in SDC D, E, or F
- Nonbuilding structures in SDC D, E, or F when the exemption above does not apply

The *Seismic Design Manual* consists of nine parts addressing various topics related to the design and construction of seismic force-resisting systems of structural steel and structural steel acting compositely with reinforced concrete. Part 1 stipulates the specific editions of the specifications, codes, and standards referenced in this Manual, and provides a discussion of general design considerations related to seismic design. Part 2 provides some guidance on structural analysis procedures employed. For the design of systems not detailed for seismic resistance, see Part 3. Parts 4 through 7 apply to the various types of seismic force-resisting systems, including design examples. Part 8 discusses other systems, such as diaphragm chords and collectors, that are important in seismic design. For applicable AISC seismic standards, see Part 9.

REFERENCE

AISC (2022a), *Specification for Structural Steel Buildings*, ANSI/AISC 360-22, American Institute of Steel Construction, Chicago, Ill.

AISC (2022b), *Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-22, American Institute of Steel Construction, Chicago, Ill.

AISC (2023), *Steel Construction Manual*, 16th Ed., American Institute of Steel Construction, Chicago, Ill.

ASCE (2022), *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-22, American Society of Civil Engineers, Reston, Va.

PART 1

GENERAL DESIGN CONSIDERATIONS

1.1 SCOPE

The design considerations summarized in this Part apply in general to the design and construction of steel buildings for seismic applications. The specific editions of specifications, codes, and other references listed below are referenced throughout this Manual.

1.2 APPLICABLE SPECIFICATIONS, CODES, AND OTHER REFERENCES

Specifications, Codes, and Standards for Structural Steel Buildings

Subject to the requirements in the applicable building code and the contract documents, the design, fabrication, and erection of structural steel buildings is governed as indicated in AISC *Specification* Sections A1 and B2 and AISC *Seismic Provisions* Sections A2 and B2 as follows:

1. ASCE/SEI 7: *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, ASCE/SEI 7-22. Available from the American Society of Civil Engineers, ASCE/SEI 7 provides the general requirements for loads, load factors, and load combinations (ASCE, 2022).
2. AISC *Specification: Specification for Structural Steel Buildings*, ANSI/AISC 360-22. This standard provides the general requirements for design and construction of structural steel buildings, is included in Part 16 of the AISC *Steel Construction Manual*, and is also available at **www.aisc.org** (AISC, 2022a).
3. AISC *Seismic Provisions: Seismic Provisions for Structural Steel Buildings*, ANSI/AISC 341-22. This standard provides the design and construction requirements for seismic force-resisting systems in structural steel buildings, is included in Part 9 of this Manual, and is also available at **www.aisc.org** (AISC, 2022b).
4. ANSI/AISC 358: *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*, ANSI/AISC 358-22. This standard specifies design, detailing, fabrication, and quality criteria for connections that are prequalified in accordance with the AISC *Seismic Provisions* for use with special and intermediate moment frames. It is included in Part 9 of this Manual and is also available at **www.aisc.org** (AISC, 2022c).
5. ANSI/AISC 342: *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*, ANSI/AISC 342-22. This standard provides design criteria in conjunction with ASCE/SEI 41 for the retrofit of existing steel structures subjected to seismic loads and is available at **www.aisc.org** (AISC, 2022d).
6. AISC *Code of Standard Practice: Code of Standard Practice for Steel Buildings and Bridges*, ANSI/AISC 303-22. This document provides the standard of custom and usage for the fabrication and erection of structural steel, is included in Part 16 of the AISC *Steel Construction Manual*, and is also available at **www.aisc.org** (AISC, 2022e).

Other referenced standards include:

1. RCSC *Specification: Specification for Structural Joints Using High-Strength Bolts*, reprinted in Part 16 of the AISC *Steel Construction Manual* with the permission of the Research Council on Structural Connections and available at **www.boltcouncil.org**, provides the additional requirements specific to bolted joints with high-strength bolts (RCSC, 2020).
2. AWS D1.1/D1.1M: *Structural Welding Code—Steel*, AWS D1.1/D1.1M:2020 (AWS, 2020b). Available from the American Welding Society, AWS D1.1/D1.1M provides additional requirements specific to welded joints. Requirements for the proper specification of welds can be found in AWS A2.4: *Standard Symbols for Welding, Brazing, and Nondestructive Examination* (AWS, 2020a).
3. AWS D1.8/D1.8M: *Structural Welding Code—Seismic Supplement*, AWS D1.8/D1.8M:2021. Available from the American Welding Society, AWS D1.8/D1.8M acts as a supplement to AWS D1.1/D1.1M and provides additional requirements specific to welded joints in seismic applications (AWS, 2021).
4. ACI 318: *Building Code Requirements for Structural Concrete and Commentary*, ACI 318-19. Available from the American Concrete Institute, ACI 318 provides additional requirements for reinforced concrete, including composite design and the design of steel-to-concrete anchorage (ACI, 2019).

Other AISC Reference Documents

The AISC *Steel Construction Manual* (AISC, 2023), referred to as the AISC *Manual*, is available from AISC at **www.aisc.org**. This publication provides design recommendations and specification requirements for various topics related to steel building design and construction.

1.3 SEISMIC DESIGN OVERVIEW AND DESIGN CONSIDERATIONS

Performance Goals

Seismic design is the practice of proportioning and detailing a structure so that it can withstand shaking from an earthquake event with acceptable performance. The AISC *Seismic Provisions for Structural Steel Buildings* are intended to provide a means of designing structures constructed to respond to maximum considered earthquake ground shaking, as defined in ASCE/SEI 7, with low probability of collapse, while potentially sustaining significant inelastic behavior and structural damage. Fundamental to seismic design is the practice of proportioning and detailing the structure so that it can withstand large deformation demands, accommodated through inelastic behavior of structural elements that have been specifically designed to withstand this behavior acceptably. This requires careful proportioning of the structural system so that inelastic behavior occurs in pre-selected elements that have appropriate section properties to sustain large inelastic deformation demands without loss of strength, and ensuring that connections of structural elements are adequate to develop the required strength of the connected members.

Performance appropriate to the function of the structure is a fundamental consideration for the seismic design. Potential considerations are post-earthquake reparability and serviceability for earthquakes of different severity. Most structures are designed only with an expectation of collapse prevention to minimize risk to life when subject to a maximum considered earthquake, rather than ensuring either the feasibility of repair or post-earthquake utility. Buildings assigned to Risk Categories III and IV, as defined in ASCE/SEI 7, are expected to withstand severe earthquakes with limited levels of damage, and in some cases, allow post-earthquake occupancy. The criteria of the AISC *Seismic Provisions*, when used together with the requirements of ASCE/SEI 7, are intended to provide performance appropriate to the structure's risk category[1]. For some buildings, performance that exceeds these expectations may be appropriate. In those cases, designers must develop supplementary criteria to those contained in the AISC *Seismic Provisions* and ASCE/SEI 7.

Building performance is not a function of the structural system alone. Many building structures have exhibited ill effects from damage to nonstructural components, including breaks in fire protection systems and impaired egress, which have precluded building functions and thus impaired performance. Proper consideration of the behavior of nonstructural components is essential to enhanced building performance. Industrial and nonbuilding structures often contain elements that require some measure of protection from large deformations.

Generally, seismic force-resisting systems (SFRS) are classified into three levels of inelastic response capability, designated as ordinary, intermediate, or special, depending on the level of ductility that the system is expected to provide. A system designated as ordinary is designed and detailed to provide limited ability to exhibit inelastic response without failure or collapse. The design requirements for such systems, including limits on proportioning and detailing, are not as stringent as those for systems classified as intermediate or special. Ordinary systems rely on limited ductility and overstrength for collapse prevention when subject to a maximum considered earthquake. Structures such as these must be designed for higher force demands with commensurately less stringent ductility and member stability requirements.

Some steel structures achieve acceptable seismic performance by providing ductility in specific structural elements that are designed to undergo nonlinear deformation without strength loss and to dissipate seismic energy. Examples of ductile steel structures include special moment frames, eccentrically braced frames, and buckling-restrained braced frames. The ability of these structures to deform inelastically, without strength loss or instability, permits them to be designed for lower forces than structures with ordinary detailing.

Enhanced performance, relative to that provided by conformance to the AISC *Seismic Provisions* and ASCE/SEI 7, can be a required consideration for certain nuclear structures and critical military structures, but is beyond the scope of this Manual. Critical structures generally are designed to remain elastic, even for large infrequent seismic events.

Applicable Building Code

National model building codes are published so that state and local authorities may adopt the code's prescriptive provisions to standardize design and construction practices in their

[1]Codes have historically used occupancy category. This classification was changed to risk category in ASCE/SEI 7-10 and IBC 2012. Where classification by occupancy category is still employed, the more stringent of the two is used.

jurisdiction. The currently used model code in the U.S. for the structural design of buildings and nonbuilding structures is the *International Building Code* (ICC, 2024). Often times the adopted provisions are amended based on jurisdictional requirements to develop local building codes (e.g., California Building Code and the Building Code of New York City). Local codes are then enforced by law and any deviation must be approved by the local building authority. As the local code provisions may change between jurisdictions, the AISC *Specification* and AISC *Seismic Provisions* refer to this code as the applicable building code.

The primary performance objective of these model codes is that of "life safety" for building occupants for all the various demands to which the building will be subjected. To satisfy this objective for structures required to resist strong ground motions from earthquakes, these codes reference ASCE/SEI 7 for seismic analysis and design provisions. Seismic design criteria in this standard prescribe minimum requirements for both the strength and stiffness of the SFRS and the structural elements they include. The seismic design criteria in ASCE/SEI 7 for the most part are based on the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA, 2020).

The seismic design of nonbuilding structures is addressed separately in ASCE/SEI 7, Chapter 15. Nonbuilding structures are defined as all self-supporting structures, other than buildings, that carry gravity loads and that may be required to resist the effects of seismic loads, with certain exclusions. Buildings are defined as structures whose intended use includes shelter of human occupants. ASCE/SEI 7 develops an appropriate interface with building structures for those types of nonbuilding structures that have dynamic behaviors similar to buildings. There are other nonbuilding structures that have little similarity to buildings in terms of dynamic response, which are not specifically covered by AISC documents.

Risk Category and Seismic Design Category

In ASCE/SEI 7, the expected performance of a structure is determined by assigning it to a risk category. There are four risk categories (I, II, III, and IV), based on the risk posed to society as a consequence of structural failure or loss of function. In seismic design, the risk category is used in conjunction with parameters that define the intensity of design ground shaking in determining the importance factor and the seismic design category for which a structure must be designed. There are six seismic design categories, designated by the letters A through F. Structures assigned to Seismic Design Category (SDC) A are not anticipated to experience ground shaking of sufficient intensity to cause unacceptable performance, even if they are not specifically designed for seismic resistance. Structures in SDC B or C can experience motion capable of producing unacceptable damage when the structures have not been designed for seismic resistance. Structures in SDC D are expected to experience intense ground shaking capable of producing unacceptable performance in structures that have unfavorable structural systems and that have not been detailed to provide basic levels of inelastic deformation response without failure. Structures assigned to SDC E and F are located within a few miles of major active faults capable of providing large-magnitude earthquakes and ground motions with peak ground accelerations exceeding 0.6g. Even well-designed structures with extensive inelastic response capability can be severely damaged under such conditions, requiring careful selection and proportioning of structures.

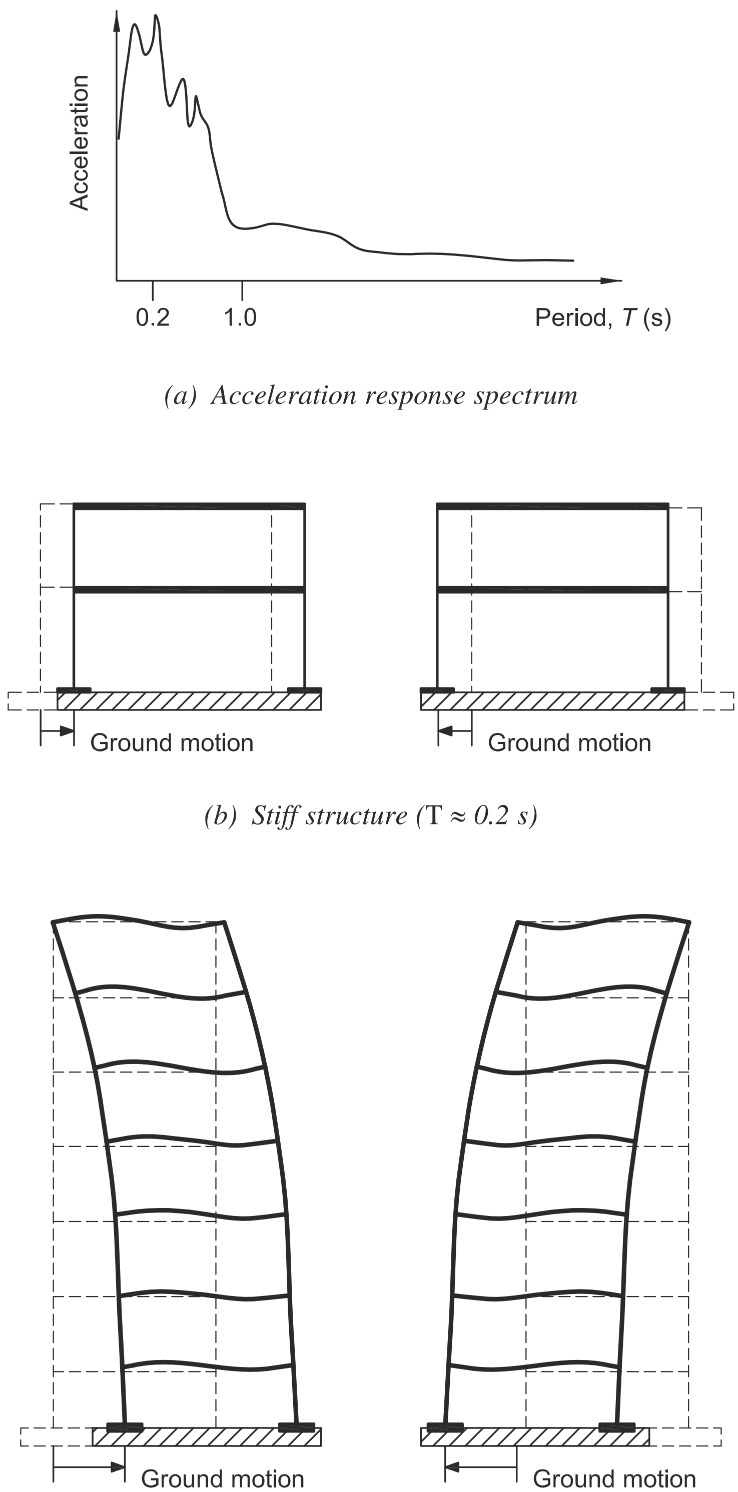
Earthquake Ground Motion and Response Spectrum

An earthquake causes ground motions that may propagate from the hypocenter in any direction. These motions produce horizontal and vertical ground accelerations at the earth's surface, which in turn cause structural accelerations. While it is possible to use earthquake ground motions recorded in past earthquakes to simulate the behavior of structures, the required analysis procedures are complex, and the analysis results are sensitive to the characteristics of the individual ground motions selected, which may not actually be similar to those a structure will experience in the future. To simplify the uncertainty and complexity associated with using recorded motions to predict a structure's response, response earthquake spectra are used. A response spectrum for a given earthquake ground motion indicates the maximum (absolute value), expressed either as acceleration, velocity, or displacement, that an elastic single-degree-of-freedom (SDOF) oscillator will experience as a function of the structure's period and equivalent damping factor. Figure 1-1(a) shows an example of an acceleration response spectrum. On average, low-rise buildings [Figure 1-1(b)] tend to have short periods, while tall structures tend to be flexible with longer periods [Figure 1-1(c)]. For a given ground motion, short period structures tend to experience higher acceleration, and therefore, higher inertial force (mass times acceleration), than do longer period structures. However, long period structures generally experience greater displacement.

Multi-story buildings are multi-degree-of-freedom systems with multiple modes of vibration. Each mode has a characteristic deflected shape and period. Because earthquake ground motion contains energy caused by vibration across an entire spectrum of frequencies, each frequency that corresponds to a mode imparts energy into the structure. Figure 1-2 shows an example of a five-story building frame and the modal information for the first four modes. Although the mode shapes are shown separately, the actual building motion will consist of combined response in each of the several modes. Using the modal shape of the structure for each mode and the effective percentage of the structure's mass mobilized when vibrating in that mode, it is possible to use the same SDOF response spectrum discussed previously to determine the maximum response for each mode. These maxima are then combined to estimate the total maximum response based on the participation of each mode. These maxima for the various modes will generally occur at different points in time. Modal combination rules approximately account for this effect. Detailed information about structural response using modal analysis can be found in Chopra (2016).

Maximum Considered Earthquake and Design Basis Earthquake

Ground motion hazards in ASCE/SEI 7 are defined as maximum considered earthquake ground motions. They are based on the proximity of the site to active faults, the activity of these faults, projected magnitude of the event these faults can produce, and the regional and local geology at a site. The design intent of ASCE/SEI 7 is to ensure that ordinary occupancy structures (structures assigned to Risk Categories I and II) have no greater than a 10% chance of collapse should they experience maximum considered earthquake shaking. Except for regions located within a few miles of major active faults, such as some sites in coastal California, the maximum considered earthquake is selected with an annual frequency that will provide a uniform collapse risk of 1% probability in 50 years (denoted MCE_R). In regions close to major active faults, the MCE_R is capped by a conservative



(a) Acceleration response spectrum

*(b) Stiff structure (*T ≈ *0.2 s)*

*(c) Flexible structure (*T > *1.0 s)*

Fig. 1-1. Earthquake acceleration and structure response.

deterministic estimate of the ground motion resulting from a maximum magnitude earthquake on the nearby fault, resulting in a higher collapse risk. The MCE_R is represented by a generalized elastic acceleration response spectrum. This response spectrum is subsequently reduced by two-thirds to represent the response for the design basis earthquake for which a structure is designed. Additional information about this reduction can be found in ASCE/SEI 7, Section C11.8.3.

Systems Defined in ASCE/SEI 7

A steel SFRS is generally classified into three levels of expected inelastic response capability, designated as ordinary, intermediate, or special, depending on the level of ductility that the system is expected to provide. Systems designated as ordinary are designed and detailed to provide limited ductility, and the requirements are not as stringent as those systems classified as intermediate or special. In some cases, an SFRS can be classified as a "structure not specifically detailed for seismic resistance" in accordance with the applicable building code. Each classification is characterized by the following seismic performance factors:

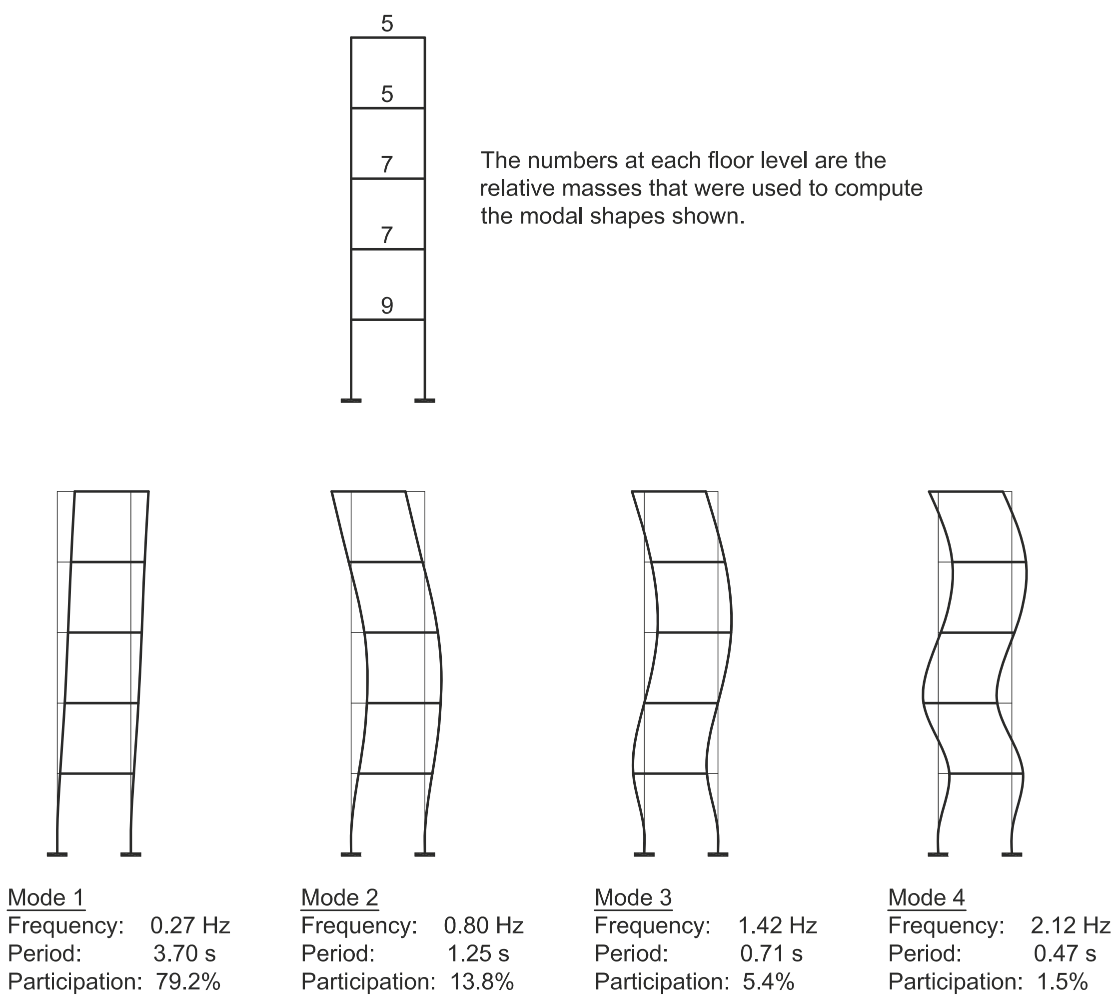


Fig. 1-2. Vibration modes for a multi-degree-of-freedom building caused by application of a typical earthquake acceleration design spectrum.

- Response modification coefficient, R
- Overstrength factor, Ω_0
- Deflection amplification factor, C_d

When used in combination, these factors quantitatively outline the expected performance of an SFRS. Other factors that influence the performance are the importance factor, I_e, and redundancy factor, ρ. These factors are discussed in the following.

Designing to meet the requirements of the AISC *Seismic Provisions* is mandatory for structures where they have been specifically referenced in ASCE/SEI 7, Table 12.2-1. For steel structures, typically this occurs in SDC D and higher where R is greater than 3. However, there are instances where an R less than 3 is assigned to a system and the AISC *Seismic Provisions* are still required. See the Scope section at the front of this Manual for additional discussion.

Systems where R is greater than 3 are intended for buildings that are designed to meet the requirements of both the AISC *Seismic Provisions* and the AISC *Specification*. The use of R greater than 3 in the calculation of the seismic base shear requires the use of a seismically designed and detailed system that is able to provide the level of ductility commensurate with the value of R selected in the design. This level of ductility is achieved through a combination of proper material and section selection, the use of low width-to-thickness members for the energy dissipating elements of the SFRS, detailing member connections to resist forces and deformations associated with the inelastic capacity of the system, and providing for system lateral stability at the large deformations expected in a major earthquake. Consider the following three examples:

1. Special concentrically braced frame (SCBF) systems—As shown in Figure 1-3, SCBF systems are generally configured so that energy dissipation will occur by tension yielding and/or compression buckling in the braces. The surrounding columns, beams, and associated connections between these elements must then be proportioned to remain essentially elastic as they undergo these deformations.

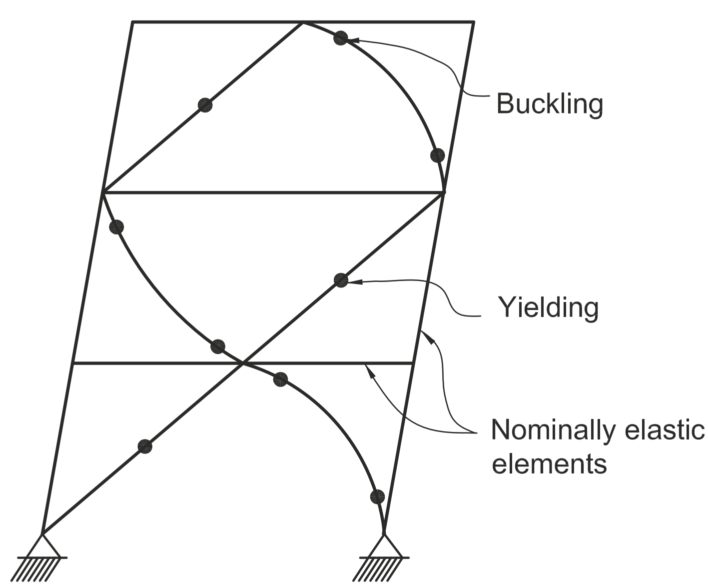


Fig. 1-3. Ductile braced frames.

2. Eccentrically braced frame (EBF) systems—As shown in Figure 1-4, EBF systems are generally configured so that energy dissipation will occur by shear and/or flexural yielding in the link. The beam outside the link, connections, braces, and columns must then be proportioned to remain essentially elastic as the link is subject to inelastic deformations.
3. Special moment frame (SMF) systems—As shown in Figure 1-5, SMF systems are generally configured so that energy dissipation will occur by flexural yielding in the girders near, but outside of, the connection of the girders to the columns. The connections of the girders to the columns and the columns themselves must then be proportioned to remain essentially elastic as the girders are subject to inelastic deformations.

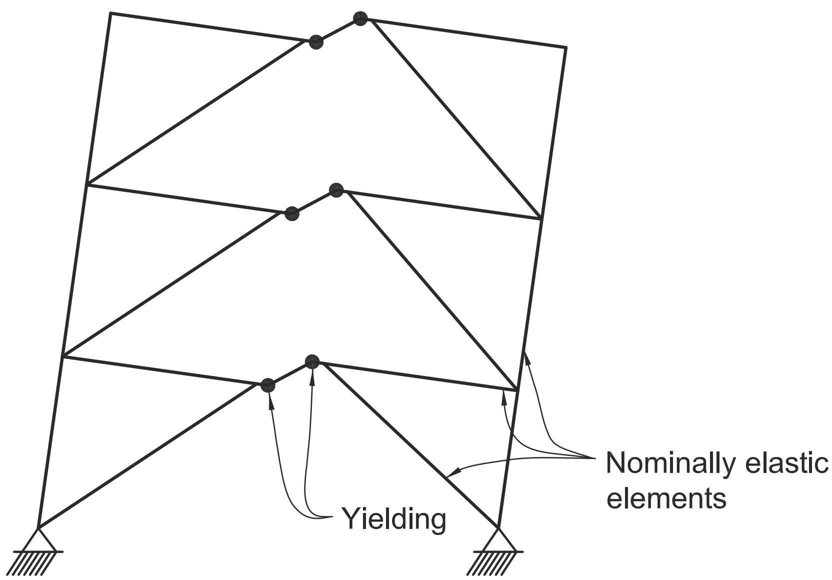


Fig. 1-4. Ductile eccentrically braced frames.

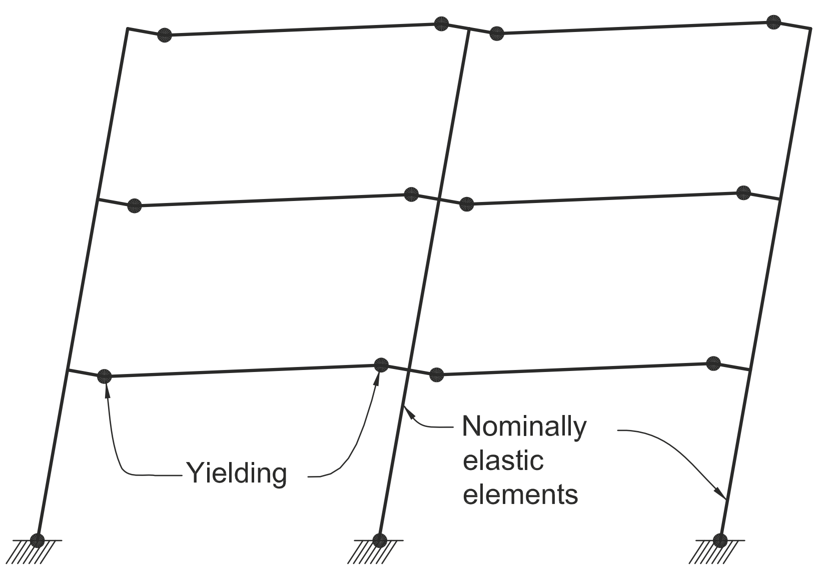


Fig. 1-5. Ductile moment frames.

Seismic Performance Factors

Response Modification Coefficient, R

The seismic design category is used, along with the SFRS type, to establish a minimum level of inelastic, ductile response that is required of a structure. The corresponding expected system behavior is codified in the form of an *R* factor, which is a response modification coefficient applied to the lateral force to adjust a structure's required lateral strength considering its inelastic response capability.

The response modification coefficient, *R*, accounts for ductility and overstrength in the SFRS. This factor is positioned in the denominator of the equation used to calculate the seismic base shear for the structure and, therefore, higher *R* values correspond to reduced seismic design forces. These seismic design forces are used with an elastic design model and, as such, are intended to acknowledge the benefit of ductility and overstrength with regard to the overall resistance of the SFRS. Structures designed with a large value of *R* must have extensive capability to withstand large inelastic deformation demands during design level shaking. Structures designed with an *R* approximating 1.0 are anticipated to experience design shaking while remaining essentially elastic. Figure 1-6 shows the relationship between *R* and the design-level forces, along with the corresponding lateral deformation of the structural system (FEMA, 2020).

Factors that determine the magnitude of the response modification coefficient are the vulnerability of the gravity load-resisting system to a failure of elements in the SFRS, the level and reliability of the inelastic deformation the system can attain, and potential backup frame resistance such as that which is provided by dual-frame systems. As illustrated in Figure 1-6, in order for a system to utilize a higher value of *R*, other elements of the system must have adequate strength and deformation capacity to remain stable at the maximum lateral deformation levels. If the system redundancy and system overstrength cannot be achieved, a lower value of *R* should be incorporated in the design and detailing of the structure. Values of *R* for all structural systems are defined in ASCE/SEI 7, Table 12.2-1. Tables 1-9a and

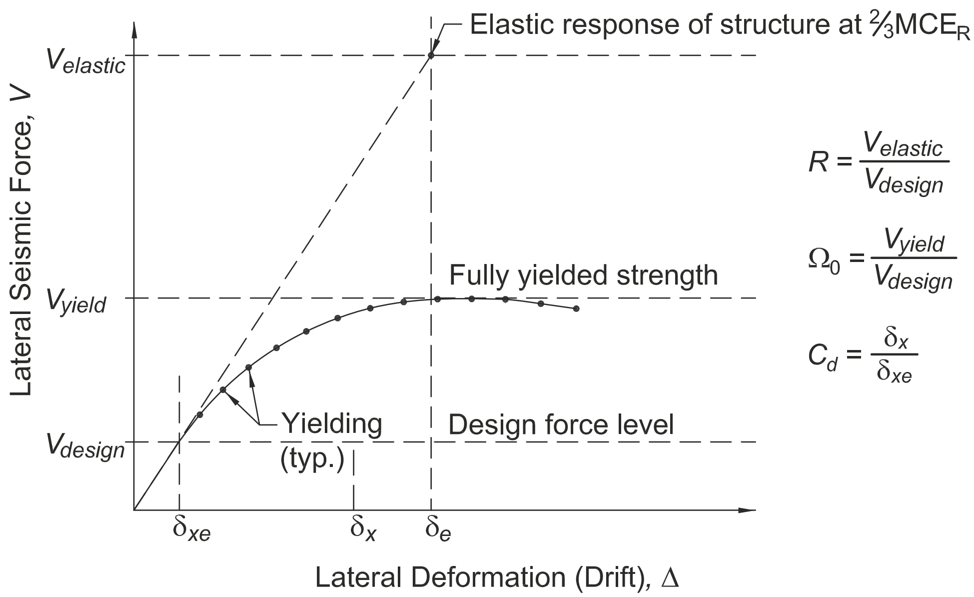


Fig. 1-6. Relationship between R*, design level forces, and lateral deformation.*

1-9b in this Part summarize the R factors and other factors specified in ASCE/SEI 7 for steel and composite systems. More detailed discussion on the system design parameters can be found in FEMA (2020).

R = *3 Applications*

For structures assigned to SDC B and C in ASCE/SEI 7, the designer may choose to solely use the AISC *Specification* to design and detail the structure. The resulting systems (assigned an R of 3) have ductility associated with conventional steel framing not specifically detailed for high seismic resistance. It is important to note that even steel structures not specifically designed or detailed for seismic resistance possess some inherent amount of seismic resistance, which may be adequate to resist a limited amount of seismic demand.

It is recognized that when the designer has the option to design a building to meet the AISC *Specification* with $R = 3$, such a design will generally be more cost effective than the same structure designed in accordance with the AISC *Seismic Provisions* using a higher value of R. The extra fabrication, erection, and inspection required to achieve the high ductility commensurate with the higher R value can cost more than the additional steel tonnage required by the $R = 3$ system.

The $R = 3$ option is not generally available for composite steel-concrete systems. For composite systems, the designer must follow the requirements outlined in ASCE/SEI 7, Table 12.2-1.

Deflection Amplification Factor, C_d

The elastic story drifts calculated under reduced lateral forces are multiplied by the deflection amplification factor, C_d, to better estimate the total story drifts likely to result from the design earthquake ground motion. These amplified story drifts are used to verify compliance with the allowable story drift, to investigate separation requirements between adjacent structures, and to determine seismic demands on elements of the structure that are not part of the SFRS and on nonstructural components within the structure.

Overstrength Seismic Load and Capacity-Limited Seismic Load Effect

Most SFRS rely on dissipation of earthquake energy through varying levels of inelastic response in the structure. Steel seismic system definitions in the AISC *Seismic Provisions* designate the elements intended to dissipate the majority of this energy through ductile inelastic response and those that are intended to remain essentially elastic. Overstrength seismic loads, E_{mh}, are prescribed for certain load combinations in ASCE/SEI 7 and in the AISC *Seismic Provisions* for the design of those elements of the SFRS that are intended to remain essentially elastic. Overstrength seismic loads incorporate an amplification (overstrength) factor, Ω_0, that is prescribed by ASCE/SEI 7 for each given system. ASCE/SEI 7 and the AISC *Seismic Provisions* introduced a new term, the capacity-limited seismic load, E_{cl}, which defines the lateral seismic load level associated with the maximum expected capacity of the designated yielding elements for the system. ASCE/SEI 7 provides specific direction as to when each of these elevated seismic force levels are to be considered. The capacity-limited seismic load, E_{cl}, represents an upper bound for the horizontal seismic loads on the SFRS and, therefore, E_{mh} need not exceed E_{cl}. These special seismic load combinations, involving either E_{cl} or E_{mh}, are invoked for members or connections whose inelastic behavior may cause poor system performance. Failure of these elements could lead to unacceptable behavior, and they are, therefore, protected against large inelastic demands by application of the overstrength factor.

Members and connections requiring the special seismic load combinations including overstrength or the capacity-limited horizontal seismic load effect in ASCE/SEI 7 include the following (the applicable section of ASCE/SEI 7 is provided in parentheses):

1. Elements supporting discontinuous walls or frames (Section 12.3.3.4)
2. Collectors for structures in SDC C through F (Section 12.10.2.1)
3. Batter piles (Section 12.13.8.4)
4. Pile anchorage (Section 12.13.8.5)
5. Pile splices (Section 12.13.8.6)

In the AISC *Seismic Provisions*, the application of the overstrength factor, Ω_0, is addressed using the term overstrength seismic load. The overstrength seismic load refers to the use of the ASCE/SEI 7 load combinations that include Ω_0. When overstrength seismic load is specified, it is acceptable for E_{mh} to either be based on the overstrength factor, Ω_0, or be equal to the capacity-limited seismic load, E_{cl}. For some situations, the capacity-limited seismic load must be used, in which case the capacity-limited horizontal seismic load effect, E_{cl}, is substituted for E_{mh} in the special seismic load combinations in ASCE/SEI 7. See AISC *Seismic Provisions* Section B2 for more information.

Sections of the AISC *Seismic Provisions* where it is permissible to apply either the overstrength seismic load or the capacity-limited seismic load for the design of certain elements or connections include:

Section D1.4a—Required compressive and tensile strength of columns
Section D1.6—Required strength of connections between components of built-up members
Section D2.5b—Required strength of column splices
Section D2.6a—Required axial strength of column bases
Section D2.6b—Required shear strength of column bases
Section D2.6c—Required flexural strength of column bases
Sections E3.4a and G3.4a—Moment ratio check for special moment frames and composite special moment frames (also referred to as the strong-column/weak-beam calculation)
Sections E3.4c and G3.4c—Required column strength at unbraced beam-to-column connections for special moment frames and composite special moment frames
Section E5.4a—Required strength of columns in ordinary cantilever column systems
Section E6.4a—Required strength of columns in special cantilever column systems
Section F1.2—Determination of eccentric moments in members for ordinary concentrically braced frames, if an eccentricity is present
Section F1.4a—Required strength of beams in V-braced and inverted V-braced ordinary concentrically braced frames
Section F1.4c—Required strength of brace connections, struts, and columns in multi-tiered ordinary concentrically braced frames
Section F1.5c—Required strength of beams and their connections in ordinary concentrically braced frames
Section F1.6a—Required strength of diagonal brace connections in ordinary concentrically braced frames

Section F2.4a—Required strength of compression braces in special concentrically braced frames when the exception to the lateral force distribution requirement is used
Section F2.6b—Required strength of diaphragm collector forces in special concentrically braced frames
Section F2.6c—Required strength for the limit state of bolt slip in brace connections with oversized holes in special concentrically braced frames
Section F3.6b—Required strength of diaphragm collector forces in eccentrically braced frames
Section F3.6c—Required strength for the limit state of bolt slip in brace connections with oversized holes
Section F4.4c—Required strength of braces in buckling-restrained braced frames when the exception to the lateral force distribution requirement is used
Section F4.6b—Required strength of diaphragm collector forces in buckling-restrained braced frames
Section H2.6b—Required strength of diaphragm collector forces in composite special concentrically braced frames
Section H3.6a—Required strength of diaphragm collector forces in composite eccentrically braced frames

Sections of the AISC *Seismic Provisions* where the application of the capacity-limited seismic load for the design of certain elements or connections is required:

Section E1.6b—Required shear strength of beam-to-column connections in ordinary moment frames
Section E1.7a—Required strength for truss members and connections in OMF composed of structural steel trusses and structural steel columns
Sections E2.6d and G2.6d—Required shear strength of beam-to-column connections in intermediate moment frames and composite intermediate moment frames
Section E3.6d and G3.6d—Required shear strength of beam-to-column connections in special moment frames and composite special moment frames
Section E4.3b—Required strength of nonspecial segment members and connections in special truss moment frames
Section F1.4c—Required strength of multi-tiered ordinary braced frame columns when the exception to the typical requirements for tension-only bracing is used
Section F2.3—Required strength of columns, beams, struts, and connections in special concentrically braced frames
Sections F3.3—Required strength of diagonal braces and their connections, beams outside links, and columns in eccentrically braced frames
Sections F4.3—Required strength of columns, beams, struts, and connections in buckling-restrained braced frames
Sections F5.3 and F5.6b—Required strength of horizontal and vertical boundary elements and connections in special plate shear walls
Section H8.3c—Required strength of composite walls in CC-PSW/CF
Section H8.8b—Required shear strength for the coupling beam-to-wall connection in CC-PSW/CF
Section H8.9a—Required strength for the composite wall-to-foundation connections in CC-PSW/CF

See the applicable sections of the AISC *Seismic Provisions* for specific requirements.

Redundancy Factor, ρ

Redundancy is an important property for structures designed with the expectation that damage will occur. Redundant structures have alternative load paths so that if some elements are severely damaged and lose load carrying capacity, other elements and load paths will be able to continue to provide necessary resistance. Adequate redundancy is ensured when a large number of elements are expected to yield or buckle throughout the structure in a progressive manner before formation of a collapse mechanism occurs and when no one element is required to provide the full seismic resistance of the structure. To encourage provision of a minimum level of redundancy in the structure, ASCI/SEI 7, Section 12.3.4, stipulates a redundancy factor, ρ, based on the structure's configuration and the number of independent seismic force-resisting elements present. When structures do not satisfy minimum criteria, this factor amplifies the required strength of the lateral system. The elastic analysis of the SFRS is performed using the total design lateral force, V, based on the tabulated value of R, and ρ is applied to the resulting Q_E member force effects, where Q_E is the effect of horizontal seismic forces.

Maximum Force Delivered by the System

The maximum force delivered by the system is a concept used in several applications in the practice of seismic design. The maximum force delivered by the system is often one of the limits for required strength of a seismic force-resisting element. For example, a thorough discussion of how this force may be determined for SCBF brace connections is contained in the AISC *Seismic Provisions* Commentary Section F2.6c.

Load Effects and Load Combinations

The vertical seismic load effect, E_v, from ASCE/SEI 7, Section 12.4.2.2, is:

$$E_v = 0.2S_{DS}D \qquad \text{(ASCE/SEI 7, Eq. 12.4-4a)}$$

The horizontal seismic load effect, E_h, from ASCE/SEI 7, Section 12.4.2.1, is:

$$E_h = \rho Q_E \qquad \text{(ASCE/SEI 7, Eq. 12.4-3)}$$

The horizontal seismic load effect including overstrength, E_{mh}, from ASCE/SEI 7, Section 12.4.3.1, is:

$$E_{mh} = \Omega_0 Q_E \qquad \text{(ASCE/SEI 7, Eq. 12.4-7)}$$

The basic load combinations with seismic load effects from ASCE/SEI 7, Section 2.3.6 (for LRFD) and Section 2.4.5 (for ASD), are used, with E_v and E_h as defined in Section 12.4.2.

LRFD	ASD
Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on L): $1.2D + E_v + E_h + 0.5L + 0.15S$ $= 1.2D + 0.2S_{DS}D + \rho Q_E + 0.5L + 0.15S$ $= (1.2 + 0.2S_{DS})D + \rho Q_E + 0.5L + 0.15S$	Load Combination 8 from ASCE/SEI 7, Section 2.4.5: $1.0D + 0.7E_v + 0.7E_h$ $= 1.0D + 0.7(0.2S_{DS}D) + 0.7\rho Q_E$ $= (1.0 + 0.14S_{DS})D + 0.7\rho Q_E$

LRFD	ASD
Load Combination 7 from ASCE/SEI 7, Section 2.3.6: $0.9D - E_v + E_h$ $= 0.9D - 0.2S_{DS}D + \rho Q_E$ $= (0.9 - 0.2S_{DS})D + \rho Q_E$	Load Combination 9 from ASCE/SEI 7, Section 2.4.5: $1.0D + 0.525E_v + 0.525E_h + 0.75L + 0.1S$ $= 1.0D + 0.525(0.2S_{DS}D) + 0.525\rho Q_E$ $+ 0.75L + 0.1S$ $= (1.0 + 0.105S_{DS})D + 0.525\rho Q_E$ $+ 0.75L + 0.1S$ Load Combination 10 from ASCE/SEI 7, Section 2.4.5: $0.6D - 0.7E_v + 0.7E_h$ $= 0.6D - 0.7(0.2S_{DS}D) + 0.7\rho Q_E$ $= (0.6 - 0.14S_{DS})D + 0.7\rho Q_E$

The basic load combinations with seismic load effects including overstrength from ASCE/SEI 7, Section 2.3.6 (for LRFD) and Section 2.4.5 (for ASD), are used, with E_v and E_{mh} as defined in Section 12.4.3.

LRFD	ASD
Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on L): $1.2D + E_v + E_{mh} + 0.5L + 0.15S$ $= 1.2D + 0.2S_{DS}D + \Omega_0 Q_E + 0.5L + 0.15S$ $= (1.2 + 0.2S_{DS})D + \Omega_0 Q_E + 0.5L + 0.15S$	Load Combination 8 from ASCE/SEI 7, Section 2.4.5: $1.0D + 0.7E_v + 0.7E_{mh}$ $= 1.0D + 0.7(0.2S_{DS}D) + 0.7\Omega_0 Q_E$ $= (1.0 + 0.14S_{DS})D + 0.7\Omega_0 Q_E$
Load Combination 7 from ASCE/SEI 7, Section 2.3.6: $0.9D - E_v + E_{mh}$ $= 0.9D - 0.2S_{DS}D + \Omega_0 Q_E$ $= (0.9 - 0.2S_{DS})D + \Omega_0 Q_E$	Load Combination 9 from ASCE/SEI 7, Section 2.4.5: $1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.1S$ $= 1.0D + 0.525(0.2S_{DS}D) + 0.525\Omega_0 Q_E$ $+ 0.75L + 0.1S$ $= (1.0 + 0.105S_{DS})D + 0.525\Omega_0 Q_E$ $+ 0.75L + 0.1S$ Load Combination 10 from ASCE/SEI 7, Section 2.4.5: $0.6D - 0.7E_v + 0.7E_{mh}$ $= 0.6D - 0.7(0.2S_{DS}D) + 0.7\Omega_0 Q_E$ $= (0.6 - 0.14S_{DS})D + 0.7\Omega_0 Q_E$

Where the capacity-limited seismic load effect, E_{cl}, is considered, E_{mh} need not be larger than E_{cl}. Where the capacity-limited seismic load is required, it is intended that E_{cl} replace E_{mh}. The basic load combinations using E_{cl} in place of E_{mh} are:

LRFD	ASD
Load Combination 6 from ASCE/SEI 7, Section 2.3.6 (including the permitted 0.5 factor on L): $1.2D + E_v + E_{mh} + 0.5L + 0.15S$ $= 1.2D + 0.2S_{DS}D + E_{cl} + 0.5L + 0.15S$ $= \left(1.2 + 0.2S_{DS}\right)D + E_{cl} + 0.5L + 0.15S$	Load Combination 8 from ASCE/SEI 7, Section 2.4.5: $1.0D + 0.7E_v + 0.7E_{mh}$ $= 1.0D + 0.7\left(0.2S_{DS}D\right) + 0.7E_{cl}$ $= \left(1.0 + 0.14S_{DS}\right)D + 0.7E_{cl}$
Load Combination 7 from ASCE/SEI 7, Section 2.3.6: $0.9D - E_v + E_{mh}$ $= 0.9D - 0.2S_{DS}D + E_{cl}$ $= \left(0.9 - 0.2S_{DS}\right)D + E_{cl}$	Load Combination 9 from ASCE/SEI 7, Section 2.4.5: $1.0D + 0.525E_v + 0.525E_{mh} + 0.75L + 0.1S$ $= 1.0D + 0.525\left(0.2S_{DS}D\right) + 0.525E_{cl}$ $+ 0.75L + 0.1S$ $= \left(1.0 + 0.105S_{DS}\right)D + 0.525E_{cl}$ $+ 0.75L + 0.1S$
	Load Combination 10 from ASCE/SEI 7, Section 2.4.5: $0.6D - 0.7E_v + 0.7E_{mh}$ $= 0.6D - 0.7\left(0.2S_{DS}D\right) + 0.7E_{cl}$ $= \left(0.6 - 0.14S_{DS}\right)D + 0.7E_{cl}$

Building Joints

Expansion Joints

Expansion joints in a structure are provided to limit the effects of thermal expansion and contraction on the function of the facility and to avoid any resulting damage to structural or architectural components. The number and location of building expansion joints is a design issue not fully treated in technical literature.

- The AISC *Specification* considers expansion joints a serviceability issue, and Section L6 states that "The effects of thermal expansion and contraction of a building shall be considered."
- ASCE/SEI 7 also considers expansion joints a serviceability issue indicating in Section 1.3.2 that "Structural systems, and members thereof, shall be designed under service loads to have adequate stiffness to limit deflections, lateral drift, vibration, or any other deformations that adversely affect the intended use and performance of buildings and other structures based on requirements set forth in the applicable codes and standards, or as specified in the project design criteria."

Typical locations of expansion joints include:

- Separating wings of L-, U-, and T-shaped buildings
- At additions to existing buildings
- At locations where interior heating conditions change, such as where heated offices abut an unheated warehouse
- To break very long structures into shorter structures

The width of an expansion joint is determined from the basic thermal expansion expression for the material used for the structural frame:

$$\Delta_L = \alpha L \Delta_T \tag{1-1}$$

where

L = length subject to the temperature change, in.
Δ_L = change in length, in.
Δ_T = design temperature change, °F
α = $6.5 \times 10^{-6}/°\text{F}$, coefficient of linear thermal expansion for steel structures

See AISC *Manual* Table 17-11 for additional information on coefficients of expansion.

Seismic Joints

Seismic joints are similar in form to expansion joints but are the result of very different structural considerations. They must accommodate movement in both orthogonal directions simultaneously, and their spacing is not typically affected by building length or size. Seismic joints are used to separate an irregular structure into multiple regular structures in an effort to provide better seismic performance of the overall building.

The design of seismic joints is complex and includes efforts by all members of the design team to ensure that the joint is properly sized, adequately sealed from weather, and safe to walk on, as well as to provide for adequate movement of other systems crossing the joint and means to maintain the fire ratings of the floor, roof, and wall systems. Seismic joints are costly and architecturally undesirable, so they should be incorporated with discretion.

When seismic joints are determined to be necessary or desirable for a particular building, the locations of the joints are often obvious and inherent. Many of the locations appropriate for expansion joints are also appropriate for seismic joints. Requirements for determining the seismic separation between buildings are prescribed in ASCE/SEI 7, Section 12.12.

The width of seismic joints in modern buildings can vary from just a few inches to several feet, depending on building height and stiffness. Joints in more recent buildings tend to be much wider than their predecessors. This is due to several major factors, the most important of which is changes in the codes. Other contributing factors are the lower lateral stiffness of many modern buildings and the greater recognition by engineers of the magnitude of real lateral deformations induced by an earthquake.

Seismic joints often result in somewhat complicated structural framing conditions. In the simplest of joints, separate columns are placed at either side of the joint to provide the necessary structural support. This is common in parking structures. When double columns are not acceptable, either the structure must be cantilevered from more widely spaced columns or seated connections must be used. In the case of seated connections, there is the temptation to limit the travel of the sliding element, because longer sliding surfaces using Teflon plates or similar devices are costly and the seat element may interfere with other elements of the building. It is strongly recommended that seated connections be designed to allow for movements that exceed those calculated for the design basis earthquake to allow for the effects of greater earthquakes and because the consequences of the structure falling off of the seat may be disastrous. Where this is not possible, restraint cables such as those often used on bridges should be considered.

Building Separations

Separations between adjacent buildings that are constructed at different times, have different ownership, or are otherwise not compatible with each other may be necessary and unavoidable if both buildings are located at or near the common property line. ASCE/SEI 7, Section 12.12.2, prescribes required setbacks for buildings from property lines. An exception can be made where justified by rational analysis based on inelastic response to design ground motions.

Building Drift

Story drift is the maximum lateral displacement within a story (i.e., the displacement of one floor relative to the floor below caused by the effects of seismic loads). Buildings subjected to earthquakes need drift control to limit damage to fragile nonstructural elements and to limit second-order effects on the overall strength and stability of the structure. It is expected that the design of moment-resisting frames and the design of tall, narrow shear-wall or braced-frame buildings will be governed at least in part by drift considerations.

The allowable story drift limits are defined in ASCE/SEI 7, Table 12.12-1, and are a function of the seismic lateral force-resisting system and the building risk category. The prescribed story drift limits are applicable to each story. They must not be exceeded in any story, even though the drift in other stories may be well below the limit.

Deflection Compatibility

ASCE/SEI 7, Section 12.12.4, prescribes requirements for deformation compatibility for SDC D through F to ensure that the SFRS provides adequate deformation control to protect elements of the structure that are not part of the SFRS. This is intended to ensure that these components and the support connections for these components are detailed to accommodate the expected movement due to story drift, while still supporting the gravity loads.

Lowest Anticipated Service Temperature

Most structural steels can fracture either in a ductile or in a brittle manner. The mode of fracture is governed by the material temperature at fracture, the rate at which the loads are applied, and the magnitude of the constraints that would prevent plastic deformation. Fracture toughness is a measure of the energy required to cause an element to fracture; the more energy that is required, the tougher the material (i.e., it takes more energy to fracture a ductile material than a brittle material). Additionally, lower temperatures have an adverse impact on material ductility. Fracture toughness for materials can be established by using fracture-mechanics test methods.

Traditionally, the fracture toughness for structural steels has been primarily characterized by testing Charpy V-notch (CVN) specimens at different temperatures [ASTM E23 (ASTM, 2018)]. The CVN test produces failures at very high strain rates. If testing is carried out over a range of temperatures, the results of energy absorbed versus temperature can be plotted to give an S-curve as shown in Figure 1-7. Usually, three specimens are tested at a given temperature, and the average value is used to construct the S-curve.

Carbon and low-alloy steels exhibit a change in fracture behavior as the temperature falls, with the failure mode changing from ductile to brittle. At high temperatures, the fracture is characterized by pure ductile tearing. At low temperatures, the fracture surface is characterized by cleavage fractures. The decrease in fracture toughness at low temperatures decreases the fracture capacity of the member, resulting in poorer cyclic behavior.

AISC *Seismic Provisions* Commentary Section A3.4 acknowledges that in structures with exposed structural steel, demand critical welds may be subjected to service temperatures less than 50°F on a regular basis. In these cases, the AISC *Seismic Provisions* Commentary suggests that the minimum qualification temperature provided in AWS D1.8/D1.8M Annex A be adjusted such that the test temperature for the CVN toughness qualification tests be no more than 20°F above the lowest anticipated service temperature (LAST).

It is recognized that the LAST is defined differently in different industries. For example, the current AASHTO CVN toughness requirements are specified to avoid brittle fracture in steel bridges above the LAST, which is defined in terms of three temperature zones. In arctic offshore applications, the LAST can be either the minimum design temperature or a selected value below the design temperature, depending upon the consequences of failure.

The AISC *Seismic Provisions* are intended to ensure ductile performance for a low probability earthquake event. The LAST is normally defined to ensure ductile performance for a low probability temperature extreme. The direct combination of two low probability events would be statistically very unlikely. As a result, the definition of LAST need not be excessively restrictive for seismic applications. For purposes of the AISC *Seismic Provisions*, the LAST may be considered to be the lowest one-day mean temperature compiled from National Oceanic and Atmospheric Administration data. For more information, go to **www.noaa.gov** and **www.climate.gov**.

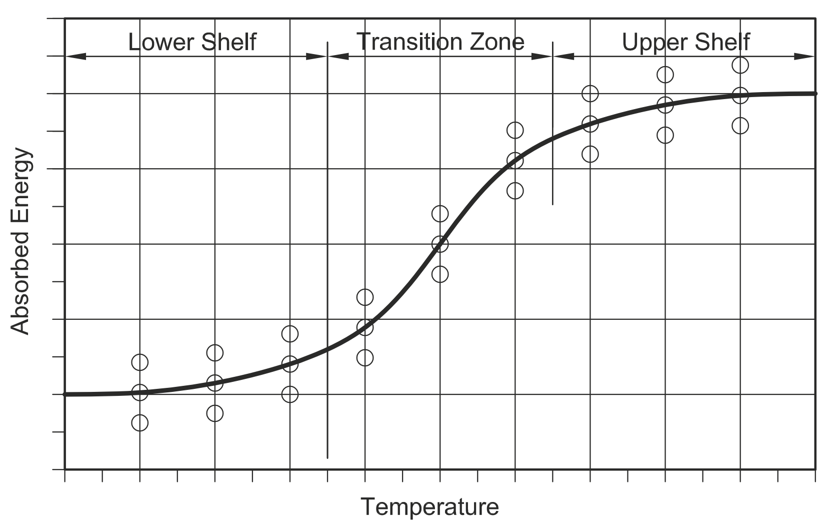


Fig. 1-7. Typical Charpy V-notch test results.

Quality Control and Quality Assurance

The *International Building Code* (ICC, 2024) refers to the 2022 AISC *Specification* and the 2022 AISC *Seismic Provisions* for all quality requirements for structural steel. The scope statement in AISC *Seismic Provisions* Section J1 gives the following explanation for quality control and quality assurance:

> Quality control (QC), as specified in this chapter, shall be provided by the fabricator and erector. Quality assurance (QA), as specified in this chapter, shall be provided by others when required by the authority having jurisdiction (AHJ), applicable building code, purchaser, owner, or engineer of record, and when required, responsibilities shall be specified in the contract documents.

When ductile seismic response should be assured and the AISC *Seismic Provisions* govern the design, fabrication, and erection, steel framing needs to meet special quality requirements as appropriate for the various components of the structure. These requirements, applicable only to members of the SFRS, are provided in:

- ANSI/AISC 341, *Seismic Provisions for Structural Steel Buildings*
- AWS D1.8/D1.8M, *Structural Welding Code—Seismic Supplement*
- ANSI/AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications*
- ANSI/AISC 342, *Seismic Provisions for Evaluation and Retrofit of Existing Structural Steel Buildings*
- 2024 *International Building Code*, Chapter 17 (ICC, 2024)

Additional quality requirements are specified in:

- ANSI/AISC 360, *Specification for Structural Steel Buildings*
- ANSI/AISC 303, *Code of Standard Practice for Steel Buildings and Bridges*
- AWS D1.1/D1.1M, *Structural Welding Code—Steel*
- RCSC *Specification for Structural Joints Using High-Strength Bolts*

The requirements of AISC *Seismic Provisions* Chapter J specify QC and QA special requirements for all responsible parties related to the following:

- Fabricator and erector documents
- Quality assurance agency documents
- Inspection and nondestructive testing personnel
- Inspection tasks
- Welding inspection and nondestructive testing
- Inspection of high-strength bolting
- Other steel structure inspections
- Inspection of composite structures
- Inspection of H-piles

To meet the requirements of the *International Building Code*, as part of the contract documents, the registered design professional in responsible charge must prepare a "statement of special inspections," which is termed the quality assurance plan in the AISC *Seismic Provisions*. The quality assurance plan should be prepared by the engineer of record and made a part of the contract documents. The plan should contain, at a minimum, a written description of qualifications, procedures, quality inspections, resources, and records to be used to provide assurance and supporting documentation that the structure complies with the engineer's quality requirements, specifications, and contract documents. Chapter J of the AISC *Seismic Provisions* provides the minimum acceptable requirements for a quality assurance plan for the SFRS, including requirements for the contract documents, quality assurance agency documents, inspection points, and frequencies, along with special requirements for weld and bolt inspections.

AISC *Seismic Provisions* Chapter J has specific requirements for nondestructive testing of welds, in addition to those in AISC *Specification* Section N5.5, which must be shown on the contract documents. Quality assurance requirements for bolting include verifying that faying surfaces meet the specification requirements and that the bolts are properly tensioned per the RCSC *Specification*.

Design Document Requirements

Structural Design Document Requirements

For systems not requiring seismic detailing, structural design documents are to meet the requirements in AISC *Specification* Section A4 and in the AISC *Code of Standard Practice*. Fabrication and erection documents should follow the design documents to convey specified information for fabrication and erection. For systems designed to meet the AISC *Seismic Provisions*, additional requirements are provided in AISC *Seismic Provisions* Section A4, with supplementary discussion in the *Seismic Provisions* Commentary Section A4. It is important to define all structural elements in the building that resist seismic loads, including struts, collectors, chords, diaphragms, and trusses. Also, the SFRS members should be identified in the design documents. If the SFRS includes other materials, these elements should be defined as such where the steel connects to them.

SFRS Member and Connection Material Specifications

SFRS material requirements are discussed in AISC *Seismic Provisions* Section A3.1 and in the material sections of the various prequalified connections in ANSI/AISC 358. Wide-flange shapes will generally be ASTM A992/A992M material. ASTM A992/A992M has a specified maximum yield stress and a maximum yield-to-tensile ratio to ensure ductility, along with a limit on the carbon equivalent to ensure weldability. Material requirements for the connection elements must be consistent with the prequalified details in ANSI/AISC 358. Bolt material grade, size, location, and tensioning must be shown on the design documents. Bolts often are designed as bearing-type connections with standard holes, and all bolts are required to be pretensioned and have Class A faying surfaces per AISC *Seismic Provisions* Section D2.2(d). AISC *Specification* Section J3.2 stipulates that bolting conforms to the provisions of the RCSC *Specification*, except for where those provisions differ from the AISC *Specification*. AISC *Seismic Provisions* Section D2.3 on welded joints refers to AISC *Specification* Chapter J. AISC *Specification* Section J2 stipulates that welding conforms to the provisions of AWS D1.1/D1.1M, except for where those provisions differ from the AISC *Specification*. AISC *Seismic Provisions* Section A3.4a requires that all welds in the SFRS be made with filler metals meeting the requirements specified in AWS D1.8/D1.8M, clauses 6.1, 6.2, and 6.3.

Demand Critical Welds

In the AISC *Seismic Provisions*, welds are designated as demand critical based on consideration of the inelastic strain demand and the consequence of failure. The location of these demand critical welds is given in the AISC *Seismic Provisions* and in ANSI/AISC 358 in the section applicable to the designated SFRS. As specified in AISC *Seismic Provisions* Section A3.4b, demand critical welds are to be made with filler metals meeting the requirements of AWS D1.8/D1.8M, clauses 6.1, 6.2, and 6.3.

There are a number of other quality control and quality assurance items associated with demand critical welds that are covered in the AISC *Seismic Provisions* and AWS D1.8/D1.8M. Items such as use of backing bars and run-off tabs, including requirements for trimming and finishing of run-off tabs, are specifically addressed.

Locations and Dimensions of Protected Zones

Protected zones are designated by the AISC *Seismic Provisions* for different systems and generally are areas encompassing the plastic hinging region. The FEMA/SAC testing has demonstrated the sensitivity of these areas to fracture caused by discontinuities resulting from welding, penetrations, changes in section, or construction-caused notches (Ricles et al., 2003). Fabrication and erection work, and the subsequent work by other trades, have the potential to cause discontinuities in the SFRS. AISC *Seismic Provisions* Sections D1.3 and I2.1 provide detailed requirements for the protected zone.

The locations and dimensions of these protected zones are specified in the AISC *Seismic Provisions* and in ANSI/AISC 358 for each SFRS. For example, according to AISC *Seismic Provisions* Section F2.5c, the protected zone for special concentrically braced frames includes “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” as well as “elements that connect braces to beams and columns.” For eccentrically braced frames, AISC *Seismic Provisions* Section F3.5c defines the protected zone as the link. In any case, the requirements in AISC *Seismic Provisions* Sections D1.3 and I2.1 must be satisfied.

When located in the protected zone, defects or discontinuities are required to be repaired by the responsible contractor to the satisfaction of the engineer of record. The AISC *Seismic Provisions* require that the protected zones be shown on the design documents. The contractor needs to use this information to control construction activities in this area.

Additional Structural Design Document Detail Requirements in the Provisions

The following are some of the additional requirements from the AISC *Seismic Provisions* that may affect structural design document details:

1. SFRS column splice requirements are given in AISC *Seismic Provisions* Section D2.5a. The splices need to be located away from beam-to-column connections, with the provisions stipulating 4 ft or more away from the connection; however, in general, splices should be in the middle third of the column (see Exceptions in AISC *Seismic Provisions* Section D2.5a). Because of the splice strength requirements in AISC *Seismic Provisions* Section D2.5 and Chapters E and F, it is important that the splice

be fully detailed on the design documents. Where bolted splices are used, there must be plates or channels on both sides of the web.

2. Column splice requirements for columns that are not part of the SFRS are given in the AISC *Seismic Provisions* Section D2.5c. The minimum shear forces required to be developed in these splices will require a special column splice, and this detail should also be shown on the design documents.
3. SFRS column bases must meet the requirements of AISC *Seismic Provisions* Section D2.6 and anchor rod embedment and reinforcing steel should be designed according to ACI 318. Anchor rod sizes and locations, along with washer requirements, hole sizes, and base plate welds, must meet these design requirements and must be shown on the design documents. Special embedment used for base fixity must also be shown on the structural design documents. The Commentary to AISC *Seismic Provisions* Section D2.6 gives a good discussion along with examples of how to develop these forces. For column bases that are not part of the SFRS, some consideration should be given to developing a limited amount of base shear. AISC *Seismic Provisions* Section D2.6b stipulates the required shear strength for column bases, including those not designated as part of the SFRS.
4. Width-to-thickness ratios of SFRS members must be less than those that are resistant to local buckling in order to achieve the required inelastic deformations. While the width-to-thickness ratios given in the AISC *Specification* Tables B4.1a and B4.1b for compact sections are adequate to prevent buckling before the onset of strain hardening, tests have shown that they are not adequate for the required inelastic performance in several SFRS. AISC *Seismic Provisions* Tables D1.1a and D1.1b give the limiting width-to-thickness ratios for moderately ductile and highly ductile members. Classification of members as moderately or highly ductile may govern member size for the various systems.
5. Requirements for stability bracing of beams are provided for each system. The bracing required is stipulated in AISC *Seismic Provisions* Section D1.2 and depends on whether the beam is moderately or highly ductile. Special bracing is required adjacent to plastic hinge locations. If the bracing requirement cannot be met by the floor slab and other normal floor framing elements, then additional bracing members and associated connections should be shown. For example, special moment frame beams require bracing that satisfies the provisions for highly ductile members as given in AISC *Seismic Provisions* Section D1.2b. While the floor slab typically will brace the top flange, additional braces should be shown where required with the necessary connections.

AWS D1.8/D1.8M Structural Welding Code—Seismic Supplement

AWS D1.8/D1.8M, clause 1.4.1, lists the information that the engineer of record is required to provide on the contract documents related to welding of the SFRS. Additionally, gouges and notches are not permitted, and weld contours should provide smooth transitions. AWS D1.8/D1.8M provides recommended details for transitions.

AWS D1.8/D1.8M contains a number of other special requirements that should be specifically referenced in the contract documents. In addition to the filler metal requirements mentioned previously, demand critical welds have the following requirements:

- Manufacturer's certificates of conformance for filler metals
- Special restrictions on care and exposure of electrodes
- Supplemental welder qualification for restricted access welding for bottom flange welding through access holes
- Special weld sequence for bottom flange welding through access holes
- Supplementary requirements for qualification of ultrasonic testing technicians

Composite Systems

The 2022 AISC *Seismic Provisions* for the seismic design of composite structural steel and reinforced concrete buildings are based upon the 1994 *NEHRP Provisions* (FEMA, 1994) and subsequent modifications made in the 1997, 2000, 2003, 2009, 2015, and 2020 *NEHRP Provisions* (FEMA, 2020) and in ASCE/SEI 7. Because composite systems are comprised of integrated steel and concrete components, both the AISC *Specification* and ACI 318 form an important basis for provisions related to composite construction.

There is, at present, limited experience in the U.S. with composite building systems subjected to extreme seismic loads. Extensive design and performance experience with this type of construction in Japan clearly indicates that composite systems, due to their inherent rigidity and toughness, can equal or exceed the performance of buildings comprised of reinforced concrete systems or structural steel systems (Deierlein and Noguchi, 2004; Yamanouchi et al., 1998). Composite systems have been extensively used in tall buildings throughout the world.

Careful attention to all aspects of the design is necessary in the design of composite systems, particularly with respect to the general building layout and detailing of members and connections. Composite connection details are illustrated throughout this Manual to convey the basic character of the force transfer in composite systems. However, these details should not necessarily be treated as design standards. The cited references provide more specific information on the design of composite connections. For a general discussion of these issues and some specific design examples, refer to Viest et al. (1997).

The design and construction of composite elements and systems continues to evolve in practice. Except where explicitly stated, the AISC *Seismic Provisions* are not intended to limit the application of new systems for which testing and analysis demonstrate that the structure has adequate strength, ductility, and toughness. It is generally anticipated that the overall behavior of the composite systems herein will be similar to that for counterpart structural steel systems or reinforced concrete systems and that inelastic deformations will occur in conventional ways, such as flexural yielding of beams in fully restrained moment frames or axial yielding and/or buckling of braces in braced frames.

When systems have both ductile and nonductile elements, the relative stiffness of each should be properly modeled; the ductile elements can deform inelastically, while the nonductile elements remain nominally elastic. When performing an elastic analysis, member stiffness should be reduced to account for the degree of cracking at the onset of significant yielding in the structure. Additionally, it is necessary to account for material overstrength that may alter relative strength and stiffness.

Parts 6 and 7 of this Manual address the design of members and connections for composite moment-frame and braced-frame systems, respectively, as well as guidelines for traversing through the AISC *Seismic Provisions* and AISC *Specification* relative to each specific building system.

Wind and Seismic Design

Members and connections of the main wind force-resisting system are generally not permitted to be subject to large inelastic strains under wind loading. In contrast, designing for seismic effects (including designing with $R = 3$) as discussed in this Part and in Part 2 is based on inelastic behavior in the SFRS, although reduced forces are computed that permit the use of traditional force-based design equations for member selection. Thus, design for wind or seismic effects considers different ranges of structural response but utilizes the same design equations.

It is often advantageous to determine which of the two loadings will govern the required strengths of the elements of the lateral force-resisting system early in the design process. In some cases, a simplified analysis approach is possible for the lower design loads, thus reducing engineering effort without affecting the final design. There are conditions when one loading governs the required strength for the entire lateral force-resisting system, but very often the required strengths for some elements are governed by wind effects and others by seismic effects. In any case, for frames designed with a response modification coefficient, R, corresponding to a system type defined in the AISC *Seismic Provisions*, the proportioning rules and detailing requirements for that system must be followed. This may result in member sizes larger than those required to meet the force demands from wind effects.

For the design of the lateral force-resisting system, comparisons may be made on the basis of force demands for each load. For flexible structures, it is convenient to first select member sizes to control drift for both wind and seismic effects, then to check the strength of elements for both loads, keeping in mind the applicable limitations and proportioning rules for the system as discussed above.

Forces on the lateral force-resisting system must be compared for each member. A comparison of wind base shear to seismic base shear is informative but can be misleading. In general, it is more informative to compare story shears and overturning moments because the lateral force distributions for wind and seismic effects can be very different, with seismic response often inducing larger overturning moments for the same base shear. Additionally, for elements that are required to be designed for the overstrength seismic load or capacity-limited seismic load, such as columns, brace connections, etc., a simple base shear comparison using the basic load combinations is misleading. Similarly, for structures that require a redundancy factor greater than one, the base shear comparison is insufficient because this effect is captured in the member design load combinations. Also, limitations on member slenderness and compactness requirements often require members that are substantially larger than those required for strength demands. The commentary to ASCE/SEI 7 also discusses specific situations and considerations when comparing wind and seismic effects (e.g., ASCE/SEI 7, Section C12.8.2, regarding period determination).

Regardless of which load produces higher story shears and overturning moments, designers must check seismic drift and seismic stability, as well as wind serviceability criteria. Further information on serviceability criteria can be found in AISC *Specification* Chapter L and ASCE/SEI 7, Appendix C.

The design of cladding and other components represents a separate case. These elements must resist forces that are subject to amplification from dynamic effects (both wind and seismic), and a complete analysis for forces on cladding and components for both wind and seismic forces is often necessary. Refer to Parker (2008) for more information.

1.4 IDENTIFICATION OF SFRS ELEMENTS AND SAMPLE CONNECTION DETAILS

Identification of SFRS Elements

As required by AISC *Seismic Provisions* Section A4.1, structural design documents and specifications must include designation of the SFRS and its associated members, including collectors and chords, and their connections. AISC *Seismic Provisions* Chapters A through D contain general requirements related to the SFRS, and AISC *Seismic Provisions* Chapters E through H contain requirements specific to system type. Figure 1-8 shows a typical plan with these elements identified.

Sample Connection Details

Connection design details are to be created based on the requirements of AISC *Seismic Provisions* Section A4.2. AISC *Code of Standard Practice* Section 3.2.3 provides three options for connection design details. Figure 1-9 and the accompanying notes satisfy the appropriate level of detail for Option 1, where the complete connection design is shown in the structural design documents. This figure shows a welded unreinforced flange-welded web (WUF-W) moment connection as an example of a fully developed connection detail.

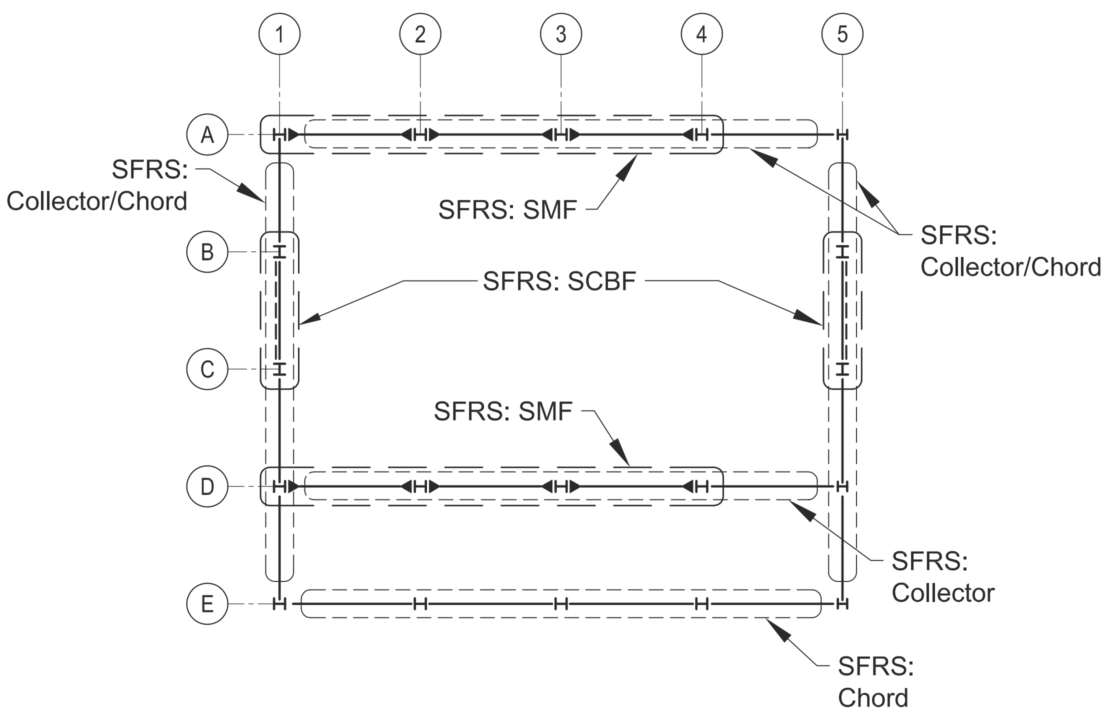
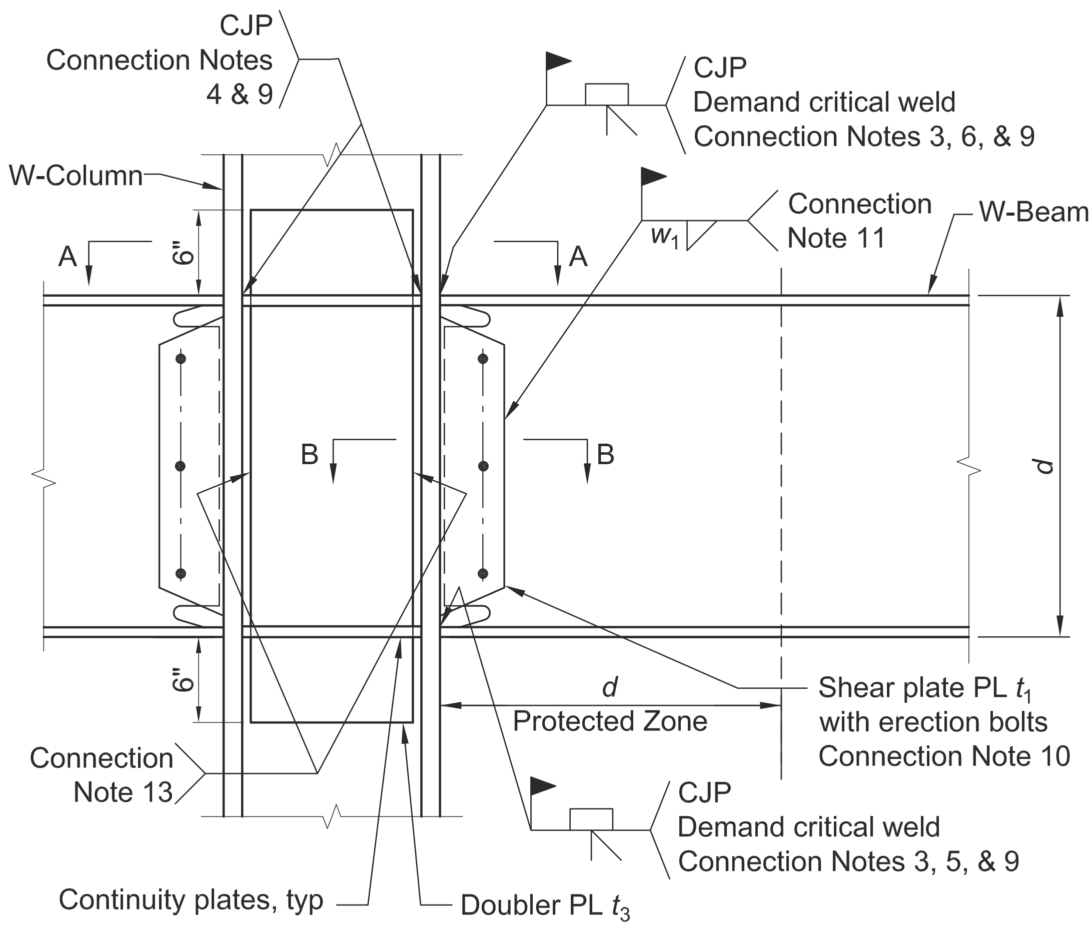
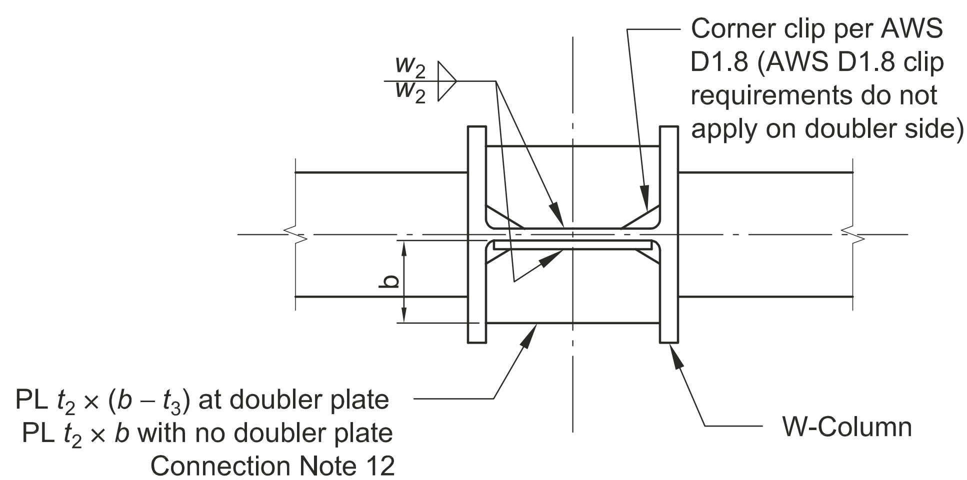


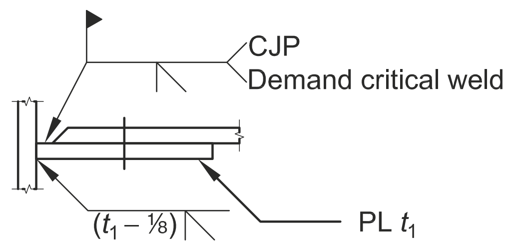
Fig.1-8. Typical plan identifying SFRS elements.





Section A-A

Fig. 1-9. Beam-to-column special moment frame connection (WUF-W) as a sample connection detail.



Section B-B

Connection Schedule							
Column Size	**Beam Size**	**Shear Plate**		**Continuity Plates**			**Doubler plate thickness t_3, in.**
		Thickness t_1, in.	**Fillet weld w_1, in.**	**Thickness t_2, in.**	**Width b, in.**	**Fillet weld w_2, in.**	
W14×370	W24×76	½	7⁄16	Not required	Not required	Not required	Not required
W14×257	W24×76	½	7⁄16	¾	6	½	½

Connection Notes

1. This connection is part of a seismic force-resisting system.
2. See Connection Schedule for connection parameters.
3. Weld access holes must conform to the requirements of AWS D1.8, clause 6.11.1.2.
4. Steel backing at the continuity plate may be removed (Connection Note 7) or left in place (Connection Note 8).
5. Steel backing at the bottom flange must be removed (Connection Note 7).
6. Steel backing at the top flange may be removed (Connection Note 7) or left in place (Connection Note 8).
7. Where steel backing is removed, the root pass is backgouged to sound weld metal and back welded with a minimum 5⁄16-in. reinforcing fillet. The toe of the reinforcing fillet does not need to be located on the continuity plate base metal.
8. Where steel backing is left in place, it has a 5⁄16-in. fillet to the column flange. No weld should be made from the backing to the beam flange or continuity plate.
9. Weld tabs at beam flanges and continuity plates must be removed in accordance with AWS D1.8, except at the outboard ends of continuity-plate-to-column welds. Weld tabs and weld metal need not be removed closer than 1⁄4 in. from the continuity plate edge.
10. Fabricate single plate per ANSI/AISC 358, Figure 8.3. It is acceptable to use horizontal short-slotted holes in the plate for erection bolts.
11. Weld shear plate to beam web on three sides. See ANSI/AISC 358, Figure 8.3, for additional information.
12. When a doubler plate is required, clip stiffener plate corners to clear doubler plate to W-shape column weld and column fillet. When no doubler plate is required, clip stiffener plate corners per AWS D1.8.
13. Provide weld at the web doubler plate per AWS D1.8, clause 4.3.
14. W-shapes are ASTM A992/A992M, connection plates are ASTM A572/A572M Gr. 50, and weld electrodes are E70XX.
15. This example is dependent on AISC *Seismic Provisions*, ANSI/AISC 358, and AWS D1.8 for complete detailing requirements.

Fig. 1-9 (continued). Beam-to-column special moment frame connection (WUF-W) as a sample connection detail.

1.5 DESIGN TABLE DISCUSSION

Guidance on Calculations Including Interpolation

Refer to AISC *Manual* Part 2 for discussion of tabulated values, including rounding and linear interpolation. When noted in this Manual, linear interpolation between tabulated values may lead to unconservative results.

Seismic Weld Access Hole Configurations

Table 1-1. Workable Weld Access Hole Configurations for Beams

Sixteen configurations are given based upon the minimum seismic weld access hole profile in accordance with the alternate geometry provisions of AWS D1.8/D1.8M, Figure 6.2. This table is suitable for beam-to-column connections where the alternate hole configuration per AWS D1.8/D1.8M is stipulated by AISC *Seismic Provisions* or ANSI/AISC 358. If this alternate hole configuration is not required, then the typical weld access holes in the AISC *Seismic Provisions* may be as provided in the AISC *Specification*, AWS D1.1/D1.1M, or AWS D1.8/D1.8M. This table is intended to be used in conjunction with Table 1-3 for quick selection of weld access hole geometry for W-shape beams when the seismic weld access hole is used. A workable seismic access hole configuration from Table 1-1 is given in Table 1-3 for each shape listed. The weld access hole is applicable regardless of the member ductility requirements, if any. Where an asterisk is shown, no configuration shown in Table 1-1 meets all criteria for the seismic hole configuration.

AISC *Specification* Section J1.6 provides general requirements for weld access holes. It should be noted that the geometries shown in Table 1-1 represent only one set of configurations that satisfy the dimensions and tolerances in AWS D1.8/D1.8M, Figure 6.2. Other configurations that comply with AWS D1.8/D1.8M, Figure 6.2 may also be used. The special seismic weld access hole is required for beams in ordinary moment frames per AISC *Seismic Provisions* Section E1.6b(c) and for beams in welded unreinforced flange-welded web (WUF-W) moment connections per ANSI/AISC 358.

Member Ductility Requirements

Table 1-2. Summary of Member Ductility Requirements

This table summarizes the member ductility requirements of AISC *Seismic Provisions* Section D1.1 that are required by AISC *Seismic Provisions* Chapters E, F, G, and H. Additional ductility requirements may be found in AISC *Seismic Provisions* sections specific to each SFRS.

Local Buckling Requirements

Table 1-3a. Sections that Satisfy Seismic Width-to-Thickness Requirements, W-Shapes, Moderately Ductile

Table 1-3b. Sections that Satisfy Seismic Width-to-Thickness Requirements, W-Shapes, Highly Ductile

These tables summarize the width-to-thickness requirements of W-shapes based on member type for both moderately and highly ductile applications. For each shape in Table 1-3a, the requirements for moderately ductile members are presented for members in moment frames, braced frames, and shear wall systems. For each shape in Table 1-3b, the requirements for highly ductile members are presented on the left-hand page for members in moment frames and on the right-hand page for members in braced frames and shear wall systems. See Table 1-2 for a summary of member ductility requirements, indicated by the SFRS, per the AISC *Seismic Provisions*. A wide-flange section satisfies the width-to-thickness requirements if its corresponding flange and web ratios are less than or equal to the limits listed in Table 1-A, which summarizes the requirements in AISC *Seismic Provisions* Tables D1.1a and D1.1b. Note that W-shapes that do not satisfy either moderately or highly ductile width-to-thickness ratios for any of the steel strengths incorporated are not included in Tables 1-3a and 1-3b.

Diagonal brace W-shapes that satisfy the moderately or highly ductile width-to-thickness requirements per AISC *Seismic Provisions* Table D1.1a are indicated with a "•" in the column labeled "Diagonal Braces" for F_y = 50 ksi (ASTM A992/A992M and ASTM A913/A913M, where applicable). For beams, columns, and links with F_y = 50 ksi (ASTM A992/A992M and ASTM A913/A913M, where applicable), and for columns with F_y = 65 ksi (ASTM A913/A913M) and F_y = 70 ksi (ASTM A913/A913M), the limiting web width-to-thickness ratio is a function of a member's required axial strength, P_r, using LRFD or ASD load combinations. For these cases, the member will satisfy the width-to-thickness requirements if P_r is less than or equal to the value tabulated for $P_{r\,max}$. Where "NL" is indicated, the values of P_r are not limited by seismic width-to-thickness ratios and are instead limited by the member available strength.

Higher strength steels of F_y = 65 ksi (ASTM A913/A913M) and F_y = 70 ksi (ASTM A913/A913M) are included in Table 1-3b for highly ductile members but not in Table 1-3a for moderately ductile members. According to AISC *Seismic Provisions* Section A3.1, these higher strength steels are permitted only for columns in specific systems, and Table 1-2 indicates that columns in each of these systems either have no ductility requirements or must meet the requirements for highly ductile members.

Also provided is the maximum spacing of beam bracing for moderately ductile and highly ductile beams, $L_{b\,max}$, where for moderately ductile beams, $L_{b\,max} = 0.17 r_y E/(R_y F_y)$, and for highly ductile beams, $L_{b\,max} = 0.086 r_y E/(R_y F_y)$.

Interpolation between values in this table may produce an incorrect result. Refer to *Guidance on Calculations Including Interpolation* in this part.

Table 1-A
Limiting Width-to-Thickness Ratios for W-Shape Flanges and Webs in Compression

	Member	Limiting Width-to-Thickness Ratio	
		Flange, b/t	Web, h/t_w
Moderately Ductile	Diagonal Brace	$0.38\sqrt{\dfrac{E}{R_yF_y}}$	$1.49\sqrt{\dfrac{E}{R_yF_y}}$
Moderately Ductile	Moment Frame Beam or Column	$0.38\sqrt{\dfrac{E}{R_yF_y}}$	$5.4(1-C_a)^{2.3}\sqrt{\dfrac{E}{R_yF_y}}$ [a]
Moderately Ductile	Beam or Column not in Moment Frame, EBF Link[b]	$0.38\sqrt{\dfrac{E}{R_yF_y}}$	For $C_a \le 0.113$[a] $3.76(1-3.05C_a)\sqrt{\dfrac{E}{R_yF_y}}$ For $C_a > 0.113$ $2.61(1-0.49C_a)\sqrt{\dfrac{E}{R_yF_y}} \ge 1.56\sqrt{\dfrac{E}{R_yF_y}}$
Highly Ductile	Diagonal Brace	$0.30\sqrt{\dfrac{E}{R_yF_y}}$	$1.49\sqrt{\dfrac{E}{R_yF_y}}$
Highly Ductile	Moment Frame Beam or Column	$0.30\sqrt{\dfrac{E}{R_yF_y}}$	$2.5(1-C_a)^{2.3}\sqrt{\dfrac{E}{R_yF_y}}$ [a]
Highly Ductile	Beam or Column not in Moment Frame, Chord in STMF Special Segment, EBF Link, SPSW VBE & HBE	$0.30\sqrt{\dfrac{E}{R_yF_y}}$	For $C_a \le 0.113$[a] $2.45(1-1.04C_a)\sqrt{\dfrac{E}{R_yF_y}}$ For $C_a > 0.113$ $2.26(1-0.38C_a)\sqrt{\dfrac{E}{R_yF_y}} \ge 1.56\sqrt{\dfrac{E}{R_yF_y}}$

[a] $C_a = \dfrac{\alpha_s P_r}{R_y F_y A_g}$

where
- A_g = gross area, in.^{2}
- E = modulus of elasticity of steel, ksi
- F_y = specified minimum yield stress, ksi
- P_r = required axial strength using LRFD or ASD load combinations, kips
- R_y = ratio of the expected yield stress to the specified minimum yield stress
- α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD

[b] Applies to EBF links meeting the exception in AISC *Seismic Provisions* Section F3.5b.1.

Table 1-B
Limiting Width-to-Thickness Ratios for Angle Legs in Compression

	Member	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio
Moderately Ductile	Diagonal Brace	b/t	$0.38\sqrt{\frac{E}{R_yF_y}}$
Highly Ductile	Diagonal Brace, Chords in STMF Special Segment	b/t	$0.30\sqrt{\frac{E}{R_yF_y}}$

Table 1-4. Sections that Satisfy Seismic Width-to-Thickness Requirements, Angles

Angles with F_y = 50 ksi (ASTM A572/A572M Grade 50), including both single- and double-angle configurations, that satisfy AISC *Seismic Provisions* local buckling requirements for use as diagonal braces in SCBF, OCBF, EBF, and the special segment of STMF chords are indicated with a "•" in the corresponding column. An angle satisfies these requirements if the greatest leg width-to-thickness ratio is less than or equal to the corresponding limits listed in Table 1-B, which is summarized from the requirements in AISC *Seismic Provisions* Tables D1.1a and D1.1b. Note that angles that do not satisfy either moderately or highly ductile width-to-thickness ratios are not included in Table 1-4.

Table 1-5a. Sections that Satisfy Seismic Width-to-Thickness Requirements, Rectangular HSS

Table 1-5b. Sections that Satisfy Seismic Width-to-Thickness Requirements, Square HSS

These tables summarize the width-to-thickness requirements of rectangular and square HSS for ASTM A500/A500M Grade C and ASTM A1085/A1085M material with F_y = 50 ksi. Note that, because ASTM A1085/A1085M material is available only in Grade A, the tables do not include any grade designation.

Sections that satisfy the AISC *Seismic Provisions* moderately or highly ductile width-to-thickness requirements for use as braces are indicated with a "•" in the corresponding column. For beams and columns, the limiting width-to-thickness ratio is a function of a member's required axial strength, P_r, using LRFD or ASD load combinations. For these cases, the member will satisfy the width-to-thickness requirements if P_r is less than or equal to the value tabulated for $P_{r\,max}$. Where "NL" is indicated, the values of P_r are not limited by seismic width-to-thickness ratios and are instead limited by the member available strength. A rectangular or square HSS satisfies these requirements if its flange and web width-to-thickness ratios are less than or equal to the corresponding limits listed in Table 1-C, which is summarized from the requirements of AISC *Seismic Provisions* Tables D1.1a and D1.1b. Note that HSS sections that do not satisfy either moderately or highly ductile width-to-thickness ratios are not included in Table 1-5a or 1-5b.

Interpolation between values in this table may produce an incorrect result. Refer to *Guidance on Calculations Including Interpolation* in this part.

Table 1-6. Sections that Satisfy Seismic Width-to-Thickness Requirements, Round HSS

This table summarizes the width-to-thickness requirements of round HSS for ASTM A500/A500M Grade C and ASTM A1085/A1085M material with F_y = 50 ksi. Note that, because ASTM A1085/A1085M material is available only in Grade A, the table does not include any grade designation.

Sections that satisfy the AISC *Seismic Provisions* local buckling requirements for use as columns, beams, or braces in SCBF, columns or braces in EBF, and braces in OCBF are indicated with a "•" in the corresponding column. A round HSS satisfies these requirements if its width-to-thickness ratio is less than or equal to the corresponding limit listed in Table 1-D. Note that round HSS sections that do not satisfy either moderately or highly ductile width-to-thickness ratios are not included in Table 1-6.

Table 1-7. Sections that Satisfy Seismic Width-to-Thickness Requirements, Pipes

Pipes with F_y = 35 ksi (ASTM A53/A53M Grade B) that satisfy AISC *Seismic Provisions* local buckling requirements for use as braces or columns in SCBF and EBF and braces in OCBF are indicated with a "•" in the corresponding column. A pipe satisfies these requirements if its width-to-thickness ratio, D/t, is less than or equal to the corresponding limit listed in Table 1-D. Note that pipes that do not satisfy either moderately or highly ductile width-to-thickness ratios are not included in Table 1-7.

Table 1-C
Limiting Width-to-Thickness Ratios for Rectangular and Square HSS Walls in Compression

	Member	Limiting Width-to-Thickness Ratio	
		Flange, *b/t*	**Web, *h/t***
Moderately Ductile	**Diagonal Brace**	$0.76\sqrt{\dfrac{E}{R_y F_y}}$	$0.76\sqrt{\dfrac{E}{R_y F_y}}$
	Beam, Column	$1.00\sqrt{\dfrac{E}{R_y F_y}}$	For $C_a \le 0.113$[a] $3.76(1-3.05C_a)\sqrt{\dfrac{E}{R_y F_y}}$ For $C_a > 0.113$ $2.61(1-0.49C_a)\sqrt{\dfrac{E}{R_y F_y}} \ge 1.56\sqrt{\dfrac{E}{R_y F_y}}$
Highly Ductile	**Diagonal Brace**	$0.65\sqrt{\dfrac{E}{R_y F_y}}$	$0.65\sqrt{\dfrac{E}{R_y F_y}}$
	Beam, Column	$0.55\sqrt{\dfrac{E}{R_y F_y}}$	For $C_a \le 0.113$[a] $2.45(1-1.04C_a)\sqrt{\dfrac{E}{R_y F_y}}$ For $C_a > 0.113$ $2.26(1-0.38C_a)\sqrt{\dfrac{E}{R_y F_y}} \ge 1.56\sqrt{\dfrac{E}{R_y F_y}}$

[a] $C_a = \dfrac{\alpha_s P_r}{R_y F_y A_g}$

where

A_g = gross area, in.^{2}
E = modulus of elasticity of steel, ksi
F_y = specified minimum yield stress, ksi
P_r = required axial strength using LRFD or ASD load combinations, kips
R_y = ratio of the expected yield stress to the specified minimum yield stress
α_s = LRFD-ASD force level adjustment factor = 1.0 for LRFD and 1.5 for ASD

Table 1-D
Limiting Width-to-Thickness Ratios for Round HSS and Pipe Walls in Compression

	Member	Width-to-Thickness Ratio	Limiting Width-to-Thickness Ratio
Moderately Ductile	Diagonal Brace	D/t	$0.062\dfrac{E}{R_yF_y}$
	Beam, Column	D/t	$0.07\dfrac{E}{R_yF_y}$
Highly Ductile	Diagonal Brace	D/t	$0.053\dfrac{E}{R_yF_y}$
	Beam, Column	D/t	$0.038\dfrac{E}{R_yF_y}$

Strength of Steel Headed Stud Anchors

Table 1-8. Nominal Horizontal Shear Strength and 25% Reduced Nominal Horizontal Shear Strength for One Steel Headed Stud Anchor

The nominal shear strength of steel headed stud anchors is given in Table 1-8, in accordance with AISC *Specification* Chapter I. This table provides the nominal shear strength for one steel headed stud anchor embedded in a solid concrete slab or in a composite slab with decking, as given in AISC *Specification* Section I8.2a. The nominal shear strength with the 25% reduction as specified in AISC *Seismic Provisions* Section D2.8 for intermediate or special SFRS of Sections G2, G3, G4, H2, H3, H5, and H6 is also given in Table 1-8. According to the User Note in AISC *Seismic Provisions* Section D2.8, the 25% reduction is not necessary for gravity or collector components in structures with intermediate or special SFRS designed for the overstrength seismic load. Nominal horizontal shear strength values are presented based upon the position of the steel anchor, profile of the deck, and orientation of the deck relative to the steel anchor. See AISC *Specification* Commentary Figure C-I8.1.

Interpolation between values in this table may produce an incorrect result. Refer to *Guidance on Calculations Including Interpolation* in this part.