Seismic Design of Structures According to ASCE/SEI 7-22

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Credit: 3 PDH

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1. Introduction

This course describes the ASCE/SEI 7 procedures for determining the required seismic strength, stiffness, and detailing of structures in the Seismic Design Category (SDC) B through SDC F. The list below indicates these steps and identifies the section in which these steps are discussed.

1) Select the structural system
2) Identify system design coefficients
3) Check for configuration irregularities
4) Calculate seismic loads
5) Analyze and design structural elements
6) Check drift and stability
7) Design diaphragms
8) Detail connections and other elements

2. Select the structural system

Over many years, engineers have observed that some structural systems perform better in earthquakes than others. Based on these observations, the selection of a structural system type is a key and early step in the seismic design criteria for buildings. Structural systems are categorized based on three characteristics:

- Material of construction (e.g., concrete, masonry, steel, or wood)
- How lateral forces induced by earthquake shaking are resisted by the structure
- The relative quality of earthquake-resistant design and detailing

Ductility is the ability of some structural systems to experience deformations beyond those that cause them to develop their peak strength while continuing to carry the load. Brittle structural systems have no ductility. They will deform elastically until the applied load is equal to their ultimate strength, then fail suddenly and lose the load-carrying ability. Structures with limited ductility may be able to retain load-carrying capability up to a deformation 50% larger than the deformation at which they develop peak strength. Highly ductile structures may be able to withstand deformations up to 4 or 5 times those at which peak strength is achieved without loss of load-carrying capability. In reality, most structural systems have some ductility. ASCE/SEI 7 categorizes systems with superior ductility as special, systems with limited ductility as ordinary, and systems with intermediate levels of ductility as intermediate. ASCE/SEI 7 permits the design of intermediate and special systems with less strength than ordinary systems. However, to qualify as an intermediate or special system, the design must follow rigorous detailed requirements that can result in greater construction costs. This course describes these requirements in more detail.

NOTE: ASCE/SEI 7 Table 12.2-1 defines the applicable structural systems for seismic resistance, associated height restrictions, applicability to different SDC$_s$, and applicable design coefficients (R, C_d and Ω_o).

Table 12.2-1. Design Coefficients and Factors for Seismic Force-Resisting Systems.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R[a]	Overstrength Factor, Ω_0[b]	Deflection Amplification Factor, C_d[c]	Structural System Limitations Including Structural Height, h_n, Limits (ft)[d] — Seismic Design Category: B	C	D[e]	E[e]	F[f]
A. BEARING WALL SYSTEMS									
1. Special reinforced concrete shear walls[g,h]	14.2	5	2½	5	NL	NL	160	160	100
2. Reinforced concrete ductile coupled walls[g]	14.2	8	2½	8	NL	NL	160	160	100
3. Ordinary reinforced concrete shear walls[g]	14.2	4	2½	4	NL	NL	NP	NP	NP
4. Detailed plain concrete shear walls[g]	14.2	2	2½	2	NL	NP	NP	NP	NP
5. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
6. Intermediate precast shear walls[g]	14.2	4	2½	4	NL	NL	40[i]	40[i]	40[i]
7. Ordinary precast shear walls[g]	14.2	3	2½	3	NL	NP	NP	NP	NP
8. Special reinforced masonry shear walls	14.4	5	2½	3½	NL	NL	160	160	100
9. Intermediate reinforced masonry shear walls	14.4	3½	2½	2¼	NL	NL	NP	NP	NP
10. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP	NP	NP
11. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP	NP	NP
12. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
13. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
14. Ordinary reinforced AAC masonry shear walls	14.4	2	2½	2	NL	35	NP	NP	NP
15. Ordinary plain AAC masonry shear walls	14.4	1½	2½	1½	NL	NP	NP	NP	NP
16. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	6½	3	4	NL	NL	65	65	65
17. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	6½	3	4	NL	NL	65	65	65
18. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2	2½	2	NL	NL	35	NP	NP
19. Light-frame (cold-formed steel) wall systems using flat strap bracing	14.1	4	2	3½	NL	NL	65	65	65
20. Cross-laminated timber shear walls	14.5	3	3	3	65	65	65	65	65
21. Cross-laminated timber shear walls with shear resistance provided by high-aspect-ratio panels only	14.5	4	3	4	65	65	65	65	65
B. BUILDING FRAME SYSTEMS									
1. Steel eccentrically braced frames	14.1	8	2	4	NL	NL	160	160	100
2. Steel special concentrically braced frames	14.1	6	2	5	NL	NL	160	160	100
3. Steel ordinary concentrically braced frames	14.1	3¼	2	3¼	NL	NL	35[j]	35[j]	NP[j]
4. Special reinforced concrete shear walls[g,h]	14.2	6	2½	5	NL	NL	160	160	100
5. Reinforced concrete ductile coupled walls[g]	14.2	8	2½	8	NL	NL	160	160	100
6. Ordinary reinforced concrete shear walls[g]	14.2	5	2½	4½	NL	NL	NP	NP	NP
7. Detailed plain concrete shear walls[g]	14.2 and 14.2.2.7	2	2½	2	NL	NP	NP	NP	NP
8. Ordinary plain concrete shear walls[g]	14.2	1½	2½	1½	NL	NP	NP	NP	NP
9. Intermediate precast shear walls[g]	14.2	5	2½	4½	NL	NL	40[i]	40[i]	40[i]
10. Ordinary precast shear walls[g]	14.2	4	2½	4	NL	NP	NP	NP	NP
11. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	160	160	100
12. Steel and concrete composite special concentrically braced frames	14.3	5	2	4½	NL	NL	160	160	100
13. Steel and concrete composite ordinary braced frames	14.3	3	2	3	NL	NL	NP	NP	NP
14. Steel and concrete composite plate shear walls	14.3	6½	2½	5½	NL	NL	160	160	100
15. Steel and concrete composite special shear walls	14.3	6	2½	5	NL	NL	160	160	100
16. Steel and concrete composite ordinary shear walls	14.3	5	2½	4½	NL	NL	NP	NP	NP
17. Special reinforced masonry shear walls	14.4	5½	2½	4	NL	NL	160	160	100

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Table 1. Design coefficients and factors for seismic force-resisting systems.

18. Intermediate reinforced masonry shear walls	14.4	4	2½	4	NL	NL	NP	NP	NP
19. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
20. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
21. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
22. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
23. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
24. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
25. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	2½	2½	NL	NL	35	NP	NP
26. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
27. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100
28. Steel and concrete coupled composite plate shear walls	14.3	8	2½	5½	NL	NL	160	160	100
C. MOMENT-RESISTING FRAME SYSTEMS									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35[k]	NP[k]	NP[k]
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP[l]	NP[l]	NP[l]
5. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame[n]	14.1	3½	3[o]	3½	35	35	35	35	35
D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Steel special concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls[g,h]	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Reinforced concrete ductile coupled walls[g]	14.2	8	2½	8	NL	NL	NL	NL	NL
5. Ordinary reinforced concrete shear walls[g]	14.2	6	2½	5	NL	NL	NP	NP	NP
6. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
7. Steel and concrete composite special concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
8. Steel and concrete composite plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
9. Steel and concrete composite special shear walls	14.3	7	2½	6	NL	NL	NL	NL	NL
10. Steel and concrete composite ordinary shear walls	14.3	6	2½	5	NL	NL	NP	NP	NP
11. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
12. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
13. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	NL	NL	NL
14. Steel special plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL
15. Steel and concrete coupled composite plate shear walls	14.3	8	2½	5½	NL	NL	NL	NL	NL
E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES	12.2.5.1								
1. Steel special concentrically braced frames[p]	14.1	6	2½	5	NL	NL	35	NP	NP
2. Special reinforced concrete shear walls[g,h]	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP

continues

Table 12.2-1 *(Continued)*. Design Coefficients and Factors for Seismic Force-Resisting Systems.

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, R^a	Overstrength Factor, Ω_0^b	Deflection Amplification Factor, C_d^c	Structural System Limitations Including Structural Height, h_n, Limits (ft)[d] — Seismic Design Category: B	C	D[e]	E[e]	F[f]
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP
5. Steel and concrete composite special concentrically braced frames	14.3	5½	2½	4½	NL	NL	160	100	NP
6. Steel and concrete composite ordinary braced frames	14.3	3½	2½	3	NL	NL	NP	NP	NP
7. Steel and concrete composite ordinary shear walls	14.3	5	3	4½	NL	NL	NP	NP	NP
8. Ordinary reinforced concrete shear walls[g]	14.2	5½	2½	4½	NL	NL	NP	NP	NP
F. SHEAR WALL-FRAME INTERACTIVE SYSTEM WITH ORDINARY REINFORCED CONCRETE MOMENT FRAMES AND ORDINARY REINFORCED CONCRETE SHEAR WALLS[g]	12.2.5.8 and 14.2	4½	2½	4	NL	NP	NP	NP	NP
G. CANTILEVERED COLUMN SYSTEMS DETAILED TO CONFORM TO THE REQUIREMENTS FOR:	12.2.5.2								
1. Steel special cantilever column systems	14.1	2½	2½	2½	35	35	35	35	35
2. Steel ordinary cantilever column systems	14.1	1¼	1¼	1¼	35	35	NP[l]	NP[l]	NP[l]
3. Special reinforced concrete moment frames[m]	12.2.5.5 and 14.2	2½	2½	2½	35	35	35	35	35
4. Intermediate reinforced concrete moment frames	14.2	1½	1½	1½	35	35	NP	NP	NP
5. Ordinary reinforced concrete moment frames	14.2	1	1¼	1	35	NP	NP	NP	NP
6. Timber frames	14.5	1½	1½	1½	35	35	35	NP	NP
H. STEEL SYSTEMS NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE, EXCLUDING CANTILEVER COLUMN SYSTEMS	14.1	3	3	3	NL	NL	NP	NP	NP

[a] Response modification coefficient, R, for use throughout the standard. Note that R reduces forces to a strength level, not an allowable stress level.
[b] Where the tabulated value of the overstrength factor, Ω_0, is greater than or equal to 2½, Ω_0 is permitted to be reduced by subtracting the value of 1/2 for structures with flexible diaphragms.
[c] Deflection amplification factor, C_d, for use in Sections 12.8.6, 12.8.7, 12.9.1.2, and 12.12.
[d] NL = Not Limited, and NP = Not Permitted. For metric SI units, multiply by 0.348 m/ft and round to the nearest 0.1 m.
[e] See Section 12.2.5.4 for a description of seismic force-resisting systems limited to buildings with a structural height, h_n, of 240 ft (73.2 m) or less.
[f] See Section 12.2.5.4 for seismic force-resisting systems limited to buildings with a structural height, h_n, of 160 ft (48.8 m) or less.
[g] In Section 2.3 of ACI 318. A shear wall is defined as a structural wall.
[h] In Section 2.3 of ACI 318. The definition of "special structural wall" includes precast and cast-in-place construction.
[i] An increase in structural height, h_n, to 45 ft (13.7 m) is permitted for single-story storage warehouse facilities.
[j] Steel ordinary concentrically braced frames are permitted in single-story buildings up to a structural height, h_n, of 60 ft (18.3 m) where the dead load of the roof does not exceed 20 lb/ft² (0.96 kN/m²) and in penthouse structures.
[k] See Section 12.2.5.7 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[l] See Section 12.2.5.6 for limitations in structures assigned to Seismic Design Categories D, E, or F.
[m] In Section 2.3 of ACI 318, the definition of "special moment frame" includes precast and cast-in-place construction.
[n] Cold-formed steel—special bolted moment frames shall be limited to one story in height in accordance with ANSI/AISI S400.
[o] Alternately, the seismic load effect including overstrength, E_{mh}, is permitted to be based on the expected strength determined in accordance with ANSI/AISI S400.
[p] Ordinary moment frame is permitted to be used in lieu of intermediate moment frame for Seismic Design Category B or C.
[q] Structural height, h_n, shall not be less than 60 ft (18.3m).

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Based on past historic performance, some structural systems that have little ductility are prohibited from use in seismic design categories associated with intense earthquake shaking or immediate post-earthquake occupancy. Still, other systems are only permitted for use for buildings of limited height or weight.

The portion of the structure that is specifically designed to provide the required earthquake resistance is called the seismic force-resisting system (SFRS). Structures assigned to SDC A can use any type of SFRS if the system is complete and provides minimum specified strength. Buildings assigned to SDC B or higher must utilize one of the specific SFRSs or combinations of these systems listed in Table 12.2-1 of the ASCE/SEI 7-22 and comply with all of the design rules applicable to the selected system. The SFRS of a building resists other lateral loads (e.g., wind), but this chapter focuses on seismic considerations.

ASCE/SEI 7-22 Table 12.2-1 lists more than 90 structural systems, providing designers with a wide range of choices and classifies structural systems for buildings into one of six broad categories:

1) Bearing wall systems
2) Building frame systems
3) Moment-resisting frame systems
4) Dual systems and shear wall frame interactive systems
5) Cantilever column systems
6) Systems not specifically designed for seismic resistance

The sections below describe key structural requirements for these systems

2.1 Wall Systems

Wall systems include structures in which masonry, concrete, wood-frame, structural steel, composite steel and concrete, or cold-formed steel walls provide lateral resistance to wind and earthquake forces. Wall systems can be classified as bearing wall systems or building frame systems, depending on whether the walls carry a substantial portion of the gravity loading of the building or they rely on them only to resist lateral loads. The building code requires that wall systems that carry substantial portions of the vertical load of a building be designed with higher strength than those that do not so that they will experience less damage in response to strong shaking.

The primary factor affecting the classification of a structural system of concrete or masonry walls as plain, detailed, ordinary, intermediate, or special is the quantity and detailing of reinforcing steel contained in the wall. Figure 1 shows an exaggerated, deformed shape for a typical concrete or masonry wall subjected to lateral forces from an earthquake illustrating the types of damage that may occur. Typical damage includes diagonal cracking due to shear in coupling beams and piers, flexural cracking at the bases of vertical piers and compressive crushing and spalling at the corners of piers, accompanied by buckling and potentially fracturing of vertical reinforcing steel.

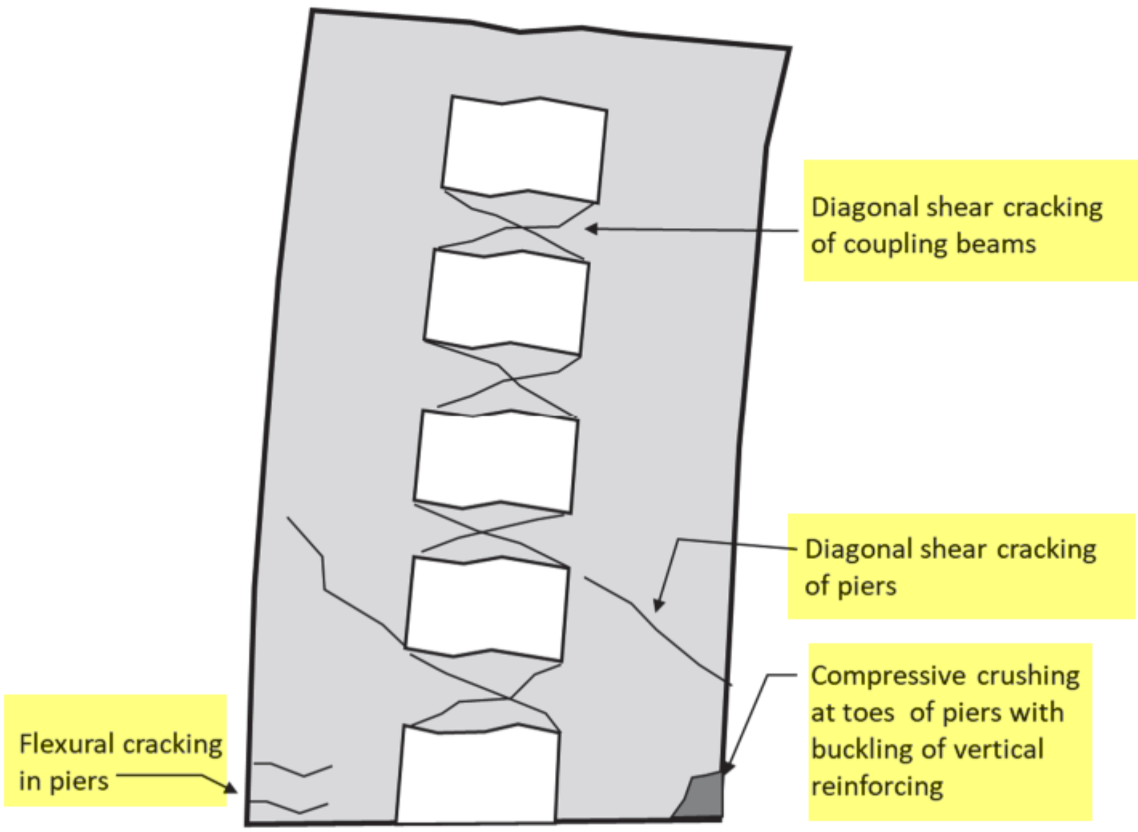

Figure 1. Deformed shape and typical damage patterns in multi-story concrete or masonry

Plain masonry (unreinforced) and concrete walls are not provided with reinforcing to resist seismic forces and the types of damage indicated in the figure and can rapidly lose strength in earthquake shaking. Detailed plain walls are provided with nominal reinforcing at openings, such as those for doors and windows. This reinforcing is primarily intended to prevent cracking originating at the corners of the opening but is not effective in resisting repeated straining of the wall into the inelastic range.

NOTE: ASCE/SEI 7-22 and ACI 318-19 introduced a new class of special concrete shear wall system, called the ductile coupled wall system. The coupled wall system is required to incorporate coupling beams, meeting specific dimensional criteria, over openings. These *coupling beams serve as a benign means of dissipating earthquake energy and permit use of reduced design forces relative to other concrete wall systems.*

Walls of light-frame construction, including both wood and cold-formed steel, are categorized with regard to their ability to resist inelastic response, primarily, based on the type of sheathing used to provide lateral resistance. Traditional systems of plaster and gypsum board sheathing have limited ability to provide repeated resistance to lateral deformation, once the plaster or gypsum product cracks. Walls with these sheathing materials are limited to SDC B, C and D, and in SDC D are only permitted for low-rise structures. Walls incorporating plywood or structural panel sheathing attached with appropriate fasteners can resist many cycles of large lateral displacement and are permitted in all SDCs and are permitted to be designed for reduced forces, relative to walls with ordinary sheathing materials.

2.2 Braced Frame Systems

The most common braced frame systems are constructed from steel. Steel braced frames are a common type of structural steel building frame system. Figure 2 shows common types of steel braced frame systems. Braced frame systems that are specifically detailed for seismic resistance must meet the criteria of AISC 341, Seismic Provisions for Steel Structures. This is required for braced frames in SDC D, E, or F and permitted for other SDCs. AISC 341 does not permit single diagonal braced frames with more than 50% of the braces in a story and in a frame line aligned in one direction because if the braces are overloaded, and buckle, the frame will lose lateral resistance. Similarly, K-braced frames are prohibited by AISC 341 because under lateral loads, the compression braces can buckle, and the tensile braces will then place large, concentrated loads on the columns at mid-height, potentially resulting in column buckling and collapse

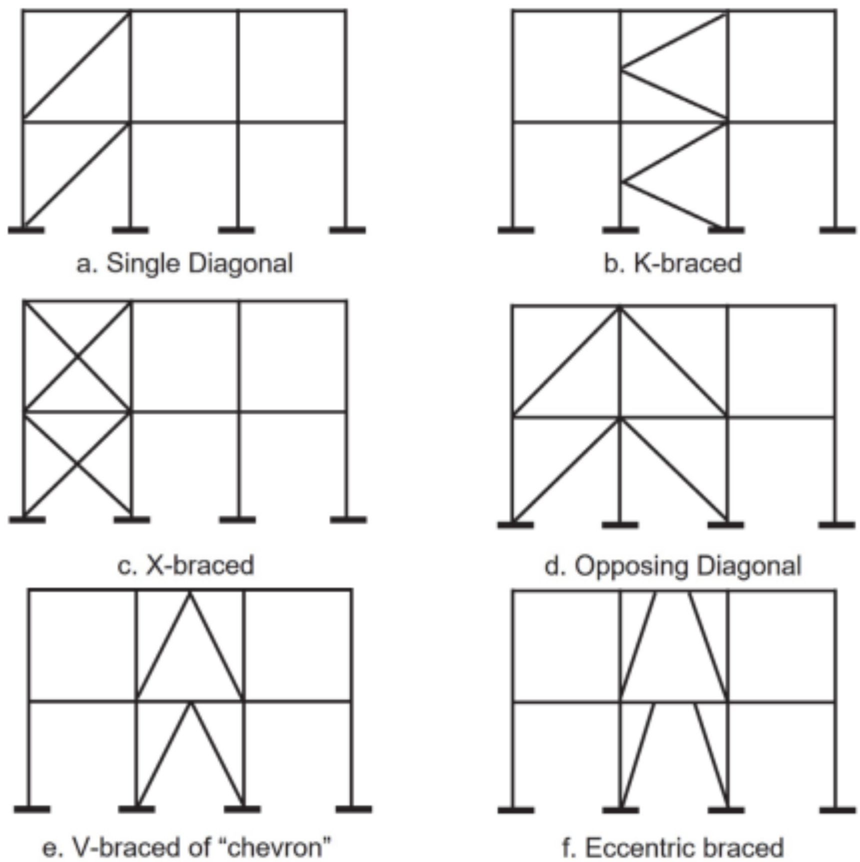

Figure 2. Common types of steel braced frame systems.

Concentrically braced frames are configured such that the beams, columns and braces intersect at common points. Eccentrically braced frames (Figure 8-2(f)) are configured such that brace-to-beam connections are offset from each other, or offset from beam-column intersections, such that nonlinear behavior is accommodated through ductile flexural or shear yielding of the beams, rather than the braces themselves. Buckling-restrained braced frames use braces consisting of a central steel core that can yield in tension or compression, braced by an outer sleeve. This newer system is highly tolerant of repeated nonlinear cyclic loading, and, like the eccentrically braced frame, is permitted to be designed for reduced strength relative to other concentrically braced frame types.

2.3 Moment-resisting Frame Systems

Moment-resisting frame systems (also called moment frames) can be constructed of structural steel, reinforced concrete, or a combination of steel and reinforced concrete called composite construction. Moment frames derive their lateral resistance through the rigid or semi-rigid connection of their beams and columns. This results in the lateral deformation pattern illustrated in Figure 3. This deformation pattern occurs simultaneously with the development of shearing forces and bending moments in the beams and columns, and axial forces associated with overturning in the columns. ACI 318 and AISC 341 respectively describe the detailing required of special, intermediate, and ordinary moment frames of reinforced concrete, steel, and composite construction.

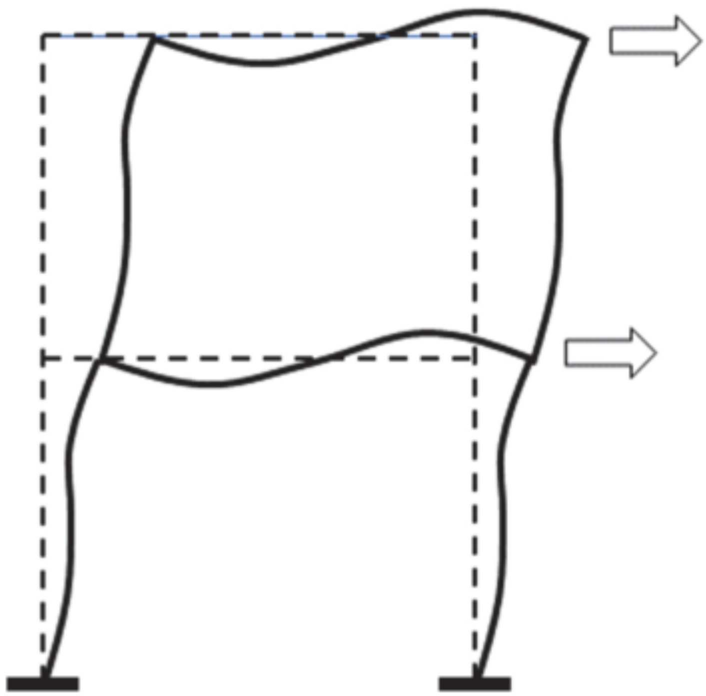

Figure 3. Typical deformed shape of moment-resisting frame responding to lateral forces.

Special moment frames detailed in accordance with ACI 318 and AISC 341 can provide large inelastic response, permitting them to be designed for low lateral forces, relative to other systems, and allowing use of these systems without height restrictions, even in SDC D, E, and F. Both ACI 318 and AISC 341 require design of beams and columns such that flexural yielding of the frame will occur mostly in the beams, rather than the columns, to minimize damage to columns. Both specifications also restrict the materials that can be used in special moment frames, and the locations of splices in members to ensure that nonlinear behavior is achievable.

Both ACI and AISC recognize intermediate and ordinary moment frame systems. These systems have relaxed detailing criteria relative to special moment-resisting frames but are required to have greater strength than special moment frames, to limit the amount of nonlinear behavior experienced in design shaking.

2.4 Dual Systems

Dual systems are a combination of a concrete, steel, or composite moment-resisting frame system and a concrete or masonry wall, or braced steel frame system. In SDC C, D, E and F, design of dual systems requires a structural analysis that considers the interaction between the moment frame and other elements. The moment frame must be designed to resist at least 25% of the total required seismic forces and have sufficient strength to resist the forces predicted by analysis. The intent of this requirement is that the moment frame will act as a redundant (i.e., back-up) system, that will be capable of resisting earthquake forces should the primary system (walls or braced frames) become extensively damaged. Like special moment frame systems, dual systems that incorporate a special moment-resisting frame can be constructed without height limit in SDC D, E, and F. In SDC B, ASCE/SEI 7 permits a type of dual system known as a frame-wall interactive system. In this system, it is not necessary that the moment frame be capable of resisting 25% of the total seismic design forces.

2.5 Cantilever Column Systems

Cantilever column systems are a special form of moment-resisting frame in which there are no beams connected to the column tops to restrain them against rotation. These systems derive their lateral resistance solely from the fixity against rotation at the column base. The simple single degree of freedom (SDOF) structure is an example of a cantilever column system. Both concrete and steel cantilever columns systems are permitted by the code. Detailing of the cantilevered columns can conform to the criteria for special, intermediate, or ordinary moment-resisting frame systems. Regardless, the limits on height are very restrictive and these structures must be designed to remain nearly elastic in response to design earthquake shaking. This is because these systems often have low redundancy, that is, formation of a single hinge, at the column base, results in formation of a plastic mechanism. In addition, these systems tend to be quite flexible, and can quickly develop large P-delta effects and instability.

3. Identify Design Coefficients

ASCE/SEI 7-22 Table 12.2-1 specifies the values of three design coefficients used to determine the required strength and stiffness for the seismic force-resisting system of a structure:

- ***R*** is the response modification coefficient that accounts for the ability of some seismic force resisting systems to respond to earthquake shaking in a ductile manner without loss of load carrying capacity. ***R*** values range from 1 for systems that have no ability to provide ductile response to 8 for systems that are capable of highly ductile response. The R factor is used to reduce the required design strength for a structure.

- C_d is the deflection amplification coefficient. It is used to adjust computed lateral displacements for the structure determined using linear analysis procedures to the anticipated inelastic lateral displacement that will occur in design earthquake shaking. The C_d factors assigned to the various structural systems are typically similar but smaller that the ***R*** coefficients, which accounts in an approximate manner for the effective damping and energy dissipation that can be mobilized during inelastic response of highly ductile systems. The more ductile a system is, the greater will be the difference between the value of ***R*** and C_d.

- Ω_o is an over strength coefficient used to account for the fact that the actual seismic forces on some elements of a structure can significantly exceed those indicated by analysis using the design seismic forces. For most structural systems, the Ω_o coefficient will have a value between 2 and 3.

4. Check for Configuration Irregularities

The values of the design coefficients (R, C_d, and Ω_o) specified in ASCE/SEI 7 were developed for systems that have regular configuration, with deformation and nonlinear response well-distributed throughout the structure and limited torsion about the vertical axis. Also, some of the analysis procedures used to determine the required strength of structural systems are incapable of predicting response reliably when the structures are not regular. To the extent that structures have non-uniform distribution of strength or stiffness and discontinuous structural systems, the assumptions that underlie the design procedures can become invalid. These conditions are known as irregularities, and structures that have one or more of these irregularities are termed irregular structures.

Some irregularities require modification of the analysis procedures used to determine required strength and story drift. Some irregularities trigger requirements for portions of the structure to be provided with greater strength to counteract the negative effects of the irregularity. Some irregularities have led to such poor performance in past earthquakes that they are prohibited from use in structures assigned to SDC E or SDC F. ASCE/SEI 7 identifies two basic categories of structural irregularity: horizontal and vertical.

4.1 Horizontal irregularities

Horizontal irregularities include the following types:

- Torsional irregularity: This condition exists when the distribution of vertical elements of the SFRS within a story, including braced frames, moment frames and walls, is such that when the building is pushed to the side by wind or earthquake forces, it will tend to twist as well as deflect horizontally. Torsional irregularity is deemed to exist if: 75% of the lateral strength at a story is located on one side of the center of mass or the drift in a story at the ends exceeds 120% of the average story drift. Presence of this irregularity requires explicit consideration of inherent and accidental torsion when determining required strength and story drift and strengthening of some elements of the seismic force-resisting system. It can also affect the assessed redundancy factor.

- Reentrant corner irregularity: This is a geometric condition that occurs when a building with a rectangular plan shape has a missing corner or when a building is formed by multiple connecting wings. Figure 4 illustrates this irregularity.

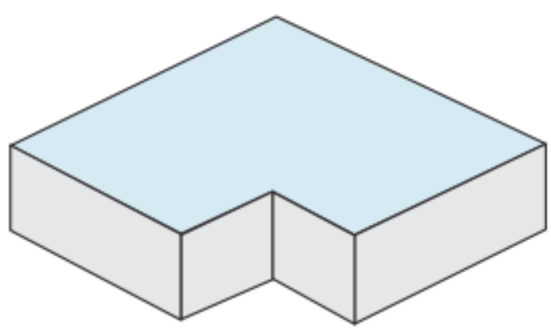

Figure 4. Reentrant corner irregularity

- Diaphragm discontinuity irregularity: This occurs when a floor or roof has a large open area as can occur in buildings with large atriums. Figure 5 illustrates this irregularity

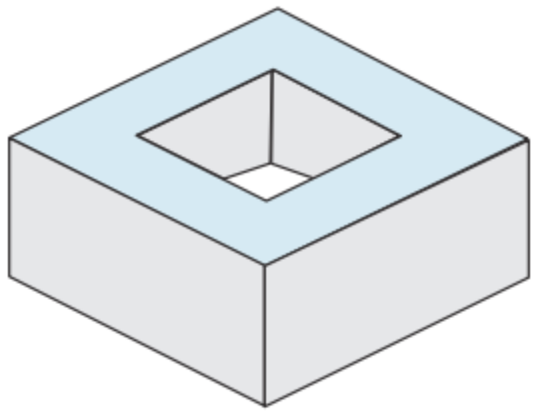

Figure 5. Diaphragm discontinuity irregularity

- Out-of-plane offset irregularity: This occurs when the vertical elements of the SFRS, such as braced frames or shear walls, are not aligned vertically from story to story. Figure 6 illustrates this irregularity

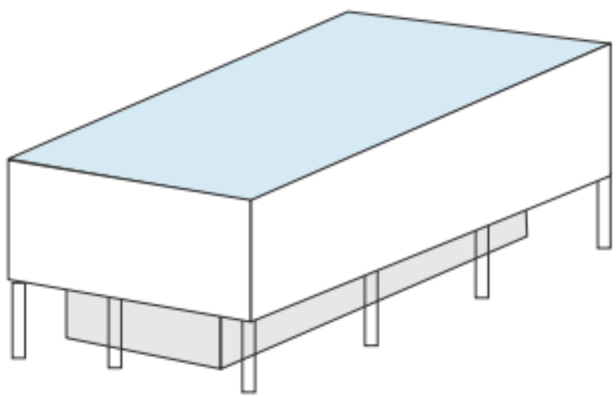

Figure 6. Out-of-plane offset irregularity

- Nonparallel systems irregularity: This occurs when the SFRS does not include a series of frames or walls that are oriented at 90-degree angles with each other.

NOTE: ASCE/SEI 7-22 removes the extreme torsional irregularity, a type of horizontal irregularity that was included in previous editions of ASCE/SEI 7.

4.2 Vertical irregularities

Vertical irregularities include the following types:

- Soft story irregularity: This occurs when the stiffness of one story is substantially less than that of the stories above. This commonly occurs at the first story of multi-story moment frame buildings where the architectural design calls for a tall lobby area. It also can occur in multi-story bearing wall buildings when the first story walls are punched with a number of large openings relative to the stories above, such as for a garage or glass storefront. Figure 7 illustrates these two conditions. An extreme soft story irregularity is deemed to exist when the stiffness of a story is less than 60% of the story above. Extreme soft story irregularity is an extreme version of the soft story irregularity that is prohibited in SDC E and SDC F structures.

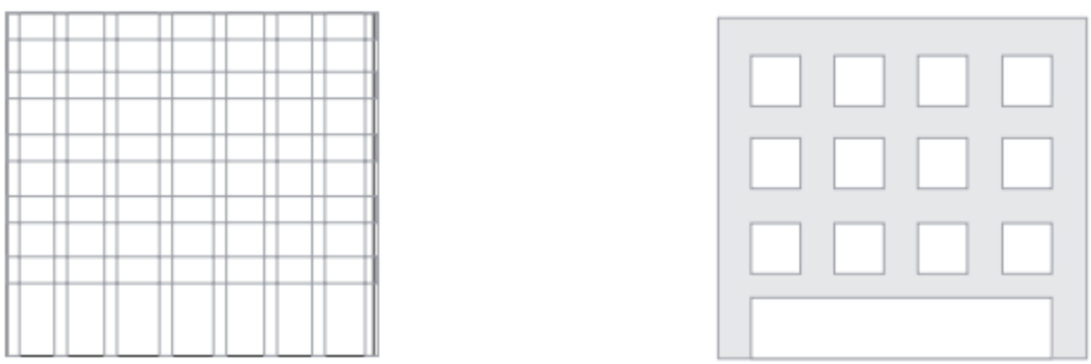

Figure 7. Examples of buildings with a soft first story, a common type of stiffness irregularity.

- Vertical geometric irregularity: This occurs where the width in plan of the SFRS is more than 130% larger in one or more stories than it is in adjacent stories. Figure 8 illustrates this condition.

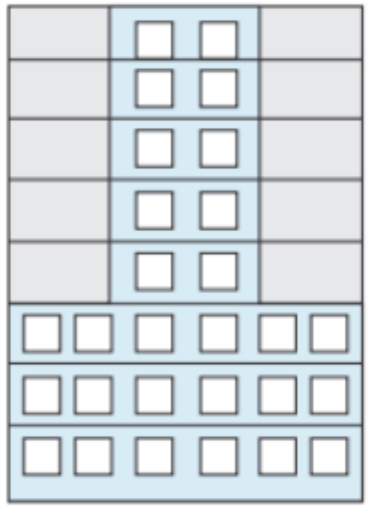

Figure 8. Vertical geometric irregularity.

- In-plane discontinuity irregularity: This occurs when the vertical elements of the SFRS, such as its walls or braced frames, do not align vertically within a given line of framing or the frame or wall has a significant setback. Figure 9 provides an example of this irregularity.

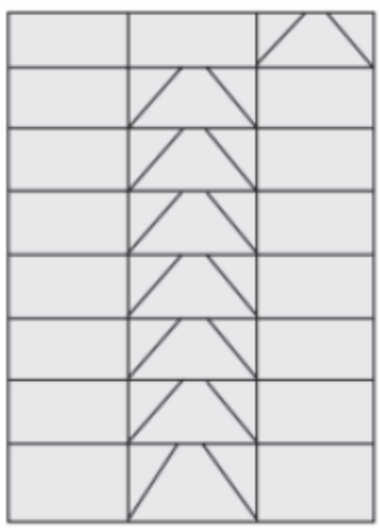

Figure 9. Example of an in-plane discontinuity irregularity

- Weak story irregularity: This occurs when the strength of the walls or frames that provide lateral resistance in one story is substantially less than that of the walls or frames in the adjacent stories. This irregularity often accompanies a soft-story irregularity but does not always. It is prohibited in SDC E and F structures, where the story stiffness of one story is less than 80% of the story above. Extreme weak story irregularity is an extreme version of the weak-story irregularity that is prohibited in SDC E and SDC F structures.

5. Calculate Seismic Loads

ASCE/SEI 7 requires that structures have adequate strength to resist specified design earthquake forces in combination with other loads. Earthquake shaking induces both horizontal and vertical forces in structures. These forces vary during an earthquake and, for brief periods ranging from a few tenths of a second to a few seconds, they can become very large. In structures assigned to SDC D, E, or F, these forces easily can exceed the forces associated with supporting the building weight and contents. In keeping with the basic design philosophy of accepting damage but attempting to avoid collapse, the design seismic forces specified by ASCE/SEI 7 are less than those which would enable a structure to remain undamaged by design earthquake shaking.

Typically, engineers design structures so that only some of the structural elements (e.g., beams, columns, walls, braces) and their connections provide the required seismic resistance. For example, the braced frame structures in Figure 2: a, b, c, e, and f each have three bays, but only one of the bays has bracing. The braces, beams, and columns that the braces connect to would be designed to resist seismic forces, while the other beams and columns would not. The ASCE/SEI 7 standard specifies the magnitude of earthquake design forces and the required combinations of seismic forces with other loads, including dead and live loads that must be used to design the SFRS.

The magnitude of the specified earthquake forces and how they are calculated depends on the SDC, the type of structural system that is used, the configuration of the structure, and the type of element or connection being designed. These are described briefly below. For SDC A structures, ASCE/SEI 7 simply requires that structures be designed with adequate strength to resist 1% of the weight of the structure, applied as a lateral force in each direction, at each level.

NOTE: ASCE/SEI 7-22 Chapter 2 specifies the required combinations of seismic loads with other design loadings, including dead and live.

5.1 Base Shear Strength

Figure 10(a) displays a simple, multiple degree of freedom structure consisting of a single cantilever column with masses, having weights W1, W2, and W3, lumped at 3 levels. If the top mass of the structure is displaced to the side, as illustrated in Figure 10(b), then released, the structure will respond in free vibration, holding the deformed shape illustrated. This shape is termed the fundamental mode shape of the structure.

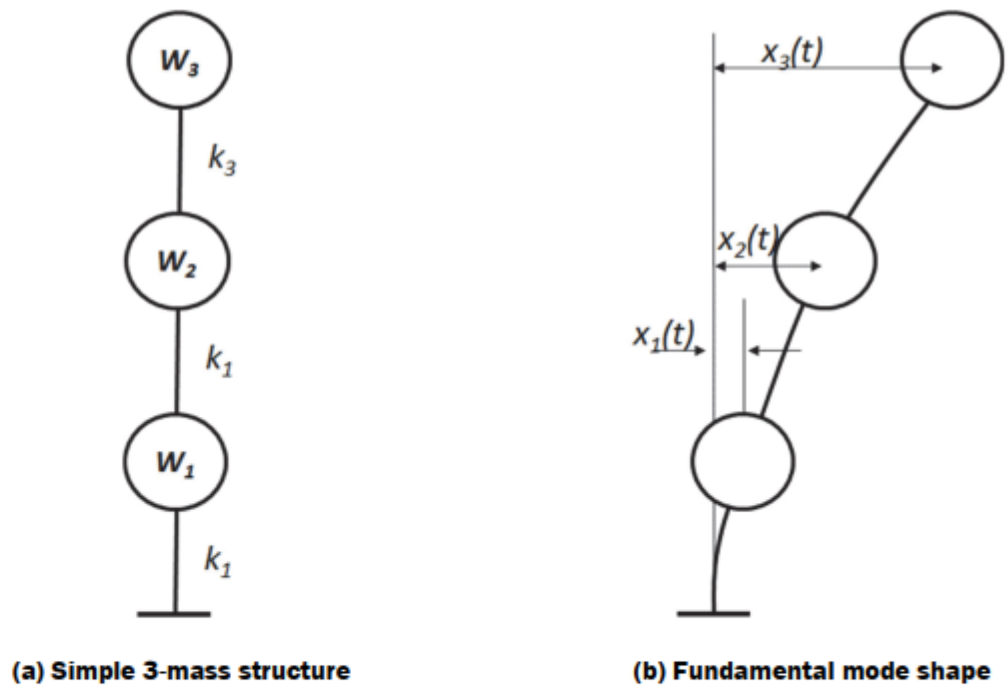

Figure 10. Simple multi-degree of freedom (MDOF) structure in free vibration

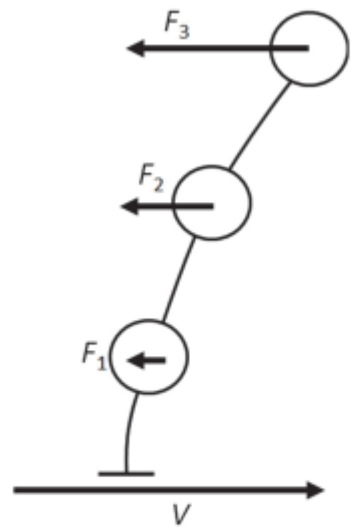

Figure 11. Forces acting on MDOF structure in free vibration.

When the ground shakes, inertial force acts on each mass, at a magnitude of acceleration times the mass, as shown in Figure 11. Because the base of the structure is not moving relative to the ground, equilibrium must be provided by a reaction force, denoted V in the figure, that is equal in magnitude to the sum of the F forces. This reaction force, V, is termed the base shear.

NOTE: The base shear equations shown here as Equations 8-1 through 8-6 appear in ASCE/SEI 7 as Equations 12.8-1 through 12.8-6.

ASCE/SEI 7 requires design of structures for a minimum base shear force, V, given by the equation:

$$V = C_S W$$

where:

C_s = the seismic response coefficient
W = the seismic weight of the structure

The seismic response coefficient, C_s, depends on the fundamental period of vibration (T) of the structure, the risk category, and the type of SFRS used. For structures with fundamental periods of vibration less than the mapped value of T_L at their site, the seismic response coefficient, C_s, is taken as the lesser of the value given by:

$$C_s = \frac{S_{DS}}{(R/I_e)}$$

$$C_s = \frac{S_{D1}}{(R/I_e)T}$$

where:
S_{DS} = the spectral response acceleration parameter obtained from the USGS online database,
S_{D1} = the spectral response acceleration parameter obtained from the USGS online database,
R = the response modification coefficient,
I_e = an importance factor, the value of which depends on the risk category, and
T = the fundamental period of vibration of the structure

The seismic weight is equal to the weight of the structure and all permanently attached nonstructural components and systems including cladding, roofing, partitions, ceilings, and MEP equipment. In storage and warehouse occupancies, W also includes 25 percent of the design storage load. For buildings with a flat roof in areas susceptible to a ground snow load of 30 pounds per square foot (psf) or more, the seismic weight also includes 20 percent of the uniform design snow load.

The quantity R/I_e in the above equations is an expression of the permissible amount of inelastic structural response or ductility. The value of R is determined from the ASCE/SEI 7 standard Table 12.2-1 based on the selected seismic force-resisting system. For buildings in Risk Category I or II, the importance factor, Ie, has a value of 1.0. For structures in Risk Categories III and IV, the importance factors are 1.25 and 1.5, respectively. Thus, for structures in higher risk categories, less inelastic behavior is permitted, which is consistent with the desired reduced risk of damage.

For structures with a fundamental period of vibration greater than T_L, the value of C_s is determined using this equation:

$$C_s = \frac{S_{D1}T_L}{(R/I_e)T^2}$$

Regardless of fundamental period, the value of the base shear coefficient for any structure, cannot be taken as less than the value obtained from the following equation:

$$C_S = 0.044 S_{DS} I_e$$

On sites close to major active faults, where ground motions can have large impulsive components, the base shear coefficient cannot be taken less than:

$$C_s = 0.5\frac{S_1}{(R/I_e)}$$

In the above equation, the parameter S_1 is the value of the 1-second MCE_R spectral response acceleration at the site assuming conditions corresponding to Site Class BC.

5.2 Redundancy

The strength design of structures assigned to SDC D, E, and F is subject to consideration of redundancy. A structure is sufficiently redundant if the notional removal of any single element in the SFRS (e.g., a shear wall or brace) does not reduce the lateral strength of the structure by more than one third and does not create an extreme torsional irregularity. If the configuration of an SFRS meets certain prescriptive requirements, a rigorous check of the redundancy is not required. If a structure does not meet these prescriptive requirements or the minimum strength and irregularity criteria described above, the redundancy factor, ρ, applies, and required strength of all elements and their connections comprising the SFRS, except diaphragms, must be increased by 30 percent.

5.3 Vertical Earthquake Forces

Structures in SDC C, D, E, and F must also be designed for the effects of vertical shaking. All members in these SDCs must be designed for vertical seismic forces, whether or not they are part of the designated SFRS. Vertical seismic load effect, E_v, can be determined from either of two equations:

$$E_v = 0.2S_{DS}D$$

$$E_v = 0.3S_{av}D$$

In the equation, S_{DS} is the horizontal design spectral acceleration at short periods and S_{av} is the vertical spectral response acceleration at short period, derived in accordance with Section 11.9.2 of ASCE/SEI 7. D is dead load.

6. Analyze and Design Structural Elements

ASCE/SEI 7 requires structural analysis to determine the strength required of each beam, brace, column, and wall of the SFRS. The code permits use of several different approaches. These include equivalent lateral force procedure, simplified equivalent lateral force procedure, modal response spectral analysis, linear response history analysis, and nonlinear response history analysis.

6.1 Equivalent Lateral Force Procedure

The most used approach is known as Equivalent Lateral Force (ELF) procedure, in which the required seismic base shear force V, is applied as a series of vertically distributed static forces to a mathematical model of the structure and the individual seismic demand, E, on each element is determined and ultimately combined with dead, live and other prescribed forces for determination of required strength.

The static seismic design force applied at each story is given by the equation:

$$F_i = \frac{w_i h_i^k}{\sum_{j=i}^{n} w_j h_j^k} V$$

Note: The above equation discussed here appears in two parts in ASCE/SEI 7-22. ASCE/SEI 7 Equation 12.8-11 computes the seismic design force at each level, F_x, as the product of a vertical force distribution factor, C_{vx}, and the base shear V. ASCE/SEI 7 Equation 12.8-11 defines the value of C_{vx} and takes the form of the above equation.

In the above equation, the superscript "*k*" has a value of unity for structures with a fundamental period, *T*, less than or equal to 0.5 second, has a value of two for structures with a fundamental period greater than or equal to 2.5 seconds, and has a value that is linearly interpolated from these values for structures with a fundamental period that falls between these values. The value of the period can be determined using either a series of approximate formula that depend on the type of SFRS used or methods of structural dynamics that directly consider the distribution of the structural mass and stiffness. The above equation is intended to represent the distribution of inertial forces associated with free vibration in the natural modes of the structure with the term $\left(w_i h_i^k / \sum_{i=1}^{n} w_i h_i^k \right)$ approximating the relative amplitude and contribution of each dominant mode shapes at each level.

The fundamental period, *T*, seismic base shear force, *V*, and individual story forces, F_i, must be computed and applied independently in the two primary orthogonal directions of response. The major vertical elements of the SFRS (i.e., frames or walls) will be aligned in these two orthogonal directions in most structures. However, when this is not the case, any two orthogonal axes may be used. The story forces, F_i, are applied as static loads, and an elastic analysis is performed to determine the distribution of seismic forces in the various beams, columns, braces, and walls that form the vertical elements of the SFRS.

In Seismic Design Category (SDC) C, D, E, and F, structures with vertical seismic force-resisting elements (e.g., shear walls, braced frames, moment frames, or combinations of these systems) located in plan such that they can experience significant seismic forces as a result of shaking in either of the major orthogonal building axes must be designed considering this behavior. An example of such a structure is one with columns common to intersecting braced frames or moment frames aligned in different directions. Another example is a structure with vertical elements aligned in two or more directions that are not orthogonal to each other. Design of these structures requires considering that forces can be incident in any direction. This requirement can be satisfied by considering 100 percent of the specified design forces applied along one primary axis simultaneously with 30 percent of the specified design forces in an orthogonal direction. When this approach is used, at least two load cases must be considered consisting of 100 percent of the specified forces in direction A taken with 30 percent of the specified forces in direction B and 30 percent of the specified forces in direction A taken with 100 percent of the forces in direction B where directions A and B are, respectively, orthogonally oriented to each other.

Structures can experience torsional excitation of their seismic force-resisting systems due to imbalance in the placement of live loads, nominal differences in strength and stiffness in lateral elements on each side of the structure, difference in arrival times of ground motion at different sides of the structure, and other effects. Consideration of accidental torsion is intended to ensure that all structures are configured with minimum torsional resistance.

The analysis of torsionally irregular structures in SDC B and all structures in SDC C, D, E, and F that do not have flexible diaphragms, typically composed of wood sheathing or untopped metal deck, must consider the effects of accidental torsion. In the ELF method, accidental eccentricity is accounted for by applying the lateral forces (F_i) at each level at a location that is displaced from the center of mass of the level by a distance equal to 5 percent of the width of the level perpendicular to the direction of application of the force. Figure 12 illustrates this concept. If the structure is not symmetrical, the 5 percent displacement of the point of application of the forces must be taken to both sides of the center of mass, and the design seismic forces on the elements must be taken as the highest forces obtained from either point of application. The purpose of this eccentric application of the forces is to account for any potential unbalanced loading that may occur if, for example, one side of a building is occupied during earthquake shaking while the other side is vacant. This requirement also is intended to ensure that all structures have a minimum amount of resistance to torsional effects.

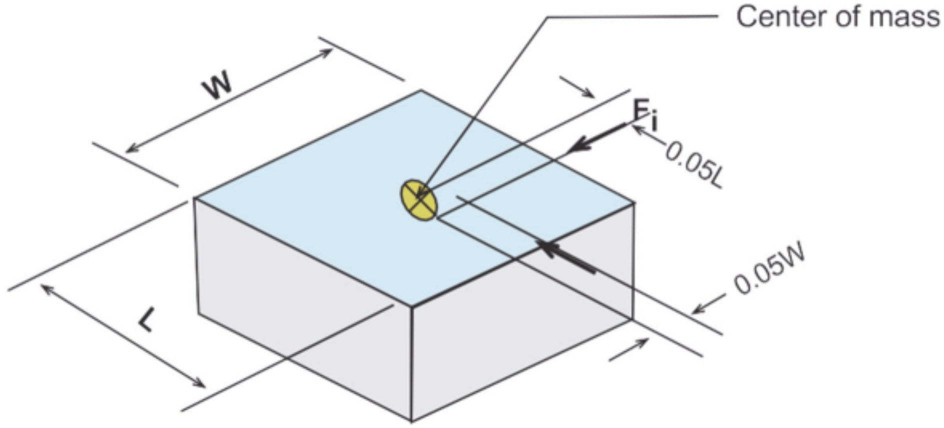

Figure 12. Eccentric application of story forces

The design seismic forces on some elements in irregular structures must be amplified by the Ω_o over strength coefficient as described earlier. The purpose of design using these amplified forces is to avoid damage to elements whose failure could result in widespread damage and collapse of the structure. An example of such an element is the column beneath a discontinuous shear wall.

6.2 Simplified Equivalent Lateral Force Procedure

The simplified ELF method is applicable to low-rise structures that have stiff seismic force-resisting systems using walls or braced frames. The base shear force equations are simplified relative to those in the standard ELF method, as is the vertical force distribution formula. Further, it is not required to compute story drift. However, the simplified method requires design for larger forces to ensure that assumptions used to create the simple procedure do not compromise safety.

6.3 Modal Response Spectrum Analysis

Modal response spectrum analysis is a hybrid between ELF and dynamic methods. MDOF structures like that illustrated in Figure 11 will have as many natural modes of vibration as they have individual dynamic degrees of freedom. For this purpose, a dynamic degree of freedom can be thought of as a unique direction of motion associated with an individual mass. The simple two-dimensional MDOF structure illustrated in Figure 11 has three dynamic degrees of freedom consisting of independent lateral translation of each of the three masses. If that structure were three-dimensional, rather than two dimensional, it would have nine dynamic degrees of freedom consisting of two orthogonal directions of lateral displacement and one of twisting about a vertical axis for each mass.

Figure 13 illustrates the three mode shapes for the three independent natural modes of vibration for the two-dimensional structure previously illustrated in Figure 11. When subjected to earthquake ground motions, each of these modes will be excited, resulting in inertial forces on the masses, in each mode, as also illustrated in the figure. In modal response spectrum analysis, the engineer creates a mathematical model of the structure that includes a representation of the structural geometry, stiffness, and mass. This model is used to determine the natural periods of vibration for each mode and the mode shapes. These data are then used, together with the design response spectrum for the site to determine the modal forces on the structure. The structure is then analyzed for these modal forces to determine the force in each member and connection due to each mode of response. Finally, these forces are added, typically using a square root sum of squares approach, to estimate the likely maximum combined forces, considering that the maximum forces in each mode are unlikely to occur simultaneously. Finally, the forces are scaled up such that the base shear, V, obtained from the analyses is not less than the base shear obtained from ELF analysis. While modal response spectrum analysis has the advantage that it can result in less conservative design forces for some structures, the root sum of squares combination of force results in a loss of sign for forces and displacements, which some engineers find to be disadvantageous.

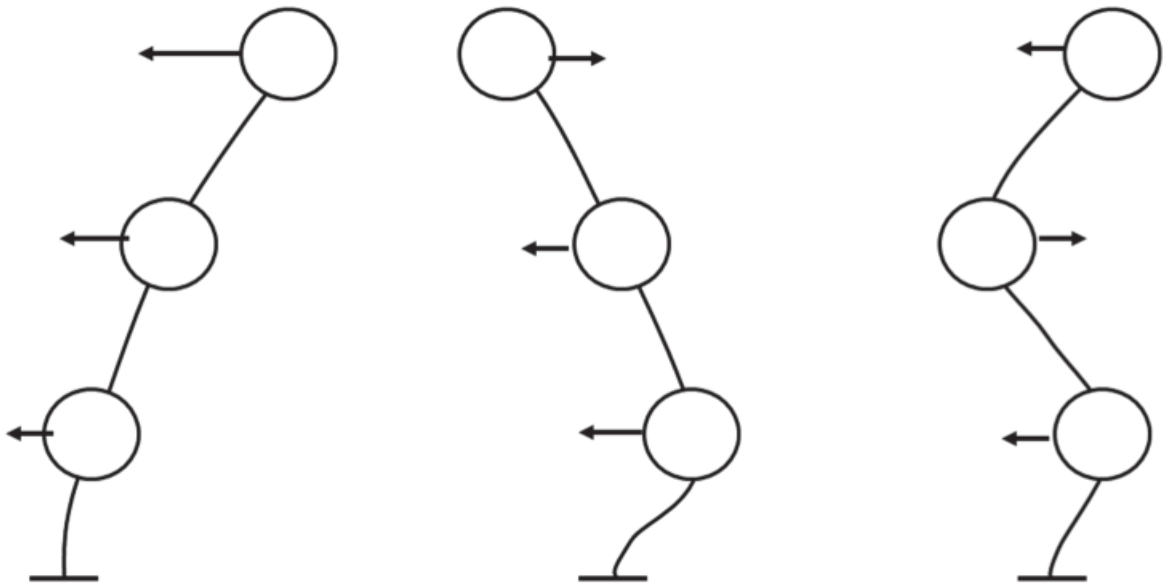

Figure 13. Mode shapes and inertial forces associated with free vibration of MDOF structure

<u>NOTE:</u> New in ASCE/SEI 7-22: Removal of Required Design Strength Reduction for Modal Response Spectrum Analysis

Modal response spectrum analysis was once thought to be superior to ELF in its ability to predict structural response. Under earlier editions of ASCE/SEI 7, a reduction in the required design strength was permitted with this technique. However, reliability studies indicated that this strength reduction resulted in structures that could not perform as well as structures designed using the ELF method, so the reduction was removed in ASCE/SEI 7-22.
ASCE/SEI 7-22 Section 12.9.1 covers the requirements for modal response spectrum analysis.

6.4 Linear Response History Analysis

Linear response history analysis is another technique permitted by ASCE/SEI 7. In this approach, a mathematical model of the structure is constructed and subjected to a suite of ground motions that have been scaled or matched to the design response spectrum for the site. The computer software used for this analysis does a numerical integration of the equation of motion for the structure, and the maximum forces in each member and connection for each ground motion are determined. These are averaged, and then scaled such that the total base shear force, *V*, is equal to that obtained from the ELF method. Finally, as with the ELF methods, these forces are combined with the forces from dead, live, and other loads. Linear response history analysis is advantageous, relative to modal response spectrum analysis because it preserves the sign of earthquake forces and displacements, which some engineers find advantageous for connection design. Also, it can eliminate some conservatism associated with the root sum of squares summation approach used in modal response spectrum analysis. However, it is computationally complex and requires manipulation of the results form a suite of ground motions, requiring more effort.

Note: ASCE/SEI 7-2 Section 12.9.2 covers the requirements for linear response history analysis

6.5 Nonlinear Response History Analysis

Nonlinear response history analysis (NLRHA) is the fourth method of analysis permitted by ASCE/SEI 7. NLRHA is like linear response history analysis except that the stiffness of members and connections is modified throughout the analysis to simulate the occurrence of cracking, yielding, buckling and other damage. NLRHA is a complex technique that calculates the forces and deformations induced in a structure in response to a suite of earthquake records and accounts explicitly for the dynamic properties of the structure, as well as the damage caused by earthquake response. Members and connections are evaluated in two groups. Members and connections that have inherent ductility and an ability to yield while continuing to carry load are evaluated based on the level of nonlinear deformation predicted by the analysis. Elements that have limited or no ductility are evaluated based on the amount of force predicted by the ground motions. It is commonly used in performance-based design approaches, high-rise buildings, and structures with energy dissipation systems or seismic isolation.

Note: To elaborate on this technique, the engineer must read the: ***NEHRP Technical Brief No. 4: Nonlinear Structural Analysis for Seismic Design:*** *This report addresses provides clear and concise guidance for conducting nonlinear structural analysis for seismic design of buildings. Published 2010.*

7. Check Drift and Stability

Unless the simplified ELF analysis procedure is used, structures must be evaluated to ensure that their anticipated lateral deflection in response to earthquake shaking does not exceed acceptable levels or result in P-delta instability. Two evaluations are required: the first is an evaluation of the adequacy of the story drift of the structure at each level, and the second is an evaluation of stability.

Story drift is a measure of how much one floor or roof level displaces under load relative to the floor level immediately below. It is typically expressed as a ratio of the difference in deflection between two adjacent floors divided by the height of the story that separates the floors. Figure 14 illustrates the concept of story drift, showing this as the quantity δi, the drift that occurs under the application of the design seismic forces.

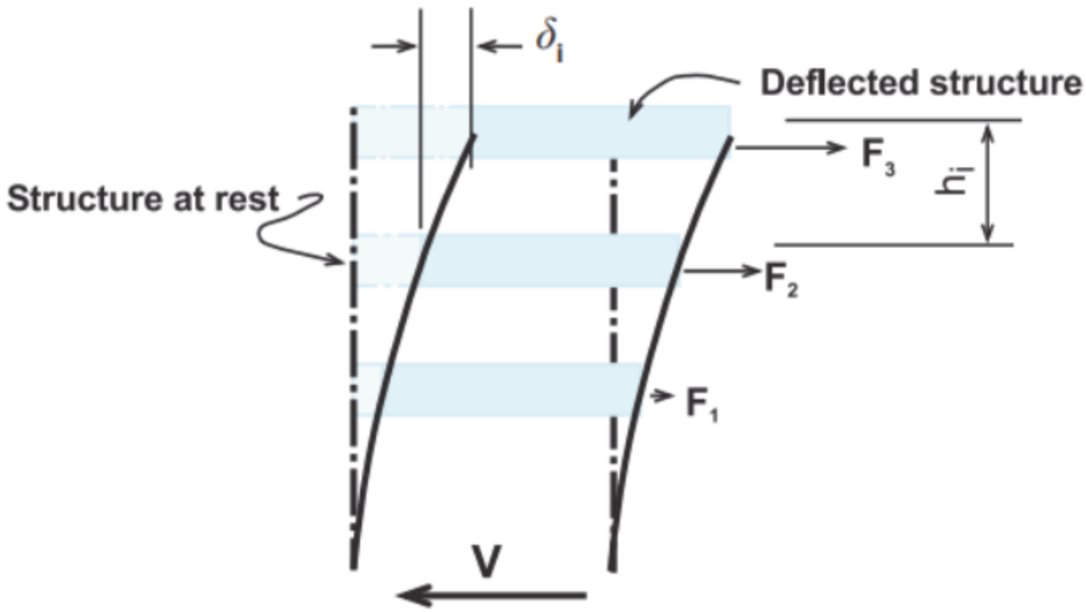

Figure 14. Story drift.

ASCE/SEI 7 sets maximum permissible story drift limits based on risk category and construction type. The adequacy of a structure in this respect is determined by calculating the design story drift, Δ using the equation:

$$\Delta = \frac{C_d \delta_i}{I_e} \leq \Delta_a h_i$$

where:
δ_i = the computed story drift under the influence of the design seismic forces,
C_d = the deflection amplification coefficient described above, and
I_e = the occupancy importance factor.

The acceptable drift ratio, Δ_a, varies from 0.007 to 0.025 depending on risk category of the building and construction type.

Drift is also an important consideration for structures constructed near one another. In response to strong ground shaking, structures located close together can hit one another, an effect known as pounding. Pounding can induce forces in a structure at the area of impact and has been known to cause the collapse of some structures. Therefore, ASCE/SEI 7 requires sufficient separation of adjacent structures and from property lines so that pounding will not occur if the structure experiences the design drifts determined using the above Equation. In addition, ASCE/SEI 7 requires evaluation of a structural stability under the anticipated lateral deflection by calculating the quantity θ for each story:

$$\theta = \frac{P_x \Delta}{V_x h_x C_d}$$

where:
P_x = the weight of the structure above the story being evaluated,
Δ = the design story drift determined using Equation 8-10,
V_x = the sum of the lateral seismic design forces above the story,
h_x = the story height, and
C_d = the deflection amplification coefficient described earlier.

If the calculated value of θ at each story is less than or equal to 0.1, the structure is considered to have adequate stiffness and strength to provide stability. If the value of θ exceeds 0.1, the lateral force analysis must include explicit consideration of P-delta effects. These effects are an amplification of forces that occurs in structures when they undergo large lateral deflection. The limiting value for θ (θ_{max}) is calculated as:

$$\theta_{max} = \frac{0.5}{\beta C_d} \leq 0.25$$

If the structure exceeds this limiting value, it is considered potentially unstable and must be redesigned unless NLRHA is used to demonstrate that the structure is adequate. In the equation for θ_{max}, β is calculated as the ratio of the story shear demand under the design seismic forces to the story shear strength. It can conservatively be assumed to have a value of 1.0. This requirement can become a controlling factor in areas of moderate seismicity for flexible structures like steel moment frames.

8. Design Diaphragms

In addition to determining the seismic forces (E) on the vertical elements of the SFRS, the building code requires determination of the seismic forces on the horizontal elements, typically called diaphragms. In most structures, the diaphragms consist of the floors and roofs acting as large horizontal beams that distribute the seismic forces to the various vertical elements. Diaphragms are categorized as being rigid, flexible, or of intermediate stiffness depending on the relative amounts of deflection that occur in the structure when it is subjected to lateral loading. Figure 15 shows the deflected shape of a simple single-story rectangular building under the influence of lateral forces in one direction. The roof diaphragm has deflection δ_L at the left side, δ_R at the right side and δ_C at its center. If the deflection at the center of the diaphragm, δ_C, exceeds twice the average of deflections δ_L and δ_R at the ends, the diaphragm must be considered flexible. ASCE/SEI 7 permits diaphragms of untopped wood sheathing or steel deck to be considered flexible regardless of the computed deflection. Diaphragms consisting of reinforced concrete slabs or concrete-filled metal deck that meet certain length-to-width limitations can be considered perfectly rigid. Other diaphragms must be considered to be of intermediate stiffness.

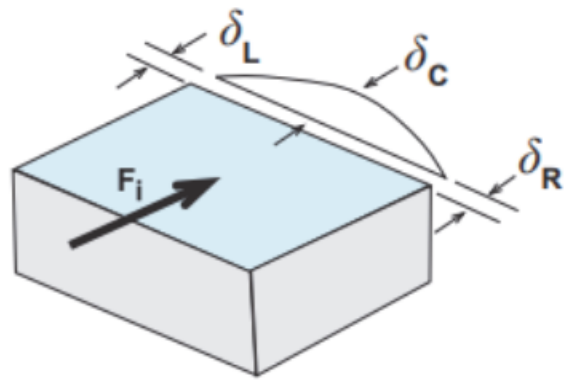

Figure 15. Deflection of diaphragm under lateral loading

A flexible diaphragm is considered to distribute forces to the supporting vertical elements of the SFRS using so-called tributary mass assumptions, much in the same way as a simple beam spanning between the vertical elements. For other diaphragms, the distribution of forces to the vertical elements of the SFRS must be considered based on the relative rigidity of the vertical elements and the diaphragms using methods of structural analysis.

Diaphragms that are not flexible must be designed for two types of forces. The fist type are the inertial forces associated with the weight of the diaphragm itself and the acceleration the building transmits to the diaphragm as the building responds to shaking. The second type are transfer forces, associated with redistribution of forces in vertical elements of the SFRS above and below the diaphragm, based on the relative stiffness of these elements. Regardless of whether diaphragms are classified as flexible, rigid or of intermediate stiffness, there are three procedures available to determine the required diaphragm inertial forces. The first of these has been in ASCE/SEI 7 for many years and uses the equation:

$$F_{px_i} = \frac{\sum_{j=i}^{n} F_j}{\sum_{j=i}^{n} w_j} w_{px_i}$$

In this equation, F_{px} is the total force to be applied to the diaphragm at level i, F_j is the seismic design force at each level j determined from this Equation (it was mentioned earlier), w_{px} is the

seismic weight of the structure tributary to the diaphragm at level *i*, and *wj* is the seismic weight at each level *j* of the structure.
The static seismic design force applied at each story is given by the equation:

$$F_i = \frac{w_i h_i^k}{\sum_{j=i}^{n} w_j h_j^k} V$$

The second approach was introduced in ASCE/SEI 7-16 and more accurately accounts for both the ability of some diaphragms to exhibit ductile behavior and the differences in shaking experienced by horizontal levels supported by different structural systems. This procedure is required for precast concrete diaphragm systems in SDC C, D, E, and F and is permitted for wood sheathed and bare metal deck diaphragms. In this procedure, design diaphragm inertial forces are determined using the equation:

$$F_{px} = \frac{C_{px}}{R_s} w_{px}$$

In the above equation, C_{px} is computed using approximate modal mass participation factors determined using a series of equations associated with different structural system types, the number of stories, the design peak ground acceleration, and the occupancy importance factor. R_s, termed the diaphragm force reduction factor, is determined from a table of values for different diaphragm types and is intended to account for the ability of some diaphragms to exhibit ductility and nonlinear deformation.

The third approach for determining inertial diaphragm forces is applicable only to single story buildings with rigid vertical seismic force-resisting elements, including steel braced frames and masonry or concrete walls, and having either wood or steel deck diaphragms without concrete topping. For these buildings, it is permitted to compute the design inertial diaphragm forces as the lesser of that obtained from theses equations:

$$F_{px} = \frac{S_{DS}}{R_{diaph}/I_e} w_{px}$$

$$F_{px} = \frac{S_{D1}}{\left(R_{diaph}/I_e\right) T_{diaph}} w_{px}$$

In the equation, R_{diaph} is a measure of the available diaphragm ductility, taken as 4.5 for wood diaphragms and specially detailed steel deck diaphragms and 1.5 for ordinary steel deck diaphragms; T_{diaph} is an estimate of the fundamental period of the diaphragm, computed based on the diaphragm span length; and other factors are as previously defined.

Regardless of the procedure used to compute diaphragm design forces, diaphragms must be designed for shear and flexure. It is common to use the analogy of a beam when designing diaphragms, where the diaphragm web is assumed to carry the shear forces and boundary elements at the diaphragm edges are assumed to resist flexure in the form of concentrated tension and compression forces, commonly called chord forces. Beams located near the edges of the diaphragms can be designed and connected to carry these chord forces, in combination with other loads, or other continuous elements, such as a band of reinforcing steel can be used for this purpose

Another important diaphragm element is the collector, sometimes also called a drag strut. These elements are used to "drag" load from the diaphragm web into discrete vertical elements of the SFRS, such as isolated walls or frames. As with chords, it is common to use floor or roof support beams as collectors. Figure 16 illustrates these important diaphragm elements.

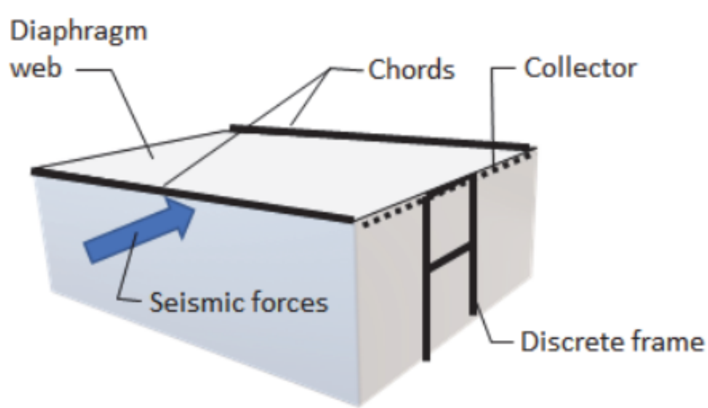

Figure 16. Diaphragm elements

Note: Useful Guidelines
The following guides provide guidance on analysis, design, and detailing of Diaphragms:

- *Seismic Design of Cast-in-Place Concrete Diaphragms, Chords, and Collectors (NIST GCR 16-917-42)*
- *Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms (NIST GCR 11-917-10)*
- *Seismic Design of Precast Concrete Diaphragms (NIST GCR 17-917-47)*

9. Detail Connections and Other Elements

The final step in the design process, once the SFRS has been designed, is to detail the structure. Detailing refers to ensuring that the details, including connections of elements, bends and spacing of reinforcing, and similar items, conform with all applicable building code requirements. Table 2-1 contains material-specific standards that contain the applicable requirements, as referenced by ASCE/SEI 7-22 Table 12.2-1, such as ACI 318, TMS 502, AISC Specifications, and the National Design Specification.

Topic Area	Standard		Publisher
New buildings	ASCE/SEI 7	*Minimum Design Loads and Associated Criteria for Buildings and Other Structures*	Structural Engineering Institute (SEI) of ASCE
Existing buildings	ASCE/SEI 41	*Seismic Evaluation and Retrofit of Existing Buildings*	Structural Engineering Institute (SEI) of ASCE
Steel	AISC 360	*Specification for Steel Buildings*	American Institute of Steel Construction (AISC)
Steel	AISC 341	*Seismic Provisions for Steel Buildings*	American Institute of Steel Construction (AISC)
Steel	AWS D.8	*Seismic Supplement*	American Welding Society (AWS)
Reinforced concrete	ACI 318	*Building Code Requirements for Reinforced Concrete*	American Concrete Institute (ACI)
Wood-frame	NDS	*National Design Specification*	American Wood Council (AWC)
Wood-frame	SDPWS	*Special Design Provisions for Wind and Seismic*	American Wood Council (AWC)
Cold-formed steel	AISI S100	North American Specification for the Design of Cold Formed Steel Structural Members	American Iron and Steel Institute (AISI)
Reinforced masonry	TMS 402/602	*Building Code Requirements and Specification for Masonry Structures*	The Masonry Society (TMS)

Table 2. Standards & codes related to design of earthquake-resistant structures

9.1 Concrete and Masonry Walls

Figure 17 illustrates some of the special reinforcement required to conform to the criteria for special reinforced concrete walls, as specified in ACI 318. These include: two curtains of vertical and horizontal reinforcement throughout the wall; closely spaced, closed stirrups or hoops in beams over the openings of walls (called coupling beams); provision of special diagonal tension reinforcement capable of carrying 100% of the seismic shear forces in coupling beams with low aspect ratios; and provision of closely spaced hoops around vertical reinforcing in those portions of concrete walls and piers that are anticipated to experience high strains during earthquake shaking (called boundary zones). TMS 503 specifies different requirements for special reinforced masonry walls because it is not possible to place the same types of reinforcing in masonry walls. To compensate for this, ASCE/SEI 7 specifies higher design forces for special masonry walls as opposed to special concrete walls.

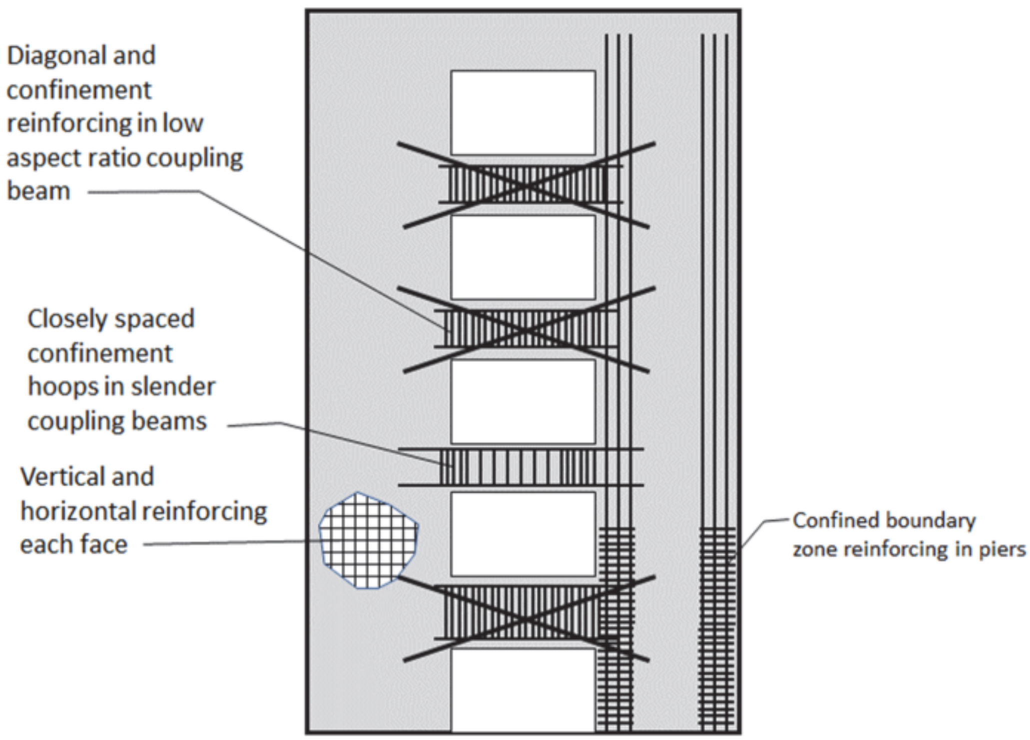

Figure 17. Typical reinforcing requirements in special reinforced concrete walls.

Note: The following guides provide specific recommendations on analysis, design, and detailing of concrete and masonry walls.

- *Cast-in-Place Concrete Special Structural Walls and Coupling Beams (NIST GCR 11-917-11)*
- *Steel Special Concentrically Braced Frames (NIST GCR 13-917-24)*
- *Special Reinforced Masonry Walls (NIST GCR 14-917-31)*

9.2 Steel Braced Frames

Steel braced frames in SDC D, E, and F must conform to the detailing requirements of AISC 341. Depending on the classification of the braced frame as ordinary, intermediate, or special, AISC 341 requires that columns, braces, and beams meet member compactness criteria, to avoid premature buckling and fracture and requires that connections of braces to beams and columns be designed sufficiently strong to develop the full expected strength of the brace in compression and tension. This ensures that the braces can buckle and yield, modes of behavior that permit inelastic deformation of the structure, while maintaining a substantial portion of its lateral resistance. In addition, gusset plate brace connections of special concentrically braced frames must be designed to accommodate the out-of-plane rotations resulting from brace buckling.

Note: Useful Resources
The following guides provide specific recommendations on analysis, design, and detailing of steel braced frames.

- *Steel Special Concentrically Braced Frames (NIST GCR 13-917-24)*
- *Buckling Restrained Steel Braced Frames (NIST GCR 15-917-34)*

9.3 Moment Frames

AISC 341 requires special compactness criteria (i.e., control of the slenderness of webs and flanges) in steel beams and columns, so that local buckling and strength degradation can be minimized. Further, AISC 341 requires the use of beam-column connection details that have been proven by testing and analysis to be capable of developing the required nonlinear behavior. AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames* provide criteria for design and construction of prequalified connection details demonstrated to have the necessary robustness.

ACI 318 requires provision of closely spaced hoop lateral reinforcement in beams, columns, and beam-column joints of special moment frames. As illustrated in Figure 8-18 these are required at the end zones of beams and columns, throughout the height of beam-column joints, and at locations where reinforcing splices occur. Hoops must have sufficient cross ties and area of reinforcement to effectively confine the concrete and prevent its crushing under large compressive strains, as well as brace the longitudinal steel against compression buckling. The shear capacity of beams must exceed the shear associated with development of flexural plastic hinges at both ends together with gravity shears. This ensures that nonlinear behavior will occur primarily through flexural, rather than shear yielding, and that concrete is sufficiently confined that during the formation of plastic flexural hinges, the concrete will retain its strength.

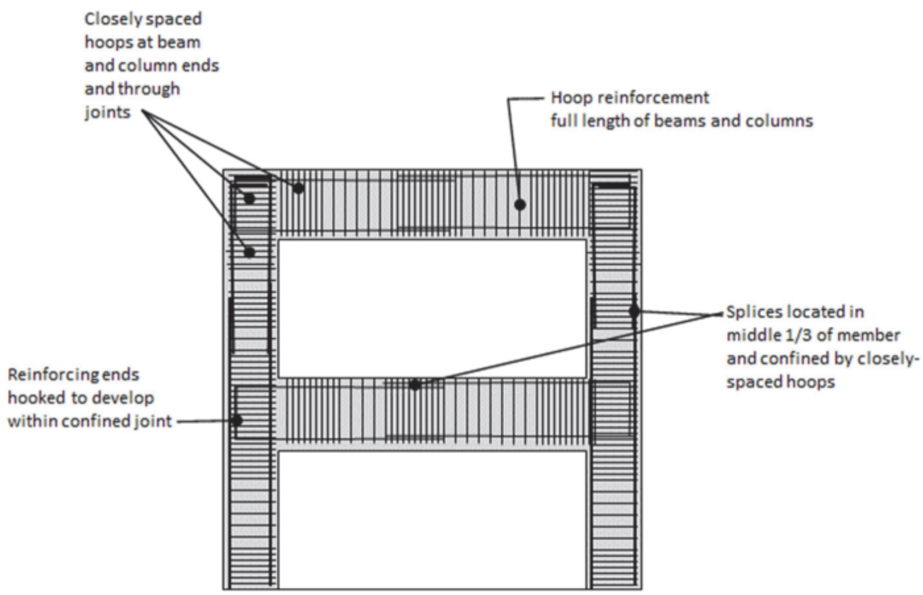

Figure 18. Typical reinforcing for special concrete moment frame.

In addition to detailing the SFRS, it is also necessary to design and detail the gravity load-carrying system to ensure that the gravity load-bearing elements have deformation compatibility with the SFRS. Particularly for special and intermediate systems, elements of the SFRS are specifically detailed to ensure that they have superior deformation capacity. In structures having these systems, the gravity system must "go along for the ride." That is, although the elements of the gravity system are not relied on to carry lateral forces, they will deflect to the same extent that the lateral system does, and as a result they will inevitably carry some lateral forces.

The same industry specifications that govern the design and detailing criteria for elements of the lateral force-resisting system also have criteria for detailing the gravity system elements for deformation compatibility. ACI 318, for example, requires that gravity load-carrying columns have sufficient ties to ensure that they can develop the shear strength associated with the seismic story drifts. Similarly, AISC 341 requires that column splices be designed with sufficient shear capacity. In some cases, it may be more economical to stiffen the SFRS to protect the gravity load-bearing elements than it is to detail the gravity elements for compatibility.

Note: Useful Resources
The following guides provide specific recommendations on analysis, design, and detailing of moment frames.

- *Reinforced Concrete Special Moment Frames (NIST GCR 16-917-40)*
- *Reinforced Steel Special Moment Frames (NIST GCR 16-917-41)*

9.4 Light-Frame Systems

Traditional light-frame construction, comprised either of wood or cold-formed steel, relies on repetitively framed horizontal members (i.e., joists or rafters) to span between load-bearing walls framed by closely spaced studs. These members transfer gravity loads between them by direct compressive bearing. For example, in Figure 19, which shows a typical section through the exterior wall of a two-story wood structure, the roof rafters and floor joists bear on the top plates of the stud walls, and the plates transfer load by bearing on the ends of the vertical studs. Detailing of such structures for seismic resistance typically requires assuring a continuous load path for shear and tensile forces, in addition to the compressive forces by which gravity loads are traditionally transferred in bearing.

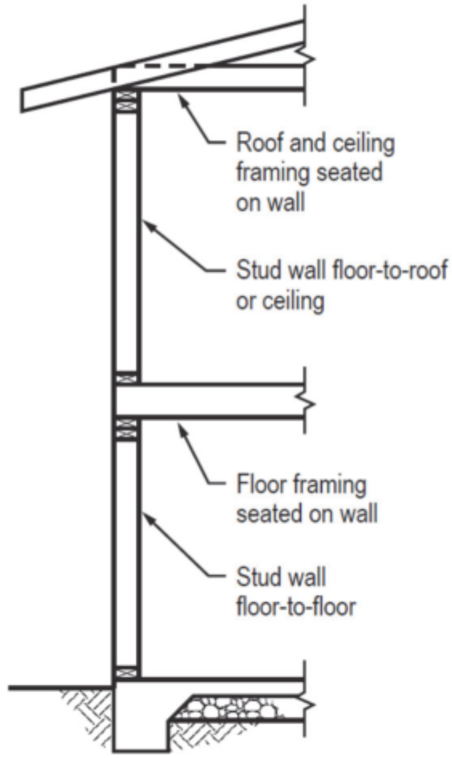

Figure 19. Section through perimeter of two-story wood framed building (NIST, 2014)

Important seismic detailing includes the use of blocking between joists on top of walls to prevent rolling of the joists and transfer shear forces from the diaphragm or walls above to walls below (Figure 21), the use of hold down-type devices at the ends of walls to resist tensile forces associated with overturning (Figure 22) and the use of blocking, together with metal straps to transfer tensile forces from one point in the structure to another (Figure 8-20). In addition, the applicable industry standards including the *Special Design Provisions for Wind and Seismic* published by the AWC and the *North American Standard for Seismic Design of Cold Formed Steel*, published by AISI specify other detailing requirements including minimum framing sizes, permissible fastener types and spacing limits, and requirements for bracing and blocking of members.

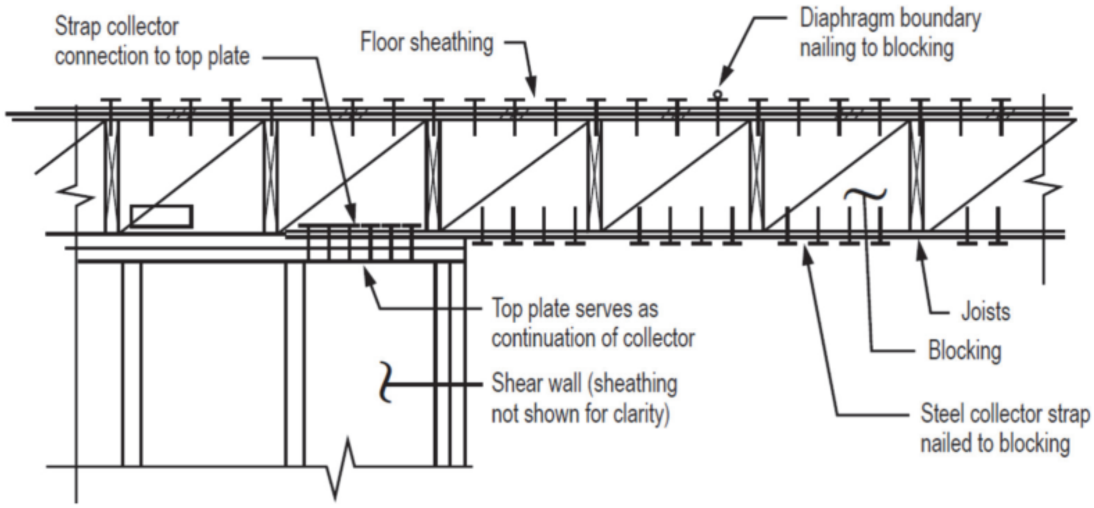

Figure 20. Use of blocking and steel straps to transfer tensile forces in the structure (from

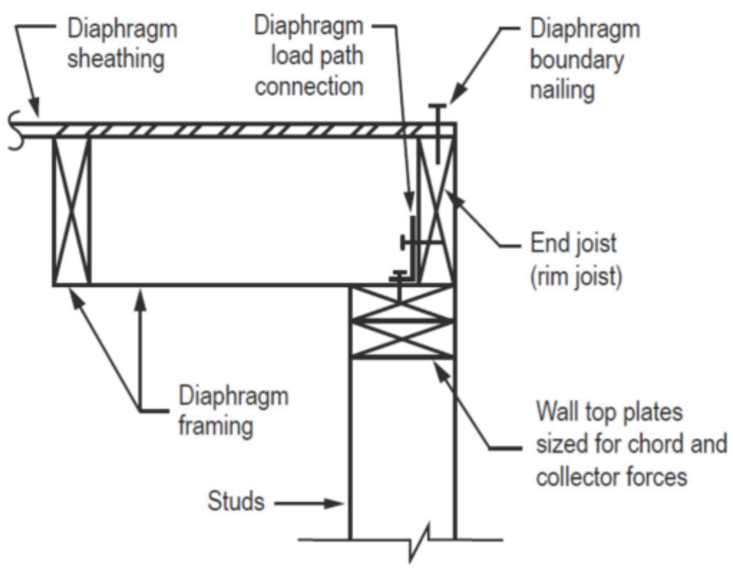

Figure 21. Use of blocking (or rim joist) to transfer shear loads and prevent joist roll-over

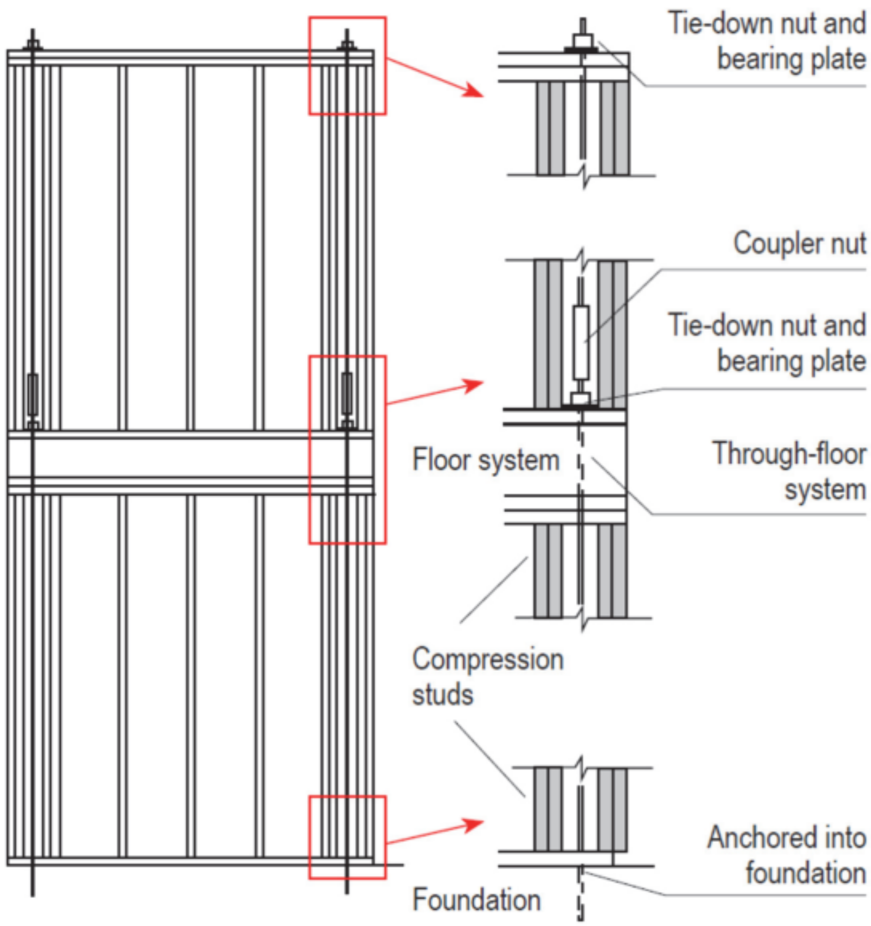

Figure 22. Use of hold down devices to resist overturning loads (from NIST, 2014).

Note: Useful Resources
The following guides provide specific recommendations on analysis, design, and detailing of light frames.

- *Wood Light-Frame Diaphragm Systems (NIST GCR 14-917-32)*
- *Cold-Formed Steel Lateral-Load Resisting Systems (NIST GCR 16-917-38)*

References

1) ASCE, 2022, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-22, American Society of Civil Engineers, Structural Engineering Institute, Reston, Virginia.

2) ASCE, 2017, Seismic Evaluation and Retrofit of Existing Buildings, ASCE/SEI 41-17, American Society of Civil Engineers, Structural Engineering Institute, Reston, Virginia.

3) AISC. Seismic provisions for structural steel buildings (ANSI/AISC 341-22). Chicago (IL): American Institute of Steel Construction, Inc.; 309 pp.

4) Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary, ACI , Farmington Hills, Michigan, USA

5) USGS, 2022d, Seismic Design Ground Motions, U.S. Geological Survey, Reston, Virginia, https://doi.org/10.5066/f7nk3c76, accessed September 24, 2022.