Seismic Design and Construction of Dwellings and Townhouses According to the International Residential Code (IRC-2024a)

Course No: S08-007
Credit: 8 PDH

Ibrahim M. Metwally, Ph.D., P.E.

CED engineering.com
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Continuing Education and Development, Inc.

P: (877) 322-5800
info@cedengineering.com

www.cedengineering.com

This course was adapted from the Federal Emergency Management Agency, Publication No. FEMA P-232, "Homebuilders' Guide to Earthquake-Resistant Design and Construction", which is in the public domain.

COURSE CONTENT

1- Introduction

This course presents a plain-language overview of design and construction provisions important to the earthquake performance of one- and two-family detached dwellings and townhouses (referred to as dwellings and townhouses in this guide) constructed under the International Residential Code (IRC) (ICC, 2024a). The primary intended audiences for this course are homebuilders, tradespeople working in the home building industry, and building department plan checkers and inspectors. Secondary audiences include architects, engineers, and homeowners with construction knowledge.

2- Fundamental Concepts of Earthquake-Resistant Design

This section explains fundamental concepts of earthquake-resistant design applicable to most buildings including dwellings and townhouses designed in accordance with the IRC. The topics covered include building response to earthquake ground shaking, site and soil characteristics, load path, and configuration irregularities.

2.1 Building Response to Earthquake Ground Shaking

It is useful to start the discussion of fundamental concepts with a basic description of how buildings respond to earthquake ground shaking. The series of drawings in Figure 2-1 illustrates how a building responds. Before the earthquake occurs, the building is stationary, resisting only the vertical or gravity loads associated with the weight of the building, its occupants, and its contents. When the ground starts to move during an earthquake, the foundation of the building moves sideways but the roof and upper stories try to remain stationary due to the inertia of the building. By the time the roof starts to move in the direction of the foundation, the foundation is already moving back towards its initial position, therefore the roof and foundation are moving in opposite directions. This cycle repeats until the earthquake ends and the building movements slow and then stop. If the shaking has been severe enough, the building may be damaged and may have a residual lean.

To perform adequately in an earthquake, a building must be both strong enough and stiff enough. Strength allows the building to resist earthquake forces, and stiffness prevents the building from deflecting too much under those forces. The forces exerted by earthquake ground motion are resisted by the strength of the building, whereas the deflection is resisted by the stiffness of the building. An analogy that can be used for this is a fishing pole. The amount the pole bends is dependent on how stiff the pole is; whether the pole breaks or not is dependent on how strong the pole is. A strong building is unlikely to fall down, but if it lacks stiffness, it could deflect significantly and sustain considerable damage as a result. Stiffness is measured in buildings in terms of the horizontal drift, a measure of deflection, in a particular story (Figure 2-2). The stiffer the dwelling or townhouse, the less it will move or deflect during an earthquake. The less a building deflects, the less damage there will be to finish materials, resulting in lower repair costs.

Actual earthquakes can generate forces (Figure 2-3, green line) considerably higher than those used for code-prescribed design (Figure 2-3, blue line). In Figure 2-3, the vertical axis is a measure of the earthquake forces experienced by the building, and the horizontal axis measures the building period, which is proportional to its height. Nevertheless, design for code earthquake forces has generally prevented loss of life and therefore satisfies the purpose of the code. Remember that the primary goal of the building code is to prevent loss of life; damage due to earthquakes should be expected. There are two items to consider relative to expected damage.

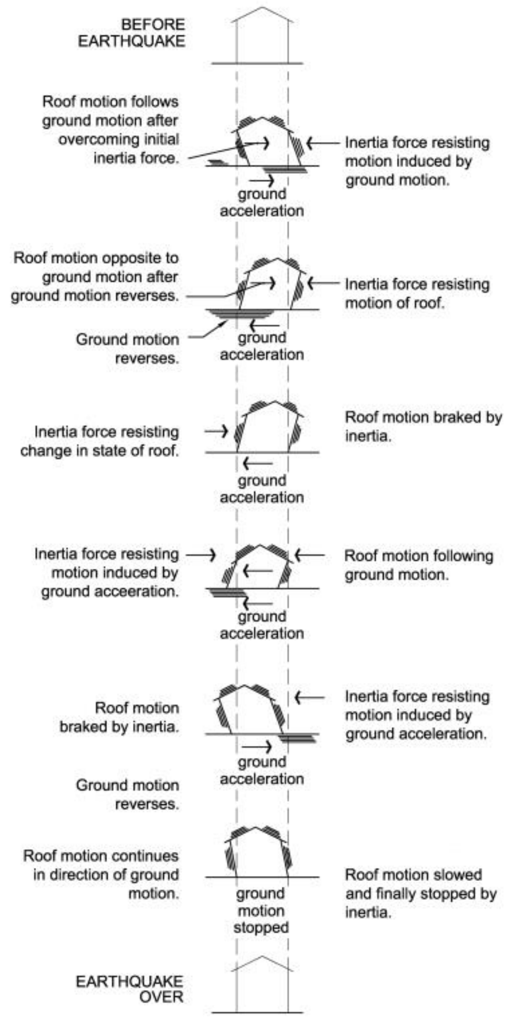


Figure 2-1 Forces induced in a dwelling due to earthquake ground motion

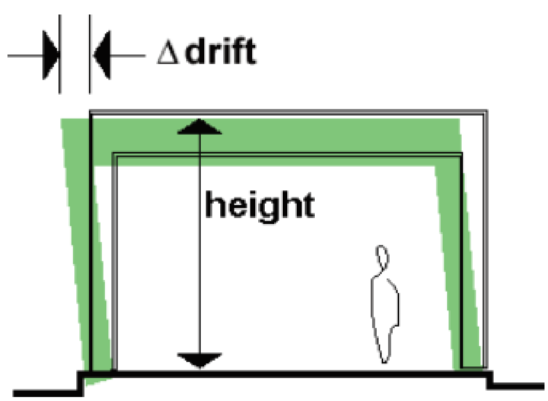


Figure 2-2 Illustration of building drift

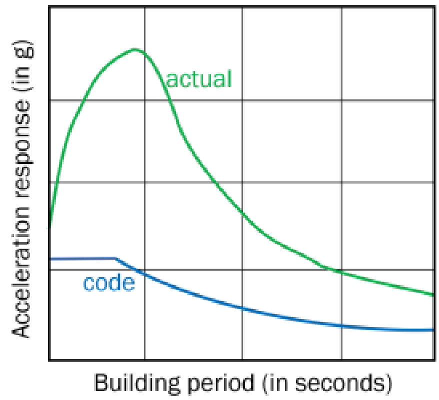


Figure 2-3 Concept of actual vs. code earthquake forces.

First, dwellings and townhouses tend to generally perform well in earthquakes even when designed to the minimum code forces because they are typically stronger than recognized in code-level design, and they often are constructed with ductile earthquake-resisting systems. In residential construction, the wall finish materials and interior walls often add significant strength, in addition to the strength provided by the required bracing materials, while the weight of floors and roofs bearing on bracing walls helps to resist overturning loads. Dwellings and townhouses constructed from ductile earthquake-resisting systems generally perform well during earthquakes because they can deform without breaking. An example of ductility is given in Figure 2-4. The ductile metal spoon simply bends while the brittle plastic spoon breaks. Non-ductile (i.e., brittle) materials like poorly reinforced concrete can break or fail without warning. Ductility is the characteristic of a material like steel that fails only after considerable deformation has occurred.

Second, it is because of expected damage that this guide presents above-code recommendations that describe techniques intended to improve the performance of a dwelling or townhouse during an earthquake and result in less damage and reduced cost of repair. Because increased stiffness also generally results in increased earthquake forces, the above-code recommendations made in this guide simultaneously increase both strength and stiffness.

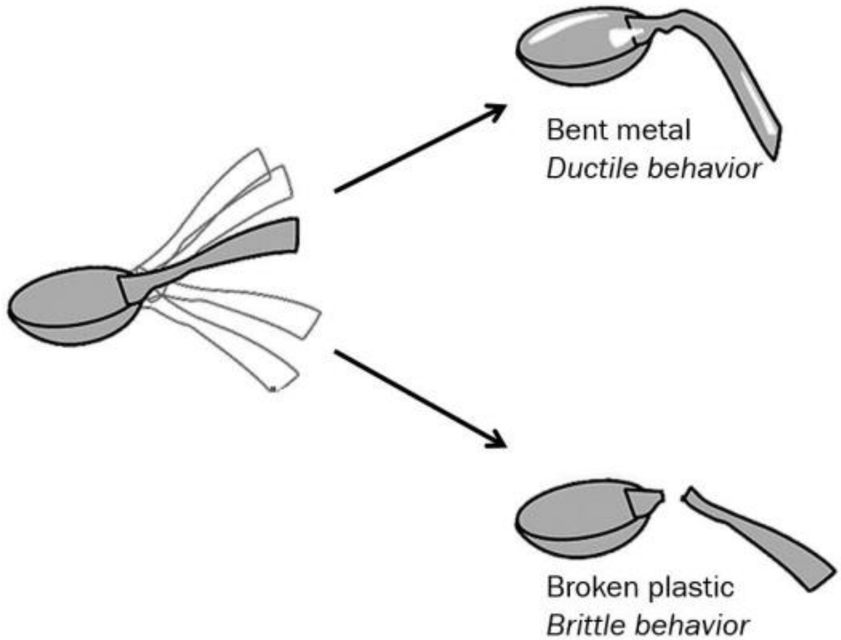


Figure 2-4 Concept of ductility.

2.2 Site and Soil Characteristics

Characteristics of the lot or site on which a building is constructed also affect how the building will perform during an earthquake. Certain types of soil amplify earthquake ground motions, which in turn amplify the shaking experienced by the building. Some types of soil slide, liquefy, densify, or settle due to earthquake ground motions, all of which will result in loss of vertical support for the building and vertical settlement. Some soils on hillsides can cause landslides that similarly result in loss of vertical support for the building. Fault rupture on a site can result in both horizontal and vertical offsets of the supporting ground. It is most desirable to build on lots or sites with stable, solid geologic formations. Deep and unbroken rock formations, referred to as bedrock, minimize earthquake damage whereas deep soft sedimentary soils result in larger earthquake forces and deflections being transferred to the dwelling or townhouse.

Sites located above known faults and in landslide-prone areas warrant special attention. No matter how well-designed, a building cannot accommodate earthquake ground motions at a site directly on top of an earthquake fault where the ground on either side of the fault moves in opposite directions horizontally or vertically. Check with the local building department or online maps published by the U.S. Geological survey (USGS) to determine where known faults are located. New buildings are generally not built within 50 feet of a known fault; when they are built in this zone, additional investigation and restrictions on building location may apply. This is partially due to uncertainty regarding where the fault rupture will propagate to the ground surface and partially because, even at a distance of 50 feet, ground shaking and resulting damage can be significant. A building damaged by direct fault movement is shown in Figure 2-5

Sites where landslides are likely to occur should be avoided. An example of damage due to a hillside collapsing in a landslide is shown in Figure 2-6. if there is concern regarding a particular building site, the owner can find further information by reviewing published landslide, liquefaction, and fault rupture maps, engaging a geotechnical engineer or geologist to inspect the site, or checking with the local building department to better understand risks associated with the site. Maps are available from the USGS and cover the entire country. In addition, some seismically active states provide site hazard mapping online.

Figure 2-5 A dwelling damaged due to fault movement in the 2014 South Napa Earthquake. (from Geotechnical Extreme Events Reconnaissance, GEER).

Figure 2-6 Dwellings damaged due to a landslide at the site during 1964 Anchorage Earthquake (from National Information Service for Earthquake Engineering, NISEE).

2.3 Load Path

For a building to remain stable, a load (i.e., force) applied at any point on the structure must have a path allowing load transfer through each building part down to the building foundation and supporting soils. The term "load path" is used to describe this transfer of load through the building systems (e.g., floors, roof-ceilings, bracing walls). To understand the concept of a load path, a house can be represented by the chain shown in Figure 2-7. The chain is pulled at the top and the load is transferred from one link to the next until it is transferred to the ground. If any link is weak or missing, the chain will not transfer the load to the ground and failure will result. Likewise, dwellings and townhouses must have complete and adequate load paths to successfully transfer earthquake loads and other imposed loads to the ground.

Figure 2-7 Chain illustrating the load path concept.

The example house shown in Figure 2-8 will be used to discuss the load path. The arrows provide a simplified depiction of earthquake or wind loads pushing horizontally on the house. While earthquake loads can also act vertically, the vertical load is small enough that it does not generally affect the behavior of dwellings and townhouses; as a result, for purposes of this guide vertical load behavior is ignored. Although wind and earthquake loads can occur in any horizontal direction, IRC design procedures generally apply the loads in each of the two principal building directions (i.e., parallel to the length and parallel to the width), one at a time, and this discussion of loading will utilize this convention.

Internally, the house has to transfer loads from the upper portions of the structure to the foundation. For the example house, this is accomplished by transferring the loads:

1. From the roof-ceiling system and its connections to the second story bracing wall system,
2. From the second-story wall bracing system and its connections to the lower floor-ceiling system,
3. From the floor-ceiling system and its connections to the first story bracing wall system, and
4. From the first-story bracing wall system and its connections to the foundation system, and
5. From the foundation system to the ground (soil).

The following discussion focuses primarily on the connections between the various building systems. The systems themselves are addressed only briefly here but are discussed in detail in Chapter 4 to Chapter 9 of this guide.

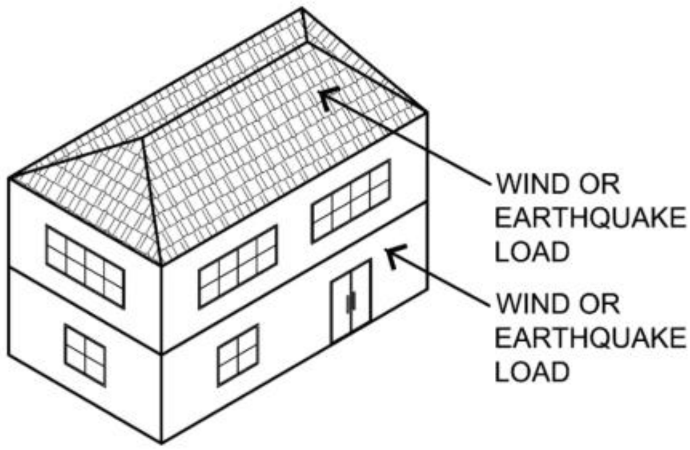


Figure 2-8 Lateral loads induced in a building due to wind or earthquakes.

2.3.1 Roof-Ceiling and Floor Systems

Roof-ceiling and floor systems are called diaphragms. In earthquakes, they behave like a beam loaded horizontally on its side. In the example house, the roof-ceiling system will resist horizontal earthquake loads generated from the weight of the roof, ceiling, and top half of the second-story walls. The series of arrows in Figure 2-9a depicts this horizontal load. The roof-ceiling system deflects horizontally under the load and transfers the load to the supporting walls at both ends. The single arrows at the roof-ceiling system ends depict the reaction loads to the supporting wall bracing. Within the roof-ceiling system, the load is carried primarily by the roof sheathing and its fastening to the roof framing. See section 8 of this course for a discussion of roof-ceiling systems.

Similarly, the floor system will resist horizontal earthquake loads generated from its weight and the weight of walls above and below the floor. As shown in Figure 2-9b, it will deflect and transfer the load to the supporting wall bracing in much the same way as the roof-ceiling system. Again, the loading is carried by the floor sheathing and fastening to the floor framing. See Chapter 5 of this course for a discussion of floor systems.

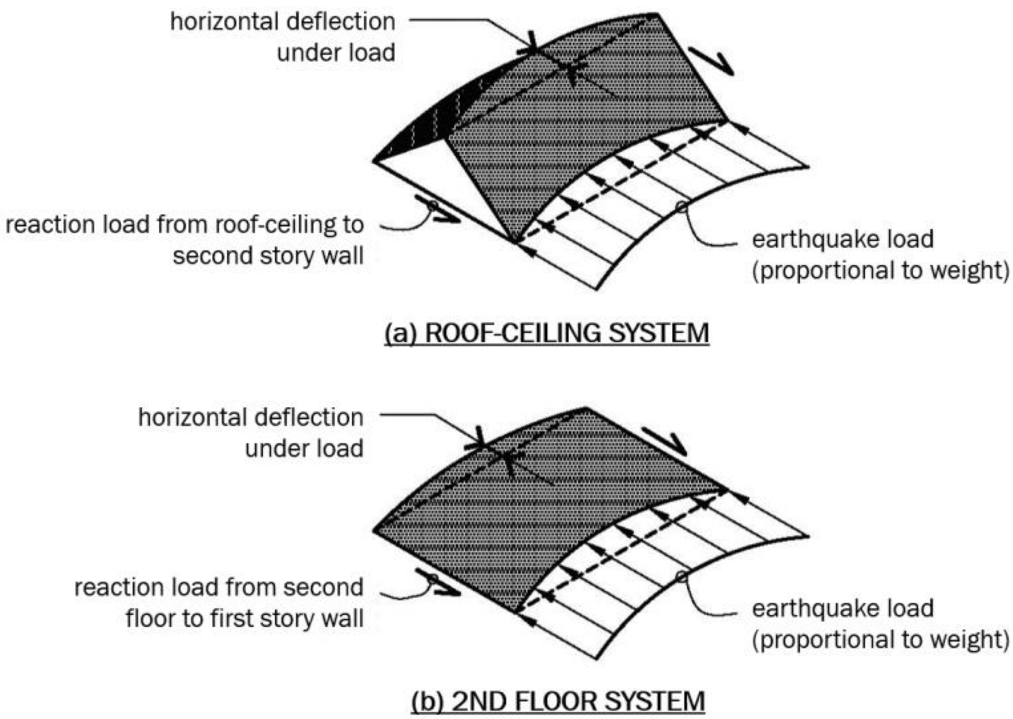


Figure 2-9 Loading and deflection of roof-ceiling and floor systems

2.3.2 Wall Bracing

The roof-ceiling reaction load is transferred into the second-story wall bracing as depicted by the arrow at the top of the wall in Figure 2-10a. The wall deflects under this load and transmits the load to the wall base and through the floor system to the first-story wall. Resistance to the wall load is provided by the wall sheathing and its fastening to the wall framing.

The first-story wall bracing resists loads from both the second-story wall and the second-story floor system as depicted by the arrow at the top of the wall in Figure 2-10b. The wall deflects under this load and transmits the load to the wall base and the foundation. Again, resistance to the wall load is provided by sheathing and its fastening. Figure 2-11 provides an exploded view of the example house that illustrates the combination of roof-ceiling, floor, and wall bracing and their connection to the foundation below.

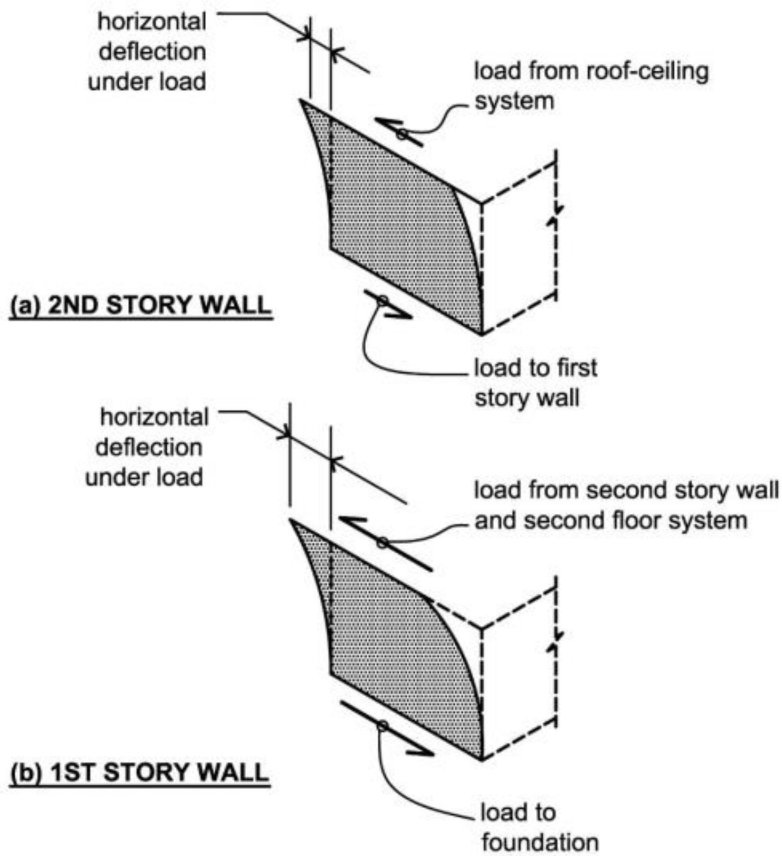


Figure 2-10 Loading and deflection of wall bracing systems

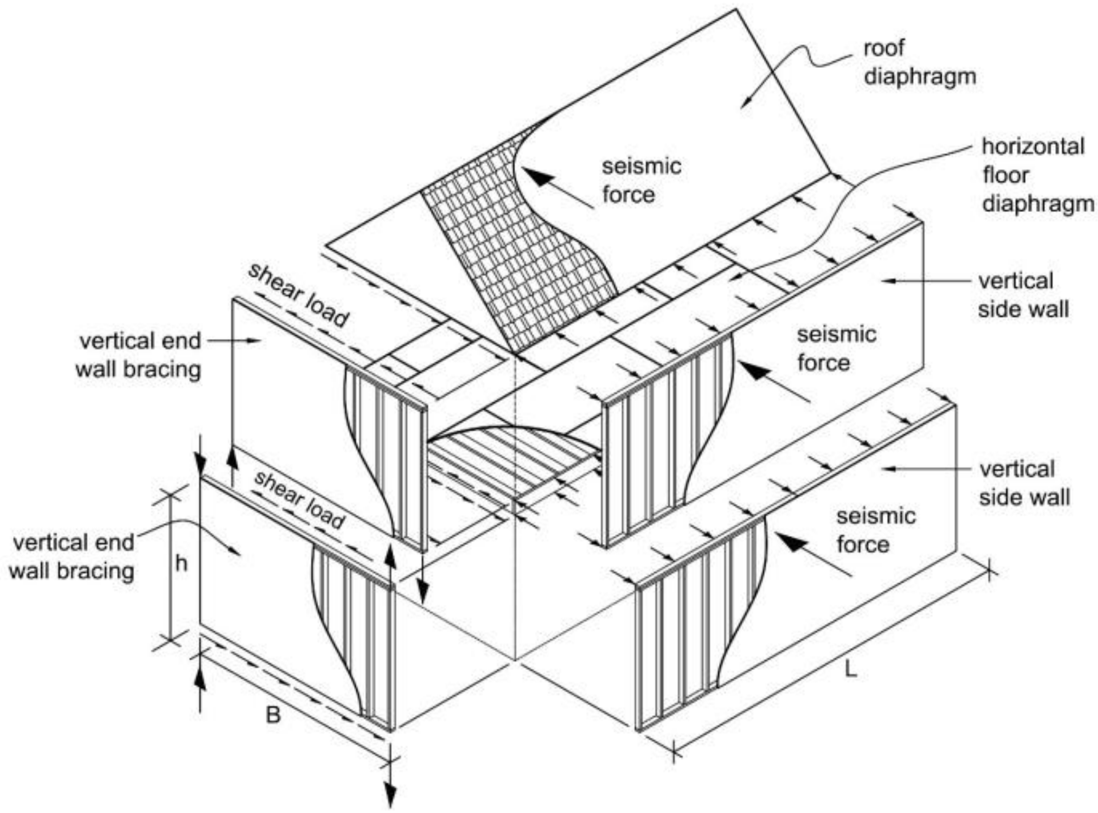


Figure 2-11 Load transfer between components in a building.

2.3.3 Connections Between Systems

A complete load path for earthquake loads requires not only adequate roof-ceiling, floor, and wall bracing but also adequate connection between these systems. Connections between systems must resist two primary types of loads: horizontal sliding loads (Figure 2-12) and overturning loads (Figure 2-13). These are discussed in Sections 2.3.4 and 2.3.5.

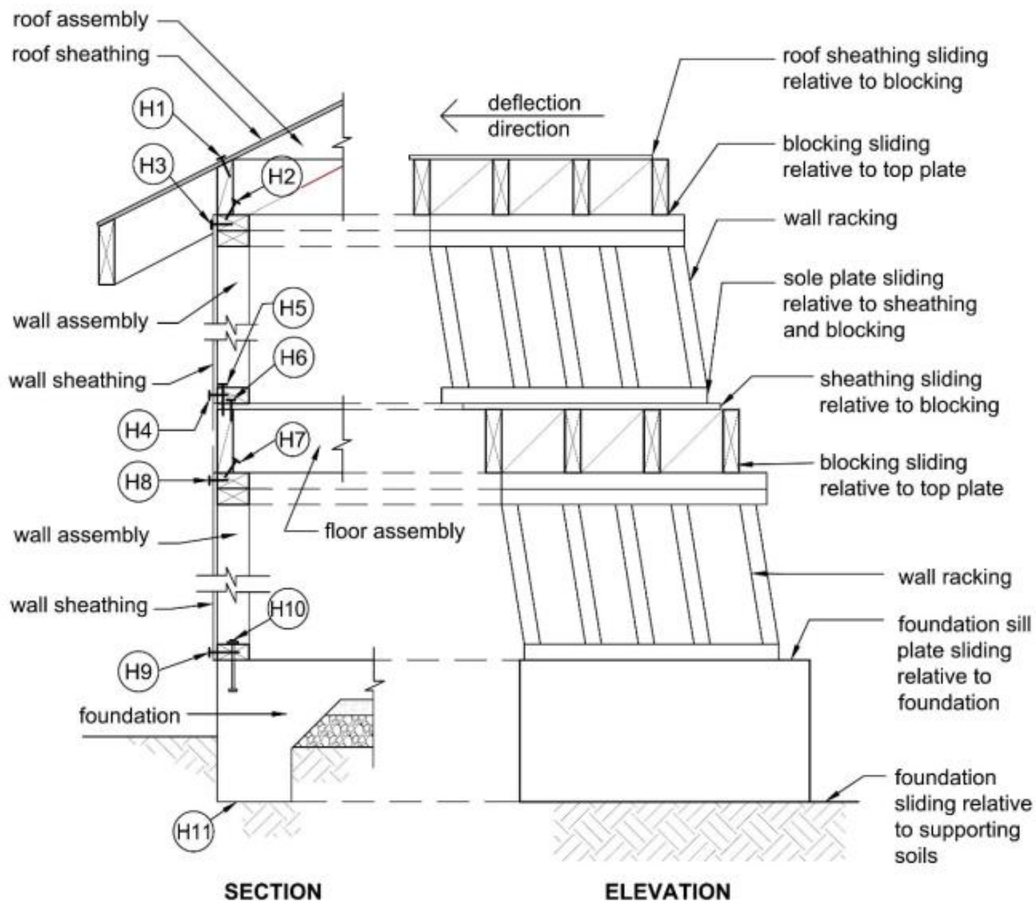


Figure 2-12 Horizontal load path connections and deformations

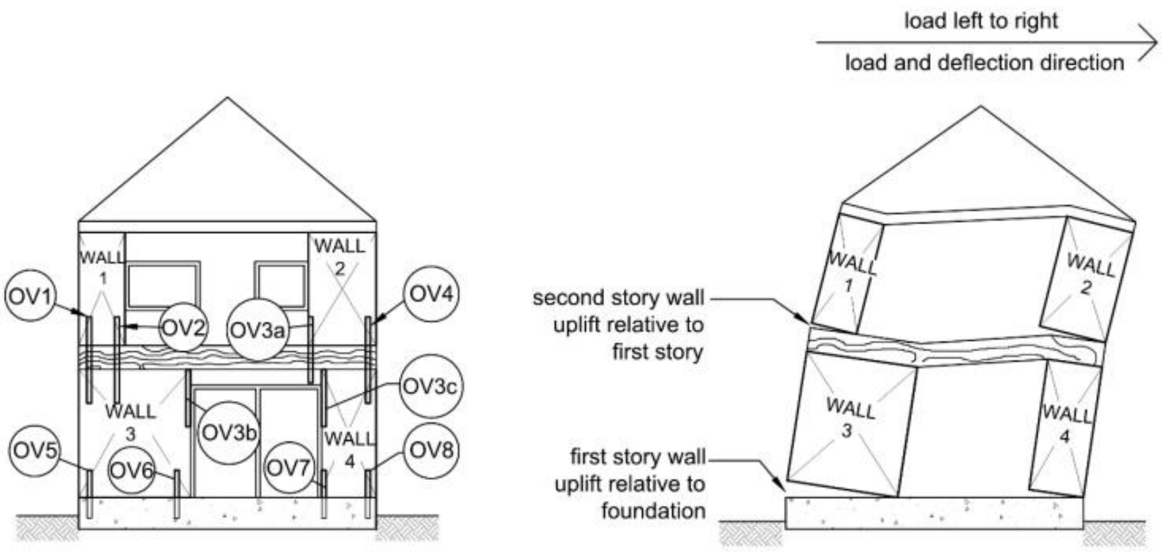


Figure 2-13 Overturning load path connections and deflections, showing load from left to right. Load will also reverse and act from right to left

2.3.4 Load Path Connection for Horizontal Sliding

Figure 2-12 depicts the end wall bracing at the left side of the house illustrated in Figure 2-8 through Figure 2-11 and provides a detailed illustration of one possible path for horizontal sliding (shear) loads from the roof assembly to the foundation. The left-hand portion of the figure shows a section through the end wall in which each of the “links” in the load path is given a number, H1 through H11 (H designates horizontal loading), corresponding to a connection or mechanism used to transfer the horizontal loads. The right-hand side of the figure shows an elevation of the same wall and illustrates the horizontal deflection that will occur if the adequate connection is not made. Section 3.4 of this guide will provide further detail of IRC fastening requirements for the Figure 2-12 horizontal load path.

2.3.5 Load Path Connection for Overturning

Because the horizontal loads are applied high on the house and resisted at the foundation, the bracing walls experience roll-over behavior, resulting in overturning loads in the bracing walls. Figure 2-13 illustrates one possible load path for overturning loads. The left-hand side of the figure shows a wall elevation in which each of the “links” in the overturning load path is given a number, OV1 to OV8 (OV designates overturning loads), corresponding to locations with uplift or downward loads due to overturning. The right-hand side of the diagram shows an elevation of the same wall that illustrates overturning deformations that will occur with earthquake loading from the left to the right. Uplift or tension occurs at one end of a wall simultaneously with downward force or compression at the other end. If not adequately anchored down, bracing walls could roll-over. Section 3.4 of this guide provides further details of IRC requirements for the Figure 2-13 overturning load path.

2.3.6 Accumulation of Loads in Systems and Connections

Wind and earthquake loads increase or accumulate towards the bottom of the dwelling or townhouse. This is particularly applicable to loads in the wall bracing systems and their connections for horizontal loads and uplift and downward loads due to wall overturning. For example, overturning connections must be sized to resist all of the loads generated above the connection location. In a two-story dwelling, the second-floor uplift connection, such as OV1 in Figure 2-13, will need to resist loads from the second story. The first-story uplift connection, such as OV5 in Figure 2-13, will have to resist the uplift loads from the second story plus the additional uplift from the first story. It can generally be expected that OV5 will need to resist a load two to four times that resisted by OV1. Downward loads at the opposite ends of walls and horizontal loads accumulate similarly.

2.4 Configuration Irregularities

The shape (i.e., configuration) of a building affects its response to wind and earthquake loads. For earthquake resistance, the ideal building has:

- ♣ A simple rectangular shape,
- ♣ Bracing walls distributed uniformly and symmetrically through the house,
- ♣ No large concentrations of weight,
- ♣ Bracing walls at upper stories located immediately above bracing walls in stories below,
- ♣ Wall bracing lengths that increase in lower story levels compared to the story above, and
- ♣ No split-levels or other floor level offsets.

A version of an ideal dwelling is shown in Figure 2-14. This ideal shape results in a uniform distribution of loads and deformations throughout the dwelling, which permits resisting elements to contribute equally to earthquake resistance. With good distribution of bracing walls, earthquake loads can be resisted very close to where they are generated, which reduces the need for transfer of earthquake loads through floor and roof systems to other portions of the dwelling or townhouse. This helps reduce the poor performance that often results when such transfers are required.

While the ideal shape is attractive from the standpoint of earthquake resistance, dwellings and townhouses with irregularities are much more common than those without. Deviations from the ideal shape are called configuration irregularities. As the configuration deviates from the ideal shape, loads and deformations are concentrated, which causes localized damage that can result in premature local or even complete failure of the dwelling or townhouse. Large open great rooms and walls of nothing but windows are examples of common configuration irregularities.

For purposes of discussion, configuration irregularities are often grouped as horizontal irregularities (variations between different areas of the floor plan) and vertical irregularities (variations from story to story). The discussion that follows will follow this pattern.

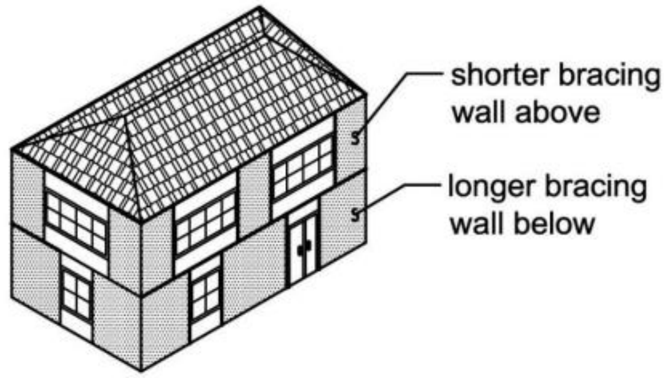


Figure 2-14 Ideal building shape for earthquake resistance.

2.4.1 Horizontal Irregularities

Horizontal irregularities concentrate earthquake load and deformation in a particular area of a dwelling or townhouse. A common cause is a center of mass at a location different from the center of the resisting elements (i.e., bracing walls). This can be due to non-uniform building weight distribution, non-uniform distribution of bracing walls, or an irregular plan. Two common examples are dwellings or townhouses with one exterior wall completely filled with windows with no wall bracing provided (Figure 2-15) and dwellings or townhouses with a large masonry chimney at one end, with the chimney creating a concentration of weight and therefore seismic force. Dwellings and townhouses with horizontal irregularities generally experience rotation in addition to the expected horizontal movement. Rotation, as illustrated in Figure 2-16, magnifies the deflection, resulting in increased damage

Other common horizontal irregularities occur in T- and L-shaped dwellings and townhouses that concentrate loads at the corners where the different wings connect. Figure 2-17 illustrates the concentration of loads in an L-shaped dwelling. The noted location of load concentration is where damage would be anticipated. Adequate interconnection of the dwelling wings is required for good performance. Without adequate interconnection, poor performance and additional damage or failure would be expected for structures with such plan irregularities.

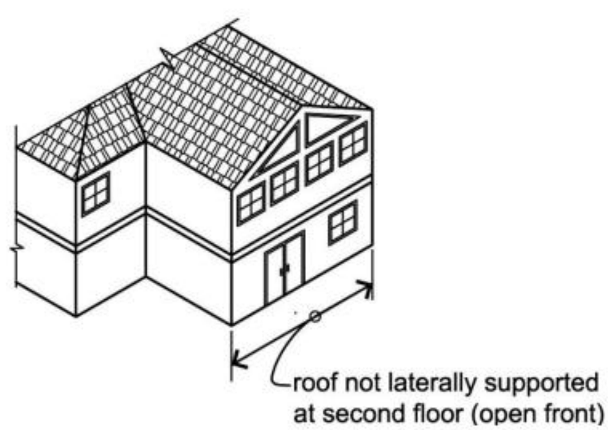


Figure 2-15 Irregular building shape with open front (no second-story bracing walls supporting roof on right-hand portion).

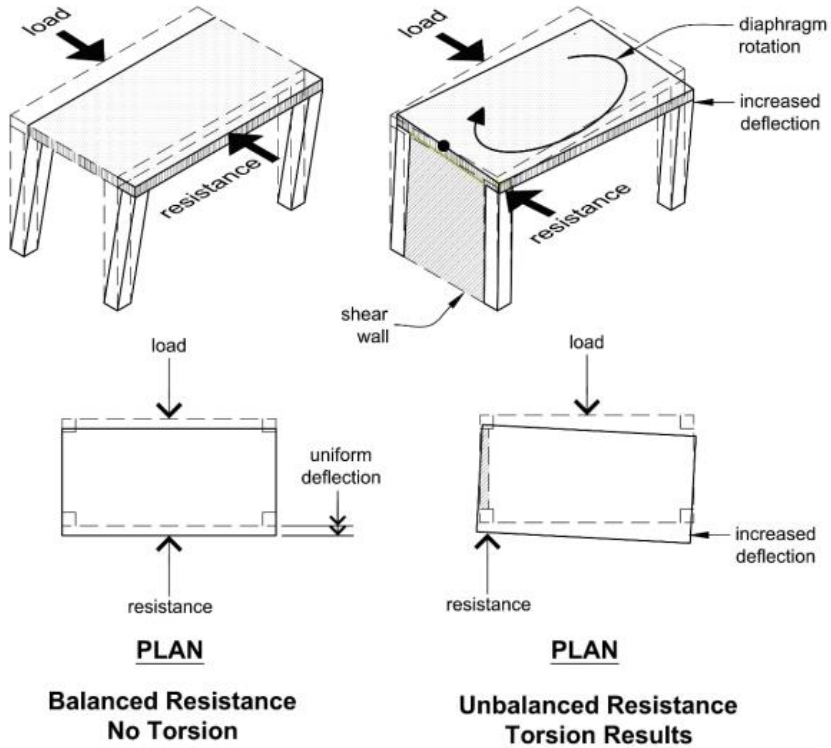


Figure 2-16 Rotational response and resistance to torsion

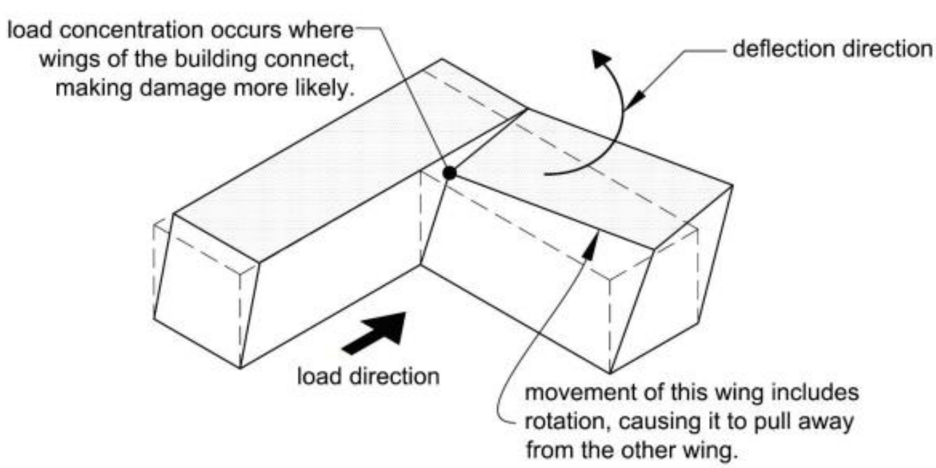


Figure 2-17 Concentration of loads and possible damage and failure due to irregular plan

2.4.2 Vertical Irregularities

Vertical irregularities concentrate damage in one story of a multistory dwelling or townhouse. This occurs when the stiffness or strength of any one story is significantly lower than that of adjacent stories. When the stiffness of a story is significantly lower, the deformations associated with the earthquake loads tend to be concentrated at that "soft" story, as illustrated in Figure 2-18. The shape on the left (normal) provides a uniform stiffness in each story, while the shape in the middle illustrates a soft story with the deflection being concentrated in that story. If the deflections get large enough, they can cause complete failure of the soft story, as illustrated in the shape on the right.

Many multi-story light-frame residential dwellings and townhouses exhibit soft/weak story behavior to some extent because the first stories feature relatively large window and door openings and fewer bracing walls than the second stories, where bedrooms and bathrooms are located. Soft/weak first story behavior also was observed in the analysis of the model house used for this guide (see Appendix C). Figure 2-19 provides an exaggerated illustration of deformation concentrated in the first story of the model house.

Cripple walls around the perimeter of a crawlspace also can result in soft/weak story behavior. Because the interior walls in the story above do not generally extend down into the crawlspace, there is less wall bracing in the crawlspace than in the story above. While IRC provisions attempt to provide adequate cripple wall strength, cripple wall stories still tend to be somewhat soft, with higher drift and potentially higher damage.

Soft/weak story irregularities have been the primary cause of story failure or collapse and earthquake fatalities in wood light-frame buildings in the United States. To date, story failure has only been observed in buildings that would not meet the current IRC bracing requirements or would fall outside the scope of the IRC. Figure 2-20 illustrates soft- and weak-story behavior in a single-family dwelling. Figure 2-21 illustrates the loss of a soft and weak story in multi-family housing. Figure 2-22 illustrates soft- and weak-story behavior in a detached dwelling with a cripple wall.

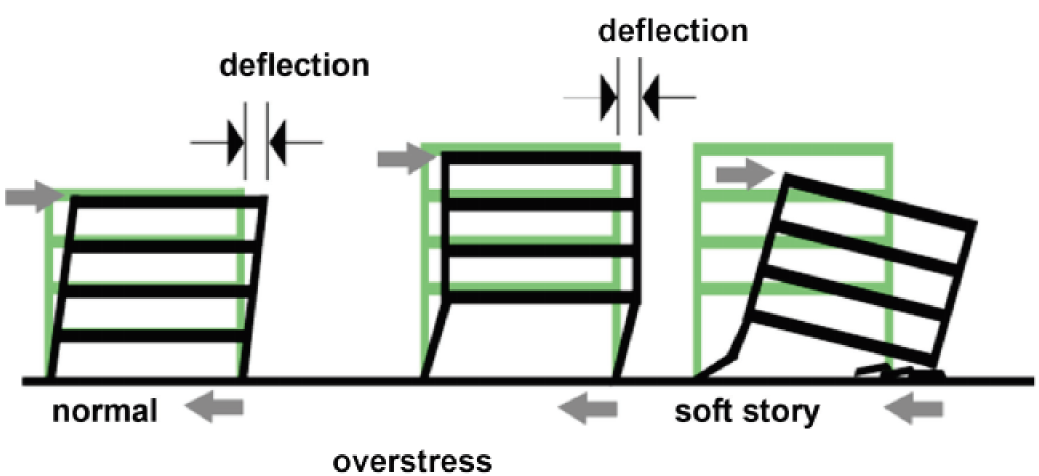


Figure 2-18 Deformation pattern due to soft-story behavior. Deformation is concentrated in The first story

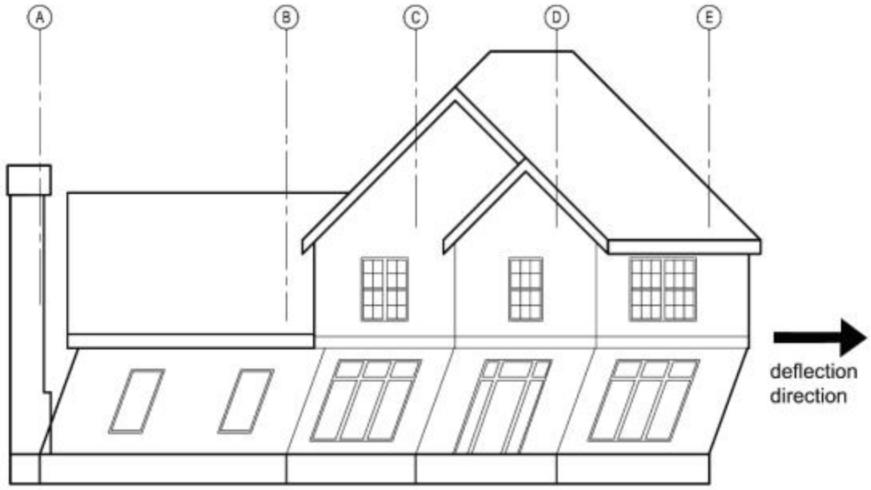


Figure 2-19 Change in deformation patterns associated with soft-story irregularity

Figure 2-20 House experiencing soft- and weak-story behavior in the 1989 Loma Prieta Earthquake (from USGS).

Figure 2-21 Multi-family housing with soft story failure in the 1989 Loma Prieta Earthquake (from NISEE).

Figure 2-22 Cripple wall failure in the 2014 South Napa Earthquake (from FEMA P-1024)

2.4.3 Additional Resources

Those interested in further reading on the topic of fundamental concepts of earthquake-resistant design is encouraged to read FEMA P-749, *Earthquake-Resistant Design Concepts* (FEMA, 2022).

3. IRC Overarching Topics

The IRC addresses the design and construction of dwellings and townhouses up to a maximum of three stories above grade plane (average finished ground level), although some materials and systems are limited to fewer than three stories. Applicability of most seismic-related provisions is determined by the SDC assigned based on the building location. The provisions of the IRC are prescriptive, meaning that instead of performing engineering calculations, the user selects applicable design and construction requirements from tables and text. The table and text provisions have been pre-engineered. In order to pre-engineer the solutions, it is necessary to establish scope limits and additional requirements for higher SDCs.

This section introduces overarching topics including SDCs, as well as the IRC seismic scoping limitations that are used throughout the IRC seismic provisions and this guide. This section also discusses use of engineered design in an otherwise prescriptive dwelling or townhouse, load path connection requirements that form the links between the building elements discussed in later chapters, and alternative dwelling construction types recognized by the IRC.

3.1 Seismic Design Category

The IRC designates the level of potential earthquake ground shaking hazard by assigning a dwelling or townhouse to an SDC based on its location. The SDCs are A, B, C, D0, D1, D2, and E, with SDC A representing the lowest level of earthquake hazard and SDC E representing the highest.

As stated in §R301.2.2, the seismic provisions of the IRC are in general applicable to dwellings and townhouses in SDC D0, D1 and D2 and to townhouses in SDC C; this is strictly true of the provisions in R301.2.2 and often but not always true for the balance of the seismic provisions scattered through the IRC. As a result, there are generally no seismic provisions applicable to dwellings and townhouses located in SDC A and B and dwellings located in SDC C; for these, use of applicable wind provisions is thought to provide acceptable seismic performance. A limited number of exceptions to this general rule of grouping by SDC can be found in the scattered seismic provisions. SDC E regions have such a high level of earthquake hazard that, with a few exceptions (see Section 3.2.1), dwellings and townhouses in these regions fall outside the scope of the IRC and must be designed using the engineering provisions of the *International Building Code* (IBC)(ICC, 2024a).

Consistent with the applicability of seismic provisions just described, the discussion and examples presented in this guide have a primary focus on dwellings and townhouses located in SDCs D0, D1, and D2, and townhouses located in SDC C. Application of the recommendations in this guide to all SDCs from SDC A to SDC E will improve the resistance of a dwelling to earthquake forces, wind forces, and possibly the effects of other natural hazards.

The IRC provides maps identifying the applicable SDC. The 2024 IRC Seismic Design Category maps, which designate SDCs for the United States and U.S. territories, are provided in Figures R301.2.2.1(1) through (7), which are reproduced for easy reference in Appendix B of the guide. The legend correlates the SDC with the horizontal earthquake acceleration expected on the dwelling or townhouse in terms of gravity (g). A value of 100% g generates horizontal earthquake forces equal to the full weight pushing horizontally (e.g., an object weighing 100 pounds experiences a 100-pound horizontal force). Web-based tools that assist with reading the IRC maps are discussed in Appendix B.

When using the earthquake maps associated with the building codes, the reader should be aware that local soil conditions have a major impact on the earthquake ground shaking hazard for each site, as discussed in Section 2.2 of this guide. The IRC map incorporates an assumed default for soil condition based on the most conservative of Site Classes C, CD, and D. If the soil conditions are known, it is possible to use the provisions of the IBC and ASCE/SEI 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures* (ASCE, 2022), to assign an SDC. The SDC assigned using the IBC provisions may be lower than that assigned using the IRC. Where poor soil conditions are known to exist, §R401.4 requires soil testing and, for a site located in SDC C or higher, also requires that the soils engineer determine the SDC. This is because a higher SDC may be applicable for very poor soil sites.

The local building department may be able to assist in determining the applicable SDC. Check with the local building department to determine if any further special regulations apply to a particular building site.

3.2 IRC Seismic Scoping Limits

Dwellings and townhouses are constructed in a wide variety of configurations using a variety of materials. It is not feasible for the IRC to address every possible variation, so scoping limits are provided to define the materials and configurations that are permitted to be constructed using the IRC. When the materials or configuration fall outside the IRC scope, an engineered design is required. IRC scoping limitations reflect the desire to provide equal earthquake performance for dwellings and townhouses designed using the IRC provisions as provided for those with an engineered design using the IBC. Use of the IRC for dwellings and townhouses that are intended to be scoped out of the IRC may result in inadequate earthquake performance.

The IRC addresses design and construction of dwellings and townhouses up to a maximum of three stories above grade plane (average finished ground level), although some materials and systems are limited to fewer than three stories (Section 3.2.7). Townhouse buildings addressed by the IRC are required to have three or more townhouse units, with each unit separated by walls extending through the full height of the building, and each unit having a yard or public way on not less than two sides (usually the front and back).

The provisions of the IRC are prescriptive, meaning that engineering calculations are not necessary. Rather, the user selects applicable construction requirements from tables and text, which have been pre-engineered. As previously discussed, the earthquake provisions of the IRC include scoping limits and additional requirements for higher SDCs.

The approaches to IRC earthquake scoping limits are varied and distributed through the various IRC chapters:

1. §R301.2.2 Includes a number of IRC seismic scope provisions and limitations,
2. Requirements limiting construction types and materials to lower SDCs are found throughout the IRC. As one example of such a limitation, wood light-frame bracing walls with let-in bracing are limited to lower SDCs per Table R602.10.3(3), and
3. Added requirements for higher SDCs are also found throughout the chapters of the IRC. As one example of an additional requirement, steel plate washers are required on anchor bolts in higher SDCs per §R602.11.1.

The balance of Section 3.2 highlights the limitations and requirements of §R301.2.2, as noted in Item 1 above. Discussion of system-specific limits, as discussed in Items 2 and 3 above, is provided in subsequent chapters.

3.2.1 Seismic Design Category E

In general, design using the IRC is prohibited in SDC E, and the user is referred to the engineered design requirements of the IBC. However, there are two methods by which dwellings and townhouses designated as SDC E in IRC Figures R301.2.2.1(1) through R301.2.2.1(7) may be designed in accordance with IRC provisions:

♣ *Method 1:* If site soil conditions are known, §R301.2.2.1.2, Item 1 permits determination of the SDC in accordance with the IBC, which may result in a lower SDC.

♣ *Method 2:* If the restrictions of §R301.2.2.1.2, Item 2 are met, dwellings and townhouses located in SDC E may be reclassified to SDC D2 and designed using the IRC provisions. §R301.2.2.1.2, Item 2 requirements dictate a regular dwelling or townhouse configurations with minimal irregularities.

3.2.2 Weight Limitations

§R301.2.2.2 specifies maximum weights for assemblies including the combined roof and ceiling, the combined floor and ceiling, exterior light-frame walls, interior light-frame walls, and veneer. Because earthquake loads are proportional to the weight of the dwelling or townhouse, an upper bound on assembly weight provides an upper bound on earthquake loads. The specified maximum assembly weights relate directly to the weights considered in developing the IRC earthquake bracing provisions. The effect of the maximum weights is the exclusion of heavier finish materials when using the IRC provisions. Where heavier finish materials are to be used, an engineered design must be provided. Note that the weight limitations of §R301.2.2.2 are applicable to roof-ceiling, floor, and framed wall assemblies that commonly occur in masonry and concrete wall dwellings and townhouses, as well as wood light-framed dwellings and townhouses.

3.2.3 Stone and Masonry Veneer Limitations

Per §R301.2.2.3, stone and masonry veneer are limited by provisions found in §R702.1 and §R703 of the IRC. These provisions regulate both the location (extent), the thickness, and the unit weight of veneer. Requirements vary as a function of SDC. Further discussion is provided in Chapter 7 of this guide

3.2.4 Masonry Construction Limitations

Per §R301.2.2.4, masonry construction in SDC D0 and D1 is limited by §R606.12.1 and masonry construction in SDC D2 is further limited by §R606.12.4. Additional seismic provisions for masonry are found in §R606,12. A notable limitation is found in Table R606.12.2.1, which limits masonry wall construction to one story buildings in SDC D0, D1, and D2. Further discussion can be found in Section 6.4 of this guide.

3.2.5 Concrete Construction Limitations

Per §R301.2.2.5, concrete exterior wall construction is permitted in SDC C in accordance with §R608. Concrete exterior wall construction in SDC D0, D1, and D2 falls outside of the IRC scope and is required to be in accordance with the PCA 100 Standard, *Prescriptive Design of Exterior*

Concrete Walls for One- and Two-Family Dwellings (PCA, 2017) or the ACI 318 Standard, *Building Code Requirements for Structural Concrete* (ACI, 2019). PCA 100 includes prescriptive design provisions, while ACI 318 must be used as part of an engineered design. Further discussion can be found in Section 6.5 of this guide.

3.2.6 Configuration Limitations

§R301.2.2.6 places limits on dwelling and townhouse configuration irregularities. These limits are discussed in Section 3.4.3 of this guide.

3.2.7 Height Limitations

§R301.2.2.7 provides limits for number of stories based on building system. Height limitations are summarized in Table 3-1. For completeness, Table 3-1 also includes previously noted information from other sections related to concrete and masonry wall buildings.

Table 3-1 Height Limitations

Building Type	**Limitations**
Wood Light-Frame Buildings	▪ Limited to three stories above grade plane or the limits of Table R602.10.3(1) or R602.10.3(3), if more restrictive ▪ Buildings in SDC D_2 exceeding two stories require design in accordance with accepted engineering practice for wind and earthquake loads
Cold-Formed Steel Buildings	▪ Limited to three stories above grade plane or the limits of AISI S230 (AISI, 2019), if more restrictive
Masonry Wall Buildings §R301.2.2.4	▪ Table R606.12.2.1 limits masonry wall construction to two story buildings for townhouses in in SDC C ▪ Table R606.12.2.1 limits masonry wall construction to one story buildings in SDC D_0, D_1, and D_2. Further discussion can be found in Section 6.4 of this guide
Concrete (ICF) Wall Buildings §R301.2.2.5	▪ Concrete exterior wall construction in SDC D_0, D_1, and D_2 falls outside of the IRC scope
Structural Insulated Panel Wall Buildings	▪ Limited to two stories above grade plane

3.2.8 Story Height Limitation

§R301.3 provides a scope limitation that is not related solely to earthquake loads but rather applies in all SDCs. This section limits story height by limiting the wall clear height and the height of the floor assembly. This serves to limit both the lateral earthquake and wind loads and the resulting overturning loads.

3.3 Engineered Design Incorporated into IRC Prescriptive Construction

§R301.1.3 allows for the inclusion of engineered structural elements in dwellings and townhouses that otherwise conform to the IRC. This is permitted for elements that exceed the limits of or otherwise do not conform to the provisions of the IRC. Examples include metal plate connected roof trusses, engineered wood joists and beams, engineered shear walls, and other items that fall outside of IRC scoping limitations. The engineering design is permitted to be limited to design of the nonconforming element, with the additional requirement that the element performance be compatible with the performance of the surrounding conventional systems. Design for compatibility should include providing a load path into and out of the element and making sure that deformation under load is consistent with the surrounding construction or can otherwise be tolerated.

The IRC also requires design in accordance with accepted engineering practice when the general earthquake limitations are not met (e.g., weight limitations, dwelling and townhouse configuration limitations, building system limitations, story height limitations). §R301.1.3 permits the design to be limited to just the elements that do not conform to the IRC limitations. Increased assembly weight and story height will globally increase earthquake loads, generally making the engineered design of the entire dwelling or townhouse necessary. Design of portions of the dwelling or townhouse is particularly applicable when an irregularity such as a cantilever, setback, or open front occurs. The extent of design is left to the judgment of the designer and building code official, with the requirement that the engineered portions be compatible with the performance of the conventional framed system. When multiple irregularities occur, an engineered design of the entire dwelling or townhouse may become necessary in order to provide adequate performance. The IRC requires that engineering design methods be used but does not specify whether this must be done by a registered design professional. State or local law governs who can perform the design; the reader is advised to check with the local building department for requirements.

3.4 Load Path Connections and Irregular Configurations

3.4 Load Path Connections and Irregular Configurations

The concepts of load path and configuration irregularities were first introduced in Section 2 of this course. Section 3.4 continues the discussion by introducing the IRC seismic provisions and limitations that address these topics.

3.4.1 Load Path Connections for Horizontal Sliding

Figure 3-1 depicts the end wall bracing at one side of a dwelling and provides a detailed illustration of one possible path for horizontal loads from the roof assembly to the foundation. The left-hand portion of the figure shows a section through the end wall in which each of the “links” in the load path is given a number, H1 through H11 (H designates horizontal loading), corresponding to a connection or mechanism used to transfer the horizontal loads. The right-hand side of the figure shows an elevation of the same wall and illustrates the horizontal deflection that will occur if adequate connection is not made. Table 3-2 provides a detailed summary of the IRC load path connections for the Figure 3-1 wall. The fastening described is largely from Table R602.3(1).

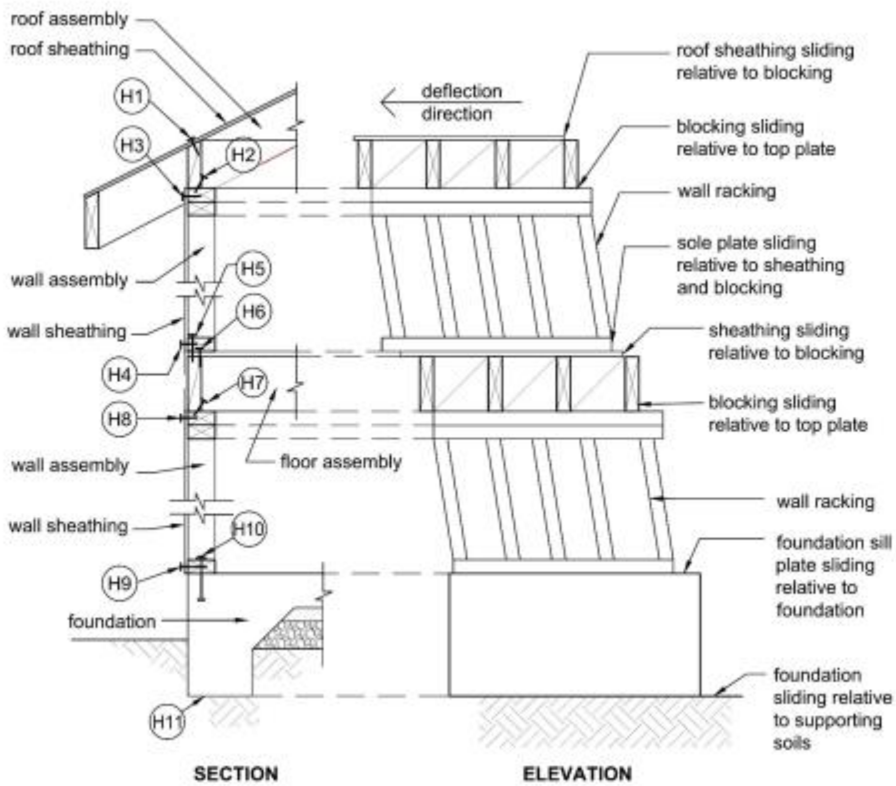
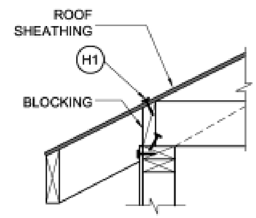
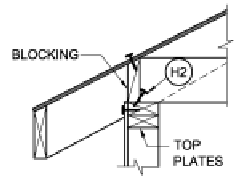
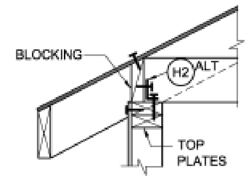
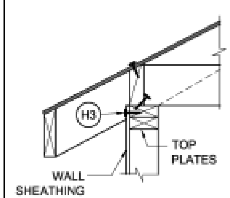
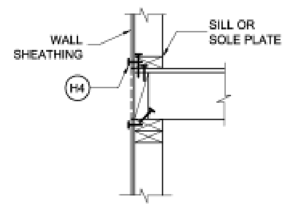
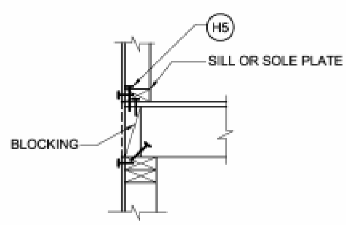
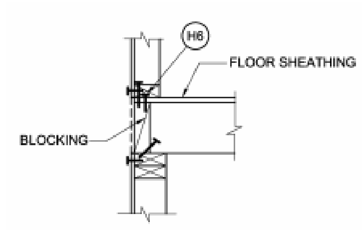
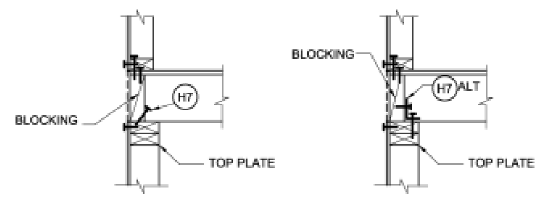
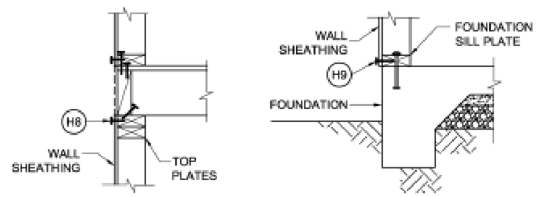
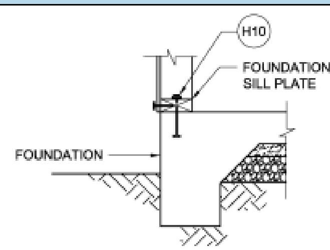
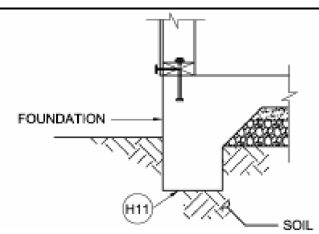


Figure 3-1 Horizontal load path connections and deformations

Table 3-2 Load Path Connections for Horizontal Sliding

Item[a]	Minimum Fastening per IRC Table R602.3(1) and Discussion[b]	Illustration
H1 (31)	Sheathing[c] — Nailing[d] 3/8" to 1/2" — 8d common @ 6" 19/32" to 3/4" — 8d common @ 6" 7/8" to 1 1/4" — 10d common @ 6" ▪ Resists roof sheathing sliding with respect to blocking below. Develops bracing strength of roof sheathing. ▪ Six-inch nail spacing applies to supported sheathing edges and blocking (including above walls). Twelve-inch spacing applies at other panel supports. ▪ Rafter blocking is not always required by IRC; however, sheathing should be nailed to blocking where blocking is provided.	
H2 (1)	▪ Four 8d box (0.113" × 2 1/2") or three 8d common (0.131" × 2 1/2") toenails each block. ▪ Resists rafter blocking sliding with respect to wall top plate. ▪ ***Use of angle clips in lieu of toenails is a recommended above-code measure.*** ▪ Rafter blocking is not always required by IRC; however, it should be fastened where provided.	 
H3 & H4 (31, 32, 33)	Sheathing[c] — Nailing[d] 3/8" to 1/2" — 8d common @ 6" 19/32" to 3/4" — 8d common @ 6" 7/8" to 1 1/4" — 10d common @ 6" ▪ Provides wall racking resistance, develops the bracing strength of the wall. ▪ Six-inch nail spacing applies to sheathing edges. Twelve-inch spacing applies at other studs.	 
H5 (16, 15)	At Braced Wall Panels ▪ Three 16d box (0.135" × 3 1/2") or two 16d common (0.162" × 3 1/2") face nails each 16 inches on center (space evenly). Between Braced Wall Panels ▪ One 16d box (0.135" × 3 1/2") face nail at 12 inches on center or one 16d sinker (0.148" × 3 1/4") face nail at 16 inches on center. ▪ Resists wall sole plate sliding with respect to sheathing and blocking or rim joist below.	

Item[a]	Minimum Fastening per IRC Table R602.3(1) and Discussion[b]	Illustration
H6 (31, 32, 33)	<u>Sheathing[c]</u> — <u>Nailing[d]</u> 3/8" to 1/2" — 8d common @ 6" 19/32" to 3/4" — 8d common @ 6" 7/8" to 1 1/4" — 10d common @ 6" ▪ Resists floor sheathing sliding with respect to blocking below, develops bracing strength of floor sheathing. ▪ Six-inch nail spacing applies to supported sheathing edges and blocking. Twelve-inch spacing applies at other panel supports.	
H7 (23)	▪ 8d box (0.113" × 2 1/2") at 4 inches on center (4 at 16") or 8d common (0.131" × 2 1/2") (3 at 16") toenails each block. ▪ Resists joist blocking sliding with respect to wall top plate. ▪ ***Use of angle clips in lieu of toenails is a recommended above-code measure.***	
H8 & H9 (31, 32, 33)	<u>Sheathing[c]</u> — <u>Nailing[d]</u> 3/8" to 1/2" — 8d common @ 6" 19/32" to 3/4" — 8d common @ 6" 7/8" to 1 1/4" — 10d common @ 6" ▪ Provides wall racking resistance, develops bracing strength of wall sheathing. ▪ Six-inch nail spacing applies to all sheathing edges. Twelve-inch spacing applies at other studs.	

Item[a]	Minimum Fastening per IRC Table R602.3(1) and Discussion[b]	Illustration
H10	▪ Anchor bolts in accordance with Sections R403.1.6 and R403.1.6.1. Steel plate washers in accordance with R602.11.1. Requirements vary by SDC. See Chapter 4 of this guide for further discussion. ▪ Resists foundation sill plate sliding with respect to slab-on-grade or other foundation.	
H11	▪ Foundation embedment in accordance with §403.1.4 provides for development of lateral bearing and friction, which permits transfer of loads between the foundation and supporting soil. ▪ Resists foundation sliding relative to soil (grade).	

[a] Item number in IRC Table R602.3(1) shown in parenthesis.
[b] Alternatives with other nail sizes are provided in IRC Table R602.3(1).
[c] Wood structural panel sheathing; see IRC Table R602.3(1) for other sheathing materials.
[d] Common nail diameter and length: 6d: 0.113" × 2"; 8d: 0.131" × 2½"; 10d: 0.148" × 3".

3.4.2 Load Path Connections for Wall Overturning

Because the horizontal loads from earthquakes are applied high on the house and resisted at the foundation, the bracing walls experience roll-over behavior, resulting in overturning loads in the bracing walls. Uplift or tension occurs at one end of a wall simultaneously with downward force or compression at the other end. If not adequately anchored down, bracing wall could roll-over. Figure 3-2 illustrates one possible load path for overturning loads. The left-hand side of the figure shows a wall elevation in which each of the "links" in the overturning load path is given a number, OV1 to OV8 (OV designates overturning loads), corresponding to locations with uplift or downward loads due to overturning. The right-hand side of the diagram shows an elevation of the same wall that illustrates overturning deformations that will occur with earthquake loading from the left to the right. Figure 3-3 provides an enlarged detail of the overturning connections seen in Figure 3-2.

The IRC only specifies connections (i.e., hold-down straps or brackets) to resist overturning loads for a small number of bracing methods. These conditions are provided in Table 3-3. Table 3-4 provides a detailed summary of the load path connections for the Figure 3-3 wall. Where overturning straps or brackets are not provided, the connectors used to resist horizontal loads in most IRC designs will be required to resist overturning loads as well. The very limited use of overturning straps and brackets is a major difference between IRC prescriptive design and engineered design, in which connectors for the specific purpose of resisting overturning loads are systematically provided.

Table 3-3 Conditions Requiring Hold-downs

Condition		IRC Reference
Wall Bracing Method	Alternate braced wall panel (ABW)	§R602.10.6.1
	Portal frame with hold-downs (PFH)	§R602.10.6.2
	Portal frame at garage (PFG) door openings in SDC A, B and C	§R602.10.6.3
	Continuously sheathed portal frame (CS-PF)	§R602.10.6.4
	Wood structural panels with stone or masonry veneer (BV-WSP)	§R602.10.6.5,2
End Conditions 2 and 5 shown in Figure R602.10.7		Figure R602.10.7
For SDC D_0, D_1, and D_2 when braced wall panels (BWP) are not at the end of braced wall lines (BWL) and when Exception 1.2 is used		§R602.10.2.2.1

Use of straps (hold-downs) to resist overturning loads is graphically depicted in Figure 3-2. However, overturning can be resisted by connections employing such other devices as bolts, nails, or hold down brackets. Because different device types may deform differently under load, it is preferable to use the same type of device for an entire story level. Variations in connector type from story to story are acceptable.

When considering overturning in an engineered design, it is customary to include the effect of dead load (i.e., weight of the house) in reducing uplift and overturning loads. However, this level of calculation detail is beyond the scope of the IRC provisions. Hold-downs should be provided wherever they are required by the IRC, irrespective of dead load.

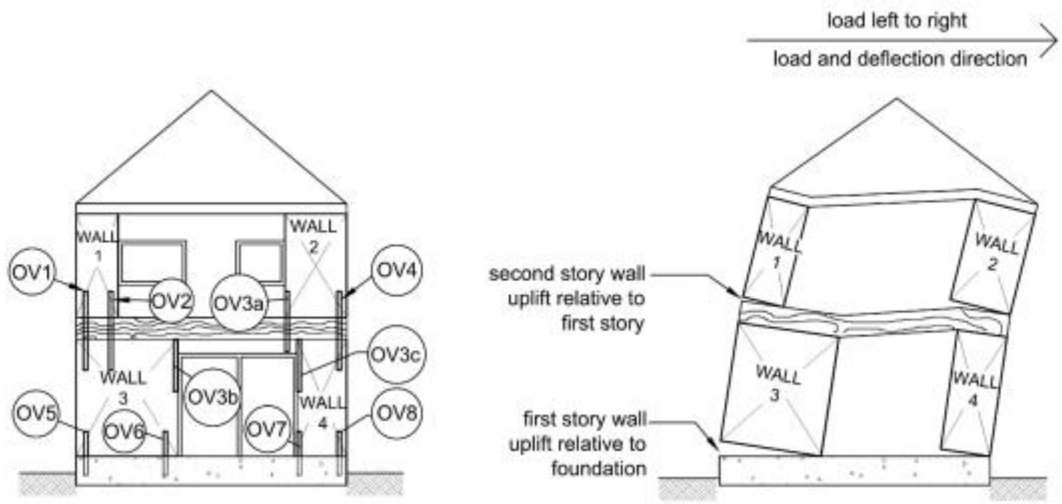
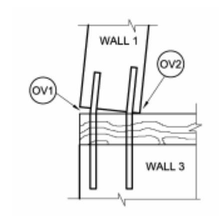
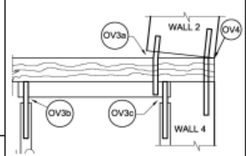
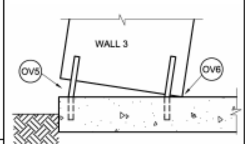
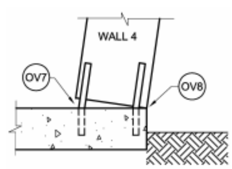


Figure 3-2 Overturning load path connections (left) and deflections (right, showing load from left to right. Load will also reverse and act from right to left.

Table 3-4 Load Path Connections for Overturning

Item	Overturning Load Path Description and Discussion	Illustration
OV1	▪ When Wall 1 is loaded from left to right, the wall tries to overturn causing the lower left corner to uplift. This illustration shows a hold-down strap restraining this uplift. The hold-down strap carries tension from an end post or studs in the second-story wall to an end post or studs in the first-story wall, which in turn must be anchored to the foundation (OV5)	
OV2	▪ When Wall 1 is loaded from left to right and an uplift load occurs at OV1, an approximately equal downward load occurs at OV2. This load will be in the post or studs at the end of the wall and will push down on the floor framing and first-story wall. This load will be transmitted through a first-story post to the foundation. ▪ When Wall 1 is loaded from right to left, there is an uplift load in the hold-down strap at OV2. Because this end of the wall is not aligned with a wall end in the first story, attention is needed to make sure that a post is added in the first-story wall for strap nailing. The first-story post can have edge nailing to the wall sheathing and transfer the uplift into the first-story wall or, alternatively, can be anchored directly to the foundation with an additional hold-down anchor. One or the other of these anchorage methods is needed to complete the load path.	
OV3	▪ When Wall 2 is loaded from left to right, the wall tries to overturn causing the lower left corner to uplift. The location of OV3 over a first-story header makes the load path more complex than OV1. Hold-down strap OV3a carries the uplift load from the Wall 2 end post to the first-story header. Because the uplift load can be larger than the minimum load on the header, straps OV3b and OV3c are shown tying the header down to the first-story posts or studs. If this is not done, it might be possible for the header to pull up. ▪ When Wall 2 is loaded from right to left, a downward load occurs at OV3a. This downward load adds to the load already in the header and the studs supporting the header. When Wall 2 extends more than a foot over the header, the condition is considered an irregularity and is subject to limitations in SDC D_1 and D_2.	
OV4	▪ When Wall 2 is loaded from left to right and an uplift load occurs at OV3a, an approximately equal downward load occurs at OV4. This load will be in the post or studs at the end of the wall and will push down on the floor framing and first-story wall. This load will be transmitted through a first-story post to the foundation.	

Item	Overturning Load Path Description and Discussion	Illustration
OV5	▪ When Wall 3 is loaded from left to right, the wall tries to overturn causing the lower left corner to uplift. This illustration shows a hold-down strap restraining this uplift. The hold-down strap carries tension from an end post or studs in the first-story wall to the foundation. The uplift load in the first-story end post or studs is a combination of the second-story uplift load from OV1 and the uplift load accumulated over the height of the first story. ▪ Hold-downs anchored to the foundation should be used only where substantial continuous foundations are provided. Hold-downs anchored to existing foundations that are weak or that do not meet current dimensional requirements require engineering guidance. Hold-downs anchored to isolated footings require engineering guidance.	
OV6	▪ When Wall 3 is loaded from left to right and an uplift load occurs at OV5, an approximately equal downward load occurs at OV6. This load will be in the post or studs at the end of the wall and will push down on the foundation. This load will be a combination of the downward load OV2 from Wall 1 and the load accumulated over the height of Wall 3. An exact engineering calculation would adjust this downward load based on the narrower width of Wall 2 and the uplift from the hold-down at OV3b.	
OV7	▪ When Wall 4 is loaded from left to right, the wall tries to overturn causing the lower left corner to uplift. This illustration shows a hold-down strap restraining this uplift. The hold-down strap carries tension from an end post or studs in the first-story wall to the foundation. The uplift load in the first-story end post or studs is a combination of the second-story uplift load from OV3c and the uplift load accumulated over the height of the first story.	
OV8	▪ When Wall 4 is loaded from left to right and an uplift load occurs at OV7, an approximately equal downward load occurs at OV8. This load will be in the post or studs at the end of the wall and will push down on the foundation. This load will be a combination of the downward load OV4 from Wall 2 and the load accumulated over the height of Wall 4.	

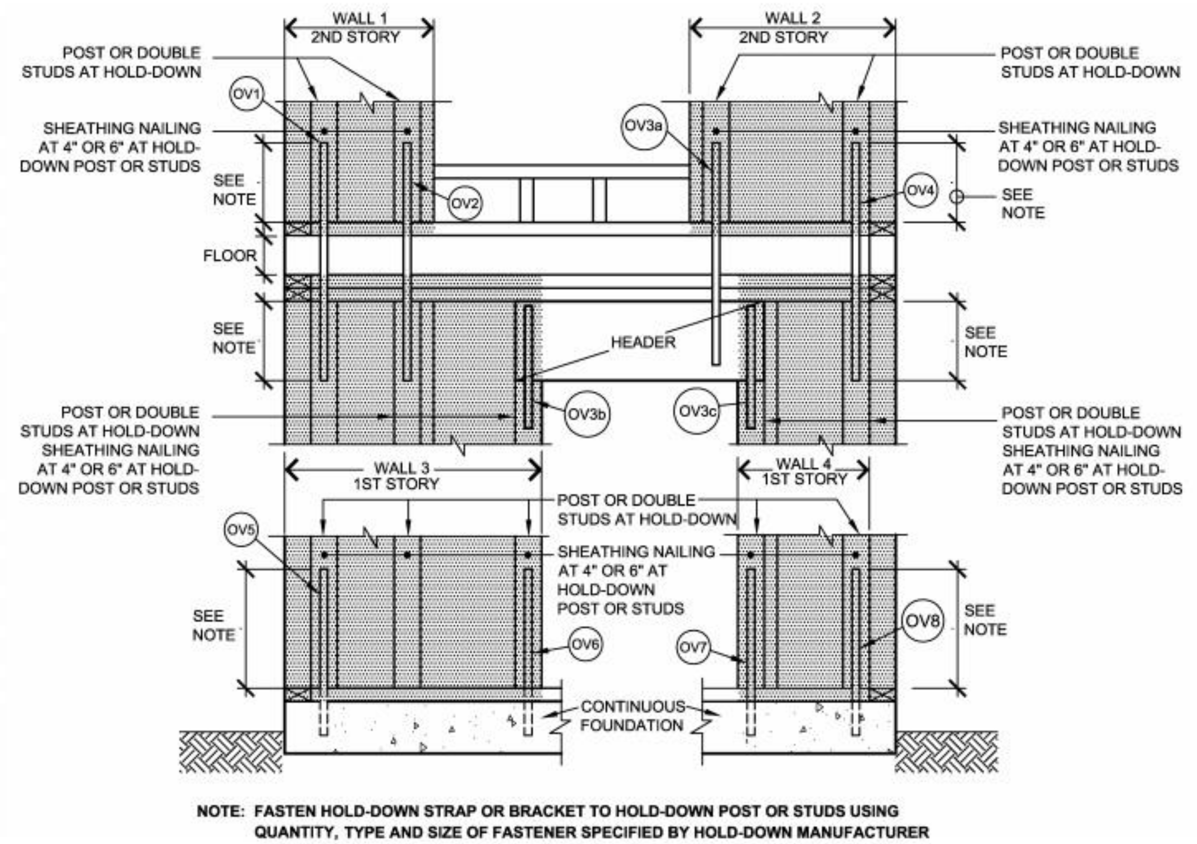


Figure 3-3 Detail of overturning load path connections

Recommendation: Hold-down Devices

Where not already required by the IRC (see Table 3-3), the use of hold-down connectors (e.g., brackets, straps) is recommended as an above-code measure to improve load transfer and thereby decrease damage. For most basic IRC wall bracing types, the provisions rely on building weight and the fasteners resisting horizontal loads to also resist overturning. However, the use of hold-down brackets and straps provides more reliable earthquake performance.

3.4.3 Configuration Irregularities

Section 2 of this course introduced configuration irregularities with a discussion of configurations that can cause concentrations of seismic damage. Section 3.4.3 continues this discussion by introducing the IRC approaches to limiting irregular configurations in dwellings and townhouses. The two broad approaches used by the IRC are:

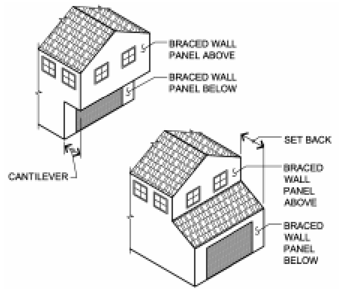
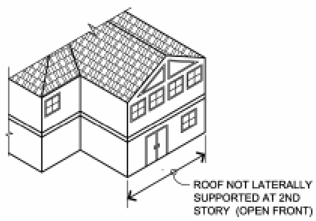
♣ *Approach 1*: The more explicit approach is found in §R301.2.2.6, which directly limits irregular building shapes. Where irregularities occur, design of all or portions of the wind and earthquake bracing system is required to be engineered. Exceptions allow less significant irregularities within IRC prescriptive designs. For one- and two-family detached dwellings, these provisions are applicable in SDC D0, D1, and D2. For townhouses, these provisions are applicable in SDC C, D0, D1, and D2. The IRC limitations on irregularities are derived from those required by the IBC and ASCE/SEI 7 for engineered buildings. Table 3-5 provides detailed discussion of the §R301.2.2.6 irregularity provisions.

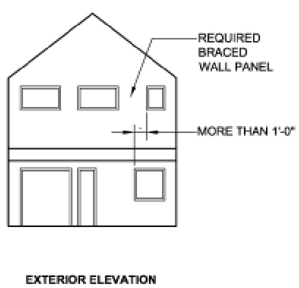
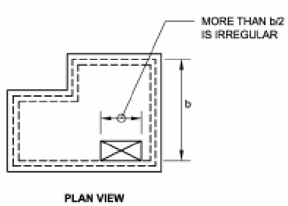
♣ *Approach 2*: The other IRC approach requires the distribution of wall bracing. Along with braced wall lines at exterior walls in wood light-frame dwellings and townhouses, interior braced wall lines must be added so that the spacing between wall braced lines (BWLs) does not exceed limits specified in Chapter 6. Maximum spacing between braced wall panels in a braced wall line is also regulated. The required distribution of braced wall lines and braced wall panels results in the bracing walls being distributed throughout, rather than concentrated in limited portions of the structure. This allows the earthquake load to be resisted in the area where it is developed. Good distribution of bracing walls helps to provide better earthquake performance.

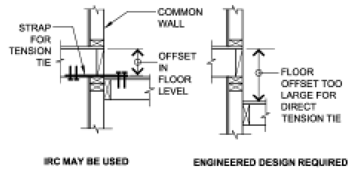
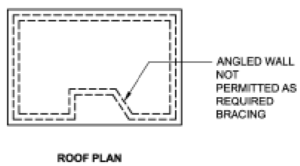
Recommendation: Irregularity Limitations for SDC A, B, and C

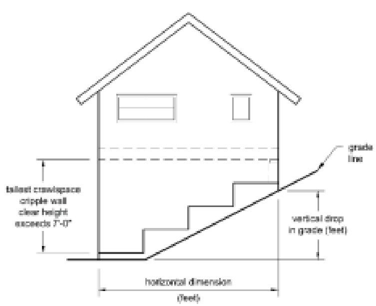
The concentration of damage as a result of irregular building shape is equally applicable to earthquake loading in all SDCs and to wind loading. To date, the IRC has limited application of irregularity provisions to areas of high earthquake hazard. Use of the irregularity limitations in SDCs A, B, and C and for wind loading will contribute to better building performance and is recommended as an above-code measure.

Table 3-5 IRC Configuration Irregularities (§R301.2.2.6)

Irregularity Number and Description	Illustration	Use IRC if	Engineer if	Discussion
1: Exterior shear wall lines or braced wall panels are not in one plane vertically from the foundation to the uppermost story in which they are required		The out-of-plane offset does not exceed four times the nominal depth of the floor joists and the detailing requirements of the exception are met.	The out-of-plane offset exceeds that permitted or the detailing provisions are not followed.	Braced wall panels are intended to be stacked in order to maintain uniform strength and stiffness at each story and to aid in continuity of the load path. Support of braced wall panels on cantilevers and setbacks can reduce their strength, stiffness and continuity.
2: Section of floor or roof is not laterally supported by shear walls or braced wall lines on all edges. Also called an open front irregularity.		A section of floor or roof extends not more than six feet beyond the braced wall line, and the cantilever does not support a braced wall panel. This is often used for a roof overhang over an entry deck.	The floor or roof extension is greater than six feet or is supporting a braced wall panel.	Placement of bracing walls at the edge of each floor or roof section aids in the uniform distribution of bracing and aids in ensuring continuity of the load path. Where bracing walls are not provided at all edges, rotational behavior (plan irregularity) may result.

Irregularity Number and Description	Illustration	Use IRC if	Engineer if	Discussion
3: The end of a braced wall panel occurs over an opening in the wall below and ends at a horizontal distance greater than 1 foot horizontal past the edge of opening. *This provision is applicable to shear walls and braced wall panels with or without offset out of plane as permitted by the exception to Item 1 above.*		The braced wall panel does not extend more than one foot over the header below, or if the header meets the requirements of the exception.	The braced wall panel extends more than 1 foot over a header and the header is not selected in accordance with exception requirements, or if the entire braced wall panel falls on the clear span of the header.	When earthquake loads are applied to braced wall panels, large downward loads develop at the panel ends due to overturning (see Table 2). If the downward load falls on a header that is not strong or stiff enough, the effectiveness of the bracing is reduced and localized damage may occur.
4: An opening in a floor or roof exceeds the lesser of 12 feet or 50% of the least floor or roof dimension.		Floor and roof openings are kept to minimum size, such as standard stair openings.	Stair openings are enlarged, such as to create entry foyers or two-story great rooms or accommodate large skylights.	Large floor and roof openings can affect the uniform distribution of earthquake loads to bracing walls, resulting in increased deformations and concentration of damage in the floor, roof and bracing wall systems.

Irregularity Number and Description	Illustration	Use IRC if	Engineer if	Discussion
5: Portions of a floor level are vertically offset. Also called a split-level irregularity.		Floor framing on either side of a common wall is close enough in elevation so that straps or other similar devices can provide a direct tension tie between framing members on each side of the wall.	Floor framing on either side of a common wall cannot be directly tied together.	This irregularity results from observed earthquake damage in which one of two floor or roof levels pulled away from a common wall, resulting in local collapse. The direct tie limits the distance that either floor system can pull away, reducing likelihood of losing vertical support. This configuration is somewhat common in townhouse construction.
6: Shear walls and braced wall lines do not occur in two perpendicular directions.		Required braced wall panels are oriented in the house longitudinal and transverse directions.	Required bracing walls fall at angles other than longitudinal and transverse.	Some dwellings have walls that fall at an angle to the main transverse and longitudinal directions (often at 45 degrees). Where angled walls are not required for bracing, this is not a concern. Walls used for bracing must be aligned in the longitudinal or transverse direction. When the angle of bracing walls varies, the earthquake loads in the walls vary from those assumed in developing the IRC provisions. Non-standard load path detailing may also be required.

Irregularity Number and Description	Illustration	Use IRC if	Engineer if	Discussion
7: Stories above grade partially or completely braced by wood wall framing in accordance with §R602 or steel wall framing in accordance with §R603 include masonry or concrete construction.	None.	Concrete or masonry construction within a light-frame house is limited to those items listed in the exception (fireplaces, chimneys, and veneer).	Other concrete or masonry construction is mixed with light-frame walls in any story above grade.	The wall bracing requirements for wood or steel light-frame walls are proportioned to resist earthquake loads from light-frame wall systems only. Introduction of concrete or masonry will increase earthquake loads beyond the wall bracing capacity. In addition, introduction of concrete or masonry walls will likely affect the distribution of wall stiffness, causing a plan irregularity.
8: Dwellings or townhouses are identified as hillside light-frame construction as per §R301.2.2.6, Item 8, engineered design shall be provided from the floor diaphragm above the cripple walls through the foundation.		The average slope along all sides of the dwelling is less than one vertical in five horizontal, or the tallest cripple wall is less than seven feet, or if more than 50% of the underfloor area is occupied, having interior wall finishes.	The above criteria are not met.	Hillside dwellings were observed to be very vulnerable to earthquake damage up to collapse in the 1994 Northridge Earthquake. Based on FEMA P-1100 studies, the criteria were established to identify hillside dwellings most vulnerable to significant earthquake damage.

§R602.11.2 and Figure R602.11.2 provide specific detailing to reduce the effect of a concentration of load due to stepped concrete or masonry foundation walls, another configuration irregularity. As illustrated in Figure 3-4, a direct tie to the tallest foundation segment provides uniform stiffness along the wall line.

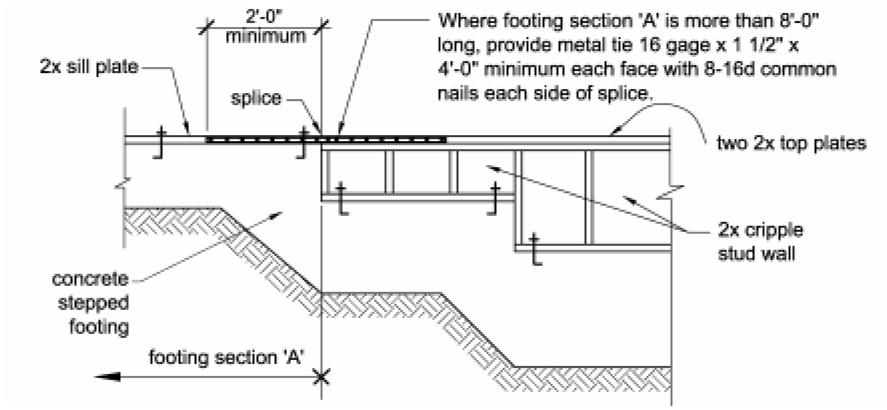


Figure 3-4 Detailing required for stepped foundation walls (ICC, 2024a).

Recommendation: Length of Cripple Wall Sheathing

Cripple walls are a common location where soft/weak stories occur. This is particularly true where perimeter cripple walls have no inside face sheathing and under-floor basement areas have few or no interior bracing walls. Although the IRC has increased bracing wall lengths beyond those required by earlier codes, further increasing length and providing interior braced wall lines will help to limit deformation and damage. Additional length of wood structural panel sheathing at the building perimeter and interior is recommended as an above-code measure to limit cripple wall deformation.

3.5 Alternative Dwellings

In recent code update cycles, provisions have been added for some alternative dwelling configurations and construction methods. Table 3-6 introduces a few of these.

Table 3-6 Alternative Dwelling Provisions

Dwelling Type	IRC Reference	Discussion
Repurposed intermodal shipping containers	§R301.1.4	Addresses shipping containers repurposed as buildings or structures, a common trend in some regions. Rather than bringing design provisions into the IRC, the provisions of IBC Chapter 31 are referenced. These provisions include both a regular engineered approach and a simplified engineered approach. These can be applied to either single repurposed containers, or structures constructed of multiple containers.
Tiny houses	Appendix BB	Included in the scope are dwellings of 400 square feet or less. The provisions of the IRC apply except as modified by the appendix. Modifications address items such a minimum dimensions, lofts, and stairs that are unique to the reduced size of tiny houses. Structural provisions of the IRC are not modified in the appendix chapter. As a result, tiny houses are required to conform to all of the requirements that are discussed in this guide. While the plan dimensions of tiny houses may create some challenges in incorporating required wall bracing, the many IRC Chapter 6 bracing methods that incorporate slender walls should meet this need.
Light straw-clay construction	Appendix BI	Prescriptive provisions limited to use in low SDCs.
Strawbale construction	Appendix BJ	Prescriptive provisions limited to use in low SDCs.
Cob construction (monolithic adobe)	Appendix BK	Prescriptive provisions limited to use in low SDCs.
3D printed building construction	Appendix BM	3D printing involves automated equipment that deposits material in a layer-by-layer fashion. It is anticipated that engineered designs will be provided. In any case, the method of design must be approved under §R104.2.2.1 or IBC §104.2.3 (alternative materials, design and methods of construction and equipment).
Hemp-lime (hempcrete) construction	Appendix BL	Prescriptive provisions limited to use in low SDCs.

4- Foundations and Foundation Walls

Foundations are the interface between a building and the soils supporting it. Foundations primarily provide support for vertical gravity loads from the weight of a dwelling or townhouse and its contents, but they also provide resistance to horizontal sliding resulting from earthquake ground motions and vertical overturning loads at the ends of braced walls. This section presents the seismic requirements as presented in IRC Chapter 4 for foundations and foundation walls constructed using the two most common foundation materials: concrete and masonry.

§R403 addresses foundations, including footings and combined footings and stem walls. §R404 addresses foundation walls, including basement walls, retaining walls, and stem walls. §R403 is always applicable, except for the unusual occurrences of crushed stone footings. Because stem walls are addressed in both §R403 and §R404, the IRC requires that all applicable provisions from both sections be met. In particular, §R404 requirements are applicable starting with a maximum unsupported wall height of five feet or a maximum unbalanced fill height of four feet. This chapter presents the §R403 provisions first, followed by §R404. Both will often be applicable.

Terminology

The following are defined for the purpose of discussion in section 4 of this coursee. The reader is cautioned that the IRC does not always use these terms consistently.

Footing: The foundation element that sits directly on the supporting soils, most commonly constructed of concrete, but in some cases constructed of brick masonry or crushed rock.

Foundation Wall: A wall element constructed on top of a footing and including basement, retaining, and stem walls. Walls meeting any of these descriptions are required to conform to §R404 foundation wall requirements.

4.1 The Role of Foundations in Earthquakes

When earthquake ground motion occurs, the resulting ground movements, velocities, and accelerations are imparted to the foundation and, in turn, transferred to the dwelling or townhouse. How well the dwelling or townhouse performs during an earthquake depends on how well the foundation is able to provide:

♣ Continued vertical support,
♣ Friction and passive bearing at the soil-to-foundation interface to minimize movement and damage,
♣ Anchorage at the foundation-to-dwelling or townhouse interface to minimize movement and damage, and
♣ Strength and stiffness sufficient to resist both horizontal loads and vertical loads resulting from racking and overturning of bracing walls within the dwelling or townhouse.

The foundation of the dwelling or townhouse must resist the sliding (Figures 4-1) and overturning (Figure 4-2) actions associated with an earthquake. The soil surrounding a foundation can resist sliding using a combination of friction along the bottom and bearing along the sides of the foundation; therefore, a wider and deeper foundation provides greater friction and greater bearing resistance than a shallow and narrow foundation. The whole building overturning action illustrated in Figure 4-2 is resisted at the foundation in two ways. The portion of the foundation being pushed downward will bear against the soil below, so a wider footing will provide more surface area to resist that downward load. At the uplift end of the foundation, the weight of the foundation plus any soil located above a footing helps to resist the loads trying to pull the foundation out of the ground; therefore, a deep inverted T-shaped foundation will provide greater resistance to uplift than a shallow footing or than a foundation having a simple rectangular foundation.

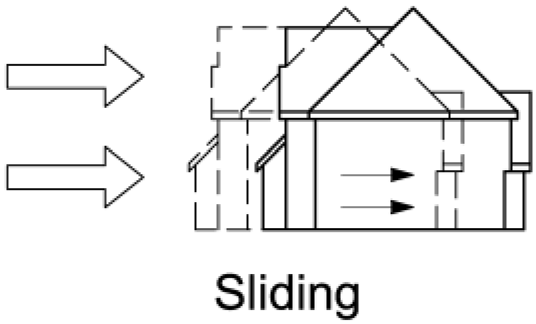


Figure 4-1 Sliding action resisted by foundation.

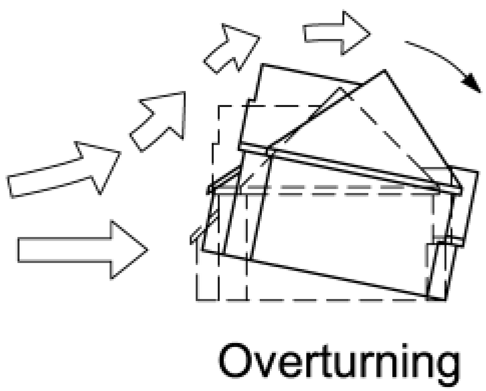


Figure 4-2 Overturning action resisted by foundation

4.2 Conditions Not Addressed in This Section

The following foundation types, while permitted by the IRC, are not addressed by this section.

♣ Preservative-treated wood foundations and foundation walls. §R401.1 requires that these be designed and installed in accordance with the American Wood Council (AWC) permanent wood foundation standard (AWC, 2021), and §R401.1 requires that these be engineered in SDC D0, D1, and D2.

♣ Foundation systems such as pilings, drilled piers, and grade beams that require the involvement of a licensed design professional.

Frost protection of foundations is also not discussed in this guide; where required by the code (see IRC §R403.1.4.1) or local regulations, foundations must either extend below the frost line or be protected from frost using approved methods.

4.3 General Foundation Requirements

When selecting a foundation system, it is important to consider site topography, soil conditions, retained soil height, loading from the dwelling or townhouse above, frost depth, and termite and decay exposure. Regardless of SDC, all dwellings and townhouses require a continuous foundation extending at least 12 inches below undisturbed soil along all exterior walls (Figure 4-3). Alternately, other foundation systems designed to accommodate all loads and approved by the building official are permitted.

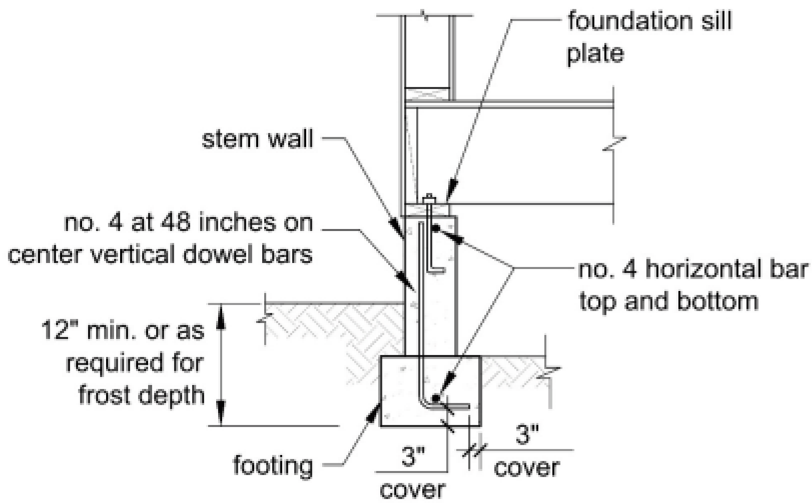


Figure 4-3 Perimeter foundation with separately placed footing and stem wall.

The IRC addresses foundations (i.e., footings with and without stem walls) in §R403 and foundation, basement, stem, and retaining walls in §R404. This chapter is similarly organized. When using the IRC, it is important that all applicable provisions be met; for stem walls this means meeting applicable provisions of both §R403 and §R404. The foundation and retaining wall provisions of IRC §R404 become mandatory for wall clear heights of 5 feet and greater and for walls retaining unbalanced fill of 4 feet and greater; the stem wall provisions apply for shorter walls and less retained soil.

§R403.1.2 contains a general requirement that buildings located in SDCs D0, D1, and D2 be supported on continuous concrete, solid masonry, or fully grouted masonry footings if they are to be designed in accordance with the prescriptive provisions of the IRC. Other foundation types are permitted if they are engineered. As an example, engineered foundation systems might include systems permitted in the IRC, but limited by the IRC to SDC A through C, or other foundation systems not included in the IRC, such as drilled piers or other deep foundations.

§R403.1.2 also contains a general rule applicable to buildings located in SDCs D0, D1, and D2 that requires interior braced wall lines to be supported on a continuous foundation when the spacing between parallel exterior wall lines exceeds 50 feet. For a two-story dwelling or townhouse in SDC D2, however, a continuous foundation is required below all interior braced walls, even when the distance between exterior walls does not exceed 50 feet. An exception permits spacing of braced wall lines supported on continuous foundations at up to 50 feet where three additional conditions can be met.

Those conditions are:

♣ The distance between braced wall lines does not exceed twice the building width measured parallel to the interior braced wall line,
♣ In dwellings or townhouses having either a crawl space or basement, cripple walls do not exceed 4 feet in height, and
♣ In dwellings or townhouses having a crawl space or basement, first-floor interior braced walls are supported on double joists, beams, or blocking as shown in Figure 4-4

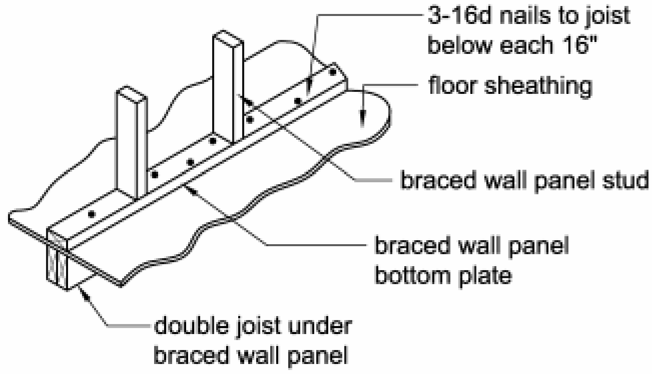


Figure 4-4 Interior braced wall on floor framing

In the 2024 IRC, this information regarding required continuous footings has been moved into new Table R403.1.2 for easier use. This table is intended to communicate the same criteria as the previous IRC edition regarding continuous foundations under interior braced wall lines. One clarification is appropriate when using the new table. Where Footnote a says "Buildings shall be permitted to have interior braced wall panels supported on continuous foundations at intervals not exceeding 50 feet (15,240 mm) provided that when the following conditions are all met," it is more correct to say that the 50-foot spacing applies to braced wall lines, not braced wall panels.

Recommendation: Continuous Foundation at Braced Wall Lines

The main consequence of not providing a continuous foundation below a first-story interior braced wall line is that the floor must be strong enough to transfer earthquake loads from that interior wall to a parallel perimeter foundation. This transfer of lateral loads can be accomplished using a wood-framed and wood-sheathed floor system, but this solution will impart additional stresses in the floor and into foundation walls at the perimeter that would not occur if a foundation was provided below the interior braced wall line. Vertical loads due to the overturning in the braced wall segments also must be transferred to the perimeter foundation through bending action. It is recommended as an above-code measure that interior braced wall lines be supported directly on a continuous foundation, even when not specifically required by the IRC.

For slab-on-grade construction in SDCs D_0, D_1, and D_2, when the conditions described above require an interior braced wall line to have a footing, the footing depth along that interior wall must be at least 12 inches below the top of the slab as shown in Figure 4-5.

Recommendation: Reinforcement at Interior Thickened Slab

Although Figure R403.1.3 implies that interior thickened slab footings do not require reinforcing, it is recommended that this interior footing type be reinforced as for a perimeter thickened slab footing. This recommended reinforcing is reflected in Figure 4-5.

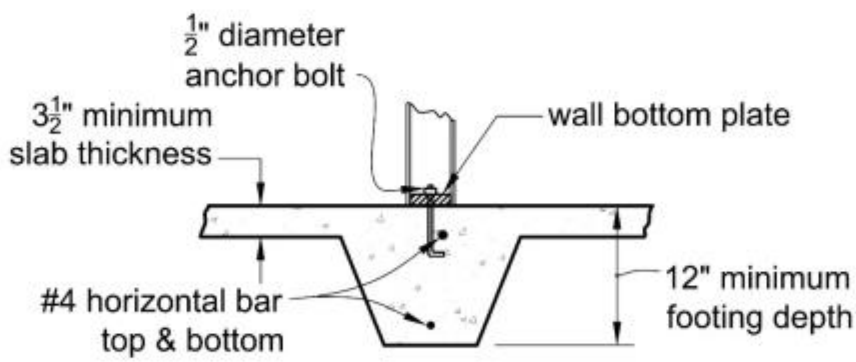


Figure 4-5 Footing requirements for interior braced wall line on slab-on-grade

Regardless of SDC, the minimum specified concrete strength for foundations (and foundation walls) is 2,500 pounds per square inch (psi) with higher strength necessary when located in a moderate or severe weathering probability area. Table R402.2 identifies where higher concrete strength is required as a function of the Figure R301.2(1) weathering probability map. 2,500 psi refers to a measure of the concrete compressive strength. To enable a concrete foundation to resist all of the possible loads to which it may be exposed, compressive strength needs to be complemented with tensile strength. Since concrete is unable to resist tension stresses without cracking, steel reinforcing bars are added to resist tension. Reinforcing is particularly valuable when resisting cyclic earthquake loads because, within the span of a few seconds, the loads may start by causing compression and then reverse to cause tension in the same location.

4.4 Footings and Stem Walls – Concrete

Section 4.4 addresses concrete slab-on-grade (slab-on-ground) footings as seen in Figure 4-5 and concrete footings and stem walls as seen in Figure 4-6. §R403.1.1 provides minimum footing and stem wall size information and references figures illustrating construction for SDC A, B, and C. In these referenced figures, no reinforcing steel (rebar) is required. In SDC D0, D1, and D2, §R403.1.3 specifies minimum reinforcement of concrete footings and stem walls. Separate subsections within §R403.1.3 address reinforcing of foundations consisting of a footing and a stem wall and reinforcing of the footing along the perimeter of a slab-on-ground. Figure R403.1.3 also illustrates various foundations and reinforcing steel conditions commonly encountered in dwellings and townhouses.

Recommendation: Minimum Foundation Reinforcement in SDC C

To obtain above-code performance in SDC C, it is recommended that the minimum foundation reinforcing requirements for SDCs D_0, D_1, and D_2 also be used in SDC C. This added reinforcing will provide better footing performance whether it is resisting earthquake loads or loads induced by differential soil settlement, expansive soils, or frost heave.

Typically, the bottom portion of a concrete footing must have one horizontal No. 4 reinforcing bar located 3 to 4 inches up from the bottom of the concrete (i.e., clear from the soil along the bottom of the footing). When a foundation consists of both a footing and short stem wall, two No. 4 continuous horizontal reinforcing bars are required: one in the bottom of the footing and within 12 inches of the top of the stem wall as shown in Figure 4-6 (§R403.1.3 and Figure R403.1.3). Information regarding detailing of reinforcing bars in concrete can be found in §R608.5.4. Further discussion follows. Included are minimum lap splice lengths and standard hook dimensions. Not included is treatment of horizontal bars at footing or stem wall corners and intersections. Figure 4-7 provides a standard detail for these locations.

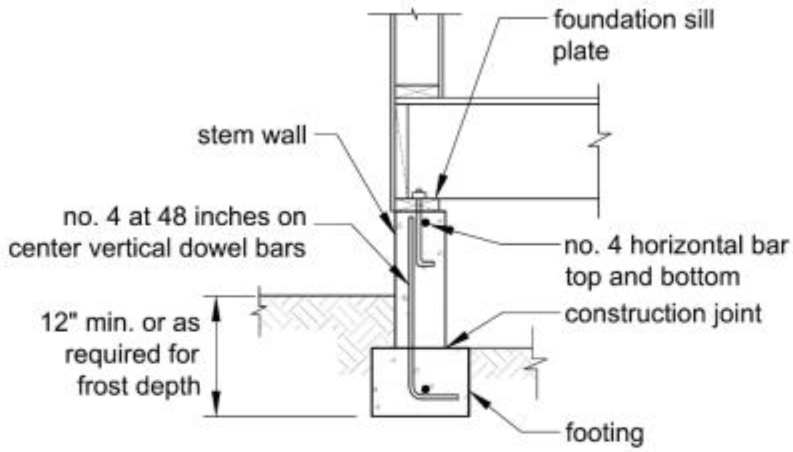


Figure 4-6 Recommended minimum reinforcement for concrete footings and stem walls.

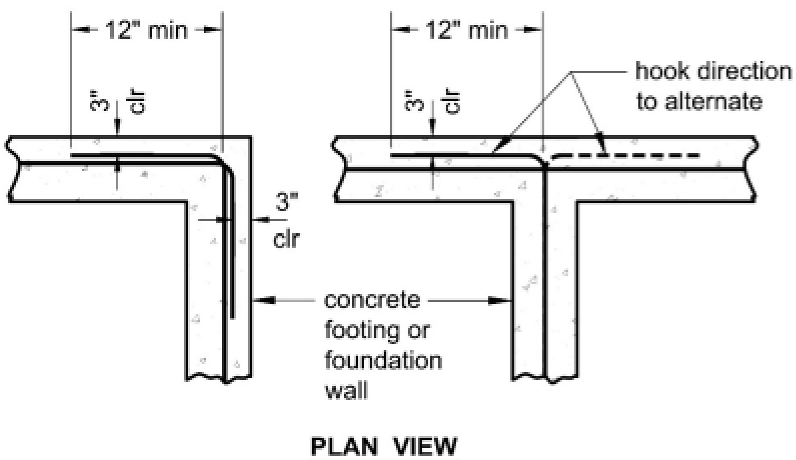


Figure 4-7 Horizontal reinforcing lap at corners and intersections

In SDCs D0, D1, and D2, No. 4 vertical reinforcing is required at 48-inch maximum spacing if a construction joint (also called a "cold joint") occurs between the footing and a concrete stem wall (§R403.1.3.1 and Figure R403.1.3). This is referred to as a "two-pour" foundation because the concrete for the footing and the concrete for the wall are poured at separate times. These vertical bars (also called "dowels") must extend a minimum of 14 inches into the stem wall and be hooked at the bottom into the footing as shown in Figure 4-6. These dowels provide a very important connection, resisting sliding along the joint between the two separate concrete pours. Sliding along a similar construction joint between a slab-on-grade and a separately poured footing below the slab edge occurred during the 1994 Northridge Earthquake: Severe damage to dwellings was observed in Simi Valley and San Fernando Valley, California.

For concrete stem walls, Figure R403.1.3 specifies that the thickness of the stem wall be determined in accordance with §R404.1.4.2 (concrete foundation walls). This section permits a 6-

inch minimum thickness for a maximum wall height of 4 feet 6 inches and requires a minimum 7 1/2 inch thickness for taller walls.

When a dwelling or townhouse has a concrete slab-on-grade with a thickened edge forming its perimeter footing (also called a “turned-down slab edge”), one No. 4 horizontal reinforcing bar is required at the top and bottom of this footing as shown in Figure 4-8. The exception to this is when the slab and footing are poured at the same time; in this case, a single No. 5 bar or two No. 4 bars located in the middle third of the combined slab and footing depth may be used. For slab-on-grade construction in SDCs D0, D1, and D2, interior bearing walls and interior braced walls required to have a continuous foundation must have the concrete slab thickened to 12 inches to form a footing as shown in Figure 4-5

In SDC D0, D1, and D2, where a slab-on-grade thickened edge footing is constructed using a lower and upper concrete pour, as shown in Figure 4-8, vertical dowels are required between the upper and lower pour. Similar to the vertical rebar dowels between the concrete footing and stem wall, these vertical dowels prevent sliding along the construction joint between pours. The vertical reinforcing is required to be not less than No. 3 bars and spaced no more than 48 inches on center. A standard hook (6-inch hook length for No. 3 bar, 8-inch for No. 4) is required at the top and bottom.

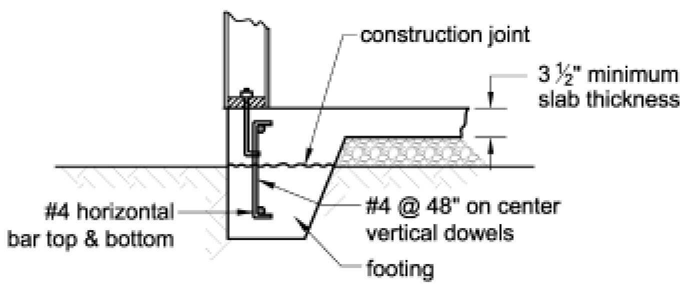


Figure 4-8 Use of vertical dowels to connect a slab-on-grade to a separately poured footing. Correct vertical dowel to #3 and add standard hook top and bottom.

§R403.1.3.5 and §R608.5.4 provide details of reinforcing steel installation. The purpose of the reinforcing is to tie together the concrete sections and to provide continuous strength along the length of the concrete foundations. To get the benefit of the reinforcing steel, the following requirements must be met:

♣ Grade of reinforcing steel needs to comply with one of the following standards: ASTM A615, A706M, or A996M. ASTM A996 bars produced from rail steel are required to be Type R. The minimum yield strength of reinforcing steel is required to be 40,000 psi (Grade 40). The supplier of the reinforcing steel should be able to provide documentation for this requirement.

♣ Reinforcing steel needs to be near the center in the stem wall thickness and also meet the minimum cover requirements.

♣ Reinforcing steel needs to be adequately secured in place and the concrete cover (minimum distance from the face of the rebar to the face of the foundation concrete) meet specified minimums ranging from 1 1/2 inches when not cast against existing soil to 3 inches when cast against existing soil.

♣ Where it is not possible to provide a single reinforcing bar for the full length or height of the foundation, continuity is provided using a minimum overlap between the end of one reinforcing bar and the start of the next bar. Table R608.5.4(1) provides the required lap length, and Figure R608.5.4(1) illustrates a lap splice and requirements for maximum spacing. It is acceptable for the lapped bars to be in contact with each other. If multiple bars are lapped at the same location, it is important to get several inches of clear distance between pairs of lapped bars so that concrete can encase the laps.

♣ From §R608.5.4.5, where standard hooks are required, the geometry of the standard hooks is required to meet the placement requirements of Figure R608.5.4(2) and the geometric requirements (bend diameter and extension length) of Figure R608.5.4(3)

4.5 Footings and Stem Walls – Masonry

Masonry foundation requirements are more dependent on SDC than concrete foundation requirements.
♣ Masonry stem walls used in combination with continuous concrete footings are permitted for all SDCs, as discussed in §R403.1.2. Further discussion of this combination follows below.
♣ With certain scoping limits and added seismic detailing, masonry foundation and basement retaining walls are also permitted in all SDCs as addressed in §R404.1. Discussion is provided in Section 4.11 of this guide.

♣ Due to its low strength and stability, rubble stone masonry is not allowed for dwellings or townhouses in SDCs D0, D1, and D2 or townhouses in SDC C. See §R404.1.8.

♣ Also with low strength and stability, masonry pier and curtain wall foundations (concrete foundations supporting masonry piers and curtain walls as per §R404.1.5.3) are not commonly used and require special detailing in SDCs D0, D1, and D2.

Recommendation: No Rubble Stone Foundations in SDC C

Because of low strength and stability, rubble stone foundations are not recommended for use in SDC C.

Recommendation: No Masonry Pier and Curtainwall Foundations in SDC C or D

Because of low strength and stability, masonry pier and curtainwall foundations are not recommended in SDC C, D_0, D_1, or D_2.

In SDCs D0, D1, and D2, masonry stem walls also must have vertical reinforcing and must be fully grouted (i.e., grout in every cell) as required by §R403.1.3.2. See Chapter 6 and §R606 for a more detailed discussion of grouting in masonry wall construction. For a masonry stem wall supported on a concrete footing, vertical and horizontal reinforcing is required per §R403.1.3.2, similar to that required for concrete stem walls. The vertical reinforcing must be one No. 4 bar at 48-inches maximum spacing, extending into the footing with a standard hook at the bottom, similar to what is shown in Figure 4-6. This is necessary to prevent sliding along the bottom mortar joint between the concrete footing and the masonry stem wall. In addition, one horizontal No. 4 bar is required within 12 inches of the top of the stem wall. The reinforcing detailing requirements of §R403.1.3.5 and §R608.5.4, discussed in Section 4.4 of this guide, are also applicable to reinforcing for masonry stem walls.

Recommendation: Vertical Reinforcement in Masonry Stem Walls in SDC C

To prevent sliding along the bottom mortar joint between the concrete footing and the masonry stem wall, it is recommended that masonry stem walls in SDC C also use the vertical reinforcing required in SDCs D_0, D_1 and D_2.

4.6 Footing Width

Footing width is not dependent upon SDC but instead based solely on vertical load considerations. The minimum width for a concrete or masonry footing is dependent on the load bearing capacity of the soil measured in pounds per square foot (psf) and the number of stories and the weight of the building it supports. Heavier roof, floor, and wall construction results in wider footings being needed.

The minimum soil bearing capacity considered by the IRC is 1,500 psf. In this case, the minimum footing width is 12 inches but it can increase to as much as 32 inches for a three-story dwelling with brick veneer as shown in Figure 4-9. Information on minimum widths is found in §R403.1.1, Tables R403.1(1) through (3), Figure R403.1(1), and Figure R403.1.3

Bearing capacity is determined based on the soil classification determined for the site. Soil classifications and corresponding bearing capacities are listed in Table R401.4.1(1). The soil classification system is described further in Table R401.4.1(2). There are four distinct groups of soils that comprise a total of 15 separate soil classifications ranging from well graded gravels as the best and peat as the worst. Accurate determination of soil bearing capacity requires correct classification of the soil at the building site and may require the expertise of a soils engineer or geologist. However, most building departments have determined the soil classification for most sites within their jurisdiction. Accurate determination of the correct site soil classification is important not just for determining the minimum footing width but also for determining the minimum reinforcing required for concrete or masonry foundation retaining walls.

When a footing is constructed with a width (W) at the bottom that is greater than the thickness (t) of its stem wall or foundation wall (e.g., an inverted-T shape, see Figure 4-9), the minimum thickness (t) of the footing is provided in Table R403.1(1) through R403.1(3) along with the width. §R403.1.1 also specifies that the footing width projecting beyond the face of the stem wall be a minimum of two inches and a maximum of the footing thickness. This requirement results in the footing thickness increasing as the footing width increases. These dimensional requirements for an inverted-T foundation are illustrated in Figure 4-9. These minimum and maximum projection limits of the footing beyond the stem wall are consistent with design of plain concrete footings per ACI 318, *Building Code Requirements for Structural Concrete* (ACI, 2019). Note that Figures R403.1(1) and R403.1.3 do not include L-shaped footings. Based on both IRC and ACI 318 requirements, an Lshaped footing should not be permitted unless its dimensions and reinforcing are designed to account for the eccentricity of the vertical load on the footing.

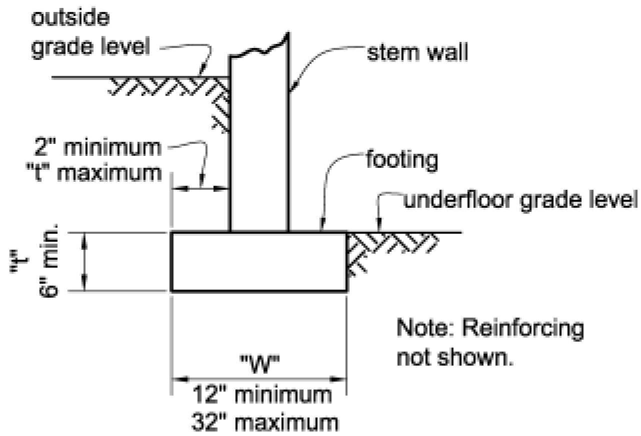


Figure 4-9 Inverted-T footing dimensions.

4.7 Special Soil Conditions

On some sites, such as those in marshy areas or bogs, the bearing capacity of the soil may be known or suspected to be less than 1,500 psf. In such a case, the IRC permits the building official to require soil testing and a report specifying appropriate foundation types and design criteria. Soil testing and reports are also necessary when it is likely that existing soil has shifting, expansive, compressive, or other unknown characteristics. Local officials often are aware of such conditions in locations where previous buildings have been constructed and may have maps identifying such areas and/or special rules that apply to foundation construction in such areas. When existing soil data are not available, such as on previously undeveloped sites, it is always prudent to obtain tests to validate bearing capacity and to determine if expansive soils are present.

In SDC C, D0, D1, or D2, where soil testing and a report are required by the building official due to the soil type, the soil report is also required to specify the short-period spectral response acceleration (*SDS*) that is used in §R301.2.2.1 to determine the SDC. This is because an SDC higher than shown in the IRC maps might be applicable at poor soil sites.

When expansive soils that exhibit large changes in volume (usually in response to changes in moisture content) are encountered, foundations must be designed in accordance with IBC Chapter 18 because the prescriptive requirements of the IRC assume no such special conditions are present. Methods to address expansive soil include foundation designs to resist the stresses caused by the soil volume changes that are likely to occur, gaps between grade beams and expansive soils, removal of the expansive soil, or stabilization of the soil by chemicals, dewatering, presaturation, or other methods. Failure to identify and adequately compensate for differential movements caused by expansive soils can result in excessive stresses on the foundation and cause cracking of even reinforced concrete or masonry foundations. Foundation movement induced by expansive soil also can result in differential movement of the dwelling or townhouse that can crack brick, gypsum wallboard, and stucco finishes. Movement of the walls can create stresses that loosen the nailed connections of wall sheathing used to provide lateral bracing. These effects can cause damage and reduce earthquake resistance. Because of this, it is very important to address expansive or other special soil conditions to limit differential foundation movement.

4.8 Special Considerations for Footings on Steep Sites

Special consideration is needed for footings constructed on sloped sites. To aid in providing sliding resistance, the bottom of footings should be level rather than sloped. The IRC does allow the bottom of footings to be sloped at a rate of not more than 1 foot vertical in 10 feet horizontal. When this maximum slope along the bottom of the footing cannot be met, the bottom must be stepped. Similarly, the top of foundations must be level but can also be stepped. Requirements for foundations on sloped sites can be found in §R403.1.5 and §R403.1.7.

Recommendation: Stepped Foundation at Slope

As an above-code measure when steps are used, the horizontal reinforcing should be provided in accordance with Figure 4-10. This reinforcing detailing will maintain the strength and stiffness of the foundation through the steps.

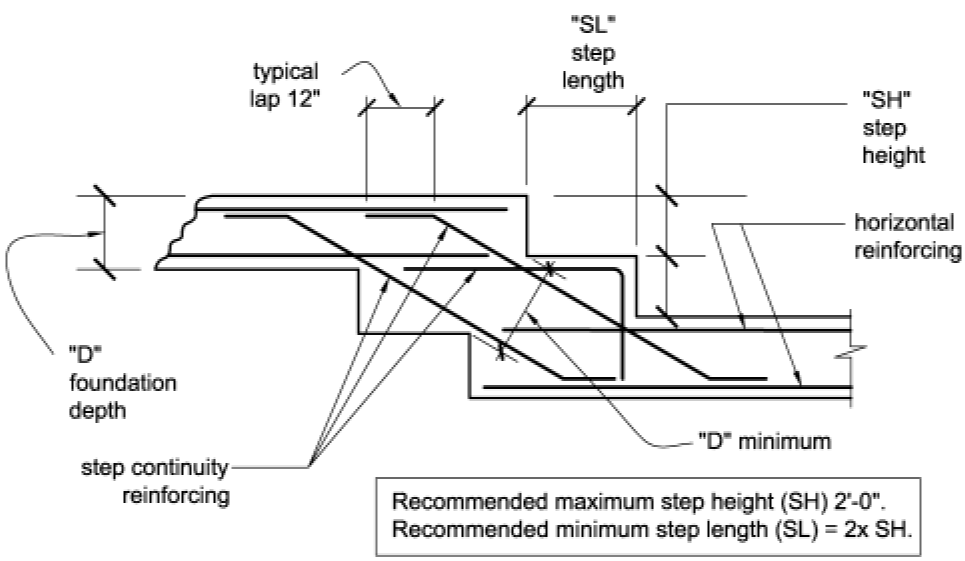


Figure 4-10 Above-code stepped foundation reinforcing detail

Special consideration also must be given to sites where the ground slopes upward or downward beyond the limits of the perimeter foundation to ensure that adequate resistance to foundation sliding and settlement are provided and to protect against the effects of drainage, erosion, and shallow failures of the sloping surface. Prescriptive rules for setbacks from either an ascending or descending slope are given in §R403.1.7. Smaller (or greater) setbacks may be approved (or required) based on a soil investigation report prepared by a qualified engineer. Because landslides can be triggered by earthquakes, any building site having natural or man-made sloping terrain above or below should be thoroughly evaluated for landslide potential even when the prescriptive setbacks of the code are met.

4.9 Special Considerations for Cut and Fill Sites

A hillside site can result in foundations being supported on soils that have very different bearing capacities. Figure 4-12 shows a situation where a portion of the foundation is supported on rock and the other side is supported on a fill that extends above the existing rock grade. This condition often occurs when soil is removed (cut) from the high side of a lot and placed on the lower side to create a level building pad. Although the IRC requires all fill soils to be designed, installed, and tested in accordance with accepted engineering practice, there are no specific criteria in the IRC regarding what should be achieved. Site-specific guidance on the design and placement of fill material is a particularly important concern in high seismic areas.

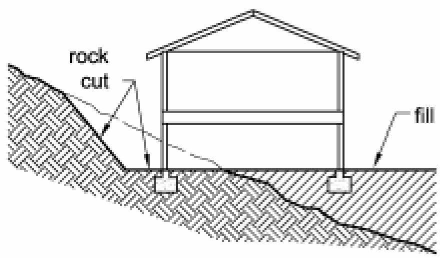


Figure 4-11 Foundation supported on a combination of cut rock and fill

Studies of damage to dwellings or townhouses located on cut-and-fill sites as a result of the 1971 San Fernando (McClure, 1973; Slosson, 1975) and the 1994 Northridge (Stewart et al., 1994 and 1995) earthquakes document the consequences of improperly installed and compacted fill. Figure 4-12 shows an example of the type of damage that occurred to slab-on-grade dwellings or townhouses during the Northridge Earthquake. Generally, these sites experienced settlement and downhill sliding of the fill portion of the site and vertical displacement along the line of transition from cut to fill.

Recommendation: Geotechnical Investigation at Cut-and-Fill Sites

To meet the intent of the IRC requirement for all fill soils to be designed, it is recommended that a geotechnical investigation be conducted and a report provided that includes specifications and requirements for shallow foundations on fill materials.

Figure 4-12 Example of damage caused in building on cut and fill site. The line of slab damage follows the transition from cut soils to fill soils

Recommendation: Avoidance of Cut-and-Fill Sites

Even with proper installation and compaction of engineered fills, earthquakes are expected to result in some differential settlement and consequent damage when a dwelling or townhouse has a portion of its foundation on a cut pad and other portions on fill. In SDCs C, D_0, D_1, and D_2, sites that require a cut-and-fill approach should be avoided, particularly if a slab-on-grade foundation is used. Where cut-and-fill grading of a site is unavoidable, increased levels of quality control should be used to ensure the optimum installation of the fill, and foundations should be designed to either accommodate or resist the expected settlement.

4.10 Additional Recommendations for Footings and Stem Walls

Foundations are the final link in the load path within a building to transfer the earthquake loads to the ground. At the foundation level, the combined lateral loads from the entire building are attempting to push the building laterally. To resist movement, the foundation pushes against the soil that surrounds it. Consequently, footing width and depth are factors that determine the resistance that can be provided by the foundation to earthquake ground motions. This is because footing width determines the horizontal surface area of the bottom of a foundation in contact with the ground and the depth determines the vertical surface area bearing against the soil on either side. These surface areas provide resistance to sliding through a combination of friction and bearing against the soil as shown in Figure 4-13. Therefore, a wider or deeper foundation will be capable of resisting greater lateral loads than a narrower or shallower foundation. Similarly, the sliding resistance of a slab-on grade dwelling or townhouse will be greater than that of one having only a perimeter foundation due to the added frictional surface area provided by the underside of the slab.

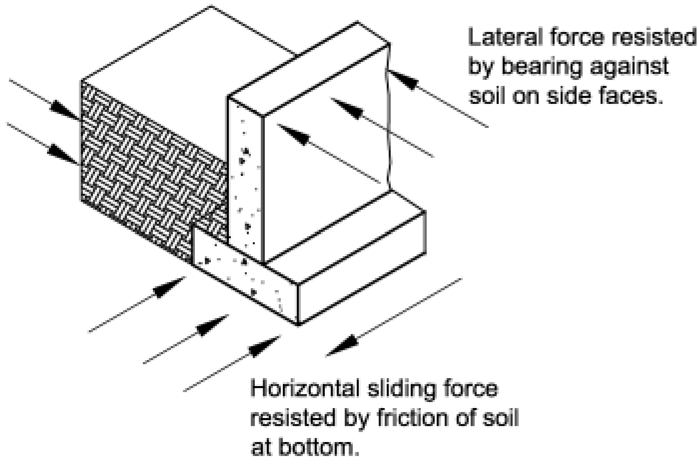


Figure 4-13 Lateral resistance provided by foundation.

Recommendation: Crawl Space Perimeter Footing Depth

For dwellings or townhouses with the first floor located above a crawl space rather than a basement, IRC §R408.6 allows finished grade under the floor to be located at the bottom of the footing except where the groundwater table is high, surface drainage is poor, or the area is prone to flooding. However, when the finished grade is located at the bottom of the footing, the vertical face of the footing on that side does not bear against soil. Therefore, as an above-code measure, in SDCs C, D_0, D_1, and D_2, it is recommended that crawl space perimeter footings maintain their entire depth below the finished grade of the crawl space. Embedment along both vertical faces of the footing provides additional bearing surface area to resist sliding loads perpendicular to the exterior walls. In all cases, minimum required clear heights between grade and the underside of floor framing need to be maintained.

4.11 Foundation Walls

§R404 addresses foundation walls including basement walls, retaining walls, and stem walls. Depending on the difference in ground level on each side of a foundation wall, certain minimum requirements for reinforcing and wall thickness will apply. Foundation wall construction is dependent to some extent on SDC, particularly for SDCs D0, D1, and D2 sites. As previously mentioned, stem walls may be required to meet the requirements of §R403 and additional applicable requirements of §R404.

Foundation walls can be constructed of concrete, masonry, or preservative-treated wood. Permanent wood foundations are not discussed in this guide. Rubble stone masonry foundation walls also are not permitted in townhouses in SDC C or dwellings or townhouses in SDC D0, D1, and D2 (§R404.1.8); however, some use in lower SDCs is permitted with restricted wall height, backfill height, and retaining pressures.

Regardless of SDC, all foundation walls must extend a minimum of 6 inches above the highest adjacent grade or, when brick veneer is used on a wall, the foundation wall must extend a minimum of 4 inches above the highest adjacent grade.

Concrete and masonry foundation and retaining walls must conform to the prescriptive requirements of the IRC or may be based on other recognized structural standards such as ACI 318, ACI 332 (ACI, 2020), or PCA 100 for concrete or TMS 402 (TMS, 2016) for masonry. When concrete or masonry walls are subject to hydrostatic pressure from groundwater or support more than 48 inches of unbalanced fill, without permanent lateral support at both the top and bottom, they must be designed using accepted engineering practice (engineered design methods). The discussion below relates only to the IRC prescriptive methods for concrete or masonry foundation and retaining walls.

Terminology

- **Plain concrete; plain masonry**: Not necessarily devoid of all reinforcing; however, they have less reinforcing than is required to be officially designated as being "reinforced."
- **Unbalanced backfill**: the difference in height of soil between the exterior and interior finish ground levels except that, when an interior concrete slab floor is present, the height is measured from the top of the slab to the exterior finished ground level.

The minimum thickness of plain concrete and plain clay or concrete masonry foundation walls ranges from 6 inches to 12 inches depending on several variables. These variables include the height of the wall, the site's soil classification, and the height of any unbalanced backfill.

Foundation wall thickness also must be at least equal to the width of the supported wall above.

4.11.1 Masonry Foundation Walls

For plain masonry foundation walls, the minimum thickness is dependent on the choice of solid masonry units or hollow units that can be either grouted or ungrouted. Generally, the minimum thicknesses for solid masonry and grouted hollow masonry are identical whereas the minimum thickness for ungrouted hollow masonry is greater. Minimum wall thickness for plain concrete and plain masonry walls, based on specific combinations of wall height, unbalanced backfill height, and soil type, are listed in Table R404.1.2.1(1).

In any SDC, masonry foundation walls are required to meet the following:

♣ For solid masonry walls, the wall type and thickness requirements of Table R404.1.2.1(1), and
♣ The wall reinforcing requirements of Tables R404.1.2.1(2) through R404.1.2.1.(4).
In SDCs D0, D1, and D2, §R404.1.4.1 imposes additional limitations on masonry foundation walls as follows:
♣ Two No. 4 horizontal reinforcing bars are required in the top 12 inches of all masonry foundation walls,
♣ Foundation walls with heights not greater than eight feet, unbalanced retaining not greater than four feet, stem thickness not less than eight inches and meeting the requirements of Table R404.1.2.1(1) can be reinforced with No. 4 vertical bars at not more than four feet on center. (This is consistent with the §R403 requirements for stem walls), and

♣ Foundation walls not meeting the above limitations will need to meet the more restrictive reinforcing requirements of Table R404.1.2.1(2) through R404.1.2.1(4).

Recommendation: Masonry Foundation Strength and Reinforcing

While not specifically discussed in §R404 for masonry foundation walls, it is recommended that the material strength and reinforcing detailing provisions discussed in Section 4.2 of this guide also apply to masonry foundation walls. This includes strength of masonry, grade of reinforcing steel, and details such as lap splices and standard hooks.

Regardless of SDC, all vertical reinforcing for these reinforced masonry or concrete foundation walls must be at least ASTM Grade 60 (yield strength of 60,000 psi). This is important to note because No. 4 bars are commonly available in Grade 40, which has a lower yield strength. In addition, the distance from the soil side face of the wall to the centerline of the vertical reinforcing must be 5 inches in an 8-inch-thick wall, 6 3/4 inches in a 10-inch-thick wall, and 8 1/2 inches in a 12-inch-thick wall.

4.11.2 Concrete Foundation Walls

§R404.1.3 contains provisions for concrete foundation walls for all SDCs. §R404.1.3.2 provisions address the following sub-categories:

♣ Foundation walls,
♣ Concrete foundation stem walls supporting above-grade concrete walls, and
♣ Concrete foundation stem walls supporting light-frame above-grade walls.

The above-noted provisions for concrete stem walls are in addition to those from §R403, already discussed. Where above-grade concrete walls occur, provisions from §R608 may apply in addition to the IRC Chapter 4 provisions.

A number of topics are addressed in §R404.1.3 concrete foundation wall provisions, including extensive discussion of materials and detailing.

For buildings in SDC D_0, D_1, and D_2, §R404.1.4.2 incorporates the following additional provisions:

♣ The horizontal reinforcing requirements of Table R404.1.3.2(1),

♣ Vertical reinforcement in accordance with Table R404.1.3.2(2), R404.1.3.2(3), R404.1.3.2(4), R404.1.3.2(5), R404.1.3.2(6), R404.1.3.2(7) or R404.1.3.2(8).

♣ Where Tables R404.1.3.2(2) through R404.1.3.2(8) permit plain concrete walls, not less than No. 4 vertical bars at a spacing not exceeding 48 inches shall be provided.

4.12 Wood-Framed Plate Anchorage to Foundation

For wood-framed construction, provisions for anchorage to foundations are found in §R403.1.6, with additional detail for plate washers in §R602.11.1. §R403.1.6 refers the user to Sections R505.3.1 and §R603.3.1 for anchorage of cold-formed steel construction to the foundation; both of these sections provide figures illustrating intended anchorage.

Terminology

When discussing anchorage of light-frame construction to foundations, the IRC uses a variety of terms, including **bottom plate**, **sole plate**, **and sill plate**. These terms are often used interchangeably. Regardless of terminology, these are plates supported on foundations or slabs on ground and are required to be anchored to the foundation.

From §R403.1.6, the following provisions apply regardless of SDC:

♣ Anchor bolts are required at exterior walls, interior braced wall panels and all wood sill plates,

♣ Bolts are to be a minimum of 1/2-inch diameter, embedded not less than seven inches into the concrete, be located in the middle third of the sill plate width, and be secured with a cut washer and nut. Use of proprietary anchors is permitted where the anchors are found to provide equivalent strength and durability,

♣ A minimum of two bolts is to be provided on each length of sill plate, and the first bolt is to be not closer than seven bolt diameters (3 1/2 inches) or further than twelve inches from each end of each section of sill. Exceptions are provided for short lengths of wall, and

♣ For interior walls not part of a braced wall panel, the sill plates are required to be fastened with approved anchors. This might include anchor bolts or a variety of available proprietary anchors (e.g., shot pins, cast in connectors).

From §R403.1.6.1, the following additional provisions apply to SDC C, D_0, D_1, and D_2:

♣ At interior braced wall line and interior bearing wall plates, anchor bolts are required to be provided at no more than 6-feet on center,

♣ For three story dwellings and townhouses, anchor bolt spacing is to be reduced to 4 feet on center,

♣ For the full length of all braced wall lines, steel plate washers are to be provided on all required anchor bolts,

♣ The plate washers are specified in §R602.11.1 to be 3-inch by 3-inch square steel plate with a minimum thickness of 0.229 inches (approximately 1/4 inch). This steel plate washer size limits the potential for splitting of bottom plates under earthquake loading (a behavior seen in past earthquakes). It is permitted to have a diagonal slot in the plate washer. This allows the plate washer to slide when the anchor bolt is not perfectly centered on the sill plate. Where a slotted hole is used, a round washer must also be provided.

♣ Special anchorage provisions are specified for stepped cripple walls.

Anchor bolts connecting wood foundation sill plates to the foundation must be installed in the foundation or stem wall. It is preferred that anchor bolts be put in place and secured against movement prior to concrete placement. As an alternative, §R403.1.6 permits anchor bolts to be installed immediately following placement of the foundation concrete and before it has hardened (also called “wet setting”). However, wet setting is not recommended because it can create a void in the concrete adjacent to the bolts, whether the anchor bolts have heads or hooks on the embedded end. The void can extend over the entire length of the bolt embedment and prevent the bolt from completely bearing against the surrounding concrete. This condition can result in movement and potentially failure of the bolt when subjected to earthquake loads. Because of this, §R403.1.6 allows wet setting of anchor bolts with the provision that where anchor bolts resist placement or the consolidation of concrete around anchor bolts is impeded, the concrete be re-vibrated to ensure full contact between the anchor bolts and concrete. Re-vibrating of the concrete around the anchor bolts is always encouraged. Additional anchor bolt requirements are discussed at the end of this section.

4.13 Quality Control

There are several construction aspects important to the earthquake performance foundations and foundation walls:

- Support on stable soils is critical to foundation performance both prior to and during earthquakes. Where foundations are not extended down to competent stable soils, cracking damage can initiate in the foundation and project up into the dwellings prior to the earthquake. Damage prior to an earthquake has been observed to increase the level of damage caused by the earthquake

- Excavation to required dimensions is also important for the foundation to function as intended. Slight over excavation sometimes helps to assure that minimum foundation dimensions are maintained. Foundations need to be cleaned of all debris and sloughing soil just prior to placing concrete.

- Adequate cover on reinforcing steel maintains the intended resistance to corrosion. Without adequate cover the reinforcing steel is likely to prematurely corrode, reducing the effectiveness of the reinforcing and causing spalling of the concrete.

- Adequate detailing of reinforcing steel is also critical to having the foundation perform as intended. Included are adequate lap splices, adequate bends around corners, adequate dimensions of intended standards hooks, and adequate embedment.

- Vibration of concrete and grout is critical for adequate consolidation of the concrete. This plays a critical role in the strength and durability of the concrete, reinforcing and anchor bolts.

- §R403.1.6 permits anchor bolts to be set in place after placement of the concrete (wet setting). Although this is not a preferred option, it is possible for wet set bolts to perform adequately provided the concrete in the vicinity of the anchor is well vibrated after the anchor has been set. Without adequate vibration, the concrete is likely to have air pockets that reduce the concrete bond to the bolt and, as a result, reduce the bolt capacity.

5- Floors

This section addresses wood-framed floor systems and concrete slab-on-grade floor systems, both of which are addressed by IRC Chapter 5.

5.1 The Role of Floor Systems in Earthquakes

Wood-frame floor systems form a horizontal diaphragm (essentially a beam loaded horizontally on its side as shown in Figure 2-9) that transfers earthquake lateral loads to supporting walls below that floor level. Loads are transferred directly to the foundation when the lowest floor is supported on a foundation. When a floor supports walls above and is supported on walls below as shown in Figure 5-1, the lateral loads in the floor system are based on the mass of the floor itself and a portion of the mass of all the walls in the stories immediately above and below the floor. See Chapter 2 of this guide for discussion of the load path.

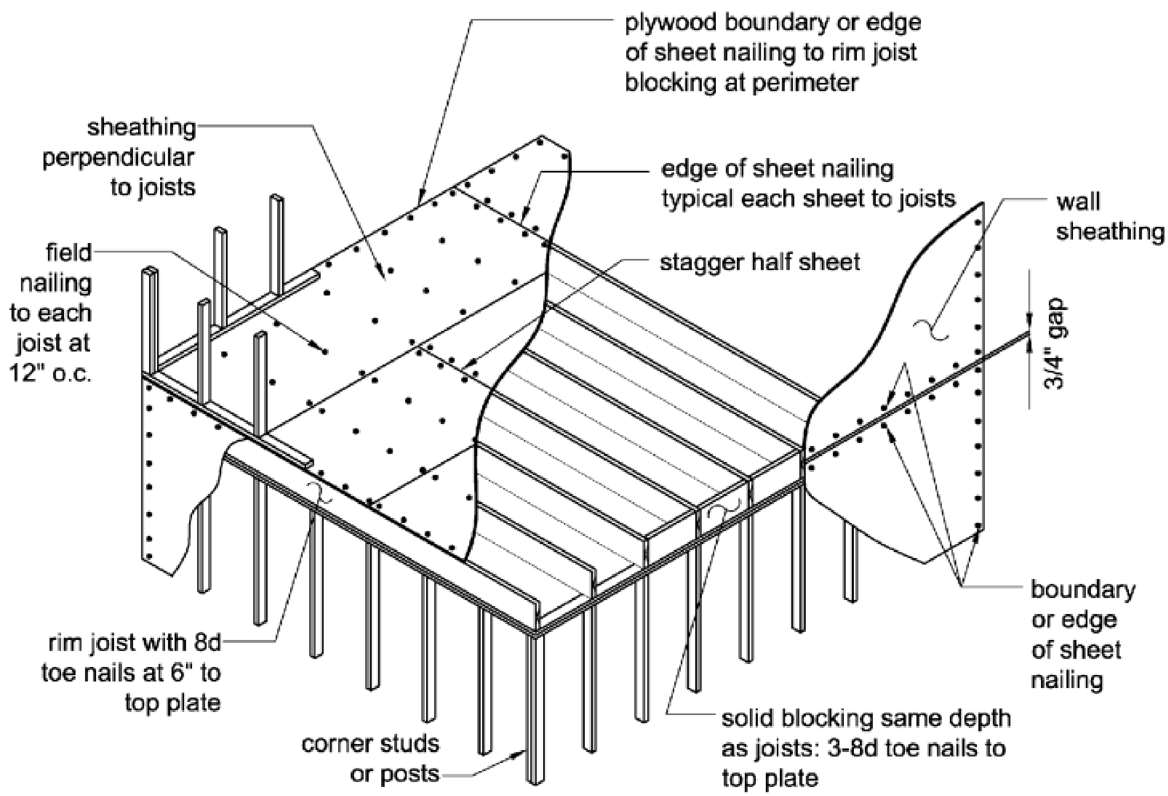


Figure 5-1 An unblocked wood light-frame floor diaphragm.

Concrete slab-on ground (slab-on-grade floors are typically constructed with a concrete thickened slab foundation (see Chapter 4 of this guide) and together these elements form the base of the building. Lateral forces from exterior and interior braced wall lines are transferred to a slab-on-grade via connections between the bottom plate of a braced wall and the slab. In turn, the concrete slab and foundation transfer those forces directly to the ground as shown in Figure 5-2.

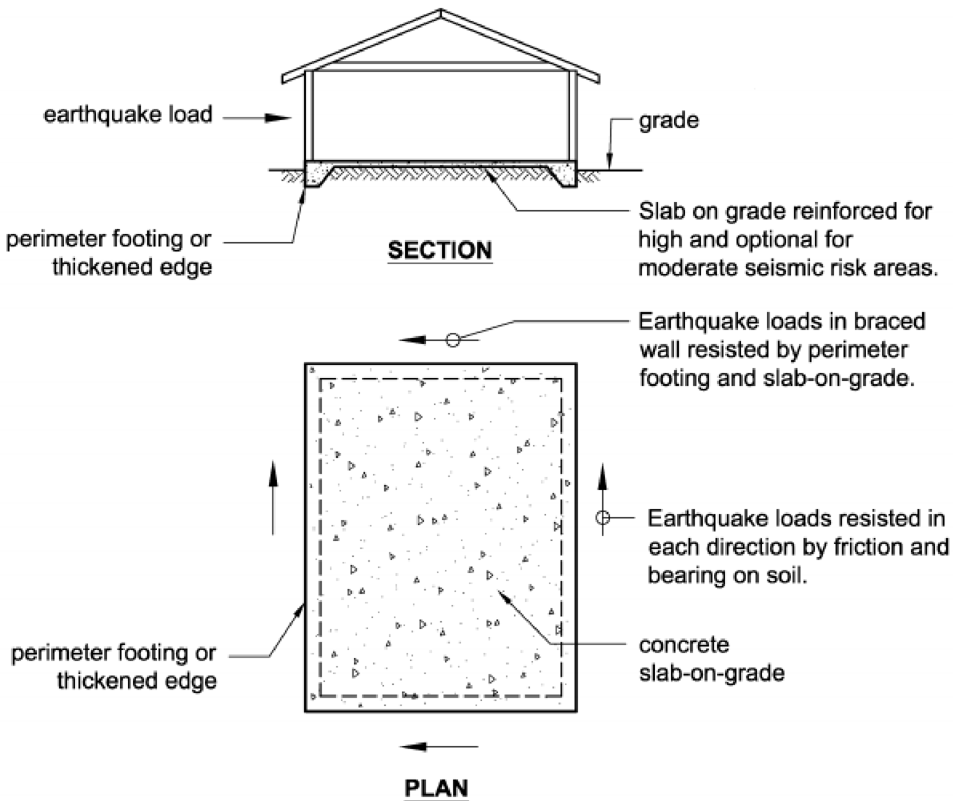


Figure 5-2 Slab-on-grade and perimeter footing transfer loads into soil.

5.2 Conditions Not Addressed in This Section

Cold-formed steel framed floor systems are permitted by the IRC but not covered in this chapter. Rather, the reader is referred to the AISI S320 for guidance. Also permitted but not discussed here are pressure-preservative treated wood floor systems on ground; information on the use of these systems are provided in IRC Chapter 5.

5.3 Wood-Frame Floor Systems

Wood-frame floors typically consist of repetitive joists or trusses, at a spacing of 16- or 24-inches on center, sheathed with boards or wood structural panels attached to the top surface (Figure 5-1). Finish materials such as gypsum board typically are applied to the bottom surface where it serves as the ceiling for a room below. Blocking between joists or trusses is used at the ends of the floor joists or trusses (or a continuous band joist can be used at the ends) and where walls occur above or below. Floor systems also include beams, girders, or headers where needed to support joists. Joists can be sawn lumber, end jointed lumber, or a variety of prefabricated (engineered) members. Examples of engineered lumber include wood I-joists, trusses, and solid rectangular structural composite members such as parallel strand lumber (PSL), laminated veneer lumber (LVL), and laminated strand lumber (LSL). Beams, girders, or headers and blocking also can be either sawn lumber or engineered lumber.

The primary design consideration in choosing the minimum size and the maximum span and spacing of floor joists, trusses, beams, girders, and headers is adequate support for dead and live vertical loads as prescribed by the code. Vertical deflection of a floor is another design consideration that can limit the maximum span of floor members. Tables in IRC Chapter 5 and similar tables in other documents such as those published by the AWC or engineered lumber manufacturers are available for use in selecting the proper combination of minimum size and maximum span and spacing of floor framing members.

5.4 Cantilevered Floors

When floor joists cantilever beyond a support, joist size and spacing are limited by prescriptive tables in IRC Chapter 5. Table R502.3.3(1) addresses cantilevered floor joists that support a roof and one story of wall (Figure 5-3) for roof spans up to 40 feet and ground snow loads up to 70 psf. Table R502.3.3(2) addresses joists cantilevered for an exterior balcony. When a floor is supporting more than a roof and one story of wall, the maximum prescriptive cantilever distance is limited by the IRC to the depth of the joist. If longer cantilevers are desired, an engineered design must be provided for that portion of the floor system.

In SDCs C, D_0, D_1, and D_2, when cantilevered floor joists support braced wall panels in the story above, the cantilevered floor is limited by several additional prescriptive requirements in §R301.2.2.6. This is because the exterior wall braced wall panels above and braced wall panels below are offset out-of-plane. When a floor cantilever supporting a braced wall does not meet the IRC limits, that portion of the dwelling or townhouse is defined as having an irregularity (see Section 3.4) that prevents the use of prescriptive wall bracing where the irregularity occurs. In such a case, engineering design must be provided to resolve the out-of-plane offset of the braced walls located in the stories above and below that floor. The maximum permitted cantilever of a second floor supporting a braced wall and roof is illustrated in Figure 5-3.

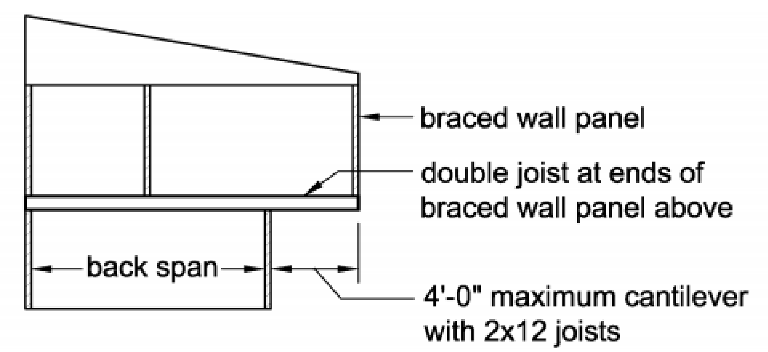


Figure 5-3 Cantilevered floor joist supporting roof and one story of wall.

The specific limits and requirements in §R301.2.2.6 for cantilevered floors in SDCs C, D_0, D_1, and D_2 that support braced walls are not particularly difficult to meet. In SDCs C, D_0, D_1, and D_2, cantilever floor joists supporting a braced wall panel may extend up to four times the nominal depth of the joist when the following set of rules is met:

♣ Joists must be 2×10 nominal or larger at 16-inch maximum spacing.

♣ The back span of the cantilever joist must be at least twice the cantilever distance.

♣ Joists must be doubled at the ends of the braced wall panel above.

♣ A continuous rim joist is connected to the end of each cantilevered joist. If that rim joist is spliced along its length, the splice must be made with either: (a) a 16-gage strap having six 16d common nails on each side of the splice or (b) by using wood blocking having the same size as the rim joist, installed between the cantilevered joists, and nailed to the rim with eight 16d common nails on each side of the splice.

♣ The cantilever end of the joist is limited to supporting uniform loads from a roof and wall above and, if supporting a header above, the header span is limited to 8 feet

These rules are illustrated in the framing plan shown in Figure 5-4. The upper portion of the figure shows joists that are continuous past the back span support, while the lower portion shows joists spliced at the back span support. Both show uplift connections from the back span to the supporting beam or wall. What is not mentioned in this list of rules is the need for connections to resist uplift at the back-span (interior) end of a cantilever joist as noted in Figure 5-4. In Table R502.3.3(1) for cantilever joists supporting a roof and wall, the uplift is determined using a back-span distance that is three times the cantilever distance (3:1). Because the minimum back span specified in the IRC Chapter 3 (see second bullet above) is only twice the cantilever distance (2:1), the uplift values in Table R502.3.3(1) would need to be increased by a factor of 1.5 just to address the gravity loads.

When the downward earthquake overturning load from the ends of a braced wall panel supported by cantilever joists are considered in addition to gravity loads, the uplift load at the back-span end of the joist will increase. Therefore, depending on the actual back-span-to-cantilever-length ratio, the back-span end of the double cantilever joists supporting the ends of a braced wall may need to provide uplift restraint as much as twice than that listed in Table R502.3.3(1). However, because the magnitude of the uplift load at the back-span end of a cantilevered joist reduces as the back-span length increases, it is possible that a cantilever joist that is continuous over its interior support will result in zero uplift at the back-span end. When cantilever joists are continuous over an interior support, the back span increases and the uplift at the end of the joist is greatly reduced. Therefore, the specific cantilever floor joist layout and ratio of the length of the back span to the cantilever will determine if and how much uplift may need to be resisted.

Recommendation: Doubling of Uplift Loads

In SDCs C, D_0, D_1, and D_2, when a braced wall is supported at the ends of cantilever joists, the back-span uplift connection required strength should be determined using engineering principles for the specific back-span and cantilever distances involved. As a minimum it is suggested that the uplift loads specified in Table R502.3.3(1) be doubled.

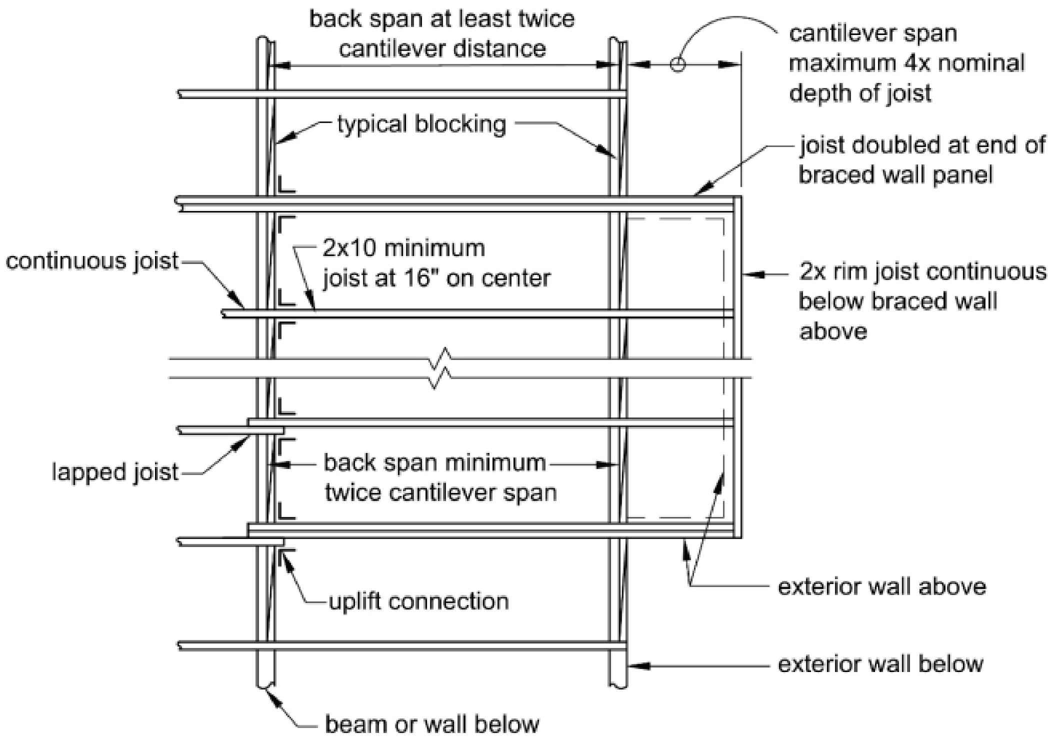


Figure 5-4 Floor framing plan showing cantilevered joists at braced wall above. Top portion shows joists continuous past left-hand support, bottom portion shows joists lapped at left-hand support.

5.5 Requirements for Blocking

It is important in floor framing construction to prevent joists (or trusses) from rotating (rolling) or displacing laterally from their intended vertical position. Rotation loads occur because, when floor sheathing is resisting lateral loads oriented perpendicular to the joist, those lateral loads are actually trying to move the top edge of the joist sideways. Note that for I-joists and trusses, the manufacturer's installation instructions will specify required blocking or other bracing.

Preventing rotation is often accomplished by installing full-depth solid blocking at the ends of joists. The ends of joists also can be restrained by attaching the joist to a continuous rim or band joist or a header or, in balloon framed walls, by attaching the joist to the side of a stud. In SDCs D_0, D_1, and D_2, additional solid blocking between joists (or trusses) is necessary at each intermediate joist support even when that location is not at the end of the joist. For example, blocking should be located at an interior girder or bearing wall where joists are continuous over that support. Blocking installed between joists supported by an interior floor girder is illustrated in Figure 5-5.

Blocking also is required below an interior braced wall line in all SDCs when joists are perpendicular to the braced wall. Although the IRC is silent regarding minimum depth and width for these blocks, the intent of this added blocking is to provide a nailing surface for the 16d common nails used to connect the bottom plate of the braced wall to the floor. This nailing is an important part of the lateral load path. Therefore, the blocking should be of a depth sufficient to allow full embedment of the 16d common nails and of sufficient width to prevent the nails from missing the block.

Assuming floor sheathing is at least 1/2-inch thick, the minimum depth of the blocking should be 1 1/2 inches. Therefore, a flat 2-inch by 4-inch block as shown in Figure 5-6 can provide sufficient depth and, when accurately placed below a wall, can provide a width that greatly reduces the potential for bottom plate nails missing the block.

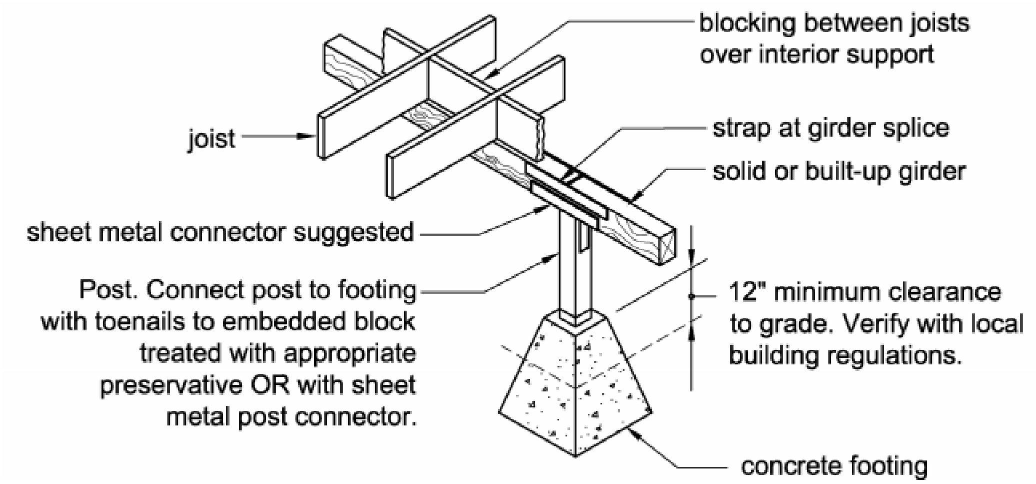


Figure 5-5 Blocking at joists interior bearing line as required in SDC D_0, D_1, and D_2.

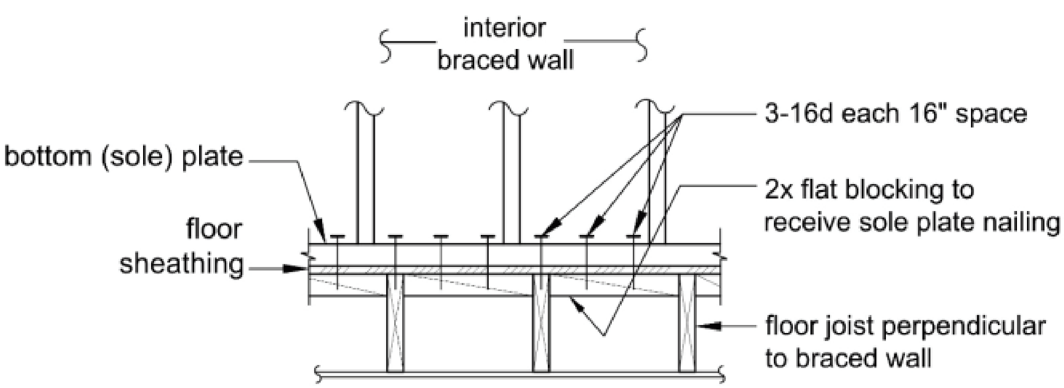


Figure 5-6 Blocking below interior braced wall in SDC A to C where floor joists are perpendicular to wall.

When an interior braced wall also is a bearing wall and joists below the wall are parallel to the wall, a double joist or a beam typically is provided in the floor below the wall. Occasionally this pair of joists may be spaced apart to allow for piping or vents passing vertically from the wall above through the floor. When this occurs, the double joists cannot be located directly below the wall's bottom plate. To provide a nailing surface for the bottom-plate connection of the braced wall above, 2× flat blocking should be installed in line with the braced wall's bottom plate between and parallel to these spaced joists as shown in Figure 5-7.

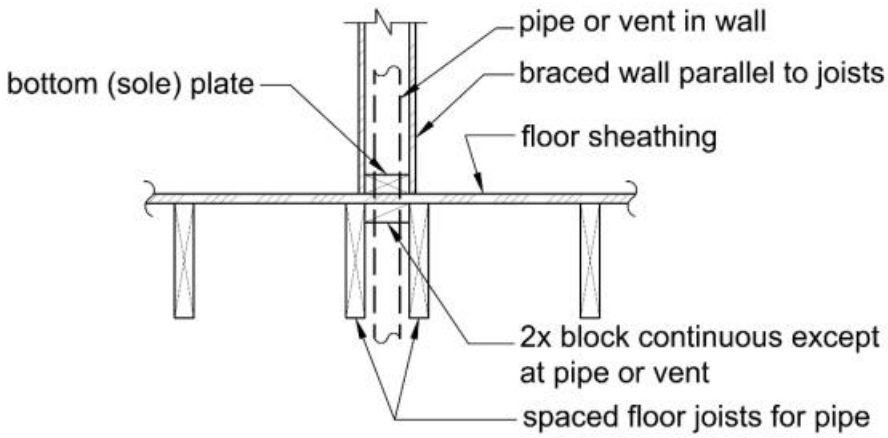


Figure 5-7 Blocking for floor joists spaced apart for piping in floor

5.6 Connection of Floor Joists to Wall Top Plate or Foundation Sill Plate Below

Floor joists (or trusses) are required to be connected to the top plate of supporting walls or to a foundation sill plate as specified in the Table R602.3(1) minimum fastening schedule and §R602.10.8. Each of these connections provides a load path to transfer loads from the floor diaphragm into the braced walls or the foundation below. Nailed connections must meet the following minimum requirements:

♣ Rim or band joists parallel to a wall or foundation require a toe-nailed connection to the wall top plate or foundation sill plate using 8d common, 10d box, or 3-inch by 0.131-inch nails at 6-inch spacing. See also Figure R602.10.8(2).

♣ Floor joists perpendicular to a wall top plate or foundation sill plate require a toe-nailed connection at each joist using four 8d box or three 8d common nails. Several other nail choices are provided. See also Figure R602.10.8(1).

♣ When blocking is installed between the floor joists, the blocking requires a toe-nailed connection to the wall top plate or foundation sill plate using three 8d common nails in each block. See Figure R602.10.8(1).

Table R602.3(1) minimum fastening schedule provides actual nail sizes in inches for both length and shank diameter. As an example, an 8d common nail is noted as 2 1/2 inches × 0.131 inches. This is intended to help ensure that proper nail sizes are used, providing the intended capacity. The builder should be able to match up this information with the dimensional information printed on the nail box or container.

Where toe nailing is used, it must be done correctly so that it can transfer the intended loads and so that the nails do not split the wood when being installed. Toe-nailed connections prescribed by the IRC should be acceptable when connecting joists to wall top plates or foundation sills that are perpendicular to the joists because these connections are not highly loaded by lateral loads. The primary lateral load transfer in a floor system occurs through the rim or band joists and through blocking that is parallel to braced walls or foundation sill plates.

Information on proper toe-nail installation is presented in Figure 5-8; however, that idealized picture of nail inclination and location is difficult to achieve in actual construction. Consequently, many toenailed connections that must transfer lateral loads may not actually perform very well.

Recommendation: Use of Light-Gage Steel Angles

In SDCs C, D_0, D_1, or D_2, it is recommended that connections between joists or blocking and wall top plates or foundation sill plates that are parallel to the joist or blocking use commercially available light-gage steel angles and nails of the correct diameter and length for the product. Many of the toe-nailed connections specified in the IRC also can be made using light-gage steel angles through which face nails are driven into the two wood framing members being connected. Although light-gage angles may require more time to install than toe nails, the angle connections should reduce splitting of the wood and can provide a more reliable connection capacity for lateral loads compared to toe nails.

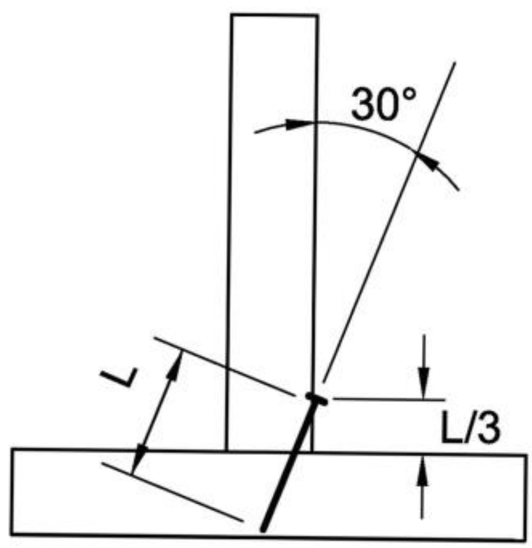


Figure 5-8 Toe-nail configuration requirements.

5.7 Floor Sheathing

Floor sheathing can be either wood (lumber) boards installed perpendicular or diagonally to the joists or wood structural panels (as subfloor or combination subfloor-underlayment) installed with the long direction of the panel perpendicular to the joists. The minimum thickness for wood board sheathing depends on joist spacing and the orientation of the boards to the joist (e.g., perpendicular or diagonal). For wood structural panels, the minimum thickness is based on joist spacing and the grade of sheathing panels selected. IRC Chapter 5 contains tables to use in determining the required minimum thickness for sheathing materials based on a variety of joist spacings.

Recommendation: Limit Use of Wood Boards Perpendicular to Joints

Wood floor sheathing boards are rarely used in modern dwelling or townhouse construction unless the underside of the floor is intended to be visible to the interior space below to achieve a specific architectural effect. As an above-code measure, wood boards installed perpendicular to joists should not be used in SDC C, D_0, D_1, or D_2 unless wood structural panels are installed over the boards because the wood board sheathing alone provides little resistance to lateral loads. In contrast, diagonally placed wood boards provide greater lateral capacity and should be acceptable for small rectangular-shaped floor areas.

For modern construction, floor sheathing typically will be wood structural panels (i.e., OSB or plywood). These panels are fastened to the joists based on a schedule prescribed in Table R602.3(1) previously discussed.

5.8 Framed Floors Using Wood Structural Panels

The lateral (seismic and wind) capacity of a floor diaphragm sheathed with wood structural panels is based on five factors:

- Sheathing thickness,
- Sheathing fastener type and size,
- Fastener spacing along supported sheathing edges,
- Presence or absence of blocking along all edges of each piece of sheathing, and
- Layout of the sheathing joints with respect to direction of lateral loading.

Below is a discussion of differences in lateral capacity resulting from variations in floor diaphragm construction.

Sheathing thickness usually is selected based on the spacing of joists and, for floors, will never be less than 3/8 inches but normally is at least 5/8 inches. Generally, thicker sheathing will provide a more comfortable floor for occupants due to reduced deflections and will have a greater lateral capacity compared to thinner sheathing using the same fastener size and spacing.

The most common sheathing fasteners used are nails with a minimum size of 6d common (0.113 inches × 2 inches) or 8d common (0.131 inches × 2 1/2 inches) for a floor sheathing thickness of up to 1/2 inch. The minimum fastener size increases with increasing sheathing thickness to a minimum of 10d common nails (0.148 inches × 3 inches) for sheathing that is 1 1/8 inches thick. Larger diameter nails will provide greater lateral capacity than smaller nails in the same thickness of sheathing because the lateral capacity of a nail is directly proportional to its diameter. Therefore, using box nails that have a smaller diameter than common nails will reduce the lateral capacity of a floor diaphragm. Both box and sinker nails with the same penny weight as common nails (i.e., 8d) are smaller and provide reduced strength. See Table R602.3(1) for permitted alternative fasteners.

Staples also can be used to fasten sheathing to framing members. Although not commonly used, Table R602.3(2) has information for specifying alternative sheathing fasteners including staples. Generally, staples of either 15 or 16 gage can be used in place of most nails with reduced spacing. However, when using staples, it is important to understand that they must be installed with the crown parallel to the length of the framing member below the sheathing edge being fastened. This is to help ensure that the staple legs are firmly embedded into the framing.

Fastener spacing for floor sheathing is typically 6 inches along continuously supported panel edges and 12 inches along supporting members not located at panel edges. Greater lateral capacity can be obtained when fastener spacing along supported edges is reduced from 6 inches to 4 or 3 inches.

The IRC requires floor diaphragms to be fastened along continuously supported panel edges. This includes where panel edges are located parallel to and over a joist and at the floor framing members forming the perimeter of the floor. The unsupported panel edges that are spanning perpendicular to the joists only need to be fastened at each joist. The floor diaphragm is also always required to be fastened to joists or blocking above all walls in the story below, as shown in previously discussed IRC figures. In engineering terms, this type of diaphragm is called an unblocked diaphragm. See Figure 5-1 for the sheathing layout and nailing pattern for a portion of an unblocked diaphragm.

Recommendation: Diaphragm Blocking

The IRC requires that wood structural panel sheathing be installed with the long dimension of the panel perpendicular to joists, but it does not specify staggering of panel joints along the short direction of the panels. Although not specifically required by the IRC, sheathing panels should be installed staggered as shown in Figure 5-1 and Figure 5-9 to achieve the greatest capacity. This staggered sheathing layout pattern causes the individual sheathing panels to interlock and makes the whole floor act as a unit.

Although not specifically required by the IRC, sheathing installed as a blocked diaphragm with blocking added at all unsupported edges (i.e., per Figure 5-9 rather than Figure 5-1) is recommended as an above-code measure.

In contrast, a fully blocked floor diaphragm means that all edges of each sheathing panel that are not located on a joist are supported on and fastened to blocking. This requires that blocking be added at all panel edges that are not already supported on framing. A blocked diaphragm will have significantly greater lateral capacity than an unblocked diaphragm having the same thickness of sheathing and attached with identical fasteners because the extra fasteners along the blocked edges provide additional capacity. Figure 5-9 shows the layout of sheathing and nailing of a portion of a blocked floor diaphragm. Fastening the sheathing to joists or blocking along all panel edges allows the shear loads being carried in the sheathing to be transferred from one panel to the next much more effectively. This, in turn, ties the floor together better and allows the braced walls below that floor to resist an earthquake more as a system than as individual walls.

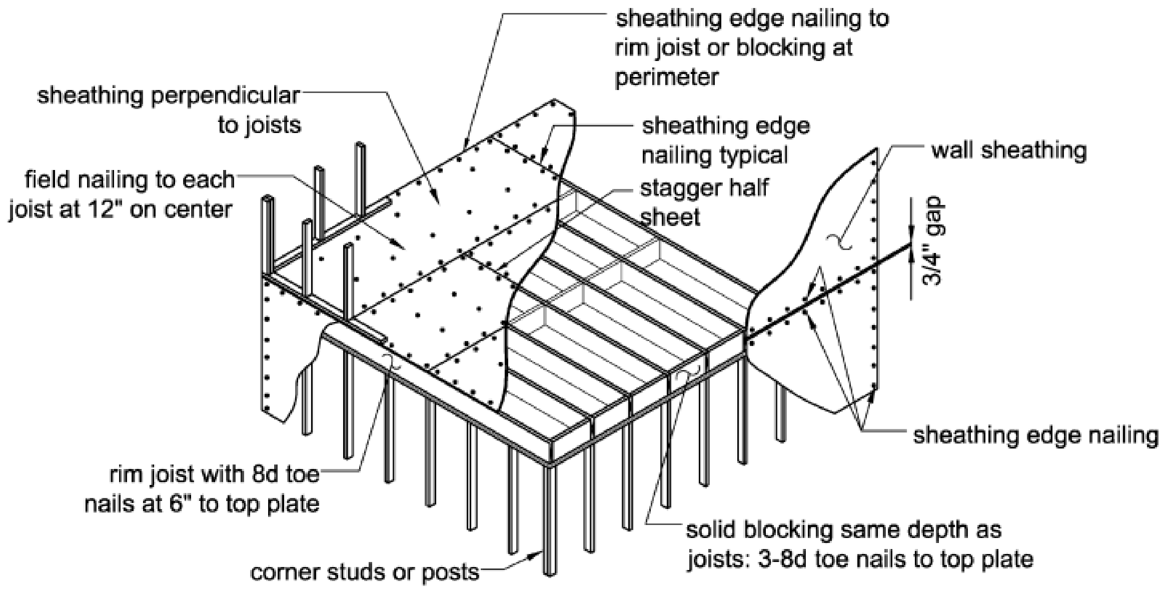


Figure 5-9 Blocked diaphragm configuration

Lateral loads in a floor diaphragm are affected by the distance between braced wall lines or between foundations located below the floor. The loads increase with increasing distance between lines of parallel braced walls or foundations. Therefore, a long and narrow floor diaphragm as shown in Figure 5-10 will have to transfer a greater load per foot along its short sides than along its long sides. In order to limit the maximum load along a short side, IRC Chapter 6 places limits on the maximum spacing between braced wall lines or foundations. These can be found in §R602.10.1.3. Additional limitations are provided relative to spacing between foundations supporting braced wall lines in §R403.1.2.

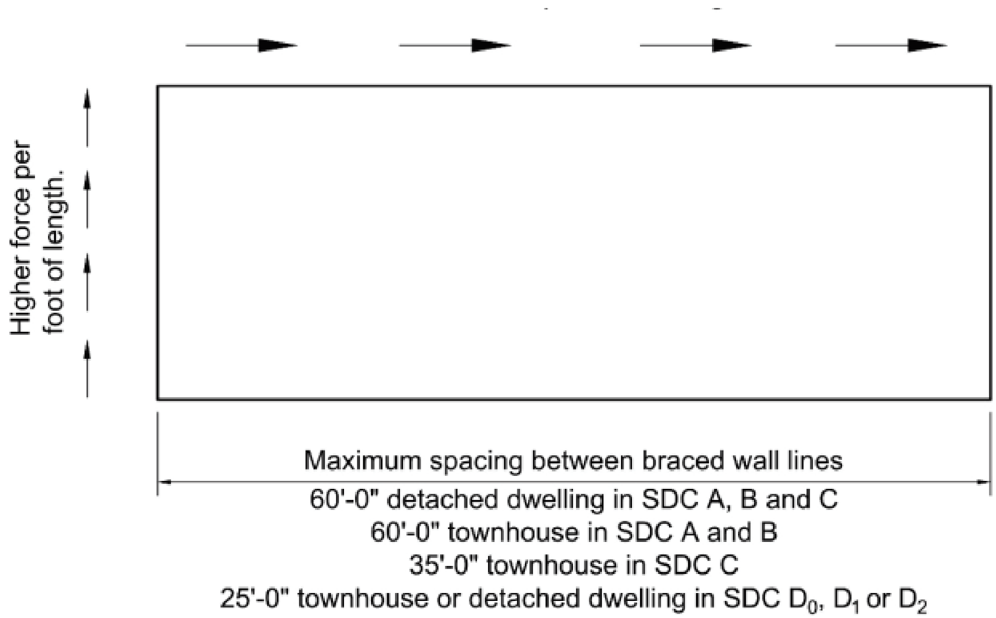


Figure 5-10 Diaphragm loads on long and short sides.

The size and location of floor openings such as for stairs or a two-story entry foyer can create concentration of lateral loads in a floor diaphragm. To address this, §R301.2.2.6 Item 4 limits openings through a floor or roof to the lesser of either 12 feet maximum or 50% of the least dimension of the floor. When openings exceed these limits, engineering of the floor or roof diaphragm is required.

Recommendation: Strapping at Floor Openings

In SDCs C, D_0, D_1, and D_2, when floor openings exceed 50% of the IRC prescriptive opening size limits, it is recommended that 16-gage straps be installed along the edges perpendicular to the joists and extended beyond the opening by at least 2 feet at each end as shown in Figure 5-11. The straps can be nailed with 10d nails into the framing members forming the perimeter of the opening and into blocking beyond the corners. The straps and additional nailing act to reinforce the diaphragm and provide a dedicated path for lateral loads to be transferred around the opening to the portions of the floor beyond. Smaller openings such as those for chimneys or duct shafts do not require any special reinforcing.

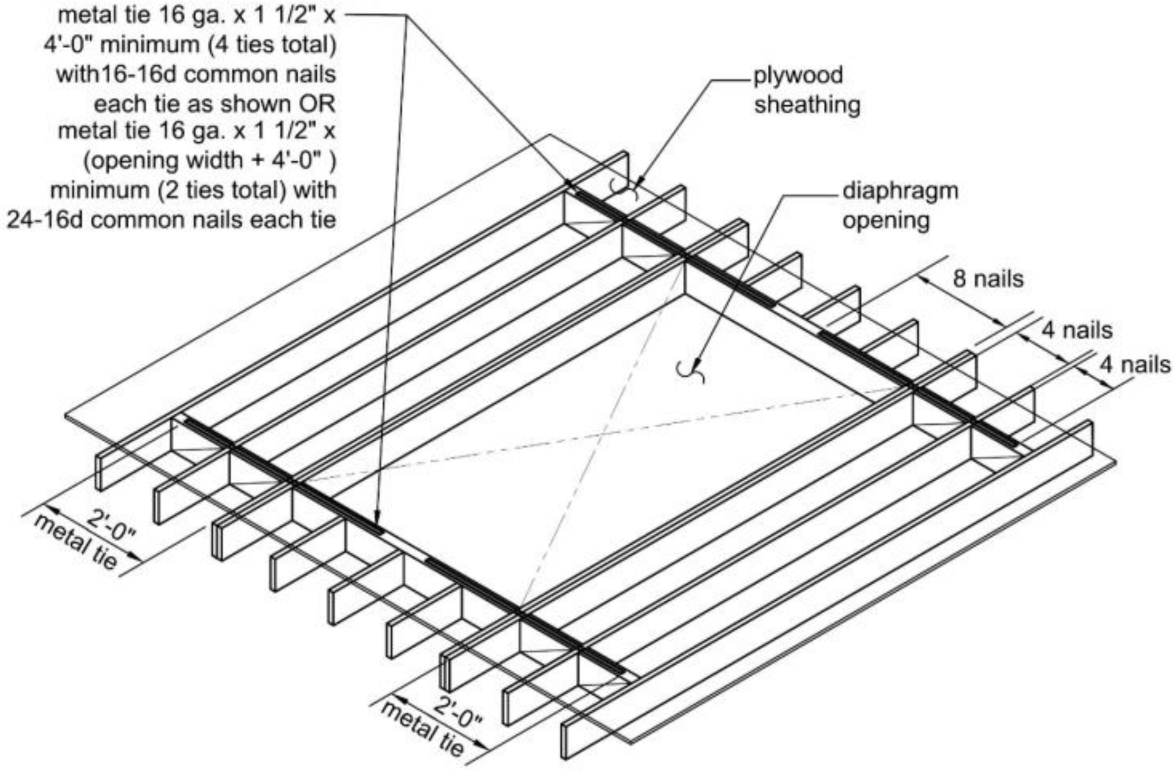


Figure 5-11 Reinforcing straps at large diaphragm openings.

5.9 Concrete Slab-On-Grade Floors

A concrete slab-on-grade (slab-on-ground) can be used as a first-floor level or a basement-level floor. The minimum thickness for a concrete slab-on-grade is 3 1/2 inches except where expansive soil is present. Where expansive soils are encountered, a design for the slab-on-grade must conform to IBC Chapter 18. See Section 4 of this course for a discussion of the effects of expansive soil.

The minimum concrete compressive strength is 2,500 pounds per square inch (psi). Floor slabs having an exterior surface exposed to the weather in areas of moderate to severe concrete weathering must have higher compressive strength as specified in Table R402.2. A map in IRC Chapter 3 identifies locations where moderate and severe weathering of concrete is expected to occur.

In the absence of expansive soils, the IRC does not require reinforcing of concrete slabs. Reinforcing typically is used to provide tension capacity in concrete and thereby reduce cracking caused by a variety of loads including shrinkage. Concrete alone has very good compression capacity but has a very low capacity for tension. Therefore, adding reinforcing bars to a slab-on-grade will provide much greater resistance to tension loads originating from earthquake loads and other soil conditions that could induce tension stress in the slab.

Reinforcing is required only where a slab-on-grade is thickened along its perimeter edge or below an interior bearing wall in SDCs D_0, D_1, and D_2. When exterior braced walls are spaced more than 50 feet apart, an interior braced wall also needs a foundation as part of the slab-on-grade in SDCs D_0, D_1, and D_2. Chapter 4 of this guide provides information on where and how much reinforcing is needed in foundations provided with a slab-on-grade.

5.10 Quality Control

Aspects of construction that are important to seismic performance of floor construction are the same as those discussed in Section 6.1.8 of this course.

6- Walls

This section addresses wall design and construction, also addressed by IRC Chapter 6. This section focuses primarily on wood light-frame construction (Section 6.2), but also addresses alternatives such as cold-formed steel (CFS) construction (Section 6.3), masonry walls (Section 6.4), and concrete walls (Section 6.5). Masonry and stone veneer provisions, addressed in IRC Chapter 6 and Chapter 7, are covered in Section 7 of this course.

6.1 The Role of Walls in Earthquakes

In residential construction, walls provide the primary lateral resistance to wind and earthquake loads, providing resistance to sliding, overturning, and racking loads from an earthquake as illustrated in Figure 6-1. The walls are the principal element for transmitting the loads from the upper stories and roof to the foundation.

As a brief review of the load path discussion from Chapter 2, seismic load from the roof/ceiling is transferred into the second-story bracing wall as depicted by the arrow at the top of the wall in Figure 6-2a. The wall deflects under this load and transmits the load to the wall base and through the floor system to the first-story wall. Resistance to the wall load is provided by the wall sheathing and its fastening to the wall framing.

Similarly, the first-story wall bracing resists loads from both the second-story wall and the second story floor system as depicted by the arrow at the top of the wall in Figure 6-2b. The wall deflects under this load and transmits the load to the wall base and the foundation. Again, resistance to the wall load is provided by sheathing and its fastening. Figure 6-3 provides an exploded view of the example house that illustrates the combination of roof-ceiling, floor, and wall bracing and their connection to the foundation below.

This behavior is the same regardless of the specific construction of the walls. The strength and stiffness of the walls determine their earthquake performance.

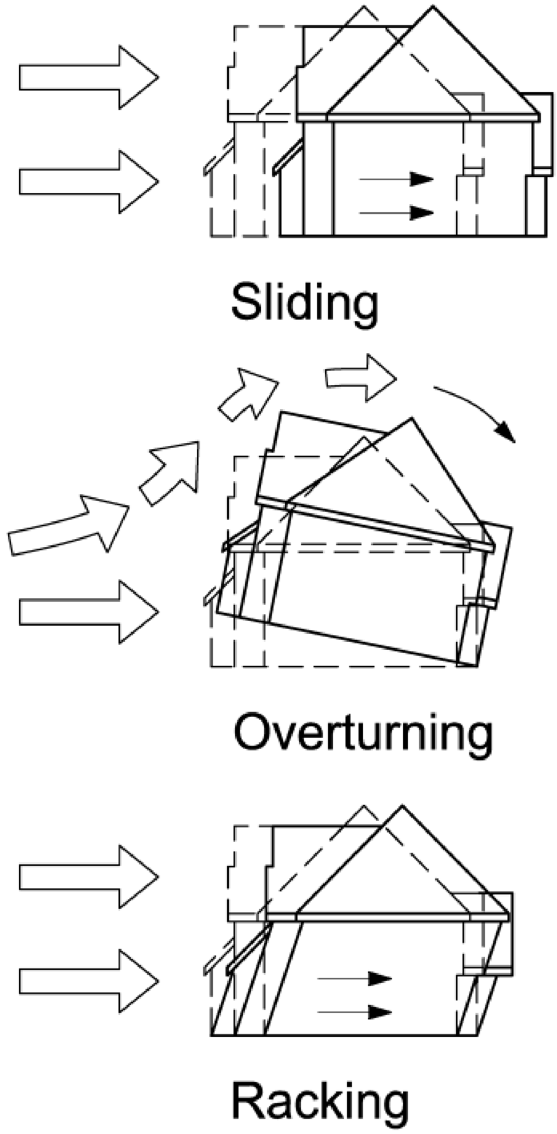


Figure 6-1 Sliding, overturning, and racking action resisted by walls and foundation

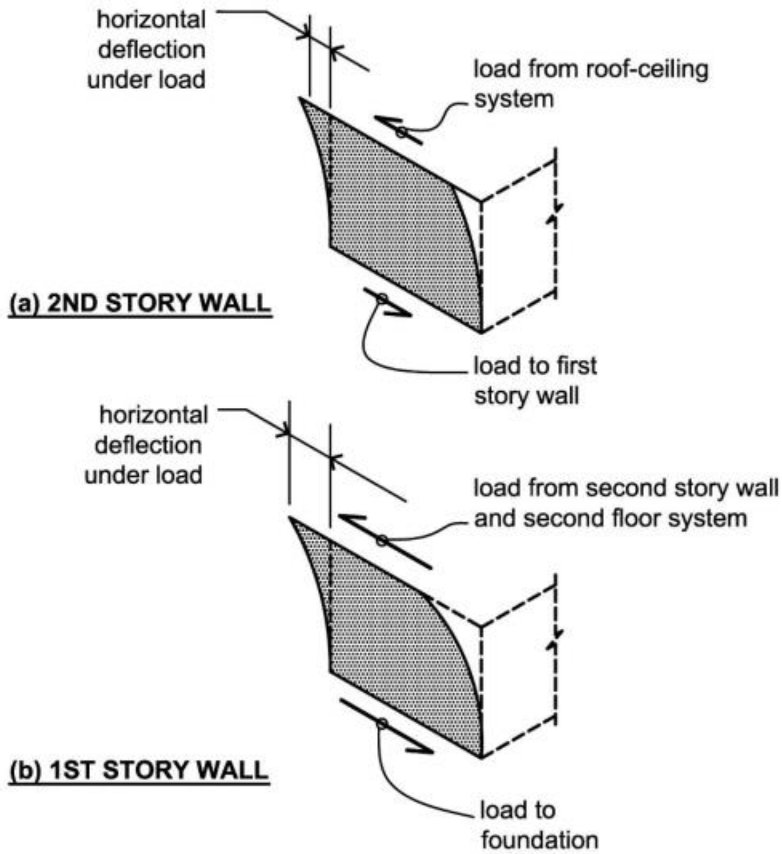


Figure 6-2 Loading and deflection of bracing wall systems

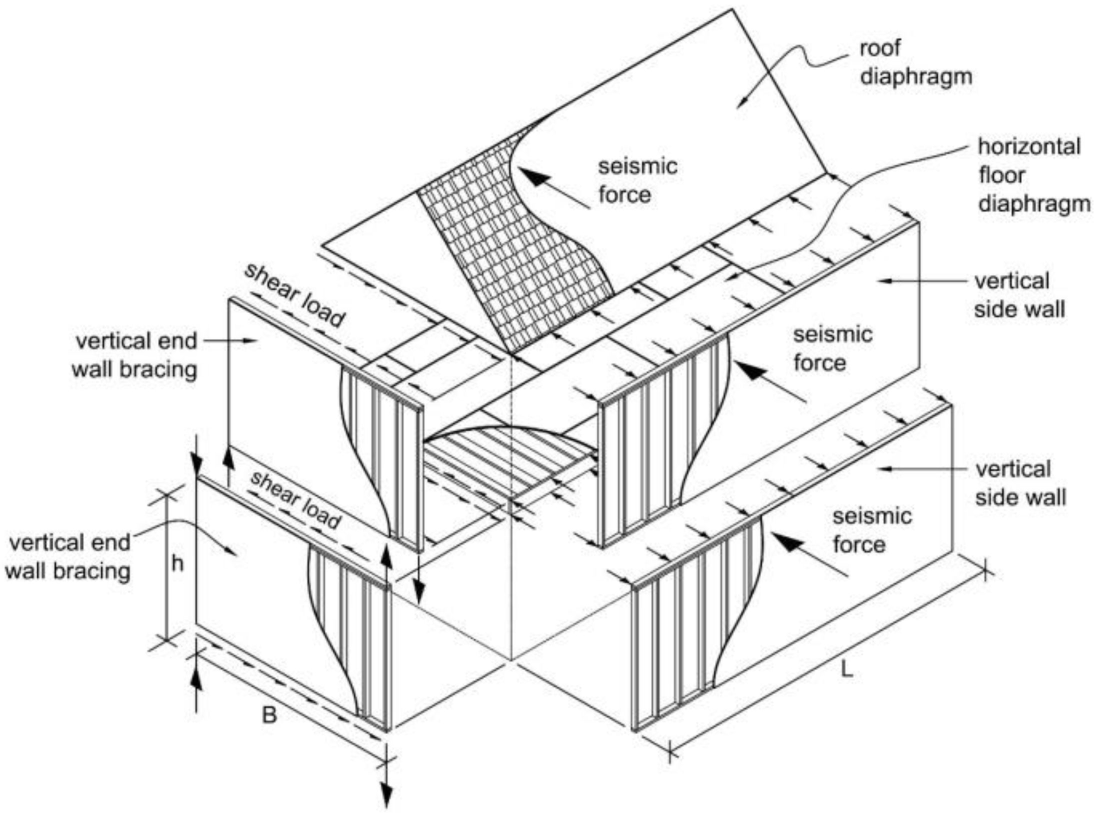


Figure 6-3 Load transfer between components in a building.

6.2 Wood Light-Frame Walls

Wood light-frame walls typically consist of lumber framing covered by sheathing material that is attached to the wood framing with nails, staples, or screws. Figure 6-4 illustrates the components of a wall that is sheathed with wood structural panels (i.e., OSB or plywood) on the outside and gypsum wallboard on the inside. The figure also shows the addition of hold-down connectors to the framing, which are required by the IRC for some specific bracing configurations. When used, hold-down connectors increase the strength and stiffness of the wall segment by providing resistance to wall overturning.

While §R602 addresses a wide range of topics related to wood light-frame wall construction, Section 6.2 focuses on the wood light-frame bracing provisions of §R602.10.

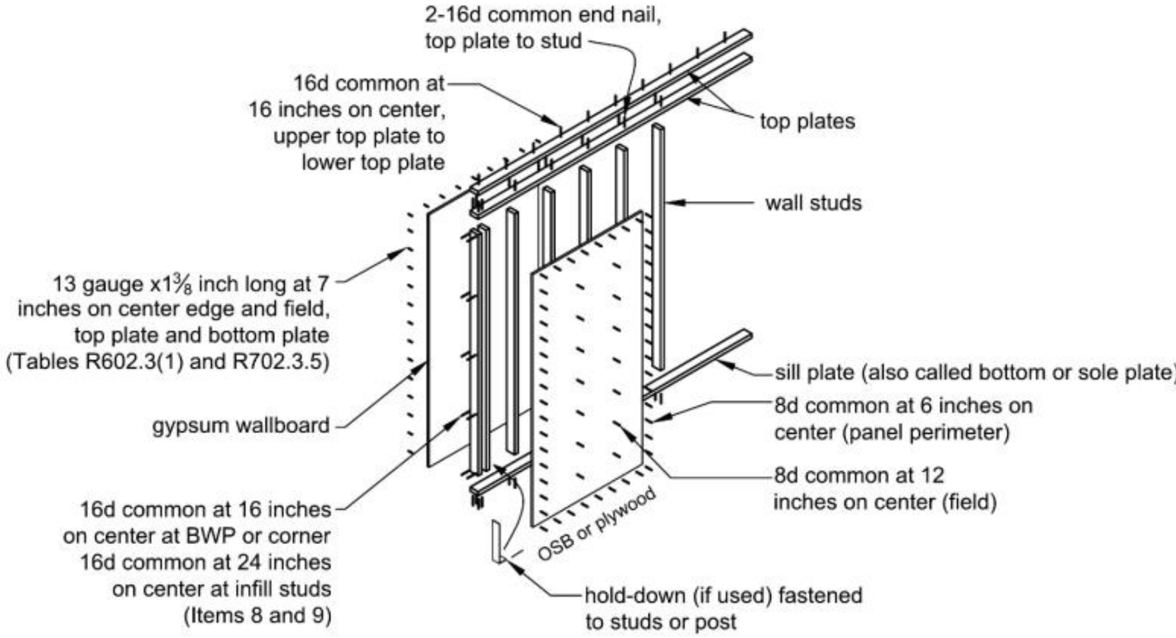


Figure 6-4 Exploded view of example residential wall segment

6.2.1 Scope Limitations

The topic of IRC scope limitations was introduced in Section 3 of this course, with a number of limitations found in IRC §R301.2.2. There are further limitations sprinkled through the IRC provisions. The following highlights some provisions of interest found in the wall bracing provisions of §R602.10.

♣ Let-in Bracing: Based on the not permitted (NP) designation in Table R602.10.3(3), let-in bracing (LIB) for townhouses is limited to single-story or the top story in SDC C, and not permitted for townhouses or dwellings in SDC D_0, D_1, or D_2.

♣ Engineering Required: Based on the NP designation in Table R602.10.3(3), engineering is required for three-story dwellings and townhouses in SDC D_2.

♣ Bracing Systems Limited: Based on the NP designation in Table R602.10.3(3), a number of bracing methods are not permitted to be used for cripple walls in SDC D_2.

♣ Garage Portal Frame and Structural Fiberboard: Based on Footnote e to Table R602.10.3(3), portal frame at garage (PFG) and continuous sheathing with structural fiberboard (CS-SFB) are not permitted in SDCs D_0, D_1, or D_2.

♣ Wall Clear Height: Based on Table R602.10.3(4), Item 1, wall clear height is limited to 10 feet with tabulated bracing and 12 feet maximum (with increased bracing wall length required).

♣ Limits on Bracing Methods: Based on Table R602.10.3(4), Items 7 to 10, additional limitations are placed on permitted bracing methods.

6.2.2 Wall Bracing Methods

Table R602.10.4 provides an overview of the wall bracing methods that are permitted. Differences in these bracing methods include sheathing materials, minimum bracing length, extent of sheathing, and anchorage at the wall base. While there are a total of 16 permitted bracing methods, these can be organized into the following six categories, as summarized in Table 6-1. Details of the construction of these wall bracing categories follow.

Acronyms

The following are the wood light-frame bracing methods from Table R602.10.4:

- **LIB:** let-in-bracing (diagonal 1×4 dapped into studs)
- **DWB:** diagonal wood boards
- **WSP:** wood structural panel (plywood or OSB)
- **BV-WSP:** wood structural panels with brick or masonry veneer
- **SFB:** structural fiber board
- **GB:** gypsum board
- **PBS:** particle board sheathing
- **PCP:** Portland cement plaster (stucco)
- **HPS:** hardboard panel siding
- **ABW:** alternate braced wall (wood structural panel with hold-downs)
- **PFH:** portal frame with hold-downs
- **PFG:** portal frame at garage
- **CS-WSP:** continuously sheathed wood structural panel
- **CS-G:** continuously sheathed wood structural panel next to garage openings
- **CS-PF:** continuously sheathed portal frame
- **CS-SFB**: continuously sheathed structural fiberboard

Table 6-1 Major Categories of Wall Bracing Methods

Category	**Wall Bracing Method**	**Reference Figure**
Intermittent (traditional) bracing	LIB, DWB, WSP, SFB, GB, PBS, PCP, HPS	Figure 6-5a
Continuous sheathing	CS-WSP, CS-G, CS-SFB	Figure 6-5b
Alternative braced wall panels	ABW	Figure 6-5c
Stone or masonry veneer exceeding the first story in SDC D_0, D_1, or D_2	BV-WSP	Figure 6-5c, similar
Portal frame bracing	PFH, PFG	Figure 6-5d
Combined continuous sheathing and portal frame	CS-PF	N/A

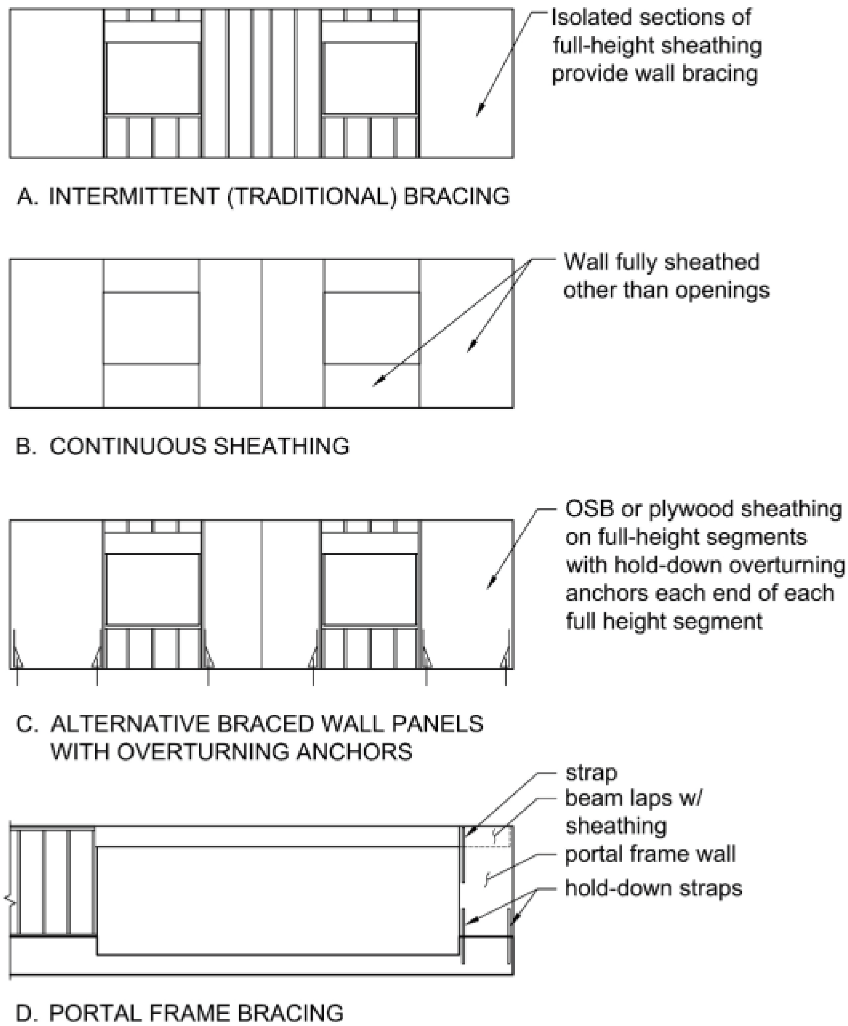


Figure 6-5 Detailing differences for different wall bracing categories

6.2.2.1 INTERMITTENT WALL BRACING CATEGORY

The intermittent wall bracing category shown in Figure 6-5A has in the past been the most common. In this method, sheathing for wall bracing is only provided where required to meet minimum bracing length requirements, and hold-down brackets or straps for overturning loads are not typically provided. Eight different methods (materials) are recognized by the IRC as acceptable for the intermittently sheathed braced wall panels. These are listed in Table 6-2. Except for one-story townhouses in SDC C, let-in bracing is not allowed to be used in regions of high earthquake hazard because it often will fail as the walls are racked during an earthquake; therefore, this method is not discussed further in this guide. Of the acceptable intermittent bracing materials, wood structural panels and diagonal lumber sheathing are known to perform better than others (i.e., withstand higher deformations while supporting higher loads).

The total required length of intermittent bracing along each braced wall line will be discussed in Section 6.1.4. There are also requirements for the minimum length for each individual section of intermittent sheathing (each braced wall panel). These vary by sheathing type and can be found in

Table R602.10.5. In general, the minimum section length should be taken as four feet (48 inches), but section lengths as short as 36 inches are permitted in some circumstances. Wall sections with smaller widths do not contribute to the IRC required bracing, and so are ignored for purposes of bracing design.

Table 6-2 Intermittently Sheathed Braced Wall Panel Construction Methods

Bracing Method	Sheathing Material
LIB	Nominal 1×4 inch continuous let-in bracing
DWB	5/8-inch minimum thickness boards applied diagonally to studs (diagonal lumber sheathing)
WSP	Wood structural panels (OSB or plywood) 3/8-inch minimum thickness
SFB	1/2- or 25/32-inch-thick structural fiberboard
GB	1/2-inch gypsum wallboard
PBS	Particleboard sheathing
PCP	Portland cement plaster
HPS	Hardboard panel siding

6.2.2.2 CONTINUOUS SHEATHING WALL BRACING

The continuous wood structural panel sheathing method (CS-WSP) requires that one face of the entire wall surface, other than door and window openings, be sheathed with wood structural panel sheathing (Figure 6-5B). Use of structural fiberboard sheathing (CS-SFB) is also permitted in lower SDCs, but not in SDC D_0, D_1, or D_2. This wall bracing configuration has greater strength and stiffness than intermittent sheathing. The increased strength and stiffness are due in part to the continuity provided by additional sheathing above and below windows and doors (which makes the wall-base connection capacity less critical) and increased overturning capacity due to required corner framing details or hold-downs at wall ends. In recognition of the improved strength and stiffness provided with continuous sheathing, §R602.10.5 permits the minimum length of bracing to be reduced below the 48-inch length generally required for intermittent bracing. The shortest braced wall panel length permitted in Table R602.10.5 for continuous sheathing methods is 24 inches.

6.2.2.3 ALTERNATIVE BRACED WALL PANELS

The wood structural panel alternative braced wall panel (ABW) bracing method, per §R602.10.6.1, is the strongest and stiffest option for resisting lateral loads. This wall configuration is illustrated in Figure 6-5c. Although sheathing, fastening, and hold-down loads for these walls are prescribed, the walls are essentially equivalent to engineered shear walls. The ABW bracing method was developed to allow use of braced wall panels narrower than the 4-foot minimum required by §R602.10.5; the IRC permits an ABW braced panel to be substituted for each 4 feet of bracing throughout the dwelling or townhouse. The minimum width for an ABW panel is 28 inches in SDC A, B, and C, and 32 inches in SDC D_0, D_1, and D_2.

6.2.2.4 WOOD STRUCTURAL PANEL BRACING WITH STONE OR MASONRY VENEER

Where stone or masonry veneer extends above the lowest story, the weight of the veneer greatly increases seismic forces on the bracing system. As a result, additional bracing is required. Bracing method BV-WSP uses wood structural panel sheathing and hold-downs, very similar to the ABW method and Figure 6-5c, to provide the additional bracing strength. This bracing method is discussed further in Section 7 of this course.

6.2.2.5 PORTAL FRAMES

Portal frames are another category for which bracing panels narrower than 48 inches can be used. Portal frames also incorporate aspects of engineered shear wall design to gain higher strength and stiffness from narrow bracing panels. The IRC includes two portal frame bracing methods, and one combination of a portal frame and continuous sheathing. They are as follows:

♣ Portal frame with hold-downs (PFH): Details of construction are found in Figure R602.10.6.2. Increased strength and stiffness come from hold-down straps at the base of the wall, as well as a special connection to the header at the top of the wall.

♣ Portal frame at garage door openings (PFG) in SDC A, B, and C: Details of construction are found in Figure R602.10.6.3. Increased strength and stiffness come from a special connection to the header at the top of the wall. This method is not available in SDC D_0, D_1, or D_2.

♣ Continuously sheathed portal frame (CS-PF): Details of construction are found in Figure R602.10.6.4. Increased strength and stiffness come from a special connection to the header at the top of the wall, in combination with the continuous sheathing.

The IRC figures noted above include significant detail in describing the required configuration and fastening. It is important that all guidance is followed in order for these bracing panels to perform as intended.

6.2.2.6 ADDITIONAL WALL BRACING CONSIDERATIONS

As discussed above, where wood structural panel bracing is used, the strength and stiffness of the wall bracing is very dependent on the extent of sheathing and anchorage at the wall base. The configuration shown in Figure 6-5a, using IRC minimum braced wall panel anchorage, is the weakest and least stiff of the wood structural panel bracing configurations. The configuration shown in Figure 6-5B adds strength and stiffness by providing continuous structural panel sheathing and added detailing or hold-downs at wall ends. The strongest and stiffest configuration is illustrated in Figure 6-5C where overturning anchors are provided at each end of each wall segment.

Recommendation: Continuous Sheathing and Hold-down Anchors

Use of the continuous sheathing (Figure 6-5b) or alternative braced wall panel with overturning anchor configurations (Figure 6-5c) can significantly increase the strength and stiffness of braced wall panels sheathed with wood structural panel or diagonal lumber sheathing and are recommended as an above-code measure to provide improved performance whether specifically required by the IRC or not. The addition of wall sheathing material above doors and above and below windows, as shown in Figure 6-5b, is of benefit regardless of the sheathing material type. Adding hold-downs as shown in Figure 6-5c is of significant benefit when in combination with wood structural panel sheathing (plywood or OSB) but of limited benefit with other sheathing materials.

When the WSP wall bracing option is used in an engineered design, the walls are designed according to empirical tables that provide allowable design loads in pounds per foot of wall length depending on the thickness and grade of the sheathing and the size and spacing of the sheathing nails. These walls rely on hold-down anchor connections to resist the overturning loads and substantial connections (e.g., nails, lag screws, bolts) along the top and bottom plates to transmit the horizontal seismic loads between the wall framing and the floor platform or foundation. This type of wall can resist allowable design loads up to 870 pounds per linear foot when sheathed on one face with wood structural panels.

6.2.3 Wall Bracing Requirements

IRC §R602.10 describes the system of wall bracing that must be provided for wood light-frame dwellings and townhouses. The system requires that braced wall lines (BWLs) be laid out using rules for location and spacing between, and then that braced wall panels be appropriately placed in each BWL. §R602.10 specifies BWL placement. Within each BWL, §R602.10.2 specifies the placement of required braced wall panels, and §R602.10.3 specifies the total required length of bracing, and §R602.10.4 through §R602.10.7 regulate the details of construction. The following discusses this bracing system in more detail.

6.2.3.1 BRACED WALL LINES

For wood light-frame dwellings and townhouses, the IRC uses the concept of a grid of braced wall lines to ensure that wall bracing is distributed through the building (Figure 6-6). The IRC requires that BWLs be provided both at exterior walls and at interior wall lines. §R602.10.1.3 specifies that interior BWLs must be added such that the spacing between BWLs does not exceed the distances specified in Table R602.10.1.3. For lower SDCs, the maximum spacing between BWLs is 60 feet. For townhouses in SDC C, this drops to 35 feet, with an allowance of up to 50 feet where other adjustments to bracing are made. For dwellings and townhouses in SDCs D_0, D_1, or D_2, the maximum BWL spacing is 25 feet, with an allowance to increase this to 35 feet. It is generally anticipated that the lines are located at the center of the respective walls. Where walls step in and out along the BWL, rules define the location of the BWL to be used while implementing BWL spacing requirements. In townhouses, the demising walls between units are BWLs.

Recommendation: BWL Spacing in SDC C

It is recommended that dwellings in SDC C be provided with BWL spacing and braced wall panel lengths as specified for SDC D_0. Use of this wall bracing will provide a better distributed system of wall bracing, which experience shows results in better earthquake performance.

6.2.3.2 BRACED WALL PANEL PLACEMENT WITHIN A BRACED WALL LINE

§R602.10.2.2 specifies the placement of required braced wall panels within a BWL, while §R602.10.2.3 specifies the minimum number of panels. Together these define the braced wall panels that will be required in each BWL. For SDCs A through C, braced wall panels are required to start at not more than ten feet from the end of the BWL and be spaced not more than 20 feet clear. Modifications apply where continuous sheathing bracing methods are used. In SDC D_0, D_1, and D_2, the basic requirement is that the first braced wall panel be located at the start of the BWL (i.e., the corner of the building). A series of exceptions are provided, based on the bracing method and where specific corner detailing is used, or hold-down brackets are added. The objective of these provisions is to provide a strong anchor at the end braced wall panels in order to improve the strength and stiffness of the BWL.

Where continuous sheathing is used, §R602.10.7 and Figure R602.10.7 provide direction.

The building corner is the ideal location for the first braced wall panel in each line because it serves to interconnect the BWL with the perpendicular BWL, providing an inherently strong and stiff configuration.

Recommendation: Braced Wall Panels at Building Corners

As an above-code measure, it is recommended that, wherever possible, braced wall panels be located at each building corner.

In general, it is required that not less than two braced wall panels be provided in each BWL. Where the BWL has a length of 16 feet or less, one braced wall panel of not less than four feet is required.

6.2.3.3 BRACED WALL PANEL TOTAL LENGTH AND ADJUSTMENTS TO TOTAL LENGTH

Within each BWL, §R602.10.3 specifies the total required length of bracing. For all buildings, the required length of bracing based on wind loading is found in Table R602.10.3(1) and modified as appropriate by the adjustment factors of Table R602.10.3(2). To look up the required length, the ultimate design wind speed, number of stories, and BWL spacing are required. The bracing length is read separately for each story of the dwelling or townhouse. Table R602.10.3(2) provides an accumulation of the many adjustments that might be applicable to the wind bracing length. For townhouses in SDC C and all dwellings and townhouses in SDC D_0, D_1, and D_2, the length of bracing based on earthquake design must also be checked, as found in Table R602.10.3(3) and adjusted per Table R602.10.3(4); the required bracing is taken as the greater of the adjusted bracing lengths from the wind and earthquake tables.

6.2.3.4 BRACED WALL PANEL DETAILS OF CONSTRUCTION

§R602.10.4 through §R602.10.7 provide details of construction for each permitted bracing method, as well as rules regulating combinations or mixing of bracing methods. The details of construction are provided in a combination of the code text, Table R602.10.4, and a series of figures.

6.2.3.5 LOAD PATH CONNECTIONS

The benefit of braced wall panels can be significantly reduced where there is an inadequate connection between the bracing wall and the construction above (roof or floor) and below (floor or foundation). These connections at the top and bottom of the braced wall panels are designated as “load path connections.” §R602.10.8 provides general requirements and details for load path connections, supplementing the requirements introduced in Chapter 3 of this guide. The following introduces these braced wall panel connections.

♣ At roofs: For dwellings and townhouses, exterior walls will be tied into the roof framing in accordance with IRC details and minimum fastening schedules. Where open ceilings occur, such that the roof framing is exposed on the interior, braced wall panels in interior BWLs will similarly extend up to the roof framing and be similarly attached to the roof. At townhouses, fire resistive provisions of §R302.2.2 require that common walls between units extend up to and are tight against roof sheathing. The above requirements will result in a load path between the roof and the braced wall panels at the full perimeter of each dwelling and each townhouse unit. In addition, with open ceilings, all interior braced wall panels will have a load path to the roof. Where typical ceilings occur, braced wall panels at interior BWLs will only extend to and attach to the ceiling. This is intended by the IRC and considered acceptable at the unit interior, as the ceiling has some ability to span horizontally and transfer load to the braced wall panels similar to the roof. §R602.10.8.2 includes provisions for connections to roof framing and a series of connection details addressing common framing conditions.

♣ At floors and ceilings: §R602.10.8 similarly includes provisions for connections to framed floors and ceilings and provide details for common configurations.

♣ To foundation or slab-on-grade: Connections between braced wall panels (and framed walls in general) and the foundations or slabs-on-ground are found in §R403.1.6. Discussion of these connections can be found in Section 4.10 of the course.

6.2.4 Cripple Wall Bracing

Cripple walls are short-framed walls that extend from the foundation to the bottom of the first floor, generally surrounding a crawlspace or basement that is partially below grade. They are most often found in the western United States. Historically, these walls have been the cause of significant failures in residential construction during earthquakes primarily due to inadequate in-plane strength or inadequate anchorage to the foundation, as discussed in Section 11.3.1 of this guide. These walls are the most highly loaded of all the light-frame walls in a dwelling or townhouse because they have to resist the entire load from the dwelling or townhouse above. Cripple wall framing and bracing are addressed in §R602.9 and §R602.10.10.

For townhouses and dwellings in SDC A and B and dwellings in SDC C, the required length of cripple wall bracing is the length required for the story above, provided that the cripple wall height is less than 4 feet (i.e., not considered an additional story). This length of bracing is required to be multiplied by 1.15 to reflect the additional lateral loads from the first-floor weight, and in addition the applicable adjustments from Table R602.10.3(2) apply.

For townhouses in SDC C and all buildings in SDC D_0 and D_1, the following additional requirements apply:

♣ The bracing method is limited to Method WSP or CS-WSP. This is based on a long history of observing cripple wall behavior in earthquakes, and the realization that the toughness of the wood structural panels is needed to have adequate earthquake performance,

♣ Clear distance between braced wall panels is reduced from 20 feet to 14 feet,

♣ Where interior braced wall panels are not supported on foundations, the length of bracing required by Table R602.10.3(3) is multiplied by 1.5. This is an adjustment that is not included in Table R602.10.3(4),

♣ Other adjustments from Table R602.10.3(4) apply. In particular for cripple walls, it is not common to have gypsum board on the interior face of the wall, resulting in an additional multiplier of 1.5 on the required bracing length.

For dwellings and townhouses in SDC D_2, the following requirements apply:

♣ Bracing length is found at the bottom of Table R602.10.3(3) specifically for cripple walls in SDC D_2.

♣ Panel spacing, interior braced wall line support and use of gypsum interior finish mirror the requirements in SDC D_0 and D_1

6.2.5 Design Examples

Examples have been developed to illustrate the required earthquake bracing for wood light-frame dwellings and townhouses. One example is a single-family dwelling, and the other is a townhouse unit. Bracing for both buildings has been determined using the provisions of §R602.10 of the 2024 IRC. These are intended to provide examples of what wall bracing requirements entail and to illustrate best practices for communicating wall bracing requirements in building plans. Figure 6-6 shows an excerpt from one of the design examples. **See Appendix C** for further information.

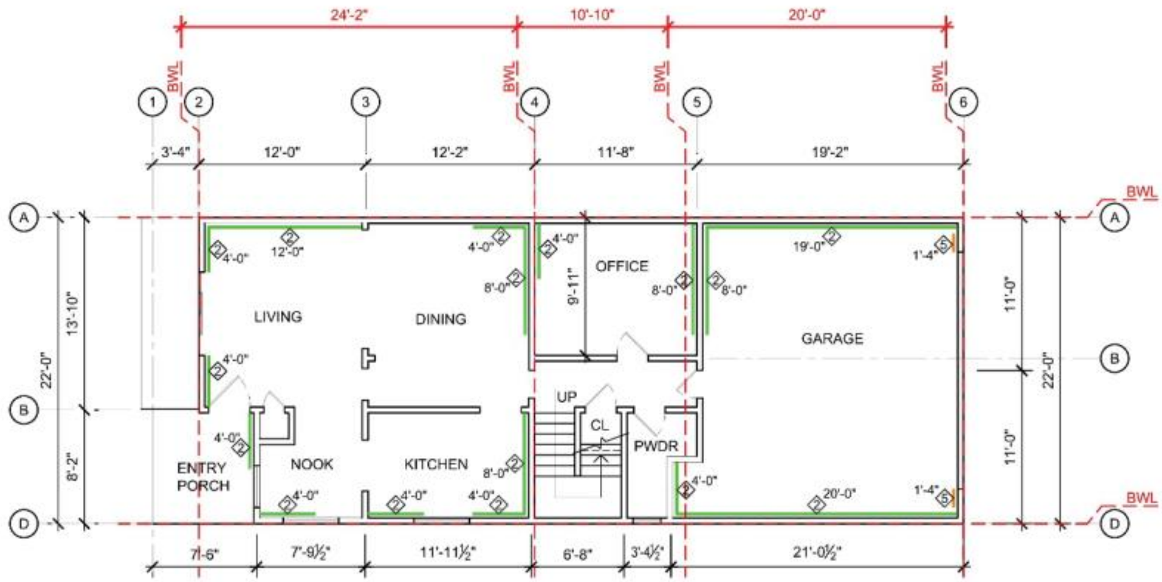


Figure 6-6 Excerpt from townhouse design example, with BWLs shown in red and braced wall panels shown in light green.

6.2.6 Additional Above-Code Measures

The following are additional recommended above-code measures for which information on both the cost and earthquake performance benefit have been developed. The single-family dwelling described in Section 6.2.5 was used as the basis for computer simulations of earthquake performance. See Appendix C for further information regarding the model dwelling and the computer analysis that form the basis of these above-code recommendations.

Improving the detailing of the braced wall system is the most effective way to obtain earthquake performance levels higher than the code minimums. It is also most effective to concentrate on the lower stories of the dwelling or townhouse since this is the area that typically has the fewest walls and experiences the highest loads during an earthquake. The following provides a discussion of the above-code recommendations, their performance impact, and their cost.

The above-code measures improve earthquake performance by reducing the drift (deflection) of the bracing walls, resulting in a reduction in damage (classified as significant, moderate, or slight damage). At the same time, the above-code measures increase the cost of construction. This has been expressed as a percent increase in the total cost of home construction (excluding the cost of the home lot). Most above-code measures that were studied increased earthquake performance while only slightly increasing home cost. Based on this ideal outcome, implementation of the above code measures is strongly encouraged.

Recommendation: Continuous Panel Sheathing

Use of continuous structural panel sheathing is recommended as an above-code measure. For this measure, the entire exterior of the dwelling or townhouse unit is sheathed in wood structural panel sheathing, including areas above doors and above and below windows. The analyses of the model dwelling used in this guide indicated that use of this above-code measure reduced the drift ratio (deflection) in all SDCs and improved the approximate performance category by one (moderate to minor or significant to moderate damage) for SDC D_0, D_1 and D_2. The cost of making this change is estimated to be approximately 2% of the cost of construction of the model house, as described in Appendix C.

Recommendation: Sheathing Over Rim Joist or Blocking

Use of wood structural panel sheathing extending over and nailed to the floor rim joist or blocking is recommended as an above-code measure. This is illustrated in Figure 6-7 and can be accomplished either by sheathing the wall with oversized panels (10-foot panels on a 9-foot wall) or by cutting and adding horizontal blocking somewhere over the height of standard-size sheets. It is important to leave a vertical gap between sheathing panels at the mid-height of the rim joist (approximately ¾ inch for green solid sawn floor framing) to allow for shrinkage of the wood floor member without causing the sheathing to buckle. The analyses of the model dwelling used in this guide indicated that use of this above-code measure in combination with continuous sheathing reduced the drift (deflection) in all SDCs and improved the approximate performance category by one (moderate to minor or significant to moderate damage) for SDC D_0, D_1 and D_2. The cost of making this change is estimated to be approximately 2% of the cost of construction of the model house, as described in Appendix C.

Recommendation: Hold-down Anchors

Use of hold-down anchors at each end of each wood structural panel wall segment is recommended as an above-code measure**.** Analytical studies of the model house used in this guide indicated that the addition of hold-down anchors reduced drift in all SDCs. The approximate performance category was only improved, however, in SDC D_1 (from "significant damage" to "moderate damage"). The cost of making this change is estimated to be approximately 4% of the cost of construction of the model house, as described in Appendix C. While this is an easy to implement above-code measure, it is the most expensive and least effective of this group of above-code recommendations.

Recommendation: Combination of Recommendations

Although not addressed in the analytical studies, use of the above-code measures in combination is thought to have a cumulative effect and is recommended. This level of connectivity will further improve earthquake performance by stiffening and strengthening the walls. Additional above-code options for increasing strength and stiffness include spacing sheathing nails closer than the standard 6 inches on center (additional detailing requirements may be applicable at 3-inch or closer spacing; see the IBC) and placing wood structural panel sheathing on both faces of walls. Use of these above-code recommendations will be most effective where the highest loads are present, such as lower stories and cripple wall levels.

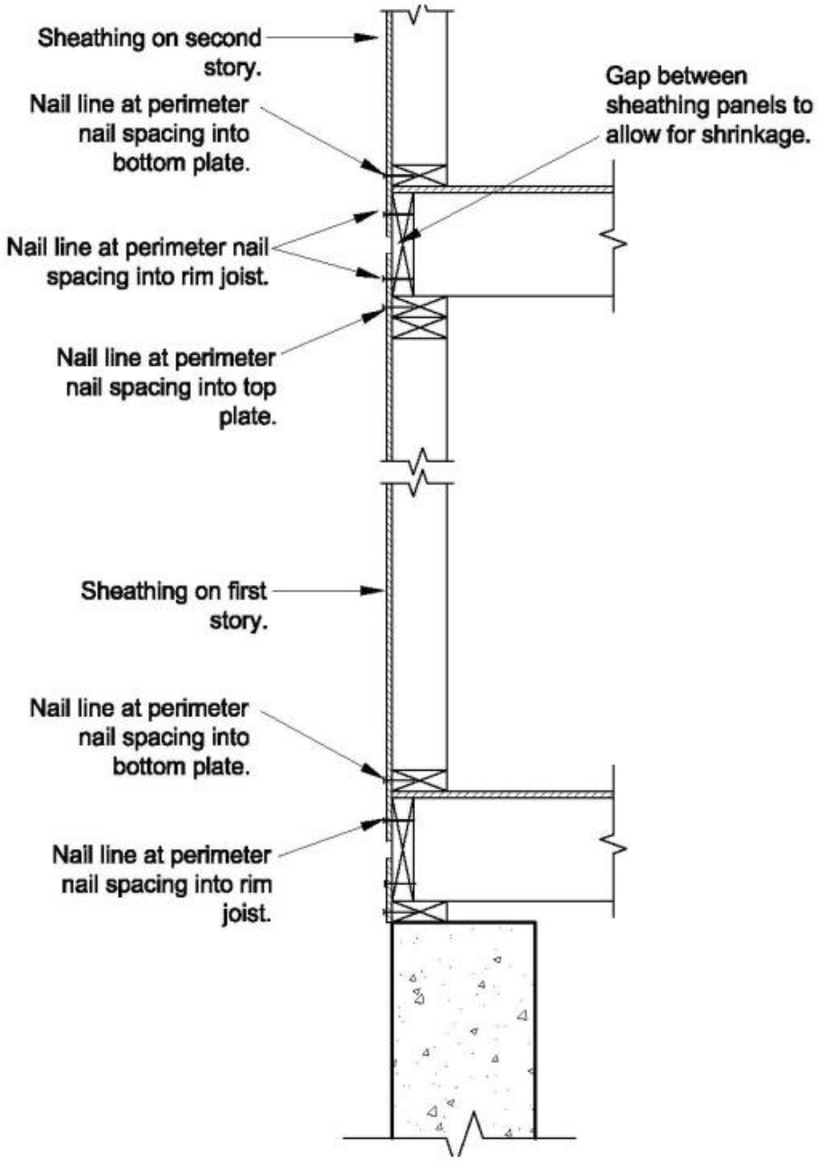


Figure 6-7 Sheathing detail for extending the sheathing over the rim joist.

6.2.7 Quality Control

Quality control for the typical light-frame wall involves inspection of three areas:

- The sheathing nails,
- The anchorage of the framing to the floor framing or foundation below, and
- The anchorage of the wall framing to the roof or floor framing above.

6.2.7.1 SHEATHING NAILS

The most common problem that adversely affects the performance of all wall types is the overdriving of the nails attaching the sheathing to the studs; this is especially a problem when pneumatic or power-driven nail guns are used. All of the nails used to attach the sheathing to the wall framing should be driven only to where the nail head is flush with the surface of the sheathing as shown in Figure 6-8. An improperly driven (overdriven) nail is also shown.

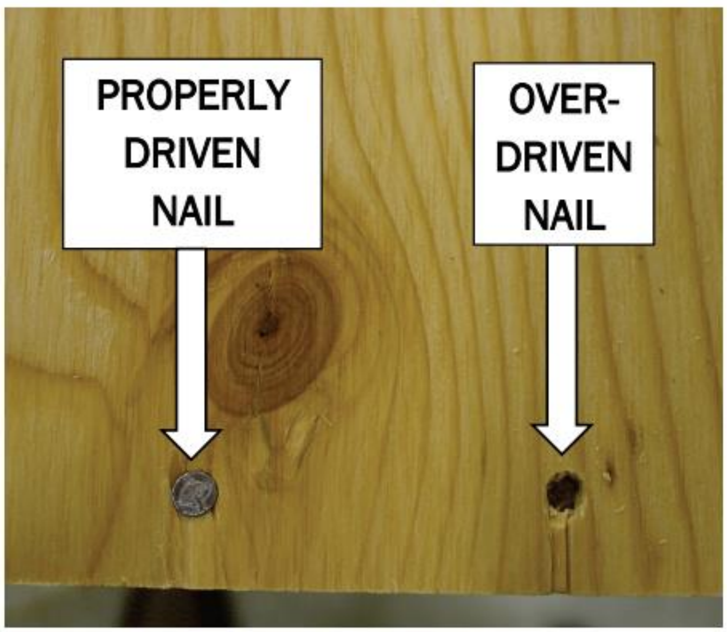


Figure 6-8 Properly driven and overdriven nails.

The most important nails are those around the perimeter of each panel of sheathing. These nails govern the strength and stiffness of the bracing. If the nails are overdriven, the strength of the connection is reduced. For instance, if 3/8-inch wood structural panel sheathing is used and the nails are overdriven 1/8 inch (typical for many pneumatic nail tools), the strength of the wall is reduced as much as 40% to 50%. It is therefore imperative that the sheathing nails be inspected to ensure that they are properly driven. Many tool manufacturers now provide an adjustment on the driving pin to allow the user to change the depth of the driving pin without replacing the part. It is recommended that where nail heads occasionally are more than 1/16 inch below the surface, an additional nail should be provided between existing nails. If a substantial number of nails are overdriven, the sheathing should be removed, and the framing checked for splitting before replacing the sheathing with proper nailing.

Nails often are located too close to the edge of the sheathing panel, which can result in the nails tearing out the side of the panel and weakening the wall. Therefore, the minimum edge distance for nailing sheathing is 3/8 inch. The larger the edge distance is, the stronger the wall will be. This is especially true for the row of nails at the bottom of the wall. If the bottom row of nails is located at the mid-height of the bottom plate for the wall, the displacement capacity of the wall is almost double that with the nails spaced at 3/8 inch. When the edges of two sheathing panels meet on a common stud (2x nominal), the minimum and maximum edge distance should be 3/8 inch to prevent the nail from splitting the edge of the stud behind the sheathing. Figure 6-9 shows a comparison of adequate and inadequate edge distance.

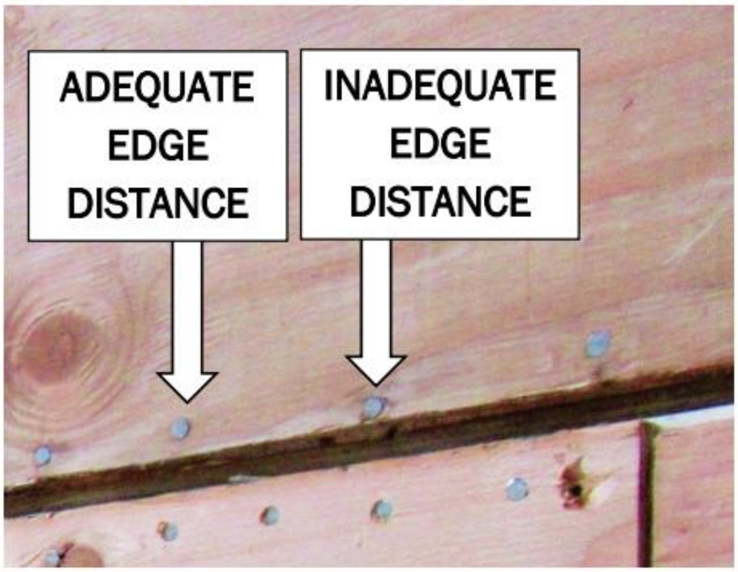


Figure 6-9 Properly spaced nail distance from edge of sheathing (edge distance).

Another common problem is that the nails are driven through the sheathing but miss the stud behind. Although it is obvious that nails not embedded into the framing provide no strength or stiffness, this problem is so common it merits emphasizing that sheathing nails must be driven into the studs and top and bottom plates of the wall and not miss the framing. The use of pneumatic nail guns make it very difficult for the operator to "feel" whether or not the nail has been driven into the stud behind. Therefore, all such nailing must be visually inspected from behind to ensure that the nails did not miss the studs.

Finally, the sheathing nailing around the perimeter of each sheet of sheathing should be symmetric about the center of the sheathing panel. This means that there should be approximately the same number of nails along each parallel side of the panel as illustrated in Figure 6-10.

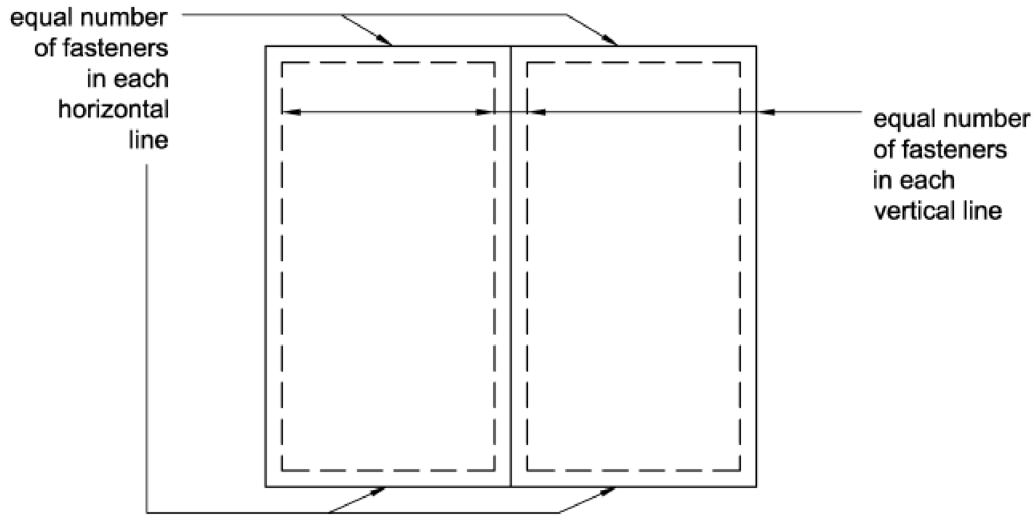


Figure 6-10 Illustration of concept of equal numbers of fasteners in line for symmetric nailing schedule.

6.2.7.2 ANCHORAGE AT BOTTOM OF WALLS

Another area to inspect is the anchorage at the bottom of the wall. For prescriptive walls, the nails must pass through the bottom plate into the floor framing (Table 3-2, Item H5). If the nails miss the floor joists, they have no capacity to resist withdrawal and sliding. If the wall is attached directly to the foundation, anchor bolts should be spaced at a maximum of 6 feet on center (Table 3-2, Item H10), and the end bolts should not be more than 12 inches or closer than 7 bolt diameters from the end of the plate. For townhouses in SDC C and dwellings and townhouses in SDC D_0, D_1, or D_2, the anchor bolts are required to have a steel plate washer between the wood sill plate and the nut. See Section 4.10 of this guide for further discussion of anchorage to foundations.

If the bracing walls have hold-downs, the hold-down connector needs to be attached to an end stud (usually a double stud or post) that has sheathing edge nailing, and the bolt or threaded rod holding the connector to the foundation or story below needs to be tightened to a snug one-quarter turn condition. Further tightening the nut does not provide better performance due to the fact that the tension in the bolt will be lost over time. A common practice to prevent the nuts from loosening during an earthquake is to use a double nut or slack take-up device (often available from the holddown manufacturer). The hold-down should connect the end stud either directly to the foundation or to the top of a stud in a wall of the story below in order to effectively transmit the overturning loads to the foundation.

6.2.7.3 ANCHORAGE AT TOP OF WALLS

The connection between the wall top plates and the roof framing or floor framing for the story above should be checked. The lateral loads are transmitted through nails or other fasteners connecting the top plate of the wall to the floor or ceiling joists above. (For vaulted ceilings, the connection might be between the top plate and the rafters.) If there is a wall on the story above that has a hold-down overturning anchor attached to the end stud, there should be an equivalent connection near the top of a stud in the story below that is upside down from the one on the wall above. A threaded rod should connect the two hold-down anchors to transmit the loads between them. If a stud is being used to transmit overturning loads from the story above, it will need to have an overturning anchor at its base to transmit the loads to the next story or foundation below.

6.3 Cold-Formed Steel Construction

The IRC permits construction with cold-formed steel (CFS) framing. This guide section discusses the general principles of earthquake-resistant design for CFS dwellings or townhouses, specific IRC requirements important to earthquake performance, and above-code measures for improved earthquake performance.

CFS construction is, in many respects, like wood light-frame construction. The IRC contains provisions similar to the wood light-frame provisions for CFS floors, roofs, and walls. The weights of CFS floor, roof, and wall assemblies are the same or slightly lower than those with wood light-frame construction and result in very similar earthquake loads. The system resisting wind and earthquake loads most commonly consists of floor and roof assemblies acting as horizontal beams carrying loads to shear walls. CFS shear walls carry wind and earthquake loads to the foundation. The IRC provides extensive illustrations of minimum construction requirements for CFS framing. One significant difference between steel and wood light-frame construction is that steel construction assumes that the framing is in-line – that is, that the rafters, studs, and floor joists align vertically so that the top tracks of the walls and other elements are not subjected to compression and bending loads. Figure 6-11 shows a CFS dwelling under construction.

Figure 6-11 Cold-formed steel house under construction (from Dietrich Metal Framing -- a Worthington Industries Company)

6.3.1 IRC Limitations and Requirements

The IRC limits the scope of CFS dwellings and townhouses permitted under the prescriptive provisions. Some scope limitations are found in IRC Chapter 3 while others are found in §R603.

For all SDCs, CFS dwellings and townhouses are limited to three stories above grade; the maximum permitted plan dimensions are 60 feet perpendicular to truss or joist span and 40 feet parallel to truss or joist span (§R505.1.1, §R603.1.1, §R804.1.1). For all SDCs, story height is limited to a 10-foot stud clear height and to a clear height plus framing height not to exceed 11 feet 7 inches (§R301.3). Although not explicitly stated in the IRC, the maximum plan dimensions are consistent with one- and two-family detached dwellings and do not accommodate townhouses, which are required to have three or more attached townhouse units. The use of one- and two-family dwellings is also consistent with the related AISI S230 standard.

For dwellings and townhouses in SDC D_0, D_1, and D_2, the IRC requires use of the AISI S230 prescriptive method in addition to the provisions of the IRC (§R301.2.2.8). As a result, the provisions in the IRC only address SDC A through C, as well as wind speeds less than 140 mph.

Wall bracing provisions for the SDC A though C scope that is fully addressed by the IRC can be found in §R603.9. The basic premise is that all sheathable areas of exterior walls be sheathed with wood structural panels (plywood or OSB). In addition, tables provide a minimum length of the braced wall line that must have full height sheathing. The length of full height sheathing varies depending on whether or not hold-downs are provided. With maximum plan dimensions for CFS buildings limited to 40 feet by 60 feet, in SDC A through C bracing is only required to be provided at the exterior walls; no interior braced wall lines or braced wall panels are required. As in wood-frame construction, load path connections are important to the performance of the bracing systems; these are detailed in a series of IRC provisions and figures.

Because dwellings and townhouses in SDC D_0, D_1, and D_2 must meet the requirements of AISI 230, the IRC does not include additional requirements for these SDCs. One exception, however, is the inclusion in §R603.9.5 and Tables R603.9.5(1) to (4) of bracing requirements for CFS buildings with stone or masonry veneer. The concept of this bracing is similar to that used for bracing method BVWSP for wood-frame construction.

6.3.2 Earthquake Requirements

There are no IRC maximum spacing limitations between lines of CFS bracing walls corresponding to the wood light-frame braced wall line spacing limits of 25 and 35 feet. With wood light-frame construction, these spacing limitations are intended to help distribute the bracing walls in proportion to the house mass (dead load) as well as to limit the loading to the floor and roof. This helps to reduce concerns regarding house rotation due to torsional irregularities and concerns regarding irregular floor and roof system shapes.

Recommendation: Maximum Braced Wall Line Spacing

For CFS dwellings and townhouses, the maximum 60-foot dwelling or townhouse dimension provides some limit for earthquake loads in the floor and roof; however, for improved earthquake performance, it is recommended as an above-code measure that interior CFS braced wall lines be added such that the distance between braced wall lines does not exceed 35 feet. This will help lower the loads within any given wall segment and better distribute the earthquake loads throughout the dwelling or townhouse. The cost associated with adding additional bracing walls would be consistent with the percentage increase in wall length.

6.3.3 Quality Control

Quality control for CFS construction can play a major role in the satisfactory performance of the dwelling or townhouse. Important steps in steel construction include:

- Ensuring proper load path connections. Just as in wood light-frame construction, the connections between the bracing walls and the floor, foundation, and roof become critical for the strength and stiffness to be the highest possible. Strong connections ensure the load path will function properly.
- Adequate lateral bracing of all floor, roof, and wall framing members is important to prevent the individual members (joists or studs) from buckling. Cold-formed members have to be braced well before they can support significant compression loads. Attaching roof, floor or wall sheathing to the stud or joist typically will provide this lateral support.
- Screws that attach the sheathing to the framing should be inspected to ensure that they are not overdriven (the resulting loss in capacity will be similar to that experienced in wood light-frame construction when nails are overdriven). Just as in wood framing, the power tools used to install the screws can easily over drive the screws, leaving the head of the screw below the surface of the sheathing material and significantly increasing the chances of pull through.

6.4 Masonry Walls

While many dwellings and townhouses use masonry walls for foundation stemwalls, some are built using masonry for the walls above grade. Figure 6-12 shows a dwelling constructed using masonry for the walls. The IRC permits construction of dwellings and townhouses with masonry walls. Provisions for masonry construction appear in §R606. Discussed below are the general principles of earthquake-resistant design for masonry wall dwellings and townhouses, specific IRC requirements important to earthquake performance, and above-code measures for improved earthquake performance.

Figure 6-12 House constructed with masonry walls

Masonry wall construction is significantly heavier than light-frame wall construction; however, in most respects, the same principles of earthquake-resistant construction apply. The system resisting wind and earthquake loads most commonly consists of light-frame floor and roof assemblies acting as horizontal beams carrying loads to masonry bracing walls. Masonry bracing walls carry wind and earthquake loads down to the foundation with masonry walls primarily resisting loads acting in their strong direction – that is, parallel to the wall length. Because masonry walls are heavier than stud framing, it follows that the earthquake loads resisted by each wall element and connection will be greater. As a result, the design of the connections is as important as the proportioning of the floor and roof systems and bracing walls. Significant earthquake loads also develop perpendicular to the wall surface (out-of-plane) due to the wall's weight. These loads tend to pull the walls away or push them toward the floor or roof, making wall to floor or roof anchorage back to the floor or roof critical. Damage from insufficient anchorage or connectivity similar to that shown in Figure 6-13 can result if these connections are not strong enough.

Figure 6-13Earthquake-damaged unreinforced masonry wall structure in Christchurch, New Zealand (from Fred Turner)

6.4.1 Scope Limitations

One of the primary approaches the IRC uses to deal with the increased earthquake loading associated with masonry wall dwellings and townhouses is to limit the scope of the dwelling or townhouse permitted without an engineered design.

Important for higher SDCs is the limitation of dwellings and townhouses in SDC D_0, D_1, and D_2 to one story. This can be found in Table R606.12.2.1, with §R606.12.1 also being referenced from §R301.2.2.4. Construction with additional stories will require an engineered design. Similarly, the provisions of Table R606.12.2.1 limit townhouses in SDC C to two stories of masonry or one story of light frame over one story of masonry. In addition, it is important that §R301.2.2.6, for townhouses in SDC C and dwellings and townhouses in SDC D_0, D_1, and D_2, requires design in accordance with accepted engineering practice for buildings with irregular configurations regardless of construction materials.

The discussion that follows is applicable to dwellings and townhouses that fall within the scope of the masonry provisions.

§R301.2.2 introduces earthquake provisions applicable to townhouses in SDC C and dwelling and townhouses in SDCs D_0, D_1, and D_2. The first scope limitation is found in §R301.2.2.2, *Weights of Materials*, which limits masonry walls to 8 inches thick and 80 pounds per square foot, which essentially prohibits the use of rubble stone masonry (§R606.4.2 specifies a minimum thickness of 16 inches for rubblestone) as well as relatively thick concrete or masonry walls.

§R301.2.2.4 triggers additional earthquake design requirements of §R606.12 for townhouses in SDC C and dwellings and townhouses in SDC D_0, D_1, and D_2. Further discussion is provided below.

A final scope limitation in §R301.3, Item 3, limits masonry house walls to a maximum story height (clear height plus floor or roof assembly height) of 13 feet 7 inches and bearing wall clear height of 12 feet. An additional 8 feet of masonry wall height is permitted for gable end walls.

It is also worth noting that §R301.2.2.6, Item 7, prohibits the mixing of light-frame and masonry construction such that light-frame walls would be required to support earthquake loads due to masonry wall construction (e.g., mixing masonry and light-frame walls on the same story.) As a result, in order to be designed and constructed under the IRC provisions, all bracing walls must be of masonry.

6.4.2 Wall Lateral Support and Anchorage

General requirements for lateral support of masonry walls for all SDCs are provided in §R606.6.4. Walls are permitted to be laterally supported by cross-walls, pilasters, buttresses, or structural frame members where walls span horizontally between supports and by floors and roofs where walls span vertically between supports.

§R606.11 requires anchorage of masonry walls to floor and roof systems in accordance with the details in Figures R606.11(1) through R606.11(3). These details, however, do not include robust anchorage of the walls to the floor and roof for out-of-plane earthquake or wind loading. In recent U.S. earthquakes, similar details have been observed to be susceptible to damage. For townhouses in SDC C and dwellings and townhouses in SDC D_0, D_1, and D_2, in addition to the figures, §R606.12 requires design of masonry wall connections per TMS 402, triggering an engineered design of the anchorage. Engineered wall anchorage should result in connections with better earthquake resistance.

6.4.3 Additional Earthquake Requirements

§R606.12.2.1 specifies the minimum length of full-height wall without openings, in order to provide adequate in-plane strength of exterior bracing walls. This is similar to the requirements for minimum length of braced wall panels found in the wood light-frame provisions of §R602.10. It is noted that the length of solid wall is only regulated for higher SDCs. The lengths are given as a percentage of the exterior wall length; there are minimum lengths of wall piers that can be counted towards this percent. As discussed in Section 6.4.1, this section and table greatly limit the scope of dwellings and townhouses that can be constructed in accordance with the IRC.

§R606.12 contains a number of additional requirements for townhouses in SDC C and dwellings and townhouses in SDC D_0, D_1, and D_2. The additional provisions are presented first for SDC C, followed by SDC D_0 and D_1, and finally SDC D_2. The provisions are cumulative. Dwellings and townhouses in SDC D_0 and D_1, are required to comply with the requirements for SDC C, plus the additional requirements for SDC D_0 and D_1. The following notes are some of the additional seismic requirements.

SDC C:

♣ §R606.12.1.1 gives minimum construction requirements for light-framed floor and roof diaphragms.

♣ §R606.12.2.2 includes a series of provisions for masonry walls that are not part of the wind and earthquake bracing system, requiring that they either be isolated from the structure or their impact on structure response determined, and providing additional minimum reinforcing requirements.

♣ §R606.12.2.3 includes the requirement that connection of masonry walls be designed in accordance with TMS 402. Although the language specifies shear walls, all exterior walls are loaded in shear and out-of-plane, and the engineered design will need to consider both load directions and their interaction. Also included are provisions for masonry columns and additional minimum reinforcing requirements.

SDC D_0 and D_1:

♣ §R606.12.3 prohibits use of autoclaved aerated concrete (AAC) masonry.

♣ §R606.12.3.1 requires design per TMS 402 of elements other than partition walls.

♣ §R606.12.3.2 and §R606.12.3.3 provide additional minimum reinforcing requirements.

♣ §R606.12.3.4 and §R606.12.3.5 include additional material and reinforcing restrictions.

SDC D_2:

♣ §R606.12.4.1 and §R606.12.4.2 include additional detailing requirements.

§R606.4.4 limits masonry thickness and height of parapets in all SDCs and sets the requirements for reinforcing of parapet walls for townhouses in SDC C and dwellings and townhouses in SDC D_0, D1, and D_2. Parapet walls are easily damaged in an earthquake and attention to placement of reinforcement and bracing to stabilize the parapet is important.

6.4.4 Above-Code Recommendations

The current IRC requirements for masonry construction reveal significant gaps between prescriptive and engineered construction, creating a significant opportunity to employ above-code measures to improve performance.

Recommendation: Solid Wall at Exterior and Interior Bracing Walls

Each exterior wall and each interior bracing wall should have at least one, and preferably two, sections of solid wall not less than 4 feet in length. Further, sections of solid wall should be placed as symmetrically as possible (Figure 6-14). The improvement requires only reasonable planning and should not result in higher construction costs.

§R403.1 requires that exterior walls be supported on continuous concrete or masonry footings and regulates minimum footing width and depth, but nothing in the IRC appears to require that other masonry walls be supported on foundations.

Recommendation: Continuous Footings

It is vital that all masonry walls be supported on substantial continuous footings extended to a depth that provides competent bearing. If this is not done the walls have a high probability of being damaged due to uneven settlement.

There are no IRC maximum spacing limitations between lines of masonry bracing walls corresponding to the light-frame limits of 25 and 35 feet. In light-frame construction, these spacing limitations are intended to help distribute the bracing walls in proportion to the dwelling or townhouse mass (dead load) as well as to limit the loading to the floor and roof, which helps prevent damage to the dwelling or townhouse from rotation due to torsional irregularities caused by irregular floor and roof system shapes.

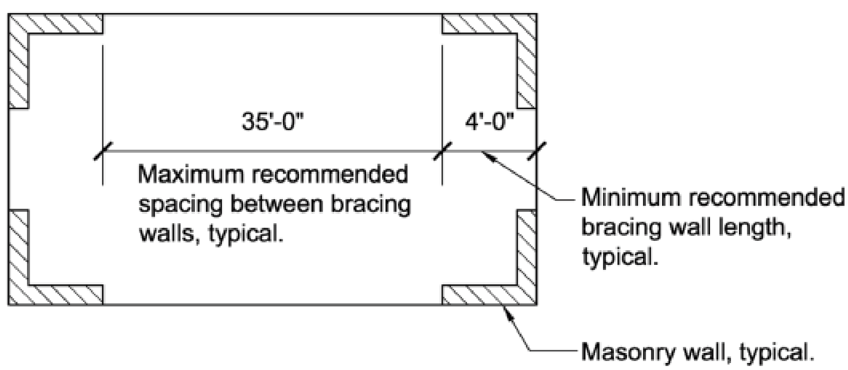


Figure 6-14 Symmetric layout of walls to distribute loads uniformly and thereby prevent torsion

Recommendation: Regular Configurations

Because prescriptive detailing is not available for the addition of interior masonry braced wall lines, it is recommended that the distribution of bracing walls be carefully balanced and that the floor and roof plans use simple rectangular shapes without indentions, bump-outs, or openings. The cost associated with this improvement is proportional to the length of wall added in relation to the initial total wall length.

Recommendation: Vertical Continuity of Walls

Solid portions of wall should be aligned (stacked) vertically from floor to floor and masonry walls should be continuous from the top of the structure to the foundation. Masonry walls not directly supported on walls below require engineered design for gravity load support. Design for earthquake and wind loads should also be provided.

Recommendation: Bond Laying

Running bond lay up of masonry units is inherently much stronger than a stack bond layup. For concrete masonry, use of open-end units at locations of vertical reinforcement and use of bond beam units for horizontal reinforcement also help to increase the interlocking of the masonry construction, thereby increasing strength. The cost associated with using running bond rather than stacked bond masonry is minimal. The cost of bond beam masonry units for placing horizontal reinforcement at the top of each wall segment would be significantly less than 1% of the cost of the original structural system but would dramatically improve the performance of the wall and connection to the floor or roof framing.

Recommendation: Design Lower Hazard Using High Hazard Provisions

Any of the many required or recommended measures for areas of high earthquake hazard would improve the performance of masonry wall dwellings or townhouses in areas of lower earthquake hazard as well as in high-wind areas. Priorities include provision for reinforcing such as that shown in Figure R606.11(2), wall anchorage using details developed to resist out-of-plane wall loads such as those shown in Figures R608.9(1) to (7) for walls, minimum length of bracing walls, and a maximum spacing of bracing wall lines.

6.4.5 Quality Control

Quality control during masonry construction can play a major role in earthquake performance.

The following should be monitored:

- Proper placement of reinforcing in masonry. Unless specifically designed otherwise, reinforcing should be located as near the centerline of the masonry cavity as possible. In no case should reinforcing be closer than 5/8 inch to a masonry unit wall.

- Anchor bolts should be secured into place before grout placement (i.e., not placed into grout following the pour).

- Excess mortar and other obstructions should be cleaned from the cavity to allow free placement of grout.

- Consolidation of the grout after it is placed into the cavity is necessary to eliminate voids.

6.5 Concrete Walls

The IRC permits construction of dwellings and townhouses with concrete walls in accordance with the provisions of §R608 for insulating concrete form (ICF) walls. ICF walls are concrete that is cast into forms that remain in place to serve as insulation. The code covers three geometries of ICF forms: flat, waffle-grid, and screen grid forms. Both the waffle and screen grid consist of concrete cast into a series of interconnected horizontal and vertical cores within the insulation. Interior and exterior wall finishes are applied over the ICF wall form. ICF walls may be used for the full height of the dwelling or townhouse or for the lower story only with light-frame walls above. The provisions of §R608 are based on the use of light-frame floor, roof, and ceiling assemblies in ICF wall dwellings and townhouses. The light-frame construction can be either wood or steel light-frame. This section discusses general principles of earthquake-resistant design for ICF wall dwellings and townhouses, specific IRC requirements important to earthquake performance, and above-code measures for improved earthquake performance.

ICF wall construction, like masonry, is significantly heavier than light-frame wall construction. In most respects, however, the same principles of earthquake-resistant construction apply. The system resisting wind and earthquake loads consists of floor and roof assemblies acting as horizontal beams carrying loads to bracing walls. ICF bracing walls carry wind and earthquake loads down to the foundation with ICF walls primarily resisting loads acting in their strong direction, parallel to the wall length (shear walls). Because ICF walls are heavier than light-frame walls, the earthquake loads to be resisted by each wall element and connection will be greater. As a result, proportioning of the floor and roof systems and bracing walls is important as are the connections between the walls and the floor and roof framing. Significant earthquake loads also develop perpendicular to the wall surface due to the wall weight. These loads tend to pull the walls away or push them toward the floor or roof, making wall to floor and wall to roof anchorage very important.

6.5.1 Scope Limitations

One way the IRC uses to address the increased earthquake loading with ICF wall dwellings or townhouses is to limit the scope of dwellings and townhouses permitted under the prescriptive provisions. For ICF, some scope limitations are found in IRC Chapter 3 while others are found in §R608.

Important for higher SDCs is the limitation of use of the IRC provisions to dwellings and townhouses in SDC A and B and dwellings in SDC C. This can be found in §R301.2.2.5 and as one of the §R608.2 limitations; other dwellings and townhouses in higher SDCs require design by a registered design professional. In high SDCs, §R301.2.2.2 limits the thickness and weight of concrete walls, and §R301.2.2.6 limits irregular building configurations; both of these are moot, however, since engineered design is already required by §R608.2.

The discussion that follows is applicable to the dwellings and townhouses that fall within the scope of the IRC concrete provisions:

♣ Per §R608.1, the IRC provisions address buildings with ICF exterior walls and light-frame (wood or CFS) interior walls,

♣ For all SDCs, concrete and ICF wall buildings are limited by §R608.2 to two stories above grade. The maximum permitted house plan dimension is 60 feet, and framing clear spans are limited to 32 feet for floors and 40 feet for roofs (§R608.2). In addition, the mean roof height is limited to 35 feet. Although not explicitly stated in the IRC, the maximum plan dimensions are consistent with one- and two-family detached dwellings and do not accommodate townhouses which are required to have three or more attached townhouse units. The use of one- and two-family dwellings is also consistent with PCA 100.

♣ Per §R608.2, the IRC provisions are permitted up to a wind speed of 160 mph in Exposure B, 136 mph in Exposure C, and 125 mph in Exposure D.

♣ Per §R608.2, the unit weight of floor plus ceiling is limited to 10 psf and roof plus ceiling to 15 psf.

♣ Per §R608.10, when designing using the IRC provisions, gable end walls are limited to light frame construction. This is consistent with the details provided in §R608.9. This is a significant variation from masonry and masonry veneer walls that both allow for additional wall height at gable ends.

♣ It is also worth noting that §R301.2.2.6, Item 7, prohibits the mixing of light-frame and concrete construction such that light-frame walls would be required to support earthquake loads due to concrete wall construction (e.g., mixing concrete and light-frame walls on the same story.) As a result, in higher SDCs, all bracing walls must be of concrete.

6.5.2 Wall Reinforcing and Anchorage

Reinforcing steel is of primary importance for concrete and masonry building performance. The reinforcing steel is what ties the components together and provides toughness to the concrete when it cracks. General detailing and installation requirements for reinforcing of ICF walls are provided in §R608.5.4. These details are important for the adequate performance of the walls. With the IRC ICF provisions being limited to lower SDCs, the provisions for minimum wall reinforcing are controlled by out-of-plane wind loading (§R608.6.2) and in-plane shear wall (bracing wall) design (§R608.7).

Also of primary importance for dwelling or townhouse performance is the connection of the ICF walls to the floors and roofs, both for bracing loads parallel to the wall and out-of-plane loads perpendicular to the wall. Discussion and an extensive set of details can be found in §R608.9. The IRC addresses three types of connections to wood light-frame floors and roofs:

♣ Top bearing connections where the light-frame floor bears on the top of the ICF wall with lightframe walls above (§R608.9.2),

♣ Ledger-bearing connections where the floor or roof is supported off the face of the ICF wall by a wood ledger (§R608.9.1), and

♣ Top-bearing connections where the light-frame roof bears on top of the ICF wall (§R608.9.3). The IRC details provide a direct tension tie from the wall to the floor or roof framing or blocking to resist loads perpendicular to the wall (this connection keeps the wall from pulling away from the floor or roof framing).

6.5.3 Additional Earthquake Provisions

Like light-frame wall bracing provisions, ICF wall provisions require a minimum length or percentage of the wall to be solid (without openings) so that the bracing wall strength is in proportion to the earthquake load. The minimum required lengths of bracing wall for ICF construction in each SDC are defined in §R608.7 and Tables R608.7.1.1(1) through R608.7.1.1(3). Minimum bracing lengths are tabulated for wind load parallel and perpendicular to the ridge. These tabulated values are permitted to be adjusted for less than maximum mean roof height and floor to ceiling height, and further adjustments based on strength are permitted. The required bracing length must be provided using wall segments not less than 2 feet in width.

The floor and roof systems act as horizontal beams carrying wind and earthquake loads to the bracing walls. For the ICF provisions, these systems are of either wood or steel light-frame, construction. The increased weight of the ICF walls results in increased earthquake load in the floor and roof systems. With the IRC ICF provisions being limited to low SDCs and lower wind speeds, IRC minimum fastening requirements are adequate. The wall connection provisions of §R608.9, already discussed in Section 7.5.2 of the guide, include the requirements for the floor and roof systems. §R608.9.2 addressing connections between walls and floor systems requires that aspects not addressed by the details be in accordance with §R502 or the *Wood Frame Construction Manual* for *One- and Two-Family Dwellings* (WFCM) (AWC, 2024) for wood systems and §R505 or AISI S230 for CFS systems. Similarly for connections to roofs, §R802 or the WFCM are referenced for wood and §R804 or AISI S230 for steel. Use of wood structural panel sheathing is required, and the IRC minimum fastening schedules of 6 inches around the perimeter of each sheet and 12 inches along intermediate supports are referenced.

6.5.4 Above-Code Recommendations

The current IRC requirements for ICF wall construction reveal some gaps between prescriptive and engineered construction creating an opportunity for above-code measures to improve performance in areas of high earthquake hazard.

There are no IRC maximum spacing limitations between lines of ICF bracing walls corresponding to the light-frame limits of 25 and 35 feet. In light-frame construction, these spacing limitations are intended to help distribute the bracing walls in proportion to the dwelling or townhouse mass (dead load) as well as to limit the loading to the floor and roof. This helps to reduce concerns regarding dwelling or townhouse rotation due to torsional irregularities and concerns regarding irregular floor and roof system shapes. For ICF wall dwellings or townhouses, the scope limitation to rectangular dwellings or townhouses reduces the likelihood of rotational behavior.

Recommendation: Balance of Bracing Walls at Perimeter

Careful balancing of bracing walls around the dwelling or townhouse perimeter is recommended to further limit torsional behavior. The maximum 60-foot dwelling or townhouse dimension will provide some limit for earthquake loads in the floor and roof. The cost of distributing the wall segments around the perimeter of the dwelling or townhouse should not result in any increased cost for construction.

Recommendation: Design Lower Hazard Using High Hazard Provisions

Any of the many required or recommended measures for areas of high earthquake hazard would improve performance of ICF wall dwelling or townhouses in areas of lower earthquake hazard as well as in high-wind areas. Most of the recommendations would improve dwelling or townhouse performance in high-wind events. Priorities include wall anchorage using details developed to resist out-of-plane wall loads such as those shown in Figures R608.9(2) to R608.9(7).

6.5.5 Quality Control

Quality control for ICF wall construction can play a major role in the satisfactory performance of the dwelling or townhouse. Important steps in ICF construction include:

- Reinforcing should be placed properly. Unless specifically designed otherwise, reinforcing should be located as near the centerline of the ICF cavity as possible but at least within the middle third of the wall.
- Anchor bolts should be secured into place before concrete placement (not placed into concrete following placement of the concrete) to prevent air pockets from forming around the bolt which reduces its strength.
- Concrete should be consolidated as it is placed into forms to prevent voids.

7- Stone and Masonry Veneers

Stone and masonry veneers are popular exterior finish materials for residential construction (Figure 7-1 and Figure 7-2). Veneer provides a durable finish for the house but, the added weight of increases the loads experienced during an earthquake. The IRC, within limits, permits use of stone and masonry veneers installed over concrete or masonry walls and over wood light-frame or cold formed steel construction. This chapter discusses general principles of earthquake-resistant design for houses with veneer, specific IRC requirements important to earthquake performance, and above code measures for improved earthquake performance. Installation of veneers is covered under IRC Chapter 7. Wall bracing requirements for dwellings or townhouses with stone or masonry veneer are found in IRC Chapter 6.

Figure 7-1 Dwelling with areas of stone veneer

Figure 7-2 Dwelling with masonry veneer.

Stone and masonry veneer are addressed in several IRC sections, including §R301.2.2 limitations on weight, §R602.10.6 wood-frame bracing methods, and the wall covering provisions of §R702.1, §R703.3, §R703.8, and §R703.12. Additional provisions for cold-formed steel framing can be found in §R603.9.5. More detailed discussion of these provisions follows. For earthquake-resistant construction using veneer, there are two major areas of concern:

♣ The increased earthquake loading on the house due to the weight of the veneer, and
♣ Adequate anchorage of the veneer.

The weight of stone and masonry veneer permitted under the IRC provisions can vary from as little as 20 pounds per square foot installed for adhered veneer to as much as 50 psf installed for a full-brick anchored veneer with 1 inch of grout. The weight of stone or masonry veneer greatly increases the overall weight of a light-frame house and, as a result, the earthquake loads. The IRC provisions rely exclusively on the strength and stiffness of the light-frame bracing systems to resist wind and earthquake loads, discounting any strength and stiffness that might be provided by the veneer. This is primarily because there is only limited understanding of the ability of the veneer to resist cyclic earthquake loading while acting in combination with the light-frame bracing systems, but also because veneer cracks and breaks at smaller displacements than required for the light-frame system to achieve its capacity. As a result, additional requirements apply to light-frame systems with veneer in areas with higher earthquake hazard.

Masonry and stone veneers have been damaged in many past earthquakes. Figure 7-3 illustrates damage that occurred during the 2014 South Napa Earthquake.

Figure 7-3 Veneer damage in the 2014 South Napa Earthquake (from L Whitehurst, EERI Learning From Earthquakes)

7.1 Scoping Limits and Bracing Provisions

The IRC incorporates provisions that limit the scope of when veneer is permitted and augments wall bracing requirements where veneer is used. The following discussion provides an overview of these limits and provisions as applicable to wood-frame construction.

7.1.1 Dwellings and Townhouses in SDC A, B, and C

Stone or masonry veneer up to five inches thick and 50 psf is permitted to be used on dwellings and townhouses in SDC A, B, and C up to a height of 30 feet above a noncombustible foundation, with another eight feet permitted at gable-end walls. Details regarding weight and height can be found in §R301.2.2.2 (Exception 2), §R301.2.2.4, and §R703.8, and Table R703.8(1). Additional provisions can be found in §R702.1 and §R703. Wall bracing is permitted to be in accordance with §R602.10 without the additional requirements of §R602.10.6.5. For townhouses in SDC C, when veneer is added to a second or third story and the wall bracing minimum length is increased using Table R602.10.3(4).

Recommendation: Use of SDC D Requirements in SDC C

It is recommended that the additional requirements applicable to one- and two-story detached dwellings in SDC D_0 also apply to townhouses in SDC C. This would include both the additional limitations on veneer thickness, weight and height, and the additional wall bracing requirements. This is recommended to be consistent with application of other IRC seismic provisions to townhouses in SDC C. Reducing weight and increasing bracing will improve the seismic performance of the townhouse and increase the likelihood that the townhouse is occupiable following an earthquake.

7.1.2 Townhouses in SDC D0, D1, and D2

Townhouses in SDC D_0, D_1, and D_2 are permitted per Table R703.8(2) to have stone or masonry veneer installed on the first story only, providing it meets the requirements of §R703.8 and Table R703.8(2), including weight and height limits. Additional provisions can be found in §R702.1 and §R703. Wall bracing is permitted to be in accordance with §R602.10 without the additional requirements of §R602.10.6.5.2 or §R602.10.6.5.3.

For townhouses in SDC D_0, D_1, and D_2 with veneer exceeding the first story in height, §R602.10.6.5 requires a design in accordance with accepted engineering practice.

7.1.3 Dwellings in SDC D0, D1, and D2

One- and two-family detached dwellings are permitted to have stone or masonry veneer as follows:

♣ In SDC D_0: Up to 4 inches thick and 40 psf is permitted to be used to the lesser of three stories and 30 feet above a noncombustible foundation, with another 8 feet permitted at gable-end walls.

♣ In SDC D_1: Up to 4 inches thick and 40 psf is permitted to be used to the lesser of three stories and 20 feet above a noncombustible foundation, with another 8 feet permitted at gable-end walls.

♣ In SDC D_2: Up to 3 inches thick and 30 psf is permitted to be used to the lesser of two stories and 20 feet above a noncombustible foundation, with another 8 feet permitted at gable-end walls.

Greater heights are permitted where the first story wall is masonry or concrete. Where veneer is in excess of these limitations, a design is required in accordance with accepted engineering practice. Additional provisions can be found in §R702.1 and §R703. Wall bracing is required to be in accordance with §R602.10 using Table R602.10.3(4) and the additional requirements of §R602.10.6.5. There are two options for the wall bracing, depending on extent of second story veneer as follows:

♣ Limited Veneer: §R602.10.6.5.3 provides intermediate bracing requirements for two-story dwellings where veneer is provided over a limited portion of the second story. This includes the veneer only extending to the second story over one wall (intended to be one entire face of the dwelling), or veneer on more than one second floor wall, but not exceeding 25% of the second floor wall area. For both of these conditions the wall bracing is limited to Type WSP or CS-WSP and a factor of 1.2 is used to increase the required length of wall bracing.

♣ More Than Limited Veneer: §R602.10.6.5.2 provides bracing requirements for any SDC D_0, D_1, or D_2 dwelling in which the veneer exceeds what is allowed in the limited veneer provisions. In this case wall bracing at both exterior and interior braced wall lines is limited to Type BV-WSP, which includes hold-downs at all levels. In addition, cripple walls are prohibited and interior braced wall lines are required to be supported on continuous foundations. The overall effect of these requirements is to provide a bracing system that is very similar to an engineered bracing system

7.2 Installation Provisions

Support requirements for anchored veneer are given in §R703.3 through §R703.8.3. Requirements for anchorage of stone or masonry veneer to the wall framing behind are given in §R703.8.4. Adequate anchorage of veneer is important for seismic performance. Anchorage using galvanized corrugated sheet metal ties or metal strand wire ties is required. The minimum gage required is No. 9 U.S. gage wire for wire ties and No. 22 U.S. gage by 7/8-inch corrugated sheet metal ties. The maximum area of veneer to be supported by each anchor is 2 2/3 square feet per tie for SDCs A, B, and C (i.e., 16 inches vertical by 24 inches horizontal spacing). The maximum area of veneer to be supported by each anchor is reduced to 2 square feet per tie for houses in SDC D_0, D_1, and D_2 (i.e., 12 inches vertical by 24 inches horizontal spacing).

The intent of using ties for masonry veneer is not to prevent the veneer from cracking but rather to prevent the veneer from pulling away from the supporting wall system. In earthquakes, the desire is to reduce the falling hazard posed by falling veneer. Use of anchors should reduce the falling hazard by holding the cracked veneer to the supporting wall.

Recommendation: Increase of WSP Length and Number of Hold-downs

The sheet metal ties or wires used to fasten veneers should be corrosion-resistant; the more corrosion-resistant, the better. Corrosion resistance fasteners should penetrate the weather-resistant barrier and sheathing and should be embedded into the wall studs. Fasteners that do not penetrate the studs (i.e., only the sheathing is penetrated) have low withdrawal resistance and significantly reduce the ability of the tie to hold the veneer to the wall.

Where veneer is limited to the first story above grade, it is recommended as an above-code measure to increase the length of wood structural panel bracing and use of hold-down devices on the braced wall panels in the first story to increase both the strength and stiffness of the first story above grade. These measures will help make the deformation behavior of the light-frame system more compatible with the veneer and reduce cracking of the veneer.

7.3 Quality Control

Quality control for stone and masonry veneers involves the following:

- All mortar joints should be completely filled and well tooled for water tightness. This can have a significant effect on the strength and durability of the masonry veneers. To augment the tooling of joints for improving moisture control (which directly affects the longer-term strength of the wall), flashing and weep holes need to be placed at the bottom of the veneer so that any water trapped between the veneer and the structural wall behind can be directed to the outside of the house. The flashing and weep holes frequently are omitted in residential construction, resulting in expensive repairs to the structural walls of the house.

- Veneer should be placed such that a 1-inch cavity is maintained between the veneer and the supporting wall. This cavity provides a drainage plane for channeling the moisture that will migrate from the outside surface of the veneer to the inside and then to the bottom of the wall where it can escape to the outside through the weep holes. This drainage plane, in combination with a weather-resistive barrier, is very important to prevent the wall sheathing from being constantly wet, which will result in mold, mildew, and decay.

- Anchor ties that hold the veneer to the wall need to be placed at the proper spacing to ensure that the area of veneer attributed to each tie does not exceed the maximum allowed for the particular SDC. For SDCs D_0, D_1, and D_2, the spacing should be such that each tie is supporting no more than two square feet of veneer. The ties also need to be placed such that the nail holding the tie to the wall is embedded into a stud and not just the sheathing material.

8- Roof-Ceiling Systems

Wood-frame roof-ceiling systems are the focus of this section. These systems are covered by IRC Chapter 8.

8.1 The Role of Roof-Ceiling Systems in Earthquakes

Wood-frame roof-ceiling systems, regardless of the pitch of the roof, form a roof diaphragm (essentially a beam loaded horizontally on its side as shown in Figure 2-9) that transfers earthquake lateral loads to braced walls in the story level immediately below the roof in the same manner that floors transfer loads from interior portions of the floor to the braced wall lines of the story below. The lateral loads in the roof-ceiling are based on the mass of the roof-ceiling assembly and a portion of the mass of the walls in the story immediately below the roof. See Section 2 of this course for discussion of the load path.

8.2 Conditions Not Addressed in This Section

Cold-formed steel (CFS) framing is permitted by the IRC for a roof-ceiling system, but this CFS framing is not discussed in this guide. Rather, the reader is referred to AISI S230 for guidance. Most of the recommendations for improving the earthquake performance of wood-frame roof-ceiling systems also apply to CFS construction since the systems are similar.

8.3 Wood-Framed Roof-Ceiling Systems

Wood-frame roof-ceiling systems typically consist of repetitive rafters and ceiling joists or prefabricated (engineered) trusses at a prescribed spacing. They are sheathed with either solid wood boards or with wood structural panels attached to the top surface of the rafter or truss. Figure 8-1 illustrates this type of roof and ceiling framing system. Roof members also can consist of repetitive beams spaced further apart than rafters, either with or without ceiling joists.

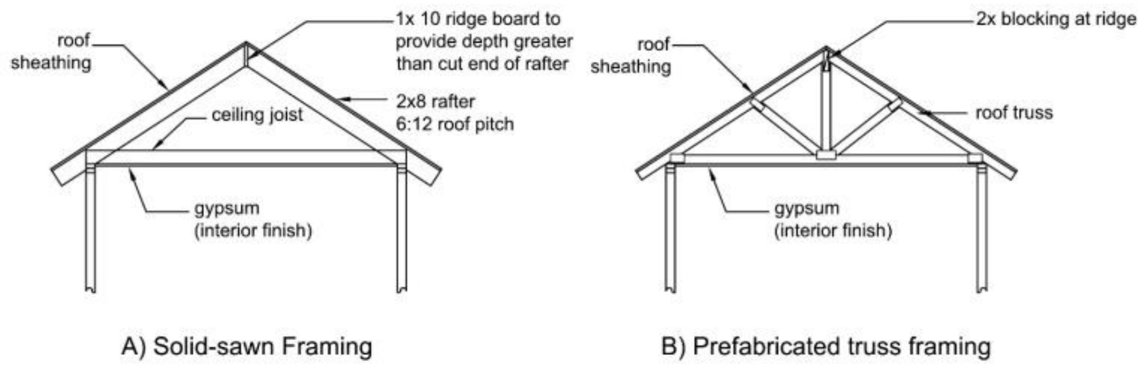


Figure 8-1 Typical light-frame roof-ceiling system.

Depending on the roof shape chosen for a dwelling or townhouse, hip and valley rafter members may be needed where intersecting rafters change the direction of their span. Depending on the slope of the roof, hip and valley rafters can experience very high loads when supporting long-span rafters. Therefore, purlins are sometimes provided below and perpendicular to rafters. The purlins, in turn, are supported by a stud or post attached to a wall or beam below. Ridge boards also are often provided at the peak of a roof where sloping rafters meet. Ceiling joists between top plates, when fastened to rafters in accordance with the minimum fastening table, form a stable roof truss system. At open or cathedral ceilings, where ceiling joists are not provided at or near the top plate level, ridge boards need to be replaced with ridge beams, capable of spanning between points of beam support on bearing walls or posts.

Blocking between rafters (or trusses) is used at the bearing points of rafters, trusses, and ceiling joists to prevent lateral movement or rolling of the framing. Finish materials such as gypsum board are typically applied to the bottom surface of ceiling joists or the bottom chord of a truss where the ceiling height corresponds to the bottom of the truss.

Rafters, purlins, ridge boards, and hip or valley members can be sawn lumber, end-jointed lumber, or any one of a variety of prefabricated (engineered) members. Examples of engineered lumber include wood I-joists or solid rectangular structural composite members such as parallel strand lumber (PSL), laminated veneer lumber (LVL), or laminated strand lumber (LSL). Roof beams and blocking can be either sawn lumber or engineered lumber.

The minimum required size and maximum span and spacing of roof rafters or beams, ceiling joists, and trusses is based on providing adequate support for dead and live vertical loads prescribed by the code. Snow loads must be considered for rafters, and attic storage must be taken into account for ceiling joists. Roof trusses must be designed for dead, live, snow and attic loads, combined in accordance with IBC requirements. Vertical deflection of rafters and ceiling joists is another design consideration that may limit the maximum span of these members. Rafter spans listed in prescriptive tables are based on the horizontal projection of the rafter rather than being measured along the slope, which would be a greater distance.

Tables in IRC Chapter 8 and similar tables in other documents such as those published by the AWC or engineered lumber manufacturers are available for use in selecting the proper combination of size, span, and spacing of most roof-ceiling framing members. Depending on the roof pitch, certain roof members require engineering to determine their size. For a roof pitch less than 3:12, the size of ridge boards and hip or valley members must be individually determined based on their spans and the span of the rafters they support.

8.4 Special Framing Considerations

Roof rafters must either be tied together at the ridge by a gusset plate or be framed to a ridge board. Ridge boards in roofs having a pitch of 3:12 or greater must be at least 1× nominal thickness and at least the same depth as the cut end of the intersecting rafters (Figure 8-2). Valley and hip members in a roof having a pitch of 3:12 or greater must be at least 2× nominal thickness and at least the same depth as the cut end of the rafters. Because the cut end depth of a rafter increases with increasing roof pitch, a 2×8 rafter will need a 1×10 nominal ridge board and a 2×10 nominal hip or valley member for a pitch up to 9:12. At a pitch exceeding 9:12, a 1×12 or 2×12 will be needed because the cut end of a 2×8 rafter will be greater than the 9 1/4 inch actual depth of a 1×10 or 2×10 nominal member. Figure 8-2 shows a 1×12 ridge board for a 12:12 pitch condition with the dimension for the cut end of a 2×8 rafter.

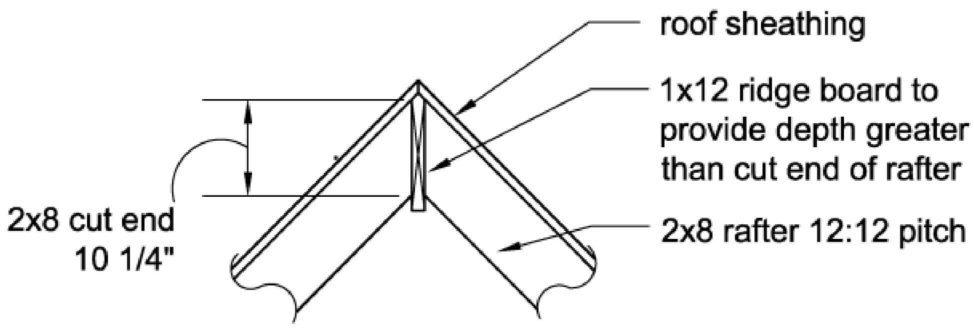


Figure 8-2 Ridge board for 12:12 pitch roof

One additional consideration when using nominal 1× ridge boards is that ceiling joists or rafter ties are needed at the top plate of the supporting walls to prevent the walls from spreading outward. The ceiling joists or rafter ties act as a brace to resist the outward thrust of the rafters at the wall support ends of rafters. In the absence of ceiling joists or ties at the wall top plate, rafters must be supported at the ridge by a beam designed for the support of rafter loads. In addition, where a nominal 1× ridge is used and a 2× valley or hip member intersects the ridge, the valley or hip member must be supported by a stud or post attached to a bearing wall below to transfer the high loads associated with hip and valley rafters.

Recommendation: Bracing of Gable Ends

A special condition can occur at the gable ends of a roof. When the exterior wall at the gable end has its double top plate level with the low ends of the roof framing, the wall studs are not continuous to the roof sheathing along the gable edge. A section view of this condition is shown in Figure 8-3. To provide above-code performance, the framing extending above the top plate to the level of the sloping gable end roof sheathing should be braced at regular intervals of not more than 4 feet on center at both the wall top plate level and along the top edge of the sloping roof edge. This bracing permits the top of the exterior wall and the framing extending above to resist the lateral loads that are acting perpendicular to the wall. Without this bracing, the framing above the double top plate could be easily displaced because a hinge can form where the gable end wall framing attaches to the top plate.

Recommendation: Continuous Wall Studs

Another above-code alternative to the framing shown in Figure 8-3 would be to provide wall studs that are continuous to a sloping double top plate located just below the roof sheathing (balloon framing). When continuous studs are provided, there is no weak location for a hinge to form. However, when this method is used, the required stud size, spacing, and maximum height must comply with §R602.3.1 or the wall studs will need to be engineered by a design professional. Bracing a gable end wall also is important for providing resistance to high winds, especially for roofs with a steep pitch.

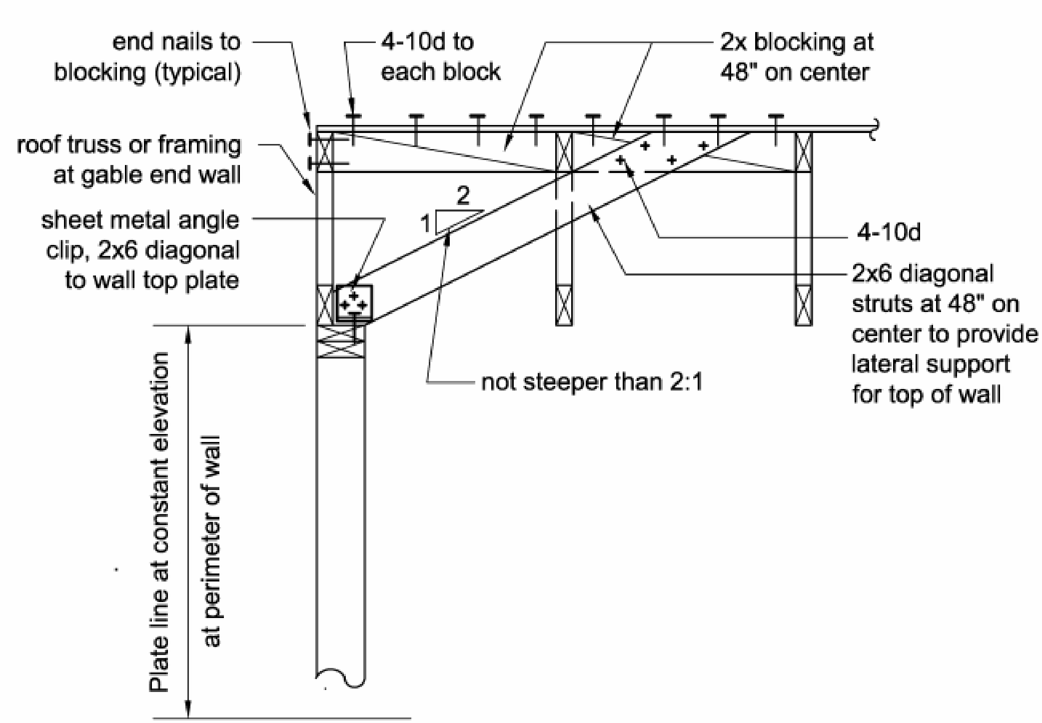


Figure 8-3 Gable end wall or gable truss bracing

8.5 Blocking and Lateral Load Paths for Roof Systems

At the intersection between the roof-ceiling framing and the top of exterior walls (Figure 8-4), there are several different needs that much be considered:

♣ Rotational restraint of the roof/ceiling framing members at their support (§R802.8 for solid-sawn framing and §R802.10.3 for trusses),

♣ Load path connections between the roof and the bracing walls (§R602.10.8.2), and

♣ Roof ventilation (§R806).

It is necessary to accommodate all three requirements at the same time, using the most restrictive of the applicable provisions.

Rotational restraint is required by §R802.8 for solid sawn framing when the depth-to-thickness ratio (height-to-width) of the framing members is greater than 5:1. If ceiling joists are attached to the face of the member, the combined width is permitted for this ratio. This will mean that unless the framing is deeper than 2×10s (or deeper members with ceiling attached), rotational restraint will not be required. Rotational restraint for trusses may be required by the truss manufacturer.

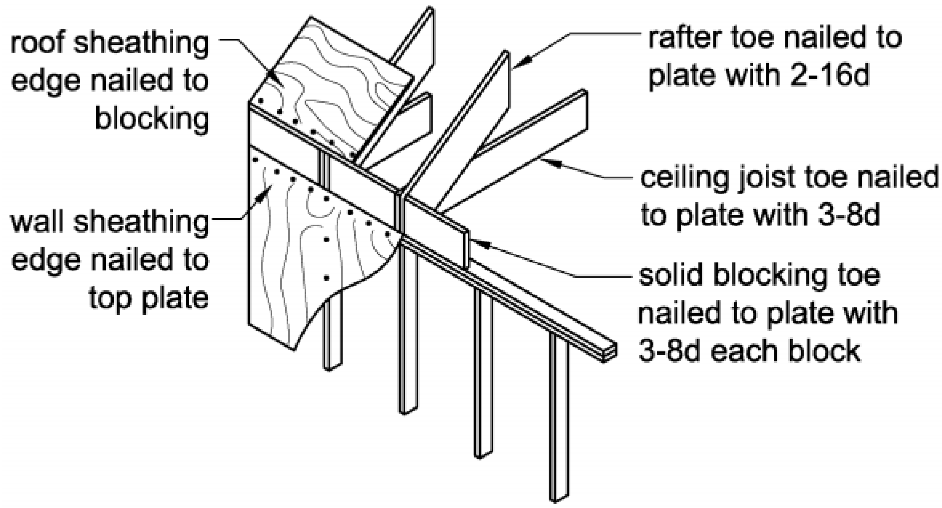


Figure 8-4 Blocking at rafters to exterior wall

Load path connections between braced wall panels and the roof are specified in §R602.10.8.2. For SDC A, B, and C, where distance from the top of the braced wall panel to the top of the rafter/truss is 9 1/4 inch or less, no blocking is required; where greater than 9 1/4 inch, Figure R602.10.8.2(1) is specified at the braced wall panels only. This detail includes solid sawn blocking but allows a 2-inchtall vent space over the blocking to accommodate required roof venting.
For SDC D_0, D_1, and D_2, §R602.10.8.2 always requires blocking over braced wall panels. Where the height from top of braced wall panel to top of rafter/truss is 15 1/4 inch or less, Figure R602.10.8.2(1) is again specified. When the height is taller, load path connections are provided by one of the following:

♣ Figure R602.10.8.2(2) with wood structural panel soffit sheathing,

♣ Figure R602.10.8.2(3) with wood structural panel infill bracing walls,

♣ Blocking as specified by the roof truss manufacturer, or

♣ Blocking in accordance with the AWC WFCM or accepted engineering practice.

The above blocking details for the load path connections should also be adequate to provide required rotational restraint; however, where rotational restraint is required, the detail will need to be used at all roof framing supports instead of just at braced wall panels.

The roof ventilation requirements of §R806 can be met in a number of ways including:

♣ The two-inch air gap shown in Detail R602.10.8.2(1),

♣ Drilling holes in solid blocking in combination with a baffle that pushes the insulation away from the vent holes,

♣ Rectangular notches cut in the top of the blocking, similar to vent holes,
♣ Installing every other block (where braced wall panels configurations permit,

♣ Vents in the plane of the roof, or

♣ Periodic ventilation openings in the ceiling soffit in Figure R602.10.8.2(2) or in the wall sheathing in Figure R602.10.8.2(3).

Recommendation: Blocking

In SDCs D_0, D_1, and D_2, blocking is always recommended between rafters/trusses along exterior wall lines to provide a surface for the edge nailing of roof sheathing and to provide a very direct load path to the top plate of the exterior braced walls. When this blocking is provided, it also is necessary to ensure that minimum attic ventilation opening requirements are met.

8.6 Connection of Ceiling Joists and Rafters to Walls Below

Ceiling joists and rafters (or trusses) are required to be connected to the top plate of supporting walls as specified in Table R602.3(1). These connections also provide a portion of the load path that transfers loads from the roof diaphragm into the braced walls below. Ceiling joists require a toenailed connection to the top plate using three 8d common nails. The same connection is commonly provided between trusses and top plates. Rafters also require a toe-nailed connection to the top plate using three 10d common nails. Blocking installed between the rafters or ceiling joists requires a toe-nailed connection to the top plate using a minimum of two 8d common nails in each block. Toe nailing must be done correctly if the needed transfer of loads is to occur; therefore, ensure that the nails do not split the wood. In high-wind areas, light-gage steel connectors often are used in place of these toe-nailed connections for rafters. The use of commercially available light gage steel connectors in place of toenails can reduce wood splitting and provide more reliable load transfer. Additional information on proper toe-nailing installation is illustrated in Figure 5-8.

In addition, §R802.11 specifies wind uplift connections between rafters/trusses and the supporting walls. For low wind uplift loads, the minimum fastening of Table R602.3(1) is sufficient. Where higher uplift occurs, the specified uplift loads can be used to select uplift connectors or otherwise design the connection.

8.7 Roof Sheathing

Wood boards installed either perpendicular or at an angle to the rafters (or trusses) or wood structural panels can be used as roof sheathing. Table R803.1 specifies the minimum thickness for lumber (wood board) roof sheathing for various spacings between roof rafters, trusses, or beams.

Recommendation: Additional Strength and Stiffness at Straight Lumber

Solid lumber roof sheathing is rarely used in modern housing construction except along roof eave overhangs, and where the underside of the sheathing is exposed as the ceiling finish. When lumber sheathing is used with the boards installed perpendicular to rafters (i.e., straight lumber sheathing) they provide a very weak diaphragm with little stiffness. It is recommended that where straight lumber sheathing is used in SDCs C, D_0, D_1, and D_2, it be overlain with wood structural panel sheathing to provide improved strength and stiffness. Alternatively, wood boards installed diagonally (diagonal lumber sheathing) could be used.

Wood structural panel roof sheathing is the most common roof sheathing used in the current construction. The minimum thickness is based on rafter (or truss) spacing and the grade of sheathing panels selected. Table R503.2.1.1(1) is used to determine the minimum required thickness for wood structural panel roof sheathing for a variety of rafter spacings. For roofs, the short direction panel joints between wood structural panels can be either staggered or not staggered. Typical wood structural panel roof sheathing installation using staggered joints is illustrated in Figure 8-5.

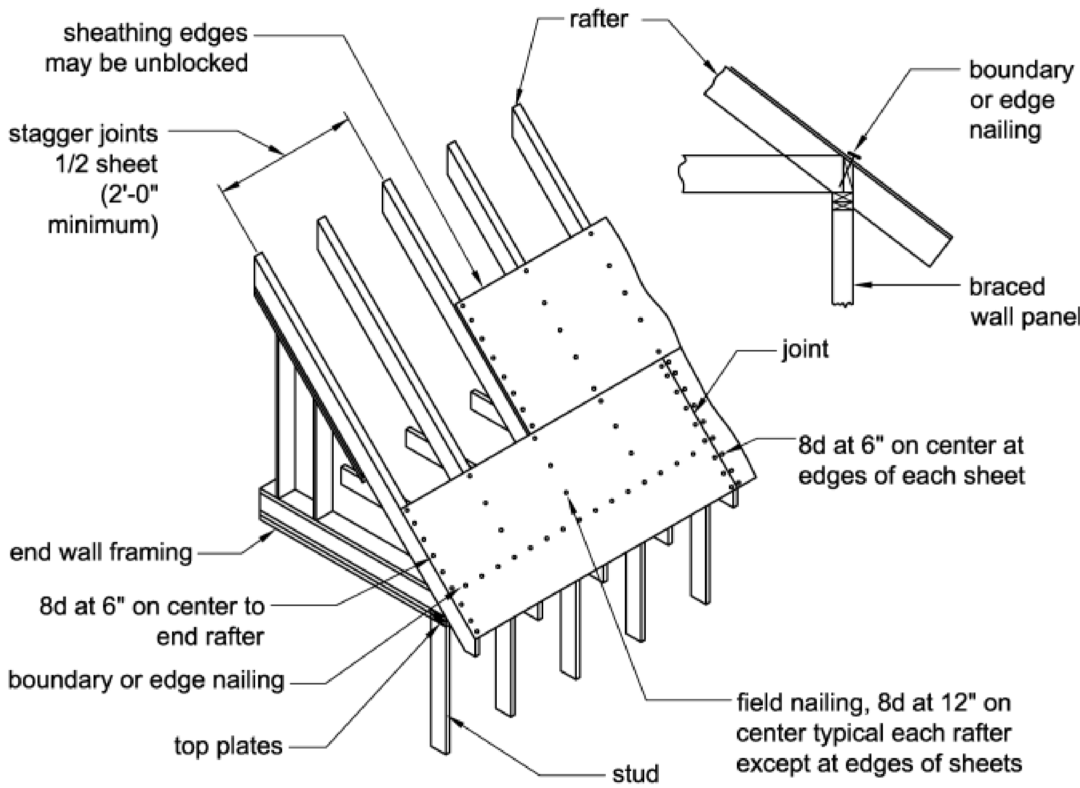


Figure 8-5 Typical roof diaphragm sheathing nailing when wood structural panels are used.

There are two roofing systems where spaced lumber sheathing is permitted in lower SDCs and prohibited in higher SDCs:

♣ Concrete and clay tiles are permitted to be installed on spaced lumber sheathing in SDC A, B, and C. Use with spaced sheathing is prohibited in SDC D_0, D_1, and D_2 per the §R905.3.1 requirement for solid sheathing.

♣ Wood shingle and shake roofs are permitted to be installed on spaced lumber sheathing in SDC A through D_1. Use with spaced sheathing is prohibited in SDC D_2 per §R803.

These restrictions on spaced lumber sheathing are intended to provide a stiffer and stronger roof diaphragm that will resist the larger lateral loads anticipated in higher SDCs. When wood boards are used, they typically are nailed to each rafter with only two 8d box or common nails. Wood structural panel sheathing results in the stiffest roof diaphragms, and the panels typically are fastened to the rafters with 8d common or RSRS-01 nails spaced at 6 inches along supported edges and 12 inches in the field of the panel. When spaced wood boards are used, even fewer nails are provided than with solid wood board sheathing and even less lateral resistance results. This makes the roof less effective in transmitting loads to braced wall panels.

Recommendation: Wood Structural Panel Sheathing

Wood structural panel sheathing is encouraged for roof sheathing in all SDCs, including with concrete and clay tiles roofs and with shingle and shake roofs.

8.8 Lateral Capacity for Wood Framed Roofs

The lateral capacity of a roof diaphragm sheathed with wood structural panels is based upon the same five factors as floor diaphragms. The effects of distance between braced wall lines below the roof and the effects of large roof openings (e.g., skylights) are also similar to those discussed for floors. The recommendations for reinforcing floor diaphragms with large openings also apply when roof openings exceed the code maximum limits. See Section 5.9 for more information on the effects of sheathing thickness, fastener size and spacing, use of blocking, and layout of wood structural panel sheathing. See Section 4.7 for more information about reinforcing diaphragms with large openings.

Although roof-ceiling systems typically are subjected to the smallest loads when a dwelling or townhouse experiences earthquake ground motion, several simple things can be done to improve the performance of the system.

Recommendation: Reinforcement of Framing at Skylights

Reinforce the framing around skylights to provide positive connections to transfer the diaphragm loads around the opening. Strapping and blocking as illustrated in Figure 5-11 can be used to strengthen the roof around large openings and the additional cost for the blocking and strapping for a typical skylight opening would add less than 0.4% to the cost of the structural portion of the project.

Recommendation: Blocking of Rafters and Joists at Top Plate

Make sure all rafters and ceiling joists are blocked at all locations where they are in contact with a top plate of a wall below (alternately a rim joist may be used). For the exterior walls of the model dwelling or townhouse, this would cost 0.8% of the structural portion of the project. If interior walls were also blocked, an additional cost of 0.7% of the structural portion of the project would be incurred.

Recommendation: Gluing of Sheathing and Blocking of Roof Framing

Glue the sheathing to the roof framing in the same way most floor sheathing is installed and block the roof as well. Construction adhesives are typically used to prevent squeaky floors, but they also strengthen and stiffen. The cost of adding adhesive to the roof system for the model dwelling or townhouse in this guide would be 1.8% of the structural system, and adding blocking to the roof system would add another 3.4% to the cost of the structural system.

8.9 Quality Control

Quality control for roof-ceiling systems involves the following:

- The most important item to monitor for quality of the roof diaphragm system is to ensure that the nails used to attach the sheathing to the framing are driven flush with the top of the sheathing and not overdriven and counter sunk into the sheathing materials. Overdriving sheathing nails has been shown to reduce the strength of shear walls and diaphragms. Evidence of a 40% to 60% loss in strength of the shear wall and diaphragm has been observed in laboratory tests of assemblies with overdriven nails. (See Section 6.2.7 of this guide for more discussion on the effects of overdriven nails and how pneumatic tools can be altered to correct for this error.)

- To ensure the quality of the roof-ceiling system, make sure blocking is installed correctly so that the wood framing is not split by the toe nailing.

- If adhesives are used to attach the sheathing, attention needs to be paid to the time between the application of the adhesive and when the nails are driven to hold the sheathing in place. Especially in hot weather, the adhesive tends to skin over or cure on the surface quickly, which reduces the adhesion between the glue and the sheathing. Check the time allowed for each specific product used.

9- Chimneys, Fireplaces, Balconies, and Decks

This section provides an overview of the IRC provisions for earthquake-resistant design and construction of chimneys, fireplaces, balconies, and decks in dwellings and townhouses. These components are covered under IRC Chapter 10, §R507, and §R502.

9.1 Chimneys and Fireplaces

IRC Chapter 10 presents requirements for masonry fireplaces and chimneys and for factory-built fireplaces and chimneys enclosed in framing. The provisions of the IRC are intended for the moderately sized fireplaces and chimneys commonly found in dwellings and townhouses.

Recommendation: Engineering of Unique Fireplaces and Chimneys

Where fireplaces or chimneys are large or oddly configured, an engineered design is encouraged in order to fully address the design of the chimney and fireplace and their influence on the dwelling and townhouse.

9.1.1 Masonry Chimneys and Fireplaces

Although the IRC permits the construction of masonry fireplaces and chimneys in earthquake-prone regions, masonry chimneys are particularly vulnerable to earthquake damage, and such damage has occurred in most moderate to severe U.S. earthquakes (Figure 9-1). Masonry fireplaces and chimneys can be heavy and rigid, and many chimneys in existing dwellings and townhouses also are brittle. The movement of the fireplace and chimney in response to earthquake ground motions can be significantly different from the movement of the light-frame dwelling or townhouse itself, creating the potential for damage to both the chimney and the dwelling or townhouse.

The IRC triggers requirements for masonry fireplace and chimney reinforcing steel and anchorage to floors, roofs, and ceilings of dwellings and townhouses in SDCs D_0, D_1, and D_2. Although these requirements cannot completely eliminate the possibility of damage to the fireplace and chimney in an earthquake, their use permits a chimney to better withstand earthquake loads and should lessen the falling hazard posed by a damaged chimney. Although it may be possible with systematic engineering design to mitigate the damage often seen in masonry chimneys and fireplaces, the lower weight and greater flexibility of factory-built fireplaces and flues and light-frame chimneys make them the better choice for light-frame dwellings and townhouses in earthquake-prone areas.

Figure 9-1 Chimney collapse in the 2011 M5.8 Virginia Earthquake (from USGS).

A substantial footing is necessary if a fireplace and chimney is to perform well under any type of loading. The footing should extend to a depth not less than that of surrounding footings. §R1003.2 contains minimum footing requirements.

§R1001 addresses masonry fireplaces and §R1003 addresses masonry chimneys; these two sections overlap because §R1001 also addresses chimneys with fireplaces. Both §R1001 and §R1003 address reinforcing and anchorage; the provisions in the two sections are identical. Per §R1001.3 and §R1003.3, reinforcing steel is required for chimneys in SDCs D_0, D_1, and D_2. The minimum amount of vertical reinforcing steel is four No. 4 bars for a chimney up to 40 inches wide (a depth of approximately 24 inches is common). An additional two No. 4 vertical bars are required for each additional flue or each additional 40 inches of width. Where the reinforcing bars cannot run full height, a lap splice of not less than 24 inches is needed. Grout, continuous from the footing to the top of the chimney, must surround the reinforcing steel. For horizontal reinforcing, a minimum of ¼-inch ties at not more than 18 inches on center is required in the mortar joints. §R1003.3 also cites the §R608 requirements for grouted masonry (discussed in Chapter 6 of this guide). Proper grouting and consolidation around the reinforcing steel is needed in order for the reinforcing and anchorage to contribute to earthquake resistance. Lack of grout and poorly consolidated grout are common contributors to earthquake damage. It is also important to note that §R608 prohibits the use of Type N masonry mortar in SDCs D_0, D_1, and D_2.

Anchorage of the masonry chimney to the framing at each above grade floor, roof, and ceiling level is required and an important step towards reducing earthquake vulnerability. §R1001.4 and §R1003.4 provide anchorage requirements applicable in SDCs D_0, D_1, and D_2. Steel straps not less than 3/16-inch by 1-inch are required to extend a minimum of 12 inches into the chimney masonry, hook around outer reinforcing bars, and extend not less than 6 inches beyond the hook. Chimney anchorage locations are illustrated in Figure 9-2 and Figure 9-3. The IRC provisions specify anchorage to a minimum of four wood ceiling or roof joists with not less than two ½-inch bolts; equivalent fastening provisions are given for CFS framing. This description does not give details of the intended configuration and does not address framing parallel to the chimney wall, although IRC Figure R1001.1 identifies the anchorage as Item "S" and shows intended anchorage locations. Figure 9-4 and Figure 9-5 illustrate implementation of this anchorage provision with detailing consistent with industry recommendations (MIA, 2004). Anchorage in general conformance with Figure 9-4 and Figure 9-5 should be consistent with the intent of the IRC.

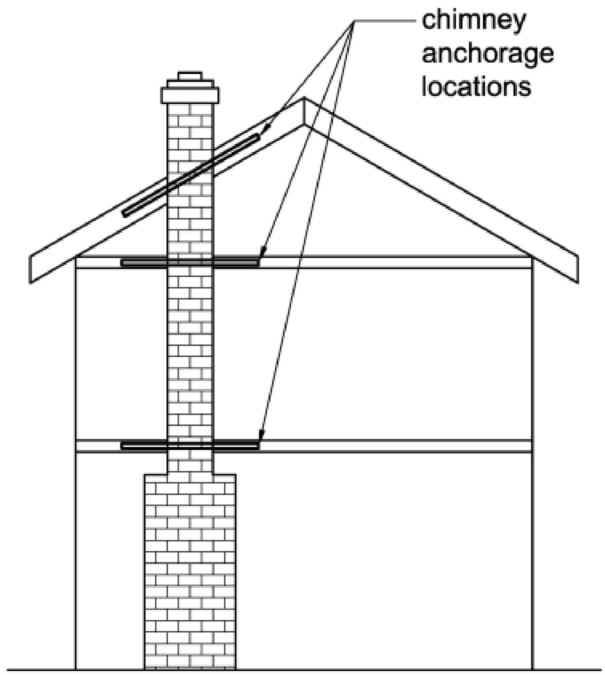


Figure 9-2 Locations for earthquake anchorage of masonry chimney at exterior dwelling and townhouse wall.

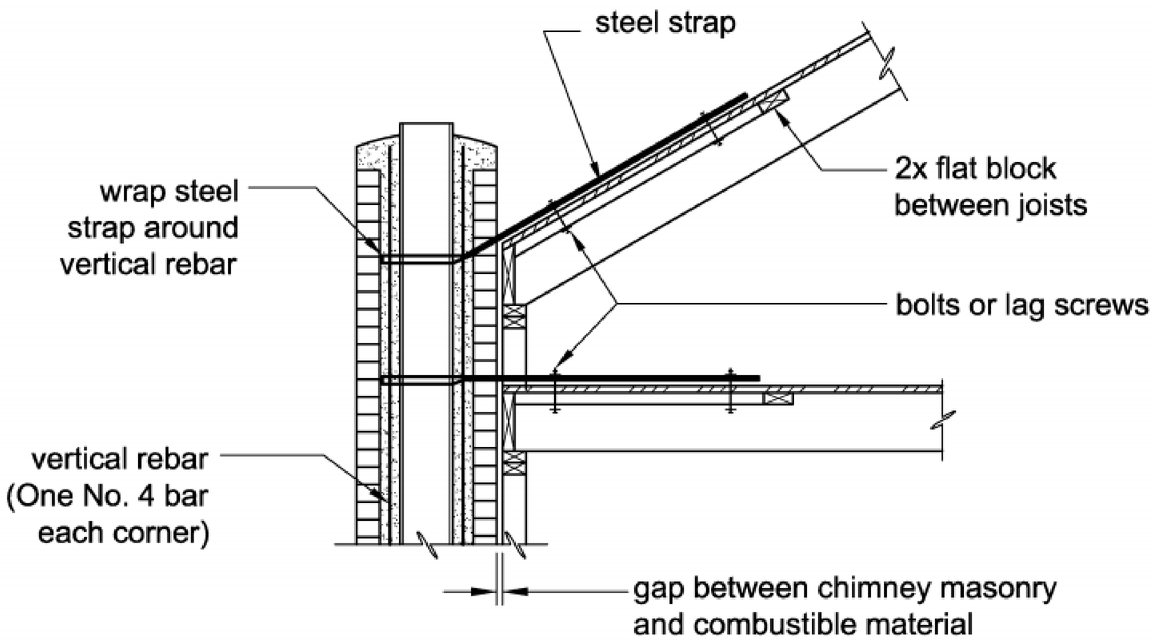


Figure 9-3 Chimney section showing earthquake anchorage

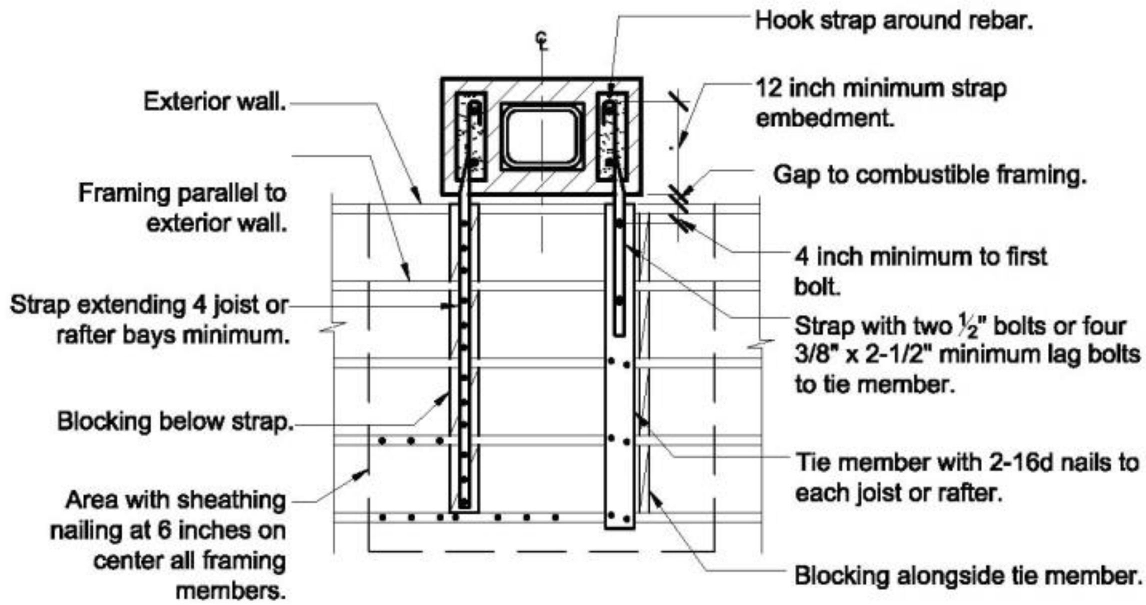


Figure 9-4 Anchorage detail for framing parallel to exterior wall.

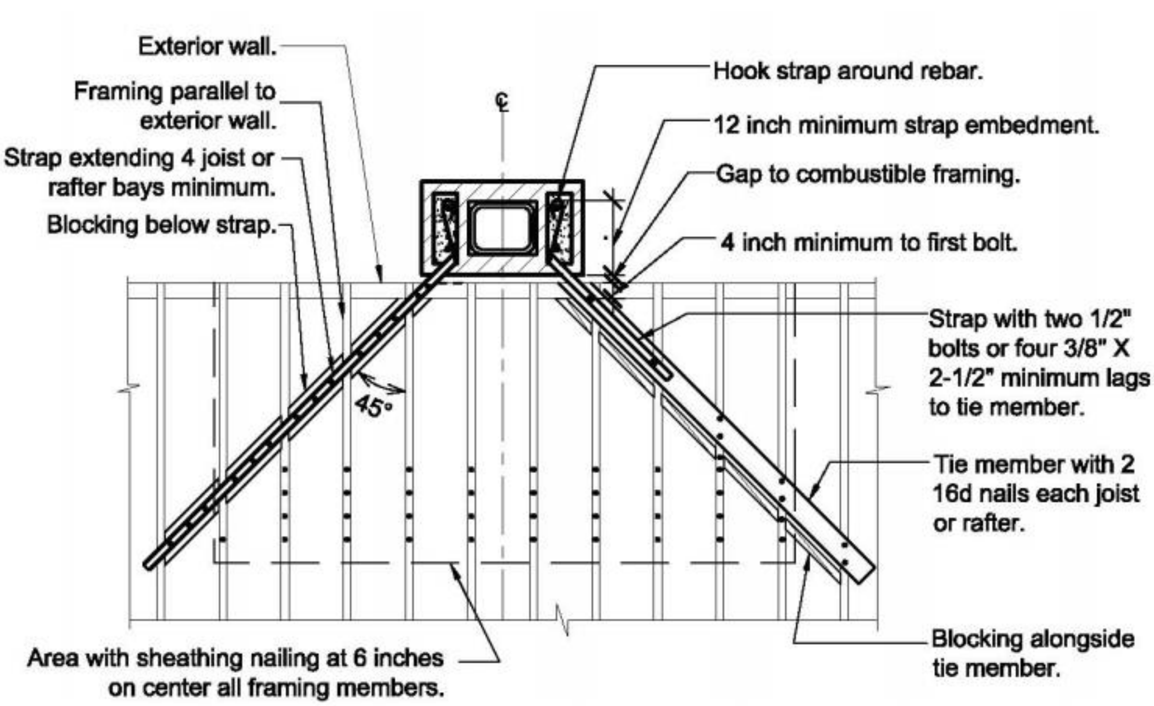


Figure 9-5 Anchorage detail for framing perpendicular to exterior wall.

To reduce the fire hazard with respect to the surrounding wood structure, §R1001.11 requires a clearance of 2 to 4 inches (depending on configuration) between combustible framing materials and the masonry chimney. Detailing of earthquake-resistant anchorage at the floor, ceiling, and roof levels needs to maintain this required clearance with only the steel straps extending across this clearance gap.

Even when fireplaces and chimneys show no signs of damage after an earthquake, the masonry or flue liner may have cracked, and inspection before reuse is recommended.

9.1.2 Factory-Built Fireplaces and Flues

Factory-built fireplaces and flues are generally installed within light-frame fireplace enclosures and chimneys. During an earthquake, the deflections of the light-frame enclosure are more compatible with those of the dwelling or townhouse, reducing the potential for or extent of earthquake damage. Detailing of framing anchorage, however, is important. At exterior walls, as shown in Figure 9-6, the framing for the chimney disrupts the typical wall and roof framing. Wall top plates are often discontinued, and studs are balloon framed to the top of the light-frame chimney. Measures should be taken to restore the continuity of the top plates and to anchor the fireplace/chimney wall framing to the floor and roof. Figure 9-7 shows how one light-frame chimney without sufficient connections behaved. No specific requirements for this construction currently exist in the IRC. Clearances to combustible wood framing remain important with factory-built fireplaces and chimneys and are typically addressed in the installation instructions.

Figure 9-6 Factory-built flue and light-frame enclosure

Figure 9-7 Collapsed factory-built chimney, light-frame enclosure, and deck after the San Simeon Earthquake (from J. Marrow).

Use of stone or masonry veneer increases the weight of the light-frame chimney, thereby increasing earthquake loads proportionately. Particular care should be taken to tie the wood framing into the floor and roof when veneer is used. Veneer attachment to the framing should be in accordance with IRC Chapter 7. See Section 7.2 of this guide for a discussion of veneer attachment.

Recommendation: Factory-Built Fireplaces and Flues

Factory-built fireplaces and flues are typically installed within light-frame fireplace enclosures and chimneys that generally perform well during earthquakes; therefore, their use in all SDCs is recommended, but particularly in higher SDCs. Special attention should be given to the detailing of the framing anchorage and to compliance with clearances to combustible wood framing addressed in installation instructions.

Recommendation: Reinforcing Steel and Chimney Anchorage

Use of reinforcing steel and chimney anchorage are recommended to improve the performance of fireplaces and chimneys across all SDCs and particularly in SDC C. Adding reinforcement and anchorage for the chimney on the model dwelling and townhouse used in this guide would increase the cost of the structural portion of the dwelling and townhouse by approximately 2%, which is approximately 0.5% of the total cost of the dwelling.

Recommendation: Addition of Bracing Walls in Vicinity of Fireplace

Both masonry and factory-built fireplaces and chimneys result in increased weight and earthquake loading as well as discontinuities in dwelling and townhouse configuration. Additional bracing walls in both directions in the vicinity of the fireplace are recommended to resist the additional earthquake load. This will result in a reduction in the amount of window opening available, but the cost of adding the wall will likely be offset by the reduced cost for windows.

9.2 Balconies and Decks

Balconies and decks often are prominent features of modern residential construction and, for many people, add much-desired living space (Figure 9-8). Over several code update cycles, the IRC provisions for exterior decks have advanced from a single provision for anchorage to supporting structures to the current §R507. This section systematically addresses many aspects of deck design and construction, including prescriptive provisions for footings, posts, beams, joists, and decking. Also included are permissible details for anchorage of decks to supporting dwellings and townhouses. The §R507 provisions serve the purpose of improving consistency of exterior deck design and construction. In addition, §R502.3.3 and Table R502.3.3(2) address cantilevered joists for exterior balconies. In terms of earthquake-resistant design, there are two primary aspects of exterior balconies and decks that deserve further discussion: anchorage for earthquake loads and the effect of added floor area beyond braced wall lines.

Terminology

For purposes of this guide, exterior balconies are considered exterior framed floors where the balcony floor framing extends from the enclosed portion of the dwelling or townhouse and cantilevers to form the balcony.

For purposes of this guide, exterior decks are considered exterior framed floors where the deck floor framing is completely exterior and may abut and be supported by but does not extend into the enclosed portion of the dwelling.

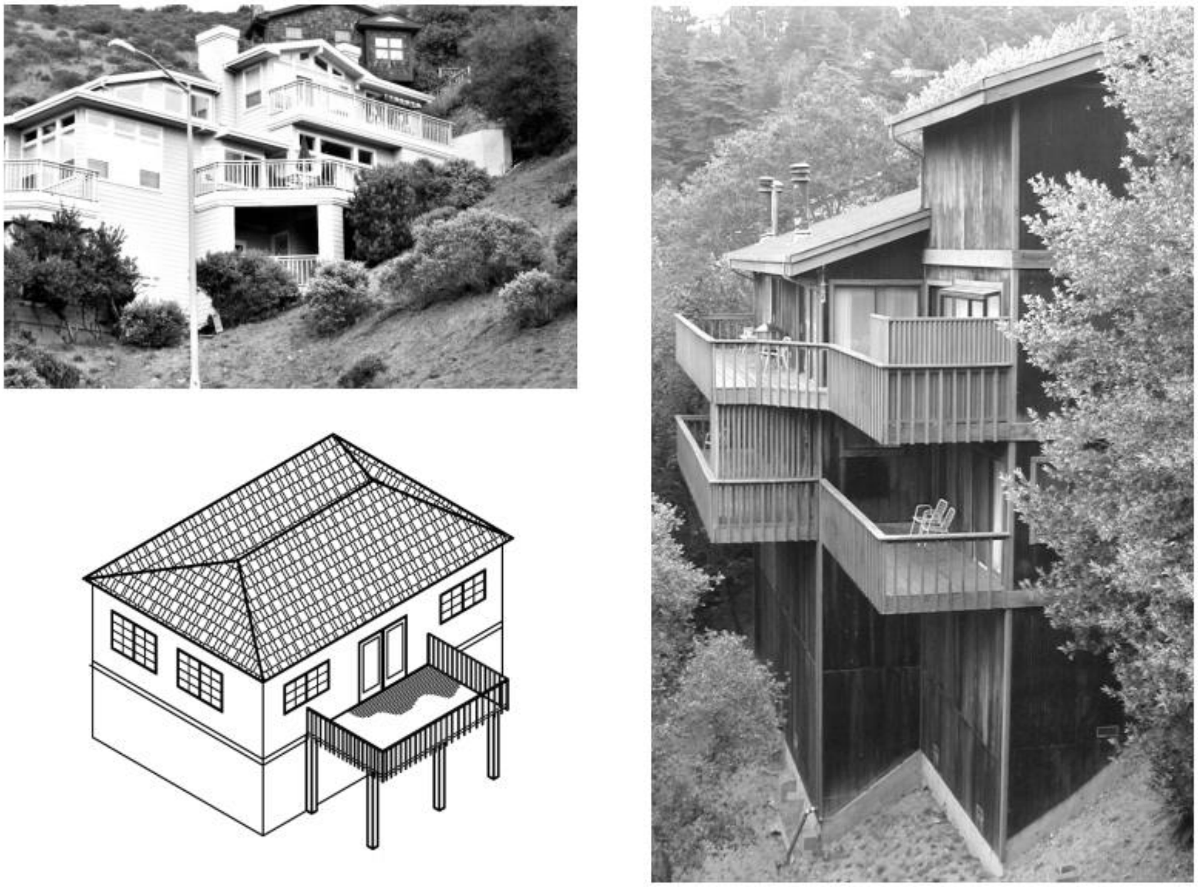


Figure 9-8 Decks and balconies in residential construction

9.2.1 Anchorage for Earthquake Loads

Where a balcony or deck is laterally supported by the dwelling or townhouse, adequate connection to the dwelling or townhouse is key to good earthquake performance. In general, balconies are inherently constructed as an extension of the interior floor system, and in typical circumstances do not require specific consideration of anchorage; should this not be the case, balcony anchorage to the dwelling or townhouse is required. Because deck framing is separate from the interior floor framing, however, deck anchorage for earthquake loads is required. The IRC includes several provisions that address deck anchorage to the dwelling or townhouse:

♣ §R507.8 includes a general requirement that positive anchorage be provided to attach exterior decks to the supporting dwelling or townhouse for resistance to both vertical gravity and horizontal loads. The intent is to protect against separation of the deck from the supporting structure. The use of toenails or nails subject to withdrawal is prohibited.

♣ §R507.9.1 provides specific direction for transfer of vertical loads using illustrated details that include a ledger at the deck perimeter, fastened to a band or rim joists at the edge of the interior floor platform. While it is always possible to instead use an engineered detail, this identifies a prescriptive method adequate to meet the IRC requirements.

♣ §R507.2 similarly provides specific permitted details for the anchorage of the deck to the dwelling or townhouse. Again, it is always permitted to instead use an engineered detail.

§R507.2 and Figure R507.9.2(1) require a hold-down or other positive connector tying the deck joists to the interior floor joists. This detail was introduced into the IRC with the specific intent of providing earthquake anchorage for the deck. It is important that the hold-down not be installed to the rim or band joist or blocking, because these can be fairly readily pulled off of the interior floor system and result in deck collapse. §507.9.2 requires a minimum of two connectors anchoring the deck to the dwelling or townhouse, irrespective of the size or complexity of the deck. As decks become larger and more complex, additional connectors are recommended

Recommendation: Engineering of Balcony-to-Dwelling Connection

Balconies and decks often are subjected to significant horizontal loads during an earthquake. As an above-code measure for dwellings and townhouses located in SDCs C, D_0, D_1, and D_2, the connections used to attach the deck to the dwelling or townhouse should be designed by a registered design professional to ensure that the loads acting on the deck are properly transferred to the framing of the dwelling and townhouse. In addition, the bracing of the deck the sides not attached to the dwelling and townhouse and for all sides of free-standing decks must be sufficient to prevent excessive horizontal drift (deflection) of the balcony or deck.

9.2.2 Added Floor Area Beyond Braced Wall Lines

The bracing provisions of the IRC have been developed considering the weight and the earthquake loads generated by the weight of the interior floor system, as well as the roof and walls. The addition of balconies and decks creates additional weight and increases earthquake loads, a fact that was not envisioned when required wall bracing lengths were determined. Although the addition of a small balcony or deck is not likely to greatly affect the earthquake performance of a dwelling and townhouse, the addition of a large balcony or deck may. Further, balconies and decks tend to concentrate the added earthquake load on one side of the dwelling and townhouse and one braced wall line. This can contribute to rotational behavior and concentrations of damage. See Section 2.4 for a discussion of plan irregularities.

The irregularity provisions of §R301.2.2.6, for SDCs C, D_0, D_1, and D_2, already envision that it is permissible for floors to extend up to 6 feet beyond the exterior braced wall line, without requiring an additional braced wall line at the exterior edge. This provision can be used to accommodate moderately sized balconies and decks. It is noted that there are related engineering design provisions that trigger at the same 6-foot dimension.

Where balconies and decks extend further than 6 feet beyond the exterior braced wall line, they fall into gray area regarding conformance with irregularity and wall bracing provisions. The IRC does not, however, specifically prohibit larger decks from being anchored to and supported by the dwelling or townhouse. It is recommended, however, that additional bracing length be added to the braced wall lines to which larger balconies and decks are anchored

Recommendation: Increase of Bracing Capacity at Deck

Where balconies or decks extending more than 6 feet from the dwelling or townhouse exterior wall are anchored to and braced by the dwelling or townhouse, it is recommended that additional bracing capacity be provided in the braced wall line to which the balcony or deck is anchored. This includes the exterior wall line at the story immediately below the deck and balcony and any additional stories between the balcony or deck and the supporting foundation.

9.2.3 Vertical Support and Moisture and Decay

Vertical load connections and moisture and decay are two issues beyond the scope of earthquake resistance but of enough significance to warrant discussion.

Deck construction is notoriously problematic, and poor details can lead to collapse. Inadequate connection between the deck and the dwelling or townhouse for vertical loads is the biggest problem. Where an engineered design is not provided, connections for vertical loads often are inadequate. The IRC provisions for vertical load anchorage of decks have been modified over the last four code update cycles, with updated provisions found in §R507.9.1 and Figure R507.9.1.3(2). Attention to adequacy of vertical load connections is imperative for safety.

Recommendation: Vertical Support at Balcony or Deck

As an above-code measure, providing a line of vertical support (posts and beams) alongside the exterior dwelling or townhouse wall is recommended as a method to reduce the load on the deck-to-dwelling or townhouse connection.

The interface between a balcony or deck and the exterior dwelling and townhouse wall is critical for the waterproofing system. Penetration of moisture at this interface can endanger not only the capacity of the connection but also the interior and exterior framing members.

Recommendation: Moisture Protection

The joints of exterior decks should be designed such that moisture will not collect in the connection area or that the connection is otherwise protected from moisture.

The references and resources list provided at the end of this guide includes references addressing deck connections and moisture and decay issues. In addition, over the last few code update cycles, several significant changes have been made to the IRC deck provisions. The reader is referred to these updates and the ICC publication *Deck Construction Based on the 2021 International Residential Code* (ICC, 2021).

10- Appliances, Equipment, Furniture, and Contents

In addition to the earthquake performance of dwellings and townhouses themselves, the performance of home appliances, equipment, furniture, and contents plays an important role in reducing the risk of injury, property loss, and interruption of home use from an earthquake. This chapter addresses requirements and recommendations for appliances and equipment, followed by recommendations and resources for bracing of furniture and contents.

10.1 Appliances and Equipment

Dwelling and townhouse appliances and equipment can contribute to earthquake injury, property loss, and interruption of home use. This is particularly true for items that could cause a fire hazard if they shift or fall, such as gas-powered water heaters, furnaces, wood stoves and fuel tanks. The IRC includes specific code provisions to help mitigate these hazards.

In general, appliances and equipment are to be installed in accordance with the manufacturer's installation instructions; installation instructions should include seismic support, bracing, and anchorage where required by code. If not allowed for in the installation instructions, the installation will have to follow manufacturer instructions and the additional code requirements. For townhouses in SDC C and dwellings and townhouses in SDC D_0, D_1, and D_2, §R301.2.2.10 requires seismic support, bracing and anchorage of appliances and equipment designed to be fixed in position, with the following exceptions:

♣ Suspended mechanical ducts, electrical conduit, automatic sprinkler systems, and plumbing systems,

♣ Where the appliance or equipment is bearing on an elevated floor or roof and the housing height is not greater than 1.5 times the width of the housing base in either direction,

♣ Where the installed weight of a suspended appliance or equipment is 50 pounds or less, or

♣ Where the installed weight is 400 pounds or less and the bottom of the appliance or equipment is 4 feet or less above the adjacent floor level.

Per §R301.2.2.10, this seismic support, bracing, and anchorage is required to resist a horizontal force acting in any direction equal to one-third of the operating weight of the item being addressed. Specific bracing requirements are given for items to be supported at their base (based on typical natural gas water heater bracing), and for items that are suspended from the structure above. These provisions are intended to facilitate the implementation of common-sense bracing solutions, with engineered design always permitted as an alternative. This IRC requirement is meant to broadly address mechanical, electrical, and plumbing appliances, as well as water heaters and thermal storage units.

Recommendation: Seismic Bracing in SDC A, B, and C

It is recommended that earthquake support, bracing, and anchorage of appliances and equipment be provided in all SDCs.

10.1.1 Water Heater Anchorage

Because water heaters have long been known to be vulnerable to earthquake damage and because there is a body of guidance developed for bracing of water heaters, this section provides available guidance.

When not properly anchored, natural gas water heaters can fall over and result in fire hazard and water damage. For many years the dangers of unbraced water heaters have been identified and bracing requirements established. As a result, there are a number of resources available to assist with bracing water heaters, some of which are discussed here.

Bracing is required by §R301.2.2.10 for new water heater installations, whether being installed in a new or existing dwelling or townhouse. Bracing is also recommended as a top priority for existing water heater installations. Anchorage may be designed by the water heater manufacturer. Kits for bracing water heaters are also available at many hardware stores. As an alternative to kits, a common method uses strapping designed to resist a horizontal force equal to one-third the operating weight of the tank, placed within the upper one-third and lower two-thirds of the appliance's height. Figure 10-1 and Figurer 10-2 provide example details for anchorage using steel straps and fasteners available from hardware stores. These figures are adapted from *Guidelines for Earthquake Bracing of Residential Water Heaters* (DSA, 2002); consult that publication for detailed installation instructions and additional cautions and limitations.

Figure 10-1 and Figure 10-2 are applicable only to water heaters with a maximum capacity of 52 gallons with two strap locations and with a maximum capacity of 75 gallons with three strap locations. Some jurisdictions place additional limits on water heater bracing details. Where water heaters are installed on a platform, the water heater base should be attached to the platform and the platform should be anchored to the floor. Placing water heaters in metal pans to retain any spilled water is a possible precaution in addition to bracing. The IRC requires that water heaters be placed in pans where loss of water would cause damage. Required clearances to walls and combustible construction need to be maintained; the water heater UL listing and local jurisdiction requirements should be verified prior to moving or installing a water heater.

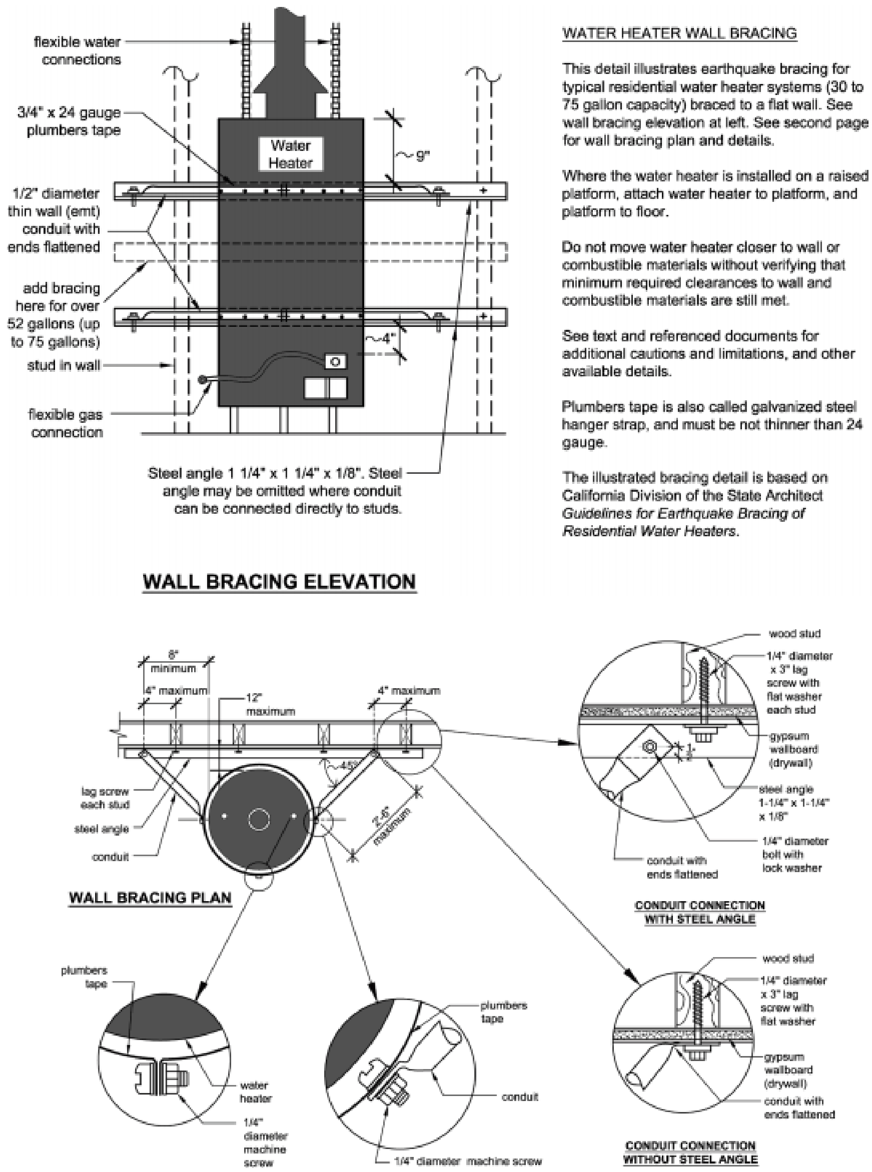


Figure 10-1 Securing a water heater with wall bracing (adapted from DSA, 2002)

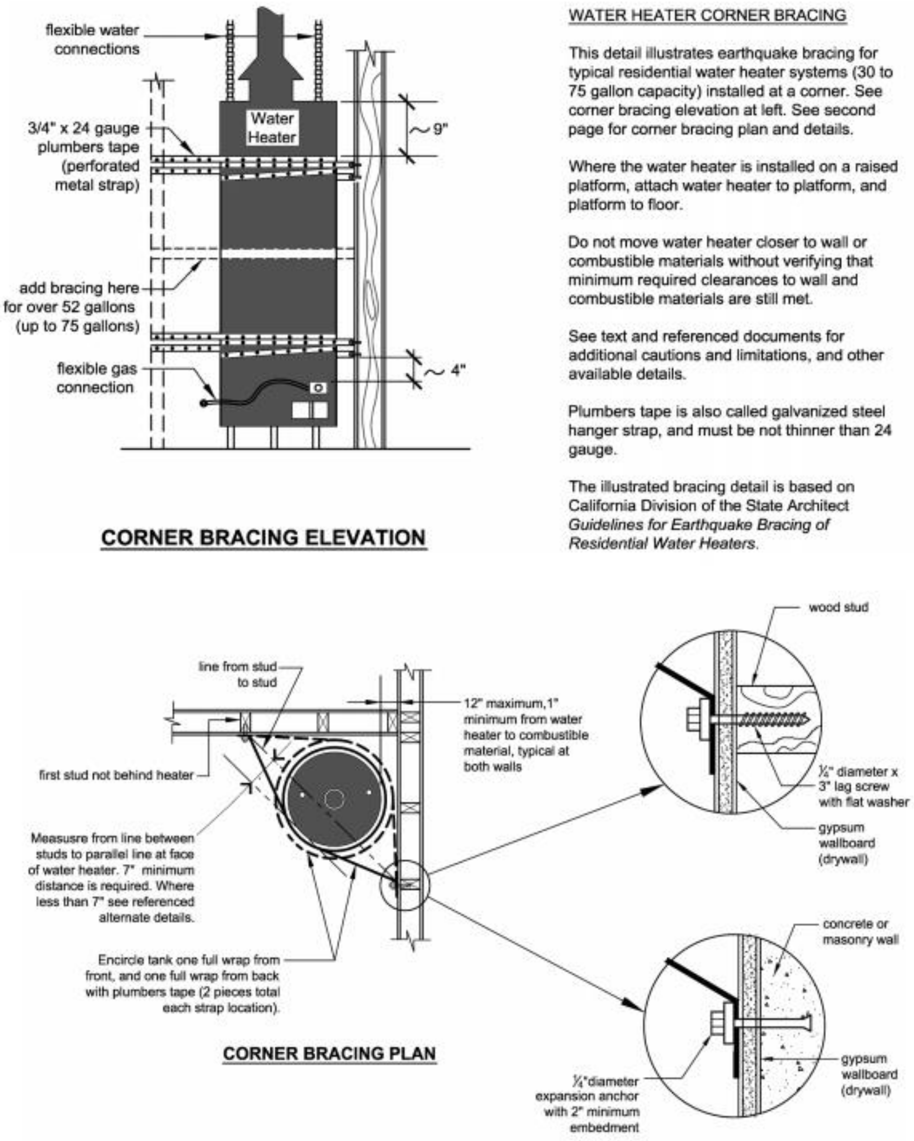


Figure 10-2 Securing a water heater with corner bracing (adapted from DSA, 2002).

10.2 Securing Other Home Furniture and Contents

In addition to appliances and equipment, anchorage of furniture and other home contents can greatly reduce the risk of injury, property loss, and interruption of home use as a result of an earthquake. Anchorage is particularly recommended for large and heavy items such as bookcases and cabinets (Figure 10-3) and for items that could cause injury if they fell (e.g., items on shelves above a bed that cannot be relocated).

Anchorage of other home contents will further reduce disruption following an earthquake. Measures to anchor computers and televisions range from very simple home fixes to specialized restraint systems available from a number of manufacturers. Locks on kitchen cabinets can help reduce the spilling and breakage of contents during an earthquake. Measures are available to secure a wide range of other home contents including pictures, mirrors, fragile objects, and fire extinguishers.

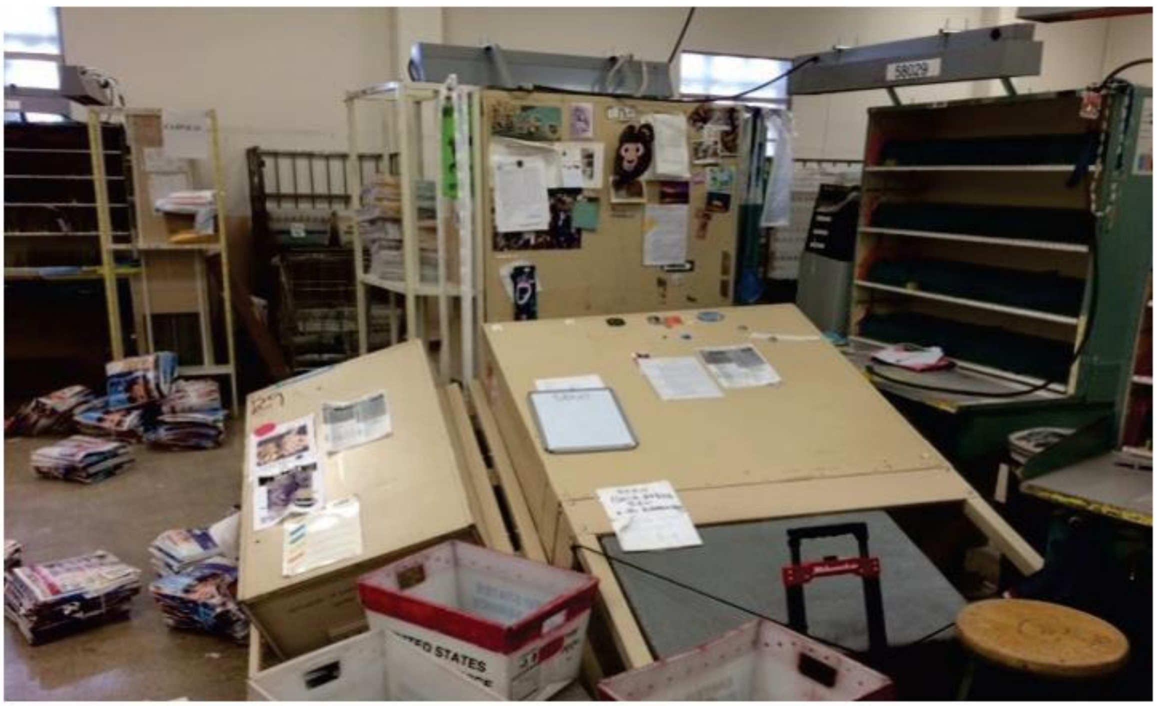


Figure 10-3 Toppling of unanchored cabinets in the 2014 South Napa Earthquake (from Glen Granholm, EERI Learning From Earthquakes).

FEMA E-74, *Reducing the Risks of Non-Structural Earthquake Damage* (FEMA, 2012b), provides guidance on furniture and contents anchorage that will be of assistance to interested users. Reproductions of FEMA E-74 pages are shown in Figure 10-4 through Figure 10-7, showing representative details for protecting common items from earthquake damage. Two different types of details are discussed in FEMA E-74:

♣ Do-it-yourself methods, which are simple generic methods for securing typical nonstructural items found in the home. Enough information is provided to enable a maintenance person with common tools and readily available materials to complete an installation.

♣ Engineered methods, which are schematic details showing common solutions for the items in question. These sketches do not contain enough information for installation; they are provided here primarily as an illustration of the scope of work required. The designation “Engineering Required” has been used for items where do-it-yourself installation is likely to be ineffective. FEMA E-74 recommends that design professionals be retained to evaluate the vulnerability of these items and design appropriate anchorage or restraint solutions, particularly where safety is an issue. Examples might include anchorage of propane tanks serving individual residences.

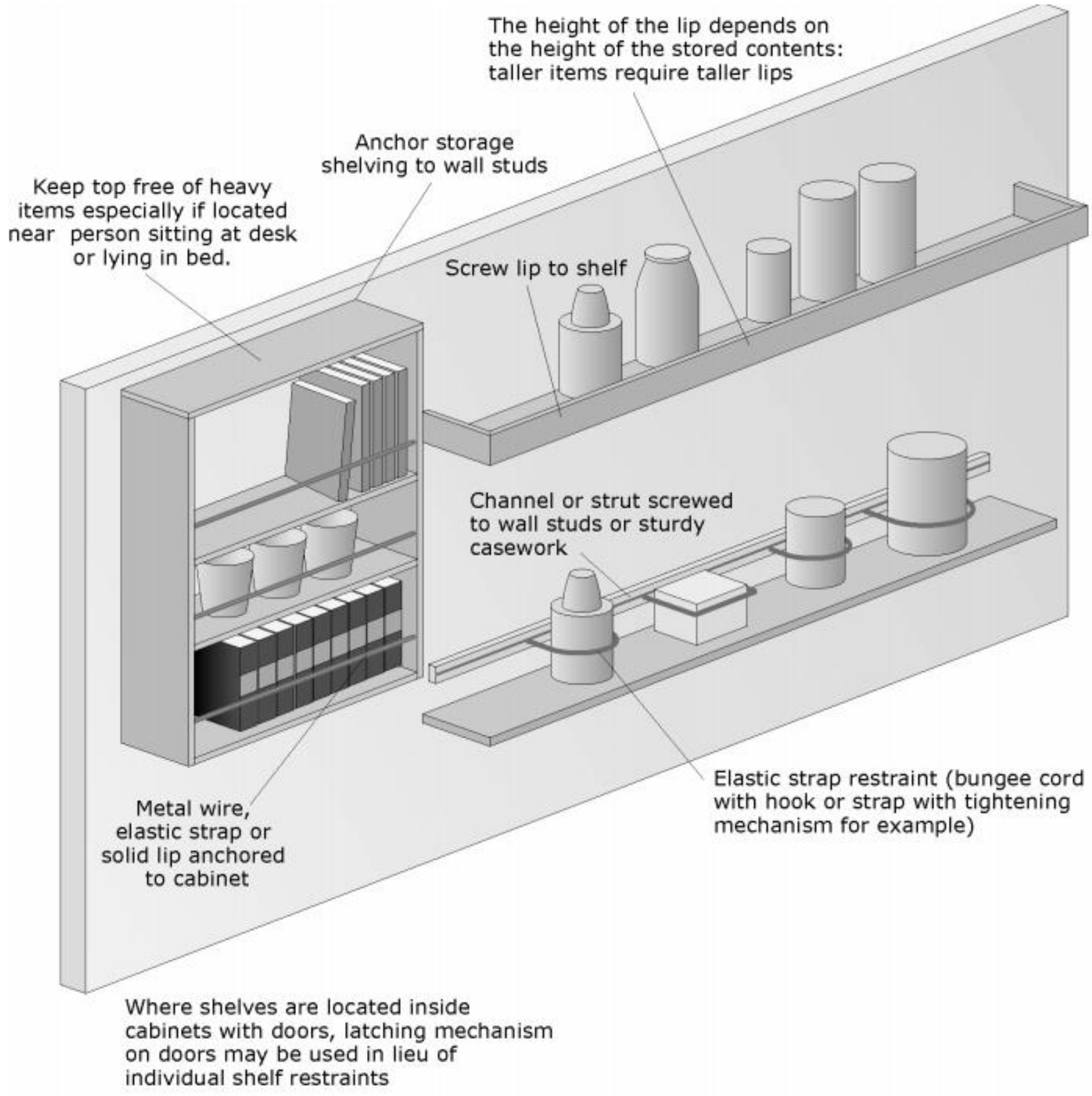


Figure 10-4 Methods for securing shelf-mounted items (from FEMA E-74).

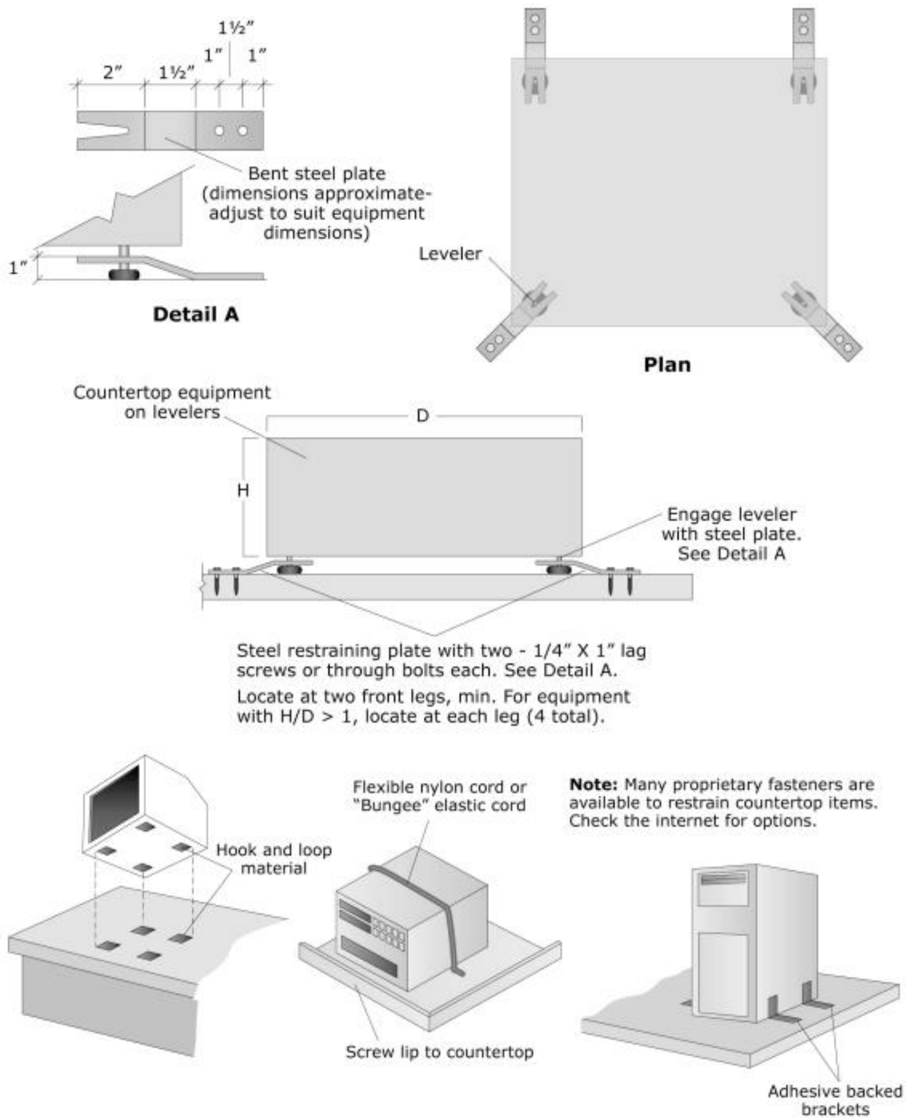


Figure 10-5 Methods for securing desktop items (from FEMA E-74).

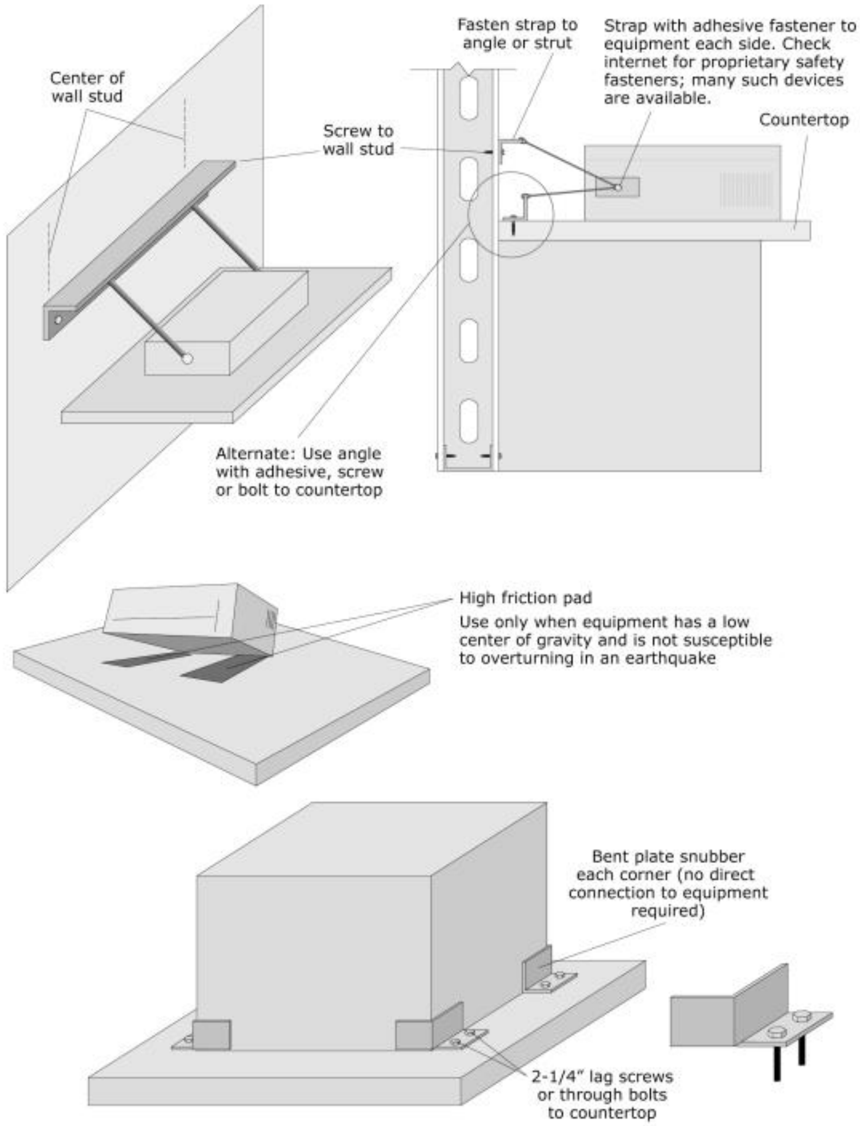


Figure 10-6 Methods for securing heavier desktop equipment (from FEMA E-74).

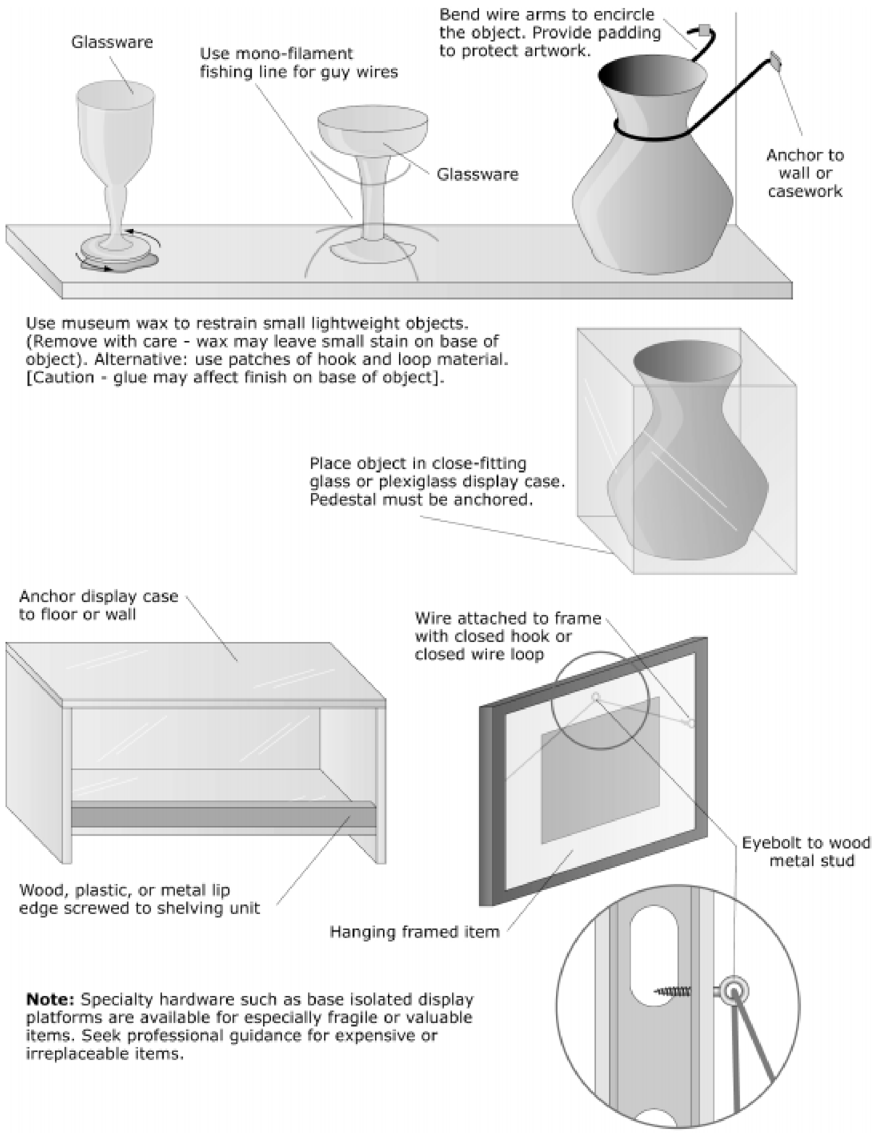


Figure 10-7 Methods for securing home artwork (from FEMA E-74).

11- Existing Dwellings and Townhouses

Over time, most dwellings and townhouses will be subject to work that extends beyond simple maintenance. Such work might include additions, alterations, repair, relocation, and change in use. This type of work could potentially reduce the earthquake resistance of an existing dwelling or townhouse. With proper consideration, however, earthquake resistance can be maintained or even increased while implementing this type of work. This chapter discusses the earthquake-resistance implications of structural work on existing buildings, applicable IRC provisions, and recommendations and references for earthquake retrofits.

The IRC was developed with the intent that it serve as a sole source for code requirements for dwellings and townhouses. The user has the ability to use provisions of the International Building Code (IBC), International Existing Building Code (IEBC) (ICC, 2023b), and other codes, but is not required to do so. Provisions affecting structural aspects of existing dwellings and townhouses are found in two primary locations in the IRC. One location is in §R102.6.1, where mandatory provisions are included that will generally be adopted by state and local jurisdictions when adopting the IRC. The second location is in IRC Appendix Chapter BO; as with all IRC appendices, this is only adopted if the jurisdiction specifically identifies the appendix chapter as being adopted in the adopting ordinance. For this reason, fewer jurisdictions adopt the appendix chapter, but it is available to those jurisdictions that wish to adopt it. It is also available for use on a voluntary basis or if approved by the building official.

Section 11.1 discusses the provisions of §R102.6.1, and the implications of these provisions for existing structures. Section 11.2 discusses the provisions of Appendix Chapter BO and its implications for existing structures. Section 11.3 provides recommendations for voluntary earthquake retrofit to existing dwellings and townhouses and identifies references of interest; homebuilders that work on existing dwellings and townhouses are encouraged to familiarize themselves with this section and communicate to homeowners the added value that seismic retrofit can provide, whether implemented as a stand-alone project or during alterations and additions can provide.

11.1 IRC Provisions

§R102.6.1 contains, in a brief statement, the current provisions of the IRC for additions, alternations, change of use, and repairs to existing dwellings and townhouses. The basic principles can be summarized in the following items:

♣ Item 1: New work is to conform to requirements for a new structure.

♣ Item 2: Existing construction can remain and need not comply to requirements for a new structure, unless otherwise stated.

♣ Item 3: Additions, alterations, and repairs are not permitted to cause the existing structure to become less compliant than the structure was prior to the new work.

♣ Item 4: Where the existing structure, together with additions, alterations, or changes of use, result in a use or occupancy, height, or means of egress outside of the scope of the IRC, it is required to comply with the IEBC.

The following discussion parses out the impact these provisions have in more detail.

11.1.1 Additions

In accordance with §R102.6.1, any new work that occurs as part of the addition is required to meet the requirements of the IRC. An addition to an existing dwelling or townhouse often results in both the removal of some existing bracing wall, roof, and floor areas and the addition of weight and, therefore, increased earthquake loading. Most additions can be categorized as horizontal additions, vertical additions, or a combination of the two. Horizontal additions generally are built along the side of an existing dwelling or townhouse. Vertical additions generally involve the addition of an upper story. Figure 10-1 illustrates horizontal and vertical additions.

Horizontal additions may create irregularities or make existing irregularities worse. Thus, the IRC building configuration irregularity provisions of §R301.2.2.6 should be reviewed to assess the conformance of the post-addition configuration.

Horizontal additions include the construction of new bracing walls at the new exterior of the dwelling or townhouse (and sometimes on the interior). A significant reconfiguration of bracing walls at the interface of existing and new construction often also occurs. All bracing walls in the addition and interface should be checked for conformance with the IRC as should any portions of the existing dwelling or townhouse where framing and bracing modifications have been made.

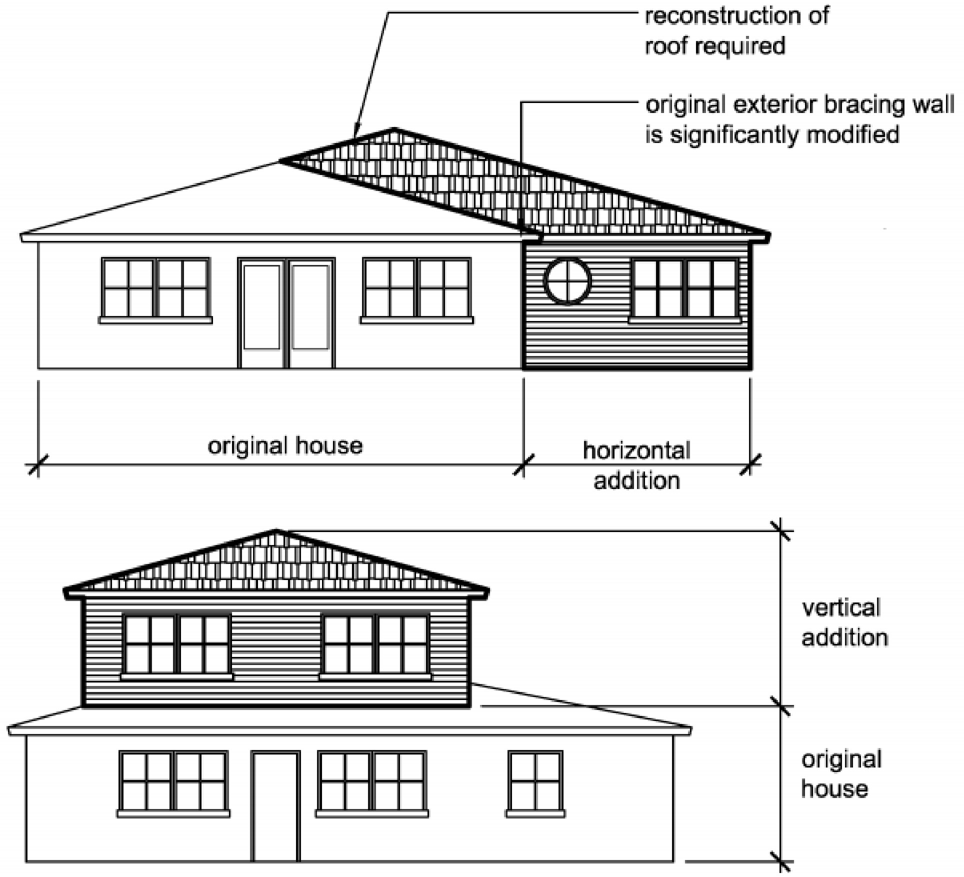


Figure 11-1 Horizontal addition (above) and vertical addition (below).

Finally, it is important that the horizontal addition and the existing dwelling or townhouse be tied together well. Ideally the level of interconnection should be the same as would occur if they had been built at the same time, but this generally cannot be practically achieved. Top plates and sill plates should be strapped or otherwise fastened between the new and existing construction to provide continuity.

Sheathing should be continuous and fastened to the same or interconnected framing members where possible. Where not possible, strapping of framing members should occur at a regular interval. Figure 11-2 shows the earthquake behavior of a dwelling with failed cripple walls and an adjoining (very small) slab-on-grade addition; a combination of cripple wall retrofit and strapping between the dwelling or townhouse and the addition would have greatly improved the performance of this dwelling or townhouse.

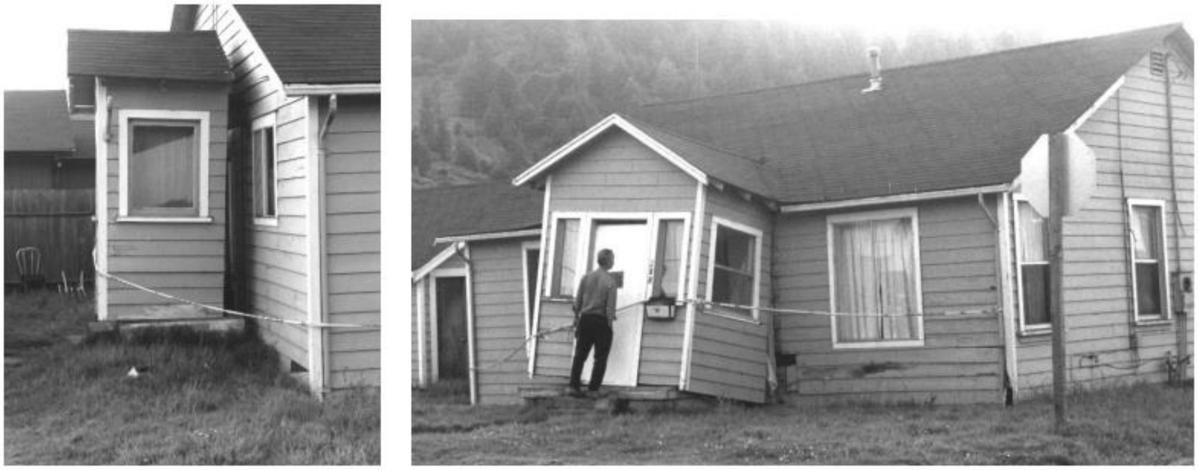


Figure 11-2 Damage to a cripple wall house with slab-on-grade additions in the 1992 Cape Mendocino Earthquake.

A vertical addition demands significantly greater consideration of gravity, earthquake and wind loads. This is because the story that is added often will more than double both the gravity and lateral loads on the existing lower story. This will very clearly make the existing structure less compliant, again triggering Item 3. Thus, when adding an upper story, the entire dwelling or townhouse, including the lower story, should be brought into conformance with current code requirements. Although it should be possible to meet IRC requirements, use of the engineered design requirements of the IBC or IEBC may result in a more practical design.

With any addition or alteration, it is important that the gravity and lateral load paths be checked in detail to ensure that they are complete and meet the requirements of the IRC or IBC. Additions and alterations often create nontypical load path details, and it is important that these details result in a complete load path with load-carrying capacity that is not less than would have resulted had typical details been used. In some cases, the new detailing deviates enough from that which is typical that engineered design should be provided.

Adding to or altering an existing dwelling or townhouse offers an opportunity to voluntarily retrofit existing portions of the dwelling or townhouse to better resist earthquake forces. Section 11.3 of this guide describes where such retrofits might be employed.

11.1.2 Alterations

In accordance with §R102.6.1, any new work that occurs as part of the alteration is required to meet the requirements of the IRC. Alterations often modify the load-resisting systems of existing dwelling or townhouses. Generally, both the systems supporting gravity loads and those supporting wind or earthquake loads are affected. From a structural standpoint, effects of concern occur in two broad groups: alterations that increase loads to the existing structure and alterations that reduce the capacity of the existing structure.

Increased loads can be from gravity dead load (i.e., self-weight of the structure) as might happen if a light roof is replaced by a heavy roof, live loads, wind loads, seismic loads, and others. For purpose of this discussion, our main concern is increased seismic loads. Where seismic loads increase, it is possible that the bracing walls become less compliant than they were prior to the alteration (Item 3). It is also possible that an increase in seismic load could occur and the wall bracing still be compliant; this might be checked by comparing the provided wall bracing with what would be required by the IRC. Where the wall bracing is found to not comply, retrofit of the wall bracing would be required. Similar questions would be appropriate for all load types and all elements of the structure, including the foundation.

Reduced capacity of the existing structure often comes about when alterations remove existing elements that provide bracing. Alterations to existing dwelling or townhouses often involve modification or removal of existing bracing walls and portions of floors and roofs. Figure 11-3 shows two alterations that remove exterior bracing walls from a dwelling or townhouse and disrupt the roof. Interior remodels often remove interior walls that provide bracing for earthquake and wind loads.

Where existing bracing walls are removed or reduced due to alterations, the remaining bracing walls should be checked for conformance with the bracing location, length, and bracing type requirements of the IRC provisions. The primary focus should be on bracing in the immediate vicinity of the alteration. If bracing deficiencies occur in other portions of a dwelling or townhouse, retrofit of those areas are encouraged.

When skylights, dormer windows, or similar openings are added to existing roofs, the openings should be checked for conformance with IRC requirements. For earthquake loading, this would include checking the opening size against permitted maximum sizes in the irregularities provisions and checking detailing against IRC requirements. The framing around the opening also should be checked for gravity load requirements such as doubled rafters and headers to support discontinued rafters. If a significant rebuilding of the roof is occurring, a broader range of IRC provisions require checking as does the completeness of the load path for gravity and lateral loads.

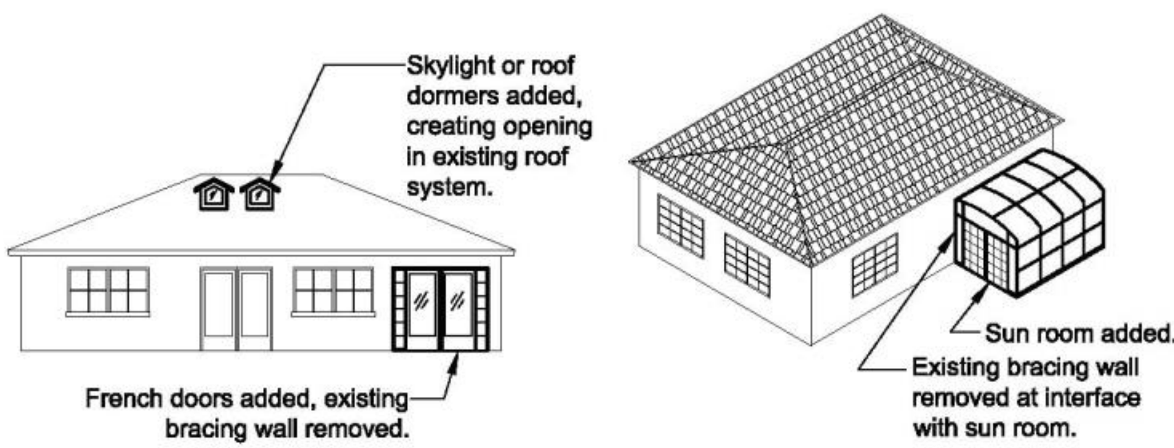


Figure 11-3 Alterations to existing house: bracing wall and roof modifications (left) and modification for the addition of sunroom (right).

11.1.3 Change in Use

In general, a change in use from a dwelling or townhouse to some other use would mean that the building would no longer fall within the scope of the IRC, which is narrowly scoped to dwellings and townhouses. This is complicated somewhat by the provisions of the exceptions to §R101.2, which allows live/work units, lodging houses, and care facilities of limited size to be located in IRC dwellings or dwelling units. At this time, the IRC does not impose any structural requirements on this type of change in use, provided the new occupancy is in accordance with §R101.2 and that an alteration or addition does not occur along with the change in use. This is consistent with these occupancies being permitted to occur in IRC dwelling units. Similarly, no requirements are triggered should an occupant of a dwelling unit be receiving custodial care within a dwelling unit. A change of an existing non-residential building to a residential use could potentially occur by showing compliance with the IRC. Given the additional detail and flexibility of the IEBC provisions, however, compliance with the IEBC may well be an easier approach.

11.1.4 Repairs

Similar to additions and alterations, repairs are subject to §R102.6.1 and Items 1 to 4. In particular for repairs, Item 1 requires that new work conform to requirements for new structures. This is more restrictive than IEBC provisions that might permit the damaged elements to be replaced in kind, provided that detailing in conformance with new requirements is used.

11.1.5 Relocation

When a structure is relocated, it is most often resupported on a new foundation that has been constructed on the new site and anchorage to the new foundation provided. The new foundation and anchorage are clearly required to conform to requirements for new construction. As part of relocation, the structure is also potentially being moved from a location with one set of environmental loads (e.g., wind, earthquake, rain, snow, flood) to a location with different loads. Should any of these environmental loads increase, the structure would become less compliant, and would potentially need to be retrofit. This could be checked by determining whether the existing dwelling meets current code requirements at the new location. For increases in seismic loads this would primarily involve checking adequacy of the wall bracing provisions. For increases in wind load, wind anchorage provisions should also be checked. If vertical gravity or live loads were to increase, such as due to increased snow loads, a wider check of structural framing would be required.

11.2 IRC Appendix Chapter BO

Across the I-codes, appendices are part of the published model code but must be specifically adopted by the state or local jurisdiction in order for the appendix chapter to be enforceable. For this reason, IRC Appendix Chapter BO is not enforceable unless it has been specifically noted as adopted in the state or local adoption ordinance. It is currently understood that IRC Appendix Chapter BO is occasionally adopted and, when adopted, further local amendments are often made as part of the adoption process. For this reason, the reader is urged to check with the local jurisdiction before using IRC Appendix Chapter BO.

As of the 2024 edition, IRC Appendix Chapter BO has been significantly amended to more systematically give direction for repair, additions, alteration, and relocation of existing dwellings and townhouses and to provide improved coordination with the IEBC. Changes to Appendix Chapter BO of structural interest include the following:

♣ Scope: §BO101.1 clarifies that repairs, alterations, additions, and relocations of dwellings and townhouses are required to comply with the requirements of the IRC for new construction, except as modified by the appendix chapter.

♣ Definitions: §BO103 definitions have been updated to supplement those already provided in IRC Chapter 2 and to better coordinate with the IEBC. Where terms of interest are not defined in Appendix Chapter BO, look for definitions in Chapter 2 of the IRC.

♣ Repair: §BO104 is updated to address use of new and existing materials in repairs, based on precedent established in the IEBC.

♣ Alterations: §BO105.4 introduces common sense provisions for alterations, based on the principles of the IEBC.

♣ Additions: §BO106 introduces common sense provisions for additions, based on the principles of the IEBC.

The §BO105.4 alteration provisions include the four basic items included in §R102.6.1, specifically discussing the requirements for alterations that decrease structural capacity and alterations that increase structural loads. Included is more detailed discussion of dead loads (self-weight of the structure), live loads, snow loads, and seismic loads. Exceptions permit common increases in dead loads from added layers of roofing and added photovoltaic panels. Wind load requirements are triggered by an increase to the projected area that will be subject to wind load, but also permit an increase in projected area of up to 5% of the existing area before triggering alteration requirements. Seismic provisions focus on two common sources of heavier loads: a new roof with increased weight or a new cladding with increased weight. The added detail of the Appendix Chapter BO provisions is intended to make the provisions more uniformly enforceable and to accommodate common and reasonable occurrences that should not trigger evaluation and retrofit.

The §BO106 provisions for additions are separated into horizontal additions (the addition built alongside the existing dwelling or townhouse), and vertical additions (increasing the height of the existing dwelling or townhouse). For a horizontal addition, new construction is to meet the IRC requirements for new construction; where the existing dwelling is altered the IRC requirements for alterations are applicable. Where a vertical addition occurs, wind and seismic loads to the existing dwelling or townhouse are very likely to increase, triggering a requirement for IRC conformance for the full dwelling or townhouse including both new and existing portions.

In addition, §BO105.4.3 provisions trigger bracing of unreinforced masonry parapet walls in SDC D_2 where a specified extent of reroofing occurs. Unreinforced masonry walls are widely known to be vulnerable to earthquake damage. Observations from past earthquakes suggest that damage often initiates at the parapet, and that without parapet anchorage, loss of the parapet often results in loss of the wall below. In high SDCs, anchorage of parapets is a cost-effective way to reduce earthquake damage. For parapets with low height-to-thickness ratios, anchorage at the roof line will adequately improve earthquake performance. For parapets with high height-to-thickness ratios, the combination of bracing in the top third of the parapet height and bracing at the roof level is required.

11.3 Earthquake Retrofit Measures

The life-safety performance of dwellings and townhouses in past earthquakes has been very good, with only a few exceptions. There are, however, certain configurations of dwellings and townhouses that have repeatedly resulted in earthquake damage, loss, and, in some cases, loss of life or injury. Several recent publications provide both helpful explanations of the vulnerable configurations and information on the implementation of retrofit measures. Homebuilders that work on existing dwellings and townhouses are encouraged to familiarize themselves with this section and communicate to homeowners the added value that seismic retrofit, implemented as a standalone project or during alterations and additions, can provide.

The following seismic retrofits are addressed in this section:

- ♣ Strengthening of weak cripple walls and anchorage to foundation (Section 11.3.1),
- ♣ Strengthening of hillside home anchorage to the foundation (Section 11.3.2),
- ♣ Strengthening of garages in living-space-over-garage homes (Section 11.3.3),
- ♣ Bracing homes supported on posts and piers (Section 11.3.4),
- ♣ Strengthening homes with unreinforced masonry walls (Section 11.3.5),
- ♣ Strengthening unreinforced stone or masonry foundations (Section 11.3.6),
- ♣ Retrofit of masonry chimneys (Section 11.3.7),
- ♣ Anchorage of elevated decks, porches, trellises, and carports (Section 11.3.8).

Because existing dwellings or townhouses vary widely in configuration and construction based on age, region, siting, and other factors, it is necessary to identify which retrofit measures are appropriate to the particular dwelling or townhouse. In deciding on voluntary retrofit measures, the configuration of the dwelling or townhouse and the potential benefit of the retrofit should be taken into consideration. The "Protect" section of FEMA P-530 provides an excellent introduction to considerations in the identification and prioritization of retrofit measures.

If a retrofit is being undertaken on a voluntary basis, the dwelling or townhouse generally will not be required to conform to all of the code requirements for new construction. The building official or authority having jurisdiction should be consulted regarding minimum requirements. It is recommended that a published, referenceable basis for the retrofit work be established (i.e., guidelines, plansets, building or residential code) be established and clearly documented. When a dwelling or townhouse is being remodeled or extensively renovated, a systematic retrofit to meet the IRC requirements for new construction may be reasonable and may be required by the authority having jurisdiction.

The remainder of this section provides an overview of the common retrofit measures listed above. Those interested in implementing specific measures are referred to FEMA P-530, FEMA P-1100, or ICC 1300, *Standard for the Vulnerability-Based Seismic Assessment and Retrofit of One- and TwoFamily Dwellings* (ICC, 2024b), and the list of references and resources at the end of this guide for further information on implementation.

11.3.1 Strengthening of Weak Cripple Walls and Anchoring to Foundation

Many dwellings or townhouses constructed before the 1950s in California, and even more recently in other parts of the United States do not have adequate anchorage of the dwelling or townhouse to the concrete or masonry foundation. During an earthquake, this can allow the wood framing to slide off the foundation and collapse. In addition, these dwellings with crawlspaces often have inadequate bracing strength in the cripple walls (walls that extend from the foundation to the underside of the lowest framed floor), resulting in racking or collapse of the cripple walls (Figure 11-4).

(a) (b)

Figure 11-4 (a) House with collapsed cripple walls (from NISEE), and (b) a house with severe damage due to cripple wall collapse (from FEMA).

RETROFIT MEASURES

Dwellings and townhouses can be retrofitted by strengthening cripple walls and adding or improving anchorage to the foundation. FEMA P-1100 and ICC 1300 provide direction for this retrofitting including both a simplified engineered retrofit method and a pre-engineered plan set that can be implemented by a contractor without involvement of an engineer. The main elements of the retrofit (seen in Figure 11-5) can include new anchor bolts to the foundation, new plywood or OSB sheathing on the inside face of the perimeter cripple walls, and new framing anchors connecting the top of the cripple walls to the floor framing above. These retrofit items can generally be implemented from the crawlspace without any disruption to occupied portions of the dwelling or townhouse.

RETROFIT IMPLEMENTATION

It is anticipated that these strengthening measures could be performed by an experienced homeowner with construction knowledge. However, working within a confined crawlspace can be difficult which is why a contractor is often desirable. Retrofit plan sets are applicable to most homes and will save the additional cost of a design professional. Where a home falls outside the scope of these plan sets, retrofit design by a registered design professional will be needed. The plan sets contain simple screening questions that will quickly identify whether or not the plan set can be used without engaging a design professional. A general contractor experienced in seismic retrofits is recommended.

Additional references include *Seismic Retrofit Training for Building Contractors and Inspectors* (ABAG) and *Earthquake Retrofit Handbook* (City of San Leandro, 1997).

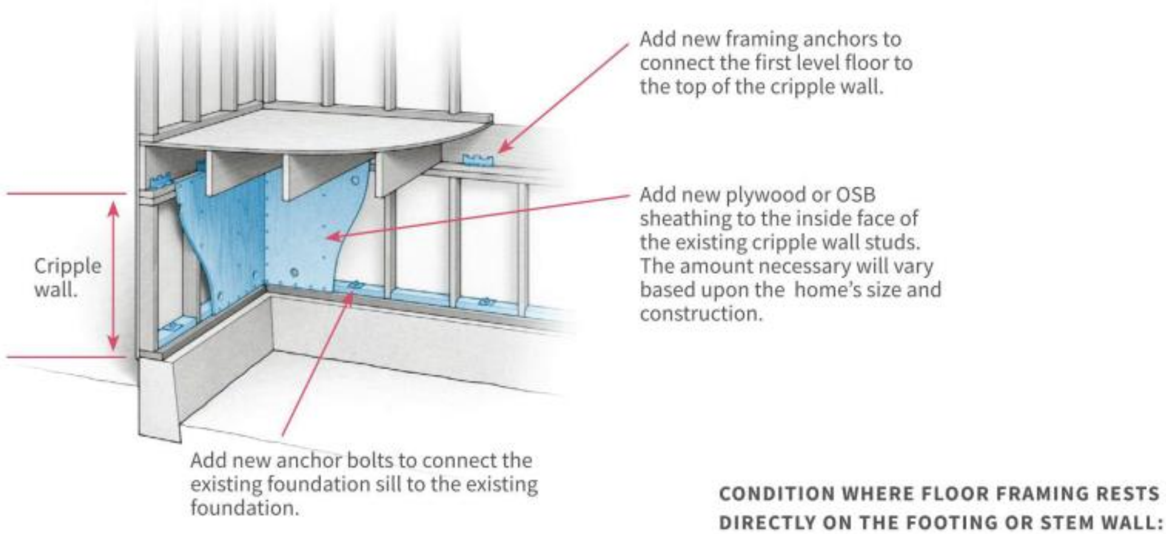


Figure 11-5 Cripple wall bracing and foundation bolting (from FEMA P-530).

11.3.2 Strengthening of Hillside Dwelling Anchorage to the Foundation

A number of dwellings on steep hillsides collapsed or were severely damaged in the 1994 Northridge Earthquake. Of particular concern are dwellings on sites with both a slope of one vertical to five horizontal or steeper and downhill wood walls or posts seven feet or taller (Figure 11-6). Many of the failures began with floor framing pulling away from the uphill foundation or foundation wall, with resulting damage as seen in Figure 11-7.

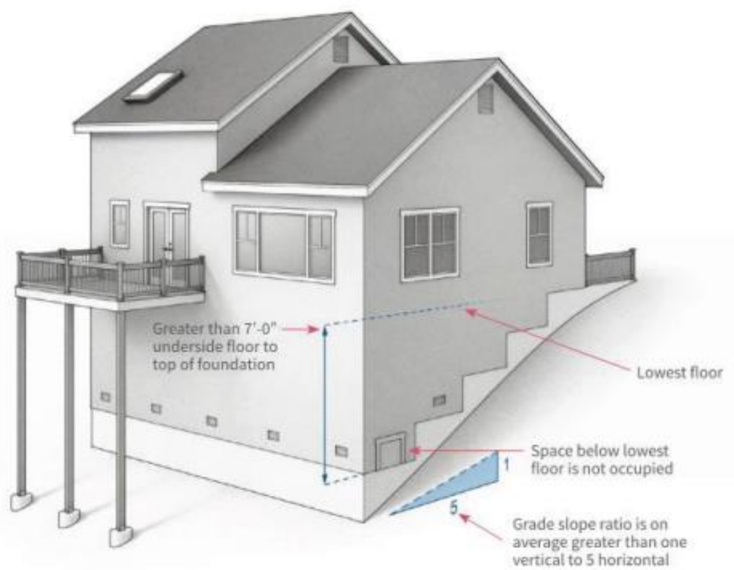


Figure 11-6 Dwelling located on a hillside site illustrating site slope and cripple wall height of concern (from FEMA P-530).

Figure 11-7 Dwellings located on hillside sites and damaged during the Northridge, California, earthquake (from FEMA P-530).

RETROFIT MEASURES

Hillside homes are retrofit by strengthening the anchorage of the home to the uphill foundation and by adding or strengthening walls surrounding the under-floor space (crawlspace). FEMA P-1100 and ICC 1300 provide direction for this retrofitting, including a simplified engineered retrofit method. Because of the large variability of existing dwelling configurations, no pre-engineered plan set is available. Strengthening work occurs primarily in the under-floor area. Retrofit elements are generally within the crawlspace and can include (Figure 11-8):

♣ Primary Anchors: large tension anchors tying each end of the floor to the uphill foundation. New concrete foundation elements are often required as part of this retrofit.

♣ Secondary Anchors: smaller tension anchors tying the floor to the uphill foundation, occurring at a regular spacing along the entire length of the uphill foundation.

♣ Shear Anchors: anchor bolts from the floor framing to the uphill foundation, occurring at a regular spacing along the entire length of the uphill foundation.

♣ Sheathing, anchor bolts, and shear clips on cripple walls on all sides of the under-floor crawlspace.

RETROFIT IMPLEMENTATION

It is anticipated that these strengthening measures will require the services of a design professional experienced in residential construction and of an experienced contractor.

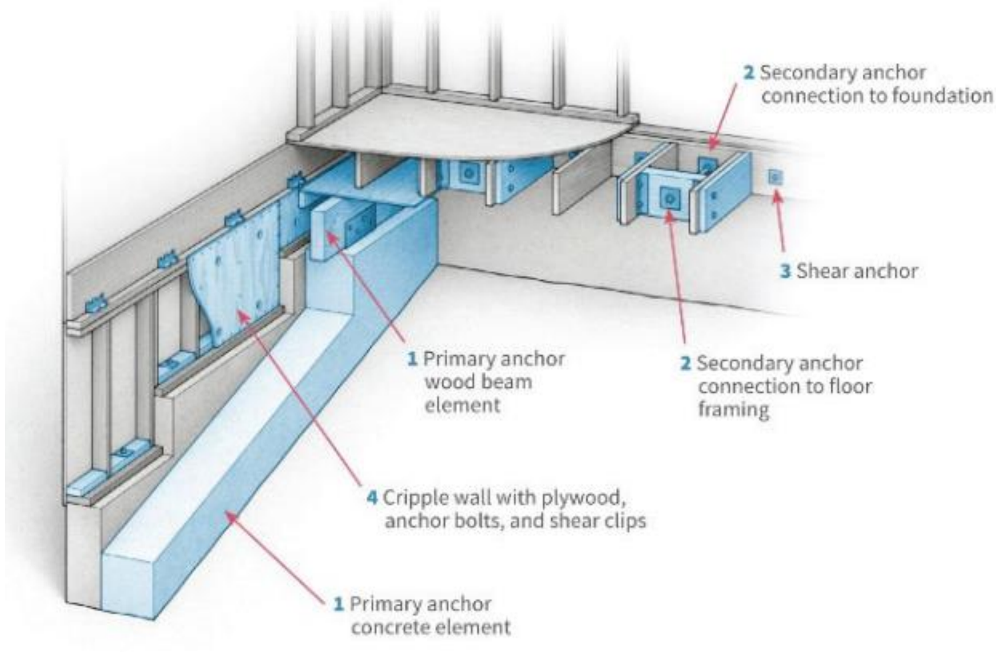


Figure 11-8 Anchorage of floor framing to the uphill foundation (from FEMA P-530)

11.3.3 Strengthening of Garages in Living-Space-Over-Garage Dwellings

Earthquake damage often is concentrated in the first story of multistory dwellings, particularly the garage area of a dwelling, which often has less earthquake bracing than other areas (Figure 11-9).

With earthquake ground shaking, homes with living space over a garage can sway sideways, resulting in significant damage and possible collapse of the home (Figure 11-10).

Figure 11-9 A common living-space-over-garage dwelling.

Figure 11-10 Damage to living-space-over-garage dwellings in the 1989 Loma Prieta Earthquake (from USGS).

RETROFIT MEASURES

Living-space-over-garage homes can be retrofit by strengthening existing walls around the garage perimeter. FEMA P-1100 and ICC 1300 provide direction for this retrofitting including both a simplified engineered retrofit method and a pre-engineered plan set that can be implemented by a contractor without involvement of an engineer. The typical locations and elements of a retrofit can be seen in the garage plan (Figure 11-11). The blue arrow indicates the retrofit elements seen in Figure 11-12.

♣ Where the length of wall adjacent to the garage door is narrow, steel columns or prefabricated shear walls can be placed in a new foundation inside of the garage door.

♣ Where the length of wall adjacent to the garage door side is at least 32 inches, shear walls detailed in accordance with the IRC can be constructed at the garage door wall.

♣ Plywood or OSB, anchor bolts, and shear clips are provided on other garage walls

RETROFIT IMPLEMENTATION

It is anticipated that these strengthening measures will require the services of an experienced contractor. Pre-engineered plan sets are applicable to some homes and will save the additional cost of a design professional. Where the home falls outside of the scope of these plan sets, the services of a registered design professional experienced in residential construction will be required.

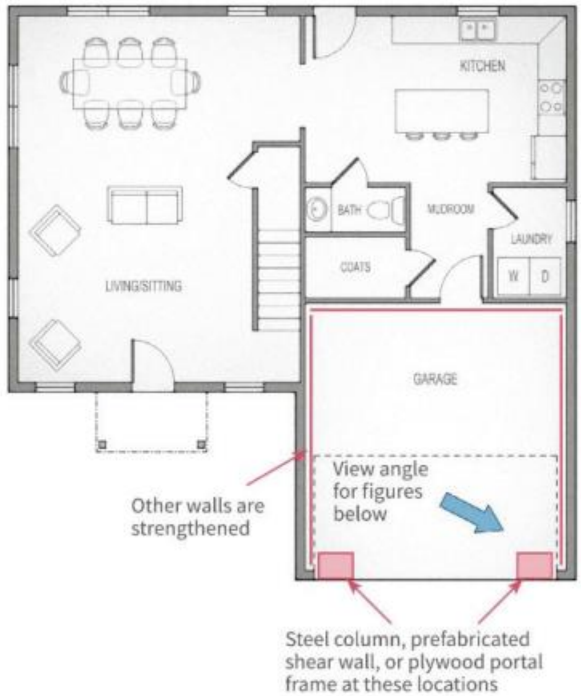


Figure 11-11 First floor plan showing location of living-space-over garage retrofit elements (from FEMA P-530)

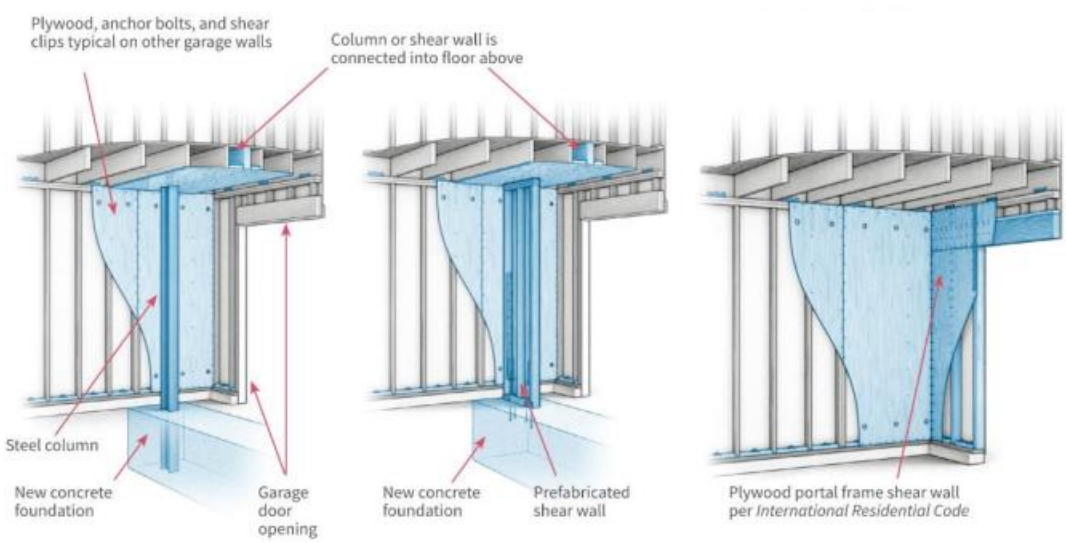


Figure 11-12 Weak- and soft-story bracing (from FEMA P-530).

11.3.4 Bracing Homes Supported on Posts and Piers

Homes supported only by posts (or posts with diagonal kickers as seen in Figure 11-13 and 11-14) are particularly susceptible to sideways motion. The posts can easily roll over or slide off of the supporting foundations, causing significant damaged and possible collapse during earthquakes.

RETROFIT MEASURES

Retrofit solutions include adding continuous foundation systems and sheathed cripple walls or raised concrete stem walls. The retrofit measures discussed in Section 11.3.1 include information for the construction of new continuous foundations and braced cripple walls at the foundation perimeter, which will provide appropriate retrofit for this construction (Figure 11-15). If the home is located in a flood hazard area and the solid walls are used as part of the seismic retrofit, flood vents will need to be added to allow flood water to equalize pressures. These details can also be used on manufactured (mobile) homes.

RETROFIT IMPLEMENTATION

Temporarily shoring the existing home and the addition of new foundations and cripple walls should only be attempted by an experienced contractor. The prescriptive plan sets available for strengthening cripple walls can be applicable and will save the additional cost of a design professional. Where a home falls outside of the scope of these plan sets, a design professional will be required. Most plan sets contain simple screening questions that will quickly identify whether or not the plan set can be used without engaging a design professional.

Figure 11-13 Dwelling with post and pier foundation (from FEMA P-530)

Figure 11-14 Earthquake damaged post and pier foundation (from FEMA P-530)

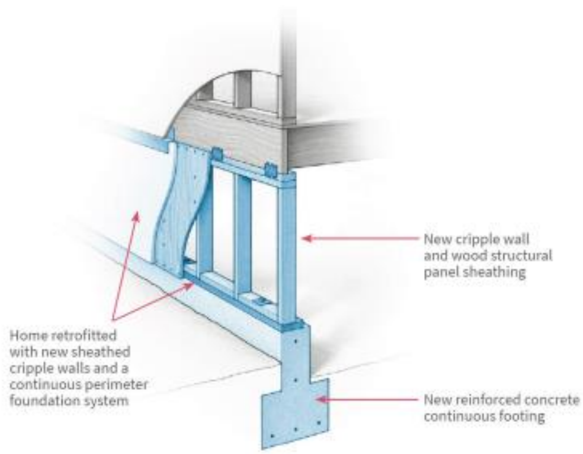


Figure 11-15 Retrofit continuous foundation with sheathed cripple wall (from FEMA P-530).

11.3.5 Strengthening Homes with Unreinforced Masonry Walls

In some regions, dwellings or townhouses constructed with unreinforced masonry walls are common, particularly in districts with older housing. Under earthquake loading, masonry walls can pull away from roof and floor framing (Figure 11-16). This is primarily a concern where no direct anchorage of the wall to the floor or roof framing exists.

Figure 11-16 Earthquake-damaged unreinforced masonry wall structure in Christchurch, New Zealand (from FEMA P-530).

RETROFIT MEASURES

This condition can be retrofitted to improve earthquake performance by providing anchorage from the wall to the floor and roof framing as shown in Figure 11-17. The anchorage seen in the lower left can be used to tie the wall to the floor and roof framing. The anchorage seen in the upper right is needed when the masonry wall extends above the roof level. The anchorage connection should be made to the joists when the joists run perpendicular to the wall and should be made to blocking and extend a substantial distance into the interior of the floor system when the joists run perpendicular.

RETROFIT IMPLEMENTATION

An engineering evaluation of the existing condition and engineered design of retrofit measures are recommended, as is the involvement of an experienced contractor.

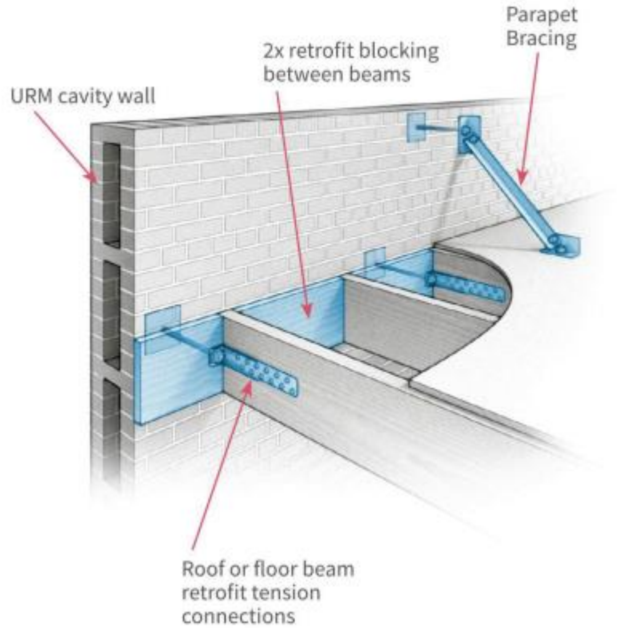


Figure 11-17 Anchorage of concrete or masonry walls to floor, roof, or ceiling framing (from FEMA P-530).

Additional references include *International Existing Building Code* Appendix Chapter A2 and *Guidelines for Seismic Evaluation and Rehabilitation of Tilt-up Buildings and Other Rigid Wall/ Flexible Diaphragm Structures* (SEAONC, 2001), and FEMA P-774, *Unreinforced Masonry Buildings and Earthquakes—Developing Successful Risk Reduction Programs* (FEMA, 2009).

11.3.6 Strengthening Unreinforced Stone or Masonry Foundations

Unreinforced stone and masonry foundations have been found to be vulnerable to earthquake damage. Older foundations can have deteriorated masonry or mortar that makes them weaker. Foundations that have signs of deterioration, diagonal cracking, or obvious settlement may be more vulnerable to earthquake damage (Figure 11-18). Earthquake damage can vary from minor to extreme depending upon the original materials used, method of construction, current condition, and the stability of the supporting soil.

RETROFIT MEASURES

Retrofitting could include partial to complete foundation removal and replacement or the addition of partial new foundations while leaving the existing foundation in place (Figure 11-19).

RETROFIT IMPLEMENTATION

A design professional is recommended to assess and design the required retrofits. Where a design professional recommends new foundations or partial replacements, the work should only be attempted by an experienced contractor.

Figure 11-18 Earthquake damage to unreinforced masonry foundations in the 2011 Mineral, Virginia Earthquake (left) and 1994 Northridge Earthquake (right) (from FEMA P-530).

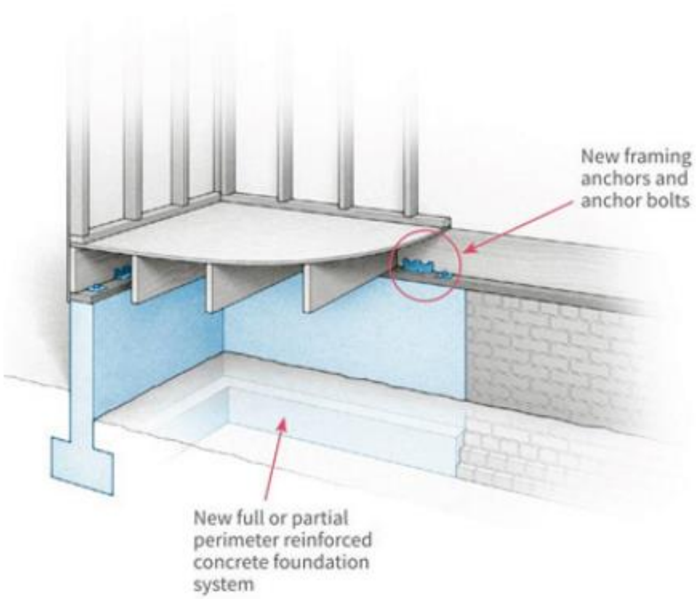


Figure 11-19 Retrofit with the addition of a new partial or full perimeter foundation (from FEMA P-530).

11.3.7 Retrofit of Masonry Chimneys

Fireplaces and chimneys in new construction were discussed in Section 9 where it was noted that masonry and concrete fireplaces are heavy, rigid, brittle, and very susceptible to earthquake damage. Due to their heavy weight, falling portions of chimneys can endanger people inside and outside of structures. Chimneys on existing dwellings and townhouses generally are even more vulnerable than new chimneys because they seldom have adequate reinforcing or anchorage to the dwelling or townhouse. Common chimney failures range from hairline fractures of masonry and flue liners to complete fracture (i.e., at the top of the firebox and at the roof line) permitting large sections of the chimney to fall away from or into the dwelling or townhouse (Figure 11-20).

Figure 11-20 Earthquake damage to existing masonry chimneys (from FEMA P-530).

RETROFIT MEASURES

FEMA P-1100 and ICC 1300 provide direction for this retrofitting including both a simplified engineered retrofit method and a pre-engineered plan set that can be implemented by a contractor without involvement of an engineer. Retrofit measures providing a range of hazard reduction include (Figure 11-21):

♣ Capping the chimney at the roof level making the fireplace no longer usable (Figure 11-21, left),

♣ Rebuilding the chimney from the top of the firebox up, either maintaining the use of the existing masonry firebox, or installing a fireplace insert (Figure 11-21, right), or

♣ Fully rebuilding the firebox and chimney in wood or steel stud construction (Figure 11-21, bottom center). Complete rebuilding a chimney must be in full compliance with the IRC.

RETROFIT IMPLEMENTATION

These retrofit measures will require the services of an experienced contractor. Where the home falls outside of the scope of the FEMA P-1100 and ICC 1300 plan sets, the services of a registered design professional experienced in residential construction will be required. It is important to note that although previous guidance suggested strapping of the chimney as a retrofit technique, chimney failures have occurred even with strapping provided. Therefore, this type of retrofit is no longer recommended.

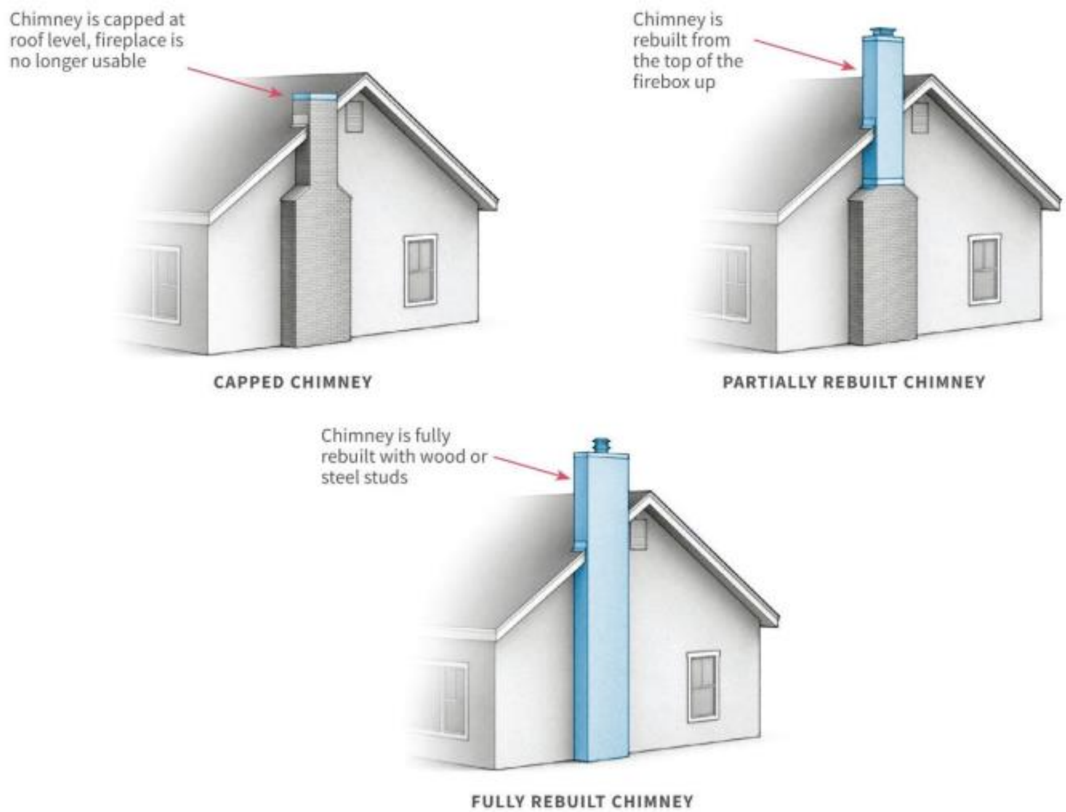


Figure 11-21 Retrofit approaches for unreinforced masonry chimneys (from FEMA P-530).

11.3.8 Anchorage of Elevated Decks, Porches, Trellises, and Carports

Elevated exterior decks, large projecting roofs, and similar structures often have little or no earthquake bracing, making full or partial collapse possible (Figure 11-22).

Figure 11-22 Earthquake damage to elevated decks and carports (from FEMA P-530).

RETROFIT MEASURES

A variety of retrofit measures can be implemented to improve earthquake bracing; such measures include:

♣ As a primary bracing measure, horizontal tension anchors can be installed perpendicular to the home's exterior wall at each end can also help control excessive sideways movement and help prevent the deck from separating away from the home (Figure 11-23, right). Anchorage guidance and details can be found in the IRC as discussed in Chapter 9 of this guide.

♣ As an additional secondary measure, properly constructed diagonal bracing can be added to the outboard end of the deck (opposite the house) to help prevent excessive sideways movement, associated damage, and in extreme cases, collapse (Figure 11-23, left).

♣ Other measures to tie together framing members can be installed as illustrated in Figure 11-24.

RETROFIT IMPLEMENTATION

It is anticipated that the strengthening of elevated decks, carports, trellises, and porches could be performed by an experienced homeowner with construction knowledge. A general contractor and possibly a design professional should be consulted in situations where the components being braced or connected are large or complicated, and where preferred by the homeowner.

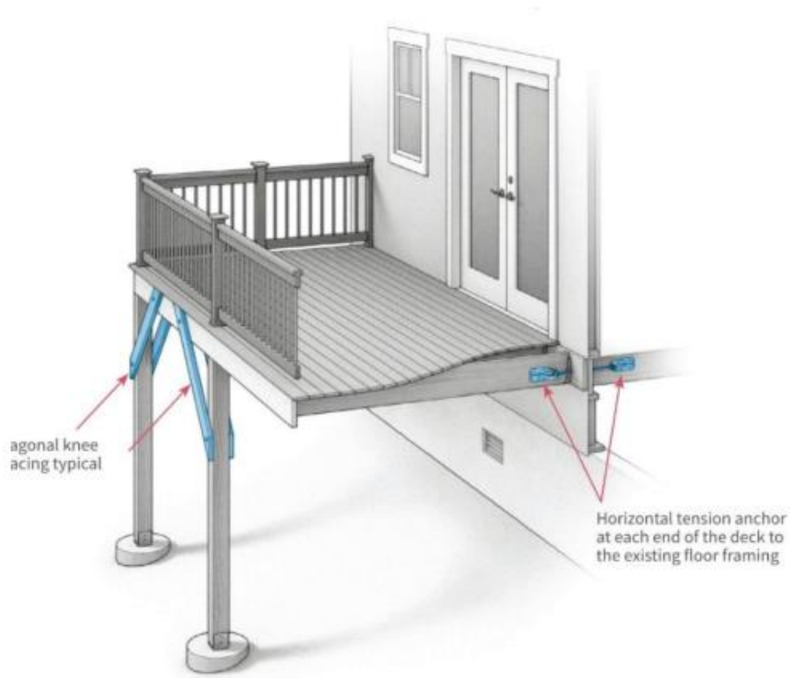


Figure 11-23 Retrofit measures for exterior decks and balconies (from FEMA P-530)

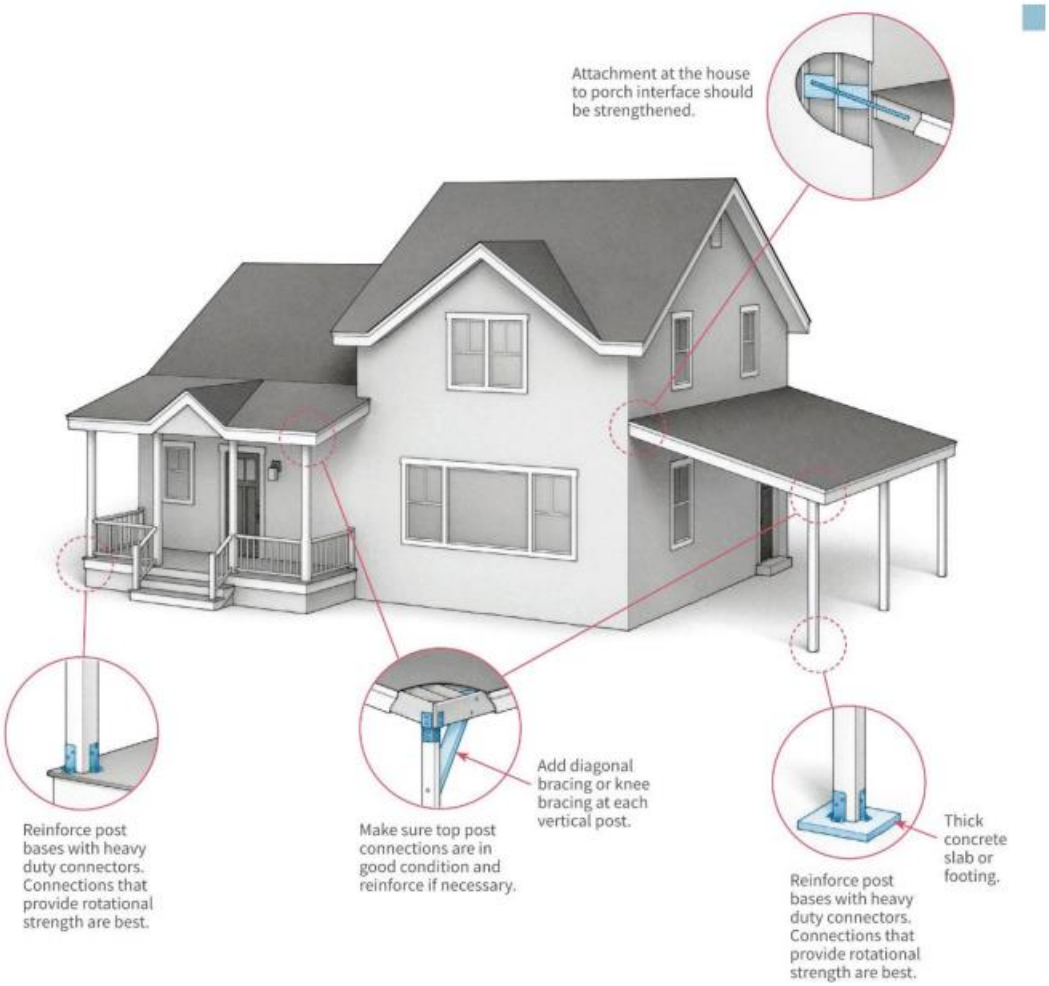


Figure 11-24 Retrofit measures for porch overhangs (from FEMA P-530).

APPENDICES

Appendix A: Checklist for Builders, Designers, and Plan Checkers

This checklist incorporates selected highlights from the provisions discussed in this guide, with an emphasis on earthquake resistance. The reader is referred to this guide and to the IRC for more complete information on earthquake design provisions. In the checklist, "C" refers to compliant, "NC" to noncompliant, and "N/A" to not applicable.

General Earthquake-Resistance Requirements

General Earthquake Scope Limitations

C	NC	N/A	
☐	☐	☐	Seismic Design Category. An SDC of A through E is assigned. §R301.2.2.1.
☐	☐	☐	Seismic Design Category. Buildings in SDCs A through D_2 may be designed per the IRC; buildings in SDC E require engineered design unless the alternate determination of SDC provisions of §R301.2.2.1.2, Item 1 or 2 is met. §R301.2.2.
☐	☐	☐	Assembly weight. Weight of roof plus ceiling, floor, interior wall and exterior wall assemblies are limited in SDCs D_0, D_1 and D_2 and townhouses in SDC C. §R301.2.2.2.
☐	☐	☐	Number of stories. The number of stories is limited by type of construction and SDC as follows:

Wood light-frame	SDC A to D_1 – three stories (§R301.2.2.7) SDC D_2 – two stories (§R301.2.2.7 & Table R602.10.3(3))
Cold-formed Steel	SDC A to C – three stories (§R301.2.2.7) SDC D_0 to D_2 – design per AISI S230 (§R301.2.2.8)
Masonry Wall	SDC A, B, dwellings in C – three stories (§R101.2) SDC C townhouses – two stories (IRC Table R606.12.2.1) SDC D_0 to D_2 – one story (§R606.12.3.1)
ICF	SDC A, B, dwellings in C – two stories (§R301.2.2.5 and R608) SDC D_0 to D_2 and townhouses in C – design per PCA 100 or ACI 318 (§R301.2.2.5)

C	NC	N/A	
☐	☐	☐	Story height. All SDCs. Building story height is limited by the following limits on bearing wall clear height plus a maximum of 16 inches for the floor framing depth:

Wood light frame	12 ft (§R301.3, Item 1 Exception)
Cold-formed steel	10 ft (§R301.3, Item 2)
Masonry	12 ft (§R301.3, Item 3)
ICF	10 ft (§R301.3, Item 4, and §R608)

C	NC	N/A	
☐	☐	☐	Veneer weight and height. The weight and height of veneer is limited as follows: Dwellings: SDC A to C – 50 psf, 30 feet + 8 feet gable end (§R301.2.2.2, R703.8, Table R703.8(1)) SDC D_0 to D_1 – 50 psf, first story only (§R703.8) SDC D_0 – 40 psf, 30 feet + 8 feet gable (Table R703.8(2)) SDC D_1 – 40 psf, 20 feet + 8 feet gable (Table R703.8(2)) SDC D_0 – 30 psf, 20 feet + 8 feet gable (Table R703.8(2)) Townhouses: SDC A to C – 50 psf, first story only (§R703.8) SDC D_0 to D_2 – engineered design required

Irregularities

The following are descriptions of seismic irregularities per §R301.2.2.6. See Table 3-5 of this guide for further discussion and illustrations.

C	NC	N/A	
☐	☐	☐	Irregularity 1: Exterior braced wall panels not in one vertical plane (stacked) from foundation to topmost story in which they are required.
☐	☐	☐	Irregularity 2: Section of floor or roof not supported by braced wall lines on all edges.
☐	☐	☐	Irregularity 3: End of braced wall panel occurs over opening in wall below and extends more than one foot beyond the edge of the opening.
☐	☐	☐	Irregularity 4: Opening in floor or roof exceeds lesser of 12 feet or 50% of least floor or roof dimension.
☐	☐	☐	Irregularity 5: Portions of floor level are vertically offset (split level).
☐	☐	☐	Irregularity 6: Braced wall lines do not occur in two perpendicular directions.
☐	☐	☐	Irregularity 7: Stories braced by light-frame walls include concrete or masonry construction.
☐	☐	☐	Irregularity 8: Hillside light-frame construction where averaged over any side the site slope exceeds 1 vertical in 5 horizontal, the downhill light frame wall or post exceeds seven feet in height, and less than 50% of the area of the lowest floor is occupied.

Above-Code Recommendations

- ☐ Apply irregularities to all SDCs because they are also applicable for wind load.
- ☐ Increase first-story strength and stiffness to mitigate weak-story irregularity.
- ☐ Increase cripple wall strength and stiffness to mitigate weak-story irregularity.

Load Path

C	NC	N/A	
☐	☐	☐	Wood Light-frame Building Anchorage to Foundation – See Section 4.12. All SDCs – Anchor bolts are to be embedded not less than 7 inches into concrete or grouted masonry units. Provide a minimum of two anchor bolts per section of foundation sill. First anchor bolt is to be not less than 3 1/2 inches or more than 12 inches from each end of the sill. Provide nut and washer. (§R403.1.6) All SDCs – Typical 1/2-inch diameter bolts at 6 feet along all exterior walls. Also, 1/2-inch bolts at 6 feet along interior braced walls and interior bearing walls supported on a continuous foundation. All interior walls to be anchored with 1/2-inch bolts at 6 feet or approved equivalent. (§R403.1.6) SDCs D_0 to D_2 and townhouses in SDC C – 1/2 inch at 4 feet along all exterior walls and along all interior bearing walls and interior bracing walls supported on a continuous foundation for buildings over two stories. (§R403.1.6.1) SDCs D_0 to D_2 – 3×3×0.229-inch steel plate washers: On all required anchor bolts in SDCs D_0 to D_2 and townhouses in SDC C. (§R403.1.6.1 and R602.11.1)
☐	☐	☐	Overturning Anchorage – The following light-frame wall bracing types require hold-down anchors to resist wall overturning: Alternate Braced Wall (ABW) – §R602.10.6.1 and Figure R602.10.6.1, Continuous Sheathing (CS-WSP) – §R602.10.2.2.1 Item 1.2, Portal Frames (PFH, PFG & CS-PF) – §R602.10.6.2, .3, and .4 and corresponding figures, and Brick veneer (BV-WSP) – §R602.10.6.5.2 and corresponding figure.

Above-Code Recommendations

- ☐ Provide 4-foot anchor bolt spacing along all exterior and interior braced wall lines for two-story houses in SDCs D_0 to D_2.
- ☐ Provide continuous foundation below interior braced walls with anchor bolts at spacing of 6 feet of less in all SDCs.
- ☐ Provide corrosion-resistant coatings for anchor bolts installed through pressure preservative treated foundation sill plates in all SDCs.
- ☐ Do not "wet set" anchor bolts; securely place anchor bolts prior to placing concrete.
- ☐ In SDC C, provide overturning anchorage as required in SDCs D_0 to D_2 for braced wall panels not located at corners and for houses with masonry veneer.
- ☐ Add tie straps between first and second story corner studs to tie the walls together in SDCs C, and D_0 to D_2.
- ☐ Use oversized sheathing panels on exterior walls and lap over rim-joist. Nail both into the plates (top and bottom) and the rim-joists in all SDCs.

C	NC	N/A	
☐	☐	☐	Minimum fastening. All SDCs per IRC Table R602.3(1) and (2). See Table 3-2 of this guide.
☐	☐	☐	Braced wall panel connection to roof. (§R602.10.8)
☐	☐	☐	Braced wall panel connection to floor and ceiling. (§R602.10.8)

Foundations and Foundation Walls

General

C	NC	N/A	
☐	☐	☐	Continuous perimeter foundations. All exterior walls are to be supported on continuous perimeter foundations. All SDCs. (§R403.1)
☐	☐	☐	Continuous interior foundations. At interior braced wall lines in buildings with plan dimensions greater than 50 ft. SDCs D_0 and D_1. (§R403.1.2)
☐	☐	☐	Continuous interior foundations. At interior braced wall lines (with allowance to increase to 50 ft. when criteria are met) SDC D_2. (§R403.1.2)
☐	☐	☐	Design in accordance with accepted engineering practice for foundation walls with more than 4 feet of unbalanced backfill and not having permanent lateral support at both the top and the bottom. (§R404.1.1)

Special Soils Conditions

C	NC	N/A	
☐	☐	☐	Low bearing capacity. Soils investigation required when building official determines that soil bearing capacities of less than 1500 psf might be present at site. All SDCs. (IRC Table R401.4.1, footnote b)
☐	☐	☐	Soil testing when expansive, compressible, or shifting soils are encountered or are likely. Where soil testing is required and SDC is C or greater using default soil assumptions, soils testing report is to include SDC determined per the IBC. (§R401.4)
☐	☐	☐	Frost protection. Footings are to be below the frost line or adequate frost protection should be provided. (§R403.1.4.1)

Concrete General

C	NC	N/A	
☐	☐	☐	Minimum concrete strength 2500 psi for all SDCs. 3000 or 3500 psi in moderate or severe weathering probability areas. (§R402.2 and Table R402.2)
☐	☐	☐	Rebar lap splices and standard hooks in accordance with §R608.5.4. Additional rebar detailing information in Figure 4-7 and Figure 4-10 of this guide.

Concrete Footings

C	NC	N/A	
☐	☐	☐	Horizontal reinforcing. In SDCs D_0 to D_2, typical one No.4 in footing. Additional No. 4 in concrete stem wall (if stem wall occurs). One No. 4 top and bottom in thickened slab footing, with alternate of one No. 5 or two No. 4 in middle third of footing height for thickened slab footings cast monolithically with slab. (§R403.1.3)
☐	☐	☐	Vertical reinforcing. In SDCs D_0 to D_2, one No. 4 at 48 inches maximum spacing where a pour joint occurs between concrete footing and concrete stem wall. (§R403.1.3)
☐	☐	☐	Adequate support of reinforcing and anchor bolts. Reinforcing concrete cover distances of 3 inches when cast against earth and 1-1/2 inches when concrete will be exposed to weather. (§R403.1)
☐	☐	☐	Clean footing excavations before casting concrete. Proper concrete consolidation. No water added to concrete mix at site.

Concrete Foundation Walls (Stem Wall, Retaining Wall, Basement Wall)

C	NC	N/A	
☐	☐	☐	Horizontal reinforcing. Dependent upon wall thickness and material. (§R404.1.3.2)
☐	☐	☐	Vertical reinforcing. Varies depending on wall height and soil type at site. ASTM Grade 60 minimum. (§R404.1.3.2)

Masonry Foundation Walls (Stem Wall, Retaining Wall, Basement Wall)

C	NC	N/A	
☐	☐	☐	Masonry foundation type. Solid clay masonry and fully grouted concrete masonry permitted in all SDCs. (§404.1.2.1) Rubble stone masonry foundation walls limited to SDC A, B and dwellings in C (§404.1.2.1)

C	NC	N/A	
☐	☐	☐	Wall thickness. Six inches minimum up to 12 inches required based on soil type at site. (IRC Table R401.1.2.1(1))
☐	☐	☐	Horizontal reinforcing per IRC Table R404.1.2(1). Vertical reinforcing per IRC Table R404.1.2,1(2) to R404.1.2,1(4).

Above-Code Recommendations

- ☐ Avoid construction of slab on grade homes on cut and fill sites where possible. Where this condition cannot be avoided, provide additional quality control for fill placement and compaction operations.
- ☐ Regardless of SDC, provide not less than one continuous horizontal No. 4 reinforcing bar in concrete footings. Provide a second No. 4 horizontal bar in stem wall if occurs. This will provide tension and bending capacity to help mitigate foundation damage due to earthquake, wind, soil movement, and frost heave.
- ☐ Regardless of SDC, remove lose debris in the construction joint between a concrete footing and a separately cast slab-on-grade.
- ☐ Regardless of SDC, provide not less than No. 4 at 4 feet on center vertical bars as dowels between a concrete footing and a separately cast slab-on-grade.

Floor Construction

Load Path

C	NC	N/A	
☐	☐	☐	Floor sheathing nailing. Floor sheathing should be edge nailed to blocking above all braced wall lines, exterior and interior, as part of the load path (IRC Table R602.3(1), Footnote i). Blocking with edge nailing needs to have a load path to top of braced wall panels.
☐	☐	☐	All SDCs. Floor framing is fastened to supporting wall top plate or foundation sill plate below per Table R602/3(1). See Table 3-2.
☐	☐	☐	SDCs D_0 to D_2, blocking or lateral restraint. Required at intermediate floor framing member supports. (§R502.7, Exception)

Cantilevered Joists

C	NC	N/A	
☐	☐	☐	All SDCs. Cantilevered floor joists conform to §R502.3.3(1) or R502.3.3.(2). See also Section 4.3 of this guide.

C	NC	N/A	
☐	☐	☐	SDCs C and D_0 to D_2, Cantilevered floor joists supporting braced wall panels above conform to §R301.2.2.6.

Light-Frame Wall Construction

Wall Bracing

C	NC	N/A	
☐	☐	☐	Braced wall line placement, length and spacing is in accordance with IRC §R602.10.1 and Figure R602.10.1.1. Braced wall line spacing: SDC A, B and dwellings in C – 60 feet on center Townhouses in SDC C – 35 feet on center with allowance for 50 feet SDC D_0 to D_2 – 25 feet on center with allowance for 35 feet
☐	☐	☐	Total length of bracing in each braced wall line meets requirements of Table R602.10.3(1) modified by R602.10.3(2) for wind and Table R602.10.3(3) modified by R602.10.3(4) for seismic.
☐	☐	☐	Braced wall panel construction details and minimum lengths for the selected bracing methods meet applicable requirements of IRC §R602.10.4 through R602.10.7 and Table R602.10.4.
☐	☐	☐	Placement of braced wall panels in braced wall lines conforms to §R602.10.2.2.
☐	☐	☐	Number of braced wall panels in each braced wall line conforms to §R602.10.2.3.

Construction Details

C	NC	N/A	
☐	☐	☐	Overdriven sheathing nails. For wood structural panel sheathing, nails are to be driven so that the top of the head is flush with the face of the sheathing. (It is recommended that where nail heads occasionally are more than 1/16 inch below the surface, an additional nail should be provided between existing nails. If a substantial number of nails are overdriven, the sheathing should be removed, and the framing checked for splitting before replacing the sheathing with proper nails.)
☐	☐	☐	Sheathing nailing to hold-down posts and studs. Hold-downs cannot carry any load unless the wall sheathing is edge-nailed to the hold-down post or stud.
☐	☐	☐	Threaded rods with properly attached nuts need to be in place before the wall sheathing is attached to the second side of the walls.

Above-Code Recommendations

- ☐ Increase first-story strength and stiffness to mitigate weak-story irregularity. Approaches include: (a) use of wood structural panel wall bracing and hold-down connectors at each end of each full height wall segment, (b) fully sheathing all exterior walls including below windows and above and below doors and providing hold-down connectors at building corners, and (c) providing more than the minimum braced wall panel length.
- ☐ Increase cripple wall strength and stiffness to mitigate weak-story irregularity by sheathing full length of exterior cripple walls.
- ☐ Use oversized sheathing panels on exterior walls to increase wall stiffness and strength. Lap the sheathing over the floor joists and nail to both the plates (top and bottom) and the floor joists.

Roof Construction

Load Path

C	NC	N/A	
☐	☐	☐	Load path connection of roof framing to walls below: SDC A to C – heel height > 9 1/4" in accordance with Figure R602.10.8.2(1) SDC D_0 to D_2 – heel height ≤ 15 1/4" in accordance with Figure R602.10.8.2(1) SDC D_0 to D_2 – heel height > 15 1/4" in accordance with Figure R602.10.8.2(2), or in accordance with truss manufacturer recommendations or AWC WFCM
☐	☐	☐	Load path connection of ceiling framing to walls below in accordance with Table R602.3(1).

Construction Details

C	NC	N/A	
☐	☐	☐	Attic ventilation per §R806.

Cold-formed Steel Construction

C	NC	N/A	
☐	☐	☐	Cold-formed steel framing. Buildings in SDCs D_0 to D_2 need to comply with the AISI *Standard for Cold-Formed Steel Framing – Prescriptive Method for One- and Two-Family Dwellings* in addition to the requirements of §R301.2.2.8.
☐	☐	☐	Load path connections. Connection of cold-formed steel framing members is different than wood light-frame connection. The IRC provisions include a significant number of specific connection details. Attention to these details is important to the building performance for all load types.

Above-Code Recommendations

- ☐ Add interior cold-formed steel braced walls such that the distance between braced wall lines does not exceed 35 feet.
- ☐ In all SDCs, apply the irregularity limitations developed for wood light-frame houses (§R301.2.2.2.2).

Masonry Wall Buildings

C	NC	N/A	
☐	☐	☐	Limited to one story for SDCs D_0 to D_2. (§R606.12.3 and Table R606.12.2.1)
☐	☐	☐	In SDCs D_0 to D_2, 9 feet between lateral supports. (§R606.12.3.1, Exception)
☐	☐	☐	Light-frame bracing walls restricted from supporting lateral loads from masonry. (§R301.2.2.6, Item 7)
☐	☐	☐	Connections to masonry shear walls and columns. For SDC D_0 to D_2 and townhouses in C, connections to masonry shear walls and columns are required to be designed in accordance with TMS 402.
☐	☐	☐	Construction quality control. Proper type of mortar for masonry being used and proper mortar mixing. (Type N mortar is prohibited in higher SDCs.) Proper placement of reinforcing. Adequate support and attachment of reinforcing and anchor bolts. Cleaning out of grout space to allow proper grout placement, including cleaning out excess mortar if necessary. Provide cleanouts if necessary for adequate cleaning. Consolidation of grout.

Above-Code Recommendations

- ☐ Each exterior wall and each interior braced wall should have one, and preferably two, sections of solid wall not less than 4 feet in length.
- ☐ Sections of solid wall should be spaced no more than 40 feet on center and should be placed as symmetrically as possible.
- ☐ All masonry walls should be supported on substantial continuous footings extended to a depth that provides competent bearing.
- ☐ The distribution of interior masonry braced walls should be carefully balanced and floor and roof plans should use simple rectangular shapes without jogs and openings.
- ☐ Apply the irregularity limitations developed for wood light-frame houses (§R301.2.2.2.2). The IRC exceptions to Irregularities 2 and 5 can be applied but the rest of the exceptions are not applicable.
- ☐ Solid portions of wall should be stocked from floor to floor and masonry walls should be continuous from the top of the structure to the foundation. Masonry walls not directly supported on walls below require engineered design for gravity load support and design for earthquake and wind loads should be provided.
- ☐ Running board lay up of masonry units should be used instead of stack bond lay up.
- ☐ For concrete masonry, use open end units at locations of vertical reinforcement and use wood beam units for horizontal reinforcing to increase the interlocking of masonry construction.
- ☐ Apply the measures required or recommended for masonry construction in areas of high earthquake risk, in areas of lower earthquake risk, and in high-wind areas. Priorities include provisions for reinforcing (IRC Figure R606.10 (2)), wall anchorage using details developed to resist out-of-plane wall loads (e.g., IRC Figures R611.8 (1) through (7)), minimum length of bracing walls, and a spacing limit for braced wall lines.

Concrete and Insulating Concrete Form Wall Buildings

C	NC	N/A	
☐	☐	☐	Limited to two stories above grade. (§R301.2.2.5 and R608)
☐	☐	☐	Minimum wall section dimensions per §R608.3 and Table 608.3.
☐	☐	☐	Maximum plan dimension of 60 feet, floor span of 32 feet, roof span of 40 feet and mean roof height of 32 feet. (§R608.2)
☐	☐	☐	Construction quality control. Proper placement of reinforcing. Adequate support and attachment of reinforcing and anchor bolts. Cleaning out of cells space to allow proper concrete placement. Consolidation of concrete.

Above-Code Recommendations

- ☐ Carefully balance bracing walls around the perimeter of the building.
- ☐ Apply the measures required or recommended for ICF houses in areas of high earthquake risk, in areas of lower earthquake risk, and in high-wind areas. Priorities include wall anchorage using details developed to resist out-of-place wall loads (e.g., IRC Figures R611.8 (2) through (7)).

Stone and Masonry Veneer

C	NC	N/A	
☐	☐	☐	Veneer. In SDC D_2, veneer is not permitted on buildings with cripple walls (IRC Table 602.10.6.5.4)
☐	☐	☐	Wood light-frame buildings with veneer conform to the following wall bracing requirements: Veneer at first story only – §R602.10.6.5.1 Limited veneer above first story – §R602.10.6.5.3 More than limited veneer above first story – §R602.10.6.5.2

Above-Code Recommendations

- ☐ Use corrosion-resistant sheet metal ties or wires to fasten veneer. Fasteners attaching ties or wires should penetrate the building paper or other weather-resistive barrier and sheathing and should be embedded in the wall studs.
- ☐ Where veneer can be used only on the first story above grade, increase the length of the structural wood panel bracing and use hold-down devices on braced wall panels in the first story.

Fireplaces and Chimneys

C	NC	N/A	
☐	☐	☐	Vertical and horizontal reinforcement requirements for SDC D_0 to D_2. Vertical reinforcing of not less than four No. 4 bars for chimneys up to 40 × 24 inches. Should extend from bottom of foundation (3 inch minimum concrete cover) to top of chimney except that splices of not less than 24 inches are acceptable. Must be placed such that reinforcing can be surrounded in grout. Horizontal ties of 1/4 inch minimum at 18 inches maximum on center in mortar joint. (IRC §R1001.3 and R1003.3)

C	NC	N/A	
☐	☐	☐	Anchorage requirements for SDCs D_0 to D_2. (IRC §R1001.4 and R1003.4)
☐	☐	☐	Type N mortar prohibited in SDCs D_0 to D_2. (§R606.2.8.3)

Appendix B: Seismic Design Category Maps

The IRC provides maps identifying the earthquake ground shaking hazard. The 2024 IRC seismic design maps (IRC Figures R301.2.2.1(1) through (7)) shown in Figure B-1 designate the SDCs for U.S. States and Territories. The legend correlates the SDC with the horizontal earthquake acceleration expected on the dwelling or townhouse in terms of gravity (g). A value of 100% g generates horizontal earthquake forces equal to the full weight pushing horizontally (an object weighing 100 lb. experiences a 100 lb. horizontal force). Web-based tools are intended to be available in 2024 that allow the user to input the street address or latitude and longitude coordinates of the site and obtain earthquake hazard information. Based on the information provided on the IRC maps, the map information will be available at: https://doi.org/10.5066/F7NK3C76.

The IRC maps have been developed in collaboration with the USGS and are based on their National Seismic Hazard Models (NSHMs), the site-specific ground motion procedures of the *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures* (FEMA, 2020c) and ASCE/SEI 7-22 (Chapter 21), and the IRC definition of SDC (Table R301.2.2.1.1).

Further explanation of both the IRC and IBC seismic hazard maps can be found in FEMA P-2192-4 (FEMA, 2023b). The following is excerpted from that discussion:

> The IRC SDC maps can be conservatively used for any site, except that use of the maps is not permitted for poor soil sites as discussed in IRC Section R401. In the updated IRC SDC maps, the same choices as in prior IRC map editions have been made by the map development team. The first choice assumes that dwelling seismic demands are controlled by short-period behavior, with mapping based on only the short-period design spectral response acceleration parameter, S_{DS}. This ignores the one-second parameter, S_{D1}, which is considered in the IBC. The second choice assumes default site (soil) conditions, defined in the 2020 *NEHRP Recommended Provisions* and ASCE/SEI 7-22 as the most critical of Site Classes C, CD, and D. Third, unlike the 2020 NEHRP Recommended Provisions and ASCE/SEI 7-22, SDC E is mapped where S_{DS} exceeds 1.25 and SDC D is subdivided into D_0, D_1, and D_2. With these choices, the mapping information from the 2020 *NEHRP Recommended Provisions* and ASCE/SEI 7-22 are translated to SDC, using Table R301.2.2.1.1.
>
> The intent of adopting SDC maps is to spare the non-technical user of the IRC from having to implement the provisions of ASCE/SEI 7 Chapter 11; however, the IRC includes provisions allowing use of the IBC and ASCE/SEI 7 to determine SDC should the user elect to do so. Because the IRC maps only incorporate S_{DS} and not S_{D1}, it is possible for the IRC maps to assign a lower SDC than would be assigned using ASCE/SEI 7 in some locations. Because the IRC maps incorporate Site Classes C, CD, and D, it is possible for the IRC maps to assign a higher SDC than would be assigned using ASCE/SEI 7 when using a specific assigned site class.

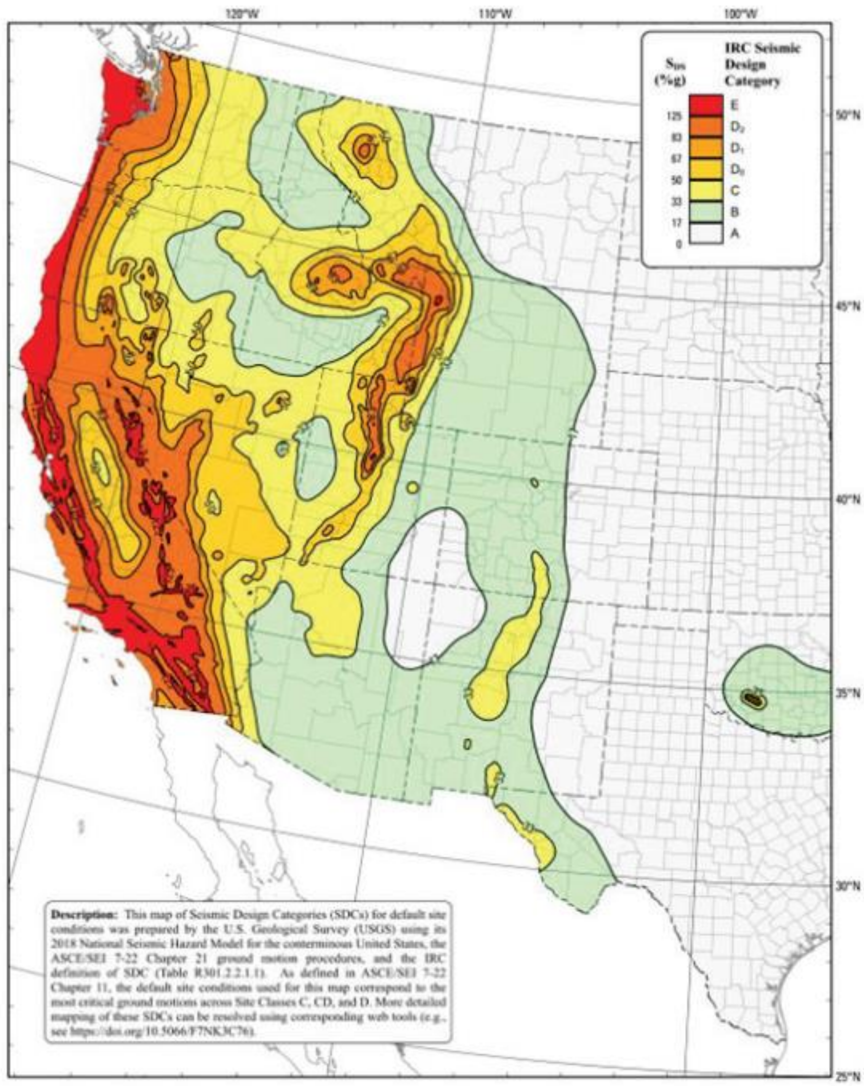


Figure B-1 **Seismic Design Categories based on 2024 IRC maps.**

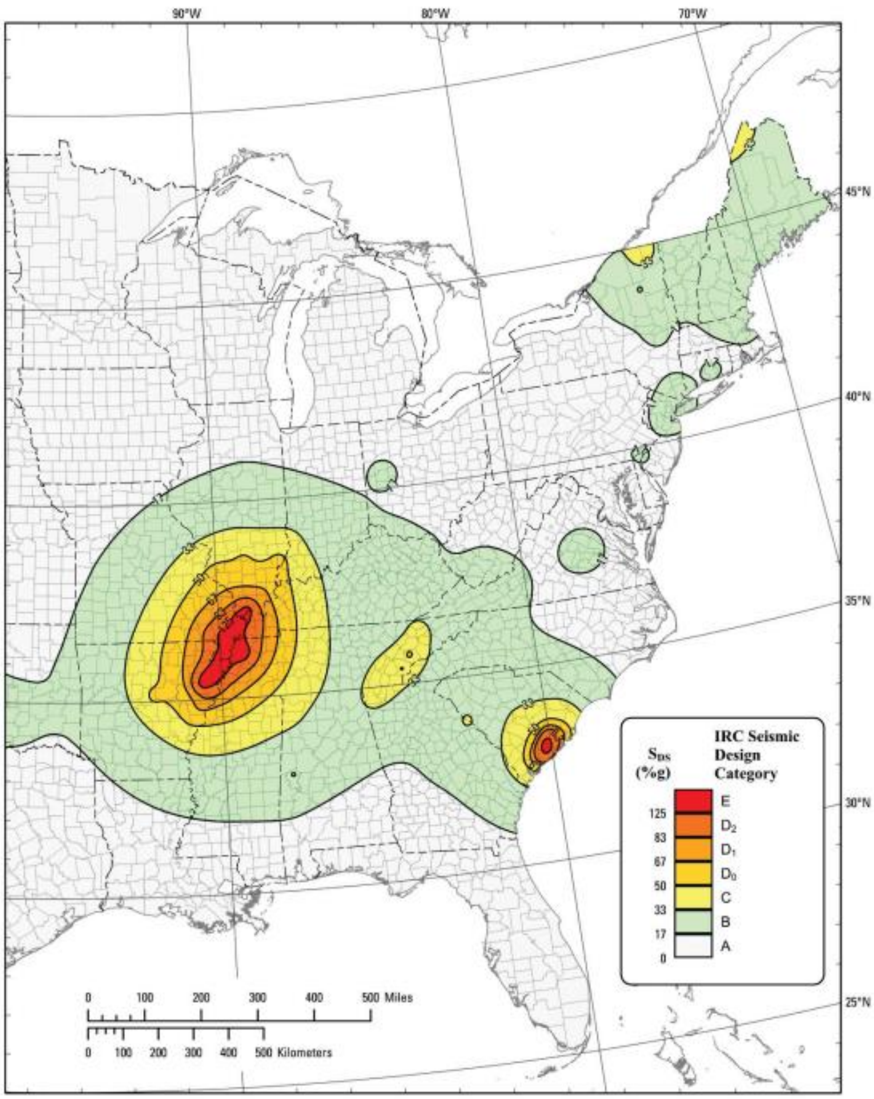
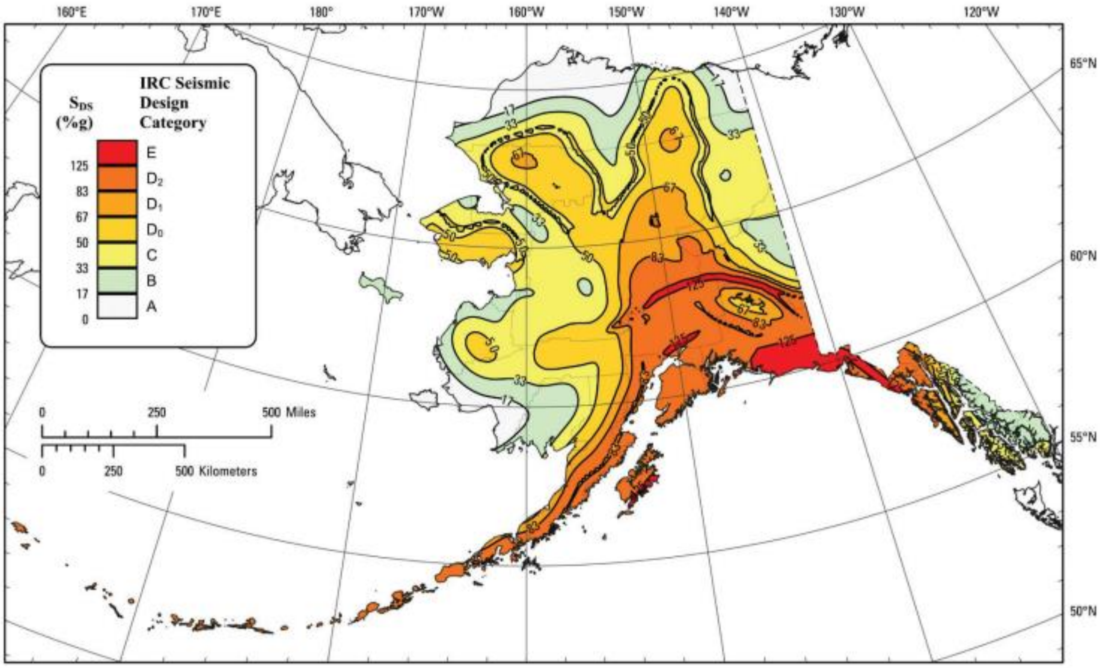


Figure B-1 Seismic Design Categories based on 2024 IRC maps (continued).



Description: This map of Seismic Design Categories (SDCs) for default site conditions was prepared by the U.S. Geological Survey (USGS) using its 2007 National Seismic Hazard Model for Alaska, the ASCE/SEI 7-22 Chapter 21 ground motion procedures, the FEMA P-2078 procedures for developing multi-period response spectra at non-conterminous U.S. sites, and the IRC definition of SDC (Table R301.2.2.1.1). As defined in ASCE/SEI 7-22 Chapter 11, the default site conditions used for this map correspond to the most critical ground motions across Site Classes C, CD, and D. More detailed mapping of these SDCs can be resolved using corresponding web tools (e.g., see https://doi.org/10.5066/F7NK3C76).

Figure B-1 Seismic Design Categories based on 2024 IRC maps (continued).

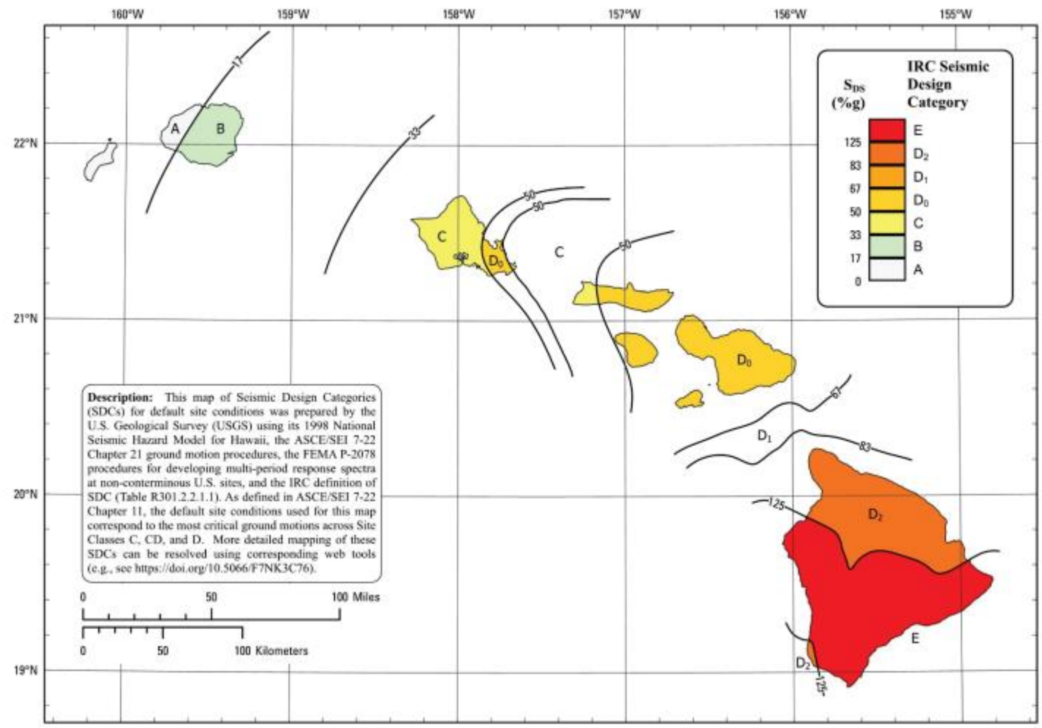
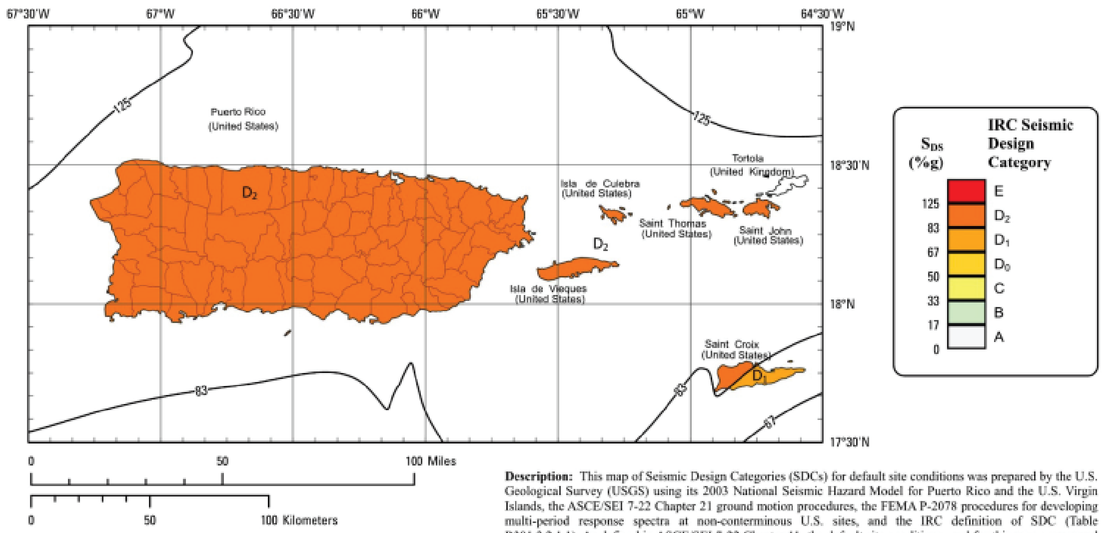
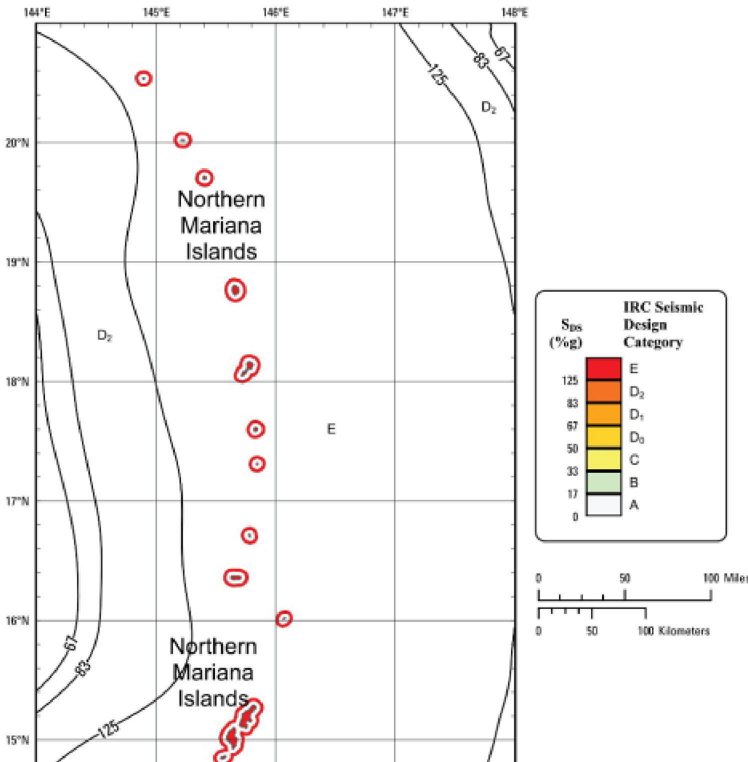


Figure B-1 Seismic Design Categories based on 2024 IRC maps (continued).



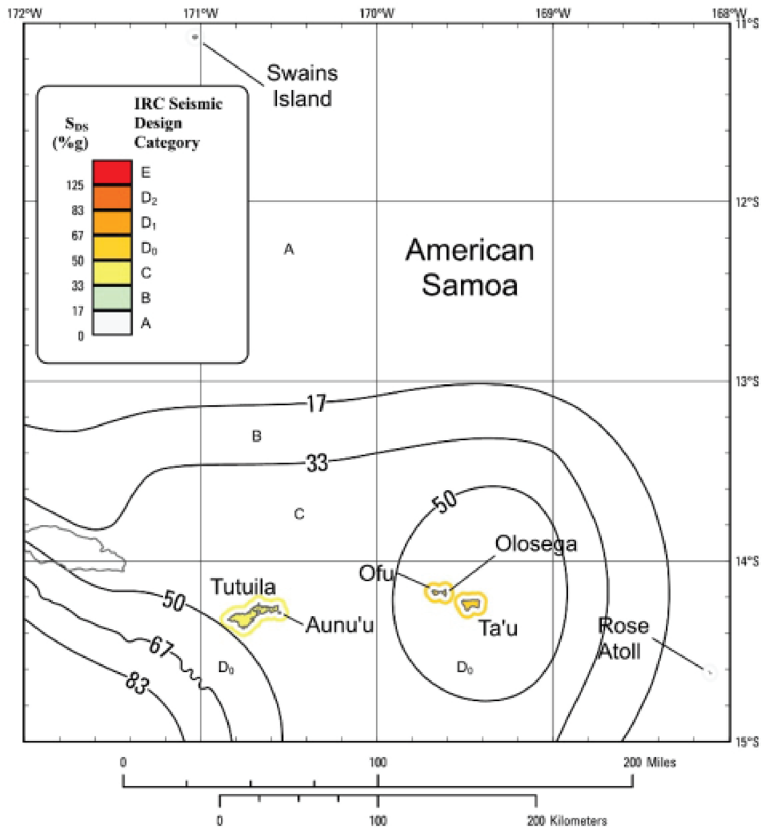
Description: This map of Seismic Design Categories (SDCs) for default site conditions was prepared by the U.S. Geological Survey (USGS) using its 2003 National Seismic Hazard Model for Puerto Rico and the U.S. Virgin Islands, the ASCE/SEI 7-22 Chapter 21 ground motion procedures, the FEMA P-2078 procedures for developing multi-period response spectra at non-conterminous U.S. sites, and the IRC definition of SDC (Table R301.2.2.1.1). As defined in ASCE/SEI 7-22 Chapter 11, the default site conditions used for this map correspond to the most critical ground motions across Site Classes C, CD, and D. More detailed mapping of these SDCs can be resolved using corresponding web tools (e.g., see https://doi.org/10.5066/F7NK3C76).

Figure B-1 **Seismic Design Categories based on 2024 IRC maps (continued).**



Description: This map of Seismic Design Categories (SDCs) for default site conditions was prepared by the U.S. Geological Survey (USGS) using its 2012 National Seismic Hazard Model for Guam and the Northern Mariana Islands, the ASCE/SEI 7-22 Chapter 21 ground motion procedures, the FEMA P-2078 procedures for developing multi-period response spectra at non-conterminous U.S. sites, and the IRC definition of SDC (Table R301.2.2.1.1). As defined in ASCE/SEI 7-22 Chapter 11, the default site conditions used for this map correspond to the most critical ground motions across Site Classes C, CD, and D.

Figure B-1 Seismic Design Categories based on 2024 IRC maps (continued).



Description: This map of Seismic Design Categories (SDCs) for default site conditions was prepared by the U.S. Geological Survey (USGS) using its 2012 National Seismic Hazard Model for American Samoa, the ASCE/SEI 7-22 Chapter 21 ground motion procedures, the FEMA P-2078 procedures for developing multi-period response spectra at non-conterminous U.S. sites, and the IRC definition of SDC (Table R301.2.2.1.1). As defined in ASCE/SEI 7-22 Chapter 11, the default site conditions used for this map correspond to the most critical ground motions across Site Classes C, CD, and D.

Figure B-1 Seismic Design Categories based on 2024 IRC maps (continued).

Appendix C: Design Examples and Cost and Benefit Data

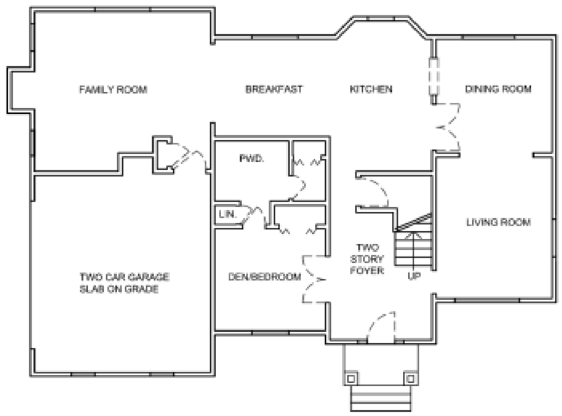
A model single-family detached dwelling and a model townhouse unit, both of wood light-frame construction, have been developed for purposes of this guide. In this appendix, these model buildings are used to illustrate wall bracing conforming to IRC Section R602.10. In addition, computer studies based on the model dwelling are used to demonstrate the benefit and cost of implementing the above-code wall bracing provisions discussed in Chapter 6 of this guide. Section C.1 introduces the model dwelling and townhouse. Section C.2 presents wall the bracing examples. Section C.3 discusses the computer modeling of above-code bracing methods and resulting benefits and costs.

C.1 Model Dwelling and Townhouse

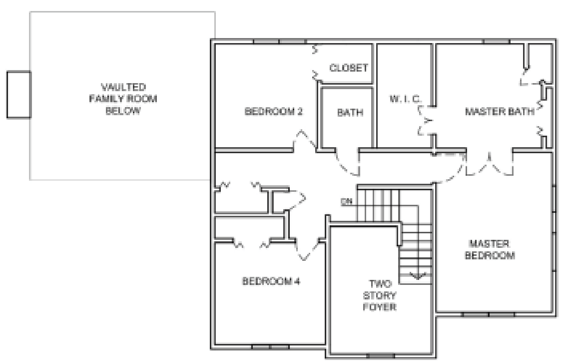
The single-family detached dwelling discussed in this appendix is of two-story wood light-frame construction. The dwelling includes both one-story and two-story portions, three bedrooms, two-and-a-half baths, and an area of approximately 2,500 square feet plus garage. The dwelling design is intended to reflect current configurations for wood light-frame dwelling construction but not necessarily any specific region of the United States. The first and second story floor plans are provided in Figure C-1.

Several variations of the dwelling are considered. These include versions of the dwelling with exterior siding of light siding materials (wood siding, or similar) and a version with full anchored brick veneer. Both slab-on-grade (slab-on-ground) foundation and crawlspace with cripple wall configurations are included. Figure C-2 provides exterior elevations with full anchored brick veneer and a slab-on-grade configuration. Figure C-3 shows the front elevation with the slab-on-grade and crawlspace configurations. Note that in the crawlspace configuration the garage remains slab-on-grade.

The townhouse discussed in this appendix is also of two-story wood light-frame construction. The first and second story floor plans are provided in Figure C-4. The townhouse is considered only with light exterior siding and a slab-on-grade configuration. Front and back elevations of the townhouse are provided in Figure C-5.



FIRST FLOOR



SECOND FLOOR

Figure C-1 **Model dwelling floor plans.**

Figure C-2 **Model dwelling elevations showing full anchored brick veneer and slab-on-grade configuration.**

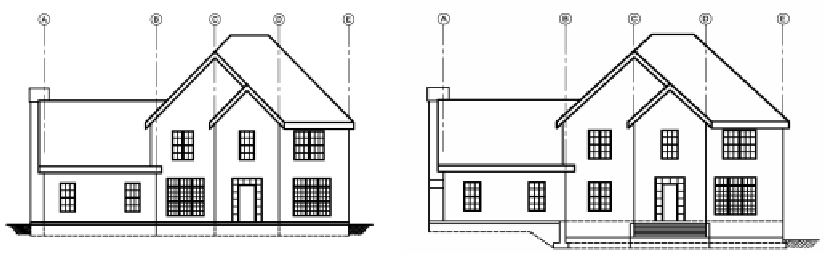
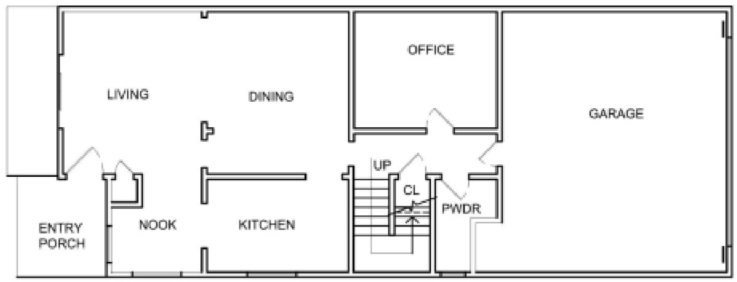
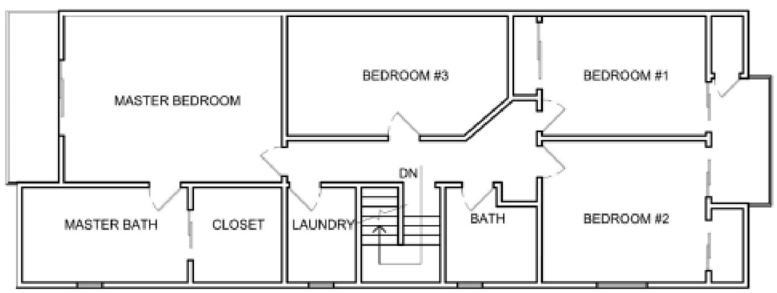


Figure C-3 **Model dwelling front elevation with (left) slab-on-grade configuration and (right) crawlspace with cripple wall configuration.**



FIRST FLOOR



SECOND FLOOR

Figure C-4 Model townhouse floor plans, showing single townhouse unit.

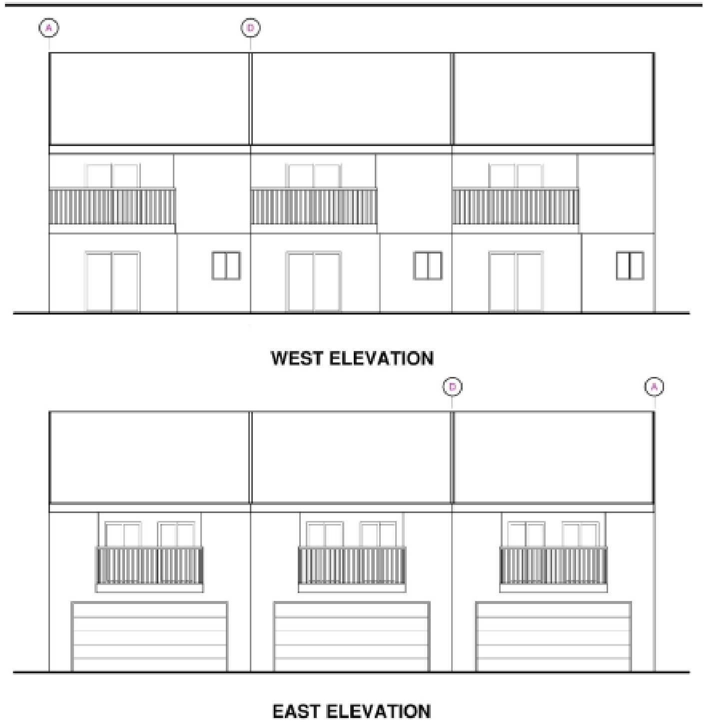


Figure C-5 Model townhouse elevations, showing three-unit townhouse building.

C.2 Design Examples

Examples have been developed to illustrate required earthquake bracing for the model wood light-frame dwelling and townhouse using the provisions of §R602.10 of the 2024 IRC. These are intended to provide examples of what wall bracing requirements entail and to illustrate best practices for communicating wall bracing requirements in building plans. The resulting model dwelling bracing designs are also used in the computer modeling and cost and benefit results discussed in Section C.3.

IRC prescriptive bracing requirements were determined for each combination of base condition, exterior finish, and SDC. Because use of veneer is not permitted on dwellings with cripple walls in

SDC D_0, D_1, and D_2 (IRC §R602.10.6.5.2), the cripple wall configuration with veneer was limited to SDC C.

Because gypsum wallboard is used in almost every U.S. residential building, it was used for the structural bracing wherever possible. Since it would be installed as a finish anyway, its use for bracing results in the least construction cost. Wood structural panel wall bracing was used when gypsum bracing did not meet applicable wall racing requirements or when the BV-WSP bracing was required. Alternative braced wall panels conforming to §R602.10.6.1 were used for the slender walls at the dwelling front and portal frame with hold-down bracing was used at the garage front for slab-on-grade conditions. The alternative braced wall panels require support directly on a continuous foundation; therefore, they could not be used in combination with cripple walls.

Several assumptions regarding the building configuration were made to make the design conform to the IRC bracing requirements.

The first assumption relates to the use of gypsum wallboard bracing (IRC Table R602.10.5, bracing method GB). §R602.10.5 requires that gypsum wallboard braced wall panels applied to one face of a wall be at least 4 feet in width. It was interpreted to mean that a continuous length of full-height wall not less than 4 feet wide would have to be available in order to use this bracing method. Interruption of the 4 foot length by perpendicular walls was interpreted to not be permitted. Where the required bracing length could not be met with 4-foot-or-longer wall segments of full-height wall available, wood structural panels (WSP) were used as bracing. The §R602.10.4.1 allowance to mix different bracing methods in SDCs A, B, and C for detached dwellings was used in some instances. Where gypsum wallboard bracing could be applied to both faces of a wall (such as at interior walls), the required minimum length of full-height sheathing is still 4 feet, as required by the IRC.

For exterior walls, GB bracing was used on the inside of the wall first. If the required length of bracing was not able to be met using only interior gypsum sheathing, the exterior rated gypsum was used as GB bracing on the exterior face as well as regular 1/2-inch gypsum wallboard was used on the interior of the wall line. If the bracing requirements for exterior wall lines could not be met with GB bracing, WSP bracing was used.

The second assumption relates to the bracing requirements used for the model dwelling in SDC C. In accordance with Section R602.10.3, Item 2, when the dwelling is located in SDC C wall bracing is determined using the wind bracing provisions of Tables R602.10.3(1) and R602.10.3(2) only; bracing using the seismic bracing provision of Tables R602.10.3(3) and R602.10.3(4) is not required. For the dwelling in SDC D_0 to D_2 and for the townhouse in SDC C to D_2, both wind and seismic bracing requirements must be checked and the most stringent requirements met.

The resulting bracing configurations are illustrated on bracing plans, as discussed below. The increased bracing length requirements for higher SDCs can be observed to result in reduced allowable door and window opening widths in some cases.

C.2.1 Dwelling in SDC C with Light Siding and Crawlspace

For the guide's model dwelling, Figure C-6a through Figure C-6c provide plans identifying the bracing required when the dwelling is in SDC C, has a light wall siding material (no stone or brick veneer), and has a crawlspace. The following are some highlights of note:

- Wall bracing is only required to be determined for wind loading; the seismic bracing provisions are not applicable. The design is based on a 110 mile-per-hour three-second-gust wind speed. The required bracing is determined using Tables R602.10.3(1) and R602.10.3(2), based on the distance between braced wall lines and on required adjustment factors. Note that the required length of cripple walls is addressed by IRC Section R602.10.10; it uses the length required for bracing in the story above and then makes a number of adjustments.

- For a detached dwelling in SDC C, the braced wall lines (BWLs) are permitted to be up to 60 feet apart. The change from a one- to two-story configuration at Line B, however, results in BWLs on Lines A, B and E, as well as Lines 1 and 8 in the west portion and Line 2 and 5 to 7 in the east portion.

- Because the crawlspace is unconditioned, wood structural panel wall bracing is used at the crawlspace level rather than gypsum board. Per §R602.10.10 the length of WSP bracing applicable for use in the first story is multiplied by 1.15 and 1.4 (Table R602.10.3(2)) to determine length required for bracing the cripple walls. Other Table R602.10.3(2) adjustments apply.

- With the exception of Line A, with the garage door, the first and second stories of the SDC C dwelling are able to be braced with gypsum board.

C.2.2 Dwelling in SDC D_2 with Light Siding and Slab-on-Grade

The bracing requirements for the same dwelling in a slab-on-grade configuration and located in SDC D_2 are shown in Figure C-7a and Figure C-7b. Notice the significant increase in designated wall bracing due to the higher seismic loads expected in SDC D_2.

- Wall bracing is required to be determined for both wind and seismic loading. The required seismic bracing is determined using Tables R602.10.3(3) and R602.10.3(4), based on the distance between braced wall lines and on required adjustment factors. Note that bracing requirements for wind need to also be checked and the most restrictive requirements used.

- For a detached dwelling in SDC D_2, the braced wall lines (BWLs) are permitted to be a maximum of 25 feet apart (with allowance for up to 35 feet with adjustments to the bracing length). In the east-west direction between Lines A and B, the BWLs are provided at no more than 25 feet on center. Between Lines B and E, however, the spacing is increased to approximately 31 feet. Because of this an adjustment factor of 1.4 is used to increase the length of the wall bracing provided.

- In addition to Line A with the garage door, the SDC D_2 dwelling requires alternate braced wall (ABW) panels in a number of first story locations.

C.2.3 Townhouse in SDC C with Light Siding and Slab-on-Grade

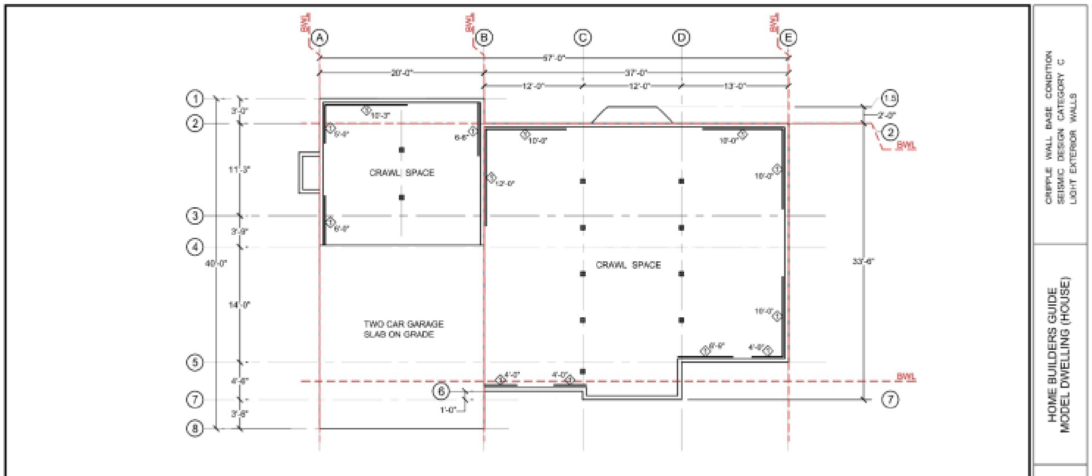
For the guide's model townhouse unit, Figures C-8a and C-8b provide plans identifying the bracing required when the townhouse unit is in SDC C and has a light wall siding material (no stone or brick veneer). The following are some highlights of note:

- Wall bracing is required to be determined for both wind and seismic loading.
- For a townhouse unit in SDC C, the braced wall lines (BWLs) are permitted to be up to 35 feet apart, but this is permitted to be increased to 50 feet provided adjustments are made to the bracing wall length. With the townhouse unit it is easy to meet the 35 foot on-center spacing requirement.
- With the exception of the garage at Line 5, the SDC C townhouse unit is able to be braced with gypsum board.

C.2.4 Townhouse in SDC D_2 with Light Siding and Slab-on-Grade

The bracing requirements for the same townhouse unit in a slab-on-grade configuration and located in SDC D_2 are shown in Figure C-9a and Figure C-9b. Notice the significant increase in designated wall bracing due to the higher seismic loads expected in SDC D_2.

- Wall bracing is required to be determined for both wind and seismic loading.
- For a townhouse unit in SDC D_2, the braced wall lines (BWLs) are permitted to be a maximum of 25 feet apart (with allowance for up to 35 feet with adjustments to the bracing length). With the townhouse unit it is easy to meet the 25 foot on center spacing requirement.
- The second story in both directions and the first story in the long direction of the unit are able to be braced using gypboard. Bracing of the first story in the short direction of the unit requires a combination of wood structural panel bracing (WSP), continuous sheathed wood structural panel bracing (CS-WSP), and portal frames with hold-downs (PFH). This is due to the combination of high seismic loads and limited available length of bracing wall. This is a common issue in townhouse units in the short direction.



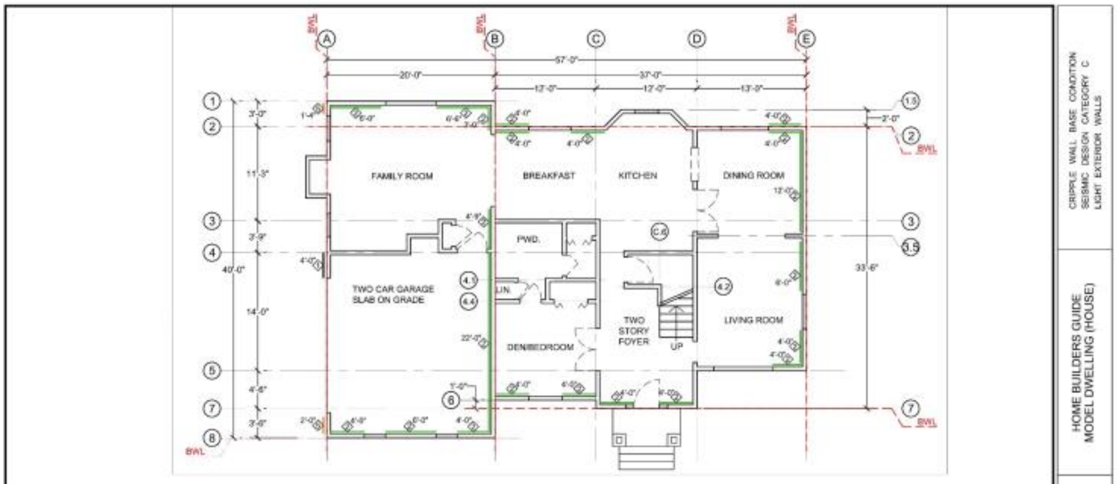
2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE[1] WALL PANEL LENGTH
	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** ⅜" sheathing with 8d common nails 6" edges & 12" field **Interior:** ⅜" sheathing with 8d common nails 6" edges & 6" field	4'-0"
	GB	R702.3.5	½" sheathing with fasteners 7" edges & 7" field	4'-0"
	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
	PFH	R602.10.6.2	⅜" sheathing	1'-4"
	ABW	R602.10.6.1	⅜" sheathing, 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

BASE	SDC	WALLS
CW	C	LIGHT

Figure C-6a **Crawl space level wall bracing plan for model dwelling with light-weight finish material located in SDC C.**



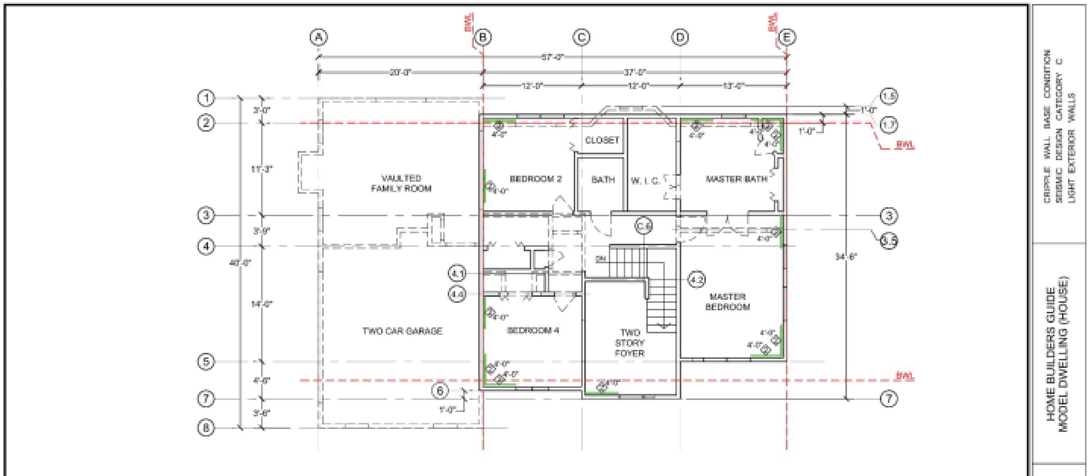
BASE	SDC	WALLS
CW	C	LIGHT

2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE(1) WALL PANEL LENGTH
1	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** ½" sheathing with 8d common nails 6" edges & 12" field **Interior:** ½" sheathing with 6d common nails 6" edges & 6" field	4'-0"
2	GB	R702.3.5	½" sheathing with fasteners 7" edges & 7" field	4'-0"
3	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
4	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
5	PFH	R602.10.6.2	⅜" sheathing	1'-4"
6	ABW	R602.10.6.1	⅜" sheathing, 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

Figure C-6b **First floor level wall bracing plan for model dwelling with light-weight finish material located in SDC C.**



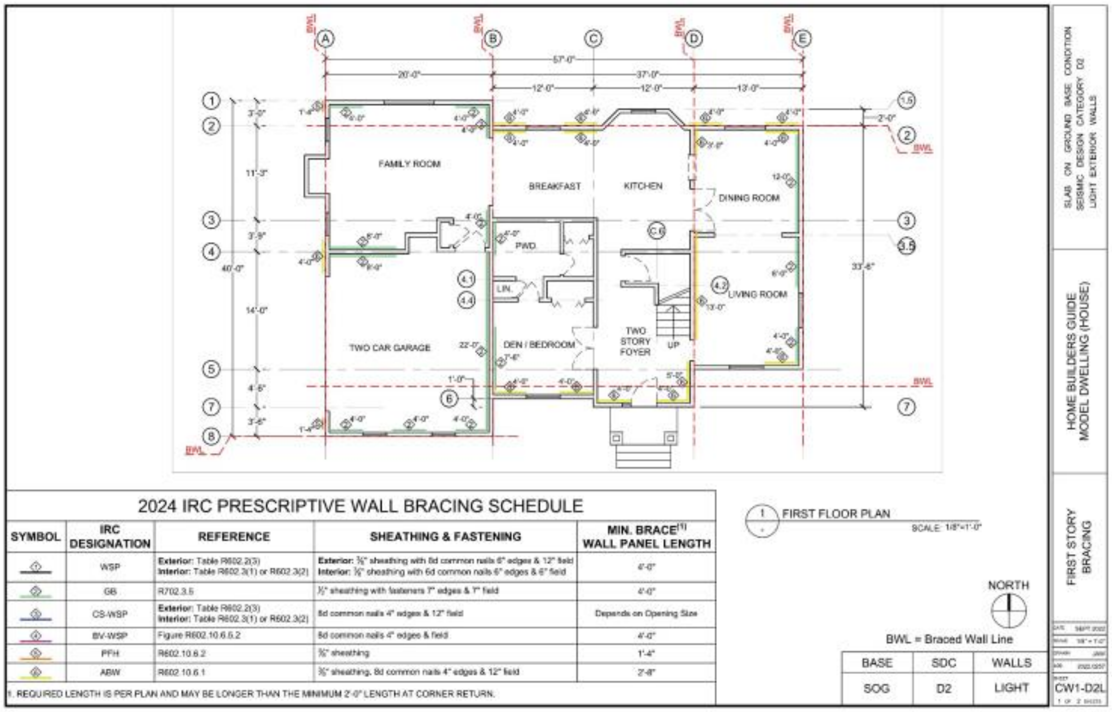
BASE	SDC	WALLS
CW	C	LIGHT

2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE[1] WALL PANEL LENGTH
1	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** 3/8" sheathing with 8d common nails 6" edges & 12" field **Interior:** 3/8" sheathing with 8d common nails 6" edges & 6" field	4'-0"
2	GB	R702.3.5	1/2" sheathing with fasteners 7" edges & 7" field	4'-0"
3	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
4	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
5	PFH	R602.10.6.2	3/8" sheathing	1'-4"
6	ABW	R602.10.6.1	3/8" sheathing, 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

Figure C-6c Second floor level wall bracing plan for model dwelling with light-weight finish material located in SDC C.

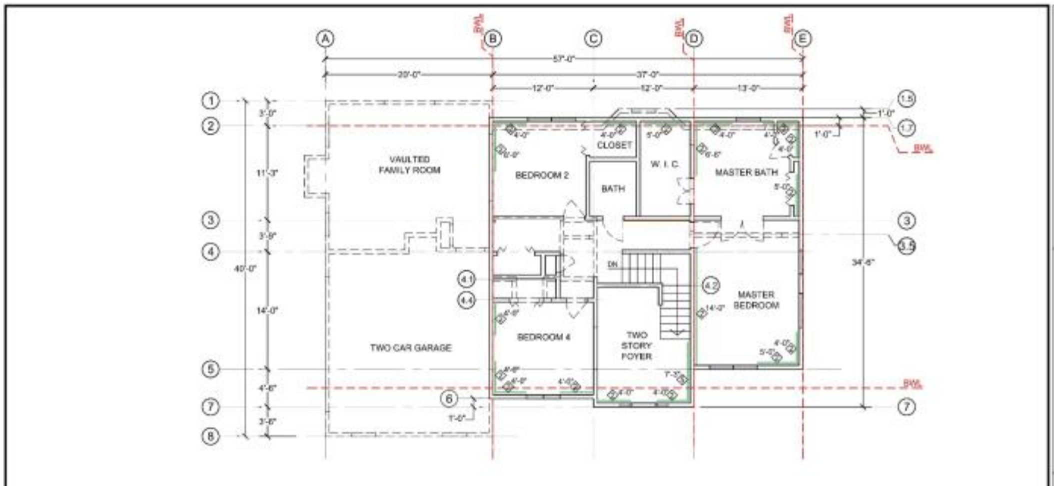


2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE[1] WALL PANEL LENGTH
1	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** 3/8" sheathing with 8d common nails 6" edges & 12" field **Interior:** 3/8" sheathing with 6d common nails 6" edges & 6" field	4'-0"
2	GB	R702.3.5	1/2" sheathing with fasteners 7" edges & 7" field	4'-0"
3	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
4	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
5	PFH	R602.10.6.2	3/8" sheathing	1'-4"
6	ABW	R602.10.6.1	3/8" sheathing, 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

Figure C-7a First floor level wall bracing plan for model dwelling with light-weight finish material located in SDC D_2.



2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE(1) WALL PANEL LENGTH
	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** 3/8" sheathing with 8d common nails 6" edges & 12" field **Interior:** 3/8" sheathing with 6d common nails 6" edges & 6" field	4'-0"
	GB	R702.3.5	1/2" sheathing with fasteners 7" edges & 7" field	4'-0"
	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
	PFH	R602.10.6.2	3/8" sheathing	1'-4"
	ABW	R602.10.6.1	3/8" sheathing 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

1 SECOND FLOOR PLAN
SCALE: 1/8"=1'-0"

NORTH

BWL = Braced Wall Line

BASE	SDC	WALLS
SOG	D2	LIGHT

SLAB ON GROUND BASE CONDITION SEISMIC DESIGN CATEGORY D2 LIGHT EXTERIOR WALLS

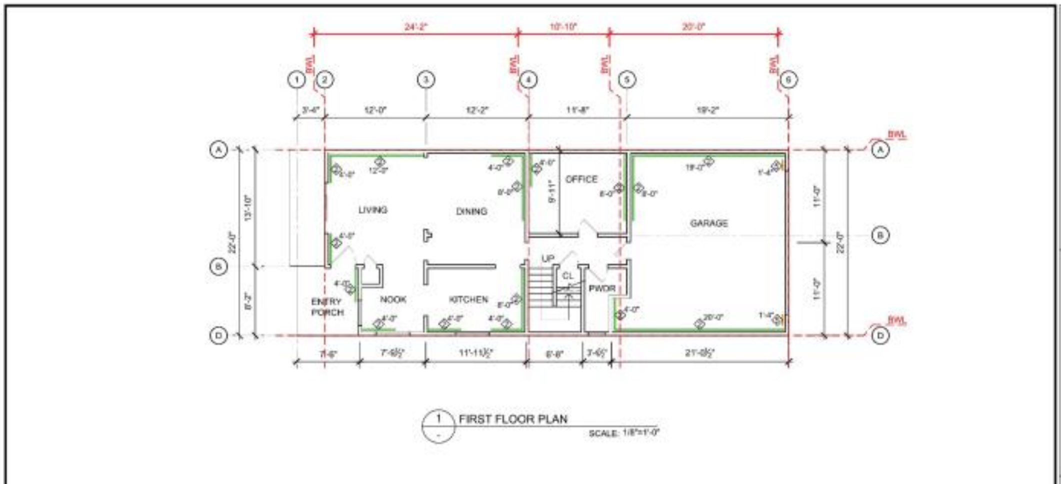
HOME BUILDERS GUIDE MODEL DWELLING (HOUSE)

SECOND STORY BRACING

CW2-D2L

Figure C-7b Second floor level wall bracing plan for model dwelling with light-weight finish material located in SDC D_2.

Homebuilders' Guide to Earthquake-Resistant Design and Construction



2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE(1) WALL PANEL LENGTH
1	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** 7/16" sheathing with 8d common nails 6" edges & 12" field **Interior:** 3/8" sheathing with 6d common nails 6" edges & 6" field	4'-0"
2	GB	R702.3.5	1/2" sheathing with fasteners 7" edges & 7" field	4'-0"
3	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
4	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
5	PFH	R602.10.6.2	7/16" sheathing	1'-4"
6	ABW	R602.10.6.1	7/16" sheathing. 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

BWL = Braced Wall Line

KEY PLAN: UNIT 3, UNIT 2, UNIT 1

NORTH

BASE	SDC	WALLS
SOG	C	LIGHT

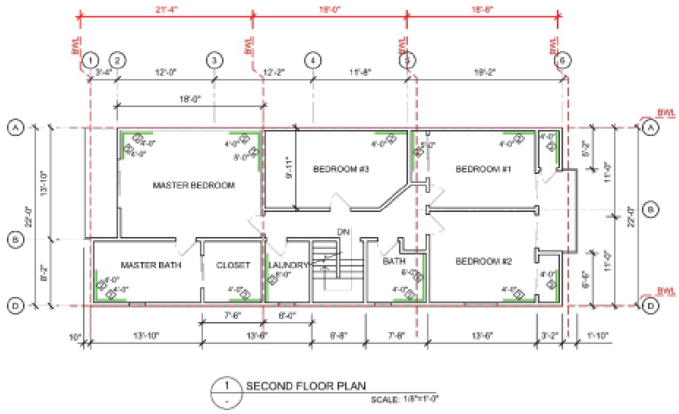
SLAB ON GROUND BASE CONDITION
SEISMIC DESIGN CATEGORY C
LIGHT EXTERIOR WALLS

HOME BUILDERS GUIDE
MODEL TOWNHOUSE

FIRST STORY
BRACING

TH-01-C

Figure C-8a First floor level wall bracing plan for model townhouse unit with light-weight finish material located in **SDC C.**



2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE[1] WALL PANEL LENGTH
	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** 3/8" sheathing with 8d common nails 6" edges & 12" field **Interior:** 3/8" sheathing with 6d common nails 6" edges & 6" field	4'-0"
	GB	R702.3.5	1/2" sheathing with fasteners 7" edges & 7" field	4'-0"
	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
	PFH	R602.10.6.2	7/16" sheathing	1'-4"
	ABW	R602.10.6.1	7/16" sheathing, 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

BWL = Braced Wall Line

UNIT 3
UNIT 2
UNIT 1
KEY PLAN

NORTH

BASE	SDC	WALLS
SOG	C	LIGHT

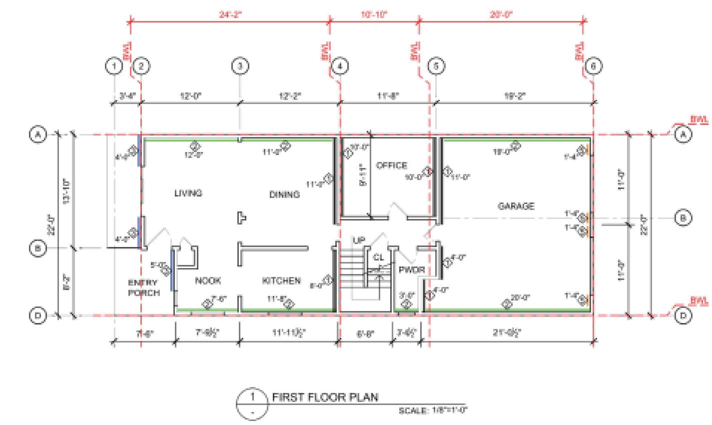
SLAB ON GROUND BASE CONDITION
SEISMIC DESIGN CATEGORY C
LIGHT EXTERIOR WALLS

HOME BUILDERS GUIDE
MODEL TOWNHOUSE

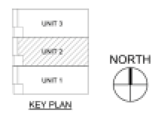
SECOND STORY
BRACING

TH-02-C

Figure C-8b Second floor level wall bracing plan for model townhouse unit with light-weight finish material located in SDC C.



BWL = Braced Wall Line



BASE	SDC	WALLS
SOG	D2	LIGHT

2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE

SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE(1) WALL PANEL LENGTH
1	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** ⅜" sheathing with 8d common nails 6" edges & 12" field **Interior:** ⅜" sheathing with 6d common nails 6" edges & 6" field	4'-0"
2	GB	R702.3.5	½" sheathing with fasteners 7" edges & 7" field	4'-0"
3	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
4	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
5	PFH	R602.10.6.2	⅜" sheathing	1'-4"
6	ABW	R602.10.6.1	⅜" sheathing, 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

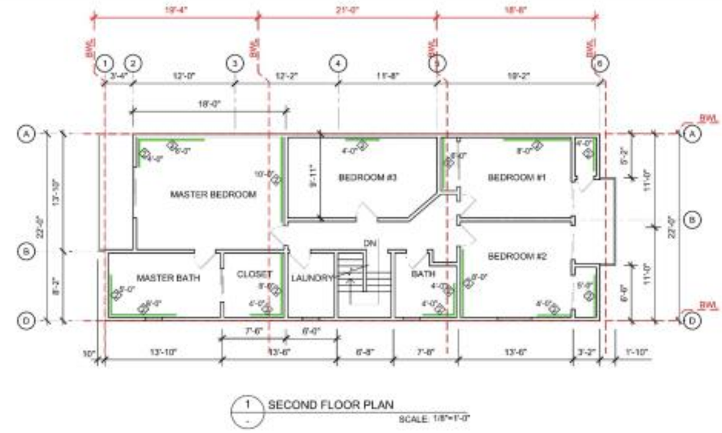
SLAB ON GROUND BASE CONDITION
SEISMIC DESIGN CATEGORY C
LIGHT EXTERIOR WALLS

HOME BUILDERS GUIDE
MODEL TOWNHOUSE

FIRST STORY
BRACING

TH-01-D2

Figure C-9a First floor level wall bracing plan for model townhouse unit with light-weight finish material located in **SDC D₂**.



2024 IRC PRESCRIPTIVE WALL BRACING SCHEDULE				
SYMBOL	IRC DESIGNATION	REFERENCE	SHEATHING & FASTENING	MIN. BRACE[1] WALL PANEL LENGTH
1	WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	**Exterior:** 7/16" sheathing with 8d common nails 6" edges & 12" field **Interior:** 3/8" sheathing with 6d common nails 6" edges & 6" field	4'-0"
2	GB	R702.3.5	1/2" sheathing with fasteners 7" edges & 7" field	4'-0"
3	CS-WSP	**Exterior:** Table R602.2(3) **Interior:** Table R602.3(1) or R602.3(2)	8d common nails 4" edges & 12" field	Depends on Opening Size
4	BV-WSP	Figure R602.10.6.5.2	8d common nails 4" edges & field	4'-0"
5	PFH	R602.10.6.2	7/16" sheathing	1'-4"
6	ABW	R602.10.6.1	7/16" sheathing. 8d common nails 4" edges & 12" field	2'-8"

1. REQUIRED LENGTH IS PER PLAN AND MAY BE LONGER THAN THE MINIMUM 2'-0" LENGTH AT CORNER RETURN.

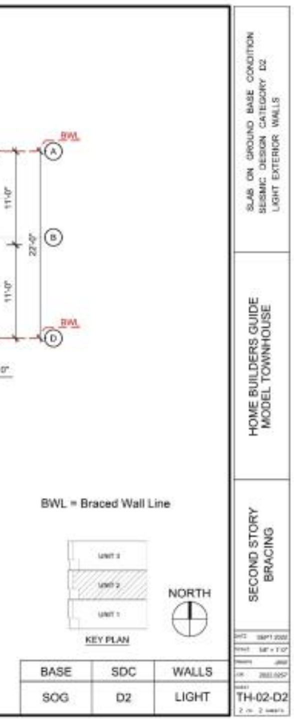


Figure C-9b **Second floor level wall bracing plan for model townhouse unit with light-weight finish material located in SDC D_2**

C.3 Computer Modeling of Above-Code Recommendations and Benefit and Cost Results

To demonstrate the cost effectiveness of implementing the above-code recommendations of this guide, non-linear computer modeling was performed to estimate the earthquake performance of the model dwelling when designed to meet minimum IRC bracing requirements, and then performed again incorporating a series of above-code recommendations. Section C.3 provides details of the computer modeling, the modeling results, and the resulting benefit-to-cost comparison.

The following above-code recommendations were evaluated:

- A wall bracing system in which all exterior walls were fully sheathed (including above doors and above and below window openings) with standard 4-feet by 8-feet oriented strand board (OSB), (Above-Code: Continuous Sheathing, Page 6-16)
- A sheathing panel system combining continuous sheathing as discussed above with having each OSB sheet lapped vertically onto to the rim joist or blocking (1/2 the joist depth) and edge-nailed along the top and bottom with a ¾-inch edge distance as well as nailed to the top and bottom plates of the wall at each floor level. (Above-Code: Sheathing Lap on Rim Joist or Blocking, Page 6-16)
- A wall bracing system with wood structural panel (OSB) sheathing and hold-down devices provided at each end of each full-height braced wall panel, (Above-Code: Hold-down Anchors, Page 6-16)
- A shear wall system with hold-down devices provided at each building corner at both the first and second story. This was seen to have modest benefit.

See Chapter 6 of this guide for further discussion of these above-code measures.

C.3.1 Computer Modeling

In order to evaluate the benefit of incorporating above-code measures, it is necessary to estimate the improvement in earthquake performance that occurs when an above-code measure is used. This was accomplished using state of the art computer modeling in which a recorded earthquake ground motion (ground acceleration) record was run through computer models depicting the model dwelling. In some modeling runs the computer model was configured to meet minimum IRC requirements, while in other modeling runs the above-code requirements were incorporated. From these computer runs, information on the maximum drift (wall in-plane deflection) occurring in each story was extracted as an indicator of the level of anticipated damage to the dwelling. Details of the computer modeling follow.

Nonlinear time-history analysis using the *Timber3D* analysis program (Pang et al., 2012) was chosen as the best available method for estimating force and deformation demands. This program was selected based on analytical studies that were verified against shake table results from the FEMA-funded CUREE-Caltech Woodframe Project and other experimental investigations. The Analysis software has been used for multiple previous large investigations such as FEMA P-1100, *Vulnerability-Based Seismic Assessment and Retrofit of One- and Two-Family Dwellings* (FEMA, 2019a) and FEMA P-2139-2, *Short-Period Building Collapse Performance and Recommendations for Improving Seismic Design, Volume 2 – Study of One-to-Four Story Wood Light-Frame Buildings* (FEMA, 2020d).

Timber3D uses non-linear hysteretic elements to simulate the individual wall segments. The parameters for the hysteretic response were primarily determined using cyclic displacement results from laboratory tests of 8-feet by 8-feet wall specimens incorporated into the modeling curve fitting techniques.

In a few cases, where matching laboratory testing was not available, the program *M-CASHEW* (Pang and Shirazi, 2013) was used to determine the hysteretic parameters. *M-CASHEW* models the sheathing as a very stiff plate, the framing members as standard elastic beam elements and the sheathing-to-framing fasteners as non-linear spring elements that are based upon experimental connection test data. The program then displaces the wall segment through a cyclic displacement pattern to simulate a cyclic test of the wall element. The hysteretic parameters for use in the *Timber3D* analysis were determined using the resulting load-displacement response from the *M-CAHSEW* model. This model was validated by comparing predicted wall performance with experimental test data for a number of tests before it was used to predict wall response for walls without experimental results.

Fifteen individual sets of hysteretic parameters were developed to describe the various types of wall bracing used. Figure C-10 shows a schematic depiction of how *Timber3D* simulates the response of a two-story dwelling and Figure C-11 illustrates the meaning of the hysteretic parameters for a wall segment. A summary of analytical modeling parameter values for each type of wall used is provided in Table C-1. The hysteretic parameters were determined for an 8-foot bracing length, except where otherwise noted in the bracing description. Because widely varying lengths are used in the dwelling, the parameters were scaled linearly for the varying bracing lengths used in the bracing design.

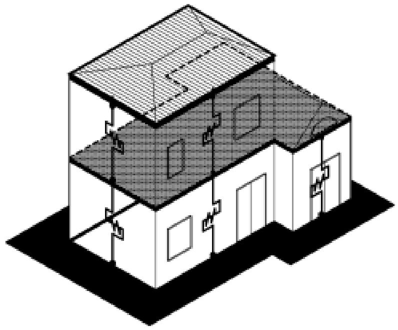


Figure C-10 Schematic of *Timber3D* analysis model of dwelling showing hysteretic springs.

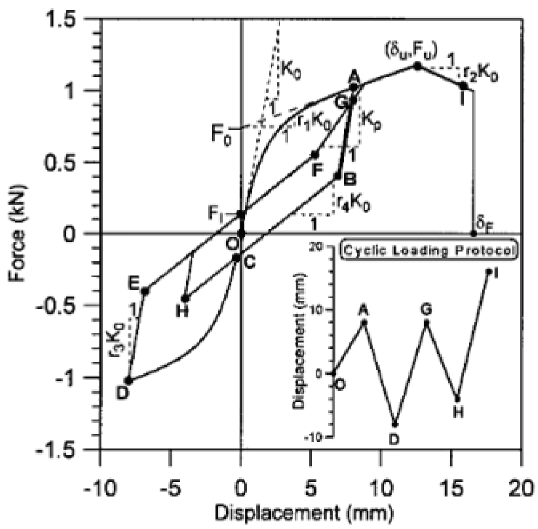


Figure C-11 Hysteretic parameters for model (Pang and Shirazi, 2013).

Hysteretic parameters currently available from laboratory testing of wall components vary based on wall boundary conditions, test set-up, and test protocol. The volume of test data for wood light-frame walls subjected to racking loads is large and the variability is significant if all the possible parameters are considered. Parameters chosen for the analysis of the model dwelling tended towards lower bounds of strength and stiffness. Future analytical studies should consider exploring upper and lower bounds.

Table C-1 Hysteretic Parameters Used for Nonlinear Dynamic Models

Property No.	Use	F_0 (k)	F_1 (k)	δ_u (in)	K_0 (k/in)	r_1	r_2	r_3	r_4	alpha	beta
1	Gypsum Wallboard (lower bound)	0.60	0.17	0.96	2.6	0.17	-0.05	1.45	0.02	0.38	1.09
2	4 ft OSB w/o Tie-Down 6" nail spacing 9 ft high	2.08	0.37	5.59	3.57	0.02	-1.01	1.05	0.04	0.85	1.05
3	4-ft OSB w/o Tie-Down 6" nail spacing 8 ft high	1.95	0.45	3.54	4.66	0.06	-0.08	1.05	0.04	0.85	1.00
4	2 ft × 10 ft Portal Frame 9 ft high	5.84	0.94	4.60	3.69	0.08	-0.78	1.12	0.11	1.00	1.00
5	2 ft high Cripple Wall	1.24	0.35	1.80	5.35	0.05	-0.10	1.01	0.10	0.80	1.10
6	4 ft OSB w/ Tie-Downs 6" nail spacing 9 ft high	1.95	0.45	3.54	4.66	0.06	-0.07	1.05	0.04	0.85	1.09
7	4 ft OSB w/ Tie-Downs 6" nail spacing 8 ft high	1.86	0.42	3.04	6.29	0.06	-0.09	1.08	0.04	0.80	1.10
8	Oversized OSB Panels 6" nail spacing 9 ft high	1.98	0.56	4.07	5.54	0.05	-0.06	1.23	0.06	0.85	1.08

Table C-1 Hysteretic Parameters Used for Nonlinear Dynamic Models (cont.)

Property No.	Use	F_0 (k)	F_1 (k)	δ_u (in)	K_0 (k/in)	r_1	r_2	r_3	r_4	alpha	beta
9	Oversized OSB Panels 6" nail spacing 8 ft high	2.46	1.29	2.61	8.57	0.11	-0.04	1.13	0.08	0.80	1.10
10	4 ft OSB w/o Tie-Down 4" nail spacing 9 ft high	2.30	0.38	2.90	3.78	0.04	0.02	1.33	0.03	0.79	1.07
11	4 ft OSB w/o Tie-Down 4" nail spacing 8 ft high	1.60	0.28	3.50	2.60	0.06	0.70	1.08	0.04	0.85	1.05
12	4 ft OSB w/ Tie-Downs 4" nail spacing 9 ft high	3.06	0.64	3.27	5.86	0.08	-0.09	1.33	0.03	0.79	1.07
13	4 ft OSB w/ Tie-Downs 4" nail spacing 8 ft high	2.69	0.59	2.64	8.19	0.05	-0.06	1.08	0.04	0.85	1.05
14	Oversized OSB Panels 4" nail spacing 9 ft high	2.30	0.38	2.90	3.78	0.04	0.02	1.33	0.03	0.79	1.07
15	Oversized OSB Panels 4" nail spacing 8 ft high	1.60	0.28	3.50	2.6	0.06	0.70	1.08	0.04	0.85	1.05

The computer modeling incorporated the following assumptions:

- The strength and stiffness contributions of exterior wall siding were not included in the computer modeling. This approach was chosen because it would lead to a lower bound on strength and an upper bound on displacement, therefore, conservative estimate of strength and deformation demand. In addition, some exterior wall siding materials are believed to have very little impact on building behavior (e.g., vinyl or fiber-cement siding) and information was not available on the contribution of some other finishes (e.g., brick veneer).

- The modeling was limited to the wall finish materials specifically noted in the bracing plans. In particular, gypsum wallboard indicated as required bracing was included in the model, while gypsum wallboard not indicated as required was not. This again will lead to a lower bound on strength and an upper bound on displacement. The out-of-plane response of the finish materials and walls was not included in the analysis due to the almost non-existence of any test data to use in validating the model predictions.

- The derived hysteretic parameters for the "continuous sheathing" above-code recommendation (addition of the wall sheathing above doors and above and below windows) reflect that this measure was found to notably increase bracing wall stiffness, but to have a negligible effect on bracing wall strength.

- The same method was used to derive hysteretic parameters for the "sheathing lapped on rim joist or blocking" above-code method to account for the continuous sheathing elements above and below the openings in the wall. This sheathing method is essentially the same as the "continuous sheathing" method, with the exception that the wall sheathing is configured to reinforce the connectivity between the wall and the floors and roof of the building.

Due to the above assumptions, and the judgment necessary to select appropriate component testing and derive parameters, the resulting model estimations of drift (deflection) must be qualified as being approximate.

Earthquake demand is represented in the computer modeling using the larger of two orthogonal horizontal acceleration records from Canoga Park for the 1994 Northridge, California, earthquake. This record was chosen because it corresponds well with the code design spectra over a range of building periods. The peak ground acceleration was scaled to be applicable to the spectral acceleration for each SDC, resulting in peak ground accelerations of 0.20g to 0.50g. The ground motion scaling used for this analysis represents the demand used as a basis for IRC compliant design for each SDC. The demand from the risk-targeted maximum considered earthquake (MCE_R) ground motion would be approximately 50% greater.

Detailed assembly weights and building weights were determined for each dwelling configuration. The analysis model spreads the resulting mass uniformly over a rigid plate used to represent each above-ground floor and roof diaphragm. This mass is then applied to the top of the wall elements based upon the tributary area to that particular wall element to simulate the gravity loads that would be resisted by the wall and therefore, the analysis includes the so call P-Δ effects to simulate the added seismic demand induced in the wall as the wall is racked.

C.3.2 Computer Modeling Results

From the nonlinear time-history analysis, peak drifts (wall in-plane deflections) in each of the bracing wall lines were extracted. In order to translate the results of the analysis into an approximation of dwelling performance, the peak drifts were translated to drift ratios (inter-story relative displacement divided by story height and multiplied by 100 to get percent). Three ranges of drift ratio and associated approximate descriptions of building performance (or associated damage) were developed. The choice of range and description of performance are based on component and full-building test results combined with the opinions of those participating in the development of this guide.

The approximate performance categories and corresponding drift ratio ranges are:

- **Minor** damage potential – Less than or equal to 0.5% story drift ratio

  The dwelling is assumed to suffer minor nonstructural damage such as cracked plaster or gypsum wallboard and hopefully would be "green-tagged" (occupancy not limited) by inspectors after an earthquake, which would permit immediate occupancy. Some repairs should still be anticipated.

- **Moderate** damage potential – Greater that 0.5% and less than or equal to 1.5% story drift ratio

  The dwelling is assumed to suffer moderate damage including possible significant damage to finish materials and associated structural damage, but the building is assumed to have some reserve capacity in terms of strength and displacement capacity. The dwelling hopefully would be "green-tagged" or, more likely, "yellow-tagged" (limited occupancy) by inspectors after an earthquake and may or may not be habitable. Significant repairs should be anticipated.

- **Significant** damage potential – Greater than 1.5% story drift ratio

  The dwelling is assumed to have significant structural and finish material damage that could result in its being "red-tagged" (occupancy prohibited) by inspectors after an earthquake. Significant repairs to most components of the building should be anticipated, and it may be more economical to replace the dwelling rather than repair it.

Use of these three categories permits an approximate comparison of the relative performance of different IRC bracing solutions and above-code recommendations. The intent is to give the reader the opportunity to make a decision of whether or not to use the above code recommendations to improve performance and reduce the risk of expensive repairs should the dwelling experience a large earthquake.

Selected results of peak drift values and approximate performance category are provided in Table C-2 for models meeting IRC bracing requirements and Table C-3 with comparison of IRC conforming models and models with above-code measures. In most cases, the drift increased with increased SDC in spite of the bracing requirements also having increased. The approximate performance category often increased from minor to moderate to significant as the SDC went from C to D_2. This resulted from application of a higher demand to a model with only nominal increases in resistance.

Although the building mass increased significantly with the addition of brick veneer, moderate increases and decreases in drift resulted. This is due to the IRC requirement for BV-WSP bracing (wood structural panel sheathing and hold-down devices at the edges of each full-height panel) for veneer in SDCs D_0, D_1 and D_2. The analysis model differentiated between wood structural panel shear walls with and without hold-down devices so the different strength, stiffness, and deformation capacity were accounted for. Because of this, the IRC bracing required for brick veneer was seen to partially compensate for the increased demand.

Table C-2 Selected Results for IRC Bracing Provisions, Slab-on-Grade Base Condition

Walls	SDC	First Story Peak Drift (%) (inch horiz./inch wall height) and Approximate Performance
Light	C	0.32 Minor
Light	D_0	0.66 Moderate
Light	D_1	2.02 Significant
Light	D_2	2.29 Significant
Veneer	C	0.43 Minor
Veneer	D_0	1.10 Moderate
Veneer	D_1	1.91 Significant
Veneer	D_2	2.63 Significant

Table C-3 Selected Results for Above-code Measures, Slab-on-Grade Base Condition and Light Walls

Above-Code Recommendation	Walls	Change in Drift Ratio and Change in Approximate Performance Category Resulting from Above-Code Measures			
		SDC C	SDC D_0	SDC D_1	SDC D_2
Continuous Sheathing	Light	0.32 to 0.26 Minor to Minor	0.66 to 0.36 Moderate to Minor	2.02 to 1.39 Significant to Moderate	2.29 to 1.46 Significant to Moderate
Hold-down Anchors in Corners Only	Light	N/A[1]	0.66 to 0.62 Moderate to Moderate	2.02 to 1.56 Significant to Significant	N/A[1]
Hold-down Anchors on All Full-Height Wall Segments	Light	N/A[1]	0.66 to 0.56 Moderate to Moderate	2.02 to 1.56 Significant to Moderate	2.29 to 1.84 Significant to Significant
Sheathing Lap on Rim Joist or Blocking	Light	N/A[1]	0.66 to 0.26 Moderate to Minor	2.02 to 1.05 Significant to Moderate	2.29 to 1.35 Significant to Moderate

[1] The above-code option for hold-down added to the building at the corners or at full-height wall panels was not applicable to conditions where the entire lateral resistance is provided by gypsum wallboard. This is because hold-downs have almost no effect on the lateral strength of gypsum wallboard sheathed walls due to the brittle nature of the sheathing and the resulting failure mode of the sheathing fasteners.

The above-code measures were applied to the dwelling analytical models with light wall siding and the slab-on-grade base condition. The measures were seen to always reduce the building drift ratio and in many instances improve the approximate performance category. In terms of effectiveness in reducing the drift ratio and improving the approximate performance category, the above code measures can be ranked from most to least effective as follows:

- "Sheathing lap on rim joist or blocking" provided the largest reduction in drift ratio and improved the approximate performance category for SDCs D_0, D_1 and D_2.
- "Continuous sheathing" provided the next largest reduction in drift ratio and improved the approximate performance category for SDCs D_0, D_1 and D_2.
- "Hold-down anchors on all full-height wall segments" provided modest reduction in drift ratio and only improved the approximate performance category for only SDC D_1.
- "Hold-down anchors in corners only" provided the least reduction in drift ratio and did not improve the approximate performance category in the two investigated SDCs (D_0 and D_1).

C.3.3 Benefit and Cost Results

The cost of implementing each above-code measure was estimated in terms of the added cost of the above-code measure as a percentage of the overall construction cost for dwelling (excluding cost of the property). The overall cost of construction for the dwelling meeting IRC requirements was estimated to be $300 a square foot, for a total cost of $820,000. This was determined based on input from construction professionals and is intended to represent an average cost per square foot mid-range housing in the larger San Francisco Bay Area. A contractor estimated the added cost to implement each of the above-code measures (based on the SDC D_1 bracing plan), also for the larger San Francisco Bay Area.

The following are the rankings of effectiveness of the above-code measures as previously discussed, with the estimated cost to implement each above-code measure expressed as a percentage of the construction cost:

- "Sheathing lap on rim joist or blocking" combined with continuous sheathing provided the largest reduction in drift ratio and improved the approximate performance category by one for SDCs D_0, D_1 and D_2. Cost: Approximately 2% of construction cost.
- "Continuous sheathing" provided the next largest reduction in drift ratio and improved the approximate performance category by one for SDCs D_0, D_1 and D_2. Cost: Approximately 2% of construction cost.
- "Hold-down anchors on all full-height wall segments" provided modest reduction in drift ratio and only improved the approximate performance category by one for SDC D_1. Cost: Approximately 4% of construction cost.
- "Hold-down anchors in corners only" provided the least reduction in drift ratio and did not improve the approximate performance category in the two investigated SDCs (D_0 and D_1). Cost: Less that 1% of construction cost.

This demonstrates that two above-code measures are available that provide the largest increase in performance at moderate cost. Use of the above-code measures in combination is thought to have a cumulative effect in improving performance and so is encouraged.

Appendix D: Other Hazards

Although this guide has been written primarily to address earthquake hazard to a dwelling or townhouse, homebuilders and homeowners need to be aware of many other natural hazards. These hazards can also affect how a dwelling or townhouse should be designed and constructed. This includes selection of the site or lot on which the dwelling or townhouse will be constructed. The following provides a short summary of other hazards that might be considered.

D.1 Wind

Wind acts on a dwelling by imparting horizontal loads similar to those of an earthquake, and design of a dwelling or townhouse might be controlled by either wind or earthquake loading. The earthquake design and construction criteria contained in the IRC and further detailed in this guide may also improve a dwelling or townhouse's resistance to wind loads. However, wind loads also act on the home's cladding, walls, roof framing, and roof coverings, and the wind provisions of the IRC should be followed carefully (due to the differences in the load path for wind versus earthquake loads). While the IRC wind provisions are believed to be effective for the range of wind speeds addressed, additional prescriptive standards are available including the *Wood Frame Construction Manual for One- and Two-Family Dwellings* (AWC, 2024) and the *Standard for Residential Construction in High-Wind Regions* (ICC 600) (ICC, 2020a). For engineers and architects, the IBC and ASCE/SEI 7 provide engineering criteria for design.

D.2 Tornado

Provisions for engineered design of structures for tornado loads are included in the IBC through adoption of ASCE/SEI 7. Design for tornados is, however, required only for structures that are assigned to Risk Category III and IV (i.e., those representing a substantial hazard to life or designated as essential facilities) and located in tornado-prone regions. Tornado design is not currently required by the IBC or IRC for typical residential structures. Those interested in providing design to resist tornados should pursue an engineered design. Installation of a storm shelter provides a practical life-safety alternative to design of the dwelling for tornado loading. Information on storm shelters can be found in *ICC/NSSA Standard for Design and Construction of Storm Shelters* (ICC 500) (ICC, 2020b) and FEMA P-361, *Safe Rooms for Tornadoes and Hurricanes* (FEMA, 2021a).

D.3 Flooding

Homebuilders and potential homeowners should determine whether a proposed dwelling is located in a flood hazard area. If there is any doubt in this regard, the local building department or floodplain management office should be consulted. As part of the National Flood Insurance Program (NFIP), FEMA publishes local flood hazard maps that delineate the extent of the flood hazard, using the Base Flood Elevation (BFE), which is determined at the specific site or lot. As part of its agreement to participate in the NFIP in order to make federally backed flood insurance available to its residents, a community adopts and enforces a series of flood design and construction requirements. Generally, these requirements mean that a dwelling located in a flood hazard area must have its lowest floor and utilities elevated to or above the BFE.

FEMA has available a series of publications on protecting dwellings from flood damage. For guidance on the flood resistant construction provisions of the IRC, the reader is referred to the document *Reducing Flood Losses Through the International Codes: Meeting the Requirements of the National Flood Insurance Program* (FEMA, 2019c). For engineers and architects, the IBC, ASCE/SEI 7, and ASCE/SEI 24, *Flood Resistant Design and Construction* (ASCE, 2014) provide engineering criteria for design.

D.4 Coastal Hazards

Homesites along the coast are subject to two different hazards from hurricanes and other coastal storms such as Northeasters — high winds and flooding from storm surge. Storm surge is caused by the storm's low pressure and winds pushing water onto land. In Hurricane Katrina, storm surge heights of almost 30 feet were measured along the coast of Mississippi. Coastal flood hazard maps similar to the flood maps described above are prepared by FEMA; however, the coastal flood hazard area is divided into two zones: (1) the coastal high hazard zone, or "V-Zone," which has significant wave heights, and (2) the coastal flood hazard zone, or "Coastal A-Zone," which does not.

Coastal dwellings must be designed and built to withstand both wind and storm surge loads. This requires elevating the dwelling or townhouse above mapped storm surge elevations and ensuring that the dwelling or townhouse can resist the loads associated with hurricane force winds. Further, since storm surge waters can flow with considerable velocity which generates substantial hydrodynamic loads on the area below the elevated floor, this area must be built to allow water to flow through without placing additional loads on the structure. For this reason, any walls below the lowest floor must be made of "breakaway" construction.

D.5 Tsunami

Provisions for engineered design of structures for tsunami loads are included in the IBC through adoption of the 2022 edition of ASCE/SEI 7. Design for tsunamis is, however, limited to critical and essential facilities such as schools, hospitals, and police and fire stations (when designated as Tsunami Risk Category III or IV), and is not currently being required for residential structures, nor are there prescriptive design measures as used in the IRC. Those interested in tsunami design should pursue an engineered design. Alternatively, a homebuilder or potential homeowner could determine whether the dwelling lot is located in a tsunami hazard zone and if so consider relocating out of the zone. Because it is cost prohibitive to design a typical residential structure to withstand tsunami loads, communities instead may rely on tsunami warning systems, which alert residents to evacuate to high ground or to a vertical evacuation refuge before tsunami waves arrive.

REFERENCES

1) **FEMA P-232, " Homebuilders' Guide to Earthquake-Resistant Design and Construction ", Federal Emergency Management Agency, Washington, D.C., Sept. 2024.**