CHAPTER 6

Lateral Resistance to Wind and Earthquake

6.1 General

The objectives in designing a building's lateral resistance to wind and earthquake forces are

- to provide a system of shear walls, diaphragms, and interconnections to transfer lateral loads and overturning forces to the foundation;
- to prevent building collapse in extreme wind and seismic events; and
- to provide adequate stiffness to the structure for service loads experienced in moderate wind and seismic events.

In light-frame construction, the lateral force-resisting system (LFRS) comprises shear walls, diaphragms, and their interconnections to form a whole-building system that may behave differently than the sum of its individual parts. In fact, shear walls and diaphragms are themselves subassemblies of many parts and connections. Thus, designing an efficient LFRS system is perhaps the greatest challenge in the structural design of light-frame buildings. In part, the challenge results from the lack of any single design methodology or theory that provides reasonable predictions of complex, large-scale system behavior in conventionally built or engineered light-frame buildings.

Designer judgment is a crucial factor that comes into play when the designer selects how the building is to be analyzed and to what extent the analysis should be assumed to be a correct representation of the true design problem. Designer judgment is essential in the early stages of design because the analytic methods and assumptions used to evaluate the lateral resistance of light-frame buildings are not in themselves correct representations of the problem. They are analogies that are sometimes reasonable but at other times depart significantly from reason and actual system testing or field experience.

This chapter focuses on methods for evaluating the lateral resistance of individual subassemblies of the LFRS (i.e., shear walls and diaphragms) and the response of the whole building to lateral loads (i.e., load distribution). Traditional design approaches as well as innovative methods, such as the *perforated shear wall design method*, are integrated into the designer's “tool box.” While the code-approved methods have generally “worked,” there is considerable opportunity for improvement and optimization. Therefore, the information and design examples presented in this chapter provide a useful guide and resource that supplement existing building code provisions. More important, the chapter is aimed at fostering a better understanding of the role of analysis versus judgment and promoting more efficient design in the form of alternative methods.

The lateral design of light-frame buildings is not a simple endeavor that provides “exact” solutions. By the very nature of the LFRS, the real behavior of light-frame buildings is highly dependent on the performance of building systems, including the interactions of structural and nonstructural components. For example, the nonstructural components in conventional housing (i.e., sidings, interior finishes, interior partition walls, and even windows and trim) can account for more than 50 percent of a building’s lateral resistance. Yet, the contribution of these components is not considered as part of the “designed” LFRS for lack of appropriate design tools and building code provisions that may prohibit such considerations. In addition, the need for simplified design methods inevitably leads to a trade-off–analytical simplicity for design efficiency.

In seismic design, factors that translate into better performance may not always be obvious. The designer should become accustomed to thinking in terms of the relative stiffness of components that make up the whole building. Important, too, is an understanding of the inelastic (nonlinear), nonrigid body behavior of wood-framed systems that affect the optimization of strength, stiffness, dampening, and ductility. In this context, the concept that more strength is better is insupportable without considering the impact on other important factors. Many factors relate to a structural system’s deformation capability and ability to absorb and safely dissipate energy from abusive cyclic motion in a seismic event. The intricate interrelationship of these several factors is difficult to predict with available seismic design approaches.

For example, the basis for the seismic response modifier R is a subjective representation of the behavior of a given structure or structural system in a seismic event (refer to Chapter 3). In a sense, it bears evidence of the inclusion of “fudge factors” in engineering science for reason of necessity (not of preference) in attempting to mimic reality. It is not necessarily surprising, then, that the amount of wall bracing in conventional homes shows no apparent correlation with the damage levels experienced in seismic events (HUD, 1999). Similarly, the near-field damage to conventional homes in the Northridge Earthquake did not correlate with the magnitude of response spectral ground accelerations in the short period range (HUD, 1999). The short-period spectral response acceleration, it will be recalled, is the primary ground motion parameter used in the design of most low-rise and light-frame buildings (refer to Chapter 3).

The apparent lack of correlation between design theory and actual outcome points to the tremendous uncertainty in existing seismic design methods for light-frame structures. In essence, a designer's compliance with accepted seismic design provisions may not necessarily be a good indication of actual performance in a major seismic event. This statement may be somewhat unsettling but is worthy of mention. For wind design, the problem is not as severe in that the lateral load can be more easily treated as a static load, with system response primarily a matter of determining lateral capacity without complicating inertial effects, at least for small light-frame buildings.

In conclusion, the designer should have a reasonable knowledge of the underpinnings of current LFRS design approaches (including their uncertainties and limitations). However, many designers do not have the opportunity to become familiar with the experience gained from testing whole buildings or assemblies. Design provisions are generally based on an "element-based" approach to engineering and usually provide little guidance on the performance of the various elements as assembled in a real building. Therefore, the next section presents a brief overview of several whole-house lateral load tests.

6.2 Overview of Whole-Building Tests

A growing number of full-scale tests of houses have been conducted to gain insight into actual system strength and structural behavior. Several researchers have recently summarized the body of research; the highlights follow (Thurston, 1994; NIST, 1998).

One whole-house test program investigated the lateral stiffness and natural frequency of a production-built home (Yokel, Hsi, and Somes, 1973). The study applied a design load simulating a uniform wind pressure of 25 psf to a conventionally built home: a two-story, split-foyer dwelling with a fairly typical floor plan. The maximum deflection of the building was only 0.04 inches and the residual deflection about 0.003 inches. The natural frequency and dampening of the building were 9 hz (0.11 s natural period) and 6 percent, respectively. The testing was nondestructive such that the investigation yielded no information on "postyielding" behavior; however, the performance was good for the nominal lateral design loads under consideration.

Another whole-house test applied transverse loads without uplift to a wood-framed house. Failure did not occur until the lateral load reached the "equivalent" of a 220 mph wind event without inclusion of uplift loads (Tuomi and McCutcheon, 1974). The house was fully sheathed with 3/8-inch plywood panels, and the number of openings was somewhat fewer than would be expected for a typical home (at least on the street-facing side). The failure took the form of slippage at the floor connection to the foundation sill plate (i.e., there was only one 16d toenail at the end of each joist, and the band joist was not connected to the sill). The connection was somewhat less than what is now required in the United States for conventional residential construction (ICC, 1998). The racking stiffness of the walls nearly doubled from that experienced before the addition of the roof framing. In addition, the simple 2x4 wood trusses were able to carry a gravity load of 135 psf–more than three times the design load of 40 psf. However, it is important to note that combined uplift and lateral load, as would be expected in high-wind conditions, was not tested. Further, the test house was relatively small and "boxy" in comparison to modern homes.

Many whole-house tests have been conducted in Australia. In one series of whole-house tests, destructive testing has shown that conventional residential construction (only slightly different from that in the United States) was able to withstand 2.4 times its intended design wind load (corresponding to a 115 mph wind speed) without failure of the structure (Reardon and Henderson, 1996). The test house had typical openings for a garage, doors, and windows, and no special wind-resistant detailing. The tests applied a simultaneous roof uplift load of 1.2 times the total lateral load. The drift in the two-story section was 3 mm at the maximum applied load while the drift in the open one-story section (i.e., no interior walls) was 3 mm at the design load and 20 mm at the maximum applied load.

Again in Australia, a house with fiber cement exterior cladding and plasterboard interior finishes was tested to 4.75 times its "design" lateral load capacity (Boughton and Reardon, 1984). The walls were restrained with tie rods to resist wind uplift loads as required in Australia's typhoon-prone regions. The roof and ceiling diaphragm was found to be stiff; in fact, the diaphragm rigidly distributed the lateral loads to the walls. The tests suggested that the house had sufficient capacity to resist a design wind speed of 65 m/s (145 mph).

Yet another Australian test of a whole house found that the addition of interior ceiling finishes reduced the deflection (i.e., drift) of one wall line by 75 percent (Reardon, 1988; Reardon, 1989). When cornice trim was added to cover or dress the wall-ceiling joint, the deflection of the same wall was reduced by another 60 percent (roughly 16 percent of the original deflection). The tests were conducted at relatively low load levels to determine the impact of various nonstructural components on load distribution and stiffness.

Recently, several whole-building and assembly tests in the United States have been conducted to develop and validate sophisticated finite-element computer models (Kasal, Leichti, and Itani, 1994). Despite some advances in developing computer models as research tools, the formulation of a simplified methodology for application by designers lags behind. Moreover, the computer models tend to be time-intensive to operate and require detailed input for material and connection parameters that would not normally be available to typical designers. Given the complexity of system behavior, the models are often not generally applicable and require "recalibration" whenever new systems or materials are specified.

In England, researchers have taken a somewhat different approach by moving directly from empirical system data to a simplified design methodology, at least for shear walls (Griffiths and Wickens, 1996). This approach applies various "system factors" to basic shear wall design values to obtain a value for a specific application. System factors account for material effects in various wall assemblies, wall configuration effects (i.e., number of openings in the wall), and interaction effects with the whole building. One factor even accounts for the fact that shear loads on wood-framed shear walls in a full brick-veneered building are reduced by as much as 45 percent for wind loads, assuming, of course, that the brick veneer is properly installed and detailed to resist wind pressures.

More recently, whole-building tests have been conducted in Japan (and to a lesser degree in the United States) by using large-scale shake tables to study the inertial response of whole, light-frame buildings (Yasumura, 1999). The tests have demonstrated whole-building stiffness of about twice that experienced by walls tested independently. The results are reasonably consistent with those reported above. Apparently, many whole-building tests have been conducted in Japan, but the associated reports are available only in Japanese (Thurston, 1994).

The growing body of whole-building test data will likely improve the understanding of the actual performance of light-frame structures in seismic events to the extent that the test programs are able to replicate actual conditions. Actual performance must also be inferred from anecdotal experience or, preferably, from experimentally designed studies of buildings experiencing major seismic or wind events (refer to Chapter 1).

6.3 LFRS Design Steps and Terminology

The lateral force resisting system (LFRS) of a home is the "whole house" including practically all structural and non-structural components. To enable a rational and tenable design analysis, however, the complex structural system of a light-frame house is usually subjected to many simplifying assumptions; refer to Chapter 2. The steps required for thoroughly designing a building's LFRS are outlined below in typical order of consideration:

1. Determine a building's architectural design, including layout of walls and floors (usually pre-determined).
2. Calculate the lateral loads on the structure resulting from wind and/or seismic conditions (refer to Chapter 3).
3. Distribute shear loads to the LFRS (wall, floor, and roof systems) based on one of the design approaches described later in this chapter (refer to Section 6.4.1).
4. Determine *shear wall* and *diaphragm* assembly requirements for the various LFRS components (sheathing thickness, fastening schedule, etc.) to resist the stresses resulting from the applied lateral forces (refer to Section 6.5).
5. Design the *hold-down restraints* required to resist overturning forces generated by lateral loads applied to the vertical components of the LFRS (i.e., shear walls).
6. Determine interconnection requirements to transfer shear between the LFRS components (i.e., roof, walls, floors, and foundation).
7. Evaluate *chords* and *collectors* (or *drag struts*) for adequate capacity and for situations requiring special detailing such as splices.

It should be noted that, depending on the method of distributing shear loads (refer to Section 6.4.1), Step 3 may be considered a preliminary design step. If, in fact, loads are distributed according to stiffness in Step 3, then the LFRS must already be defined; therefore, the above sequence can become iterative between Steps 3 and 4. A designer need not feel compelled to go to such a level of complexity (i.e., using a stiffness-based force distribution) in designing a simple home, but the decision becomes less intuitive with increasing plan complexity.

The above list of design steps introduced several terms that are defined below.

Horizontal diaphragms are assemblies such as the roof and floors that act as “deep beams” by collecting and transferring lateral forces to the *shear walls*, which are the vertical components of the LFRS. The diaphragm is analogous to a horizontal, simply supported beam laid flatwise; a shear wall is analogous to a vertical, fixed-end, cantilevered beam. Chapter 2 discussed the function of the LFRS and the lateral load path. The reader is referred to that chapter for a conceptual overview of the LFRS and to Chapter 3 for methodologies to calculate lateral loads resulting from wind and earthquake forces.

Chords are the members (or a system of members) that form a “flange” to resist the tension and compression forces generated by the “beam” action of a diaphragm or shear wall. As shown in Figure 6.1, the chord members in shear walls and diaphragms are different members, but they serve the same purpose in the beam analogy. A *collector* or *drag strut*, which is usually a system of members in light-frame buildings, “collects” and transfers loads by tension or compression to the shear resisting segments of a wall line (see Figure 6.2a).

In typical light-frame homes, special design of chord members for floor diaphragms may involve some modest detailing of splices at the diaphragm boundary (i.e., joints in the band joists). If adequate connection is made between the band joist and the wall top plate, then the diaphragm sheathing, band joists, and wall framing function as a “composite” chord in resisting the chord forces. Thus, the diaphragm chord is usually integral with the collectors or drag struts in shear walls. Given that the collectors on shear walls often perform a dual role as a chord on a floor or roof diaphragm boundary, the designer needs only to verify that the two systems are reasonably interconnected along their boundary, thus ensuring composite action as well as direct shear transfer (i.e., slip resistance) from the diaphragm to the wall. As shown in Figure 6.2b, the failure plane of a typical “composite” collector or diaphragm chord can involve many members and their interconnections.

For shear walls in typical light-frame buildings, tension and compression forces on shear wall chords are usually considered. In particular, the connection of hold-downs to shear wall chords should be carefully evaluated with respect to the transfer of tension forces to the structure below. Tension forces result from the overturning action (i.e., overturning moment) caused by the lateral shear load on the shear wall. In some cases, the chord may be required to be a thicker member to allow for an adequate hold-down connection or to withstand the tension and compression forces presumed by the beam analogy. Fortunately, most chords in light-frame shear walls are located at the ends of walls or adjacent to openings where multiple studs are already required for reasons of constructability and gravity load resistance (see cross-section "B" in Figure 6.1).

FIGURE 6.1 ***Chords in Shear Walls and Horizontal Diaphragms Using the "Deep Beam" Analogy***

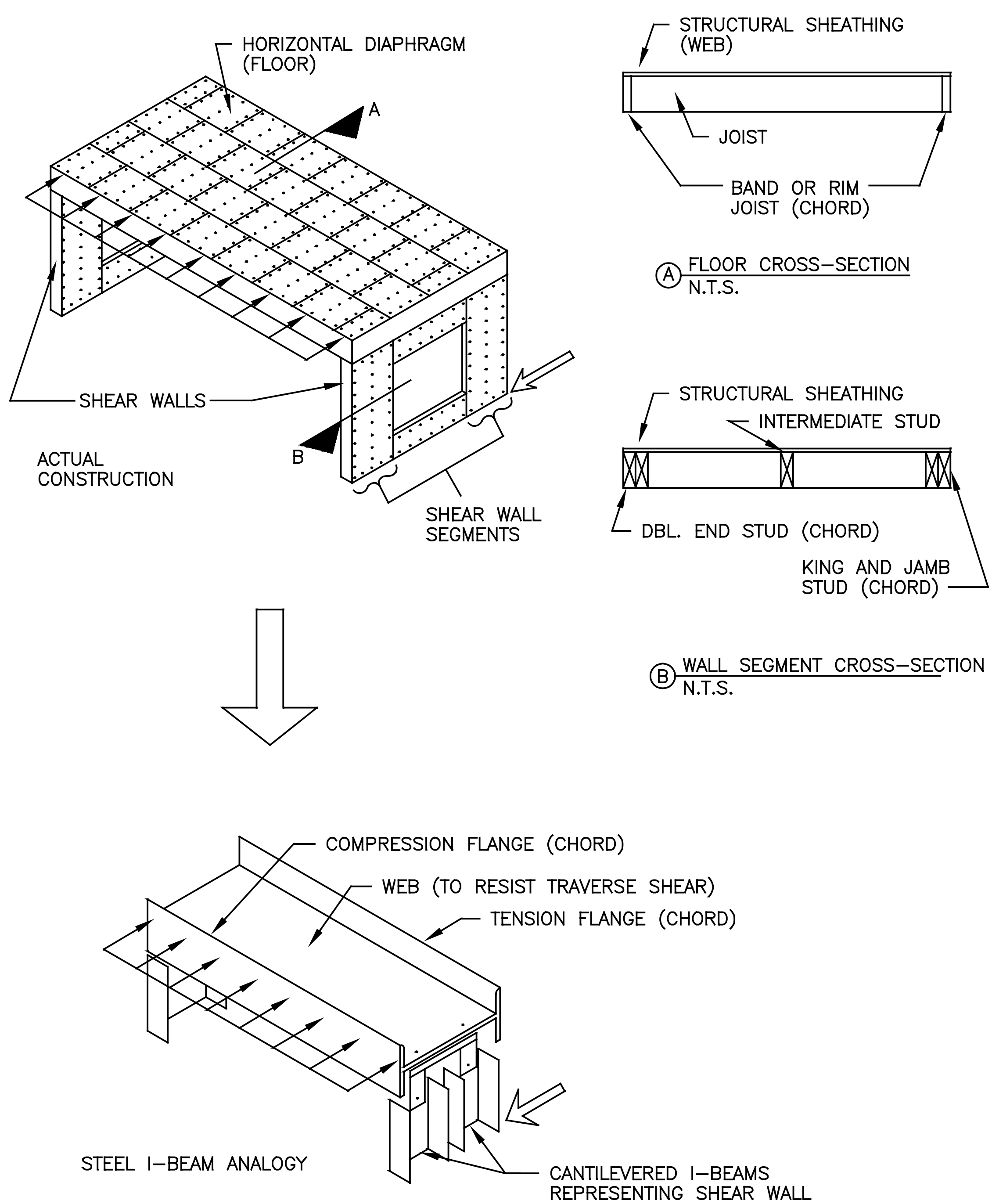
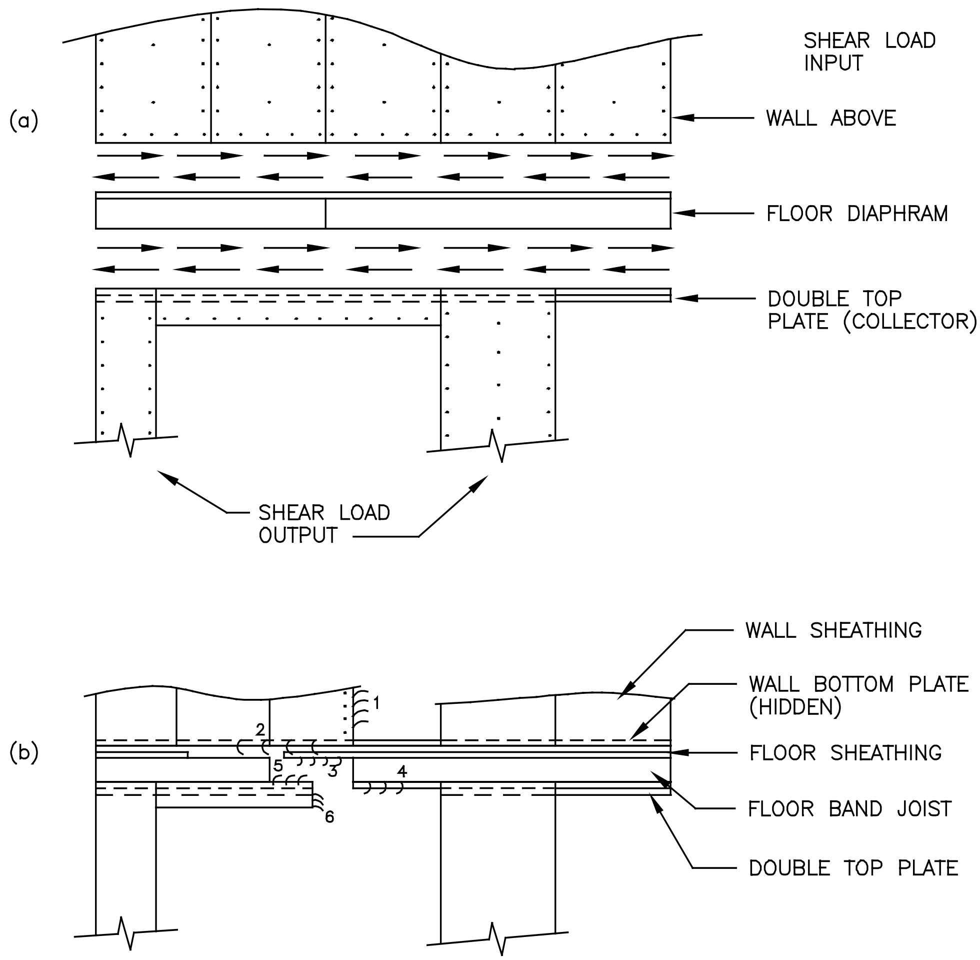


FIGURE 6.2 ***Shear Wall Collector and the Composite Failure Plane (Failure plane also applies to diaphragm chords)***



COMPOSITE FAILURE PLANE FOR A COLLECTOR:

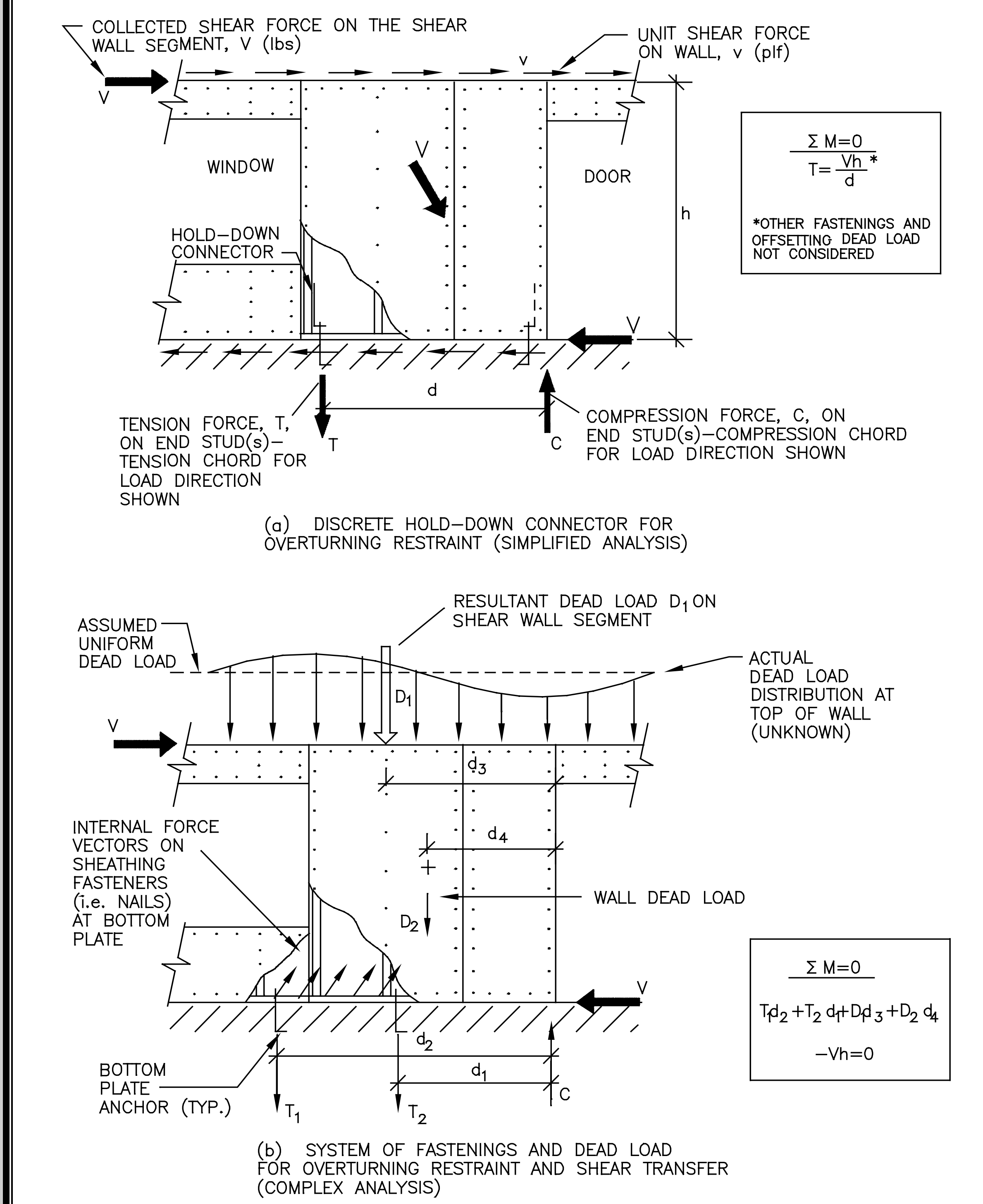
1. SHEATHING FASTENERS AT COMMON WALL STUD
2. WALL BOTTOM PLATE FASTENERS
3. FLOOR SHEATHING FASTENERS
4. DOUBLE TOP PLATE SPLICE NAILS
5. SHEATHING FASTENERS AT REGION OF DOUBLE TOP PLATE SPLICE
6. SHEATHING AND HEADER FASTENERS TO KING STUD

Hold-down restraints are devices used to restrain the whole building and individual shear wall segments from the overturning that results from the leveraging (i.e., overturning moment) created by lateral forces. The current engineering approach calls for restraints that are typically metal connectors (i.e., straps or brackets) that attach to and anchor the chords (i.e., end studs) of shear wall segments (see Figure 6.3a). In many typical residential applications, however, overturning forces may be resisted by the dead load and the contribution of many component connections (see Figure 6.3b). Unfortunately (but in reality), this consideration may require a more intensive analytic effort and greater degree of designer presumption because overturning forces may disperse through many “load paths” in a nonlinear fashion. Consequently, the analysis of overturning becomes much more complicated; the designer cannot simply assume a single load path through a single hold-down connector. Indeed, analytic knowledge of overturning has not matured sufficiently to offer an exact performance-based solution, even though experience suggests that the resistance provided by conventional framing has proven adequate to prevent collapse in all but the most extreme conditions or mis-applications (see Chapter 1 and Section 6.2).

Framing and fastenings at wall corner regions are a major factor in explaining the actual behavior of conventionally built homes, yet there is no currently recognized way to account for this effect from a performance-based design perspective. Several studies have investigated corner framing effects in restraining shear walls without the use of hold-down brackets. In one such study, cyclic and monotonic tests of typical 12-foot-long wood-framed shear walls with 2- and 4-foot corner returns have demonstrated that overturning forces can be resisted by reasonably detailed corners (i.e., sheathing fastened to a common corner stud), with the reduction in shear capacity only about 10 percent from that realized in tests of walls with hold-downs instead of corner returns (Dolan and Heine, 1997c). The corner framing approach can also improve ductility (Dolan and Heine, 1997c) and is confirmed by testing in other countries (Thurston, 1994). In fact, shear wall test methods in New Zealand use a simple three-nail connection to provide hold-down restraint (roughly equivalent to three 16d common nails in a single shear wood-to-wood connection with approximately a 1,200- to 1,500-pound ultimate capacity). The three-nail connection resulted from an evaluation of the restraining effect of corners and the selection of a minimum value from typical construction. The findings of the tests reported above do not consider the beneficial contribution of the dead load in helping to restrain a corner from uplift as a result of overturning action.

The discussion to this point has given some focus to conventional residential construction practices for wall bracing that have worked effectively in typical design conditions. This observation is a point of contention, however, because conventional construction lacks the succinct loads paths that may be assumed when following an accepted engineering method. Therefore, conventional residential construction does not lend itself readily to current engineering conventions of analyzing a lateral force resisting system in light-frame construction. As a result, it is difficult to define appropriate limitations to the use of conventional construction practices based purely on existing conventions of engineering analysis.

FIGURE 6.3 ***Two Types of Hold-Down Restraint and Basic Analytic Concepts***



6.4 The Current LFRS Design Practice

This section provides a brief overview of the current design practices for analyzing the LFRS of light-frame buildings. It highlights the advantages and disadvantages of the various approaches but, in the absence of a coherent body of evidence, makes no attempt to identify which approach, if any, may be considered superior. Where experience from whole-building tests and actual building performance in real events permits, the discussion provides a critique of current design practices that, for lack of better methods, relies somewhat on an intuitive sense for the difference between the structure as it is analyzed and the structure as it may actually perform. The intent is not to downplay the importance of engineering analysis; rather, the designer should understand the implications of the current analytic methods and their inherent assumptions and then put them into practice in a suitable manner.

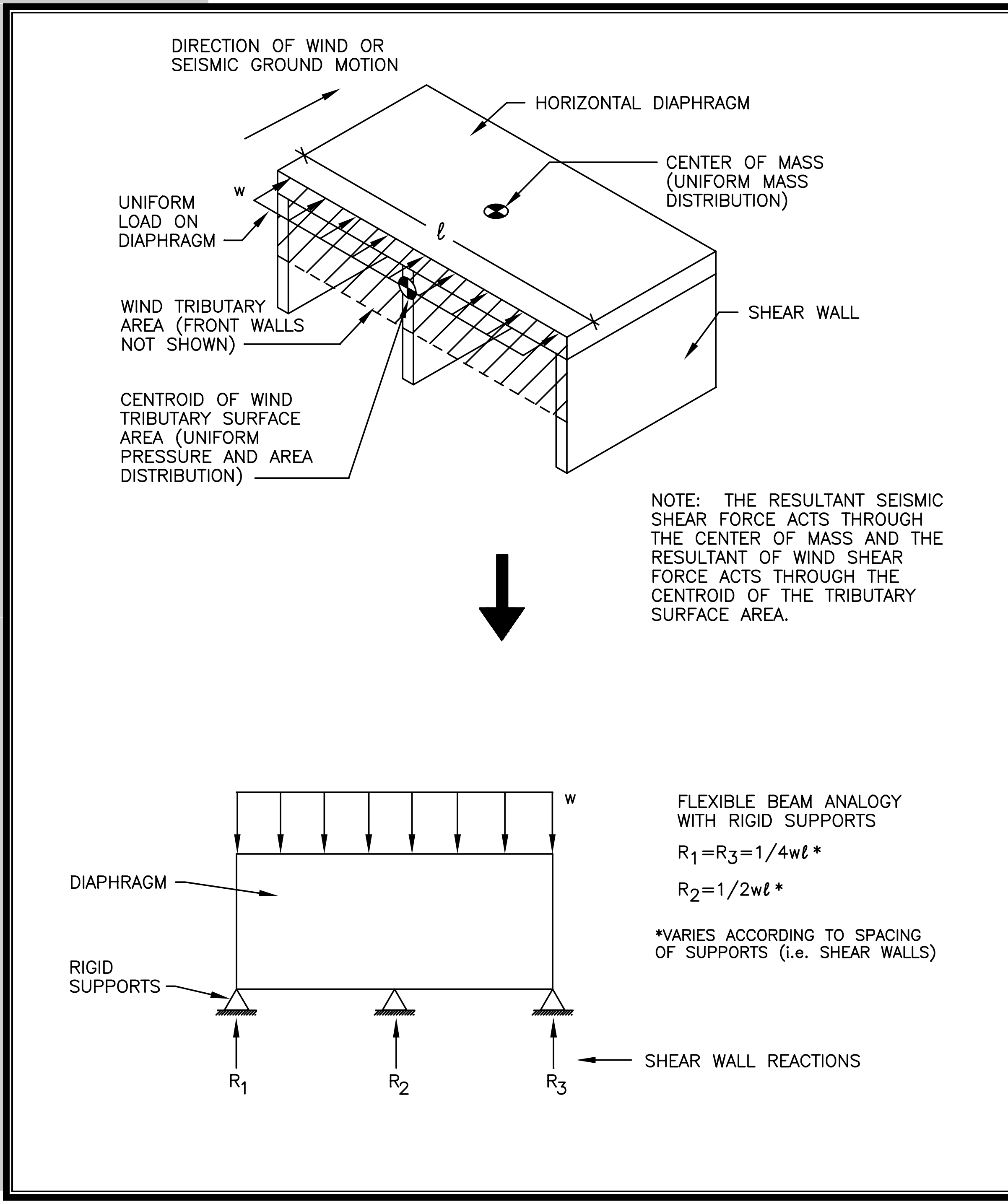
6.4.1 Lateral Force Distribution Methods

The design of the LFRS of light-frame buildings generally follows one of three methods described below. Each differs in its approach to distributing whole-building lateral forces through the horizontal diaphragms to the shear walls. Each varies in the level of calculation, precision, and dependence on designer judgment. While different solutions can be obtained for the same design by using the different methods, one approach is not necessarily preferred to another. All may be used for the distribution of seismic and wind loads to the shear walls in a building. However, some of the most recent building codes may place limitations or preferences on certain methods.

Tributary Area Approach (Flexible Diaphragm)

The *tributary area approach* is perhaps the most popular method used to distribute lateral building loads. Tributary areas based on building geometry are assigned to various components of the LFRS to determine the wind or seismic loads on building components (i.e., shear walls and diaphragms). The method assumes that a diaphragm is relatively flexible in comparison to the shear walls (i.e., a “flexible diaphragm”) such that it distributes forces according to tributary areas rather than according to the stiffness of the supporting shear walls. This hypothetical condition is analogous to conventional beam theory, which assumes rigid supports as illustrated in Figure 6.4 for a continuous horizontal diaphragm (i.e., floor) with three supports (i.e., shear walls).

FIGURE 6.4 ***Lateral Force Distribution by a "Flexible" Diaphragm (tributary area approach)***



In seismic design, tributary areas are associated with uniform area weights (i.e., dead loads) assigned to the building systems (i.e., roof, walls, and floors) that generate the inertial seismic load when the building is subject to lateral ground motion (refer to Chapter 3 on earthquake loads). In wind design, the tributary areas are associated with the lateral component of the wind load acting on the exterior surfaces of the building (refer to Chapter 3 on wind loads).

The flexibility of a diaphragm depends on its construction as well as on its aspect ratio (length:width). Long, narrow diaphragms, for example, are more flexible in bending along the their long dimension than short, wide diaphragms. In other words, rectangular diaphragms are relatively stiff in one loading direction and relatively flexible in the other. Similarly, long shear walls with few openings are stiffer than walls comprised of only narrow shear wall segments. While analytic methods are available to calculate the stiffness of shear wall segments and diaphragms (refer to Section 6.5), the actual stiffness of these systems is extremely difficult to predict accurately (refer to Section 6.2). It should be noted that if the diaphragm is considered infinitely rigid relative to the shear walls and the shear walls have roughly equivalent stiffness, the three shear wall reactions will be roughly equivalent (i.e., $R_1 = R_2 = R_3 = 1/3[w][l]$). If this assumption were more accurate, the interior shear wall would be overdesigned and the exterior shear walls underdesigned with use of the tributary area method. In many cases, the correct answer is probably somewhere between the apparent over- and under-design conditions.

The tributary area approach is reasonable when the layout of the shear walls is generally symmetrical with respect to even spacing and similar strength and stiffness characteristics. It is particularly appropriate in concept for simple buildings with diaphragms supported by two exterior shear wall lines (with similar strength and stiffness characteristics) along both major building axes. More generally, the major advantages of the tributary area LFRS design method are its simplicity and applicability to simple building configurations. In more complex applications, the designer should consider possible imbalances in shear wall stiffness and strength that may cause or rely on torsional response to maintain stability under lateral load (see *relative stiffness design approach*).

Total Shear Approach ("Eyeball" Method)

Considered the second most popular and simplest of the three LFRS design methods, the *total shear approach* uses the total story shear to determine a total amount of shear wall length required on a given story level for each orthogonal direction of loading. The amount of shear wall is then "evenly" distributed in the story according to designer judgment. While the total shear approach requires the least amount of computational effort among the three methods, it demands good "eyeball" judgment as to the distribution of the shear wall elements in order to address or avoid potential loading or stiffness imbalances. In seismic design, loading imbalances may be created when a building's mass distribution is not uniform. In wind design, loading imbalances result when the surface area of the building is not uniform (i.e., taller walls or steeper roof sections experience greater lateral wind load). In both cases, imbalances are created when the center of resistance is offset from either the center of mass (seismic design) or the resultant force center of the exterior surface pressures (wind design). Thus, the reliability of the total shear approach is highly dependent on the designer's judgment and intuition regarding load distribution and structural response. If used indiscriminately without consideration of the above factors, the total shear approach to LFRS design can result in poor performance in severe seismic or wind events. However, for small structures such as homes, the method has produced reasonable designs, especially in view of the overall uncertainty in seismic and wind load analysis.

Relative Stiffness Design Approach (Rigid Diaphragm)

The *relative stiffness approach* was first contemplated for house design in the 1940s and was accompanied by an extensive testing program to create a database of racking stiffnesses for a multitude of interior and exterior wall constructions used in residential construction at that time (NBS, 1948). If the horizontal diaphragm is considered stiff relative to the shear walls, then the lateral forces on the building are distributed to the shear wall lines according to their relative stiffness. A stiff diaphragm may then rotate some degree to distribute loads to all walls in the building, not just to walls parallel to an assumed loading direction. Thus, the relative stiffness approach considers torsional load distribution as well as distribution of the direct shear loads. When torsional force distribution needs to be considered, whether to demonstrate lateral stability of an "unevenly" braced building or to satisfy a building code requirement, the relative stiffness design approach is the only available option.

Although the approach is conceptually correct and comparatively more rigorous than the other two methods, its limitations with respect to reasonably determining the real stiffness of shear wall lines (composed of several restrained and unrestrained segments and nonstructural components) and diaphragms (also affected by nonstructural components and the building plan configuration) render its analogy to actual structural behavior uncertain. Ultimately, it is only as good as the assumptions regarding the stiffness or shear walls and diaphragms relative to the actual stiffness of a complete building system. As evidenced in the previously mentioned whole-building tests and in other authoritative design texts on the subject (Ambrose and Vergun, 1987), difficulties in accurately predicting the stiffness of shear walls and diaphragms in actual buildings are significant. Moreover, unlike the other methods, the relative stiffness design approach is iterative in that the distribution of loads to the shear walls requires a preliminary design so that relative stiffness may be estimated. One or more adjustments and recalculations may be needed before reaching a satisfactory final design.

However, it is instructional to consider analytically the effects of stiffness in the distribution of lateral forces in an LFRS, even if based on somewhat idealized assumptions regarding relative stiffness (i.e., diaphragm is rigid over the entire expanse of shear walls). The approach is a reasonable tool when the torsional load distribution should be considered in evaluating or demonstrating the stability of a building, particularly a building that is likely to undergo significant torsional response in a seismic event. Indeed, torsional imbalances exist in just about any building and may be responsible for the relatively good performance of some light-frame homes when one side (i.e., the street-facing side of the building) is weaker (i.e., less stiff and less strong) than the other three sides of the building. This condition is common owing to the aesthetic desire and functional need for more openings on the front side of a building. However, a torsional response in the case of underdesign (i.e., "weak" or "soft" story) can wreak havoc on a building and constitute a serious threat to life.

6.4.2 Shear Wall Design Approaches

Once the whole-building lateral loads have been distributed and assigned to the floor and roof diaphragms and various designated shear walls, each of these subassemblies must be designed to resist the assigned shear loads. As discussed, the whole-building shear loads are distributed to various shear walls ultimately in accordance with the principle of relative stiffness (whether handled by judgment, analytic assumptions per a selected design method, or both). Similarly, the distribution of the assigned shear load to the various shear wall segments within a given shear wall line is based on the same principle, but at a different scale. The scale is the subassembly (or shear wall) as opposed to the whole building.

The methods for designing and distributing the forces within a shear wall line differ as described below. As with the three different approaches described for the distribution of lateral building loads, the shear wall design methods place different levels of emphasis on analytic rigor and judgment. Ultimately, the configuration of the building (i.e., are the walls inherently broken into individual segments by large openings or many offsets in plan dimensions?) and the required demand (i.e., shear load) should drive the choice of a shear wall design approach and the resulting construction detailing. Thus, the choice of which design method to use is a matter of designer judgment and required performance. In turn, the design method itself imposes detailing requirements on the final construction in compliance with the analysis assumptions. Accordingly, the above decisions affect the efficiency of the design effort and the complexity of the resulting construction details.

Segmented Shear Wall (SSW) Design Approach

The *segmented shear wall design approach*, well recognized as a standard design practice, is the most widely used method of shear wall design. It considers the shear resisting segments of a given shear wall line as separate "elements," with each segment restrained against overturning by the use of hold-down connectors at its ends. Each segment is a fully sheathed portion of the wall without any openings for windows or doors. The design shear capacity of each segment is determined by multiplying the length of the segment (sometimes called segment width) by tabulated unit shear design values that are available in the building codes and newer design standards. In its simplest form, the approach analyzes each shear wall segment for static equilibrium in a manner analogous to a cantilevered beam with a fixed end (refer to Figures 6.1 and 6.3a). In a wall with multiple designated shear wall segments, the typical approach to determining an adequate total length of all shear wall segments is to divide the design shear load demand on the wall by the unit shear design value of the wall construction. The effect of stiffness on the actual shear force distribution to the various segments is simply handled by complying with code-required maximum shear wall segment aspect ratios (i.e., segment height divided by segment width). Although an inexact and circuitous method of handling the problem of shear force distribution in a shear wall line, the SSW approach has been in successful practice for many years, partly due to the use of conservative unit shear design values.

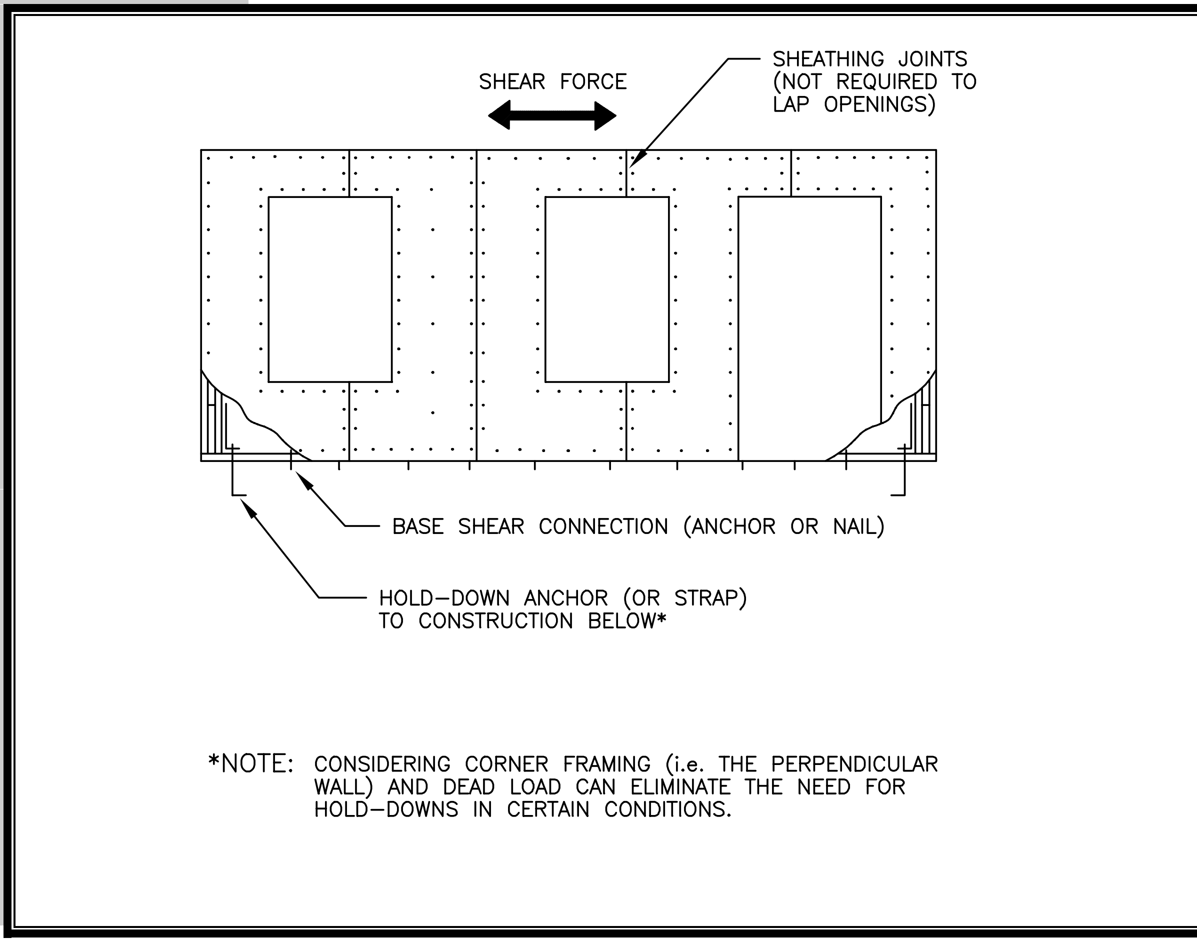
When stiffness is considered, the stiffness of a shear wall segment is assumed to be linearly related to its length (or its total design shear strength). However, the linear relationship is not realistic outside certain limits. For example, stiffness begins to decrease with notable nonlinearly once a shear wall segment decreases below a 4-foot length on an 8-foot-high wall (i.e., aspect ratio of 2 or greater). This does not mean that wall segments shorter than 4 feet in width cannot be used but rather that the effect of relative stiffness in distributing the load needs to be considered. The SSW approach is also less favorable when the wall as a system rather than individual segments (i.e., including sheathed areas above and below openings) may be used to economize on design while meeting required performance (see perforated shear wall design approach below).

As shown in Figure 6.3, it is common either to neglect the contribution of dead load or assume that the dead load on the wall is uniformly distributed as would be the case under gravity loading only. In fact, unless the wall is restrained with an infinitely rigid hold-down device (an impossibility), the uniform dead load distribution will be altered as the wall rotates and deflects upward during the application of shear force (see Figure 6.3b). As a result, depending on the rigidity of the framing system above, the dead load will tend to concentrate more toward the “high points” in the wall line, as the various segments begin to rotate and uplift at their leading edges. Thus, the dead load may be somewhat more effective in offsetting the overturning moment on a shear wall segment than is suggested by the uniform dead load assumption. Unfortunately, this phenomenon involves nonrigid body, nonlinear behavior for which there are no simplified methods of analysis. Therefore, this effect is generally not considered, particularly for walls with specified restraining devices (i.e., hold-downs) that are, by default, generally assumed to be completely rigid–an assumption that is known by testing not to hold true to varying degrees depending on the type of device and its installation.

Basic Perforated Shear Wall (PSW) Design Approach

The *basic perforated shear wall (PSW) design* method is gaining popularity among designers and even earning code recognition. The method, however, is not without controversy in terms of appropriate limits and guidance on use. A perforated shear wall is a wall that is fully sheathed with wood structural panels (i.e., oriented strand board or plywood) and that has openings or “perforations” for windows and doors. The ends of the walls–rather than each individual segment as in the segmented shear wall method–are restrained against overturning. As for the intermediate segments of the wall, they are restrained by conventional or designed framing connections such as those at the base of the wall that transfer the shear force resisted by the wall to the construction below. The capacity of a PSW is determined as the ratio of the strength of a wall with openings to the strength of a wall of the same length without openings. The ratio is calculated by using two empirical equations given in Section 6.5. Figure 6.5 illustrates a perforated shear wall.

FIGURE 6.5 ***Illustration of a Basic Perforated Shear Wall***



The PSW design method requires the least amount of special construction detailing and analysis among the current shear wall design methods. It has been validated in several recent studies in the United States but dates back more than 20 years to research first conducted in Japan (Dolan and Heine, 1997a and b; Dolan and Johnson, 1996a and 1996b; NAHBRC, 1997; NAHBRC, 1998; NAHBRC, 1999; Sugiyama and Matsumoto, 1994; Ni et al., 1998). While it produces the simplest form of an engineered shear wall solution, other methods such as the segmented shear wall design method–all other factors equal–can yield a stronger wall. Conversely, a PSW design with increased sheathing fastening can outperform an SSW with more hold-downs but weaker sheathing fastening. The point is, that for many applications, the PSW method often provides an adequate and more efficient design. Therefore, the PSW method should be considered an option to the SSW method as appropriate.

Enhancements to the PSW Approach

Several options in the form of structural optimizations (i.e., "getting the most from the least") can enhance the PSW method. One option uses multiple metal straps or ties to restrain each stud, thereby providing a highly redundant and simple method of overturning restraint. Unfortunately, this promising enhancement has been demonstrated in only one known proof test of the concept (NAHBRC, 1999). It can, however, improve shear wall stiffness and increase capacity beyond that achieved with either the basic PSW method or SSW design approach. Another option, subjected to limited study by the NAHB Research Center, calls for perforated shear walls with metal truss plates at key framing joints (NAHBRC, 1998). To a degree similar to that in the first option, this enhancement increases shear capacity and stiffness without the use of any special hold-downs or restraining devices other than conventional framing connections at the base of the wall (i.e., nails or anchor bolts). Neither of the above options applied dead loads to the tested walls, such application would have improved performance. Unfortunately, the results do not lend themselves to easy duplication by analysis and must be used at their face value as empirical evidence to justify practical design improvements for conditions limited by the tests. Analytic methods are under development to facilitate use of optimization concepts in shear wall design and construction.

In a mechanics-based form of the PSW, analytic assumptions using free-body diagrams and principles of statics can conservatively estimate restraining forces that transfer shear around openings in shear walls based on the assumption that wood-framed shear walls behave as rigid bodies with elastic behavior. As compared to several tests of the perforated shear wall method discussed above, the mechanics-based approach leads to a conservative solution requiring strapping around window openings. In a condition outside the limits for application of the PSW method, a mechanics-based design approach for shear transfer around openings provides a reasonable alternative to traditional SSW design and the newer empirically based PSW design. The added detailing merely takes the form of horizontal strapping and blocking at the top and bottom corners of window openings to transfer the calculated forces derived from free-body diagrams representing the shear wall segments and sheathed areas above and below openings. For more detail, the reader should consult other sources of information on this approach (Diekmann, 1986; ICBO, 1997; ICC, 1999).

6.4.3 Basic Diaphragm Design Approach

As described in Chapter 2 and earlier in this section, horizontal diaphragms are designed by using the analogy of a deep beam laid flatwise. Thus, the shear forces in the diaphragm are calculated as for a beam under a uniform load (refer to Figure 6.4). As is similar to the case of shear walls, the design shear capacity of a horizontal diaphragm is determined by multiplying the diaphragm depth (i.e., depth of the analogous deep beam) by the tabulated unit shear design values found in building codes. The chord forces (in the “flange” of the analogous deep beam) are calculated as a tension force and compression force on opposite sides of the diaphragm. The two forces form a force couple (i.e., moment) that resists the bending action of the diaphragm (refer to Figure 6.1).

To simplify the calculation, it is common practice to assume that the chord forces are resisted by a single chord member serving as the “flange” of the deep beam (i.e., a band joist). At the same time, bending forces internal to the diaphragm are assumed to be resisted entirely by the boundary member or band joist rather than by other members and connections within the diaphragm. In addition, other parts of the diaphragm boundary (i.e., walls) that also resist the bending tension and compressive forces are not considered. Certainly, a vast majority of residential roof diaphragms that are not considered “engineered” by current diaphragm design standards have exhibited ample capacity in major design events. Thus, the beam analogy used to develop an analytic model for the design of wood-framed horizontal diaphragms has room for improvement that has yet to be explored from an analytic standpoint.

As with shear walls, openings in the diaphragm affect the diaphragm’s capacity. However, no empirical design approach accounts for the effect of openings in a horizontal diaphragm as for shear walls (i.e., the PSW method). Therefore, if openings are present, the effective depth of the diaphragm in resisting shear forces must either discount the depth of the opening or be designed for shear transfer around the opening. If it is necessary to transfer shear forces around a large opening in a diaphragm, it is common to perform a mechanics-based analysis of the shear transfer around the opening. The analysis is similar to the previously described method that uses free-body diagrams for the design of shear walls. The reader is referred to other sources for further study of diaphragm design (Ambrose and Vergun, 1987; APA, 1997; Diekmann, 1986).

6.5 Design Guidelines

6.5.1 General Approach

This section outlines methods for designing shear walls (Section 6.5.2) and diaphragms (Section 6.5.3). The two methods of shear wall design are the segmented shear wall (SSW) method and the perforated shear wall (PSW) method. The selection of a method depends on shear loading demand, wall configuration, and the desired simplicity of the final construction. Regardless of design method and resulting LFRS, the first consideration is the amount of lateral load to be resisted by the arrangement of shear walls and diaphragms in a given building. The design loads and basic load combinations in Chapter 3, Table 3.1, are as follows:

- 0.6D + (W or 0.7E) ASD
- 0.9D + (1.5W or 1.0E) LRFD

Earthquake load and wind load are considered separately, with shear walls designed in accordance with more stringent loading conditions.

Lateral building loads should be distributed to the shear walls on a given story by using one of the following methods as deemed appropriate by the designer:

- tributary area approach;
- total shear approach; or
- relative stiffness approach.

These methods were described earlier (see Section 6.4). In the case of the tributary area method, the loads can be immediately assigned to the various shear wall lines based on tributary building areas (exterior surface area for wind loads and building plan area for seismic loads) for the two orthogonal directions of loading (assuming rectangular-shaped buildings and relatively uniform mass distribution for seismic design). In the case of the total shear approach, the load is considered as a "lump sum" for each story for both orthogonal directions of loading. The shear wall construction and total amount of shear wall for each direction of loading and each shear wall line are then determined in accordance with this section to meet the required load as determined by either the tributary area or total shear approach. The designer must be reasonably confident that the distribution of the shear walls and their resistance is reasonably "balanced" with respect to building geometry and the center of the total resultant shear load on each story. As mentioned, both the tributary and total shear approaches have produced many serviceable designs for typical residential buildings, provided that the designer exercises sound judgment.

In the case of the relative stiffness method, the assignment of loads must be based on an assumed relationship describing the relative stiffness of various shear wall lines. Generally, the stiffness of a wood-framed shear wall is assumed to be directly related to the length of the shear wall segments and the unit shear value of the wall construction. For the perforated shear wall method, the relative stiffness of various perforated shear wall lines may be assumed to be directly related to the design strength of the various perforated shear wall lines. Using the principle of moments and a representation of wall racking stiffness, the designer can then identify the center of shear resistance for each story and determine each story's torsional load (due to the offset of the load center from the center of resistance). Finally, the designer superimposes direct shear loads and torsional shear loads to determine the estimated shear loads on each of the shear wall lines.

It is common practice (and required by some building codes) for the torsional load distribution to be used only to add to the direct shear load on one side of the building but not to subtract from the direct shear load on the other side, even though the restriction is not conceptually accurate. Moreover, most seismic design codes require evaluations of the lateral resistance to seismic loads with "artificial" or "accidental" offsets of the estimated center of mass of the building (i.e., imposition of an "accidental" torsional load imbalance). These provisions, when required, are intended to conservatively address uncertainties in the design process that may otherwise go undetected in any given analysis (i.e., building mass is assumed uniform when it actually is not). As an alternative, uncertainties may be more easily accommodated by increasing the shear load by an equivalent amount in effect (i.e., say 10 percent). Indeed, the seismic shear load using the simplified method (see Equation 3.8-1 in Chapter 3) includes a factor that increases the design load by 20 percent and may be considered adequate to address uncertainties in torsional load distribution. However, the simple "20 percent" approach to addressing accidental torsion loads is not explicitly permitted in any current building code. But, for housing, where many redundancies also exist, the "20 percent" rule seems to be a reasonable substitute for a more "exact" analysis of accidental torsion. Of course, it is not a substitute for evaluating and designing for torsion that is expected to occur.

Design Example 6.5 of Section 6.6 elaborates on and demonstrates the use of the methods of load distribution described above. The reader is encouraged to study and critique them. The example contains many concepts and insights that cannot be otherwise conveyed without the benefit of a “real” problem.

6.5.2 Shear Wall Design

6.5.2.1 Shear Wall Design Values (F_s)

This section provides unfactored (ultimate) unit shear values for wood-framed shear wall constructions that use wood structural panels. Other wall constructions and framing methods are included as an additional resource. The unit shear values given here differ from those in the current codes in that they are based explicitly on the ultimate shear capacity as determined through testing. Therefore, the designer is referred to the applicable building code for "code-approved" unit shear values. This guide uses ultimate unit shear capacities as its basis to give the designer an explicit measure of the actual capacity and safety margin (i.e., reserve strength) used in design and to provide for a more consistent safety margin across various shear wall construction options. Accordingly, it is imperative that the values used in this guide are appropriately adjusted in accordance with Sections 6.5.2.2 and 6.5.2.3 to ensure an acceptable safety margin.

Wood Structural Panels (WSP)

Table 6.1 provides unit shear values for walls sheathed with wood structural panels. It should be noted again that these values are estimates of the ultimate unit shear capacity values as determined from several sources (Tissell, 1993; FEMA, 1997; NAHBRC, 1998; NAHBRC, 1999; others). The design unit shear values in today’s building codes have inconsistent safety margins that typically range from 2.5 to 4 after all applicable adjustments (Tissell, 1993; Soltis, Wolfe, and Tuomi, 1983). Therefore, the actual capacity of a shear wall is not explicitly known to the designer using the codes’ allowable unit shear values. Nonetheless, one alleged benefit of using the code-approved design unit shear values is that the values are believed to address drift implicitly by way of a generally conservative safety margin. Even so, shear wall drift is usually not analyzed in residential construction for reasons stated previously.

The values in Table 6.1 and today’s building codes are based primarily on monotonic tests (i.e., tests that use single-direction loading). Recently, the effect of cyclic loading on wood-framed shear wall capacity has generated considerable controversy. However, cyclic testing is apparently not necessary when determining design values for seismic loading of wood-framed shear walls with structural wood panel sheathing. Depending on the cyclic test protocol, the resulting unit shear values can be above or below those obtained from traditional monotonic shear wall test methods (ASTM, 1998a; ASTM, 1998b). In fact, realistic cyclic testing protocols and their associated interpretations were found to be largely in agreement with the results obtained from monotonic testing (Karacabeyli and Ceccotti, 1998). The differences are generally in the range of 10 percent (plus or minus) and thus seem moot given that the seismic response modifier (see Chapter 3) is based on expert opinion (ATC, 1995) and that the actual performance of light-frame homes does not appear to correlate with important parameters in existing seismic design methods (HUD, 1999), among other factors that currently contribute to design uncertainty.

TABLE 6.1 ***Unfactored (Ultimate) Shear Resistance (plf) for Wood Structural Panel Shear Walls with Framing of Douglas-Fir, Larch, or Southern Pine***[1,2]

			Panels Applied Direct to Framing				
				Nail Spacing at Panel Edges (inches)			
Panel Grade	Nominal Panel Thickness (inches)	Minimum Nail Penetration in Framing (inches) (APA, 1998)	Nail Size (common or galvanized box)	6	4	3	2[3]
Structural I	5/16	1-1/4	6d	821	1,122	1,256	1,333
	3/8[4]	1-3/8	8d	833	1,200	1,362	1,711
	7/16[4]	1-3/8	8d	905	1,356	1,497	1,767
	15/32	1-3/8	8d	977	1,539	1,722	1,800
	15/32	1-1/2	10d[5]	1,256	1,701	1,963	2,222

Notes:
[1]Values are average ultimate unit shear capacity for walls sheathed with Structural I wood structural panels and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3. Additional adjustments to the table values should be made in accordance with those sections. For other rated panels (not Structural I), the table values should be multiplied by 0.85.
[2]All panel edges should be backed with 2-inch nominal or wider framing. Panels may be installed either horizontally or vertically. Space nails at 6 inches on center along intermediate framing members for 3/8-inch panels installed with the strong axis parallel to studs spaced 24 inches on-center and 12 inches on-center for other conditions and panel thicknesses.
[3]Framing at adjoining panel edges should be 3-inch nominal or wider and nails should be staggered where nails are spaced 2 inches on-center. A double thickness of nominal 2-inch framing is a suitable substitute.
[4]The values for 3/8- and 7/16-inch panels applied directly to framing may be increased to the values shown for 15/32-inch panels, provided that studs are spaced a maximum of 16 inches on-center or the panel is applied with its strong axis across the studs.
[5]Framing at adjoining panel edges should be 3-inch nominal or wider and nails should be staggered where 10d nails penetrating framing by more than 1-5/8 inches are spaced 3 inches or less on-center. A double thickness of 2-inch nominal framing is a suitable substitute.

The unit shear values in Table 6.1 are based on nailed sheathing connections. The use of elastomeric glue to attach wood structural panel sheathing to wood framing members increases the shear capacity of a shear wall by as much as 50 percent or more (White and Dolan, 1993). Similarly, studies using elastomeric construction adhesive manufactured by 3M Corporation have investigated seismic performance (i.e., cyclic loading) and confirm a stiffness increase of about 65 percent and a shear capacity increase of about 45 to 70 percent over sheathing fastened with nails only (Filiatrault and Foschi, 1991). Rigid adhesives may create even greater strength and stiffness increases. The use of adhesives is beneficial in resisting shear loads from wind. Glued shear wall panels are not recommended for use in high-hazard seismic areas because of the brittle failure mode experienced in the wood framing material (i.e., splitting), though at a significantly increased shear load. Gluing shear wall panels is also not recommended by panel manufacturers because of concern with panel buckling that may occur as a result of the interaction of rigid restraints with moisture/temperature expansion and contraction of the panels.

However, construction adhesives are routinely used in floor diaphragm construction to increase the bending stiffness and strength of floors; in-plane (diaphragm) shear is probably affected by an amount similar to that reported above for shear walls.

For unit shear values of wood structural panels applied to cold-formed steel framing, the following references are suggested: *Uniform Building Code* (ICBO,1997); *Standard Building Code* (SBCCI, 1999); and *Shear Wall Values for Light Weight Steel Framing* (AISI, 1996). The unit shear values for cold-formed steel-framed walls in the previous references are consistent with the values used in Table 6.1, including the recommended safety factor or resistance factor. Table 6.2 presents some typical unit shear values for cold-formed steel-framed walls with wood structural panel sheathing fastened with #8 screws. Values for power-driven, knurled pins (similar to deformed shank nails) should be obtained from the manufacturer and the applicable code evaluation reports (NES, Inc., 1997).

TABLE 6.2 ***Unfactored (Ultimate) Unit Shear Resistance (plf) for Walls with Cold-Formed Steel Framing and Wood Structural Panels***[1,2]

Panel Grade	Panel Type and Nominal Thickness (inches)	Minimum Screw Size[3]	Screw Spacing at Panel Edges (inches)[4]			
			6	4	3	2
Structural I	7/16 OSB	#8	700	915	1,275	1,625
	15/32 plywood	#8	780	990	1,465	1,700

Notes:
[1]Values are average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Values apply to 18 gauge (43 mil) and 20 gage (33 mil) steel C-shaped studs with a 1-5/8-inch flange width and 3-1/2- to 5-1/2-inch depth. Studs spaced a maximum of 24 inches on center.
[3]The #8 screws should have a head diameter of no less than 0.29 inches and the screw threads should penetrate the framing so that the threads are fully engaged in the steel.
[4]The spacing of screws in framing members located in the interior of the panels should be no more than 12 inches on-center.

Portland Cement Stucco (PCS)

Ultimate unit shear values for conventional PCS wall construction range from 490 to 1,580 plf based on the ASTM E 72 test protocol and 12 tests conducted by various testing laboratories (Testing Engineers, Inc., 1971; Testing Engineers, Inc., 1970; ICBO, 1969). In general, nailing the metal lath or wire mesh resulted in ultimate unit shear values less than 750 plf, whereas stapling resulted in ultimate unit shear values greater than 750 plf. An ultimate design value of 500 plf is recommended unless specific details of PCS construction are known. A safety factor of 2 provides a conservative allowable design value of about 250 plf. It must be realized that the actual capacity can be as much as five times 250 plf depending on the method of construction, particularly the means of fastening the stucco lath material. Current code-approved allowable design values are typically about 180 plf (SBCCI, 1999; ICBO, 1997). One code requires the values to be further reduced by 50 percent in higher-hazard seismic design areas (ICBO, 1997), although the reduction factor may not necessarily improve performance with respect to the cracking of the stucco finish in seismic events

(HUD, 1999); refer to Chapter 1 and the discussion in Chapter 3 on displacement compatibility under seismic load. It may be more appropriate to use a lower seismic response modifier R than to increase the safety margin in a manner that is not explicit to the designer. In fact, an R factor for PCS wood-framed walls is not explicitly provided in building codes (perhaps an R of 4.5 for "other" wood-framed walls is used) and should probably be in the range of 3 to 4 (without additional increases in the safety factor) since some ductility is provided by the metal lath and its connection to wood framing.

The above values pertain to PCS that is 7/8-inch thick with nail or staple fasteners spaced 6 inches on-center for attaching the metal wire mesh or lath to all framing members. Nails are typically 11 gauge by 1-1/2 inches in length and staples typically have 3/4-inch leg and 7/8-inch crown dimensions. The above unit shear values also apply to stud spacings no greater than 24 inches on-center. Finally, the aspect ratio of stucco wall segments included in a design shear analysis should not be greater than 2 (height/width) according to current building code practice.

Gypsum Wall Board (GWB)

Ultimate capacities in testing 1/2-inch-thick gypsum wall board range from 140 to 300 plf depending on the fastening schedule (Wolfe, 1983; Patton-Mallory, Gutkowski, Soltis, 1984; NAHBRF, date unknown). Allowable or design unit shear values for gypsum wall board sheathing range from 75 to 150 plf in current building codes depending on the construction and fastener spacing. At least one building code requires the values to be reduced by 50 percent in high-hazard seismic design areas (ICBO, 1997). Gypsum wall board is certainly not recommended as the primary seismic bracing for walls, although it does contribute to the structural resistance of buildings in all seismic and wind conditions. It should also be recognized that fastening of interior gypsum board varies in practice and is generally not an 'inspected" system. Table 6.3 provides estimated ultimate unit shear values for gypsum wall board sheathing.

TABLE 6.3 ***Unfactored (Ultimate) Unit Shear Values (plf) for 1/2-Inch-Thick Gypsum Wall Board Sheathing***[1,2]

GWB Thickness	**Blocking Condition**[3]	**Spacing of Framing (inches)**	**Fastener Spacing at Pane Edges (inches)**				
			12	8	7	6	4
1/2 inch	Blocked	16	120	210	250	260	300
	Unblocked	16	80	170	200	220	250
		24	40	120	150	180	220

Notes:
[1]The values represent average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Fasteners should be minimum 1 1/2-inch drywall nails (i.e., 4d cooler) or 1-1/4-inch drywall screws (i.e., #6 size with bugle head) or equivalent with spacing of fasteners and framing members as shown.
[3]"Blocked" refers to panels with all edges fastened to framing members; "unblocked" refers to the condition where the panels are placed horizontally with horizontal joints between panels not fastened to blocking or vertically with the top and bottom edges fastened only at stud locations.

1x4 Wood Let-in Braces and Metal T-braces

Table 6.4 provides values for typical ultimate shear capacities of 1x4 wood let-in braces and metal T-braces. Though not found in current building codes, the values are based on available test data (Wolfe, 1983; NAHBRF, date unknown). Wood let-in braces and metal T-braces are common in conventional residential construction and add to the shear capacity of walls. They are always used in combination with other wall finish materials that also contribute to a wall's shear capacity. The braces are typically attached to the top and bottom plates of walls and at each intermediate stud intersection with two 8d common nails. They are not recommended for the primary lateral resistance of structures in high-hazard seismic or wind design areas. In particular, values of the seismic response modifier R for walls braced in this manner have not been clearly defined for the sake of standardized seismic design guidance.

TABLE 6.4 ***Unfactored (Ultimate) Shear Resistance (lbs) for 1x4 Wood Let-ins and Metal T-Braces[1,2]***

Type of Diagonal Brace	Ultimate Horizontal Shear Capacity (per brace)[3]
1x4 wood let-in brace (8-foot wall height)[4]	600 lbs (tension and compression)
Metal T-brace[5]	1,400 lbs (tension only)

Notes:
[1]Values are average ultimate unit shear capacity and should be multiplied by a safety factor (ASD) or resistance factor (LRFD) in accordance with Sections 6.5.2.2 and 6.5.2.3.
[2]Values are based on minimum Spruce-Pine-Fir lumber (specific gravity, G = 0.42).
[3]Capacities are based on tests of wall segments that are restrained against overturning.
[4]Installed with two 8d common nails at each stud and plate intersection. Angle of brace should be between 45 and 60 degrees to horizontal.
[5]Installed per manufacturer recommendations and the applicable code evaluation report. Design values may vary depending on manufacturer recommendations, installation requirements, and product attributes.

Other Shear-Resisting Wall Facings

Just about any wall facing, finish, or siding material contributes to a wall's shear resistance qualities. While the total contribution of nonstructural materials to a typical residential building's lateral resistance is often substantial (i.e., nearly 50 percent if interior partition walls are included), current design codes in the United States prohibit considerations of the role of facing, finish, or siding. Some suggestions call for a simple and conservative 10 percent increase (known as the "whole-building interaction factor") to the calculated shear resistance of the shear walls or a similar adjustment to account for the added resistance and whole-building effects not typically considered in design (Griffiths and Wickens, 1996).

Some other types of wall sheathing materials that provide shear resistance include particle board and fiber board. Ultimate unit shear values for fiber board range from 120 plf (6d nail at 6 inches on panel edges with 3/8-inch panel thickness) to 520 plf (10d nail at 2 inches on panel edges with 5/8-inch panel thickness). The designer should consult the relevant building code or manufacturer data for additional information on fiber board and other materials' shear resistance qualities. In one study that conducted tests on various wall assemblies for HUD, fiber board was not recommended for primary shear resistance in high-hazard seismic or wind design areas for the stated reasons of potential durability and cyclic loading concerns (NAHBRF, date unknown).

Combining Wall Bracing Materials

When wall bracing materials (i.e., sheathing) of the same type are used on opposite faces of a wall, the shear values may be considered additive. In high-hazard seismic design conditions, dissimilar materials are generally assumed to be nonadditive. In wind-loading conditions, dissimilar materials may be considered additive for wood structural panels (exterior) with gypsum wall board (interior). Even though let-in brace or metal T-brace (exterior) with gypsum wall board (interior) and fiber board (exterior) with gypsum wall board (interior) are also additive, they are not explicitly recognized as such in current building codes.

When the shear capacity for walls with different facings is determined in accordance with Sections 6.5.2.2 and 6.5.2.3, the designer must take care to apply the appropriate adjustment factors to determine the wall construction's total design racking strength. Most of the adjustment factors in the following sections apply only to wood structural panel sheathing. Therefore, the adjustments in the next section should be made as appropriate before determining combined shear resistance.

6.5.2.2 Shear Wall Design Capacity

The unfactored and unadjusted ultimate unit shear resistance values of wall assemblies should first be determined in accordance with the guidance provided in the previous section for rated facings or structural sheathing materials used on each side of the wall. This section provides methods for determining and adjusting the design unit shear resistance and the shear capacity of a shear wall by using either the perforated shear wall (PSW) approach or segmented shear wall (SSW) approach discussed in Section 6.4.2. The design approaches and other important considerations are illustrated in the design examples of Section 6.6.

Perforated Shear Wall Design Approach

The following equations provide the design shear capacity of a perforated shear wall:

$$F'_s = (F_s)C_{sp}C_{ns}x[\frac{1}{SF} or \phi] \quad \text{(units plf)} \qquad \text{Eq. 6.5-1a}$$

$$F_{psw} = (F'_s)C_{op}C_{dl}x[L] \quad \text{(units lb)} \qquad \text{Eq. 6.5-1b}$$

where,

F_{psw} = the design shear capacity (lb) of the perforated shear wall

F_s = the unfactored (ultimate) and unadjusted unit shear capacity (plf) for each facing of the wall construction; the C_{sp} and C_{ns} adjustment factors apply only to the wood structural panel sheathing F_s values in accordance with Section 6.5.2.1

F'_s = the factored and adjusted design unit shear capacity (plf) for the wall construction

C = the adjustment factors in accordance with Section 6.5.2.3 as applicable

L = the length of the perforated shear wall, which is defined as the distance between the restrained ends of the wall line

1/*SF* = the safety factor adjustment for use with ASD

ϕ = the resistance factor adjustment for use with LRFD

The PSW method (Equations 6.5-1a and b) has the following limits on its use:

- The value of F_s for the wall construction should not exceed 1,500 plf in accordance with Section 6.5.1.2. The wall must be fully sheathed with wood structural panels on at least one side. Unit shear values of sheathing materials may be combined in accordance with Section 6.5.2.1.
- Full-height wall segments within a perforated shear wall should not exceed an aspect ratio of 4 (height/width) unless that portion of the wall is treated as an opening. (Some codes limit the aspect ratio to 2 or 3.5, but recent testing mentioned earlier has demonstrated otherwise.) The first wall segment on either end of a perforated shear wall must not exceed the aspect ratio limitation.
- The ends of the perforated shear wall must be restrained with hold-down devices sized in accordance with Section 6.5.2.4. Hold-down forces that are transferred from the wall above are additive to the hold-down forces in the wall below. Alternatively, each wall stud may be restrained by using a strap sized to resist an uplift force equivalent to the design unit shear resistance F'_s of the wall, provided that the sheathing area ratio r for the wall is not less than 0.5 (see equations for C_{op} and r in Section 6.5.2.3).
- Top plates must be continuous with a minimum connection capacity at splices with lap joints of 1,000 lb, or as required by the design condition, whichever is greater.
- Bottom plate connections to transfer shear to the construction below (i.e., resist slip) should be designed in accordance with Section 6.5.2.5 and should result in a connection at least equivalent to one 1/2-inch anchor bolt at 6 feet on center or two 16d pneumatic nails 0.131-inch diameter at 24 inches on center for wall constructions with $F_sC_{sp}C_{ns}$ not exceeding 800 plf (ultimate capacity of interior and exterior sheathing). Such connections have been shown to provide an ultimate shear slip capacity of more than 800 plf in typical shear wall framing systems (NAHBRC, 1999); refer to Section 7.3.6 of Chapter 7. For wall constructions with ultimate shear capacities $F_sC_{sp}C_{ns}$ exceeding 800 plf, the base connection must be designed to resist the unit shear load and also provide a design uplift resistance equivalent to the design unit shear load.
- Net wind uplift forces from the roof and other tension forces as a result of structural actions above the wall are transferred through

the wall by using an independent load path. Wind uplift may be resisted with the strapping option above, provided that the straps are sized to transfer the additional load.

Segmented Shear Wall Design Approach

The following equations are used to determine the adjusted and factored shear capacity of a shear wall segment:

$$F'_s = F_s C_{sp} C_{ns} C_{ar} [\frac{1}{SF} \text{ or } \phi] \qquad \text{Eq. 6.5-2a}$$

$$F_{ssw} = F'_s \, x[L_s] \qquad \text{Eq. 6.5-2b}$$

where,

F_{ssw} = the design shear capacity (lb) of a single shear wall segment
F_s = the unfactored (ultimate) and unadjusted unit shear resistance (plf) for the wall construction in accordance with Section 6.5.2.1 for each facing of the wall construction; the C_{sp} and C_{ns} adjustment factors apply only to wood structural panel sheathing F_s values
F'_s = the factored (design) and adjusted unit shear resistance (plf) for the total wall construction
C = the adjustment factors in accordance with Section 6.5.2.3
L_s = the length of a shear wall segment (total width of the sheathing panel(s) in the segment)
$1/SF$ = the safety factor adjustment for use with ASD
ϕ = the resistance factor adjustment for use with LRFD

The segmented shear wall design method (Equations 6.5-2a and b) imposes the following limits:

- The aspect ratio of wall segments should not exceed 4 (height/width) as determined by the sheathing dimensions on the wall segment. (Absent an adjustment for the aspect ratio, current codes may restrict the segment aspect ratio to a maximum of 2 or 3.5.)
- The ends of the wall segment should be restrained in accordance with Section 6.5.2.4. Hold-down forces that are transferred from shear wall segments in the wall above are additive to the hold-down forces in the wall below.
- Shear transfer at the base of the wall should be determined in accordance with Section 6.5.2.5.
- Net wind uplift forces from the roof and other tension forces as a result of structural actions above are transferred through the wall by using an independent load path.

For walls with multiple shear wall segments, the design shear resistance for the individual segments may be added to determine the total design shear resistance for the segmented shear wall line. Alternatively, the combined shear capacity at given amounts of drift may be determined by using the load-deformation equations in Section 6.5.2.6.

6.5.2.3 Shear Capacity Adjustment Factors

Safety and Resistance Factors (SF and ϕ)

Table 6.5 recommends values for safety and resistance factors for shear wall design in residential construction. A safety factor of 2.5 is widely recognized for shear wall design, although the range varies substantially in current code-approved unit shear design values for wood-framed walls (i.e., the range is 2 to more than 4). In addition, a safety factor of 2 is commonly used for wind design. The 1.5 safety factor for ancillary buildings is commensurate with lower risk but may not be a recognized practice in current building codes. A safety factor of 2 has been historically applied or recommended for residential dwelling design (HUD, 1967; MPS, 1958; HUD, 1999). It is also more conservative than safety factor adjustments typically used in the design of other properties with wood members and other materials.

TABLE 6.5 Minimum Recommended Safety and Resistance Factors for Residential Shear Wall Design

Type of Construction		Safety Factor (ASD)	Resistance Factor (LRFD)
Detached garages and ancillary buildings not for human habitation		1.5	1.0
Single-family houses, townhouses, and multifamily low-rise buildings (apartments)	Seismic	2.5	0.55
	Wind	2.0	0.7

Species Adjustment Factor (C_{sp})

The ultimate unit shear values for wood structural panels in Table 6.1 apply to lumber species with a specific gravity (density), G, greater than or equal to 0.5. Table 6.6 presents specific gravity values for common species of lumber used for wall framing. For G < 0.5, the following value of C_{sp} should be used to adjust values in Table 6.1 only (APA, 1998):

$$C_{sp} = [1 - (0.5 - G)] \leq 1.0 \qquad \text{Eq. 6.5-3}$$

TABLE 6.6 Specific Gravity Values (Average) for Common Species of Framing Lumber

Lumber Species	Specific Gravity, G
Southern Yellow Pine (SYP)	0.55
Douglas Fir-Larch (DF-L)	0.50
Hem-Fir (HF)	0.43
Spruce-Pine-Fir (SPF)	0.42

Nail Size Adjustment Factor (C_{ns})

The ultimate unit shear capacities in Table 6.1 are based on the use of common nails. For other nail types and corresponding nominal sizes, the C_{ns} adjustment factors in Table 6.7 should be used to adjust the values in Table 6.1. Nails should penetrate framing members a minimum of 10D, where D is the diameter of the nail.

TABLE 6.7 Values of C_{ns} for Various Nail Sizes and Types[1]

Nominal Nail Size (penny weight)	Nail Length (inches)	Nail Type					
		Common[2]	Box[3]	Pneumatic (by diameter in inches)			
				0.092	0.113	0.131	0.148
6d	1-7/8 to 2	1.0	0.8	0.9	1.0	N/A[4]	N/A[4]
8d	2-3/8 to 2-1/2	1.0	0.8	0.5	0.75	1.0	N/A[4]
10d	3	1.0	0.8	N/A[4]	N/A[4]	0.8	1.0

Notes:
[1]The values of C_{ns} are based on ratios of the single shear nail values in NER-272 (NES, Inc., 1997) and the NDS (AF&PA, 1997) and are applicable only to wood structural panel sheathing on wood-framed walls in accordance with Table 6.1.
[2]Common nail diameters are as follows: 6d (0.113 inch), 8d (0.131 inch), and 10d (0.148 inch).
[3]Box nail diameters are as follows: 6d (0.099 inch), 8d (0.113 inch), and 10d (0.128 inch).
[4]Diameter not applicable to nominal nail size. Nail size, diameter, and length should be verified with the manufacturer.

Opening Adjustment Factor (C_{op})

The following equation for C_{op} applies only to the perforated shear wall method in accordance with Equation 6.5-1b of Section 6.5.2.2:

$$C_{op} = r/(3-2r) \qquad \text{Eq. 6.5-4}$$

where,

r = $1/(1 + \alpha/\beta)$ = sheathing area ratio (dimensionless)

α = ΣA_o / (H x L) = ratio of area of all openings ΣA_o to total wall area, H x L (dimensionless)

β = ΣL_i / L = ratio of length of wall with full-height sheathing ΣL_i to the total wall length L of the perforated shear wall (dimensionless)

Dead Load Adjustment Factor (C_{dl})

The C_{dl} factor applies to the perforated shear wall method only (Equation 6.5-1b). The presence of a dead load on a perforated shear has the effect of increasing shear capacity (Ni et al., 1998). The increase is 15 percent for a uniform dead load of 300 plf or more applied to the top of the wall framing. The dead load should be decreased by wind uplift and factored in accordance with the lateral design load combinations of Chapter 3. The C_{dl} adjustment factor is determined as follows and should not exceed 1.15:

$$C_{dl} = 1 + 0.15\left(\frac{w_D}{300}\right) \le 1.15 \qquad \text{Eq 6.5-5}$$

where,

w_D = the net uniform dead load supported at the top of the perforated shear wall (plf) with consideration of wind uplift and factoring in accordance with load combinations of Chapter 3.

Aspect Ratio Adjustment Factor (C_{ar})

The following C_{ar} adjustment factor applies only to the segmented shear wall design method for adjusting the shear resistance of interior and exterior sheathing in accordance with Equation 6.5-2a of Section 6.5.2.2:

$$C_{ar} = \frac{1}{\sqrt{0.5(a)}} \text{ for } 2.0 \le a \le 4.0 \qquad \text{Eq 6.5-6}$$

C_{ar} = 1.0 for $a < 2.0$

where,

a is the aspect ratio (height/width) of the sheathed shear wall segment.

6.5.2.4 Overturning Restraint

Section 6.3 and Figure 6.3 address overturning restraint of shear walls in conceptual terms. In practice, the two generally recognized approaches to providing overturning restraint call for

- the evaluation of equilibrium of forces on a *restrained* shear wall segment using principles of engineering mechanics; or
- the evaluation of *unrestrained* shear walls considering nonuniform dead load distribution at the top of the wall with restraint provided by various connections (i.e., sheathing, wall bottom plate, corner framing, etc.).

The first method applies to restrained shear wall segments in both the perforated and segmented shear wall methods. The first segment on each end of a perforated shear wall is restrained in one direction of loading. Therefore, the overturning forces on that segment are analyzed in the same manner as for a segmented shear wall. The second method listed above is a valid and conceptually realistic method of analyzing the restraint of typical residential wall constructions, but it has not yet fully matured. Further, the method's load path (i.e., distribution of uplift forces to various connections with inelastic properties) is perhaps beyond the practical limits of a designer's intuition. Rather than presume a methodology based on limited testing (see Section 6.3), this guide does not suggest guidelines for the second approach. However, the second method is worth consideration by a designer when attempting to understand the performance of conventional, "nonengineered" residential construction. Mechanics-based methods to assist in the more complicated design approach are under development.

Using basic mechanics as shown in Figure 6.6, the following equation for the chord tension and compression forces are determined by summing moments about the bottom compression or tension side of a restrained shear wall segment:

$$\sum M_C = 0$$

$$F'_s\,(d)(h) - T\,(x) - D_W\left(\tfrac{1}{2}d\right) - (w_D)(d)\left(\tfrac{1}{2}d\right) = 0$$

$$T = \left(\frac{d}{x}\right)\left(F'_s\,h - \tfrac{1}{2}D_W - \tfrac{1}{2}(w_D)(d)\right) + t \qquad \text{Eq. 6.5-7a}$$

$$\sum M_T = 0$$

$$C = \left(\frac{d}{x}\right)\left(F'_s\,h + \tfrac{1}{2}D_W + \tfrac{1}{2}(w_D)(d)\right) + c \qquad \text{Eq. 6.5-7b}$$

where,

T = the tension force on the hold-down device (lb)

d = the width of the restrained shear wall segment (ft); for segments greater than 4 ft in width, use d = 4 ft.

x = the distance between the hold-down device and the compression edge of the restrained shear wall segment (ft); for segments greater than 4 ft in width, use x = 4 ft plus or minus the bracket offset dimension, if any

F'_s = the design unit shear capacity (plf) determined in accordance with Equation 6.5-2a of Section 6.5.2.2 (for both the PSW and SSW methods)

h = the height of the wall (ft)

D_w = the dead load of the shear wall segment (lb); dead load must be factored and wind uplift considered in accordance with the load combinations of Chapter 3.

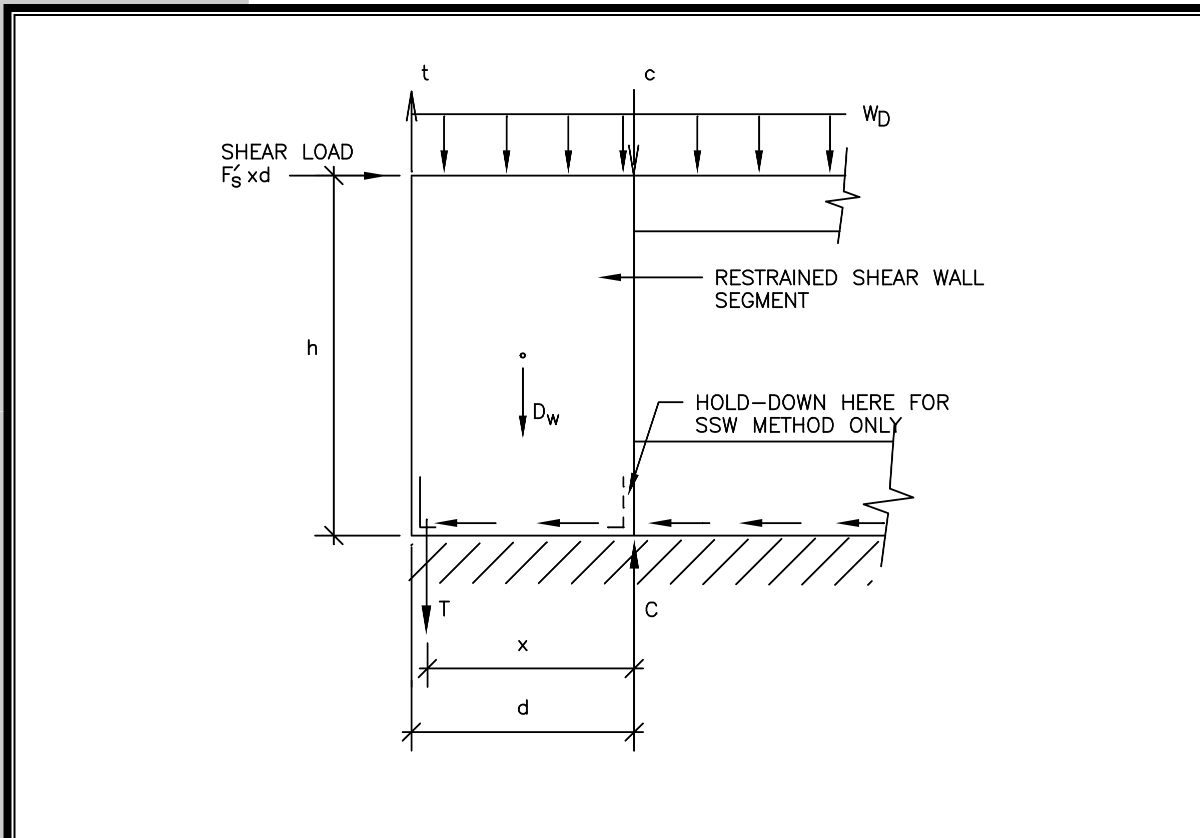
w_D = the uniform dead load supported by the shear wall segment (plf); dead load must be factored and wind uplift considered in accordance with the load combinations of Chapter 3.

t = the tension load transferred through a hold-down device, if any, restraining a wall above (lb); if there is no tension load, t = 0

c = the compression load transferred from wall segments above, if any (lb); this load may be distributed by horizontal structural elements above the wall (i.e., not a concentrated load); if there is not compression load, c = 0.

The 4-foot-width limit for *d* and *x* is imposed on the analysis of overturning forces as presented above because longer shear wall lengths mean that the contribution of the additional dead load cannot be rigidly transferred through deep bending action of the wall to have a full effect on the uplift forces occurring at the end of the segment, particularly when it is rigidly restrained from uplifting. This effect also depends on the stiffness of the construction above the wall that "delivers" and distributes the load at the top of the wall. The assumptions necessary to include the restraining effects of dead load is no trivial matter and, for that reason, it is common practice to not include any beneficial effect of dead load in the overturning force analysis of individual shear wall segments.

FIGURE 6.6 ***Evaluation of Overturning Forces on a Restrained Shear Wall Segment***



For a more simplified analysis of overturning forces, the effect of dead load may be neglected and the chord forces determined as follows using the symbols defined as before:

$$T = C = \left(\frac{d}{x}\right) F'_s\, h \qquad \text{Eq. 6.5-7c}$$

Any tension or compression force transferred from shear wall overturning forces originating above the wall under consideration must be added to the result of Equation 6.5-7c as appropriate. It is also assumed that any net wind uplift force is resisted by a separate load path (i.e., wind uplift straps are used in addition to overturning or hold-down devices).

For walls not rigidly restrained, the initiation of overturning uplift at the end stud (i.e., chord) shifts an increasing amount of the dead load supported by the wall toward the leading edge. Thus, walls restrained with more flexible hold-down devices or without such devices benefit from increased amounts of offsetting dead load as well as from the ability of wood framing and connections to disperse some of the forces that concentrate in the region of a rigid hold-down device. However, if the bottom plate is rigidly anchored, flexibility in the hold-down device can impose undesirable cross-grain bending forces on the plate due to uplift forces transferred through the sheathing fasteners to the edge of the bottom plate. Further, the sheathing nails in the region of the bottom plate anchor experience greater load and may initiate failure of the wall through an “unzipping” effect.

The proper detailing to balance localized stiffness effects for more even force transfer is obviously a matter of designer judgment. It is mentioned here to emphasize the importance of detailing in wood-framed construction. In particular, wood framing has the innate ability to distribute loads, although weaknesses can develop from seemingly insignificant details. The concern noted above has been attributed to actual problems (i.e., bottom plate splitting) only in severe seismic events and in relatively heavily loaded shear walls. For this reason, it is now common to require larger washers on bottom plate anchor bolts, such as a 2- to 3-inch-square by 1/4-inch-thick plate washer, to prevent the development of cross-grain tension forces in bottom plates in high-hazard seismic regions. The development of high cross-grain tension stresses poses less concern when nails are used to fasten the bottom plate and are located in pairs or staggered on both sides of the wood plate. Thus, the two connection options above represent different approaches. The first, using the plate washers, maintains a rigid connection throughout the wall to prevent cross grain tension in the bottom plate. The second, using nails, is a more “flexible” connection that prevents concentrated cross-grain bending forces from developing. With sufficient capacity provided, the nailing approach may yield a more “ductile” system. Unfortunately, these intricate detailing issues are not accommodated in the single seismic response modifier used for wood-framed shear walls or the provisions of any existing code. These aspects of design are not easily “quantified” and are considered matters of qualitative engineering judgment.

Finally, it is important to recognize that the hold-down must be attached to a vertical wall framing member (i.e., a stud) that receives the wood structural panel edge nailing. If not, the hold-down will not be fully effective (i.e., the overturning forces must be “delivered” to the hold-down through the sheathing panel edge nailing). In addition, the method of deriving hold-down capacity ratings may vary from bracket to bracket and manufacturer to manufacturer. For some brackets, the rated capacity may be based on tests of the bracket itself that do not represent its use in an assembly (i.e., as attached to a wood member). Many hold-down brackets transfer tension through an eccentric load path that creates an end moment on the vertical framing member to which it is attached. Therefore, there may be several design considerations in specifying an appropriate hold-down device that go beyond simply selecting a device with a sufficient rated capacity from manufacturer literature. In response to these issues, some local codes may require certain reductions to or verification of rated hold-down capacities.

6.5.2.5 Shear Transfer (Sliding)

The sliding shear at the base of a shear wall is equivalent to the shear load input to the wall. To ensure that the sliding shear force transfer is balanced with the shear capacity of the wall, the connections at the base of the wall are usually designed to transfer the design unit shear capacity F'_s of the shear wall. Generally, the connections used to resist sliding shear include anchor bolts (fastening to concrete) and nails (fastening to wood framing). Metal plate connectors may also be used (consult manufacturer literature). In what is a conservative decision, frictional resistance and "pinching" effects usually go ignored. However, if friction is considered, a friction coefficient of 0.3 may be multiplied by the dead load normal to the slippage plane to determine a nominal resistance provided by friction.

As a modification to the above rule, if the bottom plate is continuous in a perforated shear wall, the sliding shear resistance is the capacity of the perforated shear wall F_{psw}. If the bottom plate is not continuous, then the sliding shear should be designed to resist the design unit shear capacity of the wall construction F'_s as discussed above. Similarly, if the restrained shear wall segments in a segmented shear wall line are connected to a continuous bottom plate extending between shear wall segments, then the sliding shear can be distributed along the entire length of the bottom plate. For example, if two 4-foot shear wall segments are located in a wall 12 feet long with a continuous bottom plate, then the unit sliding shear resistance required at the bottom plate anchorage is (8 ft)(F'_s)/(12 ft) or 2/3(F'_s). This is similar to the mechanism by which a unit shear load is transferred from a horizontal diaphragm to the wall top plate and then into the shear wall segments through a collector (i.e., top plate). Chapter 7 addresses design of the above types of shear connections.

6.5.2.6 Shear Wall Stiffness and Drift

The methods for predicting shear wall stiffness or drift in this section are based on idealized conditions representative solely of the testing conditions to which the equations are related. The conditions do not account for the many factors that may decrease the actual drift of a shear wall in its final construction. As mentioned, shear wall drift is generally overestimated in comparison with actual behavior in a completed structure (see Section 6.2 on whole-building tests). The degree of overprediction may reach a factor of 2 at design load conditions. At capacity, the error may not be as large because some nonstructural components may be past their yield point.

At the same time, drift analysis may not consider the factors that also increase drift, such as deformation characteristics of the hold-down hardware (for hardware that is less stiff than that typically used in testing), lumber shrinkage (i.e., causing time-delayed slack in joints), lumber compression under heavy shear wall compression chord load, and construction tolerances. Therefore, the results of a drift analysis should be considered as a guide to engineering judgment, not an exact prediction of drift.

The load-drift equations in this section may be solved to yield shear wall resistance for a given amount of shear wall drift. In this manner, a series of shear wall segments or even perforated shear walls embedded within a given wall line may be combined to determine an overall load-drift relationship for the entire wall line. The load-drift relationships are based on the nonlinear behavior of wood-framed shear walls and provide a reasonably accurate means of determining the behavior of walls of various configurations. The relationship may also be used for determining the relative stiffness of shear wall lines in conjunction with the relative stiffness method of distributing lateral building loads and for considering torsional behavior of a building with a nonsymmetrical shear wall layout in stiffness and in geometry. The approach is fairly straightforward and is left to the reader for experimentation.

Perforated Shear Wall Load-Drift Relationship

The load-drift equation below is based on several perforated shear wall tests already discussed in this chapter. It provides a nonlinear load-drift relationship up to the ultimate capacity of the perforated shear wall as determined in Section 6.5.2.2. When considering shear wall load-drift behavior in an actual building, the reader is reminded of the aforementioned accuracy issues; however, accuracy relative to the test data is reasonable (i.e., plus or minus 1/2-inch at capacity).

$$\Delta = 1.8\left(\frac{0.5}{G}\right)\left(\frac{1}{\sqrt{r}}\right)\left(\frac{V_d}{F_{PSW,ULT}}\right)^{2.8}\left[\frac{h}{8}\right] \text{ (inches)} \qquad \text{Eq. 6.5-8}$$

where,

Δ = the shear wall drift (in) at shear load demand, V_d (lb)
G = the specific gravity of framing lumber (see Table 6.6)
r = the sheathing area ratio (see Section 6.5.2.3, C_{op})
V_d = the shear load demand (lb) on the perforated shear wall; the value of V_d is set at any unit shear demand less than or equal to $F_{psw,ult}$ while the value of V_d should be set to the design shear load when checking drift at design load conditions
$F_{psw,ult}$ = the unfactored (ultimate) shear capacity (lb) for the perforated shear wall (i.e., F_{psw} x *SF* or F_{psw}/ϕ for ASD and LRFD, respectively)
h = the height of wall (ft)

Segmented Shear Wall Load-Drift Relationship

APA Semiempirical Load-Drift Equation

Several codes and industry design guidelines specify a deflection equation for shear walls that includes a multipart estimate of various factors' contribution to shear wall deflection (ICBO, 1997; ICC, 1999, APA, 1997). The approach relies on a mix of mechanics-based principles and empirical modifications. The principles and modifications are not repeated here because the APA method of drift prediction is considered no more reliable than that presented next. In addition, the equation is complex relative to the ability to predict drift accurately. It also requires adjustment factors, such as a nail-slip factor, that can only be determined by testing.

Empirical, Nonlinear Load-Drift Equation

Drift in a wood structural panel shear wall segment may be approximated in accordance with the following equation:

$$\Delta = 2.2\left(\frac{0.5}{G}\right)\sqrt[4]{a}\left(\frac{V_d}{F_{SSW,ULT}}\right)^{2.8}\left[\frac{h}{8}\right] \text{ (in)} \qquad \text{Eq. 6.5-9}$$

where,

Δ = the shear wall drift (in) at load V_d (lb)
G = the specific gravity of framing lumber
a = the shear wall segment aspect ratio (height/width) for aspect ratios from 4 to 1; a value of 1 shall be used for shear wall segments with width (length) greater than height
V_d = the shear load demand (lb) on the wall; the value of V_d is set at any unit shear demand less than or equal to $F_{ssw,ult}$ while the value of V_d should be set to the design load when checking drift at design load conditions
$F_{ssw,ult}$ = the unfactored (ultimate) shear capacity (lb) of the shear wall segment (i.e., F_{ssw} x *SF* or F_{ssw}/ϕ for ASD and LRFD, respectively)
h = the height of wall (ft)

The above equation is based on several tests of shear wall segments with aspect ratios ranging from 4:1 to 1:5.

6.5.2.7 Portal Frames

In situations with little space to include sufficient shear walls to meet required loading conditions, the designer must turn to alternatives. An example is a garage opening supporting a two-story home on a narrow lot such that other wall openings for windows and an entrance door leaves little room for shear walls. One option is to consider torsion and the distribution of lateral loads in accordance with the relative stiffness method. Another possibility is the use of a portal frame.

Portal frames may be simple, specialized framing details that can be assembled on site. They use fastening details, metal connector hardware, and sheathing to form a wooden moment frame and, in many cases, perform adequately. Various configurations of portal frames have undergone testing and provide data and details on which the designer can base a design (NAHBRC, 1998; APA, 1994). The ultimate shear capacity of portal frames ranges from 2,400 to more than 6,000 pounds depending on the complexity and strength of the construction details. A simple detail involves extending a garage header so that it is end-nailed to a full-height corner stud, strapping the header to the jamb studs at the portal opening, attaching sheathing with a standard nailing schedule, and anchoring the portal frame with typical perforated shear wall requirements. The system has an ultimate shear capacity of about 3,400 pounds that, with a safety factor of 2 to 2.5, provides a simple solution for many portal frame applications for residential construction in high-hazard seismic or wind regions. Several manufacturers offer preengineered portal frame and shear wall elements that can be ordered to custom requirements or standard conditions.

6.5.3 Diaphragm Design

6.5.3.1 Diaphragm Design Values

Depending on the location and number of supporting shear wall lines, the shear and moments on a diaphragm are determined by using the analogy of a simply supported or continuous span beam. The designer uses the shear load on the diaphragm per unit width of the diaphragm (i.e., floor or roof) to select a combination of sheathing and fastening from a table of allowable horizontal diaphragm unit shear values found in U.S. building codes. Similar to those for shear walls, unit shear values for diaphragms vary according to sheathing thickness and nailing schedules, among other factors. Table 6.8 presents several of the more common floor and roof constructions used in residential construction as well as their allowable diaphragm resistance values. The values include a safety factor for ASD and therefore require no additional factoring. The aspect ratio of a diaphragm should be no greater than 4 (length/width) in accordance with current building code limits. In addition, the sheathing attachment in floor diaphragms is often supplemented with glue or construction adhesive. The increase in unit shear capacity of vertical diaphragms (i.e. shear walls) was discussed in Section 6.5.2.1 in association with Table 6.1. A similar increase to the unit shear capacity of floor diaphragms can be expected, not to mention increased stiffness when the floor sheathing is glued and nailed.

TABLE 6.8 ***Horizontal Diaphragm ASD Shear Values (plf) for Unblocked Roof and Floor Construction Using Douglas Fir or Southern Pine Framing***[1,2,3,4]

Panel Type and Application	Nominal Panel Thickness (inches)	Common Nail Size	Design Shear Value (plf)
Structural I (Roof)	5/16	6d	165
	3/8	8d	185
	15/32	10d	285
APA Sturd-I-Floor (Floor) and Rated Sheathing	7/16	8d	230
	15/32	8d	240
	19/32	10d	285

Notes:
[1]Minimum framing member thickness is 1-1/2 inches.
[2]Nails spaced at 6 inches on-center at supported panel edges and at the perimeter of the diaphragm. Nails spaced at 12 inches on-center on other framing members spaced a maximum of 24 inches on-center.
[3]"Unblocked" means that sheathing joints perpendicular to framing members are not fastened to blocking.
[4]Apply C_{sp} and C_{ns} adjustment factors to table values as appropriate (see Section 6.5.2.3 for adjustment factor values).

6.5.3.2 Diaphragm Design

As noted, diaphragms are designed in accordance with simple beam equations. To determine the shear load on a simply supported diaphragm (i.e., diaphragm supported by shear walls at each side), the designer uses the following equation to calculate the unit shear force to be resisted by the diaphragm sheathing:

$$V_{max} = \frac{1}{2} wl \qquad \text{Eq. 6.5-10a}$$

$$v_{max} = \frac{V_{max}}{d} \qquad \text{Eq. 6.5-10b}$$

where,

V_{max} = the maximum shear load on the diaphragm (plf)
w = the tributary uniform load (plf) applied to the diaphragm resulting from seismic or wind loading
l = the length of the diaphragm perpendicular to the direction of the load (ft)
v_{max} = the unit shear across the diaphragm in the direction of the load (plf)
d = the depth or width of the diaphragm in the direction of the load (ft)

The following equations are used to determine the theoretical chord tension and compression forces on a simply supported diaphragm as described above:

$$M_{max} = \frac{1}{8} wl^2 \qquad \text{Eq. 6.5-11a}$$

$$T_{max} = C_{max} = \frac{M_{max}}{d} \qquad \text{Eq. 6.5-11b}$$

where,

M_{max} = the bending moment on the diaphragm (ft-lb)
w = the tributary uniform load (plf) applied to the diaphragm resulting from seismic or wind loading
l = the length of the diaphragm perpendicular to the direction of the load (ft)
T_{max} = the maximum chord tension force (lb)
C_{max} = the maximum chord compression force (lb)
d = the depth or width of the diaphragm in the direction of the load (ft)

If the diaphragm is not simply supported at its ends, the designer uses appropriate beam equations (see Appendix A) in a manner similar to that above to determine the shear and moment on the diaphragm. The calculations to determine the unit shear in the diaphragm and the tension and compression in the chords are also similar to those given above. It should be noted that the maximum chord forces occur at the location of the maximum moment. For a simply supported diaphragm, the maximum chord forces occur at mid-span between the perimeter shear walls. Thus, chord requirements may vary depending on location and magnitude of the bending moment on the diaphragm. Similarly, shear forces on a simply supported diaphragm are highest near the perimeter shear walls (i.e., reactions). Therefore, nailing requirements for diaphragms may be adjusted depending on the variation of the shear force in interior regions of the diaphragm. Generally, these variations are not critical in small residential structures such that fastening schedules can remain constant throughout the entire diaphragm. If there are openings in the horizontal diaphragm, the width of the opening dimension is usually discounted from the width *d* of the diaphragm when determining the unit shear load on the diaphragm.

6.5.3.3 Shear Transfer (Sliding)

The shear forces in the diaphragm must be adequately transferred to the supporting shear walls. For typical residential roof diaphragms, conventional roof framing connections are often sufficient to transfer the small sliding shear forces to the shear walls (unless heavy roof coverings are used in high-hazard seismic areas or steep roof slopes are used in high-hazard wind regions). The transfer of shear forces from floor diaphragms to shear walls may also be handled by conventional nailed connections between the floor boundary member (i.e., a band joist or end joist that is attached to the floor diaphragm sheathing) and the wall framing below. In heavily loaded conditions, metal shear plates may supplement the connections. The simple rule to follow for these connections is that the shear force in from the diaphragm must equal the shear force out to the supporting wall. Floors supported on a foundation wall are usually connected to a wood sill plate bolted to the foundation wall; however, the floor joist and/or the band joist may be directly connected to the foundation wall. Chapter 7 addresses the design of these shear connections.

6.5.3.4 Diaphragm Stiffness

Diaphragm stiffness may be calculated by using semi-empirical methods based on principles of mechanics. The equations are found in most modern building codes and industry guidelines (APA, 1997; ICBO, 1997; ICC, 1999). For typical residential construction, however, the calculation of diaphragm deflection is almost never necessary and rarely performed. Therefore, the equations and their empirical adjustment factors are not repeated here. Nonetheless, the designer who attempts diaphragm deflection or stiffness calculations is cautioned regarding the same accuracy concerns mentioned for shear wall drift calculations. The stiffness of floor and roof diaphragms is highly dependent on the final construction, including interior finishes (see Section 6.2 on whole-building tests).

6.6 Design Examples

EXAMPLE 6.1 *Segmented Shear Wall Design*

Given

The segmented shear wall line, as shown in the figure below, has the following dimensions:

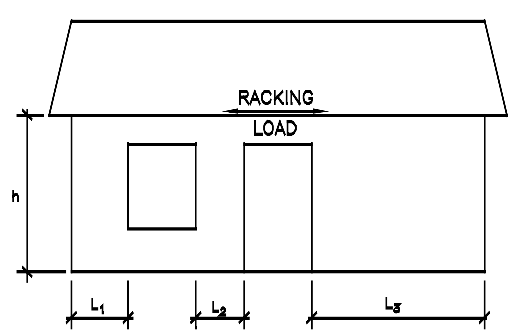
h = 8 ft
L_1 = 3 ft
L_2 = 2 ft
L_3 = 8 ft

Wall construction:

- Exterior sheathing is 7/16-inch-thick OSB with 8d pneumatic nails (0.113 inch diameter by 2 3/8 inches long) spaced 6 inches on center on panel edges and 12 inches on center in panel field
- Interior sheathing is 1/2-inch-thick gypsum wall board with #6 screws at 12 inches on center
- Framing lumber is Spruce-Pine-Fir, Stud grade (specific gravity, G = 0.42); studs are spaced at 16 inches on center.

Loading condition (assumed for illustration)

Wind shear load on wall line = 3,000 lb
Seismic shear load on wall line = 1,000 lb



Find

1. Design capacity of the segmented shear wall line for wind and seismic shear resistance.
2. Base shear connection requirements.
3. Chord tension and compression forces.
4. Load-drift behavior of the segmented shear wall line and estimated drift at design load conditions.

Solution

1. Determine the factored and adjusted (design) shear capacities for the wall segments and the total wall line (Section 6.5.2).

$F_{s,ext}$ = 905 plf OSB sheathing (Table 6.1)
$F_{s,int}$ = 80 plf GWB sheathing (Table 6.3)

The design shear capacity of the wall construction is determined as follows for each segment (Sections 6.5.2.1 and 6.5.2.2):

$F'_s = F'_{s,ext} + F'_{s,int}$
$F'_s = F_{s,ext}\, C_{sp}\, C_{ns}\, C_{ar}\, [1/SF] + F_{s,int}\, C_{ar}\, [1/SF]$

C_{sp} = [1-(0.5-0.42)] = 0.92 (Section 6.5.2.3)
C_{ns} = 0.75 (Table 6.7)
SF = 2.0 (wind) or 2.5 (seismic) (Table 6.5)

<u>Segment 1</u>

a = h/L_1 = (8 ft)/(3 ft) = 2.67 (segment aspect ratio)
C_{ar} = 1/sqrt(0.5(a)) = 0.87 (Section 6.5.2.3)

For wind design

$F'_{s,1,wind}$ = (905 plf)(0.92)(0.75)(0.87)(1/2.0) + (80 plf)(0.87)(1/2.0)
= 272 plf + 35 plf = 307 plf
$F_{ssw,1,wind}$ = $F'_s(L_1)$ = (307 plf)(3 ft) = 921 lb

For seismic design

$F'_{s,1,seismic}$ = (905 plf)(0.92)(0.75)(0.87)(1/2.5) + 0 = 218 plf
$F_{ssw,1,seismic}$ = (218 plf)(3 ft) = 654 lb

<u>Segment 2</u>

a = h/L_2 = (8 ft)/(2 ft) = 4
C_{ar} = 1/sqrt(0.5(a)) = 0.71

For wind design

$F'_{s,2,wind}$ = (905 plf)(0.92)(0.75)(0.71)(1/2.0) + (80 plf)(0.71)(1/2.0)
= 222 plf + 28 plf = 250 plf
$F_{ssw,2,wind}$ = (250 plf)(2 ft) = 500 lb

For seismic design

$F'_{s,2,seismic}$ = (905 plf)(0.92)(0.75)(0.71)(1/2.5) + 0 = 178 plf
$F_{ssw,2,seismic}$ = (178 plf)(2 ft) = 356 lb

Segment 3

$a = h/L_3 = (8 \text{ ft})/(8 \text{ ft}) = 1$
$C_{ar} = 1.0 \quad (\text{for } a < 2)$

For wind design

$F'_{s,3,wind} = (905 \text{ plf})(0.92)(0.75)(1.0)(1/2.0) + (80 \text{ plf})(1.0)(1/2.0)$
$= 312 \text{ plf} + 40 \text{ plf} = 352 \text{ plf}$
$F_{ssw,3,wind} = (352 \text{ plf})(8 \text{ ft}) = 2{,}816 \text{ lb}$

For seismic design

$F'_{s,3,seismic} = (905 \text{ plf})(0.92)(0.75)(1.0)(1/2.5) + 0 = 250 \text{ plf}$
$F_{ssw,3,seismic} = (250 \text{ plf})(8 \text{ ft}) = 2{,}000 \text{ lb}$

Total for wall line

$F_{ssw,total,wind} = 921 \text{ lb} + 500 \text{ lb} + 2{,}816 \text{ lb} = 4{,}237 \text{ lb}$
$F_{ssw,total,seismic} = 654 \text{ lb} + 356 \text{ lb} + 2{,}000 \text{ lb} = 3{,}010 \text{ lb}$

2. Determine base shear connection requirements to transfer shear load to the foundation or floor construction below the wall

The wall bottom plate to the left of the door opening is considered to be continuous and therefore acts as a distributor of the shear load resisted by Segments 1 and 2. The uniform shear connection load on the bottom plate to the left of the opening is determined as follows:

Bottom plate length = 3 ft + 3 ft + 2 ft = 8 ft

Base shear resistance required (wind) $= (F_{ssw,1,wind} + F_{ssw,2,wind})/(\text{plate length})$
$= (921 \text{ lb} + 500 \text{ lb})/(8 \text{ ft}) = 178 \text{ plf}$

Base shear resistance required (seismic) $= (F_{ssw,1,seismic} + F_{ssw,2,seismic})/(\text{plate length})$
$= (654 \text{ lb} + 356 \text{ lb})/(8 \text{ ft}) = 127 \text{ plf}$

For the wall bottom plate to the right of the door opening, the base shear connection is equivalent to $F'_{s,3,wind} = 352$ plf or $F'_{s,3,seismic} = 250$ plf for wind and seismic design respectively.

Normally, this connection is achieved by use of nailed or bolted bottom plate fastenings. Refer to Chapter 7 and Section 7.3.6 for information on designing these connections.

Notes:

1. While the above example shows that variable bottom plate connections may be specified based on differing shear transfer requirements for portions of the wall, it is acceptable practice to use a constant (i.e., worst-case) base shear connection to simplify construction. However, this can result in excessive fastening requirements for certain loading conditions and shear wall configurations.
2. For the assumed wind loading of 3,000 lb, the wall has excess design capacity (i.e., 4,237 lb). The design wind load may be distributed to the shear wall segments in proportion to their design capacity (as shown in the next step for hold-down design) to reduce the shear connection loads accordingly. For seismic design, this should not be done and the base shear connection design should be based on the design capacity of the shear walls to ensure that a "balanced design" is achieved (i.e., the base connection capacity meets or exceeds that of the shear wall). This approach is necessary in seismic design because the actual shear force realized in the connections may be substantially higher than anticipated by the design seismic load calculated using an R factor in accordance with Equation 3.8-1 of Chapter 3. Refer also to the discussion on R factors and overstrength in Section 3.8.4 of Chapter 3. It should be realized that the GWB interior finish design shear capacity was not included in determining the design shear wall capacity for seismic loading. While this is representative of current building code practice, it can create a situation where the actual shear wall capacity and connection forces experienced are higher than those used for design purposes. This condition (i.e., underestimating of the design shear wall capacity) should also be considered in providing sufficiently strong overturning connections (i.e., hold-downs) as covered in the next step.

3. Determine the chord tension and compression (i.e., overturning) forces in the shear wall segments (Section 6.5.2.4)

Basic equation for overturning (Equation 6.5-7c)

$T = C = (d/x)(F'_s)(h)$

<u>Segment 1</u>

h = 8 ft
d = 3 ft
x = d – (width of end studs + offset to center of hold-down anchor bolt)*
= 3 ft – (4.5 in + 1.5 in)(1ft/12 in) = 2.5 ft

*If an anchor strap is used, the offset dimension may be reduced from that determined above assuming a side-mounted hold-down bracket. Also, depending on the number of studs at the end of the wall segment and the type of bracket used, the offset dimension will vary and must be verified by the designer.

$F'_{s,1,wind}$ = 307 plf
$F'_{s,1,seismic}$ = 218 plf

T = C = (3 ft / 2.5 ft)(307 plf)(8 ft) = 2,947 lb (wind)
T = C = (3 ft / 2.5 ft)(218 plf)(8 ft) = 2,093 lb (seismic)

Segment 2

h = 8 ft
d = 2 ft
x = 2 ft – 0.5 ft = 1.5 ft
$F'_{s,2,wind}$ = 250 plf
$F'_{s,2,seismic}$ = 178 plf

T = C = (2 ft / 1.5 ft)(250 plf)(8 ft) = 2,667 lb (wind)
T = C = (2 ft / 1.5 ft)(178 plf)(8 ft) = 1,899 lb (seismic)

Segment 3

h = 8 ft
d = 8 ft
x = 8 ft – 0.5 ft = 7.5 ft
$F'_{s,2,wind}$ = 352 plf
$F'_{s,2,seismic}$ = 250 plf

T = C = (8 ft / 7.5 ft)(352 plf)(8 ft) = 3,004 lb (wind)
T = C = (8 ft / 7.5 ft)(250 plf)(8 ft) = 2,133 lb (seismic)

Notes:

1. In each of the above cases, the seismic tension and compression forces on the shear wall chords are less than that determined for the wind loading condition. This occurrence is the result of using a larger safety factor to determine the shear wall design capacity and the practice of not including the interior sheathing (GWB) design shear capacity for seismic design. Thus, the chord forces based on the seismic shear wall design capacity may be under-designed unless a sufficient safety factor is used in the manufacturer's rated hold-down capacity to compensate. In other words, the ultimate capacity of the hold-down connector should be greater than the overturning force that could be created based on the ultimate shear capacity of the wall, including the contribution of the interior GWB finish. This condition should be verified by the designer since the current code practice may not provide explicit guidance on the issue of balanced design on the basis of system capacity (i.e., connector capacity relative to shear wall capacity). This issue is primarily a concern with seismic design because of the higher safety factor used to determine design shear wall capacity and the code practice not to include the contributing shear capacity of the interior finish.
2. The compression chord force should be recognized as not being a point load at the top of the stud(s) comprising the compression chord. Rather, the compression chord force is accumulated through the sheathing and begins at the top of the wall with a value of zero and increases to C (as determined above) at the base of the compression chord. Therefore, this condition will affect how the compression chord is modeled from the standpoint of determining its capacity as a column using the column equations in the NDS.
3. The design of base shear connections and overturning forces assume that the wind uplift forces at the base of the wall are offset by 0.6 times the dead load (ASD) at that point in the load path or that an additional load path for uplift is provided by metal strapping or other means.
4. As mentioned in Step 2 for the design of base shear connections, the wind load on the designated shear wall segments may be distributed according to the design capacity of each segment in proportion to that of the total shear wall line. This method is particularly useful when the design shear capacity of the wall line is substantially higher than the shear demand required by the wind load as is applicable to this hypothetical example. Alternatively, a shear wall segment may be eliminated from the analysis by not specifying restraining devices for the segment (i.e., hold-down brackets). If the former approach is taken, the wind load is distributed as follows:

Fraction of design wind load to Segment 1:

$F_{ssw,1,wind}/F_{ssw,total,wind}$ = (921 lb)/(4,237 lb) = 0.22

Fraction of wind load to Segment 2:

$F_{ssw,2,wind}/F_{ssw,total,wind}$ = (500 lb)/(4,237 lb) = 0.12

Fraction of wind load to Segment 3:

$F_{ssw,3,wind}/F_{ssw,total,wind}$ = (2,816 lb)/(4,237 lb) = 0.66

Thus, the unit shear load on each shear wall segment due to the design wind shear of 3,000 lb on the total wall line is determined as follows:

Segment 1: 0.22(3,000 lb)/(3 ft) = 220 plf
Segment 2: 0.12(3,000 lb)/(2 ft) = 180 plf
Segment 3: 0.66(3,000 lb)/(8 ft) = 248 plf

Now, the overturning forces (chord forces) determined above and the base shear connection requirements determined in Step 2 may be recalculated by substituting the above values, which are based on the design wind loading. This approach only applies to the wind loading condition when the design wind loading on the wall line is less than the design capacity of the wall line. As mentioned, it may be more efficient to eliminate a designed shear wall segment to bring the total design shear capacity more in line with the design wind shear load on the wall. Alternatively, a lower capacity shear wall construction may be specified to better match the loading condition (i.e., use a thinner wood structural sheathing panel, etc.). This decision will depend on the conditions experienced in other walls of the building such that a single wall construction type may be used throughout for all exterior walls (i.e., simplified construction).

4. Determine the load-drift behavior of the wall line.

Only the load-drift behavior for wind design is shown below. For seismic design, a simple substitution of the design shear capacities of the wall segments and the safety factor for seismic design (as determined previously) may be used to determine a load-drift relationship for use in seismic design.

The basic equation for load-drift estimation of a shear wall segment is as follows:

$$\Delta = 2.2\left(\frac{0.5}{G}\right)\sqrt[4]{a}\left(\frac{V_d}{F_{SSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \quad \text{(Equation 6.5-9)}$$

h = 8 ft
G = 0.42 (Spruce-Pine-Fir)

Aspect ratios for the wall segments

$a_1 = 2.67$
$a_2 = 4.0$
$a_3 = 1.0$

$F_{ssw,ult,1,wind} = F_{ssw,1,wind}(SF)$ = (921 lb)(2.0) = 1,842 lb
$F_{ssw,ult,2,wind} = F_{ssw,2,wind}(SF)$ = (500 lb)(2.0) = 1,000 lb
$F_{ssw,ult,3,wind} = F_{ssw,3,wind}(SF)$ = (2,816 lb)(2.0) = 5,632 lb

Therefore, the total ultimate capacity of the wall for wind loading is

$F_{ssw,ult,wall,wind}$ = 1,842 lb + 1,000 lb + 5,632 lb = 8,474 lb

Substituting the above values into the basic load-drift equation above, the following load-drift equations are determined for each segment:

Segment 1: $\Delta_1 = 2.41\text{x}10^{-9} (V_{d,1,wind})^{2.8}$ (inches)
Segment 2: $\Delta_2 = 1.45\text{x}10^{-8} (V_{d,2,wind})^{2.8}$ (inches)
Segment 1: $\Delta_3 = 2.41\text{x}10^{-10} (V_{d,3,wind})^{2.8}$ (inches)

Realizing that each segment must deflect equally (or nearly so) as the wall line deflects, the above deflections may be set equivalent to the total wall line drift as follows:

$\Delta_{wall} = \Delta_1 = \Delta_2 = \Delta_3$

Further, the above equations may be solved for V_d as follows:

Segment 1: $V_{d,1,wind} = 1{,}196\ (\Delta_{wall})^{0.36}$
Segment 2: $V_{d,2,wind} = 630\ (\Delta_{wall})^{0.36}$
Segment 3: $V_{d,3,wind} = 1{,}997\ (\Delta_{wall})^{0.36}$

The sum of the above equations must equal the wind shear load (demand) on the wall at any given drift of the wall as follows:

$V_{d,wall,wind} = V_{d,1,wind} + V_{d,2,wind} + V_{d,3,wind} = 3{,}823\ (\Delta_{wall})^{0.36}$

Solving for Δ_{wall} , the following final equation is obtained for the purpose of estimating drift and any given wind shear load from zero to $F_{ssw,ult,wall,wind}$:

$\Delta_{wall} = 9.32\text{x}10^{-11}(V_{d,wall,wind})^{2.8}$

For the design wind load on the wall of 3,000 lb as assumed in this example, the wall drift is determined as follows:

$\Delta_{wall} = 9.32\text{x}10^{-11}(3{,}000)^{2.8} = 0.51$ inches

Note: This analysis, as with most other methods of determining drift, may overlook many factors in the as-built construction that serve to increase or decrease drift. As discussed in Section 6.2, whole building tests seem to confirm that drift is generally over-predicted.

Conclusion

In this example, the determination of the design shear capacity of a segmented shear wall was presented for seismic design and wind design applications. Issues related to connection design for base shear transfer and overturning forces (chord tension and compression) were also discussed and calculations were made to estimate these forces using a conventional design approach. In particular, issues related to capacity-based design and "balanced design" of connections were discussed. Finally, a method to determine the load-drift behavior of a segmented shear wall line was presented. The final design may vary based on designer decisions and judgments (as well as local code requirements) related to the considerations and calculations as given in this example.

EXAMPLE 6.2 *Perforated Shear Wall Design*

Given

The perforated shear wall, as shown in the figure below, is essentially the same wall used in Example 6.1. The following dimensions are used:

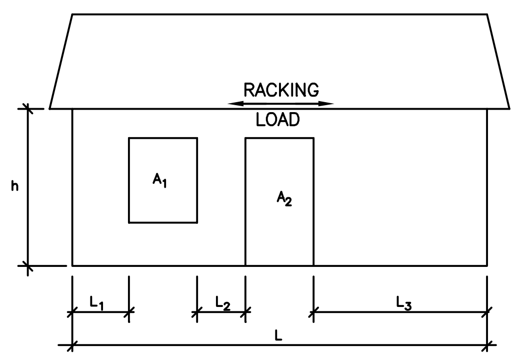
h = 8 ft
L_1 = 3 ft
L_2 = 2 ft
L_3 = 8 ft
L = 19 ft
A_1 = 3.2 ft x 5.2 ft = 16.6 sf (rough window opening area)
A_2 = 3.2 ft x 6.8 ft = 21.8 sf (rough door opening area)

Wall construction:

- Exterior sheathing is 7/16-inch-thick OSB with 8d pneumatic nails (0.113 inch diameter by 2 3/8 inches long) spaced 6 inches on center on panel edges and 12 inches on center in panel field
- Interior sheathing is 1/2-inch-thick gypsum wall board with #6 screws at 12 inches on center
- Framing lumber is Spruce-Pine-Fir, Stud grade (specific gravity, G = 0.42); studs are spaced at 16 inches on center.

Loading condition (assumed for illustration):

Wind shear load on wall line = 3,000 lb
Seismic shear load on wall line = 1,000 lb



Find

1. Design capacity of the perforated shear wall line for wind and seismic shear resistance.
2. Base shear connection requirements.
3. Chord tension and compression forces.
4. Load-drift behavior of the perforated shear wall line and estimated drift at design load conditions.

Solution

1. Determine the factored and adjusted (design) shear capacity for the perforated shear wall line.

$F'_s = F_s\, C_{sp}\, C_{ns}\, [1/SF]$ (Eq. 6.5-1a)

C_{sp} = [1-(0.5-0.42)] = 0.92 (Section 6.5.2.3)
C_{ns} = 0.75 (Table 6.7)
SF = 2.0 (wind design) or 2.5 (seismic design) (Table 6.5)

$F_s = F_{s,ext} + F_{s,int}$ (Section 6.5.2.1)

$F_{s,ext}$ = 905 plf (Table 6.1)
$F_{s,int}$ = 80 plf (Table 6.3)

For wind design

$F_{s,wind}$ = 905 plf + 80 plf = 985 plf
$F'_{s,wind}$ = (985 plf)(0.92)(0.75)(1/2.0) = 340 plf

For seismic design

$F_{s,seismic}$ = 905 plf + 0 plf = 905 plf
$F'_{s,seismic}$ = (905 plf)(0.92)(0.75)(1/2.5) = 250 plf

The design capacity of the perforated shear wall is now determined as follows:

$F_{psw} = F'_s\, C_{op}\, C_{dl}\, L$ (Eq. 6.5-1b)

where,

$C_{op} = r/(3-2r)$
$r = 1/(1+\alpha/\beta)$
$\alpha = \Sigma A_o/(h \times L) = (A_1 + A_2)/(h \times L)$
$= (16.6\text{ sf} + 21.8\text{ sf})/(8\text{ ft})(19\text{ ft}) = 0.25$
$\beta = \Sigma L_i/L = (L_1 + L_2 + L_3)/L$
$= (3\text{ ft} + 2\text{ ft} + 8\text{ ft})/(19\text{ ft}) = 0.68$
$r = 1/(1+0.25/0.68) = 0.73$
$C_{op} = 0.73/(3-2(0.73)) = 0.47$

$C_{dl} = 1 + 0.15(w_D/300) \leq 1.15$

Assume for the sake of this example that the roof dead load supported at the top of the wall is 225 plf and that the design wind uplift force on the top of the wall is 0.6(225 plf) – 400 plf = -265 plf (net design uplift). Thus, for wind design in this case, no dead load can be considered on the wall and the C_{dl} factor does not apply for calculation of the perforated shear wall resistance to wind loads. It does apply to seismic design, as follows:

$w_D = 0.6*(225 \text{ plf}) = 135 \text{ plf}$

*The 0.6 factor comes from the load combinations 0.6D + (W or 0.7E) or 0.6D – W_u as given in Chapter 3.

$C_{dl} = 1 + 0.15(135/300) = 1.07$

For wind design,

$F_{psw,wind} = (340 \text{ plf})(0.47)(1.0)(19 \text{ ft}) = 3{,}036 \text{ lb}$

For seismic design,

$F_{psw,seismic} = (250 \text{ plf})(0.47)(1.07)(19 \text{ ft}) = 2{,}389 \text{ lb}$

Note: In Example 6.1 using the segmented shear wall approach, the design shear capacity of the wall line was estimated as 4,237 lb (wind) and 3,010 lb (seismic) when all of the segments were restrained against overturning by use of hold-down devices. However, given that the design shear load on the wall is 3,000 lb (wind) and 1,000 lb (seismic), the perforated shear wall design capacity as determined above is adequate, although somewhat less than that of the segmented shear wall. Therefore, hold-downs are only required at the wall ends (see Step 3).

2. Determine the base shear connection requirement for the perforated shear wall.

If the wall had a continuous bottom plate that serves as a distributor of the shear forces resisted by various portions of the wall, the base shear connection could be based on the perforated shear wall's design capacity as determined in Step 1 as follows:

For wind design,

Uniform base shear = (3,036 lb)/19 ft = 160 plf

For seismic design,

Uniform base shear = (2,389 lb)/19 ft = 126 plf

However, <u>the wall bottom plate is not continuous in this example</u> and, therefore, the base shears experienced by the portions of the wall to the left and right of the door opening are different as was the case in the segmented shear wall design approach of Example 6.1. <u>As a conservative solution, the base shear connection could be designed to resist the design unit shear capacity of the wall construction, $F'_{s,wind}$ = 340 plf or $F'_{s,seismic}$ = 250 plf.</u> Newer codes that recognize the perforated shear method may require this more conservative approach to be used when the bottom plate is not continuous such that it serves as a distributor (i.e., similar in function to a shear wall collector except shear transfer is out of the wall instead of into the wall). Of course, the bottom plate must be continuous and any splices must be adequately detailed in a fashion similar to collectors (see Example 6.3).

As an alternative, the portion of the wall to the left of the door opening can be treated as a separate perforated shear wall for the left-to-right loading condition. In doing so, the design shear capacity of the left portion of the wall may be determined to be 1,224 lb and the base shear connection required is (1,224 lb)/8ft = 153 plf, much less than the 340 lb required in the wind load condition. The right side of the wall is solid sheathed and, for the right-to-left loading condition, the base shear is equivalent to the design shear capacity of the wall or 340 plf. These calculations can also be performed using the seismic design values for the perforated shear wall. This approach is based on the behavior of a perforated shear wall where the leading edge and the immediately adjacent shear wall segments are fully restrained as in the segmented shear wall approach for one direction of loading. Thus, these segments will realize their full unit shear capacity for one direction of loading. Any interior segments will contribute, but at a reduced amount do to the reduced restraint condition. This behavior is represented in the adjustment provided by the C_{op} factor which is the basis of the perforated shear wall method. Unfortunately, the exact distribution of the uplift forces and shear forces within the wall are not known. It is for this reason that they are assigned conservative values for design purposes. Also, to accommodate potential uplift forces on the bottom plate in the regions of interior perforated shear wall segments, <u>the base shear connections are required to resist an uplift load equivalent to the design unit shear capacity of the wall construction</u>. In the case of this example, the base shear connection would need to resist a shear load of 340 plf (for the wind design condition) and an uplift force of 340 plf (even if under a zero wind uplift load).

Testing has shown that for walls constructed similar to the one illustrated in this example, a bottom plate connection of 2 16d pneumatic nails (0.131 inch diameter by 3 inches long) at 16 inches on center or 5/8-inch-diameter anchor bolts at 6 feet on center provides suitable shear and uplift resistance – at least equivalent to the capacity of the shear wall construction under conditions of no dead load or wind uplift (NAHBRC, 1999). For other conditions, this connection must be designed following the procedures given in Chapter 7 using the conservative assumptions as stated above.

As an alternative base connection that eliminates the need for hold-down brackets at the ends of the perforated shear wall, straps can be fastened to the individual studs to resist the required uplift force of 340 plf as applicable to this example. If the studs are spaced 16 inches on center, the design capacity of the strap must be (340 plf)(1.33 ft/stud) = 452 lb per stud. If an uplift load due to wind uplift on the roof must also be transferred through these straps, the strap design capacity must be increased accordingly. In this example, the net wind uplift at the top of the wall was assumed to be 265 plf. At the base of the wall, the uplift is 265 plf – 0.6(8 ft)(8 psf) = 227 plf. Thus, the total design uplift restraint must provide 340 plf + 227 plf = 567 plf. On a per stud basis (16 inch on center framing), the design load is 1.33 ft/stud x 567 plf = 754 lb/stud. This value must be increased for studs adjacent to wall openings where the wind uplift force in increased. This can be achieved by using multiple straps or by specifying a larger strap in these locations. Of course, the above combination of uplift loads assumes that the design wind uplift load on the roof occurs simultaneously with the design shear load on the wall. However, this condition is not usually representative of actual conditions depending on wind orientation, building configuration, and the shear wall location relative to the uplift load paths.

3. Determine the chord tension and compression forces

The chord tension and compression forces are determined following the same method as used in Example 6.1 for the segmented shear wall design method, but only for the first wall segment in the perforated shear wall line (i.e. the restrained segment). Therefore, the tension forces at the end of the wall are identical to those calculated in Example 6.1 as shown below:

Left end of the wall (Segment 1 in Example 6.1):

T = 2,947 lb (wind design)
T = 2,093 lb (seismic design)

Right end of the wall (Segment 3 in Example 6.1):

T = 3,004 lb (wind design)
T = 2,133 lb (seismic design)

Note: One tension bracket (hold-down) is required at each the end of the perforated shear wall line and not on the interior segments. Also, refer to the notes in Example 6.1 regarding "balanced design" of overturning connections and base shear connections, particularly when designing for seismic loads.

4. Determine the load-drift behavior of the perforated shear wall line.

The basic equation for load-drift estimation of a perforated shear wall line is as follows (Section 6.5.2.6):

$$\Delta = 1.8\left(\frac{0.5}{G}\right)\left(\frac{1}{\sqrt{r}}\right)\left(\frac{V_d}{F_{PSW,ULT}}\right)^{2.8}\left(\frac{h}{8}\right) \qquad \text{(Eq. 6.5-8)}$$

h = 8 ft
G = 0.42(specific gravity for Spruce-Pine-Fir)
r = 0.73 (sheathing area ratio determined in Step 1)

$F_{psw,ult,wind}$ = $(F_{psw,wind})$(SF) = (3,036 lb)(2.0) = 6,072 lb
$F_{psw,ult,seismic}$ = $(F_{psw,seismic})$(SF) = (2,389 lb)(2.5) = 5,973 lb

Substituting in the above equation,

$\Delta_{wind} = 6.4\text{x}10^{-11}(V_{d,wind})^{2.8}$
$\Delta_{seismic} = 6.7\text{x}10^{-11}(V_{d,seismic})^{2.8}$

For the design wind load of 3,000 lb and the design seismic load of 1,000 lb (assumed for the purpose of this example), the drift is estimated as follows:

$\Delta_{wind} = 6.4\text{x}10^{-11}(3{,}000)^{2.8} = 0.35$ inches
$\Delta_{seismic} = 6.7\text{x}10^{-11}(1{,}000)^{2.8} = 0.02$ inches

Note: The reader is reminded of the uncertainties in determining drift as discussed in Example 6.1 and also in Chapter 6. For seismic design, some codes may require the design seismic drift to be amplified (multiplied by) a factor of 4 to account for the potential actual forces that may be experienced relative to the design forces that are determined using an R factor; refer to Chapter 3 for additional discussion. Thus, the amplified drift may be determined as 4 x 0.02 inches = 0.08 inches. However, if the seismic shear load is magnified (i.e., 4 x 1,000 lb = 4,000 lb) to account for a possible actual seismic load (not modified for the seismic response of the shear wall system), the seismic drift calculated in the above equation becomes 0.8 inches which is an order of magnitude greater. The load adjustment is equivalent to the use of an R of 1.5 instead of 6 in Equation 3.8-1 of Chapter 3. However, this latter approach of magnifying the load is not currently required in the existing building codes for drift determination. As mentioned, drift is not usually considered in residential design. Finally, the above equations may be used to determine a load-drift curve for a perforated shear wall for values of V_d ranging from 0 to $F_{psw,ult}$. While the curve represents the non-linear behavior of a perforated shear wall, it should only be considered as a representation, and not an exact solution.

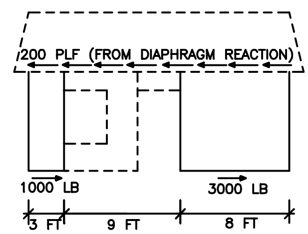
Conclusion

In this example, the determination of the design shear capacity of a perforated shear wall was presented for seismic design and wind design applications. Issues related to connection design for base shear transfer and overturning forces (chord tension) were also discussed and calculations (or conservative assumptions) were made to estimate these forces. In particular, issues related to capacity-based design and "balanced design" of connections were discussed. Finally, a method to determine the load-drift behavior of a perforated shear wall line was presented. The final design may vary based on designer decisions and judgments (as well as local code requirements) related to the considerations and calculations as given in this example.

EXAMPLE 6.3 *Shear Wall Collector Design*

Given

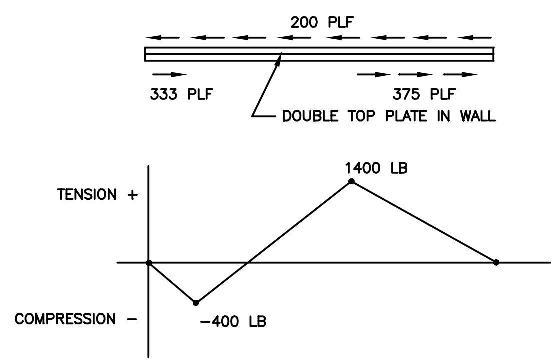
The example shear wall, assumed loading conditions, and dimensions are shown in the figure below.



Find The maximum collector tension force

Solution

1. The collector force diagram is shown below based on the shear wall and loading conditions in the figure above.



The first point at the interior end of the left shear wall segment is determined as follows:

200 plf (3 ft) – 333 plf (3 ft) = - 400 lb (compression force)

The second point at the interior end of the right shear wall segment is determined as follows:

- 400 lb + 200 plf (9 ft) = 1,400 lb (tension force)

The collector load at the right-most end of the wall returns to zero as follows:

1,400 lb – 375 plf (8 ft) + 200 plf (8 ft) = 0 lb

Conclusion

The maximum theoretical collector tension force is 1,400 lb at the interior edge of the 8-foot shear wall segment. The analysis does not consider the contribution of the "unrestrained" wall portions that are not designated shear wall segments and that would serve to reduce the amount of tension (or compression) force developed in the collector. In addition, the load path assumed in the collector does not consider the system of connections and components that may share load with the collector (i.e., wall sheathing and connections, floor or roof construction above and their connections, etc.). Therefore, the collector load determined by assuming the top plate acts as an independent element can be considered very conservative depending on the wall-floor/roof construction conditions. Regardless, it is typical practice to design the collector (and any splices in the collector) to resist a tension force as calculated in this example. The maximum compressive force in the example collector is determined by reversing the loading direction and is equal in magnitude to the maximum tension force. Compressive forces are rarely a concern when at least a double top plate is used as a collector, particularly when the collector is braced against lateral buckling by attachment to other construction (as would be generally necessary to deliver the load to the collector from somewhere else in the building).

EXAMPLE 6.4 *Horizontal (Floor) Diaphragm Design*

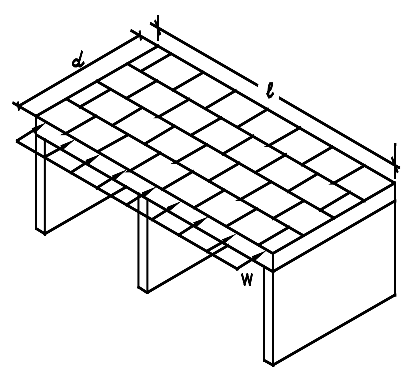
Given

The example floor diaphragm and its loading and support conditions are shown in the figure below. The relevant dimensions and loads are as follows:

d = 24 ft
l = 48 ft
w = 200 plf (from wind or seismic lateral load)*

*Related to the diaphragm's tributary load area; see Chapter 3 and discussions in Chapter 6.

The shear walls are equally spaced and it is assumed that the diaphragm is flexible (i.e. experiences beam action) and that the shear wall supports are rigid. This assumption is not correct because the diaphragm may act as a "deep beam" and distribute loads to the shear wall by "arching" action rather than bending action. Also, the shear walls cannot be considered to be perfectly rigid or to exhibit equivalent stiffness except when designed exactly the same with the same interconnection stiffness and base support stiffness. Regardless, the assumptions made in this example are representative of typical practice.



Find

1. The maximum design unit shear force in the diaphragm (assuming simple beam action) and the required diaphragm construction.
2. The maximum design moment in the diaphragm (assuming simple beam action) and the associated chord forces.

Solution

1. The maximum shear force in the diaphragm occurs at the center shear wall support. Using the beam equations in Appendix A for a 2-span beam, the maximum shear force is determined as follows:

$$V_{max} = \frac{5}{8} w\left(\frac{l}{2}\right) = \frac{5}{8}(200\ \text{plf})\left(\frac{48\ \text{ft}}{2}\right) = 3{,}000\ \text{lb}$$

The maximum design unit shear in the diaphragm is determined as follows:

$$v_{max} = \frac{V_{max}}{d} = \frac{3{,}000\ \text{lb}}{24\ \text{ft}} = 125\ \text{plf}$$

From Table 6.8, the lightest unblocked diaphragm provides adequate resistance. Unblocked means that the panel edges perpendicular to the framing (i.e., joists or rafters) are not attached to blocking. The perimeter, however, is attached to a continuous member to resist chord forces. For typical residential floor construction a 3/4-inch-thick subfloor may be used which would provide at least 240 plf of design shear capacity. In typical roof construction, a minimum 7/16-inch-thick sheathing is used which would provide about 230 plf of design shear capacity. However, residential roof construction does not usually provide the edge conditions (i.e., continuous band joist of 2x lumber) associated with the diaphragm values in Table 6.8. Regardless, roof diaphragm performance has rarely (if ever) been a problem in light-frame residential construction and these values are often used to approximate roof diaphragm design values.

Note: The shear forces at other regions of the diaphragm and at the locations of the end shear wall supports can be determined in a similar manner using the beam equations in Appendix A. These shear forces are equivalent to the connection forces that must transfer shear between the diaphragm and the shear walls at the ends of the diaphragm. However, for the center shear wall, the reaction (connection) force is twice the unit shear force in the diaphragm at that location (see beam equations in Appendix A). Therefore, the connection between the center shear wall and the diaphragm in this example must resist a design shear load of 2 x 125 plf = 250 plf. However, this load is very dependent on the assumption of a "flexible" diaphragm and "rigid" shear walls.

2. The maximum moment in the diaphragm also occurs at the center shear wall support. Using the beam equations in Appendix A, it is determined as follows:

$$M_{max} = \frac{1}{8}w\left(\frac{l}{2}\right)^2 = \frac{1}{8}(200\text{plf})\left(\frac{48\text{ft}}{2}\right)^2 = 14{,}400\text{ft} - \text{lb}$$

The maximum chord tension and compression forces are at the same location and are determined as follows based on the principle of a force couple that is equivalent to the moment:

$$T = C = \frac{M_{max}}{d} = \frac{14{,}400\text{ft} - \text{lb}}{24\text{ft}} = 600\text{lb}$$

Therefore, the chord members (i.e., band joist and associated wall or foundation framing that is attached to the chord) and splices must be able to resist 600 lb of tension or compression force. Generally, these forces are adequately resisted by the framing systems bounding the diaphragm. However, the adequacy of the chords should be verified by the designer based on experience and analysis as above.

Conclusion

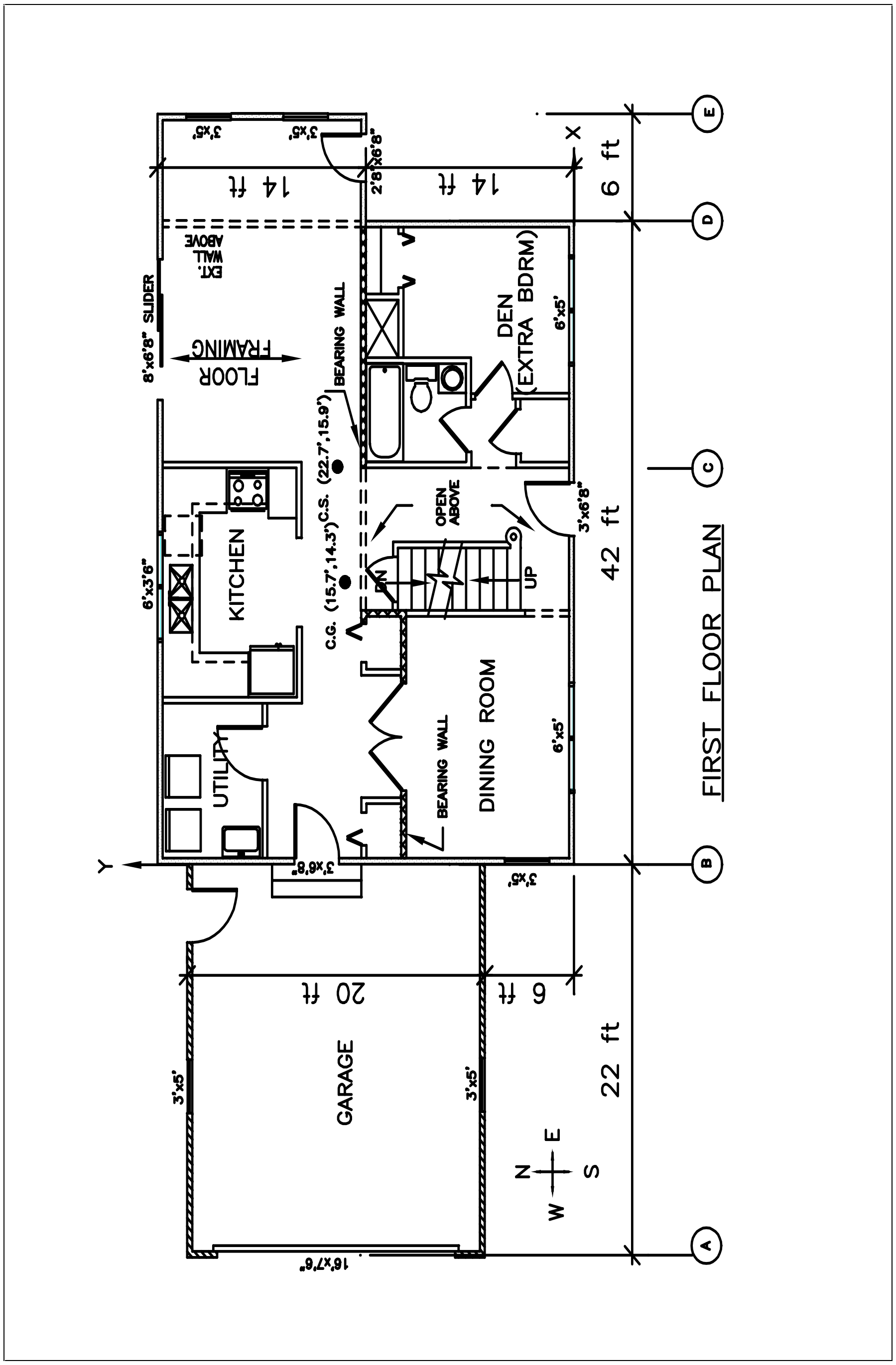
In this example, the basic procedure and principles for horizontal diaphragm design were presented. Assumptions required to conduct a diaphragm analysis based on conventional beam theory were also discussed.

EXAMPLE 6.5 *Horizontal Shear Load Distribution Methods*

Given

General

In this example, the first floor plan of a typical two-story house with an attached garage (see Figure below) is used to demonstrate the three methods of distributing shear loads discussed in Chapter 6, Section 6.4.2. The first story height is 8 ft (i.e., 8 ft ceiling height). Only the load in the North-South (N-S) direction is considered in the example. In a complete design, the load in the East-West (E-W) direction would also need to be considered.



Lateral Load Conditions

The following design N-S lateral loads are determined for the story under consideration using the methods described in Chapter 3 for wind and seismic loads. A fairly high wind load and seismic load condition is assumed for the purpose of the example.

Design N-S Wind Lateral Load (120 mph gust, exposure B)

House:	17,411	lb total story shear
Garage:	3,928	lb total story shear
Total:	21,339	lb

Design N-S Seismic Lateral Load (mapped S_s = 1.5g)

House:	7,493	lb total story shear (tributary weight is 37,464 lb)
Garage:	1,490	lb total story shear (tributary weight is 7,452 lb)
Total:	8,983	lb

Designation of Shear Walls in N-S Direction

Initially, there are four N-S lines designated in the first story for shear wall construction. The wall lines are A, B, D, and E. If needed, an interior wall line may also be designated and designed as a shear wall (see wall line C in the figure above).

The available length of full-height wall segments in each N-S shear wall line is estimated as follows from the floor plan:

Wall Line A:	2 ft + 2 ft	= 4 ft	(garage return walls)
Wall Line B:	~~1.33 ft~~* + 11 ft + 9 ft	= 20 ft	(garage/house shared wall)
Wall Line D:	14 ft	= 14 ft	(den exterior wall)
Wall Line E:	2 ft + 3 ft + 2 ft	= 7 ft	(living room exterior wall)
Total:		= 45 ft	

*The narrow 1.33 ft segment is not included in the analysis due to the segment's aspect ratio of 8 ft/1.33 ft = 6, which is greater than the maximum allowable of 4. Some current building codes may restrict the segment aspect ratio to a maximum of 2 or 3.5 depending on the code and the edition in local use. In such a case, many of the useable shear wall segments would be eliminated (i.e., all of the 2 ft segments). Thus, the garage opening wall would require larger segments, a portal frame (see Section 6.5.2.7), or transfer of the garage shear load to the house by torsion (i.e., treat the garage as a cantilever projecting from the house under a uniform lateral load).

Find

1. Using the "total shear method" of horizontal shear load distribution, determine the total length of shear wall required and the required shear wall construction in the N-S direction.
2. Using the "tributary area method" of horizontal shear load distribution, determine the shear resistance and wall construction required in each N-S shear wall line.
3. Using the "relative stiffness method" of horizontal shear load distribution, determine the shear loads on the N-S shear wall lines.

Solution

1. Using the total shear approach, determine the unit shear capacity required based on the given amount of available shear wall segments in each N-S wall line and the total N-S shear load.

In this part of the example, it is assumed that the wall lines will be designed as segmented shear wall lines. From the given information, the total length of N-S shear wall available is 45 ft. It is typical practice in this method to not include segments with aspect ratios greater than 2 since stiffness effects on the narrow segments are not explicitly considered. This would eliminate the 2 ft segments and the total available length of shear wall would be 45 ft – 8 ft = 37 ft in the N-S direction.

The required design unit shear capacity of the shear wall construction and ultimate capacity is determined as follows for the N-S lateral design loads:

Wind N-S

$F'_{s,wind}$ = (21,339 lb)/37 ft = 576 plf
$F_{s,\ wind} = (F'_{s,wind})(SF)$ = (576 plf)(2.0) = 1,152 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the shear walls must meet or exceed 1,152 plf. Assuming that standard 1/2-thick GWB finish is used on the interior wall surfaces (80 plf minimum from Table 6.3), the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,wind} = F_{s,ext} + F_{s,int}$
$F_{s,ext}$ = 1,152 plf – 80 plf = 1,072 plf

From Table 6.1, any of the wall constructions that use a 4 inch nail spacing at the panel perimeter exceed this requirement. By specifying and 3/8-thick Structural I wood structural panel with 8d common nails spaced at 4 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors. For example, assume the following framing lumber and nails are used in the shear wall construction:

lumber species:	Spruce-Pine-Fir (G=0.42)	C_{sp} = 0.92
nail type:	8d pneumatic, 0.113-inch-diameter	C_{ns} = 0.75

Thus, values in Table 6.1 would need to be multiplied by (0.92)(0.75) = 0.69. This adjustment requires a 15/32-inch-thick sheathing with the 8d nails (i.e., 1,539 plf x 0.69 = 1,062 plf which is close enough to the required 1,072 plf for practical design purposes). Alternatively, a 7/16-inch thick wood structural panel sheathing could be used in accordance with footnote 5 of Table 6.1; however, the horizontal joint between panels would need to be blocked. In extreme lateral load conditions, it may be necessary (and more efficient) to consider a "double sheathed" wall construction (i.e., structural wood panels on both sides of the wall framing) or to consider the addition of an interior shear wall line (i.e., design the interior walls along wall line C as shear walls).

Seismic N-S

$F'_{s,seismic}$ = (8,983 lb)/37 ft = 243 plf
$F_{s,\ seismic}$ = ($F'_{s,seismic}$)(SF) = (243 plf)(2.5) = 608 plf

Thus, the unfactored (ultimate) and unadjusted unit shear capacity for the wall line must meet or exceed 608 plf. Since seismic codes do not permit the consideration of a 1/2-thick GWB interior finish, the required ultimate capacity of the exterior sheathing is determined as follows:

$F_{s,seismic} = F_{s,ext}$ = 608 plf

From Table 6.1, any of the wood structural panel wall constructions that use a 6 inch nail spacing at the panel perimeter exceed this requirement. By specifying 3/8-inch-thick Structural I wood structural panels with 8d common nails spaced at 6 inches on center on the panel edges (12 inches on center in the panel field), the design of the wall construction is complete and hold-down connections and base shear connections must be designed. If a different nail is used or a framing lumber species with G < 0.5, then the values in Table 6.1 must be multiplied by the C_{ns} and C_{sp} factors as demonstrated above for the N-S wind load case.

The base shear connections may be designed in this method by considering the total length of continuous bottom plate in the N-S shear wall lines. As estimated from the plan, this length is approximately 56 feet. Thus, the base connection design shear load (parallel to the grain of the bottom plate) is determined as follows:

Base wind design shear load = (21,339 lb)/(56 ft) = 381 plf
Base seismic design shear load = (8,983 lb)/(56 ft) = 160 plf

The base shear connections may be designed and specified following the methods discussed in Chapter 7 – Connections. A typical 5/8-inch-diameter anchor bolt spaced at 6 feet on center or standard bottom plate nailing may be able to resist as much as 800 plf (ultimate shear capacity) which would provided a "balanced" design capacity of 400 plf or 320 plf for wind and seismic design with safety factors of 2.0 and 2.5, respectively. Thus, a conventional wall bottom plate connection may be adequate for the above condition; refer to Chapter 7 for connection design information and the discussion in Section 7.3.6 for more details on tested bottom plate connections.

If the roof uplift load is not completely offset by 0.6 times the dead load at the base of the first story wall, then strapping to transfer the net uplift from the base of the wall to the foundation or construction below must be provided.

The hold-down connections for the each shear wall segment in the designated shear wall lines are designed in the manner shown in Example 6.1. Any overturning forces originating from shear walls on the second story must also be included as described in Section 6.4.2.4.

Notes:

1. The contribution of the interior walls to the lateral resistance is neglected in the above analysis for wind and seismic loading. As discussed in Chapter 6, these walls can contribute significantly to the lateral resistance of a home and serve to reduce the designated shear wall loads and connection loads through alternate, "non-designed" load paths. In this example, there is approximately 40 ft of interior partition walls in the N-S direction that each have a minimum length of about 8 ft or more (small segments not included). Assuming a design unit shear value of 80 plf / 2 = 40 plf (safety factor of 2), the design lateral resistance may be at least 40 ft x 40 plf = 1,600 lb. While this is not a large amount, it should factor into the design consideration, particularly when a lateral design solution is considered to be marginal based on an analysis that does not consider interior partition walls.
2. Given the lower wind shear load in the E-W direction, the identical seismic story shear load in the E-W direction, and the greater available length of shear wall in the E-W direction, an adequate amount of lateral resistance should be no problem for shear walls in the E-W direction. It is probable that some of the available E-W shear wall segments may not even be required to be designed and detailed as shear wall segments. Also, with hold-down brackets at the ends of the N-S walls that are detailed to anchor a common corner stud (to which the corner sheathing panels on each wall are fastened with the required panel edge fastening), the E-W walls are essentially perforated shear wall lines and may be treated as such in evaluating the design shear capacity of the E-W wall lines.
3. The distribution of the house shear wall elements appears to be reasonably "even" in this example. However, the garage opening wall could be considered a problem if sufficient connection of the garage to the house is not provided to prevent the garage from rotating separately from the house under the N-S wind or seismic load. Thus, the garage walls and garage roof diaphragm should be adequately attached to the house so that the garage and house act as a structural unit. This process will be detailed in the next part of this example.

2. Determine the design shear load on each wall line based on the tributary area method.

Following the tributary area method of horizontal force distribution, the loads on the garage and the house are treated separately. The garage lateral load is assumed to act through the center of the garage and the house load is assumed to act through the center of the house. The extension of the living room on the right side of the plan is only one story and is considered negligible in its impact to the location of the real force center; although, this may be considered differently by the designer. Therefore, the lateral force (load) center on the garage is considered to act in the N-S direction at a location one-half the distance between wall lines A and B (see the given floor plan diagram). Similarly, the N-S force center on the house may be considered to act half-way between wall lines B and D (or perhaps a foot or less farther to the right to compensate for the living room "bump-out"). Now, the N-S lateral design loads are assigned to wall lines A, B, and D/E as follows:

<u>Wall Line A</u>

Wind design shear load = 1/2 garage shear load = 0.5(3,928 lb) = 1,964 lb
Seismic design shear load = 0.5(1,490 lb) = 745 lb

<u>Wall Line B</u>

Wind design shear load = 1/2 garage shear load + 1/2 house shear load
= 1,964 lb + 0.5(17,411 lb) = 10,670 lb
Seismic design shear load = 745 lb + 0.5(7,493 lb) = 4,492 lb

Wall Line D/E

Wind design shear load = 1/2 house shear load = 0.5(17,411 lb) = 8,706 lb
Seismic design shear load = 0.5(7,493 lb) = 3,747 lb

Based on the design shear loads above, each of the wall lines may be designed in a fashion similar to that used in Step 1 (total shear method) by selecting the appropriate wall construction to meet the loading demand. For example, the design of wall line B would proceed as shown below (using the perforated shear wall method in this case) for the required wind shear load.

The following equations are used to determine the required ultimate shear capacity, F_s, of the wall construction (interior and exterior sheathing type and fastening):

$F'_s = [(F_{s,ext})(C_{sp})(C_{ns}) + F_{s,int}]x[1/SF]$ (based on Eq. 6.5-1a)
$F_{psw} = F'_s\, C_{op}\, C_{dl}\, [L]$ (Eq. 6.5-1b)

Substituting the first equation above into the second,

$$F_{psw} = [(F_{s,ext})(C_{sp})(C_{ns}) + F_{s,int}]\ [1/SF]\ C_{op}\ C_{dl}\ [L]$$

To satisfy the design wind shear load requirement for Wall Line B,

$$F_{psw} \geq 10{,}670 \text{ lb}$$

Assume that the wall construction is the same as used in Example 6.2. The following parameters are determined for Wall Line B:

$C_{sp} = 0.92$ (Spruce-Pine-Fir)
$C_{ns} = 0.75$ (8d pneumatic nail, 0.113-inch-diameter)
$C_{dl} = 1.0$ (zero dead load due to wind uplift)
SF = 2.0 (wind design safety factor)
$C_{op} = 0.71$ (without the corner window and narrow segment)*
L = 28 ft – 1.33 ft – 3 ft = 23.67 ft (length of perforated shear wall line)*
$F_{s,int} = 80$ plf (Table 6.3, minimum ultimate unit shear capacity)

*The perforated shear wall line begins at the interior edge of the 3’ x 5’ window opening because the wall segment adjacent to the corner exceeds the maximum aspect ratio requirement of 4. Therefore, the perforated shear wall is “embedded” in the wall line.

Substituting the values above into the equation for F_{psw} , the following value is obtained for $F_{s,ext}$:

$$10{,}670 \text{ lb} = [(F_{s,ext})(0.92)(0.75) + 80 \text{ plf}]\ [1/2.0]\ (0.71)\ (1.0)\ [23.67 \text{ ft}]$$

$$F_{s,ext} = 1{,}724 \text{ plf}$$

By inspection in Table 6.1, the above value is achieved for a shear wall constructed with 15/32-inch-thick Structural 1 wood structural panel sheathing with nails spaced at 3 inches on the panel edges. The value is 1,722 plf which is close enough for practical purposes (particularly given that contribution of interior walls is neglected in the above analysis). Also, a thinner sheathing may be used in accordance with Footnote 5 of Table 6.1. As another alternative, wall line B could be designed as a segmented shear wall. There are two large shear wall segments that may be used. In total they are 20 ft long. Thus, the required ultimate shear capacity for wall line B using the segmented shear wall method is determined as follows:

$F'_s = (F_{s,ext} C_{sp} C_{ns} + F_{s,int}) C_{ar} [1/SF]$	(based on Eq. 6.5-2a)
$F_{ssw} = F'_s \times L$	(Eq. 6.5-2b)
$F_{ssw} \geq 10{,}670$ lb	(wind load requirement on wall line B)

Substituting the first equation into the second

$$F_{ssw} = (F_{s,ext} C_{sp} C_{ns} + F_{s,int}) C_{ar} [1/SF] \times L$$

The following parameter values are used:

$C_{sp} = 0.92$	(same as before)
$C_{ns} = 0.75$	(same as before)
$C_{ar} = 1.0$	(both segments have aspect ratios less than 2)*
SF = 2.0	(for wind design)
L = 20 ft	(total length of the two shear wall segments)*
$F_{s,int} = 80$ plf	(minimum ultimate unit shear capacity)

*If the wall segments each had different values for C_{ar} because of varying adjustments for aspect ratio, then the segments must be treated independently in the equation above and the total length could not be summed as above to determine a total L.

Now, solving the above equations for $F_{s,ext}$ the following is obtained:

$$10{,}670 \text{ lb} = [(F_{s,ext})(0.92)(0.75) + 80 \text{ plf}](1.0)[1/2.0](20 \text{ ft})$$

$$F_{s,ext} = 1{,}430 \text{ plf}$$

By inspection of Table 6.1 using the above value of $F_{s,ext}$, a 4 inch nail spacing may be used to meet the required shear loading in lieu of the 3 inch nail spacing used if the wall were designed as a perforated shear wall. However, two additional hold down brackets would be required in Wall Line B to restrain the two wall segments as required by the segmented shear wall design method.

Wall Line A poses a special design problem since there are only two narrow shear wall segments to resist the wind design lateral load (1,964 lb). Considering the approach above for the segmented shear wall design of Wall Line B and realizing that $C_{ar} = 0.71$ (aspect ratio of 4), the following value for $F_{s,ext}$ is obtained for Wall Line A:

$$F_{ssw} = (F_{s,ext} C_{sp} C_{ns} + F_{s,int}) C_{ar} [1/SF] \times L$$

$$1{,}964 \text{ lb} = [(F_{s,ext})(0.92)(0.75) + 0^*](0.71)[1/2.0](4 \text{ ft})$$

*The garage exterior walls are assumed not to have interior finish. The shared wall between the garage and the house, however, is required to have a fire rated wall which is usually satisfied by the use of 5/8-thick gypsum wall board. This fire resistant finish is placed over the wood structural sheathing in this case and the impact on wall thickness (i.e. door jamb width) should be considered by the architect and builder.

Solving for $F_{s,ext}$,

$$F_{s,ext} = 2{,}004 \text{ plf}$$

By inspecting Table 6.1, this would require 15/32-inch-thick wood structural panel with nails spaced at 2 inches on center and would require 3x framing lumber (refer to footnote 3 of Table 6.1). However, the value of C_{ns} (=0.75) from Table 6.7 was based on a 0.113-inch diameter nail for which the table does not give a conversion relative to the 10d common nail required in Table 6.1. Therefore, a larger nail should be used at the garage opening. Specifying an 8d common nail or similar pneumatic nail with a diameter of 0.131 inches (see Table 6.7), a C_{ns} value of 1.0 is used and $F_{s,ext}$ may be recalculated as above to obtain the following:

$$F_{ssw} = (F_{s,ext}\ C_{sp}\ C_{ns} + F_{s,int})\ C_{ar}\ [1/SF] \text{ x } L$$

$$1{,}964 \text{ lb} = [(F_{s,ext})(0.92)(1.0) + 0](0.71)[1/2.0](4 \text{ ft})$$

$$F_{s,ext} = 1{,}503 \text{ plf}$$

Inspecting Table 6.1 again, it is now found that 15/32-inch-thick wood structural panel sheathing with 8d common nails spaced at 4 inches on center provides an ultimate rated unit shear capacity of 1,539 plf > 1,503 plf. This design does not require the use of 3x framing lumber which allows the same lumber to be used for all wall construction. The only added detail is the difference in nail type and spacing for the garage return walls. From the standpoint of simplicity, the easiest solution would be to increase the width of the garage shear wall segments; however, design simplicity is not always the governing factor. Also, a portal frame system may be designed based on the information and references provided in Section 6.5.2.7.

Finally, the garage should be adequately tied to the building to ensure that the garage section and the house section act as a structural unit. This may be achieved by fastening the end rafter or truss top chord in the roof to the house framing using fasteners with sufficient withdrawal capacity (i.e. ring shank nails or lag screws). The same should be done for the end studs that are adjacent to the house framing. Ideally, the garage roof diaphragm may be tied into the house second floor diaphragm by use of metal straps and blocking extending into the floor diaphragm and garage roof diaphragm a sufficient distance in each direction (i.e., 4 feet). With sufficient connection to the house end wall and floor diaphragm, the garage opening issue may be avoided completely. The connection load to the house discussed above can then be determined by treating the garage roof diaphragm as a cantilevered horizontal beam on the side of the home with a fixed end moment at the connection to the house. The fixed end moment (assuming the garage opening provides no lateral shear resistance) is determined based on the beam equation for a cantilever beam (see Appendix A). For the wind load on the garage, the fixed end moment due to lateral load is (3,928 lb)(11 ft) = 43,208 ft-lb. This moment may be resisted by a strap at either side of the garage roof with about a 2,500 lb design tension capacity (i.e. 43,208 ft-lb/18 ft = 2,400 lb). Preferably, the strap would be anchored to the garage roof diaphragm and house floor diaphragm as described above. Alternatively, this moment could be resisted by numerous lag screws or similar fasteners attaching the garage framing to the house framing. By this method, the garage end walls would require no special shear wall design. Of course, connections required to resist wind uplift and transverse shear loads on the garage door and return walls would still be required.

For the seismic design lateral loads in this example, the garage opening is not so severely loaded. The design seismic load on the Wall Line A is 745 lb. Using the approach above (and substituting a safety factor of 2.5 for seismic design), the value of $F_{s,ext}$ determined is 905 plf which is much less than the 2,004 plf determined for the design wind shear load condition assumed in this example. By inspecting Table 6.1, 7/16-inch-thick Structural 1 sheathing is sufficient and the pnuematic nails used on the rest of the building's shear walls may be used. However, this requires the two garage return walls to be restrained with two hold-down brackets each as in the segmented shear wall design method. For the seismic load, the garage opening wall (Wall Line A) may be suitably designed as a perforated shear wall and eliminate the need for two of the four hold-downs. A portal frame may also be considered for the garage opening (see Section 6.5.2.7).

Wall Line D/E may be designed in a similar fashion to the options discussed above. In fact, Wall Line E may be eliminated as a designed shear wall line provided that a collector is provided to bring the diaphragm shear load into the single wall segment in wall line D (see the dotted line on the floor plan figure). Of course, Wall line D must be designed to carry the full design shear load assigned to that end of the building. Collector design was illustrated in Example 6.3. The connections for overturning (i.e., hold-downs) and base shear transfer must be designed as illustrated in Examples 6.1 and 6.2. As an additional option, Wall Line C may be designed as an interior shear wall line and the wood structural panel sheathing would be placed underneath the interior finish. This last option would relieve some of the load on the house end walls and possibly simplify the overall shear wall construction details used in the house.

3. Determine the shear loads on the N-S shear wall lines using the relative stiffness method and an assumed shear wall construction for the given seismic design condition only.

Assume that the shear wall construction will be as follows:

- 7/16-inch OSB Structural I wood structural panel sheathing with 8d common nails (or 0.131-inch diameter 8d pneumatic nails) spaced at 4 inches on center on the panel edges and 12 inches in the panel field.
- Douglas-fir wall framing is used with 2x studs spaced at 16 inches on center.
- Walls are designed as perforated shear wall lines and adequate hold-downs and base shear connections are provided.

It will be further assumed that the house and garage are sufficiently tied together to act as a structural unit. It must be remembered that the relative stiffness design approach is predicated on the assumption that the horizontal diaphragm is rigid in comparison to the supporting shear walls so that the forces are distributed according to the relative stiffness of the shear wall lines. This assumption is exactly opposite to that assumed by use of the tributary area method.

As given for the design example, the following design seismic shear loads apply to the first story of the example building:

Design N-S Seismic Lateral Load (mapped $S_s = 1.5g$)

House: 7,493 lb total story shear (tributary weight is 37,464 lb)
Garage: 1,490 lb total story shear (tributary weight is 7,452 lb)
Total: 8,983 lb total story shear (total tributary weight is 44,916 lb)

Locate the center of gravity

The first step is to determine the center of gravity of the building at the first story level. The total seismic story shear load will act through this point. For wind design, the process is similar, but the horizontal wind forces on various portions of the building (based on vertical projected areas and wind pressures) are used to determine the force center for the lateral wind loads (i.e., the resultant of the garage and house lateral wind loads).

Establishing the origin of an x-y coordinate system at the bottom corner of Wall Line B of the example first floor plan, the location of the center of gravity is determined by taking weighted moments about each coordinate axis using the center of gravity location for the garage and house portions. Again, the "bump-out" area in living room is considered to have negligible impact on the estimate of the center of gravity since most of the building mass is originating from the second story and roof which does not have the "bump-out" in the plan.

The center of gravity of the garage has the (x,y) coordinates of (-11 ft, 16 ft). The center of gravity of the house has the coordinates (21 ft, 14 ft).

Weighted moments about the y-axis:

$$
\begin{aligned}
X_{cg,building} &= [(X_{cg,garage})(\text{garage weight}) + (X_{cg,house})(\text{house weight})]/(\text{total weight}) \\
&= [(-11\text{ ft})(7{,}452\text{ lb}) + (21\text{ ft})(37{,}464\text{ lb})]/(44{,}916\text{ lb}) \\
&= 15.7\text{ ft}
\end{aligned}
$$

Weighted moments about the x-axis:

$$
\begin{aligned}
Y_{cg,building} &= [(Y_{cg,garage})(\text{garage weight}) + (Y_{cg,house})(\text{house weight})]/(\text{total weight}) \\
&= [(16\text{ ft})(7{,}452\text{ lb}) + (14\text{ ft})(37{,}464\text{ lb})]/(44{,}916\text{ lb}) \\
&= 14.3\text{ ft}
\end{aligned}
$$

Thus, the center of gravity for the first story is located at the (x,y) coordinates of (15.7 ft, 14.3 ft). The approximate location on the floor plan is about 4 inches north of the center bearing wall line and directly in front of the stair well leading down (i.e., about 5 feet to the left of the center of the house).

Locate the center of resistance

The center of resistance is somewhat more complicated to determine and requires an assumption regarding the shear wall stiffness. Two methods of estimating the relative stiffness of segmented shear walls are generally recognized. One method bases the segmented shear wall stiffness on it's length. Thus, longer shear walls have greater stiffness (and capacity). However, this method is less appealing when multiple segments are included in one wall line and particularly when the segments have varying aspect ratios, especially narrow aspect ratios which affect stiffness disproportionately to the length. The second method bases the segmented shear wall stiffness on the shear capacity of the segment, which is more appealing when various shear wall constructions are used with variable unit shear values and when variable aspect ratios are used, particularly when the unit shear strength is corrected for narrow aspect ratios. The method based on strength is also appropriate for use with the perforated shear wall method, since the length of a perforated shear wall has little to do with its stiffness or strength. Rather, the amount of openings in the wall (as well as its construction) govern its stiffness and capacity. Therefore, the method used in this example will use the capacity of the perforated shear wall lines as a measure of relative stiffness. The same technique may be used with a segmented shear wall design method by determining the shear capacity of each shear wall line (comprised of one or more shear wall segments) as shown in Example 6.1.

First, the strength of each shear wall line in the building must be determined. Using the perforated shear wall method and the assumed wall construction given at the beginning of Step 3, the design shear wall line capacities (see below) are determined for each of the exterior shear wall lines in the building. The window and door opening sizes are shown on the plan so that the perforated shear wall calculations can be done as demonstrated in Example 6.2. It is assumed that no interior shear wall lines will be used (except at the shared wall between the garage and the house) and that the contribution of the interior partition walls to the stiffness of the building is negligible. As mentioned, this assumption can overlook a significant factor in the lateral resistance and stiffness of a typical residential building.

PSW 1: F_{psw1} = 7,812 lb (Wall Line D)
PSW 2: F_{psw2} = 3,046 lb (Wall Line E)
PSW 3: F_{psw3} = 14,463 lb (North side wall of house)
PSW 4: F_{psw4} = 9,453 lb (North side of garage)
PSW 5: F_{psw5} = 182 lb (Wall Line A; garage opening)
PSW 6: F_{psw6} = 9,453 lb (South side wall of garage)
PSW 7: F_{psw7} = 9,687 lb (Wall Line B)
PSW 8: F_{psw8} = 11,015 lb (South side wall of house at front)

The center of stiffness on the y-coordinate is now determined as follows using the above PSW design shear capacities for wall lines oriented in the E-W direction:

$$\begin{aligned} Y_{cs} &= [(F_{psw3})(Y_{psw3}) + (F_{psw4})(Y_{psw4}) + (F_{psw6})(Y_{psw6}) + (F_{psw8})(Y_{psw8})]/(F_{psw,E\text{-}W}) \\ &= [(14{,}463\text{ lb})(28\text{ ft})+(9{,}453\text{ lb})(26\text{ ft})+(9{,}453\text{ lb})(6\text{ ft})+(11{,}015\text{ lb})(0\text{ ft})]/(44{,}384\text{ lb}) \\ &= 15.9\text{ ft} \end{aligned}$$

The center of stiffness on the x-coordinate is determined similarly considering the wall lines oriented in the N-S direction:

$$\begin{aligned} X_{cs} &= [(F_{psw1})(X_{psw1}) + (F_{psw2})(X_{psw2}) + (F_{psw5})(X_{psw5}) + (F_{psw7})(X_{psw7})]/(F_{psw,N\text{-}S}) \\ &= [(7{,}812\text{ lb})(42\text{ ft})+(3{,}046\text{ lb})(48\text{ ft})+(182\text{ lb})(-22\text{ ft})+(9{,}687\text{ lb})(0\text{ ft})]/(20{,}727\text{ lb}) \\ &= 22.7\text{ ft} \end{aligned}$$

Therefore, the coordinates of the center of stiffness are (22.7 ft, 15.9 ft). Thus, the center of stiffness is located to the right of the center of gravity (force center for the seismic load) by 22.7 ft – 15.7 ft = 7 ft. This offset between the center of gravity and the center of resistance will create a torsional response in the N-S seismic load direction under consideration. For E-W seismic load direction, the offset (in the y-coordinate direction) is only 15.9 ft – 14.3 ft = 1.6 ft which is practically negligible from the standpoint of torsional response. It should be remembered that, in both loading directions, the influence of interior partitions on the center of stiffness (and thus the influence on torsional response) is not considered. To conservatively account for this condition and for possible error in locating the actual center of gravity of the building (i.e., accidental torsion), codes usually require that the distance between the center of gravity and the center of stiffness be considered as a minimum of 5 percent of the building dimension perpendicular to the direction of seismic force under consideration. This condition is essentially met in this example since the offset dimension for the N-S load direction is 7 feet which is 10 percent of the E-W plan dimension of the house and attached garage.

Distribute the direct shear forces to N-S walls

The direct shear force is distributed to the N-S walls based on their relative stiffness without regard to the location of the center of stiffness (resistance) and the center of gravity (seismic force center), or the torsional load distribution that occurs when they are offset from each other. The torsional load distribution is superimposed on the direct shear forces on the shear wall lines in the next step of the process.

The direct seismic shear force of 8,983 lb is distributed as shown below based on the relative stiffness of the perforated shear wall lines in the N-S direction. As before, the relative stiffness is based on the design shear capacity of each perforated shear wall line relative to that of the total design capacity of the N-S shear wall lines.

Direct shear on PSW1, PSW2, PSW5, and PSW7 is determined as follows:

(total seismic shear load on story)[(F_{psw1})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(7,812 lb)/(20,727 lb)]
= (8,983 lb)[0.377]
= 3,387 lb

(total seismic shear load on story)[(F_{psw2})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(3,046 lb)/(20,727 lb)]
= (8,983 lb)[0.147]
= 1,321 lb

(total seismic shear load on story)[(F_{psw5})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(182 lb)/(20,727 lb)]
= (8,983 lb)[0.009]
= 81 lb

(total seismic shear load on story)[(F_{psw7})/($F_{psw,N\text{-}S}$)] = (8,983 lb)[(9,687 lb)/(20,727 lb)]
= (8,983 lb)[0.467]
= 4,195 lb

Distribute the torsion load

The torsional moment is created by the offset of the center of gravity (seismic force center) from the center of stiffness or resistance (also called the center of rigidity). For the N-S load direction, the torsional moment is equal to the total seismic shear load on the story multiplied by the x-coordinate offset of the center of gravity and the center of stiffness (i.e., 8,983 lb x 7 ft = 62,881 ft-lb). The sharing of this torsional moment on all of the shear wall lines is based on the torsional moment of resistance of each wall line. The torsional moment of resistance is determined by the design shear capacity of each wall line (used as the measure of relative stiffness) multiplied by the square of its distance from the center of stiffness. The amount of the torsional shear load (torsional moment) distributed to each wall line is then determined by the each wall's torsional moment of resistance in proportion to the total torsional moment of resistance of all shear wall lines combined. The torsional moment of resistance of each shear wall line and the total for all shear wall lines (torsional moment of inertia) is determined as shown below.

Wall Line	F_{psw}	Distance from Center of Resistance	$F_{psw}(d)^2$
PSW1	7,812 lb	19.3 ft	2.91×10^6 lb-ft^2
PSW2	3,046 lb	25.3 ft	1.95×10^6 lb-ft^2
PSW3	14,463 lb	12.1 ft	2.12×10^6 lb-ft^2
PSW4	9,453 lb	10.1 ft	9.64×10^5 lb-ft^2
PSW5	182 lb	44.7 ft	3.64×10^5 lb-ft^2
PSW6	9,453 lb	9.9 ft	9.26×10^5 lb-ft^2
PSW7	9,687 lb	22.7 ft	4.99×10^6 lb-ft^2
PSW8	11,015 lb	15.9 ft	2.78×10^6 lb-ft^2
Total torsional moment of inertia (*J*)			1.70×10^7 lb-ft^2

Now, the torsional shear load on each wall is determined using the following basic equation for torsion:

$$V_{WALL} = \frac{M_T d(F_{WALL})}{J}$$

where,

V_{WALL} = the torsional shear load on the wall line (lb)
M_T = the torsional moment* (lb-ft)
d = the distance of the wall from the center of stiffness (ft)
F_{WALL} = the design shear capacity of the segmented or perforated shear wall line (lb)
J = the torsional moment of inertia for the story (lb-ft^2)

*The torsional moment is determined by multiplying the design shear load on the story by the offset of the center of stiffness relative to the center of gravity perpendicular to the load direction under consideration. For wind design, the center of the vertical projected area of the building is used in lieu of the center gravity.

Now, the torsional loads may be determined as shown below for the N-S and E-W wall lines. For PSW1 and PSW2 the torsion load is in the reverse direction of the direct shear load on these walls. This behavior is the result of the center of shear resistance being offset from the force center which causes rotation about the center of stiffness. (Center of shear resistance and center of stiffness may be used interchangeably since the shear resistance is assumed to represent stiffness.) If the estimated offset of the center of gravity and the center of stiffness is reasonably correct, then the torsional response will tend to reduce the shear load on PSW1 and PSW2. However, codes generally do not allow the direct shear load on a wall line to be reduced due to torsion – only increases should be considered.

The following values for use in the torsion equation apply to this example:

M_T = (8,983 lb)(7 ft) = 62,881 ft-lb
J = 1.70 x 10^7 lb-ft^2

The torsional loads on PSW5 and PSW7 are determined as follows:

V_{psw5} = (62,881 ft-lb)(44.7 ft)(182 lb) / (1.70 x 10^7 lb-ft^2)
= 30 lb

V_{psw7} = (62,881 ft-lb)(22.7 ft)(9,687 lb) / (1.70 x 10^7 lb-ft^2)
= 813 lb

These torsional shear loads are added to the direct shear loads for the N-S walls and the total design shear load on each wall line may be compared to its design shear capacity as shown below.

N-S Wall Lines	Wall Design Capacity, F_{psw} (lb)	Direct Shear Load (lb)	Torsional Shear Load (lb)	Total Design Shear Load (lb)	Percent of Design Capacity Used
PSW1	7,812	3,387	na*	3,387	43% (ok)
PSW2	3,046	1,321	na*	1,321	43% (ok)
PSW5	182	81	30	111	61% (ok)
PSW7	9,687	4,195	813	5,008	52% (ok)

*The torsional shear load is actually in the reverse direction of the direct shear load for these walls, but it is not subtracted as required by code practice.

While all of the N-S shear wall lines have sufficient design capacity, it is noticeable that the wall lines on the left side (West) of the building are "working harder" and the walls on the right side (East) of the building are substantially over-designed. The wall construction could be changed to allow a greater sheathing nail spacing on walls PSW1 and PSW2. Also, the assumption of a rigid diaphragm over the entire expanse of the story is very questionable, even if the garage is "rigidly" tied to the house with adequate connections. It is likely that the loads on Walls PSW5 and PSW7 will be higher than predicted using the relative stiffness method. Thus, the tributary area method (see Step 2) may provide a more reliable design and should be considered along with the above analysis. Certainly, reducing the shear wall construction based on the above analysis is not recommended prior to "viewing" the design from the perspective of the tributary area approach. Similarly, the garage opening wall (PSW5) should not be assumed to be adequate simply based on the above analysis in view of the inherent assumptions of the relative stiffness method in the horizontal distribution of shear forces. For more compact buildings with continuous horizontal diaphragms extending over the entire area of each story, the method is less presumptive in nature. But, this qualitative observation is true of all of the force distribution methods demonstrated in this design example.

Conclusion

This seemingly simple design example has demonstrated the many decisions, variables, and assumptions to consider in designing the lateral resistance of a light-frame home. For an experienced designer, certain options or standardized solutions may become favored and developed for repeated use in similar conditions. Also, an experienced designer may be able to effectively design using simplified analytical methods (i.e. the total shear approach shown in Step 1) supplemented with judgment and detailed evaluations of certain portions or unique details as appropriate.

In this example, it appears that a 7/16-inch-thick Structural I wood structural panel sheathing can be used for all shear wall construction to resist the required wind shear loading. A constant sheathing panel edge nail spacing is also possible by using 3 inches on center if the perforated shear wall method is used and 4 inches on center if the segmented shear wall method is used (based on the worst-case condition of Wall Line B). The wall sheathing nails specified were 8d pneumatic nails with a 0.113 inch diameter. In general, this wall construction will be conservative for most wall lines on the first story of the example house. If the seismic shear load were the only factor (i.e., the wind load condition was substantially less than assumed), the wall construction could be simplified even more such that a perforated shear wall design approach with a single sheathing fastening requirement may be suitable for all shear wall lines. The garage opening wall would be the only exception.

Finally, numerous variations in construction detailing in a single project should be avoided as it may lead to confusion and error in the field. Fewer changes in assembly requirements, fewer parts, and fewer special details should all be as important to the design objectives as meeting the required design loads. When the final calculation is done (regardless of the complexity or simplicity of the analytic approach chosen and the associated uncertainties or assumptions), the designer should exercise judgment in making reasonable final adjustments to the design to achieve a practical, well-balanced design. As a critical final consideration, the designer should be confident that the various parts of the structural system are adequately “tied together” to act as a structural unit in resisting the lateral loads. This consideration is as much a matter of judgement as it is a matter of analysis.

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