Chapter 9 Commentary

CONCRETE STRUCTURE DESIGN REQUIREMENTS

9.1 GENERAL

9.1.2 References. The main concern of Chapter 9 is the proper detailing of reinforced concrete construction for earthquake resistance. The bulk of the detailing requirements in this chapter are contained in ACI 318. The commentary for ACI 318 contains a valuable discussion of the rationale behind detailing requirements and is not repeated here.

9.2 GENERAL DESIGN REQUIREMENTS

9.2.1 Classification of shear walls. In the 2000 *Provisions*, shearwalls are classified by the amount and type of detailing required. This classification was developed to facilitate assigning shearwalls to seismic design categories.

9.2.2 Modifications to ACI 318. The modifications noted for ACI 318 are: changes in load factors necessary to coordinate with the equivalent yield basis of this document, additional definitions and provisions necessary for seismic design requirements for structural systems composed of precast elements, and changes that incorporate certain features of the detailing requirements for reinforced concrete that have been adopted into the 1997 *Uniform Building Code* and the 2000 *International Building Code*.

Procedures for design of a seismic-force-resisting structural system composed of precast elements interconnected predominately by dry joints require prior acceptance testing of modules of the generic structural system because with the existing state-of-knowledge, it is inappropriate to propose code provisions without such verification.

The complexity of structural systems, configurations, and details possible with precast concrete elements requires:

1. Selecting functional and compatible details for connections and members that are reliable and can be built with acceptable tolerances;
2. Verifying experimentally the inelastic force-deformation relationships for welded, bolted, or grouted connections proposed for the seismic resisting elements of the building; and
3. Analyzing the building using those connection relationships and the inelastic reversed cyclic loading effects imposed by the anticipated earthquake ground motions.

Research conducted to date (Cheok and Lew, 1991; Elliott et al, 1992; Englekirk, 1987; French et al, 1989; BSSC, 1987; Hawkins and Englekirk, 1987; Jayashanker and French, 1988; Mast, 1992; Nakaki and Englekirk, 1991; Neille, 1977; New Zealand Society, 1991; Pekau and Hum, 1991; Powell et al, 1993; Priestley, 1991; Priestley and Tao, 1992; Stanton et al, 1986; Stanton et al, 1991) documents concepts for design using dry connections and the behavior of structural systems and subassemblages composed of precast elements both at and beyond peak strength levels for nonlinear reversed cyclic loadings.

Use of prestressing tendons. Sec. 9.2.2.1.4 defines conditions under which prestressing tendons can be used, in conjunction with deformed reinforcing bars, in frames resisting earthquake forces. As documented in Ishizuka and Hawkins (1987), if those conditions are met no modification is necessary to the R and C_d factors of Table 4.3-1 when prestressing is used. Satisfactory seismic performance can be obtained when prestressing amounts greater than those permitted by Sec. 9.2.2.1.4 are used. However, as documented by Park and Thompson (1977) and Thompson and Park (1980) and as required by the combination of New Zealand Standards 3101:1982 and 4203:1992, ensuring satisfactory performance requires modification of the R and C_d factors.

9.3 SEISMIC DESIGN CATEGORY B

Special details for ductility and toughness are not required in Seismic Design Category B.

9.3.1 Ordinary moment frames. Since ordinary frames are permitted only in Seismic Design Categories A and B, they are not required to meet any particular seismic requirements. Attention should be paid to the often overlooked requirement for joint reinforcement in Sec.11.11.2 of ACI 318.

9.4 SEISMIC DESIGN CATEGORY C

A frame used as part of the seismic-force-resisting system in Seismic Design Category C is required to have certain details that are intended to help sustain integrity of the frame when subjected to deformation reversals into the nonlinear range of response. Such frames must have attributes of intermediate moment frames. Structural (shear) walls of buildings in Seismic Design Category C are to be designed in accordance with the requirements of ACI 318.

9.4.1.1 Moment frames. The concept of moment frames for various levels of hazard zones and of performance is changed somewhat from the provisions of ACI 318. Two sets of moment frame detailing requirements are defined in ACI 318, one for "regions of high seismic risk" and the other for "regions of moderate seismic risk." For the purposes of this document, the "regions" are made equivalent to Seismic Design Categories in which "high risk" means Seismic Design Categories D and E and "moderate risk" means Seismic Design Category C. This document labels these two frames the "special moment frame" and the "intermediate moment frame," respectively.

The level of inelastic energy absorption of the two frames is not the same. The *Provisions* introduce the concept that the *R* factors for these two frames should not be the same. The preliminary version of the *Provisions* (ATC 3-06) assigned the *R* for ordinary frames to what is now called the intermediate frame. In spite of the fact that the *R* factor for the intermediate frame is less than the *R* factor for the special frame, use of the intermediate frame is not permitted in the higher Seismic Design Categories (D, E, and F). On the other hand, this arrangement of the *Provisions* encourages consideration of the more stringent detailing practices for the special frame in Seismic Design Category C because the reward for use of the higher *R* factor can be weighed against the higher cost of the detailing requirements. The *Provisions* also introduce the concept that an intermediate frame may be part of a dual system in Seismic Design Category C.

The differences in the performance basis of the requirements for the two types of frames might be summarized briefly as follows (see the commentary of ACI 318 for a more detailed discussion of the requirement for the special frame):

1. The shear strength of beams and columns must not be less than that required when the member has yielded at each end in flexure. For the special frame, strain hardening and other factors are considered by raising the effective tensile strength of the bars to 125 percent of specified yield. For the intermediate frame, an escape clause is provided in that the calculated shear using double the prescribed seismic force may be substituted. Both types require the same minimum amount and maximum spacing of transverse reinforcement throughout the member.
2. The shear strength of joints is limited and special provisions for anchoring bars in joints exist for special moment frames but not intermediate frames. Both frames require transverse reinforcement in joints although less is required for the intermediate frame.
3. Closely spaced transverse reinforcement is required in regions of potential hinging (typically the ends of beams and columns) to control lateral buckling of longitudinal bars after the cover has spalled. The spacing limit is slightly more stringent for columns in the special frame.
4. The amount of transverse reinforcement in regions of hinging for special frames is empirically tied to the concept of providing enough confinement of the concrete core to preserve a ductile response.

These amounts are not required in the intermediate frame and, in fact, for beams stirrups may be used in lieu of hoops.

5. The special frame must follow the strong column/weak beam rule. Although this is not required for the intermediate frame, it is highly recommended for multistory construction.
6. The maximum and minimum amounts of reinforcement are limited to prevent rebar congestion and to assure a nonbrittle flexural response. Although the precise limits are different for the two types of frames, a great portion of practical, buildable designs will satisfy both.
7. Minimum amounts of continuous reinforcement to account for moment reversals are required by placing lower limits on the flexural strength at any cross section. Requirements for the two types of frames are similar.
8. Locations for splices of reinforcement are more tightly controlled for the special frame.
9. In addition, the special frame must satisfy numerous other requirements beyond the intermediate frame to assure that member proportions are within the scope of the present research experience on seismic resistance and that analysis, design procedures, qualities of the materials, and inspection procedures are at the highest level of the state of the art.

9.5 SEISMIC DESIGN CATEGORIES D, E, AND F

The requirements conform to current practice in the areas of highest seismic hazard.

9.6 ACCEPTANCE CRITERIA FOR SPECIAL PRECAST STRUCTURAL WALLS BASED ON VALIDATION TESTING

9.6.1. Notation

Symbols additional to those in Chapter 21 of ACI 318 are defined:

A_h = area of hysteresis loop

E_1,E_2 = peak lateral resistance for positive and negative loading, respectively, for third cycle of loading sequence.

f_1 = live load factor defined in 9.6.2.3.

h_w = height of column of test module, in. or mm.

K, K' = initial stiffness for positive and negative loading, respectively, for first cycle

θ_1,θ_2 = drift ratios at peak lateral resistance for positive and negative loading, respectively, for third cycle of loading sequence..

θ_1',θ_2' = drift ratios for zero lateral load for unloading at stiffness K, K' from peak positive and negative lateral resistance, respectively, for third cycle of loading sequence. (Figure C9.6.2.4)

Δ = lateral displacement, in. or mm. See Figures. C9.6.2.2.1, C9.6.2.2.2 and C9.6.2.2.3

Δ_a = allowable story drift, in. or mm. See Table 9.5.2.8 of SEI/ASCE 7-02

9.6.2 Definitions

9.6.2.1 Coupling elements. Coupling elements are connections provided at specific intervals along the vertical boundaries of adjacent structural walls. Coupled structural walls are stiffer and stronger than the same walls acting independently. For cast-in-place construction effective coupling elements are typically coupling beams having small span-to-depth ratios. The inelastic behavior of such beams is normally controlled by their shear strength. For precast construction, effective coupling elements can be precast beams connected to the adjacent structural walls either by post-tensioning, ductile mechanical devices, or grouted-in-place reinforcing bars. The resultant coupled construction can be either emulative of cast-in-place construction or non-emulative(jointed). However, for precast construction coupling beams can also be omitted and mechanical devices used to connect directly the vertical boundaries of adjacent structural walls[2,3].

9.6.2.2 Drift ratio. The definition of the drift ratio, θ, is illustrated in Figure C9.6.2.2.1 for a three panel wall module. The position of the module at the start of testing, with only its self-weight acting, is indicated by broken lines. The module is set on a horizontal foundation support that is centered at A and is acted on by a lateral force *H* applied at the top of the wall. The self-weight of the wall is distributed uniformly to the foundation support. However, under lateral loading, that self-weight and any axial gravity load acting at the top of the wall cause overturning moments on the wall that are additional to the overturning moment Hh_w and can affect deformations. The chord AB of the centroidal axis of the wall is the vertical reference line for drift measurements.

For acceptance testing a lateral force *H* is applied to the wall through the pin at B. Depending on the geometric and reinforcement characteristics of the module that force can result in the module taking up any one, or a combination, of the deformed shapes indicated by solid lines in Figures C9.6.2.2.1, C9.6.2.2.2 and C9.6.2.2.3.

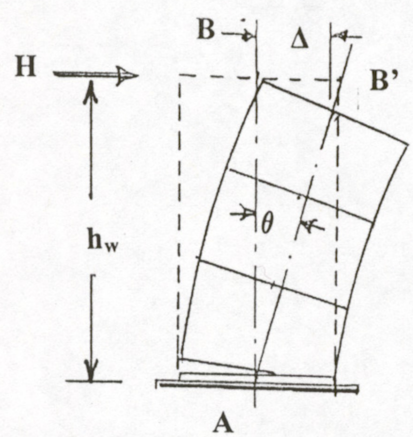

Fig. C9.6.2.2.1 Definition of drift ratio θ.

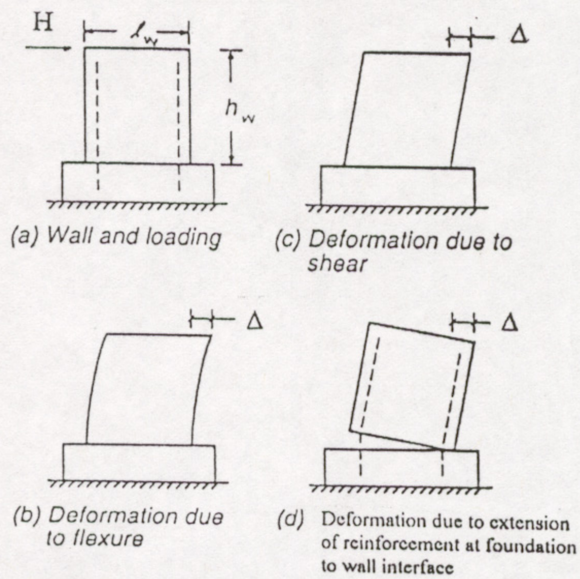

Fig. C9.6.2.2.2 Typical wall deformation components.

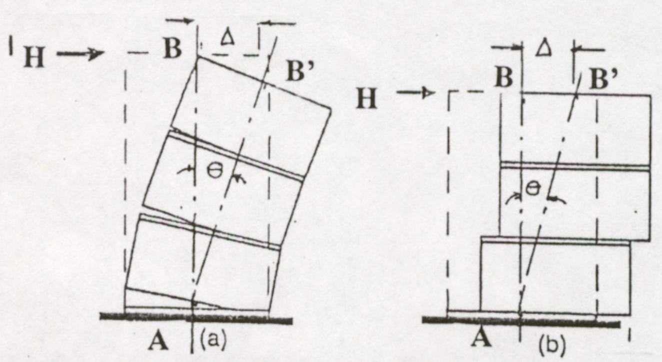

Fig. C9.6.2.2.3 Undesirable deformations along horizontal joints:
(a) excessive gap opening between panels; (b) shear slip.

Figure C9.6.2.2.2 illustrates several possible components of the displacement Δ for a wall that is effectively solid while Figure C9.6.2.2.3 illustrates two possibly undesirable components of the displacement Δ. Regardless of the mode of deformation of the wall, the lateral force causes the wall at B to displace horizontally by an amount Δ. The drift ratio is the angular rotation of the wall chord with respect to the vertical and for the setup shown equals Δ / h_w where h_w is the wall height and is equal to the distance between the foundation support at A and the load point at B.

Where prestressing steel is used in wall members, the stress f_{ps} in the reinforcement at the nominal and the probable lateral resistance shall be calculated in accordance with Sec. 18.7 of ACI 318.

9.6.2.3 Global toughness. These provisions describe acceptance criteria for special precast structural walls based on validation testing. The requirements of Sec. 21.2.1.5 of ACI 318 concerning toughness cover both to the energy dissipation of the wall system which, for monolithic construction, is affected primarily by local plastic hinging behavior and the toughness of the prototype structure as a whole. The latter is termed "global toughness" in these provisions and is a condition that does not apply to the walls alone. That global toughness requirement can be satisfied only though analysis of the performance of the prototype structure as a whole when the walls perform to the criteria specified in these provisions.

The required gravity load for global toughness evaluations is the value given by these provisions. For conformity with Sec. 9.2.1 of ACI 318, UBC 1997, IBC 2003 and NFPA 5000, the required gravity load is $1.2D + f_1L$ where the seismic force is additive to gravity forces and 0.9D where the seismic force counteracts gravity forces. D is the effect of dead loads, L is the effect of live loads, and f_1 is a factor equal to 0.5 except for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf (4.79 kN/m^2) where f_1 equals 1.0.

9.6.2.5 Relative energy dissipation ratio. This concept is illustrated in Figure C9.6.2.5 for the third loading cycle to the limiting drift ratio required by Sec. 9.6.7.4, 9.6.7.5 or 9.6.7.6, as appropriate.

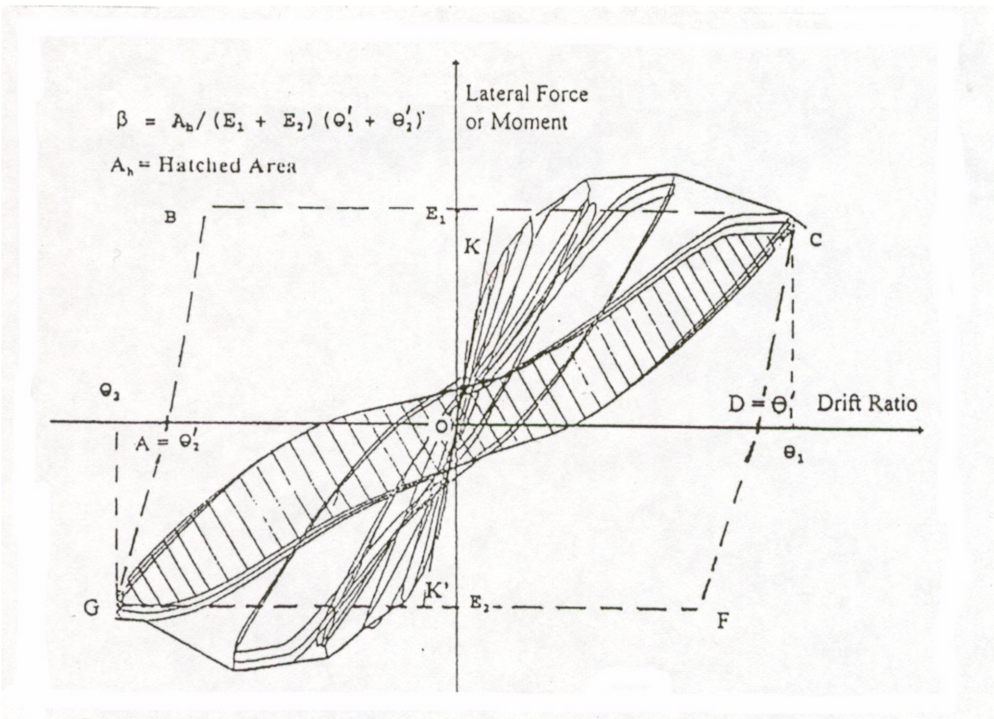

Fig. C9.6.2.5 Relative energy dissipation ratio.

For Figure C9.6.2.5, it is assumed that the test module has exhibited different initial stiffnesses, K and K', for positive and negative lateral forces and that the peak lateral resistances for the third cycle for the positive and negative loading directions, E_1 and E_2, also differ. The area of the hysteresis loop for the third cycle, A_h, is hatched. The circumscribing figure consists of two parallelograms, ABCD and DFGA. The slopes of the lines AB and DC are the same as the initial stiffness, K, for positive loading and the slopes of the lines DF and GA are the same as the initial stiffness, K', for negative loading. The relative energy dissipation ratio concept is similar to the equivalent viscous damping concept used in Sec. 13.9.3 of the *2000 NEHRP Provisions* and *Commentary* for required tests of seismic isolation systems.

For a given cycle the relative energy dissipation ratio, β, is the area, A_h, inside the lateral force-drift ratio loop for the module, divided by the area of the effective circumscribing parallelograms ABCD and DFGA. The areas of the parallelograms equal the sum of the absolute values of the lateral force strengths, E_1 and E_2, at the drift ratios θ_1 and θ_2 multiplied by the sum of the absolute values for the drift ratios θ_1' and θ_2'.

9.6.3 Scope and general requirements. While only ACI Committee 318 can determine the requirements necessary for precast walls to meet the provisions of Sec. 21.2.1.5 of ACI 318, Sec. 1.4 of

ACI 318 already permits the building official to accept wall systems, other than those explicitly covered by Chapter 21 of ACI 318, provided specific tests, load factors, deflection limits, construction procedures and other pertinent requirements have been established for acceptance of such systems consistent with the intent of the code. The purpose of these provisions is to provide a framework that establishes the specific tests, load factors, deflection limits and other pertinent requirements appropriate for acceptance, for regions of high seismic risk or for structures assigned to high seismic performance or design categories, of precast wall systems, including coupled wall systems, not satisfying all the requirements of Chapter 21 of ACI 318. For regions of moderate seismic risk or for structures assigned to intermediate seismic performance or design categories, less stringent provisions than those specified here are appropriate.

These provisions assume that the precast wall system to be tested has details differing from those prescribed by Sec. 21.7 of ACI 318 for conventional monolithic reinforced concrete construction. Such walls may, for example, involve the use of precast elements, precast prestressed elements, post-tensioned reinforcement, or combinations of those elements and reinforcement.

For monolithic reinforced concrete walls a fundamental design requirement of Chapter 21 of ACI 318 is that walls with h_w/l_w exceeding1.0 be proportioned so that their inelastic response is dominated by flexural action on a critical section located near the base of the wall. That fundamental requirement is retained in these provisions. The reason is that tests on modules, as envisioned in these provisions, cannot be extrapolated with confidence to the performance of panelized walls of proportions differing from those tested for the development of Chapter 21of ACI 318 if the shear-slip displacement pattern of Figure C9.6.2.2.3, or the shear deformation response of Figure C9.6.2.2.2, governs the response developed in the test on the module. Two other fundamental requirements of Chapter 21 of ACI 318 are for ties around heavily strained boundary element reinforcement and the provision of minimum amounts of uniformly distributed horizontal and vertical reinforcement in the web of the wall. Ties around boundary element reinforcement to inhibit its buckling in compression are required where the strain in the extreme compression fiber is expected to exceed some critical value. Minimum amounts of uniformly distributed horizontal and vertical reinforcement over the height and length of the wall are required to restrain the opening of inclined cracks and allow the development of the drift ratios specified in Sec. 9.6.7.4, 9.6.7.5 and 9.6.7.6. Deviations from those tie and distributed reinforcement requirements are possible only if a theory is developed that can substantiate reasons for such deviations and that theory is tested as part of the validation testing.

9.6.3.1. These provisions are not intended for use with existing construction or for use with walls that are designed to conform to all the requirements of Sec. 21.7 of ACI 318. The criteria of these provisions are more stringent than those for walls designed to Sec. 21.7 of ACI 318. Some walls designed to 21.7, and having low height to length ratios, may not meet the drift ratio limits of Eq. 9.6.1 because their behavior may be governed by shear deformations. The height to length ratio of 0.5 is the least value for which Eq. 9.6.1 is applicable.

9.6.3.3. For acceptance, the results of the tests on each module must satisfy the acceptance criteria of Sec. 9.6.9. In particular, the relative energy dissipation ratio calculated from the measured results for the third cycle between the specified limiting drift ratios must equal or exceed 1/8. For uncoupled walls, relative energy dissipation ratios increase as the drift ratio increases. Tests on slender monolithic walls have shown relative energy dissipation ratios, derived from rotations at the base of the wall, of about 40-45 percent at large drifts. The same result has been reported even where there has been a significant opening in the web of the wall on the compression side. For 0.020 drift ratios and walls with height to length ratios of 4, relative energy dissipation ratios have been computed as 30, 18, 12, and 6 percent, for monolithic reinforced concrete, hybrid reinforced/post-tensioned prestressed concrete with equal flexural strengths provided by the prestressed and deformed bar reinforcement, hybrid reinforced/post-tensioned prestressed concrete with 25 percent of the flexural strength provided by deformed bar reinforcement and 75 percent by the prestressed reinforcement, and post-tensioned prestressed concrete special structural walls, respectively. Thus, for slender precast uncoupled walls of emulative or non-emulative design it is to be anticipated that at least 35 percent of the flexural capacity at the base of the wall needs to be provided by deformed bar reinforcement if the requirement of a relative energy dissipation ratio of 1/8 is to be achieved. However, if more than about 40 percent of the flexural capacity at the base of the wall is provided by deformed bar reinforcement, then the self-centering capability of the wall following a major event is lost and that is one of the prime advantages gained with the use of post-tensioning. For squat walls with height to length ratios between 0.35 and 0.69 the relative energy dissipation has been reported[13] as remaining constant at 23 percent for drifts between that for first diagonal cracking and that for a post-peak capacity of 80 percent of the peak capacity. Thus, regardless of whether the behavior of a wall is controlled by shear or flexural deformations a minimum relative energy dissipation ratio of 1/8 is a realistic requirement.

For coupled wall systems, theoretical studies and tests have demonstrated that the 1/8 relative energy dissipation ratio can be achieved by using central post-tensioning only in the walls and appropriate energy dissipating coupling devices connecting adjacent vertical wall boundaries.

9.6.3.3.4. The SEI/ASCE 7-02 allowable story drift limits are the basis for the drift limits of IBC 2003 and NFPA 5000. Allowable story drifts, Δ_a, are specified in Table 1617.3 of IBC 2003 and likely values are discussed in the Commentary to Sec. 9.6.7.4. The limiting initial drift ratio consistent with Δ_a equals $\Delta_a/\phi C_d h_w$, where ϕ is the strength reduction factor appropriate to the condition, flexure or shear, that controls the design of the test module. For example, for Δ_a/h_w equal to 0.015, the required deflection amplification factor C_d of 5, and ϕ equal to 0.9, the limiting initial drift ratio, corresponding to B in Figure C9.6.9.1, is 0.0033. The use of a ϕ value is necessary because the allowable story drifts of the IBC are for the design seismic load effect, E, while the limiting initial drift ratio is at the nominal strength, E_n , which must be greater than E/ϕ. The load-deformation relationship of a wall becomes significantly non-linear before the applied load reaches E_{nt}. While the load at which that non-linearity becomes marked depends on the structural characteristics of the wall, the response of most walls remains linear up to about 75 percent of E_{nt}.

9.6.3.3.5. The criteria of Sec. 9.6.9 are for the test module. In contrast, the criterion of Sec. 9.6.3.3.5 is for the structural system as a whole and can be satisfied only by the philosophy used for the design and analysis of the building as a whole. The criterion adopted here is similar to that described in the last paragraph of R21.2.1 of ACI 318 and the intent is that test results and analyses demonstrate that the structure, after cycling three times through both positive and negative values of the limiting drift ratio specified in Sec. 9.6.7.4, 9.6.7.5 or 9.6.7.6, as appropriate, is still capable of supporting the gravity load specified as acting on it during the earthquake.

9.6.4 Design procedure

9.6.4.1. The test program specified in these provisions is intended to verify an existing design procedure for precast structural walls for a specific structure or for prequalifying a generic type of special precast wall system for construction in general. The test program is not for the purpose of creating basic information on the strength and deformation properties of such systems for design purposes. Thus, the test modules should not fail during the validation testing, a result that is the opposite of what is usually necessary during testing in the development phase for a new or revised design procedure. For a generic precast wall system to be accepted based on these provisions, a rational design procedure is to have been developed prior to this validation testing. The design procedure is to be based on a rational consideration of material properties and force transfer mechanisms, and its development will usually require preliminary and possibly extensive physical testing that is not part of the validation testing. Because special wall systems are likely to respond inelastically during design-level ground shaking, the design procedure must consider wall configuration, equilibrium of forces, compatibility of deformations, the magnitudes of the lateral drifts, reversed cyclic displacements, the relative values of each limiting engineering design criteria (shear, flexure and axial load) and use appropriate constitutive laws for materials that include considerations of effects of cracking, loading reversals and inelasticity.

The effective initial stiffness of the structural walls is important for calculating the fundamental period of the prototype structure. The procedure used to determine the effective initial stiffness of the walls is to be verified from the validation test results as described in Sec. 9.6.7.11.

Provisions Sec. 9.6.4.1.1 through 9.6.4.1.3 state the minimum procedures to be specified in the design procedure prior to the start of testing. The Authority Having Jurisdiction may require that more details be provided in the design procedure than those of Sec. 9.6.4.1.1 through 9.6.4.1.3 prior to the start of testing.

9.6.4.2. The justification for the small number of test modules, specified in Sec. 9.6.5.1 is that a previously developed rational design procedure is being validated by the test results. Thus, the test modules for the experimental program must be designed using the procedure intended for the prototype wall system and strengths must be predicted for the test modules before the validation testing is started.

9.6.5 Test modules.

9.6.5.1. One module must be tested for each limiting engineering design criterion, such as shear, or axial load and flexure, for each characteristic configuration of walls. Thus, in accordance with 9.6.4.3 if the test on the module results in a maximum shear stress of $3\sqrt{f_c'}$ then the maximum shear stress that can be used in the prototype is that same value. Each characteristic in-plane configuration of walls, or coupled walls, in the prototype structure must also be tested. Thus, as a minimum for one-way structural walls, two modules with the configuration shown in Figure C9.6.2.2.1, and, for one way coupled walls, two modules with the configuration shown in either Figure C9.6.5.1(a) or in Figure C9.6.5.1(b), must be tested. In addition, if intersecting wall systems are to be used then the response of the wall systems for the two orthogonal directions needs to be tested. For two-way wall systems and coupled wall-frame systems, testing of configurations other than those shown in Figures C9.6.2.2.1 and C9.6.5.1 may be appropriate when it is difficult to realistically model the likely dominant earthquake deformations using orthogonal direction testing only.

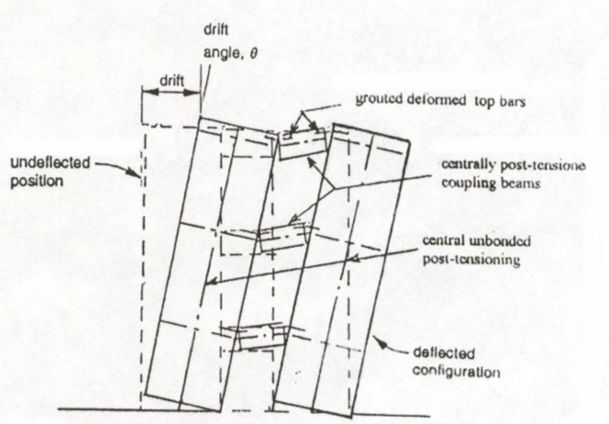

Fig. C9.6.5.1(a) Coupled wall test module with coupling beams.

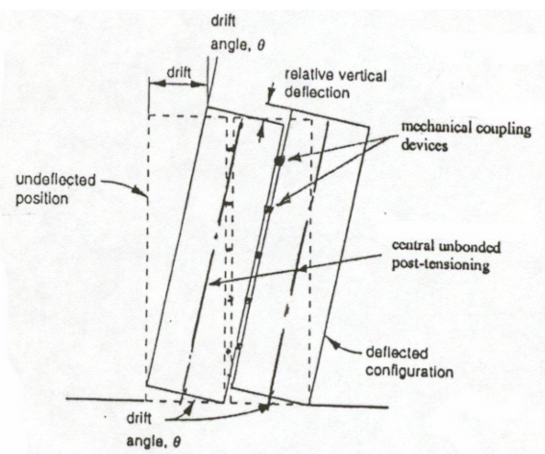

Fig. C9.6.5.1(b) Coupled wall test module with vertical mechanical couplers.

This provision should not be interpreted as implying that only two tests will need to be made to qualify a generic system. During the development of that system it is likely that several more tests will have been made, resulting in progressive refinements of the mathematical model used to describe the likely performance of the generic structural wall system and its construction details. Consequently, only one test of each module type for each limiting engineering design condition, at a specified minimum scale and subjected to specific loading actions, may be required to validate the system. Further, as stated in Sec. 9.6.9.1, if any one of those modules for the generic wall system fails to pass the validation testing required by these provisions, then the generic wall system has failed the validation testing

In most prototype structures, a slab is usually attached to the wall and, as demonstrated by the results of the PRESSS building test, the manner in which the slab is connected to the wall needs to be carefully considered. The connection needs to be adequate to allow the development of story drifts equal to those anticipated in these provisions. However, in conformity with common practice for the sub-assemblage tests used to develop the provisions of Chapter 21of ACI 318, there is no requirement for a slab to be attached to the wall of the test module. The effect of the presence of the slab should be examined in the development program that precedes the validation testing.

9.6.5.3. Test modules need not be as large as the corresponding walls in the prototype structure. The scale of the test modules, however, must be large enough to capture all the complexities associated with the materials of the prototype wall, its geometry and reinforcing details, load transfer mechanisms, and joint locations. For modules involving the use of precast elements, for example, scale effects for load transfer through mechanical connections should be of particular concern. The issue of the scale necessary to capture fully the effects of details on the behavior of the prototype should be examined in the development program that precedes the validation testing.

9.6.5.4. It is to be expected that for a given generic precast wall structure, such as an unbonded centrally post-tensioned wall constructed using multiple precast or precast pretensioned concrete wall panels, validation testing programs will initially use specific values for the specified strength of the concrete and reinforcement in the walls, the layout of the connections between panels, the location of the post-tensioning, the location of the panel joints, and the design stresses in the wall. Pending the development of an industry standard for the design of such walls, similar to the standard for special hybrid moment frames, specified concrete strengths, connection layouts, post-tensioning amounts and locations, etc., used for such walls will need to be limited to the values and layouts used in the validation testing programs.

9.6.5.5. For walls constructed using precast or precast/prestressed panels and designed using non-emulative methods, the response under lateral load can change significantly with joint opening (Figure C9.6.2.2.2(d) and Figure C9.6.2.2.3(a)). The number of panels used to construct a wall depends on wall height and design philosophy. If, in the prototype structure, there is a possibility of horizontal joint opening under lateral loading at a location other than the base of the wall, then the consequences of that possibility need to be considered in the development and validation test programs. Joint opening at locations other than the base can be prevented through the use of capacity design procedures.

9.6.5.6. The significance of the magnitude of the gravity load that acts simultaneously with the lateral load needs to be addressed during the validation testing if the development program suggests that effect is significant.

9.6.5.7. Details of the connection of walls to the foundation are critical, particularly for non-emulative wall designs. The deformations that occur at the base of the wall due to plastic hinging or extension of the reinforcing bars or post-tensioning steel crossing the wall to foundation interface, (Figure C9.6.2.2.2(d)), are in part determined by details of the anchorage and the bonding of those reinforcements on either side of the interface. Grout will be normally used to bed panels on the foundation and the characteristics of that grout in terms of materials, strength and thickness, can have a large effect on wall performance. The typical grout pad with a thickness of 1 inch (25 mm) or less can be expected to provide a coefficient of friction of about 0.6 under reversed loadings. Pads with greater thickness and without fiber reinforcement exhibit lesser coefficients of friction. Adequate frictional resistance is essential to preventing undesirable shear-slip deformations of the type shown in Figure C9.6.2.2.3(b).

9.6.5.8. The geometry of the foundations need not duplicate that used in the prototype structure. However, the geometric characteristics of the foundations (width, depth and length) need to be large enough that they do not influence the behavior of the test module.

9.6.6 Testing agency. In accordance with the spirit of the requirements of Sec.1.3.5 and 1.4 of ACI 318, it is important that testing be carried out by a recognized independent testing agency, approved by the agency having jurisdiction and that the testing and reporting be supervised by a registered design professional familiar with the proposed design procedure and experienced in testing and seismic structural design.

9.6.7 Test method. The test sequence is expressed in terms of drift ratio, and the initial ratio is related to the likely range of linear elastic response for the module. That approach, rather than testing at specific drift ratios of 0.005, 0.010, etc., is specified because, for modules involving prestressed concrete, the likely range of elastic behavior varies with the prestress level.

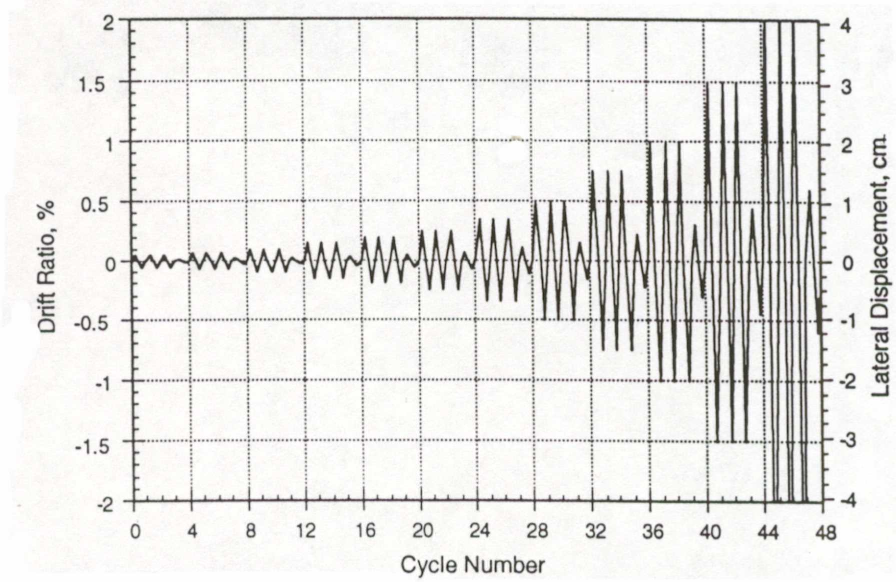

Fig. C9.6.7 Example of specified test sequence.

An example of the test sequence specified in Sec. 9.6.7.2 through 9.6.7.6 is illustrated in Figure C9.6.7. The sequence is intended to ensure that displacements are increased gradually in steps that are neither too large nor too small. If steps are too large, the drift capacity of the system may not be determined with sufficient accuracy. If the steps are too small, the system may be unrealistically softened by loading repetitions, resulting in artificially low maximum lateral resistances and artificially high maximum drifts. Also, when steps are too small, the rate of change of energy stored in the system may be too small compared with the change occurring during a major event. Results, using such small steps, can mask undesirable brittle failure modes that might occur in the inelastic response range during a major event. Because significant diagonal cracking is to be expected in the inelastic range in the web of walls, and in particular in squat walls, the pattern of increasing drifts used in the test sequence can markedly affect diagonal crack response in the post-peak range of behavior.

The drift capacity of a building in a major event is not a single quantity, but depends on how that event shakes the structure. In the forward near field, a single pulse may determine the maximum drift demand, in which case a single large drift demand cycle for the test module would give the best estimation of the drift capacity. More often, however, many small cycles precede the main shock and that is the scenario represented by the specified loading.

There is no requirement for an axial load to be applied to the wall simultaneously with the application of the lateral displacements. In many cases it will be conservative not to apply axial load because, in general, the shear capacity of the wall and the resistance to slip at the base of the wall increase as the axial load on the wall increases. However, as the height of the wall increases and the limiting drift utilized in the design of the wall increases, the likelihood of extreme fiber crushing in compression at maximum drift increases, and the importance of the level of axial load increases. The significance of the level of axial loading should be examined during the development phase.

9.6.7.4. For the response of a structure to the design seismic shear force, current building codes such as UBC-97, IBC 2003 or NFPA 5000 , or recommended provisions such as 2000 *Provisions*, SEI/ASCE 7-02 and FEMA 273 specify a maximum allowable drift. However, structures designed to meet that drift limit may experience greater drifts during an earthquake equal to the design basis earthquake and are likely to experience greater drifts during an earthquake equal to the maximum credible earthquake. In addition to the characteristics of the ground motion, actual drifts will depend on the strength of the structure, its initial elastic stiffness, and the ductility expected for the given lateral load resisting system. Specification of suitable limiting drifts for the test modules requires interpretation and allowance for uncertainties in the assumed ground motions and structural properties.

In IBC 2003, the design seismic shear force applied at the base of a building is related directly to its weight and the design elastic response acceleration, and inversely to a response modification factor, *R*. That *R* factor increases with the expected ductility of the lateral force resisting system of the building. Special structural walls satisfying the requirements of Sec. 21.2 and 21.7 are assigned an *R* value of 6 when used in a building frame system and a value of 5 when used in a bearing wall system. They are also assigned allowable story drift ratios that are dependent on the hazard to which the building is exposed. When the design seismic shear force is applied to a building, the building responds inelastically and the resultant computed drifts, (the design story drifts), must be less than a specified allowable drift. Additional guidance is given in FEMA 356 where the deformations for rectangular walls with height to length ratios greater than 2.5, and flanged wall sections with height to length ratios greater than 3.5, are to be assumed to be controlled by flexural actions. When structural walls are part of a building representing a substantial hazard to human life in the event of a failure, the allowable story drift ratio for shear controlled walls is 0.0075 and for flexure controlled walls is a function of the plastic hinge rotation at the base of the wall. For flexure controlled walls values range up to a maximum of about 0.02 for walls with confined boundary elements with low reinforcement ratios and shear stress less than $3\sqrt{f_c'}$

To compensate for the use of the *R* value, IBC Sec. 1617.4.6 requires that the drift determined by an elastic analysis for the code-prescribed seismic forces be multiplied by a deflection amplification factor, C_d ,to determine the design story drift and that the design story drift must be less than the allowable story drift. In building frame systems, structural walls satisfying the requirements of Sec. 21.7of ACI 318 are assigned a C_d value of 5. However, research[8] has found that design story drift ratios determined in the foregoing manner may be too low. Drift ratios of 6 times IBC-calculated values, (rather than 5), are more representative of the upper bounds to expected drift ratios. The value of 6 is also in agreement with the finding that the drift ratio of an inelastic structure is approximately the same as that of an elastic structure with the same initial period. For flexure controlled walls the value of 6/5 times the present IBC limits on calculated drift ratio, would lead to a limit on real drift ratios of up to 0.024.

Duffy et al. reviewed experimental data for shear walls to define post-peak behavior and limiting drift ratios for walls with height to length ratios between 0.25 and 3.5. Seo et al. re-analyzed the data of Duffy et al. together with data from tests conducted subsequent to the analysis of Duffy et al. Duffy et al. established that for squat walls with web reinforcement satisfying ACI 318-02 requirements and height to length ratios between 0.25 and 1.1, there was a significant range of behavior for which drifts were still reliable in the post-peak response region. Typically the post-peak drift increased by 0.005 for a 20 percent degradation in capacity under cyclic loading. For greater values of degradation, drifts were less reliable. That finding has also been confirmed through tests conducted by Hidalgo et al.[13] on squat walls with effective height to length ratios ranging between 0.35 and 1.0. Values of the drift ratio of the walls at inclined cracking and at peak capacity varied little with web reinforcement. By contrast, drifts in the post-peak range were reliable to a capacity equal to 80 percent of the peak capacity and were 0.005 greater than the drifts at peak capacity provided the walls contained horizontal and vertical web reinforcement equal to 0.25 percent.

From an analysis of the available test data, and from theoretical considerations for a wall rotating flexurally about a plastic hinge at its base, Seo et al concluded that the limiting drift at peak capacity increased almost linearly with the height to length ratio of the wall. When the additional post peak drift capacity for walls with adequate web reinforcement was added to the drift at peak capacity, then the total available drift capacity in percent was given by the following equation:

$$1.0 \le 0.67\left[h_w / l_w\right] + 0.5 \le 3.0$$

where h_w is the height of the wall, and l_w is the length of the wall. The data from the tests of Hidalgo et al. suggest that while that formula is correct for squat walls the lower limit on drift can be decreased to 0.8 as specified in these provisions and that the use of that formula should be limited to walls with height to length ratios equal to or greater than 0.5. For wall height to length ratios less than 0.5 the behavior is controlled principally by shear deformations, (Figure C9.6.2.2.2(c)), and Eq. 9.6.1 should not be used. The upper value of 0.030 for the drift ratio was somewhat optimistic because the data were for walls with height to length ratios equal to or less than 3.5 and subsequent tests have shown that the upper limit of 2.5, as specified in Eq. 9.6.1, is a more realistic limit.

9.6.7.5. The design capacity for coupled wall systems must be developed by the drift ratio corresponding to that for the wall with the least h_w/l_w value. However, it is desirable that testing be continued to the drift given by Eq. 9.6.1 for the wall with the greatest h_w/l_w in order to assess the reserve capacity of the coupled wall system.

9.6.7.6. The drift limits of Eq. 9.6.1 are representative of the maximum that can be achieved by walls designed to ACI 318. The use of smaller drift limits is appropriate if the designer wishes to use performance measures less than the maximum permitted by ACI 318. Examples are the use of reduced shear stresses so that the likelihood of diagonal cracking of the wall is minimized or reduced compressive stresses in the boundary elements of the wall so that the risk of crushing is reduced. Non-linear time history analyses for the response to a suite of maximum considered earthquake (MCE) ground motions, rather than 1.5 times a suite of the corresponding design basis earthquake (DBE) ground motions, is required because the drifts for the response to the MCE motion can be significantly larger than 1.5 times the drifts for the response to the DBE motions.

9.6.7.10. In many cases, data additional to the minimum specified in Sec. 9.6.7.7 may be useful to confirm both design assumptions and satisfactory response. Such data include relative displacements, rotations, curvatures, and strains.

9.6.8 Test report.

The test report must be sufficiently complete and self-contained for a qualified expert to be satisfied that the tests have been designed and carried out in accordance with these criteria, and that the results satisfy the intent of these provisions. Sec.9.6.8.1.1 through 9.6.8.1.11 state the minimum evidence to be contained within the test report. The Authority having Jurisdiction or the registered design professional supervising the testing may require that additional test information be reported.

9.6.9 Test module acceptance criteria.

The requirements of this clause apply to each module of the test program and not to an average of the results of the program. Figure C9.6.9.1 illustrates the intent of this clause.

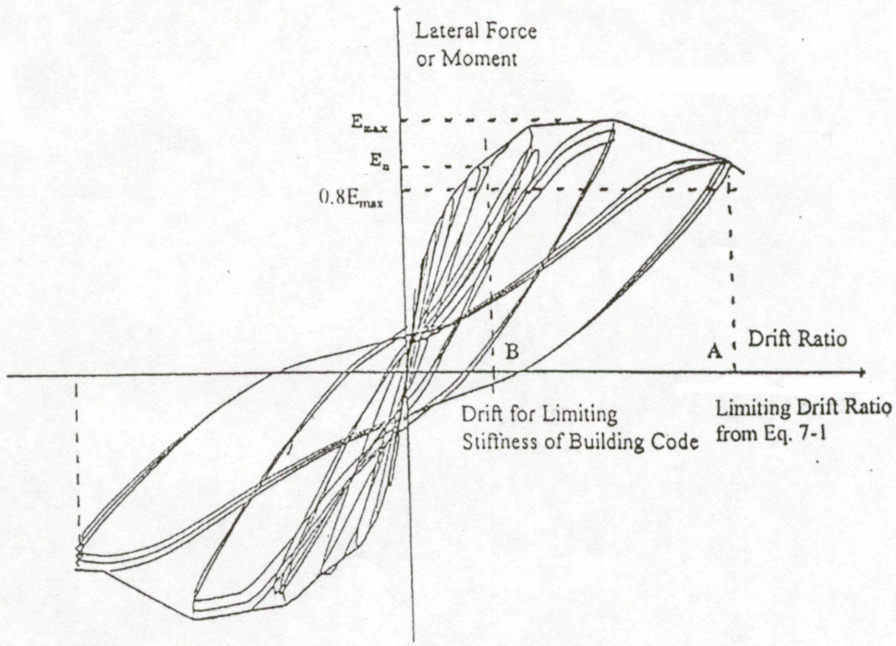

Fig. C9.6.9.1 Quantities used in evaluating acceptance criteria.

9.6.9.1.1. Where nominal strengths for opposite loading directions differ, as is likely for C-, L- or T-shaped walls, the criterion of Sec. 9.6.9.1.1 applies separately to each direction.

9.6.9.1.2. At high cyclic-drift ratios, strength degradation is inevitable. To limit the level of degradation so that drift ratio demands do not exceed anticipated levels, a maximum strength degradation of $0.20E_{max}$ is specified. Where strengths differ for opposite loading directions, this requirement applies independently to each direction.

9.6.9.1.3. If the relative energy dissipation ratio is less than 1/8, there may be inadequate damping for the building as a whole. Oscillations may continue for some time after an earthquake, producing low-cycle fatigue effects, and displacements may become excessive.

If the stiffness becomes too small around zero drift ratio, the structure will be prone to large displacements for small lateral force changes following a major earthquake. A hysteresis loop for the third cycle between peak drift ratios of 1/10 times the limiting drift ratio given by Eq. 9.6.1, that has the form shown in Figure C9.6.9.1, is acceptable. At zero drift ratio, the stiffnesses for positive and negative loading are about 11 percent of the initial stiffnesses. Those values satisfy Sec. 9.6.9.1.3. An unacceptable hysteresis loop form would be that shown in Figure C9.6.9.1.3 where the stiffness around zero drift ratio is unacceptably small for both positive and negative loading.

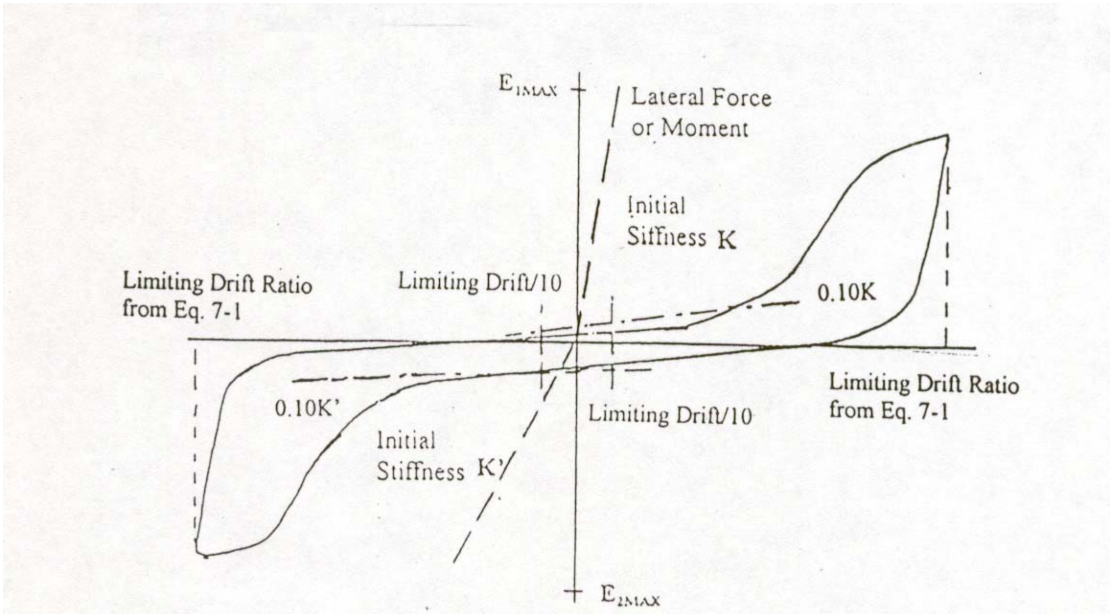

Figure C 9.6.9.1.3 Unacceptable hysteretic behavior

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Appendix to Chapter 9

UNTOPPED PRECAST DIAPHRAGMS

Although not directly addressed in the code, untopped precast components have been used as diaphragms in high seismic regions. Untopped hollow-core planks with grouted joints and end chords have performed successfully both in earthquakes and in laboratory tests, (Elliot et al., 1992; Menegotto, 1994; Priestley et al., 1999). Experience has also demonstrated the unsuccessful use of cast-in-place concrete topping as diaphragms (Iverson and Hawkins, 1994). Where problems have occurred, they have not been inherently with the precast construction, but the result of a failure to address fundamental requirements of structural mechanics.

This section provides conditions that are intended to ensure that diaphragms composed of precast components are designed with attention to the principles required for satisfactory behavior. Each condition addresses requirements that should be considered for all diaphragms, but which are particularly important in jointed construction. Specific attention should be paid to providing a complete load path that considers force transfer across all joints and connections.

A9.2 DESIGN REQUIREMENTS

A9.2.1 Configuration. Out-of-plane offsets in the vertical elements of the seismic-force-resisting system place particularly high demands on the diaphragm in providing a continuous load path. Untopped precast diaphragms are not suitable for this condition. It must be recognized that the demand on diaphragms in buildings with these plan irregularities requires special attention. In accordance with Sec. 4.6.3.2 the design force for the diaphragm should be increased by at least 25 percent when such irregularities are present in structures assigned to Seismic Design Category D, E, or F.

A9.2.2 Diaphragm demand. Following the principle that the diaphragm is not generally an appropriate location for inelastic behavior and, in particular, for untopped precast diaphragms, specific direction is provided that elastic models should be used for diaphragm analysis. Connections are subject to a combination of load effects (Fleischman et al., 1998). The distribution of loads may change after yielding, and therefore the design of the diaphragm should avoid yielding.

Since the diaphragm is not generally an appropriate location for inelastic behavior, it should be designed to a level of strength that is intended to ensure that the ductility and yield strength of the seismic-force-resisting system can be mobilized before the diaphragm yields. While research (Fleischman et al., 1998) suggests that the diaphragm demand will not exceed twice the equivalent lateral forces used for the vertical system design, Table 4.3-1 prescribes an overstrength factor, Ω_0, and Sec. 4.3.3 prescribes a redundancy factor, ρ, for the systems that should be used. If an analysis of the probable strength of the seismic-force-resisting system is made to determine a lower demand on the diaphragm, the design force used should still be sufficient to attempt to ensure that the diaphragm remains elastic. For that reason a 1.25 factor is specified.

A9.2.3 Mechanical connections. Although the design procedures prescribed in these sections are intended to ensure elastic behavior at the level of the code design forces, it is recognized that catastrophic events may exceed code requirements. Under such circumstances, it is important that the connections possess ductility under reversed cyclic loading. The intent, in these sections, is for the connection capacity to be limited by steel yielding of the connector and not by brittle concrete failure or weld fracture.

Substantiating experimental evidence to demonstrate through testing and evaluation that mechanical connections satisfy the principles specified in ACI T1.1-01 and ATC-24, and can develop the required capacity and ductility, should meet the following criteria:

Test Procedures:

1. Prior to testing, a design procedure should have been developed for prototype connections having the generic form that is to be tested for acceptance.
2. That design procedure should be used to proportion the test specimens.
3. Specimens should not be less than two-thirds scale.
4. Test specimens should be subject to a sequence of reversing cycles having increasing limiting displacements.
5. Three fully reversed cycles should be applied at each limiting displacement.
6. The maximum load for the first sequence of three cycles should be 75 percent of the calculated nominal strength of the connection, E_n.
7. The stiffness of the connection should be defined as 75 percent of the calculated nominal strength of the connection divided by the corresponding measured displacement, δ_m.
8. Subsequent to the first sequence of three cycles, limiting displacements should be incremented by values not less than 1.0, and not more than 1.25 times δ_m.

Acceptance Criteria:

1. The connection should develop a strength, E_{max}, greater than its calculated nominal strength, E_n.
2. The strength, E_{max}, should be developed at a displacement not greater than $3\delta_m$.
3. For cycling between limiting displacements not less than $3\delta_m$, the peak force for the third loading cycle for a given loading direction should not be less than $0.8\ E_{max}$ for the same loading direction.

Results of reversed cyclic loading tests on typical connections are reported in Spencer (1986) and Pincheira et al. (1998).

A9.2.4 Cast-in-place strips. Successful designs may include a combination of untopped precast components with areas of concrete topping in locations of high force demand or concentration. Such topping can allow for continuity of reinforcement across joints. For such designs, the requirements for topping slab diaphragms apply to the topped portions.

A9.2.5 Deformation compatibility. An important element in the *Provisions* is attention to deformation compatibility requirements. Reduction in effective shear and flexural stiffness for the diaphragm is appropriate in evaluating the overall effects of drift on elements that are not part of the seismic-force-resisting system. This approach should encourage the use of more vertical elements to achieve shorter spans in the diaphragm and result in improved system redundancy and diaphragm continuity. Redundancy will also improve the overall behavior should any part of the diaphragm yield in a catastrophic event.

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Chapter 10 Commentary

COMPOSITE STEEL AND CONCRETE STRUCTURE DESIGN REQUIREMENTS

10.1 GENERAL

The 1994 Edition of the *NEHRP Recommended Provisions* included a new chapter on composite steel and concrete structures. The requirements in that chapter have been updated and incorporated in Part II of the 1997 Edition of the AISC *Seismic Provisions*. This edition of the *NEHRP Recommended Provisions* includes by reference Part II of the AISC *Seismic Provisions (1997)* together with the underlying AISC-LRFD (1999) and ACI 318 (1999) standards. Part II of the AISC *Seismic Provisions* provides definitions for composite systems consistent with the system designations in Table 4.3-1 and specifies requirements for the seismic design of composite systems and components.

10.4 SEISMIC DESIGN CATEGORIES D, E, AND F

In general, available research shows that properly detailed composite elements and connections can perform as well as, or better than, structural steel and reinforced concrete components. However, due to the lack of design experience with certain types of composite structures in high seismic risk areas, usage of composite systems in Seismic Design Categories D and above requires documentation (substantiating evidence) that the proposed system will perform as intended by Part II of the AISC *Seismic Provisions* and as implied by the *R* values in Table 4.3-1. It is intended that the substantiating evidence consist of a rational analysis that considers force transfer between structural steel, reinforced concrete, and composite elements and identifies locations in the structure required to sustain inelastic deformations and dissipate seismic energy. Design of composite members and connections to sustain inelastic deformations must be based on models and criteria substantiated by test data. For many composite components, test data and design models are available and referenced in the commentary to the AISC *Seismic Provisions* – Part II (1997).

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Chapter 11 Commentary

MASONRY STRUCTURE DESIGN REQUIREMENTS

11.1.2 References. The main concern of Chapter 11 is the proper detailing of masonry construction for earthquake resistance. The bulk of the detailing requirements in this chapter are contained in ACI 530/ASCE 5/TMS 402. The commentary for ACI 530/ASCE 5/TMS 402 contains a valuable discussion of the rationale behind detailing requirements that is not repeated here.

11.2.1.5.1 Shear keys. Shear keys provide resistance to the movement of shear walls when yielding of the reinforcing steel occurs. This phenomenon was observed in tests by Klingner. (Leiva and Klingner 1991). There has been no field verification of shear wall movement under seismic events. The shear key requirements are based on judgment and sizes are based on current construction procedures.

11.2.2.3 Article 1.3 permits the use of structural clay wall-tile meeting the requirements of ASTM C 34. At the time of publication, it was felt that the existing detailing requirements for masonry elements did not adequately address the brittle nature of clay wall-tile units.

11.2.2.11 The nominal shear strength of coupling beams must be equal to the shear caused by development of a full yield hinge at each end of the coupling beams. This nominal shear strength is estimated by dividing the sum of the calculated yield moment capacity of each end of the coupling beams, M_1 and M_2, by the clear span length, L.

A coupling beam may consist of a masonry beam and a part of the reinforced concrete floor system. Reinforcement in the floor system parallel to the coupling beam should be considered as a part of the coupling beam reinforcement. The limit of the minimum width of floor that should be used is six times the floor slab thickness. This quantity of reinforcement may exceed the limits of Sec. 3.2.3.5 but should be used for the computation of the normal shear strength.

11.2.2.12 The theory used for design of beams has a limited applicability to deep beams. Shear warping of the cross section and a combination of diagonal tension stress and flexural tension stress in the body of the deep beam requires that deep beam theory be used for design of members that exceed the specified limits of span to depth ratio. Analysis of wall sections that are used as beams generally will result in a distribution of tensile stress that requires the lower one-half of the beam section to have uniformly distributed reinforcement. The uniform distribution of reinforcement resists tensile stress caused by shear as well as flexural moment.

The flexural reinforcement for deep beams must meet or exceed the minimum flexural reinforcement ratio of Sec. 3.2.4.3.2. Additionally, horizontal and vertical reinforcement must be distributed throughout the length and depth of deep beams and must provide reinforcement ratios of at least $0.0007bd$. Distributed flexural reinforcement may be included in the calculations of the minimum distributed reinforcement ratios.

11.2.2.13 Corrugated sheet metal ties are prohibited from use in Seismic Design Categories E and F due to their decreased capacity in transferring loads.

11.2.2.14 Masonry pryout refers to a failure mode of a shear anchor in which the embedded end of the anchor moves opposite to the direction of applied shear, prying out a roughly semi-conical body of masonry (concrete, as applicable) behind the anchor. It is not the same as a "breakout," which refers to a failure mode of a shear anchor in which a body of masonry (or concrete, as applicable) is broken off between the anchor and a free edge, in the direction of applied shear.

11.4 GLASS-UNIT MASONRY AND MASONRY VENEER

Chapters 11 and 12 of ACI 530-95/ASCE 5-95/TMS 402-95 were introduced into the 1997 *Provisions* to address design of glass-unit masonry and masonry veneer. Direct reference is made to these chapters for design requirements. Investigations of seismic performance have shown that architectural components meeting these requirements perform well (Jalil, Kelm, and Klingner, 1992; and Klingner, 1994).

11.5 PRESTRESSED MASONRY

Allowable stress provisions are set forth in MSJC Chapter 4. There are no strength design provisions for prestressed masonry. There is a paucity of data on the cyclic testing of prestressed shear walls. There is only one published report of cyclic testing of prestressed shear walls in-plane using a testing protocol similar to the sequential phased displacement method used in the TCCMaR program. This report considers specimens both partially and fully grouted using only prestressed bar reinforcing. There is no published in-plane cyclic test data using prestressed strand, nor any published data using prestressed reinforcing in combination with mild steel reinforcing. There is some additional unpublished data on in-plane testing of prestressed masonry shear walls using prestressed bars only.

The data shows that solid grouted prestressed masonry shear walls subjected to in-plane cyclic displacements perform as an essentially elastic system with stiffness degradation in each cycle. Little energy is dissipated in the hysterisis loops. Although reasonably large displacements can be reached, there is essentially no ductile behavior. The data on partially grouted walls is sparse and shows inability to reach large displacement before failure. The data shows that MSJC Eq. 3-21 provides a reasonable estimate for the shear capacity for solid grouted walls.

The TCCMaR research showed that the ductility of a masonry wall loaded in-plane was highly dependent on the level of axial load and the amount of reinforcing. Ductile behavior declines significantly at axial loads in excess of 100 psi; ductile behavior also declines significantly when the reinforcement ratio is high. The addition of prestressing to a wall with mild steel reinforcing will decrease the ductility.

Because of the limited data and the potential for non-ductile, prestressed masonry shear walls are restricted to Seismic Design Categories A and B and the R factor is set at 1½. As more research becomes available, these restrictions could be eased.

11.6 ANCHORING TO MASONRY

This section covers cast-in-place headed anchor bolts and bent-bar anchors (J- or L-bolts) in grout. General background information on this topic is given in CEB, 1995.

The tensile capacity of a headed anchor bolt is governed by yield and fracture of the anchor steel or by breakout of a roughly conical volume of masonry starting at the anchor head and having a fracture surface oriented at 45 degrees to the masonry surface. Steel capacity is calculated using the effective tensile stress area of the anchor (that is, including the reduction in area of the anchor shank due to threads). Masonry breakout capacity is calculated using expressions adapted from concrete design, which use a simplified design model based on a stress of $4\sqrt{f'_m}$ uniformly distributed over the area of that right circular cone, projected onto the surface of the masonry. Reductions in breakout capacity due to nearby edges or adjacent anchors are computed in terms of reductions in those projected areas (Brown and Whitlock, 1983).

The tensile capacity of a bent-bar anchor bolt (J- or L-bolt) is governed by yield and fracture of the anchor steel, by tensile cone breakout of the masonry, or by straightening and pullout of the anchor from the masonry. Capacities corresponding to the first two failure modes are calculated as for headed anchor bolts. Pullout capacity is calculated as proposed by Shaikh (1996). Possible contributions to tensile pullout capacity due to friction are neglected.

The tensile breakout capacity of a headed anchor is usually much greater than the pullout capacity of a J- or L-bolt. The designer is encouraged to use headed anchors when anchor tensile capacity is critical.

The shear capacity of a headed or a bent-bar anchor bolt is governed by yield and fracture of the anchor steel or by masonry shear breakout. Steel capacity is calculated using the effective tensile stress area (that-is, threads are conservatively assumed to lie in the critical shear plane). Shear breakout capacity is calculated as proposed by Brown and Whitlock, 1983.

Under static shear loading, bent-bar anchor bolts (J- or L-bolts) do not exhibit straightening and pullout. Under reversed cyclic shear, however, available research suggests that straightening and pullout may occur. Headed anchor bolts are recommended for such applications (Malik et al., 1982).

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Chapter 12 Commentary

WOOD STRUCTURE DESIGN REQUIREMENTS

12.1 GENERAL

12.1.2 References. Wood construction practices have not been codified in a form that is standard throughout the country. The 2003 *Provisions* incorporates by reference the *AF&PA ASD/LRFD Supplement, Special Design Provisions for Wind and Seismic (SDPWS)* and *the 2003 International Residental Code (IRC).* Many wood frame structures are a combination of engineered wood and "conventional" light-frame construction. Wood also is used in combination with other materials (American Institute of Timber Construction, 1985; Breyer, 1993; Faherty and Williamson, 1989; Hoyle and Woeste, 1989; Somayaji, 1992; Stalnaker and Harris, 1989). The requirements of the model building codes were used as a resource in developing the requirements introduced in the 1991 *Provisions* and further modified since then. The general requirements of Chapter 12 cover construction practices necessary to provide a performance level of seismic resistance consistent with the purposes stated in Chapter 1. These requirements also may be related to gravity load capacity and wind force resistance which is a natural outgrowth of any design procedure. For the 2003 *Provisions*, the reference documents continue to be grouped according to their primary focus into three subsections: Sec. 12.1.2.1, Engineered Wood Construction; Sec. 12.1.2.2, Conventional Construction; and Sec. 12.1.2.3, Materials Standards.

12.2 DESIGN METHODS

Prior to the publication of AF&PA/ASCE 16, typical design of wood frame structures followed the American Forest and Paper Association (AF&PA) *National Design Specification for Wood Construction* (NDS) (AF&PA, 1991). The NDS is based on "allowable" stresses and implied factors of safety. However, the design procedure provided by the *Provisions* was developed on the premise of the resistance capacity of members and connections at the yield level (ASCE, 1988; Canadian Wood Council, 1990 and 1991; Keenan, 1986). In order to accommodate this difference in philosophy, the 1994 and prior editions of the *Provisions* made adjustments to the tabulated "allowable" stresses in the reference documents.

With the completion of the *Load and Resistance Factor Standard for Engineered Wood Construction* (AF&PA/ASCE, 1995), the modifications and use of an "allowable" stress based standard was no longer necessary. Therefore, the 1997 *Provisions* included the LRFD standard by reference (AF&PA/ASCE 16) and used it as the primary design procedure for engineered wood construction. The use of AF&PA/ASCE 16 continues in the 2003 *Provisions.*

Conventional light-frame construction, a prescriptive method of constructing wood structures, is allowed for some design categories. These structures must be constructed according to the requirements set forth in Sec. 12.4 and applicable reference documents. If the construction deviates from these prescriptive requirements, the engineered design requirements of Sec. 12.2 and 12.3 and AF&PA/ASCE 16 must be followed. If a structure that is classified as conventional construction contains some structural elements that do not meet the requirements of conventional construction, the elements in question can be engineered without changing the rest of the structure to engineered construction. The extent of design to be provided must be determined by the responsible registered design professional; however, the minimum acceptable extent is often taken to be force transfer into the element, design of the element, and force transfer out of the element. This does not apply to a structure that is principally an engineered structure with minor elements that could be considered conventional. When more than one braced wall line or diaphragm in any area of a conventional residence requires design, the nature of the construction may have changed, and engineered design might be appropriate for the entire seismic-force-resisting system. The absence of a ceiling diaphragm may also create a configuration that is non-conventional. The requirement for engineering portions of a conventional construction structure to maintain lateral-force resistance and stiffness is added to provide displacement compatibility.

Alternate strength of members and connections. It remains the intent of the *Provisions* that load and resistance factor design be used. When allowable stress design is to be used, however, the factored resistance of members and connections subjected to seismic forces acting alone or in combination with other prescribed loads shall be determined using a capacity reduction factor, ϕ, times 2.16 times the allowable stresses permitted in the *National Design Specification for Wood Construction* (NDS) and supplements (AF&PA, 1991). The allowable stresses used shall not include a duration of load factor, C_D. The value of the capacity reduction factor, ϕ, shall be as follows:

Wood members	
In flexure	$\phi = 1.00$
In compression	$\phi = 0.90$
In tension	$\phi = 1.00$
In shear and torsion	$\phi = 1.00$
Connectors	
Anchor bolts, bolts, lag bolts, nails, screws, etc.	$\phi = 0.85$
Bolts in single shear in members of a seismic-force-resisting system	$\phi = 0.40$

These "soft" conversions from allowable stress design values to load and resistance factor design values first appeared in Sec. 9.2 in the 1994 *Provisions*. An alternative method of calculating soft conversions is provided in ASTM D 5457-93. The reader is cautioned, however, that the loads and load combinations to be used for conversion are not specified so it is incumbent upon the user to determine appropriate conversion values. Wood frame structures assigned to Seismic Design Category A, other than one- and two-family dwellings, must comply with Sec. 12.4 or if engineered need only comply with the reference documents and Sec. 1.5. Exceptions addressing one- and two-family detached dwellings appear in Sec.

12.2.1 Seismic Design Categories B, C, and D. Seismic Design Categories B, C, and D were combined in the 1997 *Provisions*. At the same time, subsections on material limitations and anchorage requirements were moved. This was based on the philosophy that detailing requirements should vary based on R value rather than seismic design category.

Structures assigned to Seismic Design Categories B, C, and D are required to meet the minimum construction requirements of Sec. 12.4 (Sherwood and Stroh, 1989) or must be engineered using standard design methods and principles of mechanics. Conventional light-frame construction requirements were modified in the 1991 *Provisions* to limit the spacing between braced wall lines based on calculated capacities to resist the loads and forces imposed.

Engineered structures assigned to Seismic Design Categories B, C, and D are required to conform to the provisions of Sec. 12.2 and 12.3. Included in these sections are general design limitations, limits on wood resisting forces contributed by concrete or masonry, shear wall and diaphragm aspect ratio limitations, and requirements for distribution of shear to vertical resisting elements.

12.2.2 Seismic Design Categories E and F. If the provisions of Chapter 12 apply, Seismic Design Category E and F structures require an engineered design. Conventional construction is not considered rigorous enough for structures expected to be functional following a major seismic event. For Seismic Design Category E and F structures, close attention to load path and detailing is required.

Structures assigned to Seismic Design Category E and F require blocked diaphragms. Structural-use panels must be applied directly to the framing members; the use of gypsum wallboard between the structural-use panels and the framing members is prohibited because of the poor performance of nails in gypsum. Restrictions on allowable shear values for structural-use shear panels when used in conjunction with concrete and masonry walls are intended to provide for deformation compatibility of the different materials.

12.2.3.1 Discussion of cyclic test protocol is included in ATC (1995), Dolan (1996), and Rose (1996).

12.2.3.2 and 12.2.3.7 The mid-span deflection of a simple-span, blocked wood structural panel diaphragm uniformly nailed throughout may be calculated by use of the following formula:

$$\Delta = \frac{5vL^3}{8bEA} + \frac{vL}{4Gt} + 0.188Le_n + \frac{\sum(\Delta_c X)}{2b}$$

where:

Δ = the calculated deflection, in. (mm).

v = maximum shear due to factored design loads in the direction under consideration, lb/ft (kN/m).

L = diaphragm length, ft (m).

b = diaphragm width, ft (m).

E = elastic modulus of chords, psi (MPa).

A = area of chord cross-section, in.^{2} (mm^2).

Gt = panel rigidity through the thickness, lb/in. (N/mm).

e_n = nail deformation, in. (mm).

$\Sigma(\Delta_c X)$ = sum of individual chord-splice slip values on both sides of the diaphragm, each multiplied by its distance to the nearest support, in. (mm).

If not uniformly nailed, the constant 0.188 in the third term must be modified accordingly. See ATC 7 (Applied Technology Council, 1981).

This formula was developed based on engineering principles and monotonic testing. Therefore, it provides an estimate of diaphragm deflection due to loads applied in the factored resistance shear range. The effects of cyclic loading and resulting energy dissipation may alter the values for nail deformation in the third term, as well as chord splice effects of the fourth term, if mechanically-spliced wood chords are used. The formula is not applicable to partially-blocked diaphragms.

The deflection of a blocked wood structural panel shear wall may be calculated by use of the following formula.

$$\Delta = \frac{8vh^3}{bEA} + \frac{vh}{Gt} + 0.75he_n + \frac{h}{b}d_a$$

where:

Δ = the calculated deflection, in. (mm).

v = maximum shear due to factored design loads at the top of the wall, lb/ft (kN/m).

h = shear wall height, ft (m).

b = shear wall width, ft (m).

E = elastic modulus of boundary element (vertical member at shear wall boundary), psi (MPa).

A = area of boundary element cross-section (vertical member at shear wall boundary), in.^{2} (mm^2).

Gt = panel rigidity through the thickness, lb/in. (N/mm).

e_n = nail deformation, in. (mm).

d_a = deflection due to anchorage details (rotation and slip at hold downs), in. (mm).

Guidance for use of the above two equations can be found in the references.

One stipulation is that there are no accepted rational methods for calculating deflections for diaphragms and shear walls that are sheathed with materials other than wood structural panel products fastened with nails. Therefore, if a rational method is to be used, the capacity of the fastener in the sheathing material must be validated by acceptable test procedures employing cyclic forces or displacements. Validation must include correlation between the overall stiffness and capacity predicted by principles of mechanics and that observed from test results. A diaphragm or shear wall sheathed with dissimilar materials on the two faces should be designed as a single-sided wall using the capacity of the stronger of the materials and ignoring the weaker of the materials.

TABLE C12.2A
"e_n" FASTENER SLIP EQUATIONS FOR USE IN CALCULATING DIAPHRAGM AND SHEAR WALL DEFLECTION DUE TO FASTENER SLIP

Fastener	Minimum Penetration (in.)	Maximum Fastener Loads - V_n (lb/fastener)	Fastener Slip, e_n (in.)[1]	
			Fabricated w/green (>19% m.c.) lumber	Fabricated w/dry ($\leq$ 19% m.c.) lumber
6d common nail	1-1/4	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
8d common nail	1-3/8	220	$(V_n/857)^{1.869}$	$(V_n/616)^{3.018}$
10d common nail	1-1/2	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$
14-ga staple	1 to <2	140	$(V_n/902)^{1.464}$	$(V_n/596)^{1.999}$
14-ga staple	$\geq$2	170	$(V_n/674)^{1.873}$	$(V_n/461)^{2.776}$

For SI: 1 inch = 25.4 mm, 1 pound = 4.448 N.

1.Values apply to plywood and OSB fastened to lumber with a specific gravity of 0.50 or greater except that the slip shall be increased by 20 percent when plywood is not Structural I.

TABLE C12.2B
VALUES OF *Gt* FOR USE IN CALCULATING DEFLECTION OF WOOD STRUCTURAL PANEL DIAPHRAGMS AND SHEAR WALLS

PANEL TYPE	MINIMUM THICKNESS (in.)	SPAN RATING	VALUES OF *Gt* (lb/in. panel depth or width)						
			STRUCTURAL I			OTHER			
			3-ply Plywood	4- and 5-ply Plywood[1]	OSB	3-ply Plywood	4-ply Plywood	5-ply Plywood[1]	OSB
Sheathing	3/8	24/0	32,500	41,500	77,500	25,000	32,500	37,500	77,500
	7/16	24/16	35,000	44,500	83,500	27,000	35,000	40,500	83,500
	15/32	32/16	35,000	44,500	83,500	27,000	35,000	40,500	83,500
	19/32	40/20	37,000	47,500	88,500	28,500	37,000	43,000	88,500
	23/32	48/24	40,500	51,000	96,000	31,000	40,500	46,500	96,000
Single Floor	19/32	16 oc	35,000	44,500	83,500	27,000	35,000	40,500	83,500
	19/32	20 oc	36,500	46,000	87,000	28,000	36,500	42,000	87,000
	23/32	24 oc	39,000	49,500	93,000	30,000	39,000	45,000	93,000
	7/8	32 oc	47,000	59,500	110,000	36,000	47,000	54,000	110,000
	1-1/8	48 oc	65,500	83,500	155,000	50,500	65,500	76,000	155,000

PANEL TYPE	Thickness (in.)	VALUES OF *Gt* (lb/in. panel depth or width)		
		STRUCTURAL I	OTHER	
		All Plywood Grades	Marine	All Other Plywood Grades
Sanded Plywood	¼	31,000	31,000	24,000
	11/32	33,000	33,000	25,500
	3/8	34,000	34,000	26,000
	15/32	49,500	49,500	38,000
	½	50,000	50,000	38,500
	19/32	63,500	63,500	49,000
	5/8	64,500	64,500	49,500
	23/32	65,500	65,500	50,500
	¾	66,500	66,500	51,000
	7/8	68,500	68,500	52,500
	1	95,500	95,500	73,500
	1-1/8	97,500	97,500	75,000

For SI: 1 inch = 25.4 mm, 1 pound/inch of panel depth or width = 0.1751 N/mm.

1. Applies to plywood with 5 or more layers; for 5 ply/3 layer plywood, use values for 4 ply.

Effect of Green Lumber Framing on Diaphragms and Shear Walls: A recent study of wood structural panel shear walls (APA Report T2002-53) fabricated with wet lumber and tested when dry shows that shear stiffness is affected to a much larger degree than shear strength when compared to control specimens fabricated with dry lumber and tested when dry. The shear strength of walls fabricated with wet lumber showed negligible reductions (0-7 percent) when compared to control specimens. The shear stiffness of walls fabricated with wet lumber was always reduced when compared to control specimens. Observed reductions in stiffness were consistent with predicted stiffness reductions based on use of Eq. C12.2A and nail slip values specified in Table C12.2A. For example, measured deflection of a standard wall configuration at the shear wall factored unit shear value was approximately 2.5 times the deflection of the control specimen and predicted deflections were within 0.05 inches of the test deflection for both the fabricated wet specimen and control specimen.

As a result of these tests, direct consideration of shear wall stiffness is recommended in lieu of applying shear wall strength reductions when wood structural panel shear walls are fabricated with wet lumber (e.g. moisture content > 19 percent). To address reduced shear stiffness for shear walls fabricated with wet lumber, story drift calculations should be based on e_n values for lumber with moisture content > 19 percent to determine compliance with allowable story drift limits of the Provisions. A similar relationship can be expected when analyzing the deflection of diaphragms.

The designer should keep in mind that deflection equations are verified for walls with wood structural panel sheathing only and does not address the increased stiffness provided by finish materials such as gypsum and stucco. The CUREE-Caltech Woodframe project illustrated that finishes such as gypsum wallboard and stucco increase the stiffness of the walls. While these deflection equations are currently the best estimate of wood structural panel wall deflection, actual wall deflections will likely be less than predicted deflections due to the presence of finish materials in typical wall construction.

12.2.3.11 and 12.2.3.12. Tie-down devices should be based on cyclic tests of the connection to provide displacement capacity that allows rotation of the end post without significant reduction in the shear wall resistance. The tie-down device should be stronger than the lateral capacity of the wall so that the mechanism of failure is the sheathing fasteners and not a relatively brittle failure of the wall anchorage. For devices for which the published resistance is in allowable stress design values, the nominal strength shall be determined by multiplying the allowable design load by 1.3. The nominal strength of a tie-down device may be determined as the average maximum test load resisted without failing under cyclic loading. In that case, the average should be based on tests of at least three specimens.

Calculations of deflection of shear walls should include the effects of crushing under the compression chord, uplift of the tension chord, slip in the tie-down anchor with respect to the post, and shrinkage effects of the platforms, which primarily consist of floor framing members. Movement associated with these variables can be significant and neglecting their contribution to the lateral displacement of the wall will results in a significant under-estimation of the deflection. Custom tie-down devices are permitted to be designed using methods for the particular materials used and AF&PA/ASCE 16 under alternative means and methods.

Tie-down devices that permit significant vertical movement between the tie-down and the tie-down post can cause failure in the nails connecting the shear wall sheathing to the sill plate. High tension and tie-down rotation due to eccentricity can cause the bolts connecting the tie-down bracket to the tie-down post to pull through and split the tie-down post. Devices that permit such movement include heavily loaded, one-sided, bolted connections with small dimensions between elements resisting rotation due to eccentricity. Any device that uses over-drilled holes, such as most bolted connections, will also allow significant slip to occur between the device and the tie-down post before load is restrained. Both the NDS and the steel manual specify that bolt holes will be over-drilled as much as 1/16 in. (2 mm). This slip is what causes much of the damage to the nails connecting the sheathing to the sill plate. Friction between the tie-down post and the device cannot be counted on to resist load because relaxation in the wood will cause a loss of clamping and, therefore, a loss in friction over time. This is why all tests should be conducted with the bolts "finger tight" as opposed to tightening with a wrench.

Cyclic tests of tie-down connections must follow a pattern similar to the sequential phased displacement (SPD) tests used by Dolan (1996) and Rose (1996). These tests used full wall assemblies and therefore induced deflection patterns similar to those expected during an earthquake. If full wall assembly tests are not used to test the tie-down devices, it must be shown that the expected rotation as well as tension and compression are used. This is to ensure that walls using the devices will be able to deform in the intended manner. This allows the registered design professional to consider compatibility of deformations when designing the structure.

Splitting of the bottom plate of the shear walls has been observed in tests as well as in structures subjected to earthquakes. Splitting of plates remote from the end of the shear wall can be caused by the rotation of individual sheathing panels inducing upward forces in the nails at one end of the panel and downward forces at the other. With the upward forces on the nails and a significant distance perpendicular to the wall to the downward force produced by the anchor bolt, high cross-grain bending stresses occur. Splitting can be reduced or eliminated by use of large plate washers that are sufficiently stiff to reduce the eccentricity and by use of thicker sill plates. Thicker sill plates (3 in. nominal, 65 mm) are recommended for all shear walls for which Table 12.2-3a (or 12.2-3b) requires 3 in. nominal (65 mm) framing to prevent splitting due to close nail spacing. This is to help prevent failure of the sill plate due to high lateral loading and cross-grain bending.

The tendency for the nut on a tie-down bracket anchor bolt to loosen significantly during cycled loading has been observed in some testing. One tested method of limiting the loosening is to apply adhesive between the nut and tie-down bolt.

A logical load path for the structure must be provided so that the forces induced in the upper portions of the structure are transmitted adequately through the lower portions of the structure to the foundation.

In the 2003 *Provisions* update cycle anchorage provisions were divided into two distinct subsections to separately address anchorage for uplift and anchorage for in-plane shear. The title section was clarified to address both traditional segmented shear walls and perforated shear walls.

A prior *Provisions* requirement that nuts on both uplift anchors and in-plane shear anchors be prevented from loosening prior to covering the framing, was deleted. This provision was originally based on observed backing-off of nuts in a small number of cyclic tests of shear walls but in the large number of tests conducted since that time this phenomenon has not been observed to occur. It was felt that retaining the existing requirement for tightening the nuts prior to closing in the framing was sufficient to address this issue.

A prior *Provisions* requirement for the nominal strength of a tiedown to be equal to or exceed the factored resistance of the shear wall times Ω_o / 1.3, was replaced with simpler wording that has an equivalent effect and is intended primarily as a statement of design philosophy. The new language in Sec 12.2.3.11 only refers to the nominal strength of the tiedown and the nominal strength of the shear wall. Nominal strengths for typical nailed wood structural panel shear walls are set forth in Table 4.3A column B of AF&PA *ASD/LRFD Supplement, Special Design Provisions for Wind and Seismic*. In addition, similar language making the nominal strength of in-plane shear anchorage match the nominal strength values of the shear walls was added, to provide a basis for design of in-plane shear connections that is consistent with requirements for uplift anchorage. The capacity-based nominal strength have been introduced primarily as a statement of design philosophy, with the intent of forcing sheathing nailing to be the controlling failure mechanism. The complexity of load paths in wood frame buildings suggest that additional study is needed to achieve reliable development of desired failure mechanisms.

Plate washers are now specifically permitted to have a diagonal slot not exceeding 1-3/4 inches in length to facilitate placement within the width of the sill plate.

12.2.3.14 Sheathing nails should be driven flush with the surface of the panel, and not further. This could result in the nail head creating a small depression in, but not fracturing, the first veneer. This requirement is imposed because of the significant reduction in capacity and ductility observed in shear walls constructed with over-driven nails. It is advised that the edge distance for sheathing nails be increased as much as possible along the bottom of the panel to reduce the potential for the nails to pull through the sheathing.

12.3 GENERAL DESIGN REQUIREMENTS FOR ENGINEERED WOOD CONSTRUCTION

Engineered construction for wood structures as defined by the *Provisions* encompasses all structures that cannot be classified as conventional construction. Therefore, any structure exceeding the height limitations or having braced walls spaced at intervals greater than those prescribed in Table 12.4-1 or not conforming to the requirements in Sec. 12.4 must be engineered using standard design methods and principles of mechanics. Framing members in engineered wood construction are sized based on calculated capacities to resist the loads and forces imposed. Construction techniques that utilize wood for lateral force resistance in the form of diaphragms or shear walls are discussed further in Sec. 12.4. Limitations have been set on the use of wood diaphragms that are used in combination with concrete and masonry walls or where torsion is induced by the arrangement of the vertical resisting elements. A load path must be provided to transmit the lateral forces from the diaphragm through the vertical resisting elements to the foundation. It is important for the registered design professional to follow the forces down, as for gravity loads, designing each connection and member along the load path.

Although wood moment resisting frames are not specifically covered in the *Provisions*, they are not excluded by them. There are several technical references for their design, and they have been used in Canada, Europe, and New Zealand. Wood moment resisting frames are designed to resist both vertical loads and lateral forces. Detailing at columns to beam/girder connections is critical in developing frame action and must incorporate effects of member shrinkage. Detailed information can be obtained from the national wood research laboratories. There are many references that describe the engineering practices and procedures used to design wood structures that will perform adequately when subjected to lateral forces. The list at the end of this *Commentary* chapter gives some, but by no means all, of these.

Deformation compatibility The registered design professional should visualize the deformed shape of the structure to ensure that the connections provide the necessary ductility to allow the probable deflection demand placed on the structure. Unlike steel or other metal structures, wood is not a ductile material and virtually all of the ductility achieved in the structure is in the connections. The planned failure mechanism of wood structures must be through the connections, including the nailing of structural panels; otherwise the failure will be brittle in nature. The philosophy of strong, elastic columns and yielding beams cannot be projected from steel to wood structures. To enable a wood structure to deform and dissipate energy during a seismic event, the connections must be the weak link in the structure and must be ductile. Recent earthquakes, such as that in Northridge, California, have shown failures due to the fact that consideration of deformation compatibility was neglected.

As an example of a compatibility issue, consider the deformation compatibility between a tie-down connector to the tie-down post and the edge nailing of shear wall sheathing to the tie-down post and adjacent bottom plate. Recent testing and observations from the Northridge earthquake have suggested that the tie-down post experiences notable displacement before significant load can be carried through the tie-down connector. This is due, among other things, to the oversizing of the bolt holes in the tie-down post and the deformation and rotation of the tie-down bracket. Anchor bolts connecting the bottom plate to the foundation below tend to attempt to carry the shear wall uplift as the tie-down post moves. The sheathing, however, is nailed to both the bottom plate, which is held in place, and the tie-down post, which is being pulled up. The result is a large deformation demand being placed on the nails connecting the sheathing to the framing. This often results in the nails pulling out of the sheathing at the tie-down post corner and sometimes results in an unzipping effect where a significant portion of the remaining sheathing nailing fails as high loads cause one nailed connection to fail and move on to overstress the next nail. The most effective solution currently known is to limit the slip and deformation at the tie-down post by using a very stiff nailed or screwed tie-down.

Because this is an area where understanding of compatibility issues is just starting to develop, the Sec. 12.3.2 provision uses the wording “shall be considered in design” in lieu of the originally proposed “provision shall be made to ensure…” The intent is to provide guidance while not requiring the impossible.

If necessary, the stiffness of the wood diaphragms and shear walls can be increased with the use of adhesives (if adhesives are to be used). However, it should be noted that there are no rational methods for determining deflections in diaphragms that are constructed with non-wood sheathing materials. If the nail stiffness values or shear stiffness of non-wood sheathing materials is determined in a scientific manner, such as through experimental cyclic testing, the calculations for determining the stiffness of shear panels will be considered validated.

Limitation on forces contributed by concrete or masonry. Due to the significant difference in in-plane stiffness between wood and masonry or concrete systems, the use of wood members to resist the seismic forces produced by masonry and concrete is not allowed. This is due to the probable torsional response such a structure will exhibit. There are two exceptions where wood can be considered to be part of the seismic-load-resisting system. The first is where the wood is in the form of a horizontal truss or diaphragm and the lateral loads do not produce rotation of the horizontal member. The second exception is in structures of two stories or less in height. In this case, the capacity of the wood shear walls will be sufficient to resist the lower magnitude loads imposed. Five restrictions are imposed on these structures to ensure that the structural performance will not include rotational response and that the drift will not cause failure of the masonry or concrete portions of the structure.

Shearwalls and Diaphragms. Many wood-framed structures resist seismic forces by acting as a "box system." The forces are transmitted through diaphragms, such as roofs and floors, to reactions provided by shear walls. The forces are, in turn, transmitted to the lower stories and to the final point of resistance, the foundations. A shear wall is a vertical diaphragm generally considered to act as a cantilever from the foundation.

A diaphragm is a nearly horizontal structural unit that acts as a deep beam or girder when flexible in comparison to its supports and as a plate when rigid in comparison to its supports. The analogy to a girder is somewhat more appropriate since girders and diaphragms are made up as assemblies (American Plywood Association, 1991; Applied Technology Council, 1981). Sheathing acts as the "web" to resist the shear in diaphragms and is stiffened by the framing members, which also provide support for gravity loads. Flexure is resisted by the edge elements acting like "flanges" to resist induced tension or compression forces. The "flanges" may be top plates, ledgers, bond beams, or any other continuous element at the perimeter of the diaphragm.

The "flange" (chord) can serve several functions at the same time, providing resistance to loads and forces from different sources. When it functions as the tension or compression flange of the "girder," it is important that the connection to the "web" be designed to accomplish the shear transfer. Since most diaphragm "flanges" consist of many pieces, it is important that the splices be designed to transmit the tension or compression occurring at the location of the splice and to recognize that the direction of application of seismic forces can reverse. It should also be recognized that the shear walls parallel to the flanges may be acting with the flanges to distribute the diaphragm shears. When seismic forces are delivered at right angles to the direction considered previously, the "flange" becomes a part of the reaction system. It may function to transfer the diaphragm shear to the shear wall(s), either directly or as a drag strut between segments of shear walls that are not continuous along the length of the diaphragm.

For shear walls, which may be considered to be deep vertical cantilever beams, the "flanges" are subjected to tension and compression while the "webs" resist the shear. It is important that the "flange" members, splices at intermediate floors, and the connection to the foundation be detailed and sized for the induced forces.

The "webs" of diaphragms and shear walls often have openings. The transfer of forces around openings can be treated similarly to openings in the webs of steel girders. Members at the edges of openings have forces due to flexure and the higher web shear induced in them and the resultant forces must be transferred into the body of the diaphragm beyond the opening.

In the past, wood sheathed diaphragms have been considered to be flexible by many registered design professionals and model code enforcement agencies. The newer versions of the model codes now recognize that the determination of rigidity or flexibility for determination of how forces will be distributed is dependent on the relative deformations of the horizontal and vertical force-resisting elements. Wood sheathed diaphragms in structures with wood frame shear walls with various types of sheathing may be relatively rigid compared with the vertical resisting system and, therefore, capable of transmitting torsional lateral forces. A diaphragm is considered to be flexible if its deformation is two or more times that of the vertical force-resisting elements subjected to the same force.

Discussions of these and other topics related to diaphragm and shear wall design, such as cyclic testing and pitched or notched diaphragms, may be found in the references.

The capacity of shear walls must be determined either from tabulated values that are based on experimental results or from standard principles of mechanics. The tables of allowable values for shear walls sheathed with other than wood or wood-based structural-use panels were eliminated in the 1991 *Provisions* as a result of re-learning the lessons from past earthquakes and testing on the performance of structures sheathed with these materials during the Northridge earthquake. In the 1997 *Provisions* values for capacity for shear walls sheathed with wood structural panels were reduced from monotonic test values by 10 percent to account for the reduction in capacity observed during cyclic tests. This decision was reviewed for the 2000 edition of the *Provisions* due to the availability of an expanded data set of test results. The reduction was removed for the 2000 *Provisions* when the effect of the test loading protocol was determined to be the cause of the initial perceived reductions. Capacities for diaphragms were not reduced from the monotonic test values because the severe damage that occurred in shear walls has not been noted in diaphragms in recent earthquakes.

The *Provisions* are based on assemblies having energy dissipation capacities which were recognized in setting the *R* factors. For diaphragms and shear walls utilizing wood framing, the energy dissipation is almost entirely due to nail bending. Fasteners other than nails and staples have not been extensively tested under cyclic load application. When screws or adhesives have been tested in assemblies subjected to cyclic loading, they have had a brittle mode of failure. For this reason, adhesives are prohibited for wood framed shear wall assemblies in SDC C and higher and only the tabulated values for nailed or stapled sheathing are recommended. If one wished to use shear wall sheathing attached with adhesives, as an alternate method of construction in accordance with Sec. 1.1.2.5, caution should be used (Dolan and White, 1992; Foschi and Filiatrault, 1990). The increased stiffness will result in larger forces being attracted to the structure. The anchorage connections and adjoining assemblies must, therefore, be designed for these increased forces. Due to the brittle failure mode, these walls should be designed to remain elastic, similar to unreinforced masonry. The use of adhesives for attaching sheathing for diaphragms increases their stiffness, and could easily change the diaphragm response from flexible to rigid.

Horizontal distribution of shear. The *Provisions* define when a diaphragm can be considered to be flexible or rigid. The purpose is to determine whether the diaphragm should have the loads proportioned according to tributary area or stiffness. For flexible diaphragms, the loads should be distributed according to tributary area whereas for rigid diaphragms, the loads should be distributed according to stiffness.

The distribution of seismic forces to the vertical elements (shear walls) of the seismic-force-resisting system is dependent, first, on the stiffness of the vertical elements relative to that of the horizontal elements and, second, on the relative stiffness of the various vertical elements if they have varying deflection characteristics. The first issue is discussed in detail in the *Provisions*, which define when a diaphragm can be considered flexible or rigid and set limits on diaphragms that act in rotation or that cantilever. The second is largely an issue of engineering mechanics, but is discussed here because significant variations in engineering practice currently exist.

In situations where a series of vertical elements of the seismic-force-resisting system are aligned in a row, seismic forces will distribute to the different elements according to their relative stiffness.

Typical current design practice is to distribute seismic forces to a line of wood structural panel sheathed walls in proportion to the lengths of the wall segments such that each segment carries the same unit load. Wood structural panel sheathed wall segments without openings can generally be calculated to have a stiffness in proportion to the wall length when: the tie-down slip is ignored, the wood structural panel sheathing is selected from standard selection tables, and the aspect ratio limits of the *Provisions* are satisfied. For stiffness to be proportional to the wall length, the average load per nail for a given nail size must be approximately equal. Conversely, a wall could be stiffened by adding nails and reducing the calculated average load per nail. When including tie-down slip from anchors with negligible slip (1/16 in. [2 mm] or less), the assumption of wall stiffness proportional to length is still fairly reasonable. For larger tie-down slip values, wall stiffness will move towards being proportional to the square of the wall length; more importantly, however, the anchorage will start exhibiting displacement compatibility problems. For shear walls with aspect ratios higher than 2/1, the stiffness is no longer in proportion to the length and equations are not available to reasonably calculate the stiffness. For a line of walls with variations in tie-down slip, chord framing, unit load per nail, or other aspects of construction, distribution of load to wall segments will need to be based on a deflection analysis. The shear wall and diaphragm deflection equations that are currently available are not always accurate. As testing results become available, the deflection calculation formulas will need to be updated and design assumptions for distribution of forces reviewed.

Torsional diaphragm force distribution. A diaphragm is flexible when the maximum lateral deformation of the diaphragm is more than two times the average story drift. Conversely, a diaphragm will be considered rigid when the diaphragm deflection is equal to or less than two times the story drift. This is based on a model building code definition that applies to all materials.

For flexible diaphragms, seismic forces should be distributed to the vertical force-resisting elements according to tributary area or simple beam analysis. Although rotation of the diaphragm may occur because lines of vertical elements have different stiffness, the diaphragm is not considered stiff enough to redistribute seismic forces through rotation. The diaphragm can be visualized as a single-span beam supported on rigid supports.

For diaphragms defined as rigid, rotational or torsional behavior is expected and results in redistribution of shear to the vertical force-resisting elements. Requirements for horizontal shear distribution are in Sec. 5.2.4. Torsional response of a structure due to irregular stiffness at any level within the structure can be a potential cause of failure. As a result, dimensional and diaphragm ratio limitations are provided for different categories of rotation. Also, additional requirements apply when the structure is deemed to have a torsional irregularity in accordance with Table 4.3-2, Item 1a or 1b.

In order to understand limits placed on diaphragms acting in rotation, it is helpful to consider two different categories of diaphragms. Category I includes rigid diaphragms that rely on force transfer through rotation to maintain stability. An example would be an open front structure with shear walls on the other three sides. For this more structurally critical category, applicable limitations are:

Diaphragm may not be used to resist forces contributed by masonry or concrete in structures over one story.

The length of the diaphragm normal to the opening may not exceed 25 ft (to perpendicular shear walls), and diaphragm *L/b* ratios are limited as noted.

Additional limitations apply when rotation is significant enough to be considered a torsional irregularity.

Category II includes rigid diaphragms that have two or more supporting shear walls in each of two perpendicular directions but, because the center of mass and center of rigidity do not coincide, redistribute forces to shear walls through rotation of the diaphragm. These can be further divided into Category IIA where the center of rigidity and mass are separated by a small portion of the structure's least dimension and the magnitude of the rotation is on the order of the accidental rotation discussed in Sec. 5.2.4.2. For this level of rotation, an exception may result in no particular limitations being placed on diaphragm rotation for Category IIA. Category IIB, rigid diaphragms with eccentricities larger than those discussed in Sec. 5.2.4.2, are subject to the following limitations:

Diaphragm may not be used to resist forces contributed by masonry or concrete in structures over one story.

Additional limitations apply when rotation is significant enough to be considered a torsional irregularity.

Because flexible diaphragms have very little capacity for distributing torsional forces, further limitation of aspect ratios is used to limit diaphragm deformation such that rigid behavior will occur. The resulting deformation demand on the structure also is limited. Where diaphragm ratios are further limited, exceptions permit higher ratios where calculations demonstrate that higher diaphragm deflections can be tolerated. In this case, it is important to determine the effect of diaphragm rigidity on both the horizontal distribution and the ability of other structural elements to withstand resulting deformations.

Proposals to prohibit wood diaphragms acting in rotation were advanced following the 1994 Northridge earthquake. To date, however, the understanding is that the notable collapses in the Northridge earthquake occurred in part because of lack of deformation compatibility between the various vertical resisting elements rather than because of the inability of the diaphragm to act in rotation.

Diaphragm cantilever. Limitations concerning diaphragms that cantilever horizontally past the outermost shear wall (or other vertical element) are related to but distinct from those imposed because of diaphragm rotation. Such diaphragms can be flexible or rigid and for rigid diaphragms can be Category I, IIA or IIB. Both the limitations based on diaphragm rotation (if applicable) and the following limit on diaphragm cantilever must be considered:

Diaphragm cantilever may not exceed the lesser of 25 ft or two thirds of the diaphragm width.

Relative stiffness of vertical elements. In situations where a series of vertical elements of the seismic-force-resisting system are aligned in a row, the forces will distribute to the different elements according to their relative stiffnesses. This behavior needs to be taken into account whether it involves a series of wood structural panel shear walls of different lengths, a mixture of wood structural panel shear walls with diagonal lumber or non-wood sheathed shear walls, or a mixture of wood shear walls with walls of some other material such as concrete or masonry.

Diaphragm aspect ratio. The *L*/*b* for a diaphragm is intended to be the typical definition for aspect ratio. The diaphragm span, *L*, is measured perpendicular to the direction of applied force, either for the full dimension of the diaphragm or between supports as appropriate. The width, *b*, is parallel to the applied force (see Figure C12.3-1).

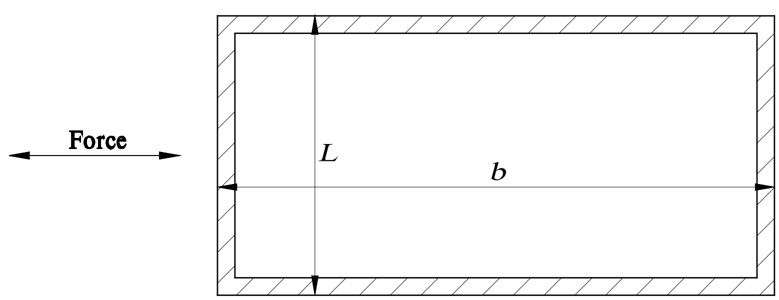


Figure C12.3-1 Diaphragm dimension definitions.

Single and double diagonally sheathed lumber diaphragms. Diagonally sheathed lumber diaphragms are addressed by the *Provisions* because they are still used for new construction in some regions. Shear resistance is based on a soft conversion from the model code allowable stress loads and capacities to *Provisions* strength loads for regions with high spectral accelerations. This will allow users in the western states, where this construction is currently being used, to continue with little or no change in requirements; at the same time, reasonable values are provided for regions with lower spectral

Shear wall aspect ratio. The *h/b* for a shear wall is intended to be the typical definitions for aspect ratio. The *h* of the shear wall is the clear story height (see Figure C12.3-2). The alternate definition of aspect ratio is only to be used where specific design and detailing is provided for force transfer around the openings. It is required that the individual wall piers meet the aspect ratio requirement (see Figure C12.3-3) and that the overall perforated wall also meet the aspect ratio requirement. Use of the alternate definition involves the design and detailing of chord and collector elements around the opening, and often results in the addition of blocking, strapping, and special nailing. As noted, the design for force transfer around the opening must use a rational analysis and be in accordance with AF&PA/ASCE 16, which discusses design principles for shear walls, diaphragms, and boundary elements.

In general, unit shear values for wood structural panel sheathing have been based on tests of shear wall panels with aspect ratios of 2/1 to 1/1. Narrower wall segments (that is, with aspect ratios greater than 2/1) have been a recent concern based on damage observations following the Northridge earthquake and based on results of recent research (Applied Technology Council, 1995; White and Dolan, 1996). In response, various limitations on aspect ratios have been proposed. In the *Provisions*, an aspect ratio adjustment, 2*b/h*, is provided to account for the reduced stiffness of narrow shear wall segments. This adjustment is based on a review of numerous tests of narrow aspect ratio walls by Technical Subcommittee 7. The maximum 3.5/1 aspect ratio is recommended based on constructability issues (placement of tie-downs) as well as reduced stiffness of narrower shear wall segments.

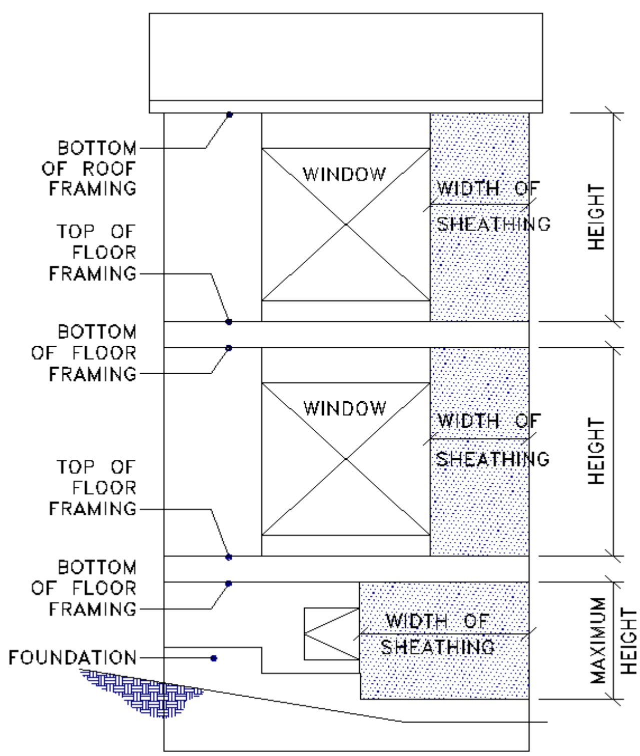


Figure C12.3-2 Typical shear wall height-to-width ratio.

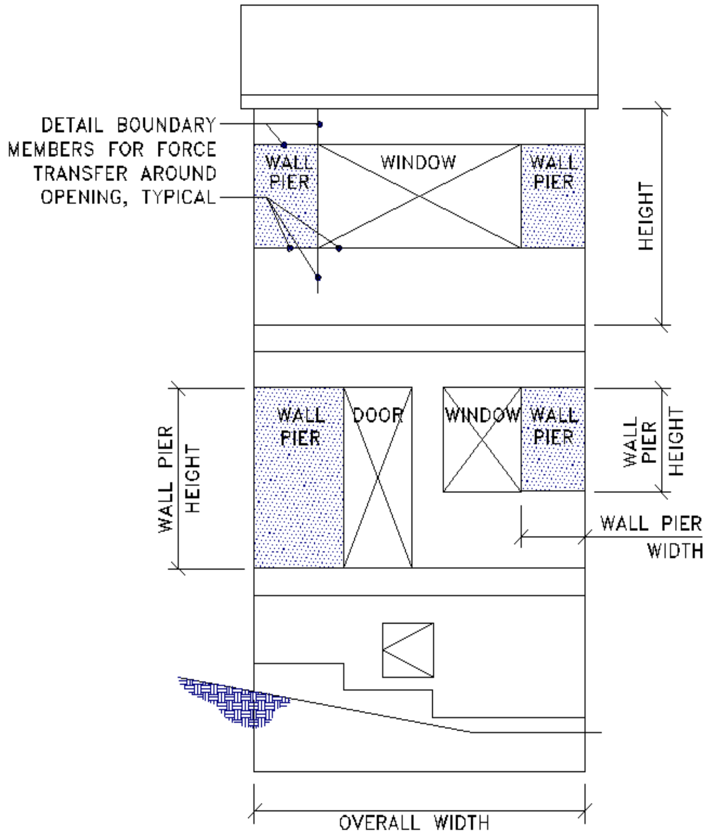


Figure C12.3-3 Alternate shear wall height-to-width ratio with design for force transfer around openings.

Single and double diagonally sheathed lumber shear walls. Diagonally sheathed lumber shear walls are addressed by the *Provisions* because they are still used for new construction in some regions. Resistance values are based on a soft conversion from the model code allowable stress loads and capacities to *Provisions* strength loads for regions with high spectral accelerations. This will allow users in the western states, where this construction is currently being used, to continue with little or no change in requirements; at the same time, reasonable values are provided for regions with lower spectral accelerations.

Perforated shear walls (PSW). In a traditional engineering approach for design of shear walls with openings, design force transfer around the openings involves developing a system of piers and coupling beams within the shear wall. Load paths for the shear and flexure developed in the piers and coupling beams generally require blocking and strapping extending from each corner of the opening to some distance beyond. This approach often results in shear wall detailing that is not practical to construct.

The perforated shear wall approach utilizes empirically based reductions of wood structural panel shear wall capacities to account for the presence of openings that have not been specifically designed and detailed for moment resistance. This method accounts for the capacity that is inherent in standard construction, rather than relying on special construction requirements. It is not expected that sheathed wall areas above and below openings behave as coupling beams acting end to end, but rather that they provide local restraint at their ends. As a consequence significantly reduced capacities are attributed to interior perforated shear wall segments with limited overturning restraint.

Example 1 and Example 2 provide guidance on the application of the perforated shear wall approach.

Perforated Shear Wall Limitations. Perforated shear wall design provisions are applicable to wood structural panel shear walls having characteristics identified in this section.

1. The requirement that perforated shear wall segments be provided at each end of the perforated shear wall ensures that a minimum length of full height sheathing, conforming to applicable aspect ratio limits, is included at each end of a perforated shear wall.
2. A factored shear resistance not to exceed 0.64 klf, based on tabulated LRFD values, is provided to identify a point beyond which other means of shear wall design are likely to be more practical. Connection requirements associated with unadjusted shear resistance greater than 0.64 klf will likely not be practical as other methods of shear wall design will be more efficient.
3. Each perforated shear wall segment must satisfy the requirements for shear wall aspect ratios. The $2b/h$ adjustment for calculation of unadjusted factored shear resistance only applies when shear wall segments with h/b greater than 2:1 but not exceeding 3.5:1 are used in calculating perforated shear wall resistance. When shear wall segments with h/b greater than 2:1 are present in a perforated shear wall, but not utilized in calculation of perforated shear wall resistance, calculation of unadjusted factored shear resistance should not include the $2b/h$ adjustment. In many cases, due to the conservatism of the $2b/h$ adjustment, it is advantageous to simply ignore the presence of shear wall segments with h/b greater than 2:1 when calculating perforated shear wall resistance.
4. No out-of-plane offsets are permitted in a perforated shear wall. While the limit on out-of-plane offsets is not unique to perforated shear walls, it is intended to clearly indicate that a perforated shear wall shall not have out-of-plane (horizontal) offsets.
5. Collectors for shear transfer to each perforated shear wall segment provide for continuity between perforated shear wall segments. This is typically achieved through continuity of the wall double top plates or by attachment of perforated shear wall segments to a common load distributing element such as a floor or roof diaphragm.
6. Uniform top-of-wall and bottom-of-wall elevations are required for use of the empirical shear adjustment factors.
7. Limiting perforated shear wall height to 20 ft addresses practical considerations for use of the method as wall heights greater than 20 ft are uncommon.

    a. The width, L, of a perforated shear wall and widths L_1, L_2 and L_3 of perforated shear wall segments are shown in Figure C12.3-4. In accordance with the limitations and anchorage requirements, perforated shear wall segments and overturning restraint must be provided at each end of the perforated shear wall.

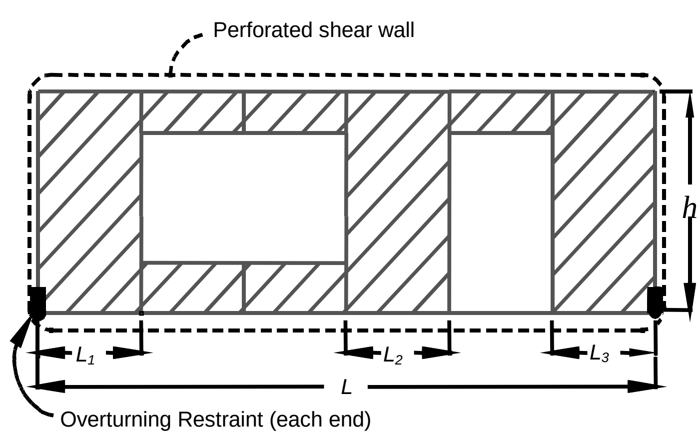


Figure C12.3-4 Perforated shear wall.

Perforated shear wall resistance. Opening adjustment factors are used to reduce shear wall resistance, based on the percent full-height sheathing and the maximum opening height ratio.

Opening adjustment factors are based on the following empirical equation for shear capacity ratio, *F*, which relates the ratio of the shear capacity for a wall with openings to the shear capacity of a fully sheathed wall (Sugiyama, 1981):

$$F = \frac{4}{3 - 2r} \qquad \text{(C12.3-1a)}$$

$$r = \frac{1}{1 + \dfrac{A_o}{h \sum L_i}} \qquad \text{(C12.3-1b)}$$

where:

r = sheathing area ratio,

A_o = total area of openings,

h = wall height,

ΣL_i = sum of the width of full-height sheathing.

Agreement between Eq. C12.3-1a and tabulated opening adjustment factors is achieved by recognizing that the tabulated opening adjustment factors are: (1) derived based on an assumption that the height of all openings in a wall are equal to the maximum opening height; and, (2) applied to the sum of the widths of the shear wall segments meeting applicable height-to-width ratios. The assumption that the height of all openings in a wall are equal to the maximum opening height conservatively simplifies tabular presentation of shear capacity adjustment factors for walls with more than one opening height.

Early verification of Eq. C12.3-1a was based on testing of one-third and full-scale shear wall assemblies (Yasumura, 1984; Sugiyama, 1994). More recently, substantial U.S. verification testing of the influence of openings on shear strength and stiffness has taken place (APA, 1996; Dolan and Johnson, 1996; Dolan and Heine, 1997; NAHB-RC, 1998) indicating shear wall performance is consistent with predictions of Eq. C12.3-1a. Results of cyclic testing indicate that the loss in strength due to cyclic loading is reduced for shear walls with openings, indicating good performance relative to that of shear walls without openings. Figure C12.3-5 provides a graphical summary of some recent U.S. verification testing. Data from monotonic tests of 12-ft shear walls (APA, 1996), monotonic and cyclic tests of long shear walls with unsymmetrically placed openings (Dolan and Johnson, 1996), and monotonic and cyclic tests of 16-ft and 20-ft shear walls with narrow wall segments (NAHB-RC, 1998).

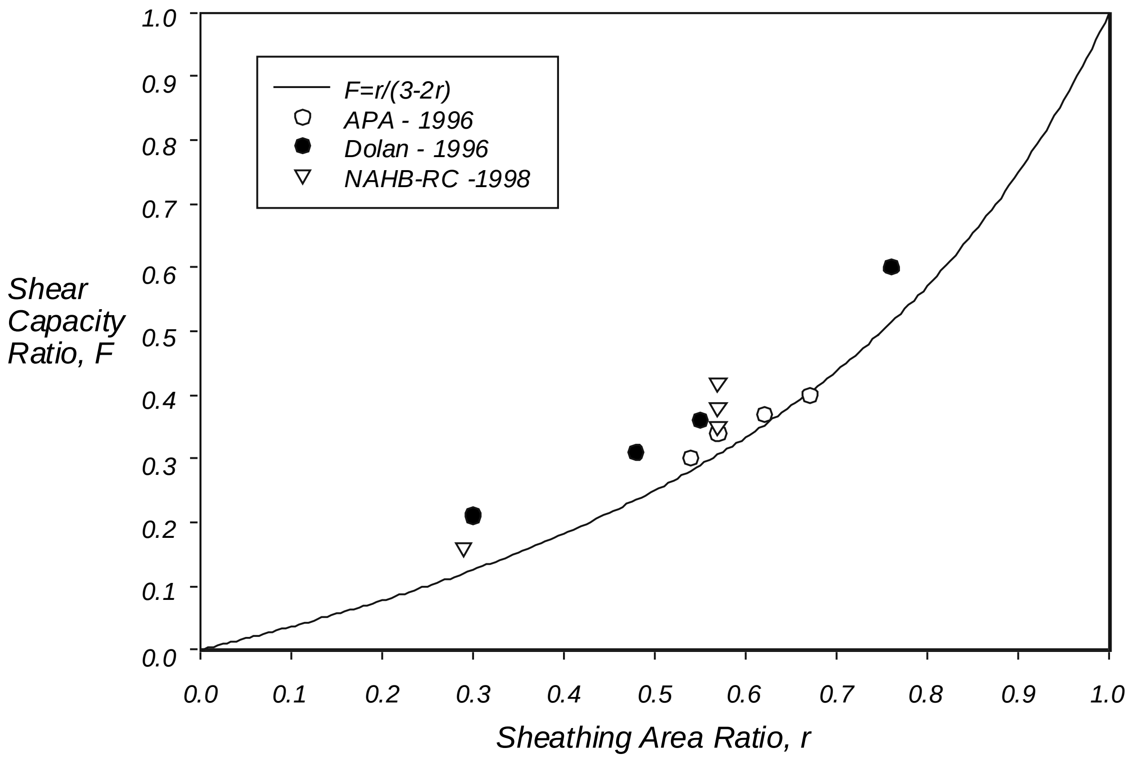


Figure C12.3-5 Shear capacity ratios (actual and predicted).

Eq. C12.3-1a for shear load ratio, F, has been shown to be a good approximation of the stiffness ratio of a wall with openings to that of a fully sheathed wall. Accordingly, the deflection of a perforated shear wall can be calculated as the deflection of an equivalent length fully sheathed wall, divided by the shear load ratio, F.

The percent full-height sheathing and the maximum opening height ratio are used to determine an opening adjustment factor. Maximum opening height is the maximum vertical dimension of an opening within the perforated shear wall. A maximum opening height equal to the wall height is used where structural sheathing is not present above or below window openings or above door openings. The percent full-height sheathing is calculated as the sum of the widths of perforated shear wall segments divided by the total width of the shear wall. Sections sheathed full-height which do not meet aspect ratio limits for wood structural panel shear walls are not considered in calculation of percent full-height sheathing.

PSW Anchorage and load path. Anchorage for uplift at perforated shear wall ends, shear, uplift between perforated shear wall ends, and compression chord forces are prescribed to address the non-uniform distribution of shear within a perforated shear wall.

Prescribed forces for shear and uplift connections ensure that the capacity of the wall is governed by the sheathing to framing attachment (shear wall nailing) and not bottom plate attachment for shear and/or uplift. Shear and uplift forces approach the unadjusted factored shear resistance of the perforated shear wall segment as the shear load approaches the shear resistance of the perforated shear wall. A continuous load path to the foundation based on this requirement and consideration of other forces (for example, from the story above) shall be maintained. The magnitude of shear and uplift varies as a function of overturning restraint provided and aspect ratio of the shear wall segment.

Uplift anchorage at perforated shear wall ends. Anchorage for uplift forces due to overturning are required at each end of the perforated shear wall. A continuous load path to the foundation based on this requirement and consideration of other forces (for example, from the story above) shall be maintained. In addition, compression chords of perforated shear wall segments are required to transmit compression forces equal to the required tension chord uplift force.

PSW Anchorage for in-plane shear. It is required that fastening be provided along the length of the sill plate of wall sections sheathed full-height to resist distributed shear, *v*, and uplift, *t*, forces. The resistance required for the shear connection is the average shear over the perforated shear wall segments, divided by the adjustment factor. This resistance will approach the unadjusted factored shear resistance of the wall as the shear wall demand approaches the maximum resistance. This shear fastening resistance conservatively accounts for the non-uniform distribution of shear within a perforated shear wall, since it represents the shear that can only be achieved when full overturning restraint is provided.

Provisions require that distributed fastening for shear, *v*, and uplift, *t*, be provided over the length of full-height sheathed wall sections. With no other specific requirements, the fastening between the full height segments will be controlled by minimum construction fastening requirements. For bottom plates on wood platforms this would only require one 16-penny nail at 16 in. on center. In some cases, it may be preferable to extend a single bottom plate fastening schedule across the entire length of the perforated shear wall rather than to require multiple fastening schedules.

Uplift anchorage between perforated shear wall ends. The resistance required for distributed uplift anchorage, *t*, is the same as the required shear resistance, *v*. The adequacy of the distributed uplift anchorage can be demonstrated using principles of mechanics and recent testing that determined the capacity of shear wall segments without uplift anchorage. A 4-ft wide shear wall segment with distributed anchorage of the base plate in lieu of an uplift anchor device provided about 25 percent of the resistance of a segment with uplift anchorage; an 8-ft wide shear wall segment resisted about 45 percent. When these are combined with the resistance adjustment factors, overturning resistance based on the unadjusted factored shear resistance is adequate for perforated shear wall segments with full height openings on each side. Conceptually the required distributed uplift resistance is intended to provide the same resistance that anchor bolts spaced at 2 ft on center provided for tested assemblies. While in the tested assemblies the bottom plates were fastened down, for design it is equally acceptable to fasten down the studs with a strap or similar device, since the studs will in turn restrain the bottom plate.

PSW Load path. A continuous load path to the foundation is required for the uplift resistance, *T*; the compression resistance, *C*; the unit shear resistance, *v*; and the unit uplift resistance, *t*. Consideration of accumulated forces (for example, from the stories above) is required. Where shear walls occur at the same location at each floor (stack), accumulation of forces is reasonably straightforward. Where shear walls do not stack, attention will need to be paid to maintaining a load path for tie-downs at each end of the perforated shear wall, for compression resistance at each end of each perforated shear wall segment, and for distributed forces *v* and *t* at each perforated shear wall segment. Where ends of shear perforated shear wall segments occur over beams or headers, the beam or header will need to be checked for the vertical tension and compression forces in addition to gravity forces. Where adequate collectors are provided at lower floor shear walls, the total shear wall load need only consider the average shear in the perforated shear wall segments above, and not the average shear divided by the adjustment factor.

Example 1 Perforated Shear Wall

Problem Description: The perforated shear wall illustrated in Figure C12.4-4 is sheathed with 15/32" wood structural panel with 10d common nails with 4 in. perimeter spacing. All full-height sheathed sections are 4 ft wide. The window opening is 4 ft high by 8 ft wide. The door opening is 6.67 ft high by 4 ft wide. Sheathing is provided above and below the window and above the door. The wall length and height are 24 ft and 8 ft, respectively. Tie-downs provide overturning restraint at the ends of the perforated shear wall and anchor bolts are used to restrain the wall against shear and uplift between perforated shear wall ends. Determine the shear resistance adjustment factor for this wall.

Solution: The wall defined in the problem description meets the application criteria outlined for the perforated shear wall design method. Tie-downs provide overturning restraint at perforated shear wall ends, and anchor bolts provide shear and uplift resistance between perforated shear wall ends. Perforated shear wall height, factored shear resistances for the wood structural panel shear wall, and aspect ratio of full height sheathing at perforated shear wall ends meet requirements of the perforated shear wall method.

The process of determining the shear resistance adjustment factor involves determining percent full-height sheathing and maximum opening height ratio. Once these are known, a shear resistance adjustment factor can be determined from tabulated reduction factors.

From the problem description and Figure C12.3.-4:

Percent full-height sheathing

$$= \frac{\text{Sum of perforated shear wall segment widths, } \Sigma L}{\text{Length of perforated shear wall, } L}$$

$$= \frac{4\text{ ft} + 4\text{ ft} + 4\text{ ft}}{24\text{ ft}} \times 100 = 50\%$$

Maximum opening height ratio

$$= \frac{\text{Maximum opening height}}{\text{Wall height, } h}$$

$$= \frac{6.67\text{ ft}}{8\text{ ft}} = \frac{5}{6}$$

For a maximum opening height ratio of 5/6 (or maximum opening height of 6.67 ft when wall height, *h*, equals 8 ft) and percent full-height sheathing equal to 50 percent, a shear resistance adjustment factor of $C_O = 0.57$ is obtained.

Note that if wood structural panel sheathing were not provided above and below the window or above the door the maximum opening height would equal the wall height, *h*.

Example 2 Perforated Shear Wall

Problem description. Figure C12.4-6 illustrates one face of a 2-story building with the first and second floor walls designed as perforated shear walls. Window heights are 4 ft and door height is 6.67 ft. A trial design is performed in this example based on applied loads, V. For simplification, dead load contribution to overturning and uplift restraint is ignored and the effective width for shear in each perforated shear wall segment is assumed to be the sheathed width. Framing is Douglas fir. After basic perforated shear wall resistance and force requirements are calculated, detailing options to provide for adequate unit shear, v, and unit uplift, t, transfer between perforated shear wall ends are covered. Figure C12.3-7 illustrates possible methods for achieving the required unit shear and uplift transfer. Configuration A considers the condition where a continuous rim joist is present at the second floor. Configuration B considers the case where a continuous rim joist is not provided, as when floor framing runs perpendicular to the perforated shear wall with blocking between floor framing members.

Perforated shear wall resistance and force requirements:

Second floor wall. Determine wood structural panel sheathing thickness and fastener schedule needed to resist applied load, $V = 2.25$ kips, from the roof diaphragm such that the shear resistance of the perforated shear wall is greater than the applied force. Also determine anchorage and load path requirements for uplift force at ends, in plane shear, uplift between wall ends, and compression.

$$\text{Percent full-height sheathing} = \frac{4\text{ ft} + 4\text{ ft}}{16\text{ ft}} \times 100 = 50\%$$

$$\text{Maximum opening height ratio} = \frac{4\text{ ft}}{8\text{ ft}} = \frac{1}{2}$$

Shear resistance adjustment factor, $C_0 = 0.80$

Try 15/32 in. rated sheathing with 8d common nails (0.131 by 2-1/2 in.) at 6 in. perimeter spacing.

Unadjusted shear resistance (LRFD) = 0.36 klf

Adjusted shear resistance

= (unadjusted shear resistance)(C_0)

= (0.36 klf)(0.80) = 0.288 klf

Perforated shear wall resistance

= (Adjusted Shear Resistance)(ΣL_i)

= (0.288 klf)(4 ft + 4 ft) = 2.304 kips

2.304 kips > 2.25 kips ✔ OK

Required resistance due to story shear forces, V:

Overturning at shear wall ends, T:

$$T = \frac{Vh}{C_0 \Sigma L_i} = \frac{2.250\text{ kips }(8\text{ ft})}{0.80\ (4\text{ ft} + 4\text{ ft})} = 2.813\text{ kips}$$

In-plane shear, v:

$$v = \frac{V}{C_0 \Sigma L_i} = \frac{2.250\text{ kips}}{0.80\ (4\text{ ft} + 4\text{ ft})} = 0.352\text{ klf}$$

Uplift, t, between wall ends: $t = v = 0.352$ klf

Compression chord force, C, at each end of each perforated shear wall segment:

$C = T = 2.813$ kips

First floor wall. Determine wood structural panel sheathing thickness and fastener schedule needed to resist applied load, $V = 2.60$ kips, at the second floor diaphragm such that the shear resistance of the perforated shear wall is greater than the applied force. Also determine anchorage and load path requirements for uplift force at ends, in-plane shear, uplift between wall ends, and compression.

$$\text{Percent full-height sheathing} = \frac{4\text{ ft} + 4\text{ ft}}{12\text{ ft}} \times 100 = 67\%$$

Shear resistance adjustment factor, C_0=0.67

Unadjusted shear resistance (LRFD) = 0.49 klf

Adjusted shear resistance

= (Unadjusted Shear Resistance)(C_o)

= (0.49 klf)(0.67) = 0.328 klf

Perforated shear wall resistance

= (Adjusted Shear Resistance)(ΣL_i)

= (0.328 klf)(4 ft + 4 ft) = 2.626 kips

2.626 kips > 2.600 kips ✔ OK

Required resistance due to story shear forces, V:

Overturning at shear wall ends, T:

$$T = \frac{Vh}{C_0 \Sigma L_i} = \frac{2.600 \text{ kips (8 ft)}}{0.67 \text{ (4 ft + 4 ft)}} = 3.880 \text{ kips}$$

When maintaining load path from story above,

T = T from second floor + T from first floor

= 2.813 kips + 3.880 kips = 6.693 kips

In-plane shear, v:

$$v = \frac{V}{C_0 \Sigma L_i} = \frac{2.600 \text{ kips}}{0.67 \text{ (4 ft + 4 ft)}} = 0.485 \text{ klf}$$

Uplift, t, between wall ends:

$t = v = 0.485$ klf

Uplift, t, can be cumulative with 0.352 klf from story above to maintain load path. Whether this occurs depends on detailing for transfer of uplift forces between end walls.

Compression chord force, C, at each end of each perforated shear wall segment:

$C = T = 3.880$ kips

When maintaining load path from story above, C = 3.880 kips + 2.813 kips = 6.693 kips.

Tie-downs and posts and the ends of perforated shear wall are sized using calculated force, T. The compressive force, C, is used to size compression chords as columns and ensure adequate bearing.

Configuration A – Continuous Rim Joist (see Figure C12.3-7)

Second floor. Determine fastener schedule for shear and uplift attachment between perforated shear wall ends. Recall that $v = t = 0.352$ klf.

Wall bottom plate (1 ½ in. thickness) to rim joist. Use 20d box nail (0.148 by 4 in.). Lateral resistance $\phi\lambda Z'$ = 0.254 kips per nail and withdrawal resistance $\phi\lambda W'$ = 0.155 kips per nail.

Nails for shear transfer

= (shear force, v)/$\phi\lambda Z'$

= 0.352 klf/0.254 kips per nail

= 1.39 nails per foot

Nails for uplift transfer

= (uplift force, t)/$\phi\lambda W'$

= 0.352 klf/0.155 kips per nail

= 2.27 nails per foot

Net spacing for shear and uplift

= 3.3 inches on center

Rim joist to wall top plate. Use 8d box nails (0.113 by 2-1/2 in.) toe-nailed to provide shear transfer. Lateral resistance $\phi\lambda Z'$ = 0.129 kips per nail.

Nails for shear transfer

= (shear force, v)/$\phi\lambda Z'$

= 0.352 klf/0.129 kips per nail

= 2.73 nails per foot

Net spacing for shear

= 4.4 inches on center

See detail in Figure C12.3-7 for alternate means a shear transfer (such as a metal angle or plate connector).

Transfer of uplift, t, from second floor in this example is accomplished through attachment of second floor wall to the continuous rim joist which has been designed to provide sufficient strength to resist the induced moments and shears. Continuity of load path is provided by tie-downs at the ends of the perforated shear wall.

First floor. Determine anchorage for shear and uplift attachment between perforated shear wall ends. Recall that $v = t = 0.485$ klf.

Wall bottom plate (1 ½ in. thickness) to concrete. Use ½ in. anchor bolt with lateral resistance $\varphi\lambda Z'$ = 1.34 kips.

Bolts for shear transfer

= (shear force, v)/$\phi\lambda Z'$

= 0.485 klf/1.34 kips per bolt

= 0.36 bolts per ft

Net spacing for shear

= 33 in. on center

Bolts for uplift transfer. Check axial capacity of bolts for $t = v = 0.485$ klf and size plate washers accordingly. No interaction between axial and lateral load on anchor bolt is assumed (that is, the presence of axial tension is assumed not to affect lateral strength).

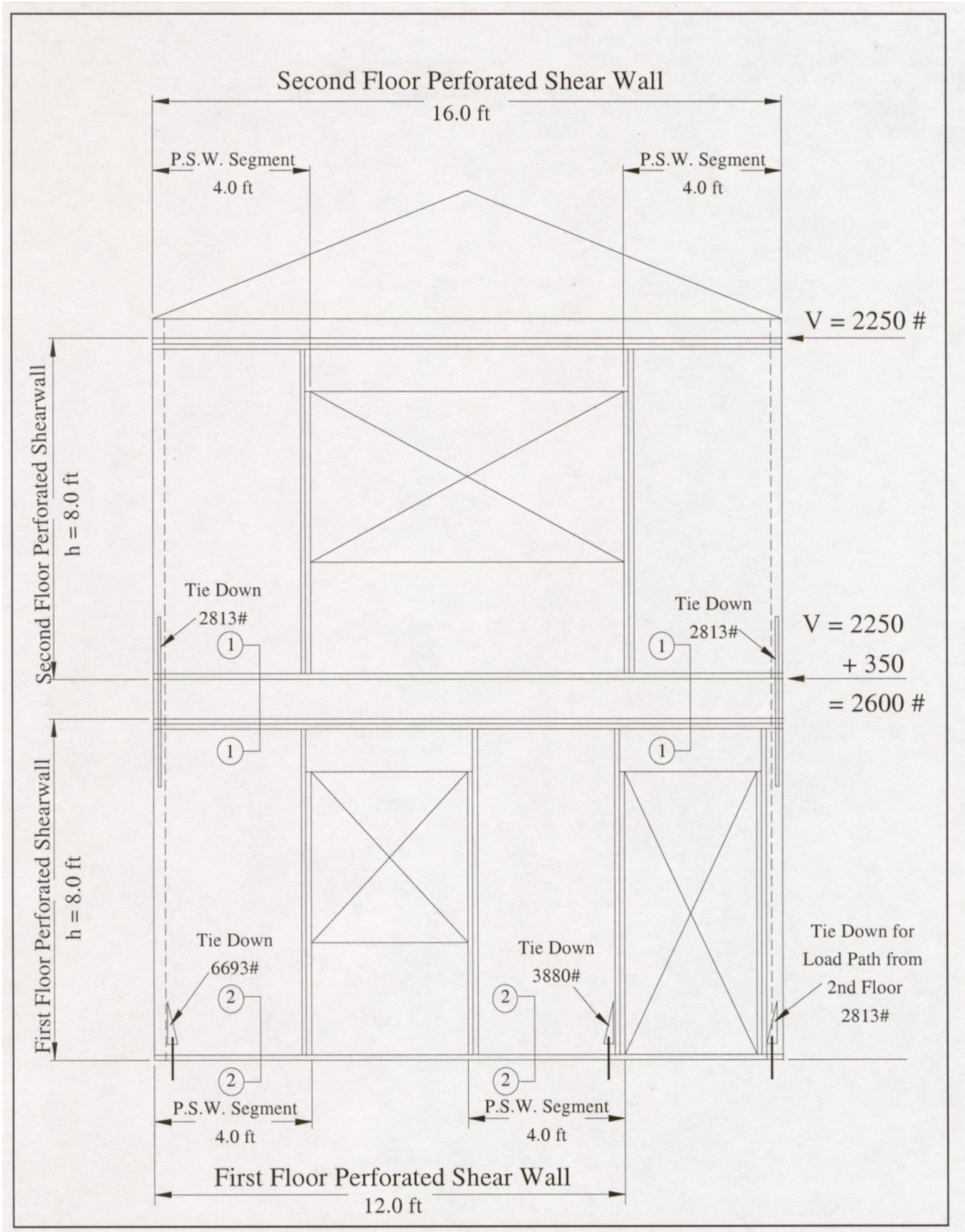


Figure C12.3-6 Elevation for perforated shear wall Example 2.

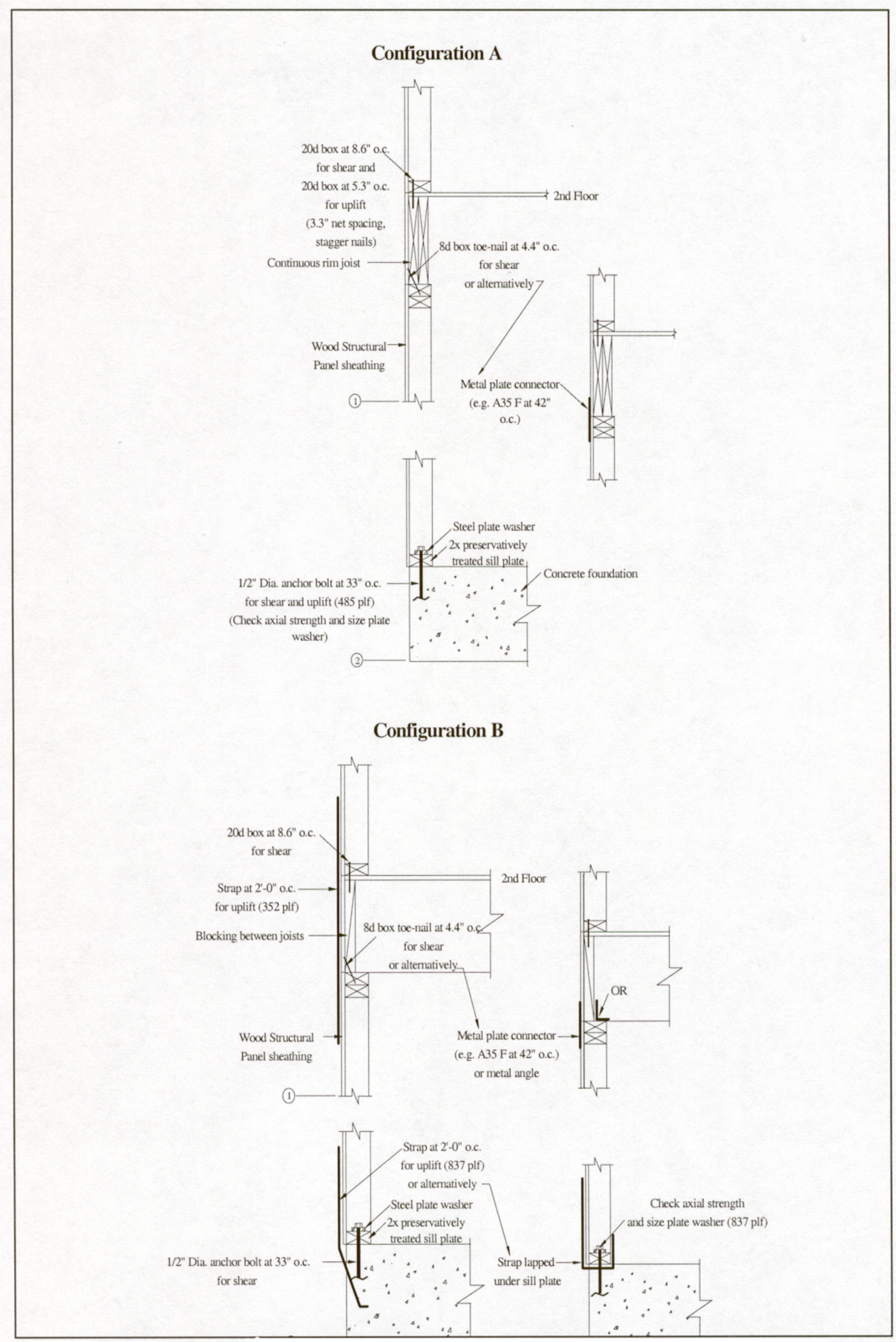


Figure C12.3-7 Details for perforated shear wall Example 2.

12.3.1 Framing. All framing that is designed as part of an engineered wood structure must be designed with connectors that are able to transfer the required forces between various components. These connectors can be either proprietary hardware or some of the more conventional connections used in wood construction. However, these connectors should be designed according to accepted engineering practice to ensure that they will have the capacity to resist the forces. The requirement of columns and posts being framed to full end bearing requires that the force transfer from the column to the base be accomplished through end grain bearing of the wood, not through placing the bolts or other connectors in shear. This requirement is included to ensure adequate capacity for transfer of the vertical forces due to both gravity and overturning moment. Alternatively, the connection can be designed to transfer the full loading through placing the bolts or other connectors in shear neglecting all possible bearing.

The anchorage connections used in engineered wood construction must be capable of resisting the forces that will occur between adjacent members (beams and columns) and elements (diaphragms and shear walls). These connections can utilize proprietary hardware or be designed in accordance with principles of mechanics. Inadequate connections are often the cause of structural failures in wood structures, and the registered design professional is cautioned to use conservative values for allowable capacities since most published values are based on monotonic, not cyclic, load applications (U.S. Department of Agriculture, National Oceanic and Atmospheric Administration, 1971). Testing has shown that some one-sided bolted connections subject to cyclic loading, such as tie-down devices, do not perform well. This was substantiated by the poor performance of various wood frame elements in structures in the January 1994 Northridge earthquake.

Concrete or masonry wall anchorages using toe nails or nails subject to withdrawal are prohibited by the *Provisions*. It has been shown that these types of connections are inadequate and do not perform well (U.S. Department of Agriculture, National Oceanic and Atmospheric Administration, 1971). Ledgers subjected to cross-grain bending or tension perpendicular to grain also have performed poorly in past earthquakes, and their use is now prohibited by the *Provisions*.

12.4 CONVENTIONAL LIGHT-FRAME CONSTRUCTION

The *Provisions* intend that a structure using conventional construction methods and complying with the requirements of this section be deemed capable of resisting the seismic forces imposed by the *Provisions*.

Repetitive framing members such as joists, rafters, and studs together with sheathing and finishes comprise conventional light-frame construction. The subject of conventional construction is addressed in each of the model codes. It is acknowledged and accepted that, for the most part, the conventional construction provisions in the model codes concerning framing members and sheathing that carry gravity loads are adequate. This is due to the fact that the tables in the model codes giving allowable spans have been developed using basic principles of mechanics. For seismic lateral force resistance, however, experience has shown that additional requirements are needed.

To provide lateral force resistance in vertical elements of structures, wall bracing requirements have been incorporated in conventional construction provisions of the model codes. With a few exceptions, these generally have been adequate for single family residences for which conventional construction requirements were originally developed. While the model building codes have been quite specific as to the type of bracing materials to be used and the amount of bracing required in any wall, no limits on the number or maximum separation between braced walls have been established. This section of the *Provisions* introduces the concept of mandating the maximum spacing of braced wall lines. By mandating the maximum spacing of braced wall lines and thereby limiting the lateral forces acting on these vertical elements, these revisions provide for a seismic-force-resisting system that will be less prone to overstressing and the requirements can be applied and enforced more uniformly than previous model building code requirements. While specific elements of light-frame construction may be calculated to be overstressed, there is typically a great deal of redundancy and uncounted resistance in such structures and they have generally performed well in past earthquakes. The experience in the

Northridge earthquake was, however, less reassuring, especially for those residences relying on gypsum board or stucco for lateral force resistance. The light weight of conventional construction, together with the large energy dissipation capacity of the multiple fasteners used and inherent redundancy of the system are major factors in the observed good performance where wood or wood-based panels were used.

12.4.1 Limitations

12.4.1.1 General. The scope of this section specifically excludes prescriptive design of structures with concrete or masonry walls above the basement story, with the exception of veneer, in order to maintain the light weight of construction that the bracing requirements are based on. Wood braced wall panels and diaphragms as prescribed in this section are not intended to support lateral forces due to masonry or concrete construction. Prescriptive (empirical) design of masonry walls is allowed for in Chapter 11; however, design of structures combining masonry wall construction and wood roof and floor diaphragm construction must have an engineered design. In regions of high seismic activity, past earthquakes have demonstrated significant problems with structures combining masonry and wood construction. While engineered design requirements do address these problems, the prescriptive requirements in the model codes do not adequately address these problems. Masonry and concrete basement walls are permitted to be constructed in accordance with the requirements of the IRC.

12.4.1.2 Irregular structures. This section was added to the 1997 *Provisions* to clarify the definition of irregular (unusually shaped) structures that would require the structure to be designed for the forces prescribed in Chapter 5 in accordance with the requirements of Sec. 12.3 and 12.4. The descriptions and diagrams provide the registered design professional with several typical irregularities that produce torsional response, or result in forces considered high enough to require an engineered design and apply only to structures assigned to Seismic Design Category C or D.

Structures with geometric discontinuities in the lateral-force-resisting system have been observed to sustain more earthquake and wind damage than structures without discontinuities. They have also been observed to concentrate damage at the discontinuity location. For Seismic Design Categories C and D, this section translates applicable irregularities from Tables 4.3-2 and 4.3-3 into limitations on conventional light-frame construction. If the described irregularities apply to a given structure, it is required that either the entire structure or the non-conventional portions be engineered in accordance with the engineered design portions of the *Provisions*. The irregularities are based on similar model code requirements. While conceptually these are equally applicable to all seismic design categories, they are more readily accepted in areas of high seismic risk, where damage due to irregularities has been observed repeatedly.

Application of engineered design to non-conventional portions rather than to the entire structure is a common practice in some regions. The registered design professional is left to judge the extent of the portion to be designed. This often involves design of the nonconforming element, force transfer into the element, and a load path from the element to the foundation. A nonconforming portion will sometimes have enough of an impact on the behavior of a structure to warrant that the entire seismic-force-resisting system receives an engineered design.

12.4.1.2.1 Out-of-plane offset. This limitation is based on Item 4 of Table 4.3-2 and applies when braced wall panels are offset out-of-plane from floor to floor. In-plane offsets are discussed in another item. Ideally braced wall panels would always stack above of each other from floor to floor with the length stepping down at upper floors as less length of bracing is required.

Because cantilevers and set backs are very often incorporated into residential construction, the exception offers rules by which limited cantilevers and setbacks can be considered conventional. Floor joists are limited to 2 by 10 (actual: 12 by 93 in.; 38 by 235 mm) or larger and doubled at braced wall panel ends in order to accommodate the vertical overturning reactions at the end of braced wall panels. In addition the ends of cantilevers are attached to a common rim joist to allow for redistribution of load. For rim joists that cannot run the entire length of the cantilever, the metal tie is intended to transfer vertical shear as well as to provide a nominal tension tie. Limitations are placed on gravity loads to be carried by cantilever or setback floor joists so that the joist strength will not be exceeded. The roof loads discussed are based on the use of solid sawn members where allowable spans limit the possible loads. Where engineered framing members such as trusses are used, gravity load capacity of the cantilevered or setback floor joists should be carefully evaluated.

12.4.1.2.2 Unsupported diaphragm. This limitation is based in Item 1 of Table 4.3-2, and applies to open-front structures or portions of structures. The conventional construction bracing concept is based on using braced wall lines to divide a structure up into a series of boxes of limited dimension, with the seismic force to each box being limited by the size. The intent is that each box be supported by braced wall lines on all four sides, limiting the amount of torsion that can occur. The exception, which permits portions of roofs or floors to extend past the braced wall line, is intended to permit construction such as porch roofs and bay windows. Walls for which lateral resistance is neglected are allowed in areas where braced walls are not provided.

12.4.1.2.3 Opening in wall below. This limitation is based on Item 4 of Table 4.3-3 and applies when braced wall panels are offset in-plane. Ends of braced wall panels supported on window or door headers can be calculated to transfer large vertical reactions to headers that may not be of adequate size to resist these reactions. The exception permits a 1 ft extension of the braced wall panel over a 4 by 12 (actual: 32 by 113 in.; 89 by 286 mm) header on the basis that the vertical reaction is within a 45 degree line of the header support and therefore will not result in critical shear or flexure. All other header conditions require an engineered design. Walls for which lateral resistance is neglected are allowed in areas where braced walls are not provided.

12.4.1.2.4 Vertical offset in diaphragm. This limitation results from observation of damage that is somewhat unique to split-level wood frame construction. If floors on either side of an offset move in opposite directions due to earthquake or wind loading, the short bearing wall in the middle becomes unstable and vertical support for the upper joists can be lost, resulting in a collapse. If the vertical offset is limited to a dimension equal to or less than the joist depth, then a simple strap tie directly connecting joists on different levels can be provided, eliminating the irregularity. The IRC, Sec. 502.6.1, provides requirements for tying of floor joists.

12.4.1.2.5 Non-perpendicular walls. This limitation is based on Item 5 of Table 4.3-2 and applies to nonperpendicular braced wall lines. When braced wall lines are not perpendicular to each other, further evaluation is needed to determine force distributions and required bracing.

12.4.1.2.6 Large diaphragm opening. This limitation is based on Item 3 of Table 4.3-2 and attempts to place a practical limit on openings in floors and roofs. Because stair openings are essential toresidential construction and have long been used without any report of life-safety hazards resulting, these are felt to be acceptable conventional construction. See Sec. 12.4.3.7 for detailing requirements for permitted openings.

12.4.1.2.7 Stepped foundation. This limits a condition that can cause a torsional irregularity per Item 1 of Table 4.3-2. Where heights of braced wall panels vary significantly, distribution of lateral forces will also vary. If a structure on a hill is supported on 2-ft-high, braced cripple wall panels on one side and 8-ft-high panels on the other, torsion and redistribution of forces will occur. An engineered design for this situation is required in order to evaluate force distribution and provide adequate wall bracing and anchor bolting. This limitation applies specifically to walls from the foundation to the floor. While gable-end walls have similar variations in wall heights, this has not been observed to be a significant concern in conventional construction. See Sec. 12.4.3.6 for detailing requirements for permitted foundation stepping.

12.4.2 Braced walls

12.4.2.1 Spacing between braced wall lines. Table 12.4-1 prescribes the spacing of braced wall lines and number of stories permitted for conventional construction structures. Figures C12.4-1 and C12.4-

2 illustrate the basic components of the lateral bracing system. Information in Tables 12.4-1 and 12.4-2 was first included in the 1991 *Provisions*.

12.4.2.2 Braced wall line sheathing. Table 12.4-2 prescribes the minimum length of bracing along each 25 ft (7.6 m) length of braced wall line. Total height of structures has been reduced to limit overturning of the braced walls so that significant uplift is not generally encountered. The height limit will accommodate 8 to 10 ft (2.4 to 3 m) story heights.

12.4.3 Detailing requirements. The intent of this section is to rely on the traditional light-frame conventional construction materials and fastenings as prescribed in the references for this chapter. Braced wall panels are not required to be aligned vertically or horizontally (within the limits prescribed in Sec. 12.4.1) but stacking is desirable where possible. With the freedom provided for non-alignment it becomes important that a load path be provided to transfer lateral forces from upper levels through intermediate vertical and horizontal resisting elements to the foundation. Connections between horizontal and vertical resisting elements are prescribed. In structures two or three stories in height, it is desirable to have interior braced wall panels supported on a continuous foundation. See Figures C12.4-3 through C12.4-13 for examples of connections.

The 1997 *Provisions* incorporated some of the wall anchorage, top plate, and braced wall panel connection requirements from the model building codes. These are included for completeness of the document and to clarify the requirement for the registered design professional. Additional requirements for foundations supporting braced wall panels has also been added to provide guidance and clarity for the registered design professional.

SEISMIC PERFORMANCE CATEGORY	MAXIMUM WALL SPACING (FEET) *
C,D AND E	25
B	35
A	NOT REQUIRED

* REFER TO TABLE 12.4-2 FOR MINIMUM LENGTH OF WALL BRACING

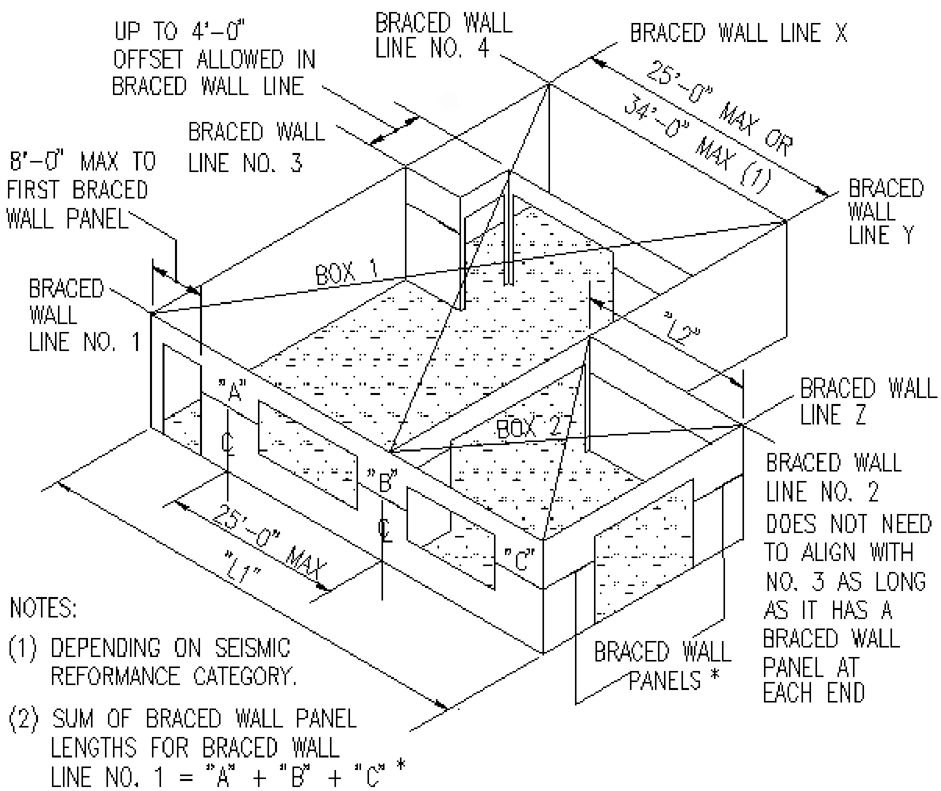


Figure C12.4-1 Acceptable one-story bracing example.

SEISMIC PERFORMANCE CATEGORY	MAXIMUM WALL SPACING (FEET) *
C AND D	25
B	35
A	NOT REQUIRED

*REFER TO TABLE 12.4-2 FOR MINIMUM LENGTH OF WALL BRACING

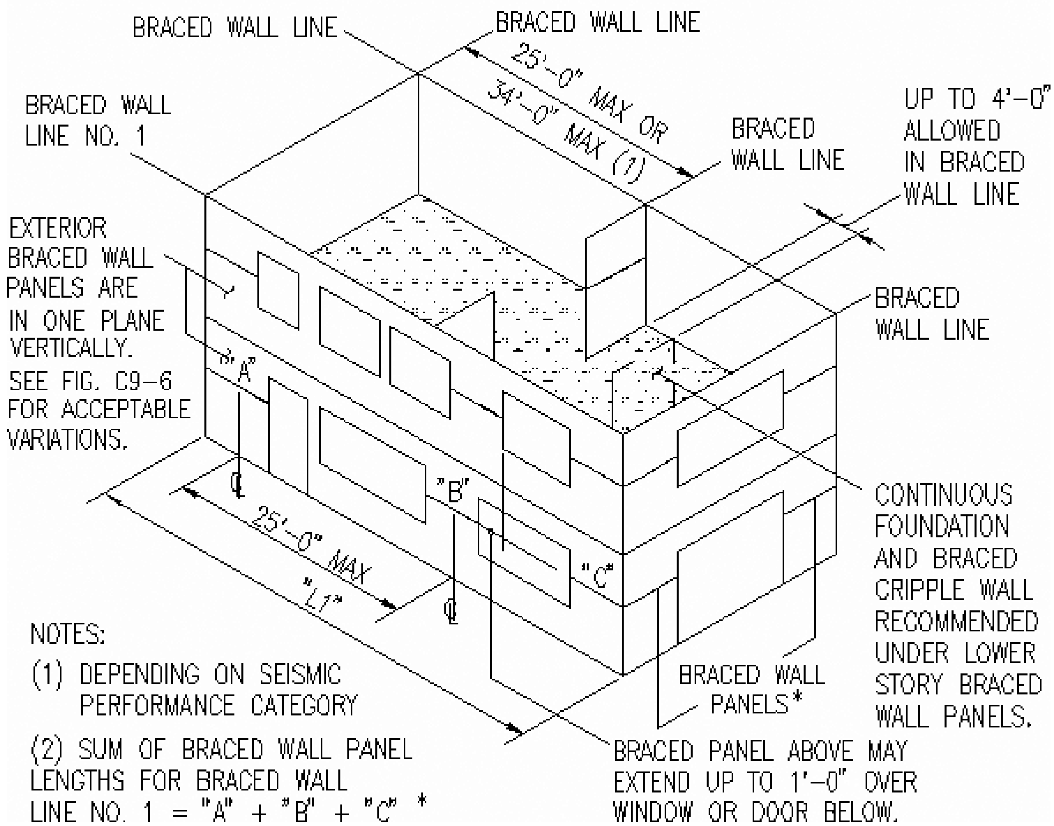
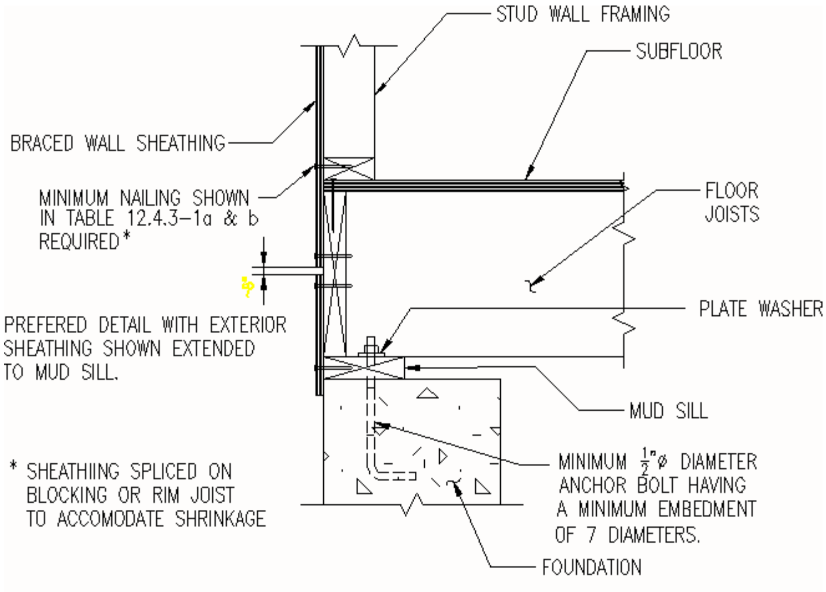


Figure C12.4-2 Acceptable two-story bracing example.



NUMBER OF STORIES	SPACING OF ANCHOR BOLTS (FEET)
ONE OR TWO	6
OVER TWO	4

Figure C12.4-3 Wall anchor detail.

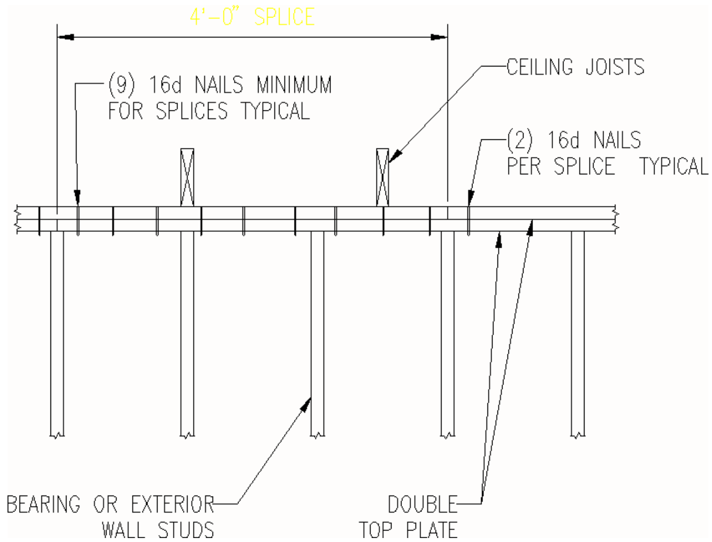


Figure C12.4-4 Double top splice.

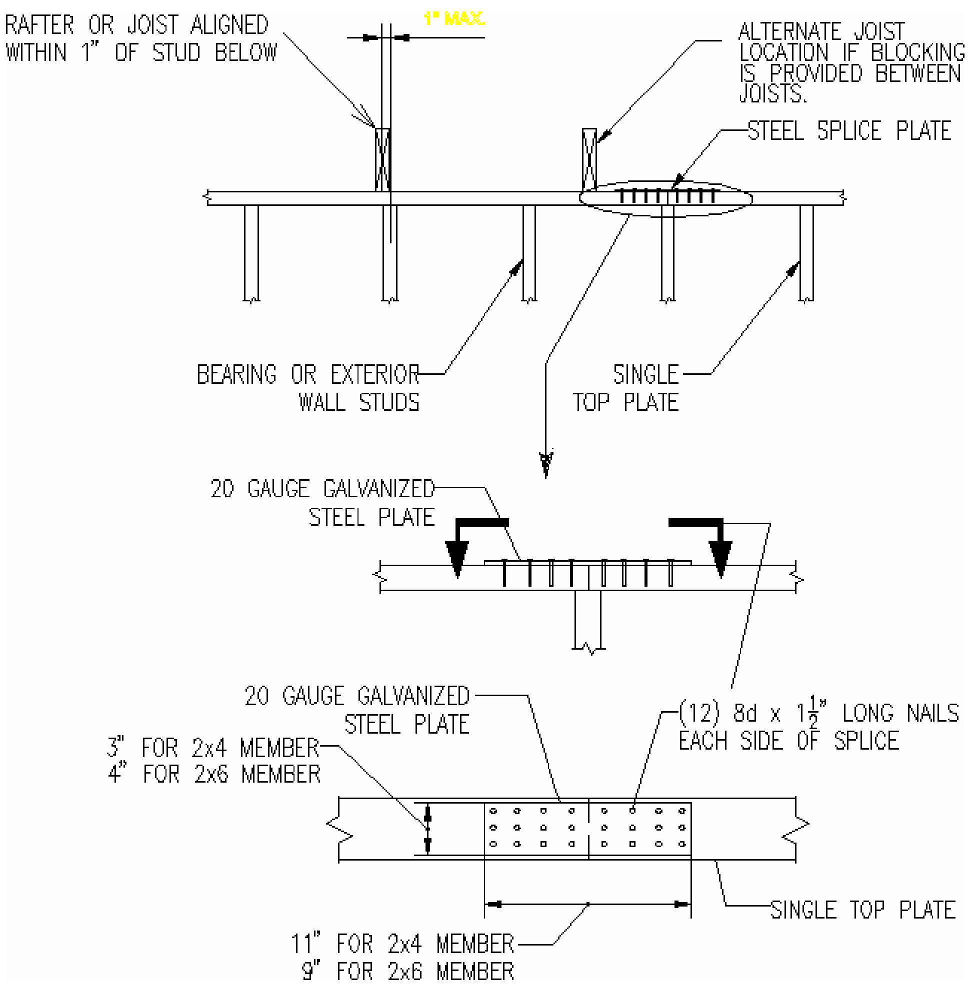


Figure C12.4-5 Single top splice.

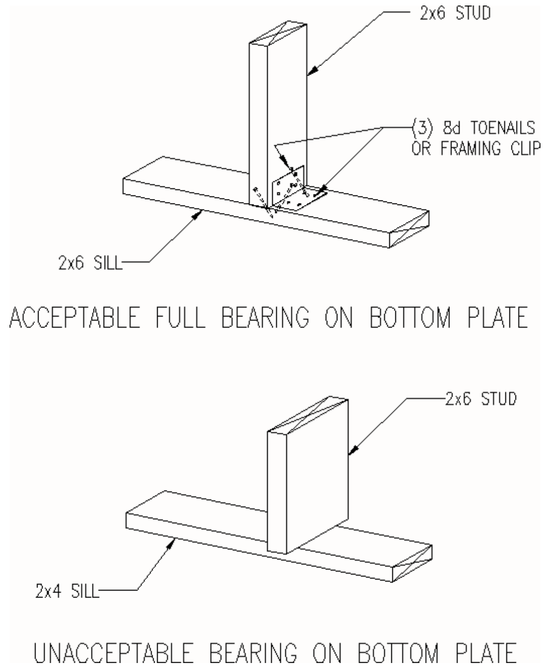


Figure C12.4-6 Full bearing bottom plate.

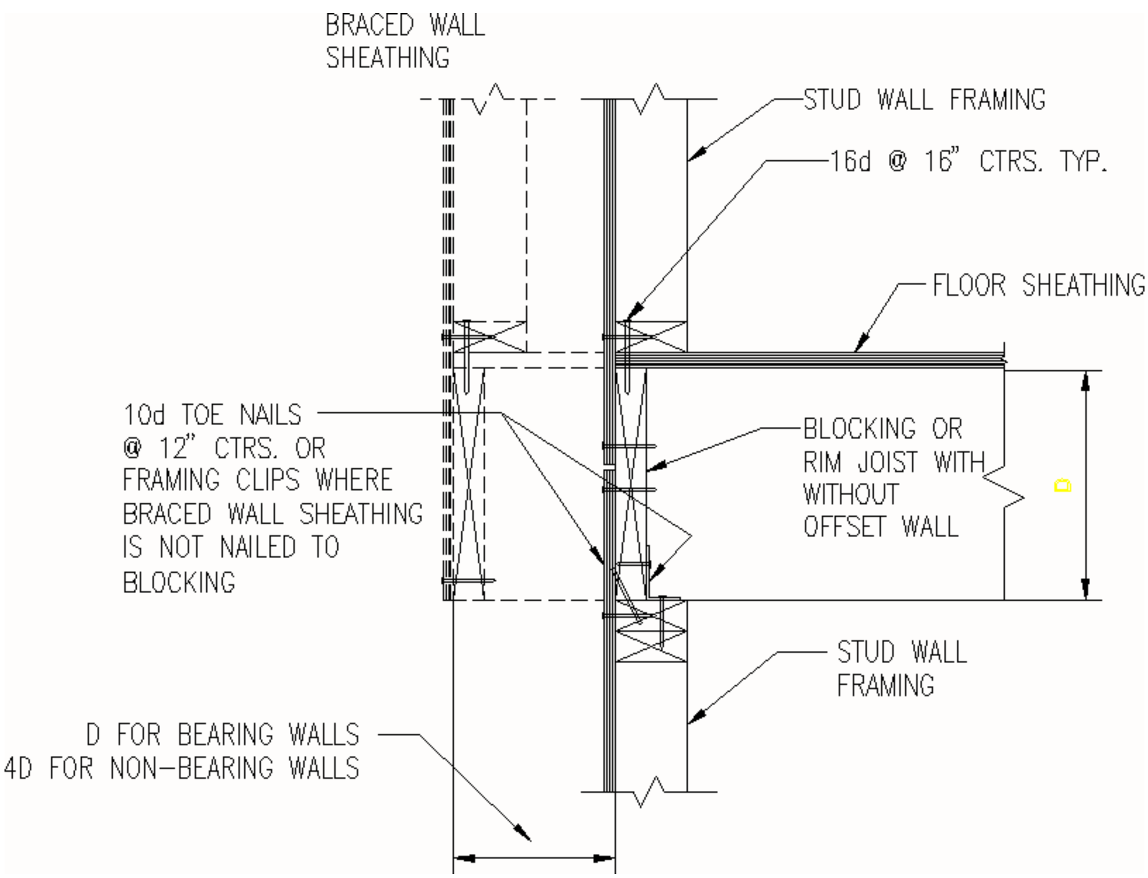


Figure C12.4-7 Exterior braced wall.

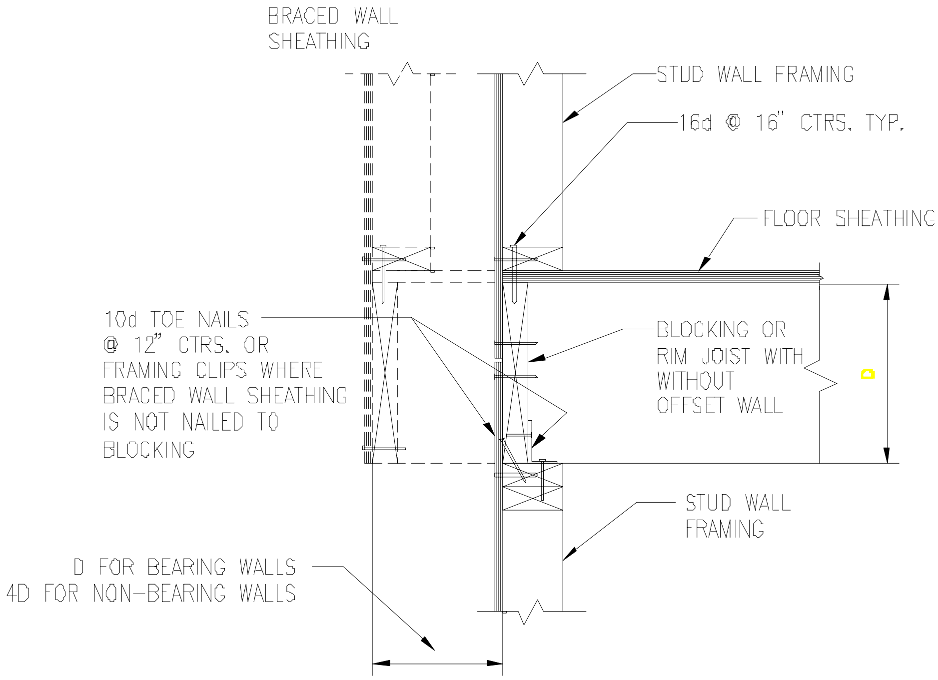


Figure C12.4-8 Interior braced wall at perpendicular joist.

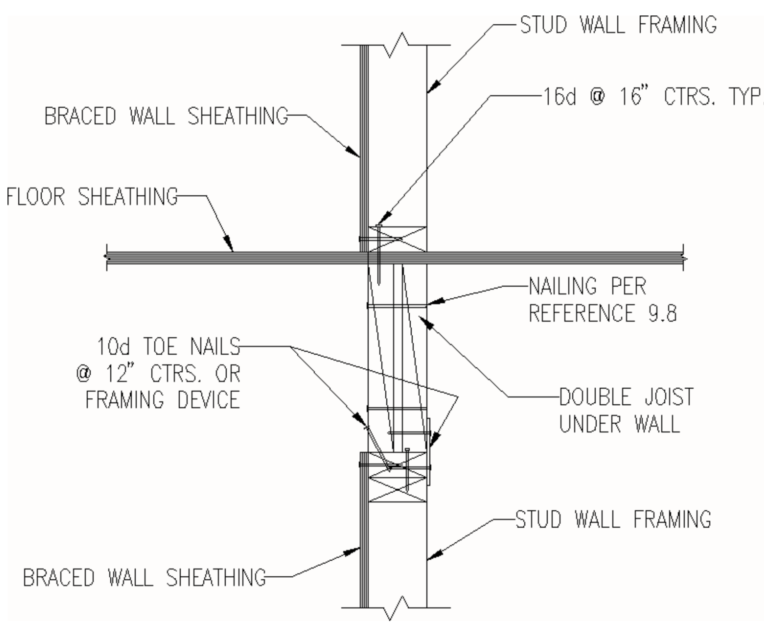


Figure C12.4-9 Interior braced wall at parallel joist.

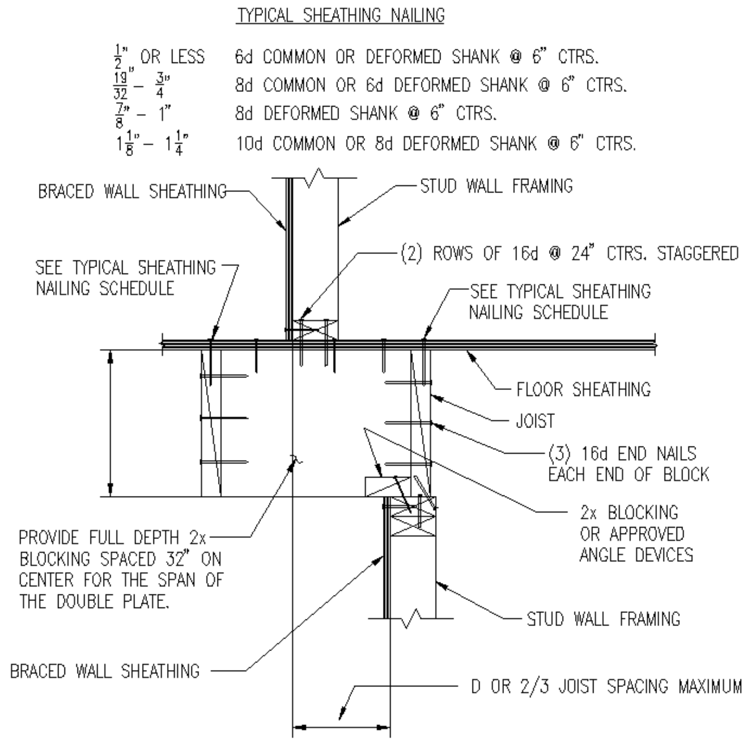


Figure C12.4-10 Offset at interior braced wall.

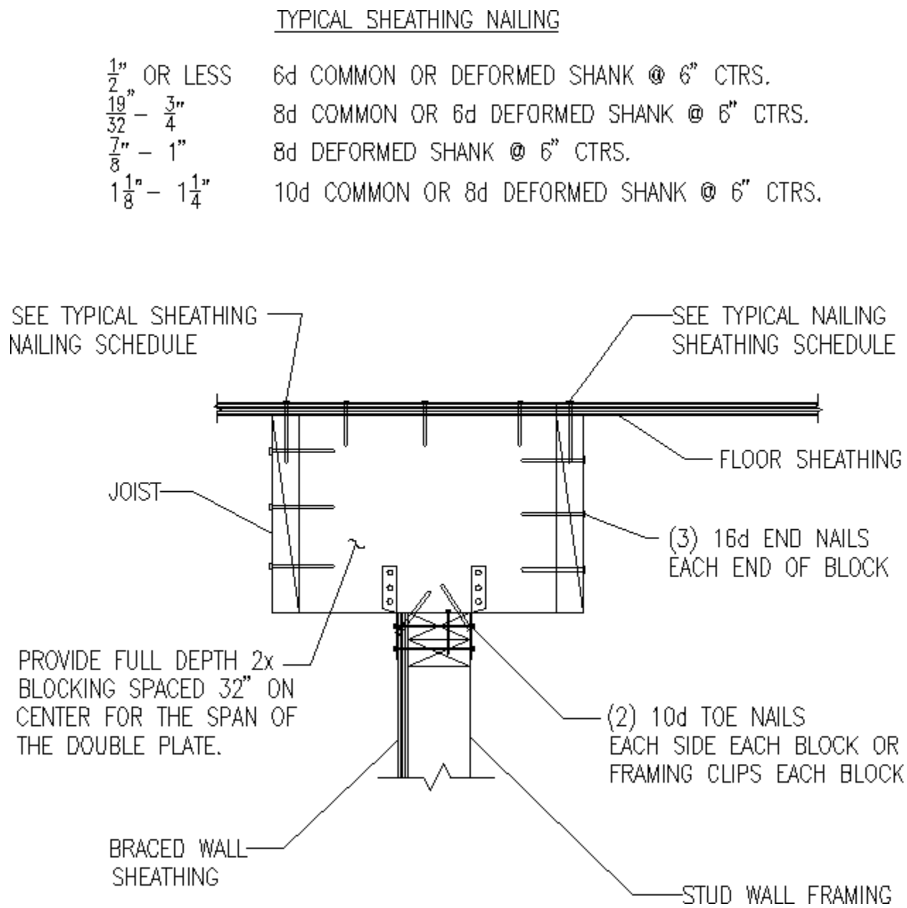


Figure C12.4-11 Diaphragm connection to braced wall below

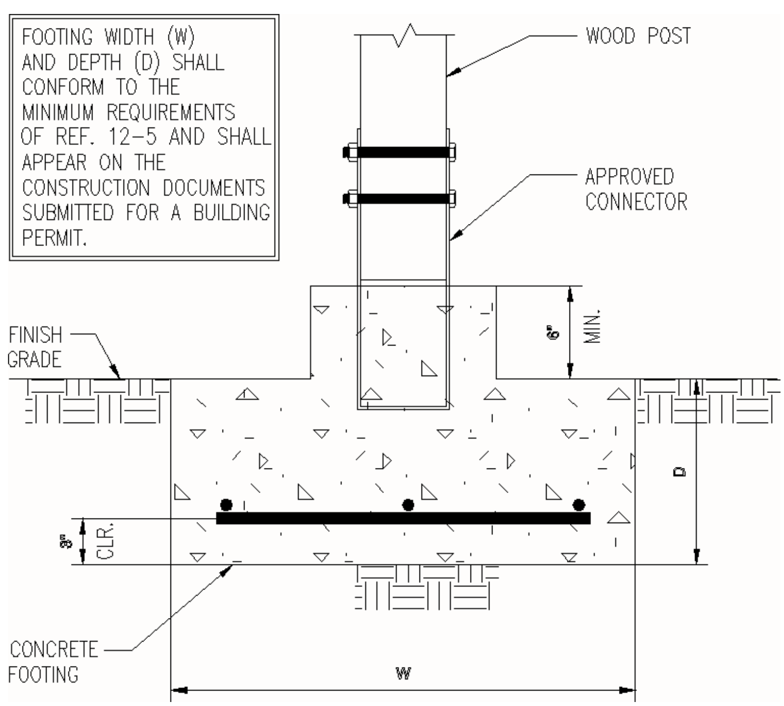


Figure C12.4-12 Post base detail.

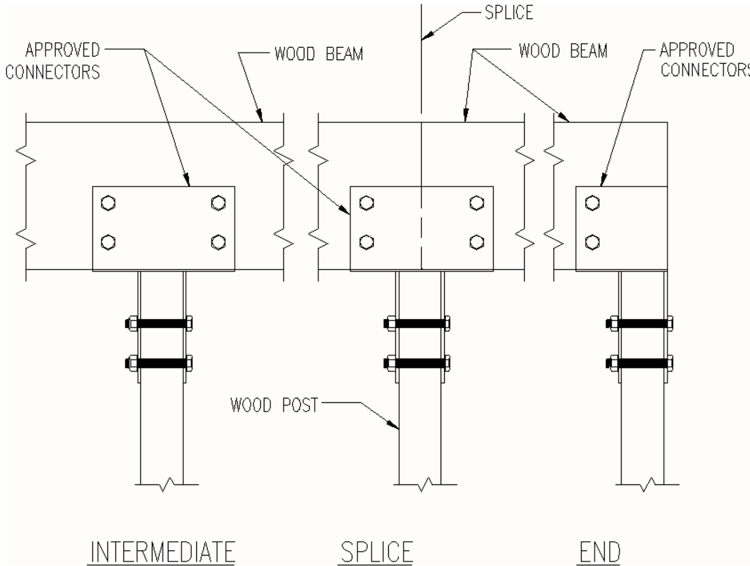


Figure C12.4-13 Wood beam connection to post.

12.4.3.4 Braced wall panel connections. The exception provided in this section of the *Provisions* is included due to the difficulty in providing a mechanism to transfer the diaphragm loads from a truss roof system to the braced wall panels of the top story. This problem has been considered by the Clackamas County, Oregon Building Codes Division, and an alternate to the CABO Building Code Sec. 402.10 was written in 1993, and revised September 5, 1995. The details shown in Figure C12.4-14 through C12.4-17 are provided as suggested methods for providing positive transfer of the lateral forces from the diaphragm through the web sections of the trusses to the top of the braced wall panels below.

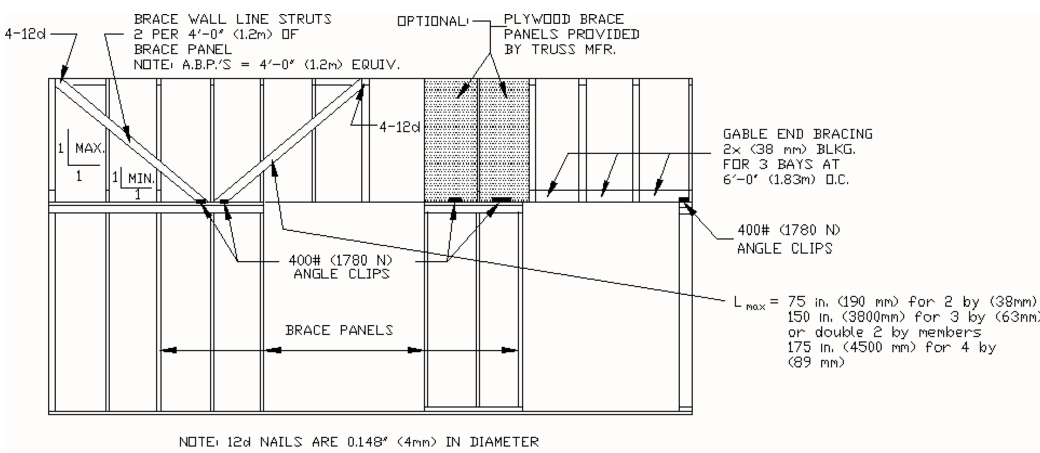


Figure C12.4-14 Suggested methods for transferring roof diaphragm load s to braced wall panels.

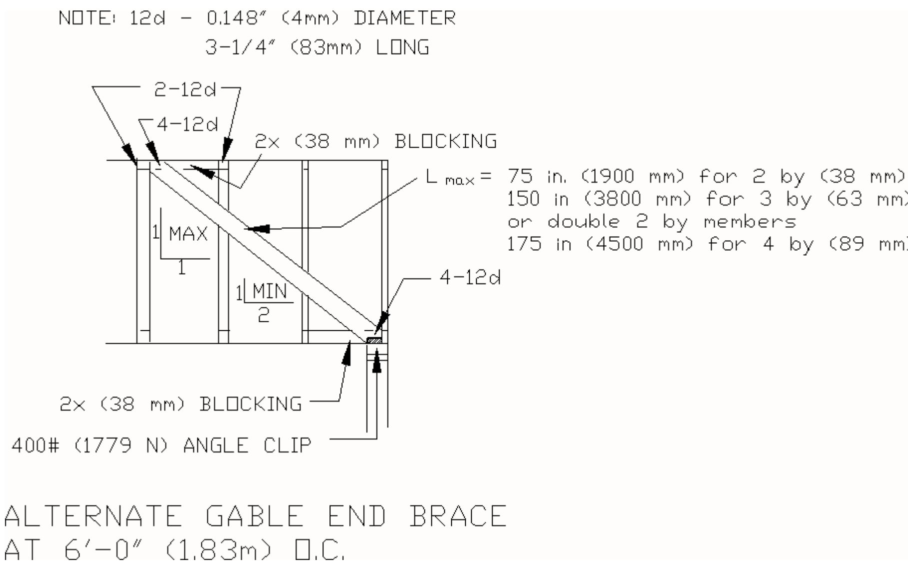


Figure C12.4-15 Alternate gable end brace.

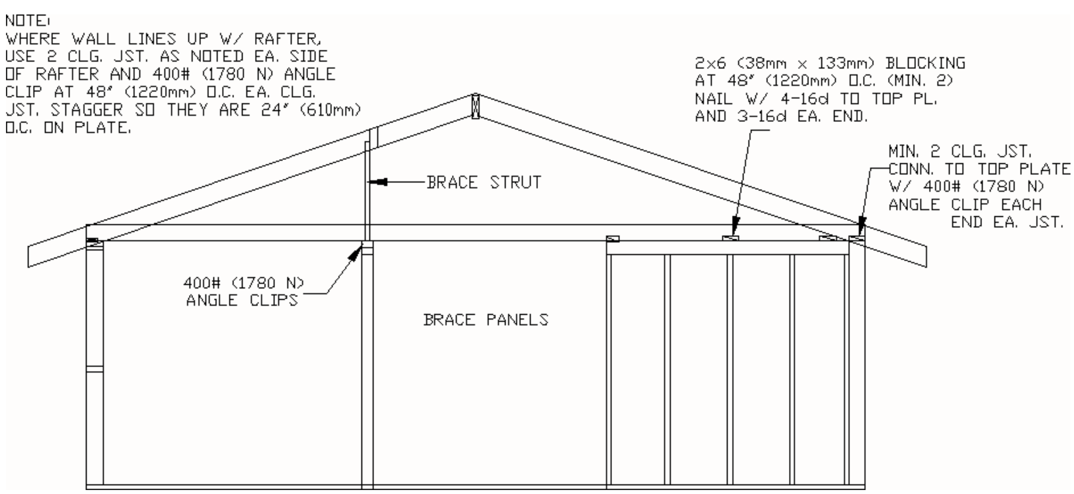


Figure C12.4-16 Wall parallel to truss bracing detail.

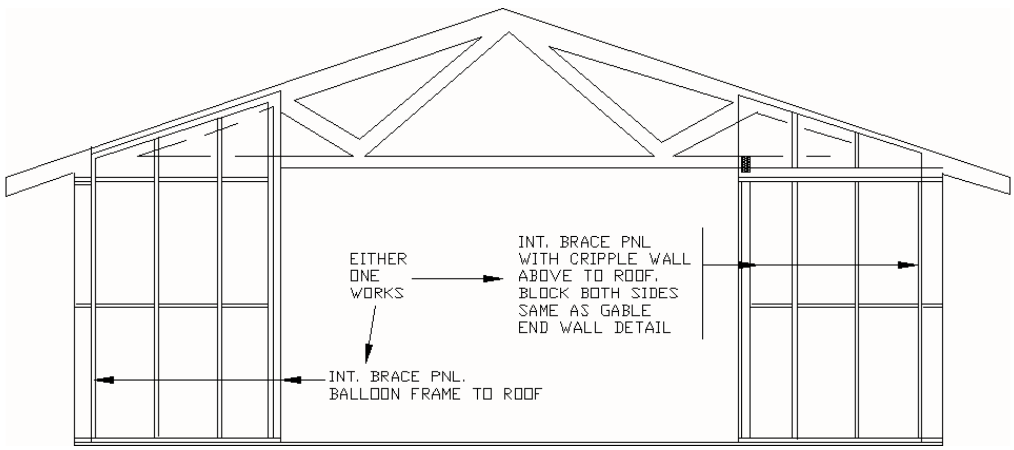


Figure C12.4-17 Wall parallel to truss alternate bracing detail.

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APA B The Engineered Wood Association. 1996. *Wood structural panel shear walls with gypsum wallboard and window/door openings*, Report 157. Tacoma, Washington: APA.

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Chapter 13 Commentary

SEISMICALLY ISOLATED STRUCTURE DESIGN REQUIREMENTS

13.1 GENERAL

Seismic isolation, commonly referred to as base isolation, is a design concept based on the premise that a structure can be substantially decoupled from potentially damaging earthquake motions. By substantially decoupling the structure from the ground motion, the level of response in the structure can be reduced significantly from the level that would otherwise occur in a conventional, fixed-base building. .

The potential advantages of seismic isolation and the recent advancements in isolation-system products already have led to the design and construction of over 200 seismically isolated buildings and bridges in the United States. A significant amount of research, development, and application activity has occurred over the past 20 years. The following references provide a summary of some of the work that has been performed: Applied Technology Council (ATC, 1986, 1993, and 2002), ASCE Structures Congress (ASCE, 1989, 1991, 1993, and 1995), EERI Spectra (EERI, 1990), Skinner, et al. (1993), U.S. Conference on Earthquake Engineering (1990 and 1994), and World Conference on Earthquake Engineering (1988, 1992, 1996, and 2000).

In the mid-1980s, the initial applications identified a need to supplement existing codes with design requirements developed specifically for seismically isolated buildings. Code development work occurred throughout the late 1980s. The status of U.S. seismic isolation design requirements as of May 2003 is as follows:

1. In late 1989, the Structural Engineers Association of California (SEAOC) State Seismology Committee adopted an “Appendix to Chapter 2” of the SEAOC Blue Book entitled, “General Requirements for the Design and Construction of Seismic-Isolated Structures.” These requirements were submitted to the International Conference of Building Officials (ICBO) and were adopted by ICBO as an appendix of the 1991 *Uniform Building Code* (*UBC*). The most current version of these regulations may be found in the ASCE-7-02 (ASCE, 2003) and the 2003 International Building Code (ICC, 2003)..
2. In 1991 the Federal Emergency Management Agency (FEMA) initiated a 6-year program to develop a set of nationally applicable guidelines for seismic rehabilitation of existing buildings. These guidelines (known as the *NEHRP Guidelines for the Seismic Rehabilitation of Buildings*) were published as FEMA 273. In 2000, FEMA 273 was republished, with minor amendments, as FEMA 356, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. The design and analysis methods of the *NEHRP Guidelines* and the FEMA *Prestandard* parallel closely methods required by the *NEHRP Recommended Provisions* for new buildings, except that more liberal design is permitted for the superstructure of a rehabilitated building.

A general concern has long existed regarding the applicability of different types of isolation systems. Rather than addressing a specific method of base isolation, the *Provisions* provides general design requirements applicable to a wide range of possible seismic isolation systems.

Although remaining general, the design requirements rely on mandatory testing of isolation-system hardware to confirm the engineering parameters used in the design and to verify the overall adequacy of the isolation system. Some systems may not be capable of demonstrating acceptability by test and, consequently, would not be permitted. In general, acceptable systems will: (1) remain stable for required design displacements, (2) provide increasing resistance with increasing displacement, (3) not degrade under repeated cyclic load, and (4) have quantifiable engineering parameters (such as force-deflection characteristics and damping).

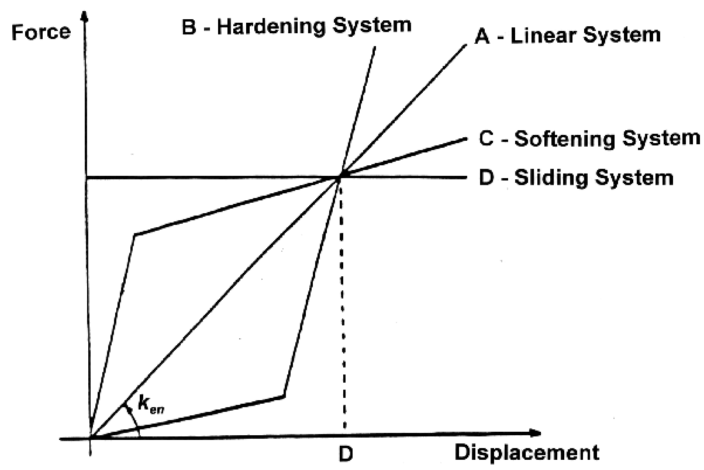

Figure C13.1-1 Idealized force-deflection relationships for isolation systems (stiffness effects of sacrificial wind-restraint systems not shown for clarity).

Conceptually, there are four basic types of isolation system force-deflection relationships. These idealized relationships are shown in Figure C13.1-1 with each idealized curve having the same design displacement, D_D, for the design earthquake. A linear isolation system is represented by Curve A and has the same isolated period for all earthquake load levels. In addition, the force generated in the superstructure is directly proportional to the displacement across the isolation system.

A hardening isolation system is represented by Curve B. This system is soft initially (long effective period) and then stiffens (effective period shortens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a hardening system, the superstructure is subjected to higher forces and the isolation system to lower displacements than a comparable linear system.

A softening isolation system is represented by Curve C. This system is stiff initially (short effective period) and softens (effective period lengthens) as the earthquake load level increases. When the earthquake load level induces displacements in excess of the design displacement in a softening system, the superstructure is subjected to lower forces and the isolation system to higher displacements than a comparable linear system.

A sliding isolation system is represented by Curve D. This system is governed by the friction force of the isolation system. Like the softening system, the effective period lengthens as the earthquake load level increases and loads on the superstructure remain constant.

The total system displacement for extreme displacement of the sliding isolation system, after repeated earthquake cycles, is highly dependent on the vibratory characteristics of the ground motion and may exceed the design displacement, D_D. Consequently, minimum design requirements do not adequately define peak seismic displacement for seismic isolation systems governed solely by friction forces.

13.1.1 Scope. The requirements of Chapter 13 provide isolator design displacements, shear forces for structural design, and other specific requirements for seismically isolated structures. All other design requirements including loads (other than seismic), load combinations, allowable forces and stresses, and horizontal shear distribution are covered by the applicable sections of the *Provisions* for conventional, fixed-base structures.

13.2 GENERAL DESIGN REQUIREMENTS

13.2.1 Occupancy importance factor. Ideally, most of the lateral displacement of an isolated structure will be accommodated by deformation of the isolation system rather than distortion of the structure above. Accordingly, the lateral-load-resisting system of the structure above the isolation system should be designed to have sufficient stiffness and strength to avoid large, inelastic displacements. For this reason, the *Provisions* contains criteria that limit the inelastic response of the structure above the isolation system. Although damage control for the design-level earthquake is not an explicit objective of the *Provisions*, an isolated structure designed to limit inelastic response of the structural system also will reduce the level of damage that would otherwise occur during an earthquake. In general, isolated structures designed in conformance with the *Provisions* should be able:

1. To resist minor and moderate levels of earthquake ground motion without damage to structural elements, nonstructural components, or building contents; and
2. To resist major levels of earthquake ground motion without failure of the isolation system, without significant damage to structural elements, without extensive damage to nonstructural components, and without major disruption to facility function.

The above performance objectives for isolated structures considerably exceed the performance anticipated for fixed-base structures during moderate and major earthquakes. Table C13.2-1 provides a tabular comparison of the performance expected for isolated and fixed-base structures designed in accordance with the *Provisions*. Loss of function is not included in Table C13.2-1. For certain (fixed-base) facilities, loss of function would not be expected to occur until there is significant structural damage causing closure or restricted access to the building. In other cases, the facility could have only limited or no structural damage but would not be functional as a result of damage to vital nonstructural components and contents. Isolation would be expected to mitigate structural and nonstructural damage and to protect the facility against loss of function.

Table C13.2-1 Protection Provided by NEHRP Recommended *Provisions* for Minor, Moderate, and Major Levels of Earthquake Ground Motion

Risk Category	Earthquake Ground Motion Level		
	Minor	Moderate	Major
Life safety[a]	F, I	F, I	F, I
Structural damage[b]	F, I	F, I	I
Nonstructural damage[c] (contents damage)	F, I	I	I

[a] Loss of life or serious injury is not expected.
[b] Significant structural damage is not expected.
[c] Significant nonstructural (contents) damage is not expected.
F indicates fixed base; I indicates isolated.

13.2.3.1 Design spectra. Site-specific design spectra must be developed for both the design earthquake and the maximum considered earthquake if the structure is located at a site with S_1 greater than 0.60 or on a Class F site. All requirements for spectra are in Sec. 3.3 and 3.4.

13.2.4 Procedure selection. The design requirements permit the use of one of three different analysis procedures for determining the design-level seismic loads. The first procedure uses a simple, lateral-force formula (similar to the lateral-force coefficient now used in conventional building design) to prescribe peak lateral displacement and design force as a function of spectral acceleration and isolated-building period and damping. The second and third methods, which are required for geometrically complex or especially flexible buildings, rely on dynamic analysis procedures (either response spectrum or time history) to determine peak response of the isolated building.

The three procedures are based on the same level of seismic input and require a similar level of performance from the building. There are benefits in performing a more complex analysis in that slightly lower design forces and displacements are permitted as the level of analysis becomes more sophisticated. The design requirements for the structural system are based on the design earthquake, a severe level of earthquake ground motion defined as two-thirds of the maximum considered earthquake. The isolation system—including all connections, supporting structural elements, and the "gap"—is required to be designed (and tested) for 100 percent of maximum considered earthquake demand. Structural elements above the isolation system are not required to be designed for the full effects of the design earthquake , but may be designed for slightly reduced loads (that is, loads reduced by a factor of up to 2.0) if the structural system has sufficient ductility, etc., to respond inelastically without sustaining significant damage. A similar fixed-base structure would be designed for loads reduced by a factor of 8 rather than 2.

This section delineates the requirements for the use of the equivalent lateral force procedure and dynamic methods of analysis. The limitations on the simplified lateral-force design procedure are quite severe at this time. Limitations cover the site location with respect to active faults; soil conditions of the site, the height, regularity and stiffness characteristics of the building; and selected characteristics of the isolation system. Response-history analysis is required to determine the design displacement of the isolation system (and the structure above) for the following isolated structures:

1. Isolated structures with a "nonlinear" isolation system including, but not limited to, isolation systems utilizing friction or sliding surfaces, isolation systems with effective damping values greater than about 30 percent of critical, isolation systems not capable of producing a significant restoring force, and isolation systems that restrain or limit extreme earthquake displacement;
2. Isolated structures with a "nonlinear" structure (above the isolation system) including, but not limited to, structures designed for forces that are less than those specified by the *Provisions* for "essentially-elastic" design; and
3. Isolated structures located on Class F site (that is, very soft soil).

Lower-bound limits on isolation system design displacements and structural-design forces are specified by the *Provisions* in Sec. 13.4 as a percentage of the values prescribed by the equivalent-lateral-force design formulas, even when dynamic analysis is used as the basis for design. These lower-bound limits on key design parameters ensure consistency in the design of isolated structures and serve as a "safety net" against gross under-design. Table C13.2-2 provides a summary of the lower-bound limits on dynamic analysis specified by the *Provisions*.

13.2.4.3 Variations in material properties: For analysis, the mechanical properties of seismic isolators are generally based on values provided by isolator manufacturers. The properties are evaluated by prototype testing, which often occurs shortly after the isolators have been manufactured, and checked with respect to the values assumed for design. Unlike conventional materials whose properties do not vary substantially with time, seismic isolators are composed of materials whose properties will generally vary with time. Because (a) mechanical properties can vary over the life span of a building, and (b) the testing protocol of Section 13.6 cannot account for the effects of aging, contamination, scragging (temporary degradation of mechanical properties with repeated cycling), temperature, velocity effects, and wear, the engineer-of-record must account for these effects by explicit analysis. One strategy for accommodating these effects makes use of property modification factors, which was introduced by Constantinou et al. (1999) in the AASHTO Guide Specification for Seismic Isolation Design (AASHTO, 1999). Constantinou et al. (1999) also provides information on variations in material properties for sliding isolation systems. Thompson et al. (2000) and Morgan et al. (2001) provide information on variations in material properties for elastomeric bearings.

Table C13.2-2 Lower-Bound Limits on Dynamic Procedures Specified in Relation to ELF Procedure Requirements

Design Parameter	ELF Procedure	Dynamic Procedure	
		Response Spectrum	Response History
Design displacement – D_D	$D_D = (g/4\pi^2)(S_{D1}T_D/B_D)$	–	–
Total design displacement - D_T	$D_T \geq 1.1D$	$\geq 0.9D_T$	$\geq 0.9D_T$
Maximum displacement – D_M	$D_M = (g/4\pi^2)(S_{M1}T_M/B_M)$	–	–
Total maximum displacement - D_{TM}	$D_{TM} \geq 1.1D_M$	$\geq 0.8D_{TM}$	$\geq 0.8D_{TM}$
Design shear – V_b (at or below the isolation system)	$V_b = k_{Dmax}D_D$	$\geq 0.9V_b$	$\geq 0.9V_b$
Design shear – V_s (“regular” superstructure)	$V_s = k_{Dmax}D_D/R_I$	$\geq 0.8V_s$	$\geq 0.6V_s$
Design shear – V_s (“irregular” superstructure)	$V_s = k_{Dmax}D_DR_I$	$\geq 1.0V_s$	$\geq 0.8V_s$
Drift (calculated using R_I for C_d)	$0.015h_{sx}$	$0.015h_{sx}$	$0.020h_{sx}$

13.2.5 Isolation system

13.2.5.1 Environmental conditions. Environmental conditions that may adversely affect isolation system performance should be thoroughly investigated. Significant research has been conducted on the effects of temperature, aging, etc., on isolation systems since the 1970s in Europe, New Zealand, and the United States.

13.2.5.2 Wind forces. Lateral displacement over the depth of the isolator zone resulting from wind loads should be limited to a value similar to that required for other story heights.

13.2.5.3 Fire resistance. In the event of a fire, the isolation system should be capable of supporting the weight of the building, as required for other vertical-load-supporting elements of the structure, but may have diminished functionality for lateral (earthquake) load.

13.2.5.4 Lateral-restoring force. The isolation system should be configured with a lateral-restoring force sufficient to avoid significant residual displacement as a result of an earthquake, such that the isolated structure will not have a stability problem so as to be in a condition to survive aftershocks and future earthquakes.

13.2.5.5 Displacement restraint. The use of a displacement restraint is not encouraged by the *Provisions*. Should a displacement restraint system be implemented, explicit analysis of the isolated structure for maximum considered earthquake is required to account for the effects of engaging the displacement restraint.

13.2.5.6 Vertical-load stability. The vertical loads to be used in checking the stability of any given isolator should be calculated using bounding values of dead load and live load and the peak earthquake demand of the maximum considered earthquake. Since earthquake loads are reversible in nature, peak earthquake load should be combined with bounding values of dead and live load in a manner which produces both the maximum downward force and the maximum upward force on any isolator. Stability of each isolator should be verified for these two extreme values of vertical load at peak maximum considered earthquake displacement of the isolation system.

13.2.5.7 Overturning. The intent of this requirement is to prevent both global structural overturning and overstress of elements due to local uplift. Uplift in a braced frame or shear wall is acceptable so long as the isolation system does not disengage from its horizontal-resisting connection detail. The connection details used in some isolation systems are such that tension is not permitted on the system. If the tension capacity of an isolation system is to be utilized to resist uplift forces, then component tests should be performed to demonstrate the adequacy of the system to resist tension forces at the design displacement.

13.2.5.8 Inspection and replacement. Although most isolation systems will not need to be replaced after an earthquake, it is good practice to provide for inspection and replacement. After an earthquake, the building should be inspected and any damaged elements should be replaced or repaired. It is advised that periodic inspections be made of the isolation system.

13.2.5.9 Quality control. A test and inspection program is necessary for both fabrication and installation of the isolation system. Because base isolation is a developing technology, it may be difficult to reference standards for testing and inspection. Reference can be made to standards for some materials such as elastomeric bearings (ASTM D 4014). Similar standards are required for other isolation systems. Special inspection procedures and load testing to verify manufacturing quality should be developed for each project. The requirements will vary with the type of isolation system used.

13.2.6 Structural system

13.2.6.1 Horizontal distribution of force

13.2.6.2 Building separations. A minimum separation between the isolated structure and a rigid obstruction is required to allow free movement of the superstructure in all lateral directions during an earthquake. Provision should be made for lateral motion greater than the design displacement, since the exact upper limit of displacement cannot be precisely determined.

13.2.7 Elements of structures and nonstructural components. To accommodate the differential movement between the isolated building and the ground, provision for flexible utility connections should be made. In addition, rigid structures crossing the interface (such as stairs, elevator shafts and walls) should have details to accommodate differential motion at the isolator level without sustaining damage sufficient to threaten life safety.

13.3 EQUIVALENT LATERAL FORCE PROCEDURE

13.3.2 Minimum lateral displacements. The lateral displacement given by Eq. 13.3-1 approximates peak design earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_D, and equivalent viscous damping, β_D, and the lateral displacement given by Eq. 13.3-3 approximates peak maximum considered earthquake displacement of a single-degree-of-freedom, linear-elastic system of period, T_M, and equivalent viscous damping, β_{DM}.

Equation 13.3-1 is an estimate of peak displacement in the isolation system for the design earthquake. In this equation, the spectral acceleration term, S_{D1}, is the same as that required for design of a conventional fixed-base structure of period, T_D. A damping term, B_D, is used to decrease (or increase) the computed displacement when the equivalent damping coefficient of the isolation system is greater (or smaller) than 5 percent of critical damping. Values of coefficient B_D (or B_M for the maximum considered earthquake) are given in Table 13.3-1 for different values of isolation system damping, β_D (or β_M).

A comparison of values obtained from Eq. 13.3-1 and those obtained from nonlinear time-history analyses are given in Kircher et al. (1988) and Constantinou et al. (1993).

Consideration should be given to possible differences in the properties of the isolation system used for design and the properties of isolation system actually installed in the building. Similarly, consideration should be given to possible changes in isolation system properties due to different design conditions or load combinations. If the true deformational characteristics of the isolation system are not stable or vary with the nature of the load (being rate-, amplitude-, or time-dependent), the design displacements should be based on deformational characteristics of the isolation system that give the largest possible deflection (k_{Dmin}), the design forces should be based on deformational characteristics of the isolation system that give the largest possible force (k_{Dmax}), and the damping level used to determine design displacements and forces should be based on deformational characteristics of the isolation system that represent the minimum amount of energy dissipated during cyclic response at the design level.

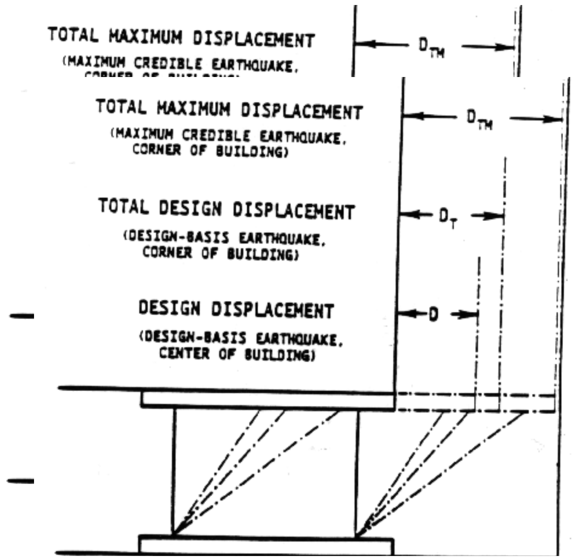

Figure C13.3-1 Displacement terminology.

The configuration of the isolation system for a seismically isolated building or structure should be selected in such a way as to minimize any eccentricity between the center of mass of the superstructure and the center of rigidity of the isolation system. In this way, the effect of torsion on the displacement of isolation elements will be reduced. As for conventional structures, allowance for accidental eccentricity in both horizontal directions must be considered. Figure C13.3-1 defines the terminology used in the *Provisions*. Equation 13.3-5 (or Eq. 13.3-6 for the maximum considered earthquake) provides a simplified formulae for estimating the response due to torsion in lieu of a more refined analysis. The additional component of displacement due to torsion increases the design displacement at the corner of the structure by about 15 percent (for a perfectly square building in plan) to about 30 percent (for a very long, rectangular building) if the eccentricity is 5 percent of the maximum plan dimension. Such additional displacement, due to torsion, is appropriate for buildings with an isolation system whose stiffness is uniformly distributed in plan. Isolation systems that have stiffness concentrated toward the perimeter of the building or certain sliding systems that minimize the effects of mass eccentricity will have reduced displacements due to torsion. The *Provisions* permits values of D_T as small as $1.1D_D$, with proper justification.

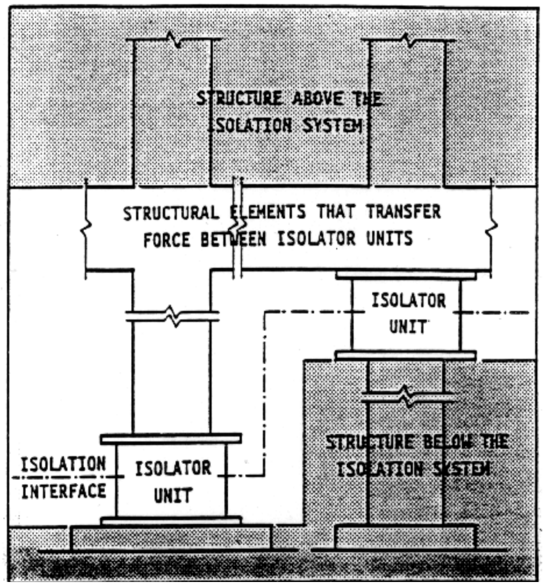

Figure C13.3-2 Isolation system terminology

13.3.3 Minimum lateral forces. Figure C13.3-2 defines the terminology below and above the isolation system. Equation 13.3-7 gives peak seismic shear on all structural components at or below the seismic interface without reduction for ductile response. Equation 13.3-8 specifies the peak seismic shear for design of structural systems above the seismic interface. For structures that have appreciable inelastic-deformation capability, this equation includes an effective reduction factor of up to 2 for response beyond the strength-design level.

The basis for the reduction factor is that the design of the structural system is based on strength-design procedures. A factor of at least 2 is assumed to exist between the design-force level and the true-yield level of the structural system. An investigation of 10 specific buildings indicated that this factor varied between 2 and 5 (ATC, 1982). Thus, a reduction factor of 2 is appropriate to ensure that the structural system remains essentially elastic for the design earthquake .

In Sec. 13.3.3.2, the limitations given on V_S ensure that there is at least a factor of 1.5 between the nominal yield level of the superstructure and (1) the yield level of the isolation system, (2) the ultimate capacity of a sacrificial wind-restraint system which is intended to fail and release the superstructure during significant lateral load, or (3) the break-away friction level of a sliding system.

These limitations are essential to ensure that the superstructure will not yield prematurely before the isolation system has been activated and significantly displaced.

The design shear force, V_S, specified by the requirements of this section ensures that the structural system of an isolated building will be subjected to significantly lower inelastic demands than a conventionally designed structure. Further reduction in V_S, such that the inelastic demand on a seismically isolated structure would be the same as the inelastic demand on a conventionally designed structure, was not considered during development of these requirements but may be considered in the future.

If the level of performance of the isolated structure is desired to be greater than that implicit in these requirements, then the denominator of Eq. 13.3-8 may be reduced. Decreasing the denominator of Eq. 13.3-8 will lessen or eliminate inelastic response of the superstructure for the design-basis event.

13.3.4 Vertical distribution of forces. Equation 13.3-9 describes the vertical distribution of lateral force based on an assumed triangular distribution of seismic acceleration over the height of the structure above the isolation interface. Constantinou et al. (1993) provides a good summary of recent work which demonstrates that this vertical distribution of force will always provide a conservative estimate of the distributions obtained from more detailed, nonlinear analysis studies.

13.3.5 Drift limits. The maximum story drift permitted for design of isolated structures varies depending on the method of analysis used, as summarized in Table C13.3-1. For comparison, the drift limits prescribed by the *Provisions* for fixed-base structures also are summarized in Table C13.3-1.

Table C13.3-1 Comparison of Drift Limits for Fixed-Base and Isolated Structures

Structure	Seismic Use Group	Fixed-Base	Isolated
Buildings (other than masonry) four stories or less in height with component drift design	I	$0.025h_{sx}/(C_d/R)$	$0.015h_{sx}$
	II	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
Other (non-masonry) buildings	I	$0.020h_{sx}/(C_d/R)$	$0.015h_{sx}$
	II	$0.015h_{sx}/(C_d/R)$	$0.015h_{sx}$
	III	$0.010h_{sx}/(C_d/R)$	$0.015h_{sx}$

Drift limits in Table C13.3-1 are divided by C_d/R for fixed-base structures since displacements calculated for lateral loads reduced by R are factored by C_d before checking drift. The C_d term is used throughout the *Provisions* for fixed-base structures to approximate the ratio of actual earthquake response to response calculated for "reduced" forces. Generally, C_d is 1/2 to 4/5 the value of R. For isolated structures, the R_I factor is used both to reduce lateral loads and to increase displacements (calculated for reduced lateral loads) before checking drift. Equivalency would be obtained if the drift limits for both fixed-base and isolated structures were based on their respective R factors. It may be noted that the drift limits for isolated structures are generally more conservative than those for conventional, fixed-base structures, even when fixed-base structures are designed as Seismic Use Group III buildings.

13.4 DYNAMIC PROCEDURES

This section specifies the requirements and limits for dynamic procedures. The design displacement and force limits on response spectrum and response history procedures are given in Table C13.2-1.

A more-detailed or refined study can be performed in accordance with the analysis procedures described in this section. The intent of this section is to provide procedures which are compatible with the minimum requirements of Sec. 13.3. Reasons for performing a more refined study include:

1. The importance of the building.
2. The need to analyze possible structure/isolation-system interaction when the fixed-base period of the building is greater than one third of the isolated period.
3. The need to explicitly model the deformational characteristics of the lateral-force-resisting system when the structure above the isolation system is irregular.
4. The desirability of using site-specific ground-motion data, especially for soft soil types (Site Class F) or for structures located where S_1 is greater than 0.60.
5. The desirability of explicitly modeling the deformational characteristics of the base-isolation system. This is especially important for systems that have damping characteristics that are amplitude-dependent, rather than velocity-dependent, since it is difficult to determine an appropriate value of equivalent viscous damping for these systems.

Sec. 13.2.4 of this commentary discusses other conditions which require use of the response history procedure.

When response history analysis is used as the basis for design, the design displacement of the isolation system and design forces in elements of the structure above are to be based on the maximum of the results of not less than three separate analyses, each using a different pair of horizontal time histories. Each pair of horizontal time histories should:

1. Be of a duration consistent with the design earthquake or the maximum considered earthquake,
2. Incorporate near-field phenomena, as appropriate, and
3. Have response spectra for which the square-root-of-the-sum-of-the-squares combination of the two horizontal components equals or exceeds 1.3 times the "target" spectrum at each spectral ordinate.

The average value of seven time histories is a standard required by the nuclear industry and is considered appropriate for nonlinear response history analysis of seismically isolated structures.

13.5 DESIGN REVIEW

Review of the design and analysis of the isolation system and design review of the isolator testing program is mandated by the *Provisions* for two key reasons:

1. The consequences of isolator failure could be catastrophic.
2. Isolator design and fabrication technology is evolving rapidly and may be based on technologies unfamiliar to many design professionals.

The *Provisions* requires review to be performed by a team of registered design professionals that are independent of the design team and other project contractors. The review team should include individuals with special expertise in one or more aspects of the design, analysis, and implementation of seismic isolation systems.

The review team should be formed prior to the development of design criteria (including site-specific ground shaking criteria) and isolation system design options. Further, the review team should have full access to all pertinent information and the cooperation of the design team and regulatory agencies involved with the project.

13.6 TESTING

The design displacements and forces developed from the *Provisions* are predicated on the basis that the deformational characteristics of the base isolation system have been previously defined by a comprehensive set of tests. If a comprehensive amount of test data are not available on a system, major design alterations in the building may be necessary after the tests are complete. This would result from variations in the isolation-system properties assumed for design and those obtained by test. Therefore, it is advisable that prototype systems be tested during the early phases of design, if sufficient test data is not available on an isolation system.

Typical force-deflection (or hysteresis) loops are shown in Figure C13.6-1; also included are the definitions of values used in Sec. 13.6.2.

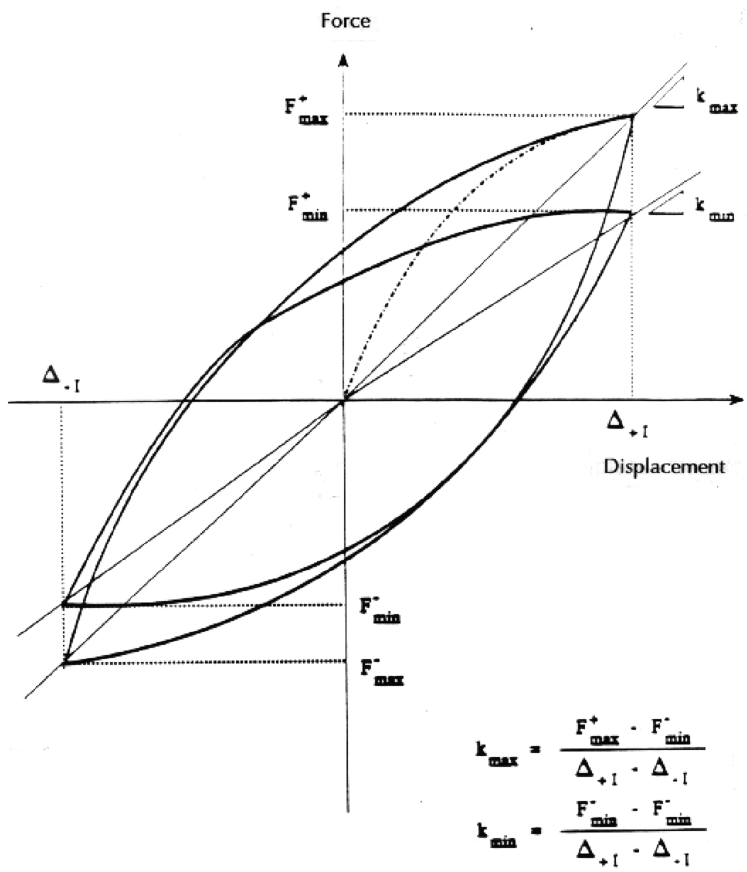

Figure C13.6-1 The effect of stiffness on an isolation bearing.

The required sequence of tests will verify experimentally:

1. The assumed stiffness and capacity of the wind-restraining mechanism;
2. The variation in the isolator's deformational characteristics with amplitude (and with vertical load, if it is a vertical load-carrying member);
3. The variation in the isolator's deformational characteristics for a realistic number of cycles of loading at the design displacement; and
4. The ability of the system to carry its maximum and minimum vertical loads at the maximum displacement.

Force-deflection tests are not required if similarly sized components have been tested previously using the specified sequence of tests.

Variations in effective stiffness greater than 15 percent over 3 cycles of loading at a given amplitude, or greater than 20 percent over the larger number of cycles at the design displacement, would be cause for rejection. The variations in the vertical loads required for tests of isolators which carry vertical, as well as lateral, load are necessary to determine possible variations in the system properties with variations in overturning force. The appropriate dead loads and overturning forces for the tests are defined as the average loads on a given type and size of isolator for determining design properties and are the absolute maximum and minimum loads for the stability tests.

13.6.4 Design properties of the isolation system

13.6.4.1 Maximum and minimum effective stiffness. The effective stiffness is determined from the hysteresis loops shown in Figure C13.6-1). Stiffness may vary considerably as the test amplitude increases but should be reasonably stable (within 15 percent) for more than 3 cycles at a given amplitude.

The intent of these requirements is to ensure that the deformational properties used in design result in the maximum design forces and displacements. For determining design displacement, this means using the lowest damping and effective-stiffness values. For determining design forces, this means using the lowest damping value and the greatest stiffness value.

13.6.4.2 Effective damping. The determination of equivalent viscous damping is reasonably reliable for systems whose damping characteristics are velocity dependent. For systems that have amplitude-dependent, energy-dissipating mechanisms, significant problems arise in determining an equivalent viscous-damping value. Since it is difficult to relate velocity and amplitude-dependent phenomena, it is recommended that when the equivalent-viscous damping assumed for the design of amplitude-dependent, energy-dissipating mechanisms (such as pure-sliding systems) is greater than 30 percent, then the design-basis force and displacement should be determined using the response history procedure, as discussed in *Commentary* Sec. 13.2.4.

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Chapter 14 Commentary

NONBUILDING STRUCTURE DESIGN REQUIREMENTS

14.1 GENERAL

14.1.1 Scope. Requirements concerning nonbuilding structures were originally added to the 1994 *Provisions* by the 1991-94 *Provisions* Update Committee (PUC) at the request of the BSSC Board of Direction to provide building officials with needed guidance. In recognition of the complexity, nuances, and importance of nonbuilding structures, the BSSC Board established 1994-97 PUC Technical Subcommittee 13 (TS13), Nonbuilding Structures, in 1995. The duties of TS13 were to review the 1994 *Provisions* and *Commentary* and recommend changes for the 1997 Edition. The subcommittee comprised individuals possessing considerable expertise concerning various specialized nonbuilding structures and representing a wide variety of industries concerned with nonbuilding structures.

Building codes traditionally have been perceived as minimum standards of care for the design of nonbuilding structures and building code compliance of these structures is required by building officials in many jurisdictions. However, requirements in the industry standards are often at odds with building code requirements. In some cases, the industry standards need to be altered while in other cases the building codes need to be modified. Registered design professionals are not always aware of the numerous accepted standards within an industry and may not know whether the accepted standards are adequate. It is hoped that Chapter 14 of the *Provisions* appropriately bridges the gap between building codes and existing industry standards.

One of the goals of TS13 was to review and list appropriate industry standards to serve as a resource. These standards had to be included in the appendix. The subcommittee also has attempted to provide an appropriate link so that the accepted industry standards can be used with the seismic ground motions established in the *Provisions*. It should be noted that some nonbuilding structures are very similar to a building and can be designed employing sections of the *Provisions* directly whereas other nonbuilding structures require special analysis unique to the particular type of nonbuilding structure.

The ultimate goal of TS13 was to provide guidance to develop requirements consistent with the intent of the *Provisions* while allowing the use of accepted industry standards. Some of the referenced standards are consensus documents while others are not.

One good example of the dilemma posed by the conflicts between the *Provisions* and accepted design practice for nonbuilding structures involves steel multilegged water towers. Historically, such towers have performed well when properly designed in accordance with American Water Works Association (AWWA) standards, but these standards differ from the *Provisions* that tension-only rods are required and the connection forces are not amplified. However, industry practice requires upset rods that are preloaded at the time of installation, and the towers tend to perform well in earthquake areas.

In an effort to provide the appropriate interface between the *Provisions* requirements for building structures, nonstructural components, and nonbuilding structures; TS13 recommended that nonbuilding structure requirements be placed in a separate chapter. The PUC agreed with this change. The 1997 *Provisions* Chapter 14 now provides registered design professionals responsible for designing nonbuilding structures with a single point of reference.

Note that building structures, vehicular and railroad bridges, electric power substation equipment, overhead power line support structures, buried pipelines and conduits, tunnels, lifeline systems, nuclear power plants, and dams are excluded from the scope of the nonbuilding structure requirements. The excluded structures are covered by other well established design criteria (e.g., electric power substation equipment, power line support structures, vehicular and railroad bridges), are not under the jurisdiction of local building officials (e.g., nuclear power plants, and dams), or require technical considerations beyond the scope of the *Provisions* (e.g., piers and wharves, buried pipelines and conduits, tunnels, and lifeline systems). Since many components of lifeline systems can be designed in accordance with the *Provisions*, the following information is provided to clarify why lifeline systems are excluded from the scope of the *Provisions*.

Seismic design for a lifeline system will typically require consideration of factors that are unique to or particularly important to that specific system. Seismic design requirements for lifeline systems will typically differ from those for buildings individual structural components for the following reasons:

1. **Physical characteristics.** A building consists of structural and non-structural components within a single site, whereas lifeline systems consist of networks of multiple and spatially distributed linked components (primarily non-building structures and equipment, and possibly some buildings as well.)

2. **Stakeholders.** The stakeholders in the continued operation of a building after an earthquake are a relatively small group of building owners, tenants, and insurers. Lifeline systems provide essential services to a community (e.g., electric power,, communications, transportation, natural gas, water, wastewater, and liquid fuel). Therefore, stakeholders in the seismic performance of such systems are the businesses and residents of the region served by the system, business clients/vendors outside of the region whose continued operation will be impacted by the conditions of the businesses/residents within the region, and the lifeline system's owners and insurers.

3. **Performance.**. Acceptable seismic performance of a building is typically measured by whether life safety of building occupants has been adequately protected (in accordance with minimum building code design provisions.) In addition, for those relatively few buildings for which performance based design has been considered, acceptable seismic performance will also be measured by how well post-earthquake functionality and return-to-service requirements of the building tenants have been met.

   The ability of a lifeline system to maintain an acceptable level of service after an earthquake will depend, not only on the seismic performance of its various spatially dispersed components, but also on the redundancy and service capacity of these components (e.g., number of lanes within roadway elements). To the extent that a lifeline system is comprised of redundant components of sufficient service capacity, it can maintain an acceptable level of service to a community even if some of the redundant components are damaged during the earthquake. In addition, except for certain transportation structures (e.g., bridges and tunnels), earthquake damage to the lifeline system components generally do not result in direct life-safety consequences. Therefore, acceptable seismic performance for a lifeline system is typically based on: (a) whether the system provides an adequate level of service to its users after an earthquake; (b) whether economic losses related to direct damage, lost revenue from an inoperable system, and liability exposure are within tolerable limits; and (c) whether any adverse political, legal, social, administrative, or environmental consequences are experienced. For these reasons, acceptable seismic performance requirements for lifeline systems are best established through interaction with the appropriate stakeholders, including the lifeline agency, its customers or users, and appropriate regulatory interests.

The definition of what constitutes a component of a lifeline system is often complicated. Components of utility lifeline systems are typically identical to components that might be found in industrial or commercial applications. A good example of this overlap are aboveground storage tanks that are common in large industrial or manufacturing facilities as well as water and liquid hydrocarbon transportation systems. Because of this similarity, a clear definition is needed to determine when design in accordance with the recommended approach for lifeline systems should be give preference over requirements in the *Provisions.* Three criteria are considered for determining whether the design of a particular nonbuilding structure can be treated as a component of a lifeline system.

1. **Spatial distribution.** As noted above, lifeline systems are typically spatially-distributed systems that provide services considered essential to community activities and include electric power, communications, water, waste-water, natural gas, liquid fuel, and transportation systems. Fixed facilities, such as power plants, compressor stations, metering stations, are typically treated as nodes

of a lifeline system and are designed in accordance with these Provisions.

2. **Definition by legal boundary.** Portions of utility lifeline systems upstream of the point defining the legal boundary for ownership and responsibility for maintenance and repair shall be considered as part of a lifeline system. The physical elements of transportation lifeline systems not excluded in the *Provisions* and owned and maintained by a transportation agency are also considered part of a lifeline system.

   Defining lifeline system components by a legal boundary is most appropriate for utility systems that deliver electric power, natural gas, electric power, wastewater and some telecommunication services. Existing regulatory provisions commonly specify a specific interface between the portions of these systems that is under the control of the service provider and the portions of the system under control of the building or facility owner. For electric power, natural gas, and water systems, this boundary is typically the customer's side of the meter. The other typical boundary is the property line. Those components under control of the service provider can be considered as part of a lifeline system.

   It is common for the design and maintenance of physical elements of transportation lifeline systems to fall under the jurisdiction of a governmental or government-regulated entity. Two common examples include state highway departments and port authorities. In such cases, the definition of a lifeline system by legal boundary for these situations is defined by the jurisdiction of these agencies.

3. **Definition by expertise.** Historically, the primary audience of the *Provisions* has been the structural engineering community and building code organizations seeking to modify their seismic provisions. As a result of this focus, the *Provisions* are best suited for the seismic design and performance of individual structures. Since most new construction for lifeline systems address adding components to existing systems, rational design approaches should consider the overall system performance in design of new components and the benefits of improved seismic performance in comparison with the performance of the system for other natural and other hazards, such as man-made threats. The geographically diverse nature of lifeline systems often requires that earthquake hazards be defined by one or more scenario events instead of the probabilistic ground motion hazards defined in the *Provisions*. These additional considerations often require special expertise in addition to that of the structural engineering profession that is dominant audience for the *Provisions*.

14.1.2 References

American Concrete Institute (ACI):

ACI 350, *Environmental Engineering Concrete Structures*, 2001.

ACI 350.3 *Seismic Design of Liquid-Containing Concrete Structures,* 2001.

American Society of Civil Engineers (ASCE):

Guidelines for Seismic Evaluation and Design of Petrochemical Facilities, 1997.

Housner, G.W. *Earthquake Pressures in Fluid Containers*, California Institute of Technology, 1954.

Miller, C. D., S. W. Meier, and W. J. Czaska, *Effects of Internal Pressure on Axial Compressive Strength of Cylinders and Cones,* Structural Stability Research Council Annual Technical Meeting, June 1997.

Rack Manufacturers Institute:

Specification for the Design, Testing, and Utilization of Industrial Steel Storage Racks, 1997 (Reaffirmed 2002).

Troitsky, M.S., *Tubular Steel Structures*, 1990. (Troitsky)

Wozniak, R. S., and W. W. Mitchell, *Basis of Seismic Design Provisions for Welded Steel Oil Storage Tanks*, 1978 Proceedings—Refining Dept., Vol. 57, American Petroleum Institute, Washington, D.C., May 9, 1978. (Wozniak)

14.1.2.2 Other references

While not cited directly in the Provisions or Commentary, the user may find these other references related to nonbuilding structures helpful.

ACI 307,	*Standard Practice for the Design and Construction of Cast-In-Place Reinforced Concrete Chimneys*, 1995.
ANSI K61.1	*Safety Requirements for the Storage and Handling of Anhydrous Ammonia*, American National Standards Institute, 1999.
API 2510	*Design and Construction of Liquefied Petroleum Gas Installation*, American Petroleum Institute, 19952001.
ASCE Petro	*Design of Secondary Containment in Petrochemical Facilities* (Petrochemical Energy Committee Task Report), American Society of Civil Engineers, 1997.
ASME B31.8	*Gas Transmission and Distribution Piping Systems*, American Society of Mechanical Engineers, 1995.
ASME B96.1	*Welded Aluminum-Alloy Storage Tanks*, American Society of Mechanical Engineers, 1993.
ASME STS-1	*Steel Stacks*, American Society of Mechanical Engineers, 2001.
ASTM F 1159	*Standard Practice for the Design and Manufacture of Amusement Rides and Devices* (ASTM F 1159-97a), American Society for Testing and Materials, 1997.
ASTM C 1298	*Standard Guide for Design and Construction of Brick Liners for Industrial Chimneys* (ASTM C 1298-95), American Society for Testing and Materials, 1995.
DOT 49CFR193	*Liquefied Natural Gas Facilities: Federal Safety Standards* (Title 49CFR Part 193), U.S. Department of Transportation, 2000.
NFPA 30	*Flammable and Combustible Liquids Code*, National Fire Protection Association, 12000.
NFPA 58	*Storage and Handling of Liquefied Petroleum Gas*, National Fire Protection Association, 2001.
NFPA 59	*Storage and Handling of Liquefied Petroleum Gases at Utility Gas Plants*, National Fire Protection Association, 2001.
NFPA 59A	*Production, Storage and Handling of Liquefied Natural Gas (LNG)*, National Fire Protection Association, 2001.
NCEL R-939	Ebeling, R. M., and Morrison, E. E., *The Seismic Design of Waterfront Retaining Structures*, Naval Civil Engineering Laboratory, 1993.
NAVFAC DM-25.1	*Piers and Wharves*, U.S. Naval Facilities Engineering Command, 1987.
TM 5-809-10	*Seismic Design for Buildings*, U.S. Army Corps of Engineers, 1992, Chapter 13 only.

14.1.5 Nonbuilding structures supported by other structures. This section has been developed to provide an appropriate link between the requirements for nonbuilding structures and those for inclusion in the rest of the *Provisions*—especially the requirements for architectural, mechanical, and electrical components.

14.2 GENERAL DESIGN REQUIREMENTS

14.2.1 Seismic use groups and importance factors. The Importance Factors and Seismic Use Group classifications assigned to nonbuilding structures vary from those assigned to building structures.

Buildings are designed to protect occupants inside the structure whereas nonbuilding structures are not normally "occupied" in the same sense as buildings, but need to be designed in a special manner because they pose a different sort of risk in regard to public safety (that is, they may contain very hazardous compounds or be essential components in critical lifeline systems). For example, tanks and vessels may contain materials that are essential for lifeline functions following a seismic event (such as fire-fighting or potable water), potentially harmful or hazardous to the environment or general health of the public, biologically lethal or toxic, or explosive or flammable (posing a threat of consequential or secondary damage).

If not covered by the authority having jurisdiction, Table 14.2-1 may be used to select the importance factor *(I)*. The value shall be determined by taking the larger of the value from the approved Standard or the value selected from Table 14.2-1. It should be noted that a single value of importance factor may not apply to an entire facility. For further details, refer to ASCE Petro. The use of a secondary containment system, when designed in accordance with an acceptable National Standard, could be considered as an effective means to contain hazardous substances and thus reduce the hazard classification.

The specific definition of material hazard and what constitutes a hazard is being developed in the *International Building Code* process. The hazards will be predicated on the quantity and type of hazardous material.

The importance factor is not intended for use in making economic evaluations regarding the level of damage, probabilities of occurrence, or cost to repair the structure. These economic decisions should be made by the owner and other interested parties (insurers, financiers, etc.). Nor it is intended for use for purposes other than that defined in this provision.

Examples are presented below demonstrate how this table may be applied.

Example 1. A water storage tank used to provide pressurized potable water for a process within a chemical plant where the tank is located away from personnel working within the facility.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues implied in the matrix:

- Seismic Use Group: Neither the structure nor the contents are critical, therefore use Seismic Use Group I.
- Function: The water storage tank is neither a designated ancillary structure for post-earthquake recovery, nor identified as an emergency back-up facilities for a Seismic Use Group III structure, therefore use F-I.
- Hazard: The contents are not hazardous, therefore use H-I.
- This tank has an importance factor of 1.0.

Example 2. A steel storage rack is located in a retail store in which the customers have direct access to the aisles. Merchandise is stored on the upper racks. The rack is supported by a slab on grade.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues in the matrix:

- Seismic Use Group: Neither the structure nor the contents are critical, therefore use Seismic Use Group I.
- Function: The storage rack is neither used for post-earthquake recovery, nor required for emergency back-up, therefore use F-I.
- Hazard: The contents are not hazardous. However, its use could cause a substantial public hazard during an earthquake. Subject to the local authority's jurisdiction it is H-II.
- According to Sec. 14.3.5.2 the importance factor for storage racks in occupancies open to the general public must be taken as 1.5.
- Use an importance factor of 1.5 for this structure.

Example 3. A water tank is located within an office building complex to supply the fire sprinkler system.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues in the matrix:

- Seismic Use Group: The office building is assigned to Seismic Use Group I.
- Function: The water tank is required to provide water for fire fighting. However since the building is not a Seismic Use Group III structure, the water is used neither for post-earthquake recovery, nor for emergency back-up, so use F-I.
- Hazard: The content and its use are not hazardous to the public, therefore use H-I.
- Use an importance factor of 1.0 for this water structure.

Example 4. A petrochemical storage tank is to be constructed within a refinery tank farm near a populated city neighborhood. An impoundment dike is provided to control liquid spills.

Table 14.2-1 Seismic Use Groups and Importance Factors for Nonbuilding Structures

Seismic Use Group	**I**	**II**	**III**
Function	F-I	F-II	F-III
Hazard	H-I	H-II	H-III
Importance Factor	I = 1.0	I = 1.25	I = 1.5

Address each of the issues in the matrix:

- Seismic Use Group: The LNG tank is assigned to Seismic Use Group III.
- Function: The tank is neither required to provide post-earthquake recovery nor used for emergency back-up for a Seismic Use Group III structure, so use F-I.
- Hazard: The tank contains a substantial quantity of high explosive and is near a city neighborhood. Despite the diking, it is considered hazardous to the public in the event of an earthquake, so use H-III.
- Use an importance factor of 1.5 for this structure.

14.2.3 Design basis. The design basis for nonbuilding structures is based on either adopted references, approved standards, or these *Provisions*. It is intended that the *Provisions* applicable to buildings apply to nonbuilding structures, unless specifically noted in this Chapter.

14.2.4 Seismic force-resisting system selection and limitations. Nonbuilding structures similar to buildings may be designed in accordance with either Table 4.3-1 or Table 14.2-2, including referenced design and detailing requirements. For convenience, Table 4.3-1 requirements are repeated in Table 14.2-2.

Table 14.2-2 of the 2000 NEHRP Provisions for nonbuilding structures similar to buildings prescribed R, Ω_o, and C_d values to be taken from Table 4.3-1, but prescribed less restrictive height limitations than those prescribed in Table 4.3-1. This inconsistency has been corrected. Nonbuilding structures similar to buildings which use the same R, Ω_o, and C_d values as buildings now have the same height limits, restrictions and footnote exceptions as buildings. The only difference is that the footnote exceptions for buildings apply to metal building like systems while the exceptions for nonbuilding structures apply to pipe racks. In addition, selected nonbuilding structures similar to buildings have prescribed an option where both lower R values and less restrictive height limitations are specified. This option permits selected types of nonbuilding structures which have performed well in past earthquakes to be constructed with less restrictions in Seismic Design Categories D, E and F provided seismic detailing is used and design force levels are considerably higher. It should be noted that revised provisions are considerably more restrictive than those prescribed in Table 4.3-1.

Nonbuilding structures not similar to buildings should be designed in accordance Table 14.2-3 requirements, including referenced design and detailing requirements.

Nonbuilding structures not referenced in either Table 14.2-2, Table 14.2-3, or Table 4.3-1 may be designed in accordance with an adopted reference, including its design and detailing requirements.

It is not consistent with the intent of the *Provisions* to take design values from one table or standard and design and/or detailing provisions from another.

14.2.5 Structural analysis procedure selection. Nonbuilding structures that are similar to buildings should be subject to the same analysis procedure limitations as building structures.

Nonbuilding structures that are not similar to buildings should not be subject to these procedure limitations. However, they should be subject to any procedure limitations prescribed in specific adopted references.

For nonbuilding structures supporting flexible system components, such as pipe racks, the supported piping and platforms are generally not regarded as rigid enough to redistribute seismic forces to the supporting frames.

For nonbuilding structures supporting rigid system components, such as steam turbine generators (STG's) and Heat Recovery Steam Generators (HRSG's), the supported equipment, ductwork, and other components (depending on how they are attached to the structure) may be rigid enough to redistribute seismic forces to the supporting frames. Torsional effects may need to be considered in such situations.

14.2.9 Fundamental period. The rational methods for period calculation contained in the *Provisions* were developed for building structures. If the nonbuilding structure has dynamic characteristics similar to those of a building, the difference in period is insignificant. If the nonbuilding structure is not similar to a building structure, other techniques for period calculation will be required. Some of the references for specific types of nonbuilding structures contain more accurate methods for period determination.

Equations 5.2-6, 5.2-7, and 5.2-7 are not recommended because they are not relevant for the commonly encountered nonbuilding structures.

14.3 NONBUILDING STRUCTURES SIMILAR TO BUILDINGS

Nonbuilding structures exhibit behavior similar to that of building structures; however, their function and performance are different. Although the *Provisions* for buildings are used as the primary basis for design, this section identifies appropriate exceptions, modifications, and additions for selected nonbuilding structures similar to buildings.

14.3.1 Electrical power generating facilities. Electrical power plants closely resemble building structures, and their performance in seismic events has been good. For reasons of mechanical performance, lateral drift of the structure must be limited. The lateral bracing system of choice has been the concentrically braced frame. The height limits on braced frames in particular can be an encumbrance to the design of large power generation facilities.

14.3.3 Piers and wharves. Current industry practice recognizes the distinct differences between the two categories of piers and wharves described in the *Provisions*. The piers and wharves with public occupancy, described in paragraph (a) are commonly treated as the "foundation" for buildings or building-like structures, and design is performed using the *Provisions*. The design is likely to be under the jurisdiction of the local building official.

Piers and wharves where occupancy by the general public is not a consideration, as described in paragraph (b), are often treated differently. In many cases, they do not fall under the jurisdiction of building officials, and utilize other design approaches more common to this industry.

Economics plays a major role in the design decisions associated with these structures. These economic decisions may be affected not only by the wishes of the owners, but also by overlapping jurisdictional entities with local, regional, or state interests in commercial development.

In the cases where the Building Officials have jurisdiction, they typically do not have experience analyzing pier and wharf structures. In these instances, they have come to rely on and utilize the other design approaches that are more common in the industry.

Major ports and marine terminals in seismic regions of the world routinely design structures as described in paragraph (b). The design of these often uses a performance-based approach, with criteria and methods that are very different than those used for buildings, as provided in the Provisions.

Design approaches most commonly used are generally consistent with the practices and criteria described in the following documents:

- Working Group No. 34 of the Maritime Navigation Commission (PIANC/MarCom/WG34), 2001, *Seismic Design Guidelines for Port Structures*, A. A. Balkema, Lisse, Netherlands, 2001.
- Ferritto, J., Dickenson, S., Priestley N., Werner, S., Taylor, C., Burke D., Seelig W., and Kelly, S., 1999, *Seismic Criteria for California Marine Oil Terminals*, Vol.1 and Vol.2, Technical Report TR-2103-SHR, Naval Facilities Engineering Service Center, Port Hueneme, CA.
- Priestley, N.J.N., Frieder Siebel, Gian Michele Calvi, *Seismic Design and Retrofit of Bridges*, 1996, New York.
- *Seismic Guidelines for Ports*, by the Ports Committee of the Technical Council on Lifeline Earthquake Engineering, ASCE, edited by Stuart D. Werner, Monograph No. 12, March 1998, published by ASCE, Reston, VA.
- *Marine Oil Terminal Engineering and Maintenance Standards*, California State Lands Commission, Marine Facilities Division, May 2002.

These alternative approaches have been developed over a period of many years by working groups within the industry, and consider the historical experience and performance characteristics of these structures that are very different than building structures.

The main emphasis of the performance-based design approach is to provide criteria and methods that depend on the economic importance of a facility. Adherence to the performance criteria in the documents listed above is expected to provide as least as much inherent life-safety, and likely much more, than for buildings designed using the Provisions. However, the philosophy of these criteria is not to provide uniform margins of collapse for all structures. Among the reasons for the higher inherent level of life-safety for these structures are the following:

- These structures have relatively infrequent occupancy, with few working personnel and very low density of personnel. Most of these structures consist primarily of open area, with no enclosed building structures which can collapse onto personnel. Small control buildings on marine oil terminals or similar secondary structures are commonly designed in accordance with the local building code.
- These pier or wharf structures are typically constructed of reinforced concrete, prestressed concrete, and/or steel and are highly redundant due to the large number of piles supporting a single wharf deck unit. Tests done for the Port of Los Angeles at the University of California at San Diego have shown that very high ductilities (10 or more) can be achieved in the design of these structures using practices currently used in California ports.
- Container cranes, loading arms, and other major structures or equipment on the piers or wharves are specifically designed not to collapse in an earthquake. Typically, additional piles and structural members are incorporated into the wharf or pier specifically to support that item.
- Experience has shown that seismic “failure” of wharf structures in zones of strong seismicity is indicated not by collapse, but by economically unrepairable deformations of the piles. The wharf deck generally remains level or slightly tilting but shifted out of position. Complete failure that could cause life-safety concerns has not been known to ever occur historically due to earthquake loading.

- The performance-based criteria of the listed documents include repairability of the structure. This service level is much more stringent than collapse prevention and would provide a greater margin for life-safety.

- Lateral load design of these structures is often governed by other marine loading conditions, such as mooring or berthing.

14.3.4 Pipe racks. Free standing pipe racks supported at or below grade with framing systems that are similar in configuration to building systems should be designed to satisfy the force requirements of Sec. 5.2. Single column pipe racks that resist lateral loads should be designed as inverted pendulums. See ASCE Petro.

14.3.5 Steel storage racks. This section is intended to assure comparable results from the use of the RMI Specification, the NEHRP *Provisions*, and the IBC code approaches to rack structural design.

For many years the RMI has been working with the various committees of the model code organizations and with the Building Seismic Safety Council and its Technical Subcommittees to create seismic design provisions particularly applicable to steel storage rack structures. The 1997 RMI Specification is seen to be in concert with the needs, provisions, and design intent of the building codes and those who use and promulgate them, as well as those who engineer, manufacture, install, operate, use, and maintain rack structures. The RMI Specification, now including detailed seismic provisions, is essentially self-sufficient.

The changes proposed here are compatible and coordinated with those in the 2000 *International Building Code*.

14.3.5.2 Importance factor. Until recently, storage racks were primarily installed in low-occupancy warehouses. With the recent proliferation of warehouse-type retail stores, it has been judged necessary to address the relatively greater seismic risk that storage racks may pose to the general public, compared to more conventional retail environments. Under normal operating conditions, retail stores have a far higher occupancy load than an ordinary warehouse of a reasonable size. Failure of a storage rack system in the retail environment is much more likely to cause personal injury than a similar failure in a storage warehouse. Therefore, to provide an appropriate level of additional safety in areas open to the public, Sec 14.3.5.2 now requires that storage racks in occupancies open to the general public be designed with an importance factor equal to 1.50. Storage rack contents, while beyond the scope of the *Provisions*, pose a potentially serious threat to life should they fall from the shelves in an earthquake. Restraints should be provided to prevent the contents of rack shelving open to the general public from falling in strong ground shaking.

14.4 NONBUILDING STRUCTURES NOT SIMILAR TO BUILDINGS

Nonbuilding structures not similar to buildings exhibit behavior markedly different from that of building structures. Most of these types of structures have adopted references that address their unique structural performance and behavior. The ground motion in the *Provisions* requires appropriate translation to allow use with industry standards. Such translation is provided in this section.

14.4.2 Earth retaining structures. In order to properly develop and implement methodologies for the design of earth retaining structures, it is essential to know and understand the nature of the applied loads. Concerns have been raised concerning the design of nonyielding walls and yielding walls for bending, overturning, sliding, etc., taking into account the varying soil types, importance, and site seismicity. See Sec. 7.5.1 in the *Commentary*.

14.4.3 Stacks and chimneys. The design of stacks and chimneys to resist natural hazards is generally governed by wind design considerations. The exceptions to this general rule involve locations with high seismicity, stacks and chimneys with large elevated masses, and stacks and chimneys with unusual geometries. It is prudent to evaluate the effect of seismic loads in all but those areas with the lowest seismicity. Although not specifically required, it is recommended that the special seismic details required elsewhere in the *Provisions* be evaluated for applicability to stacks and chimneys.

Guyed steel stacks and chimneys are generally light weight. As a result, the design loads due to natural hazards are generally governed by wind. On occasion, large flares or other elevated masses located near the top may require an in-depth seismic analysis. Although Chapter 6 of Troitsky does not specifically address seismic loading, it remains an applicable methodology for resolution of seismic forces that are defined in these *Provisions*.

14.4.7 Tanks and vessels. Methods of seismic design of tanks, currently adopted by a number of industry standards, have evolved from earlier analytical work by Jacobsen, Housner, Veletsos, Haroun, and others. The procedures used to design flat bottom storage tanks and liquid containers is based on the work of Housner, Wozniak, and Mitchell. The standards for tanks and vessels have specific requirements to safeguard against catastrophic failure of the primary structure based on observed behavior in seismic events since the 1930s. Other methods of analysis using flexible shell models have been proposed but are presently beyond the scope of these *Provisions.*

These methods entail three fundamental steps:

1. The dynamic modeling of the structure and its contents. When a liquid-filled tank is subjected to ground acceleration, the lower portion of the contained liquid, identified as the impulsive component of mass W_I, acts as if it were a solid mass rigidly attached to the tank wall. As this mass accelerates, it exerts a horizontal force, P_I, against the wall that is directly proportional to the maximum acceleration of the tank base. This force is superimposed on the inertia force of the accelerating wall itself, P_w. Under the influence of the same ground acceleration, the upper portion of the contained liquid responds as if it were a solid mass flexibly attached to the tank wall. This portion, which oscillates at its own natural frequency, is identified as the convective component W_c, and exerts a force P_c on the wall. The convective component oscillations are characterized by the phenomenon of sloshing whereby the liquid surface rises above the static level on one side of the tank, and drops below that level on the other.
2. The determination of the frequency of vibration, w_I, of the tank structure and the impulsive component; and the natural frequency of oscillation (sloshing), w_c, of the convective component.
3. The selection of the design response spectrum. The response spectrum may be site-specific or it may be constructed deterministically on the basis of seismic coefficients given in national codes and standards. Once the design response spectrum is constructed, the spectral accelerations corresponding to w_I and w_c are obtained and are used to calculate the dynamic forces P_I, P_w, and P_c.

Detailed guidelines for the seismic design of circular tanks, incorporating these concepts to varying degrees, have been the province of at least four industry standards: AWWA D100 for welded steel tanks (since 1964); API 650 for petroleum storage tanks; AWWA D110 for prestressed, wire-wrapped tanks (since 1986); and AWWA D115 for prestressed concrete tanks stressed with tendons (since 1995). In addition, API 650 and API 620 contain provisions for petroleum, petrochemical, and cryogenic storage tanks. The detail and rigor of analysis employed by these standards have evolved from a semi-static approach in the early editions to a more rigorous approach at the present, reflecting the need to factor in the dynamic properties of these structures.

The requirements in Sec 14.4.7 are intended to link the latest procedures for determining design level seismic loads with the allowable stress design procedures based on the methods in these Provisions. These requirements, which in many cases identify specific substitutions to be made in the design equations of the national standards, will assist users of the *Provisions* in making consistent interpretations.

ACI has published a document, ACI 350.3-01 titled "*Seismic Design of Liquid-Containing Concrete Structures.*" This document, which covers all types of concrete tanks (prestressed and non-prestressed, circular and rectilinear), has provisions made consistent with the seismic guidelines of the 2000 *Provisions*. This ACI document serves as both a practical "how-to" loading reference and a guide to supplement application of Chapter 21 "Special Provisions for Seismic Design" of ACI 318.

14.4.7.1 Design basis. Two important tasks of TS 13 were (a) to partially expand the coverage of nonbuilding structures in the *Provisions*; and (b) to provide comprehensive cross-references to all the applicable industry standards. It is hoped that this endeavor will bring about a standardization and consistency of design practices for the benefit of both the practicing engineer and the public at large.

In the case of the seismic design of nonbuilding structures, standardization requires adjustments to industry standards to minimize existing inconsistencies among them. However, the standardization process should recognize that structures designed and built over the years in accordance with industry standards have performed well in earthquakes of varying severity.

Of the inconsistencies among industry standards, the ones most important to seismic design relate to the base shear equation. The traditional base shear takes the following form:

$$V = \frac{ZIS}{R_w} CW$$

An examination of those terms as used in the different references reveals the following:

- *ZS*: The "seismic zone coefficient," *Z*, has been rather consistent among all the standards by virtue of the fact that it has traditionally been obtained from the seismic zone designations and maps in the model building codes.

  On the other hand, the "soil profile coefficient," *S*, does vary from one standard to another. In some standards these two terms are combined.

- *I*: The importance factor, *I*, has also varied from one standard to another, but this variation is unavoidable and understandable owing to the multitude of uses and degrees of importance of liquid-containing structures.

- *C*: The coefficient *C* represents the dynamic amplification factor that defines the shape of the design response spectrum for any given maximum ground acceleration. Since coefficient *C* is primarily a function of the frequency of vibration, inconsistencies in its derivation from one standard to another stem from at least two sources: differences in the equations for the determination of the natural frequency of vibration, and differences in the equation for the coefficient itself. (For example, for the shell/impulsive liquid component of lateral force, the steel tank standards use a constant design spectral acceleration (namely, a constant *C*) that is independent of the "impulsive" period *T*.) In addition, the value of *C* will vary depending on the damping ratio assumed for the vibrating structure (usually between 2 percent and 7 percent of critical).

  Where a site-specific response spectrum is available, calculation of the coefficient *C* is not necessary – except in the case of the convective component (coefficient C_c) which is assumed to oscillate with 0.5 percent of critical damping, and whose period of oscillation is usually high (greater than 2.5 sec). Since site-specific spectra are usually constructed for high damping values (3 percent to 7 percent of critical); and since the site-specific spectral profile may not be well-defined in the high-period range, an equation for C_c applicable to a 0.5 percent damping ratio is necessary in order to calculate the convective component of the seismic force.

- R_w: The "response modification factor," R_w, is perhaps the most difficult to quantify, for a number of reasons. While R_w is a compound coefficient that is supposed to reflect the ductility, energy-dissipating capacity, and redundancy of the structure, it is also influenced by serviceability considerations, particularly in the case of liquid-containing structures.

In the *Provisions* the base shear equation for most structures has been reduced to $V = C_sW$, where the seismic response coefficient, C_s, replaces the product $\frac{ZSC}{R_w}$. C_s is determined from the design spectral response acceleration parameters S_{DS} and S_{D1} (at short periods and at a period of 1 sec, respectively) which, in turn, are obtained from the mapped MCE spectral accelerations S_s and S_1 obtained from the seismic maps. As in the case of the prevailing industry standards, where a site-specific response spectrum is available, C_s is replaced by the actual spectral values of that spectrum.

As part of its task, TS 13 has introduced a number of provisions, in the form of bridging equations, each designed to provide a means of properly applying the design criteria of a particular industry standard in the context of these *Provisions*. These provisions are outlined below and are identified with particular types of liquid-containing structures and the corresponding standards. Underlying all these provisions is the understanding that the calculation of the periods of vibration of the impulsive and convective components is left up to the industry standards. Defining the detailed resistance and allowable stresses of the structural elements for each industry structure has also been left to the approved standard except in instances where additional information has led to additional requirements.

It is intended that, as the relevant national standards are updated to conform to these *Provisions*, the "bridging" equations of Sec. 14.4.7.6, 14.4.7.7, and 14.4.7.9 will be eliminated.

14.4.7.2 Strength and ductility. As is the case for building structures, ductility and redundancy in the lateral support systems for tanks and vessels are desirable and necessary for good seismic performance. Tanks and vessels are not highly redundant structural systems and, therefore, ductile materials and well-designed connection details are needed to increase the capacity of the vessel to absorb more energy without failure. The critical performance of many tanks and vessels is governed by shell stability requirements rather than by yielding of the structural elements. For example, contrary to building structures, ductile stretching of the anchor bolts is a desirable energy absorption component when tanks and vessels are anchored. The performance of cross-braced towers is highly dependent on the ability of the horizontal compression struts and connection details to fully develop the tension yielding in the rods. In such cases, it is also important to assure that the rods stretch rather than fail prematurely in the threaded portion of the connection and that the connection of the rod to the column does not fail prior to yielding of the rod.

14.4.7.3 Flexibility of piping attachments. The performance of piping connections under seismic deformations is one of the primary weaknesses observed in recent seismic events. Tank leakage and damage occurs when the piping connections cannot accommodate the movements the tank experiences during the a seismic event. Unlike the connection details used by many piping designers, which connections impart mechanical loading to the tank shell, piping systems in seismic areas should be designed in such a manner as to impose only negligible mechanical loads on the tank connection for the values shown in Table 14.4-1.

In addition, interconnected equipment, walkways, and bridging between multiple tanks must be designed to resist the loads and displacements imposed by seismic forces. Unless multiple tanks are founded on a single rigid foundation, walkways, piping, bridges, and other connecting structures must be designed to allow for the calculated differential movements between connected structures due to seismic loading assuming the tanks and vessels respond out of phase.

14.4.7.4 Anchorage. Many steel tanks can be designed without anchors by using the annular plate procedures given in the national standards. Tanks that must be anchored because of overturning potential could be susceptible to shell tearing if not properly designed. Ideally, the proper anchorage design will provide both a shell attachment and embedment detail that will yield the bolt without tearing the shell or pulling the bolt out the foundation. Properly designed anchored tanks retain greater reserve strength to resist seismic overload than do unanchored tanks.

Premature failure of anchor bolts has been observed where the bolt and attachment are not properly aligned (that is, the anchor nut or washer does not bear evenly on the attachment). Additional bending stresses in threaded areas may cause the anchor to fail before yielding.

14.4.7.5 Ground-supported storage tanks for liquids

14.4.7.5.1 Seismic forces. The response of ground storage tanks to earthquakes is well documented by Housner, Mitchell and Wozniak, Veletsos, and others. Unlike building structures, the structural response is strongly influenced by the fluid-structure interaction. Fluid-structure interaction forces are categorized as sloshing (convective mass) and rigid (impulsive mass) forces. The proportion of these forces depends on the geometry (height-to-diameter ratio) of the tank. API 650, API 620, AWWA D100, AWWA D110, AWWA D115, and ACI 350.3 provide the necessary data to determine the relative masses and moments for each of these contributions.

The *Provisions* stipulate that these structures shall be designed in accordance with the prevailing approved industry standards, with the exception of the height of the sloshing wave, d_s, which is to be calculated using Eq. 14.4-9 of these *Provisions*.

$$\delta_s = 0.5DIS_{ac}$$

This equation utilizes a spectral response coefficient $S_{ac} = \frac{1.5S_{D1}}{T_c}$ for $T_c < 4.0$ sec., and $S_{ac} = \frac{6S_{D1}}{T_c^2}$ for $T_c > 4.0$ sec. The first definition of S_a represents the constant-velocity region of the response spectrum and the second the constant-displacement region of the response spectrum, both at 0.5 percent damping. In practical terms, the latter is the more commonly used definition since most tanks have a fundamental period of liquid oscillation (sloshing wave period) greater than 4.0 sec.

Small diameter tanks and vessels are more susceptible to overturning and vertical buckling. As a general rule, the greater the ratio of H/D, the lower the resistance is to vertical buckling. When $H/D > 2$, the overturning begins to approach “rigid mass” behavior (the sloshing mass is small). Large diameter tanks may be governed by additional hydrodynamic hoop stresses in the middle regions of the shell.

The impulsive period (the natural period of the tank components and the impulsive component of the liquid) is typically in the 0.25 to 0.6 second range. Many methods are available for calculating the impulsive period. The Veletsos flexible-shell method is commonly used by many tank designers. (For example, see “Seismic Effects in Flexible Liquid Storage Tanks” by A. S. Veletsos.)

14.4.7.5.2 Distribution of hydrodynamic and inertia forces. Most of the methods contained in the industry standards for tanks define reaction loads at the base of the shell and foundation interface. Many of the standards do not give specific guidance for determining the distribution of the loads on the shell as a function of height. The design professional may find the additional information contained in ACI 350.3 helpful.

The overturning moment at the base of the shell as defined in the industry standards is only the portion of the moment that is transferred to the shell. It is important for the design professional to realize that the total overturning moment must also include the variation in bottom pressure. This is important when designing pile caps, slabs, or other support elements that must resist the total overturning moment. See Wozniak or TID 7024 for further information.

14.4.7.5.3 Freeboard. Performance of ground storage tanks in past earthquakes has indicated that sloshing of the contents can cause leakage and damage to the roof and internal components. While the effect of sloshing often involves only the cost and inconvenience of making repairs, rather than catastrophic failure, even this limited damage can be prevented or significantly mitigated when the following items are considered:

1. Effective masses and hydro-dynamic forces in the container.
2. Impulsive and pressure loads at
   a. Sloshing zone (that is, the upper shell and edge of the roof system),
   b. Internal supports (roof support columns, tray-supports, etc.), and
   c. Equipment (distribution rings, access tubes, pump wells, risers, etc.).
3. Freeboard (which depends on the sloshing wave height).

A minimum freeboard of $0.7\delta_s$ is recommended for economic considerations but is not required.

Tanks and vessels storing biologically or environmentally benign materials do not typically require freeboard to protect the public health and safety. However, providing freeboard in areas of frequent seismic occurrence for vessels normally operated at or near top capacity may lessen damage (and the cost of subsequent repairs) to the roof and upper container.

The estimate given in the Provision Sec. 14.4.7.5.3 is based on the seismic design event as defined by the Provisions. Users of the Provisions may estimate slosh heights different from those recommended in the national standards.

If sloshing is restricted because the freeboard provided is less than the computed sloshing height, δ_s, the sloshing liquid will impinge on the roof in the vicinity of the roof-to-wall joint, subjecting it to a hydrodynamic force. This force may be approximated by considering the sloshing wave as a hypothetical static liquid column having a height, δ_s. The pressure exerted on any point along the roof at a distance y_s above the at-rest surface of the stored liquid, may be assumed equal to the hydrostatic pressure exerted by the hypothetical liquid column at a distance $\delta_s - y_s$ from the top of that column

Another effect of a less-than-full freeboard is that the restricted convective (sloshing) mass "converts" into an impulsive mass thus increasing the impulsive forces. This effect should be taken account in the tank design. Preferably, sufficient freeboard should be provided whenever possible to accommodate the full sloshing height.

14.4.7.5.6 Sliding resistance. Steel ground-supported tanks full of product have not been found to slide off foundations. A few unanchored, empty tanks have moved laterally during earthquake ground shaking. In most cases, these tanks may be returned to their proper locations. Resistance to sliding is obtained from the frictional resistance between the steel bottom and the sand cushion on which bottoms are placed. Because tank bottoms usually are crowned upward toward the tank center and are constructed of overlapping, fillet-welded, individual steel plates (resulting in a rough bottom), it is reasonably conservative to take the ultimate coefficient of friction as 0.70 (U.S. Nuclear Regulatory Commission, 1989, pg. A-50) and, therefore, a value of tan 30^o (= 0.577) is used. The vertical weight of the tank and contents as reduced by the component of vertical acceleration provides the net vertical load. An orthogonal combination of vertical and horizontal seismic forces following the procedure in Sec. 5.2 may be used.

14.4.7.5.7 Local shear transfer. The transfer of seismic shear from the roof to the shell and from the shell to the base is accomplished by a combination of membrane shear and radial shear in the wall of the tank. For steel tanks, the radial shear is very small and is usually neglected; thus, the shear is assumed to be carried totally by membrane shear. For concrete walls and shells, which have a greater radial shear stiffness, the shear transfer may be shared. The user is referred to the ACI 350 commentary for further discussion.

14.4.7.5.8 Pressure stability. Internal pressure may increase the critical buckling capacity of a shell. Provision to include pressure stability in determining the buckling resistance of the shell for overturning loads is included in AWWA D100. Recent testing on conical and cylindrical shells with internal pressure yielded a design methodology for resisting permanent loads in addition to temporary wind and seismic loads. See Miller et al., 1997.

14.4.7.5.9 Shell support. Anchored steel tanks should be shimmed and grouted to provide proper support for the shell and to reduce impact on the anchor bolts under reversible loads. The high bearing pressures on the toe of the tank shell may cause inelastic deformations in compressible material (such as fiberboard), creating a gap between the anchor and the attachment. As the load reverses, the bolt is no longer snug and an impact of the attachment on the anchor can occur. Grout is a structural element and should be installed and inspected as if it is an important part of the vertical- and lateral-force-resisting system.

14.4.7.5.10 Repair, alteration, or reconstruction. During their service life, storage tanks are frequently repaired, modified or relocated. Repairs or often related to corrosion, improper operation, or overload from wind or seismic events. Modifications are made for changes in service, updates to safety equipment for changing regulations, installation of additional process piping connections. It is imperative these repairs and modifications are properly designed and implemented to maintain the structural integrity of the tank or vessel for seismic loads as well as the design operating loads.

The petroleum steel tank industry has developed specific guidelines in API 653 that are statutory requirements in some states. It is the intent of TS 13 that the provisions of API 653 also be applied to other liquid storage tanks (water, wastewater, chemical, etc.) as it relates to repairs, modifications or relocation that affects the pressure boundary or lateral force resisting system of the tank or vessel.

14.4.7.6 Water and water treatment structures

14.4.7.6.1 Welded steel. The AWWA design requirements for ground-supported steel water storage structures are based on an allowable stress method that utilizes an effective mass procedure considering two response modes for the tank and its contents:

1. The high-frequency amplified response to seismic motion of the tank shell, roof, and impulsive mass (that portion of liquid content of the tank that moves in unison with the shell), and
2. The low-frequency amplified response of the convective mass (that portion of the liquid contents in the fundamental sloshing mode).

The two-part AWWA equation incorporates the above modes, appropriate damping, site amplification, allowable stress response modification, and zone coefficients. In practice, the typical ground storage tank and impulsive contents will have a natural period, T, of 0.1 to 0.3 sec. The sloshing period typically will be greater than 1 sec (usually 3 to 5 seconds depending on tank geometry). Thus, the substitution in the *Provisions* uses a short- and long-period response as it applies to the appropriate constituent term in the AWWA equations.

14.4.7.6.2 Bolted steel. The AWWA Steel Tank Committee is responsible for the content of both the AWWA D100 and D103 and have established equivalent load and design criteria for earthquake design of welded and bolted steel tanks.

14.4.7.7 Petrochemical and industrial liquids

14.4.7.7.1 Welded steel. The American Petroleum Institute (API) also uses an allowable stress design procedure and the API equation has incorporated an R_w factor into the equations directly.

The most common damage to tanks observed during past earthquakes include:

- Buckling of the tank shell near the base due to excessive axial membrane forces. This buckling damage is usually evident as “elephant foot” buckles a short distance above the base, or as diamond shaped buckles in the lower ring. Buckling of the upper ring has also been observed.
- Damage to the roof due to impingement on the underside of the roof of sloshing liquid with insufficient freeboard.
- Failure of piping or other attachments that are overly restrained..
- Foundation failures.

The performance of floating roofs during earthquakes has been good, with damage usually confined to the rim seals, gage poles, and ladders. Similarly the performance of open tops with top wind girder stiffeners designed per API 650 has been good.

14.4.7.9 Elevated tanks for liquids and granular materials. There are three basic lateral-load resisting systems for elevated water tanks that are defined by their support structure. Multi-leg braced steel tanks (trussed towers), small diameter single-pedestal steel tanks (cantilever columns), and large diameter single-pedestal tanks of steel or concrete construction (load-bearing shear walls). Unbraced multi-leg tanks are not commonly built. Behavior, redundancy, and resistance to overload of these types of tanks are not the same. Multi-leg and small diameter pedestal have higher fundamental periods (typically over 2-sec) than the shear wall type tanks (typically under 2-sec). Lateral load failure mechanism is usually by bracing failure for multi-leg tanks, compression buckling of small diameter steel tanks, compression or shear buckling of large diameter steel tanks, and shear failure of large diameter concrete tanks. In order to utilize the full strength of these structures adequate connection, welding, and reinforcement details must be provided. The R-factor used with elevated tanks is typically less than that for comparable lateral load-resisting systems for other purposes in order to provide a greater margin of safety.

14.4.7.9.3 Transfer of lateral forces into support tower. The lateral transfer of load for tanks and vessels siting on grillage or support beams should consider the relative stiffness of the support beams and the shear transfer at the base of the shell, which is not typically uniform around the base of the tank. In addition, when tanks and vessels are supported on discrete points on grillage or beams, it is common for the vertical loads to vary due to settlements or variations in construction. This variation in load should be considered when analyzing the combined vertical and horizontal loads.

14.4.7.9.4 Evaluation of structures sensitive to buckling failure. Nonbuilding structures that have low or negligible structural redundancy for lateral loads need to be evaluated for a critical level of performance to provide sufficient margin against premature failure. Reserve strength for loads beyond the design loads can be limited. Tanks and vessels supported on shell skirts or pedestals that are governed by buckling are examples of structures that need to be evaluated at this critical condition. Such structures include single pedestal water towers, process vessels, and other single member towers.

The additional evaluation is based on a scaled maximum considered earthquake. This critical earthquake acceleration is defined as the design spectral response acceleration, S_a, which includes site factors. The I/R coefficient is taken as 1.0 for this critical check. The structural capacity of the shell is taken as the critical buckling strength (that is, the factor of safety is 1.0). Vertical or orthogonal earthquake combination need not be made for this critical evaluation since the probability of critical peak values occurring simultaneously is very low.

14.4.7.9.6 Concrete pedestal (composite) tanks. A composite elevated water-storage tank is a structure comprising a welded steel tank for watertight containment, a single pedestal concrete support structure, foundation, and accessories. Lateral load-resisting system is that of a load-bearing concrete shear wall. Seismic provisions in ATC 371R-98 are based on ASCE 7-95, which used NEHRP 1994 as the source document. Seismic provisions in the proposed AWWA standard being prepared by committee D170 are based on ASCE 7-98, which used NEHRP 1997 as the source document.

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Appendix to Chapter 14

OTHER NONBUILDING STRUCTURES

PREFACE: The following sections were originally intended to be part of the Nonbuilding Structures Chapter of this Commentary. The *Provisions* Update Committee felt that given the complexity of the issues, the varied nature of the resource documents, and the lack of supporting consensus resource documents, time did not allow a sufficient review of the proposed sections required for inclusion into the main body of the chapter.

The Nonbuilding Structures Technical Subcommittee, however, expressed that what is presented herein represents the current industry accepted design practice within the engineering community that specializes in these types of nonbuilding structures.

The *Commentary* sections are included here so that the design community specializing in these nonbuilding structures can have the opportunity to gain a familiarity with the concepts, update their standards, and send comments on this appendix to the BSSC.

It is hoped that the various consensus design standards will be updated to include the design and construction methodology presented in this Appendix. It is also hoped that industry standards that are currently not consensus documents will endeavor to move their standards through the consensus process facilitating building code inclusion.

A14.1 GENERAL

Agrawal P. K., and J. M. Kramer, Analysis of Transmission Structures and Substation Structures and Equipment for Seismic Loading, Sargent & Lundy Transmission and Substation Conference, December 2, 1976. (Agrawal)

American Society of Civil Engineers (ASCE):

ANSI/ASCE 10, *Design of Latticed Transmission Structures*, 1997. (ASCE 10)

ASCE Manual 72, *Tubular Pole Design Standard*, 1991 (ASCE 72).

ASCE Manual 74, *Guidelines for Electrical Transmission Line Structural Loading*, 2000. (ASCE 74).

ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, 1995. (ASCE 7).

ASCE Manual 91, *The Design of Guyed Electrical Transmission Structur*es, 1997. (ASCE 91)

Substation Structure Design Guide, 2000. (ASCE Substation)

Li, H.-N., S. Wang, M. Lu, and Q. Wang, “Aseismic Calculations for Transmission Towers,” *ASCE Technical Council on Lifeline Earthquake Engineering*, *Monograph No. 4*, August 1991. (ASCE Li)

Steinhardt, O. W., “Low Cost Seismic Strengthening of Power Systems,” *Journal of The Technical Councils of ASCE*, April 1981. (ASCE Steinhardt)

Amiri, G. G. and G. G. McClure, “Seismic Response to Tall Guyed Telecommunication Towers,” Paper No. 1982, Eleventh World Conference on Earthquake Engineering, Elsevier Science Ltd., 1996. (Amiri)

Australian Standards:

Australian Standard 3995, *Standard Design of Steel Lattice Towers and Masts*, 1994. (AS 3995)

Canadian Standards Association (CSA):

Antennas, Towers, and Masts, 1994. (CSA S37)

Earthquake Engineering Research Institute (EERI):

Li, H.-N., L. E. Suarez, and M. P. Singh, “Seismic Effects on High-Voltage Transmission Tower and Cable Systems,” Fifth U.S. National Conference on Earthquake Engineering, 1994. (EERI Li)

Federal Emergency Management Agency (FEMA):

Earthquake Resistant Construction of Electric Transmission and Telecommunication Facilities Serving the Federal Government, FEMA Report No. 202, September 1990. (FEMA 202)

Galvez, C. A., and G. G. McClure, “A Simplified Method for Aseismic Design of Self-Supporting Latticed Telecommunication Towers,” Seventh Canadian Conference on Earthquake Engineering, Montreal, 1995. (Galvez)

Institute of Electrical and Electronics Engineers (IEEE):

National Electrical Safety Code, ANSI C2, New Jersey, 1997. (NESC)

IEEE Standard 693, *Recommended Practices for Seismic Design of Substations*, Power Engineering Society, Piscataway, New Jersey, 1997 (IEEE 693).

IEEE Standard 751, *Trial-Use Design Guide for Wood Transmission Structures*, Power Engineering Society, Piscataway, New Jersey, 1991. (IEEE 751)

Long, L.W., Analysis of Seismic Effects on Transmission Structures, IEEE Paper T 73 326-6, April 1973. (IEEE Long).

Lum, W. B., N. N. Nielson, R. Koyanagi, and A. N. L. Chui, “Damage Survey of the Kasiki, Hawaii Earthquake of November 16, 1993,” Earthquake Spectra, November 1984. (Lum)

Lyver, T. D., W. H. Mueller, and L. Kempner, Jr., *Response Modification Factor, R_w, for Transmission Towers*, Research Report, Portland State University, Portland, Oregon, 1996. (Lyver)

National Center for Earthquake Engineering Research (NCEER):

The Hanshin-Awaji Earthquake of January 17, 1995—Performance of Lifelines, National Center for Earthquake Engineering Research, Technical Report NCEER-95-0015, State University of New York at Buffalo, November 3, 1995. (NCEER 95-0015)

Rural Electrical Administration (REA):

Bulletin 1724E-200, *Design Manual for High Voltage Transmission Lines*, 1992. (REA 1724).

Bulletin 65-1, *Design Guide for Rural Substations*, 1978 (REA 65-1).

Bulletin 160-2, *Mechanical Design Manual for Overhead Distribution Lines*, 1982. (REA 160)

Telecommunications Industry Association (TIA):

TIA/EIA 222F, *Structural Standards for Steel Antenna Towers and Antenna Supporting Structures*, 1996. (TIA 222)

A14.2.1 Buried Structures. This section was placed in the Appendix to Chapter 14 for the following reasons:

1. The material may serve as a starting point for continued development.
2. The comments stimulated by consideration of this section will provide valuable input so that this section may be further developed and then incorporated in the *Provisions* in the future.
3. It was determined by TS 13 and the *Provisions* Update Committee that it would be premature to incorporate this section into the *Provisions* for the 2000 edition.
4. Accepted industry standards are in the process of incorporating seismic design methodology reflecting the *Provisions*.

It is not the intent of the *Provisions* Update Committee to discourage incorporation of this section into a building code or to minimize the importance of this section. Placing this section in the appendix indicates only that this section requires further development.

Seismic forces on buried structures may include forces due to: soil displacement, seismic lateral earth pressure, buoyant forces related to liquefaction, permanent ground displacements from slope instability, lateral spread movement, fault movement, or dynamic ground displacement caused by dynamic strains from wave propagation. Identification of appropriate seismic loading conditions is dependent upon subsurface soil conditions and the configuration of the buried structure. Conditions related to permanent ground movement can often be avoided by careful site selection for isolated buried structures such as tanks and vaults. Relocation is often impractical for long buried structures such as tunnels and pipelines.

Wave propagation strains are a significant seismic force condition for buried structures if local site conditions (for instance, deep surface soil deposits with low shear wave velocities) can support the propagation of large amplitude seismic waves. Wave propagation strains tend to be most pronounced at the junctions of dissimilar buried structures (such as a pipeline connecting with a building) or at the interfaces of different geologic materials (such as a pipeline passing from rock to soft soil).

Loading conditions related to liquefaction require detailed subsurface information that can be used to assess the potential for liquefaction and, for long buried structures, the length of structure exposed to liquefaction effects. In addition, the assessment of liquefaction requires specifying an earthquake magnitude that is consistent with the definition of ground shaking. It is recommended that one refer to Chapter 7 of this C*ommentary* for additional guidance in determining liquefaction potential and seismic magnitude. Providing detailed structural design procedures in this area is beyond the scope of this document.

Loading conditions related to lateral spread movement and slope instability can be defined in terms of lateral soil pressures or prescribed ground displacements. In both cases, sufficient subsurface investigation in the vicinity of the buried structure is necessary to estimate the amount of movement, the direction of movement relative to the buried structure, and the portion of the buried structure exposed to the loading conditions. Definition of lateral spread loading conditions requires special geotechnical expertise and specific procedures in this area are beyond the scope of this document.

Defining the loading conditions for fault movement requires specific location of the fault and an estimate of the earthquake magnitude on the fault that is consistent with the ground shaking hazard in the *Provisions*. Identification of the fault location should be based on past earthquake movements, trenching studies, information from boring logs, or other accepted fault identification techniques. Defining fault movement conditions requires special seismological expertise. Additional guidance can be found in the Chapter 7 of this *Commentary*.

It may not be practically feasible to design a buried structure to resist the effects of permanent ground deformation. Alternative approaches in such cases may include relocation to avoid the condition, ground improvements to reduce the loads, or implementing special procedures or design features to minimize the impact of damage (such as remote controlled or automatic isolation valves that provide the ability to rapidly bypass damage or post-earthquake procedures to expedite repair). The goal of providing procedures or design features as an alternative to designing for the seismic loadings is to change the hazard and function classification of the buried structure such that it is not classified as Seismic Use Group II or III.

It is recommended that one refer to Chapter 7 of this C*ommentary* for additional guidance in determining liquefaction potential and determining seismic magnitude.

Buried structures are subgrade structures such as tanks, tunnels, and pipes. Buried structures that are designated as Seismic Use Group II or III, or are of such a size or length to warrant special seismic design as determined by the registered design professional, must be identified in the geotechnical report.

Buried structures must be designed to resist minimum seismic lateral forces determined from a substantiated analysis using approved procedures. Flexible couplings must be provided for buried structures requiring special seismic considerations where changes in the support system, configuration, or soil condition occur.

The requirement for and value of flexible couplings should be determined by the "properly substantiated analysis and approved procedures." It is assumed that the need for flexible couplings refers to buried piping or conduits. The prior wording of Sec. A14.2.3 was far too broad in requiring flexible couplings where changes in the support system, configuration or soil condition occur. These broad requirements could result in flexible couplings installed at locations where permanent ground displacement is expected or at transitions between aboveground supported pipe and buried pipe. As currently available flexible couplings are not generally designed to match the ultimate strength properties of the piping or conduit, the prior requirements potentially introduce a weak point in the piping or conduit system. The original focus of the prior requirements was penetrations of buried service lines into a building or other structure. Properly designed flexible couplings can be an effective means to limit forces at connections to buried structures. However, special care is needed to make sure the design loads and displacements are adequately specified. There are several other alternative to providing sufficient flexibility at connections to buried structures that are more robust in terms of margin above their design levels.