4.2 Tier 2 Analysis

4.2.1 General

Four analysis procedures are provided in this section:

- Linear Static Procedure (LSP),
- Linear Dynamic Procedure (LDP),
- Special Procedure, and
- Procedure for Nonstructural Components.

All building structures, except unreinforced masonry (URM) bearing wall buildings with flexible diaphragms, shall be evaluated by either the Linear Static Procedure (LSP) of Section 4.2.2.1 or the Linear Dynamic Procedure (LDP) of Section 4.2.2.2. The acceptability criteria for both the LSP and LDP are provided in Section 4.2.4. Out-of-plane forces on walls shall be calculated in accordance with Section 4.2.5.

If original design calculations are available, the results may be used; an appropriate scaling factor, however, to relate the original design base shear to the pseudo lateral force of this Handbook shall be applied.

Unreinforced masonry (URM) bearing wall buildings with flexible diaphragms shall be evaluated in accordance with the requirements of the Special Procedure defined in Section 4.2.6 directly.

The demands on nonstructural components shall be calculated in accordance with Section 4.2.7. These demands shall be compared with the acceptance criteria included in the Procedures for Nonstructural Components in Section 4.8.

4.2.2 Analysis Procedures for LSP & LDP

The Linear Static or Linear Dynamic Procedure shall be performed as required by the Procedures of Section 4.3 through 4.6.

The Linear Dynamic Procedure shall be used for:

- buildings taller than 100 ft,
- buildings with mass, stiffness, or geometric irregularities as specified in Sections 4.3.2.2, 4.3.2.3, and 4.3.2.5.

4.2.2.1 Linear Static Procedure (LSP)

The Linear Static Procedure shall be performed as follows:

- A mathematical building model shall be developed in accordance with Section 4.2.3;
- The pseudo lateral force shall be calculated in accordance with Section 4.2.2.1.1;
- The lateral forces shall be distributed vertically in accordance with Section 4.2.2.1.2;
- The building or component forces and displacements using linear, elastic analysis methods shall be calculated;
- Diaphragm forces shall be calculated in accordance with Section 4.2.2.1.3, if required.
- The component actions shall be compared with the acceptance criteria of Section 4.2.4.5.

Commentary:

In the Linear Static Procedure, the building is modeled with linearly-elastic stiffness and equivalent viscous damping that approximate values expected for loading to near the yield point. Design earthquake demands for the Linear Static Procedure are represented by static lateral forces whose sum is equal to the pseudo lateral force defined by Equation (3-1). The magnitude of the pseudo lateral force has been selected with the intention that when it is applied to the linearly elastic model of the building it will result in design displacement amplitudes approximating maximum displacements that are expected during the design earthquake. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations to those expected during the design earthquake. If the building responds inelastically to the design earthquake, as will commonly be the case, the calculated internal forces will exceed those that would develop in the yielding building.

The component forces in yielding structures calculated from linear analysis represent the total (linear and nonlinear) deformation of the component. The acceptability criteria reconciles the calculated forces with component capacities using component ductility related factors, m. The linear procedures linear procedures represent a rough approximation of the non-linear behavior of the actual structure and ignores redistribution of forces and other non-linear effects. In certain cases alternative acceptable approaches are presented that may provide wide variation in the results. This is expected, considering the limitations of the linear analysis procedures.

4.2.2.1.1 Pseudo Lateral Force

The pseudo lateral force applied in a Linear Static Procedure shall be calculated in accordance with Section 3.5.2.1.

The fundamental period of vibration of the building for use in Equation (3-1) shall be calculated as follows:

- For a one-story building with a single span flexible diaphragm, in accordance with Equation (4-1).

$$T = (0.1\Delta_w + 0.078\Delta_d)^{0.5} \quad (4\text{-}1)$$

where:

Δ_w and Δ_d are in-plane wall and diaphragm displacements in inches due to a lateral force equal to the weight tributary to diaphragm in the direction under consideration, or

- For multiple-span diaphragms, a lateral force equal to the weight tributary to the diaphragm span under consideration shall be applied to each span of the diaphragm to calculate a separate period for each diaphragm span. The period that maximizes the pseudo lateral force shall be used for design of all walls and diaphragm spans in the building, or
- Based on an eigenvalue (dynamic) analysis of the mathematical model of the building, or
- In accordance with Section 3.5.2.4.

4.2.2.1.2 Vertical Distribution of Seismic Forces

The pseudo lateral force calculated in accordance with Section 4.2.2.1.1 shall be distributed vertically in accordance with Equations (4-2) and (4-3).

$$F_x = C_{vx}V \qquad (4\text{-}2)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (4\text{-}3)$$

where:

k = 1.0 for $T \leq 0.5$ second,
= 2.0 for $T > 2.5$ seconds,
Linear interpolation shall be used for intermediate values of k;
C_{vx} = Vertical distribution factor,
V = Pseudo lateral force (Section 4.2.2.1.1),
w_i = Portion of the total building weight W located on or assigned to floor level i,
w_x = Portion of the total building weight W located on or assigned to floor level x,
h_i = Height (ft) from the base to floor level i,
h_x = Height (ft) from the base to floor level x.

4.2.2.1.3 Floor Diaphragms

The effects of inertial forces, calculated in accordance with Equation (4-4), developed at the level under consideration and horizontal forces resulting from offsets in, or changes in stiffness of, the vertical lateral-force-resisting elements above and below the diaphragm shall be considered in the analyses. Forces resulting from offsets in, or changes in stiffness of, the vertical lateral-force-resisting elements shall be equal to the elastic forces without reduction, unless smaller forces can be justified by rational analysis.

$$F_{px} = \frac{1}{C} F_i \frac{w_x}{\sum_{i=1}^{n} w_i} \qquad (4\text{-}4)$$

where:

F_{px} = Total diaphragm force at level x,
F_i = Lateral load applied at floor level i defined by Equation (4-2),
w_i = Portion of the total building weight W located or assigned to floor level i,
w_x = Portion of the total building weight W located or assigned to floor level x,
C = Modification Factor defined in Table 3-4.

The lateral forces on flexible diaphragms shall be distributed along the span of the diaphragm, based on the distribution of mass and displaced shape of the diaphragm.

4.2.2.1.4 Determination of Deformations

Structural deformations and story drifts shall be calculated using lateral forces in accordance with Equations (3-1), (4-2) and (4-4).

4.2.2.2 Linear Dynamic Procedure (LDP)

The Linear Dynamic Procedure shall be performed as follows:

- Develop a mathematical building model in accordance with Section 4.2.3;
- Develop a response spectrum for the site in accordance with Section 4.2.2.2.2;
- Perform a response spectrum analysis of the building;
- Modify the actions and deformations in accordance with Section 4.2.2.2.3;
- Compute diaphragm forces in accordance with Section 4.2.2.2.4, if required;
- Compute the component actions in accordance with Section 4.2.4.3;
- Compare the component actions with the acceptance criteria of Section 4.2.4.5.

Modal responses shall be combined using the SRSS (square root sum of the squares) or CQC (complete quadratic combination) method to estimate the response quantities. The CQC shall be used when modal periods associated with motion in a given direction are within 25%. The number of modes considered in the response spectrum analysis shall be sufficient to capture at least 90% of the participating mass of the building in each of the building's principal horizontal axes.

Multidirectional excitation effects shall be considered in accordance with Section 4.2.3.5. Alternatively, the SRSS method may be used to combine multidirectional effects. The CQC method shall not be used for combination of multidirectional effects.

4.2.2.2.2 Ground Motion Characterization

The seismic ground motions shall be characterized for use in the LDP by developing:

- A mapped response spectrum in accordance with Section 3.5.2.3.1, or
- A site-specific response spectrum in accordance with Section 3.5.2.3.2.

4.2.2.2.3 Modification of Demands

With the exception of diaphragm actions and deformations, all actions and deformations calculated using the Linear Dynamic Procedure shall be multiplied by the modification factor, C, defined in Table 3-4.

Commentary:

Note that, in contrast to NEHRP and the UBC, the results of the response spectrum analysis are not scaled to the pseudo lateral force of the LSP. Such scaling is unnecessary since the LSP is based on the use of actual spectral acceleration values from proper response spectra and is not reduced by R values used in traditional code design.

4.2.2.2.4 Floor Diaphragms

Floor diaphragms shall be analyzed for (1) the seismic forces calculated by dynamic analysis, and (2) the horizontal forces resulting from offsets in, or changes in stiffness of, the vertical seismic framing elements above and below the diaphragm. The seismic forces calculated by dynamic analysis shall not be less than 85% of the forces calculated using Equation (4-4). Forces resulting from offsets in, or changes in stiffness of, the vertical lateral-force-resisting elements shall be taken to be equal to the elastic forces without reduction, unless smaller forces can be justified by rational analysis.

4.2.3 Mathematical Model for LSP & LDP

4.2.3.1 Basic Assumptions

Buildings with stiff or rigid diaphragms shall be modeled two-dimensionally if torsional effects are either sufficiently small to be ignored or indirectly captured; alternatively, a three-dimensional model may be developed. If torsional effects are not sufficiently small to be ignored or indirectly captured, a three-dimensional model of the building shall be developed.

Lateral-force-resisting frames in buildings with flexible diaphragms shall be modeled and analyzed as two-dimensional assemblies of components; alternatively, a three-dimensional model shall be used with the diaphragms modeled as flexible elements.

4.2.3.2 Horizontal Torsion

The effects of horizontal torsion shall be considered in a Tier 2 analysis. The total torsional moment at a given floor level shall be equal to the sum of the following two torsional moments:

- Actual torsion resulting from the eccentricity between the centers of mass and the centers of rigidity of all floors above and including the given floor, and
- Accidental torsion produced by horizontal offset in the centers of mass, at all floors above and including the given floor, equal to a minimum of 5% of the horizontal dimension at the given floor level measured perpendicular to the direction of the applied load.

The effects of accidental torsion shall not be used to reduce force and deformation demands on building components.

A building is considered torsionally irregular if the building has stiff or rigid diaphragms and the ratio $\delta_{max}/\delta_{avg}$ due to total torsional moment exceeds 1.2. In torsionally irregular buildings, the effect of accidental torsion shall be amplified by the factor, A_x, given in Equation (4-5).

$$A_x = \left(\frac{\delta_{max}}{1.2\delta_{avg}}\right)^2 \qquad (4\text{-}5)$$

where:

δ_{max} = the maximum displacement at any point of diaphragm at level x;

δ_{avg} = the algebraic average of displacements at the extreme points of the diaphragm at level x;

A_x = shall be greater than or equal to 1.0 and need not exceed 3.0.

If the ratio, η, of the maximum displacement at any point on any floor diaphragm (including torsional amplification), to the average displacement, exceeds 1.50, a three-dimensional model shall be developed for a Tier 2 analysis. When $\eta < 1.5$, the forces and displacements calculated using two-dimensional models shall be increased by the maximum value of η calculated for the building.

4.2.3.3 Primary and Secondary Components

Components shall be classified as either primary or secondary in accordance with Section 1.3.

Only the stiffness of primary components need be included in the mathematical building model. If secondary components are modeled, the total stiffness of the secondary components shall be no greater than 25% of the total stiffness of the primary components calculated at each level of the building.

Commentary:

The classification of components and elements should not result in a change in the regularity of a building. That is, components and elements should not be selectively assigned as either primary or secondary to change the configuration of a building from irregular to regular.

4.2.3.4 Diaphragms

Diaphragm deformations shall be estimated using the seismic forces computed in this Section. Mathematical models of buildings with stiff diaphragms shall explicitly include diaphragm flexibility. Mathematical models of buildings with rigid diaphragms shall explicitly account for the rigidity of the diaphragms. For buildings with flexible diaphragms at each floor level, the vertical lines of seismic framing may be considered independently, with seismic masses assigned on the basis of tributary area.

The in-plane deflection of the diaphragm shall be calculated for an in-plane distribution of lateral force consistent with the distribution of mass, as well as all in-plane lateral forces associated with offsets in the vertical seismic framing.

4.2.3.5 Multidirectional Excitation Effects

Buildings shall be analyzed for seismic forces in any horizontal direction. Seismic displacements and forces shall be assumed to act nonconcurrently in the direction of each principal axis of a building, unless the building is torsionally irregular as defined in Section 4.2.3.2 or one or more components form part of two or more intersecting elements, in which case multidirectional excitation effects shall be considered.

Multidirectional (orthogonal) excitation shall be evaluated by applying 100% of the seismic forces in one horizontal direction plus 30% of the seismic forces in the perpendicular horizontal direction.

4.2.3.6 Vertical Acceleration

The effects of vertical excitation on horizontal cantilevers and prestressed elements shall be considered using static or dynamic analysis methods. Vertical earthquake motions shall be characterized by a spectrum with ordinates equal to 67% of those of the horizontal spectrum in Section 3.5.2.3.1. Alternatively, vertical response spectra are developed using site-specific analysis may be used.

4.2.4 Acceptance Criteria for LSP & LDP

4.2.4.1 General Requirements

Component actions shall be computed according to Section 4.2.4.3; gravity loads as well as seismic forces shall be considered. Component strengths shall be computed in accordance with Section 4.2.4.4. Component actions and strengths then shall be compared with the acceptance criteria in Section 4.2.4.5.

4.2.4.2 Component Gravity Loads

Component gravity forces shall be calculated in accordance with Equation (4-6) and (4-7).

$$Q_G = 1.1(Q_D+Q_L+Q_S) \tag{4-6}$$

$$Q_G = 0.9\ Q_D \tag{4-7}$$

where:

Q_D = Dead load,

Q_L = Effective live load, equal to 25% of the unreduced design live load but not less than the measured live load,

Q_S = Effective snow load, equal to either 70% of the full design snow load or, where conditions warrant and approved by the regulatory agency, not less than 20% of the full design snow load, except that where the design snow load is 30 pounds per square foot or less, Q_S = 0.0.

4.2.4.3 Component Actions

Actions shall be classified as either deformation-controlled or force-controlled. A deformation-controlled action shall be defined as an action that has an associated deformation that is allowed to exceed the yield value; the maximum associated deformation is limited by the ductility capacity of the component. A force-controlled action shall be defined as an action that has an associated deformation that is not allowed to exceed the yield value; actions with limited ductility shall be considered force-controlled.

Commentary:

Global deformation of a structure is primarily due to the elastic and inelastic deformations associated with the deformation-controlled actions. The maximum force in force-controlled components are governed by the capacity of deformation-controlled components.

Consider actions in beams and columns of a reinforced concrete moment frame. Flexural moment are typically a deformation-controlled action. Shear forces in beams and axial forces in columns are force-controlled actions. The yielding of deformation-controlled actions (beam moment in this example), controls the forces that can be delivered to the force-controlled actions (beam shear & column axial force in this example).

Consider a braced frame structure. The axial force in the diagonal braces are deformation-controlled actions. The force in brace connections and axial force in columns are force-controlled actions. Yielding and buckling of braces control the maximum force that can be delivered to the connections and columns.

Typical deformation- and force-controlled actions are listed below where 'M' designates moment, 'V' designates shear force, and 'P' designates axial load.

	Deformation-Controlled	Force-Controlled
Moment Frames		
Beams	M	V
Columns	M	P, V
Joints	--	V
Shear Walls	M, V	P
Braced Frames		
Braces	P	--
Beams	--	P
Columns	--	P
Shear Link	V	P, M

4.2.4.3.1 Deformation-Controlled Actions

Deformation-controlled design actions, Q_{UD}, shall be calculated according to Equation (4-8).

$$Q_{UD} = Q_G \pm Q_E \qquad (4\text{-}8)$$

where:

Q_{UD} = Action due to gravity loads and earthquake forces,

Q_G = Action due to gravity forces as defined in Section 4.2.4.2,

Q_E = Action due to earthquake forces calculated using forces and analysis models described in either Section 4.2.2.1 or Section 4.2.2.2.

4.2.4.3.2 Force-Controlled Actions

Method 1

Force-controlled actions, Q_{UF} shall be calculated as the sum of forces due to gravity and the maximum force that can be delivered by deformation-controlled actions.

Method 2

Alternatively, force-controlled actions may be calculated according to Equation (4-9) or Equation (4-10). Equation (4-9) shall be used when the forces contributing to Q_{UF} are delivered by yielding components of the seismic framing system. Equation (4-10) shall be used for all other evaluations.

$$Q_{UF} = Q_G \pm \frac{Q_E}{CJ} \qquad (4\text{-}9)$$

$$Q_{UF} = Q_G \pm \frac{Q_E}{C} \qquad (4\text{-}10)$$

where:

Q_{UF} = Actions due to gravity loads and earthquake forces,

C = Modification Factor defined in Table 3-4,

J = a force-delivery reduction factor given by Equation (4-11) and shall not exceed 2.5 for buildings being evaluated to the Life Safety Performance Level and 2.0 for buildings being evaluated to the Immediate Occupancy Performance Level.

$$J = 1.5 + S_{DS} \qquad (4\text{-}11)$$

where:

S_{DS} = Design short-period spectral acceleration parameter, calculated in accordance with Section 3.5.2.3.1.

Method 3

For the evaluation of buildings analyzed using pseudo lateral force of Equation (3-2), Equation (4-10), with C=1.0, shall be used.

Commentary:

Force-controlled actions are those actions that provide little deformation to the entire building through inelastic behavior. Because of the limited ductility associated with force-controlled actions, inelastic action in these elements may cause a sudden partial or total collapse of the structure.

There are three methods for determining force-controlled actions. The first method is the sum of forces due to gravity and the maximum force that can be delivered by deformation-controlled actions. Q_{UF} for a brace connection would be equal to the axial force capacity of the brace member. Q_{UF} for shear in a beam would be equal to gravity shear plus the shear force associated with development of flexural moment capacity at the ends of the beam. Q_{UF} for axial force in a moment frame column would be equal to the sum of maximum shear forces that can be developed in the beams supported by the columns. If it can be shown that the deformation-controlled action can be developed before the failure of the associated force-controlled action, then the failure will not occur due to the fact the yielding of the deformation-controlled components will limit the demand on the force-controlled component. This method is recommended as the method to use in evaluating force-controlled components.

The second and third methods provide conservative estimates of force-controlled actions due to a design earthquake. Equation (4-9) may be used if other yielding elements in the building will limit the amount of force that can be delivered to the force-controlled component. Equation (4-10) is used if the force-controlled component is the "weak link" and, thus, must be evaluated for full earthquake force. Equation (4-10) must also be used if foundation sliding controls the behavior of the building as assumed by Equation (3-2).

4.2.4.3.3 Connections

Connections shall be evaluated as force-controlled actions. Alternatively, hold-down anchors used to resist overturning forces in wood shear wall buildings may be evaluated as deformation-controlled actions using the appropriate m-factors specified in Table 4-6.

4.2.4.3.4 Foundation/Soil Interface

Actions at the soil-foundation interface shall be considered force-controlled as defined in Section 4.2.4.3.2. The value of the earthquake force in Section 4.2.4.3.2 may be multiplied by a factor of 2/3 for buildings being evaluated for the Immediate Occupancy Performance Level and 1/3 for the Life Safety Performance Level.

Commentary:

This criteria allows the earthquake component of the total force at soil foundations interface to be reduced, because limited uplifting of the foundation is permitted. Foundation compressive loads can also be calculated using the reduced earthquake loads. Alternatively, the compressive soil pressure can be calculated by considering the equilibrium of forces with the foundations in uplifted condition.

4.2.4.4 Component Strength

Component strength for all actions shall be taken as the expected strength, Q_{CE}. Unless calculated otherwise, the expected strength shall be assumed equal to the nominal strength multiplied by 1.25. Alternatively, if allowable stresses are used, nominal strengths shall be taken as the allowable values multiplied by the following values:

Steel	1.7
Masonry	2.5
Wood	2.0

Except for wood diaphragms and wood and masonry shear walls, the allowable values shall not include a one-third increase for short term loading.

When calculating capacities of deteriorated elements, the evaluating design professional shall make reductions in the material strength, section properties, and other parameters as approved by the authority having jurisdiction to account for the deterioration.

> **Commentary:**
>
> The *1997 NEHRP Recommended Provisions for Seismic Regulations of New Buildings and Other Structures* provides component capacities for use in strength design or load and resistance factor design. These include nominal strength for wood, concrete, masonry and steel. Note that the resistance factors (ϕ), which are used in ultimate strength code design, are not used in calculating capacities of members when the LSP or LDP is used.

4.2.4.5 Acceptance Criteria for the LSP & LDP

4.2.4.5.1 Deformation-Controlled Actions

The acceptability of deformation-controlled primary and secondary components shall be determined in accordance with Equation (4-12).

$$Q_{CE} \geq \frac{Q_{UD}}{m} \qquad (4\text{-}12)$$

where:

- Q_{UD} = Action due to gravity and earthquake loading per Section 4.2.4.3.1.
- m = Component demand modifier to account for the expected ductility of the component; the appropriate *m*-factor shall be chosen from Tables 4-3 to 4-6 based on the level of performance and component characteristics; Interpolation shall be permitted in Tables 4-3 to 4-6; m = 1.0 for all components in buildings analyzed using Equation (3-2).
- Q_{CE} = Expected strength of the component at the deformation level under consideration. Q_{CE} shall be calculated in accordance with Section 4.2.4.4 considering all co-existing actions due to gravity and earthquake loads.

> **Commentary:**
>
> The *m*-factors in Tables 4-3 to 4-6 were developed using the values in FEMA 273 as a starting point, and modified so that this document provides comparable results to FEMA 178 for life safety performance level. Considering the effect of factor C (for short period structures) and different capacities used in the two documents, it can be shown that, for equivalent results with FEMA 178, the value of *m* for life safety level of performance should be in the range of 0.7 to 0.9 times the value of *R*.
>
> Note that the acceptability criteria and use of *m*-factors is applicable to the LSP and LDP only. *m*-factors are not used in conjunction with evaluating walls for out-of-plane forces or nonstructural elements or when using the Special Procedures for unreinforced masonry bearing walls with flexible diaphragms.

4.2.4.5.2 Force-Controlled Actions

The acceptability of force-controlled primary and secondary components shall be determined in accordance with Equation (4-13).

$$Q_{CE} \geq Q_{UF} \qquad (4\text{-}13)$$

where:

- Q_{UF} = Action due to gravity and earthquake loading; Q_{UF} shall be calculated in accordance with Section 4.2.4.3.2.
- Q_{CE} = Expected strength of the component at the deformation level under consideration Q_{CE} shall be calculated in accordance with Section 4.2.4.4 considering all co-existing actions due to gravity and earthquake loads.

4.2.5 Out-of-plane Wall Forces

Out-of-plane wall forces shall be computed in accordance with this section when triggered by the Procedures of Section 4.3 through 4.6.

Walls shall be anchored to each diaphragm for a minimum force of:

- $400S_{DS}$ pounds per foot of wall or
- χS_{DS} times the unit weight of the wall where χ shall be taken as 0.4 for Life Safety and 0.6 for Immediate Occupancy.

Forces shall be developed into the diaphragm. For flexible diaphragms, the anchorage forces shall be taken as 2 times those specified above and shall be developed into the diaphragm by continuous diaphragm cross ties. Diaphragms may be partitioned into a series of subdiaphragms. Each subdiaphragm shall be capable of transmitting the shear forces due to wall anchorage to a continuous diaphragm tie. Subdiaphragms shall have aspect ratios of 3 or less. Where wall panels are stiffened for out-of-plane behavior by pilasters and similar stiffening elements, anchors shall be provided at each such element and the distribution of out-of-plane forces to wall anchors and diaphragm ties shall consider the stiffening effect.

A wall shall have adequate strength to span between locations of out-of-plane support when subjected to out-of-plane forces equal to $0.4S_{DS}$ times the unit weight of the wall, over its area.

Strength of members and connections shall be taken as ϕ times the nominal strength.

Commentary:

Values of ϕ and nominal strengths may be obtained from *1997 NEHRP Recommended Provisions for Seismic Regulations for New Buildings*.

4.2.6 Special Procedure

4.2.6.1 General

Unreinforced masonry bearing wall buildings with flexible diaphragms being evaluated to the Life Safety Performance Level shall be evaluated in accordance with the requirements of this section.

The evaluation requirements of Chapter 2 shall be met prior to conducting this special procedure.

This special procedure shall apply to unreinforced masonry bearing wall buildings with the following characteristics:

- Flexible diaphragms at all levels above the base of the structure;
- A minimum of two lines of walls in each principal direction, except for single-story buildings with an open front on one side.

A Tier 3 evaluation shall be conducted for buildings not meeting the requirements of this section.

4.2.6.2 Cross Walls

4.2.6.2.1 General

Cross walls shall not be spaced more than 40 feet on center measured perpendicular to the direction under consideration and should be present in each story of the building. Cross walls shall extend the full story height between diaphragms.

Exceptions:

- Cross walls need not be present at all levels in accordance with Section 4.2.6.3.1, Equation (4-18),
- Cross walls that meet the following requirements need not be continuous:
  - Shear connections and anchorage at all edges of the diaphragm shall meet the requirements of Section 4.2.6.3.6;
  - Cross walls shall have a shear capacity of $0.6S_{D1}\Sigma W_d$ and shall interconnect the diaphragm to the foundation;

- Diaphragms spanning between cross walls that are continuous shall comply with the following equation:

$$\frac{2.5S_{D1}W_d + V_{ca}}{2v_u D} \le 2.5 \qquad (4\text{-}14)$$

4.2.6.2.2 Shear Capacity

Within any 40 feet measured along the span of the diaphragm, the sum of the cross wall shear capacities shall greater than or equal to 30% of the diaphragm shear capacity of the strongest diaphragm at or above the level under consideration.

4.2.6.2.3 Aspect Ratio

Cross walls shall have a length-to-height ratio between openings equal to or greater than 1.5.

4.2.6.3 Diaphragms

4.2.6.3.1 Demand-Capacity Ratios

Demand-capacity ratios shall be calculated for a diaphragm at any level in accordance with the following equations:

Diaphragms without cross walls at levels immediately above or below:

$$DCR = \frac{2.5S_{D1}W_d}{\sum v_u D} \qquad (4\text{-}15)$$

Diaphragms in a one-story building with cross walls:

$$DCR = \frac{2.5S_{D1}W_d}{\sum v_u D + V_{cb}} \qquad (4\text{-}16)$$

Diaphragms in a multi-story building with cross walls at all levels:

$$DCR = \frac{2.5S_{D1}\sum W_d}{\sum(\sum v_u D + V_{cb})} \qquad (4\text{-}17)$$

Roof diaphragms and the diaphragms directly below if coupled by cross walls:

$$DCR = \frac{2.5S_{D1}\sum W_d}{\sum(\sum v_u D)} \qquad (4\text{-}18)$$

where:

v_u = unit shear strength of the diaphragm calculated in accordance with Section 4.2.4.4.

4.2.6.3.2 Acceptability Criteria

The intersection of diaphragm span between walls, L, and the demand-capacity ratio, DCR, shall be located within Region 1, 2, or 3 on Figure 4-1.

4.2.6.3.3 Chords

An analysis for diaphragm flexure need not be made and chords need not be provided.

4.2.6.3.4 Collectors

Where walls do not extend the length of the diaphragm, collectors shall be provided. The collectors shall be able to the transfer diaphragm shears calculated in accordance with Section 4.2.6.3.6 into the shear walls.

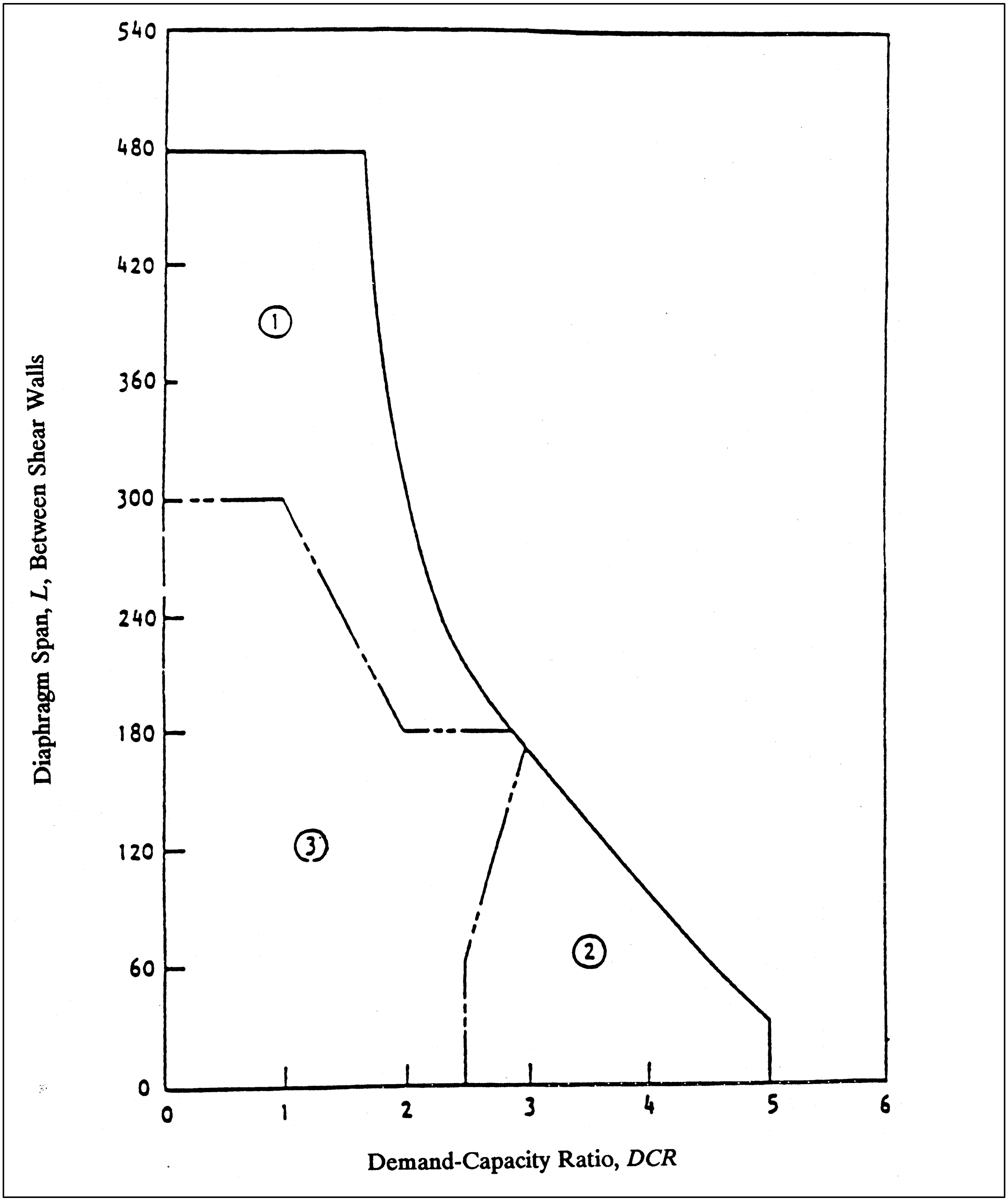
4.2.6.3.5 Diaphragm Openings

Diaphragm forces at corners at openings shall be investigated.

The diaphragm shall have the tensile capacity to develop the strength of the diaphragm at opening corners.

The demand-capacity ratio shall be calculated and evaluated in accordance with Sections 4.2.6.3.1 and 4.2.6.3.2 for the portion of the diaphragm adjacent to an opening using the opening dimension as the diaphragm span.

The demand-capacity ratio shall be calculated and evaluated in accordance with Sections 4.2.6.3.1 and 4.2.6.3.2 for openings occurring in the end quarter of the diaphragm span. The diaphragm capacity, $v_u D$, shall be based on the net depth of the diaphragm.



4.2.6.3.6 Diaphragm Shear Transfer

Figure 4-1. Diaphragm Span, L, Between Shear Walls (ft)

Diaphragms shall be connected to shear walls at each end and shall be able to developing the minimum of the forces calculated in accordance with Equations (4-19) and (4-20).

$$V_d = 1.5 S_{D1} C_p W_d \qquad (4\text{-}19)$$

$$V_d = v_u D \qquad (4\text{-}20)$$

Table 4-1 Horizontal Force Factor, C_p

Configuration of Materials	C_p
Roofs with straight or diagonal sheathing and roofing applied directly to the sheathing, or floors with straight tongue-and-groove sheathing	0.50
Diaphragm with double or multiple layers of boards with edges offset, and blocked structural panel systems.	0.75

4.2.6.4 Shear Walls

4.2.6.4.1 Shear Wall Actions

The walls story force distributed to a shear wall at any diaphragm level shall be determined in accordance with the following equations:

For buildings without cross walls:

$$F_{wx} = S_{D1}(W_{wx} + 0.5 W_d) \qquad (4\text{-}21)$$

but not exceed,

$$F_{wx} = S_{D1} W_{wx} + v_u D \qquad (4\text{-}22)$$

For buildings with cross walls in all levels:

$$F_{wx} = 0.75 S_{D1}(W_{wx} + 0.5 W_d) \qquad (4\text{-}23)$$

but need not exceed,

$$F_{wx} = 0.75 S_{D1}\left(W_{wx} + \Sigma W_d\left(\frac{v_u D}{\Sigma(\Sigma v_u D)}\right)\right) \qquad (4\text{-}24)$$

and need not exceed,

$$F_{wx} = 0.75 S_{D1} W_{wx} + v_u D \qquad (4\text{-}25)$$

The wall story shear shall be calculated in accordance with Equation (4-26).

$$V_{wx} = \Sigma F_{wx} \qquad (4\text{-}26)$$

4.2.6.4.2 Shear Wall Strengths

The shear wall strength shall be calculated in accordance with Equation (4-27).

$$V_a = 0.67 v_{me} D t \qquad (4\text{-}27)$$

where:

D = In-plane width dimension of masonry (in.),
t = Thickness of wall (in.),
v_{me} = expected masonry shear strength (psi) given by Equation (4-28),

$$v_{me} = \frac{0.75\left(0.75 v_{te} + \frac{P_{CE}}{A_n}\right)}{1.5} \qquad (4\text{-}28)$$

where:

v_{te} = Average bed-joint shear strength (psi) determined in accordance with Section 2.2 and not to exceed 100 psi;
P_{CE} = Expected gravity compressive force applied to a wall or pier component stress;
A_n = Area of net mortared/grouted section (in^2).

The rocking shear strength shall be calculated in accordance with Equations (4-29) and (4-30)

For walls without openings:

$$V_r = 0.9(P_D + 0.5 P_W)\frac{D}{H} \qquad (4\text{-}29)$$

For walls with openings:

$$V_r = 0.9 P_D \frac{D}{H} \qquad (4\text{-}30)$$

4.2.6.4.3 Shear Wall Acceptance Criteria

The acceptability of unreinforced masonry shear walls shall be determined in accordance with Equations (4-31), (4-32), and (4-33).

When $V_r < V_a$,

$$0.6V_{wx} < \Sigma V_r \quad (4\text{-}31)$$

When $V_a < V_r$, V_{wx} shall be distributed to the individual wall piers, V_p, in proportion to D/H and equation (4-32) and (4-33) shall be met.

$$V_p < V_a \quad (4\text{-}32)$$

$$V_p < V_r \quad (4\text{-}33)$$

If $V_p < V_a$ and $V_p > V_r$ for any pier, the pier shall be omitted from the analysis and the procedure repeated.

4.2.6.5 Out-of-Plane Demands

The unreinforced masonry wall height-to-thickness ratios shall be less than those set forth in Table 4-2.

The following limitations shall apply to Table 4-2:

- For buildings within Region 1 of Figure 4-1 as defined in Section 4.2.6.3.2, height to thickness ratios in column A of Table 4-2 may be used if cross walls comply with the requirements of Section 4.2.6.2 are present in all stories.
- For buildings within Region 2 of Figure 4-1 as defined in Section 4.2.6.3.2, height-to-thickness ratios in column A may be used.
- For buildings within Region 3 of Figure 4-1 as defined in Section 4.2.6.3.2, height-to-thickness in column B may be used.

Table 4-2. Allowable Height-to-Thickness Ratios of Unreinforced Masonry Walls

Wall Type	Regions of Moderate Seismicity	Regions of High Seismicity	
		A	B
Top story of multi-story building	14	14	9
First story of multi-story building	18	16	15
All other conditions	16	16	13

4.2.6.6 Wall Anchorage

Anchors shall be capable of developing the maximum of:

- $2.5S_{D1}$ times the weight of the wall, or
- 200 pounds per lineal foot, acting normal to the wall at the level of the floor or roof.

Walls shall be anchored at the roof and all floor levels at a spacing of equal to or less than 6 feet on center.

At the roof and all floor levels, anchors shall be provided within 2 feet horizontally from the inside corners of the wall.

The connection between the walls and the diaphragm shall not induce cross-grain bending or tension in the wood ledgers.

4.2.6.7 Buildings with Open Fronts

Single-story buildings with an open front on one side shall have cross walls parallel to the open front. The effective diaphragm span, L_i, for use in Figure 4-1, shall be calculated in accordance with Equation (4-34).

$$L_i = 2L\left(\frac{W_w}{W_d} + 1\right) \quad (4\text{-}34)$$

The diaphragm demand-capacity ratio shall be calculated in accordance with Equation (4-35).

$$DCR = \frac{2.5 S_{D1}(W_d + W_w)}{(v_u D + V_c)} \qquad (4\text{-}35)$$

4.2.7 Demands on Nonstructural Components

The seismic forces on nonstructural components shall be calculated in accordance with Equations (4-36), (4-37) and (4-38) when triggered by the Procedures in Section 4.8.

$$F_p = 0.4 a_p S_{DS} W_p (1 + 2x/h)/R_p \qquad (4\text{-}36)$$

F_p shall not be greater than:

$$F_p = 1.6 S_{DS} W_p \qquad (4\text{-}37)$$

and F_p shall not be taken as less than:

$$F_p = 0.3 S_{DS} W_p \qquad (4\text{-}38)$$

where:

- F_p = Seismic design force centered at the component's center of gravity and distributed relative to the component's mass distribution,
- S_{DS} = Design short-period spectral acceleration, as determined from Section 3.5.2.3.1,
- a_p = Component amplification factor from Table 4-7,
- W_p = Component operating weight,
- R_p = Component response modification factor, that varies from 1.0 to 6.0 (select appropriate value from Table 4-7),
- x = Height in structure of highest point of attachment of component. For components at or below grade x shall be taken as 0,
- h = Average roof height of structure relative to grade.

The force (F_p) shall be applied independently, longitudinally, and laterally in combination with service loads associated with the component. When positive and negative wind loads exceed F_p for nonstructural exterior walls, these wind loads shall govern the analysis. Similarly, when the building code horizontal loads exceed F_p for interior partitions, these building code loads shall govern the analysis.

Drift ratios (D) shall be determined in accordance with the following Equations (4-39) or (4-40).

For two connecting points on the same building or structural system:

$$D_r = (d_{xA} - d_{yA})/(X - Y) \qquad (4\text{-}39)$$

For two connection points on separate buildings or structural systems:

$$D_p = d_{xA} + d_{xB} \qquad (4\text{-}40)$$

where:

- D_p = Relative displacement,
- D_r = Drift ratio,
- X = Height of upper support attachment at level x as measured from grade,
- Y = Height of lower support attachment at level y as measured from grade,
- δ_{xA} = Deflection at building level x of building A, determined by elastic analysis,
- δ_{yA} = Deflection at building level y of Building A, determined by elastic analysis,
- δ_{xB} = Deflection at building level x of building B, determined by elastic analysis.

The effects of seismic displacements shall be considered in combination with displacements caused by other loads, as appropriate.