the internal volume and the flexibility of the *building* envelope (see Appendix A), or

d) if a dynamic approach to wind action is used, C_g is a value that is appropriate for the turbulence of the wind and the size and natural frequency of the structure (see Appendix A).

4.1.7.2. Dynamic Effects of Wind

1) Except as provided in Sentence (2), *buildings* whose height is greater than 4 times their minimum effective width, which is defined in Sentence (3), or greater than 60 m, and *buildings* whose lowest natural frequency is less than 1 Hz, as determined by rational analysis (see Appendix A), shall be designed

a) by experimental methods for the danger of dynamic overloading, vibration and the effects of fatigue, or

b) by using a dynamic approach to the action of wind gusts (see Appendix A).

2) *Buildings* whose lowest natural frequency is less than ¼ Hz, as determined by rational analysis, shall be designed by experimental methods in accordance with Clause (1)(a). (See Appendix A.)

3) The effective width, w, of a *building* shall be calculated using

$$w = \frac{\sum h_i w_i}{\sum h_i}$$

where the summations are over the height of the *building* for a given wind direction, h_i is the height above *grade* to level i, as defined in Sentence 4.1.7.1.(5), and w_i is the width normal to the wind direction at height h_i; the minimum effective width is the lowest value of the effective width considering all possible wind directions.

4.1.7.3. Full and Partial Loading

1) *Buildings* and structural members shall be capable of withstanding the effects of

a) the full wind loads acting along each of the 2 principal horizontal axes considered separately,

b) the wind loads as described in Clause (a) but with 100% of the load removed from any portion of the area,

c) the wind loads as described in Clause (a) but considered simultaneously at 75% of their full value, and

d) the wind loads as described in Clause (c) but with 50% of these loads removed from any portion of the area.

(See Appendix A.)

4.1.7.4. Interior Walls and Partitions

1) In the design of interior walls and *partitions*, due consideration shall be given to differences in air pressure on opposite sides of the wall or *partition* which may result from

a) pressure differences between the windward and leeward sides of a *building*,

b) stack effects due to a difference in air temperature between the exterior and interior of the *building*, and

c) air pressurization by the mechanical services of the *building*.

4.1.8. Earthquake Load and Effects

4.1.8.1. Analysis

1) The deflections and specified loading due to earthquake motions shall be determined according to the requirements in this Subsection, except that the requirements in this Subsection need not be considered in design if S(0.2), as defined in Sentence 4.1.8.4.(7), is less than or equal to 0.12.

4.1.8.2. Notation

1) In this Subsection

A_r = response amplification factor to account for type of attachment of mechanical/electrical equipment, as defined in Sentence 4.1.8.18.(1),
A_x = amplification factor at level x to account for variation of response of mechanical/electrical equipment with elevation within the *building*, as defined in Sentence 4.1.8.18.(1),
B_x = ratio at level x used to determine torsional sensitivity, as defined in Sentence 4.1.8.11.(9),
B = maximum value of B_x, as defined in Sentence 4.1.8.11.(9),
C_p = seismic coefficient for mechanical/electrical equipment, as defined in Sentence 4.1.8.18.(1),
D_{nx} = plan dimension of the *building* at level x perpendicular to the direction of seismic loading being considered,
e_x = distance measured perpendicular to the direction of earthquake loading between centre of mass and centre of rigidity at the level being considered (see Appendix A),
F_a = acceleration-based site coefficient, as defined in Sentence 4.1.8.4.(4),
F_t = portion of V to be concentrated at the top of the structure, as defined in Sentence 4.1.8.11.(6),
F_v = velocity-based site coefficient, as defined in Sentence 4.1.8.4.(4),
F_x = lateral force applied to level x, as defined in Sentence 4.1.8.11.(6),
h_i, h_n, h_x = the height above the base (i = 0) to level i, n, or x respectively, where the base of the structure is the level at which horizontal earthquake motions are considered to be imparted to the structure,
h_s = interstorey height (h_i - h_{i-1}),
I_E = earthquake importance factor of the structure, as described in Sentence 4.1.8.5.(1),
J = numerical reduction coefficient for base overturning moment, as defined in Sentence 4.1.8.11.(5),
J_x = numerical reduction coefficient for overturning moment at level x, as defined in Sentence 4.1.8.11.(7),
Level i = any level in the *building*, i = 1 for first level above the base,
Level n = level that is uppermost in the main portion of the structure,
Level x = level that is under design consideration,
M_v = factor to account for higher mode effect on base shear, as defined in Sentence 4.1.8.11.(5),
M_x = overturning moment at level x, as defined in Sentence 4.1.8.11.(7),
N = total number of *storeys* above exterior *grade* to level n,
$\bar{N}_{60}$ = Average Standard Penetration Resistance for the top 30 m, corrected to a rod energy efficiency of 60% of the theoretical maximum,
PGA = Peak Ground Acceleration expressed as a ratio to gravitational acceleration, as defined in Sentence 4.1.8.4.(1),
PI = plasticity index for clays,
R_d = ductility-related force modification factor reflecting the capability of a structure to dissipate energy through reversed cyclic inelastic behaviour, as given in Article 4.1.8.9.,
R_o = overstrength-related force modification factor accounting for the dependable portion of reserve strength in a structure designed according to these provisions, as defined in Article 4.1.8.9.,
S_p = horizontal force factor for part or portion of a *building* and its anchorage, as given in Sentence 4.1.8.18.(1),
S(T) = design spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T, as defined in Sentence 4.1.8.4.(7),
$S_a(T)$ = 5% damped spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T, as defined in Sentence 4.1.8.4.(1),
SFRS = Seismic Force Resisting System(s) is that part of the structural system that has been considered in the design to provide the required resistance to the earthquake forces and effects defined in Subsection 4.1.8.,

s_u = average undrained shear strength in the top 30 m of *soil*,
T = period in seconds,
T_a = fundamental lateral period of vibration of the *building* or structure in seconds in the direction under consideration, as defined in Sentence 4.1.8.11.(3),
T_x = floor torque at level x, as defined in Sentence 4.1.8.11.(10),
V = lateral earthquake design force at the base of the structure, as determined by Article 4.1.8.11.,
V_d = lateral earthquake design force at the base of the structure, as determined by Article 4.1.8.12.,
V_e = lateral earthquake elastic force at the base of the structure, as determined by Article 4.1.8.12.,
V_{ed} = lateral earthquake design elastic force at the base of the structure, as determined by Article 4.1.8.12.,
V_p = lateral force on a part of the structure, as determined by Article 4.1.8.18.,
$\bar{V}_s$ = average shear wave velocity in the top 30 m of *soil* or *rock*,
W = *dead load*, as defined in Article 4.1.4.1., except that the minimum *partition* load as defined in Sentence 4.1.4.1.(3) need not exceed 0.5 kPa, plus 25% of the design snow load specified in Subsection 4.1.6., plus 60% of the storage load for areas used for storage, except that *storage garages* need not be considered storage areas, and the full contents of any tanks (see Appendix A),
W_i, W_x = portion of W that is located at or is assigned to level i or x respectively,
W_p = weight of a part or portion of a structure, e.g., cladding, *partitions* and appendages,
δ_{ave} = average displacement of the structure at level x, as defined in Sentence 4.1.8.11.(9), and
δ_{max} = maximum displacement of the structure at level x, as defined in Sentence 4.1.8.11.(9).

4.1.8.3. General Requirements

1) The *building* shall be designed to meet the requirements of this Subsection and of the design standards referenced in Section 4.3.

2) Structures shall be designed with a clearly defined load path, or paths, that will transfer the inertial forces generated in an earthquake to the supporting ground.

3) The structure shall have a clearly defined Seismic Force Resisting System(s) (SFRS), as defined in Article 4.1.8.2.

4) The SFRS shall be designed to resist 100% of the earthquake loads and their effects. (See Appendix A.)

5) All structural framing elements not considered to be part of the SFRS must be investigated and shown to behave elastically or to have sufficient non-linear capacity to support their gravity loads while undergoing earthquake-induced deformations calculated from the deflections determined in Article 4.1.8.13.

6) Stiff elements that are not considered part of the SFRS, such as concrete, masonry, brick or pre-cast walls or panels, shall be
a) separated from all structural elements of the *building* such that no interaction takes place as the *building* undergoes deflections due to earthquake effects as calculated in this Subsection, or
b) made part of the SFRS and satisfy the requirements of this Subsection.
(See Appendix A.)

7) Stiffness imparted to the structure from elements not part of the SFRS, other than those described in Sentence (6), shall not be used to resist earthquake deflections but shall be accounted for
a) in calculating the period of the structure for determining forces if the added stiffness decreases the fundamental lateral period by more than 15%,

b) in determining the irregularity of the structure, except the additional stiffness shall not be used to make an irregular SFRS regular or to reduce the effects of torsion (see Appendix A), and
c) in designing the SFRS if inclusion of the elements not part of the SFRS in the analysis has an adverse effect on the SFRS (see Appendix A).

8) Structural modelling shall be representative of the magnitude and spatial distribution of the mass of the *building* and of the stiffness of all elements of the SFRS, including stiff elements that are not separated in accordance with Sentence 4.1.8.3.(6), and shall account for
a) the effect of cracked sections in reinforced concrete and reinforced masonry elements,
b) the effect of the finite size of members and joints,
c) sway effects arising from the interaction of gravity loads with the displaced configuration of the structure, and
d) other effects that influence the lateral stiffness of the *building*.
(See Appendix A.)

4.1.8.4. Site Properties

1) The peak ground acceleration (PGA) and the 5% damped spectral response acceleration values, $S_a(T)$, for the reference ground conditions (Site Class C in Table 4.1.8.4.A.) for periods T of 0.2 s, 0.5 s, 1.0 s, and 2.0 s, shall be determined in accordance with Subsection 1.1.3. and are based on a 2% probability of exceedance in 50 years.

Table 4.1.8.4.A.
Site Classification for Seismic Site Response
Forming Part of Sentences 4.1.8.4.(1) to (3)

Site Class	Ground Profile Name	Average Properties in Top 30 m, as per Appendix A		
		Average Shear Wave Velocity, $\bar{V}_s$ (m/s)	Average Standard Penetration Resistance, $\bar{N}_{60}$	*Soil* Undrained Shear Strength, s_u
A	Hard *rock*(1)(2)	$\bar{V}_s > 1500$	n/a	n/a
B	*Rock*(1)	$760 < \bar{V}_s \leq 1500$	n/a	n/a
C	Very dense *soil* and soft *rock*	$360 < \bar{V}_s < 760$	$\bar{N}_{60} > 50$	$s_u > 100$ kPa
D	Stiff *soil*	$180 < \bar{V}_s < 360$	$15 \leq \bar{N}_{60} \leq 50$	50 kPa $< s_u \leq$ 100 kPa
E	Soft *soil*	$\bar{V}_s < 180$	$\bar{N}_{60} < 15$	$s_u < 50$ kPa
		Any profile with more than 3 m of *soil* with the following characteristics: • plasticity index: PI > 20 • moisture content: w ≥40%, and • undrained shear strength: $s_u < 25$ kPa		
F	Other *soils*(3)	Site-specific evaluation required		

Notes to Table 4.1.8.4.A.:

(1) Site Classes A and B, hard *rock* and *rock*, are not to be used if there is more than 3 m of softer materials between the *rock* and the underside of footing or mat *foundations*. The appropriate Site Class for such cases is determined on the basis of the average properties of the total thickness of the softer materials (see Appendix A).
(2) If $\bar{V}_s$ has been measured in-situ, the F_a and F_v values derived from Tables 4.1.8.4.B. and 4.1.8.4.C. may be multiplied by $(1500/\bar{V}_s)^{1/2}$.
(3) Other *soils* include:
(a) liquefiable *soils*, quick and highly sensitive clays, collapsible weakly cemented *soils*, and other *soils* susceptible to failure or collapse under seismic loading,
(b) peat and/or highly organic clays greater than 3 m in thickness,
(c) highly plastic clays (PI > 75) more than 8 m thick, and
(d) soft to medium stiff clays more than 30 m thick.

2) Site classifications for ground shall conform to Table 4.1.8.4.A. and shall be determined using $\bar{V}_s$ except as provided in Sentence (3).

3) If average shear wave velocity, $\bar{V}_s$, is not known, Site Class shall be determined from energy-corrected Average Standard Penetration Resistance, $\bar{N}_{60}$, or from *soil* average undrained shear strength, s_u, as noted in Table 4.1.8.4.A., $\bar{N}_{60}$ and s_u being calculated based on rational analysis. (See Appendix A.)

4) Acceleration- and velocity-based site coefficients, F_a and F_v, shall conform to Tables 4.1.8.4.B. and 4.1.8.4.C. using linear interpolation for intermediate values of $S_a(0.2)$ and $S_a(1.0)$.

Table 4.1.8.4.B.
Values of F_a as a Function of Site Class and $S_a(0.2)$
Forming Part of Sentence 4.1.8.4.(4)

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) \geq 1.25$
A	0.7	0.7	0.8	0.8	0.8
B	0.8	0.8	0.9	1.0	1.0
C	1.0	1.0	1.0	1.0	1.0
D	1.3	1.2	1.1	1.1	1.0
E	2.1	1.4	1.1	0.9	0.9
F	(1)	(1)	(1)	(1)	(1)

Notes to Table 4.1.8.4.B.:
(1) See Sentence 4.1.8.4.(5).

Table 4.1.8.4.C.
Values of F_v as a Function of Site Class and $S_a(1.0)$
Forming Part of Sentence 4.1.8.4.(4)

Site Class	Values of F_v				
	$S_a(1.0) \leq 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \geq 0.5$
A	0.5	0.5	0.5	0.6	0.6
B	0.6	0.7	0.7	0.8	0.8
C	1.0	1.0	1.0	1.0	1.0
D	1.4	1.3	1.2	1.1	1.1
E	2.1	2.0	1.9	1.7	1.7
F	(1)	(1)	(1)	(1)	(1)

Notes to Table 4.1.8.4.C.:
(1) See Sentence 4.1.8.4.(5).

5) Site-specific evaluation is required to determine F_a and F_v for Site Class F. (See A-4.1.8.4.(3) and Table 4.1.8.4.A. in Appendix A.)

6) For structures with a fundamental period of vibration equal to or less than 0.5 s that are built on liquefiable soils, Site Class and the corresponding values of F_a and F_v may be determined as described in Tables 4.1.8.4.A., 4.1.8.4.B., and 4.1.8.4.C. by assuming that the soils are not liquefiable. (See A-4.1.8.4.(3) and Table 4.1.8.4.A. in Appendix A.)

7) The design spectral acceleration values of S(T) shall be determined as follows, using linear interpolation for intermediate values of T:

$$\begin{aligned} S(T) &= F_aS_a(0.2) \text{ for } T \le 0.2 \text{ s} \\ &= F_vS_a(0.5) \text{ or } F_aS_a(0.2)\text{, whichever is smaller for } T = 0.5 \text{ s} \\ &= F_vS_a(1.0) \text{ for } T = 1.0 \text{ s} \\ &= F_vS_a(2.0) \text{ for } T = 2.0 \text{ s} \\ &= F_vS_a(2.0)/2 \text{ for } T \ge 4.0 \text{ s} \end{aligned}$$

4.1.8.5. Importance Factor

1) The earthquake importance factor, I_E, shall be determined according to Table 4.1.8.5.

Table 4.1.8.5.
Importance Factor for Earthquake Loads and Effects, I_E
Forming Part of Sentence 4.1.8.5.(1)

Importance Category	Importance Factor, I_E	
	ULS	SLS(1)
Low	0.8	(2)
Normal	1.0	
High	1.3	
Post-disaster	1.5	

Notes to Table 4.1.8.5.:

(1) See Article 4.1.8.13.

(2) See Appendix A.

4.1.8.6. Structural Configuration

1) Structures having any of the features listed in Table 4.1.8.6. shall be designated irregular.

2) Structures not classified as irregular according to Sentence 4.1.8.6.(1) may be considered regular.

3) Except as required by Article 4.1.8.10., in cases where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, structures designated as irregular must satisfy the provisions referenced in Table 4.1.8.6.

Table 4.1.8.6.
Structural Irregularities(1)
Forming Part of Sentence 4.1.8.6.(1)

Type	Irregularity Type and Definition	Notes
1	**Vertical Stiffness Irregularity** Vertical stiffness irregularity shall be considered to exist when the lateral stiffness of the SFRS in a *storey* is less than 70% of the stiffness of any adjacent *storey*, or less than 80% of the average stiffness of the three *storeys* above or below.	(2)(3)(4)
2	**Weight (mass) Irregularity** Weight irregularity shall be considered to exist where the weight, W_i, of any *storey* is more than 150% of the weight of an adjacent *storey*. A roof that is lighter than the floor below need not be considered.	(2)

Table 4.1.8.6. (Continued)

Type	Irregularity Type and Definition	Notes
3	**Vertical Geometric Irregularity** Vertical geometric irregularity shall be considered to exist where the horizontal dimension of the SFRS in any *storey* is more than 130% of that in an adjacent *storey*.	(2)(3)(4)(5)
4	**In-Plane Discontinuity in Vertical Lateral-Force-Resisting Element** Except for braced frames and moment-resisting frames, an in-plane discontinuity shall be considered to exist where there is an offset of a lateral-force-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the *storey* below.	(2)(3)(4)(5)
5	**Out-of-Plane Offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS.	(2)(3)(4)(5)
6	**Discontinuity in Capacity - Weak Storey** A weak *storey* is one in which the *storey* shear strength is less than that in the *storey* above. The *storey* shear strength is the total strength of all seismic-resisting elements of the SFRS sharing the *storey* shear for the direction under consideration.	(3)
7	**Torsional Sensitivity (to be considered when diaphragms are not flexible)** Torsional sensitivity shall be considered to exist when the ratio B calculated according to Sentence 4.1.8.11.(9) exceeds 1.7.	(2)(3)(4)(6)
8	**Non-orthogonal Systems** A non-orthogonal system irregularity shall be considered to exist when the SFRS is not oriented along a set of orthogonal axes.	(4)(7)

Notes to Table 4.1.8.6.:

(1) One-*storey* penthouses with a weight of less than 10% of the level below need not be considered in the application of this table.
(2) See Article 4.1.8.7.
(3) See Article 4.1.8.10.
(4) See Appendix A.
(5) See Article 4.1.8.15.
(6) See Sentences 4.1.8.11.(9), (10) and 4.1.8.12.(4).
(7) See Article 4.1.8.8.

4.1.8.7. Methods of Analysis

1) Analysis for design earthquake actions shall be carried out in accordance with the Dynamic Analysis Procedure described in Article 4.1.8.12. (see Appendix A), except that the Equivalent Static Force Procedure described in Article 4.1.8.11. may be used for structures that meet any of the following criteria:

a) in cases where $I_E F_a S_a(0.2)$ is less than 0.35,
b) regular structures that are less than 60 m in height and have a fundamental lateral period, T_a, less than 2 s in each of two orthogonal directions as defined in Article 4.1.8.8., or
c) structures with structural irregularity, of Type 1, 2, 3, 4, 5, 6 or 8 as defined in Table 4.1.8.6., that are less than 20 m in height and have a fundamental lateral period, T_a, less than 0.5 s in each of two orthogonal directions as defined in Article 4.1.8.8.

4.1.8.8. Direction of Loading

1) Earthquake forces shall be assumed to act in any horizontal direction, except that the following shall be considered to provide adequate design force levels in the structure:

a) where components of the SFRS are oriented along a set of orthogonal axes, independent analyses about each of the principal axes of the structure shall be performed,

b) where the components of the SFRS are not oriented along a set of orthogonal axes and $I_E F_a S_a(0.2)$ is less than 0.35, independent analyses about any two orthogonal axes is permitted, or

c) where the components of the SFRS are not oriented along a set of orthogonal axes and $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, analysis of the structure independently in any two orthogonal directions for 100% of the prescribed earthquake loads applied in one direction plus 30% of the prescribed earthquake loads in the perpendicular direction, with the combination requiring the greater element strength being used in the design.

4.1.8.9. SFRS Force Reduction Factors, System Overstrength Factors, and General Restrictions

1) The values of R_d and R_o and the corresponding system restrictions shall conform to Table 4.1.8.9. and the requirements of this Subsection.

2) When a particular value of R_d is required by this Article, the corresponding R_o shall be used.

3) For combinations of different types of SFRS acting in the same direction in the same *storey*, R_dR_o shall be taken as the lowest value of R_dR_o corresponding to these systems.

4) For vertical variations of R_dR_o, excluding rooftop structures not exceeding two *storeys* in height whose weight is less than the greater of 10% of W and 30% of W_i of the level below, the value of R_dR_o used in the design of any *storey* shall be less than or equal to the lowest value of R_dR_o used in the given direction for the *storeys* above, and the requirements of Sentence 4.1.8.15.(5) must be satisfied. (See Appendix A.)

5) If it can be demonstrated through testing, research and analysis that the seismic performance of a structural system is at least equivalent to one of the types of SFRS mentioned in Table 4.1.8.9., then such a structural system will qualify for values of R_d and R_o corresponding to the equivalent type in that Table. (See Appendix A.)

Table 4.1.8.9.
SFRS Ductility-Related Force Modification Factors, R_d, Overstrength-Related Force Modification Factors, R_o, and General Restrictions(1)
Forming Part of Sentence 4.1.8.9.(1)

Type of SFRS	R_d	R_o	Restrictions(2)				
			Cases Where $I_E F_a S_a(0.2)$				Cases Where $I_E F_v S_a(1.0)$
			< 0.2	≥0.2 to < 0.35	≥0.35 to ≤0.75	> 0.75	> 0.3
Steel Structures Designed and Detailed According to CSA S16(3)							
Ductile moment-resisting frames	5.0	1.5	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	3.5	1.5	NL	NL	NL	NL	NL
Limited ductility moment-resisting frames	2.0	1.3	NL	NL	60	30	30

Table 4.1.8.9. (Continued)

Type of SFRS	R_d	R_o	Restrictions(2)				
			Cases Where $I_E F_a S_a(0.2)$				Cases Where $I_E F_v S_a(1.0)$
			< 0.2	≥0.2 to < 0.35	≥0.35 to ≤0.75	> 0.75	> 0.3
Moderately ductile concentrically braced frames							
Tension-compression braces	3.0	1.3	NL	NL	40	40	40
Tension only braces	3.0	1.3	NL	NL	20	20	20
Limited ductility concentrically braced frames							
Tension-compression braces	2.0	1.3	NL	NL	60	60	60
Tension only braces	2.0	1.3	NL	NL	40	40	40
Ductile buckling-restrained braced frames	4.0	1.2	NL	NL	40	40	40
Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL	NL
Ductile plate walls	5.0	1.6	NL	NL	NL	NL	NL
Limited ductility plate walls	2.0	1.5	NL	NL	60	60	60
Conventional construction of moment-resisting frames, braced frames or plate walls							
Assembly occupancies	1.5	1.3	NL	NL	15	15	15
Other *occupancies*	1.5	1.3	NL	NL	60	40	40
Other steel SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP
Concrete Structures Designed and Detailed According to CAN/CSA-A23.3							
Ductile moment-resisting frames	4.0	1.7	NL	NL	NL	NL	NL
Moderately ductile moment-resisting frames	2.5	1.4	NL	NL	60	40	40
Ductile coupled walls	4.0	1.7	NL	NL	NL	NL	NL
Ductile partially coupled walls	3.5	1.7	NL	NL	NL	NL	NL
Ductile shear walls	3.5	1.6	NL	NL	NL	NL	NL
Moderately ductile shear walls	2.0	1.4	NL	NL	NL	60	60
Conventional construction							
Moment-resisting frames	1.5	1.3	NL	NL	15	NP	NP
Shear walls	1.5	1.3	NL	NL	40	30	30
Other concrete SFRS(s) not listed above	1.0	1.0	15	15	NP	NP	NP
Timber Structures Designed and Detailed According to CSA O86							
Shear walls							

Table 4.1.8.9. (Continued)

Type of SFRS	R_d	R_o	Restrictions(2)				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where $I_EF_vS_a(1.0)$
			< 0.2	≥0.2 to < 0.35	≥0.35 to ≤0.75	> 0.75	> 0.3
Nailed shear walls: wood-based panel	3.0	1.7	NL	NL	30	20	20
Shear walls: wood-based and gypsum panels in combination	2.0	1.7	NL	NL	20	20	20
Braced or moment-resisting frames with ductile connections							
Moderately ductile	2.0	1.5	NL	NL	20	20	20
Limited ductility	1.5	1.5	NL	NL	15	15	15
Other wood- or gypsum-based SFRS(s) not listed above	1.0	1.0	15	15	NP	NP	NP
Masonry Structures Designed and Detailed According to CSA S304.1							
Moderately ductile shear walls	2.0	1.5	NL	NL	60	40	40
Limited ductility shear walls	1.5	1.5	NL	NL	40	30	30
Conventional construction							
Shear walls	1.5	1.5	NL	60	30	15	15
Moment-resisting frames	1.5	1.5	NL	30	NP	NP	NP
Unreinforced masonry	1.0	1.0	30	15	NP	NP	NP
Other masonry SFRS(s) not listed above	1.0	1.0	15	NP	NP	NP	NP
Cold-Formed Steel Structures Designed and Detailed According to CAN/CSA-S136							
Shear walls							
Screw-connected shear walls – wood-based panels	2.5	1.7	20	20	20	20	20
Screw-connected shear walls – wood-based and gypsum panels in combination	1.5	1.7	20	20	20	20	20
Diagonal strap concentrically braced walls							
Limited ductility	1.9	1.3	20	20	20	20	20
Conventional construction	1.2	1.3	15	15	NP	NP	NP
Other cold-formed SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP

Notes to Table 4.1.8.9.:

(1) See Article 4.1.8.10.

(2) NP = system is not permitted.
NL = system is permitted and not limited in height as an SFRS; height may be limited in other Parts of the Code.
Numbers in this Table are maximum height limits in m.
The most stringent requirement governs.

(3) Higher design force levels are prescribed in CSA S16 for some heights of *buildings*.

4.1.8.10. Additional System Restrictions

1) Except as required by Clause (2)(b), structures with a Type 6 irregularity, Discontinuity in Capacity - Weak Storey, as described in Table 4.1.8.6., are not permitted unless $I_E F_a S_a(0.2)$ is less than 0.2 and the forces used for design of the SFRS are multiplied by $R_d R_o$.

2) *Post-disaster buildings* shall
a) not have any irregularities conforming to Types 1, 3, 4, 5 and 7 as described in Table 4.1.8.6., in cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35,
b) not have a Type 6 irregularity as described in Table 4.1.8.6.,
c) have an SFRS with an R_d of 2.0 or greater, and
d) have no *storey* with a lateral stiffness that is less than that of the *storey* above it.

3) For *buildings* having fundamental lateral periods, T_a, of 1.0 s or greater, and where $I_E F_v S_a(1.0)$ is greater than 0.25, walls forming part of the SFRS shall be continuous from their top to the *foundation* and shall not have irregularities of Type 4 or 5 as described in Table 4.1.8.6.

4.1.8.11. Equivalent Static Force Procedure for Structures Satisfying the Conditions of Article 4.1.8.7.

1) The static loading due to earthquake motion shall be determined according to the procedures given in this Article.

2) The minimum lateral earthquake force, V, shall be calculated using the following formula:

$$V = S(T_a) M_v I_E W / (R_d R_o)$$

except
a) for walls, coupled walls and wall-frame systems, V shall not be less than

$$S(4.0) M_v I_E W / (R_d R_o)$$

b) for moment-resisting frames, braced frames, and other systems, V shall not be less than

$$S(2.0) M_v I_E W / (R_d R_o)$$

c) for *buildings* located on a site other than Class F and having an SFRS with an R_d equal to or greater than 1.5, V need not be greater than

$$\frac{2}{3} S(0.2) I_E W / (R_d R_o)$$

3) The fundamental lateral period, T_a, in the direction under consideration in Sentence (2), shall be determined as:
a) for moment-resisting frames that resist 100% of the required lateral forces and where the frame is not enclosed by or adjoined by more rigid elements that would tend to prevent the frame from resisting lateral forces, and where h_n is in metres:
   i) $0.085 (h_n)^{3/4}$ for steel moment frames,
   ii) $0.075 (h_n)^{3/4}$ for concrete moment frames, or
   iii) 0.1 N for other moment frames,
b) $0.025 h_n$ for braced frames where h_n is in metres,

c) $0.05 (h_n)^{3/4}$ for shear wall and other structures where h_n is in metres, or
d) other established methods of mechanics using a structural model that complies with the requirements of Sentence 4.1.8.3.(8), except that
   i) for moment-resisting frames, T_a shall not be taken greater than 1.5 times that determined in Clause (a),
   ii) for braced frames, T_a shall not be taken greater than 2.0 times that determined in Clause (b),
   iii) for shear wall structures, T_a shall not be taken greater than 2.0 times that determined in Clause (c),
   iv) for other structures, T_a shall not be taken greater than that determined in Clause (c), and
   v) for the purpose of calculating the deflections, the period without the upper limit specified in Subclauses (d)(i) to (d)(iv) may be used, except that, for walls, coupled walls and wall-frame systems, T_a shall not exceed 4.0 s, and for moment-resisting frames, braced frames, and other systems, T_a shall not exceed 2.0 s.

(See Appendix A.)

4) The weight, W, of the *building* shall be calculated using the following formula:

$$W = \sum_{i=1}^{n} W_i$$

5) The higher mode factor, M_v, and its associated base overturning moment reduction factor, J, shall conform to Table 4.1.8.11.

Table 4.1.8.11.
Higher Mode Factor, M_v, and Base Overturning Reduction Factor, J(1)(2)
Forming Part of Sentence 4.1.8.11.(5)

$S_a(0.2)/S_a(2.0)$	Type of Lateral Resisting Systems	M_v for $T_a \leq 1.0$	M_v for $T_a = 2.0$	M_v for $T_a \geq 4.0$	J for $T_a \leq 0.5$	J for $T_a = 2.0$	J for $T_a \geq 4.0$
< 8.0	Moment-resisting frames	1.0	1.0	(3)	1.0	0.9	(3)
	Coupled walls(4)	1.0	1.0	1.0	1.0	0.9	0.8
	Braced frames	1.0	1.0	(3)	1.0	0.8	(3)
	Walls, wall-frame systems	1.0	1.2	1.6	1.0	0.6	0.5
	Other systems(5)	1.0	1.2	(3)	1.0	0.6	(3)
≥8.0	Moment-resisting frames	1.0	1.2	(3)	1.0	0.7	(3)
	Coupled walls(4)	1.0	1.2	1.2	1.0	0.7	0.6
	Braced frames	1.0	1.5	(3)	1.0	0.6	(3)
	Walls, wall-frame systems	1.0	2.2	3.0	1.0	0.4	0.3
	Other systems(5)	1.0	2.2	(3)	1.0	0.4	(3)

Notes to Table 4.1.8.11.:
(1) For values of M_v between fundamental lateral periods, T_a, of 1.0 s and 2.0 s and between 2.0 s and 4.0 s, the product $S(T_a) \cdot M_v$ shall be obtained by linear interpolation.
(2) Values of J between fundamental lateral periods, T_a, of 0.5 s and 2.0 s and between 2.0 s and 4.0 s shall be obtained by linear interpolation.
(3) For fundamental lateral periods, T_a, greater than 2.0 s, use the values for $T_a = 2.0$.

Table 4.1.8.11. (Continued)

(4) A "coupled wall" is a wall system with coupling beams, where at least 66% of the base overturning moment resisted by the wall system is carried by the axial tension and compression forces resulting from shear in the coupling beams.
(5) For hybrid systems, values corresponding to walls must be used or a dynamic analysis must be carried out as per Article 4.1.8.12.

6) The total lateral seismic force, V, shall be distributed such that a portion, F_t, shall be assumed to be concentrated at the top of the *building*, where F_t is equal to 0.07 T_aV but need not exceed 0.25 V and may be considered as zero where the fundamental lateral period, T_a, does not exceed 0.7 s; the remainder, V - F_t, shall be distributed along the height of the *building*, including the top level, in accordance with the following formula:

$$F_x = (V - F_t)\, W_x h_x / \left(\sum_{i=1}^{n} W_i h_i \right)$$

7) The structure shall be designed to resist overturning effects caused by the earthquake forces determined in Sentence (6) and the overturning moment at level x, M_x, shall be determined using the following equation:

$$M_x = J_x \sum_{i=x}^{n} F_i (h_i - h_x)$$

where

J_x = 1.0 for $h_x \geq 0.6h_n$, and
J_x = $J + (1 - J)(h_x / 0.6h_n)$ for $h_x < 0.6h_n$

where

J = base overturning moment reduction factor conforming to Table 4.1.8.11.

8) Torsional effects that are concurrent with the effects of the forces mentioned in Sentence (6) and are caused by the simultaneous actions of the following torsional moments shall be considered in the design of the structure according to Sentence (10):

a) torsional moments introduced by eccentricity between the centres of mass and resistance and their dynamic amplification, and
b) torsional moments due to accidental eccentricities.

9) Torsional sensitivity shall be determined by calculating the ratio B_x for each level x according to the following equation for each orthogonal direction determined independently:

$$B_x = \delta_{max} / \delta_{ave}$$

where

B = maximum of all values of B_x in both orthogonal directions, except that the B_x for one-*storey* penthouses with a weight less than 10% of the level below need not be considered,
δ_{max} = maximum *storey* displacement at the extreme points of the structure, at level x in the direction of the earthquake induced by the equivalent static forces acting at distances ± 0.10 D_{nx} from the centres of mass at each floor, and
δ_{ave} = average of the displacements at the extreme points of the structure at level x produced by the above-mentioned forces.

10) Torsional effects shall be accounted for as follows:

a) for a *building* with $B \leq 1.7$ or where $I_E F_a S_a(0.2)$ is less than 0.35, by applying torsional moments about a vertical axis at each level throughout the *building*, derived for each of the following load cases considered separately:
   i) $T_x = F_x(e_x + 0.10\, D_{nx})$, and
   ii) $T_x = F_x(e_x - 0.10\, D_{nx})$

where F_x is the lateral force at each level determined according to Sentence (6) and where each element of the *building* is designed for the most severe effect of the above load cases, or

b) for a *building* with $B > 1.7$, in cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, by a Dynamic Analysis Procedure as specified in Article 4.1.8.12.

4.1.8.12. Dynamic Analysis Procedure

1) The Dynamic Analysis Procedure shall be in accordance with one of the following methods:

a) Linear Dynamic Analysis by either the Modal Response Spectrum Method or the Numerical Integration Linear Time History Method using a structural model that complies with the requirements of Sentence 4.1.8.3.(8) (see Appendix A), or

b) Non-linear Dynamic Analysis, in which case a special study shall be performed (see Appendix A).

2) The spectral acceleration values used in the Modal Response Spectrum Method shall be the design spectral acceleration values, S(T), defined in Sentence 4.1.8.4.(7).

3) The ground motion histories used in the Numerical Integration Linear Time History Method shall be compatible with a response spectrum constructed from the design spectral acceleration values, S(T), defined in Sentence 4.1.8.4.(7). (See Appendix A.)

4) The effects of accidental torsional moments acting concurrently with the lateral earthquake forces that cause them shall be accounted for by the following methods:

a) the static effects of torsional moments due to $(\pm 0.10\, D_{nx})F_x$ at each level x, where F_x is either determined from the elastic dynamic analysis or determined from Sentence 4.1.8.11.(6) multiplied by $R_d R_o / I_E$, shall be combined with the effects determined by dynamic analysis (see Appendix A), or

b) if B, as defined in Sentence 4.1.8.11.(9), is less than 1.7, it is permitted to use a three-dimensional dynamic analysis with the centres of mass shifted by a distance of $-0.05\, D_{nx}$ and $+\,0.05\, D_{nx}$.

5) Except as provided in Sentence (6), the design elastic base shear, V_{ed}, is equal to the elastic base shear, V_e, obtained from a Linear Dynamic Analysis.

6) For structures located on sites other than Class F that have an SFRS with R_d equal to or greater than 1.5, the elastic base shear obtained from a Linear Dynamic Analysis may be multiplied by the following factor to obtain the design elastic base shear, V_{ed}:

$$\frac{2S\,(0.2)}{3S\,(T_a)} \leq 1.0$$

7) The design elastic base shear, V_{ed}, shall be multiplied by the importance factor, I_E, as determined in Article 4.1.8.5., and shall be divided by $R_d R_o$, as determined in Article 4.1.8.9., to obtain the design base shear, V_d.

8) Except as required by Sentence (9), if the base shear, V_d, obtained in Sentence (7) is less than 80% of the lateral earthquake design force, V, of Article 4.1.8.11., V_d shall be taken as 0.8 V.

9) For irregular structures requiring dynamic analysis in accordance with Article 4.1.8.7., V_d shall be taken as the larger of the V_d determined in Sentence (7) and 100% of V.

10) Except as required by Sentence (11), the values of elastic *storey* shears, *storey* forces, member forces, and deflections obtained from the Linear Dynamic Analysis, including the effect of accidental torsion determined in Sentence (4), shall be multiplied by V_d/V_e to determine their design values, where V_d is the base shear.

11) For the purpose of calculating deflections, it is permitted to use a value for V based on the value for T_a determined in Clause 4.1.8.11.(3)(d) to obtain V_d in Sentences (8) and (9).

4.1.8.13. Deflections and Drift Limits

1) Lateral deflections of a structure shall be calculated in accordance with the loads and requirements defined in this Subsection.

2) Lateral deflections obtained from a linear elastic analysis using the methods given in Articles 4.1.8.11. and 4.1.8.12. and incorporating the effects of torsion, including accidental torsional moments, shall be multiplied by R_dR_o/I_E to give realistic values of anticipated deflections.

3) Based on the lateral deflections calculated in Sentence (2), the largest inter*storey* deflection at any level shall be limited to 0.01 h_s for *post-disaster buildings*, 0.02 h_s for High Importance Category *buildings*, and 0.025 h_s for all other *buildings*.

4) The deflections calculated in Sentence (2) shall be used to account for sway effects as required by Sentence 4.1.3.2.(12). (See Appendix A.)

4.1.8.14. Structural Separation

1) Adjacent structures shall either be separated by the square root of the sum of the squares of their individual deflections calculated in Sentence 4.1.8.13.(2), or shall be connected to each other.

2) The method of connection required in Sentence (1) shall take into account the mass, stiffness, strength, ductility and anticipated motion of the connected *buildings* and the character of the connection.

3) Rigidly connected *buildings* shall be assumed to have the lowest R_dR_o value of the *buildings* connected.

4) *Buildings* with non-rigid or energy-dissipating connections require special studies.

4.1.8.15. Design Provisions

1) Except as provided in Sentences (2) and (3), diaphragms, collectors, chords, struts and connections shall be designed so as not to yield, and the design shall account for the shape of the diaphragm, including openings, and for the forces generated in the diaphragm due to the following cases, whichever one governs (see Appendix A):
- a) forces due to loads determined in Article 4.1.8.11. or 4.1.8.12. applied to the diaphragm are increased to reflect the lateral load capacity of the SFRS, plus forces in the diaphragm due to the transfer of forces between elements of the SFRS associated with the lateral load capacity of such elements and accounting for discontinuities and changes in stiffness in these elements, or
- b) a minimum force corresponding to the design-based shear divided by N for the diaphragm at level x.

2) Steel deck roof diaphragms in *buildings* of less than 4 *storeys* or wood diaphragms that are designed and detailed according to the applicable referenced design standards to exhibit ductile behaviour shall meet the requirements of Sentence (1), except that they may yield and the forces shall be
- a) for wood diaphragms acting in combination with vertical wood shear walls, equal to the lateral earthquake design force,
- b) for wood diaphragms acting in combination with other SFRS, not less than the force corresponding to $R_dR_o = 2.0$, and
- c) for steel deck roof diaphragms, not less than the force corresponding to $R_dR_o = 2.0$.

3) Where diaphragms are designed in accordance with Sentence (2), the struts shall be designed in accordance with Clause 4.1.8.15.(1)(a) and the collectors, chords and connections between the diaphragms and the vertical elements of the SFRS shall be designed for forces corresponding to the capacity of the diaphragms in accordance with the applicable CSA standards. (See Appendix A.)

4) In cases where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, the elements supporting any discontinuous wall, column or braced frame shall be designed for the lateral load capacity of the components of the SFRS they support. (See Appendix A.)

5) Where structures have vertical variations of R_dR_o satisfying Sentence 4.1.8.9.(4), the elements of the SFRS below the level where the change in R_dR_o occurs shall be designed for the forces associated with the lateral load capacity of the SFRS above that level. (See Appendix A.)

6) Where earthquake effects can produce forces in a column or wall due to lateral loading along both orthogonal axes, account shall be taken of the effects of potential concurrent yielding of other elements framing into the column or wall from all directions at the level under consideration and as appropriate at other levels. (See Appendix A.)

7) Except as provided in Sentence (8), the design forces associated with the lateral capacity of the SFRS need not exceed the forces determined in accordance with Sentence 4.1.8.7.(1) with R_dR_o taken as 1.0, unless otherwise provided by the applicable referenced design standards for elements, in which case the design forces associated with the lateral capacity of the SFRS need not exceed the forces determined in accordance with Sentence 4.1.8.7.(1) with R_dR_o taken as 1.3. (See Appendix A.)

8) If *foundation* rocking is accounted for, the design forces for the SFRS need not exceed the maximum values associated with *foundation* rocking, provided that R_d and R_o for the type of SFRS used conform to Table 4.1.8.9. and that the *foundation* is designed in accordance with Sentence 4.1.8.16.(1).

4.1.8.16. Foundation Provisions

1) *Foundations* shall be designed to resist the lateral load capacity of the SFRS, except that when the *foundations* are allowed to rock, the design forces for the *foundation* need not exceed those determined in Sentence 4.1.8.7.(1) using an R_dR_o equal to 2.0. (See Appendix A.)

2) The design of *foundations* shall be such that they are capable of transferring earthquake loads and effects between the *building* and the ground without exceeding the capacities of the *soil* and *rock*.

3) In cases where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, the following requirements shall be satisfied:

a) *piles* or *pile* caps, drilled piers, and *caissons* shall be interconnected by continuous ties in not less than two directions (see Appendix A),
b) *piles*, drilled piers, and *caissons* shall be embedded a minimum of 100 mm into the *pile* cap or structure, and
c) *piles*, drilled piers, and *caissons*, other than wood *piles*, shall be connected to the *pile* cap or structure for a minimum tension force equal to 0.15 times the factored compression load on the *pile*.

4) At sites where $I_EF_aS_a(0.2)$ is equal to or greater than 0.35, *basement* walls shall be designed to resist earthquake lateral pressures from backfill or natural ground. (See Appendix A.)

5) At sites where $I_EF_aS_a(0.2)$ is greater than 0.75, the following requirements shall be satisfied:

a) *piles*, drilled piers, or *caissons* shall be designed and detailed to accommodate cyclic inelastic behaviour when the design moment in the element due to earthquake effects is greater than 75% of its moment capacity (see Appendix A), and
b) spread footings founded on *soil* defined as Site Class E or F shall be interconnected by continuous ties in not less than two directions.

6) Each segment of a tie between elements that is required by Clauses (3)(a) or (5)(b) shall be designed to carry by tension or compression a horizontal force at least equal to the greatest factored *pile* cap or column vertical load in the elements it connects, multiplied by a factor of 0.10 $I_EF_aS_a(0.2)$, unless it can be demonstrated that equivalent restraints can be provided by other means. (See Appendix A.)

7) The potential for liquefaction of the *soil* and its consequences, such as significant ground displacement and loss of *soil* strength and stiffness, shall be evaluated based on the ground motion parameters referenced in Subsection 1.1.3. and shall be taken into account in the design of the structure and its *foundations*. (See Appendix A.)

4.1.8.17. Site Stability

1) The potential for slope instability and its consequences, such as slope displacement, shall be evaluated based on site-specific material properties and ground motion parameters referenced in Subsection 1.1.3. and shall be taken into account in the design of the structure and its *foundations*. (See Appendix A.)

4.1.8.18. Elements of Structures, Non-structural Components and Equipment

(See Appendix A.)

1) Except as provided in Sentences (2) and (8), elements and components of *buildings* described in Table 4.1.8.18. and their connections to the structure shall be designed to accommodate the *building* deflections calculated in accordance with Article 4.1.8.13. and the element or component deflections calculated in accordance with Sentence (10), and shall be designed for a lateral force, V_p, distributed according to the distribution of mass:

$$V_p = 0.3\,F_a S_a\,(0.2)\;I_E S_p W_p$$

where

F_a = as defined in Table 4.1.8.4.B.,
$S_a(0.2)$ = spectral response acceleration value at 0.2 s, as defined in Sentence 4.1.8.4.(1),
I_E = importance factor for the *building*, as defined in Article 4.1.8.5.,
S_p = $C_pA_rA_x/R_p$ (the maximum value of S_p shall be taken as 4.0 and the minimum value of S_p shall be taken as 0.7), where
- C_p = element or component factor from Table 4.1.8.18.,
- A_r = element or component force amplification factor from Table 4.1.8.18.,
- A_x = height factor $(1 + 2\,h_x / h_n)$,
- R_p = element or component response modification factor from Table 4.1.8.18., and

W_p = weight of the component or element.

2) For *buildings* other than *post-disaster buildings*, where $I_EF_aS_a(0.2)$ is less than 0.35, the requirements of Sentence (1) need not apply to Categories 6 through 21 of Table 4.1.8.18.

3) The values of C_p in Sentence (1) shall conform to Table 4.1.8.18.

4) For the purpose of applying Sentence (1) and Categories 11 and 12 of Table 4.1.8.18., elements or components shall be assumed to be flexible or flexibly connected unless it can be shown that the fundamental period of the element or component and its connection is less than or equal to 0.06 s, in which case the element or component is classified as being rigid or rigidly connected.

5) The weight of access floors shall include the *dead load* of the access floor and the weight of permanent equipment, which shall not be taken as less than 25% of the floor *live load*.

6) When the mass of a tank plus its contents or the mass of a flexible or flexibly connected piece of machinery, fixture or equipment is greater than 10% of the mass of the supporting floor, the lateral forces shall be determined by rational analysis.

7) Forces shall be applied in the horizontal direction that results in the most critical loading for design, except for Category 6 of Table 4.1.8.18., where the forces shall be applied up and down vertically.

8) Connections to the structure of elements and components listed in Table 4.1.8.18. shall be designed to support the component or element for gravity loads, shall conform to the requirements of Sentence (1), and shall also satisfy these additional requirements:

a) friction due to gravity loads shall not be considered to provide resistance to seismic forces,
b) R_p for non-ductile connections, such as adhesives or power-actuated fasteners, shall be taken as 1.0,
c) R_p for anchorage using shallow expansion, chemical, epoxy or cast-in-place anchors shall be 1.5, where shallow anchors are those with a ratio of embedment length to diameter of less than 8,
d) power-actuated fasteners and drop-in anchors shall not be used for tension loads,
e) connections for non-structural elements or components of Category 1, 2 or 3 of Table 4.1.8.18. attached to the side of a *building* and above the first level above *grade* shall satisfy the following requirements:
   i) for connections where the body of the connection is ductile, the body shall be designed for values of C_p, A_r and R_p given in Table 4.1.8.18., and all of the other parts of the connection, such as anchors, welds, bolts and inserts, shall be capable of developing 2.0 times the nominal yield resistance of the body of the connection, and
   ii) connections where the body of the connection is not ductile shall be designed for values of $C_p = 2.0$, $R_p = 1.0$ and A_r given in Table 4.1.8.18., and
f) for the purpose of applying Clause (e), a ductile connection is one where the body of the connection is capable of dissipating energy through cyclic inelastic behaviour.

9) Floors and roofs acting as diaphragms shall satisfy the requirements for diaphragms stated in Article 4.1.8.15.

10) Lateral deflections of elements or components shall be based on the loads defined in Sentence (1) and lateral deflections obtained from an elastic analysis shall be multiplied by R_p/I_E to give realistic values of the anticipated deflections.

11) The elements or components shall be designed so as not to transfer to the structure any forces unaccounted for in the design, and rigid elements such as walls or panels shall satisfy the requirements of Sentence 4.1.8.3.(6).

12) Seismic restraint for suspended equipment, pipes, ducts, electrical cable trays, etc. shall be designed to meet the force and displacement requirements of this Article and be constructed in a manner that will not subject hanger rods to bending.

13) Isolated suspended equipment and components, such as pendent lights, may be designed as a pendulum system provided that adequate chains or cables capable of supporting 2.0 times the weight of the suspended component are provided and the deflection requirements of Sentence (11) are satisfied.

Table 4.1.8.18.
Elements of Structures and Non-structural Components and Equipment
Forming Part of Sentence 4.1.8.18.(1)

Category	Part or Portion of *Building*	C_p	A_r	R_p
1	All exterior and interior walls except those in Category 2 or 3(1)	1.00	1.00	2.50
2	Cantilever parapet and other cantilever walls except retaining walls(1)	1.00	2.50	2.50
3	Exterior and interior ornamentations and appendages(1)	1.00	2.50	2.50
4	Floors and roofs acting as diaphragms(2)	-	-	-
5	Towers, *chimneys*, smokestacks and penthouses when connected to or forming part of a *building*	1.00	2.50	2.50
6	Horizontally cantilevered floors, balconies, beams, etc.	1.00	1.00	2.50
7	Suspended ceilings, light fixtures and other attachments to ceilings with independent vertical support	1.00	1.00	2.50
8	Masonry veneer connections	1.00	1.00	1.50
9	Access floors	1.00	1.00	2.50
10	Masonry or concrete fences more than 1.8 m tall	1.00	1.00	2.50
11	Machinery, fixtures, equipment, ducts and tanks (including contents)			
	that are rigid and rigidly connected(3)	1.00	1.00	1.25
	that are flexible or flexibly connected(3)	1.00	2.50	2.50
12	Machinery, fixtures, equipment, ducts and tanks (including contents) containing toxic or explosive materials, materials having a *flash point* below 38°C or firefighting fluids			
	that are rigid and rigidly connected(3)	1.50	1.00	1.25
	that are flexible or flexibly connected(3)	1.50	2.50	2.50
13	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building*	0.70	1.00	2.50
14	Flat bottom tanks (including contents) attached directly to a floor at or below *grade* within a *building* containing toxic or explosive materials, materials having a *flash point* below 38°C or firefighting fluids	1.00	1.00	2.50
15	Pipes, ducts, cable trays (including contents)	1.00	1.00	3.00
16	Pipes, ducts (including contents) containing toxic or explosive materials	1.50	1.00	3.00
17	Electrical cable trays, bus ducts, conduits	1.00	2.50	5.00
18	Rigid components with ductile material and connections	1.00	1.00	2.50
19	Rigid components with non-ductile material or connections	1.00	1.00	1.00
20	Flexible components with ductile material and connections	1.00	2.50	2.50
21	Flexible components with non-ductile material or connections	1.00	2.50	1.00

Notes to Table 4.1.8.18.:

(1) See Sentence 4.1.8.18.(8).
(2) See Sentence 4.1.8.18.(9).
(3) See Sentence 4.1.8.18.(4).