INTRODUCTION

A major revision has been proposed to the earthquake design requirements in Part 4 of the National Building Code of Canada (NBCC) in preparation for the proposed 2005 edition of the NBCC. The proposals have already been published for public comment.

The Canadian National Committee for Earthquake Engineering (CANCEE), which is responsible for the NBCC earthquake provisions, felt that it would be helpful to the engineering community if some background material and commentary were available to assist in the public review of the proposals. The collection of 10 papers presented in this issue of the *Canadian Journal of Civil Engineering* provides background information about the proposed changes. The papers have been peer reviewed in the usual manner by independent reviewers.

It is important to note that the proposed changes are quite extensive. They are published here simply as proposals. They will undoubtedly be modified due to the results of ongoing research, new information continually being brought forth from the ongoing study of both existing and potentially new earthquake data, and in response to public comment. This means that what appears in the following papers may not reflect exactly what finally appears in the 2005 NBCC.

The Canadian National Committee for Earthquake Engineering wishes to thank the Editor and Associate Editors of the Journal for this opportunity, as the Committee believes it will be of assistance in promoting discussion and generating comment in the public review process for the proposed provisions.

The rest of this introductory note details some of the reasons why CANCEE believes a major change is in order.

Reasons for CANCEE Proposed Changes

The Canadian National Committee for Earthquake Engineering is a committee of the National Research Council of Canada whose responsibility is to propose earthquake design provisions to the Part 4 — Structural Design Committee of the NBCC. The Canadian National Committee for Earthquake Engineering was formed in the early 1960s and has functioned continuously since then, providing advice on the earthquake design provisions, with the latest NBCC being the 1995 edition. Given the dramatic amount of knowledge gained about earthquakes in the last two decades, particularly the last decade, and given the evolutionary nature of the code changes within a fairly strict format, the members of CANCEE felt it was time to start over from the beginning for the proposed next version of the earthquake design requirements of the NBCC.

The following lists some of the issues that influenced CANCEE's decision to propose major changes and start fresh:

(1) Previous seismic data were from the early 1980s. Many earthquakes have occurred in Canada since then and have provided much new data.
(2) Many new earthquakes have occurred in areas where both ground and buildings were extensively instrumented (San Francisco, 1989; Northridge, 1994; Kobe, 1995).
(3) Plenty of new data have been obtained on both building and ground response and behaviour.
(4) New ground motion attenuation curves have been developed from this recent data.
(5) Old ground motion data have been re-analyzed, producing different conclusions.
(6) Evidence was found that confirmed a West Coast subduction earthquake hazard, which had not been considered in previous codes.
(7) New data indicated deep, soft soil amplification values were often unconservative in the current Codes and needed substantial revision.
(8) New data on the forces felt by building contents (i.e., mechanical, electrical, and architectural components) were obtained.
(9) Methods became available to produce building response spectra at sites rather than just ground peak accelerations. Response spectra take into account more of the characteristics of an earthquake and give a more complete measure of building behaviour.
(10) It became apparent that West Coast seismicity and East Coast seismicity were quite different and required different approaches based on the relationships between the return periods and the expected seismic ground motion values.
(11) Over the past two decades, there has been an enormous increase in both our knowledge and our experience with earthquakes. During this period, our Code has evolved and incorporated some of this knowledge. However, the changes have followed a National Building Code of Canada change process that lends itself to, and more easily accommodates, change on a clause-by-clause basis. The result of this is an earthquake code that has grown from the first "modern" NBCC requirements in 1965 to the current 1995 version, and this growth has been based on both small and substantial changes, all made to an existing framework of clauses. The result is a set of clauses with some related requirements widely separated and some important ideas attached as a subsection to some nominally related clause.

All of the above, along with a concerted new code development process in the United States, made it clear that the existing NBCC Earthquake Code, while compatible with current United States and New Zealand Codes, was the result of a long evolutionary process and that a fundamental reassessment was required. This reassessment needed to address and incorporate the new data available and the new methods used to utilize the new data, and to recognize the substantial changes being incorporated into the next generation of earthquake codes in the United States and elsewhere.

The Committee has responded to the above and has proposed the following changes:

(1) Revised seismicity maps developed using a new methodology.

(2) Specification of response spectrum values on a city-by-city basis rather than peak ground acceleration on a zonal basis. This is similar to the way snow, wind, and rain are currently presented in our climatic data. This avoids the issue of steps of values at zonal boundaries.
(3) Inclusion of the West Coast subduction earthquake hazard into ground motion values.
(4) Revised formula for calculating the base shear based on the above.
(5) Revised methods to accommodate higher mode effects for static analysis, and making a response spectrum dynamic analysis the base analysis.
(6) Revised foundation values to modify the base shear for various soil conditions.
(7) Use of a 2% in 50-year probability of exceedance rather than the current 10% in 50 years to better capture the differences in ground motions between the east and west sides of the continent.
(8) Revised force reduction values and introduction of overstrength effects to capitalize on the inherent overstrengths of most structures, introduction of new limits, and restrictions on systems.
(9) Introduction of maximum force cutoff values to account for the reduction in response due to foundation rocking or to approximate the "elastic" response.
(10) Introduction of minimum force values at long period responses.
(11) Revised deflection and drift limits in buildings to try to reduce damage.
(12) Introduction of categories for various building irregularities with various special requirements or penalties for certain undesirable cases that have proven problematic and damaging in previous earthquakes.
(13) Specification of restrictions on the type of lateral force systems that could be used in post-disaster buildings, and preclusion of certain irregularities.

The Committee has compared proposed new design values to the old values for information. However, there is currently no intention to try and calibrate the design force levels back to previous design levels as done with previous editions of the Code.

The index for the proposed new earthquake provisions is:

(1) Analysis
(2) Notation
(3) General requirements
(4) Site properties
(5) Importance factor
(6) Structural configuration
(7) Methods of analysis
(8) Direction of loading
(9) Force reduction factors, overstrength, and general restrictions
(10) Other system restrictions
(11) Equivalent static force procedure
(12) Dynamic analysis procedure
(13) Deflections and drift limits
(14) Structural separation
(15) Design provisions
(16) Foundation provisions
(17) Elements of structures, nonstructural components, and equipment

Once again, CANCEE felt that with substantial changes being proposed, it was important to obtain public comment and to provide background material that would help to explain the nature of, and the reasons for, the proposed changes.

Public comment on the proposals is invited by NRC. Copies of the proposed earthquake provisions are attached as Appendix "A" and are marked "Draft". More complete information regarding changes to all of Part 4 of the NBCC can be found on the NRC website (www.nationalcodes.ca), along with forms to be used for public comment. These forms are to be returned to NRC.

Acknowledgements

The background material containing explanations and reasons for the proposed provisions is contained in the papers presented in this section, and I would like to thank the authors for the time and effort in producing the papers, all the reviewers for their careful review, Dr. Jag Humar for acting as the Coordinating Associate Editor and assisting in the planning of this issue, and the editorial board of the CJCE for the opportunity to provide these papers to the engineering design community.

I would like to also thank all the members of CANCEE, both current and previous, for the tremendous amount of work done over the years.

The Canadian National Committee for Earthquake Engineering consists of a broad mix representing seismologists, geotechnical engineers, structural and geotechnical researchers, and structural engineers — all with a background in seismic design and analysis. Amongst the members are also representatives of the CSA material design codes for steel, concrete, masonry, and wood.

The Current Members are:

Mr. R.H. DeVall, Ph.D., P.Eng., of Read Jones Christoffersen Ltd., Dr. J.E. Adams of the National Earthquake Hazards Program and Geological Survey of Canada, Dr. D.L. Anderson, P.Eng., of the Department of Civil Engineering University of British Columbia, Ms. G.M. Atkinson, Ph.D., of the Department of Earth Sciences at Carleton University, Dr. W.D.L. Finn, P.Eng., of the Department of Civil Engineering University of British Columbia, Dr. A.G. Gilles, P.Eng., of the Department of Civil Engineering at Lakehead University, Dr. A.C. Heidebrecht, P.Eng., Professor Emeritus at the Department of Civil Engineering at McMaster University, Dr. J.L. Humar, P.Eng., of the Department of Civil Engineering and Environmental Engineering at Carleton University, Mr. E. Karacabeyli, P.Eng., of Forintek Canada Corporation, Mr. F. Knoll, Ph.D., P.Eng., of Nicolet, Chartrand, Knoll, Ltee., Mr. T.E. Little, P.Eng., of B.C. Hydro, Mr. W.E. McKevitt, Ph.D., P.Eng., of McKevitt Engineering Ltd., Dr. D. Mitchell, P.Eng., of the Department of Civil Engineering McGill University, Mr. C.J. Montgomery, Ph.D., P.Eng., of The Cohos Evamy Partners, Dr. G.C. Rogers with the Geological Survey of Canada, National Resources Canada, and the Pacific Geoscience Centre, Dr. M. Saatcioglu, P.Eng., of the Department of Civil Engineering University of Ottawa, Dr. R. Tremblay, P.Eng., of the Department of Civil, Geological and Mining Engineering at École Polytechnique de Montréal, Mr. A. Wightman, P.Eng., of Klohn Crippen Consultants Ltd., Dr. A.H. Rahman, Research Advisor, Building Envelope and Structure Institute for Research in Construction National Research Council Canada, Ms. C.R. Taraschuk, P.Eng., Technical Advisor, Canadian Codes Centre, Institute for Research in Construction, National Research Council Canada, and Mr. Bill Kendrick, P.Eng., of Westmar Consultants and Dr. Bob Sexsmith, P.Eng., of the Department of Civil Engineering University of British Columbia, both of whom recently retired from the Committee.

The CANCEE members have also met with design practitioners in Vancouver and Montreal to discuss the proposals before putting them forward. I would like to thank these people for giving so freely of their time, experience, ideas, and opinions.

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Appendix A: Part 4

Structural Design

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 by the requirements given in this Subsection except that the requirements of this Subsection need not be considered in design if S(0.2), as defined in Sentence 4.1.8.4.(6), 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 Article 4.1.8.17.(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 4.1.8.17.(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 4.1.911.(9),

C_p = Seismic coefficient for mechanical/electrical equipment, as defined in Sentence 4.1.8.17.(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 that 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 = That level which is uppermost in the main portion of the structure,

Level x = That level which 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 = The 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,

$\overline{N}_{60}$ = 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),

PT = Plasticity index for clays,

R_d = Ductility related force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour, as given in Article 4.1.8.9.,

R_o = Overstrength related force modification factor that accounts 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 4.11.8.17.(1),

S(T) = The design spectral response acceleration, expressed as a ratio to gravitational acceleration, for a period of T, as defined in Sentence 4.1.8.4.(6),

S_a(T) = The 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. - (Equivalent Static Force Procedure),

V_d = Lateral earthquake design force at the base of the structure as determined by Article 4.1.8.12. - (Dynamic Analysis Procedures),

V_e = Lateral earthquake elastic force at the base of the structure as determined by Article 4.1.8.12. – (Dynamic Analysis Procedures),

V_p = Lateral force on a part of the structure, as determined in Article 4.1.8.17.,

$\overline{V}_s$ = 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 parking garages need not be considered storage areas, and the full contents of any tanks (see Appendix A),

W_i, W_x = That portion of W which is located at or is assigned to Level i or x respectively,

W_p = The 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),

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, to 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 have sufficient nonlinear 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, not considered part of the SFRS, such as concrete, masonry, brick or precast walls or panels, shall be 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 such elements shall be 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 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.)

c) in the design of the SFRS when 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 stiffness of all elements of the SFRS, which includes 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) deformation of the panel zones in steel moment frames

c) the effect of the finite size of members and joints

d) sway effects arising from the interaction of gravity loads with the displaced configuration of the structure, and

e) other effects which influence the *buildings* lateral stiffness.

(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 0.2s, 0.5s, 1.0s, and 2.0s, are determined in accordance with Subsection 2.2.1 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.(2) and (3)

		Average Properties in Top 30 m as per Appendix A		
Site Class	***Soil* Profile Name**	***Soil* Shear Wave Average Velocity, $\overline{V}_s$ (m/s)**	**Standard Penetration Resistance, $\overline{N}_{60}$**	***Soil* Undrained Shear Strength, s_u**
A	Hard *Rock*	$\overline{V}_s > 1500$	Not applicable	Not applicable
B	*Rock*	$760 < \overline{V}_s \leq 1500$	Not applicable	Not applicable
C	Very Dense *Soil* and Soft *Rock*	$360 < \overline{V}_s < 760$	$\overline{N}_{60} > 50$	$s_u > 100$kPa
D	Stiff *Soil*	$180 < \overline{V}_s < 360$	$15 \leq \overline{N}_{60} \leq 50$	$50 < s_u \leq 100$ kPa
E	Soft *Soil*	$\overline{V}_s < 180$	$\overline{N}_{60} < 15$	$s_u < 50$ kPa
E		Any profile with more than 3 m of *soil* with the following characteristics: • Plastic index PI > 20 • Moisture content w ≥ 40%, and • Undrained shear strength $s_u < 25$ kPa		
F	(1) Others	Site Specific Evaluation Required		

Notes to Table 4.1.8.4.A

(1) 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) with thickness greater than 8 m.

d) Soft to medium stiff clays with thickness greater than 30 m.

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

3) Where average shear wave velocity, $\overline{V}_s$, is not known, Site Class shall be determined from average energy corrected Standard Penetration Resistance, $\overline{N}_{60}$, or from *soil* average undrained shear strength, s_u, as noted in Table 4.1.8.4A, and $\overline{N}_{60}$ and s_u, shall be 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 $S_a(1.0)$.

5) To determine F_a and F_v for site Class F, site specific geotechnical investigations and dynamic site response analyses shall be performed.

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

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

4.1.8.5 Importance Factor

1) The earthquake importance factor, I_E, shall be as provided for in Table 4.1.8.5.

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) For cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, structures designated as irregular must satisfy the requirements noted in Table 4.1.8.6.

4.1.8.7. Methods of Analysis

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

a) For 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 fundamental lateral periods less than 2 seconds.

c) Structures with structural irregularity, Types 1, 2, 3, 4, 5, 6 or 8 as defined in Table 4.1.8.6. that are less than

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				
	$S_a(0.2) \le 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
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				
	$S_a(1.0) \le 0.1$	$S_a(1.0) = 0.2$	$S_a(1.0) = 0.3$	$S_a(1.0) = 0.4$	$S_a(1.0) \ge 0.5$
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	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)

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

	Importance Factor, I_E	
Importance Category	**ULS**	**SLS** (1)
Low	0.8	
Normal	1.0	(2)
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

Table 4.1.8.6.
Structural Irregularities(6)
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.	(1) (3) (7)
2	**Weight (mass) Irregularity** Weight irregularity shall be considered to exist where the weight, $W_{i,}$ of any *storey* is more than 150 percent of the weight of an adjacent *storey*. A roof that is lighter than the floor below need not be considered.	(1)
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 percent of that in an adjacent *storey*.	(1) (2) (3) (7)
4	**In-plane Discontinuity in vertical lateral force-resisting element** An in-plane offset of a lateral load-resisting element of the SFRS or a reduction in lateral stiffness of the resisting element in the *storey* below.	(1) (2) (3) (7)
5	**Out-of-Plane Offsets** Discontinuities in a lateral force path, such as out-of-plane offsets of the vertical elements of the SFRS.	(1) (2) (3) (7)
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.	(1) (3) (4) (7)
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.	(5) (7)

Notes: To Table 4.1.8.6.A - Requirements for Irregular Structures
(1) See Article 4.1.8.7.
(2) See Article 4.1.8.15.
(3) See Article 4.1.8.10.
(4) See Sentences 4.1.8.11.(9), (10), and 4.1.8.12.(7)
(5) See Article 4.1.8.8
(6) One *storey* penthouses with a weight less than 10% of the level below need not be considered in applying this table.
(7) See Appendix A

20 m in height and have a fundamental lateral period less than 0.5 seconds.

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 $I_EF_aS_a(0.2)$ is less than 0.35, independent analyses about any two orthogonal axes is permitted.

c) When the components of the SFRS are not oriented along a set of orthogonal axes and $I_EF_aS_a(0.2)$ 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 associated 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 penthouses whose weight is less than 10% 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.(3) 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 in Table 4.1.8.9., then such a structural system will qualify for a

Table 4.1.8.9.
SFRS Force Modification Factors (R_d), System Overstrength Factors (R_o) and General Restrictions [(1)]
Forming Part of Sentence 4.1.8.9 (1)

			Restrictions [(2)]				
			Cases Where $I_EF_aS_a(0.2)$				Cases Where
Type of SFRS	R_d	R_o	<0.2	≥0.2 to <0.35	≥0.35 to ≤0.75	>0.75	$I_EF_vS_a(1.0)$ >0.3
Steel Structures Designed and Detailed According to CSA S16							
• 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	NP	NP
• Moderately ductile concentrically braced frames							
• Non-chevron braces	3.0	1.3	NL	NL	40	40	40
• Chevron 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							
• Non-chevron braces	2.0	1.3	NL	NL	60	60	60
• Chevron braces	2.0	1.3	NL	NL	60	60	60
• Tension only braces	2.0	1.3	NL	NL	40	40	40
• Ductile eccentrically braced frames	4.0	1.5	NL	NL	NL	NL	NL
• Ductile frame plate shearwalls	5.0	1.6	NL	NL	NL	NL	NL
• Moderately ductile plate shearwalls	2.0	1.5	NL	NL	60	60	60
• Conventional construction of moment frames, braced frames or shearwalls	1.5	1.3	NL	NL	15	15	15
• Other steel SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	NP
Concrete Structures Designed and Detailed According to 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

Table 4.1.8.9 (*concluded*).

• 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 shearwalls	3.5	1.6	NL	NL	NL	NL	NL
• Moderately ductile shearwalls	2.0	1.4	NL	NL	NL	60	60
• Conventional construction							
• Moment resisting frames	1.5	1.3	NL	NL	15	NP	NP
• Shearwalls	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 086							
• Shearwalls							
• Nailed shearwalls – wood based panel	3.0	1.7	NL	NL	30	20	20
• Shearwalls – wood based and gypsum panels in combination	2.0	1.7	NL	NL	20	20	20
• Braced or moment resisting frame 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 shearwalls	2.0	1.5	NL	NL	60	40	40
• Limited ductility shear walls	1.5	1.5	NL	NL	40	30	30
• Conventional Construction							
• Shearwalls	1.5	1.5	NL	6030	30	15	15
• Moment resisting frames	1.5	1.5	NL		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

Notes to Table 4.1.8.9.:
(1) See Article 4.1.8.10.
(2) Notes on restrictions:
(a) NP in table means not permitted.
(b) Numbers in table are maximum height limits in metres.
(c) NL in table means system is permitted and not limited in height as an SFRS. Height may be limited elsewhere in other Parts.
(d) The most stringent requirement governs.

value of R_d and R_o corresponding to the equivalent type in that Table (see Appendix A).

4.1.8.10. Other System Restrictions

1) Except as required in Clause (2)(b), structures with a Discontinuity in Capacity – Weak *Storey* as described in Table 4.1.8.6. - Type 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 1.5 R_d R_o.

2) *Post-disaster buildings* shall:

a) Not have any irregularities conforming to Table 4.1.8.6. - Type 1,3,4,5 and 7 for cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35.

b) Not have an irregularity conforming to Table 4.1.8.6. - Type 6 - Discontinuity in Capacity – Weak *Storey*.

c) Have an SFRS with a R_d of 2.0 or greater.

3) For cases where $I_E F_v S_a(1.0)$ is greater than 0.3, 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 defined in Table 4.1.8.6., except that for *buildings* less than 20 m in height and with fundamental lateral periods less than 0.5, such irregularities are permitted providing the earthquake design forces and deflections determined from the requirements of this Subsection are increased by 50%.

4.1.8.11. Equivalent Static Force Procedure For Structures Satisfying the Conditions of Article 4.1.8.6.

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

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

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

except that V shall not be taken less than

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

and for an SFRS with an R_d equal to or greater than 1.5, V need not be taken greater than (2/3) $S(0.2)I_EW/(R_dR_o)$.

3) The fundamental lateral period, T_a, in the direction under consideration in Sentence (2) shall be determined as:

a) For moment-resisting frames which resist 100% of the required lateral forces and 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
   iii) 0.1 N for other moment frames

b) $0.05\ (h_n)^{3/4}$ for other structures where h_n is in metres, or

c) 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 (a)
   ii) for braced frames, T_a shall not be taken greater than 1.5 times that determined in (b)

(See Appendix A.)

4) The weight, W, of the *building* shall be calculated in accordance with 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 $T_a \le 1.0$	M_v For $T_a \ge 2.0$	J For $T_a \le 0.5$	J For $T_a \ge 2.0$
<8.0	Moment resisting frames or “coupled walls” [3]	1.0	1.0	1.0	1.0
	Braced frames	1.0	1.0	1.0	0.8
	Walls, wall-frame systems, other systems [4]	1.0	1.2	1.0	0.7
≥8.0	Moment resisting frames or “coupled walls” [3]	1.0	1.2	1.0	0.7
	Braced frames	1.0	1.5	1.0	0.5
	Walls, wall-frame systems, other systems [4]	1.0	2.5	1.0	0.4

Notes:
(1) For values of M_v between periods of 1.0 and 2.0 s, the product $S(T_a)\cdot M_V$ shall be obtained by linear interpolation.
(2) Values of J between periods of 0.5 and 2.0 s shall be obtained by linear interpolation.
(3) 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.
(4) For hybrid systems, use values corresponding to walls or carry out a dynamic analysis 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 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 formula

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

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

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

where:
$J_x = 1.0$ for $h_x \ge 0.6h_n$
$J_x = J + (1 - J)(h_x / 0.6h_n)$ for $h_x < 0.6h_n$
and J is the base overturning moment reduction factor conforming to Table 4.1.8.11.

8) Torsional effects concurrent with the effects of the forces in Sentence (6) and due to the following shall be considered in the design of the structure as per Sentence (10):

a) Torsional moments introduced by eccentricity between the centres of mass and resistance and their dynamic amplification.

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,

δ_{max} = the 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 $\pm 0.10D_{nx}$ from the centres of mass at each floor.

δ_{ave} = the average of the displacements at the extreme points of the structure at level x produced by the above forces.

B = the 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.

10) Torsional effects shall be accounted for as follows:

a) for a *building* with B ≤ 1.7 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) $T_x = F_x(e_x + 0.1D_{nx})$
   ii) $T_x = F_x(e_x - 0.1D_{nx})$
   where F_x is the lateral force at each level as given by Sentence (6) and where each element in the *building* is designed for the most severe effect of the above load cases.

b) for a *building* with B > 1.7 by a dynamic analysis procedure as specified in Article 4.1.8.12

4.1.8.12. Dynamic Analysis Procedures

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) Nonlinear Dynamic Analysis Method, 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.(6)

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.(6) (see Appendix A).

4) The elastic base shear, V_e obtained from a Linear Dynamic Analysis shall be multiplied by the Importance factor I_E as defined in Article 4.1.8.5. and shall be divided by R_dR_o as defined in Article 4.1.8.9. to obtain the design base shear V_d.

5) If the base shear V_d obtained in Sentence 4 is less than 80% of the lateral earthquake design force, V, of Article 4.1.8.11., V_d shall be taken as 0.8V, except that 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 4 and 100% of V.

6) The values of elastic *storey* shears, *storey* forces, member forces, and deflections obtained from the Linear Dynamic Analysis shall be multiplied by V_d/V_e to determine their design values, where V_d is the base shear obtained according to Sentences (4) and (5).

7) The effects of accidental torsional moments acting concurrently with and due to the lateral earthquake forces shall be accounted for by the following methods :

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

b) if B as defined in Sentence 4.1.8.11.(9) is less than 1.7, it is permitted to use a 3-dimensional dynamic analysis with the centres of mass shifted by a distance -0.05_{nx} and $+0.05_{nx}$.

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) The largest interstorey deflection at any level based on the lateral deflections as calculated in Sentence (2) shall be limited to $0.01h_s$ for *post-disaster buildings*, 0.02 h_s for schools, and 0.025 h_s for all other *buildings*.

4) The deflections as calculated in Sentence (2) shall be used to account for sway effects as required by Sentence 4.1.3.2.(9) (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_d R_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) Diaphragms and their 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 in the diaphragm due to the following cases, whichever governs (see Appendix A):

a) Forces in the diaphragm due to loads determined in Articles 4.1.8.11. or 4.1.8.12. applied to the diaphragm 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.

b) A minimum force corresponding to the design based shear divided by N for the diaphragm at Level x.

2) For cases where $I_E F_a S_a(0.2)$ is equal to or greater than 0.35, the elements supporting any discontinuous wall, column or braced frame, shall be designed for the forces developed above the discontinuity as the lateral load capacity of the structure is developed (see Appendix A).

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

4) Where earthquake actions 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.)

5) Except as provided for in Sentence (6), the design forces in elements need not exceed the following values:

a) the forces determined in accordance with Sentence 4.1.8.7.(1) multiplied by $R_d R_o$ when the SFRS is designed for a value of R_d = 2.0 (see Appendix A).

b) the forces determined in accordance with Sentence 4.1.8.7.(1), multiplied by 1.4 $R_d R_o$ when the SFRS is designed for a value of R_d < 2.0 (see Appendix A).

6) The design forces in elements need not exceed the maximum values corresponding to *foundation* rocking as provided for in 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 need not exceed 0.5 $R_d R_o$ times those determined in Sentence 4.1.8.7.(1) (see Appendix A).

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

3) For cases where $I_E F_a S_a$ (0.2) is equal to or greater than 0.2, 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.

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 capacity of the *pile*.

4) At sites where $I_E F_a S_a$ (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 $I_E F_a S_a$ (0.2) is greater than 0.75, the following requirements shall be satisfied:

a) A *pile*, drilled pier, or *caisson* 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).

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 required by Sentence (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 0.15 $I_E F_a S_a(0.2)$, unless it can be demonstrated that equivalent restraints can be provided by other means (see Appendix A).

7) The potential for liquefaction and the consequences, such as significant ground displacements and loss of *soil* strength and stiffness, shall be evaluated based on Ground Motion Parameters referenced in Subsection 2.2.1., and shall be taken into account in the design of the structure and its *foundations* (see Appendix A).

4.1.8.17 Elements of Structures, Nonstructural Components and Equipment

1) Except as provided for in Sentence 4.1.8.17.(2) and 4.1.8.17.(8), elements and components of *buildings* as described in Table 4.1.8.17. 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 Article 4.1.8.17.(10), and, for a lateral force, V_p equal to:

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

where

F_a is defined in Table 4.1.8.4.B.

$S_a(0.2)$ is the Spectral Response Acceleration Value at 0.2 sec 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 = $C_p A_r A_x / R_p$. 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

C_p Element or Component Factor from Table 4.1.8.17.

R_p Element or Component Response Modification Factor from Table 4.1.8.17.

A_r Element or Component Force Amplification Factor from Table 4.1.8.17.

A_x Height Factor $(1 + 2\ h_x/h_n)$

W_p Weight of the component or element

and the force V_p shall be applied through the centre of mass of the element or component.

2) For non *post-disaster buildings*, where $I_E F_a S_a(0.2)$ is less then 0.35, the requirements of Sentence 4.1.8.17.(1) need not apply to Categories 6 through 21 of Table 4.1.8.17.

3) The values of C_p in Sentence 4.1.8.17.(1) shall conform to Table 4.1.8.17.

4) For the purpose of applying Sentence 4.1.8.17.(1) and Table 4.1.8.17., Category (11) and (12):

a) elements or components that are both rigid and rigidly connected are defined as those having a fundamental period for the element or component and connection less than or equal to 0.06 sec.; and

b) flexible elements or components or connections are de-

Table 4.1.8.17.
Elements of Structures and Nonstructural Components and Equipment
Forming Part of Sentence 4.1.8.17.(1)

Category		Cp	Ar	Rp
1	All exterior and interior walls except those of Category 2 and 3 (1)	1.00	1.00	2.50
2	Cantilever parapet and other cantilever walls except retaining walls (1)	1.00	2.5	2.50
3	Exterior and interior ornamentations and appendages (1)	1.00	2.5	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	2.5	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 over 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 flashpoint below 38ºC or fighting 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 flashpoint below 38ºC or firefighting fluids.	1.00	1.00	2.50
15	Pipes, ducts, cable trays (including content)	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, conduit	1.00	2.50	5.00
18	Rigid components with ductile material and connections	1.00	1.00	2.50
19	Rigid components with nonductile 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 nonductile material or connections	1.00	2.50	1.00

Notes to Table 4.1.8.17:
(1) See Sentence 4.1.8.17.(8)
(2) See Sentence 4.1.8.17.(9)
(3) See Sentence 4.1.8.17.(4)

fined as those having a fundamental period greater than 0.06 sec.

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 contents 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 result in the most critical loading for design except for Category 6, Table 4.1.8.17. where the forces shall be applied up and down vertically.

8) Connections to the structure for elements and components in Table 4.1.8.17. shall be designed to support the component or element for gravity loads, the requirements of Sentence 4.1.8.17.(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 powder-activated 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 d) than 8.

d) Powder-activated fasteners and drop-in anchors shall not be used for tension loads.

e) Connections for nonstructural elements or components of Categories 1, 2 or 3 of Table 4.1.8.17. 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 from the table, and the fasteners such as anchors, welds, bolts and inserts shall be designed for values of C_p and A_r from the table, and $R_p = 1.0$.

   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 from the table.

f) For the purposes of applying (e), a ductile connection is one where the body of the connection yields at its design load (see Appendix A).

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

10) Lateral deflections of elements or components shall be based on the loads defined in Article 4.1.8.17.(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 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, etc. may be designed as a pendulum system provided that adequate chains or cables are provided which are capable of supporting 2.0 times the weight of the suspended component and the deflection requirements of Sentence (11) are satisfied.