PROPOSED REVISED EARTHQUAKE DESIGN

PROVISIONS FROM CANCEE

Ron DeVall, June 7, 2001

INTRODUCTION

1. Over the last 20 years, the following has occurred:

   (a) Several large, well instrumented and documented earthquakes have occurred that affected developed, modern areas - Mexico City (1985), San Francisco (1989), Los Angeles (1994) and Kobe (1995).

   (b) These were well recorded (and instrumented) events and provided much new information on ground motions, site soil effects, building structural behaviour, and the behaviour of architectural and mechanical components in buildings.

   (c) Several strong motion earthquakes occurred in Canada - Yukon, Quebec.

   (d) Older Canadian seismic data was reviewed and revised using new techniques.

   (e) Several new ground motion relations (i.e., attenuation curves) were developed.

   (f) We can produce area specific response spectrums now - as opposed to just peak ground accelerations.

   (g) Materials codes have substantially changed over the last 20 years in the area of earthquake design provisions.

   (h) Other jurisdictions have undertaken extensive reviews and produced extensive revisions to their earthquake codes (i.e., New Zealand, and in particular, the United States).

2. Our current Code was first modernized in 1965 and has since been built upon, and evolved, until now. The largest changes were in 1985 (seismicity revised) and 1990 (R values introduced). Basically, it is a product of evolution, with each change built into the existing framework and format (which is driven by the NBCC change process).

3. Given Items 1 and 2 above, it was the CANCEE Committee's view that it was time for a complete review of all clauses, starting from a blank sheet of paper, and incorporating as much of the knowledge information and experience that is noted in Item 1 above as possible.

4.1.8 LOADS AND EFFECTS DUE TO EARTHQUAKES

.1 Scope

.2 Notation

.3 General Requirements

.4 Site Properties (Spectrum, Soil Effects)

.5 Importance Factor

.6 Structural Configuration (Irregularities)

.7 Methods of Analysis (Limited Static Analysis, Irregularities Leads to Dynamic Analysis for T>.5’s)

.8 Direction of Loading Clause

.9 System R Values and General Restrictions

.10 Other System Restrictions

.11 Equivalent Static Procedure

.12 Dynamic Analysis Procedure

.13 Deflection and Drift

.14 Structural Separation

.15 Design Provisions

.16 Foundation Provisions

.17 Elements of Structures, Nonstructural Components, and Equipment

The “flow is: General, Site Properties, Irregularities, R Values, Restrictions and Limits, Analysis, Deflections, Design, Foundations, Parts and Portions.

4.1.8.3 GENERAL REQUIREMENTS

Emphasizes

(a) Structure must have a clearly defined load path.

(b) Structure must have a clearly defined Seismic Force Resisting System (SFRS).

(c) The SFRS alone "resists" the earthquake loads and its stiffness alone is used to calculate the deflections.

(d) Other elements of the structure are not considered to resist earthquake loads and effects (as their stiffness may degrade substantially) but must be able to sustain their gravity loads as they are displaced through the expected deflections. This may require nonlinear behaviour.

(e) Stiff elements such as wall panels, masonry infill, etc. not part of the SFRS must be separated to prevent interaction or be made part of the SFRS.

(f) Modelling must incorporate finite joint sizes, cracked concrete section properties, steel panel zone stiffness, etc.

Site Properties

1. Contains site "spectral" response definition - i.e., spectral response for 2% in 50 years at 0.2, 0.5, 1.0, 2.0 and 4.0 seconds (sends you to climatic data).

   (a) Victoria values are: 1.2 g, 0.83 g, 0.38 g, 0.19 g, 0.095 g.

   (b) Vancouver values are: 1.0 g, 0.67 g, 0.34 g, 0.18 g, 0.09 g.

   (c) "Old" 10% in 50 year short period approximation.

   For Vancouver, was 0.2 x 3 = 0.6 g, and
   for Victoria, was 0.3 x 1.4 x 3 = 1.26 g.

   Note: For West, 2% in 50 is about a 1.7 increase over 10% in 50 for the short period range. Victoria has come "down", relatively speaking, from 1995. The increase in the East for short period range is about 2.5 for 2% in 50 vs. 10% in 50.

   (d) Values provided on a city-by-city basis, similar to wind and snow (no zones).

   (e) Note: More or less falls of at 1/T.

2. Contains soil amplification factors for various soil types (see attached).

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

S(T)	=	$F_a S_a(0.2)$ for $T \leq 0.2$ s
	=	$F_v S_a(0.5)$ or $F_a S_a(0.2)$ whichever is smaller for T = 0.5
	=	$F_v S_a(1.0)$ for T = 1.0 s
	=	$F_v S_a(2.0)$ for T = 2.0 s
	=	$F_v S_a(2.0)/2$ for $T \geq 4.0$ s

Table 4.1.9.4.A.
Site Classification for Seismic Site Response -
Forming Part of Sentences 4.1.9.4.(2) and (3)

Site Class	Soil Profile Name	Average Properties in Top 30 m as per Appendix A		
		Soil Shear Wave Average Velocity, $\bar{V}_s$ (m/s)	Standard Penetration Resistance, N_{60}	Soil Undrained~~Unconfined~~ Shear Strength, s_u
A	Hard Rock	$\bar{V}_s > 1500$	Not applicable	Not applicable
B	Rock	$760 < \bar{V}_s \leq$ ~~<~~ 1500	Not applicable	Not applicable
C	Very Dense Soil and Soft Rock	$360 < \bar{V}_s < 760$	$\bar{N}_{60} > 50$	$s_u > 100$kPa
D	Stiff Soil	$180 < \bar{V}_s < 360$	$15 \leq \bar{N}_{60} \leq 50$	$50 < s_u \leq$ ~~<~~ 100kPa
E	Soft Soil	$\bar{V}_s < 180$	$\bar{N}_{60} < 15$	$s_u < 50$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			

Notes to Table 4.1.9.4.A

Note (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) ~~Very thick S~~Soft to medium stiff clays with thickness greater than 30 m.

Table 4.1.9.4.B

Values of F_a Forming Part of Sentence 4.1.9.4~~1~~.(4)

Values of F_a as a Function of Site Class and ~~T = 0.2 s~~ $S_a(0.2)$~~Spectral Acceleration.~~

Site Class	Values of F_a				
	$S_a(0.2) \leq 0.25$	$S_a(0.2) = 0.50$	$S_a(0.2) = 0.75$	$S_a(0.2) = 1.00$	$S_a(0.2) = 1.25$
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 4.1.9.4 (B):
(1) See Sentence 4.1.9.4 (5)

Table 4.1.9.4.C

Values of F_v Forming Part of Sentence 4.1.9.4~~1~~.(4)

Values of F_v as a Function of Site Class and ~~T = 1.0 s Spectral Acceleration.~~ $S_a(1.0)$

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)

4.1.8.6 IRREGULARITIES

Defines irregularities such as:

(a) Changes in mass and/or stiffness from floor to floor.

(b) Offsets in the lateral system (i.e., wall offsets)

(c) Torsionally flexible buildings

(d) Weak storey

(e) Nonorthagonal lateral load-resisting systems

Sets the stage for limitations such as:

(a) Must do dynamic analysis

(b) Must increase forces in some cases

(c) Not allowed above one storey in some cases

R Values

- R_d = "ductility" reduction.
- R_o = system "reliable" overstrength reduction.
- Height limits taken from various clauses and put into table instead. The clauses have been deleted.
- Applied to 2% in 50 load levels - i.e., divide elastic force by (R_d x R_o).

Table 4.1.9.9.A.
SFRS Force Modification Factors (R_d), System Overstrength Factors (R_o) and General Restrictions (1)
Forming Part of Sentence 4.1.9.9 (1)

TYPE OF SFRS	R_d	R_o	RESTRICTIONS (2) Cases Where $IF_aS_a(0.2)$ ≤0.2	≥0.2 to <0.35	≥0.35 to ≤0.75	>0.75	Cases Where $IF_aS_a(1.0)$ >0.3
Steel Structures Designed and Detailed According to CSA S16-2001							
▪ 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	NL
▪ Moderately ductile concentric braced frames							
• Non-chevron braces	3.0	1.5	NL	NL	40	40	NL
• Chevron braces	3.0	1.5	NL	NL	40	40	NL
• Tension only braces	3.0	1.5	NL	NL	20	20	NL
▪ Limited ductility concentric 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	60
▪ Ductile eccentric braced frames	4.0	1.5	NL	NL	NL	NL	NL
▪ Ductile frame plate shearwall	5.0	1.6	NL	NL	NL	NL	NL
▪ Moderately ductile plate shearwall	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	60
▪ Other steel SFRS(s) not defined above	1.0	1.0	15	15	NP	NP	60
Concrete Structures Designed and Detailed According to CSA A23.3-xx							
▪ Ductile moment resisting frames	4.0	1.7	NL	NL	NL	NL	NL
▪ Moderately ductile moment resisting frames	2.5	1.4	NL	NL	NL	60(?)	60(?)
▪ Ductile coupled wall	4.0	1.7	NL	NL	NL	NL	NL
▪ Ductile partially coupled wall	3.5	1.7	NL	NL	NL	NL	NL
▪ Ductile shearwall	3.5	1.6	NL	NL	NL	NL	NL
▪ Moderately ductile shearwall	2.0	1.4	NL	NL	NL	60(?)	60

	Type of SFRS	R_d	R_o	Restrictions (2)				
				Cases Where $I F_a S_a(0.2)$				Cases Where
				≤0.2	≥0.2 to <0.35	≥0.35 to ≤0.75	>0.75	$I F_a S_a(1.0)$ >0.3
▪	Conventional construction • Moment resisting frame • Shearwalls	 1.5 1.5	 1.3 1.3	 NL NL	 NL NL	 15 15	 NP 15(?)	 60 60
▪	Other concrete SFRS(s) not listed above	1.0	1.0	15	15	NP	NP	60
Timber Structures Designed and Detailed According to CSA 086-xx								
▪	Shearwalls • Nailed shearwalls - wood based panel • Shearwalls - wood based and gypsum panels in combination	 3.0 2.0	 1.7 1.7	 NL NL	 NL NL	 20 20	 20 20	 NL 60
▪	Braced or moment resisting frame with ductile connections • Moderately ductile • Limited ductility	 2.0 1.5	 1.7 1.7	 NL NL	 NL NL	 15 NP	 15 NP	 60 60
▪	Other wood or gypsum based SFRS(s) Not listed above	1.0	1.0	15	15	NP	NP	60
Masonry Structures Designed and Detailed According to CSA S304.1-xx								
▪	Moderately ductile shearwall	2.0	1.5	NL	NL	NL	60	60
▪	Limited ductility • Shearwalls • Moment resisting frames	 1.5 1.5	 1.5 1.5	 NL NL	 NL NL	 15(?) NP	 15 NP	 60 60
▪	Unreinforced masonry	1.0	1.0	NL	15(?)	NP	NP	60
▪	Other masonry SFRS(s) not listed above	1.0	1.0	15	NP	NP	NP	(60)

Notes to Table 4.1.9.9.A:

(1) See Article 4.1.9.10.

(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.

4.1.8.10 Other System Restrictions

(a) Weak storeys are prohibited except for one storey buildings or other strict requirements such as "elastic" response in low earthquake zones.

(b) Post-disaster buildings have several irregularities prohibited in higher earthquake zones.

(c) For tall buildings in high zones, walls must be continuous to the foundations.

Static Analysis

1. For regular buildings - <60 m and T = 2 seconds.

2. Irregular buildings - <20 m and T = 0.5 seconds.

3. Base shear lower limit at T = 2 second period.

4. Short period force cut-off at 2/3 of short period value.

5. Can calculate period based on methods of mechanics except that moment and braced frames still tied back to 1.5 times code formulae.

6. Wall periods are as calculated.

7. J factor still there, but no longer limited to 0.8.

8. Higher mode shear factor M_v introduced to explicitly account for higher mode shapes. Used to be done using "spectrum" equal to PGA * $3/T^{1/2}$ - i.e., Fell of at Root T, not T.

9. For Torsion, 1.5 ∗ e term is gone, but accidental torque is still 0.1 x D.

4.1.9.11. Equivalent Static Force Procedure For Structures Satisfying the conditions of Article 4.1.9.6.

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

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

$$V = S(T_a)M_vIW/(R_dR_o)$$

except that V shall not be taken ~~as no~~ less than

$$S(2.0)M_vIW/(R_dR_o)$$

and for an SFRS with an R_d equal to or greater than 1.5, V need not be taken greater than

$$\frac{2}{3}S_a(0.2)IW/(R_dR_o)$$

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 4.1.9.3 (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)

Table 4.1.9.11.A.
Higher Mode Factor M_v and Base Overturning Reduction Factor J [1,2]
Forming Part of Sentence 4.1.9.11 (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 $T_a \geq 2.0$	J For $T_a \leq 0.5$	J For $T_a \geq 2.0$
<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. Values of M_v between periods of 1.0 and 2.0 s 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 4.1.9.12.

Dynamic Analysis

1. Take results, multiply by I and divide by R_dR_o.

2. For regular structures, cannot be less than 80% of static.

3. For irregular structures, must be greater of dynamic or static.

4. If building torsionally flexible, must apply accidental torsion statically, but can use dynamic load distribution.

Deflection and Drift

1. Multiply deflections by R_oR_d/I to get realistic deflections.

2. I value taken out of deflection calculation.

3. Limits on drift (at 2% in 50 loads - basically tightened up a bit):

   - Post-disaster - 0.01 h
   - Schools - 0.02 h
   - Normal - 0.025 h

4. Simplistically back-figuring these for “less rare” events, they tend to satisfy the drift and return period recommendations of the SEAOC “Vision 2000” requirements and their implied performance levels.

5. T = 2 second force cut-off and period tieback for frames means deflections may be conservative. Probably should allow deflections based on actual period as long periods produce big displacements. Applying “short period” forces to long period buildings (as we currently do) is probably unnecessarily conservative. Under review by Committee.

4\. Building separation is now calculated on RMSS values, not absolute sum of deflections.

4.1.8.15 Design Provisions

(a) Diaphragms and connections must stay elastic and not yield.

(b) Minimum diaphragm force.

(c) Elements supporting “cut off” walls, columns, etc. must develop the lateral capacity of the cut-off element.

(d) Special capacity design requirements for elements in two-way systems (i.e., end columns common to two orthogonal frames).

(e) Force “cut-offs” at R=1.0 levels for nonbrittle systems and 1.4 R times levels for brittle systems (R<2.0).

(f) Force cut-offs for cases where the footings rock at R=2.0 load levels.

4.1.8.17 Elements of Structures, Nonstructural Components and Equipment

(a) Basically reflects NEHRP provisions from the U.S. Based on much data from Loma Pieta (San Francisco - 1989) and Northridge (Los Angeles - 1994) earthquakes.

(b) Requirements excluded for mechanical and electrical components in low earthquake zones.

(c) Large connection factor gone.

(d) Special requirements for “nonstructural walls” hanging off the edge of buildings about the first floor.