Structural Engineering

Seismic Design of Operational and Functional Components (OFCs) of Buildings

by Rola Assi
September 2003

Structural Engineering Series No. 2003-03

Department of Civil Engineering
and Applied Mechanics

McGill University
Montreal

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Literature review report

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This report is a bibliographical review on the seismic design of operational and functional components in buildings.

Ce rapport est une revue bibliographique sur la conception sismique des composantes operationnelles et fonctionnelles dans les bâtiments.

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1. Introduction

A building is made up of various components that can be divided into two groups: structural components and operational and functional components (OFCs).

According to CSA S832-01 (CSA 2001), operational and functional components are those systems and elements housed or attached to floors, roof, and walls of a building or industrial facility, that are not part of the main or intended load bearing structural system. However, these components may contribute to the structural integrity of the building, depending on their location, type of construction, and method of fastening. Like structural components, OFCs may be subjected to large seismic forces and must be designed to resist these forces.

Some of the alternative names by which these systems are known are: non-structural components or elements, secondary systems, building attachments and nonbuilding components. According to Chen and Soong (1988), secondary systems can be classified into non-structural secondary systems and structural secondary systems. For the latter type, concern is not only to their seismic behaviour, but also to the interaction with the primary structural system.

Operational and functional components can generally be divided into three sub-components according to CSA S832-01 (CSA 2001), and Villaverde (1997): architectural (internal and external like cladding, interior partition walls, ceilings and lights, raised computer floor systems, racks and shelving, etc.), building services including mechanical and electrical systems (electrical power distribution systems, heating, ventilation, and cooling systems, fire protection systems, emergency power generation, *telecommunications, etc.) and building contents (supplies, computer systems, record* storage, etc.). OFCs represent a high percentage of the total capital investment for buildings and their failure in an earthquake can disrupt the function of the building and pose a significant safety risk to building occupants as well, so these structures are far from being secondary in importance.

In fact, the development of seismic design provisions for nonstructural components has lagged behind that of primary structures. Considerable progress has been made over the last two decades in the seismic analysis of structural systems, resulting in substantial improvement in analysis, design and construction of buildings, bridges, and other industrial facilities, under seismic excitation (Filiatrault et al. 2001b). More recently, there has been an increasing concern about the seismic performance of secondary systems attached to primary structures. A review of the typical damage sustained in recent earthquakes (McKevitt et al. 1995; Kao et al. 1999; Naeim 1999; Naeim 2000; Filiatrault et al. 2001b) highlights the fact that the performance of nonstructural components, equipment and systems is the greatest contributor to damage, losses and business interruption in most facilities. The vulnerabilities of the nonstructural components in modern buildings were not exposed until the 1964 Alaska and 1971 San Fernando earthquakes (Lagorio 1990), where it became clear that damage to nonstructural elements not only can result in major economic loss, but also can pose threat to life safety, even when nonstructural damage was not significant. In moderate earthquakes, damage to critical equipment and contents may be more important than damage to structural framework, and earthquakes of moderate intensity are more frequent than earthquakes of high intensity.

The objective of this report will be to assess the current state of knowledge in seismic design of non-structural building components as reflected in codes, standards and guidelines currently in use in Canada and in the United States.

2. Classification of non-structural components

From a structural perspective, non-structural components can be classified into either deformation sensitive (drift ratio) or acceleration sensitive (force), and many components are both deformation and acceleration sensitive (NEHRP 2000; Naeim 2001).

2.1 Deformation sensitive components

The failure of deformation sensitive components (most of architectural components, ducts, trays, line services, etc.) is caused either by excessive interstory displacement or drift, or incompatible stiffness between the building structure and the component, or interaction between adjacent structural systems and non-structural systems, or multiple structure connection points.

A good seismic performance of deformation sensitive components can be obtained by two general design strategies (Naeim 1989):

- An isolation approach, in which elements are provided with sufficient separation from the structure so that the deformation of the structure will not produce appreciable stress on the element.
- A deformation approach, in which the elements are designed with the intention that they will be able to undergo the required deformation. This can be achieved either by controlling the interstory drift of the supporting structure (which governs the design for higher structural performance) or by designing the component or system to accommodate the expected lateral displacements without damage.

2.2 Acceleration sensitive components

Acceleration sensitive components (most of electrical and mechanical components) are vulnerable to shifting or overturning if the anchorage or bracing is inadequate, and to excessive shaking. These components must be designed and anchored so as not to transfer to the structural system any forces not accounted for in the design. Any interaction with rigid elements such as walls and the structural system shall be designed so that the capacity of the structural system is not impaired by the action or failure of the rigid elements (all editions of NBCC).

The following photos (figures 1 to 6) illustrate damages to different non-structural components in previous earthquakes.

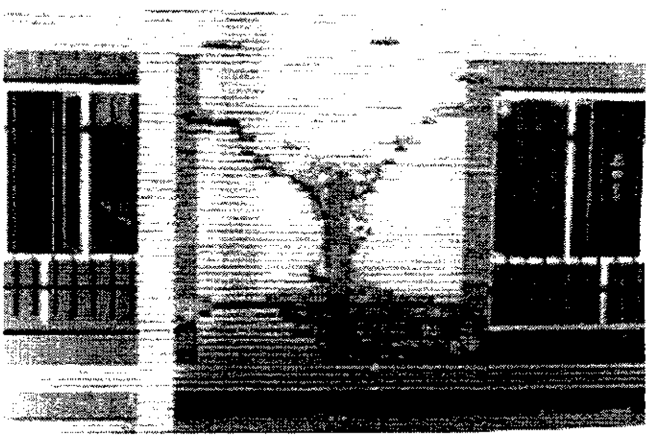

Figure 1 Damage to walls (2001 Nisqually earthquake)

http://www.amre.com/content/home/news/nisqually_quake.htm#nonstruct

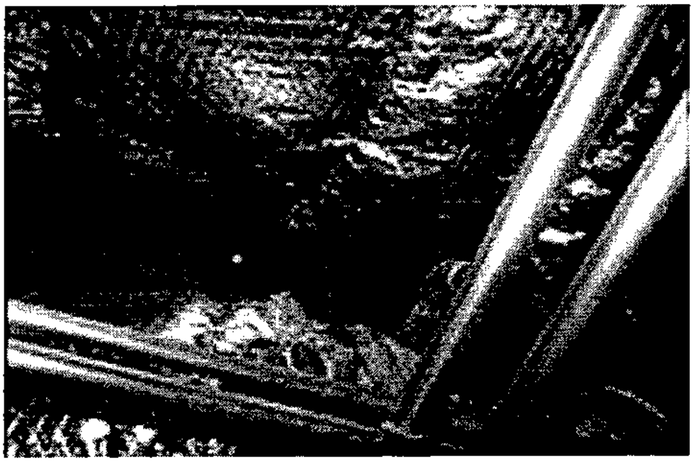

Figure 2 Damage to pipes (1994 Northridge earthquake)

(http//nisee. berkeley.edu)

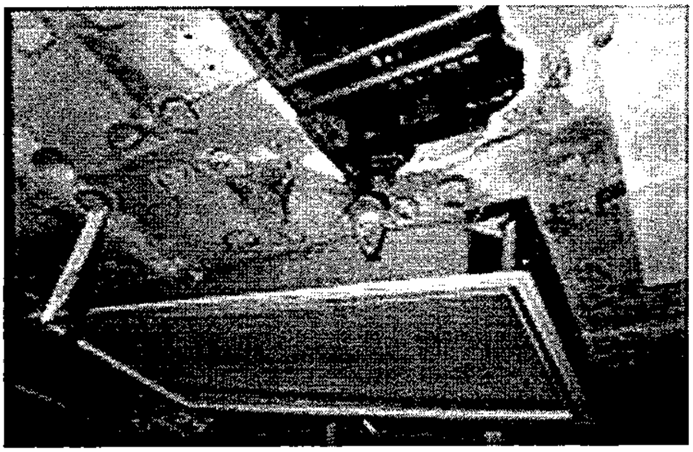

Figure 3 Damage to ceilings (1994 Northridge earthquake)

(http//nisee. berkeley.edu.)

Figure 4 Damage to unreinforced masonry cladding (1988 Saguenay earthquake) (Mitchell and Tinawi 1995)

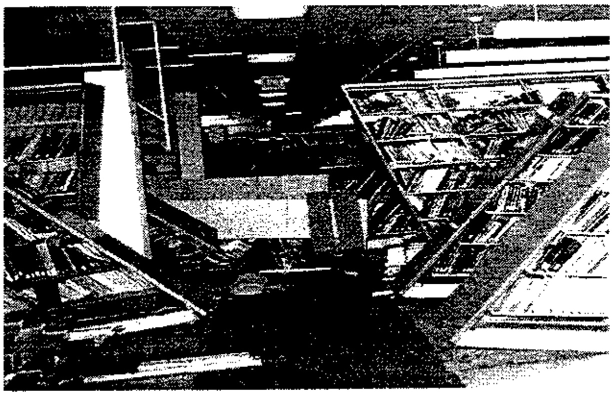

Figure 5 Damage to library content (1989 Loma Prieta earthquake) (Mitchell and Tinawi 1995)

Figure 6 Damage to emergency entrance of a hospital (1991 Costa Rican earthquake) (Mitchell and Tinawi 1995)

3. Codes for seismic design of non-structural components

The review of codes and recommended provisions, in terms of lateral force and displacement, is intended to reveal the variation in seismic design requirements for non-structural components between different codes of practice.

Architectural, mechanical, electrical and non-structural systems, components permanently attached to structures, including supporting structures and attachments, and nonbuilding components that are supported by other structures shall meet the requirements of this section.

In general, the OFC lateral seismic force, V_p, is higher than a comparative force used for the structural system for many reasons (Tauby et al. 1999; IBC 2000) such as:

- The accelerations acting on elements higher up within a building are greater than at ground level because of the dynamic response of the structure to earthquake ground motion.
- If an element is not rigid, its dynamic response is then amplified.
- Some elements lack the energy-absorbing properties of the ductile structures and hence are of limited ductility.
- Poor or lack of design of anchorage and restraint can lead to connection failure.

4. Historical overview of provisions the National Building Code of Canada, NBCC

Seismic design practice in Canada and in other countries has evolved significantly over the past fifty years. The first edition of the NBCC in 1941(NRCC 1941) contained seismic provisions in an appendix, based on concepts presented in the 1937 United States Uniform Building Code (UBC)(Heidebrecht 2003), however specific provisions for seismic design of structural and non-structural components in buildings and essential facilities were first introduced only in the 1953 edition. In all editions of the NBCC, the provisions concerning the OFCs and non-structural components are given in part 4 for design and commentary J in supplement part 4.

In the following, the evolution of provisions and recommendations of the NBCC over the years will be presented, starting from the 1953 edition until the proposed 2005 edition.

4.1 Provisions from 1953 to 1965 editions

4.1.1 Seismic force requirements

In the 1953, 1960 and 1965 editions (NRCC 1953, 1960, 1965), the seismic zoning map divided the country into four seismic regions based on earthquakes having a return period of 100 years (i.e. probability of 0.01 per annum). The minimum horizontal force for which portions of a building or structure should be designed to resist is given by the following formulae:

$$V_{1953,1960} = CW \tag{4. 1}$$

$$V_{1965} = kW \tag{4. 2}$$

Where:

V is the lateral seismic force in pounds.

W is the total dead load, including machinery and other fixed concentrated loads

C is a parameter reflecting the number of stories and seismic zoning

k is a parameter reflecting the intensity of the earthquake, the type of construction, the importance of the building, the foundation conditions and the number of stories.

It should be noted that the values of C and k only cover the architectural components, towers and tanks.

Starting from the 1965 edition, dynamic analysis was mentioned as an alternative to simple statical analysis for earthquake-resistant design.

4.1.2 Seismic displacement requirements

In these early editions of the NBCC, there were no provisions related to displacements.

4.2 Provisions of the 1970 edition

4.2.1 Seismic force requirements

The 1970 edition (NRCC 1970a, b)introduced a more refined contour map of the country based on expected ground acceleration A having a return period of 100 years. The new map was based on a computer analysis of the past earthquakes (1899-1963) throughout the country.

In this edition, buildings parts and their anchorage shall be designed for a minimum lateral force, V, as given by (4.1.7.1(6)):

$$V_{1970} = \frac{1}{4} R C_p W_p \tag{4. 3}$$

Where:

R = seismic regionalization factor which is a measure of the seismic activity and risk in the area considered

C_p = horizontal force factor for part or portion of a structure, as given in table 4.1.7.B.

W_p = the weight of a part or portion of a structure such as cladding partitions and appendages.

In this edition, it was noted in the commentary that machinery and electrical/mechanical equipment mounted within buildings should be designed to withstand the forces and displacements that arise from the seismic response of the structure, but no specific provisions were given.

4.2.2 Seismic displacement requirements

In this edition, there were no specific provisions for displacement, but it was recommended in the commentary that the interstory drift be limited $0.005h_s$, where h_s is the story height. The deflections obtained from an elastic analysis using the lateral force should be multiplied by 3.

4.3 Provisions of the 1975 and 1980 editions

4.3.1 Seismic force requirements

The same zoning maps as of the previous 1970 edition were used. In these editions (NRCC 1975a, b, 1980a, b), buildings parts and their anchorage shall be designed for a minimum lateral force as given by (4.1.9.1(12)):

$$V_{1975,1980} = AS_pW_p \tag{4. 4}$$

Where:

S_p = horizontal force factor for part or portion of a structure as given in table 4.1.9.C.

4.4.2 Seismic displacement requirements

Same as those of the 1970 edition.

4.5 Provisions of the 1990 edition

4.5.1 Seismic force requirements

In this edition (NRCC 1990a, b), the same zoning maps as in the 1985 edition were used. Buildings parts and their anchorage shall be designed for a minimum lateral force as given by (4.1.9.1.(15)):

$$V_{1990} = vS_pW_p \tag{4. 6}$$

Where

v should be determined according to section 2.2.1 except when Z_v equals to zero and Z_a is greater than zero, v should be taken as 0.05

Z_v is the velocity-related seismic zone and Z_a is the acceleration-related seismic zone

S_p = same as defined previously

For architectural components, values of S_p should conform to table 4.1.9.D and they vary between 0.7 and 6.5, while for mechanical/electrical equipment, $S_p = C_pA_rA_x$

A_x = amplification factor at level x to account for variation of response of mechanical/electrical equipment with height, and it is equal to $(1+ h_x/h_n)$

h_x, h_n = the height above level x or n respectively

A_r = response amplification factor to account for the type of attachment of mechanical/electrical component.

= 1.0 for components that are both rigid and rigidly connected.

= 2.0 for flexible components or flexibly mounted components located on ground.

= 4.5 for all other cases.

C_p = seismic coefficient for mechanical/electrical equipment, as given in table 4.1.9.E. It varies from 0.7 to 1.5.

In this edition, there was a distinction in the provisions between architectural and mechanical/electrical components. Also, a height factor was introduced for mechanical/electrical components, but not for architectural components.

4.5.2 Seismic displacement requirements

The largest interstory drift at any level based on the lateral deflections obtained from linear elastic analysis shall be limited to $0.01h_s$ for post-disaster buildings, and $0.02\ h_s$ for all other buildings. The lateral deflections obtained from an elastic analysis should be multiplied by R to give realistic values of anticipated deflections, where R reflects the capacity of the structure to dissipate energy through inelastic behavior.

4.6 Provisions of the 1995 edition

The provisions are given in section 4.1.9.1.15 of the NBCC 1995 edition, which is currently in use. The NBCC (NRCC 1995a, b) specifies design earthquake hazard at a propbability of 10% in 50 years, corresponding to a return period of 475 years. The same zoning maps as in the 1985 edition are used.

4.6.1 Seismic force requirements

The provisions use different force requirements for architectural components and for mechanical and electrical equipment.

Parts of buildings as described in table 4.1.9.1D and their anchorage shall be designed for a lateral force, V_p, distributed according to the mass distribution of the element, and equal to:

$$V_p = vIS_pW_p \text{ For architectural components} \quad (4.7)$$

$$V_p = vIC_pA_rA_xW_p \text{ For mechanical/electrical equipment} \quad (4.8)$$

Where:

ν = defined previously

I = the same importance factor as used for buildings, to establish compatibility of design risks with the structural system of post-disaster buildings and schools.

S_p = shall conform to table 4.1.9.1.D for architectural components. It varies from 0.7 to 6.5.

= $C_pA_rA_x$ for mechanical/electrical equipment

C_p = seismic coefficient for components of mechanical and electrical equipment as in table 4.1.9.1.E. It varies from 0.7 to 1.5.

A_r = 1.0 for components that are both rigid and rigidly connected and for non-brittle pipes and ducts.

= 1.5 for components located on the ground that are flexible or flexibly connected except for non-brittle pipes and ducts.

= 3.0 for all other cases

$A_x = 1.0 + (h_x/h_n)$

In this edition, the importance factor I, was introduced.

The design forces in the 1995 edition differ from the earlier editions of the NBCC due to several reasons (Tauby et al. 1999):

- Earthquake, geological, and tectonic information was analyzed using a new seismic risk approach.
- Newly developed strong seismic ground motion attenuation relations were included.
- Both horizontal acceleration and horizontal velocity have been considered.

4.6.2 Seismic displacement requirements.

The requirements in the 1995 edition are the same as those given in the 1990 edition.

4.7 Provisions of the proposed 2005 edition

The formulation of the proposed provisions (NRCC 2005) for elements of structures, non-structural components and equipment is based on the uniform hazard spectrum approach used for the design of structures (Adams and Halchuck 2003; Adams J. and Atkinson G. 2003). The new hazard spectrum model and resulting maps incorporate a significant increment of earthquake data, recent research on source zones and earthquake occurrence, together with complementary research on strong ground motion relations. In contrast to the 1985 maps, which give values for peak horizontal ground velocity and peak horizontal ground acceleration, peak horizontal spectral acceleration values ($S_a(T)$; 5% damped) are now directly specified, where T is the period. The seismic hazard at the site of the structure is included in the design force formula with the spectral value $S_a(0.2)$, which is taken from the uniform hazard spectrum at 0.2s period. Most components in buildings are stiff or rigid, and research from past earthquakes has shown that the forces on the components correlate most closely with this acceleration ordinate (NEHRP 2000).

The 2005 provisions use site values, not zone values as before, for design. These provisions are given in section 4.8.1.17 of the code and they are based on a probability of exceedence of 2% in 50 years, corresponding to a return period of approximately 2500 years. Site class C (very dense soil or soft rock) was adopted for reasons summarized in Adams and Halchuck (2003).

The following map (fig. 7) illustrates the design spectral accelerations $S_a(0.2)$ for Canada for site class C and 5% damping at a probability of 2% in 50 years.

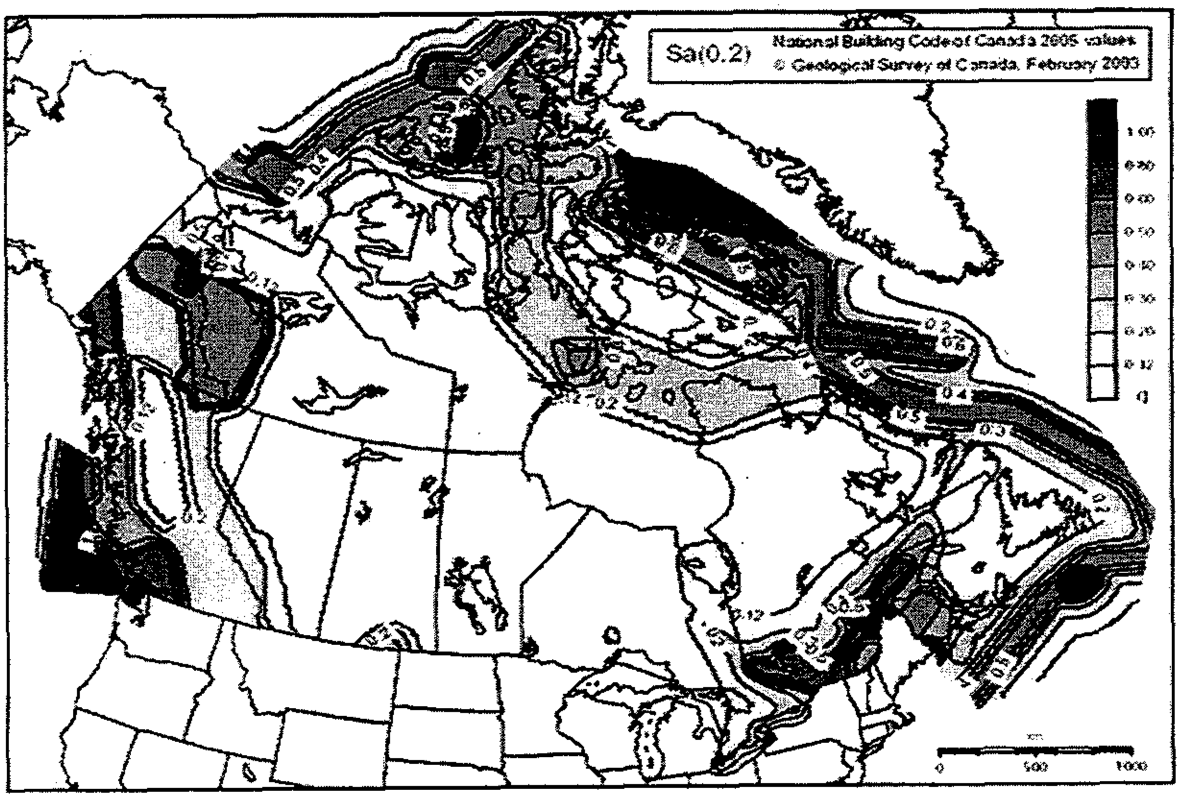

Figure 7 Sa (0.2) for Canada for site class C and 5% damping at 2% in 50 years (Adams and Halchuck 2003)

4.7.1 *Seismic force requirements*

The proposed provisions use the same force requirements for architectural components and for the mechanical and electrical equipment. Elements and components of buildings and their connections shall be designed for a lateral force V_p equal to:

$$V_p = 0.3 F_a S_a (0.2) I_E S_p W_p \tag{4. 9}$$

Where:

F_a = acceleration-based site coefficient. It is function of site class and $S_a(0.2)$. Its values vary from 0.7 to 1.4.

$S_a(0.2)$ = the spectral response acceleration value at 0.2s. it varies from 0.12 (Inuvik) to 1.2 (Victoria).

I_E = importance factor for the building.

$S_p = C_p A_r A_x / R_p$, the maximum value of S_p shall be taken as 4.0 and its minimum value shall be taken as 0.7.

C_p = element or component factor. It considers the risk to life safety associated with failure of the component and release of contents. It varies from 0.7 and 1.5.

R_p = element or component response modification factor. It represents the energy-absorption capacity of the element and its attachment. It varies from 1.25 to 5.

A_r = element or component force amplification factor. It is function of the ratio of the natural frequency of the component and the fundamental period of the structure. It varies from 1.0 to 2.5.

A_x = height factor $(1+ 2h_x/h_n)$. It considers the linear amplification of acceleration through the height of the building.

W_p = weight of the component or element.

4.7.2 Seismic displacement requirements.

The largest interstory drift at any level based on the lateral deflections obtained from linear elastic analysis 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. The lateral deflections obtained from an elastic analysis should be multiplied by R_dR_o/I_E to give realistic values of anticipated deflections, where R_o is the force overstrength factor and R_d represents the energy dissipation capacity of the element or its connections.

4.8 Correction for forces on top of buildings

In the 1970, 1975, 1980 and 1985 editions, the concentrated force on top of building is equal to:

$$F_t = 0.004V(h_n / D_s)^2 \qquad (4.10)$$

$$F_t \leq 0.15V \qquad (4.11)$$

$$F_t = 0 \text{ if } h_n/D_s \leq 3 \qquad (4.12)$$

Where

F_t is the portion of V to be concentrated at the top of the structure.

V is the lateral seismic action or force on a part or portion of the structure

h_n is the height in feet above the base.

D_s is the dimension of the lateral-force resisting system in feet in a direction parallel to the applied forces.

In the 1990, 1995 and proposed 2005 editions

$$F_t = 0.07TV \qquad (4.13)$$

$$F_t \leq 0.25V \qquad (4.14)$$

$$F_t = 0 \text{ if } T \leq 0.7s \qquad (4.15)$$

4.9 Comments on the proposed 2005 and the 1995 editions of the NBCC

There are number of similarities and differences between the two editions. Other than the obvious differences in the form there are some differences such as:

The NBCC 1995 does not account for the soil type, the near fault effect, the variation of acceleration over the height, and the component response modification factor R was implicitly accounted for.

In the 2005 edition, the amplification due to location ranges from 1.0 at ground level to 3 at the roof level, while in the 1995 edition, the amplification ranges from 1 at ground level to 2 at the roof level. Therefore, the new code provisions bring more stringent requirements for equipment at higher elevations in a building.

The appropriateness of the linear distribution of accelerations over the height needs to be evaluated by using the response spectrum analysis of buildings and data from instrumented buildings. To study the distribution of acceleration over height, Kehoe and Freeman (1998) did a comparison using dynamic analysis and assuming rigidly attached elements. They found that for buildings with higher mode effects, the floor accelerations are relatively constant over most of the height of the buildings, this is consistent with what is found by Soong and al. (1993) .

The factor used to account for the effect of equipment elevation should take into account structural rigidity and height of the structure, so the variation of response acceleration with height should be revised and evaluated as a function of building type, height and fundamental period and perhaps this may lead to correction at top levels.

In the following, we present design forces for various building components in an ordinary building in Vancouver according to the last two editions of the NBCC, so that we can assess the impact of changes on the proposed design force levels on design force levels at specific locations. Results extracted from McKevitt (2003) are presented. The indicator is the ratio of the lateral seismic force to the component weight.

4.9.1 Mechanical and electrical equipments

Table 1 Values for mechanical/electrical equipments

		F/W	
	Mech./elec. eq.	Proposed	1995 NBCC
1st floor	flexible equipment	0.3	0.3
	cable tray	0.21	0.2
	tank	0.21	0.21
	rigid equipment	0.21	0.2
Middle	flexible equipment	0.6	0.9
	ductile pipes	0.21	0.3
	non-ductile pipes	0.6	0.9
	rigid equipment	0.24	0.3
Roof	flexible equipment	0.3	0.4
	ductile pipes	0.3	0.4
	cable tray	0.45	0.4
	non-ductile pipes	0.9	1.2
	rigid equipment	0.36	0.4

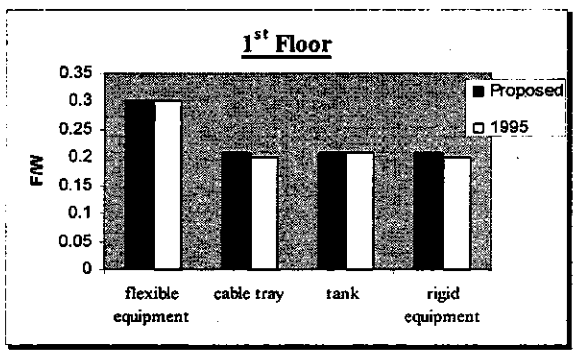

Figure 8 Comparison of forces for mechanical/electrical equipments (1st floor) according to 1995 and 2005 provisions

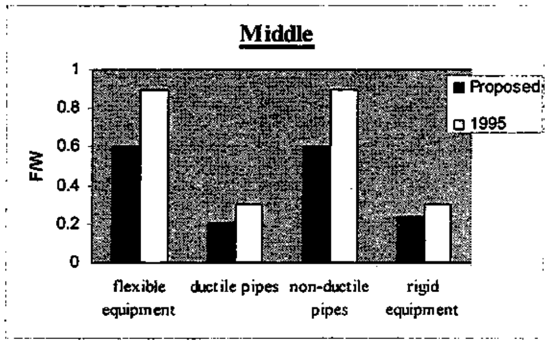

Figure 9 Comparison of forces for mechanical/electrical equipments (Middle) according to 1995 and 2005 provisions

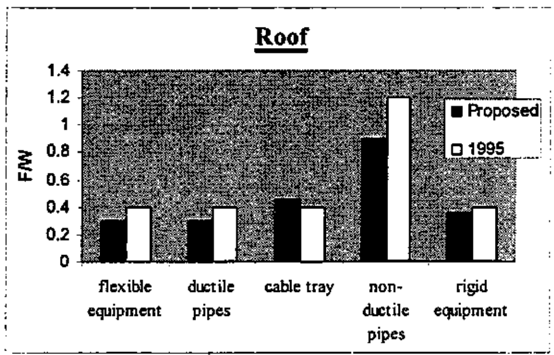

Figure 10 Comparison of forces for mechanical/electrical equipments (Roof) according to 1995 and 2005 provisions

4.9.2 Architectural components

Table 2 Values for architectural components

		F/W	
	Architectural	Proposed	1995 NBCC
1st floor	Exterior walls	0.21	0.3
	Suspended ceilings	0.21	0.3
	Cantilever wall	0.3	0.3
	Interior walls	0.21	0.3
	Balconies	0.21	0.9
Roof	Exterior walls	0.36	0.3
	Cantilever parapet	0.9	1.3
	Suspended ceilings	0.36	0.4
	Chimneys	0.9	0.9
	Interior walls	0.36	0.3
	Balconies	0.36	0.9

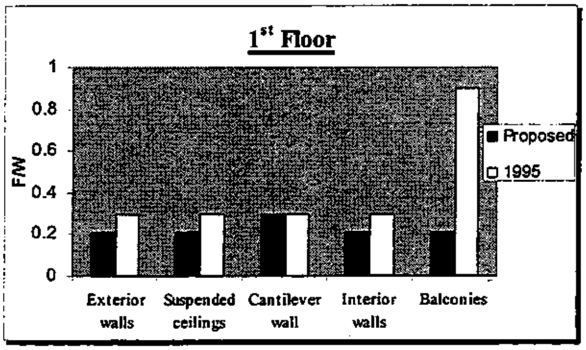

Figure 11 Comparison of forces for architectural components (1st floor) according to 1995 and 2005 provisions

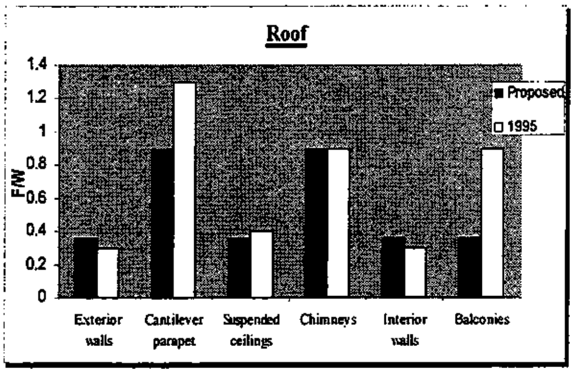

Figure 12 Comparison of forces for architectural components (Roof) according to 1995 and 2005 provisions

The most significant difference between the proposed provisions and the 1995 provisions is in the design forces for the architectural components, as a result of the inclusion of the height factor in the in the force calculation (McKevitt 2003). Also there was a reduction of forces for components located at mid-height. Very often architectural componetns are not only force sensitive, but also displacement sensitive.

Also there were changes in the provisions for displacement requirement.

4.10 Numerical examples for calculation of forces according to previous editions in the NBCC (1970-2005)

In this section, we will consider examples given in appendix G of the S832-01 guide. Calculation details are given in appendix A of this report.

4.10.1 Architectural component

It is a suspended acoustic ceiling system for office occupancy in Vancouver.

$V_{p,2005} = 0.396\ W_p$ (roof) $V_{p,2005} = 0.132\ W_p$ (ground)

$V_{p,1995} = 0.4\ W_p$

$V_{p,1990} = 1.3\ W_p$

$V_{p,1985} = 0.88\ W_p$

$V_{p,1980} = 7.85\ W_p$

$V_{p,1975} = 7.85\ W_p$

$V_{p,1970} = 1.0\ W_p$

The following graph (fig.13) illustrates the calculations

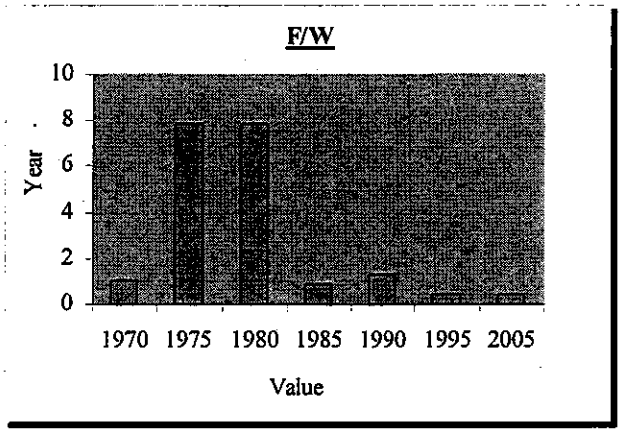

Figure 13 Comparison of forces for an architectural component starting from the 1970 edition until the proposed 2005 edition of the NBCC

4.10.2 Mechanical and electrical equipments

It is a rooftop chiller on isolators for hospitals in Victoria (Flexibly connected)

$V_{p,2005} = 1.62\ W_p$

$V_{p,1995} = 2.7\ W_p$

$V_{p,1990} = 2.7\ W_p$

$V_{p,1985} = 1.32\ W_p$

$V_{p,1980} = 7.85\ W_p$

$V_{p,1975} = 1.57\ W_p$

The following graph (fig.14) illustrates the calculations

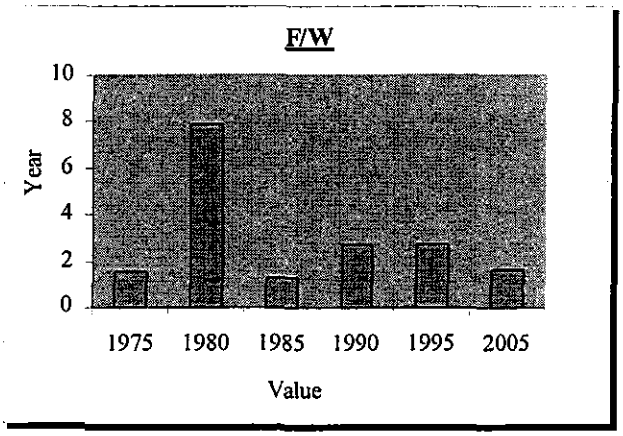

Figure 14 Comparison of forces for a mechanical component starting from the 1975 edition until the proposed 2005 edition of the NBCC

4.10.3 General remarks

We can notice that the variation of forces was not uniform, but the variability is greater for mechanical/electrical equipment.

In both cases, the proposed 2005 provisions give values less than those of the 1995 provisions.

5. Guideline S832-01: Seismic design of operational and functional components (OFCs) of buildings

The objective of the guideline is to provide information and methodology to identify the OFCs whose failure modes and consequences due to earthquakes may require mitigation, also it is intended to suggest design approaches to achieve adequate mitigation.

The recommended approach to risk assessment according to CSA (2001) is to determine the risk rating for each OFC and establish a ranking of high, moderate or low, based on numerical seismic risk rating scores, R. This rating is determined as the product of the OFC's seismic vulnerability related to probability of failure, V, and the consequences of failure related to probability of resultant death, injury, or loss of building functionality, C, if failure/malfunction occurs. The methodology is outlined in clause 6.2 of the guideline.

$$R = V \times C \tag{5. 16}$$

V is determined according to table 2 of the guideline

$$V = RG \times RB \times \frac{\sum (RS \times WF)}{10} \tag{5. 17}$$

Where

RS is the rating score, WF is the weight factor, RG depends on the type of the structural system and RB depends on the characteristics of the ground motion.

C is determined according to table 3 of the guideline

$$C = \sum RS \tag{5. 18}$$

The guideline may be used by building owners, inspectors, facility managers, engineers, architects, etc.

6. NEHRP 2000 recommended provisions for seismic regulations of new buildings

6.1 Seismic force requirements

The seismic design force provisions of the 2000 NEHRP(NEHRP 2000) have been taken from the 1997 NEHRP (NEHRP 1997) which have evolved from the design force provisions of the 1994 NEHRP, that are based on strength design (Bachman R.E. et al. 1993; Soong and al. 1993; Drake R.M. and Bachman R.E. 1994).

The principal contributor to these provisions is the Building Seismic Safety Council (BSSC). Seismic forces F_p of the 2000 NEHRP shall be determined according to the following equations:

$$F_p = \frac{0.4 a_p S_{SD} W_p}{\frac{R_p}{I_p}} \left(1 + 2\frac{z}{h}\right) \tag{6. 1}$$

$$F_{p\max} = 1.6 S_{DS} I_p W_p \tag{6. 2}$$

$$F_{p\min} = 0.3 S_{DS} I_p W_p \tag{6. 3}$$

Where:

F_p = seismic design force centered at the component's center if gravity and distributed relative to the component's mass distribution.

S_{SD} = design spectral acceleration at short period. It reflects the seismicity of the site including soil amplification effects. It is obtained from the maximum considered earthquake ground motion maps, reduced by a factor of 2/3.

a_p = component amplification factor that varies from 1 to 2.5.

I_p = component importance factor that is either 1 or 1.5. It represents the greater of the life-safety importance of the component and the hazard exposure importance of the structure. This factor indirectly accounts for the functionality of the component or structure by requiring design for a higher force level.

W_p = component reactive weight.

R_p = component response factor that varies from 1 to 5.

It considers both the overstrength and deformability of the component's structure and attachments. The engineering community is encouraged to address the issue and conduct research into the response modification factor that will advance the state of the art.

z = height in structure of point of attachment of component.

h = average roof height of the structure relative to grade elevation.

The structural period effect has been removed from these provisions.

The term C_a in the 1994 NEHRP has been replaced by the quantity $0.4S_{DS}$ to conform with changes in chapter 4 (design earthquakes ground motions) of the NEHRP 2000.

6.2 Seismic displacement requirements

For two connection points on the same structure A, or the same structural system, one on level x and the other on level y, the seismic relative displacement (D_p) shall be determined in accordance with the following:

$$D_p = \delta_{xA} - \delta_{yA} \tag{6. 4}$$

D_p is not required to be taken as greater than:

$$D_p = X - Y\left(\frac{\Delta_{aA}}{\Delta_{SX}}\right) \tag{6. 5}$$

For two connection points on separate structures A and B or separate structural systems, one at level x and the other at level y, D_p shall be determined as:

$$D_p = \left|\delta_{xA}\right| - \left|\delta_{yB}\right| \tag{6. 6}$$

Where:

D_p = relative seismic displacement the component must be designed to accommodate.

δ_{xA}= deflection at building level x of structure A, determined by an elastic analysis and multiplied by the C_d factor.

δ_{yA}= deflection at building level y of structure A, determined by an elastic analysis and multiplied by the C_d factor.

δ_{yA}= deflection at building level y of structure B, determined by an elastic analysis and multiplied by the C_d factor.

X = height of upper support attachment at level x as measured from the base.

Y = height of lower support attachment at level y as measured from the base.

Δ_{aA} = allowable story drift for structure A.

Δ_{aB} = allowable story drift for structure B.

h_{sx} = story height used in the definition of the allowable drift, Δ_a.

Δ_a/h_{sx} = allowable drift index.

7. International building Code, IBC 2000

Seismic force and displacement requirements are similar to those of the NEHRP 2000.

8. Comparison of seismic code provisions of the NBCC 2005 and NEHRP 2000

8.1 Comparison of maximum force requirements

Both the NBCC and the NEHRP consider the seismicity of the site, the height of component above floor level and whether or not occupants would be injured should the component fail, whether the component stores, uses or transports hazardous substances; whether or not the component's function is required to maintain life safety, and how the failure of one component may affect the functionality of another component.

The primary intent of the provisions is to provide minimum requirements for life safety. Functionality of equipment during or after seismic event is outside the scope of current provisions, but it is implicitly addressed in the NEHRP provisions thorough the importance factor.

Components that are both rigid and rigidly connected are defined as those having a fundamental period less than 0.06s

8.2 Comparison of seismic displacement requirements

The NBCC provisions require less calculation than the NEHRP 2000; this implies that the recommendations of the NBCC are less specific than those of the NEHRP 2000.

9. Research needs: critical review

Secondary structures should be the subject of rational seismic design, in the same way of the supporting structure and should be the continuing object of a careful performance assessment following strong earthquakes.

In recent years, the attention paid to this problem has generated better understanding of the seismic behavior of non-structural components, but there are still some areas that require further research.

An overview of the research gaps as reported in (Chen and Soong 1988; Soong 1994; Phan and Taylor 1996; Villaverde 1997; Kehoe and Freeman 1998; Singh et al. 1998; Filiatrault et al. 2001a; McKevitt 2003) is summarized as follows:

- There is a great challenge for researchers to develop accurate yet simple response calculation procedure which can be incorporated into codes and standards. In particular, research is needed to develop simplified guidelines to account in a rational way for the effect of yielding in a structure on the response of the secondary elements attached to it. Also, only a few methods that take into account the effect of the nonlinearity of non-structural components and that of their supporting structure have been proposed, so there is a need to derive simplified methods for the analysis of non-structural components, taking into account these effects. It can be noted that structures should not yield at moderate earthquakes and the effect of nonlinearity becomes important in case of high intensity earthquakes.
- On the other hand, there is a need to study the effect of structural yielding on the effectiveness of base isolation and structural control of secondary structures, and

develop simplified methods for the analysis of secondary structures that incorporate any of these techniques in their designs.

- The distribution of floor acceleration over the height needs to be reviewed in order to assess what parameters influence whether the floor accelerations are essentially constant or vary over the height of the building; there is work in progress at Stanford University concerning this issue, (Taghavi and Miranda 2002, 2003b, 2003a; Miranda and Taghavi 2003).
- Functionality represents an aspect that is still not explicitly addressed in present codes and provisions. In fact, the internal functionality is implicitly addressed in current codes through the importance factors. Assessment of function by definition of acceleration limits at which particular pieces of equipment cease to be operational can only be achieved by full-scale shake table testing of equipment and is outside of the scope of the present provisions and codes.
- Also work is needed to assess the importance of torsional motion of the supporting structure on the response of a secondary structure attached to it and to develop simplified methods of analysis to take this effect into account. In fact, this effect is important for multi-supported operational and functional components and architectural elements. Localized equipment does not feel torsion, only translation.

Villaverde (1997) is an excellent source of information of the recent state of research, while Chen and Soong (1988) is an excellent source of information on the methods of analysis.

10. Proposed research

In these days, there is an increasing concern about the seismic design of non-structural components. This stems from many reasons, such as their paramount importance to economic losses and the advent of performance-based design. Despite their importance, non-structural components have received very little attention in the past from the earthquake engineering research community, except those elements in essential facilities such as nuclear power plants.

In fact, most of the economic losses from earthquakes are due to nonstructural components failures. This stems from three reasons according to (Taghavi and Miranda *2002, 2003b):*

- Nonstructural components represent a large percentage (~65-80%) of the total construction cost in buildings.
- Damage to nonstructural components is usually triggered at levels of deformation much smaller than those required to initiate structural damage.
- The higher frequency of occurrence of seismic events with small or moderate ground motions intensities compared to that of seismic events that produce ground motions with large intensities.

The second and third reasons make economic losses from nonstructural components not only larger but potentially more frequent than losses from structural components.

Also, the trend now is towards having buildings with predictable performance that meet the performance requirements of the owners and investors. For buildings as hospitals, emergency services, critical lifelines, to prevent loss of functionality is extremely important.

Taghavi and Miranda (Taghavi and Miranda 2002; 2003b; 2003a) and Miranda and Taghavi (2003) are developing procedures to estimate floor acceleration demands. Parameters considered are the fundamental period of vibration, the damping ratio, and type of lateral resisting system in the structure.

Marsantyo et al. (1998; 2000) have investigated the maximum acceleration amplification of nonstructural systems mounted on floors through experimental and analytical works. They found that systems having low damping factor produced acceleration response exceeding the design code stipulation of the 1997 UBC and 1997 BCJ (Japenese Building Code), while isolated nonstructural systems or mounted on floors of an isolated structures satisfy the codes even with low damping.

The research project will focus on the prediction of acceleration at rooftop of buildings, with specific application to telecommunication towers mounted on top of buildings. The effect of vertical accelerations will also be studied.

Few researchers have studied the seismic response of telecommunication towers on top of buildings. Kanazawa K. and Hirata K. (2000) developed a method to evaluate the floor response spectrum considering dynamic interaction between the primary and secondary system without neglecting non-stationary or transient effects. The researchers applied the proposed method on telecommunication towers on top of buildings.

Khedr (1998) and Khedr and McClure (2000) developed a simplified method for seismic analysis of lattice telecommunication towers founded on ground, and subjected to horizontal and vertical earthquakes accelerations.

The purpose of the research will focus on predicting the accelerations on top of buildings and develop a simplified method for seismic analysis of telecommunication towers built on top of buildings.

More details will be available in the research proposal.

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Appendix A: Calculation of forces

Architectural component

It is a suspended acoustic ceiling system for office occupancy in Vancouver.

Proposed NBCC 2005

$I = 1.0$ (Clause 4.1.8.5.1) $C_p = A_r = 1.0$ $R_p = 2.5$ (Table 4.1.8.17)

$S_a(0.2) = 0.96$ (soil type C) $\Longrightarrow$ $Fa = 1.0$ $A_x = 3.0$ (roof level) $A_x = 1.0$ (ground level)

$V_{p,2005} = 0.396\ W_p$ (roof) $V_{p,2005} = 32\ W_p$ (ground)

NBCC 1995

$I = 1.0$ (Clause 4.1.9.1.10) $v = 0.2$ (Appendix C, NBC) $S_p = 2.0$ (Table 4.1.9.1.D)

$V_{p,1995} = 0.4\ W_p$

NBCC 1990

$v = 0.2$(Appendix C, NBC) $S_p = 6.5$ (Table 4.1.9.1.D)

$V_{p,1990} = 1.3\ W_p$

NBCC 1985

$v = 0.2$(Appendix C, NBC) $S_p = 4.4$ (Table 4.1.9.1.D)

$V_{p,1985} = 0.88\ W_p$

NBCC 1980

$S_P = 10$ (Table 4.1.9.C) $A = 0.785$ (table J-2 of the commentary)

$V_{p,1980} = 7.85\ W_p$

NBCC 1975

$S_P = 10$ (Table 4.1.9.C) $A = 0.785$ (table J-2 of the commentary)

$V_{p,1975} = 7.85\ W_p$

NBCC 1970

$R = 4.0$ (supplement N.4 to NBC) $C_p = 1.0$ (Table 4.1.7.1.B)

$V_p = 1.0\ W_p$

Mechanical equipment

It is a rooftop chiller on isolators for a hospital in Victoria (Flexibly connected)

NBCC 2005

$I = 1.5$ (Clause 4.1.8.5.1) $C_p = 1.0$ $A_r = R_p = 2.5$ (Table 4.1.8.17)

$S_a(0.2) = 1.2$ (soil type C) $\Longrightarrow$ $Fa = 1.0$ $A_x = 3.0$ (roof level) $A_x = 1.0$ (ground level)

$V_{p,2005} = 1.62\ W_p$ (roof) $V_{p,2005} = 0.54\ W_p$ (ground)

NBCC 1995

$I = 1.5$ (Clause 4.1.9.1.10) $\nu = 0.3$ (Appendix C, NBC) $C_p = 1.0$ (Table 4.1.9.1.E)

$A_x = 2.0$ (Clause 4.1.9.19) $A_r = 3.0$ (Clause 4.1.9.19) $S_p = 6.0$

$V_{p,1995} = 2.7\ W_p$ (roof) $V_{p,1995} = 1.35\ W_p$ (ground)

NBCC 1990

$\nu = 0.30$ (Table J-1 of the commentary) $C_p = 1.0$ $A_r = 4.5$ $A_x = 2.0$

$V_{p,1990} = 2.7\ W_p$ (roof) $V_{p,1990} = 1.35\ W_p$ (ground)

NBCC 1985

$\nu = 0.30$ (Table J-2 of the commentary) $S_p = 4.4$

$V_{p,1985} = 1.32\ W_p$

.**NBCC 1975 & NBCC 1980**

$S_P = 10$ (Table 4.1.9.C) $A = 0.785$ (table J-2 of the commentary)

$V_{p,1980} = 7.85\ W_p$

NBCC 1975

$S_P = 2.0$ $A = 0.785$ (table J-2 of the commentary)

$V_{p,1975} = 1.57\ W_p$