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Seismic Hazard Assessment and Mitigation for Buildings' Functional and Operational Components:

A Canadian Perspective

Acknowledgments

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Executive Summary

Past earthquakes have shown that the damage to operational and functional components of buildings usually cause more injuries, fatalities, and property and financial loss than those inflicted by structural damage. Operational and functional components of a building include architectural components, mechanical and electrical equipment, and building contents. There have been many incidents when a building which sustained only minor structural damage was deemed unsafe and unusable as a result of extensive damage to its operational and functional components.

Failure of such components and the debris caused by falling objects could critically affect the performance of vital facilities such as emergency command centres, fire and police stations, hospitals, power stations and water supply plants. During the 1994 Northridge earthquake in California, several major hospitals had to be evacuated, not because of structural damage, but due to failure of emergency power systems, air control units, falling ceilings, and light fixtures. In Canada, the 1988 Saguenay earthquake, the strongest event in eastern North America recorded within the last 50 years, caused very little structural damage. It has been well documented that a great majority of the injuries, property damage and economic loss was caused by the failure of operational and functional components in buildings. Failure of such components also poses serious problems for search and rescue operations after an earthquake, resulting in additional and unnecessary increases in casualties.

During the past few years, significant progress has been made towards the understanding and improvement of seismic behaviour of operational and functional components (OFCs). These include the development of provisions for seismic design of restraints of OFCs, conducting experimental investigations of OFCs, and the development of guidelines for seismic risk reduction of OFCs. Review of the current state-of-the art in the area followed by the production of design floor response spectra for representative Canadian locations form the objective of this report.

The project was initiated by reviewing the performance of OFCs during past earthquakes, then examining the current methodologies in use in the U.S. and Canada for seismic risk reduction of OFCs of buildings. Relevant findings are documented, and current OFC guidelines in the U.S. and Canada are described.

Design floor response spectra for medium-rise concrete moment resisting frame buildings were then developed. To accomplish this, two 10-storey concrete moment-resisting buildings were designed in accordance with the National Building Code of Canada, 1995 edition. One building was designed for Ottawa and the other for Vancouver, as representative sites in Eastern and Western Canada. Fifteen artificially generated earthquake records (accelerograms) were developed and 15 actual or field recorded accelerograms were selected for each of these two locations. Each building was then subjected to a total of 30 accelerograms (the 15 artificial and 15 actual input ground motions). Nonlinear analysis was carried out for each ensemble of accelerograms. Acceleration time histories of the response at each floor were determined for each earthquake record.

Based on the acceleration time histories, acceleration spectra were computed for each floor and for each analysis. It was observed that the response amplifications relative to ground excitations varied from floor to floor, and were frequency dependent.

The acceleration floor spectra for individual floors were used to develop design floor response spectra. The proposed design floor response spectra can be used by the seismic protection community in Canada to determine the seismic requirements for securing functional and operational components of buildings.

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1.0 Introduction

Past earthquakes have shown that the damage to operational and functional components of buildings usually cause more injuries, fatalities, property, and financial loss than those inflicted by structural damage. There have been many incidences that a building, which sustained only minor structural damage, was deemed unsafe and unusable as a result of extensive damage to its operational and functional components. Failure of such components also poses serious problems for search and rescue operations after the earthquake, resulting in additional and unnecessary increases in casualties.

Operational and functional components of a building include:

- **Architectural components** – parapets, claddings, partitions, stairways, lighting systems, suspended ceilings, etc.
- **Mechanical and electrical equipment** – pipes and ducts, escalators, central control panels, transformers, emergency power systems, fire protection systems, machinery, etc.
- **Building contents** – books and shelves, furniture, file cabinets, storage racks, etc.

Failure of equipment and the debris caused by falling objects could critically affect the performance of vital facilities such as emergency command centres, fire and police stations, hospitals, power stations and water supply plants. Failure of such components also poses serious problems for search and rescue operations after a seismic disaster, resulting in additional casualties. During the 1994 Northridge earthquake in California, several major hospitals had to be evacuated, not because of structural damage, but due to failure of emergency power systems, air control units, falling ceilings and light fixtures. In Canada, the 1988 Saguenay earthquake, the strongest event in eastern North America recorded within the last 50 years, caused very little structural damage. It has been well documented that a great majority of the injuries, property damage, and economic loss was caused by the failure of operational and functional components in buildings.

This project was conceived to provide Canada's seismic protection community with a state-of-the-art knowledge base on the seismic hazard assessment and mitigation for buildings' functional and operational components. The project consisted of the following tasks:

i. Technical review of analytical and experimental research on the seismic behaviour and of code/guideline development on the seismic evaluation and mitigation of functional and operational components.
ii. Preparation of a report on the technical review.
iii. Design of two typical moment resisting frame buildings, one in Vancouver and one in Ottawa, as representatives of concrete frame buildings in Western and Eastern Canada.
iv. Development of models for the moment resisting frame buildings for use in dynamic response history analyses in subsequent tasks.
v. Preparation of an interim report outlining the selection, design and modelling of buildings.
vi. Development of 15 artificial accelerograms (for ground motions) reflecting the

seismicity of Western and Eastern Canada (Vancouver and Ottawa, respectively).

vii. Selection of 15 actual accelerograms (for ground motions), representatives of Western and Eastern Canada.

viii. Nonlinear dynamic analyses of buildings under the previously selected ensemble of artificial accelerograms.

ix. Nonlinear dynamic analyses of buildings under the previously selected ensemble of recorded (actual) accelerograms.

x. Computation of floor response spectra associated with each ensemble of accelerograms (artificial and actual).

xi. Development of design floor response spectra based on the results of task (x).

xii. Preparation of a final report

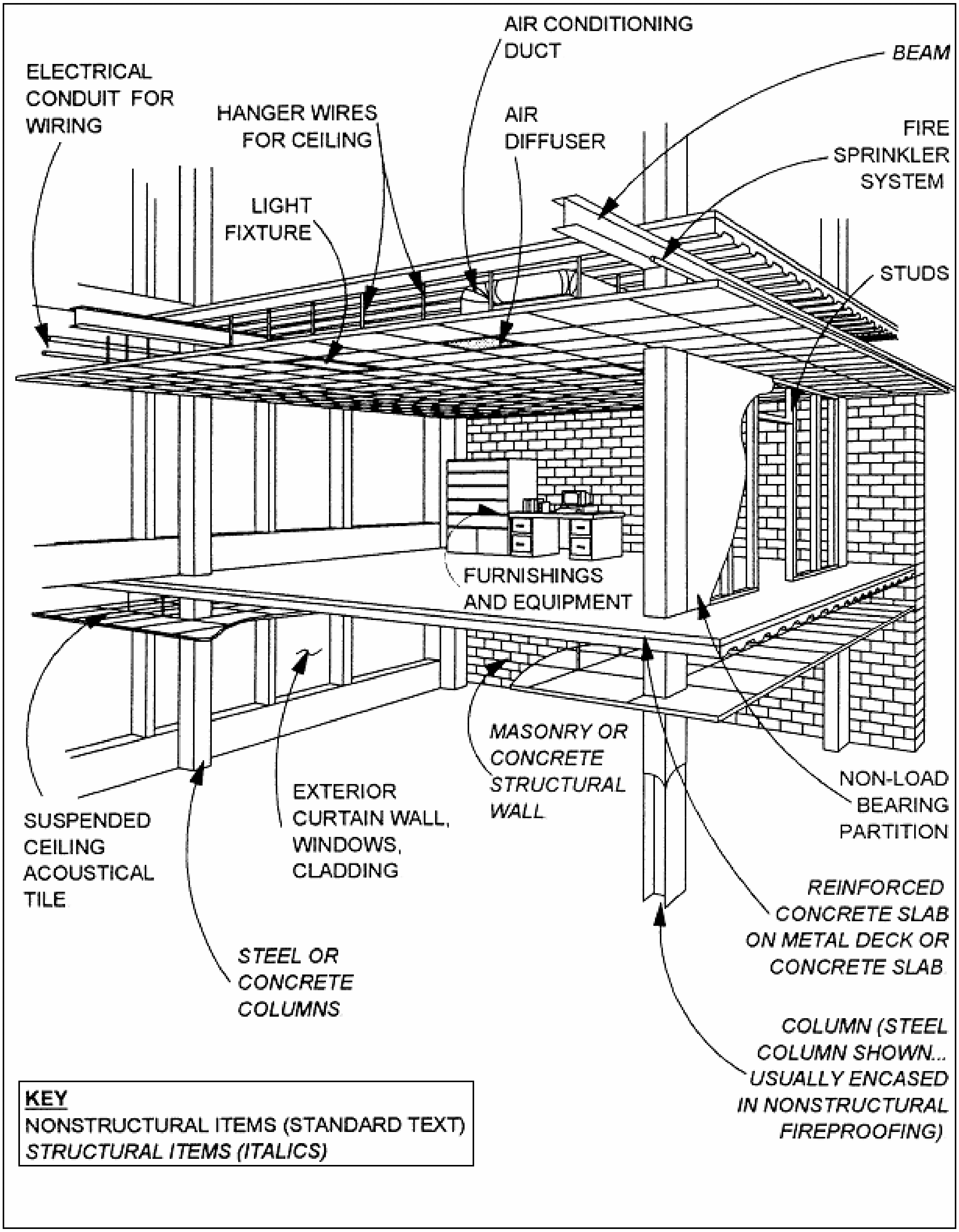
The document reports on the work done throughout the project, as stated in item (xii) above.

2.0 Experience from Past Earthquakes

Given the experience from past earthquakes, measures have been undertaken in a number of countries in order to reduce the seismic risk of operational and functional components (OFCs) during future earthquakes. However, dealing with the seismic risk of OFCs is an extremely difficult task. This is because there are many different situations that arise from: (i) a large number of different types of OFCs; (ii) different structural systems of buildings; and (iii) different locations in the building where the OFCs are attached (i.e. floor level and location at that level). To appreciate the scope of the problem associated with the different types of OFCs, Table 1 in Appendix A lists the OFCs that should be considered for reducing the seismic risk of OFCs in building structures (Moe, Foo and McClure, 1999).

Figure 1 (adopted from FEMA 74, 1994) shows non-structural and structural components of a building.

Figure 1 Non-structural and structural components of a typical building (FEMA 74, 1994)



In the U.S., extensive research and development work related to OFCs has been carried out under the National Earthquake Hazards Reduction Program (NEHRP), and sponsored by the Federal Emergency Management Agency (FEMA). Results from this research and development work have been presented in several publications (FEMA publications No. 74, 172, 241, 273, 274, 302 and 303). Among these publications, FEMA 74 and FEMA 241 are specifically for OFCs; the other publications are for building structures and both structural components and OFCs are considered.

In Canada, significant work related to OFCs has been also conducted during the last 10 years, which has been initiated and sponsored by Public Works and Government Services Canada (PWGSC, 1995; 1997; 1999). Currently, the Technical Committee on Seismic Risk Reduction of the Canadian Standards Association (CSA) is working on development of a guideline for seismic risk reduction of OFCs of buildings (Cheung et al. 1999). Types of OFCs covered in the CSA Guideline are given in Table 1 in Appendix A. This guideline will be the first official document in Canada that deals specifically with OFCs of buildings.

A brief historical review of the damage of OFCs during past earthquakes is given in FEMA 274 (Building Seismic Safety Council, 1997). In this section, the review adapted from FEMA 274 is presented first. In addition, more detailed discussion is given for the damage of OFCs during the most recent earthquakes (i.e. the 1994 Northridge, California earthquake, the 1999 Kocaeli earthquake in Turkey, and the 1999 Jiji, Taiwan earthquake), as reported by reconnaissance teams of engineers who visited the affected regions immediately after the earthquakes.

2.1 Historical Review

During the 1906 San Francisco, 1925 Santa Barbara, and 1933 Long Beach earthquakes, the collapse of unreinforced brick parapets and exterior walls caused many casualties to building occupants, pedestrians and motorists. This was demonstrated again during the 1952 Bakersfield, 1971 San Fernando, 1987 Whittier-Narrows, 1989 Loma Prieta, and 1994 Northridge earthquakes.

The 1964 Alaska earthquake first pointed out the vulnerability of modern exterior precast wall panels, elevators and suspended ceilings. The 1971 San Fernando earthquake caused collapses of metal library shelving, debris on exit stairways, failures of suspended ceilings, light fixtures, and HVAC (Heating, Ventilation and Air Conditioning) ducts. The 1989 Loma Prieta earthquake caused collapse of some heavy plaster ceilings and ornamentation, falling of lighting grids and their supported fixtures. This earthquake also caused severe economic losses created by damage of water systems.

For illustration, Figures 2 and 3 (adopted from Gates and McGavin, 1998) and Figure 4 (adopted from FEMA 74) show cases of damage to OFCs during past earthquakes. Figure 2 illustrates failure of the spring support system of mechanical equipment and Figure 3 shows an elevator control cabinet in the penthouse of a large hotel that was thrown from its base during strong earthquake on the island of Guam in 1993. Figure 4 shows failure of a piping system during the 1971 San Fernando earthquake.

Figure 2 Failure of spring support system of mechanical equipment (Gates and McGavin, 1998)

Figure 3 Failure of elevator control cabinet during the 1993 Guam earthquake (Gates and McGavin, 1998)

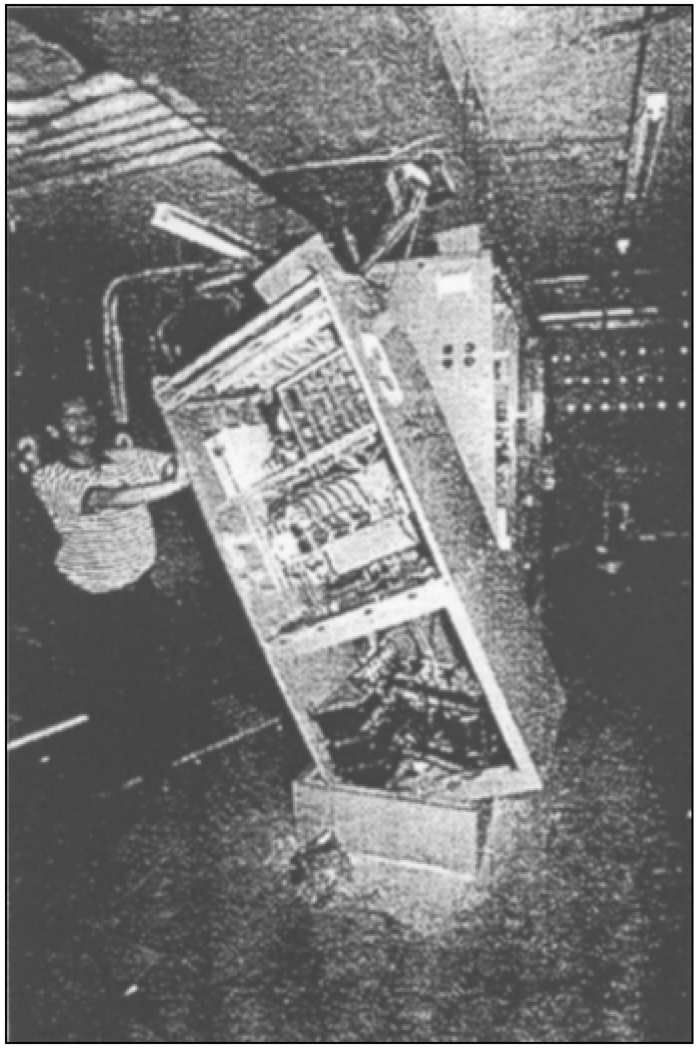
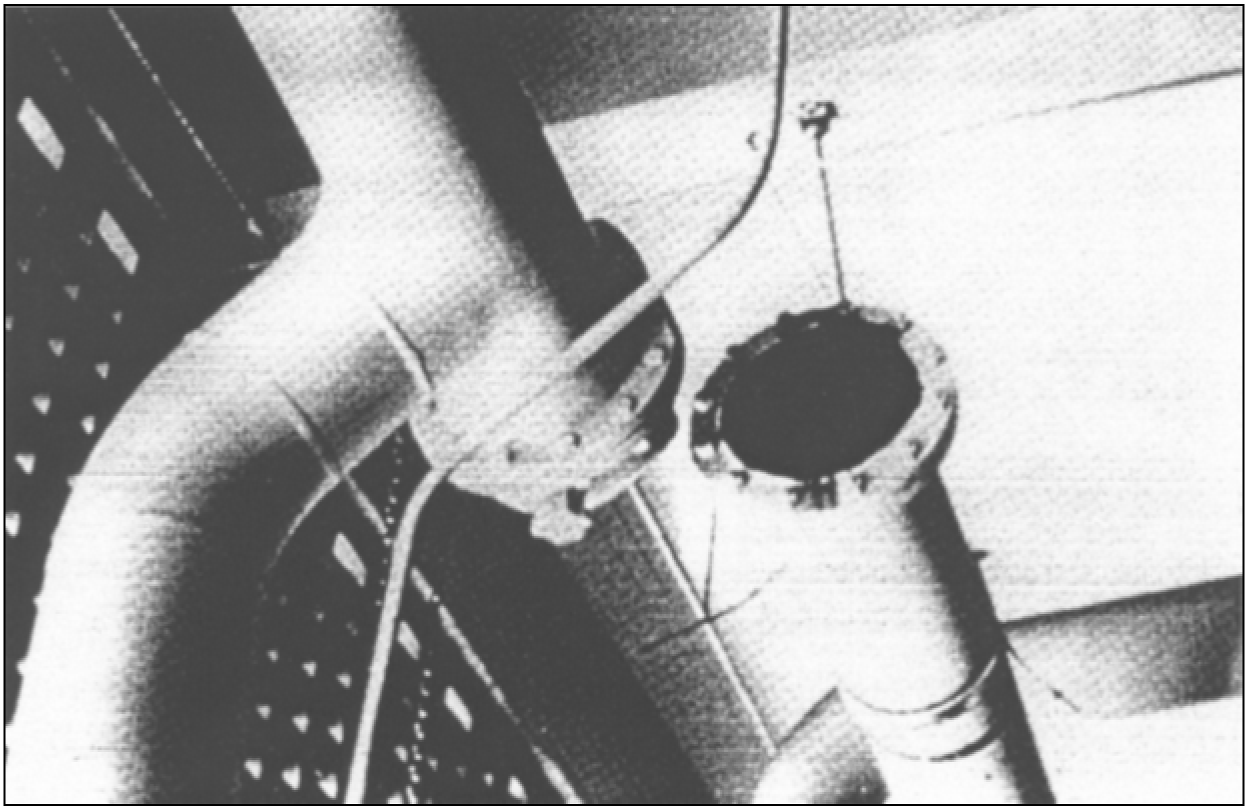


Figure 4 Failure of piping system during the 1971 San Fernando, California earthquake (FEMA 74, 1994)



In addition to the direct consequences of the OFCs failures during these earthquakes, the OFCs failures triggered other life-threatening hazards to the building structures, occupants and contents. These include fires, explosions, and release of hazardous materials as a result of damaged mechanical and electrical equipment. Selvaduray (1998) provided a summary of hazardous materials incidents at educational facilities that were caused due to damage to OFCs during a number of earthquakes in California and Japan. As reported, spilled chemicals resulted in explosions and fires in many laboratories that increased the consequences of the damage to OFCs. As reported by Selvaduray (1988), a total of 490 hazardous materials incidents were recorded during the 1989 Loma Prieta earthquake, and 387 such incidents occurred during the 1994 Northridge earthquake.

2.2 Performance of OFCs during the 1994 Northridge, California Earthquake

McKevitt et al. (1995) described failures of various types of OFCs in buildings located in the epicentral region of Northridge earthquake. As reported, non-structural damage occurred mainly to components that had inadequate or no seismic restraints. It has been observed that the majority of OFCs with restraints designed according to current code requirements (i.e. 0.4g peak horizontal acceleration for zone 4 according to the Uniform Building Code [UBC]) performed well in spite of measured ground accelerations of twice the design value. They also noted that in some situations where damage occurred to OFCs with restraints designed according to current code provisions, the failures were associated with poor design details. For illustration, Figures 5 to 11, adopted from FEMA 74 (1994), McKevitt et al. (1995), and Tremblay et al. (1995), show damage to various OFCs during the Northridge earthquake.

Figure 5 Failure of suspended ceiling and light fixtures during the 1994 Northridge, California earthquake (FEMA 74, 1994)

Figure 6 Failure of air compressor during the 1994 Northridge, California earthquake (FEMA 74, 1994)

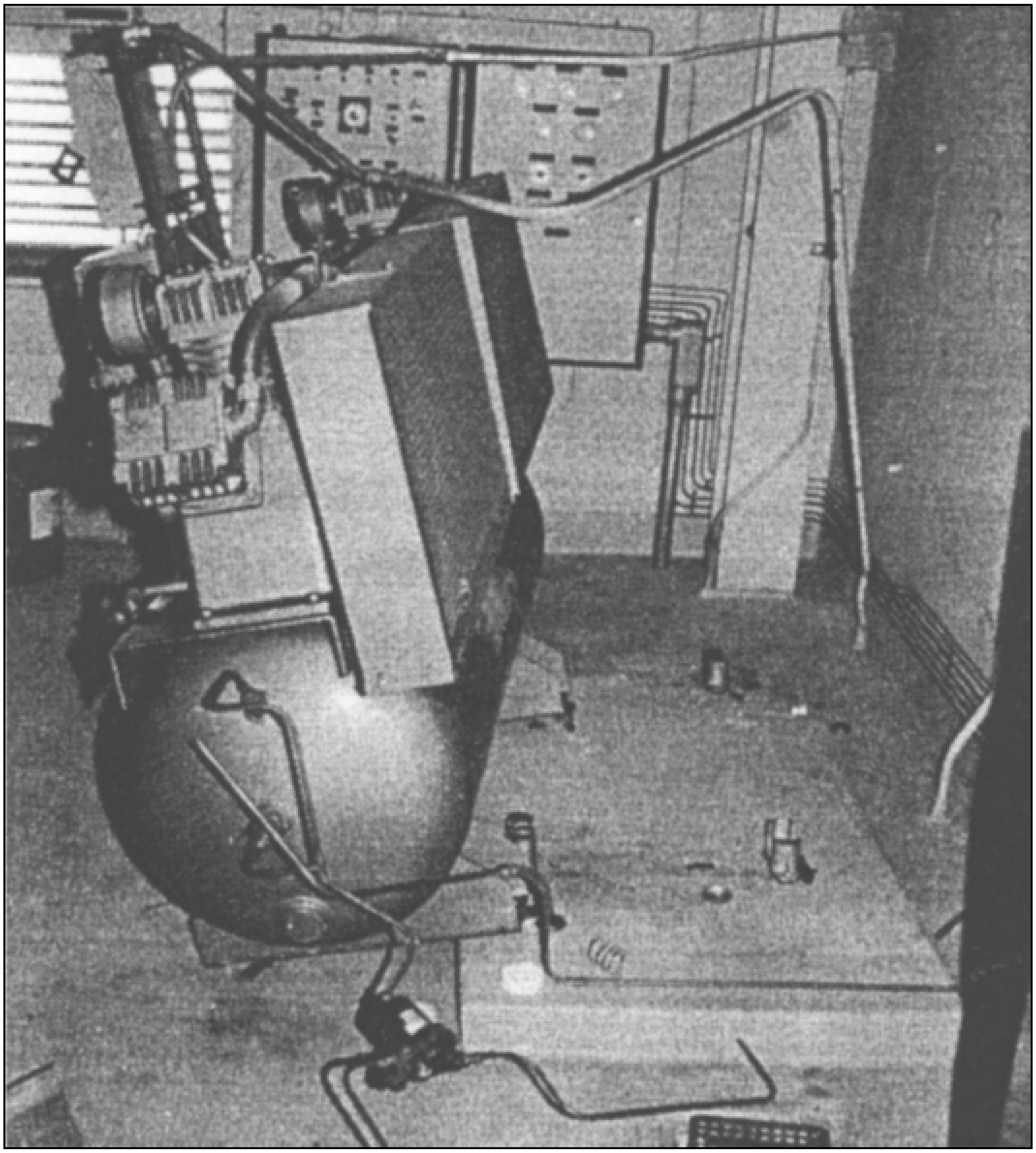


Figure 7 Failed chiller mounts during the 1994 Northridge, California earthquake (FEMA 74, 1994)

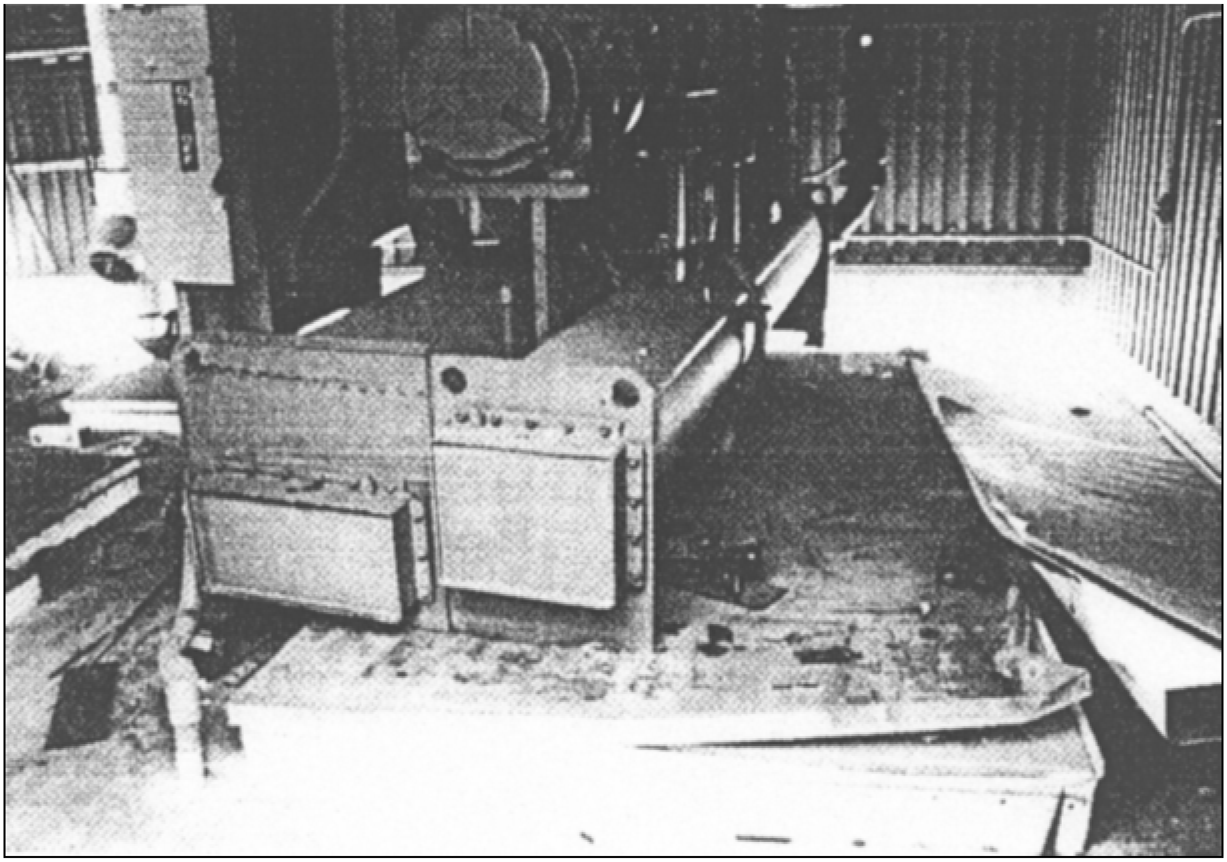


Figure 8 Failure of ducting at ventilation unit during the 1994 Northridge, California earthquake (McKevitt et al., 1995)

Figure 9 Failure of HVAC control unit support frame during the 1994 Northridge, California earthquake (McKevitt et al., 1995)

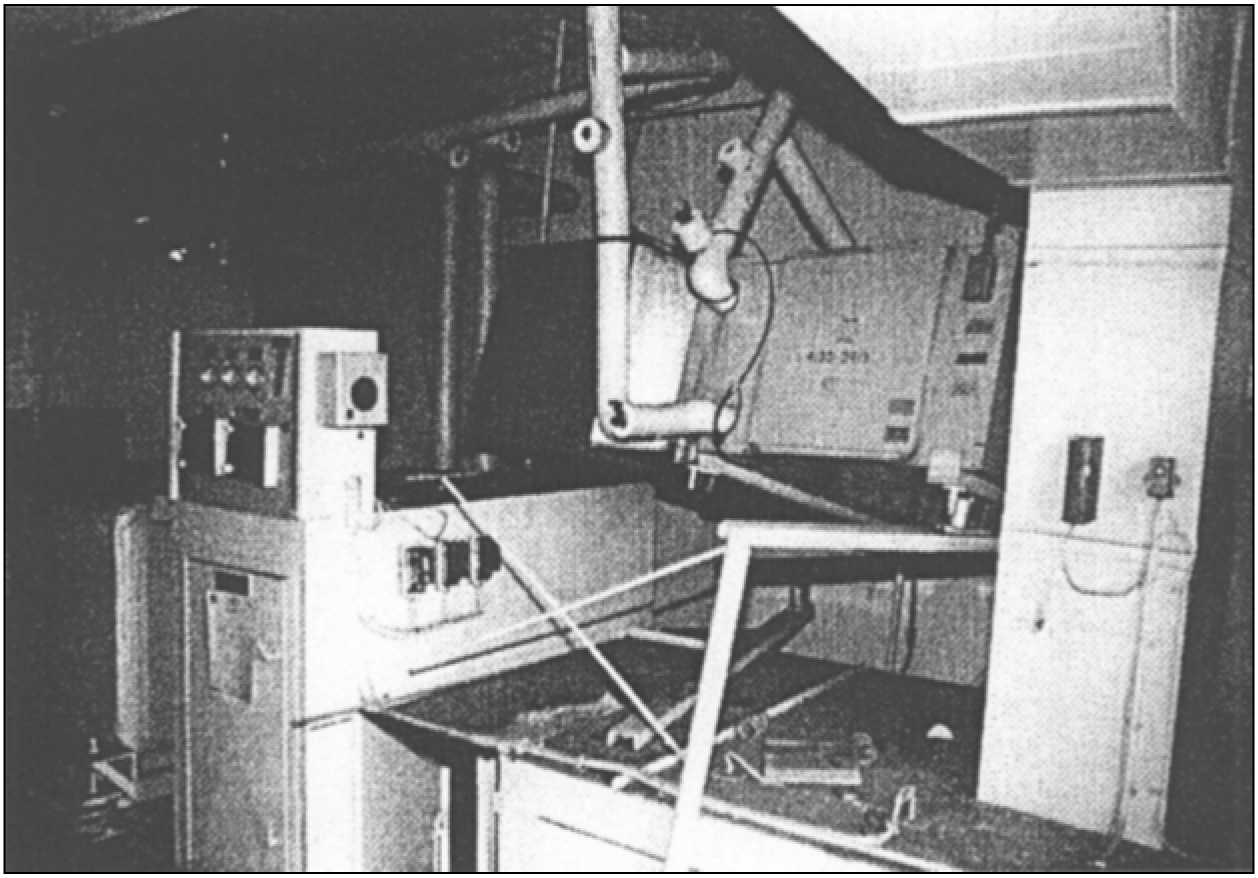


Figure 10 Racking of services attached to the penthouse roof of the Kaiser Permanente Hospital during the 1994 Northridge, California earthquake (Tremblay et al., 1995)

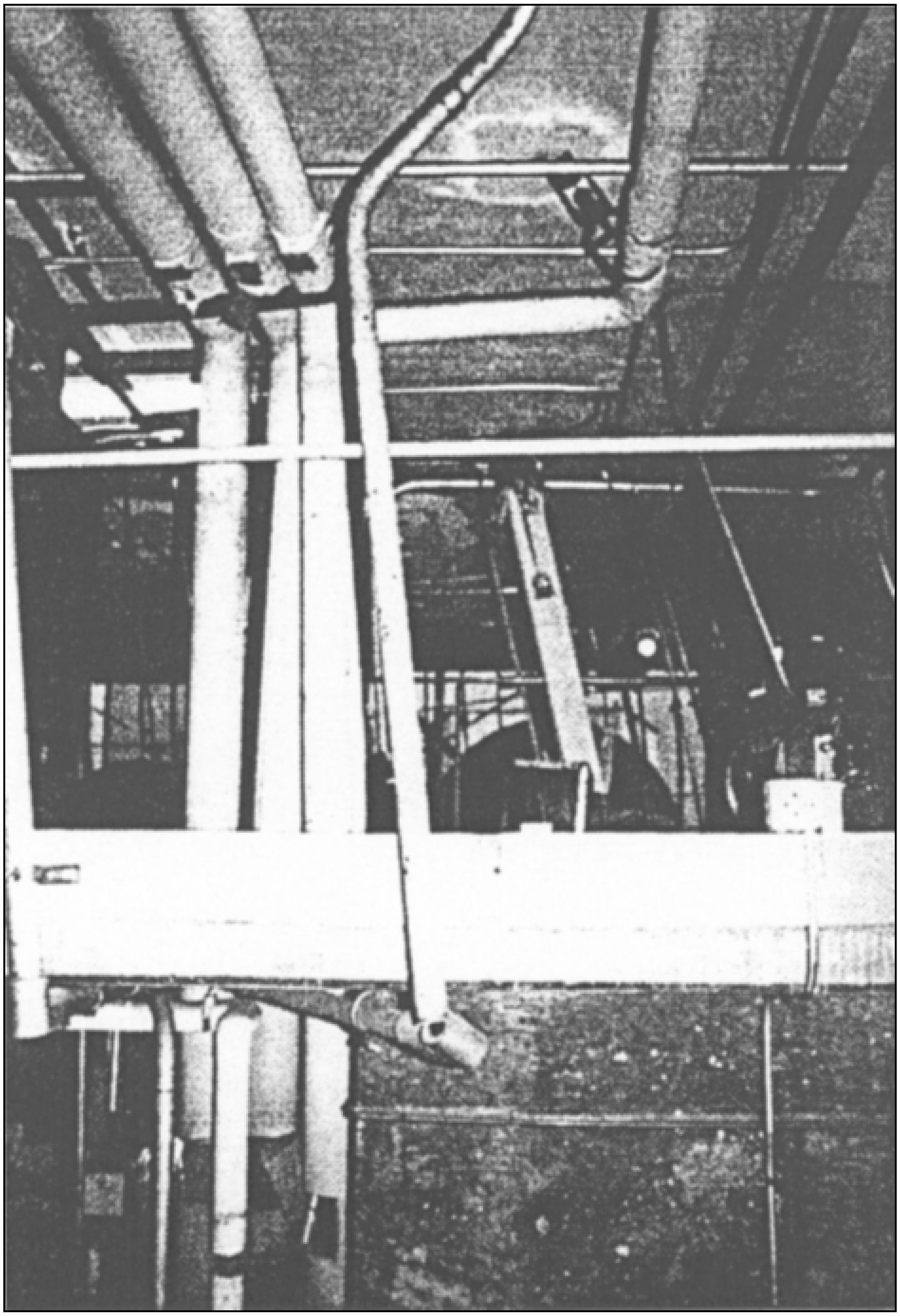


Figure 11 Damage due to inadequate restraint for pressurized tanks during the Northridge, California earthquake (McKevitt et al., 1995)

In the conclusions, McKevitt et al. (1995) pointed out that in terms of the behaviour of the OFCs during the Northridge earthquake, the most significant observations from their reconnaissance visit were the importance of the following:

- seismic restraint on ceiling systems and equipment supported in the ceilings;
- anchorage of equipment; and
- restraint and detailing of piping systems.

In addition, it is useful to mention the following two notes from the conclusions made by McKevitt et al. (1995):

- From the total number of 25 deaths during the Northridge earthquake, at least five deaths have been attributed to OFCs failures; and
- Damage to OFCs has been also responsible for many of the injuries during the earthquake, the loss of function of several hospitals, and other damage and financial loss.

Additional highlights on the damage to OFCs during the Northridge earthquake are given by Gates and McGavin (1998). As reported, sprinkler systems in many buildings failed. Glass windows with protective films to mitigate flying shards breached the window frames. It is also pointed out that mechanical and electrical equipment that was rigidly bolted or anchored to floors generally performed acceptably during the Northridge earthquake if the anchors had been designed to building code force levels. However, in many cases in which the anchor bolts in concrete floors or grade slabs were inadequate, premature failures of the anchorage systems occurred.

2.3 Performance of OFCs during the 1999 Kocaeli Earthquake in Turkey

The earthquake of 17 August 1999 resulted in over 20,000 deaths, 50,000 injured, and over $30 billion in damage. A significant portion of the damage and the casualties were attributed to the failure of non-structural elements, primarily in the form of unreinforced masonry walls (Saatcioglu, Gardner and Ghobarah, 2001). The extensive use of these brittle walls, without any seismic considerations such as proper connections or isolations and restraints, resulted in extensive damage, even if the structural framing systems remained intact in many cases. Figures 12 and 13 illustrate two reinforced concrete frame buildings, with significant damage to the non-structural masonry walls, while the reinforced concrete frame systems maintained the overall strength and stability of the structures (Saatcioglu et al, 2001).

Figure 12 Example of damage to non-structural masonry walls during the Kocaeli earthquake in Turkey (Saatcioglu et al., 2001)

Figure 13 Another example of damage to non-structural masonry walls during the Kocaeli earthquake in Turkey (Saatcioglu et al., 2001)

Damage to OFCs also played an important role on the performance of industrial facilities. A fire, caused by the rupturing of pipes that contained highly flammable substances, destroyed a major oil refinery. Figures 14 and 15 illustrate the damage to the oil refinery.

Figure 14 Damage to non-structural elements in an oil refinery during the Kocaeli earthquake in Turkey (Saatcioglu et al., 2001)

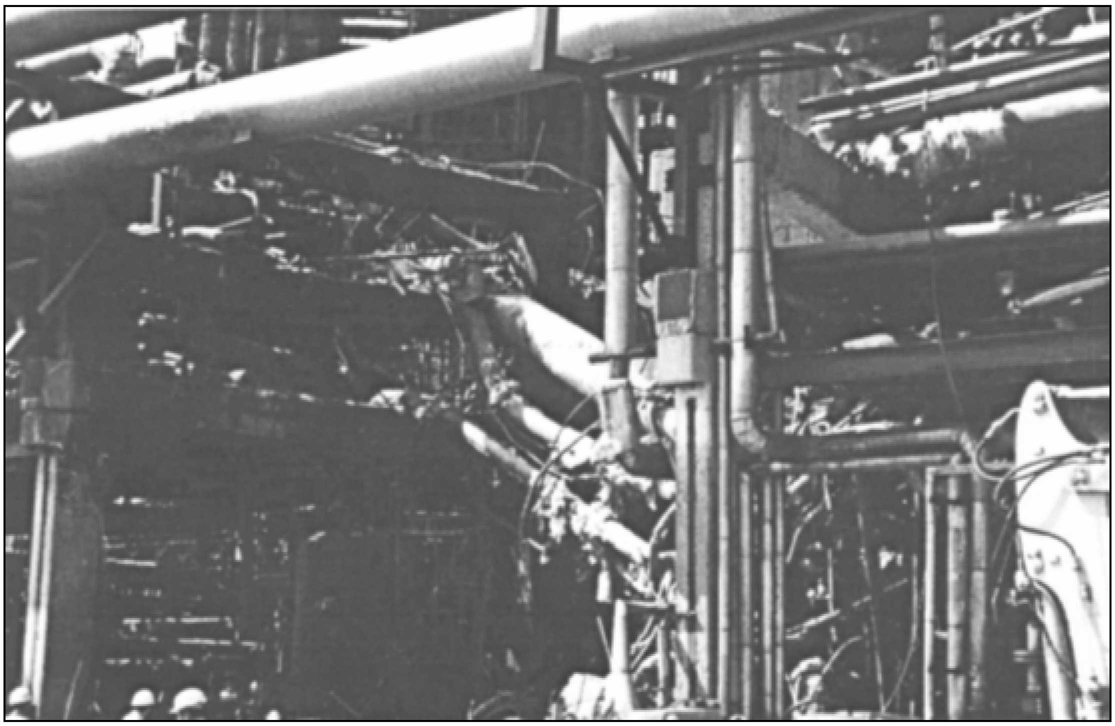
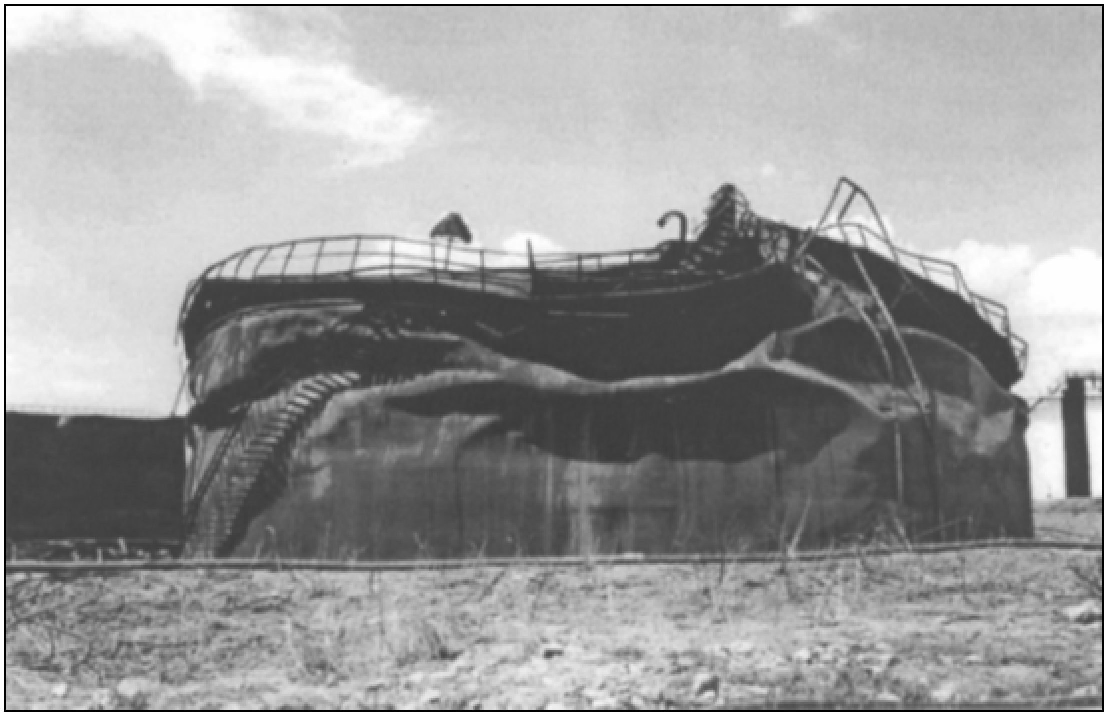
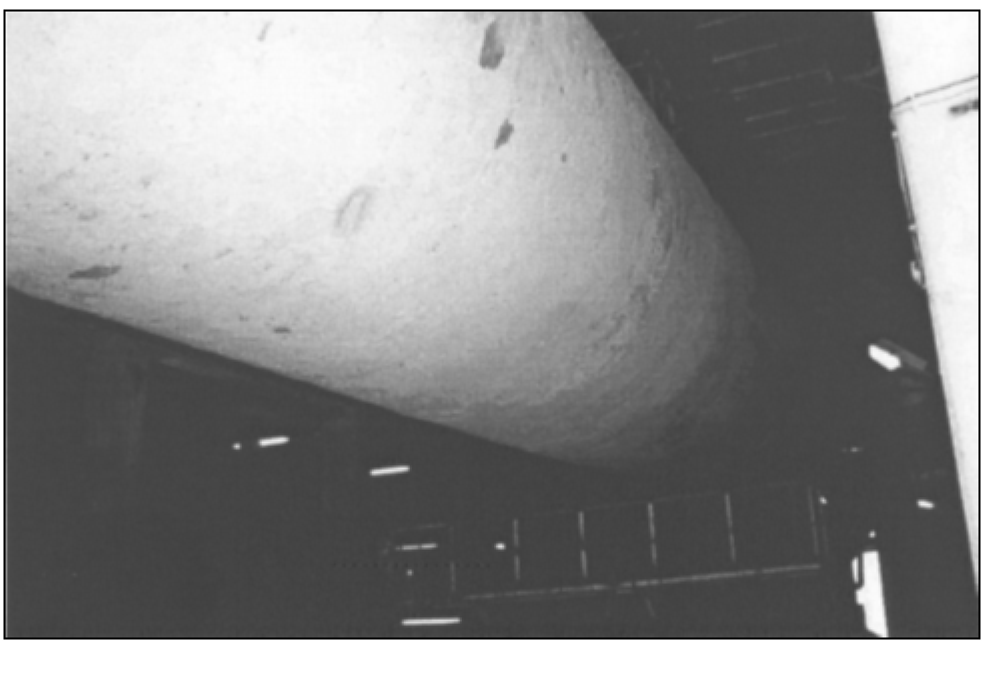


Figure 15 Damage to a storage tank in an oil refinery during the Kocaeli earthquake in Turkey (Saatcioglu et al., 2001)



Another example of OFC damage was seen in a fertilizer factory where a large kiln was displaced from its bearings, as illustrated in Figure 16.

Figure 16 A large kiln displaced from its bearings in a fertilizer factory during the Kocaeli earthquake in Turkey (Saatcioglu et al., 2001)



2.4 Performance of OFCs during the 1999 Jiji, Taiwan Earthquake

The worst casualties in the 1999 Jiji earthquake in Taiwan were a result of collapse of low-rise commercial/residential buildings with open front shops on ground level and heavy non-engineered unreinforced brick walls on upper levels. The ground level of these two, three, or four-storey buildings is commercial, while the upper floors are residential. The front of the ground floor shop is usually open.

These non-engineered, non-structural heavy walls basically changed the seismic behaviour of the reinforced concrete frame structures, which had been designed to dissipate seismic energy through displacements. The frame structures can no longer dissipate the seismic energy effectively due to their reduced displacement capabilities. Coupled with an open and potentially soft storey at the ground floor, the buildings suffered substantial damage when the weak open front failed resulting in the caving in of the heavy unreinforced brick walls.

Figures 17 to 20 illustrate a few examples of damage to and hazard of OFCs during the 1999 Jiji earthquake in Taiwan including roof-top building systems, freestanding architectural wall, sprinkler system and over-head equipment.

Figure 17 Damage to roof-top building systems during the Jiji earthquake in Taiwan

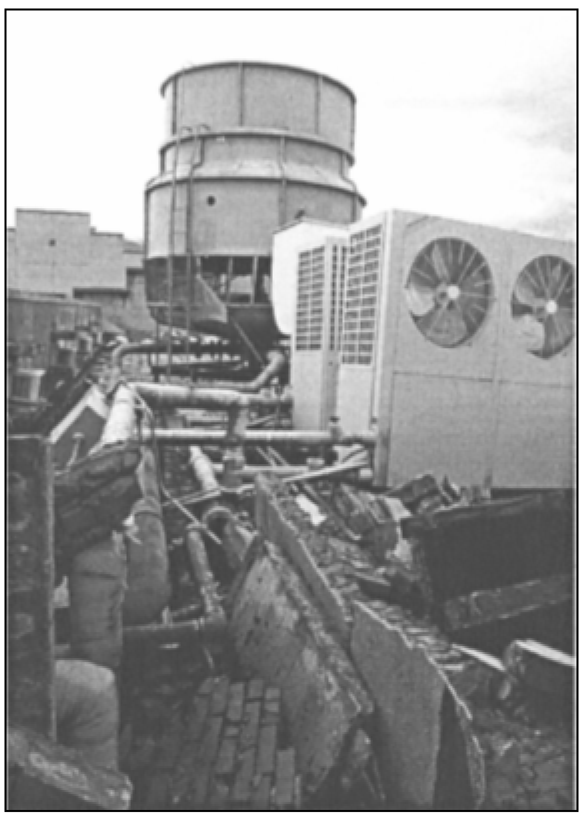


Figure 18 Collapse of free-standing non-structural wall during the Jiji earthquake in Taiwan

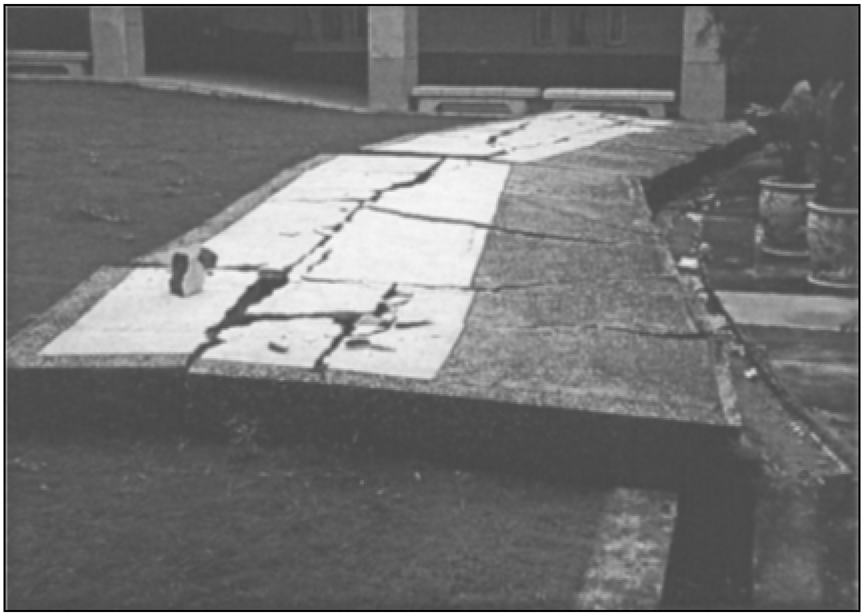


Figure 19 Damage to sprinkler system during the Jiji earthquake in Taiwan

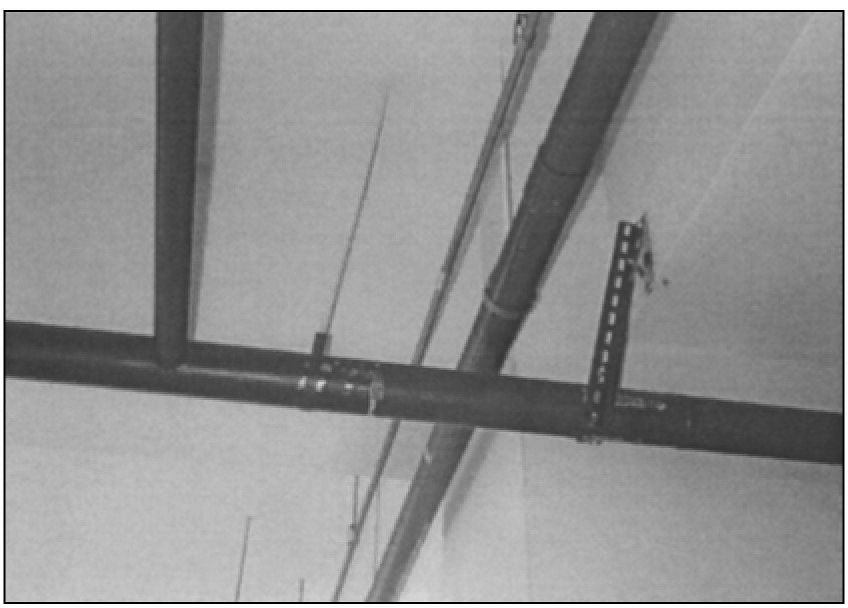


Figure 20 Hazard due to unrestrained overhead equipment during the Jiji earthquake in Taiwan.

3.0 The U.S. Approach for Seismic Risk Reduction of OFCs

Seismic provisions for OFCs can be found in:

- Uniform Building Code (UBC)
- National Earthquake Hazard Reduction Program (NEHRP)
- Industrial guidelines such as:
  - American Society of Mechanical Engineers (ASME) boiler and pressure vessel code (ASME Boiler and Pressure Vessel Code, Section III, Rules of construction of nuclear power plant components, Division 1, Appendix N, 1993, ASME, New York)
  - NFPA 13 for fire suppression systems
  - ASHRAE for mechanical components (A Practical Guide to Seismic Restraint 1999; the 1999 ASHRAE Handbook – HVAC Applications, Chapter 50 “Seismic Restraint Design” ASHRAE, 1999)
  - SMACNA (Seismic Restraint Manual, 1997)
  - Bellcore – “Network Equipment-building System (NEBS) Requirements: Physical Protection”. (Bell Communications Research.)

Provisions of the UBC and NEHRP provisions are discussed here because they are well-known worldwide and widely used. Brief discussions are also given of various industrial guidelines.

3.1 UBC Requirements for OFCs

The Uniform Building Code (UBC; International Conference of Building Officials, 1997) is the most widely used code in the U.S. for seismic design. In addition to the specifications for design of structural building components, which constitute the primary objective of the code, it also provides criteria for design of non-structural components, i.e. OFCs. This design should be conducted using both the code prescribed seismic lateral forces for the OFCs and the maximum inelastic deflections of the building structure.

3.1.1 Seismic Forces

Non-structural components are subjected to horizontal accelerations during an earthquake, resulting in lateral inertial forces. These forces are very important for both the design of non-structural component connections, as well as structural members to which the non-structural components are attached. UBC 1997 requires that the total design lateral seismic force, F_p, for a given OFC should be calculated as:

[1] $F_p = 4.0\ C_a\ I_p\ W_p$

Alternatively, F_p may be calculated using the following formula:

[2] $F_p = (a_p\ C_a\ I_p / R) \times (1 + 3\ h_x / h_r)\ W_p$

Except that F_p shall not be less than $0.7\ C_a\ I_p\ W_p$ and not more than $4C_a\ I_p\ W_p$.

The parameters in these equations are as follows:

C_a = seismic coefficient which depends on the seismic zone and the type of soil at the location of the building (Table 16-Q in UBC, 1997)
I_p = importance factor which depends on the occupancy category of the building, and has values of 1.0 or 1.5 (Table 16-K in UBC, 1997)
W_p = operating weight of the non-structural component
R_p = response modification factor of the non-structural component, ranging from 1.5 to 4 (Table 16-O in UBC 1997)
h_x = elevation of the component attachment with respect to the grade level, h_x shall not be taken less than 0.0
h_r = roof elevation of the building with respect to the grade level
a_p = in-structure component amplification factor that varies from 1.0 to 2.5 (Table 16-O in UBC, 1997)

UBC 1997 also allows calculating the seismic force F_p according to other approved methods, standards or from experimental tests. However, the value of F_p used in the design of the non-structural component should not be less than 80 percent of the values calculated according to Equations [1] and [2].

3.1.2 Maximum Lateral Deflections

It is essential that the maximum expected deflections of the OFCs and the building are taken into account in the design of OFCs. A proper consideration of these deflections in terms of providing adequate gaps between the structural components and OFCs would lead to significant reduction of earthquake damage to both structural components and OFCs.

UBC 1997 specifies the following formula for calculation of the maximum interstorey drift, Δ_M, of building structures:

[3] $\Delta_M = 0.7\ R\ \Delta_S$ where,

R = force modification factor representative of the inherent overstrength and global ductility capacity of the lateral load resisting system, ranging from 2.2 to 8.5 (Table 16N in UBC, 1997)

Δ_S = design interstorey drift resulting from the design seismic forces

UBC 1997 also allows calculation of interstorey drifts by nonlinear time history analysis. As pointed out in UBC 1997, the calculated drift should not exceed the value of 0.025 times the storey height for structures having a fundamental period of less than 0.7 second. For structures having a fundamental period of 0.7 second and greater, the calculated drift should not exceed 0.02 times the storey height.

3.2 FEMA Requirements for OFCs

As was mentioned in the introduction, a comprehensive project related to OFCs has been undertaken in the U.S., as part of the National Earthquake Hazards Reduction Program (NEHRP), and sponsored by the Federal Emergency Management Agency (FEMA). Results from this project are presented in a number of reports, referred to as FEMA reports in the further text. The reports address the following major issues related to OFCs:

1. Design requirements for OFCs of new buildings (FEMA 302, Chapter 6 – Recommended Provisions, and FEMA 303, Chapter 6 – Commentary to the Provisions; Building Seismic Safety Council, 1998).
2. Seismic evaluation of OFCs (FEMA 310, Section 4.8, Building Seismic Safety Council, 1998; FEMA 178, Sections 10.4 and 10.5, Building Seismic Safety Council, 1992).
3. Rehabilitation of OFCs in existing buildings (FEMA 273, Chapter 11 – Simplified and Systematic Rehabilitation of OFCs, and FEMA 274, Chapter 11 – Commentary; Building Seismic Safety Council, 1997).
4. Techniques for Seismic Rehabilitation of OFCs (FEMA 74, Wiss, Janney, Elstner Associates, Inc., 1994; FEMA 241, Bay Area Regional Earthquake Preparedness Project, and Office of the State Architect, 1993; FEMA 172, Chapters 5 and 6, Seismic Safety Council, 1992).

Among these reports, the most comprehensive discussion of various issues related to OFCs of building structures is presented in FEMA 273 and FEMA 274, and these represent the basis for the latest reports, i.e. FEMA 302 and 303.

The most important issues for OFCs discussed in the FEMA reports (such as performance levels, evaluation methods, etc.) are adopted by the CSA Technical Committee on Seismic Risk Reduction (2000) in the development of the Guidelines for seismic risk reduction of OFCs of buildings and these are discussed in Section 4.3 of this report. In order to avoid repetition, only the FEMA specifications for the analytical methods of calculating seismic forces and displacements of OFCs are presented, as they are somewhat different than those in the Guideline prepared by the CSA Technical Committee on Seismic Risk Reduction (2000).

3.2.1 Seismic Forces (FEMA 273, Sections 11.7.3 and 11.7.4)

According to FEMA 273 (Section 11.7.3), the seismic force for a given OFC should be calculated using the equation:

[4] $F_p = 1.6\, S_{xs}\, I_p\, W_p$ where,

F_p = Seismic design force applied horizontally at the component's centre of gravity and distributed relative to the component's mass distribution
S_{xs} = Spectral acceleration, at short periods, of the design spectrum as defined in Section 2.6.1.5 of FEMA 273
I_p = Component performance factor that is either 1.0 for Life Safety Performance Level or 1.5 for Immediate Occupancy Performance Level
W_p = Weight of the component

Alternatively, the seismic force can be calculated using the equation as specified in FEMA 273, Section 11.7.4:

[5] $F_p = 0.4\, a_p\, S_{xs}\, I_p\, W_p\, (1 + 2x/h) / R_p$

But F_p should not be taken greater than the value calculated by equation [4], and not less than:

[6] $F_p = 0.3\, S_{xs}\, I_p\, W_p$

The parameters S_{xs}, I_p and W_p are described above; the other parameters used in Equations [5] and [6] are as follows:

a_p = component amplification factor that varies between 1.0 and 2.5 (see Table 11-2 in FEMA 273)
x = average roof height of the building structure relative to the grade elevation
h = height of the point where the component is attached relative to the grade elevation (for items at or below the base, h should be taken as 0.0; see Section 6.1.3 in FEMA 302)

In the FEMA specifications, Equation [4] is treated as the “default” equation that gives conservative results (see FEMA 274 – Commentary, Sections C11.7.3 and C11.7.4); Equations [5] and [6] give more precise and generally less conservative results. For a component such as a heavy cladding, where connections are critical, the more precise analytical procedure (i.e. Equations [5] and [6]) should always be used.

3.2.2 Drift Ratios and Relative Displacements (FEMA 273, Section 11.7.5)

The drift ratio, D_r, for two connection points on the same building or structural system should be calculated using the equation as specified in FEMA 273, Section 11.7.5:

[7] $D_r=(\delta_{xA}-\delta_{yA})/(X-Y)$

Similarly, the relative displacement, D_p, of two connection points on separate buildings or structural systems should be calculated using the equation:

[8] $D_p= |\delta_{xA}| + |\delta_{xB}|$

The parameters used in Equations [7] and [8] are as follows:

X = height of upper support attachment at level “x” measured from the grade level
Y = height of lower support attachment at level “y” measured from the grade level
δ_{xA} = deflection at level “x” of building “A”
δ_{yA} = deflection at level “y” of building “A”
δ_{xB} = deflection at level “x” of building “B”

The deflections, δ, should be calculated using the design seismic loads and the stiffness properties of the structural systems.

3.3 Industrial Guidelines

Industrial guidelines are usually developed for specific equipment or systems and often offer details on the seismic restraint system of such equipment or systems. It is common practice and recommended in the guidelines that an experienced design professional be required for proper use of the guidelines, especially for conditions not covered by the details on seismic restraint systems.

American Society of Heating, Refrigerating and Air-Conditioning Engineers (ASHRAE, 1999) published a comprehensive guide for the seismic restraint of HVAC, plumbing and electrical systems. It is regarded as one of the most complete guides for seismic risk reduction of non-structural components. Code requirements as well as load calculations and distribution of the restraint are examined extensively in the guide. Illustrations are given on brace details and restraint guidance for equipment, pipes and ducts.

“Seismic Restraint Manual, Guidelines for Mechanical Systems,” published by the Sheet Metal and Air Conditioning Contractors’ National Association, Inc., (SMACNA, 1998) provides extensive details of bracing systems for a wide range of pipe, duct, and conduit installations. SMACNA is most widely used for seismic restraint of mechanical systems. SMACNA specifications are prescriptive in nature in that no determination of design force is required. The guide does not cover isolated ducts and fire sprinkler systems.

"Standard for the Installation of Sprinkler Systems," published by the National Fire Protection Association (NFPA, 1994), is referenced in most codes, including NBCC 1995. It provides guidance for restraint of fire sprinkler piping. It provides prescriptive requirements, similar to SMACNA, in terms of tables for determining the bracing system (size, spacing and attachment details) for the fire sprinkler piping. Allowance is made for piping that enters into buildings, crossing seismic joints and pipe risers. The standard does not cover equipment or duct, and applies only to rigidly mounted fire sprinkler pipe and standpipe installations.

Bell Communications Research, or Bellcore, published a document (Bell Communications Research, 1995) which provides spatial requirements, framework criteria (such as structural, geometric, wiring/cable access requirements), environmental criteria (such as temperature, humidity, fire resistance, earthquake, vibration, air quality, acoustical and illumination requirements) and test methods for network and telecommunications equipment. Detailed earthquake test methods (such as the configuration, framework and anchors, required response spectra, static and shake table testing procedure and reporting) are given in the document. Bellcore recommends that the equipment to be used in seismic Zones 1 through 4 should be tested to determine the equipment's ability to withstand earthquakes. No earthquake test is required for equipment in Zone 0.

NOTE: It is a general practice in North America to conduct shake table tests for critical or expensive equipment/systems (using the U.S. specifications) in order to verify its dynamic characteristics and its response to earthquakes. The fact remains, however, that components such as motor control centre units or emergency power systems, which meet the U.S. specifications, may not perform satisfactorily under Canadian seismicity and building construction practices. There is a pressing need for the development of Canadian specifications for the proper assessment and mitigation of seismic hazards associated with functional and operational components.

4.0 Canadian Approach For Seismic Risk Reduction of OFCs

4.1 NBCC requirements for OFCs

The National Building Code of Canada (NBCC) (Associate Committee on the National Building Code, 1995) provides minimum provisions for the safety of buildings with reference to public safety, fire protection and structural performance. The provisions for design of building structures are given in Part 4 of the Code – Structural Design, Section 1 – Structural Loads and Procedures. While this section is primarily intended for *structural* components of buildings, provisions are also given for OFCs, which are referred to as non-structural components (i.e. architectural, mechanical and electrical parts or portions of buildings). This is because public safety is ensured not only by structural but also non-structural components of buildings. The Code specifically addresses two issues related to non-structural components: (i) lateral forces that are needed for proper anchorage of these components, and (ii) deflections of building structures which structural and non-structural components can tolerate. Both of these issues are discussed briefly here.

4.1.1 Seismic Forces

The lateral inertia force, V_p, is defined as:

[9] $V_p = v\ I\ S_p\ W_p$ (see clause 4.1.9.1(15) in NBCC 1995)

where,

v = zonal velocity ratio for the location of building (from the seismic zoning maps of NBCC 1995)
I = seismic importance factor for the building ($I = 1.0$ for ordinary buildings, 1.3 for schools, and 1.5 for post-disaster buildings)
S_p = horizontal force factor for the part or portion of building and its anchorage, as given in Table 2 in Appendix A for architectural components, and Equation [10] for mechanical/electrical components
W_p = the weight of part or portion of structure

For mechanical/electrical components, the value of S_p in Equation [9] is defined as:

[10] $S_p = C_p\ A_r\ A_x$ (see Clause 4.1.9.1(19) in NBCC 1995)

where,

C_p = seismic coefficient for components of mechanical and electrical equipment as given in Table 3 in Appendix A
A_r = response amplification factor to account for type of attachment of mechanical/electrical equipment ($A_r = 1$ 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 for all other cases)
$A_x = 1.0 + (h_x / h_n)$ is amplification factor at level x to account for variation of response of mechanical/electrical equipment with elevation within the building (h_n is the height of the building above its base, and h_x is the height of the component above the base of building)

Mechanical/electrical components that are both rigid and rigidly-connected, are defined as those having a fundamental period of less than or equal to 0.6 seconds. Flexible components are those that have a fundamental period greater than 0.6 seconds. The background information for these criteria is contained in Commentary J to Part 4 of NBCC 1995.

4.1.2 Maximum Lateral Deflections

The level of maximum lateral deflection of a building structure is an important parameter for providing adequate separation between structural components and OFCs. According to NBCC 1995, the maximum lateral deflection should be computed by multiplying deflections obtained from elastic analysis of the structure (under design loads) by the force reduction factor R (Clause

4.1.9.2[2]). Having determined maximum deflections, interstorey drifts can be computed, which are essential for design of partitions and other OFCs. In order to avoid excessive deformations of both the structural components and the OFCs, NBCC 1995 limits the interstorey drift to 0.01 h_s for post-disaster buildings, and to 0.02 h_s for all other buildings, where h_s is the storey height.

4.2 CSA Guideline for Seismic Risk Reduction of OFCs

The development of a new "CSA Guideline on Seismic Risk Reduction of OFCs of Buildings" has evolved from an initiative of Public Works and Government Services Canada (PWGSC) for a project on seismic evaluation and upgrading of non-structural components in the early 1990s. The work on this project was conducted in conjunction with the Institute of Research in Construction of the National Research Council of Canada and the private sector. The results from the work were published in an internal document entitled *Guideline on Seismic Evaluation and Upgrading of Non-structural Building Components* (PWGSC, 1995). This document was restricted to normal office buildings and libraries. The PWGSC document has been rather widely used and accepted for its unique application to OFCs.

During the course of the development and application of the PWGSC guideline, it was acknowledged that there was a need for a comprehensive national standard for seismic risk reduction of non-structural building components. The Technical Committee for CSA S832-2000 was formed in September 1997 for the development of a new CSA guideline (Cheung, Foo and McClure, 2000). The Guideline provides: (a) information and methodology to identify and evaluate earthquake hazards of OFCs, and (b) design approaches to achieve adequate mitigation. It covers most buildings (new or existing, including renovations) with major occupancy classifications listed in Appendix A of NBCC 1995, such as office and residential buildings, schools and hospitals. The structural integrity of the building itself is covered by Part 4 of NBCC 1995 and is not addressed in the Guideline. As indicated in Table 1 (extracted from the Guideline), lifeline systems and utilities inside a building are covered together with their interfacing details at the building junction.

The Guideline introduced a new parametric method for conducting risk assessment of OFCs. The risk assessment of a given OFC involves the evaluation of combined effects of seismic vulnerability of the OFC and its consequences of failure under the design earthquake. To quantify the risk, the Guideline proposes the use of a parametric method, which consists of the following steps:

1. Determine the vulnerability rating, V, (Table 4 in Appendix A).
2. Determine the consequences rating, C, (Table 5 in Appendix A).
3. Calculate the seismic risk rating score, R = V x C.

A final rating score, R, of less than 16 represents a <u>low</u> seismic risk, whereas a rating of 16 to 49 represents a <u>moderate</u> risk, and a rating larger than 49 represents a <u>high</u> risk. It was noted that the parameters, the rating scales and the weighting factors given in the tables were not the final values, as the work on the validation and calibration of the method was still underway.

The Guideline suggested four mitigation options for OFCs that require modifications in order to reduce or eliminate potential risks when subjected to the design seismic motion. These options, called "the 4R's options" are as follows:

- **Restrain** – The component is attached in a manner that can accommodate movement of the component within acceptable limits when the building is subjected to an earthquake.
- **Relocate** – The component is moved to another location in the building that is away from potential human traffic areas.
- **Remove** – The component is removed from the building.
- **Replace** – The component is removed from the building and a less hazardous substitute is used in its place.

Top mitigation priority should be given to OFCs whose consequences of failure are a threat to life safety. However, once life safety is ensured, the owner/operator may influence the order of priority obtained with the seismic risk rating. In a situation where the risk is about the same, the Guideline recommends that priority be given to OFCs with highest consequences of failure rating.

The Guideline proposes procedures for assessing and mitigating seismic risk of OFCs for both the new construction and renovations, with sample applications. The Guideline also provides mitigation considerations and techniques for various types of OFCs.

5.0 Research in Seismic Risk Reduction of OFCs

Several state-of-the-art reviews on seismic performance of OFCs have been reported over the years; notably that of Chen and Soong (1988), Soong (1994) and Villaverde (1997). There were also several interesting articles on OFCs in the recent 12th World Conference on Earthquake Engineering held in Auckland, New Zealand in January 2000. OFCs have often been referred to as non-structural elements or secondary structures in the literature.

Villaverde (Villaverde, 1997) stated that, in the 1994 Northridge earthquake, several major hospitals had to be evacuated because of damage to their OFCs and not due to structural damage to the buildings. Several incidences of casualty caused by failure of OFCs were highlighted to emphasize the dire consequence of not safeguarding OFCs. Historical perspective of the seismic response and design provisions in building codes for OFCs were presented with emphasis on NEHRP and UBC provisions. Villaverde indicated that the majority of research in the area was of analytical nature and that experimental tests and field observation were scarce. The report cited other experimental investigations on light equipment, cladding systems, piping systems, critical equipment for nuclear power plants, windows and library shelving units involving either static or shake-table tests. It appeared that most experimental tests were conducted for specific equipment without co-ordination among tests and researchers, and that no consideration was given to the verification and improvement of experimental performance against field observations and conditions.

Beattie (Beattie, 2000) reported on shake table tests conducted in the BRANZ (Building Research Association of New Zealand). The following three equipment set-ups were installed and tested: an air handling unit and a duct with a flexible connection, a fire sprinkler distribution network, and a pump with pipe work suspended from the floor ceiling. Test results confirmed that input accelerations from the building structure would be amplified at these components with a resulting acceleration many times the input acceleration. Beattie also provided a summary of the design requirements contained in a newly produced design guideline for improving seismic performance of OFCs by BRANZ. It was reported that the design acceleration for OFCs, in a ductile frame on intermediate soil site, varied from 1.73 g for a four-storey building to 1.01 g for a 15-storey building. The design acceleration could be further increased according to the building importance factor and building risk factor.

Yao (Yao, 2000) presented the results of analytical and experimental investigation on seismic performance of suspended ceilings. Test results showed that the installation of sway bracing did not seem to have any impact on ceiling performance. One main observation was that the ceilings with transverse supports (cross runners) could sustain much higher accelerations before separation of the runners or the falling of panels than ceilings without the transverse supports (i.e. 2.5 g compared to 0.9 g). The transverse supports provided lateral constraints to limit the lateral spread of the runner grids and maintain the integrity of the system up to an acceleration of 2.5 g.

Villaverde (Villaverde, 2000) proposed a simplified method for seismic nonlinear analysis of OFCs, which accounts for nonlinear characteristics of components as well as the structure to which the components are attached. Required input included geometric characteristics, weights and ductility factors of the OFC and its supporting structure, and the elastic design spectra of the structure. Examples were given on the application of the proposed method. It is a rather simplified approach, however, no verification against analytical and experimental results was presented.

Marsantyo et al (Marsantyo et al., 2000) studied dynamic response of building equipment and contents mounted on floors of buildings. Both analytical and experimental (using shake table) investigations were carried out. While the equipment was fixed to the floor, building contents could be fixed to the floor, freely laid on the floor, or mounted on suspended pendulum isolation systems. It was found that, due to the acceleration amplification effect, low damping non-structural systems could produce acceleration responses exceeding the design code stipulations. It was further suggested that isolation systems might be the best solution to large acceleration responses of structural and non-structural systems.

6.0 Selections and Design of Building Structures

The design floor response spectra are to be developed for typical reinforced concrete buildings in Western and Eastern Canada. Two 10-storey concrete moment resisting frame buildings were selected and designed for this purpose to incorporate seismic conditions in Vancouver (Western Canada) and Ottawa (Eastern Canada). Figures 21 and 22 illustrate the plan and elevation views of the two buildings selected. The building frames were considered as bare frames, without any participation from non-structural elements, such as architectural and masonry enclosure and/or partition walls. A symmetrical floor plan was selected to minimize the effect of torsion. The load combinations considered were based on the current NBCC (1995).

Figure 21 Typical floor plan of the frame structures designed

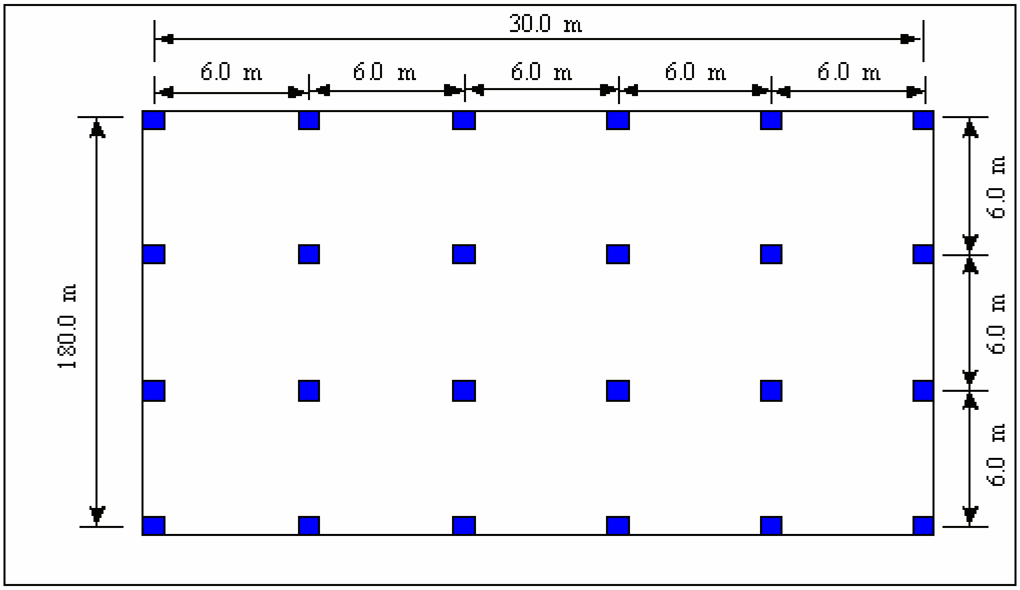
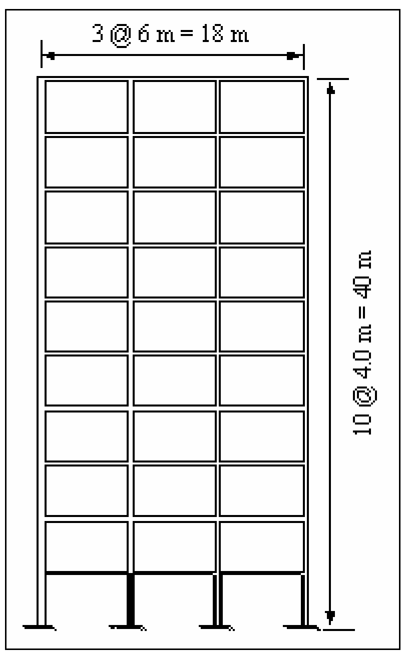


Figure 22 Elevation view of the buildings



Seismic design loads were calculated on the basis of NBCC (1995). Accordingly, the elastic design base shear, Ve, was calculated from Equation 11.

[11] $V_e = v\,S\,F\,I\,W$ where,

v = Zonal velocity ratio for a given location
S = Seismic response factor based on the fundamental period of structure
F = Foundation factor, reflecting the soil conditions
I = Importance factor
W = Weight of structure

The elastic design base shear was reduced to allow for inelastic deformability of the structure, and used as design base shear. The reduction was introduced through "R" factor. The resultant force was further modified by coefficient "U" to account for possible over-strength, past experience and the expected level of performance.

[12] $V = \frac{V_e}{R} U$ where U = 0.6

The reduction factor "R" is intended to introduce the ductility of the seismic resisting system. For ductile moment resisting elements the reduction factor is equal to 4.0 and for nominally ductile concrete frame systems it may be taken as 2.0. The structure in Vancouver was designed to be fully ductile, and hence was designed using R = 4.0. The building in Ottawa was designed for base shear associated with an R = 2.0. The fundamental period, T was computed based on the code specified empirical expression that indirectly accounted for the presence of non-structural elements. This expression is shown below for concrete frame buildings:

[13] $T = 0.075(h_n)^{3/4}$

Because of the R factors selected for the two buildings (R for a Vancouver building is twice the vale of R for an Ottawa building) and the zonal velocity ratio v specified for the two cities (v for Vancouver is twice the v for Ottawa), the design base shears for the two buildings were the same, and the difference in design was essentially due to the detailing required to attain respective ductility levels. Building designs were conducted using computer software SAP90. Two-dimensional static analyses were carried out to determine critical values of axial forces, shear forces, and bending moments at each joint, using preliminary member dimensions. This information was then used to design columns and beams.

The design was carried out following the requirements of CSA Standard A23.3 (1994) "Design of Reinforced Concrete Buildings." Chapter 21 of the same Standard was followed for seismic design and detailing. The building in Vancouver was designed for R = 4.0, and was detailed as fully ductile moment resisting frame building. On the other hand, the building in Ottawa was designed for R = 2.0, and was detailed as a nominally ductile frame building. Accordingly, the beam moments, both in positive and negative moment regions were proportioned to meet the minimum moment capacity provisions outlined in the Standard. This implies that, for the fully ductile structure, the positive beam moment near the columns was increased to 50% of the maximum negative beam moment capacity, and elsewhere in the beam, both the negative and positive moment capacities were ensured to be at least equal to 25% of the maximum negative moment capacity. This resulted in continuity in top and bottom reinforcement. Similarly, for the nominally ductile frame designed for Ottawa, the positive beam moment capacity near the column was increased to 33% of the maximum negative beam moment, and elsewhere the positive and negative moment capacities were ensured to be at least equal to 20% of the maximum negative beam moment.

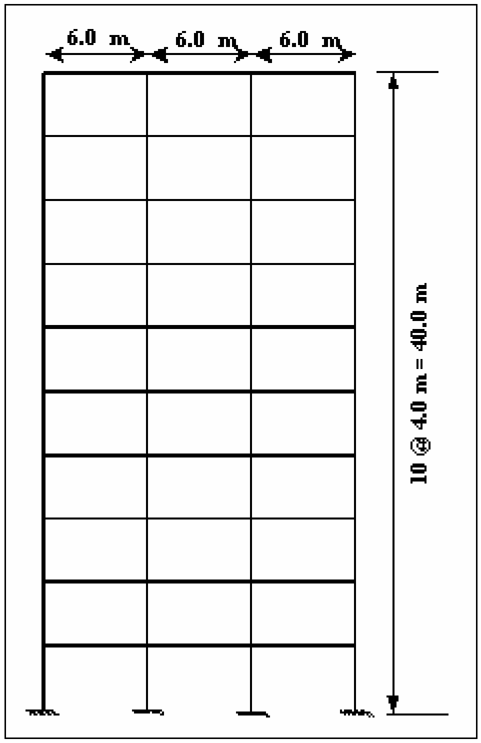
The flexural design was carried out on the basis of the strong-column weak-beam concept. Accordingly, the summation of beam moment resistances at each beam-column connection was made to be at least equal to 110% of the summation of the connecting column moments. The potential plastic hinge regions at the ends of beams and columns were confined by closely spaced transverse reinforcement. The confinement steel requirements were implemented for both ductile and nominally ductile elements. The transverse reinforcement was also designed to prevent premature shear failure. Design shear force was calculated at the formation of plastic hinges at member ends for the fully ductile frame. Beams were assumed to develop their probable moment resistances at the ends. Design shear forces at beam and column ends were calculated to equilibrate the probable moment resistances. The same approach was used to establish the seismic design shear force for nominally ductile structure, in Ottawa. This time, however, the member end moments used were the nominal resistances at the ends of beams. Once the design shear force was calculated, the required shear reinforcement was established with due considerations given to the shear resisted by concrete.

Design of members was carried out as explained above, based on the requirements of CSA Standard A23.3 (1994). Sometimes, this necessitated the revision of member sizes. When the member sizes were revised, a new frame was generated to conduct the lateral load analysis under seismic forces, and the design was revised based on the new set of forces.

7.0 Modelling Structures for Inelastic Analysis

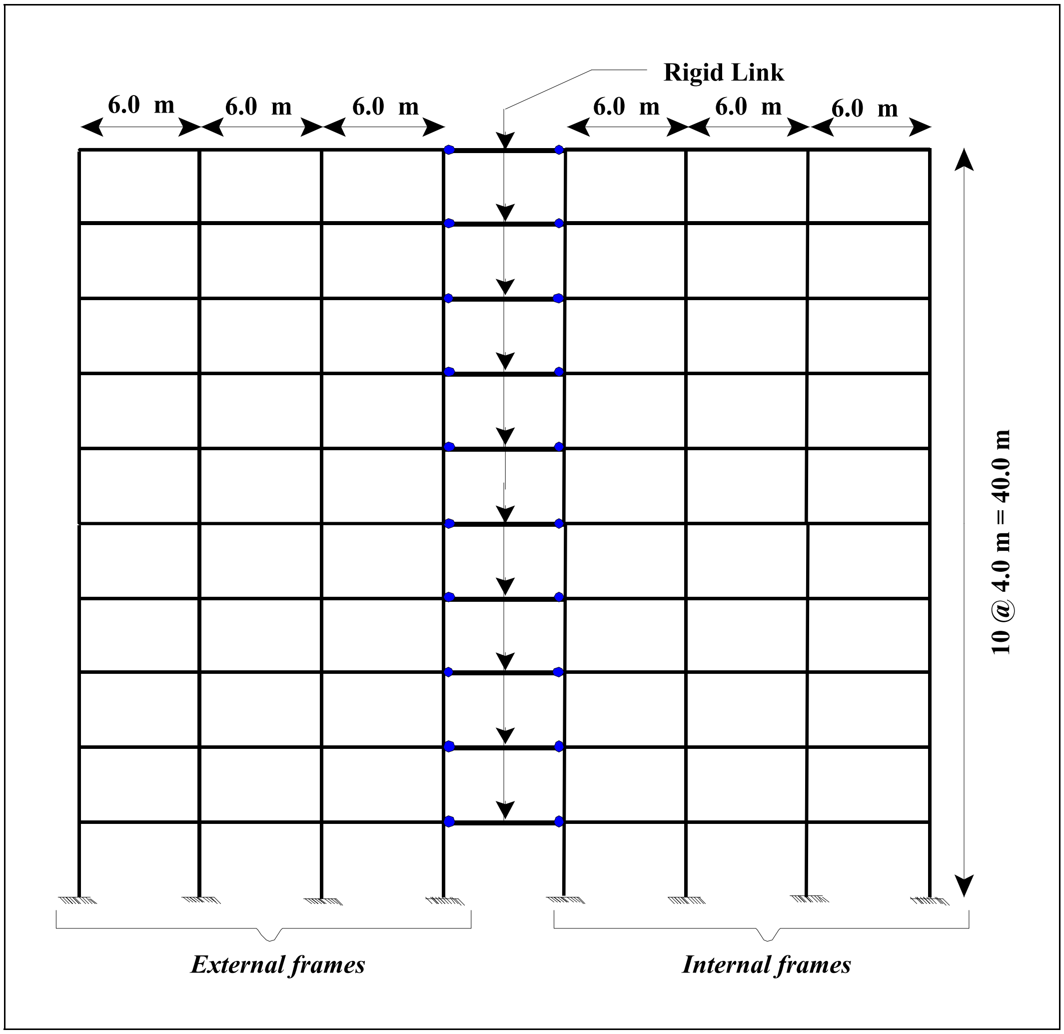
There are three stages of modelling that an analyst has to go through prior to performing dynamic inelastic analysis. The first stage involves representation of the entire structure by line elements or finite elements. When a stiffness or flexibility method of structural analysis is employed, line elements are used to model the building. Vertical lines represent the columns, and horizontal lines represent the beams. Figure 23 shows the modelling of a single for frame by line elements, for the structures selected.

Figure 23 Modelling a single frame by line elements



Structures consist of a number of frames. Therefore, the structural model consists of a combination of frame models, like the one shown in Figure 23. It may be possible to construct the structural model with as many frame models as the number of frames in the structure. Lumping similar frames into a single frame reduces the number of elements and the required analysis time, and may be necessary for a research project where extensive analysis of structures are to be conducted. It is often more convenient to lump identical frames together and represent them with a frame model that has the same properties as the combined properties of frames. Furthermore, it is more convenient and less time consuming to model a three-dimensional structure with a two-dimensional model, especially if the required analysis is a plane-frame analysis. This can be done by assuming that the building has rigid floors, causing every frame to laterally deflect by the same amount at each floor level. Rigid floor behaviour is modelled by rigid links, connecting the frames. The links do not transfer bending between the frames but do transmit equal horizontal forces to ensure equal displacements among the linked frames. The links are assigned infinite axial rigidity and zero flexural rigidity. Figure 24 illustrates a linked model for the structure shown in Figure 21.

Figure 24 Lumped frame model where external and internal frames are linked



This model includes two frames linked with axially rigid links, where one frame model represents two exterior frames lumped together with member properties twice of those of individual external frames, and the other frame model represents the remaining interior frames lumped together. Total storey mass should be assigned to each floor level, since the lumped frames are responsible from resisting the inertia forces associated with the entire structure.

A simplified approach to modelling frame structures is to model a single, typical fame with mass tributary to only to this frame. This can be done if the frames have similar strength and stiffness characteristics and the structural mass tributary to each frame is approximately the same. In the current project, this approach was adopted, and a single interior frame was model as shown in Figure 25. Each joint and each member between any two joints were numbered for data entry into the computer software used for dynamic analysis. The properties of each member, established by design performed on the basis of the seismic provisions of CSA Standard A23.3 (1994), can then be assigned to each member. The structural properties of an interior frame are shown in Figure 26.

Figure 25 Structural model of an interior frame used in dynamic analysis

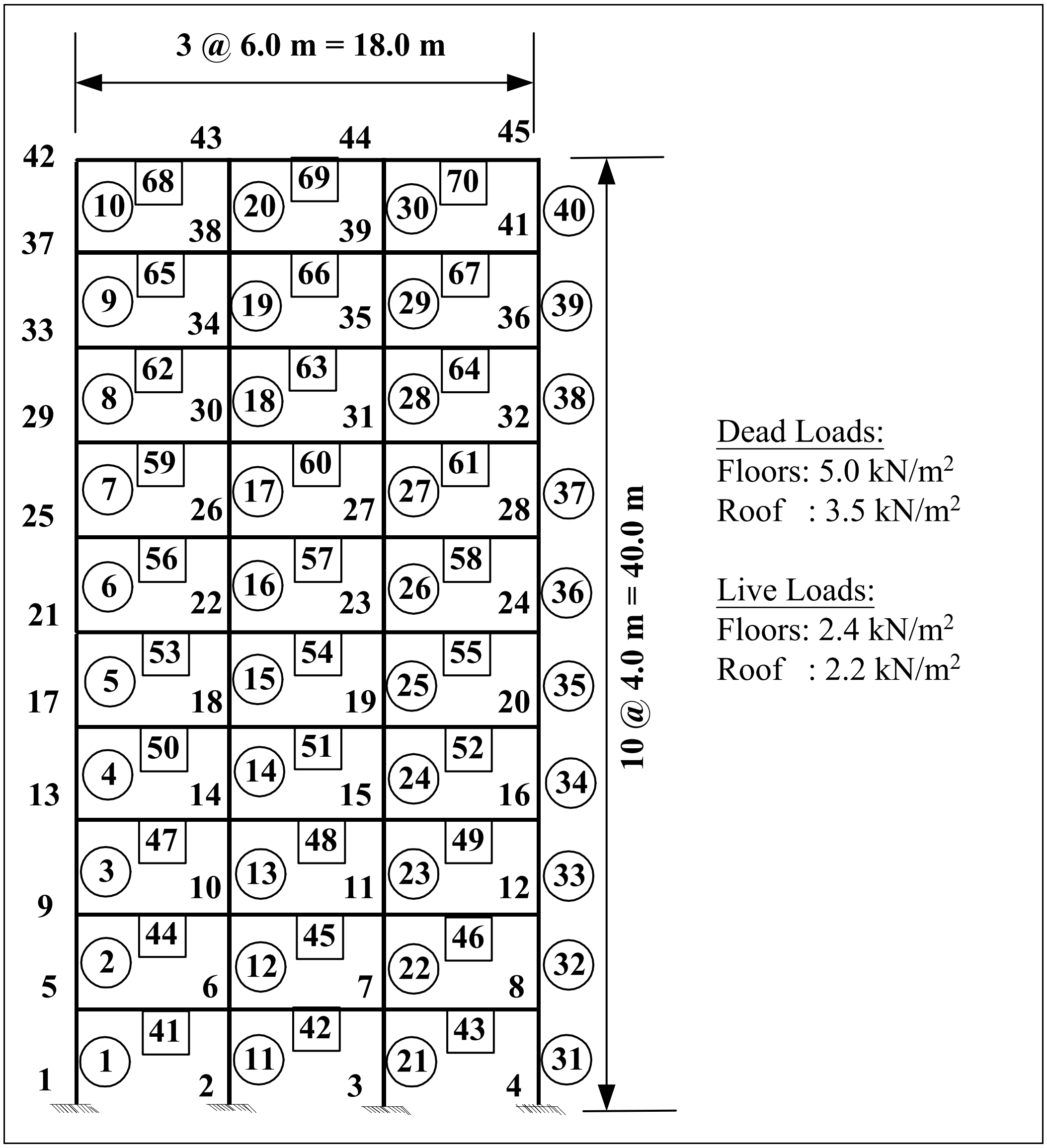
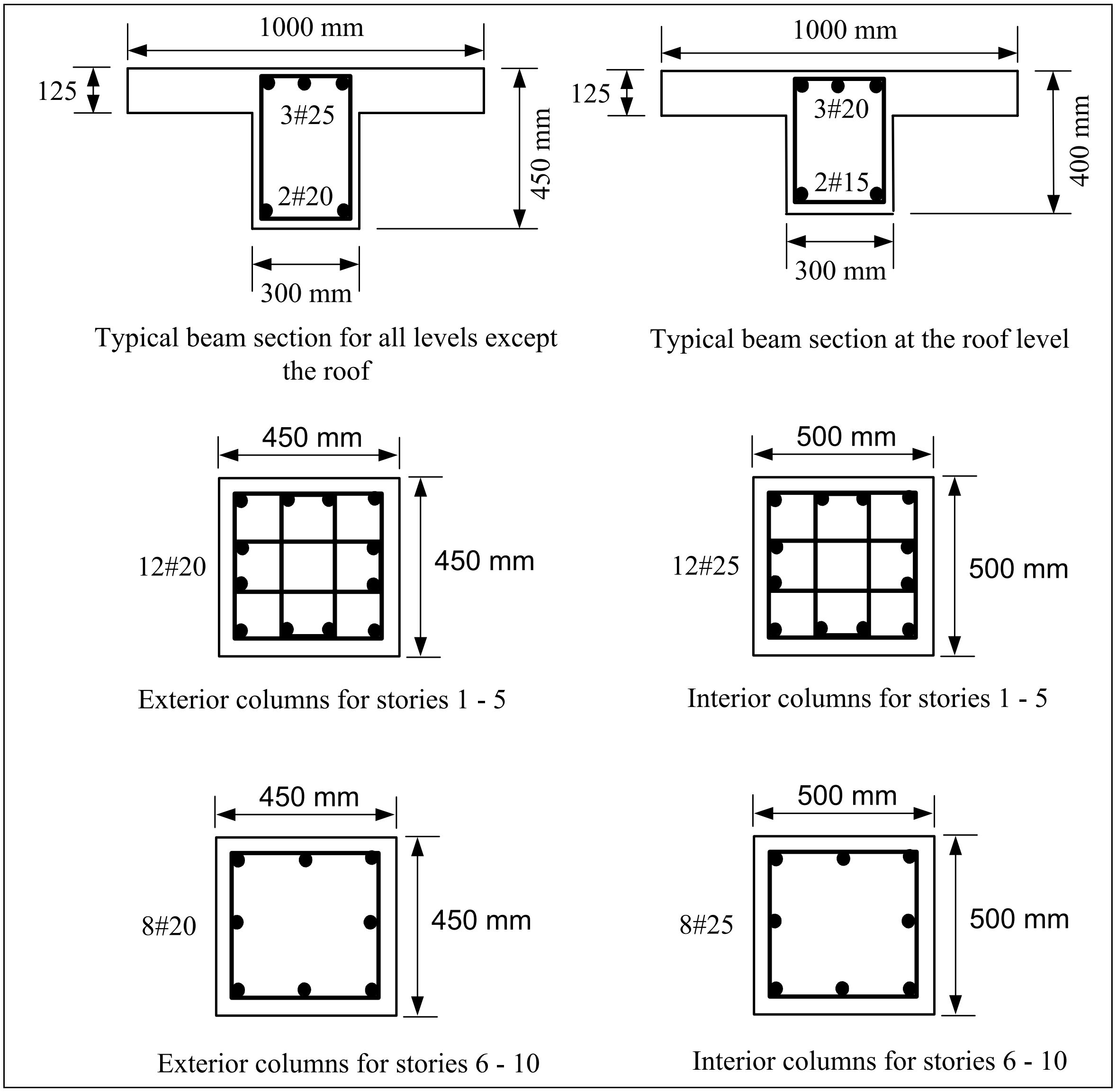
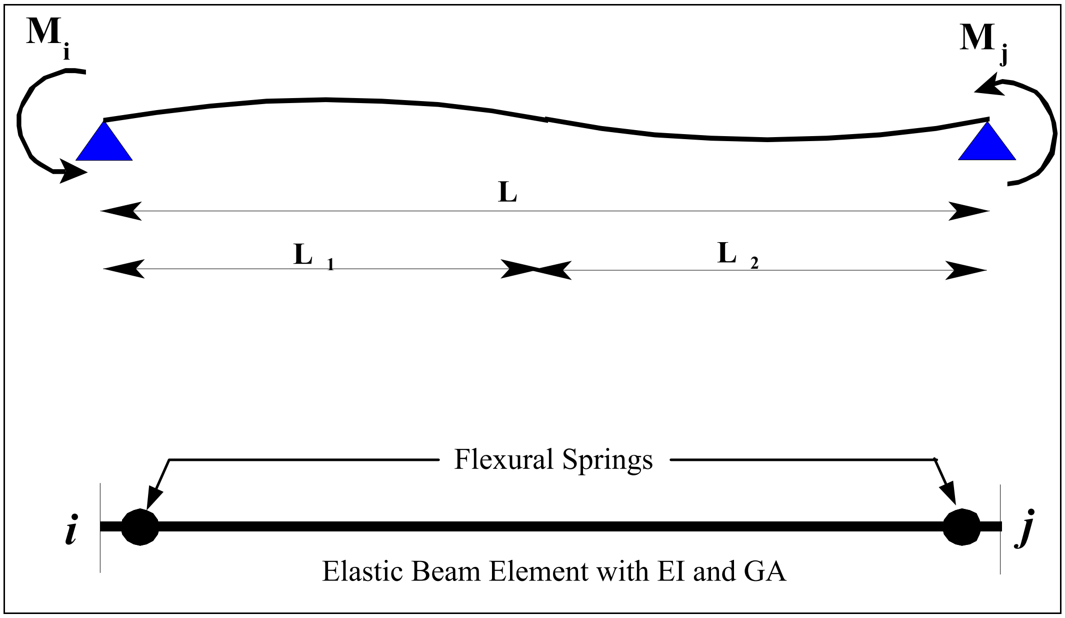


Figure 26 Sectional properties for beams (negative moment regions near the columns) and columns ($f'_c = 30$ MPa; $f_y = 400$ MPa)



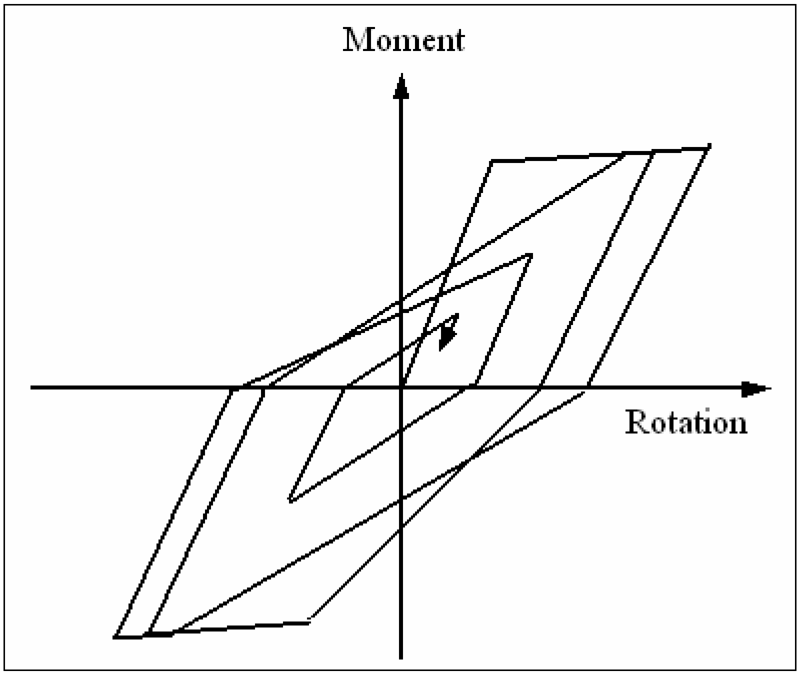
Once the structural modelling is completed, the second stage in modelling consists of the modelling of individual elements in the structure to simulate inelastic action. This is often done by introducing springs at member ends, where plastic hinges are likely to occur. The springs allow the members to yield and develop inelastic deformations, whereas the line elements simulates elastic behaviour with appropriate flexural, axial and shear rigidities (i.e., EI, AE, and GA, respectively). Depending on the type of inelasticity considered, there may be one, two, or three springs at each end, simulating different types of inelastic deformations. In a reinforced concrete frame building structure, the most predominant type of inelastic deformation is caused by flexure. Hence, the flexural spring was considered in modelling individual elements for the inelastic analysis conducted in the current research project. Figure 27 illustrates an element model with a flexural spring at member ends.

Figure 27 Element modelling for inelastic flexure



The third stage in analytical modelling deals with the simulation of hysteretic force-deformation relationships. This information is necessary to conduct dynamic inelastic response history analysis. During seismic response, the elements of a structure are subjected to inelastic deformation reversals. The previous history of loading becomes important, and the elements respond differently depending on the history of loading. For example, the behaviour of an element yielding the first time is significantly different than that of an element that has already experienced a large number of inelastic deformation cycles. This is especially true in concrete structures where strength and stiffness of structures degrade with the number, as well as the magnitude of inelastic deformation reversals. Inelastic behaviour of individual members under reversed cyclic loading is modelled by hysteretic models. These models define the force-deformation path that the element must follow for a given history of loading, and are assigned to the inelastic springs that are placed at the ends of the line elements. The hysteretic model, suggested by Takeda et al. (1970), also used in the current investigation, is shown in Figure 28.

Figure 28 Hysteretic moment-rotation relationship



The hysteretic model consists of the primary (backbone) curve and a set of rules defining various branches of unloading and reloading during seismic response. The rules are defined within the model, while the primary curve is to be computed for each member. This is done by conducting moment-curvature analysis based on the sectional characteristics specified in Figure 26. Moment-curvature analysis provides the flexural rigidity EI used in defining the elastic rigidity of the line element and the yield moment and corresponding yield deformation, which are used in defining the properties of inelastic hinges.

8.0 Dynamic Inelastic Response History Analysis

Dynamic inelastic analyses were conducted using DRAIN-2DX computer software. Time histories of response quantities are computed by numerical integration with a specified time step of 0.005 sec. The software has an option of conducting step-by-step integration either by using a constant or a variable acceleration within each time step. It computes an error measure in each step. If this measure exceeds the upper tolerance in any step, the time step is reduced and the step is repeated. If the measure is less than the lower tolerance for some specified number of steps, the time step is increased in the following step. The software also has the option of considering P-Δ effects. This is done by adding a geometric stiffness matrix to the stiffness matrix for each element, and accounting for P-Δ effects in the resisting force computation.

The structures were modelled as two-dimensional assemblages of nonlinear elements, as described earlier in Section 7.0. The model structures were specified in terms of nodal coordinates. Three degrees of freedom were used at each node, consisting of X and Y translations and R rotation about the Z-axis. Takeda's hysteretic model was used to describe element stiffnesses during the many cycles of loading, unloading and reloading, as described in Section 7.0.

The structure mass was lumped at the nodes, resulting in a diagonal mass matrix. Damping was specified as a mass and stiffness dependent viscous damping. The following expression describes the damping matrix C:

[14] $$C = \sum \alpha M + \sum \beta K_{\beta}$$

The mass dependent damping, αM, introduced translational and/or rotational dampers at each node. Stiffness dependant damping introduced dampers in parallel with elements. The damping matrix, K_{β}, for any element was set equal to the initial element stiffness. The damping ratio specified was 5% of critical damping. The ground accelerations were specified as acceleration-time pairs.

Computer software DRAIN-2DX was used to carry out response history analysis of each building, using 30 different earthquake records. The analysis provided acceleration time response histories at each floor. The data is stored for further evaluation and the computation of response spectra. Response spectra at each floor were computed for different earthquake records using computer software "Spectra." The results were further evaluated to find the mean response spectrum at each floor level by using spreadsheet software Microsoft® Excel. The results obtained in this manner are presented in the following section.

9.0 Design Response Spectra

Design response spectrum for functional and operational components of buildings can be established by computing a smoothened critical acceleration spectrum from floor response spectra. The design response spectrum provides input motion for seismic response of building contents (functional and operational components).

The two 10-storey concrete moment resisting buildings, designed in accordance with NBCC 1995 for Ottawa and Vancouver, were analyzed using computer program DRAIN-2DX to compute floor response accelerations. Details of the selections and designs of these two buildings are given in Section 6. Methodology for the modelling of these structures is presented in Section 7.

Each building was subjected to a total of 30 (i.e. 15 artificial and 15 actual) accelerograms (i.e. input ground motions). Nonlinear analysis was carried out for each ensemble of accelerograms. The analysis results provided acceleration response time histories for each floor. Based on these acceleration time histories, acceleration spectra were computed for each floor and for each analysis. This resulted in a total of 600 spectra. The mean spectral relationships were computed at each floor level. From the mean acceleration spectra for individual floors, design floor response spectra were developed and proposed for design.

9.1 Design Spectra for Ottawa (Eastern Canada)

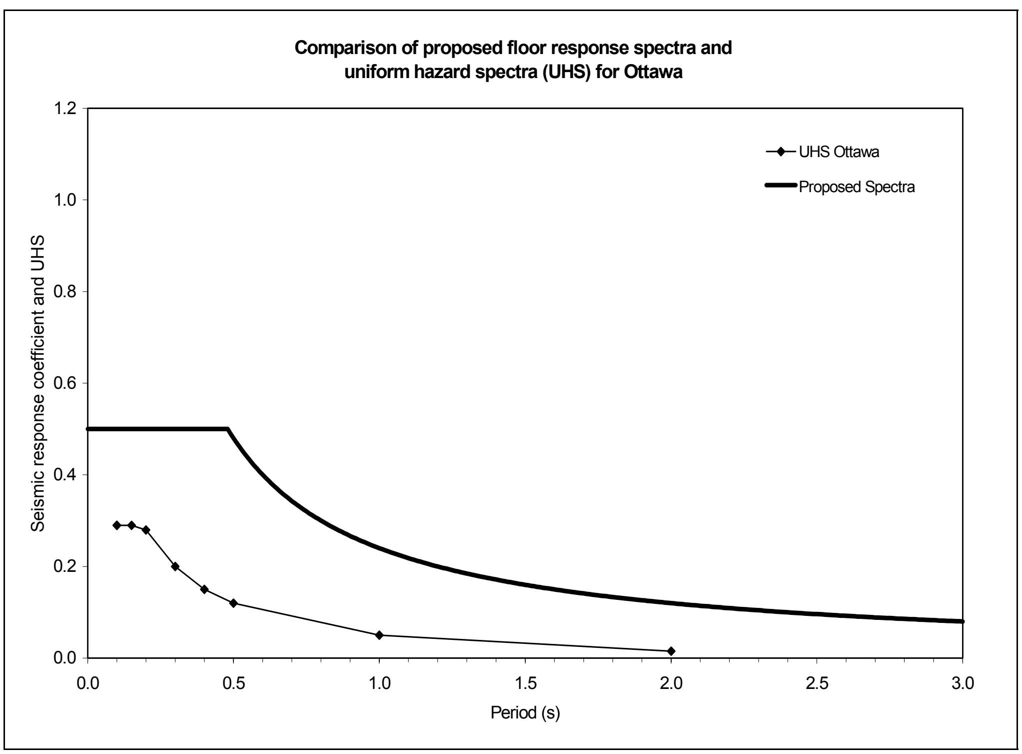
Floor mean spectral response accelerations for the building in Ottawa are given in Appendix B for actual and artificial accelerograms. The spectral relationships shown in these figures represent average values determined from 15 actual and 15 artificial accelerograms, respectively. All of the records are for 10% probability of exceedance in 50 years. The artificial records were derived for Ottawa and the actual records were obtained from various different locations in Eastern Canada. The figures also include spectral relationships for mean plus a standard deviation.

Response amplifications relative to the ground excitation vary from floor to floor and are frequency dependent. Based on these computed floor response spectra, the following equation is proposed to represent the design floor response spectra for medium-rise concrete moment frame buildings located in Ottawa:

Acceleration Spectra = $1.2\ A / T_m^{2/3} \leq 2.5\ A$

Where T_m is the period of vibration of the m^{th} mode in seconds and A is the zonal acceleration ratio for Ottawa, equal to 0.2. Figure 29 shows the proposed floor acceleration spectra. The same figure also includes the uniform hazard spectra developed by the Geological Survey of Canada for 10% probability of exceedance in 50 years for firm ground conditions in Ottawa. The comparison between the two spectral values reflects the amplification obtained for floor spectral values relative to those for ground spectral values.

Figure 29 Proposed design response spectrum for buildings functional and operational components in mid-rise concrete frame buildings in Ottawa



9.2 Design Spectra for Vancouver (Western Canada)

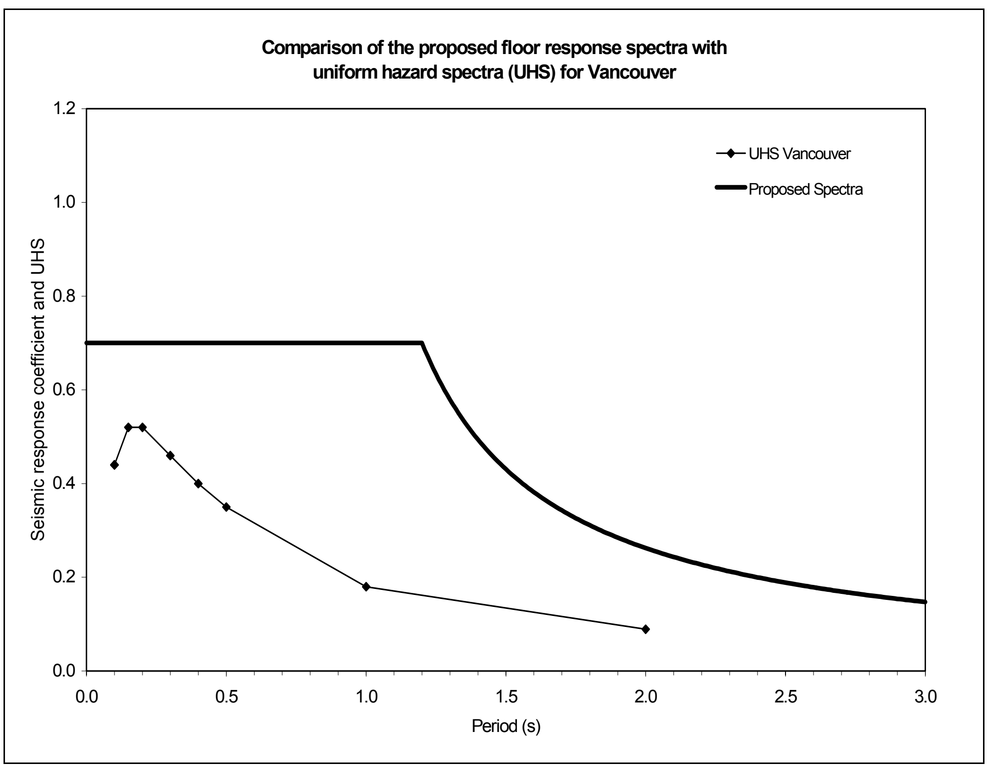
Floor mean spectral response accelerations for the building in Vancouver are given in Appendix B for actual and artificial accelerograms. The spectral relationships shown in these figures represent average values determined from 15 actual and 15 artificial accelerograms, respectively. All the records are for 10% probability of exceedance in 50 years. The artificial records were derived for Vancouver and the actual records were obtained from various different locations in Western Canada. The figures also include spectral relationships for mean plus a standard deviation.

Response amplifications relative to the ground excitation vary from floor to floor, and are frequency dependent. Based on these computed floor response spectra, the following equation is proposed to represent the design floor response spectra for medium-rise concrete moment frame buildings located in Vancouver:

Acceleration Spectra = $1.2\ A / T_m^{2/3} \leq 2.5\ A$

Where, T_m is the period of vibration of the m^{th} mode in seconds and A is the zonal acceleration ratio for Vancouver and is equal to 0.2. Figure 28 shows the proposed floor design acceleration spectra. The same figure also includes the uniform hazard spectra developed by the Geological Survey Canada for 10% probability of exceedance in 50 years for firm ground conditions in Vancouver. The comparison between the two spectral values reflects the amplification obtained for floor spectra relative to the ground spectral values.

Figure 30 Proposed design response spectrum for buildings functional and operational components in mid-rise concrete frame buildings in Vancouver



10.0 Summary and Conclusions

Recent catastrophic earthquakes were both a confirmation and a wake-up call for the seismic protection community. While changes to the building code brought about by experience have resulted in better designed, retrofitted, and constructed building structures, there has been relatively less knowledge and effort on the seismic risk reduction for buildings' operational and functional components (OFCs). Failure of OFCs has an immense impact upon life safety, property protection, emergency response, and business recovery. The notable damage to these OFCs in buildings of all types and ages demonstrated the vulnerability, particularly of older buildings, to these kinds of losses.

During the past few years, significant progress has been made towards the understanding and improvement of seismic behaviour of OFCs. These include the development of provisions for seismic design of restraints, experimental investigations, and the development of guidelines for seismic risk reduction of OFCs. In this report, the performance of OFCs during past earthquakes was reviewed. The current methodologies that are in use in the U.S. and Canada for seismic risk reduction of OFCs of buildings were presented.

Two buildings were selected and designed as representative concrete frame buildings in Western and Eastern Canada. The buildings were designed following the requirements of the National Building Code of Canada (1995) and the CSA Standard A23.3 (1994) for Vancouver and Ottawa; cities chosen to represent sites in Eastern and Western Canada. Until more data are generated for other locations in Canada, the results may be used as representative spectral values for other locations in Eastern and Western Canada with similar seismicity to Ottawa and Vancouver, respectively.

The building layout was selected to consist of four interior and two exterior frames in the short direction, each having three bays. The floor plan was selected to be symmetrical to minimize the effects of torsion. A typical interior frame was modelled as line elements, with inelastic flexural springs at member ends. Elastic properties were assigned to the line elements and inelastic hysteretic relationships were assigned to the hinges. The structural mass tributary to the frame was assigned at each floor level. The joints and elements of the model were numbered for data entry. The geometric and structural data were been prepared for dynamic inelastic response history analysis.

The scope of analysis involved:

- Generation of artificial accelerograms and selection of actual accelerograms;
- Nonlinear structural analysis to compile data on floor response accelerations; and,
- Computation of floor response spectra associated with each ensemble of accelerograms, and the eventual development of design floor response spectra.

The design floor response spectra, developed in the current investigation, can be a very valuable tool for the seismic protection community in Canada to determine the seismic requirements of securing functional and operational components of buildings in Ottawa and Vancouver, as well as buildings with similar characteristics and similar seismicity elsewhere in Canada.

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Appendix A – Tables

Table 1 Types of building components (Cheung, Foo, and McClure, 2000)

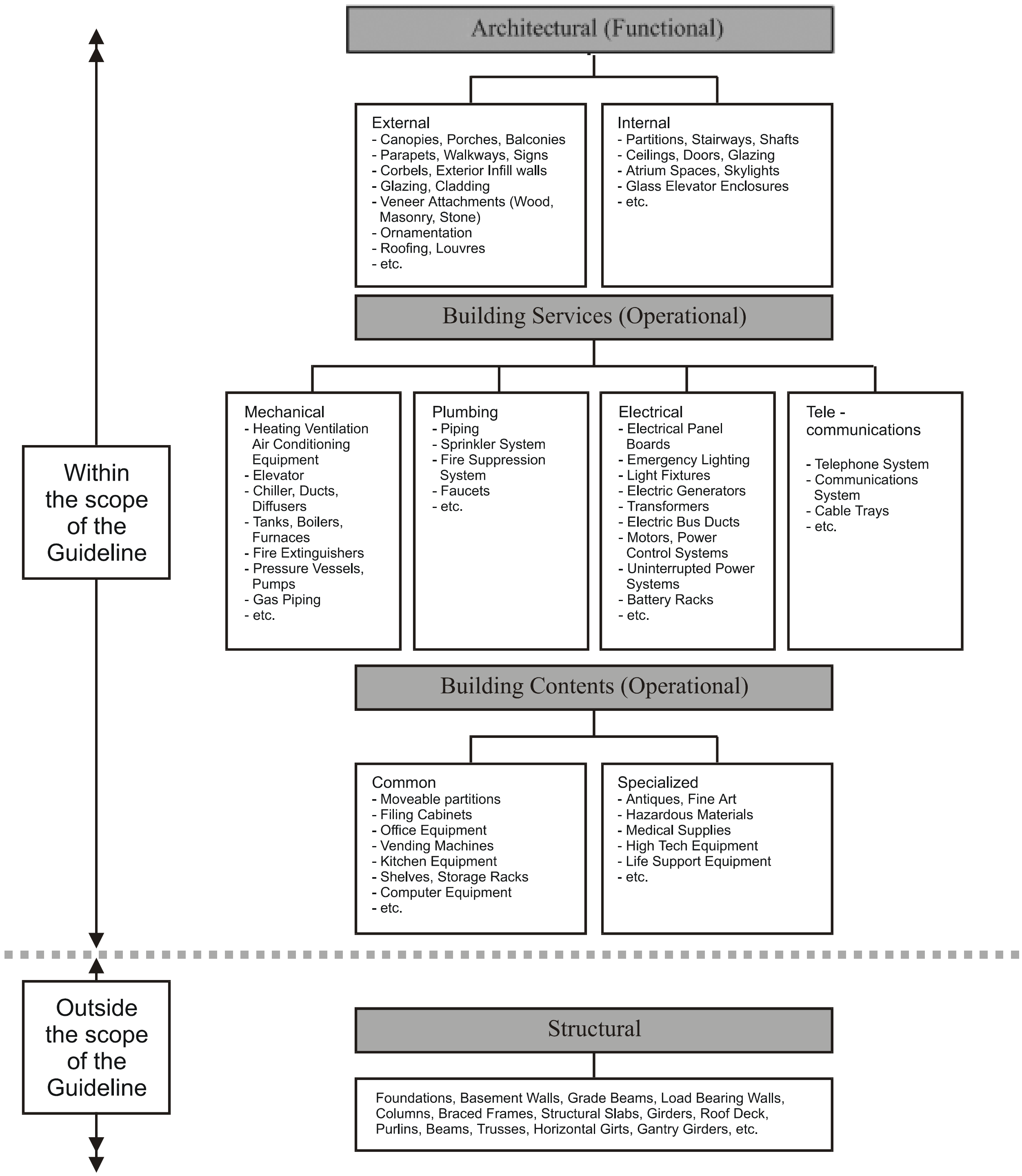


Table 2 Values of S_p for architectural parts or portions of buildings (NBCC, 1995)

Category	Architectural Part or Portion of Building	Direction of Force	Value of S_p
1	All exterior and interior walls except those of categories 2 and 3	Normal to flat Surface	1.5
2	Cantilever parapet and other cantilever walls except retaining walls	Normal to flat Surface	6.5
3	Exterior and interior ornamentations and appendages	Any direction	6.5
4	• Connections/attachments for Categories 1, 2, and 3 • The body of ductile connections/attachments • All fasteners and anchors in the ductile connection, such as bolts, inserts, welds, or dowels • Non-ductile connections/attachments	Any direction	2.5[a] 15.0
5	Floors and roofs acting as diaphragms [see Clause 4.1.9.1(21)]	Any direction	0.7
6	Towers, chimneys, smokestacks and penthouses when connected to or forming part of a building (see Appendix B)	Any direction	4.5
7	Horizontally cantilevered floors, balconies, beams, etc.	Vertical	4.5
8	Suspended ceilings, light fixtures and other attachments to ceilings with independent vertical support	Any direction	2.0
9	Masonry veneer connections	Normal to flat surface	5.0

[a] See Clause 4.1.9.1.(18) in NBCC 1995

Table 3 Values of C_p for mechanical/electrical parts or portions of buildings (NBCC, 1995)

Category	Mechanical/Electrical Part or Portion of Building	Direction of Force	Value of C_p
1	Machinery, fixtures, equipment, ducts, tanks and pipes (including contents) except as noted elsewhere in this table.	Any direction	1.0
2	Machinery, fixtures, equipment, ducts, tanks and pipes (including contents) containing toxic or explosive materials, materials having a flash point below 38°C or fire fighting fluids.	Any direction	1.5
3	Flat bottom tanks (including contents) attached directly to a floor at or below grade within a building.	Any direction	0.7
4	Flat bottom tanks (including contents) attached directly to a floor at or below grade within a building containing toxic or explosive materials, materials having a flash point below 38°C or fire fighting fluids.	Any direction	1.0

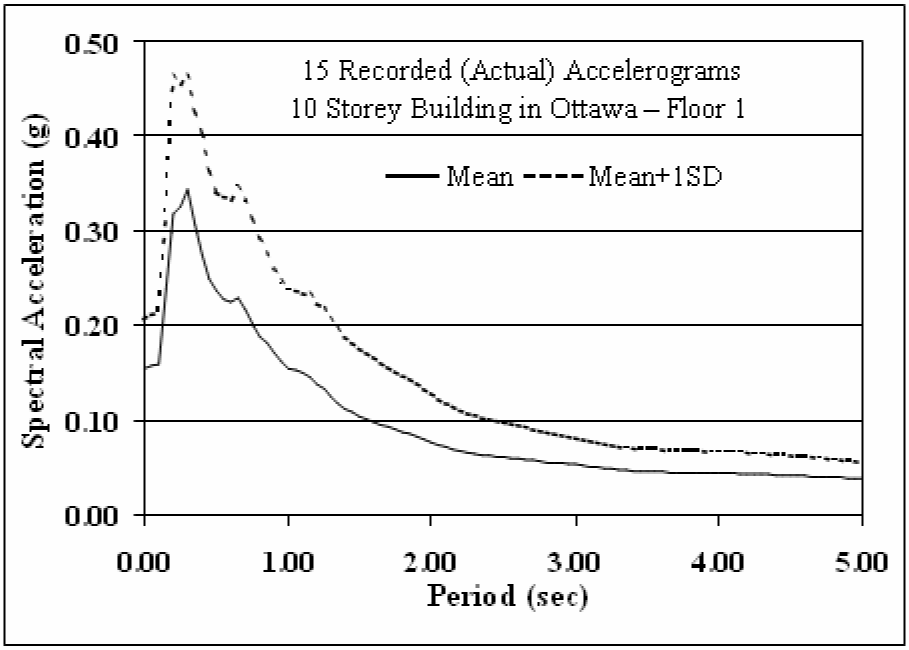
Table 4 Vulnerability Rating for OFCs (Cheung, Foo and McClure, 2000)

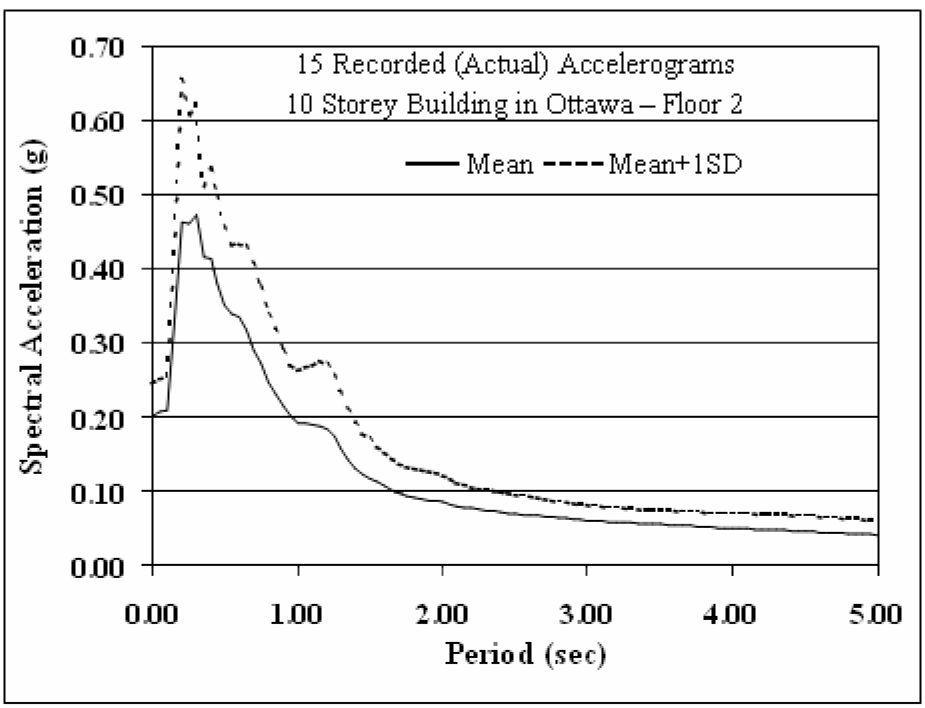
Vulnerability Parameters	Rating Scale (RS)		Weight Factor (WF)	Rating Score (RSxWF)
	Parameter range	RS		
Characteristics of ground motion and soil conditions. (Product of zonal velocity ratio, v, and Foundation factor, F, as defined in NBCC.)	v x F<0.10	1	2	
	0.10 ≤ v x F ≤ 0.20	5		
	v x F > 0.20	10		
Dynamic characteristics of building. (Period of vibration of building, T, in seconds as defined in NBCC.)	T ≥ 0.50s	1	1	
	0.50 > T > 0.25 s	5		
	T ≤ 0.25 s	10		
Lateral force resisting system of building structure. (Force modification factor, R, as defined in NBCC table 4.1.9.1.B.)	R > 3	1	2	
	2 ≤ R ≤ 3	5		
	R < 2	10		
FOC location in building. (Level 0 is ground.)	Level 0	1	1.5	
	Between levels 0 to 2	5		
	Above level 2	10		
Size and weight of FOC. (FOC weight, W_p, expressed as a percentage of weight of supporting floor, wall, ceiling, W.)	$W_p \leq 5\%$ W	1	1	
	5% W < Wp < 10% W	5		
	$W_p \geq 10\%$ W	10		
Connection details of FOC.	Appear robust	1	1.5	
	Appear doubtful	5		
	Obvious weakness	10		
Pounding/Impact effects. – Internal	Gap more than adequate	1	1	
	Gap adequate	5		
	Gap inadequate	10		
Pounding/Impact effects. – External	Gap more than adequate	1	1	
	Gap adequate	5		
	Gap inadequate	10		
	Sum (WF)			
	Sum(RSxWF)			
FINAL RATING SCORE = Sum(RSxWF) / Sum (WF)				

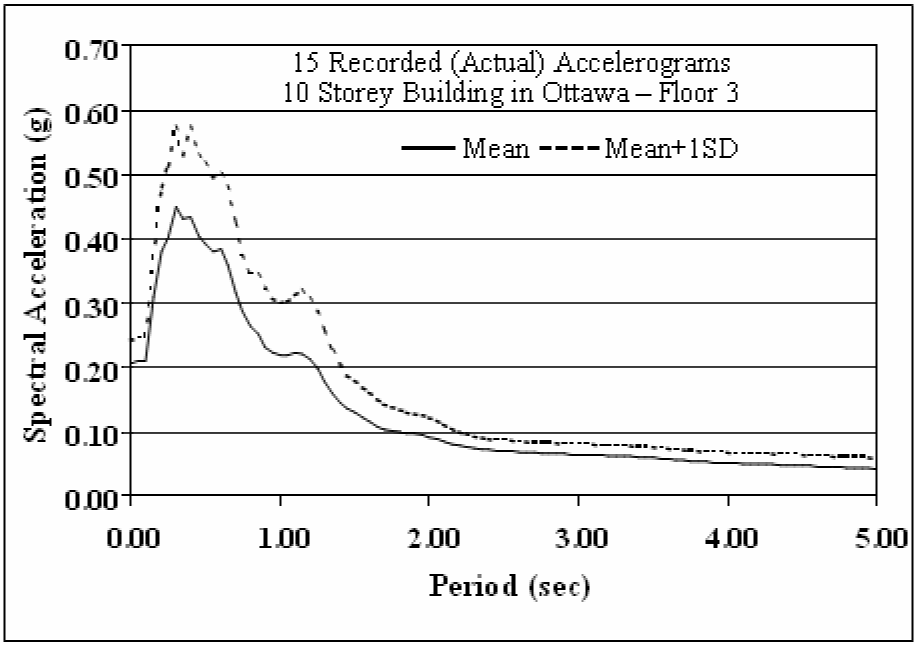
Table 5 Consequences Rating for OFCs (Cheung, Foo, and McClure, 2000)

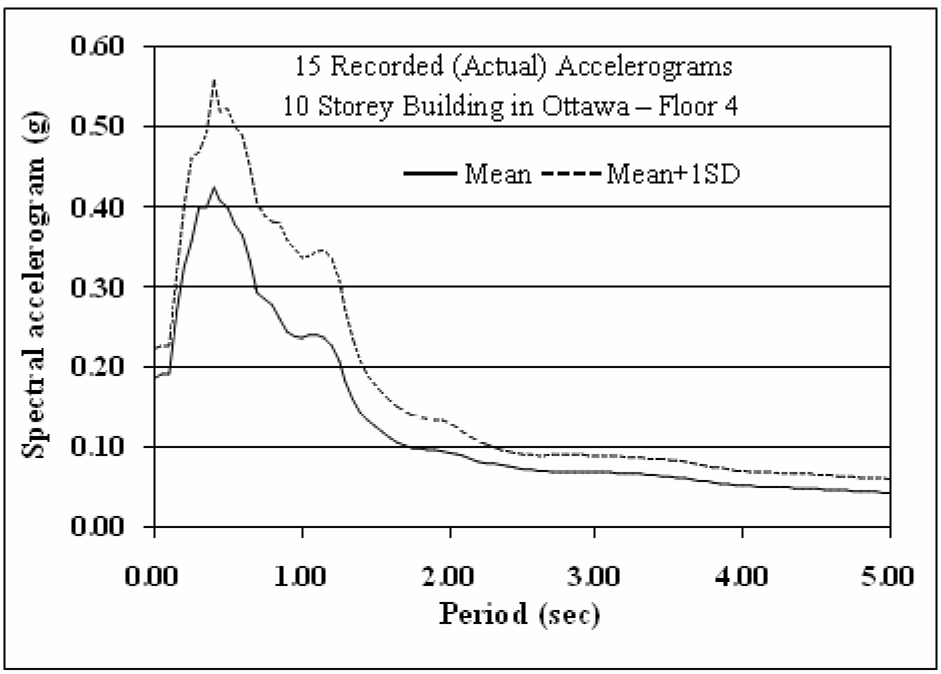
Consequences Parameters	Rating Scale (RS)		Weight Factor (WF)	Rating Score (RSxWF)
	Parameter range	**RS**		
FOC location in building. (Level 0 is ground)	Level 0	1	1.5	
	Between levels 0 to 2	5		
	Above level 2	10		
Weight of FOC. (FOC weight, W_p, expressed as a percentage of weight of supporting floor, wall, ceiling, W)	$W_p \leq 5\%$ W	1	1	
	5%W < W_p < 10% W	5		
	$W_p \geq 10\%$W	10		
Overturning of FOC. (Height of centre of gravity of FOC above floor, CG, relative to the shortest horizontal distance, H, between supports)	CG ≤ 0.5H	1	1	
	0.5H < CG < 0.75H	5		
	CG ≥ 0.75H	10		
Building occupancy. • Impact on life safety from failure of FOC.	No or minimal injury	1	3	
	Moderate injury and hospitalization	5		
	Serious injury or death	10		
Building occupancy. • Impact on building functionality from failure of FOC.	No or minimal functional loss	1	2	
	Some or moderate functional loss	5		
	Major breakdown in function	10		
	Sum (WF)			
	Sum (RSxWF)			
	FINAL RATING SCORE = Sum (RSxWF)			
	Sum (WF)			

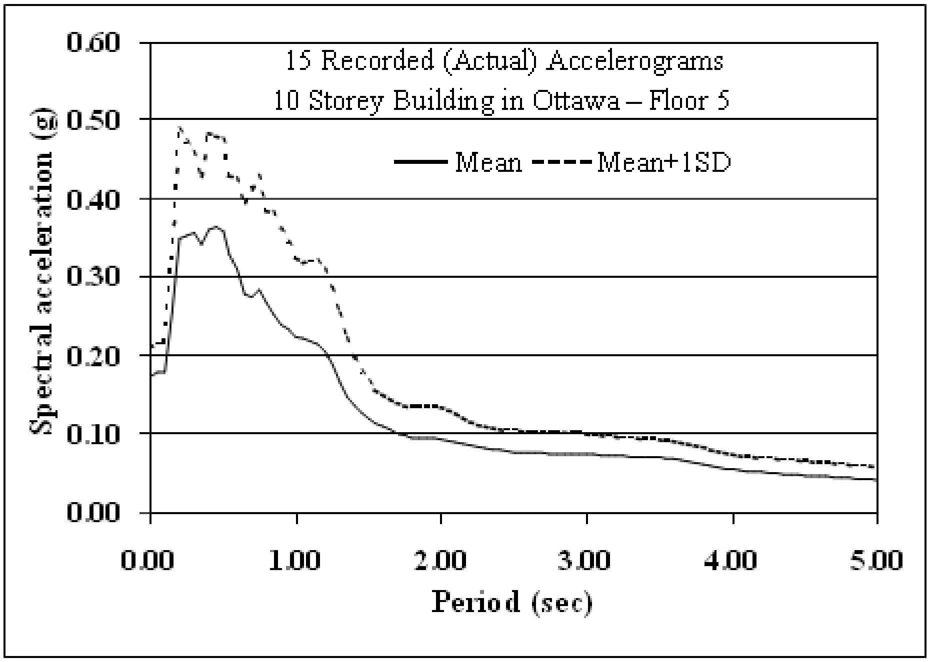
Appendix B – Mean Floor Acceleration Response Spectra for Buildings in Ottawa and Vancouver






15 Recorded (Actual) Accelerograms
10 Storey Building in Ottawa – Floor 3
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00


15 Recorded (Actual) Accelerograms
10 Storey Building in Ottawa – Floor 4
Mean
Mean+1SD
Spectral accelerogram (g)
Period (sec)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00


15 Recorded (Actual) Accelerograms
10 Storey Building in Ottawa – Floor 5
Mean
Mean+1SD
Spectral acceleration (g)
Period (sec)
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0.50
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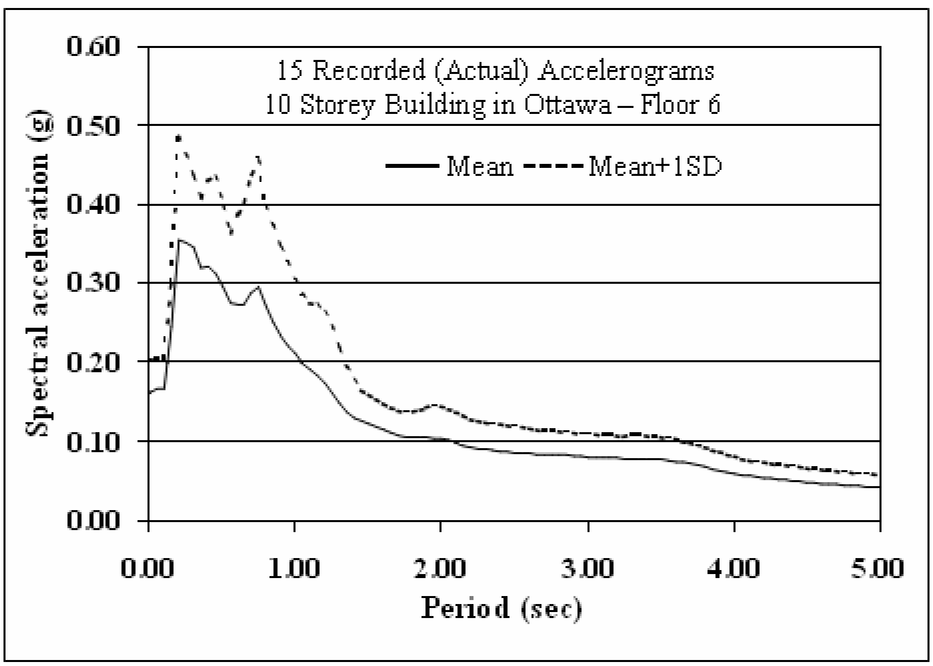
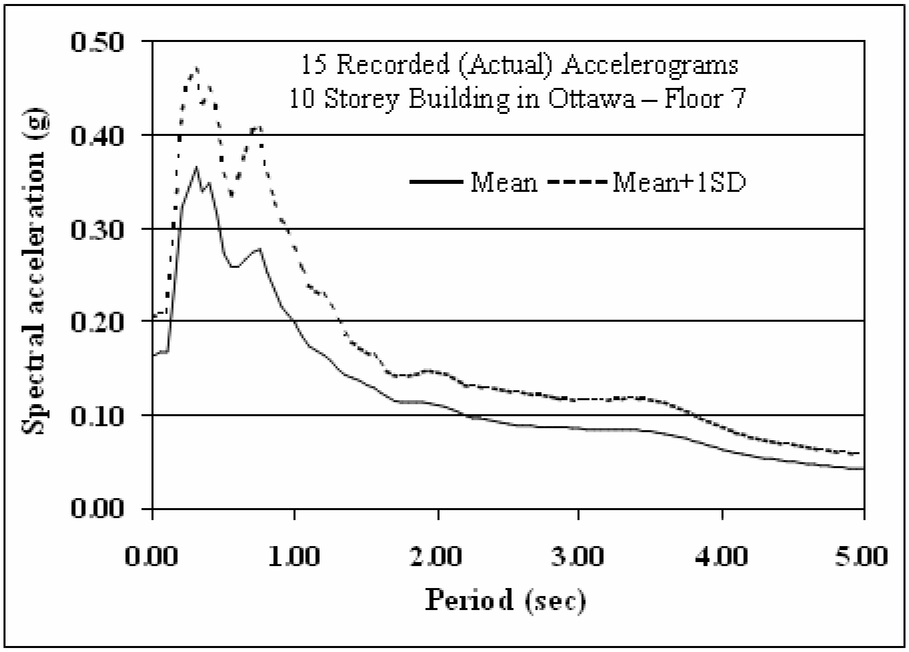
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00


15 Recorded (Actual) Accelerograms
10 Storey Building in Ottawa – Floor 6
Mean
Mean+1SD
Spectral acceleration (g)
Period (sec)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00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Mean
Mean+1SD
Spectral acceleration (g)
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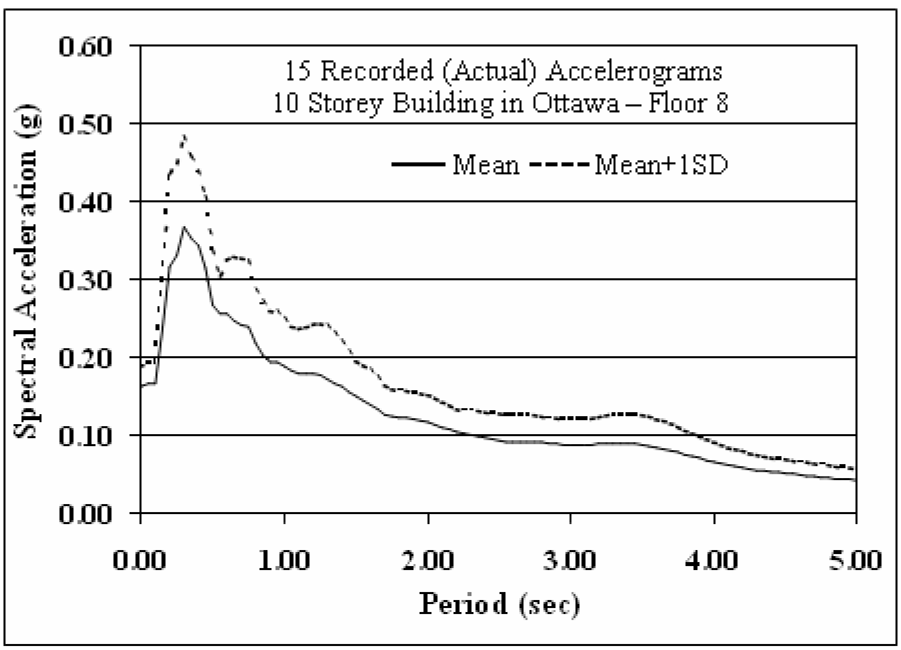
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)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10 Storey Building in Ottawa – Floor 8
Mean
Mean+1SD
Spectral Acceleration (g)
0.60
0.50
0.40
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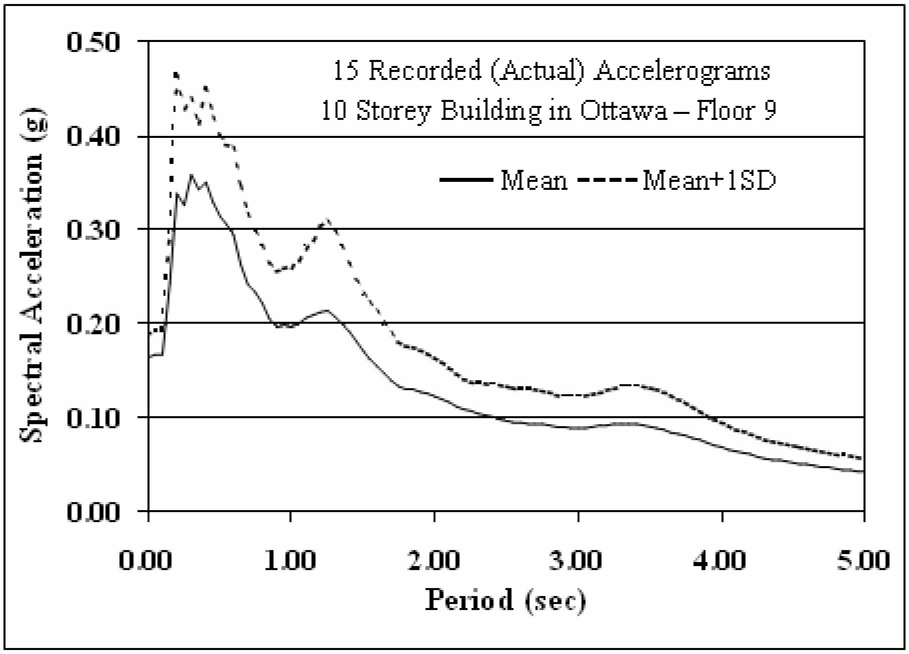
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)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10 Storey Building in Ottawa – Floor 9
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
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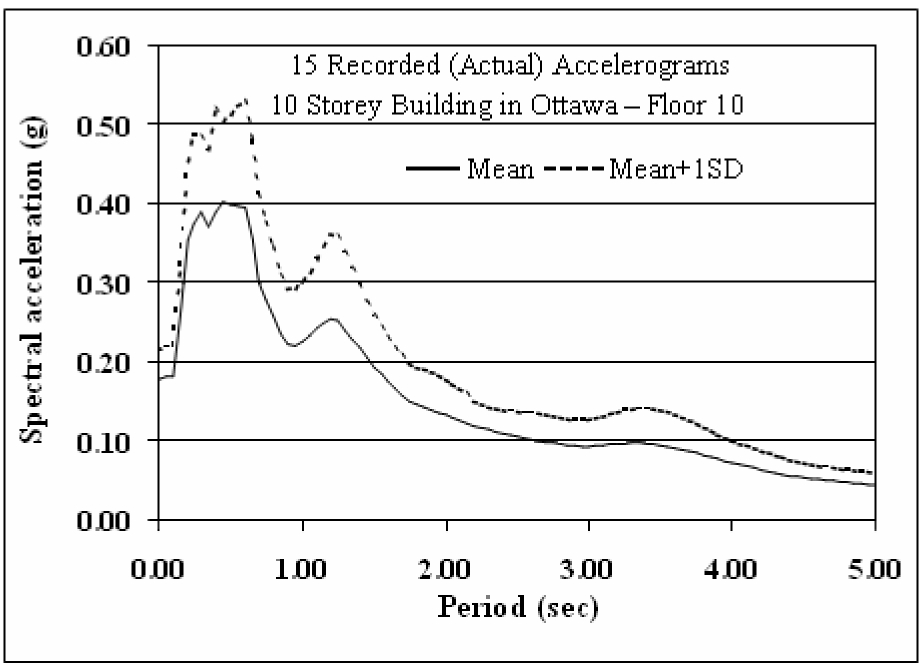
0.20
0.30
0.40
0.50
0.00
1.00
2.00
3.00
4.00
5.00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10 Storey Building in Ottawa – Floor 10
Mean
Mean+1SD
Spectral acceleration (g)
Period (sec)
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0.40
0.50
0.60
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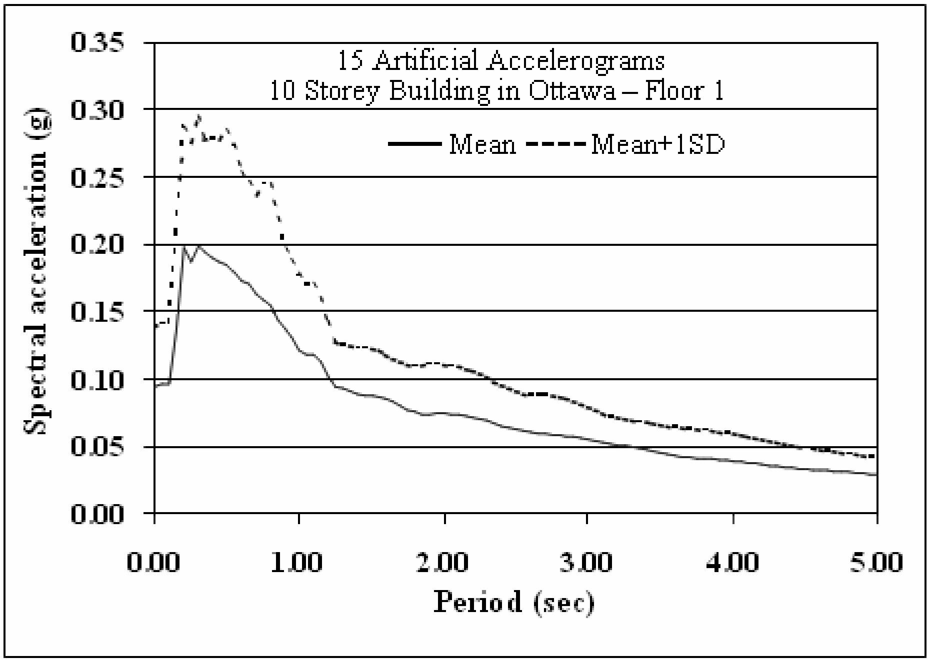
1.00
2.00
3.00
4.00
5.00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Mean
Mean+1SD
Spectral acceleration (g)
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0.25
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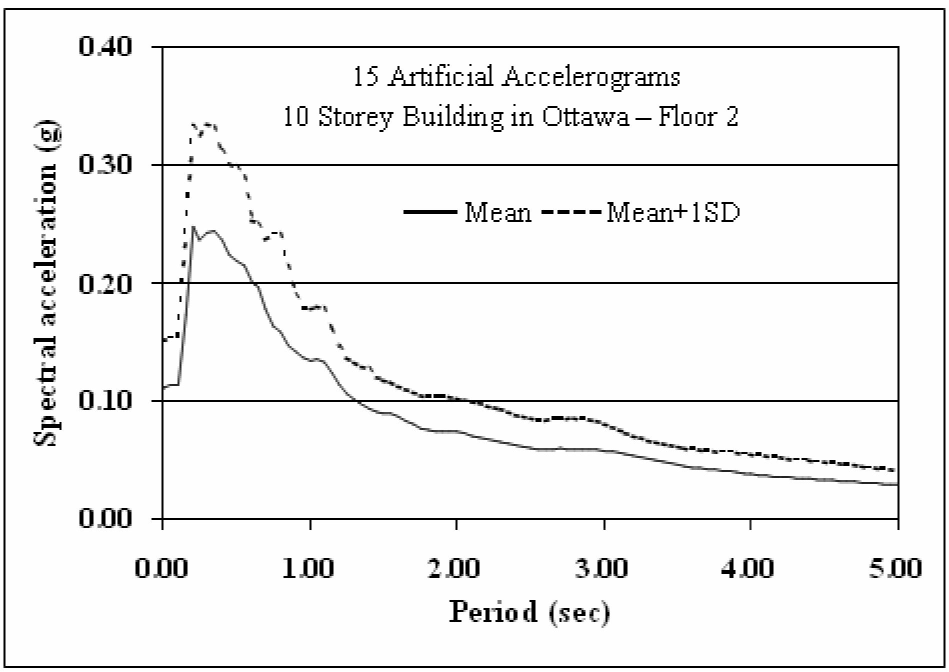
0.05
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)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Mean
Mean+1SD
Spectral acceleration (g)
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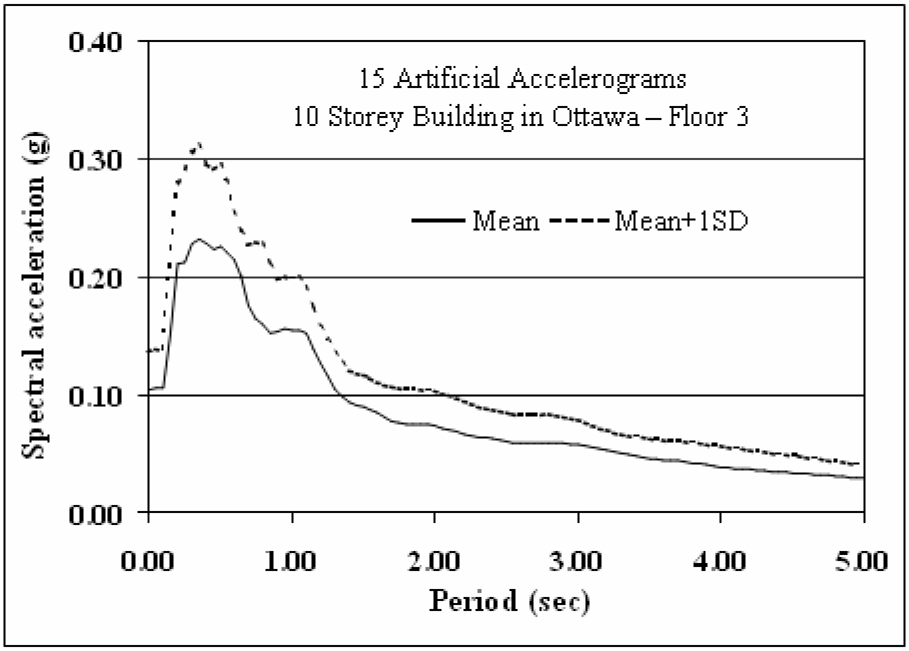
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)


15 Artificial Accelerograms
10 Storey Building in Ottawa – Floor 3
Mean
Mean+1SD
Spectral acceleration (g)
0.40
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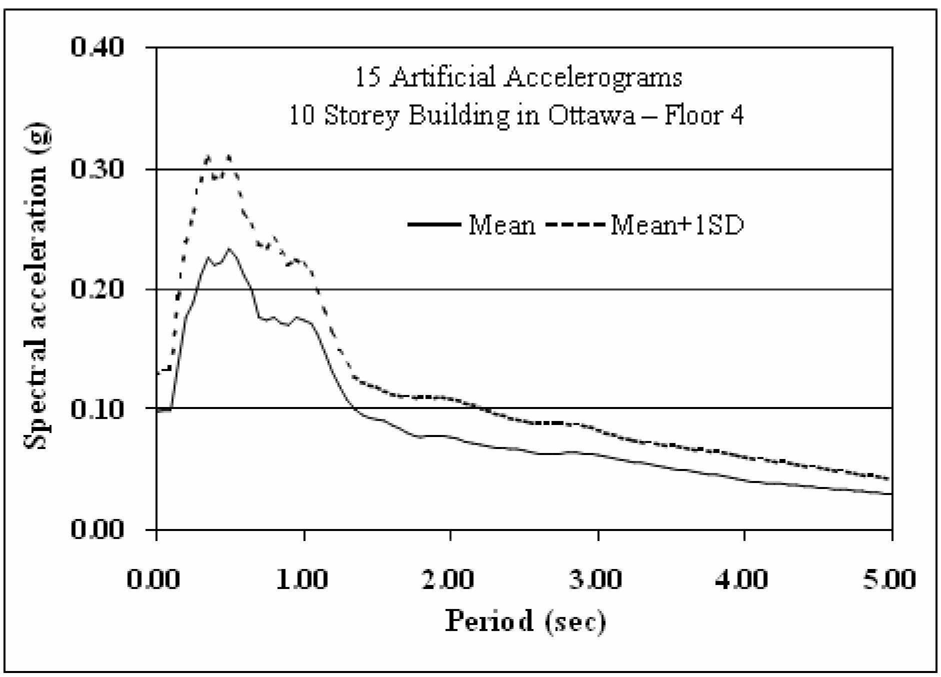
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)


15 Artificial Accelerograms
10 Storey Building in Ottawa – Floor 4
Mean
Mean+1SD
Spectral acceleration (g)
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0.10
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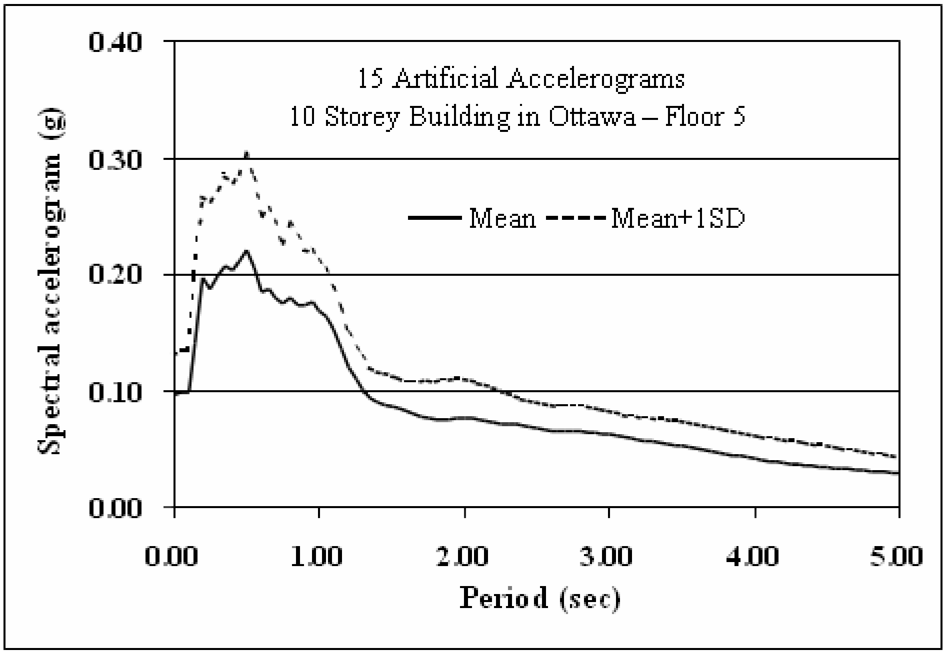
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)



15 Artificial Accelerograms
10 Storey Building in Ottawa – Floor 5
Mean
Mean+1SD
Spectral accelerogram (g)
Period (sec)
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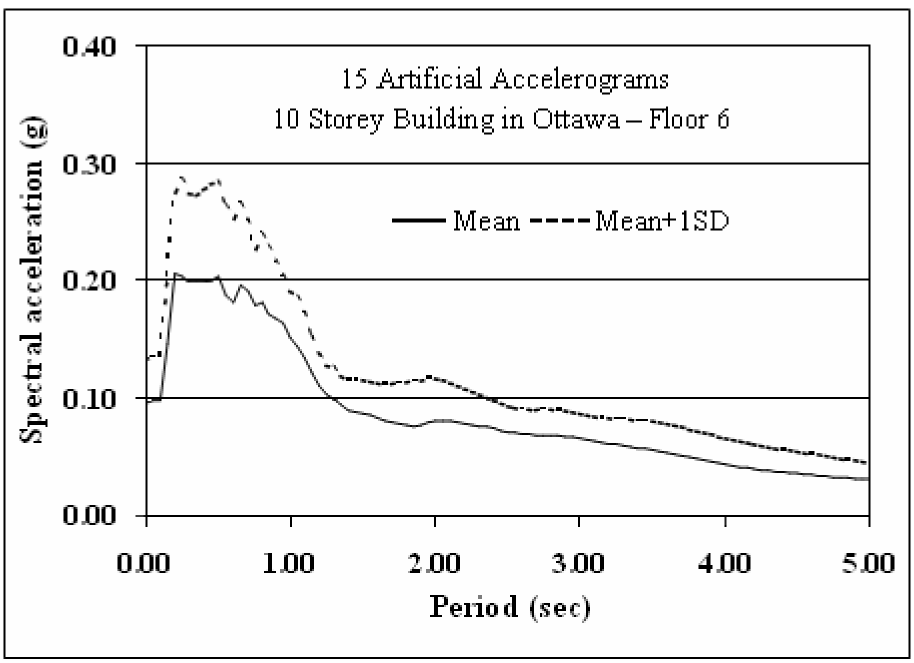
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00



15 Artificial Accelerograms
10 Storey Building in Ottawa – Floor 6
Mean
Mean+1SD
Spectral acceleration (g)
Period (sec)
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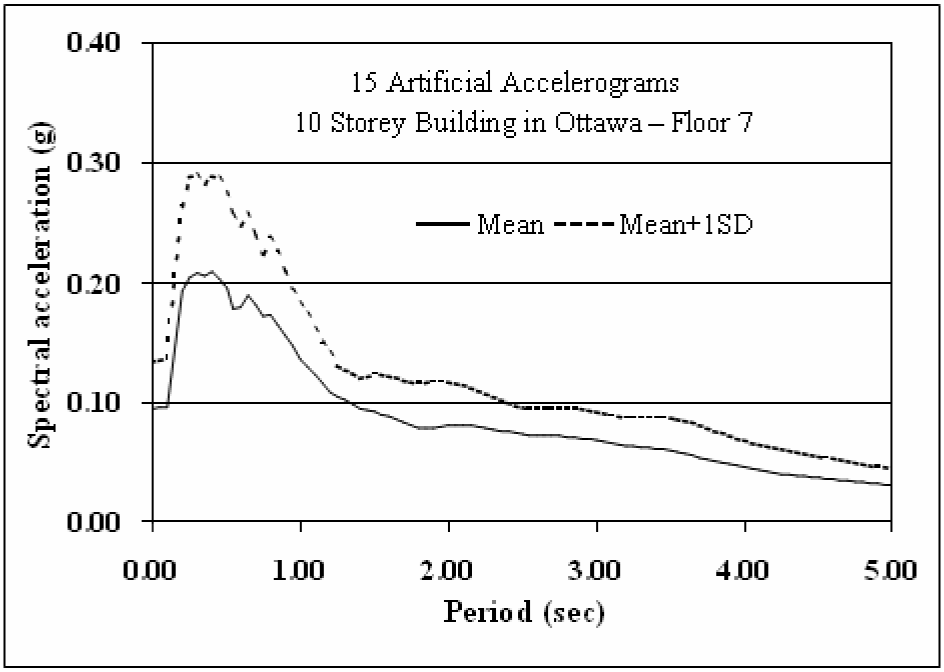
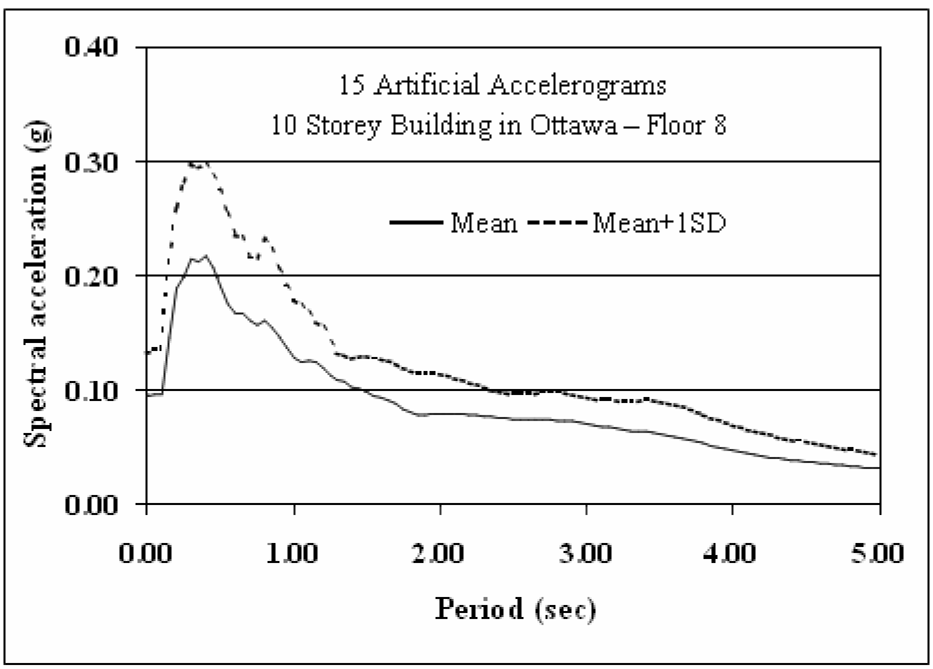
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00


15 Artificial Accelerograms
10 Storey Building in Ottawa – Floor 7
Mean
Mean+1SD
Spectral acceleration (g)
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)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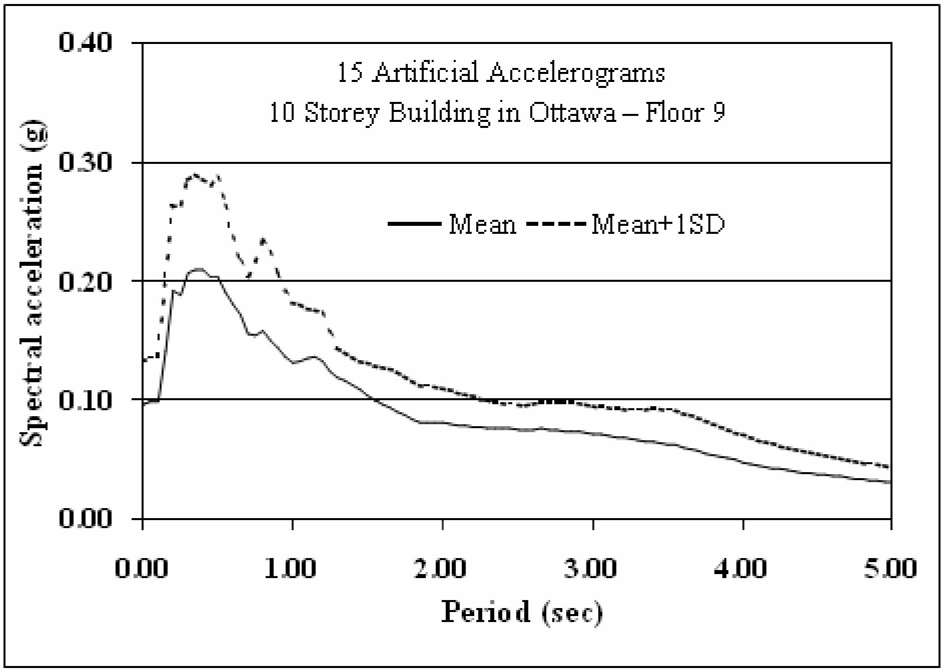
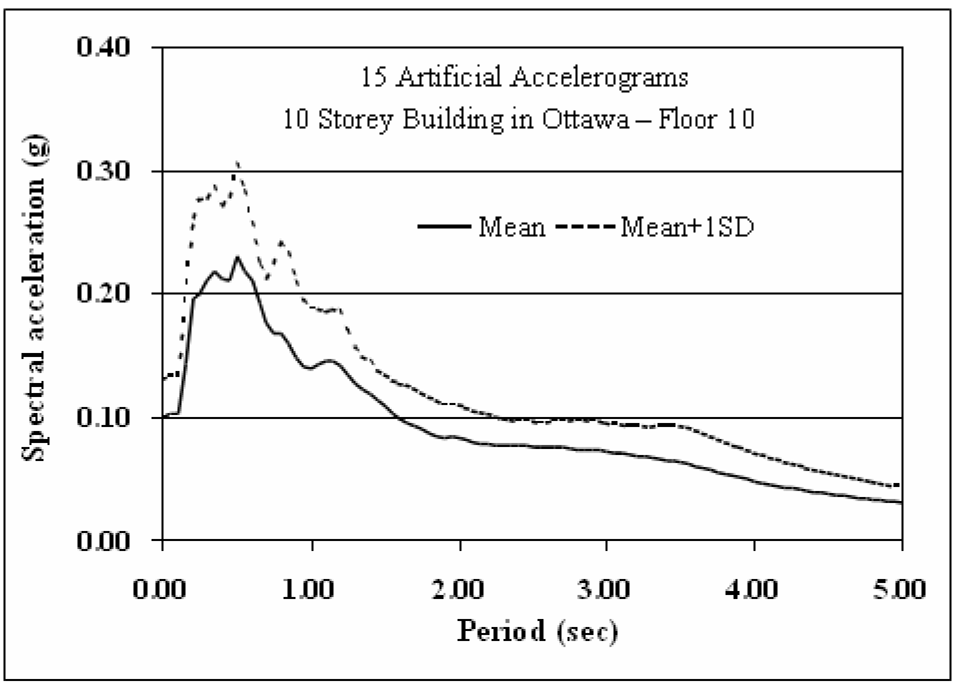
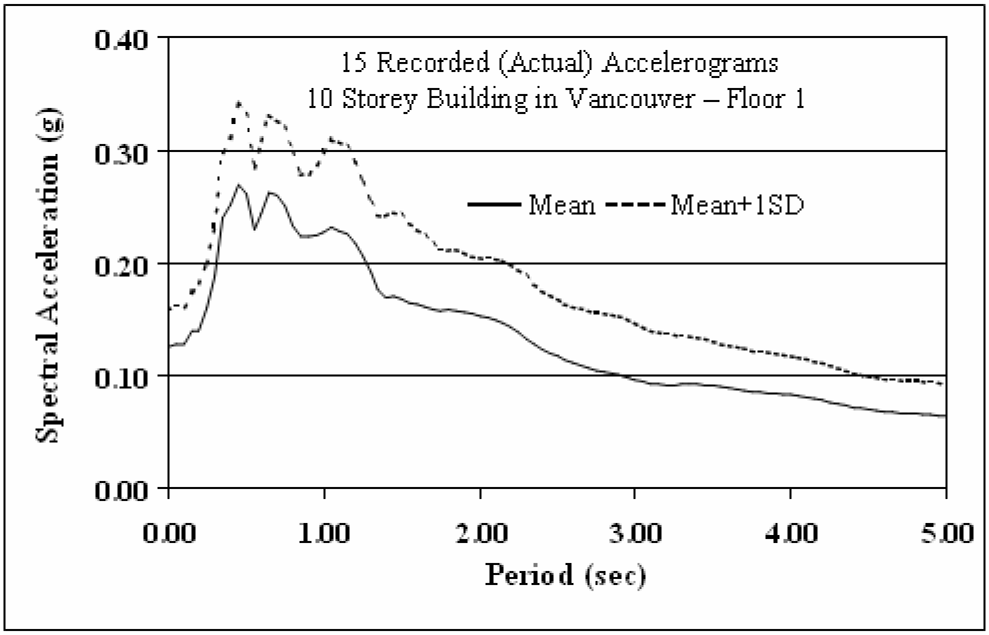
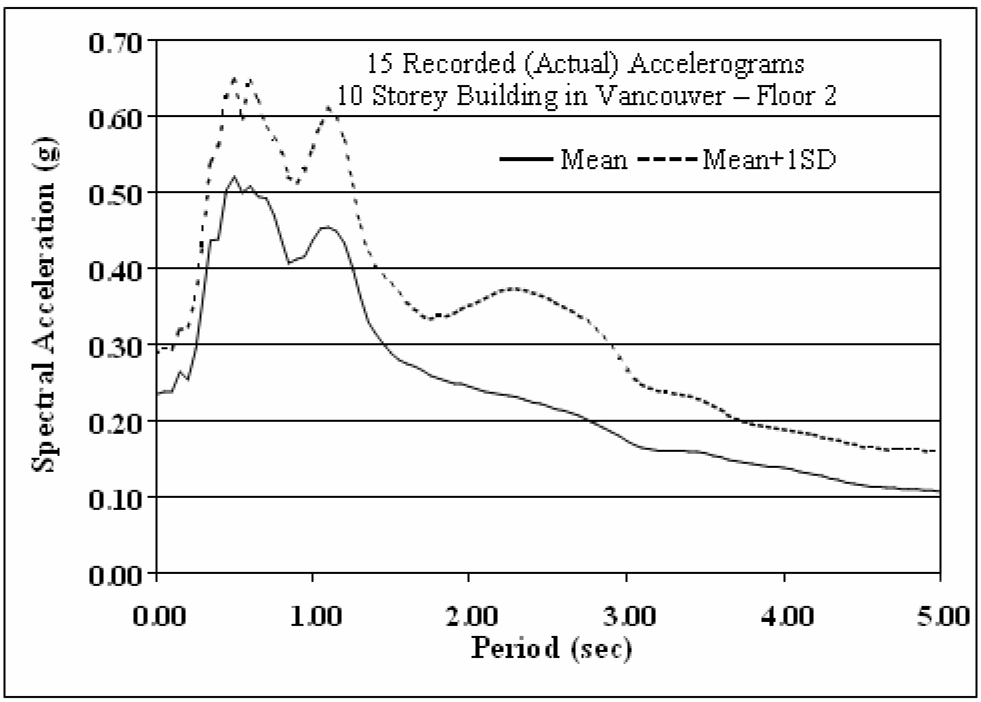
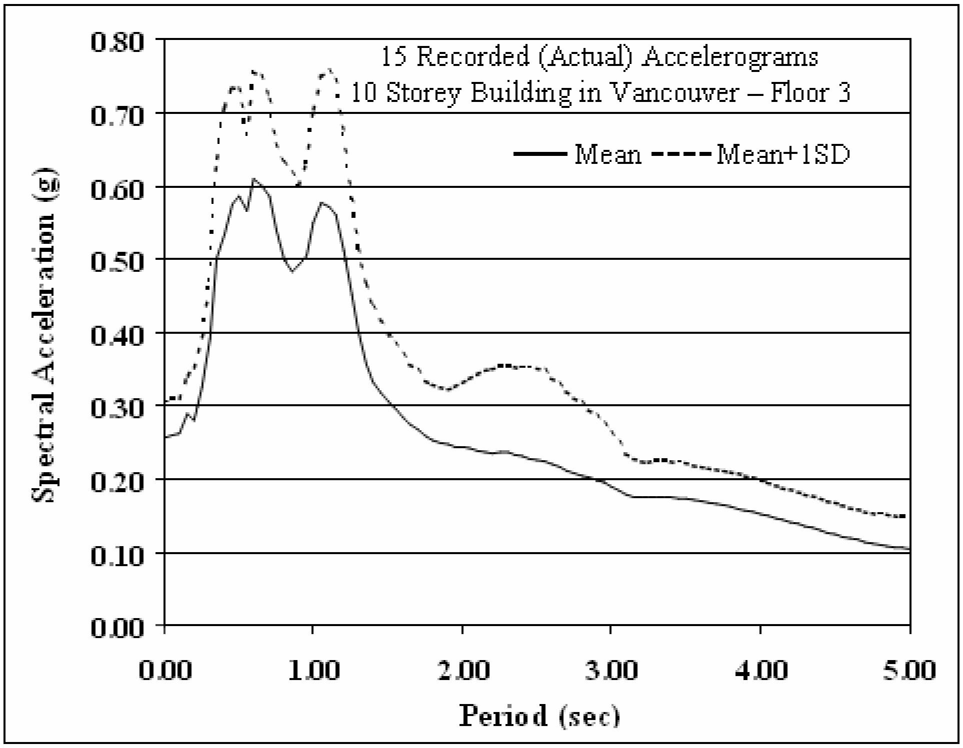
10 Storey Building in Ottawa – Floor 8
Mean
Mean+1SD
Spectral acceleration (g)
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)

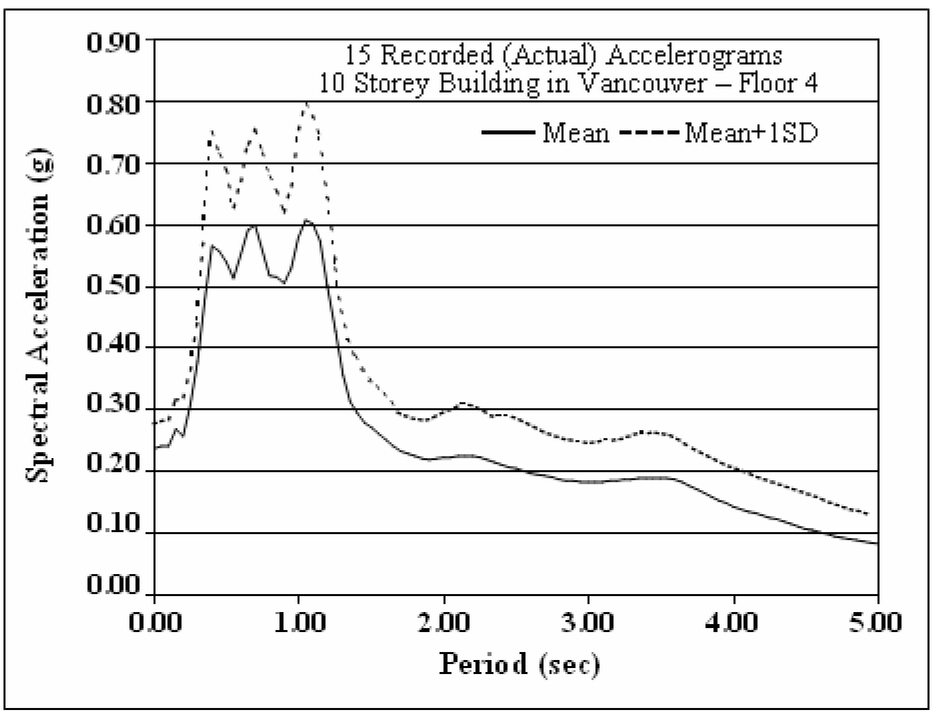

0.40
0.30
0.20
0.10
0.00
Spectral acceleration (g)
15 Artificial Accelerograms
10 Storey Building in Ottawa – Floor 9
Mean
Mean+1SD
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)

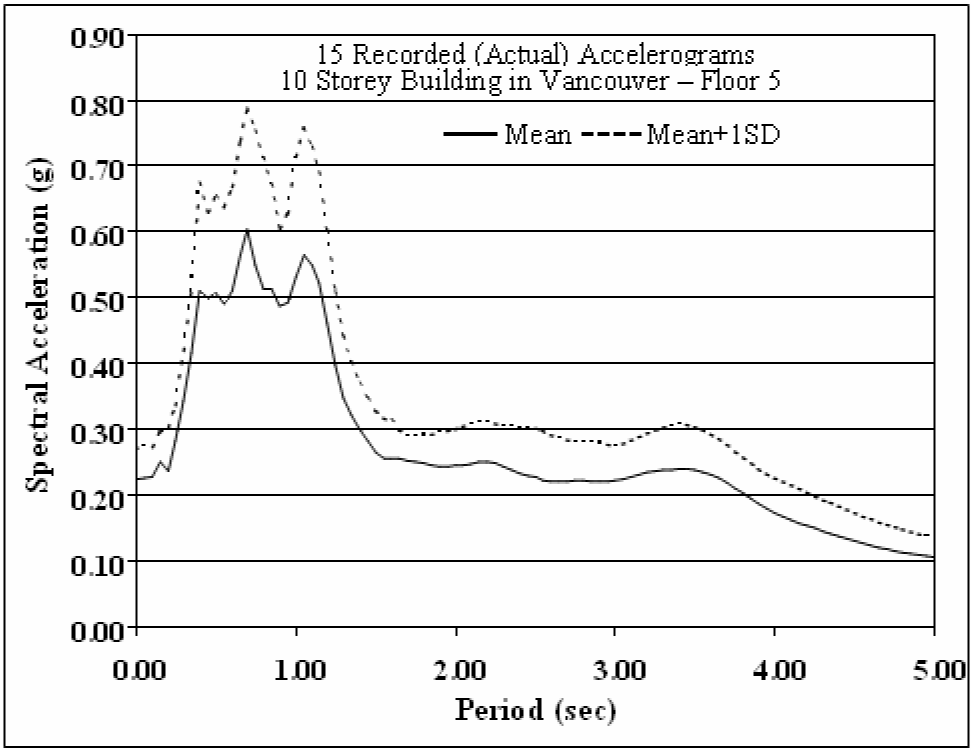

0.40
0.30
0.20
0.10
0.00
Spectral acceleration (g)
15 Artificial Accelerograms
10 Storey Building in Ottawa – Floor 10
Mean
Mean+1SD
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)

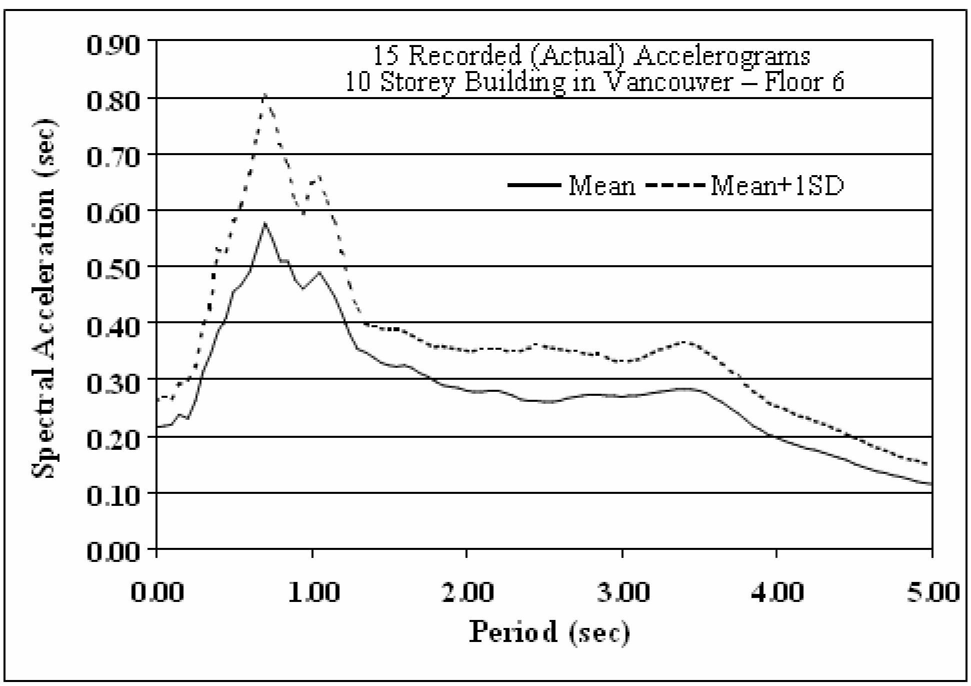

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 1
Mean
Mean+1SD
Spectral Acceleration (g)
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)

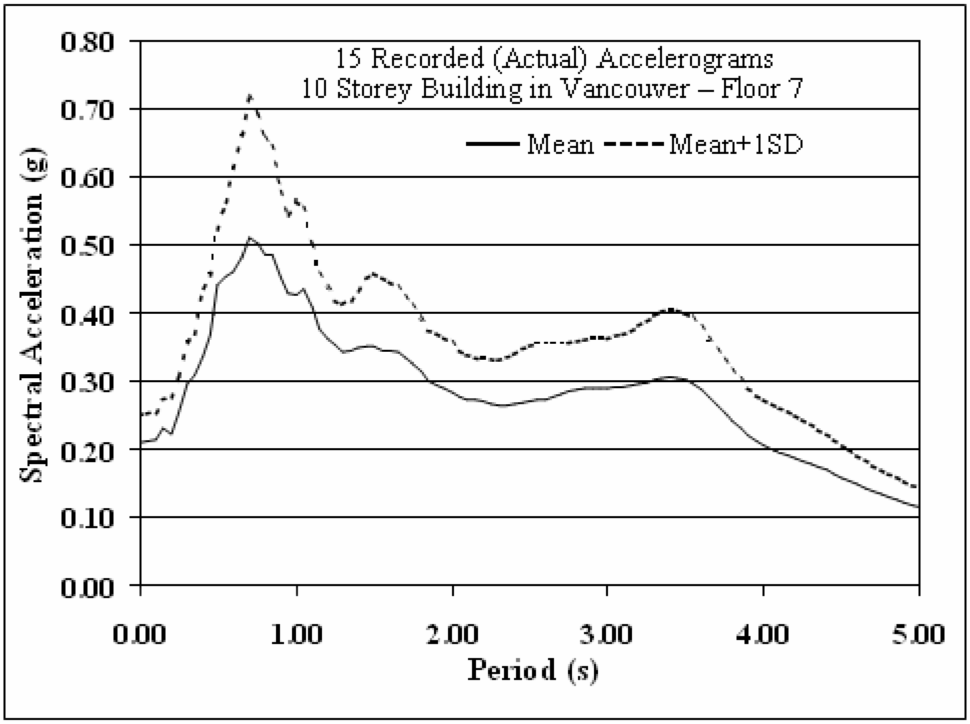

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 2
Mean
Mean+1SD
Spectral Acceleration (g)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)

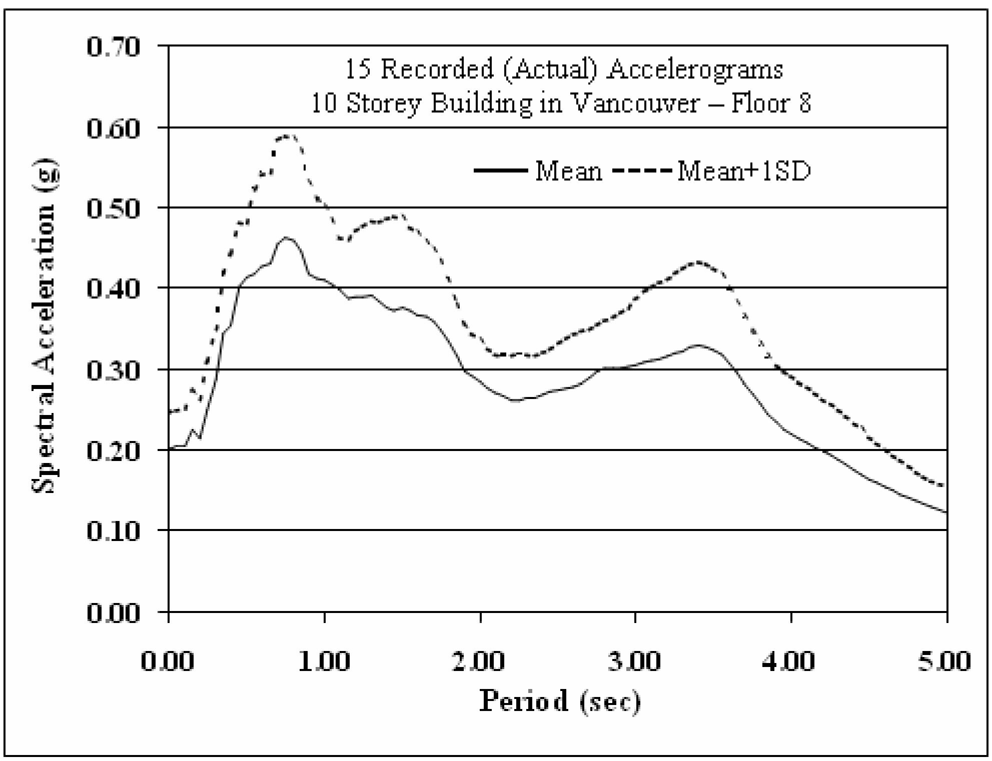

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 3
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
1.00
2.00
3.00
4.00
5.00

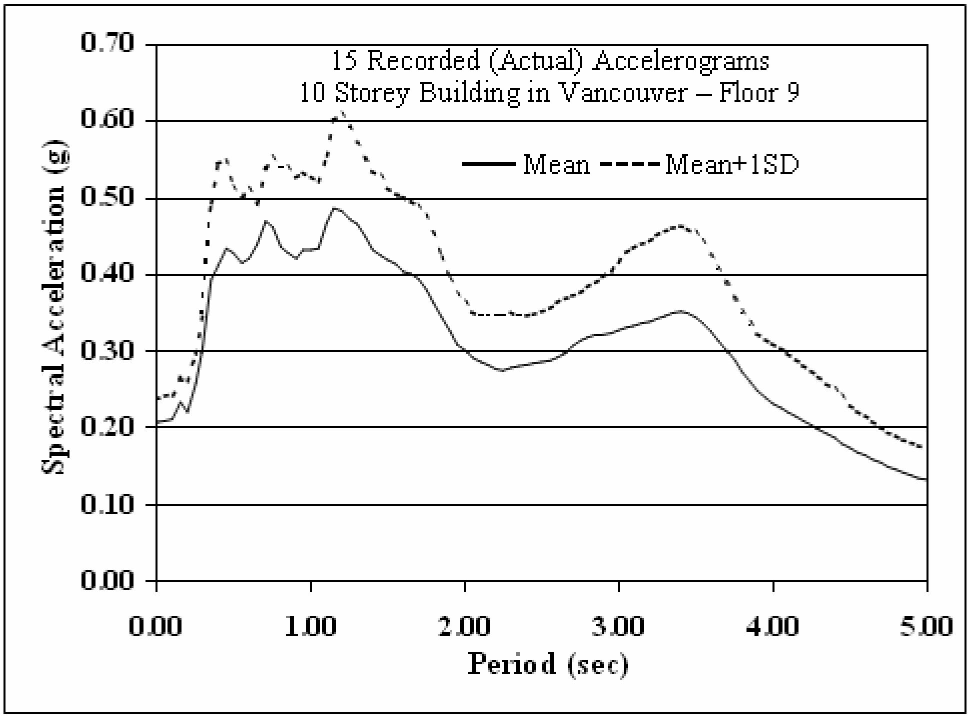

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 4
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
2.00
3.00
4.00
5.00

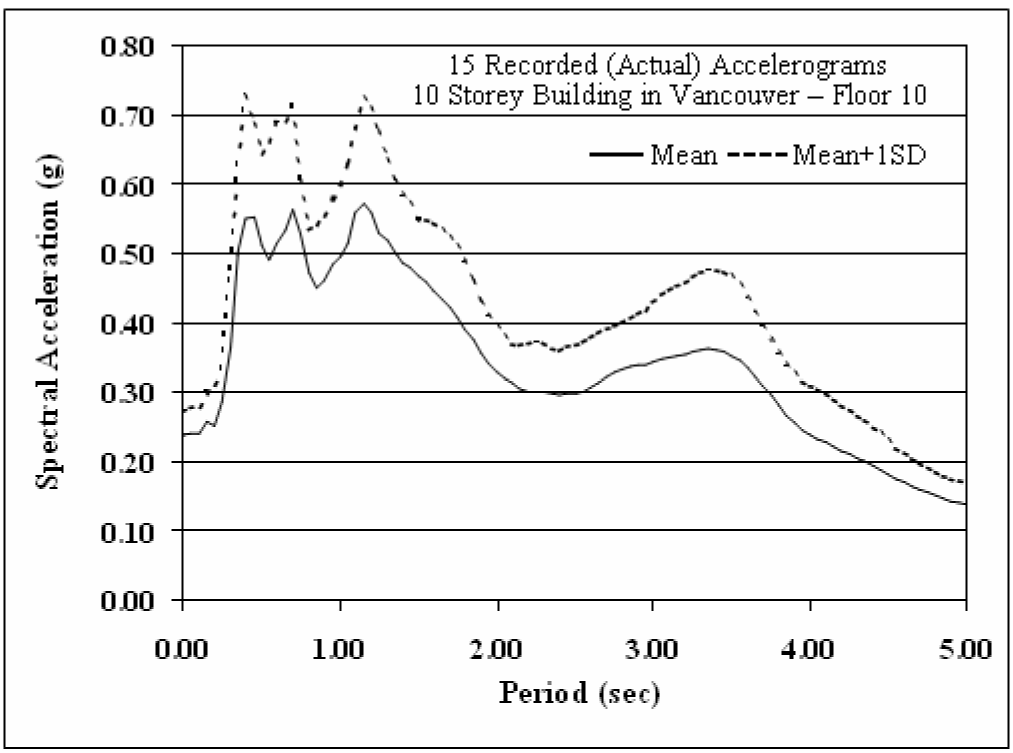

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 5
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
0.00
1.00
2.00
3.00
4.00
5.00

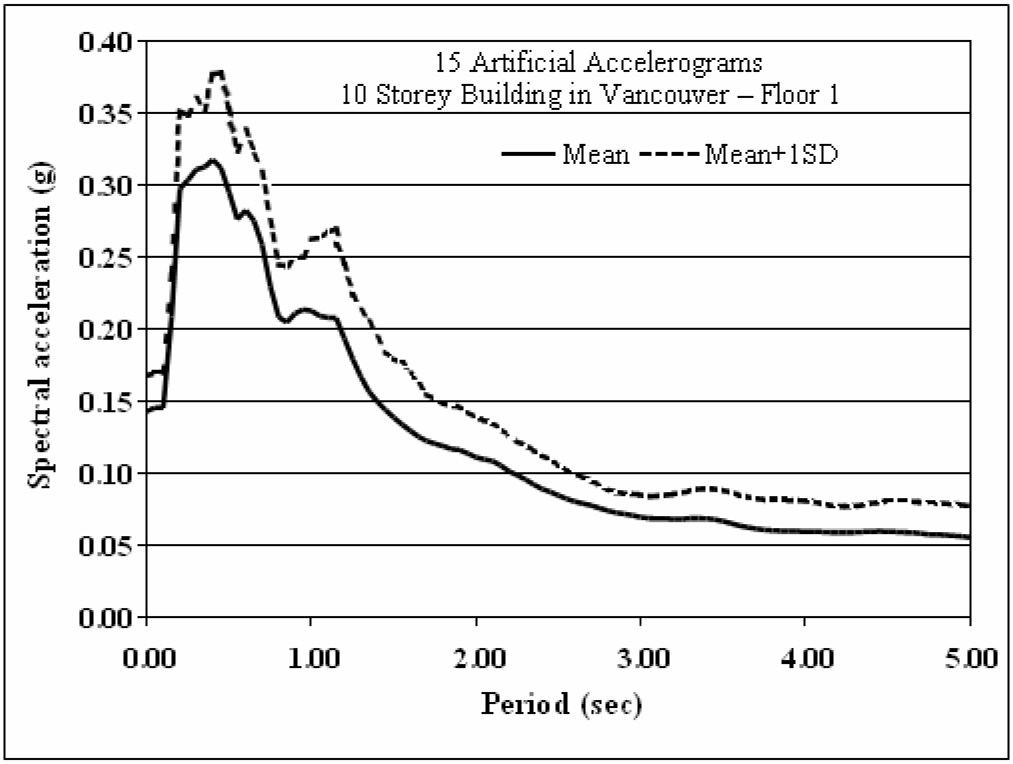

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 6
Mean
Mean+1SD
Spectral Acceleration (sec)
Period (sec)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
0.00
1.00
2.00
3.00
4.00
5.00

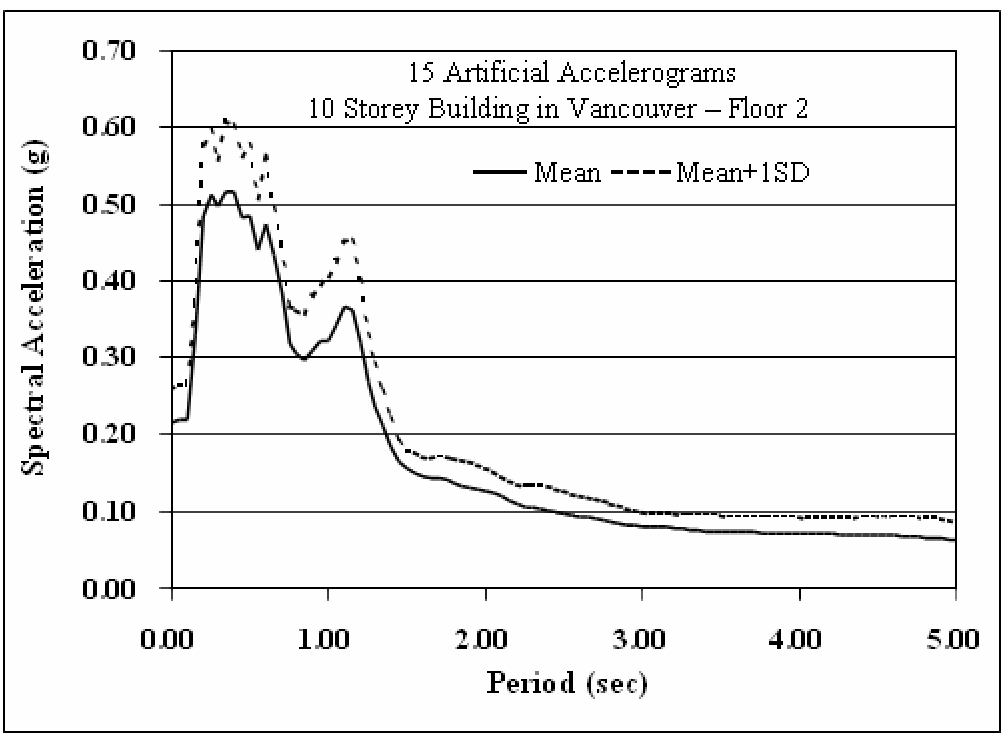

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 7
Mean
Mean+1SD
Spectral Acceleration (g)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (s)

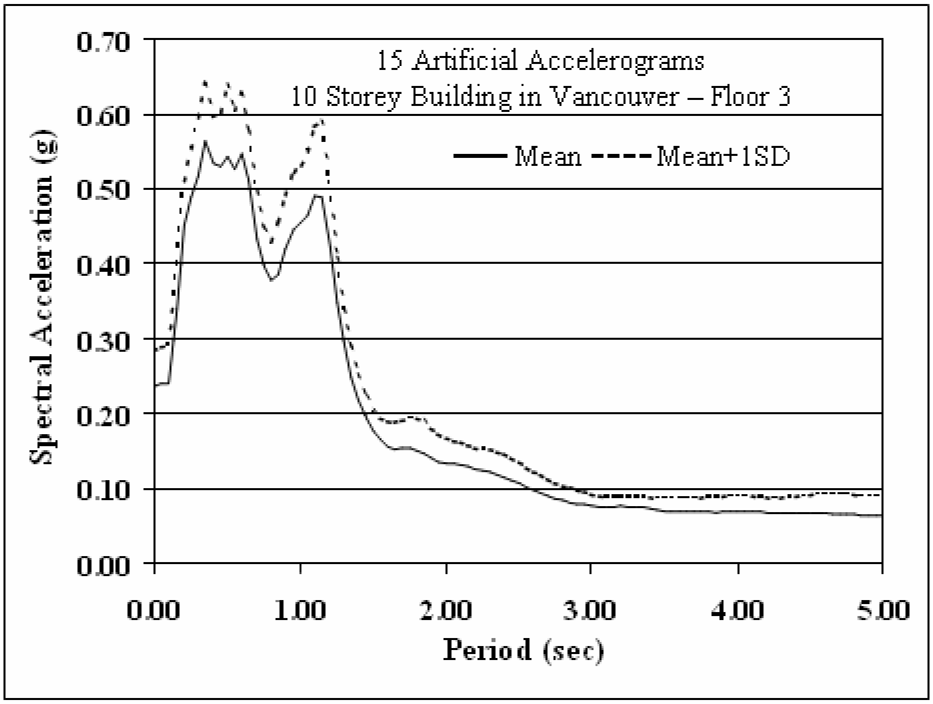

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 8
Mean
Mean+1SD
Spectral Acceleration (g)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)

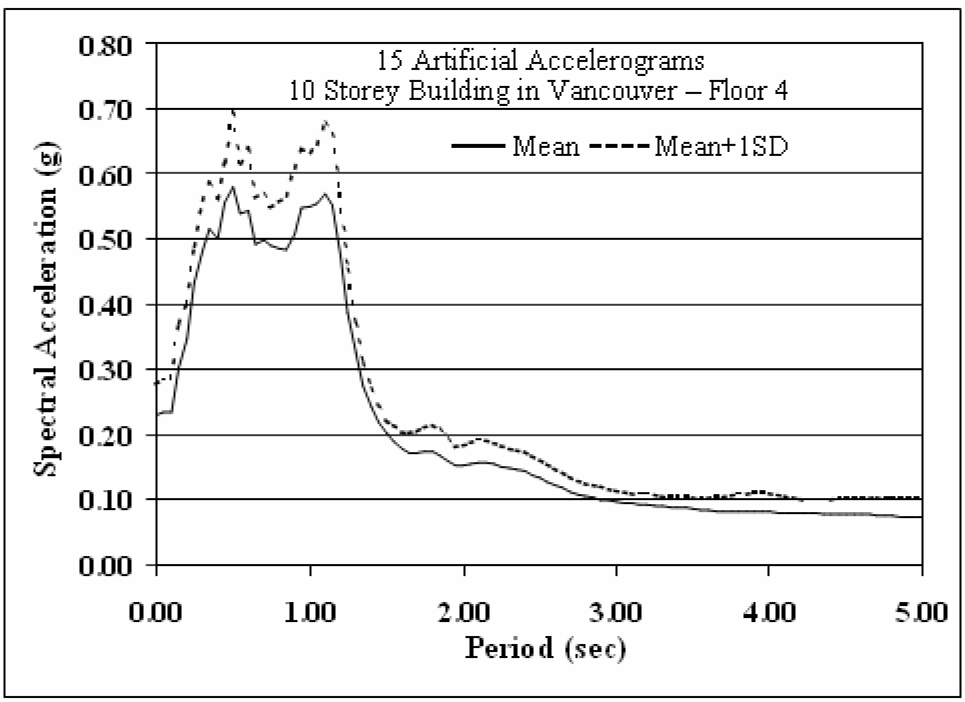

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 9
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
1.00
2.00
3.00
4.00
5.00

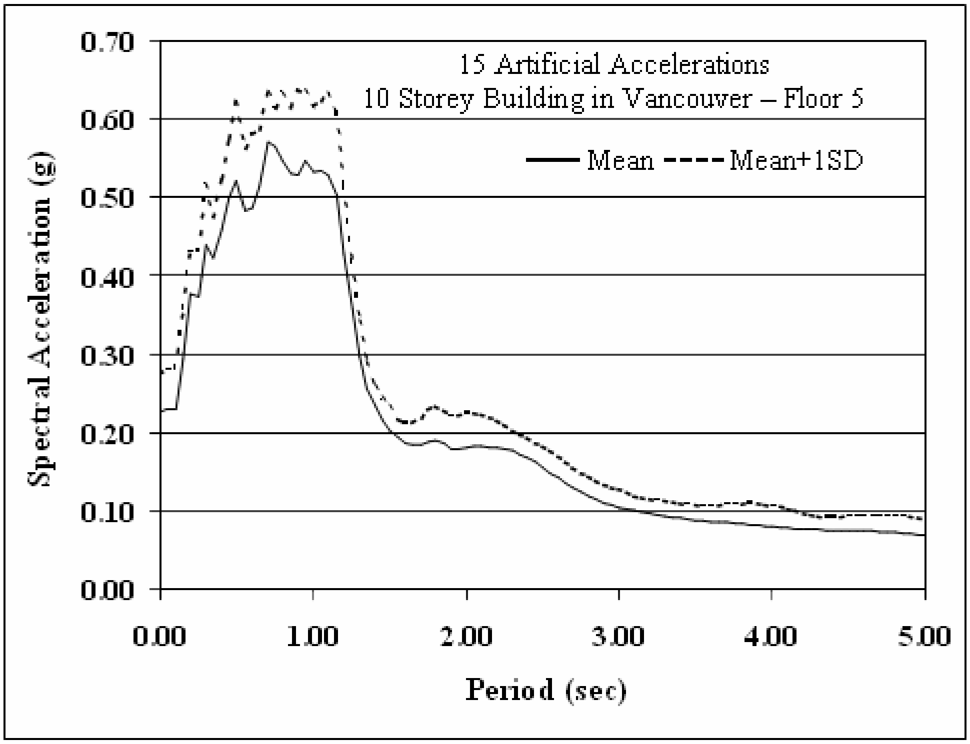

15 Recorded (Actual) Accelerograms
10 Storey Building in Vancouver – Floor 10
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
1.00
2.00
3.00
4.00
5.00

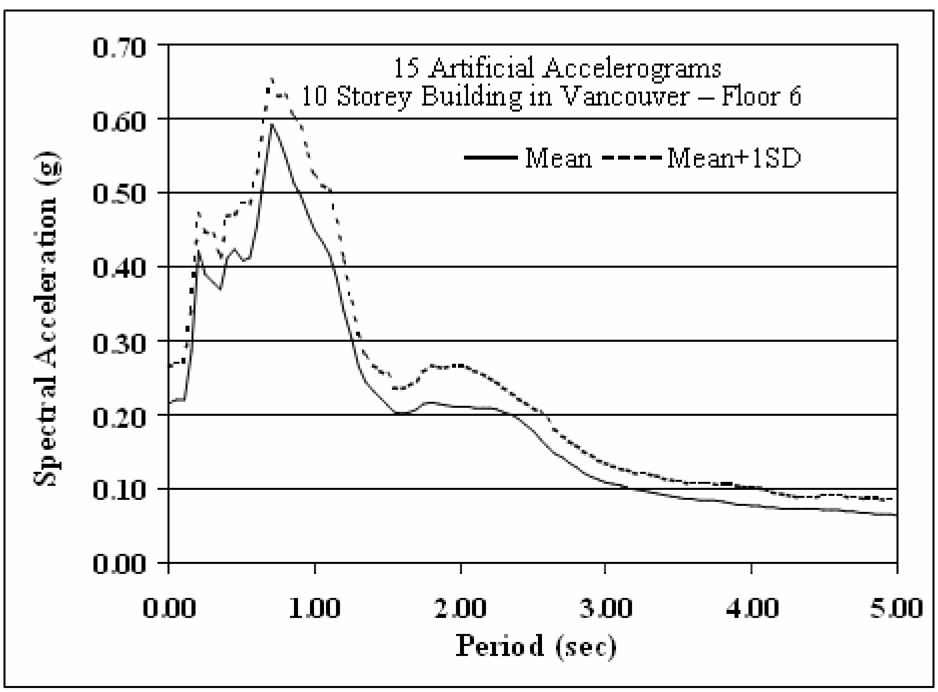

15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 1
Mean
Mean+1SD
Spectral acceleration (g)
Period (sec)
0.00
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.00
1.00
2.00
3.00
4.00
5.00

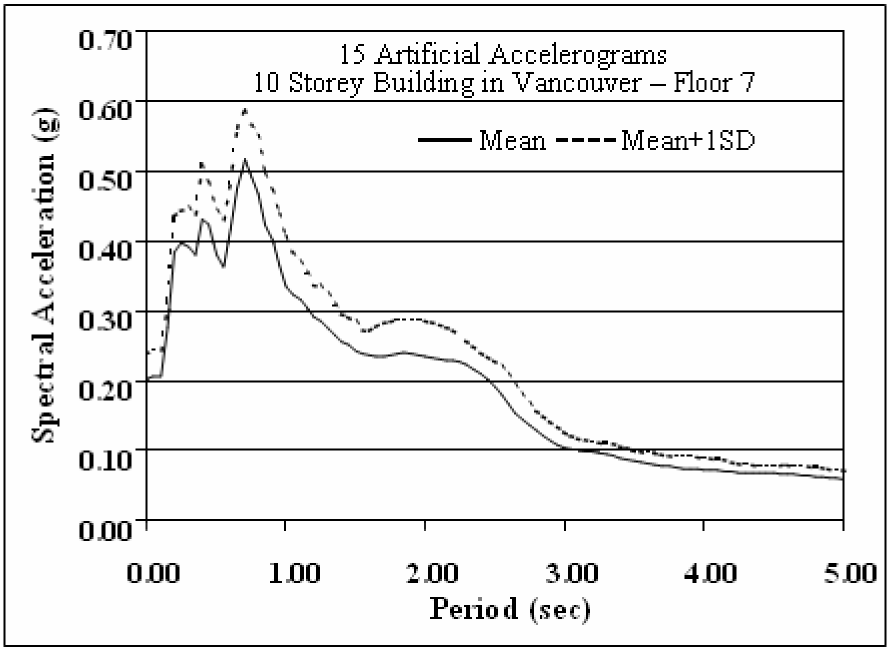

15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 2
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.00
1.00
2.00
3.00
4.00
5.00


15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 3
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00

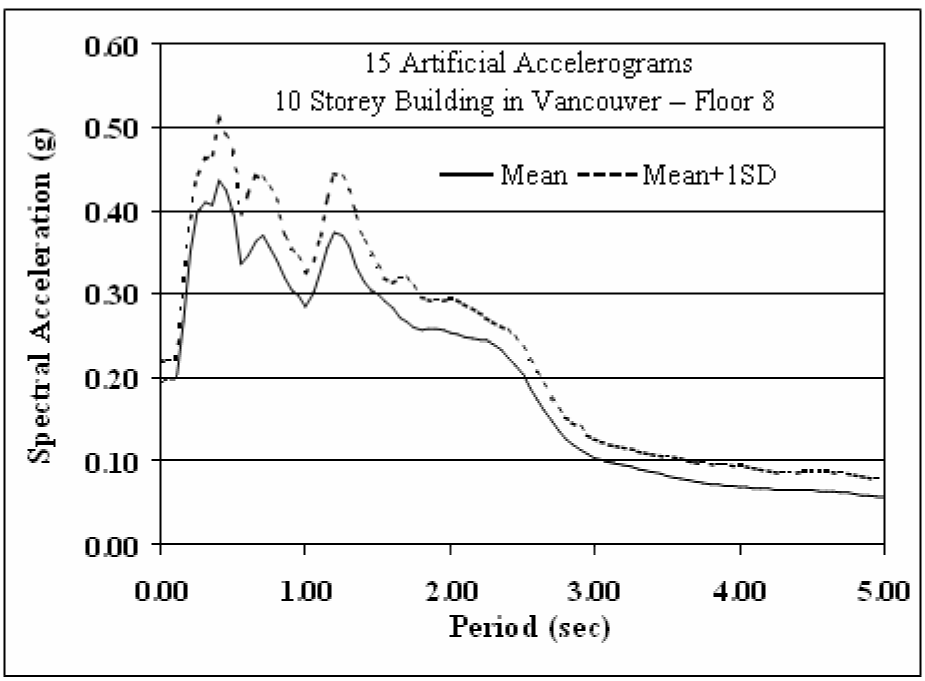

15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 4
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00


15 Artificial Accelerations
10 Storey Building in Vancouver – Floor 5
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00

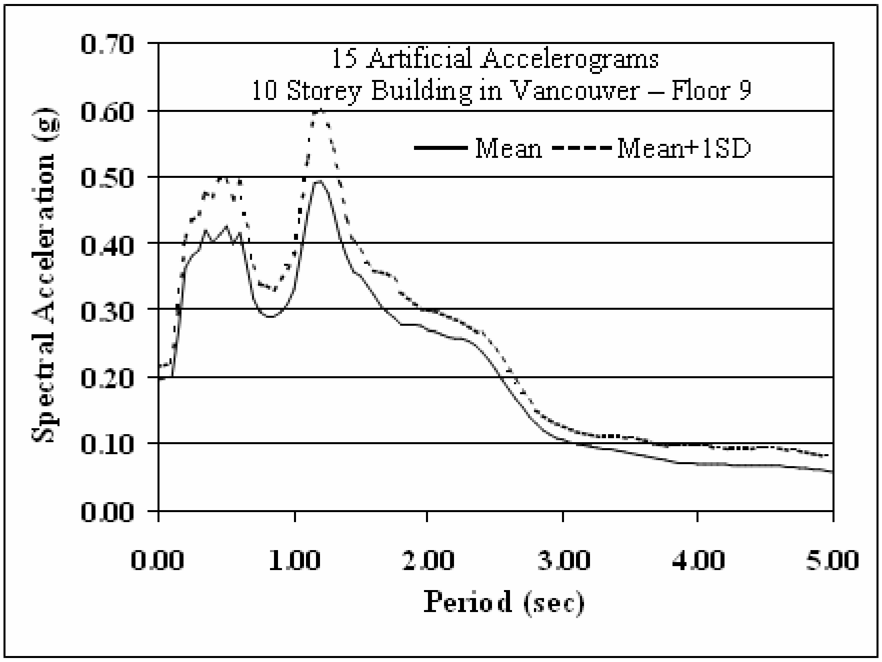

15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 6
Mean
Mean+1SD
Spectral Acceleration (g)
Period (sec)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00

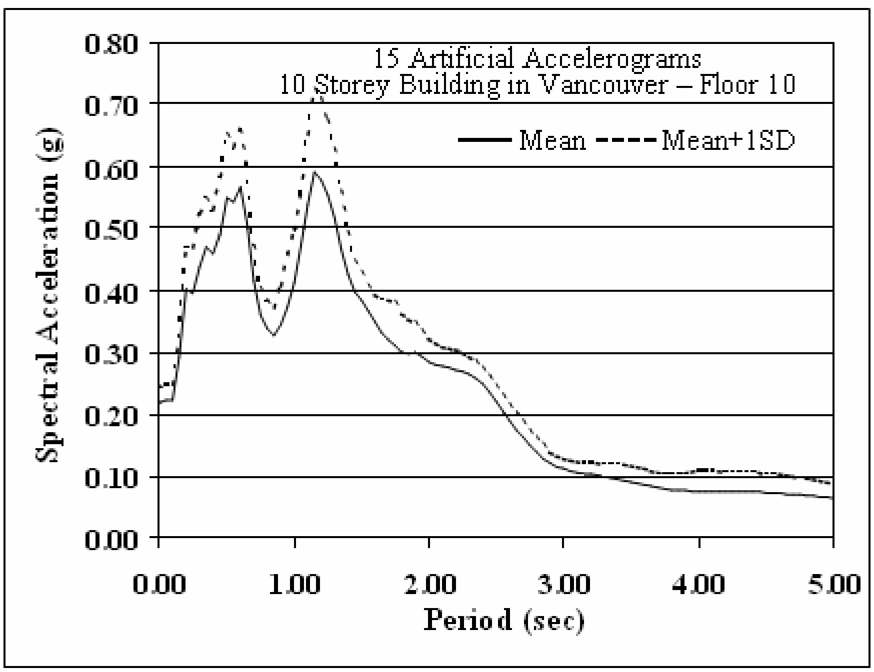


15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 7
Mean
Mean+1SD
Spectral Acceleration (g)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
Period (sec)
0.00
1.00
2.00
3.00
4.00
5.00



15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 8
Mean
Mean+1SD
Spectral Acceleration (g)
0.00
0.10
0.20
0.30
0.40
0.50
0.60
Period (sec)
0.00
1.00
2.00
3.00
4.00
5.00


15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 9
Mean
Mean+1SD
Spectral Acceleration (g)
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)


15 Artificial Accelerograms
10 Storey Building in Vancouver – Floor 10
Mean
Mean+1SD
Spectral Acceleration (g)
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.00
1.00
2.00
3.00
4.00
5.00
Period (sec)