EVALUATION OF THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS

by

Ashutosh Bagchi

B.Eng. (Civil Engineering), Jadavpur University, Calcutta, India

M.S. (Structural Eng.), Indian Institute of Technology, Madras, India

A thesis submitted to
The Faculty of Graduate Studies and Research
In partial fulfilment of the requirements
For the degree of

DOCTOR OF PHILOSOPHY

in
Engineering*

Department of Civil and Environmental Engineering
Carleton University, Ottawa, Canada
March, 2001

* The Doctoral Program in Civil and Environmental Engineering is a joint program with the University of Ottawa, administered by the Ottawa-Carleton Institute for Civil Engineering

The undersigned recommend to the Faculty of Graduate Studies and Research acceptance of the thesis

Evaluation of the Seismic Performance of Reinforced Concrete Buildings

Submitted by

Ashutosh Bagchi
B.C.E., Jadavpur University
M.S. Indian Institute of Technology

In partial fulfillment of the requirements for the degree of

Doctor of Philosophy

Professor Jagmohan L. Humar, Thesis Supervisor

Professor A. Ghobarah, External Examiner

Professor Wayne J. Parker, Chair, Civil and Environmental Engineering

Carleton University
April 2001

Abstract

The building codes in various jurisdictions, including Canada, follow a common concept in designing buildings to achieve an acceptable seismic performance. The objective underlying the concept is to ensure that the buildings designed based on code provisions should be able to resist minor earthquakes without damage, resist moderate earthquakes with some non-structural damage, and resist major earthquakes without collapse, but with some structural as well as non-structural damage. Seismic provisions in the building codes have evolved over the years to achieve this goal. Existing building codes focus on the minimum lateral load for which a structure must be designed. However, it is also necessary to include the demand and response characteristics of a structure, in the design.

The National Building Code of Canada (NBCC) is currently undergoing a review. One of the aims of this review is to permit an explicit definition of the expected seismic performance of buildings designed according to the code. The committee in charge of preparing the recommendations for seismic provisions of NBCC is considering a suggestion by which the calculation of the design lateral forces will be carried out on the basis of new seismic hazard maps of Canada. The new hazard maps, prepared by the Geological Survey of Canada (GSC), are based on the response spectral ordinates, rather than on the peak ground velocity or peak ground acceleration. It is expected that the future version of NBCC will require a building to be designed on the basis of these spectra, which are called, the uniform hazard spectra (UHS).

It is generally expected that if a structure designed on the basis of the seismic provisions of the building code is detailed properly to prevent the premature brittle

failure modes, and a capacity design principle is followed by which columns are made stronger than the beams to prevent storey sway mechanism, the structure would meet the performance objectives during expected levels of earthquakes. However, adequate studies are necessary to corroborate this idea.

The main objectives of the current research is to study the seismic performance of buildings designed according to the new seismic provisions (ie, based on the UHS based design forces). A review of the literature shows that there are not enough reports on studies on seismic performance of buildings designed according to NBCC seismic provisions. In this thesis a plan for comprehensive study of seismic performance of such buildings is presented. The following types of structural forms are identified for investigation: (a) concrete moment resisting frames, and (b) concrete shear wall systems

Static push-over analyses and nonlinear dynamic analyses are generally used to calculate the damage state in a structure. The seismic performance of a structure is determined on the basis of its damage state under an earthquake ground motion. A simplified procedure for seismic performance evaluation of buildings is also proposed.

Acknowledgement

I would like to express my sincere gratitude to my thesis supervisor Professor J.L. Huamr, Department of Civil and Environmental Engineering, Carleton University, for his guidance and inspiration in the course of this research. Without his active support and guidance I would be completely lost.

I would like to thank Professor M. Saatciaglu, Department of Civil Engineering, University of Ottawa, for his constructive comments and suggestions.

My sincere thanks are due to Professor D.T. Lau for his suggestions and encouragement.

Special thanks are due to Professor G. Atkinson, Department of Earth Sciences, Carleton University for supplying the ground motion time histories.

I would like to take this opportunity to thank my parents, sister and other relatives for their continuous support and encouragement throughout my life. I extend my sincere thanks to my wife, Monisha for her understanding, patience and inspiration.

Last but not the least, I thank my colleagues and friends for their encouragement and help.

Contents

List of Tables

List of Figures

Chapter 1

Introduction

1.1 General

Seismic loading provisions in the existing building codes focus on the minimum lateral seismic forces for which the building must be designed, but do not explicitly incorporate the demand and response characteristics (Fajfar *et al* 1996). However, the specification of the lateral forces alone is not enough to ensure the desired level of protection in a building when subjected to expected earthquakes of different intensities (Heidebrecht 1997). The design methodology must provide a means of assessing the level of protection as well as ensuring that the desired performance is achieved. With this understanding, the code writing bodies in various jurisdictions including Canada have embarked on major changes to the seismic codes.

The task of preparing and making recommendations for the seismic provisions of the National Building Code of Canada (NBCC) is entrusted to the Canadian Committee on Earthquake Engineering (CANCEE). As reported in its preliminary report, CANCEE (1996) is working on the development of a suitable format for expressing

the seismic hazard in terms of uniform hazard spectra (UHS) and on evaluating the appropriateness of the level of protection implied in the code specified design forces. The preliminary report of CANCEE (1996) presents an outline of the methodology for obtaining the elastic seismic forces from the spectral ordinates corresponding to earthquake events with a return period of 475 years. The methodology has been subsequently revised (Humar 2000) in which the design base shear calculation is based on the spectral acceleration corresponding to earthquake events with return period of 2500 years. A detailed study is required to determine if a structure designed on the basis of recommended seismic forces does indeed have the required level of protection. It is also necessary to assess the performance of these buildings under various levels of seismic hazard.

1.2 Objectives of seismic design

Severe earthquakes occur relatively infrequently. Although it is technically possible to design and construct buildings for these earthquake events, it is generally considered uneconomical and unnecessary to do so. The seismic design is performed with the anticipation that the severe earthquake would cause some damage, and a seismic design philosophy on this basis has been developed over the years. The goal of the seismic design is to limit the damage in a building to an acceptable level. The buildings designed with that goal in mind should be able to resist minor levels of earthquake ground motion without damage, resist moderate levels of earthquake ground motion without structural damage, but possibly with some non-structural damage, and resist major levels of earthquake ground motion without collapse, but

with some structural as well as non-structural damage.

As the foregoing discussion indicates, earthquake resistant design of buildings is based on the concept of acceptable levels of damage under one or more events of specified intensity. The acceptable level of damage is related to the performance objective for the building. Ideally the performance should be specified as an acceptable integrated probability of the building exceeding certain limit states during the entire spectrum of earthquake events that the building is likely to experience. Because of the complexity involved in specifying an integrated probability, requirements are often limited to one or more events of specified intensity or specified probability of exceedance. For example, the objective may be specified in the form of a requirement that the building is fully operational with little or no damage during an earthquake that has 50% probability of exceedance in 50 years; but may have moderate damage during an earthquake with 10% probability of exceedance in 50 years. The accepted level of damage also depends on the operational requirement for the building. For example, a building that is essential for post-earthquake service must be designed for a higher level of performance. In a report, the Vision 2000 committee (1995) of the Structural Engineers Association of California (SEAOC) has suggested performance objectives for buildings of different types.

1.2.1 Definition of hazard

Seismic hazards include effects like ground shaking, ground fault rupture etc., each of which can result in building damage and therefore affect the performance level achieved by a building. The damage potential of these effects is a function of the

earthquake magnitude, distance of the actual zone of fault rupture from the site, direction of fault rupture propagation, the geologic makeup of the region, and any unique geologic conditions of an individual site (Vision 2000, 1995). The goal of performance based engineering is to control the levels of damage in a building over the full spectrum of earthquake events which may occur. To facilitate practical application, the continuous spectrum of earthquake events for a given site is replaced by a series of discrete earthquake events which represent the range of seismic hazard for which a building performance is desired. These discrete earthquake events are called *earthquake design levels.*

Earthquake design levels are expressed in terms of the recurrence interval or a probability of exceedance. The recurrence interval or the return period of earthquake events of a given degree of severity is defined as the average period, expressed in years, between the occurrence of earthquakes which have the same or greater degree of severity. The probability of exceedance is a statistical representation of the chance that earthquakes with a given degree of severity or more will occur at a site within a specified number of years. As suggested in the report of the Vision 2000 Committee (1995) and in the report of NEHRP (1997), the levels of design earthquake can be defined as shown in Table 1.1. These earthquake levels are recommended for use in performance based engineering of buildings. To facilitate such use, suitable design parameters such as effective peak acceleration, elastic/inelastic response spectra etc., need to be calculated for each design event that is important to the performance objective.

Table 1.1: Design Earthquakes (Vision 2000 Committee, NEHRP)

Earthquake Design Level	Recurrence Interval	Probability of Exceedance
Frequent	43 years	50% in 30 years
Occasional	72 years	50% in 50 years
Rare	475 years	10% in 50 years
Very Rare	970 years	10% in 100 years
Extremely Rare	2500 years	2% in 50 years

1.2.2 Recommended performance levels

A performance level is defined as the maximum acceptable damage in a building when it is subjected to a specific level of earthquake. The damage suffered by structural and non-structural elements and contents as well as the availability of the site utilities necessary to building function are considered in defining the performance levels. The performance levels can be defined in qualitative terms which are meaningful to lay people including building owner, occupants and policy makers, and in quantitative terms which are useful for engineering design and evaluation. Vision 2000 report (1995) has suggested the following qualitative measures of performance levels.

- **Fully Operational:** A building can be called *fully operational* if there is essentially no damage, the building remains safe to occupy immediately after the earthquake, and all equipment and services essential to the building's basic occupancy and function are available for use. Repair is generally not required.

- **Operational:** A building can be called *operational* if there is moderate damage to its non-structural elements and contents, and light damage to structural elements, but the building remains safe to occupy immediately after the earth-

quake. Damage to some contents, utilities and non-structural elements may disrupt some normal functions. Repairs may be done at the owners and occupant's convenience.

- **Life-safe:** A building can be called *life-safe* if there is moderate damage to structural and non-structural elements and contents, but some margin against collapse remains. The structure's lateral stiffness is greatly reduced and the building would not be available for post-earthquake occupancy. The building may remain repairable, although it may not be economically practical to do so.

- **Near Collapse:** A building can be said to be *near collapse* if there is extreme damage in it, lateral and vertical load resisting capacities of the building have been substantially reduced, and the building remains unsafe for post-earthquake occupancy. Repair may not be technically or economically feasible.

- **Collapse:** A building can be assumed to have reached *collapse* if a portion or the whole of primary structural system collapses.

For engineering applications, performance levels need to be expressed in quantitative terms. Vision 2000 report (1995) provides some suggestions on the quantitative measures of performance based on drift levels. Other response/damage parameters such as, Park and Ang damage index which is discussed in detail later in this document, can also be used to quantify the performance levels. Table 1.2 provides some suggestions. These are based partly on Vision 2000 report (1995) and partly on the research papers of Park *et al* (1985a, 1985b).

Table 1.2: Performance Levels and Permissible Structural Damage (Vision 2000 Committee, Park and Ang)

Damage Parameter	Performance Level				
	Fully Oper.	Operational	Life safe	Near Collapse	Collapse
Drift					
a) Transient	< 0.2%	< 0.5%	< 1.5%	< 2.5%	> 2.5%
b) Permanent	Negligible	Negligible	< 0.5%	< 2.5%	> 2.5%
Park and Ang Damage index					
a) Element level	< 0.40	< 0.90	< 0.90	< 1.00	> 1.00
b) Storey level	< 0.25	< 0.50	< 0.70	< 0.90	> 0.90
c) Global	< 0.20	< 0.40	< 0.60	< 0.70	> 0.70

1.2.3 Building categories and design performance objectives

A design performance objective of a building can be defined as the desired performance level of the building for each earthquake design level. Design performance objectives are selected based on the buildings occupancy, the importance of functions occurring within the building, economic considerations, and historical or cultural importance of the building. Based on the occupancy, uses etc., buildings are grouped into following three categories (Vision 2000, 1995).

- **Safety Critical Facilities:** The buildings under this category contain large quantities of hazardous materials such as, toxic materials, explosives, and radioactive materials, the release of which would result in unacceptable hazard to public.

Table 1.3: Recommended performance objectives (Vision 2000 Committee)

Earthquake Design Level	Minimum performance Level		
	Safety Critical Facilities	Essential/Hazardous Facilities	Basic Facilities
Frequent	Fully operational	Fully operational	Fully operational
Occasional	Fully operational	Fully operational	Operational
Rare	Fully operational	Operational	Life-safe
Very rare	Operational	Life-safe	Near Collapse

- **Essential/Hazardous Facilities:** *Essential facilities* are those which are critical to post-earthquake operations, examples of which include hospitals, police stations, fire stations etc. *Hazardous facilities* are those which contain large quantities of hazardous materials, but the release of those materials would be contained within the boundaries of the facility, so that the impact to public would be negligible. Example of Hazardous Facilities include oil refineries, micro-chip manufacturing facilities etc.

- **Basic Facilities:** The buildings which are not classified as Safety Critical or Essential/Hazardous facilities come under this category.

The recommended performance objectives of these buildings are summarised in Table 1.3.

1.2.4 Seismic level of protection

Although the building codes (eg. NBCC) define the level of seismic protection in terms of the minimum lateral forces used in the design, it is clear that it depends on

many additional parameters. In practical terms, the seismic level of protection for buildings must be related to the performance. Performance of a building is in turn related to the maximum degree of damage that could be tolerated. Damage could be associated with various parameters like, lateral deflection, inter-storey drift and some form of damage index. Damage related performance expectation can be used to evaluate the seismic level of protection for a building. In that case, performance is measured by the damage that a building constructed in accordance with specific code requirements is likely to suffer, when subjected to earthquake ground motions associated with the seismic hazard corresponding to operational and life-safety requirements. The important aspects of the level of protection analysis are: seismic hazard, seismic design, structural damage and vulnerability function (Heidebrecht 1997). A vulnerability function can be defined as the variation of expected degree of damage with the intensity of ground motion. The seismic provisions of a building code must accommodate all these aspects in a comprehensive way.

1.3 Earlier work on performance evaluation

Although it is realised by the engineering community that the performance based seismic design is important, there is very little published work on the study of performance achieved by code designed buildings. There are some studies on seismic performance of buildings designed based on seismic provision of the US code (Anderson *et al* 1991, Anderson and Bertero 1991, Miranda and Bertero 1991, Teran-Gilmore and Bertero 1992, 1993, Bertero *et al* 1994). These studies are concerned with the performance analysis of specific structures under certain specific earthquake ground

motions. Although these studies throw some light on the performance levels achieved by a structure, they do not attempt to establish if code designed buildings, in general achieve the expected performance levels under various levels of earthquake events. There are only a few reports on the study of performance of buildings designed according to the seismic provisions of the National Building Code of Canada. Limited amount of work has been reported by Paultre and Mitchel (1987), Mitchel and Paultre (1994), Heidebrecht (1997), Heidebrecht and Naumoski (1998), Filiatrault *et al* (1997), Ghobarah *et al* (1997, 1999) and Stonehouse it et al (1999). Some of the recent studies are briefly discussed in the following paragraphs.

The study by Mitchel and Paultre (1994) focused on the ductility and overstrength present in reinforced concrete structure designed according to NBCC. Although this study was not directed to seismic performance analysis of buildings, it provided some interesting information about the behaviour of reinforced concrete moment resisting and shear wall frames, which can be used in the evaluation of seismic performance of these structures. The structures were designed using the seismic provisions of 1990 edition of NBCC. Results of a series of static push-over analyses of the structures were presented in the paper. Three different levels of material strength corresponding to the factored, nominal, and probable values were considered in the analyses. A six storey concrete moment resisting frame, and a twelve storey concrete shear wall frame were considered in the study. Two levels of structural ductility and energy dissipation capacity, one corresponding to nominally ductile frame (ie. with seismic force modification factor, R equal to 2), and the other for a ductile frame (ie, R = 4 for moment resisting frame, and R = 3.5 for shear wall frame) were considered. The buildings were assumed to be located in Montreal. It was demonstrated that

the ductile structures, which are designed with lower seismic design forces, can in fact exhibit higher capacities than the nominally ductile structures. Since seismic performance largely depends on the capacity of a structure, the ductile structures were found to have a better performance.

A study by Heidebrecht (1997) described a research framework for evaluation of the seismic level of protection. Using the methodology described there, Heidebrecht and Naumoski(1998) conducted a study on the seismic performance of two six-storey ductile moment resisting reinforced concrete frames. The performance was evaluated based on minimum transient inter-storey drift criteria as specified by Vision 2000 committee (1995). The buildings were designed based on NBCC 1995 seismic provisions and a seismic hazard corresponding to Vancouver was used in the design. All design parameters except the design drift limit were same for the two structures. One of the buildings was designed to limit the inter storey drift to 1% while the other was designed to limit it to 2% as specified by NBCC 1995. These buildings were subjected to an ensemble of 15 ground motion time histories having spectral shapes similar to those of the design level seismic ground motions expected in Vancouver. The ground motion time histories were scaled to the peak horizontal velocity. In order to determine the performance for a full range of excitations which could be expected during the life-time of a structure, various excitation levels ranging from 0.5 to 3.0 times the design level were used in the dynamic analysis. Ground excitations up to 3 times the design level were used because the uncertainty involved in estimating the peak ground motions at any location is such that the values ranging from 2 to 3 times could easily occur. Based on the results of nonlinear dynamic analysis, these structures were found to have exhibited a performance level equivalent to 'operational'

during the design level of ground motion and 'life-safe' during excitations about 3 times the design level.

A study on concrete moment resisting frames designed according to NBCC 1995 seismic provisions has been conducted by Filiatrault *et al* (1998). Experimental as well as analytical study on the performance of a ductile (qualifying for the NBCC force modification factor R = 4) and a nominally ductile (ie., R = 2) frames were carried out. Half scale models of the frames were used in the experimental and numerical analyses. It was found that both buildings met the performance expectation under the design intensity of ground motion. The ground motion intensity was doubled and it was found that the nominally ductile frame showed a plastic column-storey mechanism, while the ductile frame exhibited expected performance. The performance of the ductile structure showed the effectiveness of the capacity design philosophy adopted by the current Canadian Standards for Concrete (CSA Standard A23.3-94).

Stonehouse *et al* (1999) presented a study on seismic performance of shear wall buildings designed according to NBCC 1995 seismic provisions. The study follows the research framework designed by Heidebrecht (1997).

Linde (1998) presented an evaluation of the design provisions of Eurocode 8 and Swiss Earthquake Standard, SIA 160, for structural walls. The height of the buildings were varied from four to eight storeys. The structures were subjected to recorded ground motion and nonlinear time history analyses were performed. Design provisions of the two codes were compared and flexural resistance in Eurocode 8 was found to be more conservative than SIA 160.

Panagiotakos *et al* (1999) have observed that the estimation of peak deformation

demand is a key element in the displacement based seismic design of new structures and seismic evaluation of existing structures. Based on a number of nonlinear dynamic analyses, rules were developed for the estimation of mean and peak values of inter-srtorey drift and member chord rotation in multi storey RC frame buildings. Both bare and infilled (all but the first storey were infilled) frames were considered.

Balendra *et al* (1999) discussed the vulnerability of RC frames designed according to BS 8110. The effect of the redistribution of internal forces on overstrength and ductility demand was investigated for three-bay multi-storey reinforced concrete plane frames. Nonlinear push over analysis was used. the effect of infill panels was also considered.

Ghobarah *et al* (1999) discussed the concept of response-based damage assessment of structures. They reviewed the existing damage assessment procedures and proposed a new way of assessing structural damage. Ghobarah *et al* (1997) presented an evaluation of seismic performance of a three storey building designed for Victoria.

Bagchi (1999), Humar and Bagchi (2000) studied the performance of concrete moment resisting and shear wall frames designed according to the preliminary guidelines for seismic design proposed by CANCEE (1996). UHS-compatible ground motion time histories developed by Atkinson *et al* (1998) were used in the dynamic analyses.

The studies discussed above are not exhaustive. However, they provide very useful clues in understanding the basic behaviour of, and performance levels achieved by code designed buildings. Further study is necessary to understand the general behaviour of such buildings under design and higher levels of seismic hazard. The lateral load resisting systems of a majority of the structures considered in the above

studies comprise reinforced concrete moment resisting frames. Performance analyses of buildings with several commonly used structural forms such as, concrete moment resisting frame, concrete shear wall, steel moment resisting frame and steel braced frame system, need to be carried out in a systematic way.

1.4 NBCC Design provisions

Since the present study is related to the expected performance of buildings designed according to the provisions of the National Building Code of Canada, a brief review of the provisions the code is useful. The regulations for earthquake resistant design of buildings in Canada have been included in the National Building Code of Canada since 1953 (Heidebrecht 1997). Based on the observed damages and lessons learned from the past major earthquakes, the seismic provisions were modified over the years. The latest edition of NBCC was released in 1995. Further modifications are expected in the seismic provisions of the future edition of NBCC, expected to be released in year 2002 or 2003. These modifications would be based on the improved knowledge of the seismology of the country, the behaviour of structures under severe earthquakes that have occurred in the recent past, and results of the current research on performance-based engineering.

1.4.1 Present methodology - NBCC 1995 provisions

The 1995 edition of the National Building Code of Canada, which will be referred to as NBCC 1995, follows the seismic design philosophy explained in Section 1.2 of this thesis. The seismic design methodology suggested by the NBCC is based on the

equivalent static load approach. Structures designed to resist the equivalent static loads are expected to perform satisfactorily when a design level earthquake event occurs. Various aspects of NBCC 1995 are briefly discussed below.

The 1995 edition of NBCC adopts a two-parameter seismic zoning approach to quantify the seismic hazard across Canada. Canada is divided into seven acceleration related zones, Z_a, and seven velocity related zones, Z_v. The level of seismic risk at any site is expressed in terms of both peak horizontal ground acceleration and peak horizontal ground velocity, each with a probability of exceedance of 10% in 50 years. Two dimensionless parameters, the zonal acceleration and zonal velocity ratios, are used to define the seismic zones mentioned above. The zonal acceleration ratio, a is defined as the ratio of the peak horizontal ground acceleration to the acceleration due to gravity, g, and the zonal velocity ratio, v is defined as the ratio of the peak horizontal ground velocity to a velocity of 1 m/s. Although earthquake ground motions, in general, are multi-directional, NBCC 1995 allows a structure to be designed to resist the seismic forces acting independently along each of the two horizontal principal axes of the structure, together with associated torsional effect.

According to NBCC 1995, the minimum lateral seismic shear force, V, at the base of a structure should be calculated using the following formula.

$$V = (V_e/R)U \tag{1.1}$$

where V_e is the elastic base shear, representing the elastic response, R is a force modification factor, which actually is a measure of ductility supply in a structure, and

U is a calibration factor, which is equal to 0.6. The calibration factor, U is introduced to provide a structure with the same level of seismic protection as provided by the earlier version of the code. Often, it is viewed as a factor to account for the seismic overstrength in the structure. The equivalent elastic base shear, V_e is determined from the following formula.

$$V_e = vSIFW \tag{1.2}$$

where v is the zonal velocity ratio for a site, S is the seismic response factor, I is the importance factor, F is the foundation factor, and W is the weight of the system participating in the dynamic response.

The seismic response factor S is a representation of the idealised acceleration response of a 5% damped multiple degrees of freedom (MDOF) system, for unit value of v. The value of S depends on the fundamental period of the structure, T, and the zoning parameters Z_a and Z_v. The variations of S is shown in Fig. 1.1. The code provides empirical expressions for the determination of fundamental period, T. The importance factor, I is taken as 1.0, except for post-disaster buildings such as, hospitals, fire station, schools etc. where a higher value specified by the code is used. In order to limit the damage to non-structural elements of buildings during severe earthquakes, NBCC 1995 requires that the maximum inter-storey drift be limited to 1% of the storey height for post-disaster buildings, and 2% for all other buildings.

To account for the effect of higher modes NBCC 1995 requires that for buildings having a period higher than 0.7 s, a portion of V equal to F_t be assigned to the

topmost storey and the remaining shear $(V - F_t)$ distributed along the height using the following expression.

$$F_x = (V - F_t) w_x h_x / (\sum_{i=1}^{n} w_i h_i) \tag{1.3}$$

where F_x is the shear at storey level x; h_i, h_x are heights above base to levels i and x respectively; and w_x, w_i are the weights corresponding to levels x and i respectively. The value of F_t depends on the period of the building.

1.4.2 UHS based design - preliminary provisions

Seismic hazard can be specified in terms of either peak ground acceleration or peak ground velocity as in NBCC 1995, or as response spectral ordinates for given periods of vibration. The use of response spectral ordinates instead of peak ground motion parameters is more meaningful because of the greater engineering relevance of the former (Atkinson *et al* 1998). Thus, the new Canadian seismic hazard maps, prepared by the Geological Survey of Canada (GSC), are based on response spectral ordinates, and the future version of NBCC is expected to require that the structures be designed on the basis of these spectra, which are known as the uniform hazard spectra (UHS). The uniform hazard spectra for various levels of seismic hazard are shown in Fig. 1.2.

Based on the new ground motion maps prepared by the Geological Survey of Canada (Adams *et al* 1996), CANCEE has proposed a preliminary formulation for elastic seismic forces (CANCEE 1996). This formulation is meant to form a basis for discussion for the new edition of the building code. The hazard spectra proposed by

CANCEE are based on the actual spectra prepared by GSC and are characterised by their ordinates $S_a(0.2)$, $S_a(0.5)$, and $S_a(1.0)$, where S_a represents spectral acceleration. These spectra are available for most seismic regions of Canada. The elastic seismic base shear, V_e^* can be determined using the following expression.

$$V_e^* = S(T)M(T)IF^*W \tag{1.4}$$

in which $S(T)$ is the spectral acceleration corresponding to the period T, $M(T)$ is a multiplier to account for the effect of multiple degrees of freedom (MDOF), F^* is a new site (foundation) factor, I is the importance factor as defined in NBCC 1995 and W is the weight of the structure as defined in NBCC 1995. The spectral acceleration is obtained as follows.

$$\begin{aligned} S(T) &= S_a(0.2) \quad \text{for} \quad T \leq 0.2 \text{ s} \\ &= S_a(0.5) \quad \text{for} \quad T = 0.5 \text{ s} \\ &= S_a(1.0) \quad \text{for} \quad T = 1.0 \text{ s} \\ &= S_a(1.0)/T^k \quad \text{for} \quad T > 1.0 \text{ s} \end{aligned} \tag{1.5}$$

For $0.2 < T < 0.5$ and $0.5 < T < 1.0$, $S(T)$ is obtained by linear interpolation. The exponent k is 0.7 for the western (west of 100^0 longitude) locations and 1.6 for the eastern locations in Canada. The actual GSC spectra (Adams *et al* 1996) for 475-year earthquakes (UHS-500) and those given by Equation 1.5 are very close, and the comparison is shown in Fig. 1.3. The multiplier $M(T)$ is used to account for the effect of higher order modes. The value of $M(T)$ is taken as 1.0 for $T < 1.0$. For shear

type structures (eg. moment resisting frames) it is approximately 1.0 throughout the period range, while it is larger for cantilever type structures (eg. flexural walls). According to the preliminary reports, $M(2)$ is relatively small for cantilever type structures in western Canada (approximately 1.25), but can be larger (2 or more) for the same type of structure in eastern locations.

The values for F^* evaluated by CANCEE are based on the soil classification and are based on the NEHRP guidelines. Firm soil corresponding to Site class B is used as the reference ground condition, so that F^* is 1.0 for that site class.

The minimum lateral seismic base shear V (the design force) is determined by

$$V = (V_e^*/R)U^* \tag{1.6}$$

where R is the force modification factor as defined in NBCC 1995 and U^* is the new calibration factor which is 1.0 rather than 0.6 as specified for U in NBCC 1995. Value of U^* needs to be chosen carefully, so that the structures designed with the minimum lateral seismic base shear V have adequate level of protection. Although V provides a measure of strength of the structure, the performance level of a structure during a specified level of earthquake depends on several other factors, one of which is the calibration factor, U^*. Evidently, this factor must be selected so as to ensure the expected level of performance of a building during the design earthquake.

The seismic provisions suggested by CANCEE provide a revision of design lateral loads only. Other provisions of NBCC 1995 affecting the design of regular buildings remain essentially unchanged. The revision of lateral forces is expected to improve

the performance of the building designed using them. However, performance of a building does not depend on the design loads alone. It is generally expected that, if a structure is detailed properly to prevent premature brittle failure modes, and the capacity design philosophy (strong column and weak beam concept) is followed in the design, the structure would meet the performance objectives to be achieved during design earthquake levels. This expectation may be logical, but adequate studies are necessary to confirm it. It would also be very useful if specific design criteria can be developed to ensure that the performance objectives of a code designed building will be achieved.

1.4.3 UHS based design - revised provisions

The preliminary seismic provisions suggested by CANCEE have undergone significant revision (Humar 2000). According to NBCC 1995, the buildings are required to be life-safe under an earthquake with a return period of 475 years. However, such buildings have considerable reserve strength to sustain earthquakes of higher intensity. Fig. 1.4 shows the variation of spectral acceleration for a building having a period of 0.2 s with the recurrence interval of design earthquakes. Supposing that the building has sufficient overstrength to sustain twice the design shear without collapse, it will be observed that the building in western Canada will be able to sustain a 2400-year earthquake although it was designed to sustain a 475-year earthquake, while the building in eastern Canada will be able to sustain an earthquake with a return period of about 1600 years. Consequently, the two building have different levels of protection. Thus to design the two buildings to have the same levels of protection, the design base shear should be derived for, say a 2500-year earthquake and then reduced

by a factor to account for the reserve strength. This approach is being considered in the new version of NBCC.

The elastic base shear, V_e for a single degree-of-freedom building can be obtained from the spectral acceleration value, $S(T)$ corresponding to the period of the building, T and weight of the building, W.

$$V_e = S(T)W \tag{1.7}$$

Site-specific values of the spectral acceleration $S_a(T)$ are available from GSC. The design spectral acceleration, $S(T)$ value can be obtained from the following expressions.

$$\begin{aligned} S(T) &= F_a S_a(0.2) \quad \text{for} \quad T \leq 0.2 \text{ s} \\ &= F_v S_a(0.5) \quad \text{or} \quad F_a S_a(0.2) \quad \text{whichever is less for } T = 0.5 \text{ s} \\ &= F_v S_a(1.0) \quad \text{for} \quad T = 1.0 \text{ s} \\ &= F_v S_a(2.0) \quad \text{for} \quad T = 2.0 \text{ s} \\ &= F_v S_a(2.0)/2 \quad \text{for} \quad T \geq 4.0 \text{ s} \end{aligned} \tag{1.8}$$

Straight line interpolation can be used for intermediate values. F_a and F_v are the foundation factors corresponding to short and long period ranges, respectively. The spectral values obtained from these expressions for firm ground condition are shown in Fig. 1.5.

Apart from ductility and the importance of a structure, the effects of overstrength and higher modes for multi-storey buildings are also considered in the evaluation of

the design base shear. Following expression for the design base shear is proposed.

$$\begin{aligned} V &= V_e M_v I/(R_0 R_d) \\ &= S(T) M_v I W/(R_0 R_d), \\ &\text{but not less than } S(2.0) M_v I W/(R_0 R_d) \end{aligned} \tag{1.9}$$

where, M_v accounts for higher mode effect, I is the importance factor, R_d and R_0 account for ductility overstrength, respectively. The value of M_v depends on the period, location and type of a structure. For a moment-resisting frame the values of M_v for eastern and western Canada do not differ much. However, for a wall or wall-frame system the difference between the values of M_v for eastern and western Canada could be higher.

1.5 Objectives and scope of the present work

The changes in the current code provisions for seismic design of buildings are being made to achieve better seismic performance of buildings. However, performance objectives are not explicitly incorporated in the design provisions. Thorough research is necessary to determine if the proposed seismic provisions are adequate to ensure the expected performance and level of protection for the structures designed according to such provisions. The new guidelines and formulations proposed for the next version of NBCC must be verified and their potential effect on the seismic design and performance of buildings must be investigated in detail. The current research initiative focuses on this issue. The scope and objectives of the current investigation are outlined below.

- Assessment of the suitability of analytical models and methods that are presently available for conducting analyses of the seismic performance of buildings with common structural forms and materials, and selection of an appropriate method.
- Using the analytical tools stated in 1 above, evaluation of the strength and capacities of the buildings designed on the basis of new seismic guidelines.
- Evaluation of the damage that the above buildings may suffer when they are subjected to the ground motions corresponding to the design and higher levels of earthquakes.
- Evaluation of the qualitative performance levels achieved by the buildings under various levels of seismic hazard.
- Development of a simplified method of analysis to estimate the overall and local damage states and deformation quantities of these buildings.
- Assessment of the implications of the new seismic provisions on the design and performance of buildings.

Although an exhaustive study involving many different building configurations and a wide range of building heights and types is desirable, it may not be practical. Thus, only a few building configurations are considered in the current study.

1.6 Organization of the thesis

This thesis is organized into six chapters. Some introductory materials and objective of the thesis are presented in the current chapter. Methodologies for the evaluation

of seismic performance of buildings, background materials, damage parameters and computer softwares used in this thesis are presented in Chapter 2. Results of the performance evaluation of concrete frames and shear wall buildings are presented in Chapter 3 and 4, respectively. A simplified method of performance evaluation is presented in Chapter 5. A summary of the thesis and a list of conclusions are presented in Chapter 6. References are placed at the end of the thesis.

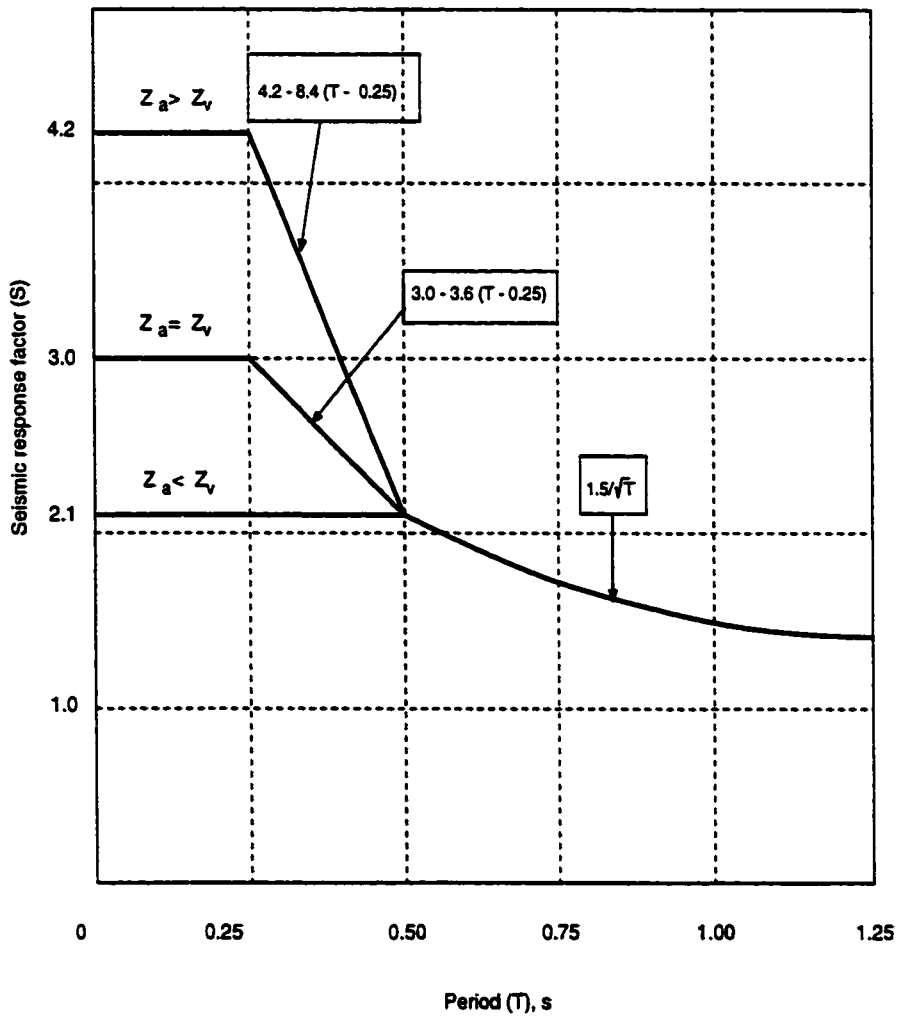


Figure 1.1: The seismic response factor (NBCC 1995)

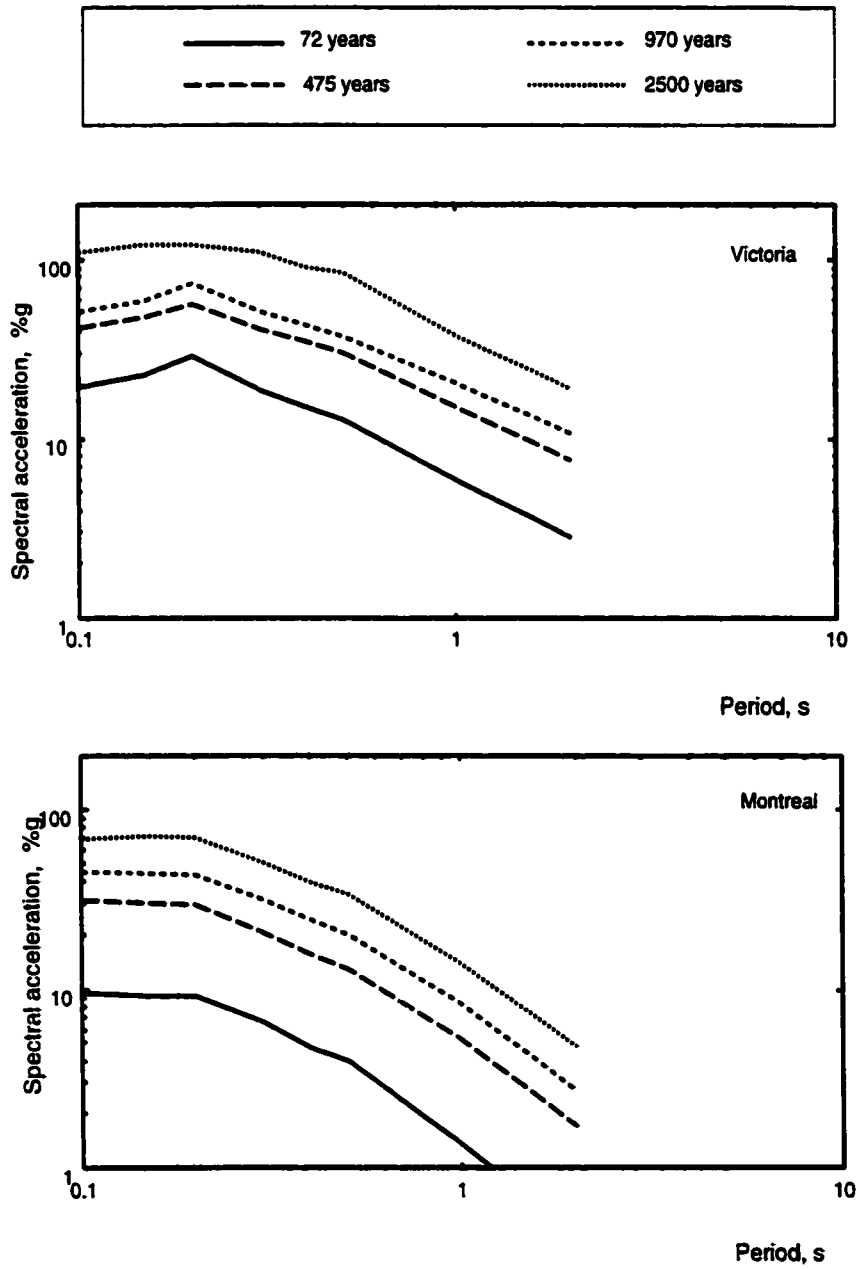


Figure 1.2: Uniform hazard spectra for various levels of seismic hazard expressed in terms of reccurence interval in years

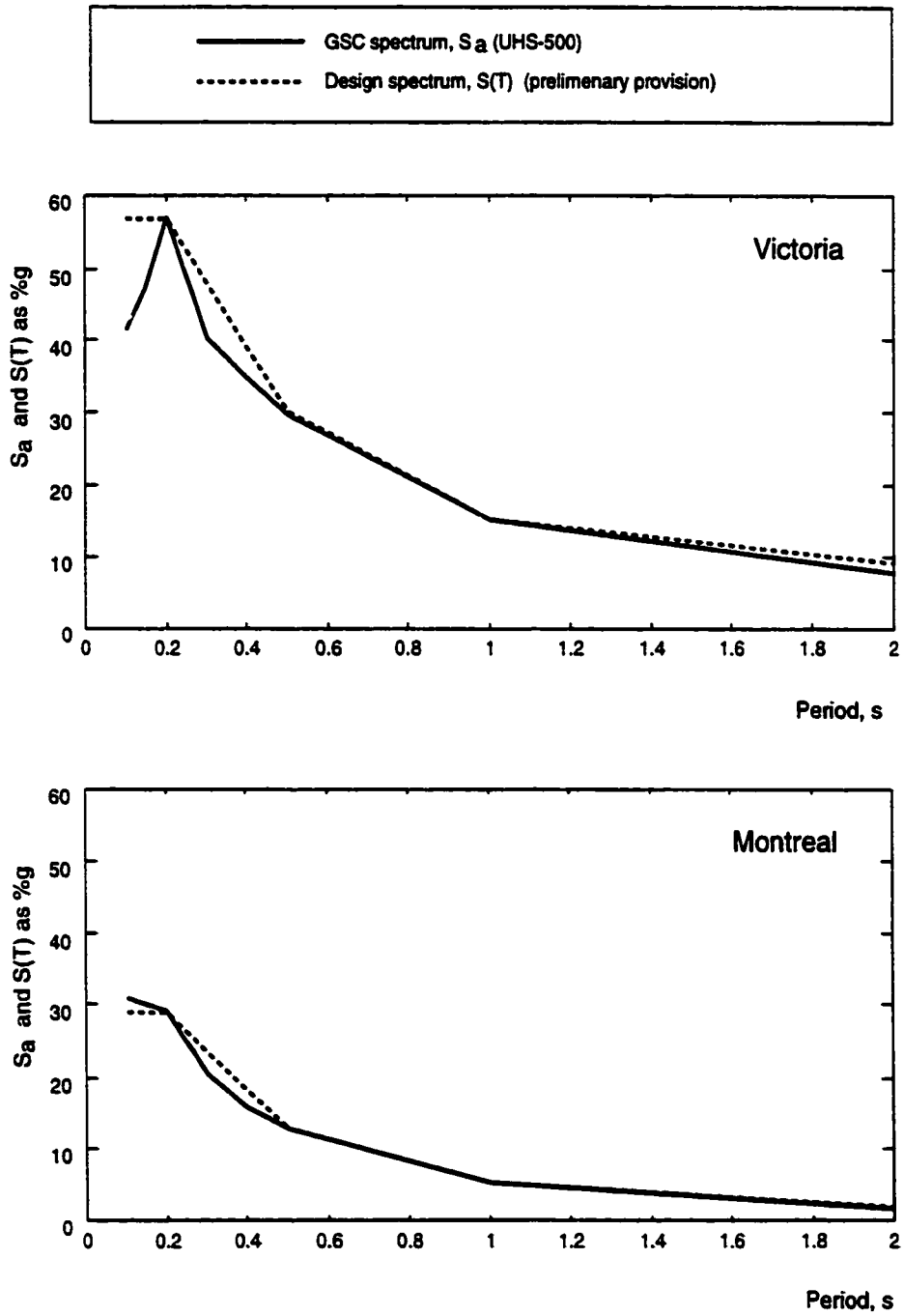


Figure 1.3: UHS-500 based spectral acceleration values (firm ground)

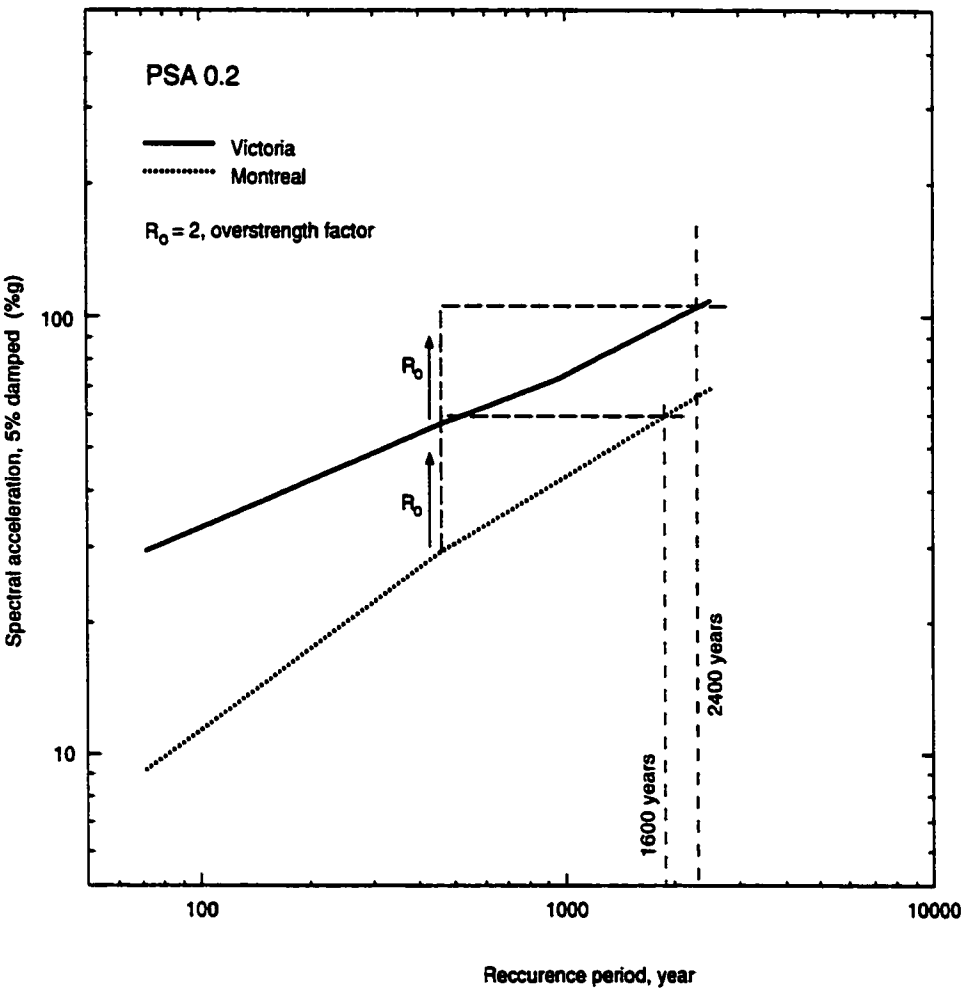


Figure 1.4: PSA 0.2 hazard curves for Victoria and Montreal showing how increasing the 10%/50 year hazard by a factor of 2 produces different level of safety

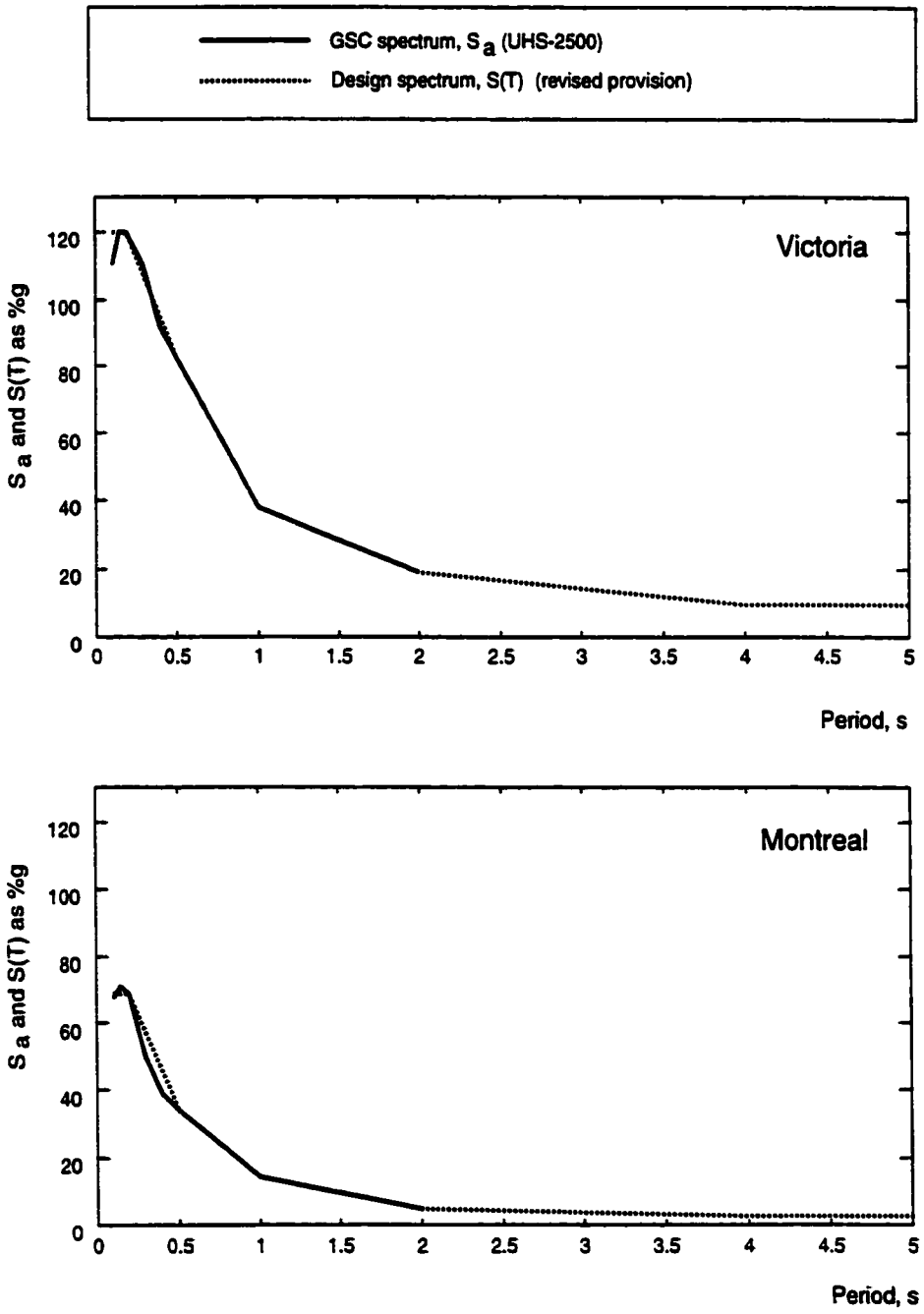


Figure 1.5: UHS-2500 based spectral acceleration values (firm ground)

Chapter 2

Methodology for performance evaluation

2.1 Structural damage parameters

Selection of appropriate damage parameters is very important for performance evaluation. Overall lateral deflection and inter-Storey drift are most commonly used damage parameters. Overall deflection is not always a good indicator of damage, but inter-Storey drift is quite useful because it is representative of the damage to the lateral load resisting system. Maximum values of member or joint rotations, curvature and ductility factors are also good indicators of damage because they can be directly related to the element deformation capacities. However, the maximum value alone of any of these parameters may not be sufficient to quantify the overall damage caused by cyclic reversal of deformation. Damage indices which take into account both the maximum deformation and cyclic effects have been developed for such cases. As an example, the damage index developed by Park and Ang (1985a) for reinforced concrete structures attempts to account for the damage caused by cyclic

deformations into the post-yield level. Park and Ang damage index can be calculated both at the element level and the global level (obtained by combining the weighted values of element level damage indices). Both indices can be used to measure the overall damage state of a structure. For materials other than reinforced concrete (eg. steel), a damage index similar to the Park and Ang damage index could be used in performance evaluation.

It is recommended by most building codes including NBCC that the seismic design of ductile moment resisting frames be based on the capacity design (weak beam and strong column) concept. This is ensured by strict strength and detailing requirements designed to avoid premature brittle failure modes. When subjected to severe ground motion, such structures show a great deal of ductility and the damage is generally distributed over the structure. Global damage index is a very useful measure of the damage in such structures. For practical application, a relation must be established between the damage indices and the damage as specified in qualitative terms (or in terms of performance level). Some commonly used damage parameters are briefly discussed in the following sections.

2.2 Displacement based damage parameters

Commonly used displacement based damage parameters are lateral drift or roof displacement, inter-Storey drift, member or joint rotations, curvature and ductility factors etc. Lateral drift and inter-Storey drift are very widely used parameters and are part of the direct output of any dynamic response analysis.

Lateral drift or roof displacement has only limited use for damage evaluation.

Since the damage in a structure depends quite a lot on the actual deformed shape of the structure, inter-Storey drift plays an important role in determining the extent of damage to columns during lateral deformation. Inter-Storey drift can also be used as a measure of non-structural damage. Maximum values of member or joint rotations, curvatures, ductility demand etc. are related to element deformation capacities, and can provide important information on the state of damage in the structure.

Although the maximum values of the displacement based damage parameters provide a good measure of damage, they do not account for the damage caused by cyclic reversal of deformation that occurs during earthquakes. Various energy based damage parameters are available to account for the cyclic reversal of deformation in the structural members.

2.3 Energy based damage parameters

2.3.1 Energy dissipation by a structure

The energy input, E_I, to a structure subjected to earthquake ground motion is dissipated in part by inelastic deformation (hysteretic energy, E_H), and in part by viscous damping. Only the hysteretic energy (E_H) is assumed to contribute to structural damage. The hysteretic to input energy ratio, E_H/E_I is an important response parameter which indicates the extent of damage in the structure. Fajfar *et al* (1992) introduced a dimensionless parameter γ, which represents the relation between hysteretic energy and the maximum displacement. This is an important energy based damage indicator.

$$\gamma = \frac{\sqrt{E_H/m}}{\omega D} \tag{2.1}$$

where E_H is the dissipated hysteretic energy, D is the maximum displacement, m is the mass of the system and ω is its natural frequency. The parameter γ also controls the reduction of displacement ductility due to low cycle fatigue.

2.3.2 Park and Ang damage index

The damage index suggested by Park and Ang (1985a) is an often used realistic damage indicators. It is defined as the linear combination of damage due to plastic deformation and hysteretic energy dissipation, and is given by

$$DM = \frac{D}{D_u} + \beta \frac{E_H}{F_y D_u} \tag{2.2}$$

where, D_u is the ultimate displacement under monotonic loading, F_y is the yield load, E_H is the energy absorbed by a structural element during the response history, and β is a constant which depends on the low cycle fatigue characteristics of the structural material. Value of β can be taken as 0.1 for reinforced concrete (Valles *et al* 1996).

The following three types of damage indices can be computed and used to define the damage level of a structure.

- *Local or element damage index:* damage index for individual structural elements such as, column, beam or shear wall elements.

- *Storey damage index:* total storey damage calculated by summing up the damage indices in the vertical and horizontal components in a storey.
- *Global or overall damage index:* the weighted sum of the damage indices of all individual elements.

The storey level damage and global damage indices are obviously functions of damages in the constituent elements. In a ductile structure with weak beams and strong columns, the damage is scattered over the structure. Any measure of overall damage should reflect the potential damage concentration on the weakest part of a building, as well as the distribution of damage in the building. Damage distribution is closely correlated with the distribution of absorbed energy. Thus, the overall damage index can be obtained as the sum of damage indices of the constituent elements (DM_i), weighted by the energy absorption factors, λ_i. The storey level damage index DM_s, and the global damage DM_g can be computed using the following expressions:

$$DM_s = \sum \lambda_i DM_i \; ; \qquad \lambda_i = E_H^i / (\sum E_H^i)_{(\text{storey})} \tag{2.3}$$

$$DM_g = \sum \lambda_s DM_s \; ; \qquad \lambda_s = E_H^s / \sum E_H^s \tag{2.4}$$

where λ_s is the storey level energy absorption factor, E_H^i is the energy absorbed by an element i, E_H^s is the total energy absorbed by all the elements in a storey s (ie., $(\sum E_H^i)$ for a storey s).

The Park and Ang damage model has been calibrated against observed structural

Table 2.1: Interpretation of the Park and Ang global damage index

Degree of damage	Physical appearance	Damage index limit
SLIGHT	Sporadic occurrences of cracking	< 0.1
MINOR	Minor cracks throughout the building, partial crushing of concrete columns	< 0.2
MODERATE	Extensive large cracks, spalling of concrete in weaker elements	< 0.5
SEVERE	Extensive crushing of concrete, disclosure of buckled reinforcements	< 0.9
COLLAPSE	Total or partial collapse of building	> 1.0

damage of nine reinforced concrete buildings. Park *et al* (1985b) classified the degree of structural damage into five categories: slight, minor, moderate, severe, and collapse. The damage index values associated with these degrees of damage in a structure are summarised in Table 2.1.

Park *et al* (1985b) have suggested that, in practical terms, a global damage index of 0.4 or less would represent a repairable damage in a structure, while a value more than 0.4 would mean that the damage is beyond repair. A value of 1.0 or greater of the global damage index represents total collapse.

The inelastic behaviour is usually confined to plastic zones near the ends of some members. In these cases, the relation between the element deformations and local plastic rotations is difficult to establish. For the element end section damage, the following modifications to the Park and Ang damage model were suggested by Kunnath

et al (1992).

$$DM = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_H \tag{2.5}$$

where θ_m maximum rotation during the loading history, θ_u is the ultimate rotation capacity of the section, θ_r is the recoverable rotation while unloading, and M_y is the yield moment. The largest value of the damage index of end sections is taken as the value of the element damage index. The modified Park and Ang damage index is used in the IDARC program (Valles *et al* 1996).

2.3.3 Equivalent ductility factor

Under cyclic loading beyond a certain level into the inelastic range, the strength of a structure deteriorates. Reduction in strength can be kept within a reasonable bound by limiting the amplitude of cyclic deformations. Cyclic loading leads to a reduction in the deformation capacity of a structure because of hysteretic energy dissipation. Fajfar *et al* (1992) have proposed a reduced ductility capacity which reflects the influence of cyclic response. This reduced ductility can be used, instead of the conventional monotonic ductility supply, in design procedures. Damage indices are included explicitly into the formula for determining the equivalent ductility factors. This would allow the designer to choose explicitly a damage limit state that is compatible with the return period of the design level of earthquake hazard. Using the park and Ang damage index (DM), the equivalent ductility factor is obtained as (Fajfar 1992),

$$\mu = \frac{\sqrt{1 + 4(DM)\beta\gamma^2\mu_u} - 1}{2\beta\gamma^2} \tag{2.6}$$

where μ_u is the ultimate monotonic ductility factor, and the other symbols have been defined earlier.

2.4 Damage parameters used in the current study

Following damage parameters are used in the current study for the evaluation of the performance of a building. These parameters give a quantitative measure of damage in a structural element which is then used to obtain a qualitative measure of damage in a building.

- Inter-storey drift and roof displacement or lateral drift
- Beam and column ductility demands
- Park-Ang damage indices (element level, storey level and global)

The following definition of ductility demand for a beam or column is used in this study: $\mu = \theta_m/\theta_y$, where θ_m is the maximum rotation at the end of a beam-column element, and θ_y is the rotation when yielding starts.

2.5 Analytical techniques of performance evaluation

The evaluation of seismic performance of any structure requires the estimation of its dynamic characteristics and the prediction of its response to the ground motion to which it could be subjected during its service life. The dynamic characteristics, namely the periods and mode shapes, are obtained through an eigenvalue analysis. Inelastic time history analyses provide the damage states of the building when it is subjected to various levels of ground motion. Static push-over analysis could be used to determine the lateral load resisting capacity of a structure and the maximum level of damage in the structure at the ultimate load.

2.5.1 Inelastic dynamic analysis

According to the seismic design philosophy followed by the current building codes, a structure is allowed to undergo deformation beyond yielding to absorb energy during severe earthquakes. A dynamic time history analysis using step by step integration is a very useful tool to determine the response, and hence the performance, of a structure subjected to a ground motion. Changes in material properties can be incorporated in this kind of analysis easily. Once a suitable model and a suite of appropriate ground motions are selected, the actual dynamic analysis to determine the dynamic response and damage parameters is relatively straight forward. The solution is carried out in incremental form using the following equation:

$$[M]\{\Delta\ddot{u}\} + [C]\{\Delta\dot{u}\} + [K_t]\{\Delta u\} = \Delta F_{eff} \tag{2.7}$$

where $[M]$ is the mass matrix of the structure, $[C]$ is the damping matrix, and $[K_t]$ is the tangent stiffness matrix; $\{\Delta u\}$, $\{\Delta \dot{u}\}$, $\{\Delta \ddot{u}\}$ are respectively the incremental vectors of displacement, velocity and acceleration in the structure; ΔF_{eff} is the vector of incremental effective dynamic forces.

2.5.2 Static push-over analysis

Static push-over analysis is basically a nonlinear static analysis which is performed by imposing an assumed distribution of lateral loads over the height of the structure and increasing the lateral loads monotonically from zero to the ultimate level corresponding to the incipient collapse of the structure. The gravity load remains constant during the analysis. The system of equations solved are:

$$[K_t]\{\Delta u\} = \Delta F \tag{2.8}$$

in which, ΔF is the vector of incremental forces. The push-over analysis is very useful in estimating the following characteristics of a structure:

- The capacity of the structure as represented by the base shear versus roof-displacement graph.
- Maximum rotation and ductility of critical members.
- The distribution of plastic hinges at the ultimate load.
- The distribution of damage in the structure, as expressed in the from of local damage indices, at the ultimate load.

- Determination of the yield lateral resistance of the structure.

The concept of yield lateral resistance could be very useful in approximately estimating the seismic response of a structure for the design purpose. Using the yield lateral resistance, the fundamental period, and inelastic spectra for single degree of freedom (SDOF) systems, the damage parameters for various strong motions earthquakes can be estimated.

The ultimate load-carrying capacity of a structure can be used to determine how much over-strength the structure has above the code specified strength requirement. Over-strength in a structure would undoubtedly affect its seismic performance and the overall level of seismic protection.

2.5.3 Simplified procedure for performance evaluation

The degree of damage that a specific building can undergo when subjected to major earthquake ground motions can be predicted through nonlinear time history analysis. However, nonlinear dynamic analysis of MDOF mathematical models is time consuming and is not practical for use in everyday design. To achieve a satisfactory balance between accuracy of analysis and applicability for everyday design, a simplified method that predicts the structural damage to an acceptable degree is necessary.

The idea of using a combination of static push-over analysis of the MDOF structure and nonlinear dynamic analysis of an equivalent SDOF structure for faster prediction of seismic damage for a given structure has been used by many authors (Fajfar

et al 1996, Mazzolini *et al* 1997). Here, a method similar to the N2 method developed by Fajfar *et al* (1996) is developed. The N2 method uses two separate mathematical models, one being the original MDOF model which is used for static push-over analysis, while the other is an equivalent SDOF model (based on the push-over analysis of the MDOF model) which is used for predicting the global damage of the actual structure for a given level of seismic hazard. The method proposed here is relatively simpler. Application of these simplified methods may be limited to building structures oscillating predominantly in the fundamental mode.

2.6 The UHS compatible ground motion records

Selection of appropriate earthquake records is very important. If actual earthquake records are used, only those records can be used whose spectra closely match the uniform hazard spectrum corresponding to the design earthquake level for the site of interest. As an alternative, the UHS compatible ground motion time histories developed by Atkinson *et al* (1998) can be used.

The UHS compatible ground motion time histories are briefly discussed in this section. These are simulated ground motion records. Generally, a simulated record, exactly matching hazard spectrum for a particular site, is composite in nature, and corresponds to simultaneous occurrence of a number of potentially damaging events. These kinds of simulated records are unrealistic. Atkinson *et al* (1998) and Atkinson (1997) have produced physically realistic time histories which not only match the hazard spectrum, but also are representative of motions for specified magnitude distance scenarios in the regions of interest. The seismic hazard spectra were gener-

ated by GSC (Adams 1997). An analysis of uncertainty was carried out during the development of those spectra, and suites of spectra corresponding to different levels of probability representing the confidence limits were developed. In the proposals under consideration for the next version of NBCC the spectra corresponding to the median or the 50th percentile confidence level would likely be used to define the seismic hazards map for Canada.

The uniform hazard spectrum can be treated as a composite of the types of earthquakes which contribute most to the hazard corresponding to a specified probability level. Generally, small earthquakes near the site are dominant contributors to the short period motions, and large earthquakes at a greater distance contribute more strongly to long period motions (Atkinson *et al* 1998). Taking these factors into consideration, a set of two ground motions was developed to match the uniform hazard spectrum for each particular site of interest. One of these ground motions corresponds to a low magnitude earthquake (M5.5, for example) at a short distance (say, R = 20 km), and is used to match the short period range (e.g., 0.1 to 0.5 seconds band) of the spectrum. The other ground motion corresponds to a high magnitude earthquake (say, M7.0), but far away from the site (say, R = 70 km), and is matched with the long period range (e.g., 0.5 to 5.0 seconds band) of the spectrum.

Atkinson *et al* (1998) have suggested that the 1/500 p.a. spectra for eastern Canadian sites can be adequately represented by an event of M5.5 for short period and an event of M7.0 for long period range of the hazard spectra. The distances at which these events are placed depend on the seismicity rate of that site area. Similarly, for western Canadian sites, M6.0 and M7.2 events can be used to match respectively the

short and long period ranges of the spectra. The Cascadia Subduction Zone effect for western Canadian sites was treated separately and an M8.5 event (rather than M8.2 used by GSC) was used to match the corresponding UHS. The spectra corresponding to the 1/500 p.a. for crustal earthquake events for Victoria and Montreal are shown in Figs. 2.1. Note that, each of the simulated spectra is scaled by a factor called, fine-tune factor (FTF), so as to match the target spectra as closely as possible. This fine-tune factor is equivalent to adjusting the number of standard deviations from the median. For the 1/1000 p.a. spectra, the same magnitudes of earthquake events as for the 1/500 p.a. spectra, can be used, but the corresponding distances or the fine-tune factors need to be adjusted. The spectra for 1/1000 p.a. events are shown in Fig. 2.2.

The UHS for 1/2500 p.a. spectra are approximated, in the case of eastern Canadian locations, by M6.0 events for short and M7.0 events for long period ranges, and in the case of western Canadian locations, by M6.5 events for short and M7.2 events for long period ranges. The spectra for 1/2500 p.a. events are shown in Fig. 2.3. Table 2.2 shows the magnitude-distance and fine-tune factors for all the three levels of seismic hazard in Victoria and Montreal.

Atkinson *et al* (1998) generated two sets of ground motion records (Trial-1 and Trial-2) for 1/500 p.a and 1/1000 p.a. events, and four sets of records for 1/2500 p.a. events. Each set contain two records, one for the short period range and the other for the long period range. These records (4 for 1/500 p.a. and 1/1000 p.a. events, and 8 for 1/2500 p.a. events) were generated using the point-source simulation of crustal, and in some cases, subcrustal earthquake sources. For western

Table 2.2: Magnitude, distance and fine tune factors (FTF) for the UHS compatible motions

Hazard level	Period-range	Victoria			Montreal		
		Magnitude	Distance	FTF	Magnitude	Distance	FTF
UHS-500	Short	6.0	20	1.10	5.5	30	0.80
	Long	7.2	70	0.80	7.0	150	0.70
UHS-1000	Short	6.0	20	1.40	5.5	20	0.80
	Long	7.2	70	1.00	7.0	100	0.80
UHS-2500	Short	6.5	30	1.20	6.0	30	0.85
	Long	7.2	70	1.25	7.0	70	0.90

Canadian sites, additional records corresponding to the Cascadia Subduction Zone scenario were developed using finite-fault simulation method. Sample time histories corresponding to 1/500 p.a. spectra for Victoria are shown in Figs. 2.4. Table 2.3 shows a summary of peak ground acceleration, a_p in $\%g$ and duration, T_d of the UHS compatible time histories. Time step is 0.01 second for all the records. Apart from the time histories mentioned in Table 2.3, there are additional time histories for Victoria which correspond to the Cascadia Subduction Zone effect. Those time histories are not considered here.

2.7 Computer modelling and analysis tools

Modelling a structure and representing it in a computer program for suitable analysis are important steps in performance evaluation. Several well established computer programs, which can be used in the modelling and analysis of a structure to evaluate its seismic performance, are available. In this section, four programs, IDARC (Valles *et al* 1996), DRAIN-2DX (Prakash *et al* 1993), DRAIN-RC (Alsiwat 1993, Shoostari

Table 2.3: UHS based ground motion time histories

Victoria						
	UHS-500		UHS-1000		UHS-2500	
Description	T_d (s)	a_p (%g)	T_d (s)	a_p (%g)	T_d (s)	a_p (%g)
Long, Trial 1	19.66	7.99	19.66	9.99	18.18	29.54
Short, Trial 1	6.00	23.17	6.00	29.49	8.53	66.70
Long, Trial 2	19.66	7.66	19.66	9.56	18.18	31.11
Short, Trial 2	6.00	25.11	6.00	31.96	8.53	67.13
Long, Trial 3					18.18	27.63
Short, Trial 3					8.53	72.31
Long, Trial 4					18.18	30.23
Short, Trial 4					8.53	48.42
Montreal						
	UHS-500		UHS-1000		UHS-2500	
Description	T_d (s)	a_p (%g)	T_d (s)	a_p (%g)	T_d (s)	a_p (%g)
Long, Trial 1	24.87	11.15	24.99	13.50	24.07	27.09
Short, Trial 1	7.82	13.95	5.90	30.48	8.88	36.58
Long, Trial 2	24.87	9.55	24.99	11.99	24.07	25.73
Short, Trial 2	7.82	16.98	5.90	29.29	8.88	44.39
Long, Trial 3					24.07	30.78
Short, Trial 3					8.88	39.96
Long, Trial 4					24.07	26.24
Short, Trial 4					8.88	37.34

1997), and COLA (Yalcin 1996) will be discussed.

IDARC is an integrated program for the inelastic dynamic analysis of concrete structures. The program can generate the moment-curvature envelopes for the reinforced concrete elements internally, if desired. If that option is adopted, the user has to provide information about the dimensions and reinforcement details of the members. DRAIN-2DX and DRAIN-RC programs do not have such an option. The moment-curvature envelopes for beams and columns must be supplied by the user for both DRAIN-2DX and DRAIN-RC models. COLA, a column analysis program, is used for generating moment-curvature envelopes, moment-axial force interaction curves etc. for reinforced concrete beam-column elements.

2.7.1 IDARC

IDARC was originally developed as a platform for nonlinear analysis of two-dimensional reinforced concrete frame structures. In IDARC various aspects of concrete behaviour can be modelled and tested. Most structural elements available in IDARC are modelled using the same basic formulation. Flexural and shear deformations are coupled in the spread plasticity formulation used in IDARC. Axial deformations are modelled using linear spring elements, which are not coupled with flexural or shear spring elements. To account for the effect of finite size members on the joint behaviour rigid end zones can be specified. The user can select the length of the rigid zones depending on the dimensions of the connecting members. The stiffness matrix is varied throughout the analysis depending on the inelastic deformations.

The moment-curvature envelops for columns, beams, and shear walls are very

important in the analysis. For reinforced concrete elements, IDARC provides a option for the user to specify the section dimensions and reinforcement details directly, and the program calculates the moment-curvature envelopes internally. A trilinear moment curvature envelop based on cracking, yield, and ultimate moments calculated by the program is used for concrete elements, unless the moment curvature envelope is explicitly defined by the user. The detailed derivation of the moment-curvature envelops for reinforced concrete elements, using a model called fiber model, are given in Valles *et al* (1996). Alternatively, the user can explicitly specify these relations (this is necessary if a structural material other than reinforced concrete is used). The current release of the program (version 4.0) includes the following types of structural elements.

1. **Column elements:** Column elements are modelled considering flexural, shear, and axial deformations. A typical column element has six degrees of freedom (three at each end).

2. **Beam elements:** Beam elements are modelled as flexural elements with shear deformations coupled. No axial deformation is considered. This element has four degrees of freedom (two at each end).

3. **Shear wall elements:** Shear wall elements are modelled considering flexural, shear, and axial deformations. A typical shear wall element has six degrees of freedom (three at each face along the vertical axis).

4. **Edge column elements:** Edge columns are monolithically connected to the shear wall elements. The behaviour of these columns is mainly dependent on

the deformation of the shear wall. This element can also be used to model other transverse elements, such as secondary shear walls that can be lumped with the corresponding column elements.

5. **Transverse beam elements:** Although the modelling of the structure is done using planar frames, it may be necessary to include the effects of strong transverse beams to obtain a realistic frame behaviour. Transverse beams are modelled in IDARC by two springs, one to provide resistance to relative vertical motions, and the second, a rotational spring to provide resistance to relative angular motion. Both springs are considered linear elastic.

6. **Rotational spring elements:** Discrete inelastic springs may be connected to beam or column ends to simulate a flexible or semi-rigid connection in the joint. In general, more than one spring may be specified at the same location, provided the number of springs used in a particular joint is less than the number of elements framing into it. The stiffness of rotational springs may be varied from a small quantity to simulate a hinge, to a large quantity to simulate a rigid connection.

7. **Visco-elastic damper elements:** Dampers are energy dissipating devices, which are used in a building for an improved seismic performance. These devices can be broadly classified as, viscous dampers, friction dampers, and hysteretic dampers. Viscous dampers exhibit an important velocity dependency. There are several kinds of viscous dampers, namely, (a) visco-elastic elements, (b) viscous walls, and (c) fluid viscous dampers. In IDARC program, visco-elastic dampers are modelled using Kelvin and Maxwell models. The Maxwell model

is recommended when the damper exhibits a strong dependency on the loading frequency.

8. **Friction damper elements:** This type of devices dissipate input energy through frictional work. Several friction dampers, or friction like devices, such as, (a) friction devices, (b) lead extrusion devices, and (c) slotted bolted connections can be modelled in IDARC. These dampers are modelled using a Wen-Bouc model without strength or stiffness degradation.

9. **Hysteretic damper elements:** Hysteretic dampers dissipate energy through inelastic deformation of the device components. Several types of hysteretic dampers have been introduced: (a) yielding steel elements, (b) shape memory alloys, and (c) eccentrically braced frames. Most of these devices are modelled using a Wen-Bouc model without strength or stiffness degradation.

10. **Infill panel elements:** The infill panel elements included in IDARC are modelled using compression struts and smooth hysterectic model. Various types of infill panels, including masonry infill panels can be modelled using this element, by changing the values of control parameters in the smooth hysteretic model.

11. **Moment releases:** A perfect hinge can be modelled as an end spring with zero stiffness. However, that may cause numerical instability due to singular stiffness matrices. Therefore, a perfect hinge is modelled by setting the hinge moment to zero and condensing out the corresponding degree of freedom.

Hysteretic behaviour of structural elements is one of the most important aspects in nonlinear analysis. The current version of IDARC program includes a number of

hysteretic response curves. Columns, beams, shear walls, and rotational springs can be modelled using (a) a three parameter Park model, or (b) a trilinear steel model, or (c) a bilinear model. Visco-elastic dampers are modelled either using a Kelvin or a Maxwell model, while friction dampers and hysteretic dampers are modelled using the Wen-Bouc model, and infill panels are modelled using the smooth hysteretic model. All these hysteretic models are discussed in Valles *et al* (1996). IDARC program can calculate the nonlinear response of a structure with the following four analysis options. The user may choose any of these four options, or a combination of a nonlinear static analysis with any of the other three options.

1. Nonlinear Static Analysis
2. Nonlinear Pushover Analysis
3. Nonlinear Dynamic Analysis
4. Nonlinear Quasi-static Analysis

In IDARC, static loads can be specified as distributed loads in beam, or as concentrated forces, or moments in the joints. The nonlinear static analysis is typically carried out in a number of loading steps. The nonlinear push-over or collapse mode analysis option performs an incremental analysis of the structure subjected to a given distribution of lateral forces. The push-over analysis can be carried out using force control or displacement control. In the force control option, the following distributions of lateral forces are available in the program: (a) uniform distribution, (b) inverted triangular distribution, (c) generalised power distribution, and (c) modal adaptive distribution.

The uniform distribution considers a constant distribution of the lateral forces throughout the height of the building. The inverse triangular distribution, often suggested in building codes, considers a linear distribution of the lateral forces. The generalised power distribution considers the variation of lateral forces with storey heights, and tries to capture different modes of deformation as well as the influence of higher modes in the response. The modal adaptive distribution is significantly different from the others discussed above. Distribution of lateral forces is often selected based on the distribution during an elastic response. However, elastic distribution of lateral forces may not be valid when a structure enters the inelastic range. If the distribution of lateral forces is not modified to account for the new stiffness distribution, the structure may respond in a significantly different way from the actual response under an earthquake. The modal adaptive distribution was developed to capture the changes in the distribution of lateral forces. Mode shapes are used in this distribution. Since the change in the stiffness matrix will change the mode shapes, the distribution of lateral forces would be changed accordingly.

The nonlinear analysis is carried out using a combination of Newmark-Beta integration method, and pseudo-force formulation. The solution is carried out in incremental form. By default, IDARC uses an unconditionally stable average acceleration method for numerical integration, but the user can opt for a linear acceleration method, if desired. The viscous damping matrix is calculated in the program using one of the following options: (a) mass proportional damping, (b) stiffness proportional damping, and (c) Rayleigh damping.

Nonlinear quasi-static analysis is basically used to test the performance of a struc-

tural component or sub-assemblage under cyclic loading. The history of cyclic loads may be applied using force or deformation control. In IDARC, the same set of equations as that used in push-over routine, is used for quasi-static routine.

The P-Delta effect can be considered in analysis options discussed above. However, in the current version of the program, the P-Delta option works only for the nonlinear dynamic analysis option, not for any other available analysis options. The inelastic deformation in flexural members is not assumed to be concentrated at the ends of the member in the form of plastic hinges. Distribution of inelastic deformation is modelled using spread plasticity and yield penetration models. The other features of IDARC include the computation of dynamic characteristics of a structure by eigenvalue analysis, damage analysis using the modified Park and Ang damage model, and determining structural response at selected instants during analysis (this is called structural response snapshots in IDARC terminology). Several types of response snapshots can be specified: (a) displacement profile, (b) element stress ratios, (c) structural collapse state, (d) damage indices, and (e) dynamic characteristics (eigenvalue analysis). Response snapshots can be requested by the user during push-over, quasi-static, or dynamic analysis.

2.7.2 DRAIN-2DX

DRAIN-2DX is a general purpose program for the dynamic analysis of planer building frames. It does not have any specialized elements for modelling reinforced concrete members. In DRAIN-2DX, a structure is modelled as a two dimensional assemblage of nonlinear elements connected at the nodes. Each node has three degrees of freedom

(x-translation, y-translation and rotation). The elements are divided into groups according to their types. The structure mass is lumped at the nodes and thus, the mass matrix is diagonal. A viscous damping matrix, C can be specified as a combination of mass proportional damping and stiffness proportional damping, in the form, $[C] = \alpha[M] + \beta[K]$, where $[M]$ and $[K]$ are mass and stiffness matrices respectively, and α and β are the specified damping constants. The program is capable of performing a number of nonlinear static and dynamic analyses, including modal analysis and time history analysis. Loading can be specified in the form of static element or nodal load patterns, ground acceleration or displacement records, force records, or response spectra.

Using DRAIN-2DX, a structure can be analysed for several different loadings. Analyses are carried out in a number of "analysis sections". In any analysis section, analysis can be performed in a number of "analysis segments", each segment consisting of either a static load increment or the application of dynamic load for a period of time. In any analysis session, the beginning state for the first segment can be any previously saved state identified by the analysis segment number, except for a static gravity analysis segment, in which the beginning state must be the initial state. Following types of analysis can be performed using DRAIN-2DX.

1. *Gravity:* Static analysis for combined element and nodal loads. The structure is assumed to remain elastic.

2. *Static:* Nonlinear static analysis for nodal loads only.

3. *Restore Static State:* At the end of dynamic analysis, the structure can be

brought back to static equilibrium by this analysis.

4. *Mode Shapes and Periods:* Calculations of mode shapes and periods based on the initial state or any other state.

5. *Response Spectrum:* Linear analysis for specified response spectra using mode shapes and periods calculated for initial state.

6. *New Ground Acceleration:* Nonlinear dynamic analysis for specified ground acceleration records.

7. *Resume Ground Acceleration:* Continuation of the preceding ground acceleration analysis for an additional time segment.

8. *New Ground Displacement:* Nonlinear dynamic analysis for specified ground displacement records.

9. *Resume Ground Displacement:* Continuation of the preceding ground displacement analysis for an additional time segment.

10. *New Dynamic Force:* Nonlinear dynamic analysis for specified force records.

11. *Resume Dynamic Force:* Continuation of the preceding dynamic force analysis for an additional time segment.

12. *New Initial Velocity:* Nonlinear dynamic analysis for specified initial modal velocities.

13. *Resume Initial Velocity:* Continuation of the preceding initial velocity analysis for an additional time segment.

P-Delta effects can be considered in these analyses, if desired. Energy calculations can also be performed for both static and dynamic analyses. For dynamic analysis, a constant or a variable time step can be used. The following types of elements are available in DRAIN-2DX for modelling a 2D structure.

1. **Inelastic truss element:** This is a simple inelastic bar element which can be used for modelling truss members, simple columns, and nonlinear support springs.

2. **Plastic hinge beam-column element:** This is a simple inelastic beam element for modelling beam-columns. Plastic deformations are assumed to occur only at the two ends of each element. Rigid zones at the ends of each element can be specified.

3. **Simple connection element:** This is an inelastic element for modelling structural connections with rotational and/or torsional flexibility.

4. **Elastic panel element:** This is a simple elastic (linear) element for modelling rectangular panels with extensional bending and/or shear stiffness.

5. **Compression/tension link element:** This is an inelastic bar element that resists only axial force, and can be used to model a cable prestressed in tension, a bearing element with initial gap, a bearing element prestressed in compression.

6. **Fiber beam-column element:** This is a complex inelastic beam-column element that can be used to model steel, reinforced concrete, or steel concrete composite elements. Although this is a very sophisticated element, implemen-

tors have not studied its performance in detail, thus caution must be exercised while using this element.

2.7.3 DRAIN-RC

DRAIN-2D (Kannan *et al* 1983), an earlier version of DRAIN-2DX, was modified at the University of Ottawa to include a reinforced concrete beam element and an infill panel element, and this modified version of DRAIN-2D is named DRAIN-RC (Alsiwat 1993, Shoostari 1998). DRAIN-2D had a beam element with degrading stiffness characteristics suitable for modelling reinforced concrete element. Following features were added to the DRAIN-RC program.

- Hysteretic models for shear and anchorage slip
- A revised flexural hysteretic model to incorporate moment-axial force interaction
- Hysteretic model for masonry infill panels
- Push-over inelastic analysis and $P - \Delta$ effect

DRAIN-RC uses modified Takeda's hysteretic model for reinforced concrete in flexure (Fig. 2.5). Hysteretic anchorage slip model developed by Saatcioglu *et al* (1987) and Alsiwat (1993), and the hysteretic model for shear developed by Ozcebe and Saatcioglu (1989) have been incorporated in DRAIN-RC. The hysteretic shear model implemented in DRAIN-RC is shown in Fig. 2.6. The hysteretic model of the strut element for infill panels used in DRAIN-RC is shown in Fig. 2.7.

With the capability to model reinforced concrete beam column elements and infill panels, DRAIN-RC becomes a very good choice for nonlinear static and dynamic analyses of reinforced concrete frames (bare as well as infilled). However, this program has some limitations. It does not have the capability to perform modal analysis, and the push-over curve can not be traced far beyond yielding when the stiffness of the system reduces significantly.

2.7.4 COLA

COLA is a computer program capable of performing inelastic anaysis of reinforced concrete columns under combined axial compression and monotonically increasing lateral loads (Yalcin 1996). This type of anaysis is required to establish the force-deformation characteristics under combined axial and lateral loads.

The software provides sectional moment-curvature analysis, anchorage slip analysis, and member analysis, including $P-\Delta$ effect. Figure 2.8 shows the derivation of the bilinear $M-\phi$ curve from the one generated by COLA. The simplified moment-axial force ($M-P$) relation obtained by using COLA is used directly in DRAIN-2DX and DRAIN-RC programs. A typical $M-P$ interaction curve is shown in Figure 2.9. This curve is characterized by (a) pure axial capacity: compression (P_{yc}) and tension (P_{yt}), (b) pure flexure capacity: positive (M_{y+}) and negative (M_{y-}), and (c) axial force and moments corresponding to the balanced conditions (Point A and B in Figure 2.9).

The program takes the geometric data, reinforcement details, material properties, and the level of axial force, as input, and based on the input data, it constructs the

concrete confinement model and stability model for compression reinforcement. The program first calculates moment and curvature at cracking, and beyond cracking, corresponding to the specifies value of extreme compression strain. Sectional anaysis continues until either tension steel fails or the extreme compressive fiber strain reaches a limiting value.

2.7.5 A comparative study

To compare the performance of the above mentioned computer programs, the programs are used to analyze a six storey frame, designed for a location in Victoria. Details of the building frame are given in Chapter 3 where an evaluation of the seismic performance of concrete moment resisting frames is presented. In this section, some results that provide a comparison of the performance of several computer programs are presented.

Program COLA is used to compute moment-curvature and moment-axial force relations for beams and columns. The effective value of EI and the yield moment etc. corresponding to a given level of material strength (factored or nominal) for each beam-column element are calculated from the moment-curvature relationship generated using COLA.

Static push-over analysis results are shown in Figs. 2.10 and 2.11, while several time history analysis plots of roof displacement are shown in Figs. 2.12 through 2.14. The results of push-over analyses show no change of behaviour on account of $P-\Delta$ effect when IDARC program is used. Evidently, IDARC is unable to account for $P-\Delta$ effect in push-over analysis. Secondly, when infill panels are used in the frame model,

IDARC program does not add the stiffness of the infill panels to the sytem stiffness, and consequently the period of the structure remains unchanged. The deformation in the infill panels are calculated from the deformation of the bare frame. This kind of treatment for infill panels is not suitable for the present study. For the reasons cited, IDARC is not used in the present study.

DRAIN-2DX is a very robust and reliable program. However, there are some limitations in the way in which DRAIN-2DX handles $P - \Delta$ effect. It uses the geometric stiffness expressions for truss element to update the stiffness of a beam column element. Thus the softening of the flexural stiffness due to the axial load is ignored. Although this is incorrect, it provides reasonably accurate results. The $P - \Delta$ effect is handled correctly in DRAIN-RC program, which uses the correct expressions for the geometric stiffness of a beam column element updating the flexural stiffness at each load step. However, it is evident from Figs. 2.10 and 2.11 that the differences in the results of push over analysis obtained by using DRAIN-2DX and DRAIN-RC are not very significant even when the $P - \Delta$ effect is included. DRAIN-RC is unable to trace the load-deformation curve in a push over analysis when the curve tends to become flat and the tangent stiffness of the system becomes very low. No such problem is associated with DRAIN-2DX. This is because the program uses a combination of load-control and displacement-control techniques in the push-over analysis to overcome numerical sensitivity when the strength of a structure reduces significantly because of yielding of many of its members.

Although the results of the time history analyses carried out with DRAIN-RC and DRAIN-2DX are quite comparable, some differences do exist. These differences can

be attributed to the built-in modelling differences in these programs (e.g., difference in hysteretic rules).

In the present study, DRAIN-2DX is used for modal analysis, while DRAIN-RC is used for inelastic dynamic analysis. Push-over analysis is carried out by both programs and the compatibility between DRAIN-2DX and DRAIN-RC models is checked by using the results of the push-over analysis of these two models. When $P-\Delta$ effect is not considered, the push-over curves obtained by DRAIN-RC should be similar to that by DRAIN-2DX. Another difference in the formullations used by thr two programs arises from the fact that while DRAIN-RC can consider inelastic shear deformation and anchorage slip, DRAIN-2DX considers elastic shear deformation and no anchorage slip. This may lead to differences in the results obtained from the two programs in the cases when shear deformation is significant or anchorage slip is considered. In the current study, anchorage slip is not considered. However, inelastic shear effect is considered.

The push-over curves for moment resiting curves tend to be flat in the post-yield region. As a result, DRAIN-RC has difficulty tracing the curves beyond yield. In addition, shear deformations are not important for moment-resisting frames. For thease reasons push-over curves obtained from DRAIN-2DX are more useful in these cases and are presented in this work. For shear wall frames, the push-over curves are not very flat in the post yield zone and DRAIN-RC can trace the entire curve. Also the effect of inelastic shear is important in these cases, and the results from DRAIN-2DX are not very accurate. For these reasons in the cases of shear walls push-over curves ontained from DRAIN-RC are presented here.

In carrying out the time history analysis the anchorage slip hysteretic model available in DRAIN-RC is not used for building models considered here since the values of anchorage slip parameters are not well defined and determination of these parameters needs a lot of care. The shear hysteretic model in DRAIN-RC is considered in the analyses of the shear wall models since the effect of inelastic shear is expected to be significant in such structures. The effect of inelastic shear is considered in in the moment resisting frames since it is not expected to be significant for such frames.

2.8 Processing of DRAIN-RC output results

DRAIN-RC program does not compute Park and Ang Damage Index. However, it reports the hysteretic energy absorption and plastic hinge rotation values. These parameters are used to separately compute the damage index for each elements in a frame. A computer program has been developed for this purpose. This programs reads all the output files produced by DRIN-RC and produces a report containing the maximum values of the following damage parameters.

- Interstorey drift at each storey level
- Beam and column ductility deamand at each storey level
- Roof displacement as a percentage of the height of the building
- Element, storey and global damage indices (Park & Ang)

When a building model is analyzed for a suite of ground motion hime history records, this program calculates the envelope values for each damage parameter men-

tioned above. A schematic diagram of the organization of the program is given in Fig. 2.15. Information about the geometry of the frame and material properies of the elements such as, the yield and ultimate deformation capacities etc. are supplied as input to the program. The program then reads the following output files of DRAIN-RC inelastic dynamic analysis corresponding to each ground motion record and precess them. Envelope values are calculated after all the results corresponding to a given level of seismic hazard are precessed.

- Output file with extension *nde* containing time history and results envelopes of nodal displacements in for specified nodes.
- Output file with extension *dcd* containing ductility demand results.
- Output file with extension *egd* containing dissipated energy factors.
- Output file with extension *hgr* containing hinge rotation results.

2.9 Evaluation of seismic performance

The methodologies used for the evaluation of seismic performance of a building are discussed earlier in this chapter. The scheme used for the evaluation of performance of buildings studied here is briefly explained in this section.

The capacity of each frame is determined through a static push over analysis. The $P-\Delta$ effect is considered in the push over analysis. The following load combination is used in the push-over analysis: $(1.0D + 0.5L + 1.0E)$, where E represents the earthquake forces, D and L represent dead load and live load effects, respectively.

The seismic performance is then determined through a series of inelastic dynamic analysis using DRAIN-RC computer program. Each frame is analyzed for three different levels of earthquake hazards. These levels correspond to earthquake return periods of 475 years (UHS-500), 970 years (UHS-1000), and 2500 years (UHS-2500). The spectra for these hazard levels were obtained from Geological Survey of Canada (Adams 1997) and the compatible ground motion time histories were generated by Atkinson (1997), Atkinson and Beresnev (1998). Four records corresponding to each of the UHS-500 and UHS-1000 events, and 8 records corresponding to UHS-2500 event are available.

Each frame is analyzed for all the ground motion time history records for a given level of seismic hazard. Push over analysis is used for determining the lateral load-resisting capacity of a structure and the envelopes of damage parameters are used for determining its seismic performance. Material strength corresponding to factored as well as nominal values are considered in the analyses.

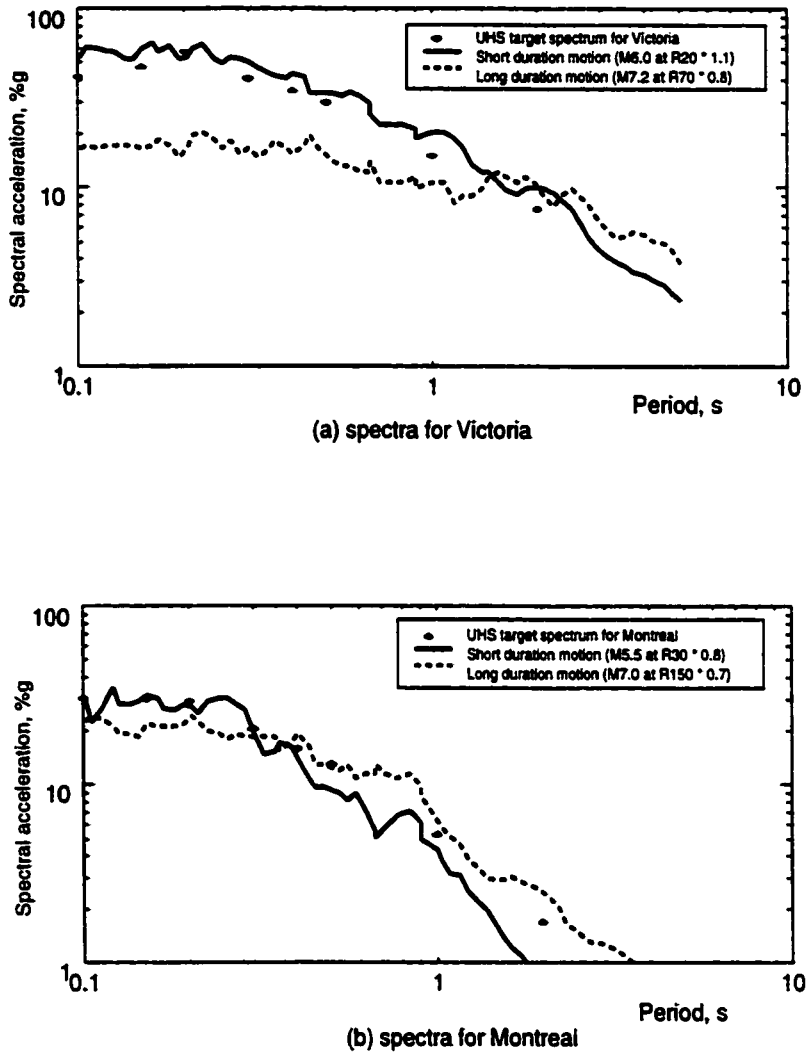
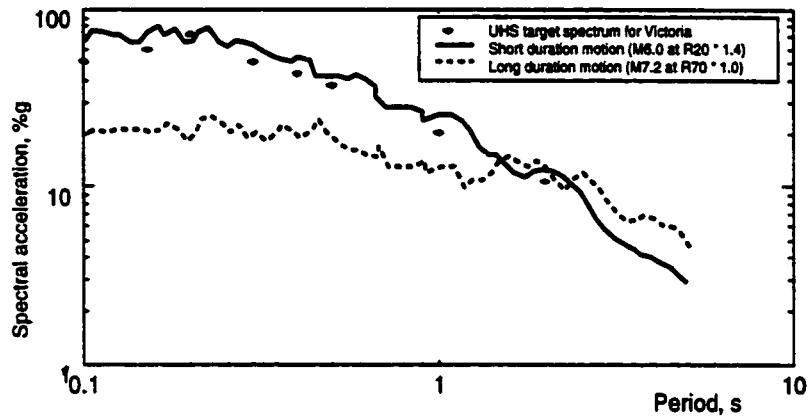
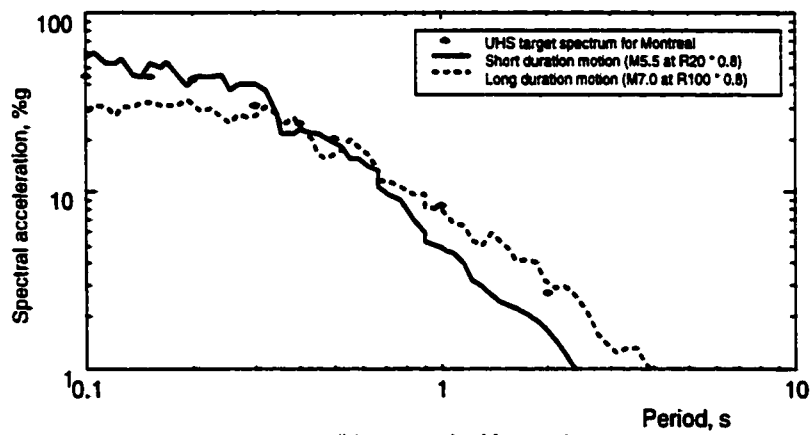


Figure 2.1: 1/500 p.a. UHS and corresponding spectra of compatible ground motions

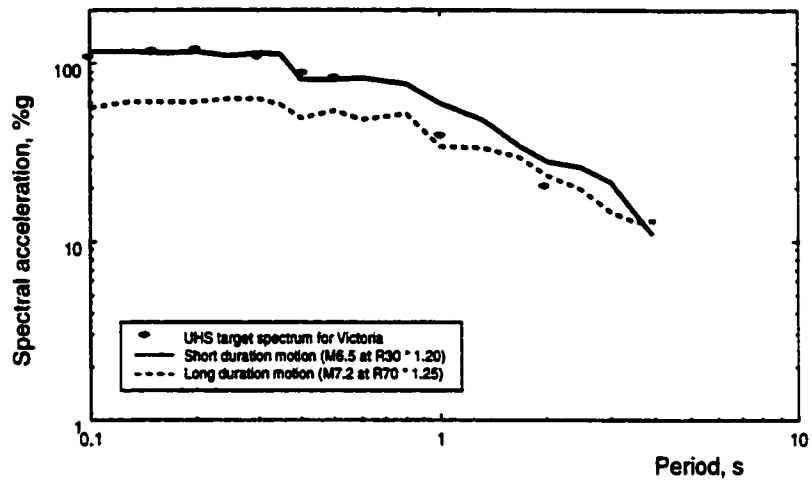


(a) spectra for Victoria

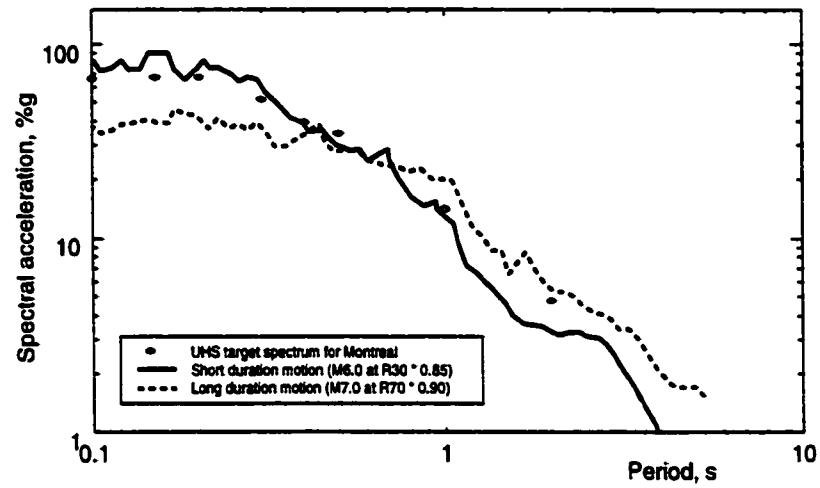


(b) spectra for Montreal

Figure 2.2: 1/1000 p.a. UHS and corresponding spectra of compatible ground motions



(a) spectra for Victoria



(c) spectra for Montreal

Figure 2.3: 1/2500 p.a. UHS and corresponding spectra of compatible ground motions

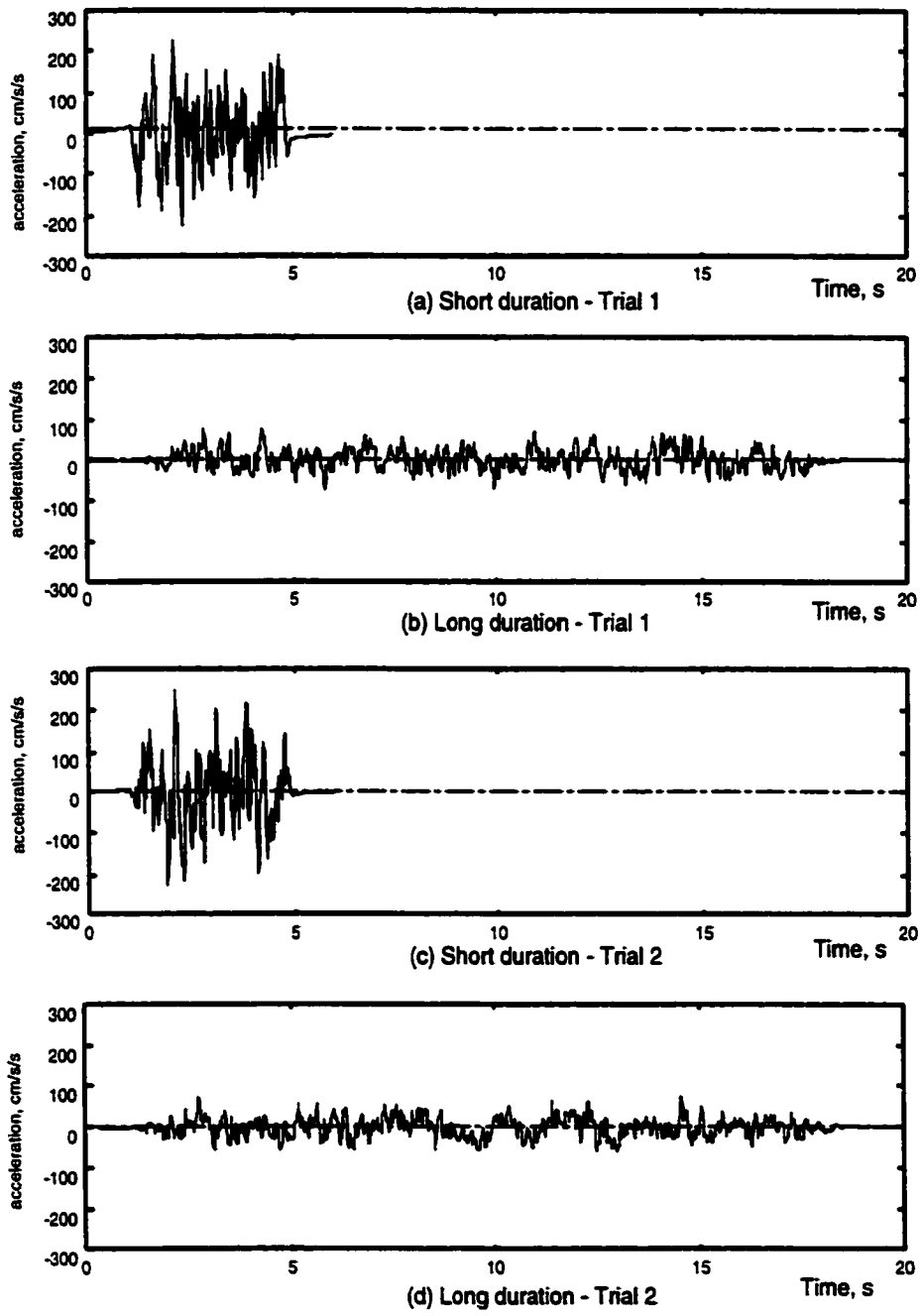


Figure 2.4: Sample UHS compatible time histories for 1/500 p.a. earthquakes in Victoria

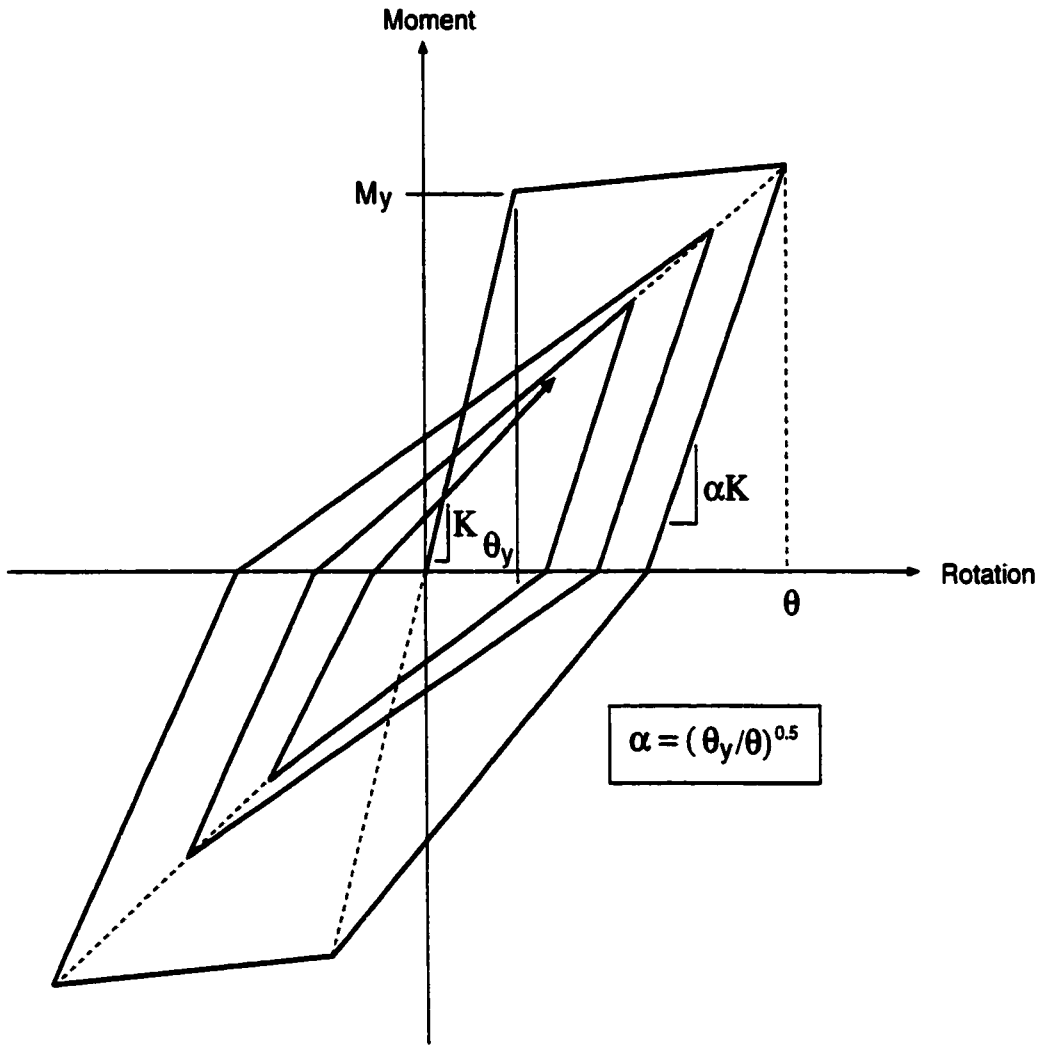


Figure 2.5: Modified Takeda's hysteretic model for reinforced concrete

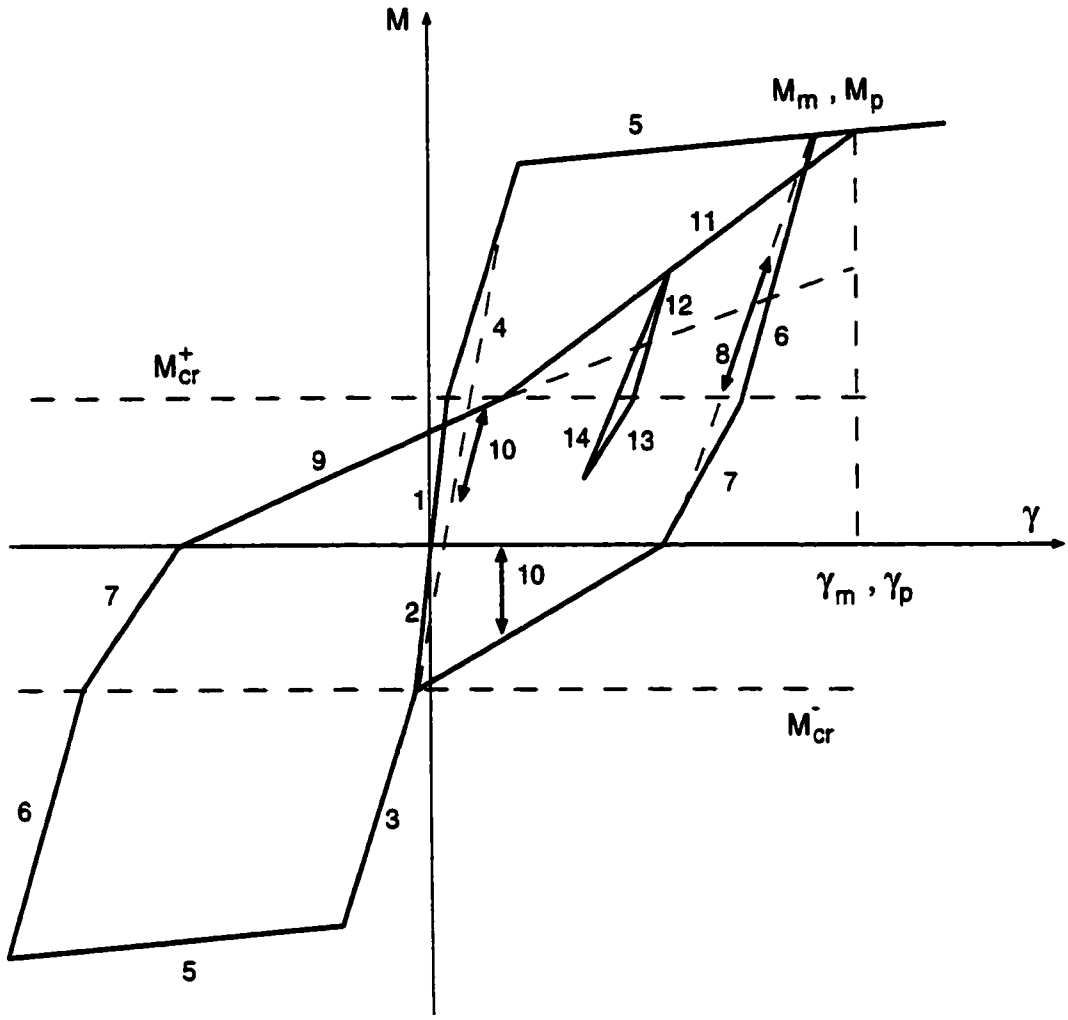


Figure 2.6: Shear hysteretic model for reinforced concrete

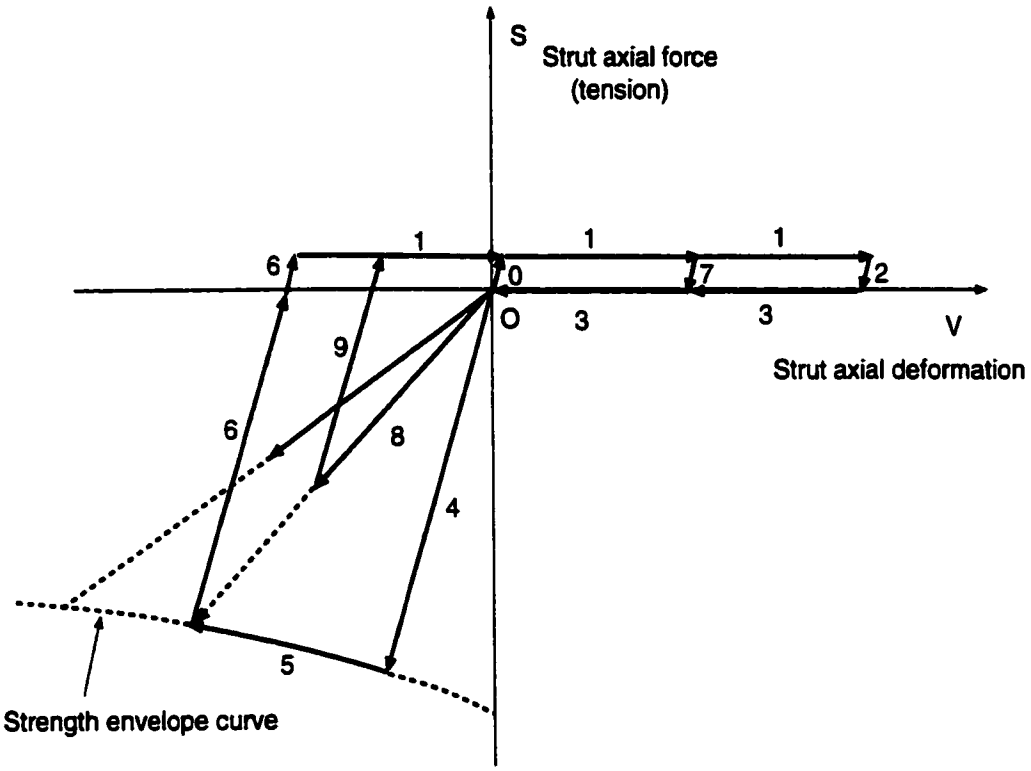


Figure 2.7: Hysteretic model of strut element for infill panel

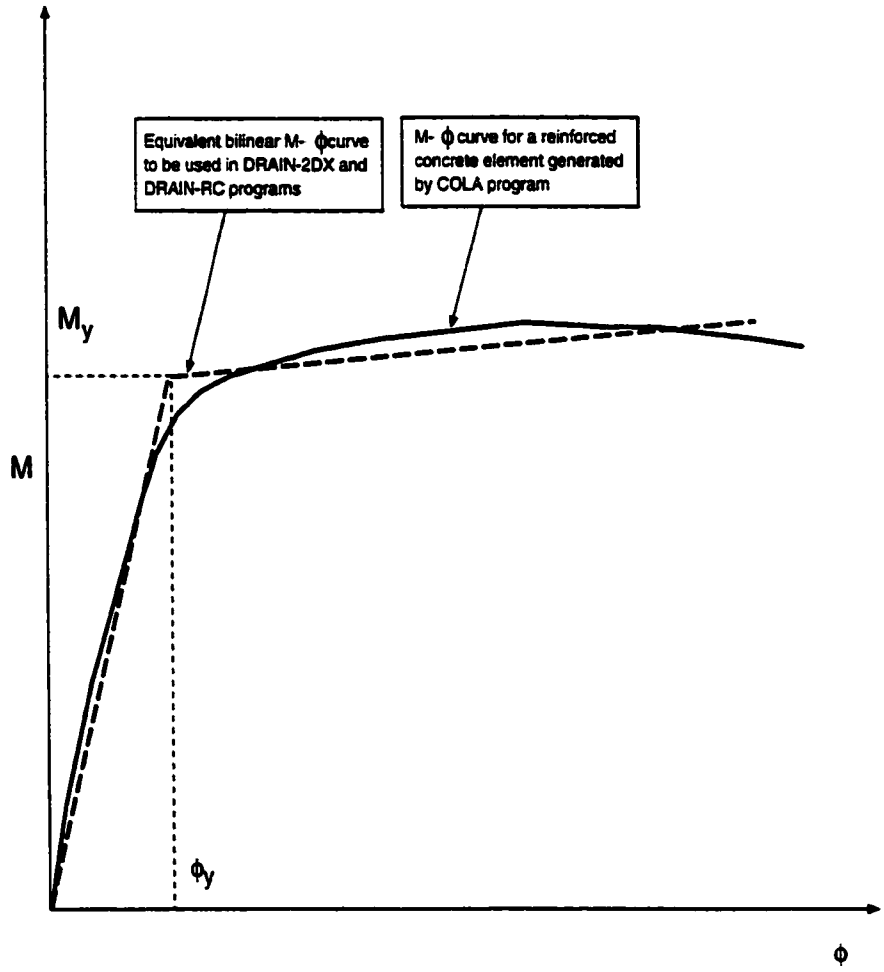


Figure 2.8: Derivation of an equivalent bilinear $M - \phi$ curve from the $M - \phi$ curve generated by COLA

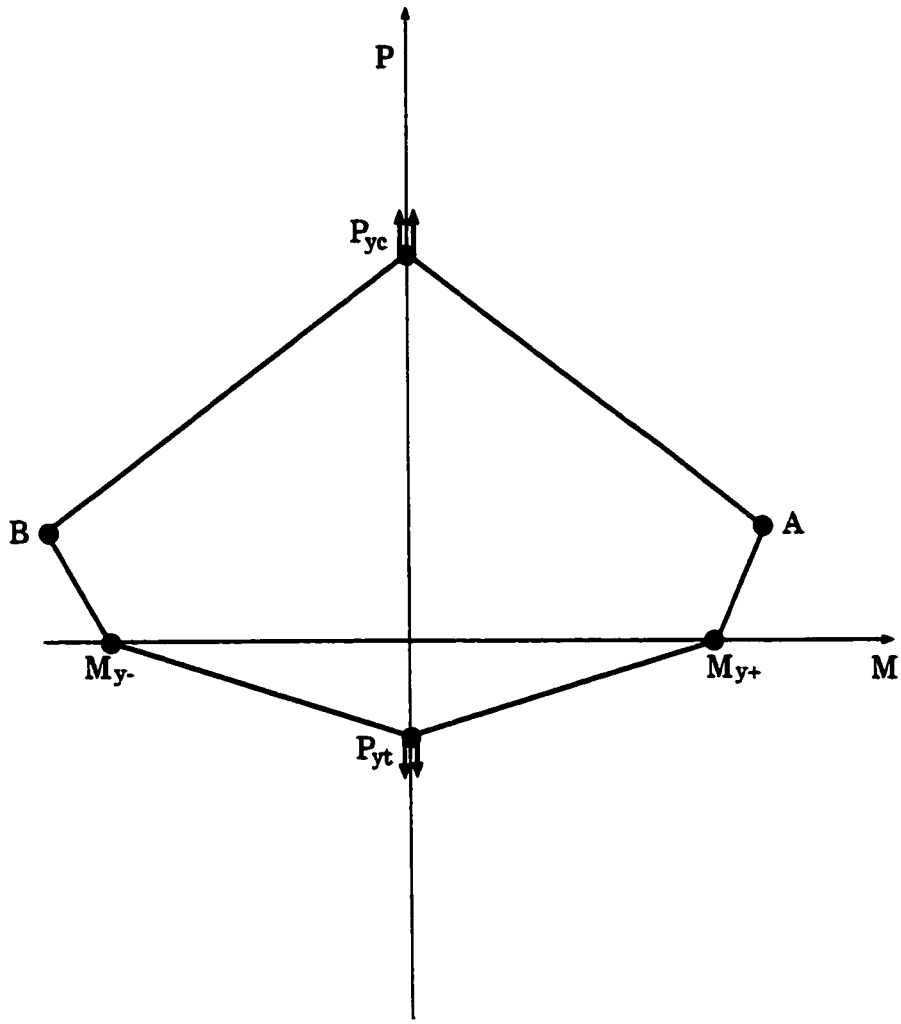


Figure 2.9: Simplified moment-axial force interaction curve for a beam-column element

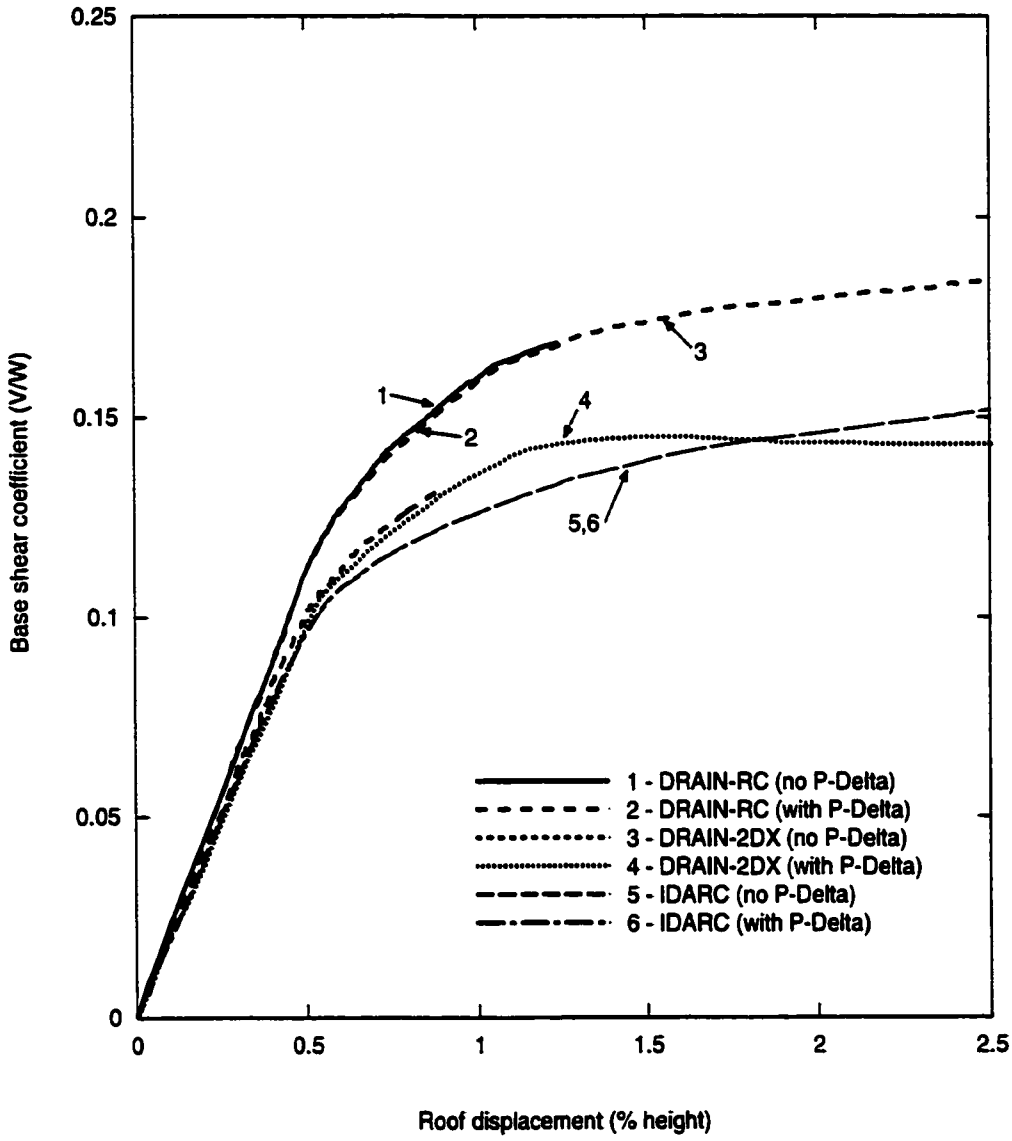


Figure 2.10: Push-over curves for the six storey bare frame in Victoria: factored material strength

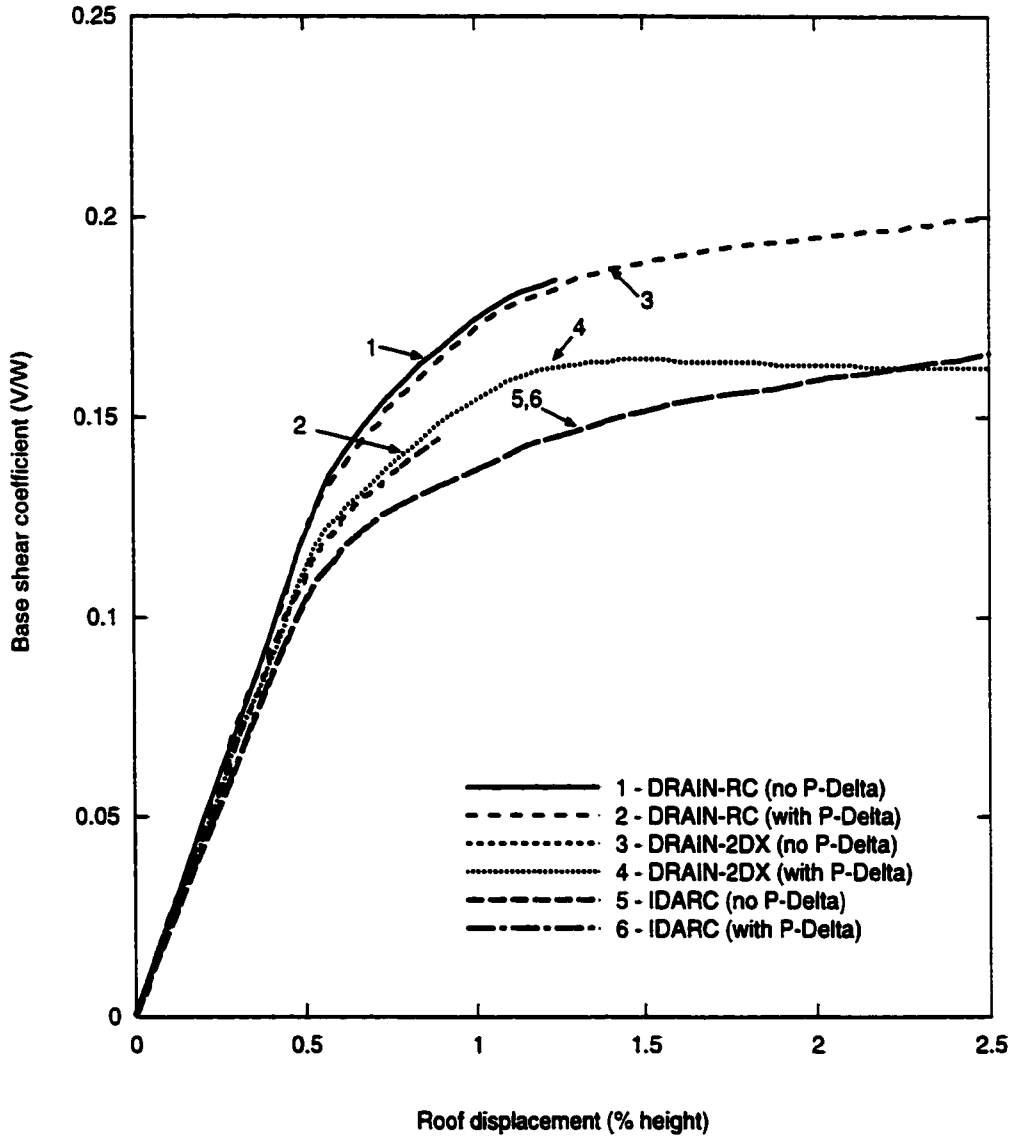
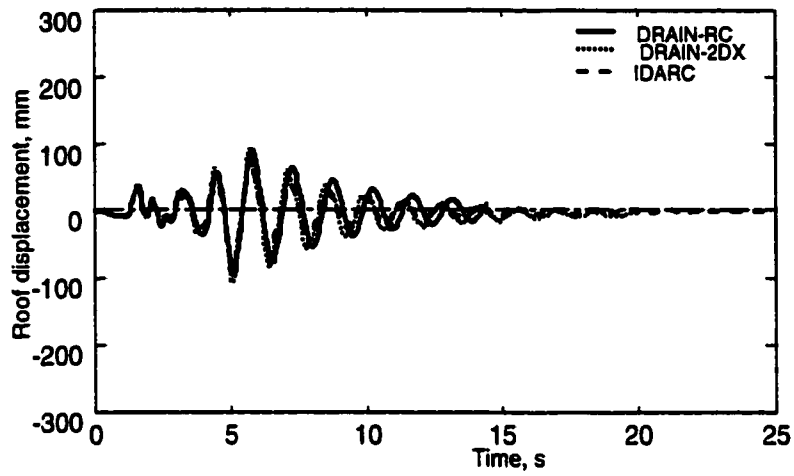
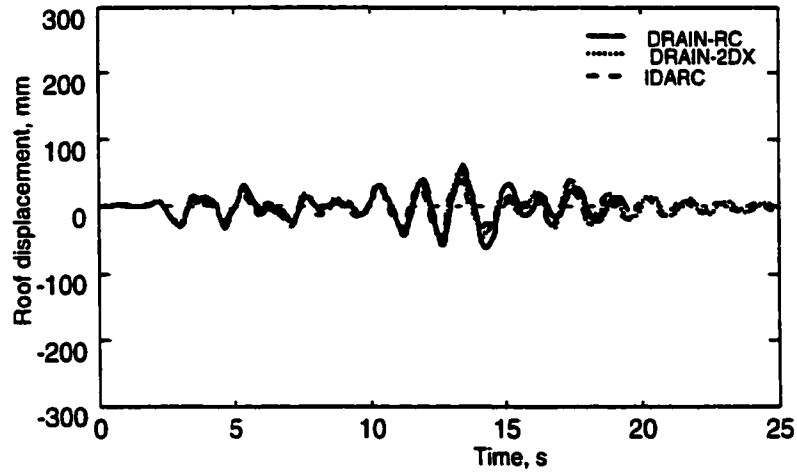


Figure 2.11: Push-over curves for the six storey bare frame in Victoria: nominal material strength

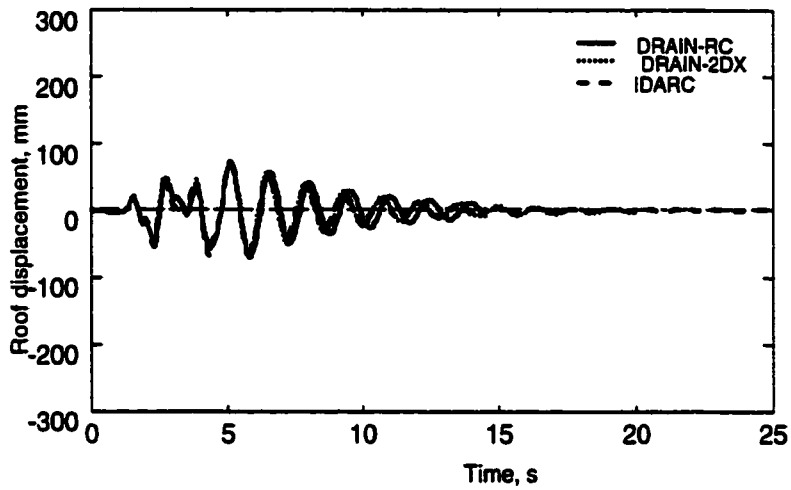


(a)

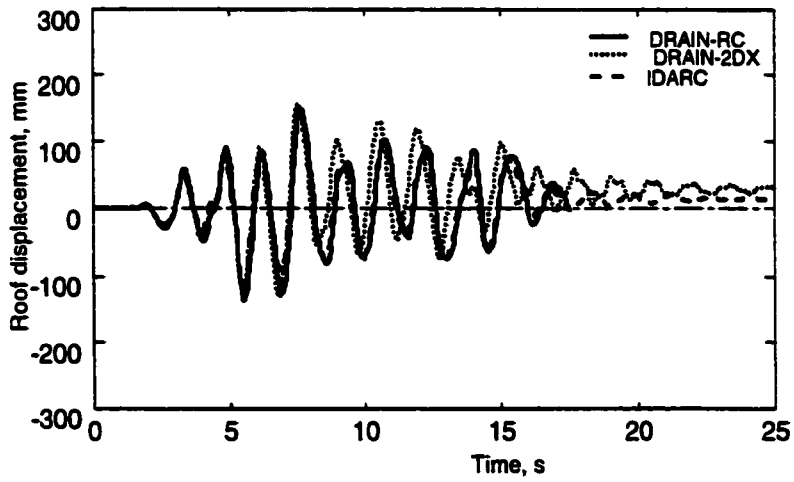


(b)

Figure 2.12: Time history analysis: UHS-500 events (nominal material strength): (a) short duration event - Trial 1, (a) long duration event - Trial 1

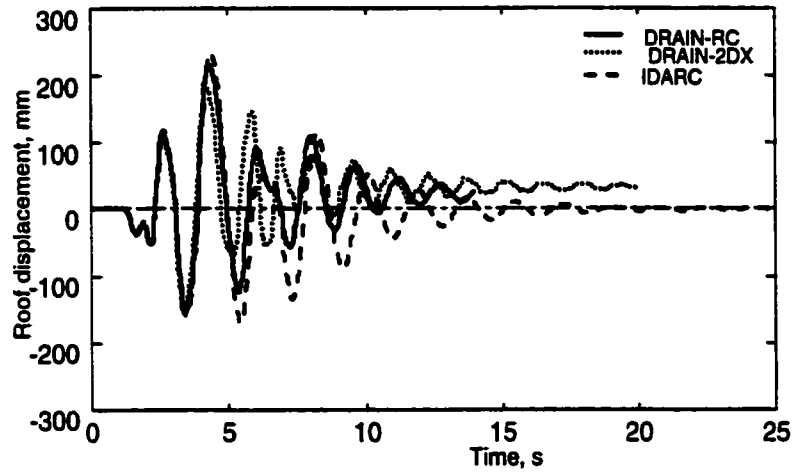


(a)

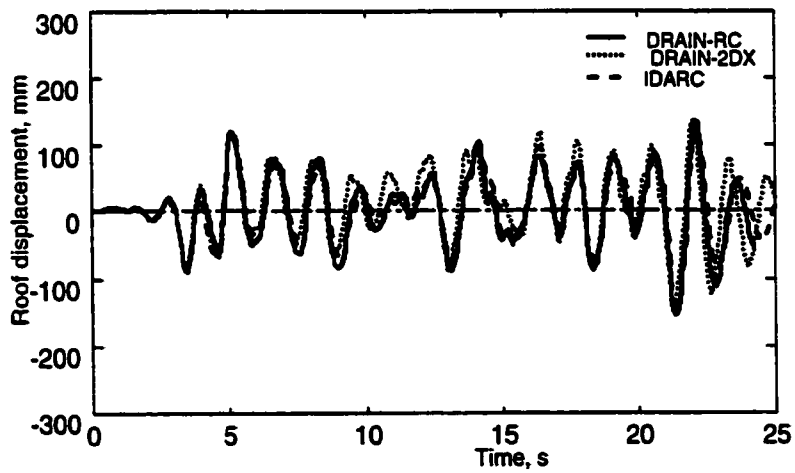


(b)

Figure 2.13: Time history analysis: UHS-1000 events (nominal material strength): (a) short duration event - Trial 1, (a) long duration event - Trial 1



(a)



(b)

Figure 2.14: Time history analysis: UHS-2500 events (nominal material strength): (a) short duration event - Trial 1, (a) long duration event - Trial 1

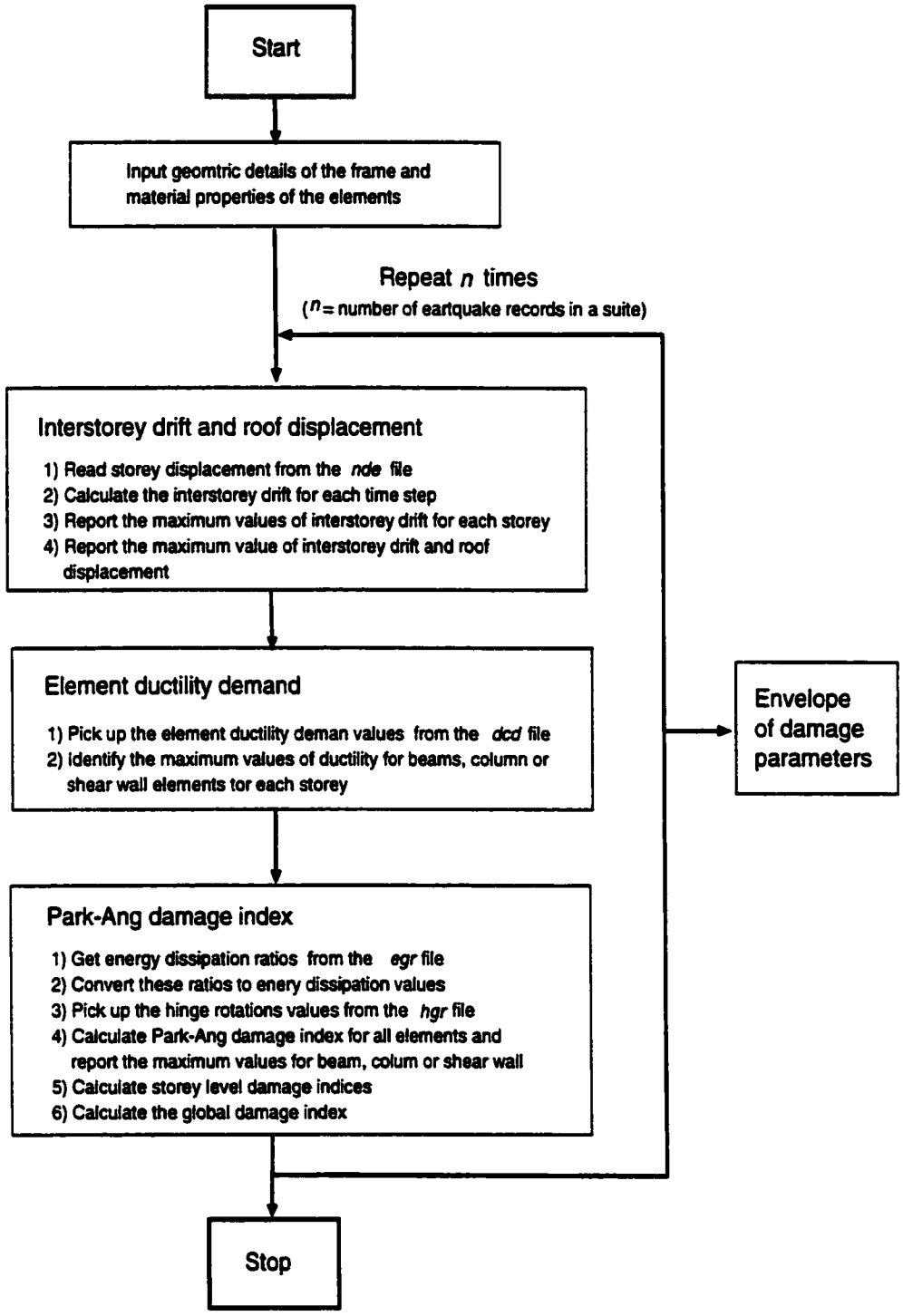


Figure 2.15: Schematic diagram of the organization of the post-processing program used for parsing and processing of DRAIN-RC dynamic anaysis output

Chapter 3

Concrete Moment Resisting Frames

3.1 Introduction

The results of a study on seismic performance of buildings with concrete moment resisting frames are presented in this chapter. Since the building configuration used in this study is simple and regular (Fig. 3.1), two dimensional structural models are sufficient to capture the structural behaviour of the buildings. Three dimensional analysis is computationally expensive and is recommended only when a structure has significant lateral eccentricity. In this study, two building heights, six and twelve storeys respectively, are considered. The buildings are assumed to be situated in Victoria in the west of Canada, and Montreal in the east of Canada. Static push-over analyses are performed to determine the lateral load-resisting capacity of a building structure, and nonlinear dynamic analyses are performed to evaluate its seismic performance. The building frames are designed according to the National Building Code of Canada (NBCC 1995). However, the design lateral forces are calculated using the

revised base-shear formulation proposed for the next version of NBCC (Humar 2000). Two levels of material strength (factored and nominal) are considered in the analyses.

The buildings considered here can be modelled as the transverse frames connected in series by rigid links. The identical frames can be lumped together to reduce the model size. For the buildings considered here, the exterior and interior ductile frames are kept similar for the sake of simplicity. Thus a single frame with lumped system properties can be used for modelling a building. Building structures with infill panels are also studied. The infill panels are modelled as diagonal struts. The number of frames with infill panels and the arrangement of the infill panels are adjusted such that the fundamental period of the building is close to the value obtained by using the NBCC expressions. Depending on period of a building as obtained from NBCC guidelines, all or only some of the ductile frames include infill panels. In those cases where some ductile frames are infilled and some are not, the analysis model is constructed by connecting the infilled and associated bare frames through rigid links.

3.2 Description of Frame models

Six and twelve storey buildings are considered and they are assumed to be located in Victoria in western Canada and Montreal in eastern Canada. A plan view of the buildings considered in this study is shown in Fig. 3.1. It has several 6 m bays in N-S direction and 3 bays in E-W direction. The E-W bays consist of 2 - 9 m office bays and a central 6 m corridor bay. The storey height is 4.85 m for the first storey and 3.65 m for all other storeys. The elevations of the frames for six and twelve storey buildings are shown in Fig. 3.2. The yield stress, f_y for reinforcing steel, and

the 28-day concrete compressive stress, f'_c are assumed to be 400 MPa and 30 MPa, respectively. The following gravity loads are used in the design. Dead load is assumed to be 3.5 kN/m^2 on the roof and 5.0 kN/m^2 on all other floors. Live load is assumed to be 2.2 kN/m^2 on the roof and 2.4 kN/m^2 on all other floors.

The seismic lateral forces are obtained using the new uniform hazard spectrum (UHS) based methodology proposed for the next version of NBCC (Humar 2000). The base shear is distributed across the height of the frame, using the procedure suggested by NBCC 1995 to obtain the floor level forces (discussed in Chapter 1).

The western Canadian location, Victoria has a higher level of seismic hazard as compared to the eastern Canadian location, Montreal. Hence for the buildings situated in Victoria all the transverse frames are assumed to be ductile lateral load-resistant, while for Montreal 50% of the transverse frames are assumed to be ductile lateral load resistant and the rest are designed to take only gravity loads. An interior transverse frame is considered for the purpose of evaluation of the seismic performance. Gravity frames (for Montreal) are not considered to be part of the lateral load resisting system, and they are not included in the analysis.

The structures are designed for the seismic loadings and associated gravity loads. Wind load is not considered in the design as the objective of this study is to evaluate the minimum level of seismic protection available to a building. In a case where wind load governs the design, the structure is expected to have a higher level of seismic protection.

For the purpose of design, the member forces are determined using a linear elastic analysis. In this analysis, the effect of cracking in concrete is accounted for by

assuming reduced moments of inertia (EI) for beams and for columns. The effective EI for a beam is assumed to be 50% of its gross EI, while for a column it is assumed to be 80% of the gross EI. Additional shear forces and bending moments due to $P - \Delta$ effect are also taken into account in the design. The assumed values of the effective EI for beams and columns as mentioned above are used in the static analysis and initial design only. A more accurate value of the effective EI for each element is then calculated using COLA. The values of the EI (effective), yield moment etc. as obtained using COLA would depend on the levels of material strength and might differ slightly for the factored and nominal values.

In general, the period of a bare frame in its fundamental mode of vibration is higher than the value obtained using the expression recommended by NBCC-1995. Non-structural elements in a frame play a significant role in stiffening the structure such that its fundamental period of vibration becomes much lower. It is presumed that the code expression for the time period takes into account the stiffening effect of non-structural elements. Infill panels can be included in a frame to simulate the effect of non-structural elements. Inclusion of infill panels brings down the period of a frame structure. At the same time, it is found in this study that the capacity of such frame is much higher than that of the corresponding bare frame.

In this study, both bare and infilled frames are considered. The number and distribution of infill panels in a frame are chosen such that the fundamental period of the structure is closer to the value recommended by NBCC-95. For concrete frames, the period is given by the expression in Equation 3.1 recommended by NBCC-95.

$$T = 0.075(h_n)^{3/4} \tag{3.1}$$

where, T is the period and h_n is the height of the building above its base.

The infill panels are modelled using equivalent struts. The effective width of these struts can be obtained from the expressions based on the theory of beams on elastic foundation (Drysdale *et al* 1994). The modulus of elasticity of masonry is assumed to be $750f_m$ for concrete blocks and $500f_m$ for clay masonry, where f_m is the compressive strength of masonry. Clay masonry with $f_m = 8.6$ MPa and a thickness of 100 mm is used in all the cases considered here.

A diagonal strut is used to model an infill panel as shown in Fig. 3.3. The equivalent strut model developed by Stafford and Smith (1966) is used in the study. As stated earlier the effective width of the strut is based on the theory of beam on elastic foundation. Parameters α_h and α_l, given by the following expressions (Drysdale *et al* 1994) are used to calculate the effective width.

$$\alpha_h = \frac{\pi}{2}\left[\frac{4E_f I_c h}{E_m t \sin(2\theta)}\right]^{1/4} \tag{3.2}$$

$$\alpha_l = \pi\left[\frac{4E_f I_b l}{E_m t \sin(2\theta)}\right]^{1/4} \tag{3.3}$$

where, E_m and E_f are elastic moduli of wall and frame material respectively; t, h, and l are respectively the thickness, height and length of the infill panel. The angle θ is defined as, $\theta = tan^{-1}(h/l)$. Effective width of the strut, w is given by,

$$w = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_l^2} \tag{3.4}$$

The modulus of elasticity of masonry, E_m is expressed as, $E_m = kf_m$, where f_m is the strength of the masonry and k is a coefficient, whose value can be assumed to be 750 for concrete blocks and 500 for clay bricks.

3.3 Design of the building frames

The lateral load-resisting frames are designed as fully ductile moment-resisting frames. The design base shear is obtained using the revised proposal for the next version of NBCC (Humar 2000). The values of the design base shear for the buildings used in this study are shown in Tables 3.2 and 3.3. The floor level lateral forces in a building are obtained by distributing the design base shear across the height according to NBCC 1995 guidelines. The floor level forces for each building are shown in Tables 3.4 through 3.7.

The frames are designed according to the capacity design philosophy, so that the total flexural capacity of the columns exceeds the sum of the flexural capacities of the beams meeting the columns at a joint. To ensure this, CSA Standard A23.3-94 requires that for reinforced concrete moment resisting frames, the following restriction (Eq. 3.5) be satisfied for the beams and columns meeting at a joint.

$$\sum M_{rc} \geq 1.1 \sum M_{nb} \tag{3.5}$$

where M_{rc} is the factored flexural resistance of a column, and M_{nb} is the nominal

flexural resistance of a beam.

The factored, nominal, and probable resistances are defined in CSA Standard A23.3 - 94, and the strength levels used in calculating these resistances are summarised in Table 3.1. The approximate relations between the flexural resistances corresponding to the three strength levels mentioned above, are given in the CSA A23.3-94 and CPCA concrete handbook (1995). Thus the nominal flexural resistance, M_n is approximately 1.20 times the factored resistance M_r, and the probable resistance, M_p is approximately 1.47 times M_r for flexure alone and 1.57 times M_r when both flexure and shear are present.

Table 3.1: Strength levels of concrete and reinforcing steel

Strength level	maximum stress in concrete	maximum stress in steel
Factored	$0.6\ f'_c$	$0.85 f_y$
Nominal	f'_c	f_y
Probable	f'_c	$1.25\ f_y$

Details of design base shear calculation are shown in Tables 3.2 and 3.3. Firm (reference) ground condition is assumed, $I = 1.0$, $R_O = 2.0$ and $R_d = 4.0$ are used for all the cases shown in Table 3.2.

The design base shear is distributed across the height of the building frames, using the procedure suggested by NBCC 1995 to obtain the floor level forces. The floor level forces in each of these buildings are shown in Tables 3.4 through 3.7.

Table 3.2: Base-shear calculation for MRF buildings, Part I

Building location	Number of storey	Period, s (NBCC)	$S(T)$	$S_a(0.2)/S_a(2.0)$	M_v	V/W
Victoria	6	0.78	0.545	5.8	1.0	0.0681
	12	1.30	0.323	5.8	1.0	0.0404
Montreal	6	0.78	0.2268	14.4	1.0	0.0284
	12	1.30	0.1124	14.4	1.06	0.0149

Table 3.3: Base-shear calculation for MRF buildings, Part II

Building location	Number of storey	Weight (W), kN	Base-shear (V), kN	F_t kN	$V - F_t$ kN
Victoria	6	3592	244.26	13.43	230.83
	12	7372	297.83	27.10	270.73
Montreal	6	7184	204.03	11.22	192.81
	12	14744	219.69	20.00	199.69

The reinforcement details for the ductile frames of the buildings in Victoria and Montreal are shown in Figs. 3.4 and 3.5.

For each building four models are considered in the sudy: (a) Bare frame model with the factored values of material strength, (b) Bare frame model with the nominal values of material strength, (d) Infilled frame model with the factored values of material strength, and (d) Infilled frame model with the nominal values of material strength.

A bare frame model contains a single frame, while depending on the case being considered, an infilled frame model may contain a single frame or two frames connected with rigid links. The Park and Ang damage indices are calculated based on the

Table 3.4: Base-shear distribution in six-storey building in Victoria

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i kN.m	F_x kN
6	22.80	442.0	10077.6	62.47
5	19.19	630.0	12089.7	58.84
4	15.50	630.0	9765.0	47.52
3	11.85	630.0	7465.0	36.34
2	8.20	630.0	5166.0	25.14
1	4.55	630.0	2866.5	13.95
Total		3592.0	47430.3	244.26

Table 3.5: Base-shear distribution in twelve-storey building in Victoria

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i kN.m	F_x kN
12	44.70	442.0	19757.4	57.19
11	41.05	630.0	25861.5	39.39
10	37.40	630.0	23562.0	35.88
9	33.75	630.0	21262.5	32.38
8	30.10	630.0	18963.0	28.88
7	26.45	630.0	16663.5	25.38
6	22.80	630.0	14364.0	21.88
5	19.15	630.0	12064.5	18.37
4	15.50	630.0	9765.0	14.87
3	11.85	630.0	7465.5	11.37
2	8.20	630.0	5166.0	7.88
1	4.55	630.0	2866.5	4.36
Total		7372.0	177761.4	297.83

Table 3.6: Base-shear distribution in six-storey building in Montreal

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i kN.m	F_x kN
6	22.80	884.0	20155.2	52.21
5	19.19	1260.0	24129.0	49.07
4	15.50	1260.0	19530.0	39.72
3	11.85	1260.0	14931.0	30.36
2	8.20	1260.0	10332.0	21.01
1	4.55	1260.0	5733.0	11.66
Total		7184.0	94810.2	204.03

Table 3.7: Base-shear distribution in twelve-storey building in Montreal

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i kN.m	F_x kN
12	44.70	884.0	39514.8	42.19
11	41.05	1260.0	51723.0	29.05
10	37.40	1260.0	47124.0	26.47
9	33.75	1260.0	42525.0	23.88
8	30.10	1260.0	37926.0	21.31
7	26.45	1260.0	33327.0	18.73
6	22.80	1260.0	28728.0	16.13
5	19.15	1260.0	24129.0	13.55
4	15.50	1260.0	19530.0	10.97
3	11.85	1260.0	14931.0	8.39
2	8.20	1260.0	10332.0	5.80
1	4.55	1260.0	5733.0	3.22
Total		14744.0	355522.8	219.69

damage in the beam-column elements in a frame. The damage in the non-structural elements or infill panels is not included in the damage indices.

3.4 Performance of buildings in western Canada

Two levels of strength for the structural materials are considered (factored and nominal material strength levels). Modal analyses are carried out at first to find out the number of infill panels that would be required to bring down the periods of the frames closer to the values recommended by NBCC. Periods of these frames are listed in Table 3.8.

The bare and infilled frame models of the six storey building in Victoria are shown in Fig. 3.6. For this building, masonry infills are considered to exist for the full height of the middle bay and every alternate frame is assumed to contain infill panels. Considering a model cosisting of an infill frame and a bare frame connected together by rigid links, period works out to be 0.77 s and 0.74 s when concrete and steel strength corresponding to their factored and nominal values respective are considered. These values are quite close to the NBCC value which is 0.78 s.

The bare and infilled frame models of the twelve storey building in Victoria are shown in Fig. 3.7. For the twelve storey building, infill panels are assumed to exist for the full height of the middle bay and all frames are infilled. In such case period becomes 1.29 s and 1.26 s for factored and nominal material strength respectively. The corresponding NBCC value is 1.3 s in this case.

Table 3.8: Fundamental periods of the frames located in Victoria

Building Model	Period (s)	
	Factored strength	Nominal strength
1. 6 storey		
Bare frame	1.47	1.37
Infilled frame	0.77	0.74
NBCC-95	0.78	0.78
2. 12 storey		
Bare frame	2.91	2.60
Infilled frame	1.29	1.26
NBCC-95	1.30	1.30

3.4.1 Lateral load-resisting capacity

Static push-over analyses are performed to estimate the lateral load-resisting capacity of each frame. Gravity loads corresponding to $(D + 0.5L)$ are used in the push-over analyses (D and L represent dead loads and live load loads, respectively). The lateral loads due to earthquake are distributed along the height of the frame according to the NBCC 1995 guidelines. The $P - \Delta$ effect is considered in the push-over analyses. Load-deformation curves are obtained from the push-over analyses of six and twelve storey frames. The lateral load-resisting capacity of each model is discussed in detail in the following paragraphs.

Six-storey building:

Push-over curves, representing the variation of base shear with the roof displacement (i.e., lateral drift) in an internal lateral load-resisting frame are shown in Fig. 3.8. Capacity curves (or push-over curves) for both bare and infilled frames are shown. In fact, there are two curves for each type of frame, one in which the

factored value of material strength is used as implied in the design, and the other in which nominal value of material strength is used. It is observed that the inclusion of infill panels drastically improves the capacity of the frame. The effect of infill panels is generally not considered in the design. In reality they contribute a great deal of strength to the overall capacity of a frame (Drysdale *et al* 1994, Fajfar *et al* 1997), provided they are positively connected to the frame.

The frames are designed on the capacity design concept and hence hinges are expected to form in the beams rather than in columns meeting at a joint. This should improve the deformation capacity of the frame and make it behave in a ductile manner. The push-over curves do indicate that the transverse frame for the six storey building behaves in a ductile manner. Push-over analysis provides the base shear for a given interstorey drift which is an important damage parameter and is used throughout this study. In the six storey frame a maximum interstorey drift of 2% occurs when the roof displacement is close to 1% (of the building height) for both bare and infilled frames. The design base shear for each of these frames, V_d is 244.3 kN. The base shear corresponding to 2% interstorey drift is $2.0V_d$ for bare frame with factored material strength, and $2.5V_d$ for the same frame with nominal material strength. The capacity of the infilled frame is significantly higher. Values of the base shear at 2% interstorey drift in the infilled frame are $4.0V_d$ and $4.8V_d$ for factored and nominal material strengths, respectively. The patterns of hinge formation in the frames are shown in Fig. 3.10.

Twelve-storey building:

The push-over curves for the interior transverse frame of a twelve storey building

located in Victoria are shown in Fig. 3.9. The design base shear, V_d is 297.8 kN for this frame. A maximum interstorey drift of 2% occurs at a roof displacement close to 1% of the building height for the bare frame and 0.8% for the infilled frame. The base shear at this level of interstorey drift (i.e., 2% of storey height) is $1.7V_d$ for the bare frame with factored material strength, $1.9V_d$ for the bare frame with nominal material strength, $3.5V_d$ for the infilled frame with factored material strength, and $4.0V_d$ for the infilled frame with nominal material strength. In this case also the infilled frame has significantly higher strength as compared to the bare frame. The pattern of hinge formation in the frames is shown in Fig. 3.11.

3.4.2 Dynamic analyses of the six-storey building in Victoria

The bare and the infilled frame models are analyzed for all three levels of earthquake hazard. The envelope values of damage parameters obtained are shown in Figs. 3.12 through 3.17. These figures show the storey-wise damage parameters for the bare and the infilled frame models. Figures 3.18 through 3.21 show the variations of some of the damage parameters of these frames, with the level of earthquake hazard. Performance of the building under different levels of seismic hazard is analyzed and is discussed in detail in the following paragraphs.

Performance under UHS-500 events:

The bare frame with either factored or nominal material strengths remains largely elastic under UHS-500 events. The columns at first storey level are slightly damaged and hinges form at the bottom of these columns. Hinges are formed in some of the beams, but the storey level damage indices are quite low and the maximum

interstorey drift is below 1% of storey height. At this level of earthquake hazard, the performance of the bare frame model can be classified as *fully operational*, and the building is expected to be safe to occupy immediately after such earthquake events.

The performance of the infilled frame model is similar to that of the bare frame. The infilled frame remains mostly elastic under UHS-500 events for both levels of material strengths (i.e., factored and nominal). Some of the structural members can be assumed to have cracked as moments in those members exceed their cracking moment values. The performance of the frames (for both nominal and factored strength) can be classified as *fully operational* as there is only minor damage, and the building is expected to be safe to occupy immediately after such earthquake events.

Performance under UHS-1000 events:

Under this level of earthquake hazard, the bare frame model suffers a minor damage with some beams and columns forming hinges. The maximum interstorey drift is below 1.0% of storey height. From Figs. 3.12 through 3.17, it is clear that the damage is only marginally more than that for UHS-500 events. The maximum beam and column ductilities are below 2, the maximum storey level Park-Ang damage index is close to 0.3 in the case of factored material strength and 0.2 in the case of nominal material strength. The performance of the building under this level of earthquake can be said to be *operational* and some repair work may be necessary to restore the normal functioning of the building.

There is a similar level of damage in the infilled frame model. The maximum beam ductility is close to 2 in both infilled frame and the associated bare frame. The maximum column ductility demand is close to 1. The maximum interstorey

drift is close to 0.5% of storey height and the storey level Park-Ang damage index is negligible. The performance level of the building can be termed as *operational.*

Performance under UHS-2500 events:

This level of earthquake hazard is very detrimental to the bare frame model. Ductility demands in some of the beams in the lower three storeys are quite high. The maximum interstorey drift is close to 2.2% for factored and 2.0% for nominal values of material strength. The maximum values of beam and cloumn ductilities are close to 6 and 5, respectively. The storey level Park-Ang damage index is close to 0.9, and the global damage index is close to 0.6. The damage parameters indicate that the building is highly damaged and its performance could be classified as *life-safe* to *near collapse.* The building would remain unsafe for post-earthquake occupancy.

Infilled frame model shows some improvement in the building's performance. Some lower storey beams and columns still suffer significant damage. The ductility demand in some beams in the infilled frame is close to 4 for factored and 3 for nominal values of material strength. The maximum value of beam ductility demand in the associated bare frame is close to 3 for both levels of material strength. Some of the columns have yielded, but the ductility demands are moderate being less than 4 in the infilled frame and less than 3 in the associated bare frame. Maximum inter-storey drift is close to 1.8% and 1.5% for factored and nominal strength, respectively. The storey level Park-Ang damage index is close to 0.8 for factored strength and close to 0.6 for nominal strength. The global damage index is close to 0.5. The performance of the building can be categorized as *life safe* under UHS-2500 earthquake events. The building would not be available for post-earthquake occupancy.

Table 3.9: Performance of the six-storey building located in Victoria

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame	Fully operational	Operational	Near collapse
Infilled frame	Fully operational	Operational	Life-safe

Seismic performance of six storey building located in Victoria is summarized in Table 3.9. Figures 3.19 through 3.21 show the variation of some of the damage indicators with seismic hazard level, for all the cases discussed here. It is noticed that the infill panels consistently improve the performance of the building. Figures 3.22 shows a sample time history plots of roof displacement for bare and infill frames having nominal level of material strengths. It is observed that the infill panels have an important role to play in the response of a framed structure.

3.4.3 Dynamic analyses of the twelve-storey building in Victoria

Inelastic dynamic analyses of the models for twelve storey frame designed for a location in Victoria are carried out for all the three levels of earthquake hazard. The envelope values of the damage parameters are shown in Figs. 3.23 through 3.26, and the variations of some damage quantities with the level of seismic hazard are shown in Figs. 3.27 through 3.30.

Performance under UHS-500 events:

From Figs. 3.23 through 3.26 it is observed that the bare frame models do not suffer any significant damage. Some of the beams at the lower storeys have yielded

but maximum beam ductility demand remains within 2, and the columns remain elastic. The maximum interstorey drift is within 1% of storey height and the storey level Park-Ang damage index is below 0.4. From Figs. 3.29 and 3.30, it is seen that the element and global level damage indices are quite low. The seismic performance of the building in this case can be classified as *operational*, and the building is expected to be safe for post-earthquake occupancy.

The infilled frame model remains elastic for both levels of material strengths. There could be some cracking in various members, but there is no hinging in any beam or column. The maximum value of interstorey drift is limited to 0.5% of storey height, and the storey level Park-Ang damage indices are negligible. The performance level achieved by the infilled frame model can be categorized as *fully operational* under UHS - 500 events, and no repair work would be necessary for normal functioning of the building after this level of earthquake events.

Performance under UHS-1000 events:

The response of the bare frame model under UHS-1000 events is quite moderate. The extent of damage is marginally higher than that under UHS-500 events. Under UHS-1000, the maximum value of interstorey drift is around 1.25% of storey height. The storey level damage indices are insignificant. The ductility demands in some beams exceeds 2.0. Columns remain mostly elastic. Performance level of the frame can be said to be *operational* in this case.

The infilled frame models show improved performance. The maximum value of interstorey drift is close to 0.5% of storey height. The beams and columns remain largely elastic. Maximum value of ductility demand in beams and columns do not

exceed 1.0. The value of storey level damage index is insignificant. The performance of the infilled frame under UHS-1000 events can be classified as *fully operational* and the building considered safe for post-earthquake occupancy.

Performance under UHS-2500 events:

Both bare and infilled frame models suffer considerable damage under this level of earthquake hazard. In the case of bare frame, the maximum value of interstorey drift approaches a value of 2.5% and 2.0% of storey height for factored and nominal strength respectively. Ductility demand in some of the beams is 5.0, and maximum ductility demand in columns is around 3.5. The values of the maximum storey level damage is close to 0.9 for both levels of material strength. The global damage index is close to 0.7. The values of maximum ductility demand and damage index in beams and columns (Figs. 3.23 through 3.26) suggest that some of them have failed. The level of performance achieved by the structure under UHS-2500 events can be categorized as *near collapse*. The building frame is assumed to have lost much of its lateral load-resisting capacity, and it is unsafe for post-earthquake occupancy.

The introduction of infill panels has a positive effect on the performance of the building. In this case, the interstorey drift is close to 1.3%. The ductility demand in some of the beams is close to 3.0, and the ductility demand in some of the first storey columns is close to 2.0. Storey level damage index is close to 0.8 for factored values of material strength and is close to 0.6 for nominal values. Figures 3.26 through 3.30 suggest that some of the beams have failed, the global damage index is close to 0.6, and the overall damage in the structure is quite high. The performance of structure can be categorized as *life-safe*, and the building would not be available for

Table 3.10: Performance of the twelve storey building located in Victoria

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame	Operational	operational	Near collapse
Infilled frame	Fully operational	Fully operational	Life-safe

post-earthquake occupancy.

Seismic performance of twelve-storey building located in Victoria under various levels of earthquake hazard is summarized in Table 3.10. It is observed that the infill panels generally help a frame in achieving a better performance.

3.5 Performance of buildings in eastern Canada

Modal analyses are carried out for the bare frame and infilled frame models. For the buildings in Montreal every alternate frame is assumed to be ductile load-resisting frame, other frames are able to carry only gravity loads. The analysis models used here include only ductile lateral load-resisting frames, not the gravity frames. The Modal analysis is used to find out the number of infill panels that is required to bring down the periods of the buildings closer to the values recommended by the building code. Periods of these models are listed in Table 3.11.

It will be observed that using factored strength, the ratio of the period obtained from a modal analysis of bare frame models to that given by the NBCC formula is 2.88 and 4.0 for the six and twelve storeys, respectively. The corresponding values for Victoria are 1.88 and 2.24, respectively. This difference could be explained as

Table 3.11: Fundamental periods of the frames in Montreal

Building Model	Period (s)	
	Factored strength	Nominal strength
1. 6 storey		
Bare frame (single)	2.25	2.16
Infilled frame	0.79	0.78
NBCC-95	0.78	0.78
2. 12 storey		
Bare frame (single)	5.25	4.95
Infilled frame	1.51	1.40
NBCC-95	1.30	1.30

follows. The NBCC formula gives the same period, 0.78 s for six-storey frames in Victoria and Montreal. Similarly, the two twelve-storey frames, located in Victoria and Montreal each have a period of 1.30 s as obtained from the NBCC formula. On the other hand, the tributary mass for a lateral load resisting frame of a building in Montreal is twice that for the frame in an identical building in Victoria. This implies that on the basis of trubutery mass alone the period obtained from a modal analysis of a building in Montreal should be $\sqrt{2}$ times the period of an identical building in Victoria. The larger difference between the modal analysis period and NBCC period for buildings in Montreal implies that greater number of infill panels will have to be assumed to obatin a match between the two sets of periods.

The bare and infilled frame models for the six storey building in Montreal are shown in Fig. 3.31. For this building, masonry panels are assumed to exist in the middle bay for the full height of the building and every ductile frame is considered to be infilled. In that case, the period becomes 0.78 s and 0.79 s for nominal and factored strength, respectively. These values are quite close to the NBCC value which

is 0.78 s.

The bare and infilled frame models for the twelve storey building in Montreal are shown in Fig. 3.32. For the twelve storey building, infill panels are assumed to exist in two exterior bays for the full height of the building and every ductile frame is considered to be infilled. In that case, the period becomes 1.51 s and 1.40 s for factored and nominal material strength, respectively; the NBCC value is 1.3 s.

3.5.1 Lateral load-resisting capacity

Static push-over analyses are performed to estimate the lateral load-resisting capacity of each frame. The effect of $P - \Delta$ is considered in the push-over analyses. Load-deformation curves are obtained from the push-over analyses of six-storey and twelve-storey frames and are discussed in more detail in the following paragraphs.

Six-storey building:

Push-over curves, representing the variation of the base shear with the roof displacement are shown in Fig. 3.33. Two pairs of curves are shown in this figure; one is for the bare frame, and the other is for the infilled frame. Two levels of material strengths (factored and nominal) are considered. The push-over curves in Fig. 3.33 indicate that the maximum inter-storey drift of 2% occurs when the roof displacement is close to 1.2% for the bare frame and close to 0.9% for the infilled frame. The design base shear, V_d is 204.0 kN. The base shear corresponding to 2% interstorey drift is $2.2V_d$ for bare frame model with factored material strength, $2.9V_d$ for the same same frame with nominal material strength. The capacity of the system with infilled frame is significantly higher. The values of base shear at 2% interstorey drift

in the infilled frame are $12.0V_d$ and $13.2V_d$ for factored and nominal values of material strength, respectively. The hinge formation in the frames is shown in Fig. 3.35.

Twelve-storey building:

The push-over curves for the models of frames for twelve storey building located in Montreal are shown in Fig. 3.34. The design base shear, V_d is 219.7 kN for the ductile frame. The maximum interstorey drift of 2% occurs at a roof displacement close to 1.1% of the building height for the bare frame and close to 0.8% for the infilled frame. The base shear corresponding to 2% interstorey drift is $2.2V_d$ for bare frame model with factored values of material strength, $2.5V_d$ for the same same frame with nominal values of material strength. The values of base shear at 2% interstorey drift in the infilled frame are $9.3V_d$ and $10.4V_d$ for factored and nominal values of material strength, respectively. The hinge formation in the frames is shown in Fig. 3.36.

3.5.2 Dynamic analyses of the six-storey building in Montreal

Four models, as discussed at the beginning of this section, are constructed for the evaluation of seismic performance of the six storey building located in Montreal. All of these models are analyzed for 3 levels of earthquake hazard (i.e., UHS-500, UHS-1000 and UHS-2500). The envelopes of the damage parameters are shown in Figs. 3.37 through 3.39. Variations of some of the damage parameters with the level of seismic hazard are shown in Figs. 3.40 and 3.41. Since the values of Park-Ang damage index are very low, their variations are not shown here. Performance of the building depends on the models considered, the level of material strength, and the level of seismic hazard. The performance achieved by the building under different

levels of seismic hazard are discussed in detail in the following paragraphs.

Performance under UHS-500 events:

All models of the frame for this building remain elastic under this level of seismic hazard. None of the beams and columns in these models form hinges, and the interstorey drift is less than 0.3% of storey height in all the four models. Since no significant damage is observed in the models considered here, the seismic performance of the building can be categorized as *fully operational* under this level of earthquake hazard.

Performance under UHS-1000 events:

Under this level of hazard also, the structural models of the building do not indicate any major damage. The frame remains elastic with none of the beams and columns forming hinges. The maximum interstorey drift is close to 0.4% in the case of bare frame models, and is less than 0.2% in the case of infilled frame models. The Park-Ang damage index values are insignificant. The performance level of the building can be said to be *fully operational* under UHS-1000 events for all the models considered here.

Table 3.12: Performance of the six-storey building in Montreal

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame	Fully operational	Fully operational	Operational
Infilled	Fully operational	Fully operational	Operational

Performance under UHS-2500 events:

The building models suffer some minor damages under this level of seismic hazard. Some beams in the first storey level of bare frame models form hinges. In the case of infilled frame models, hinges are formed in some columns at the bottom storey. Ductility demand in these columns does not exceed 2.0. The inter-storey drift ratio is less than 0.6% in all the bare frame models and close to 0.4% in the infilled frame models. The frame systems remain mostly elastic. The performance level of the building in these cases can be classified as *operational* under UHS-2500 events.

A summary of the levels of performance achieved by different models of the six storey building located in Montreal under various levels of seismic hazard is given in Table 3.12.

It is observed that the six-storey building in Monteral does not suffer significant damage even under hazard level of UHS-2500. This can be explained with the help of Figure 3.42. The design base shear has been calculated based on the period calculated using the NBCC formula (0.78 s). Considering the bare frame model with factored strength the modal analysis gives considerably higher period (2.25 s). From Figure 3.42 we can see that the spectral acceleration corresponding to the period

obtained from the modal analysis one sixth of that corresponding to the NBCC period. This means that the intensity of the dynamic forces attracted by the structure is significantly less as compared to that assumed in the design. Consequently, the dynamic response of the structure remains very low. In this case, the use of NBCC period clearly over estimates the design base shear. In case of the infilled frame the period is close to the NBCC value. Since the period of the bare frame is considerably higher than the NBCC period (2.88 times), a large number of infill panels are required to bring down the period of the structure to the NBCC level. This produces unrealistically high stiffness in the system, and the dynamic response reamins low.

In the case of the six-storey building in Victoria, the modal period and the NBCC period for the bare frame with factored strength are 1.47 s and 0.78 s, respectively. This implies that the spectral acceleration corresponding to the period obtained from modal analysis is one half of that corresponding to the NBCC period (Fig. 3.42). This indicates that the intensity of the dynamic forces attracted by the structure are reduced as compared to that assumed in the design, but not as much as in the case of Montreal. Consequently, the dynamic response is significantly higher than that of a corresponding frame in Montreal. As the period of the bare frame is much lower than that for the corresponding building in Montreal, relatively less number of infill panels would be required for the infilled frame to have a period close to the NBCC level. Thus the increase in stiffness is not as high as in the case of the building in Montreal, and the dynamic response of the infilled frame in Victoria is comparatively much higher.

3.5.3 Dynamic analyses of the twelve-storey building in Montreal

The structural models for the twelve storey building located in Montreal are analyzed to obtain their response time histories corresponding to all 3 levels of seismic hazard and the envelopes of damage parameters are shown in Figs. 3.43 through 3.45. The variations of some damage parameters with the hazard level are shown in Figs. 3.46 and 3.47. The values of Park-Ang damage index are quite insignificant, hence, their variations with the hazard levels are not shown. Performance levels achieved by the building under various levels of seismic hazard are discussed below.

Performance under UHS-500 events:

The building is found to remain elastic under this level of seismic hazard for all the cases considered here. None of the beams and columns in these models form hinges and the interstorey drift is quite low being close to 0.2% of storey height in all the six models. Since no significant damage is observed in the models considered here, the performance level of the building can be said to be *fully operational* under this level of earthquake events.

Performance under UHS-1000 events:

In this case also the building remains elastic for all the models considered. All the beams and columns remain elastic. There may be cracking in these members, but no hinge formation takes place. The values of interstorey drift remains close to 0.2%. The performance level of the building under UHS-1000 can be said to be *fully operational*

Table 3.13: Performance of the twelve storey building in Montreal

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Bare frame	Fully operational	Fully operational	Operational
Infilled frame	Fully operational	Fully operational	Operational

Performance under UHS-2500 events:

Under this level of seismic hazard, the frame models suffer some minor damage.There may be cracking in various members, but none of the members have yielded and interstorey drift ratio remains close to 0.4% of storey height. The structure remains mostly elastic. The performance level in this case can be said to be *operational*, and some minor repair work may be necessary for the normal functioning of the building.

A summary of the levels of performance achieved by different models of the twelve storey building located in Montreal under various levels of seismic hazard is given in Table 3.13. An argument similar to that for the six-storey building in can also be applied here to explain the low level of damage in the twelve-storey building in Montreal.

3.6 Discussion

Seismic performance of the building with concrete moment resisting frames as their lateral load-resisting system is presented in this chapter. Victoria and Montreal are chosen as the representative locations in terms of seismic hazard levels for western

and eastern Canada, respectively. Two building heights are considered: six storeys and twelve storeys. For Victoria, all the transverse frames are assumed to be ductile lateral load-resisting. For Montreal, only half of the transverse frames are considered ductile lateral load-resisting and the rest are assumed to be capable of resisting only gravity loads. Static push-over analyses are performed to determine the capacity of each frame. A series of nonlinear dynamic analyses are carried out for various models of a building in order to find out the extent of damage in them for a given level of seismic hazard.

Non-structural elements have significant influence in the capacity and performance of a frame. They also influence the period of a structure. Infill panel elements are used in this study to simulate the effect of non-structural elements. It is observed that the lateral load capacity of a frame increases dramatically when infill panels are included.

For all the models (bare frame or infilled), two levels of material strength (factored and nominal) are considered. Factored strength level is used to get an idea of the capacity of a structure or its seismic level of protection implied in the design, while the nominal strength level is used to obtain an estimate of the realistic behaviour of a structure. Both strength levels are taken into account while evaluating the seismic performance of a frame.

It is observed that the buildings in western Canada are more vulnerable to seismic hazard as compared to those in eastern Canada. The buildings in western Canada suffer considerable damage under the higher levels of seismic hazard. Under UHS-500 events, the buildings remain *fully operational* or *operational* depending on whether the

effect of infill panels is considered are not. At a higher level of seismic hazard, they suffer significant damage, and their their worst performance is assessed as *operational* under UHS-1000 events and *life-safe* or *near collapse* under UHS-2500 events.

The buildings in eastern Canada do not suffer significant amount of damage under the design level and higher levels of earthquake hazard. They remain *fully operational* under UHS-500 and UHS-1000, and remain *operational* under UHS-2500 events in most of the cases. This indicates that the buildings in eastern Canada when designed according to NBCC, are not vulnerable to seismic hazard in that region. This can be attributed to the fact that the base shear calculated using the NBCC formula for period is considerably over estimated. Due to the local spectral shape and the higher tributary mass, the intensity of the dynamic forces attracted by the buildings in eastern Canada are considerably less than that assumed in the design.

From the push-over curves, it is clear that the inclusion of infill panels in the building models increases the strength and stiffness of the models. The number of infill panel has been chosen such that the period of the structure is close to the NBCC 1995 values. But, this might reduce the ductility capacity of the overall system. It is interesting to note that although the pattern of yielding in the bare frame exhibits the effect of capacity design, the yielding pattern in infilled frame is not as predictable. Similar hinging pattern in infilled frames has been reported in Fajfar *et al* (1997).

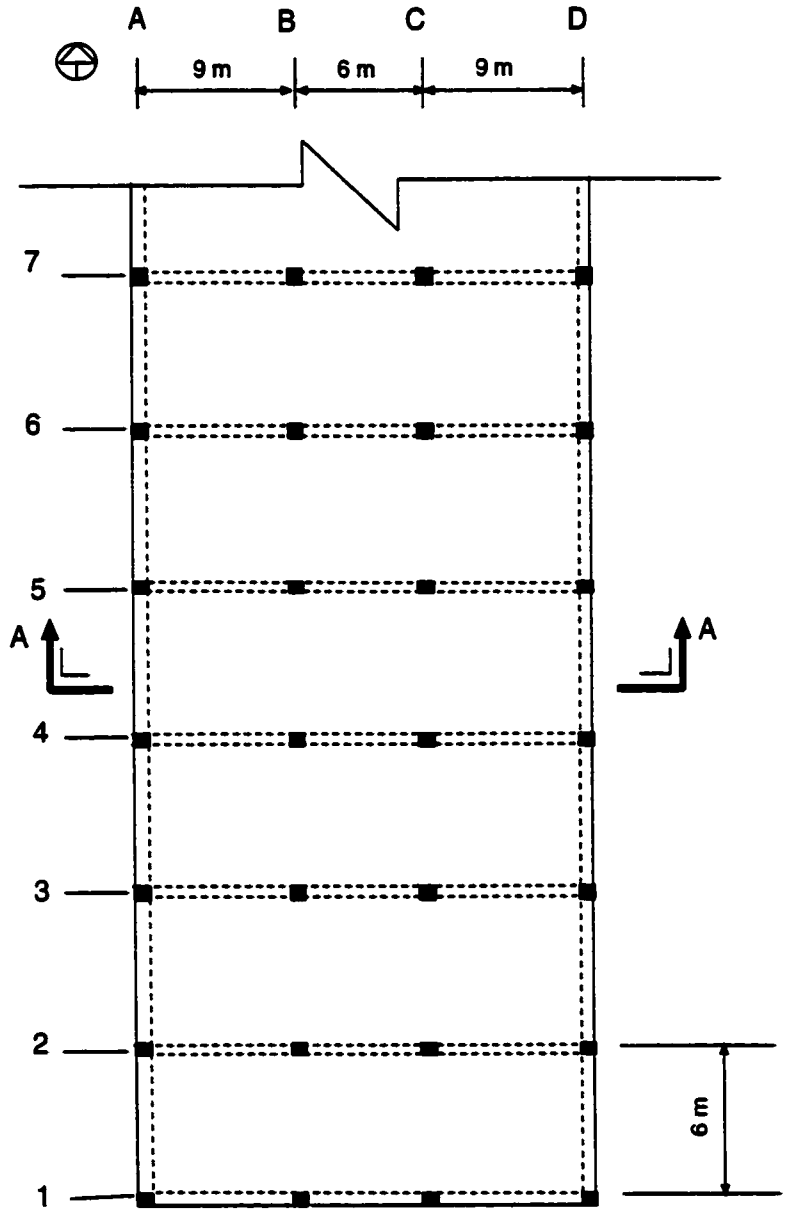
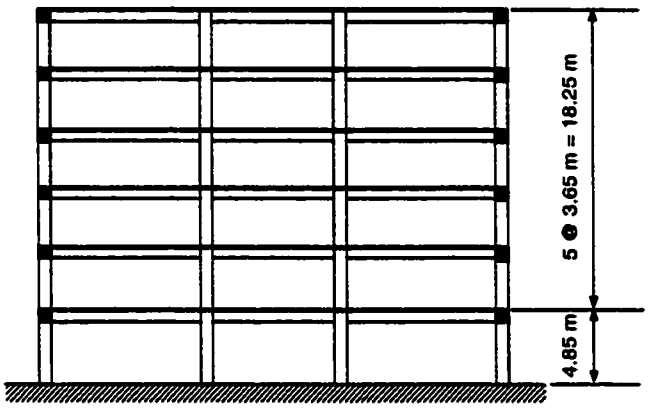
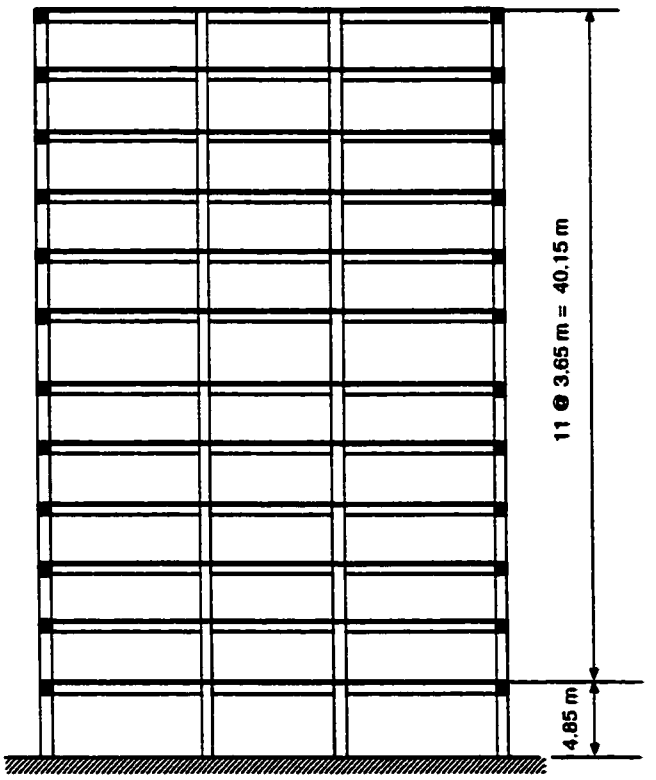


Figure 3.1: Plan view of the buildings



(a)



(b)

Figure 3.2: Elevation of transverse frames: (a) six-storey building, (b) twelve-storey building

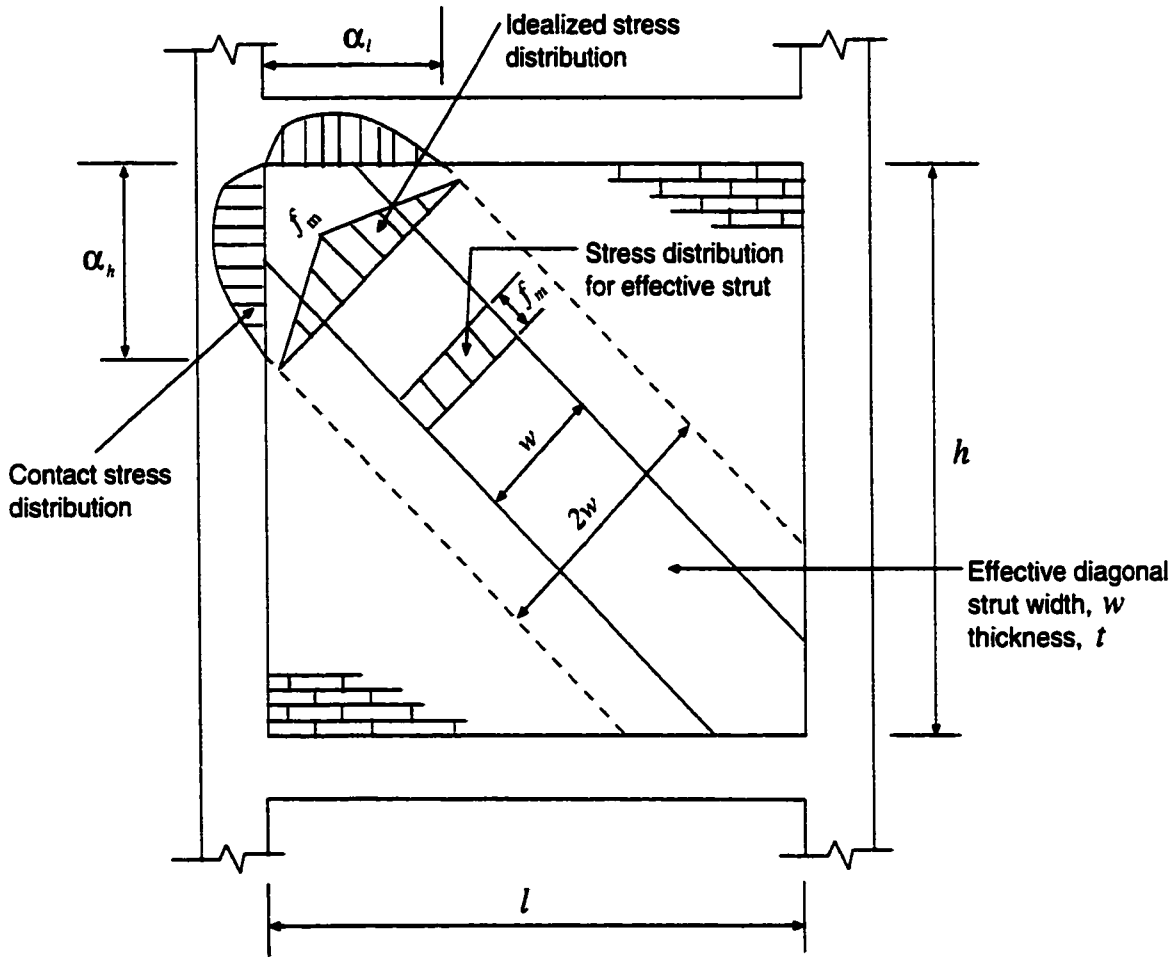
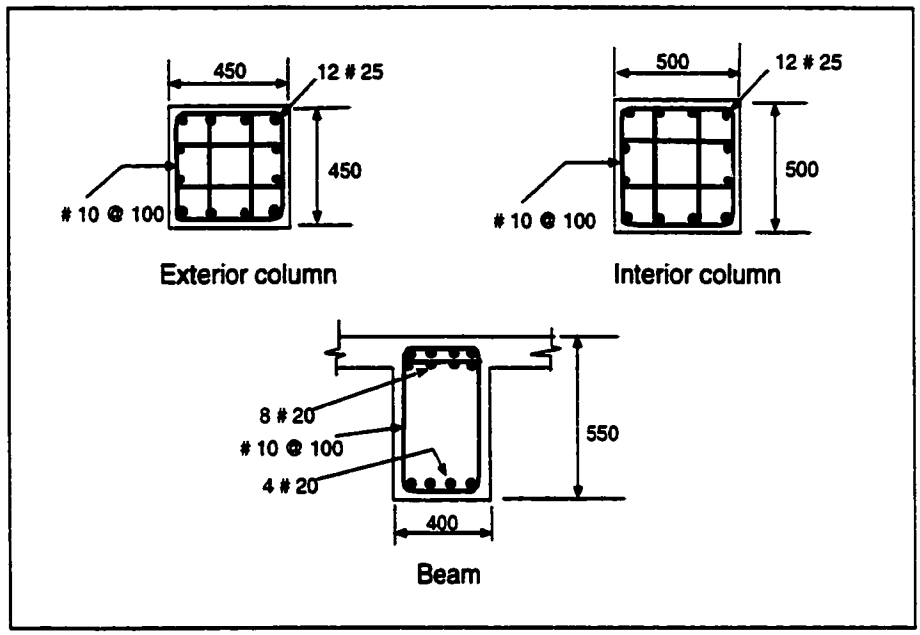
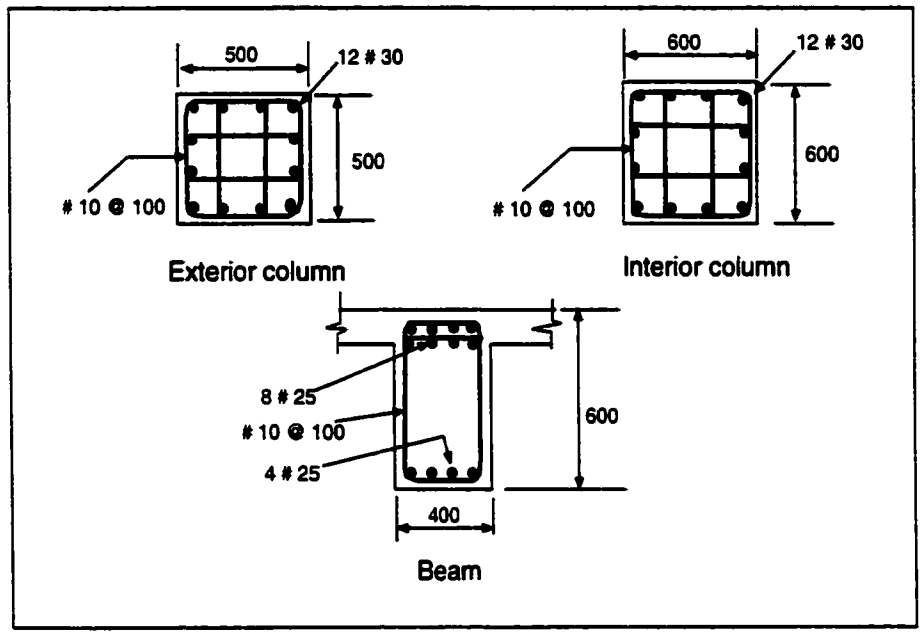


Figure 3.3: Equivalent strut model of masonry infill panel

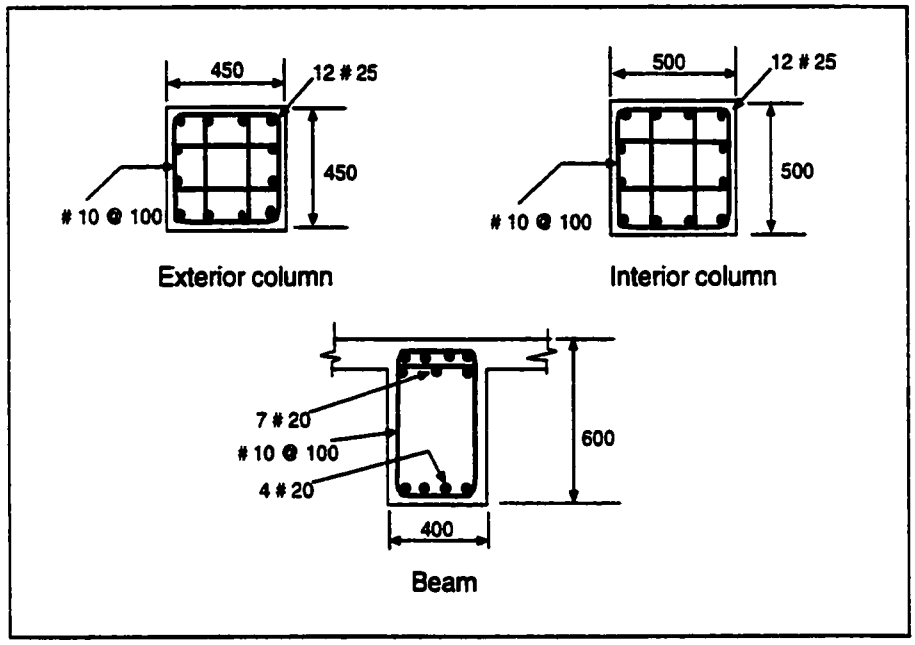


(a)

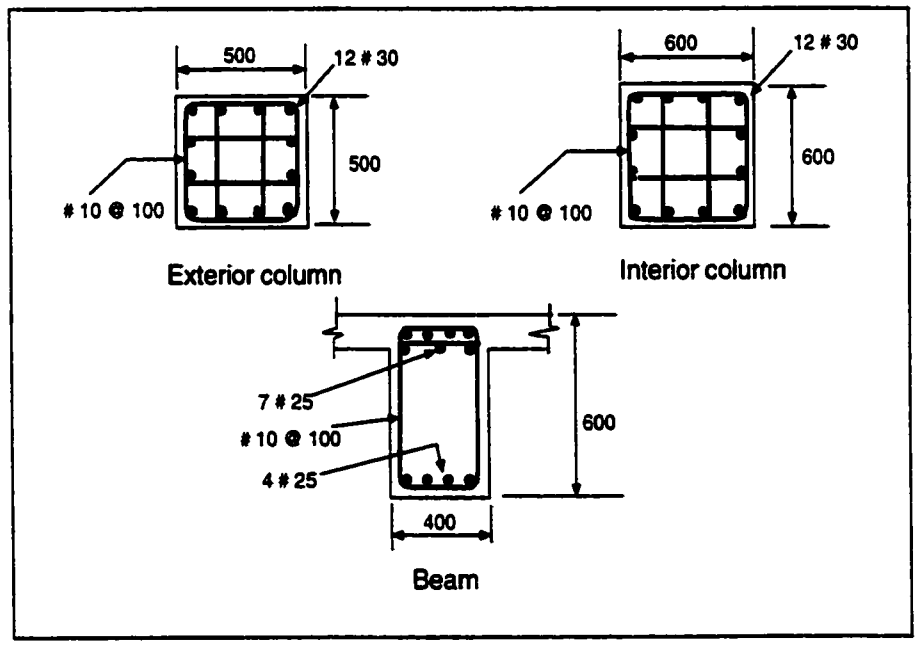


(b)

Figure 3.4: Reinforcement details for beams and columns in ductile frames of the buildings in Victoria (a) six-storey, (b) twelve-storey

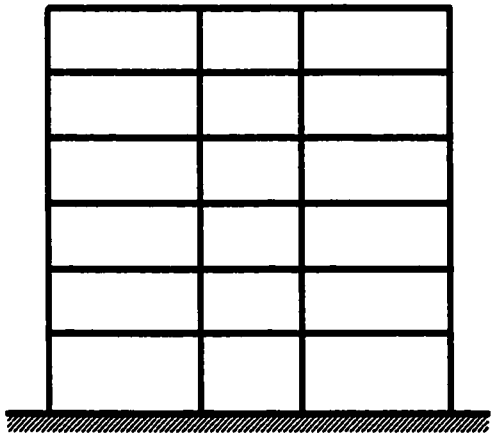


(a)

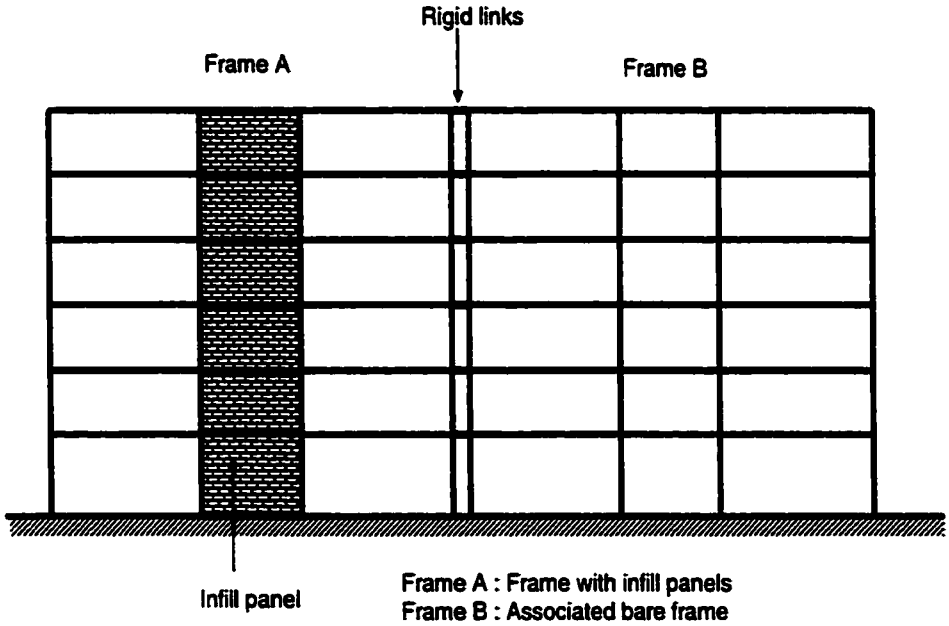


(b)

Figure 3.5: Reinforcement details for beams and columns in ductile frames of the buildings in Montreal (a) six-storey, (b) twelve-storey



(a) Bare frame model



(b) Infilled frame model

Figure 3.6: Frame models for six-storey building in Victoria

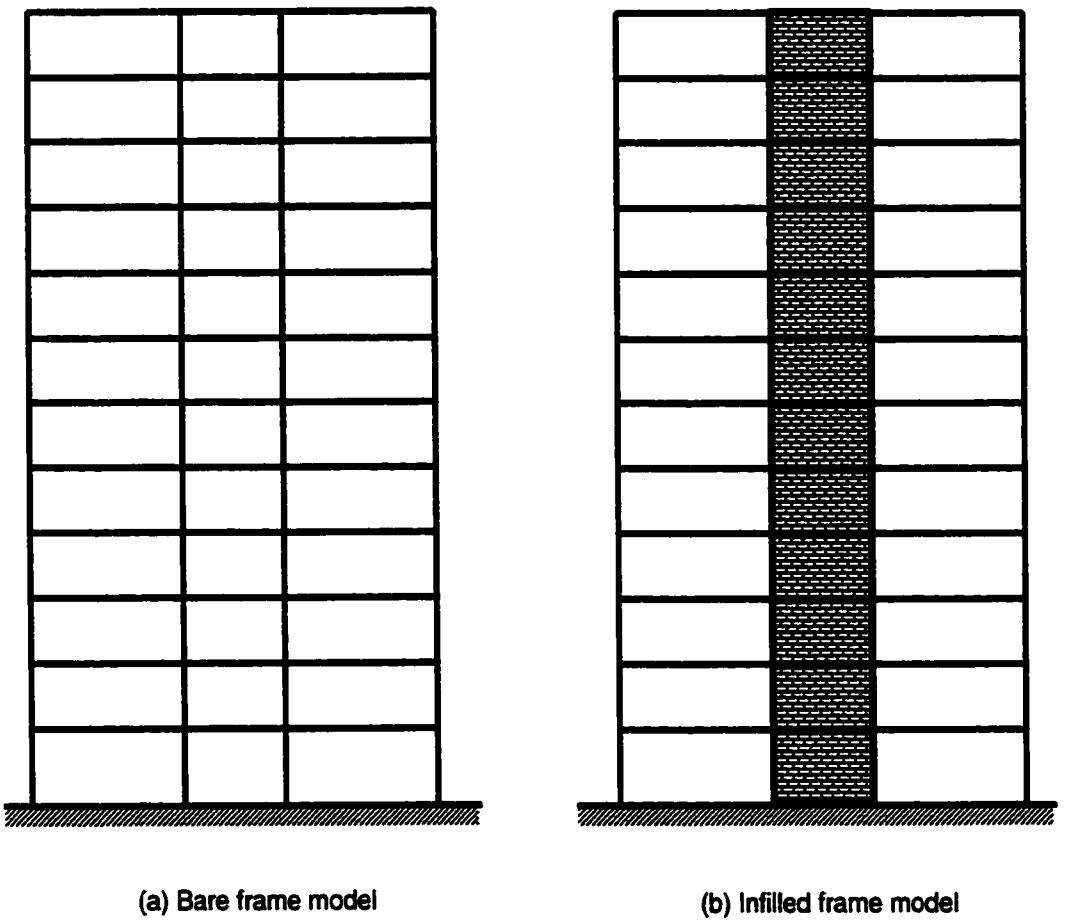


Figure 3.7: Frame models for twelve-storey building in Victoria

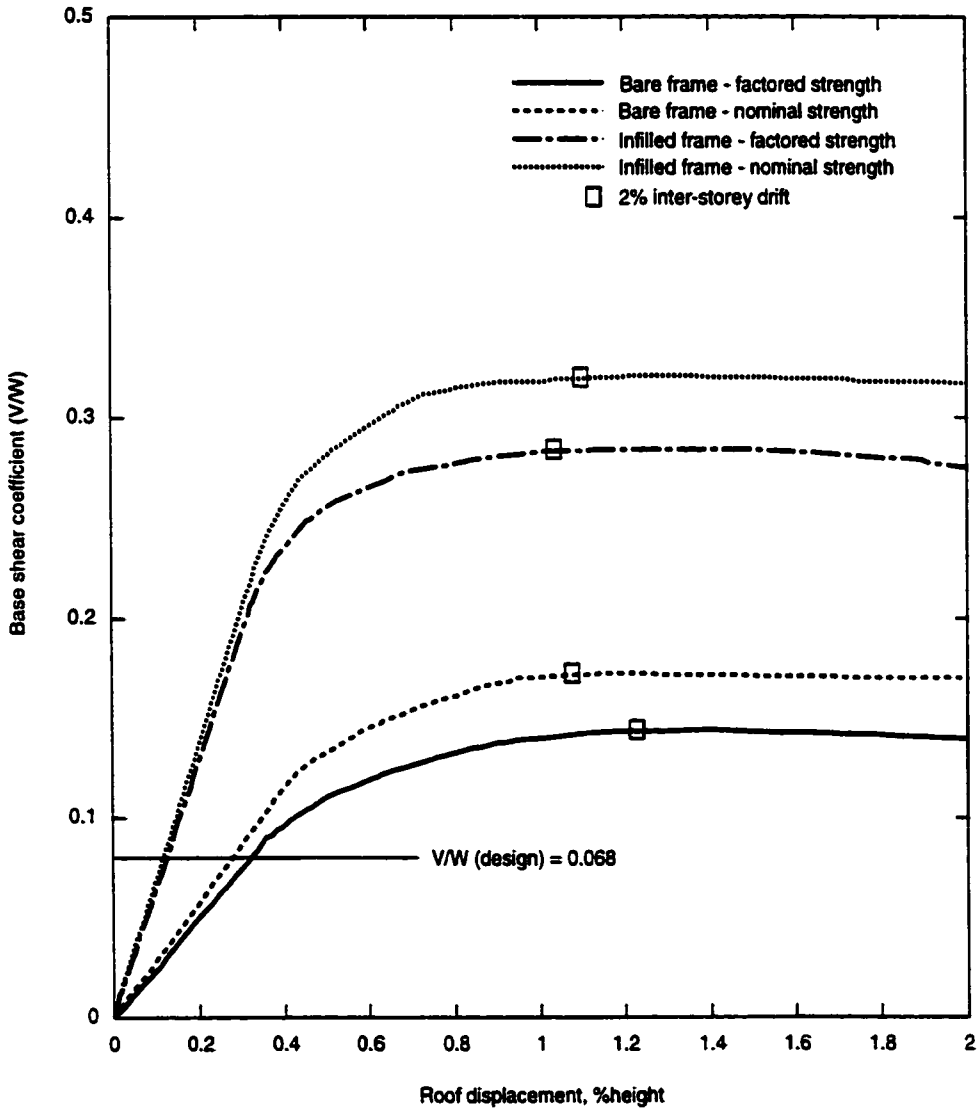


Figure 3.8: Push-over curves for six-storey building in Victoria

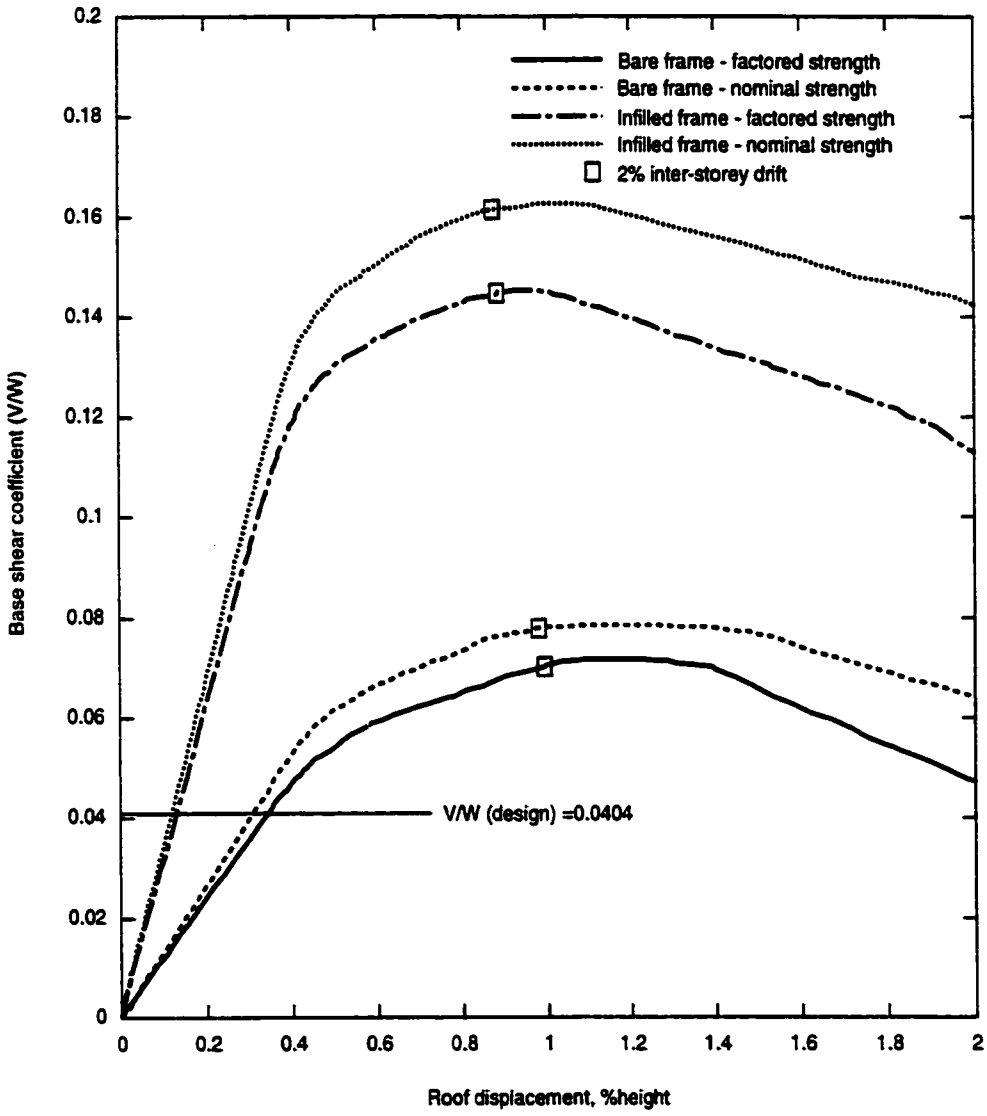
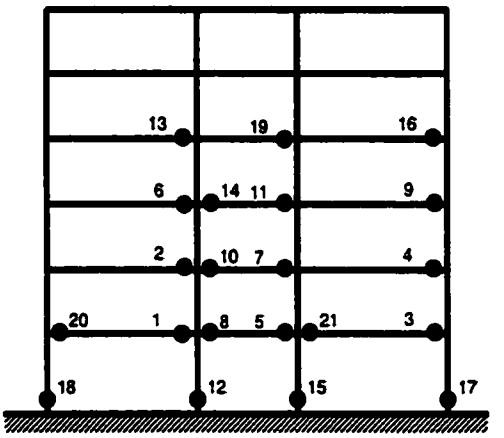
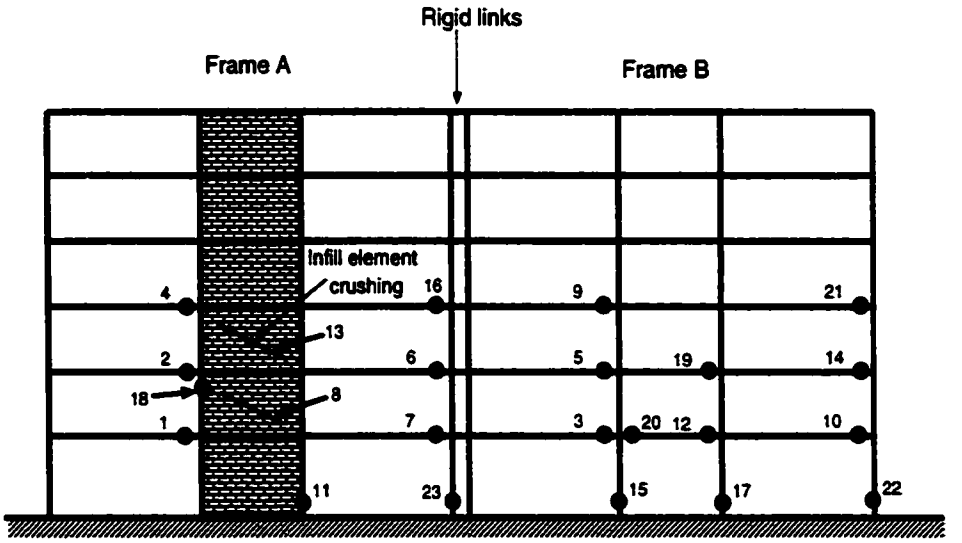


Figure 3.9: Push-over curves for twelve-storey building in Victoria



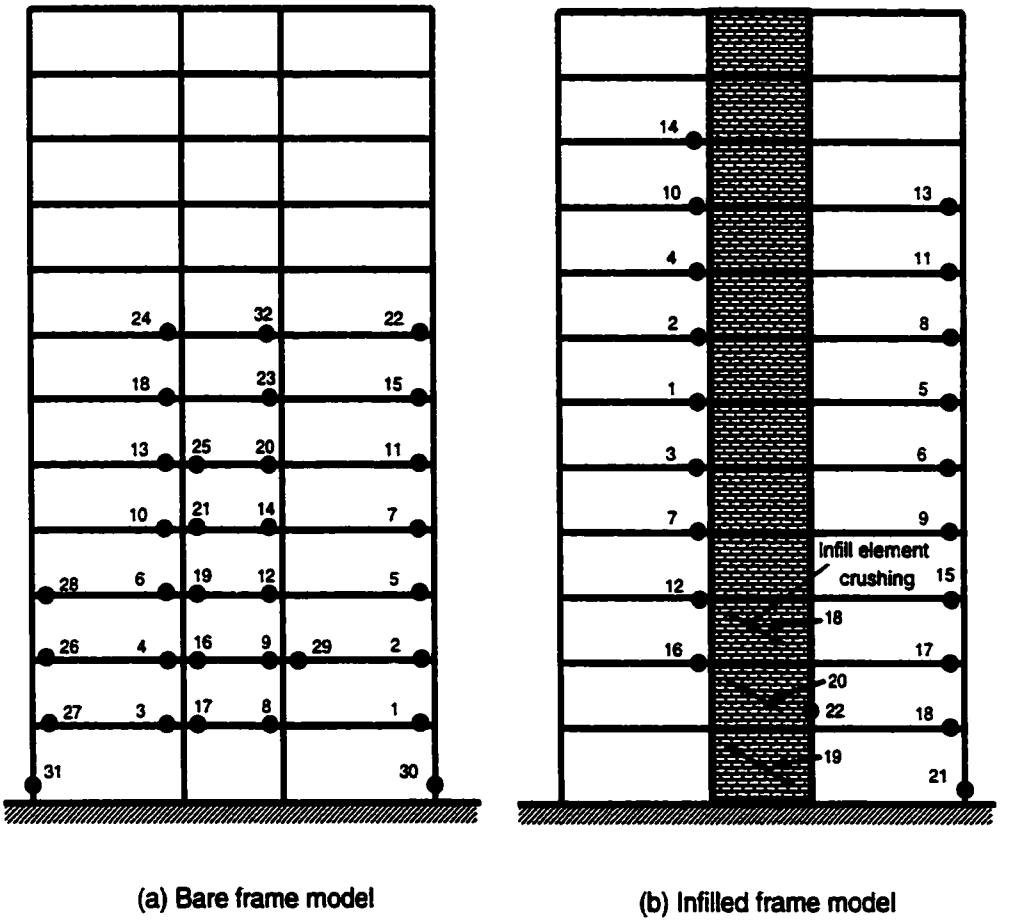
(a) Bare frame model



Frame A : Frame with infill panels
Frame B : Associated bare frame

(b) Infilled frame model

Figure 3.10: Hinge formation in the six-storey building in Victoria (push-over analysis)



(a) Bare frame model

(b) Infilled frame model

Figure 3.11: Hinge formation in the twelve-storey building in Victoria (push-over analysis)

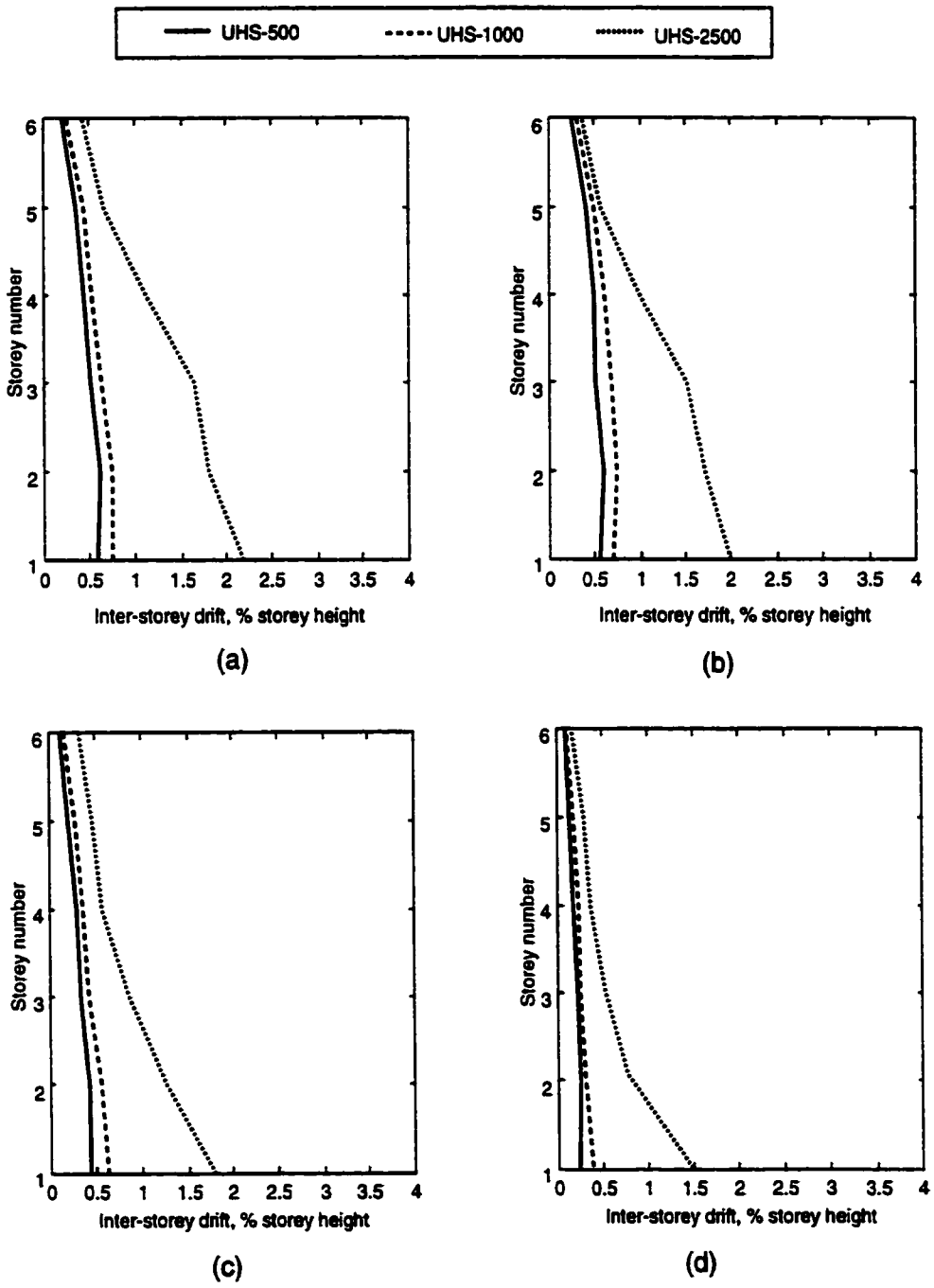
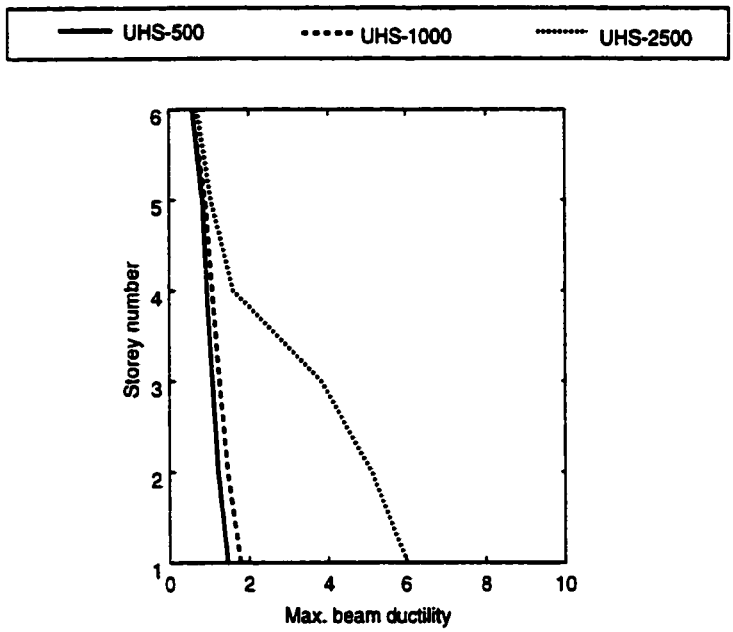


Figure 3.12: Six-storey building in Victoria - maximum inter-storey drift: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength



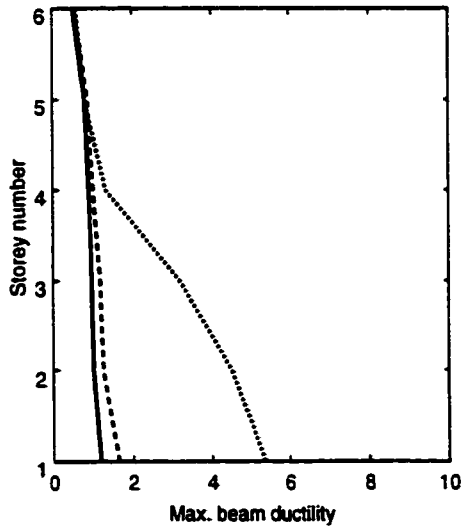


Figure 3.13: Six-storey building in Victoria - maximum beam ductility (bare frame model): (a) factored strength, (b) nominal strength

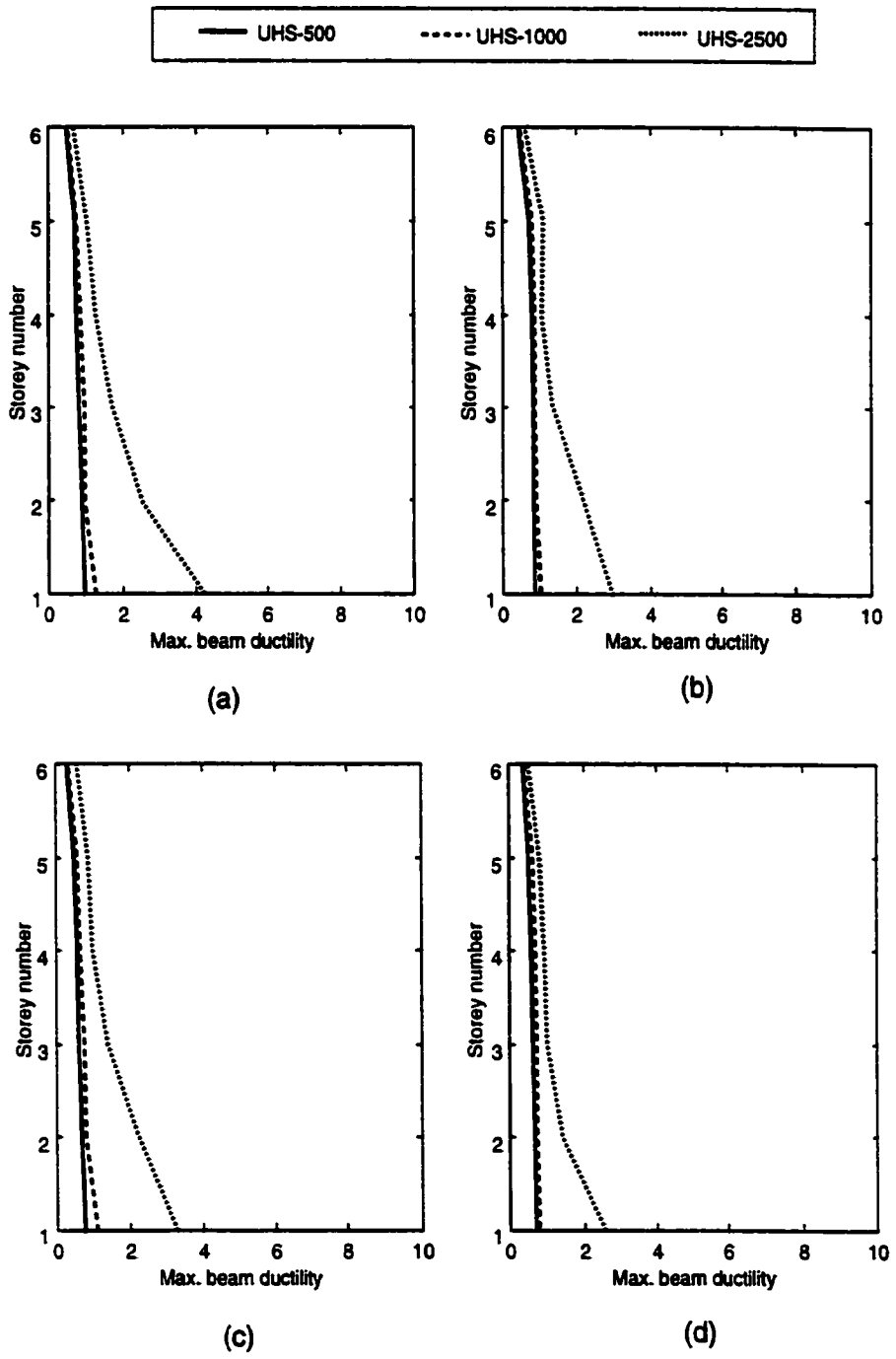
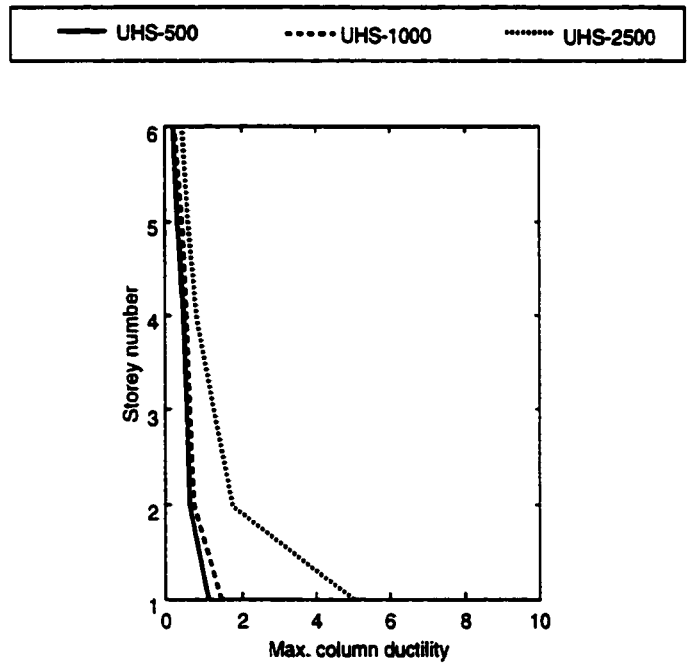
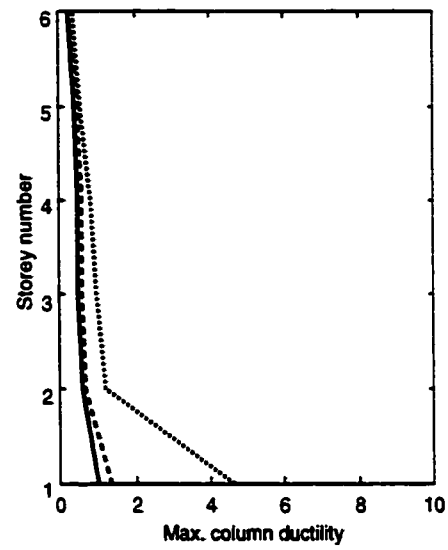


Figure 3.14: Six-storey building in Victoria - maximum beam ductility (infilled frame model): (a) Infilled frame - factored strength, (b) Infilled frame - nominal strength (c) Associated bare frame - factored strength, (d) Associated bare frame - nominal strength



(a)



(b)

Figure 3.15: Six-storey building in Victoria - maximum column ductility (bare frame model): (a) factored strength, (b) nominal strength

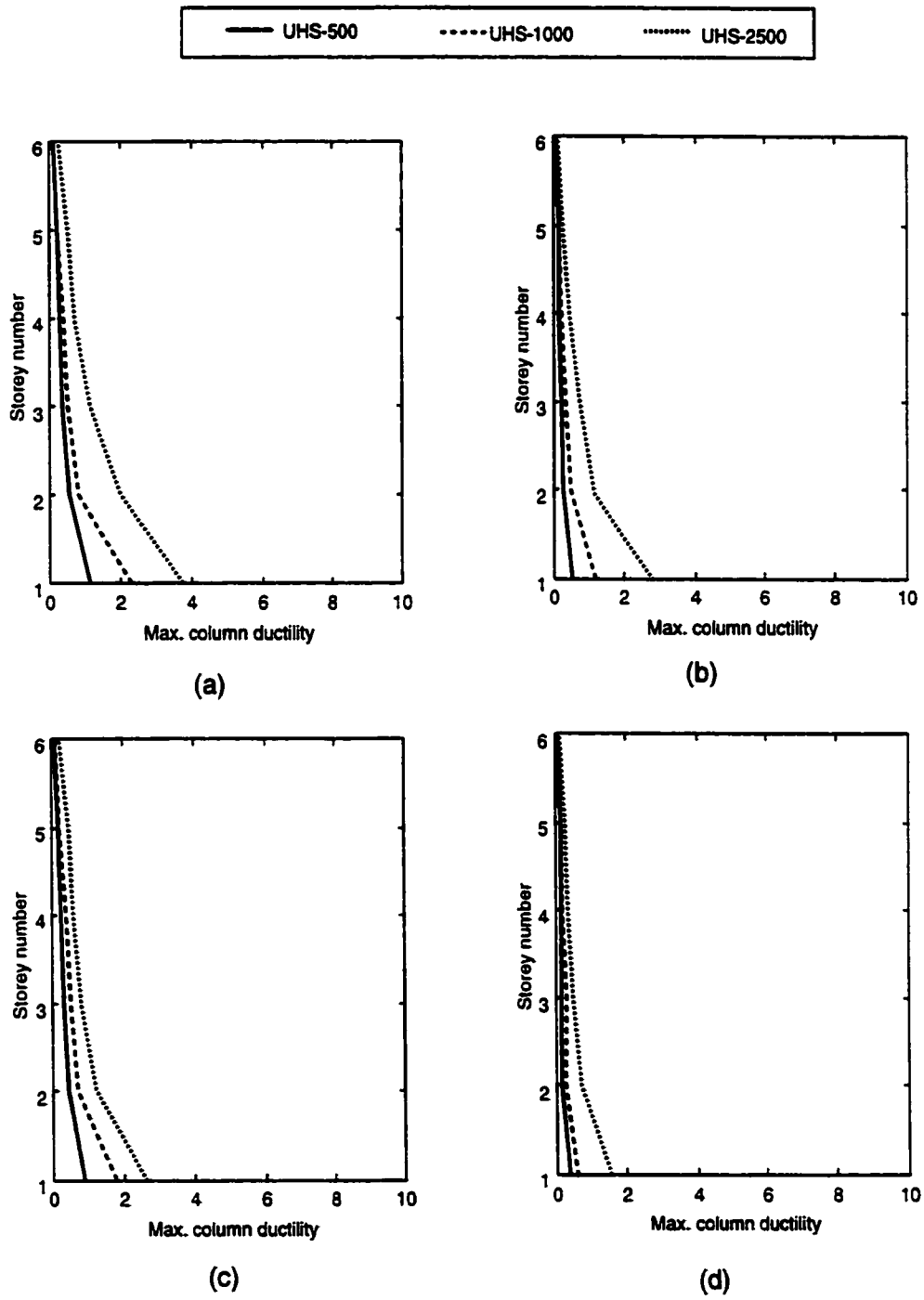


Figure 3.16: Six-storey building in Victoria - maximum column ductility (infilled frame model): (a) Infilled frame - factored strength, (b) Infilled frame - nominal strength (c) Associated bare frame - factored strength, (d) Associated bare frame - nominal strength

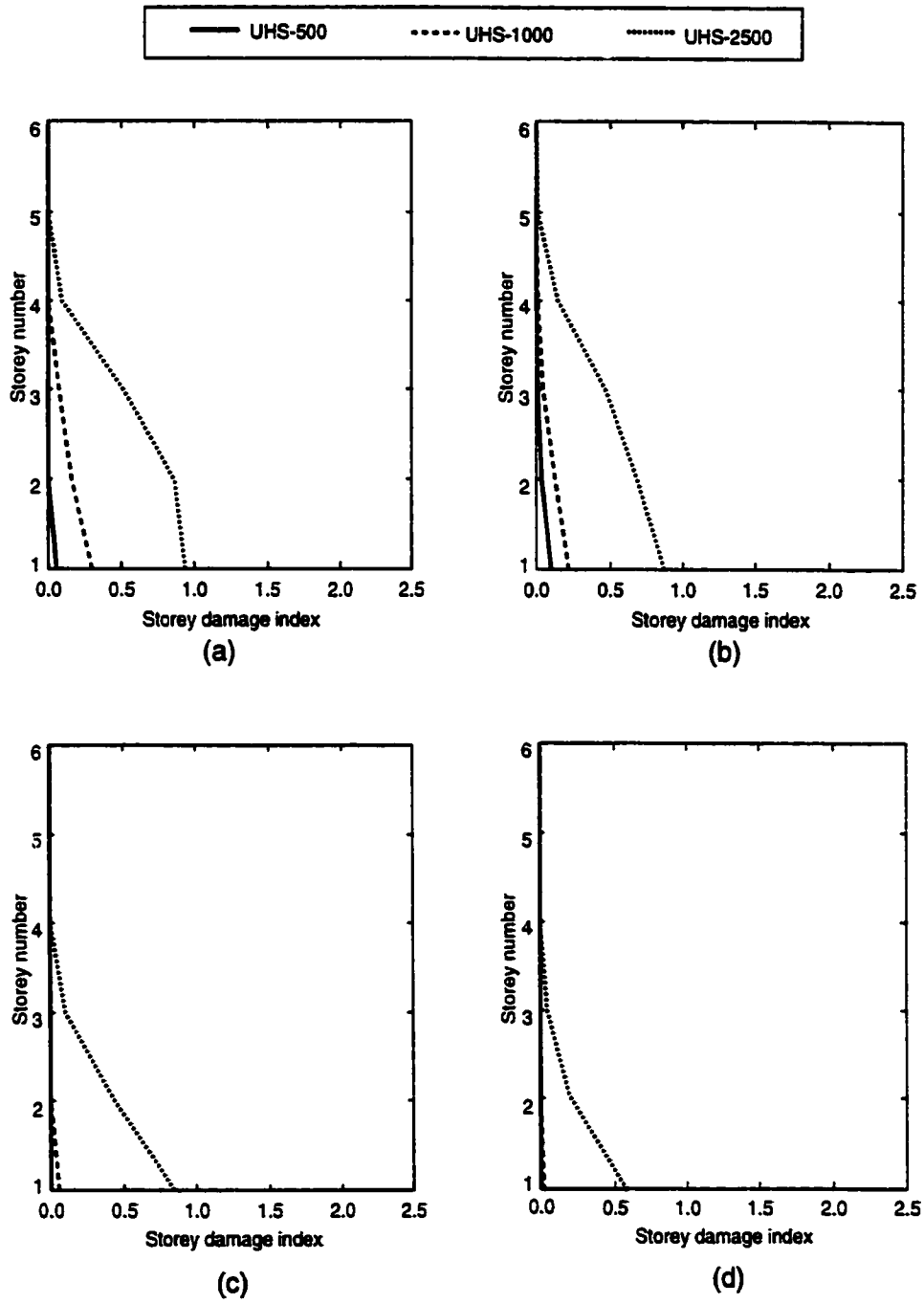


Figure 3.17: Six-storey building in Victoria - storey level Park-Ang damage index: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

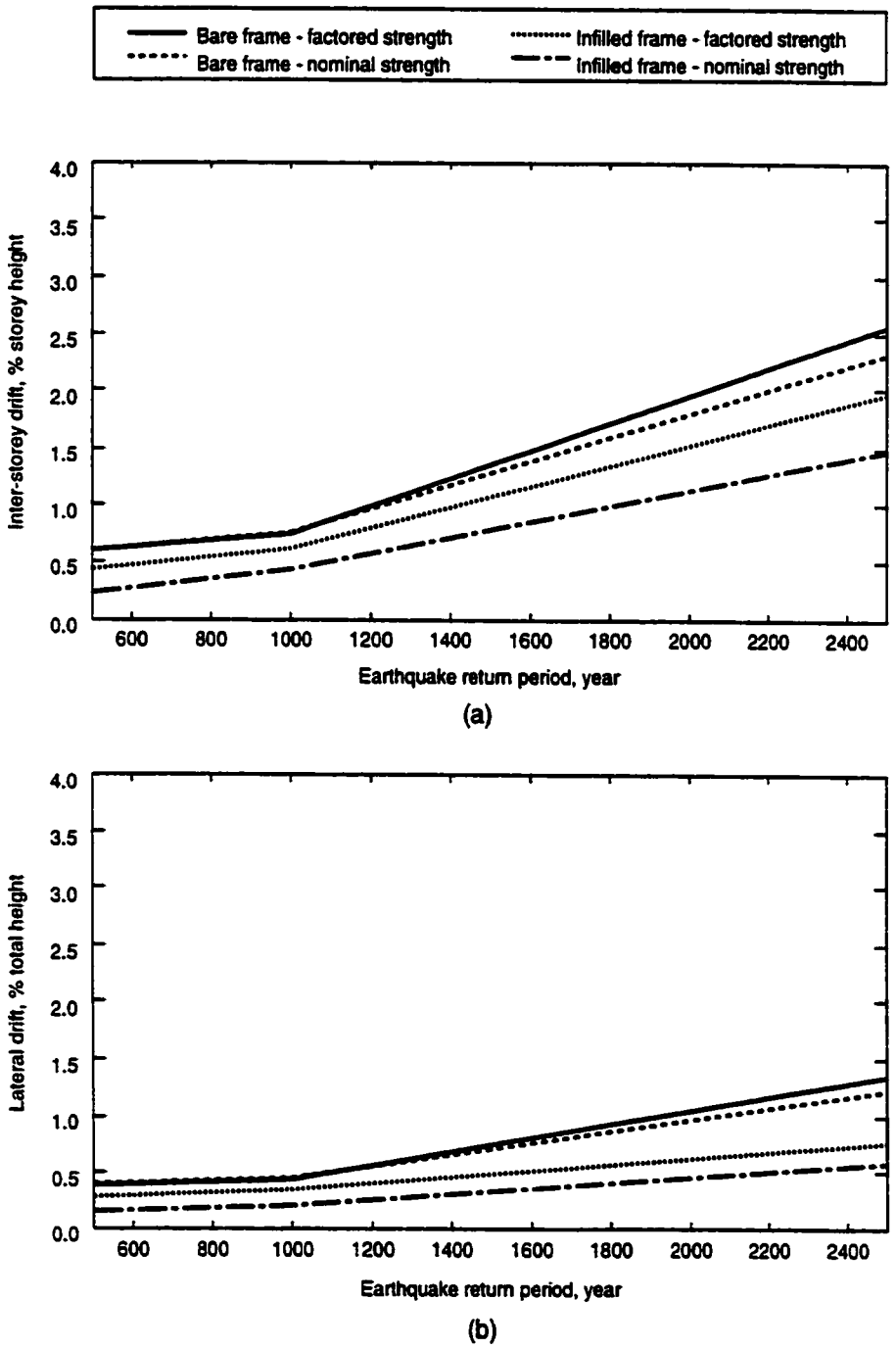


Figure 3.18: Peak displacement response in the six-storey building in Victoria: (a) inter-storey drift, (b) roof-displacement

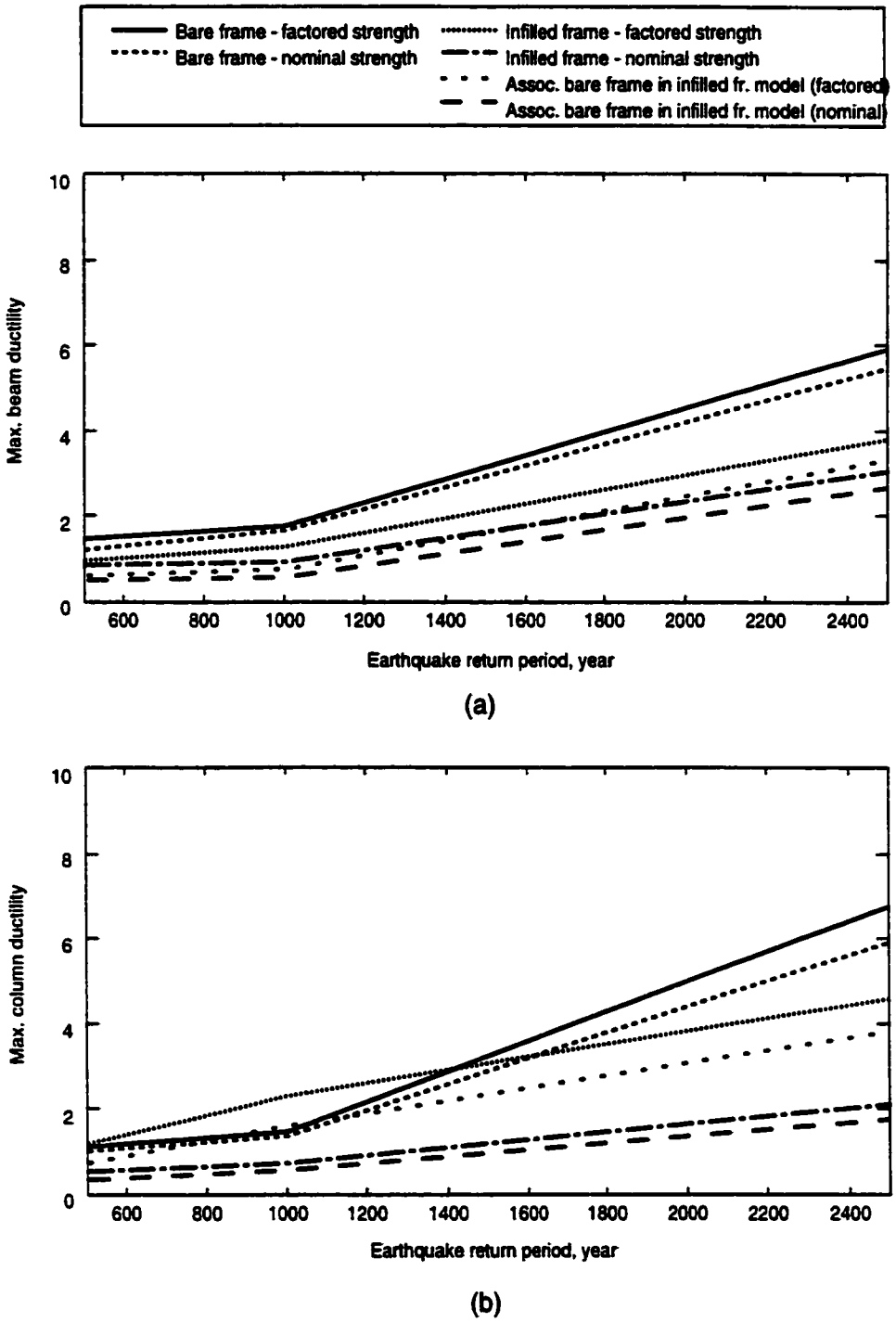


Figure 3.19: Maximum beam and column ductility demand for the six-storey building in Victoria: (a) beam, (b) column

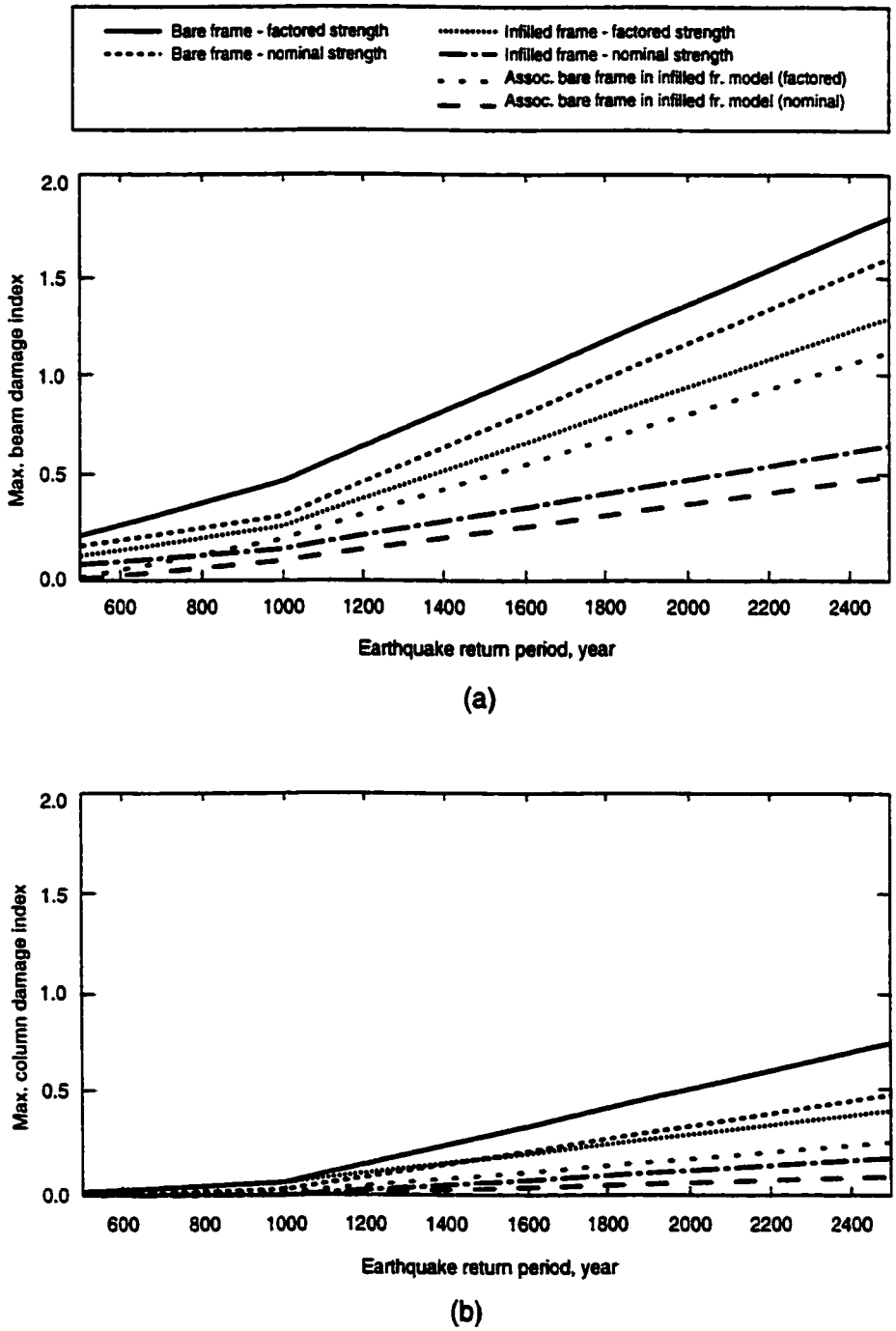


Figure 3.20: Maximum element level Park-Ang damage index for the six-storey building in Victoria: (a) beams, (b) columns

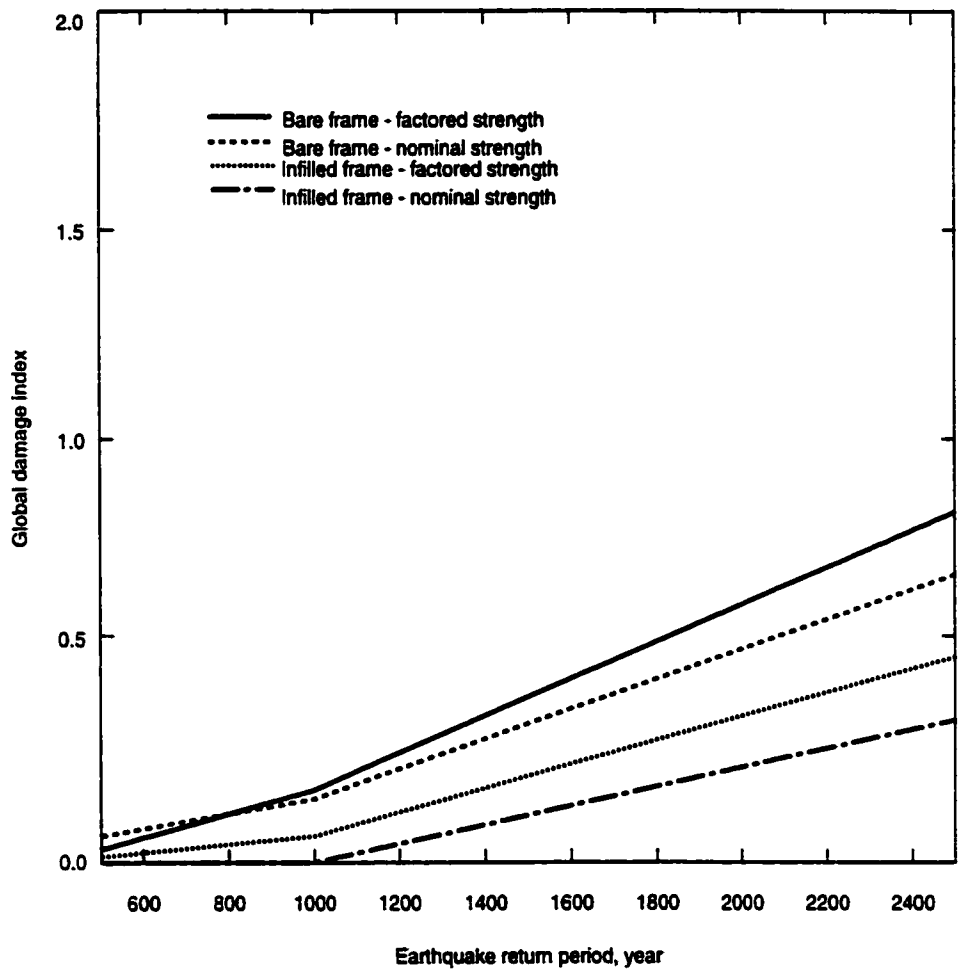
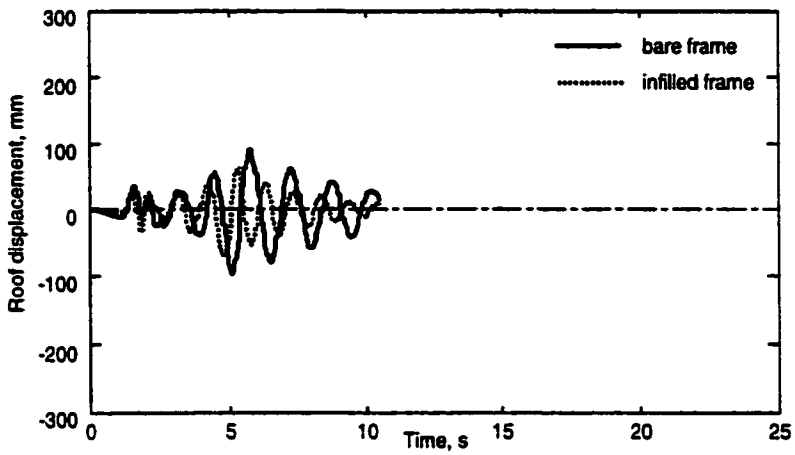
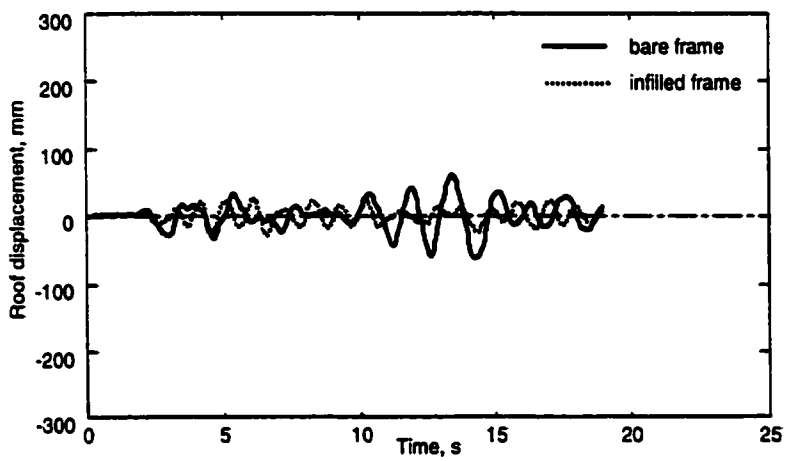


Figure 3.21: Global Park-Ang damage index for the six-storey building in Victoria



(a)



(b)

Figure 3.22: Response time histories for bare and infilled frames, UHS-500 events (nominal material strength): (a) short duration event - Trial 1, (b) long duration event - Trial 1

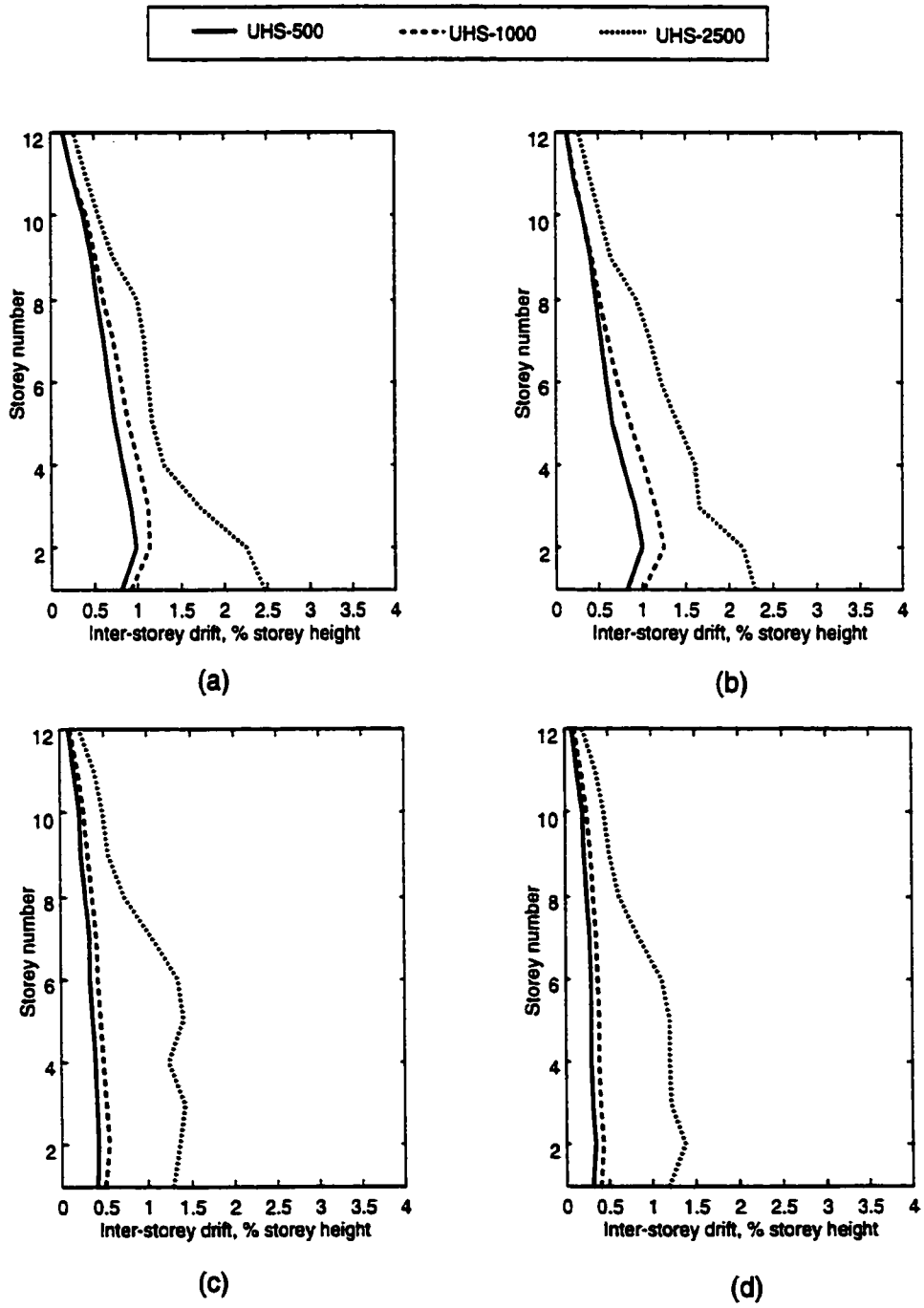


Figure 3.23: Twelve-storey building in Victoria - maximum inter-storey drift: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

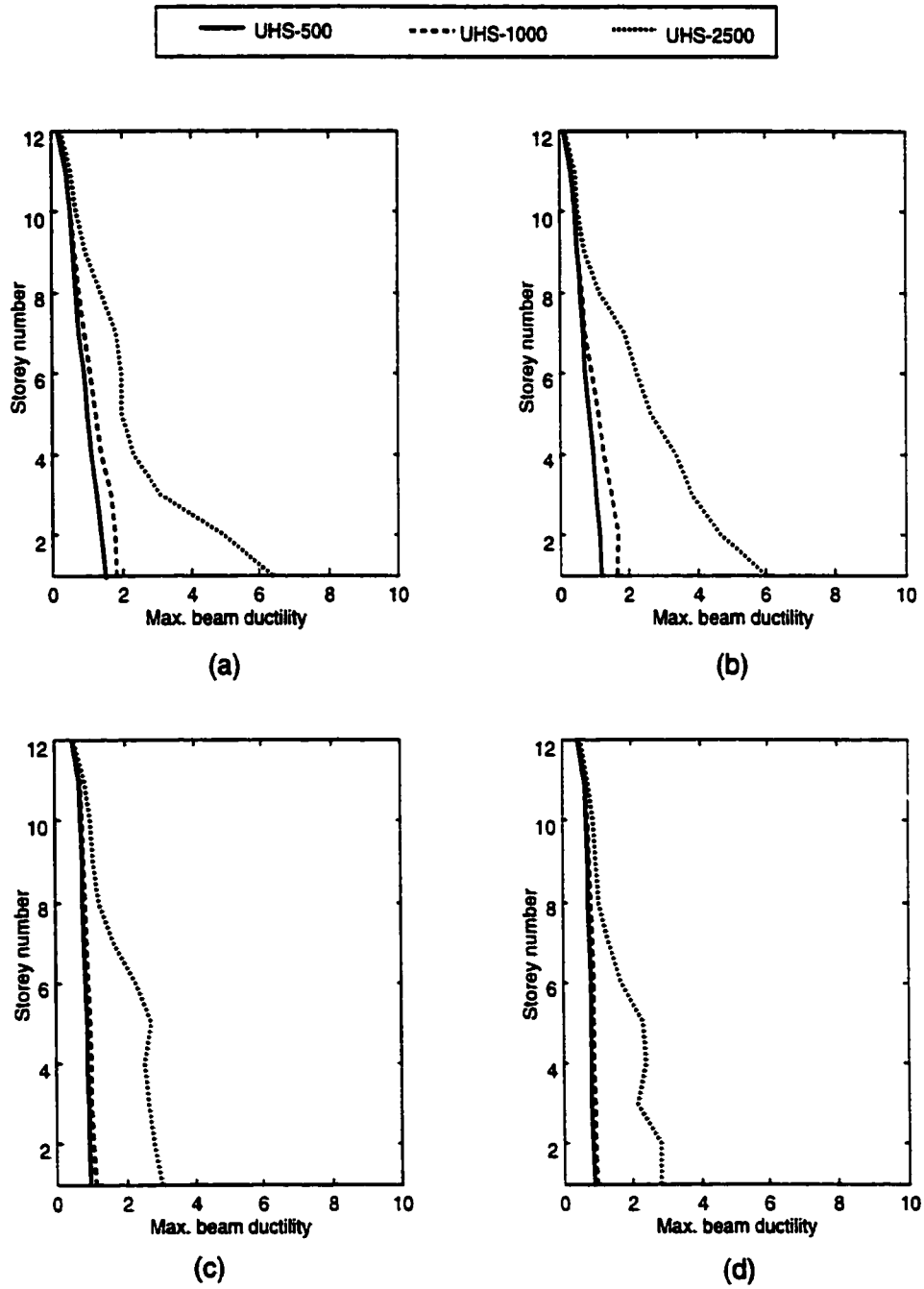


Figure 3.24: Twelve-storey building in Victoria - maximum beam ductility: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

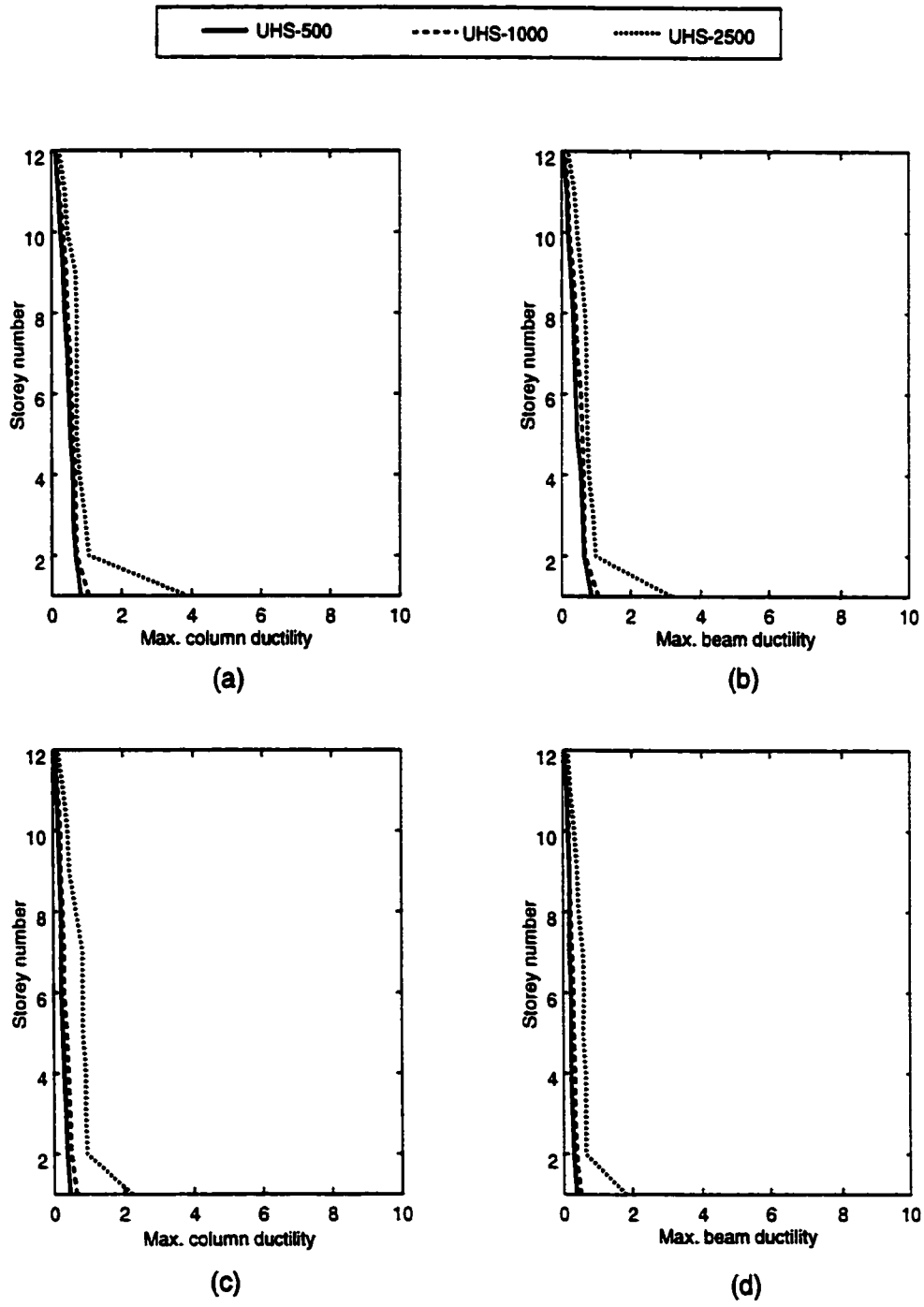


Figure 3.25: Twelve-storey building in Victoria - maximum column ductility: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

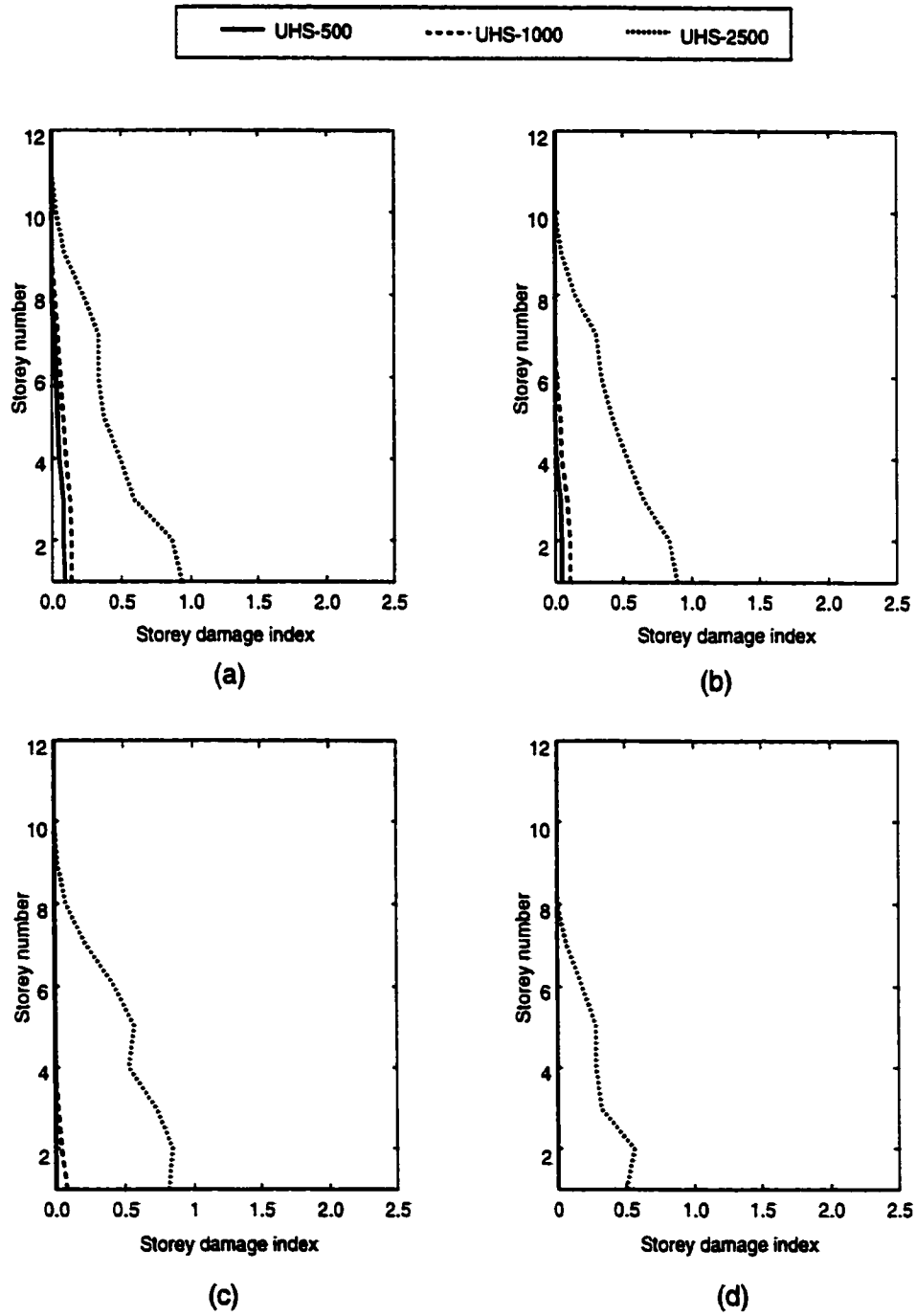


Figure 3.26: Twelve-storey building in Victoria - storey level damage index: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

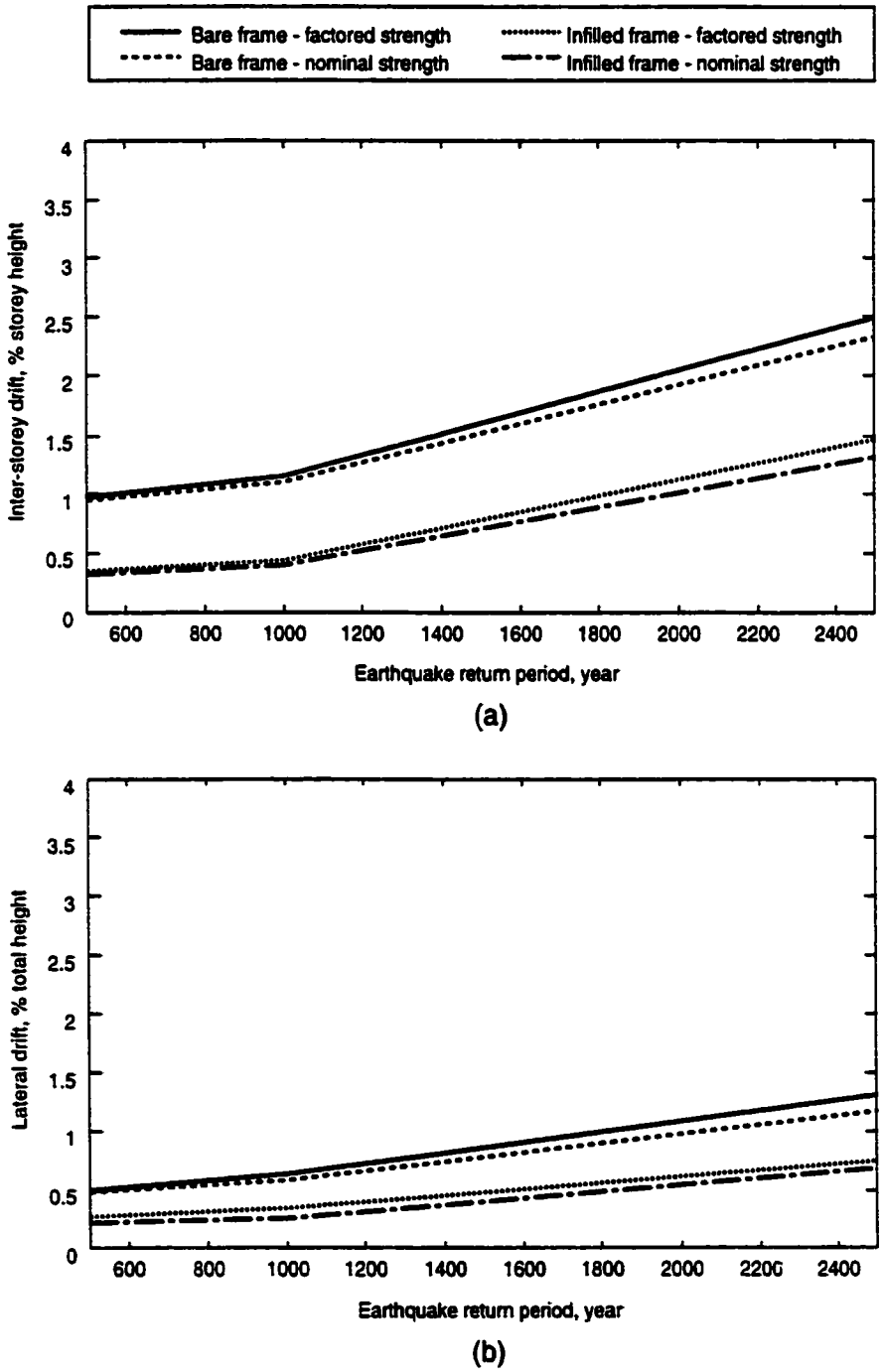


Figure 3.27: Peak displacement response in the twelve-storey building in Victoria: (a) inter-storey drift, (b) roof-displacement

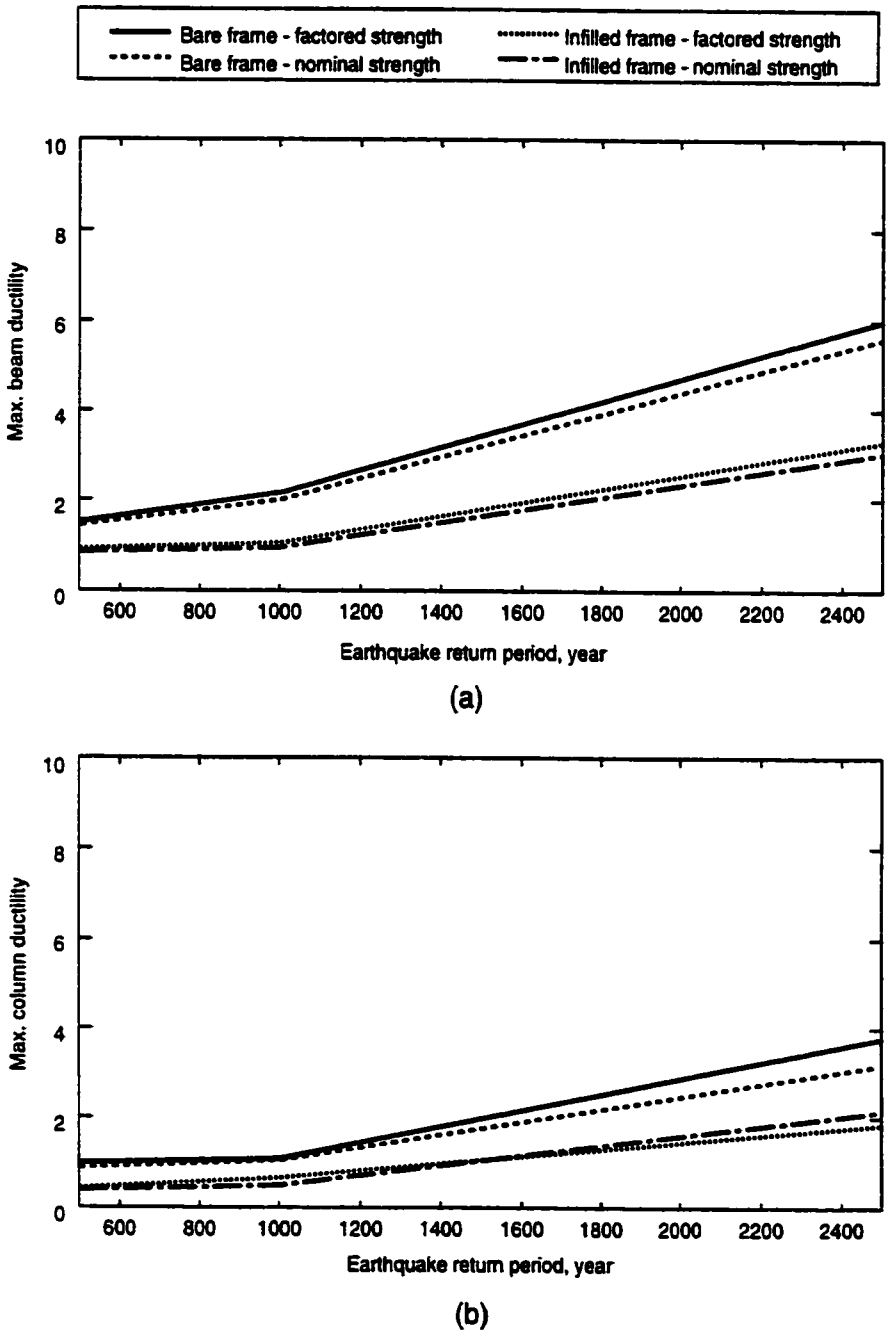


Figure 3.28: Maximum beam and column ductility for the twelve-storey building in Victoria: (a) beam, (b) column

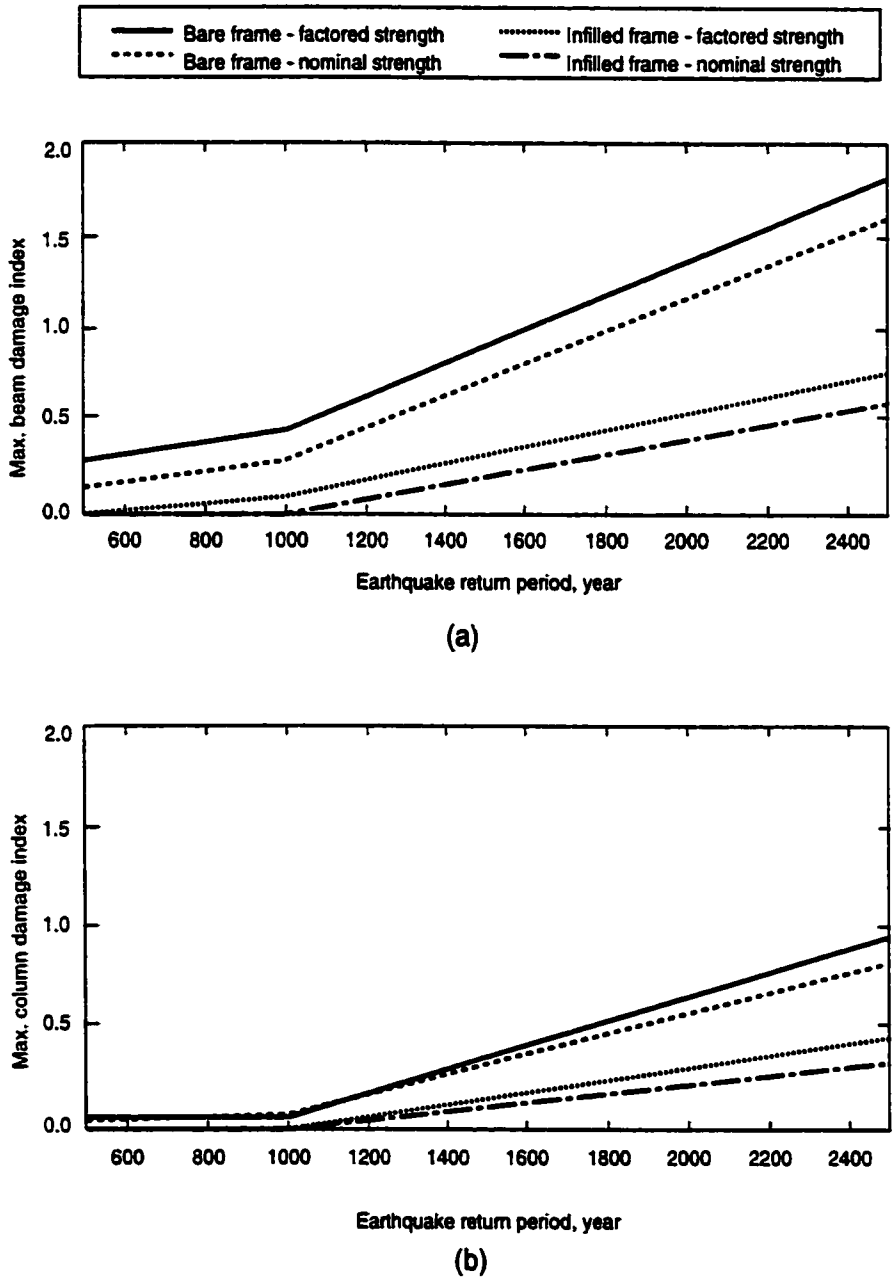


Figure 3.29: Maximum element level Park-Ang damage index for the twelve-storey building in Victoria: (a) beams, (b) columns

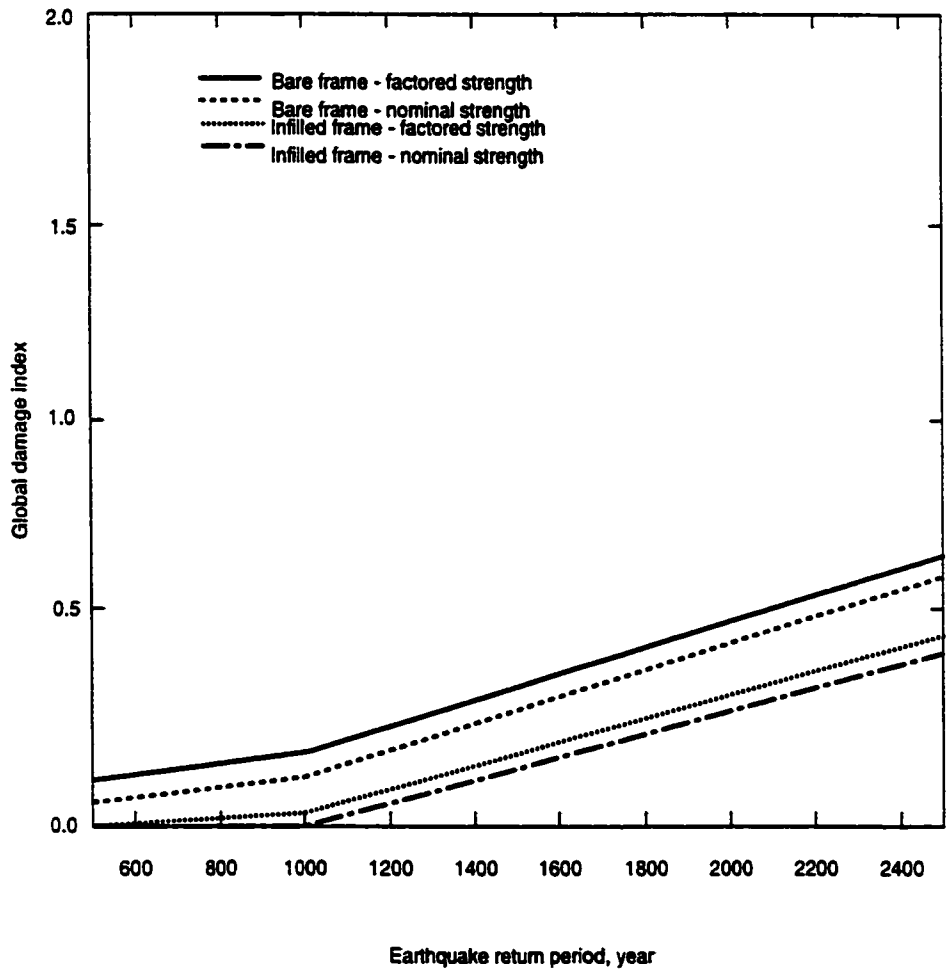
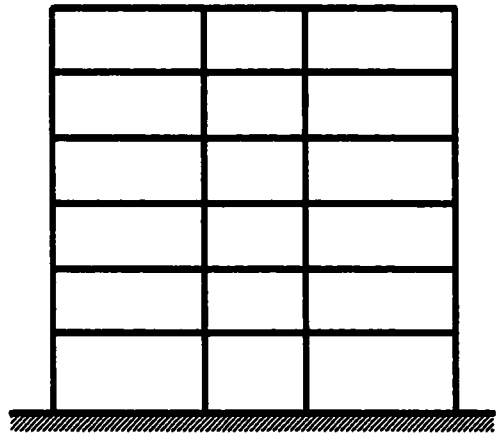
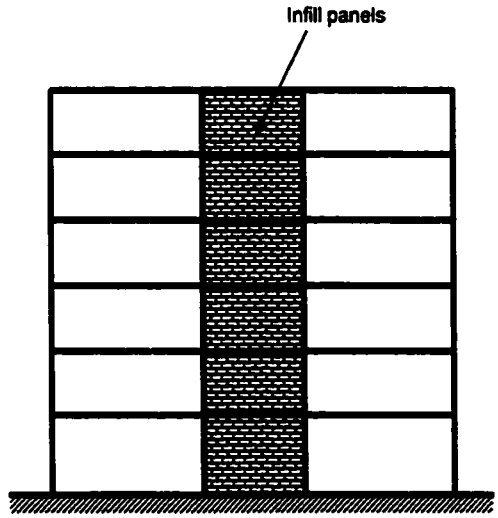


Figure 3.30: Global Park-Ang damage index for the twelve-storey building in Victoria



(a) Bare frame model



(b) Infilled frame model

Figure 3.31: Frame models for the six-storey building in Montreal

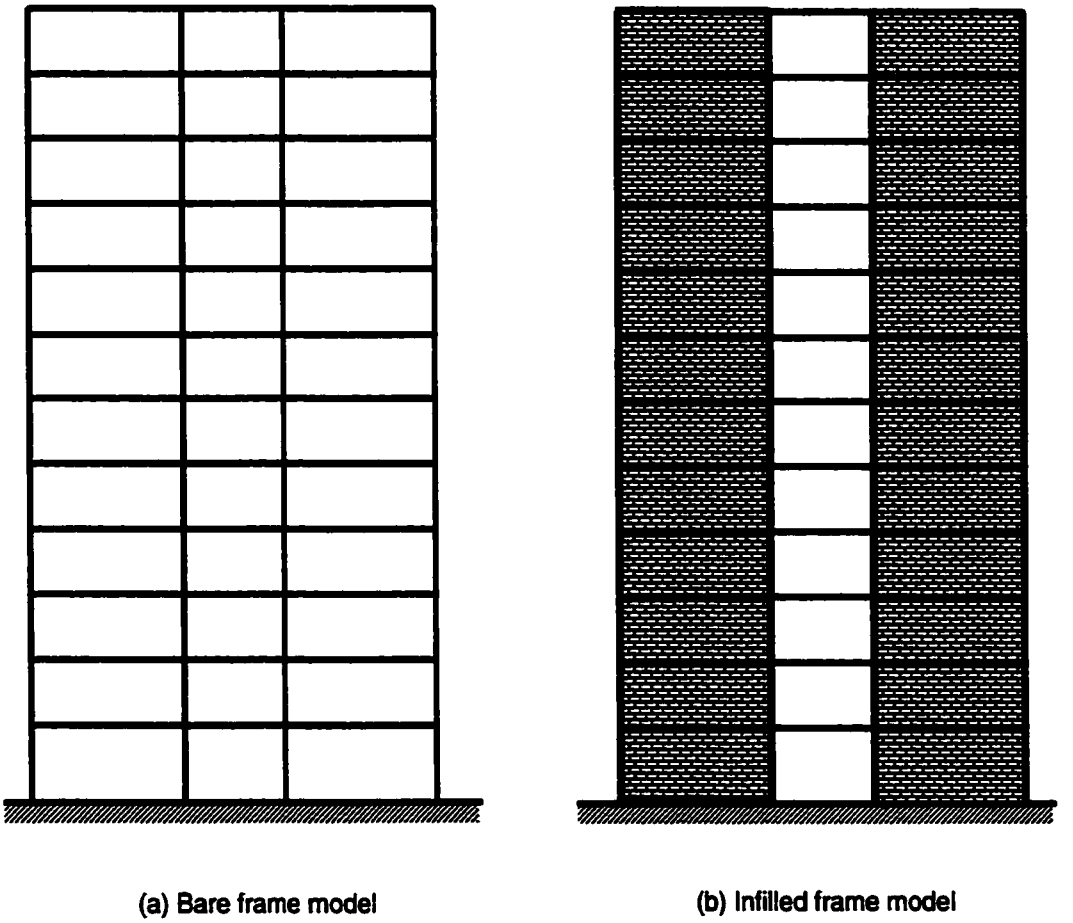


Figure 3.32: Frame models for the twelve-storey building in Montreal

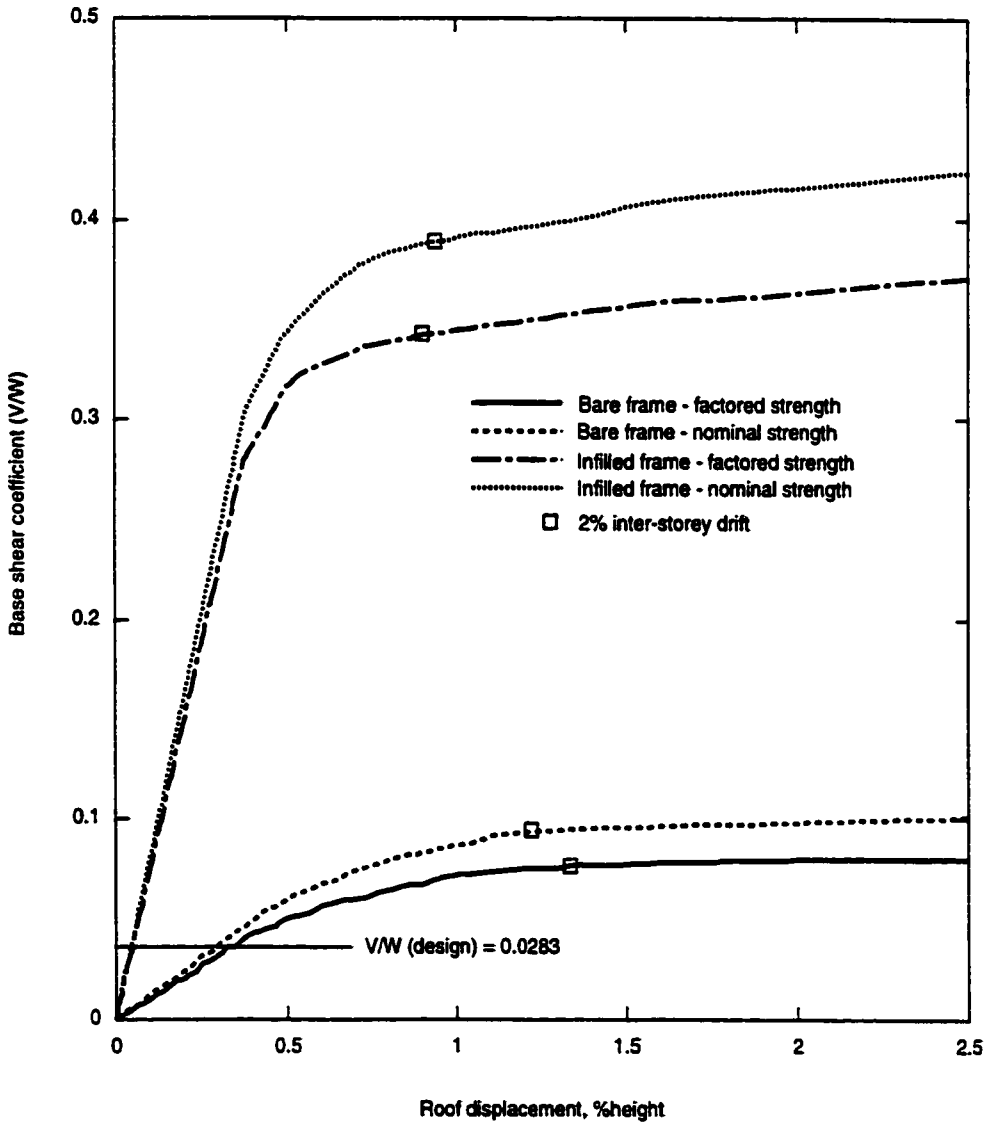


Figure 3.33: Push over curves for the six-storey building in Montreal

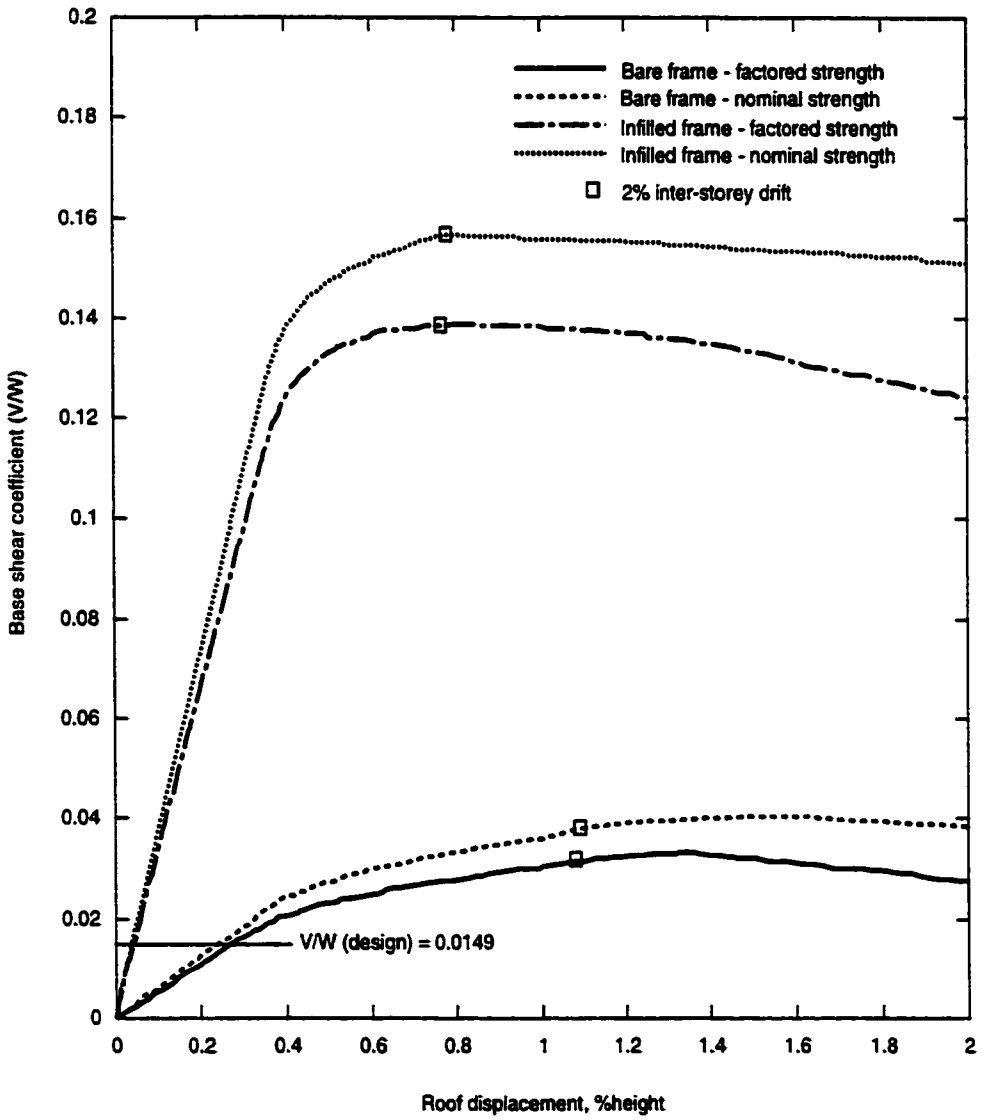
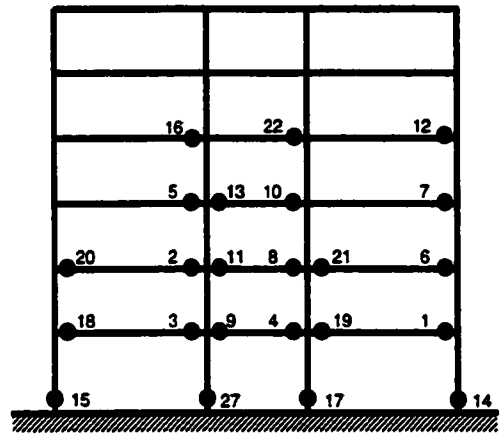
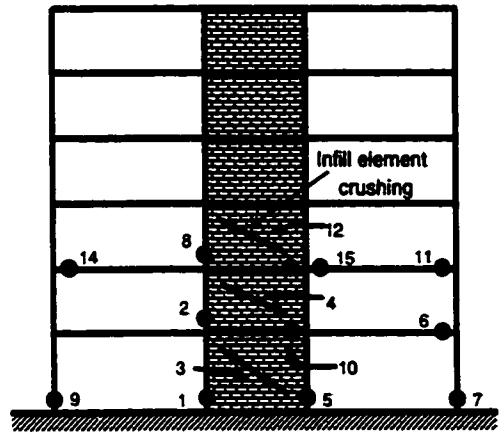


Figure 3.34: Push over curves for the twelve-storey building in Montreal

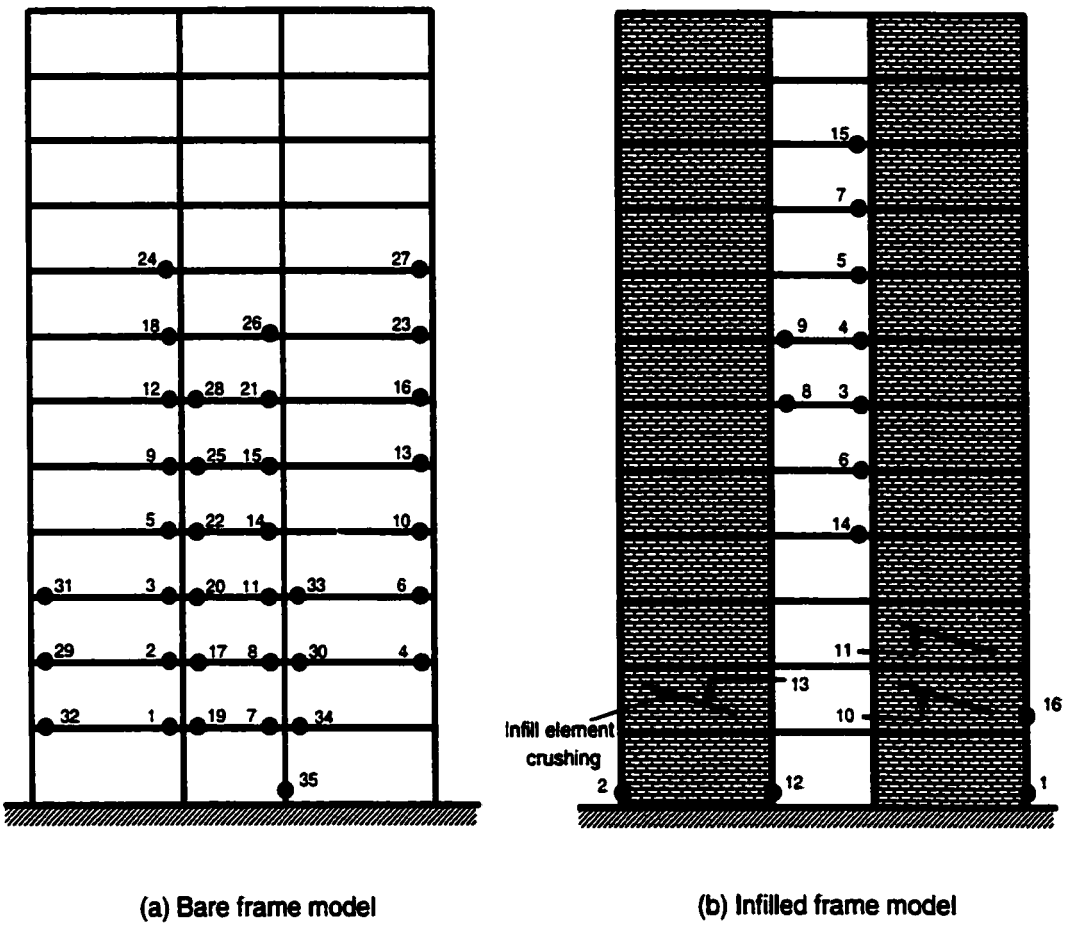


(a) Bare frame model



(b) Infilled frame model

Figure 3.35: Hinge formation in the six-storey building in Montreal (push-over analysis)



(a) Bare frame model

(b) Infilled frame model

Figure 3.36: Hinge formation in the twelve-storey building in Montreal (push-over analysis)

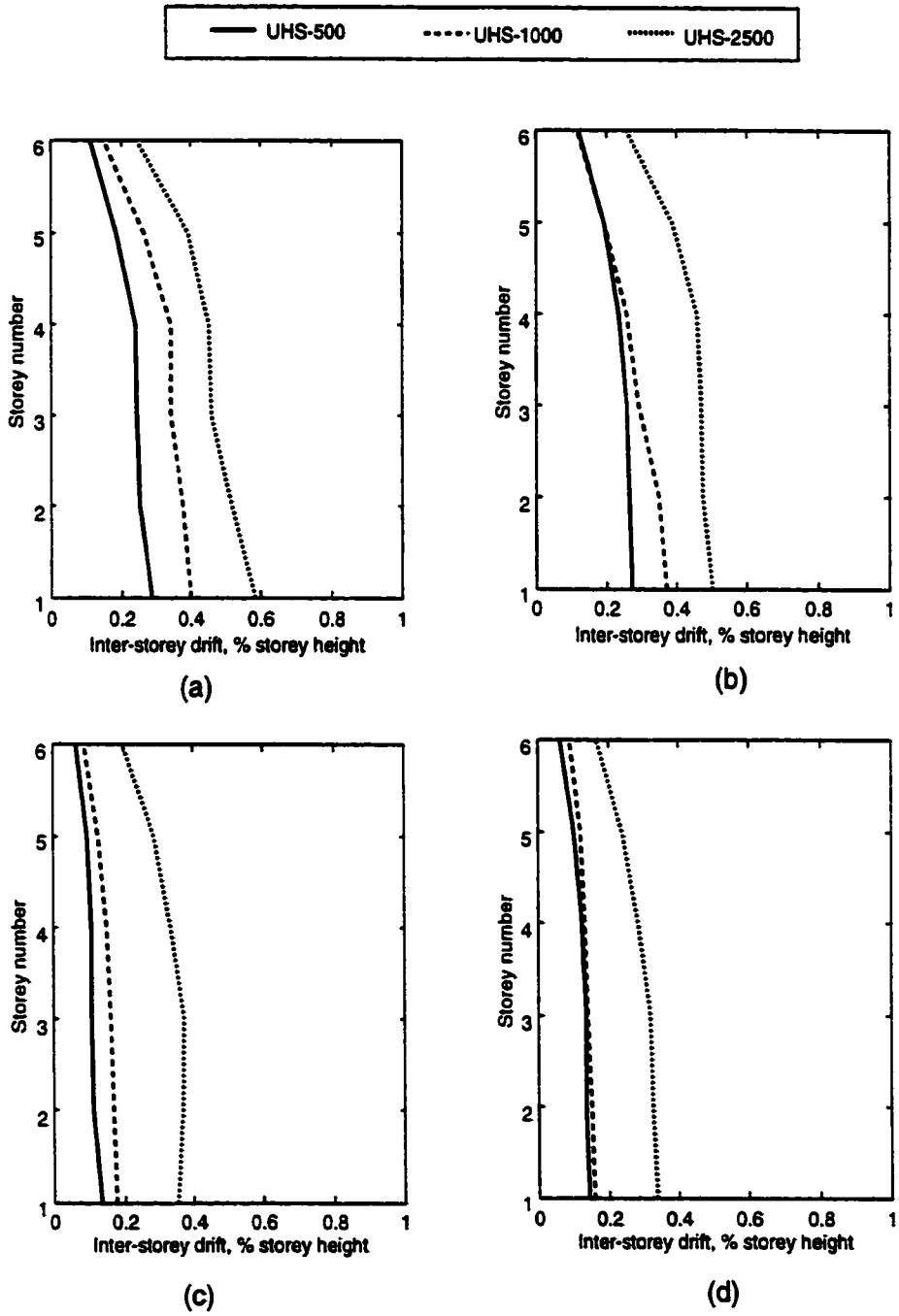


Figure 3.37: Six-storey building in Montreal - maximum inter-storey drift: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

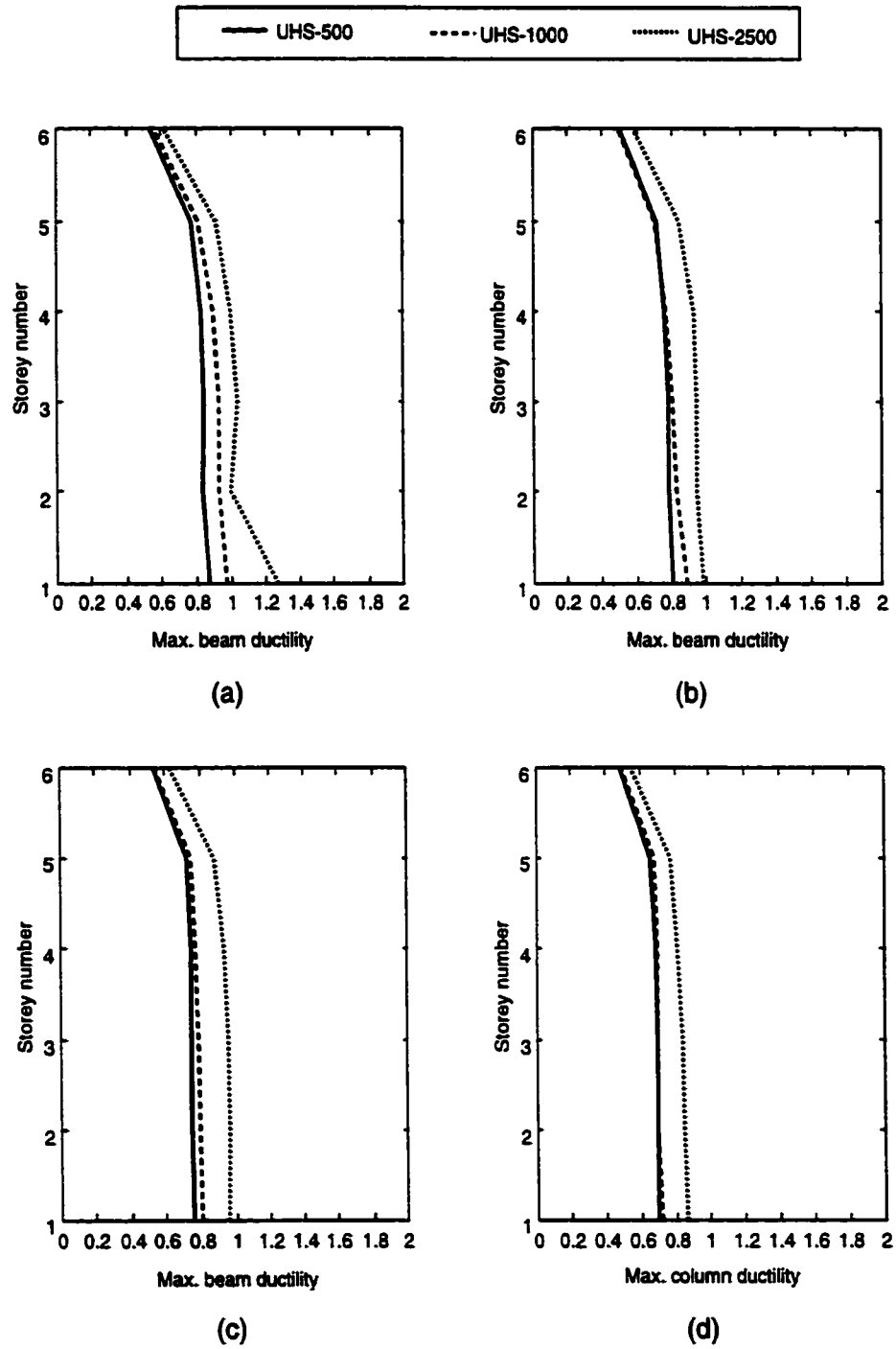


Figure 3.38: Six-storey building in Montreal - maximum beam ductility: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

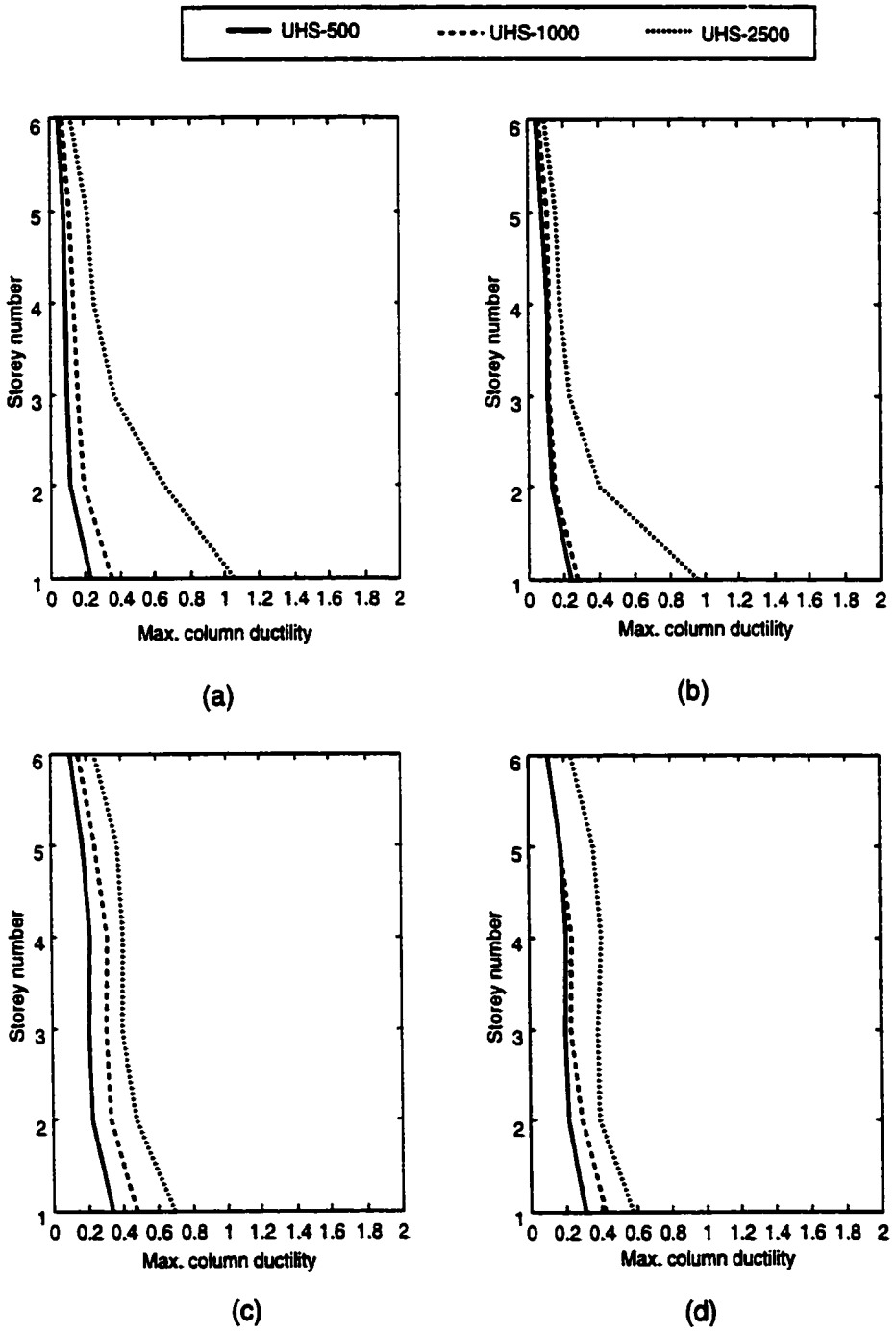
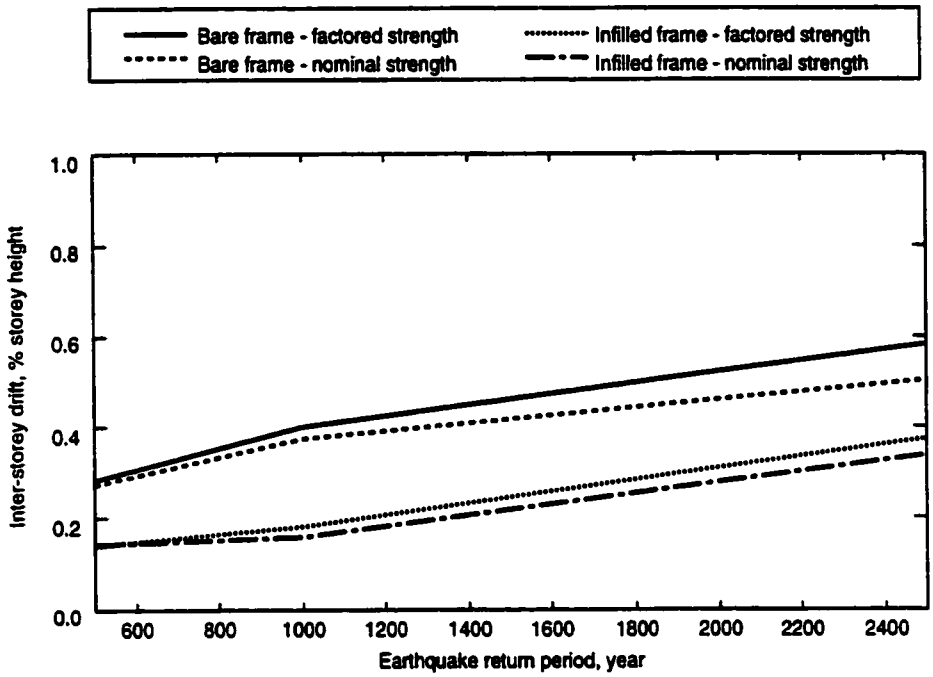
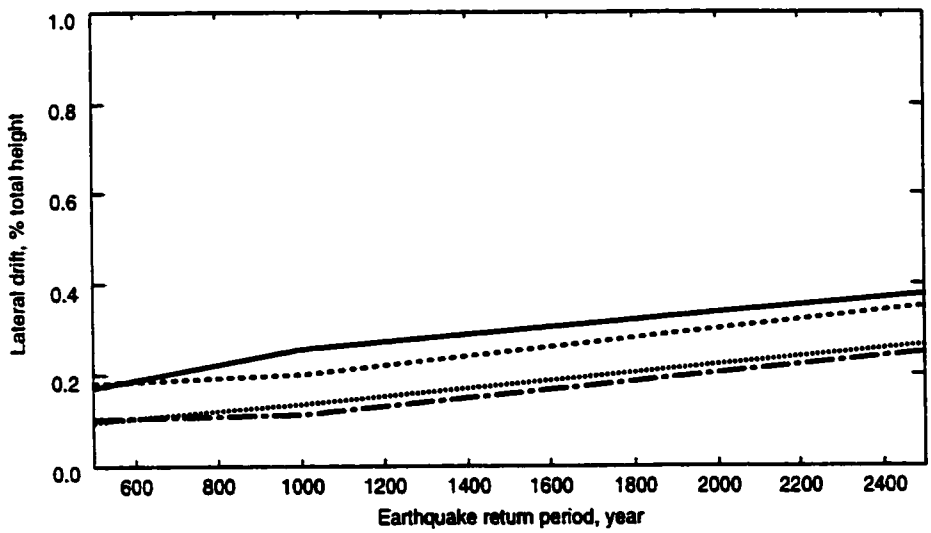


Figure 3.39: Six-storey building in Montreal - maximum column ductility: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength



(a)



(b)

Figure 3.40: Peak displacement response in the six-storey building located in Montreal: (a) inter-storey drift, (b) lateral drift

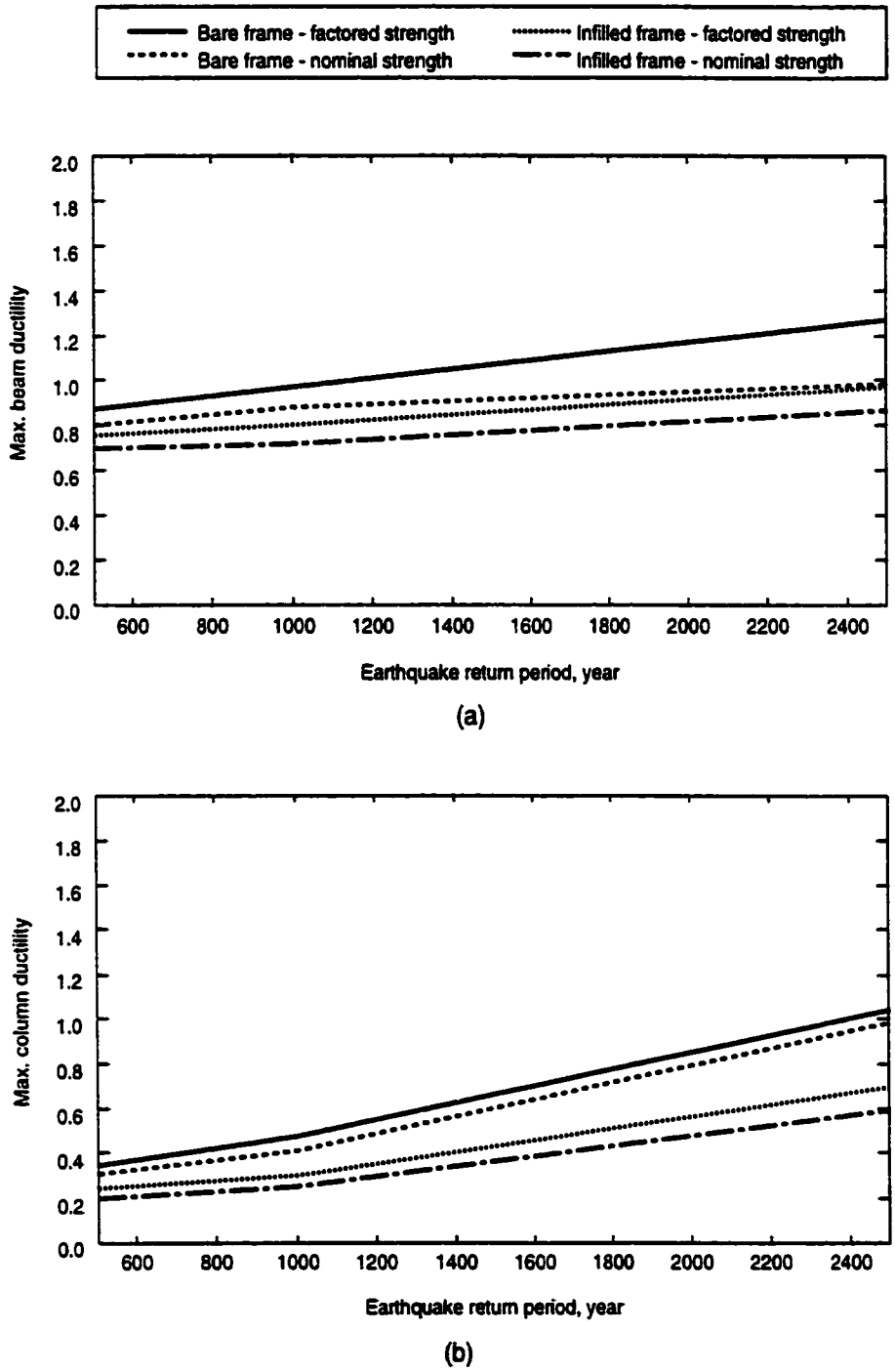


Figure 3.41: Maximum beam and column ductility for the six-storey building in Montreal: (a) beam, (b) column

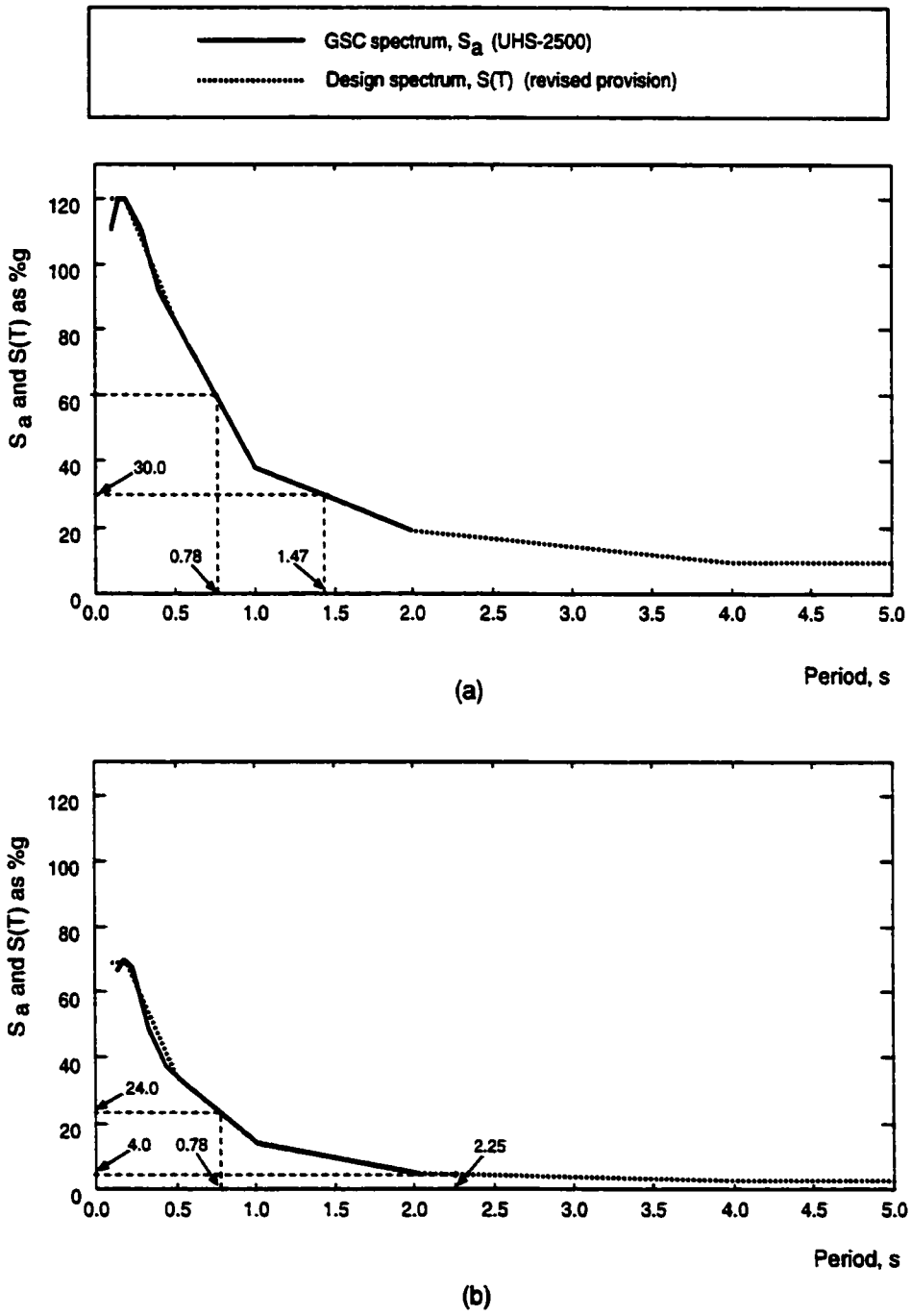


Figure 3.42: Design spectra: (a) Victoria, (b) Montreal

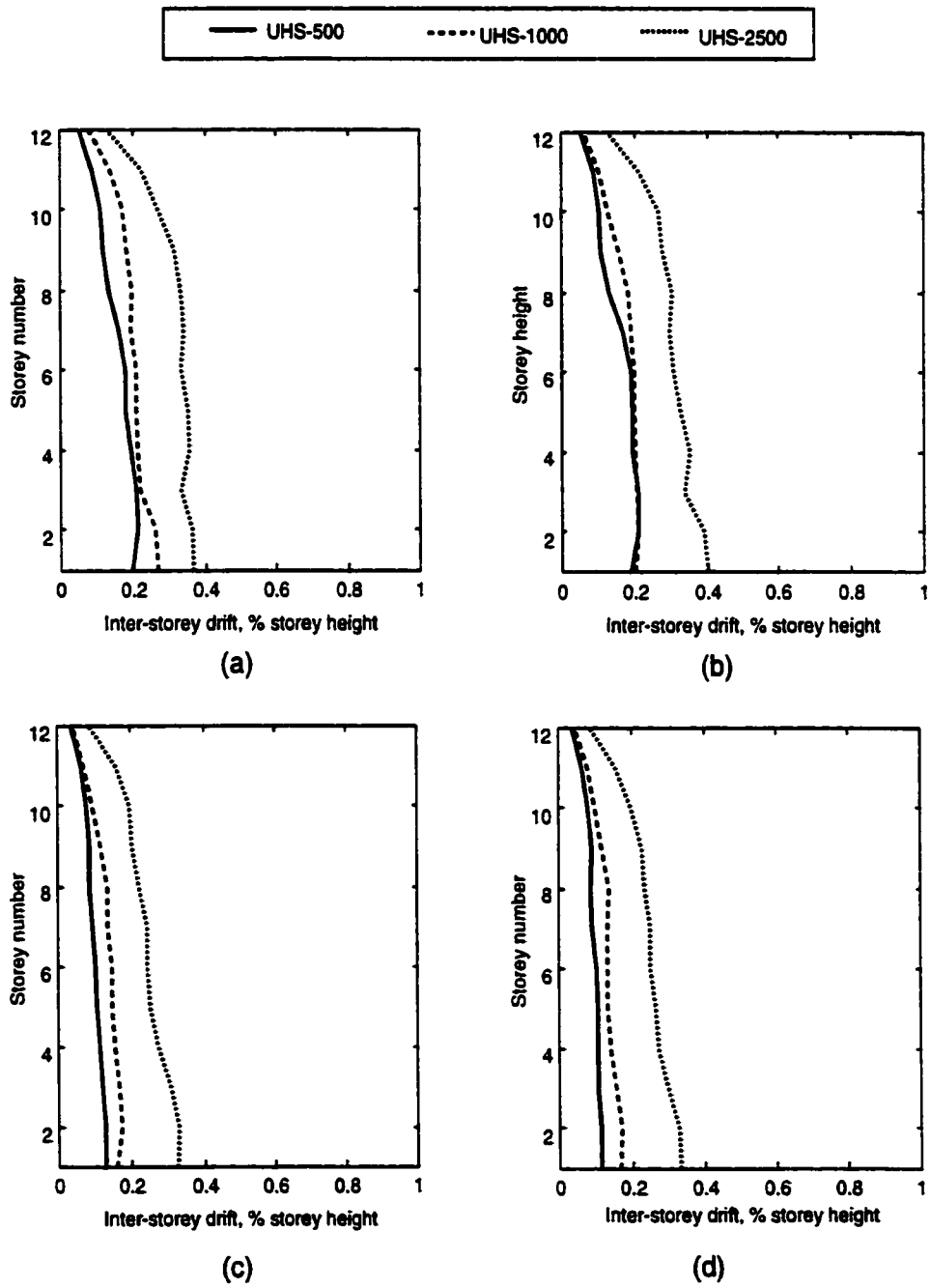


Figure 3.43: Twelve-storey building in Montreal - maximum inter-storey drift: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

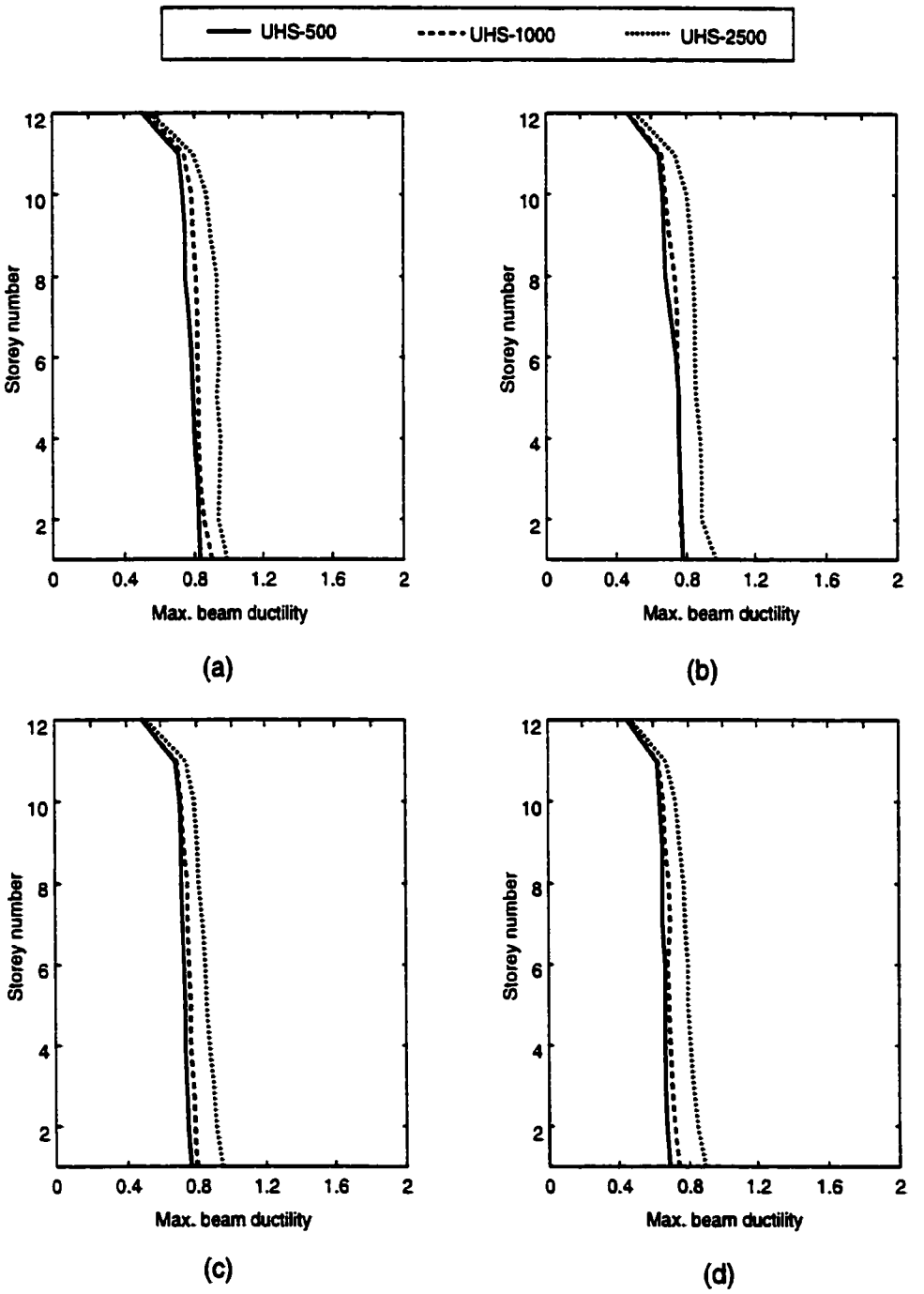


Figure 3.44: Twelve-storey building in Montreal - maximum beam ductility: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

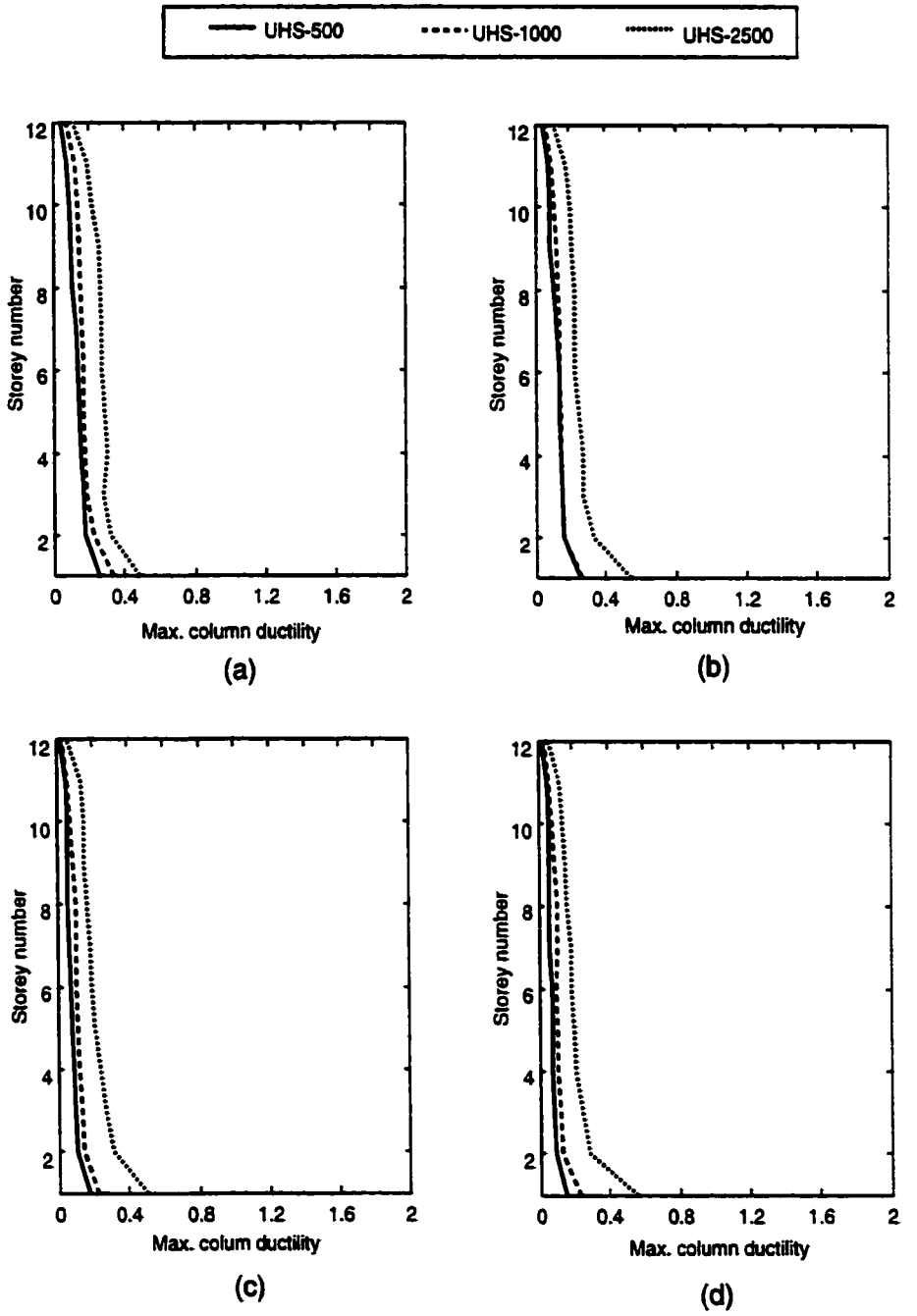


Figure 3.45: Twelve-storey building in Montreal - maximum column ductility: (a) bare frame - factored strength, (b) bare frame - nominal strength, (c) infilled frame - factored strength, (d) infilled frame - nominal strength

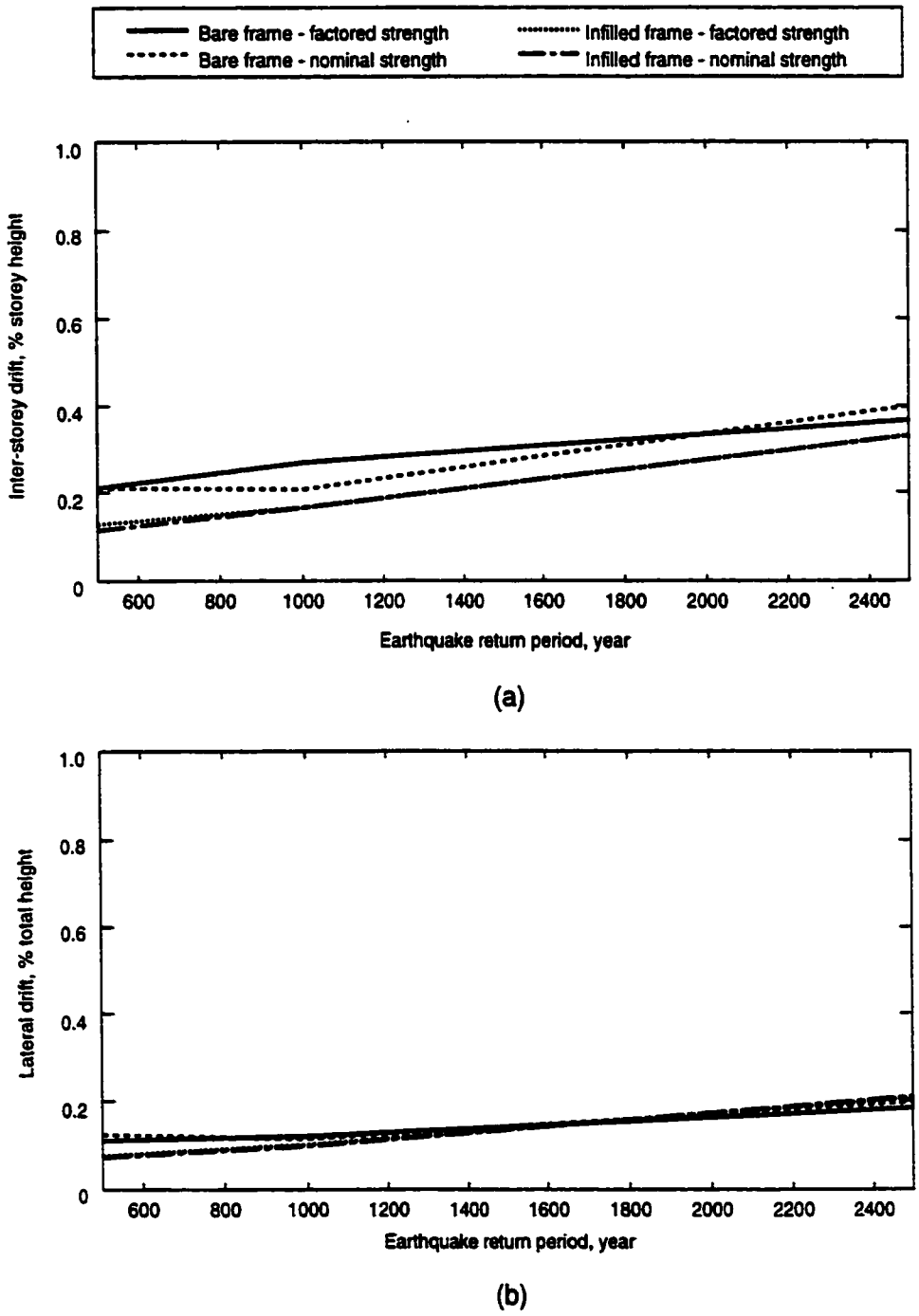


Figure 3.46: Peak displacement response in the twelve-storey building in Montreal: (a) inter-storey drift, (b) lateral drift

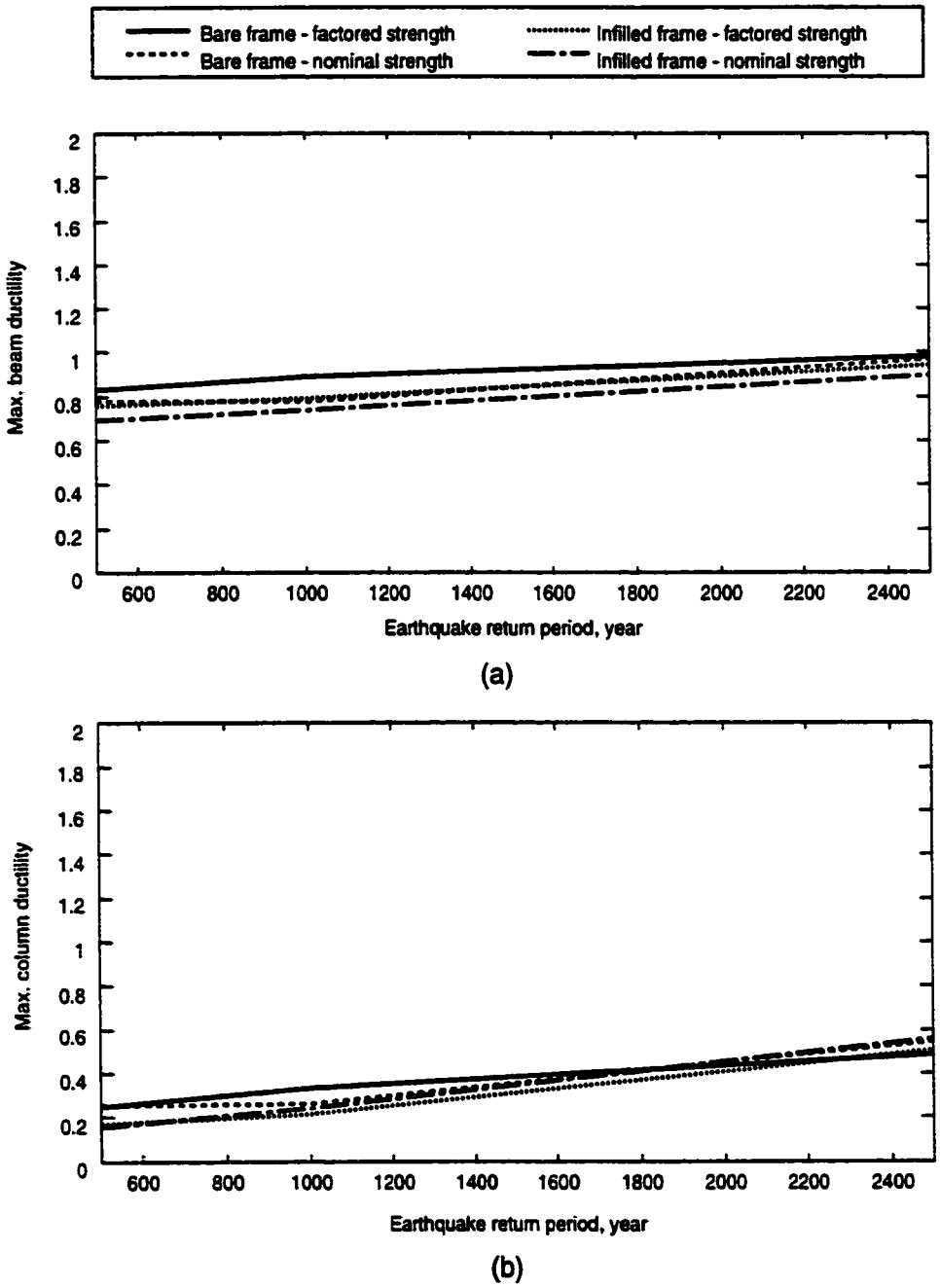


Figure 3.47: Maximum beam and column ductility for the twelve-storey building in Montreal: (a) beam, (b) column

Chapter 4

Concrete Shear Wall Buildings

4.1 Introduction

Structural wall is an important form of lateral load resisting element. When the capacity design principle (Pauley *et al* 1992) is used in the design of such walls they exhibit predominantly ductile flexural behaviour in the nonlinear range. This chapter focuses on an examination of the behaviour and seismic performance of shear wall buildings designed according to the provisions of NBCC 1995 and proposed recommendations for the next version of NBCC (Humar 2000). Twelve and a twenty storey high buildings are considered in the study. The buildings are designed for seismic conditions associated with Victoria and Montreal. Static push-over analyses are carried out to determine the capacity of each structure. The seismic performance of each structure is determined through inelastic dynamic analyses for various levels of seismic hazard. Both single wall and wall-frame models are considered for each building. Nominal and factored values material strength are considered.

4.2 Description and design of the building models

The twelve and twenty storey buildings (in Victoria and Montreal) considered in the study have the same plan and geometric configuration, differing only in heights. The plan comprises two 9 m office bays and a central 6 m bay in E-W direction, and a number of 6 m bays in N-S direction (Figure 4.1). The lateral load-resisting system in the E-W direction comprises a frame-wall in which the two side bays of 9 m each are ductile moment-resisting frames while the central 6 m bay is a shear wall. As stated later not all E-W frames resist lateral loads, and the frames that do not have a shear wall carry only gravity loads. In some models, the 9 m bays on either side of the shear wall are assumed to resist only gravity loads leaving the shear wall to carry the entire load. The storey height is 4.85 m for the first storey and 3.65 m for all the other storeys.

The yield stress, f_y for reinforcing steel, and the 28-day concrete compressive stress, f'_c are assumed to be 400 MPa and 30 MPa, respectively. The following gravity loads are used in the design. Dead load is assumed to be 3.5 kN/m^2 on the roof and 5.0 kN/m^2 on all other floors. Live load is assumed to be 2.2 kN/m^2 on the roof and 2.4 kN/m^2 on all other floors. The structures are designed for the seismic loadings and associated gravity loads. Wind load is not considered in the design (as explained in Chapter 3).

The fundamental period, T of a shear wall building is calculated using the following expression given in NBCC 1995.

$$T = 0.09h_n/\sqrt{D_s} \tag{4.1}$$

where, h_n is the total height of the building, and D_s is the shear wall width.

The seismic lateral forces are obtained using the new UHS based methodology proposed for the next version of NBCC (Equation 1.10) for base shear calculation. The suggested procedure uses a uniform hazard spectrum based on design earthquakes having a return period of 2500 years. The multi-storey effect on the base shear is higher for flexural wall as compared to moment resisting frames. M_v, the factor accounting for multi-storey effect (Equation 1.10), is greater than 1.0 for T exceeding 1.0 s, and it is higher for eastern Canada than that for western Canada. Appropriate values of M_v as suggested in Humar (2000) are used in the design of the buildings studied here.

Table 4.1: Base-shear calculation for shear wall buildings, Part I

Building location	Number of storey	Period, s (NBCC)	$S(T)$	$S_a(0.2)/S_a(2.0)$	M_v	V/W
Victoria	12	1.57	0.2725	5.8	1.11	0.0481
	20	2.00	0.1615	5.8	1.20	0.0308
Montreal	12	1.57	0.0890	14.4	1.85	0.0261
	20	2.00	0.0408	14.4	2.50	0.0162

Details of design base shear calculation are shown in Tables 4.1 and 4.2. Firm

Table 4.2: Base-shear calculation for shear wall buildings, Part II

Building location	Number of storey	Weight (W), kN	Base-shear (V), kN	F_t kN	$V - F_t$ kN
Victoria	12	7372	354.6	38.9	315.7
	20	12412	382.3	69.6	312.7
Montreal	12	14744	384.8	42.2	342.6
	20	24824	402.1	73.2	328.9

(reference) ground condition is assumed, $I = 1.0$, $R_O = 1.8$ and $R_d = 3.5$ are used for all the cases shown in Table 4.1.

The western Canadian location, Victoria, has a higher level of seismic hazard as compared to the eastern Canadian location, Montreal. For the buildings situated in Victoria every transverse frame is designed to be a lateral load-resisting frame with concrete shear wall, while for the buildings in Montreal, every alternate frame is considered to be lateral load-resisting shear wall frame. Other transverse frames are designed to carry only gravity loads and are assumed not to participate in the lateral load-resisting system. The lateral load-resisting shear wall frames are designed as fully ductile with a force modification factor, ductility factor R_d equal to 3.5 and overstrength factor R_O equal to 1.8, as provided in Humar (2000). The design base shear of these buildings are shown in Tables 4.1 and 4.2.

The design base shear is distributed across the height of the building frames, using the procedure suggested by NBCC 1995 to obtain the floor level forces. The floor level forces in each of these buildings are shown in Tables 4.3 through 4.6.

Reinforced concrete members are designed according to the provisions of CSA

Table 4.3: Base-shear distribution in twelve-storey shear wall building in Victoria

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i	F_x, kN
12	44.70	442.0	19757.4	74.00
11	41.05	630.0	25861.5	45.93
10	37.40	630.0	23562.0	41.85
9	33.75	630.0	21262.5	37.75
8	30.10	630.0	18963.0	33.68
7	26.45	630.0	16663.5	29.60
6	22.80	630.0	14364.0	25.51
5	19.15	630.0	12064.5	21.42
4	15.50	630.0	9765.0	17.34
3	11.85	630.0	7465.5	13.25
2	8.20	630.0	5166.0	9.17
1	4.55	630.0	2866.5	5.10
Total		7372.0	177761.4	354.60

standard A23.3-94 (CSA94). The frames on the side bays of a shear wall frame are designed as being fully ductile. Alternative models in which the side bays are assumed to carry only gravity loads are also analyzed. The reinforcement details for the building frames are shown in Figs. 4.2 and 4.3.

Table 4.4: Base-shear distribution in twenty-storey shear wall building in Victoria

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i	F_x, kN
20	73.90	442.0	32663.8	90.86
19	70.25	630.0	44257.5	28.81
18	66.60	442.0	41958.0	27.31
17	62.95	630.0	39658.5	25.82
16	59.30	630.0	37359.0	24.32
15	55.65	630.0	35059.0	22.82
14	52.00	630.0	32760.0	21.33
13	48.35	630.0	30460.5	19.83
12	44.70	630.0	28161.0	18.33
11	41.05	630.0	25861.5	16.84
10	37.40	630.0	23562.0	15.34
9	33.75	630.0	21262.5	13.85
8	30.10	630.0	18963.0	12.34
7	26.45	630.0	16663.5	10.85
6	22.80	630.0	14364.0	9.35
5	19.15	630.0	12064.5	7.85
4	15.50	630.0	9765.0	6.36
3	11.85	630.0	7465.5	4.86
2	8.20	630.0	5166.0	3.36
1	4.55	630.0	2866.5	1.81
Total		12412.0	480341.8	382.30

Table 4.5: Base-shear distribution in twelve-storey shear wall building in Montreal

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i	F_x, kN
12	44.70	884.0	39514.8	80.28
11	41.05	1260.0	51723.0	49.84
10	37.40	1260.0	47124.0	45.42
9	33.75	1260.0	42525.0	40.98
8	30.10	1260.0	37926.0	36.55
7	26.45	1260.0	33327.0	32.11
6	22.80	1260.0	28728.0	27.68
5	19.15	1260.0	24129.0	23.25
4	15.50	1260.0	19530.0	18.82
3	11.85	1260.0	14931.0	14.39
2	8.20	1260.0	10332.0	9.96
1	4.55	1260.0	5733.0	5.52
Total		14744.0	355522.8	384.80

4.3 Performance of buildings in western Canada

Performance of shear wall buildings is evaluated on the basis of analytical results obtained from static push-over and dynamic time history analyses carried out on the following four models.

1. Wall with factored material strength
2. Wall with nominal material strength
3. Wall and frame with factored material strength
4. Wall and frame with nominal material strength

Table 4.6: Base-shear distribution in twenty-storey shear wall building in Montreal

Floor	Height (h_i), m	Weight per frame (W_i), kN	h_iW_i	F_x, kN
20	73.90	884.0	65327.6	95.56
19	70.25	1260.0	88515.0	30.30
18	66.60	1260.0	83916.0	28.74
17	62.95	1260.0	79317.0	27.16
16	59.30	1260.0	74718.0	25.58
15	55.65	1260.0	70119.0	24.00
14	52.00	1260.0	65520.0	22.44
13	48.35	1260.0	60921.0	20.86
12	44.70	1260.0	56322.0	19.28
11	41.05	1260.0	51723.0	17.70
10	37.40	1260.0	47124.0	16.13
9	33.75	1260.0	42525.0	14.56
8	30.10	1260.0	37926.0	12.98
7	26.45	1260.0	33327.0	11.40
6	22.80	1260.0	28728.0	9.84
5	19.15	1260.0	24129.0	8.26
4	15.50	1260.0	19530.0	6.69
3	11.85	1260.0	14931.0	5.11
2	8.20	1260.0	10332.0	3.54
1	4.55	1260.0	5733.0	1.96
Total		24824.0	960683.6	402.10

Table 4.7: Fundamental periods of the frames located in Victoria

Model type	Period (s)	
	Factored strength	Nominal strength
1. 12 storey		
Single wall	2.05	1.95
Wall-frame	1.80	1.77
NBCC-95	1.60	1.60
2. 20 storey		
Single wall	3.57	3.53
Wall-frame	2.70	2.65
NBCC-95	2.60	2.60

Modal analyses are performed to determine the fundamental period of each of the above models. The periods are shown in Table 4.7.

4.3.1 Lateral load-resisting capacity

Static push-over analyses are performed to estimate the lateral load resisting capacity of each frame model. The $P - \Delta$ and shear deformation effect are considered in the analyses. The details of the analyses are given in the following paragraphs.

Twelve-storey building

Push-over curves, representing the variation of base-shear coefficient (base shear-expressed as a fraction of mass) with roof displacement(ie, lateral drift) are shown in Fig 4.4. There are two pairs of curves in the plot, one corresponding to the wall model and the other corresponding to the wall-frame model.

The design base shear, V_d is 354.6 kN in each case. The roof displacement of the wall model corresponding to a maximum interstorey drift value of 2% of building

height is close to 1.4% and 1.5% of storey height for factored and nominal strength levels, respectively. The base-shear at that drift level is $1.56V_d$ and $1.76V_d$ for factored and nominal strength levels, respectively.

The wall-frame model shows significant increase in the lateral load carrying capacity. The roof displacement corresponding to 2% inter-storey drift is close to 1.5% and 1.7% of building height for the factored and nominal strength levels respectively. The base-shear at that drift level is $2.80V_d$ and $3.22V_d$ for factored and nominal strength levels, respectively.

Twenty-storey building

The push-over curves for the twenty storey building in Victoria are shown in Fig. 4.5. The design base shear, V_d is 382.3 kN. The roof-displacement of the wall model corresponding to 2% inter-storey drift is close to 1.45% of storey height for both nominal and factored material strengths. Base shears at that drift level are $1.50V_d$ and $1.75V_d$ for factored and nominal strength levels, respectively.

The wall-frame model has higher capacity as expected. Corresponding to 2% inter-storey drift, the roof displacement is close to 1.6% and the base shear values at this drift level are $2.46V_d$ and $2.63V_d$ for factored and nominal strength levels, respectively.

4.3.2 Dynamic analyses of twelve-storey building in Victoria

Seismic time history analyses are performed for the wall and wall-frame models for three levels of seismic hazard. The envelope values of the damage parameters are

shown in Figs. 4.6 through 4.10. Figs. 4.11 through 4.14 show the variation of some of the damage parameters with the level of seismic hazard. A summary of the seismic perofmance of this building is shown in Table 4.8. Performance of these frame models is discussed in the following paragraphs.

Performance under UHS-500 events

The wall in a pure wall model remains largely elastic except that some yielding takes place in the wall at the bottom of the first storey. The maximum value of inter-storey drift remains close to 1.2% of storey height for both levels of material strength. The performance of the structure in this case can be said to be *operational.*

The wall and frame model shows a slight improvement in the damage parameters. However, there is extensive cracking in the shear wall and it yields at the bottom of the first storey. Inter-storey drift values are below 1.0% of storey height. The seismic performance in this case can be said to be *operational.*

Performance under UHS-1000 events

The damage parameters for both the wall and wall-frame models are only marginally higher than the corresponding parameters under UHS-500. The wall suffers extensive cracking and yields at the bottom of the first storey for both the wall and wall-frame models. Inter-storey drift for the wall model is close to 1.5% for both material strength levels. The drift values for the wall-frame models are lower, and are below 1.0% of storey height. Maximum values of the beams and columns ductility demand are moderate. Performance levels for both wall and wall-frame models can be said to be *operational.*

Table 4.8: Performance of the twelve-storey building in Victoria

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Single wall	Operational	Operational	Near collapse
Wall-frame	Operational	Operational	Life-safe

Performance under UHS-2500 events

The damage level in the building is quite high under this level of seismic hazard. The maximum value of inter-storey drift in the wall model is close to 2.8% and 2.5% for factored and nominal level of material strength, respectively, and the maximum value of shear wall ductility is close to 6.0. The maximum value of the storey level Park-Ang damage index is close to 0.8, and the global damage index is close to 0.7. The performance level achieved by the building can be categorized as *near collapse*.

The wall-frame model records a relatively lower level of damage. However, the inter-storey drift remains close to 2.0%. The maximum shear wall ductilities are 5.5 and 5.0 for factored and nominal values of material strength, respectively. The maximum value of beam ductility exeeds 4.0, while column ductility remains less than 2.0 except for some top storey columns that experience higher ductility demand, close to 4.0. The maximum value of the storey level Park-Ang damage index is close to 0.7, and the global damage index is close to 0.6. The performance level of the building under UHS-2500 events can be said to be *life-safe*.

4.3.3 Dynamic analyses of twenty-storey building in Victoria

Inelastic dynamic analyses of the models of twenty storey building in Victoria are carried out for different levels of seismic hazard. The envelope of damage parameters are shown in Figs. 4.15 through 4.19, and the variations of some of the damage quantities with the hazard level are shown in Figs. 4.20 through 4.23. A summary of seismic peroformance achieved by this building is shown in Table 4.9.

Performance under UHS-500 events

The wall models of the buildings show moderate amount of damage with inter-storey drift close to 1.0% and the wall remaining in the elastic range. The damage level in the wall- frame model is comparatively less, but the inter-storey drift exceeds 0.5%, and some of the beams have yielded. Columns and shear wall elements remain in the elastic range. The performance of the building can be said to be *operational* for both wall and wall-frame models.

Performance under UHS-1000 events

The damage sustained by the building UHS-1000 events is similar to that under UHS-500 events. Maximum inter-storey drift is slightly higher than 1.0 for the wall model and close to 1.0 for the wall-frame model. The wall remains elastic for all the models, only some beams have yielded in the wall-frame model. The performance level achieved by the building can be said to be *operational* for all the models considered here.

Table 4.9: Performance of the twenty storey building located in Victoria

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Single wall	Operational	Operational	Life-safe
Wall-frame	Operational	Operational	Life-safe

Performance under UHS-2500 events

The building suffers considerable damage under this level of seismic hazard. For the wall model the maximum inter-storey drift is close to 2.5% of the storey height, the wall has yielded in several places, and the maximum storey level Park-Ang damage index is 0.7 for factored values of material strength and 0.65 for nominal values of material strength. The global Park-Ang damage index is close to 0.6 for both levels of material strength. The maximum values of shear wall ductility demand are 3.0 and 2.2 for factored and nominal values of material strength, respectively. The performance level achieved by the building in this case can be said to be *life-safe*.

The wall-frame model shows some improvement in the building's seismic performance. The maximum value of inter-storey drift is close to 1.5% of storey height, ductility in some of the beams is close to 4.0, but column and shear wall ductility remain quite low. The maximum value of storey level Park-Ang damage index is close to 0.6 and the global damage index is close to 0.5. The performance level achieved by the building in this case can be categorized as *life-safe*.

4.4 Performance of buildings in eastern Canada

The magnitude of seismic hazard is quite low in the eastern Canadian locations as compared to that in the western Canadian locations. Twelve-storey and twenty-storey buildings in Montreal are considered here. Fundamental periods of these structural models are shown in Table 4.10. Following four models are analyzed for an evaluation of their seismic performance.

1. Wall with factored material strength
2. Wall with nominal material strength
3. Wall and frame with factored material strength
4. Wall and frame with nominal material strength

It is observed from Tables 4.7 and 4.10 that the period a building model for Montreal is higher than the period of a corresponding model for Victoria. This is because of the fact that the tributary mass for a ductile frame in Montreal is double that of a corresponding frame in Victoria.

Table 4.10: Fundamental periods of the frames located in Montreal

Model type	Period (s)	
	Factored strength	Nominal strength
1. 12 storey		
Single wall	2.83	2.76
Wall-frame	2.54	2.50
NBCC-95	1.60	1.60
2. 20 storey		
Single wall	5.05	4.49
Wall-frame	3.81	3.75
NBCC-95	2.60	2.60

4.4.1 Lateral load-resisting capacity

The designs of the lateral load-resisting frames are identical to that for Victoria. However, every alternate frame is designed to be a lateral load resisting shear wall frame. The push-over curves are shown in Figs. 4.24 and 4.25.

Twelve-storey building

The design base shear, V_d for a shear wall frame of the building is 384.8 kN. The maximum values of roof-displacement in the wall model corresponding to an inter-storey drift of 2% are 1.4% and 1.5% for factored and nominal levels of material strength, respectively. The corresponding values for the wall-frame models are respectively 1.5% and 1.7%. The values of base shear coefficient at this drift level for factored and nominal strength levels are respectively $1.35V_d$ and $1.46V_d$ for the wall model, and $2.10V_d$ and $2.45V_d$ for the wall-frame model.

Twenty-storey building

The design base shear, V_d for the twenty storey building frames in Montreal is 402.1 kN. The roof displacements corresponding to the maximum inter-storey drift of 2.0% are close to 1.4% for the wall model and 1.6% for the wall-frame model for both levels of material strength. The values of the base-shear this drift level for the wall model are $1.42V_d$ and $1.54V_d$ for the factored and nominal strength levels, respectively. The corresponding values for the wall-frame model are $1.68V_d$ and $1.80V_d$, respectively.

4.4.2 Dynamic analyses of twelve-storey building in Montreal

Inelastic response time histories are obtained for all of the models considered and the three levels of seismic hazard. The envelope values of the damage parameters are shown in Figs. 4.26 through 4.29. Figures 4.11 through 4.31 show the variation of some damage parameters with the level of seismic hazard. A summary of performance levels achived by the building is shown in Table 4.11. Seismic performance of various building models is discussed in the following paragraphs.

Performance under UHS-500

Under this level of seismic hazard the building does not suffer any significant damage. Both wall and wall-frame models remain elastic. There may be some sporadic cracking in the structures. The performance of the building can be described as *fully operational.*

Table 4.11: Performance of the twelve-storey building located in Montreal

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Single wall	Fully operational	Fully operational	Operational
Wall-frame	Fully operational	Fully operational	Operational

Performance under UHS-1000

The building suffers slightly more damage under this hazard level as compared to that under UHS-500. Inter-storey drift exceeds 0.5% for the wall model but remains below 0.4% for the wall-frame model. Some of the beams have just yielded in the wall-frame model, but the columns remain elastic in this case. The wall remains elastic for all the models considered. The performance of the building can be categorized as *operational* when a wall model is considered and *fully operational* for a wall-frame model.

Performance under UHS-2500

The damage suffered by the building under this level of seismic hazard is moderate. The maximum value of inter-storey drift is close to 0.8% in the wall model but below 0.6% in the wall-frame model. The wall suffers extensive cracking, but remains elastic in all the cases. Ductility in some of the beams exceed 1.5, but the columns remain elastic in the wall-frame model. The performance level achieved by the building can be said to be *operational* for both the wall and wall-frame models.

4.4.3 Dynamic analyses of twenty-storey building in Montreal

The envelope values of the damage parameters are shown in Figs. 4.32 through 4.35. Figures 4.20 through 4.37 show the variation of some damage parameters with the level of seismic hazard. A summary of performance levels achived by the building is shown is Table 4.12. Seismic performance of this building is discussed in the following paragraphs.

Performance under UHS-500

Under this level of seismic hazard the building does not suffer any significant damage. Both wall and wall-frame models remain elastic. There may be some sporadic cracking in the structures. The performance of the building can be described as *fully operational.*

Performance under UHS-1000

The building suffers slightly more damage under this hazard level as compared to that under UHS-500. Inter-storey drift exceeds 0.5% for the wall model but remains below 0.4% for the wall-frame model. Some of the beams have just yielded in the wall-frame model, but the columns remain elastic. The wall remains elastic for all the models considered. The performance of the building can be categorized as *operational* when a wall model is considered and *fully operational* for a wall-frame model.

Performance under UHS-2500

The damage suffered by the building under this level of seismic hazard is moderate. The maximum value of inter-storey drift is close to 0.8% in the wall model but below

Table 4.12: Performance of the twenty storey building located in Montreal

Model	Performance level		
	UHS-500	UHS-1000	UHS-2500
Single wall	Fully operational	Operational	Operational
Wall-frame	Fully operational	Fully operational	Operational

0.6% in the wall-frame model. The wall suffers extensive cracking, but remains elastic in all the cases. Ductility in some of the beams exceed 1.5, but the columns remain elastic in the wall-frame model. The performance level achieved by the building can be said to be *operational* for both wall and wall-frame models.

4.5 Discussion

Seismic performance of concrete shear wall buildings are presented in this chapter. Two building heights are considered: twelve storeys and twenty storeys. For the buildings in Victoria, every frame is designed to resist lateral loads, while for Montreal, every alternate frame is so designed. Two models are considered for each building: wall model and wall-frame model. In the wall model the shear wall is assumed to resist the entire lateral load. In the wall-frame model the two bays adjoining the shear wall are designed for ductility and participate in resisting the lateral load. Factored and nominal levels of the material strength are considered for each model. Static push-over analyses on these models are carried out to determine their load carrying capacity. Inelastic dynamic analyses are performed for a number of UHS-compatible ground motion time histories, and a damage envelopes are presented for each level of seismic hazard.

Results of the push-over analysis indicate that the frame-wall model has higher capacity than the single wall model for a building. However, the dynamic response of both models are close to each other and performance levels achieved by two models are found to be similar.

As in the cases of the buildings with concrete moment resisting frames, the shear wall buildings in western Canada are found to be more vulnerable to seismic hazard than those in eastern Canada. This is due to the fact that the design base-shear calculated using the NBCC formula for period is substantially over estimated for Montreal (as explained in Chapter 3). The buildings in Montreal achieve a performance level of *operational* or better under UHS-500 and UHS-1000, and *operational* under UHS-2500. The buildings in Victoria remains *operational* under UHS-500 and UHS-1000, and achieve *life-safe* or *near collapse* performance level under UHS - 2500.

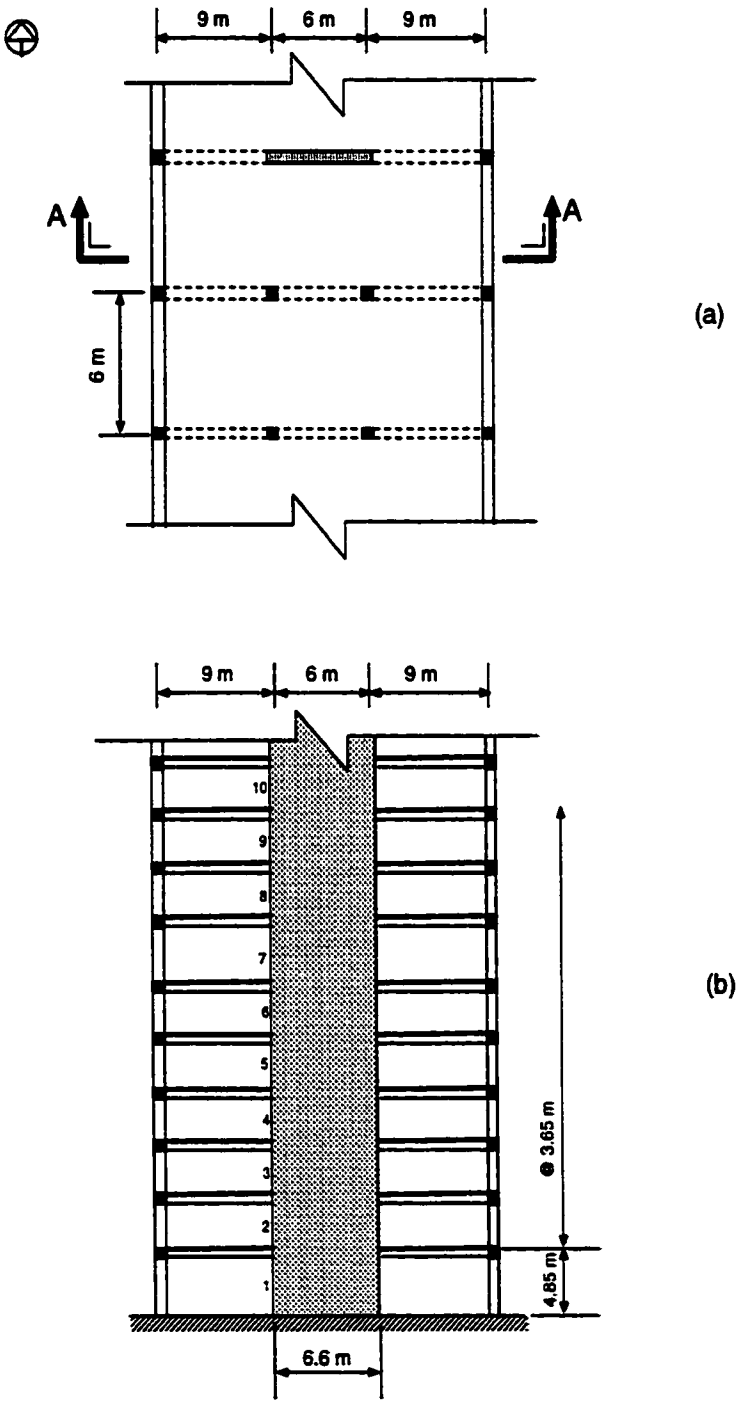


Figure 4.1: Geometric details of the shear wall buildings

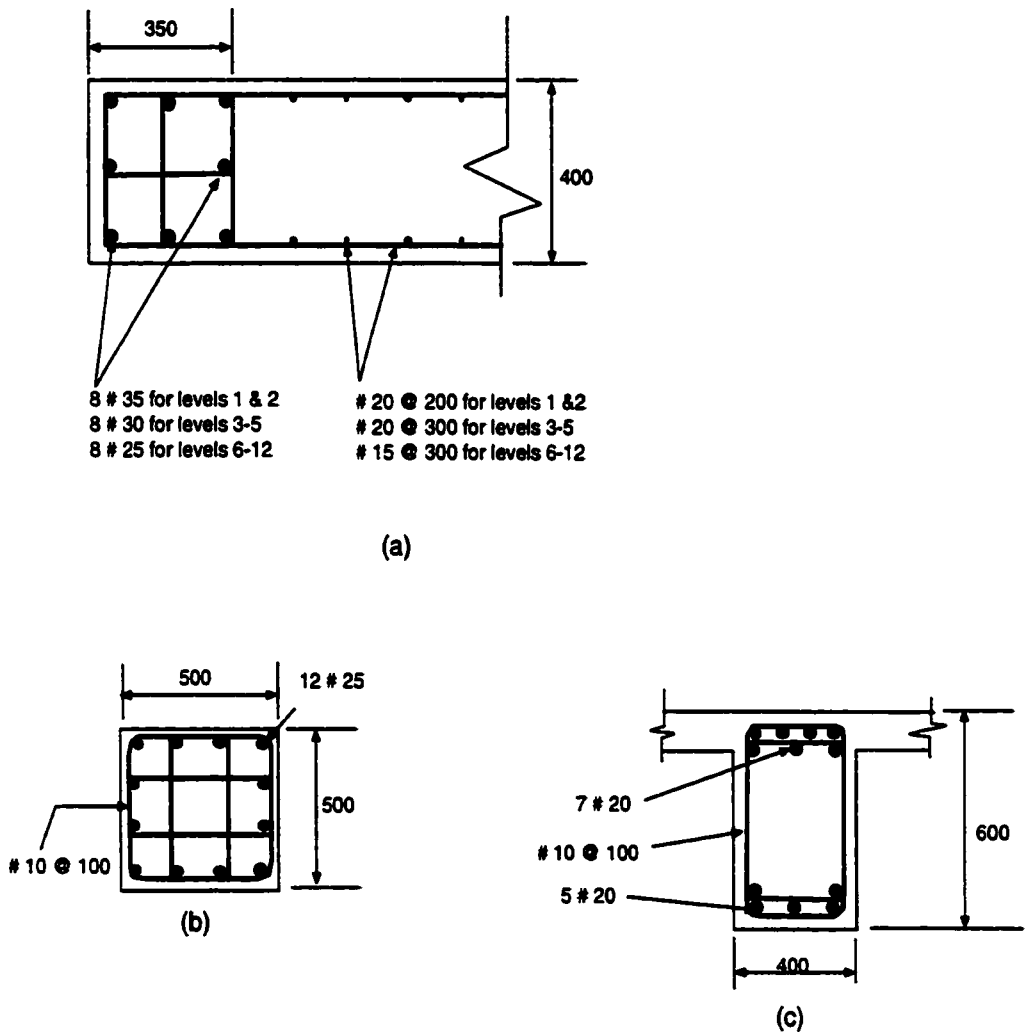


Figure 4.2: Sectional details for the twelve-storey shear wall building

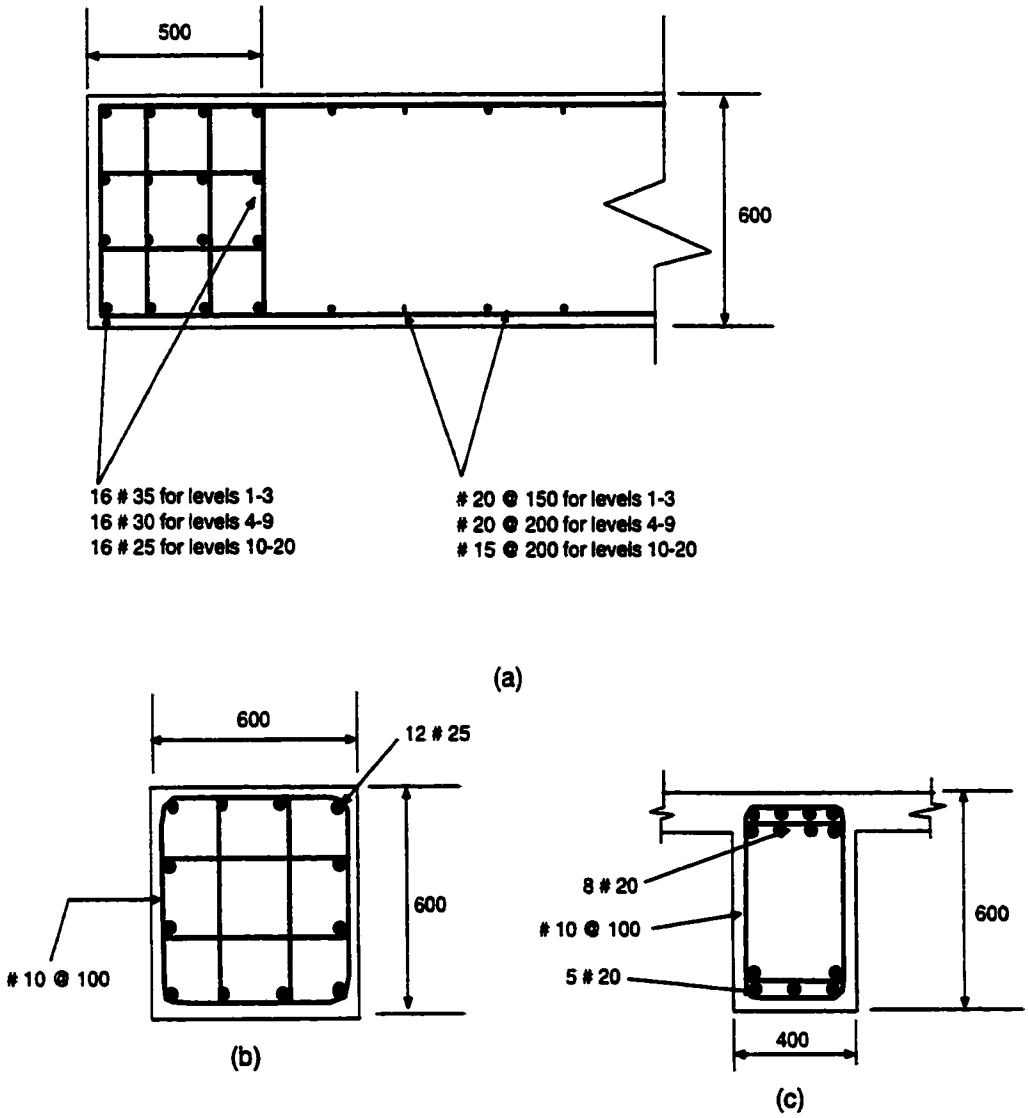


Figure 4.3: Sectional details for the twenty-storey shear wall building

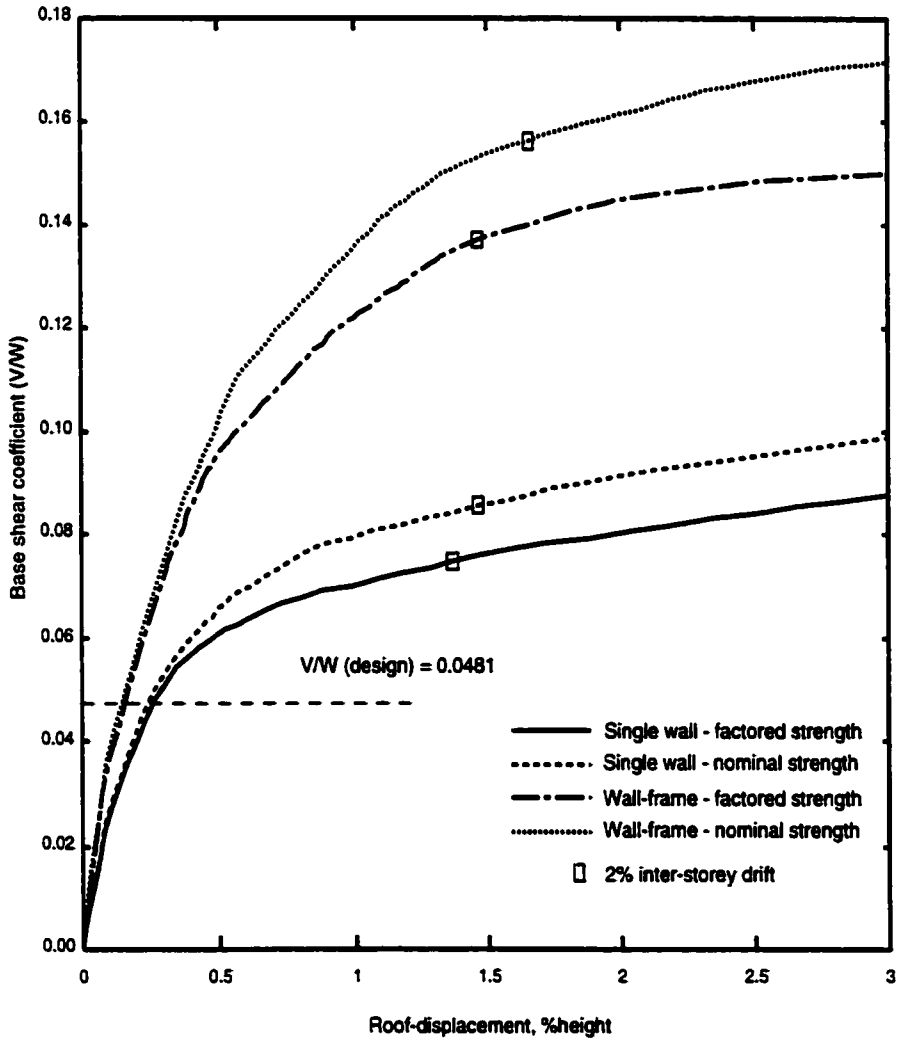


Figure 4.4: Push-over curves for the twelve-storey shear wall building in Victoria

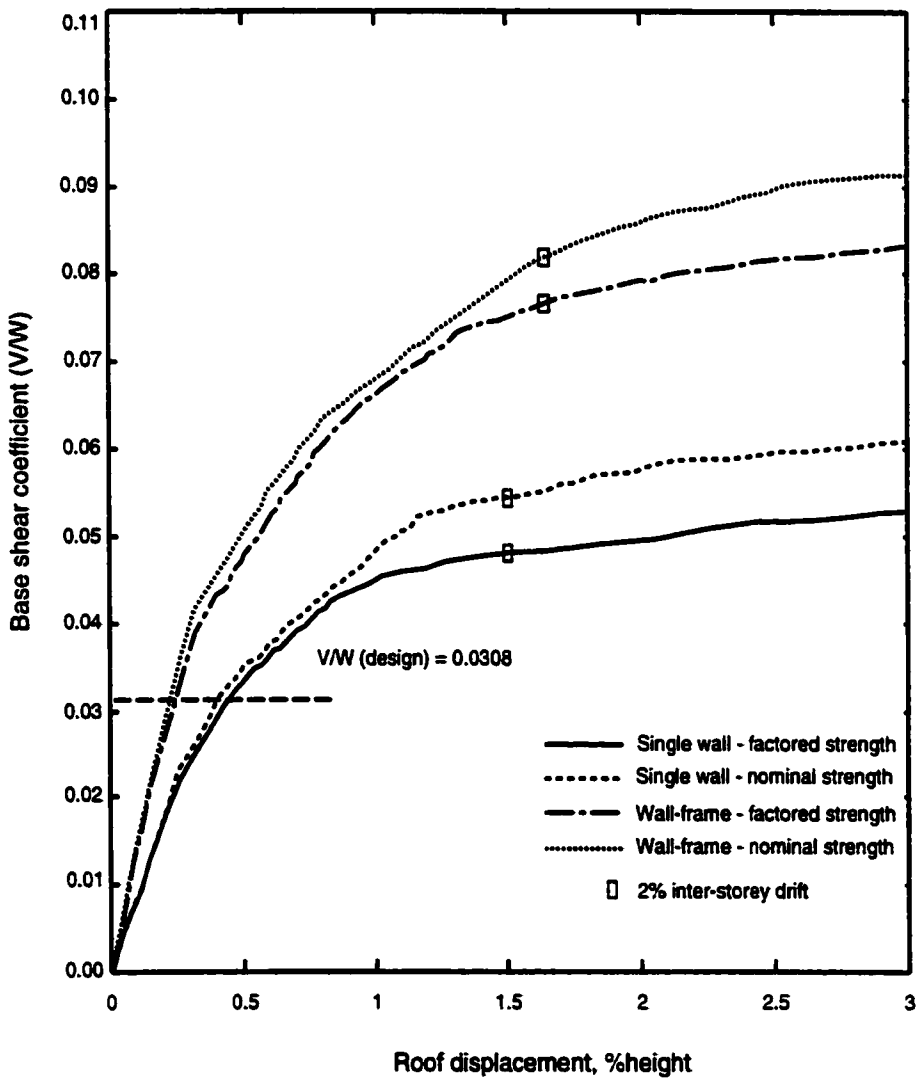


Figure 4.5: Push-over curves for the twenty-storey shear wall building in Victoria

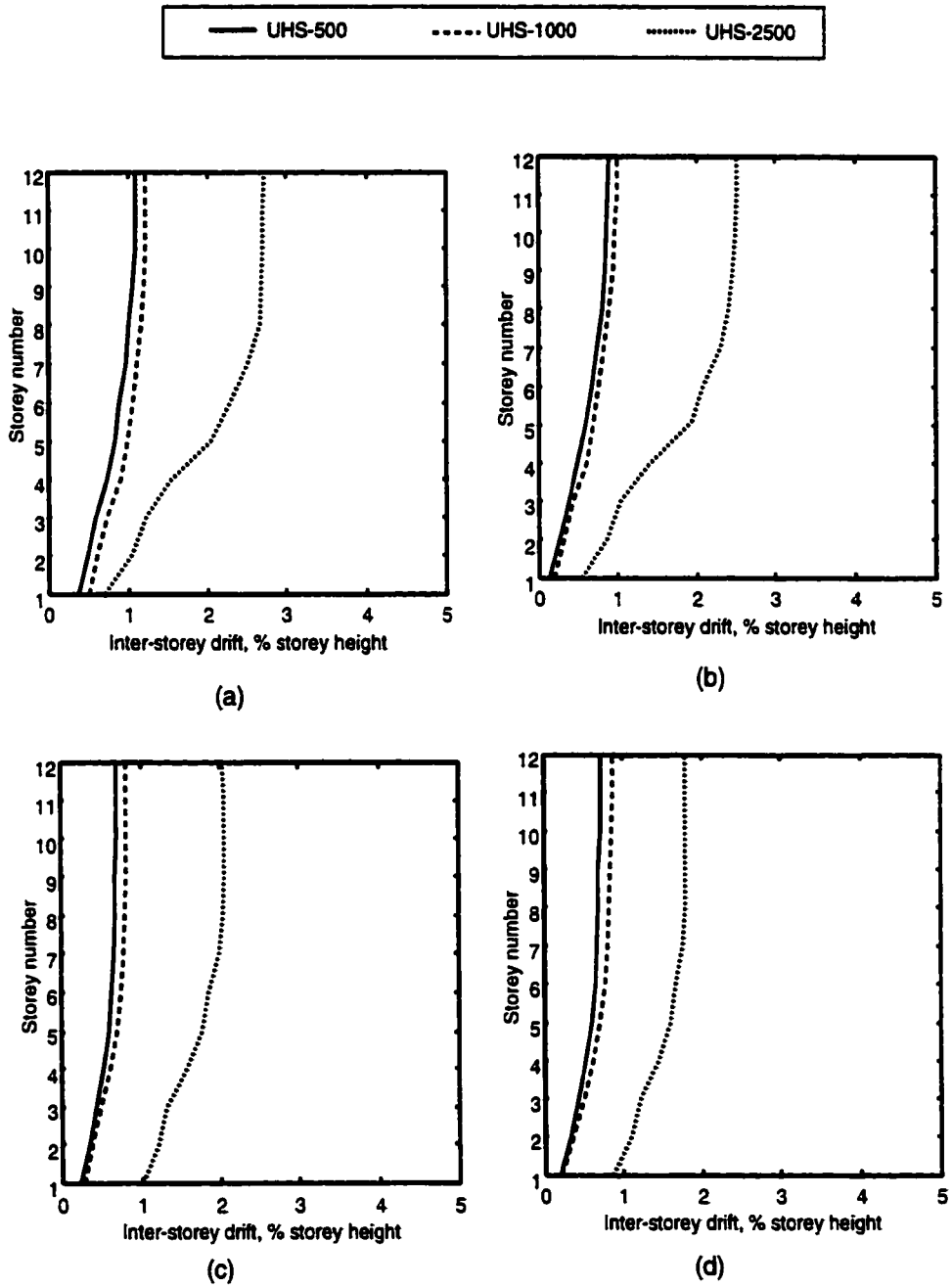


Figure 4.6: Inter-storey drift envelopes for the twelve-storey shear wall building in Victoria: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

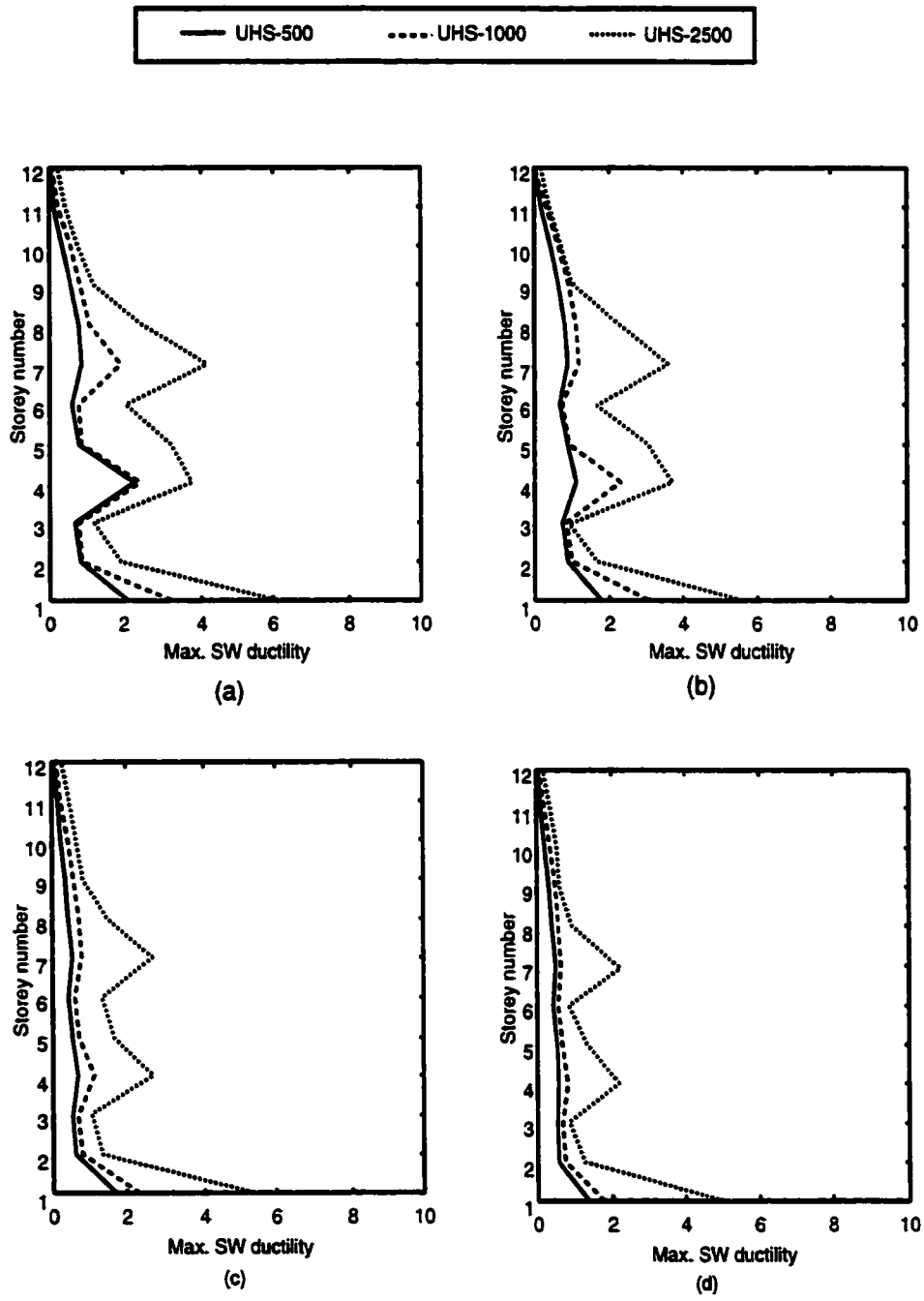


Figure 4.7: Maximum shear wall ductility demand in the twelve-storey shear wall building in Victoria: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

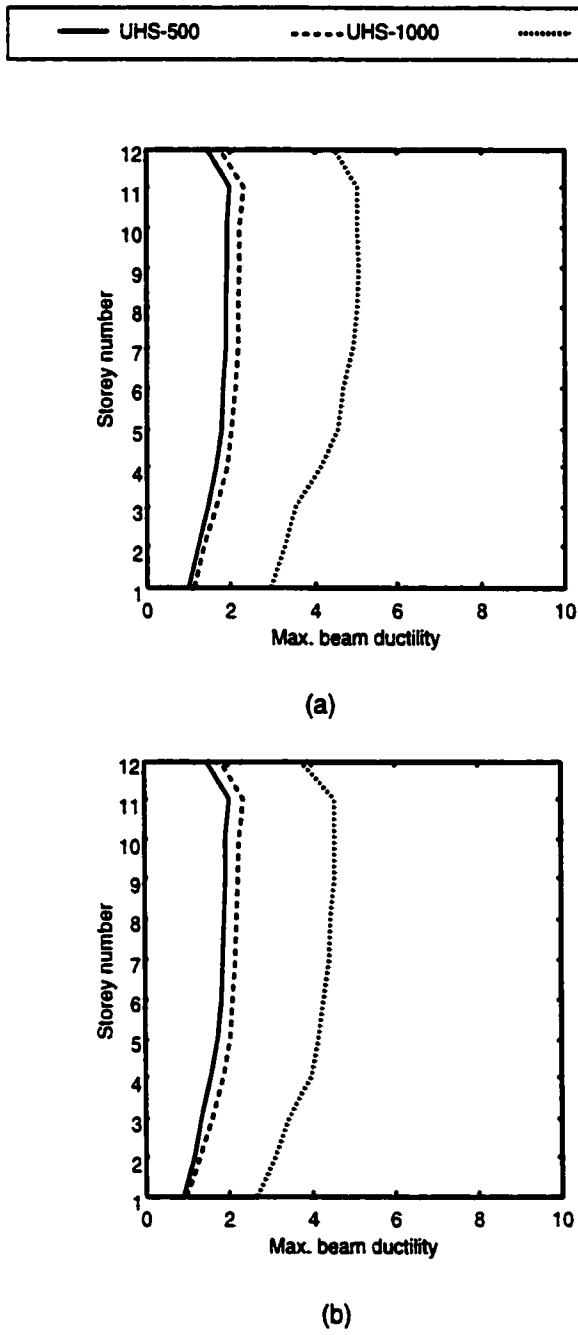


Figure 4.8: Maximum beam ductility demand in the twelve-storey shear wall building in Victoria: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength

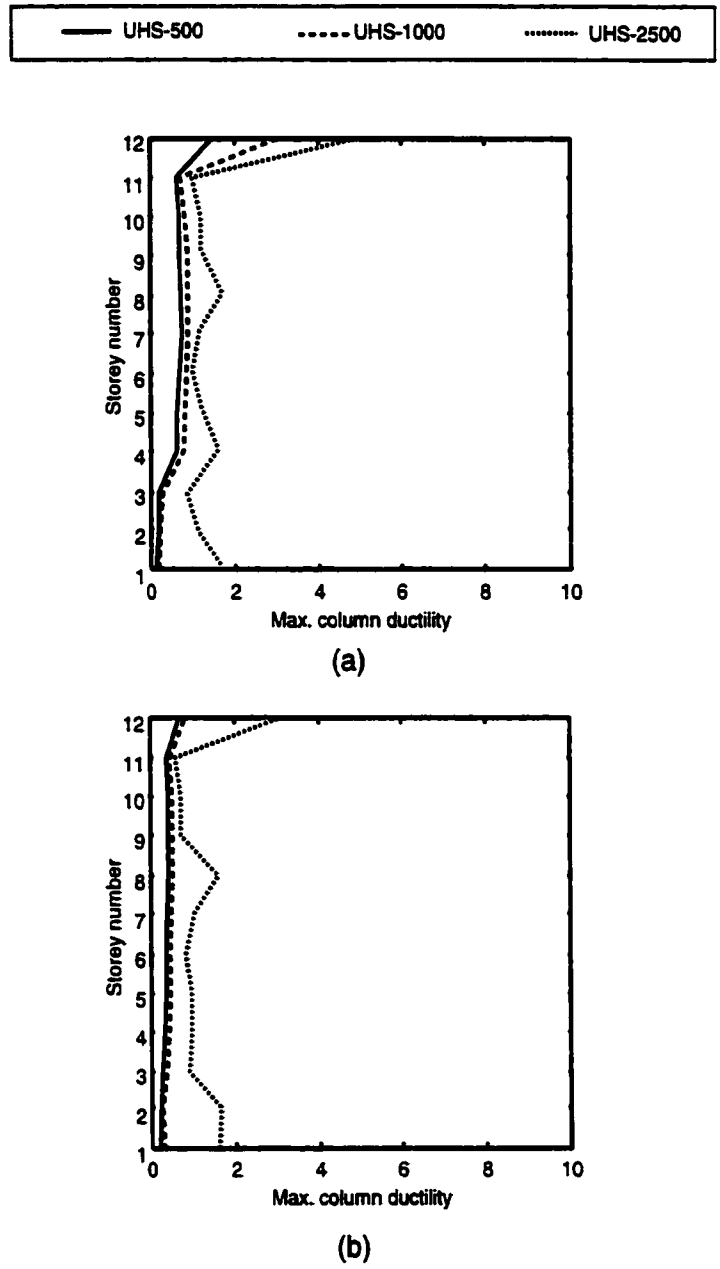


Figure 4.9: Maximum column ductility demand in the twelve-storey shear wall building in Victoria: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength

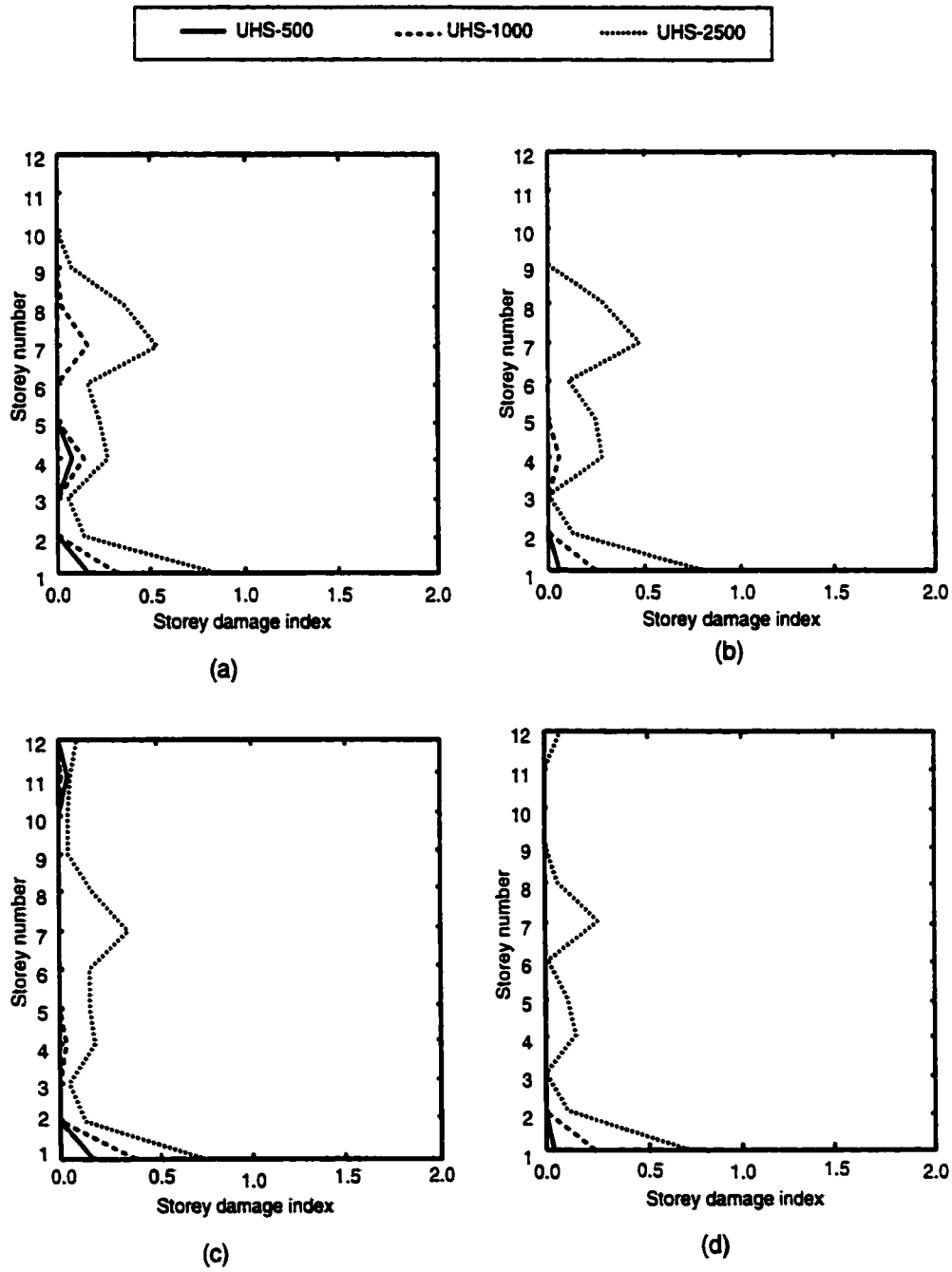


Figure 4.10: Storey level Park-Ang damage index in the twelve-storey shear wall building in Victoria: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

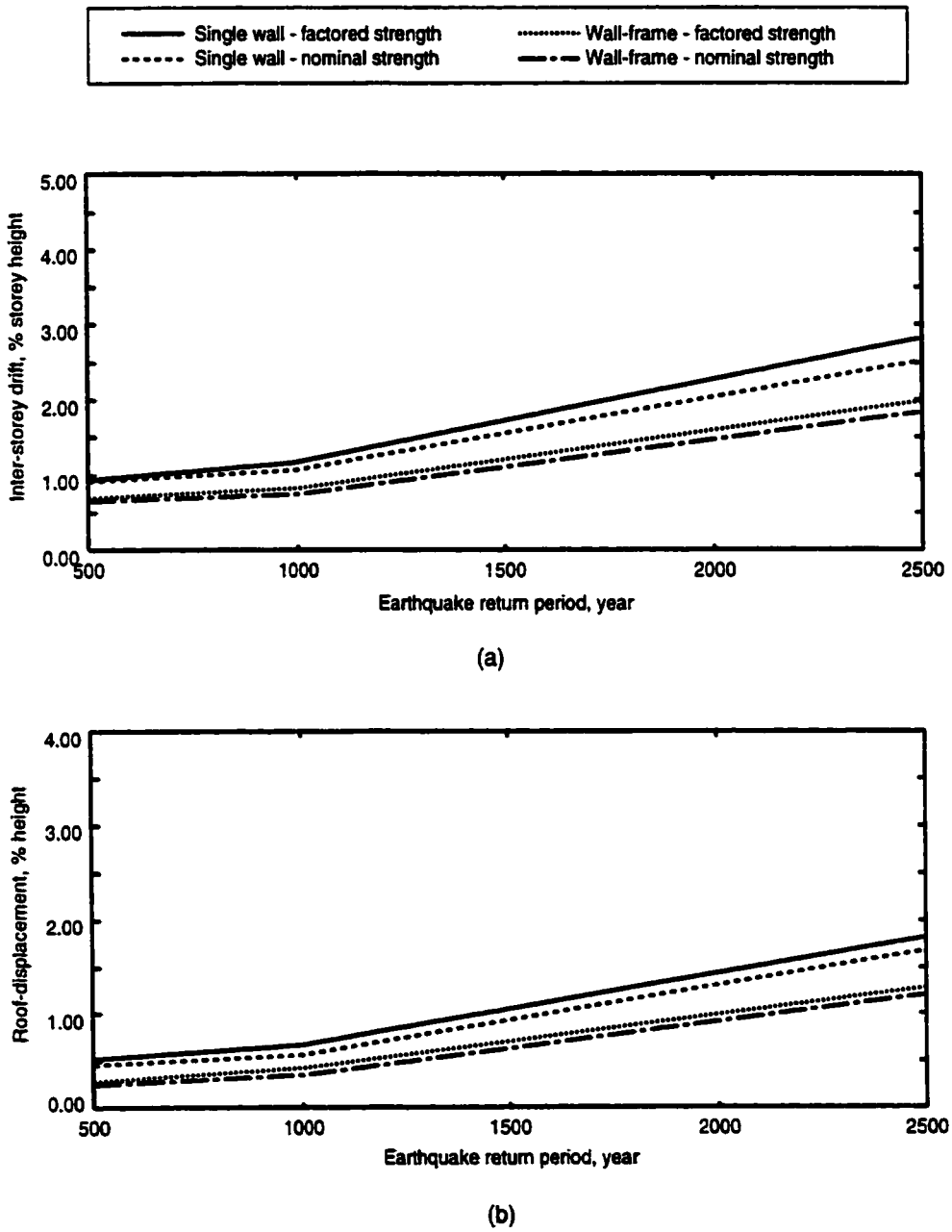


Figure 4.11: Peak displacement response in the twelve storey shear wall building in Victoria: (a) inter-storey drift, (b) roof-displacement

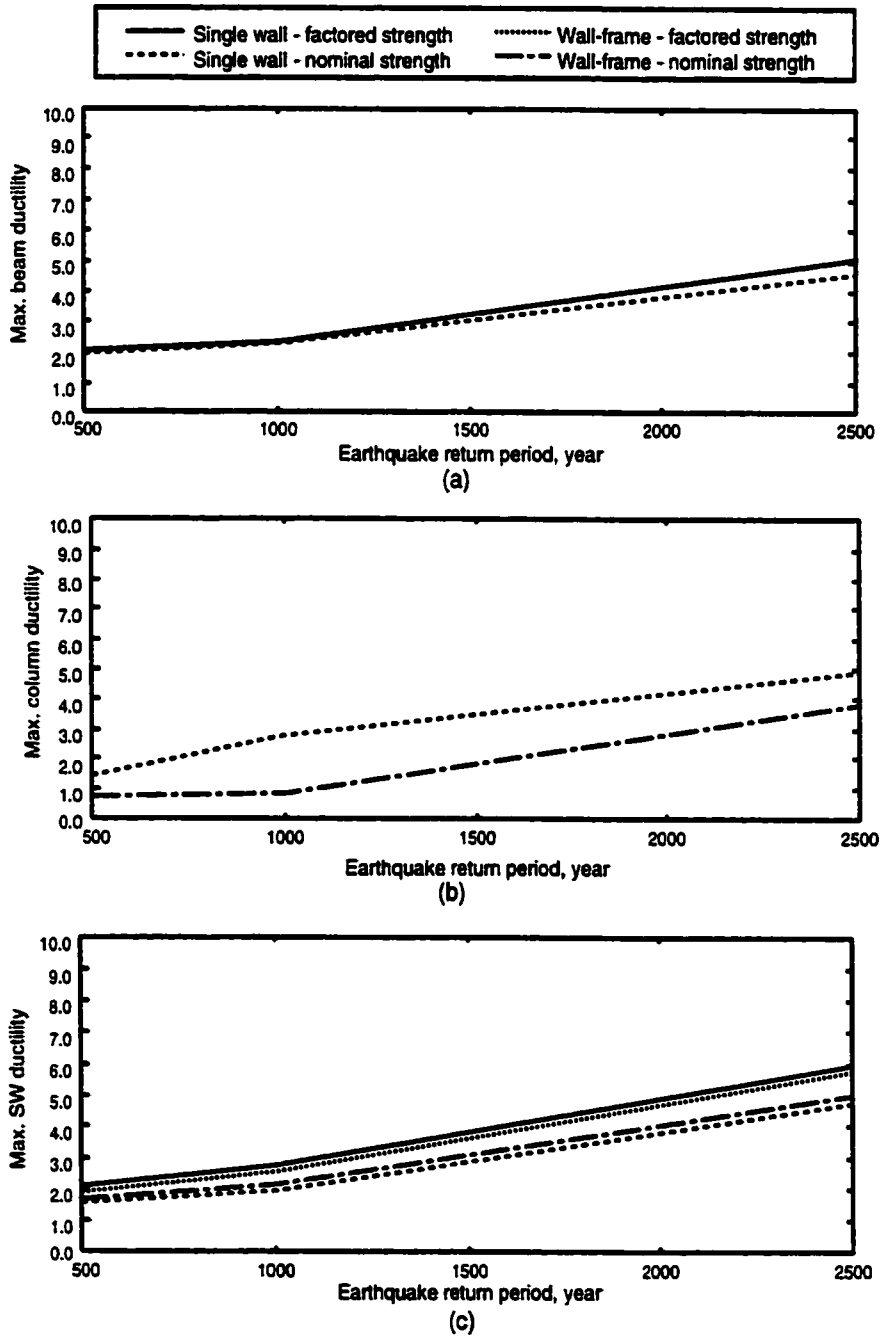
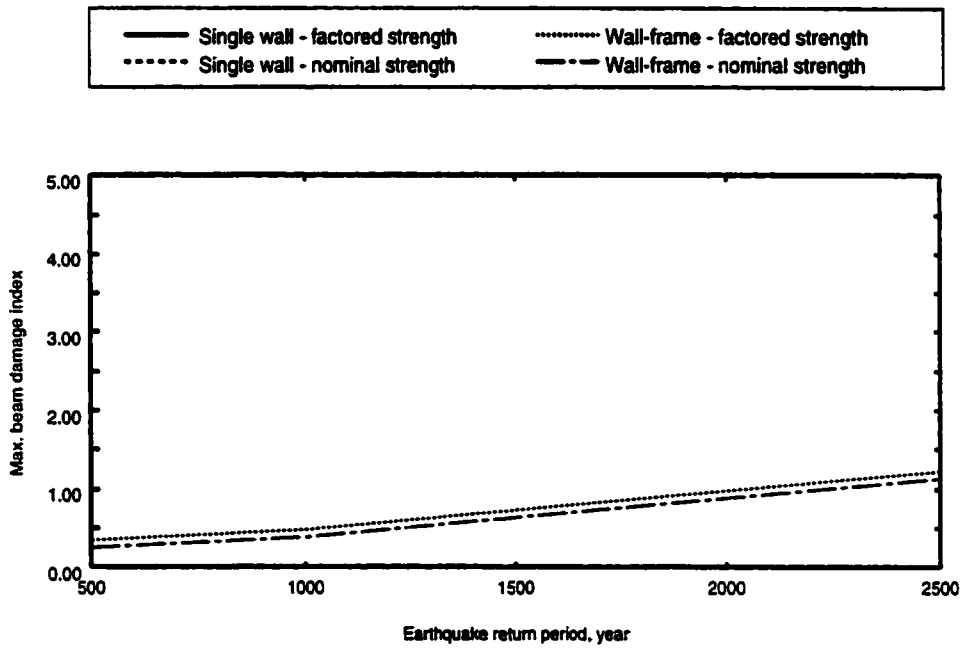
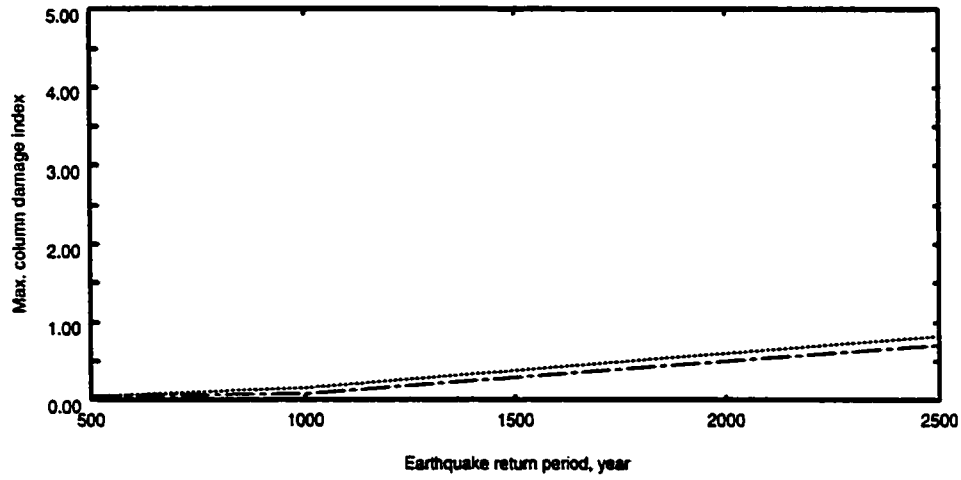


Figure 4.12: Maximum element ductility demand in the twelve-storey shear wall building in Victoria: (a) beam, (b) column, and (c) shear wall

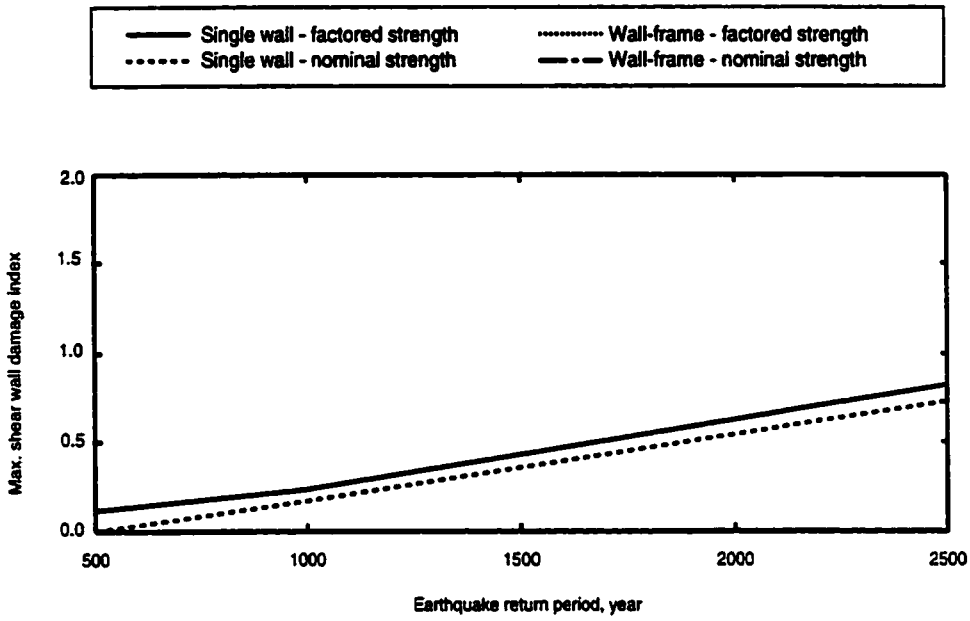


(a)

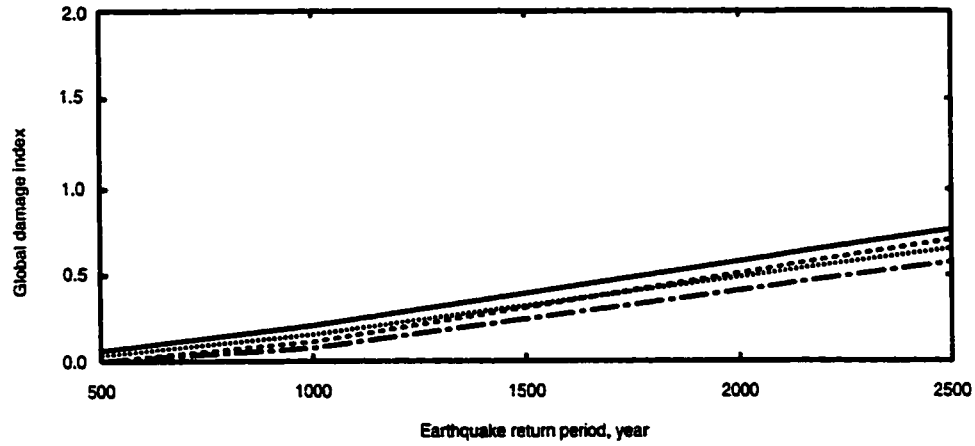


(b)

Figure 4.13: Maximum element level (beam and column) Park-Ang damage index in the twelve-storey shear wall building in Victoria: (a) beam, and (b) column



(a)



(b)

Figure 4.14: Maximum element level (shear wall) and global Park-Ang damage index in the twelve-storey shear wall building in Victoria: (a) shear wall, and (b) global

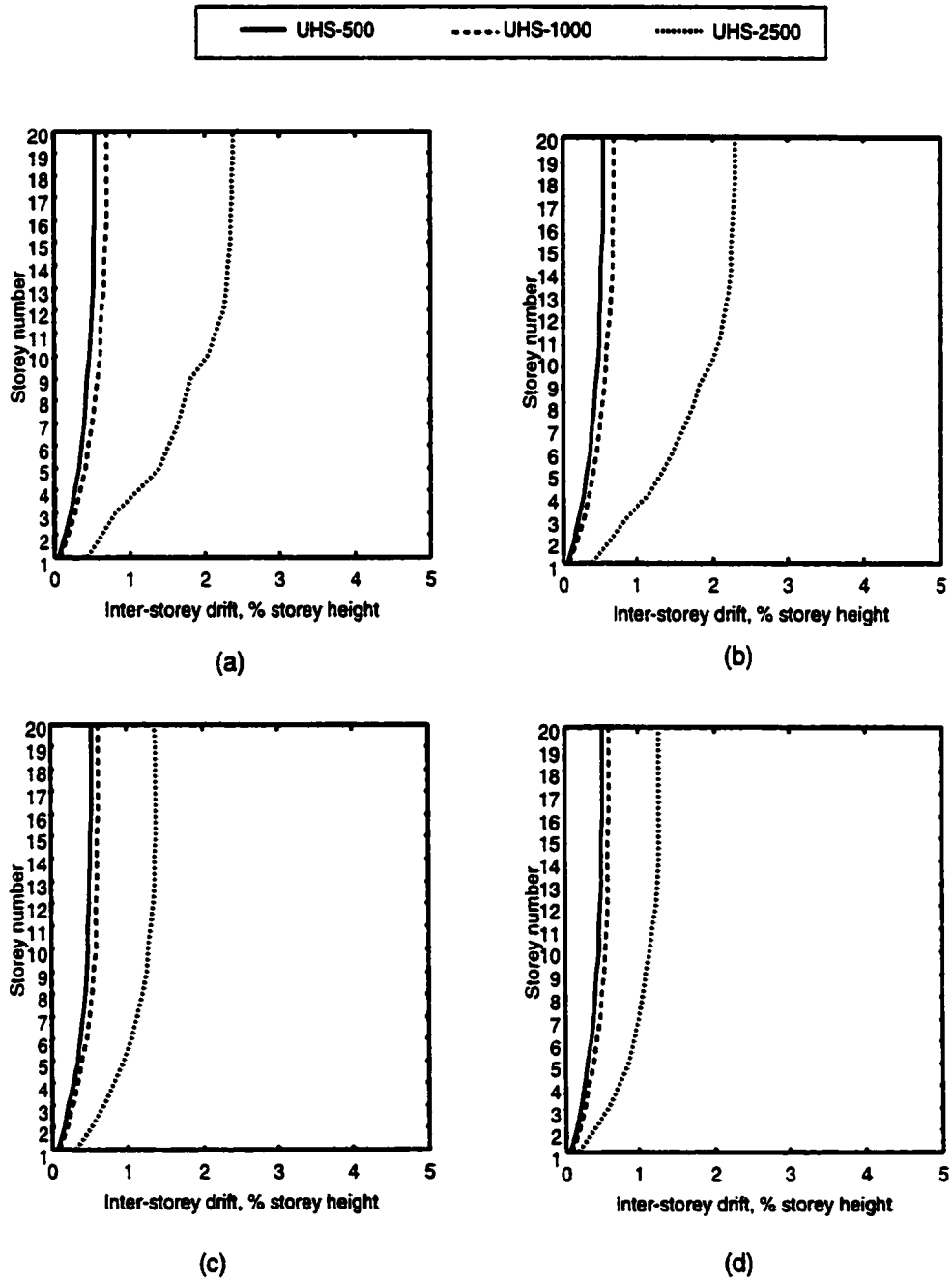


Figure 4.15: Inter-storey drift envelopes for the twenty-storey shear wall building in Victoria: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

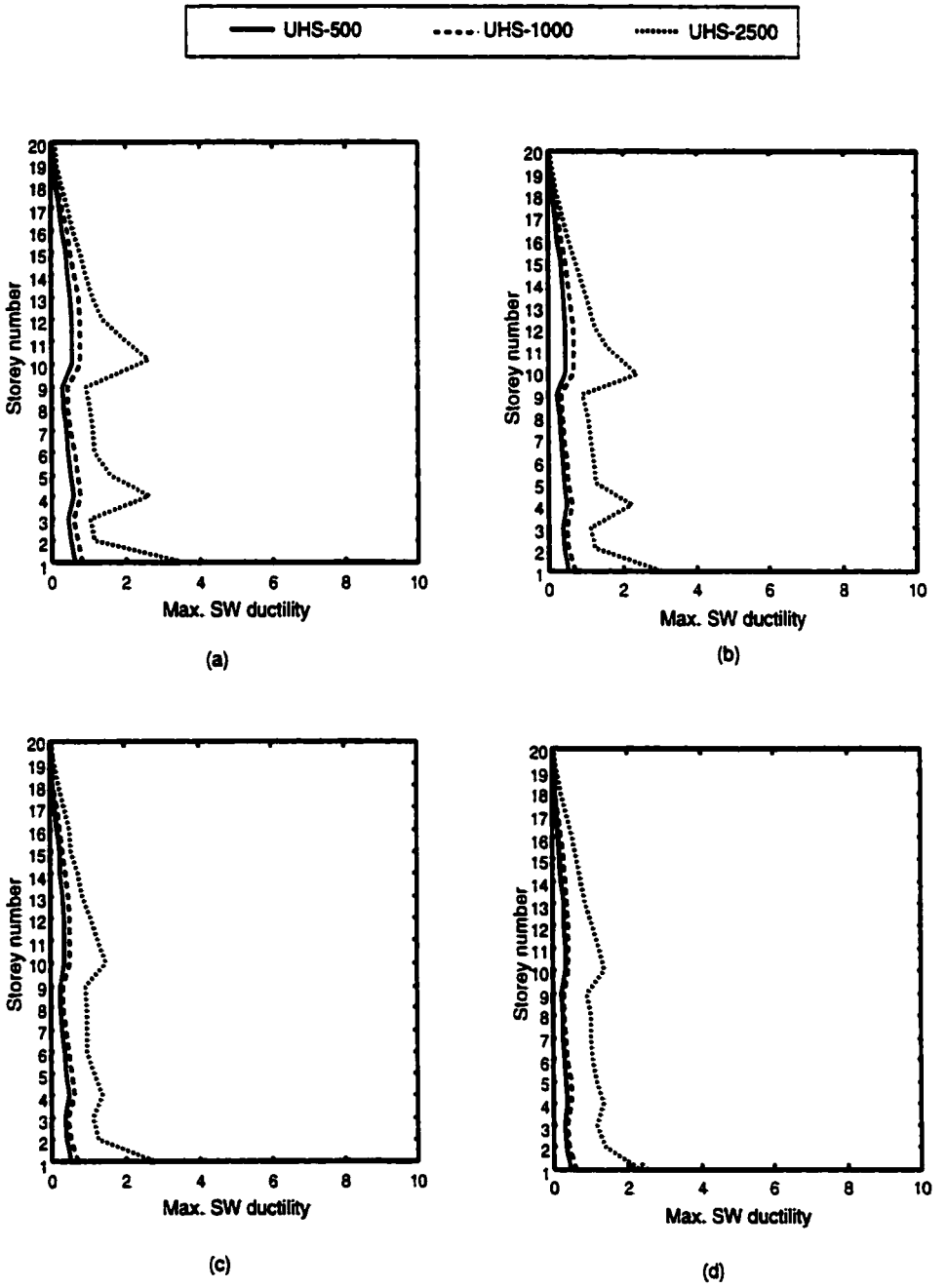
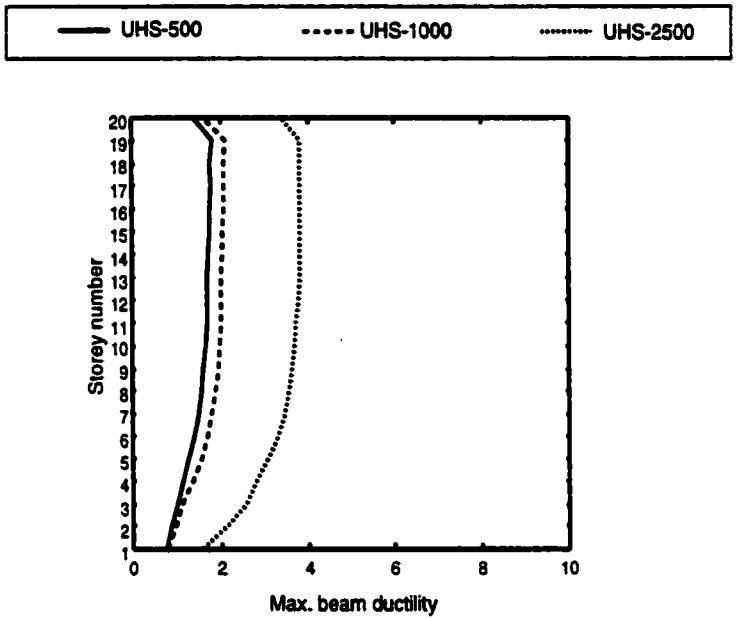


Figure 4.16: Maximum shear wall ductility demand in the twenty-storey shear wall building in Victoria: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength



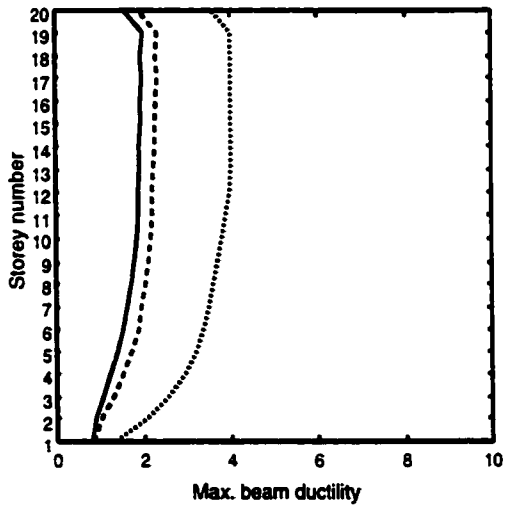


Figure 4.17: Maximum beam ductility demand in the twenty-storey shear wall building in Victoria: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength

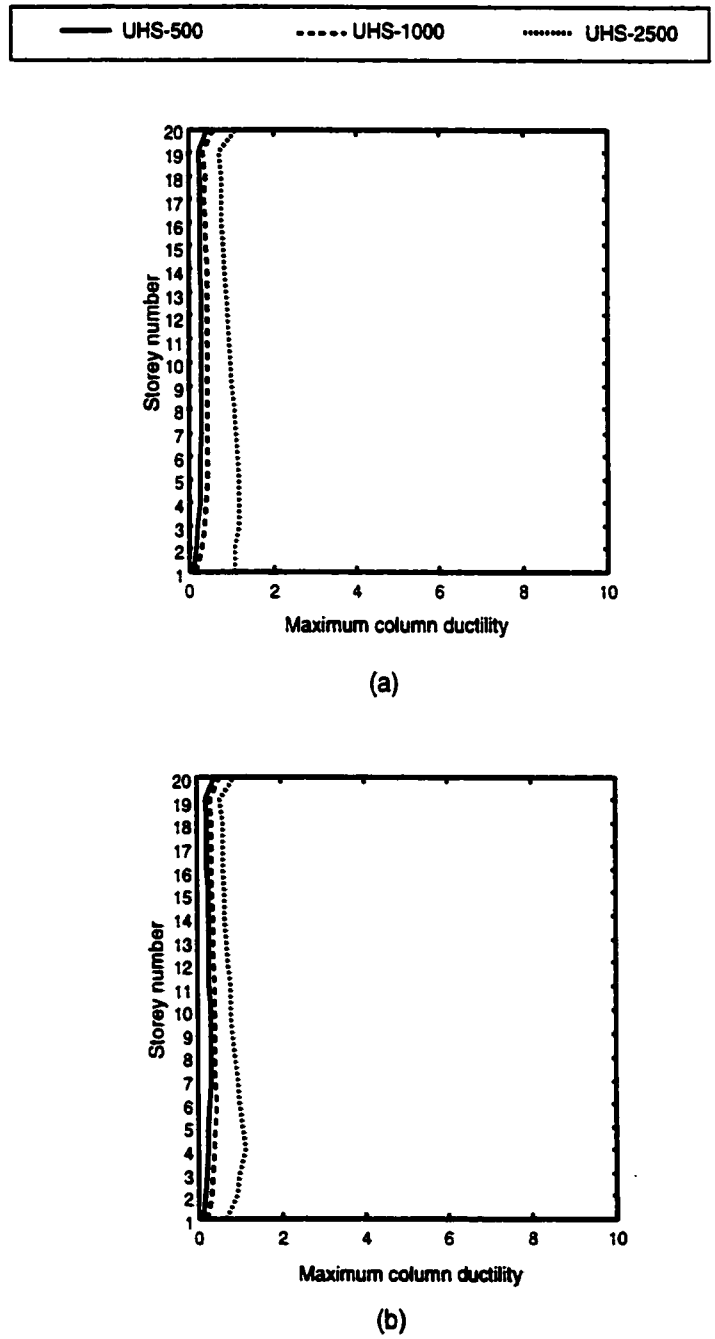


Figure 4.18: Maximum column ductility demand in the twenty-storey shear wall building in Victoria: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength

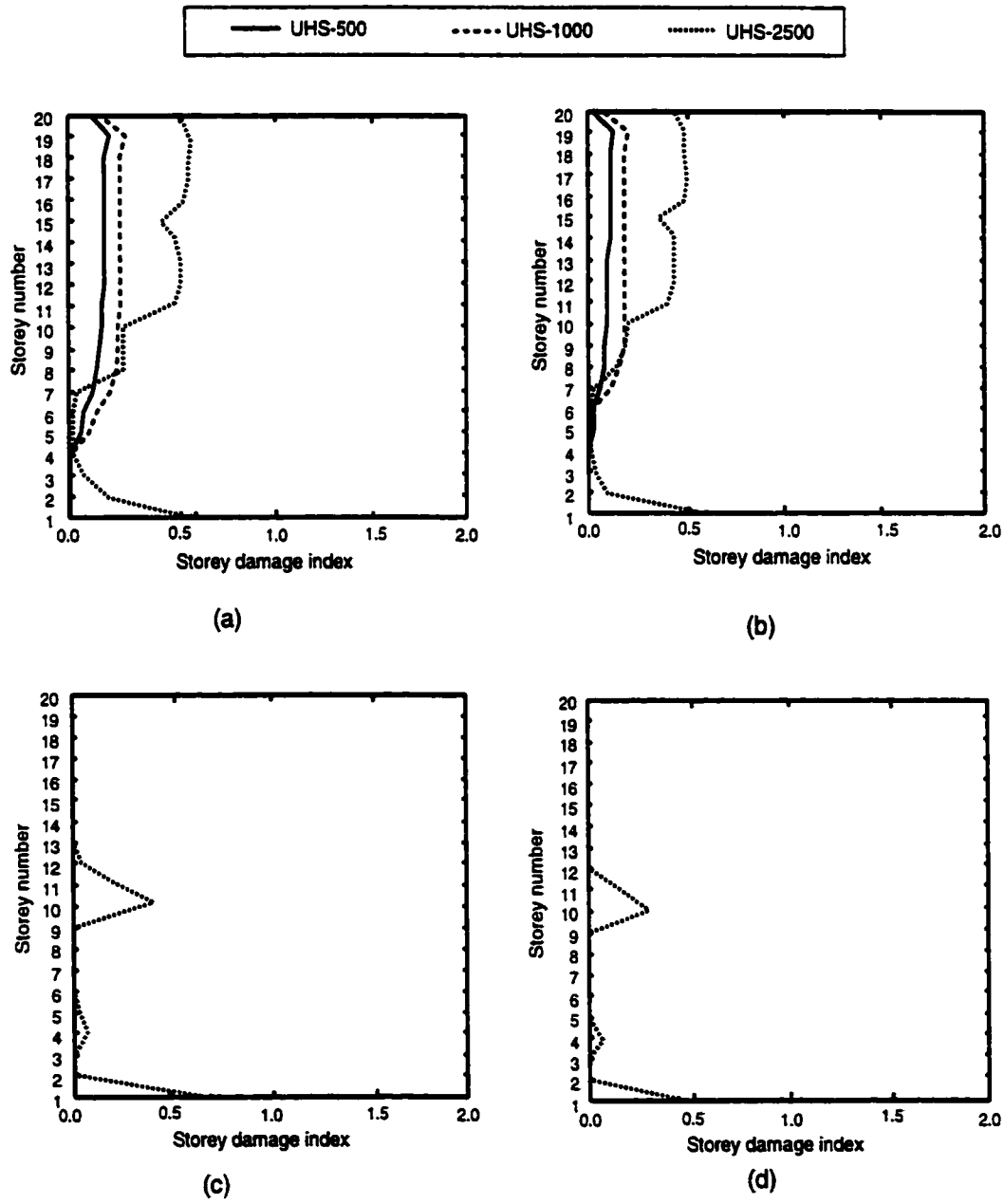


Figure 4.19: Storey level Park-Ang damage index in the twenty-storey shear wall building in Victoria: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

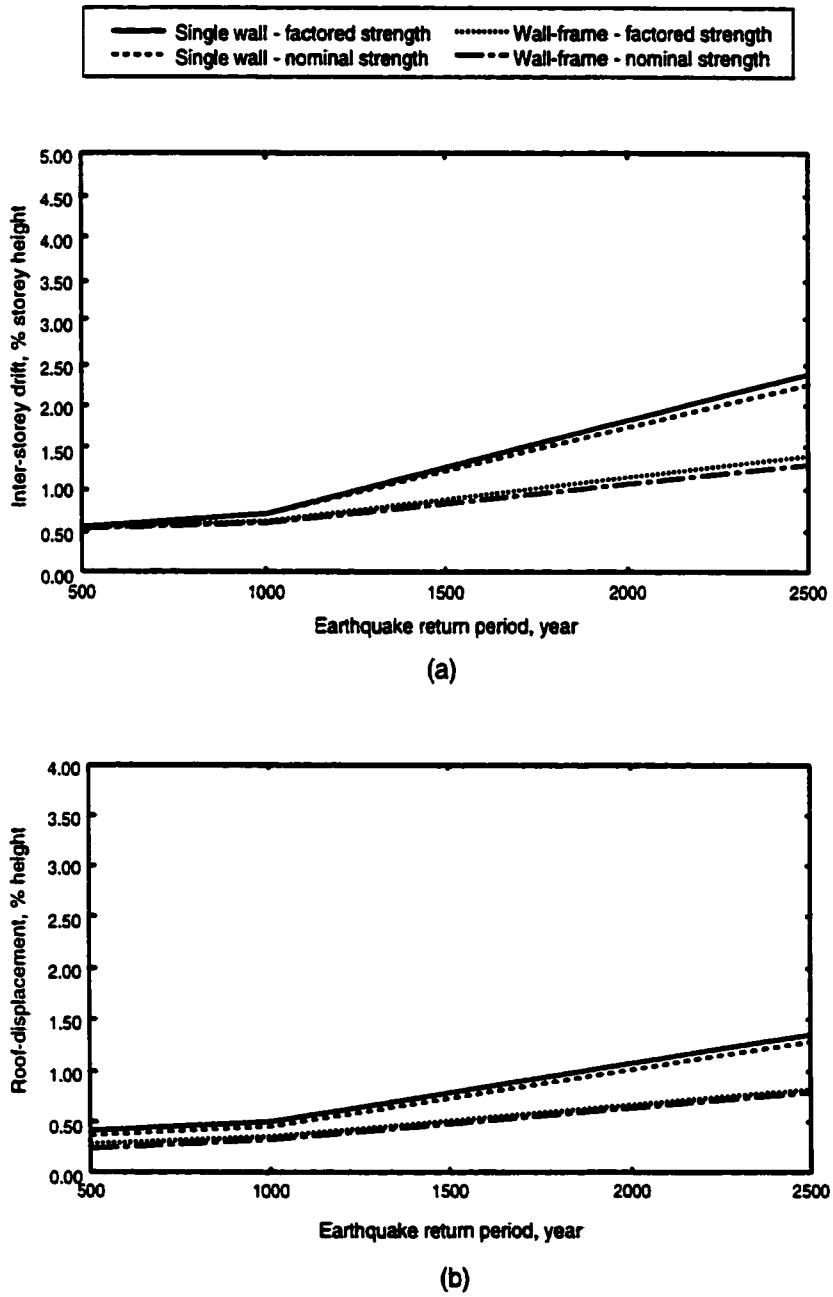


Figure 4.20: Peak displacement response in the twenty-storey shear wall building in Victoria: (a) inter-storey drift, (b) roof displacement

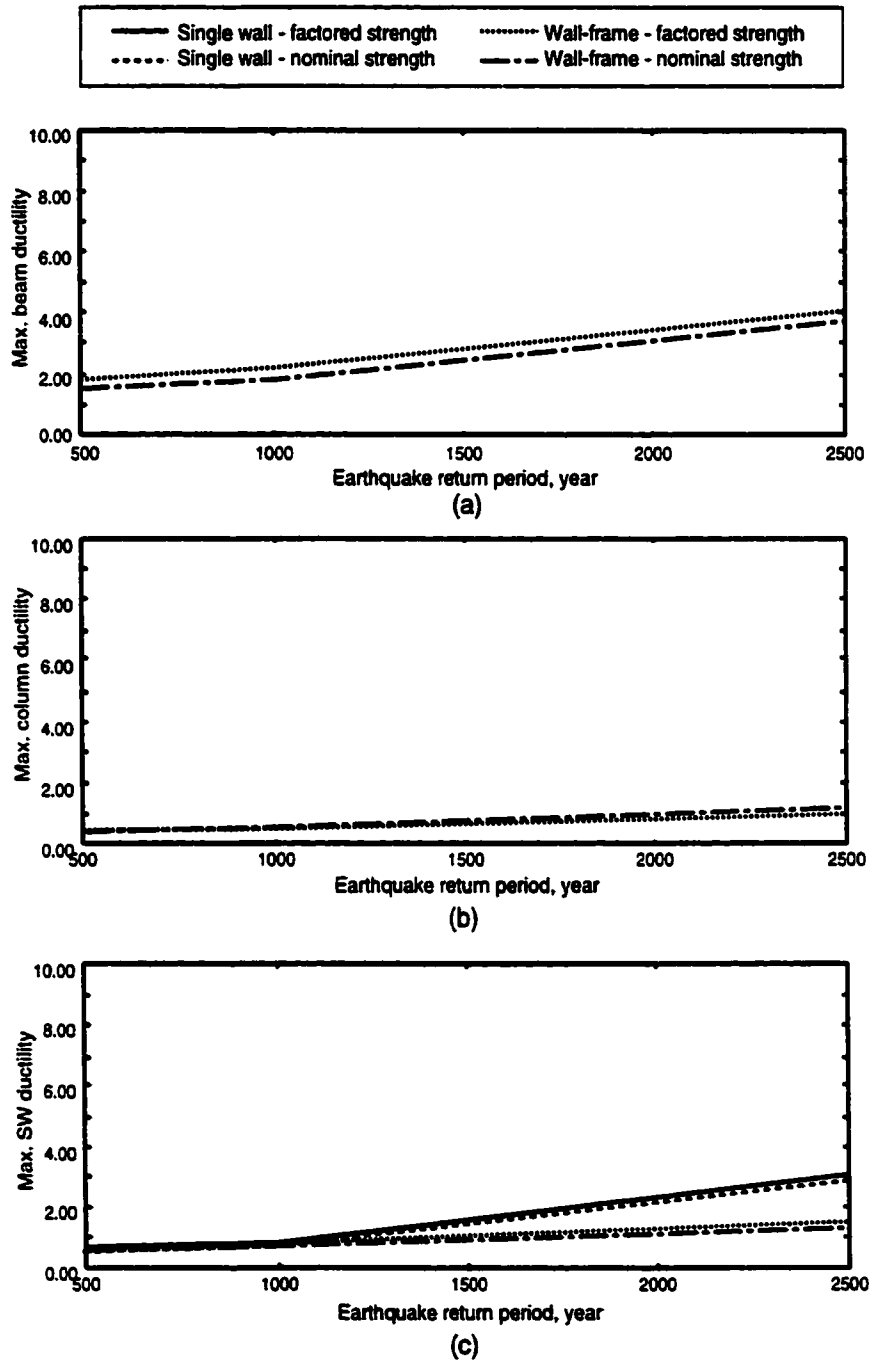


Figure 4.21: Maximum element ductility demand in the twenty-storey shear wall building in Victoria: (a) beam, (b) column, and (c) shear wall

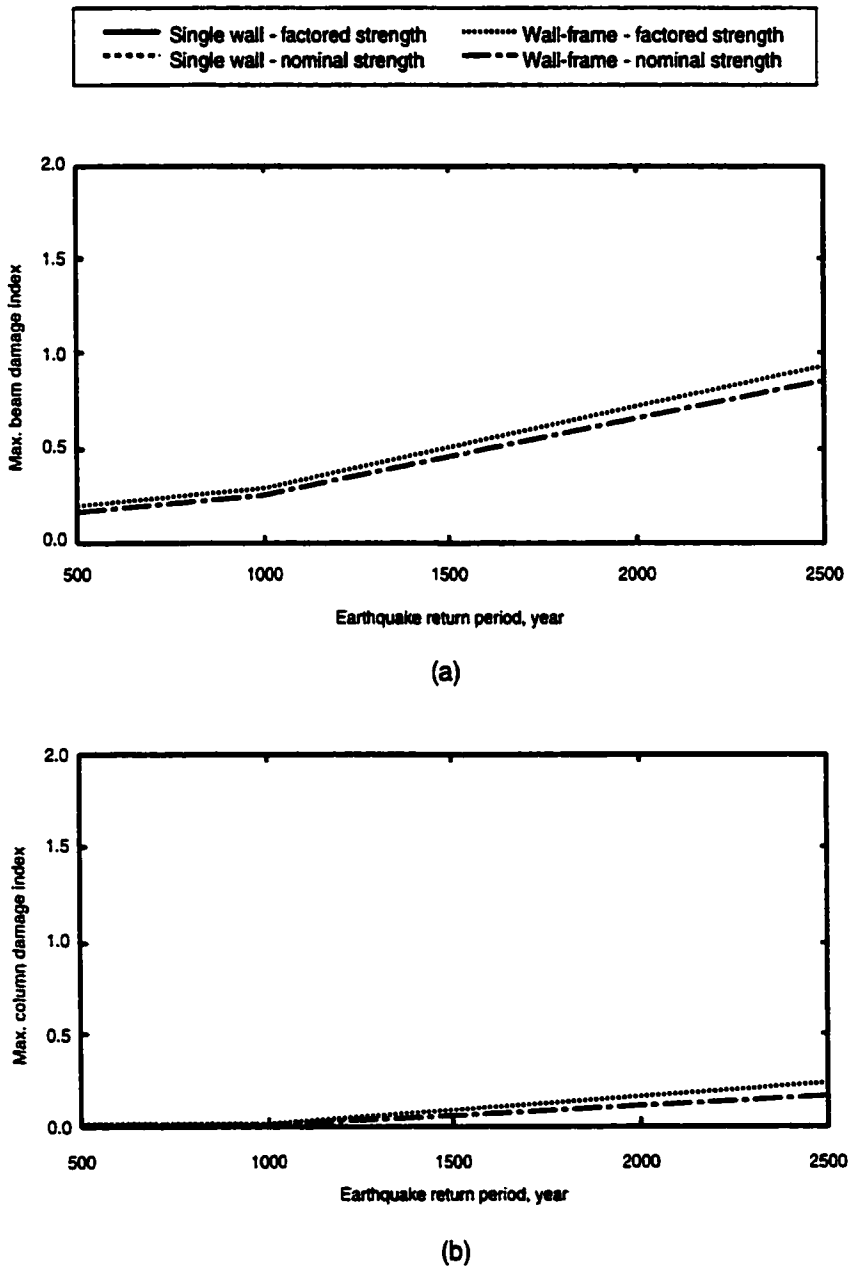
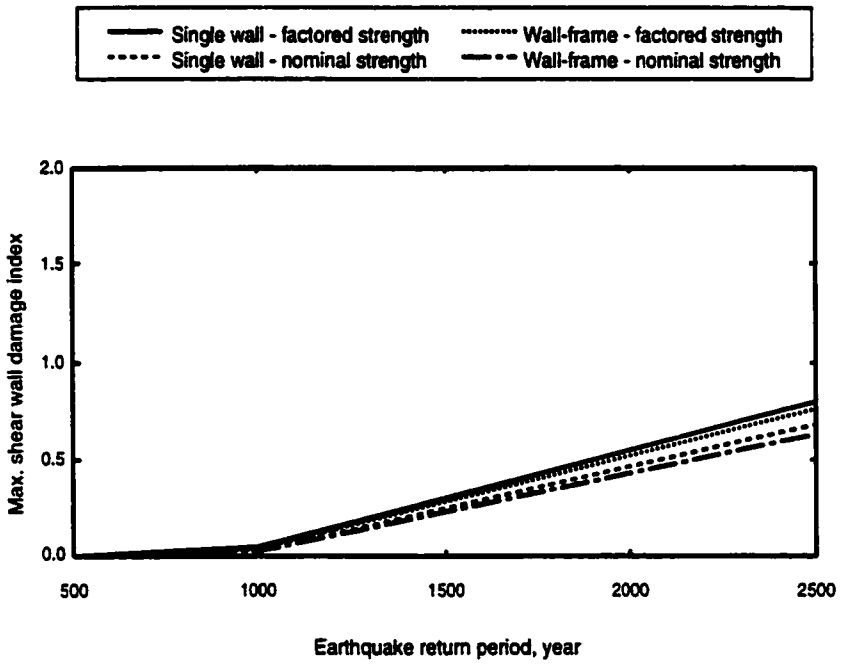
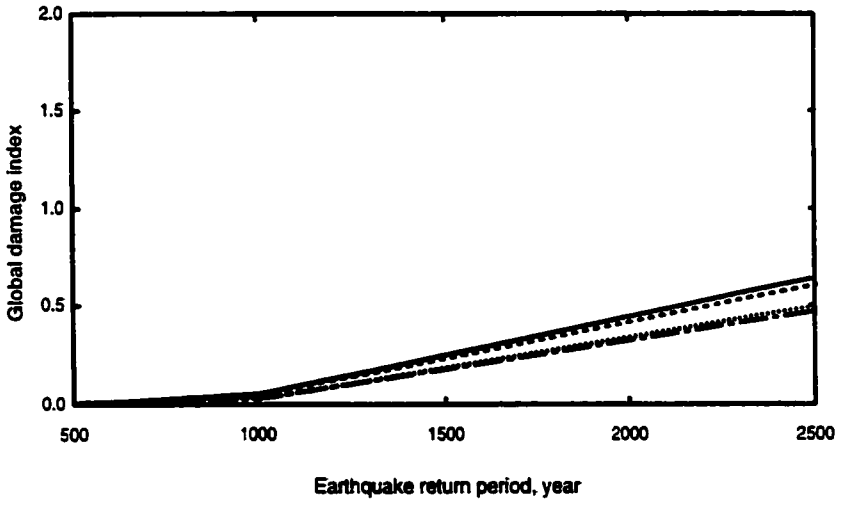


Figure 4.22: Maximum element level (beam and column) Park-Ang damage index in the twenty-storey shear wall building in Victoria: (a) beam, and (b) column

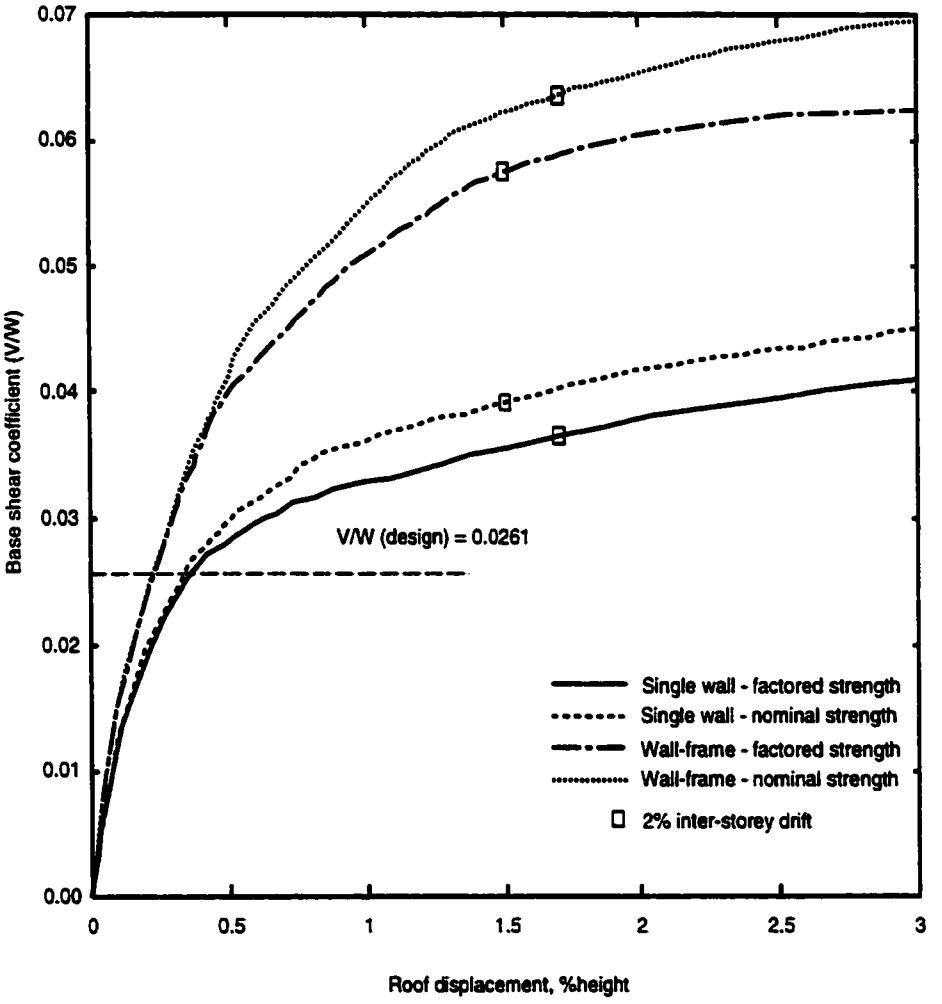


(a)



(b)

Figure 4.23: Maximum element level (shear wall) and global Park-Ang damage index in the twenty-storey shear wall building in Victoria: (a) shear wall, and (b) global


0.07
0.06
0.05
0.04
0.03
0.02
0.01
0.00
Base shear coefficient (V/W)
V/W (design) = 0.0261
Single wall - factored strength
Single wall - nominal strength
Wall-frame - factored strength
Wall-frame - nominal strength
2% inter-storey drift
0
0.5
1
1.5
2
2.5
3
Roof displacement, %height

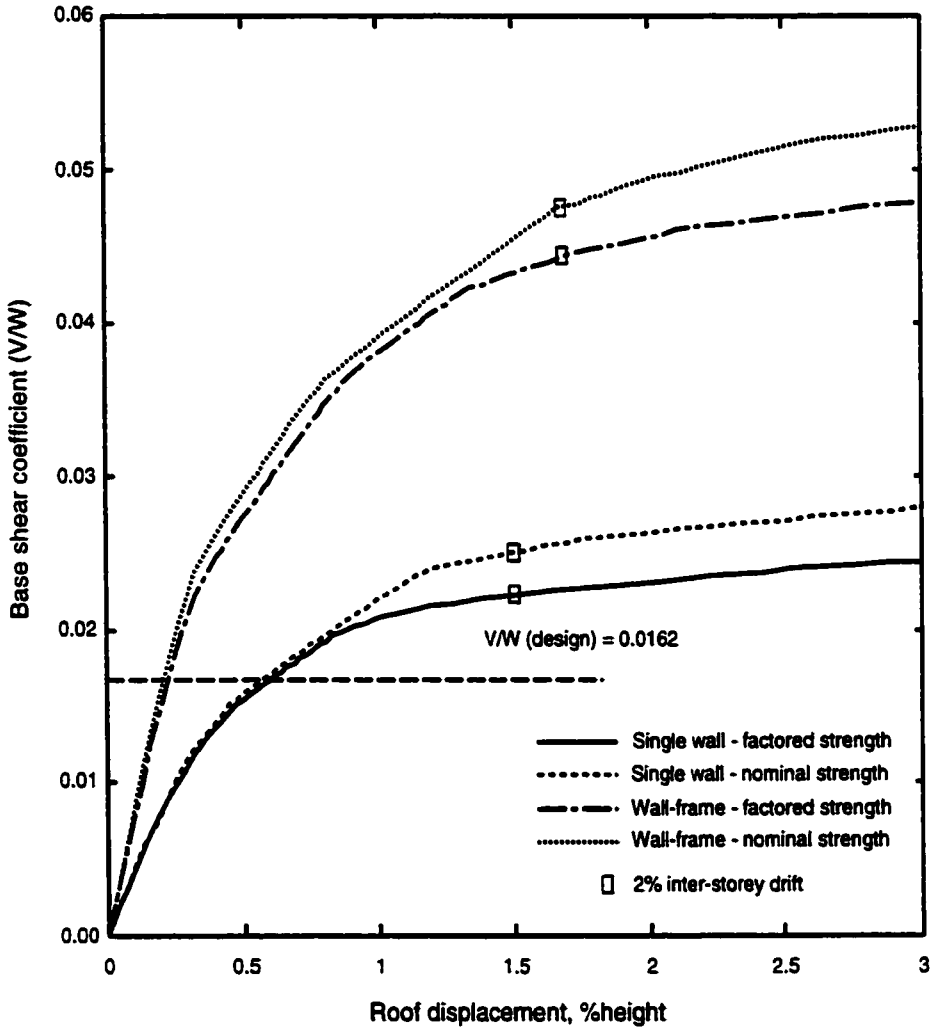


Figure 4.25: Push-over curves for the twenty-storey shear wall building in Montreal

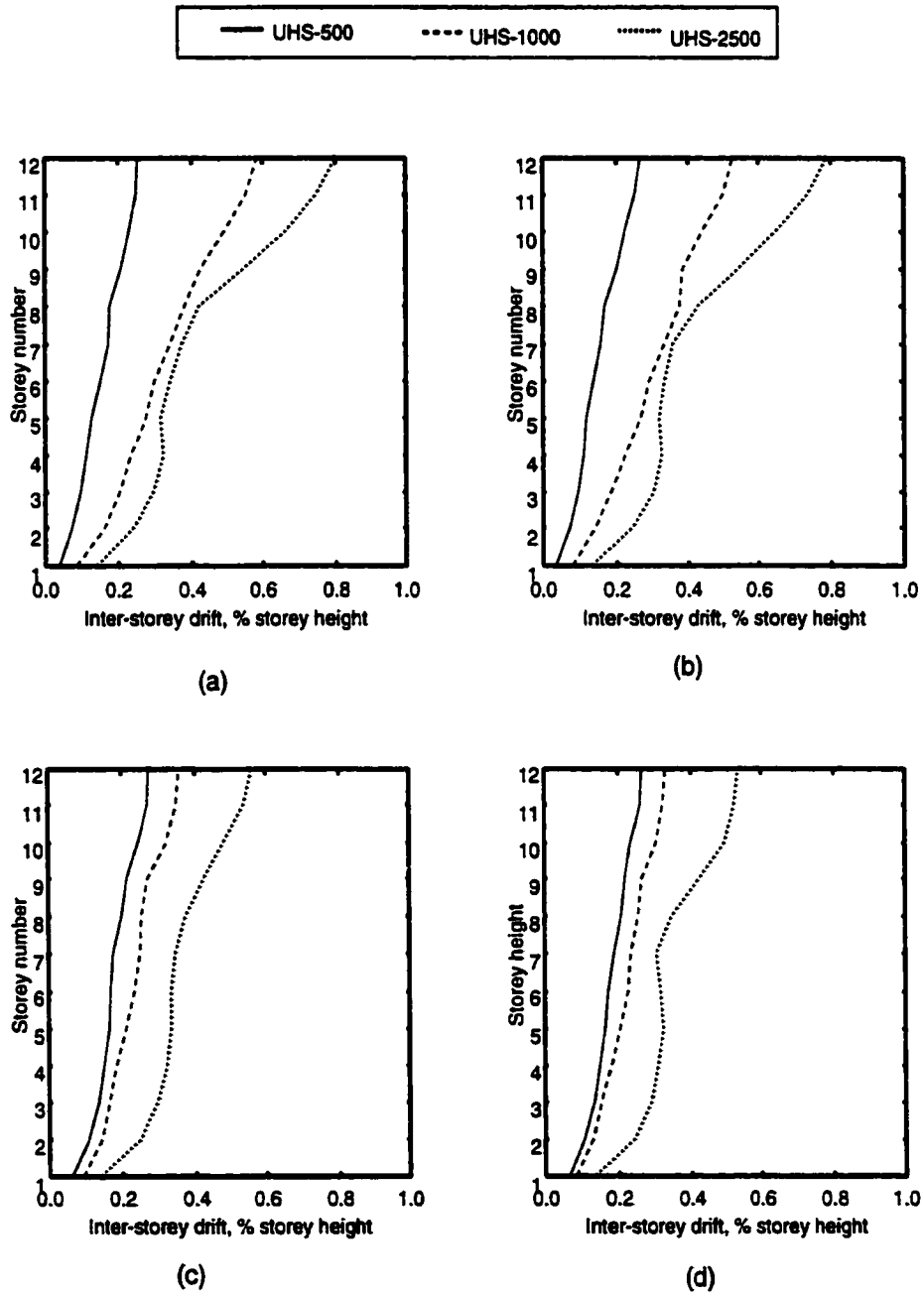


Figure 4.26: Inter-storey drift envelopes for the twelve-storey shear wall building in Montreal: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

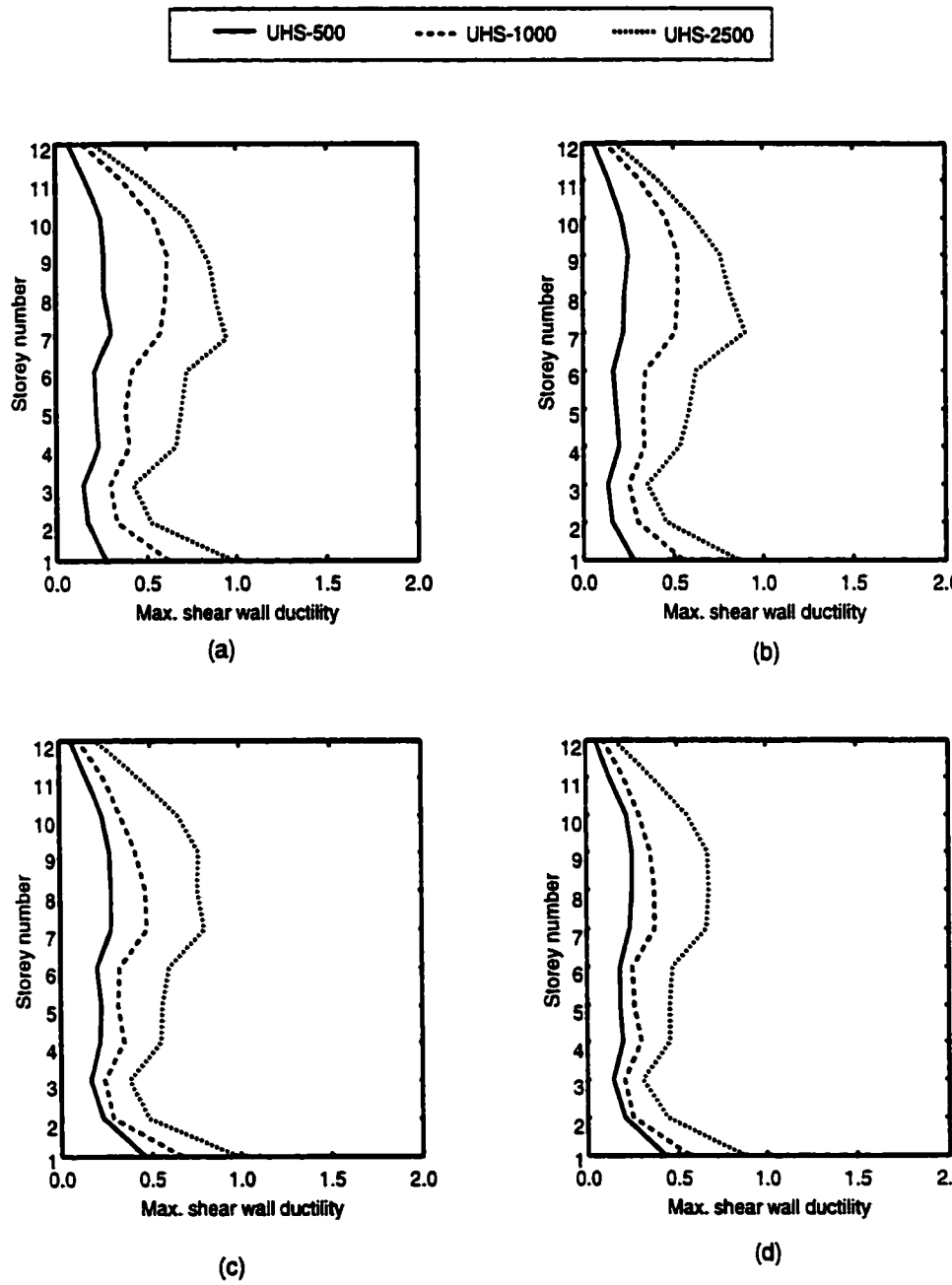
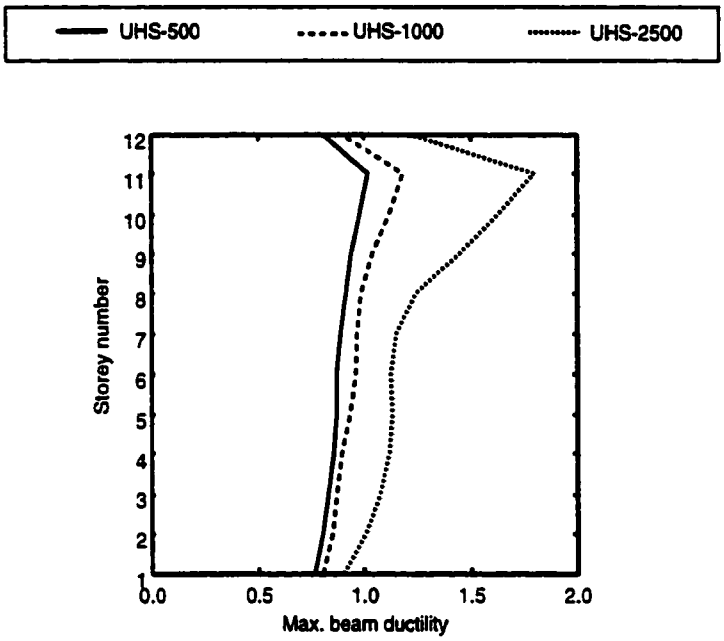
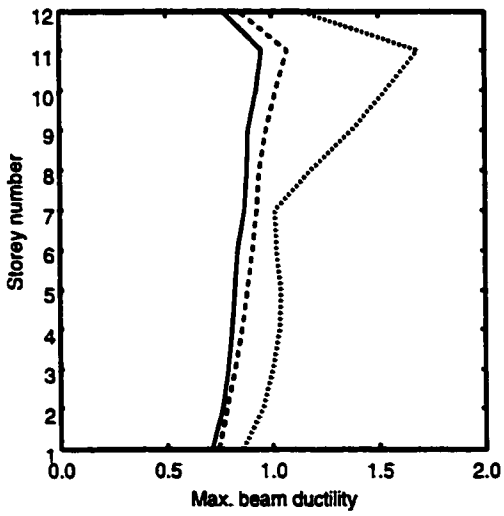


Figure 4.27: Maximum shear wall ductility demand in the twelve-storey shear wall building in Montreal: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength



(a)



(b)

Figure 4.28: Maximum beam ductility demand in the twelve-storey shear wall building in Montreal: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength

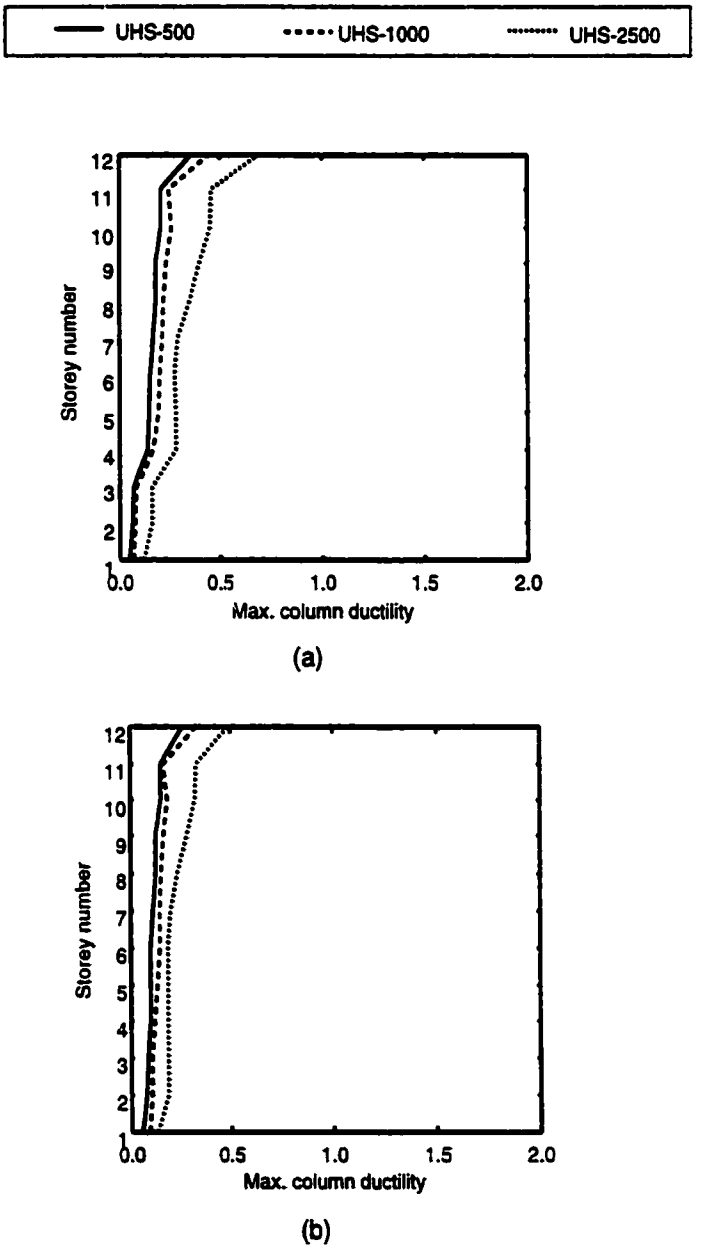
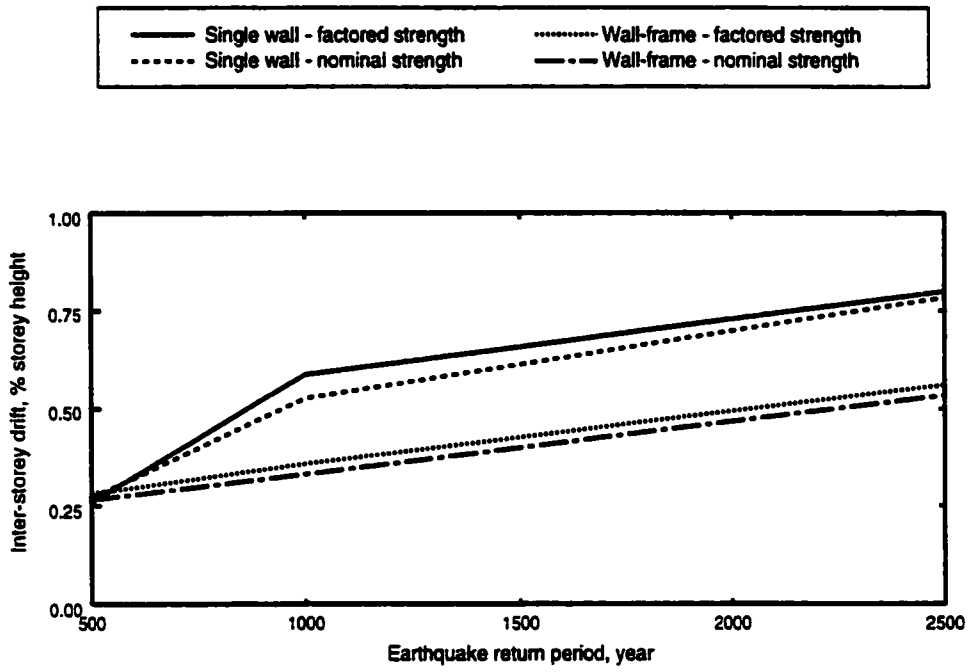
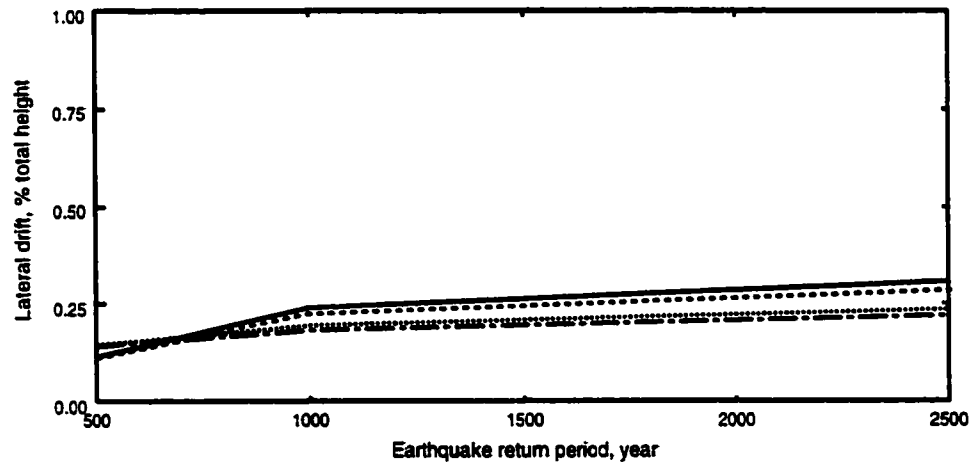


Figure 4.29: Maximum column ductility demand in the twelve-storey shear wall building in Montreal: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength



(a)



(b)

Figure 4.30: Peak displacement response in the twelve-storey shear wall building in Montreal: (a) inter-storey drift, (b) roof displacement (lateral drift)

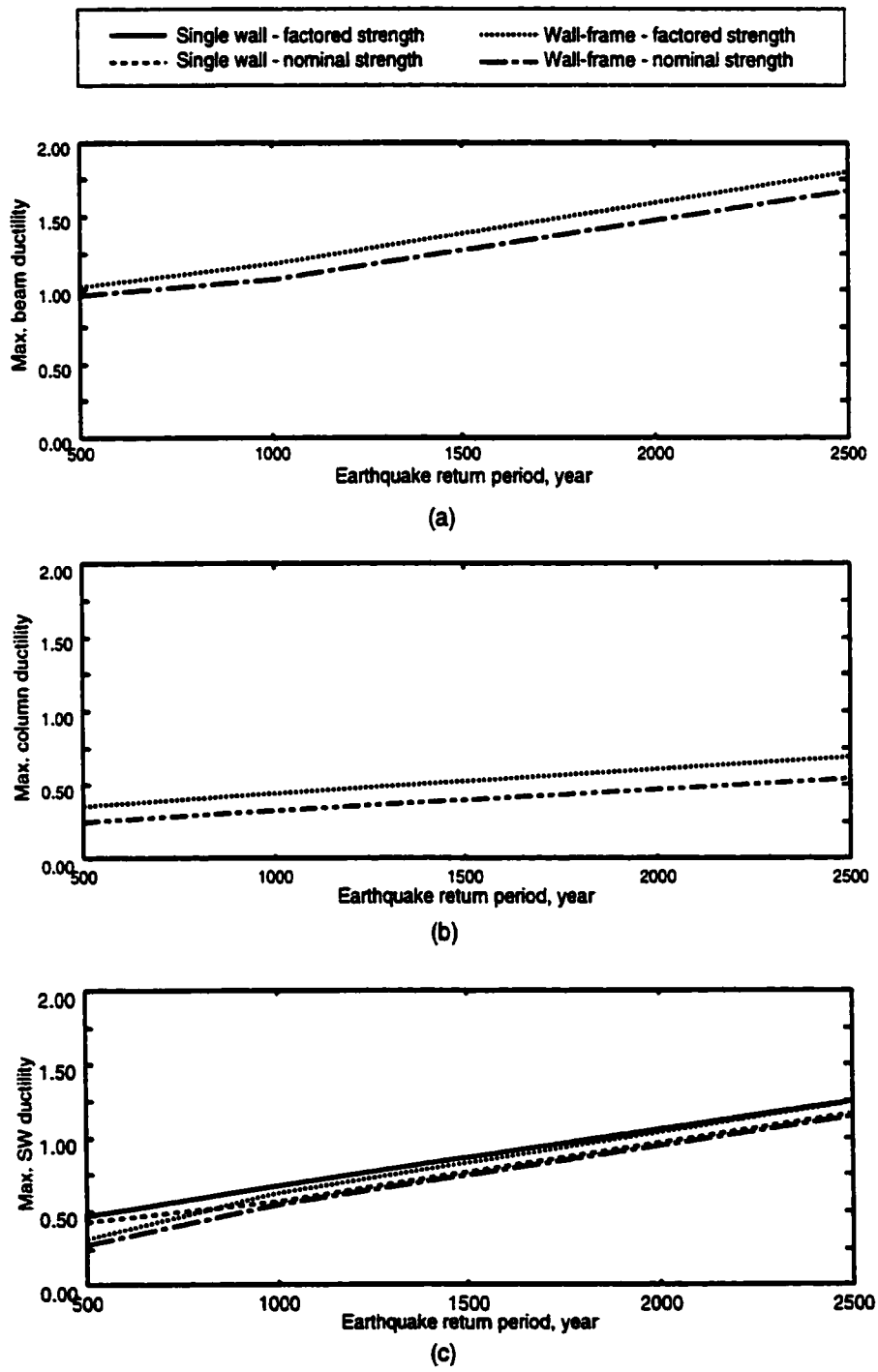


Figure 4.31: Maximum element ductility demand in the twelve storey shear wall building in Montreal: (a) beam, (b) column, and (c) shear wall

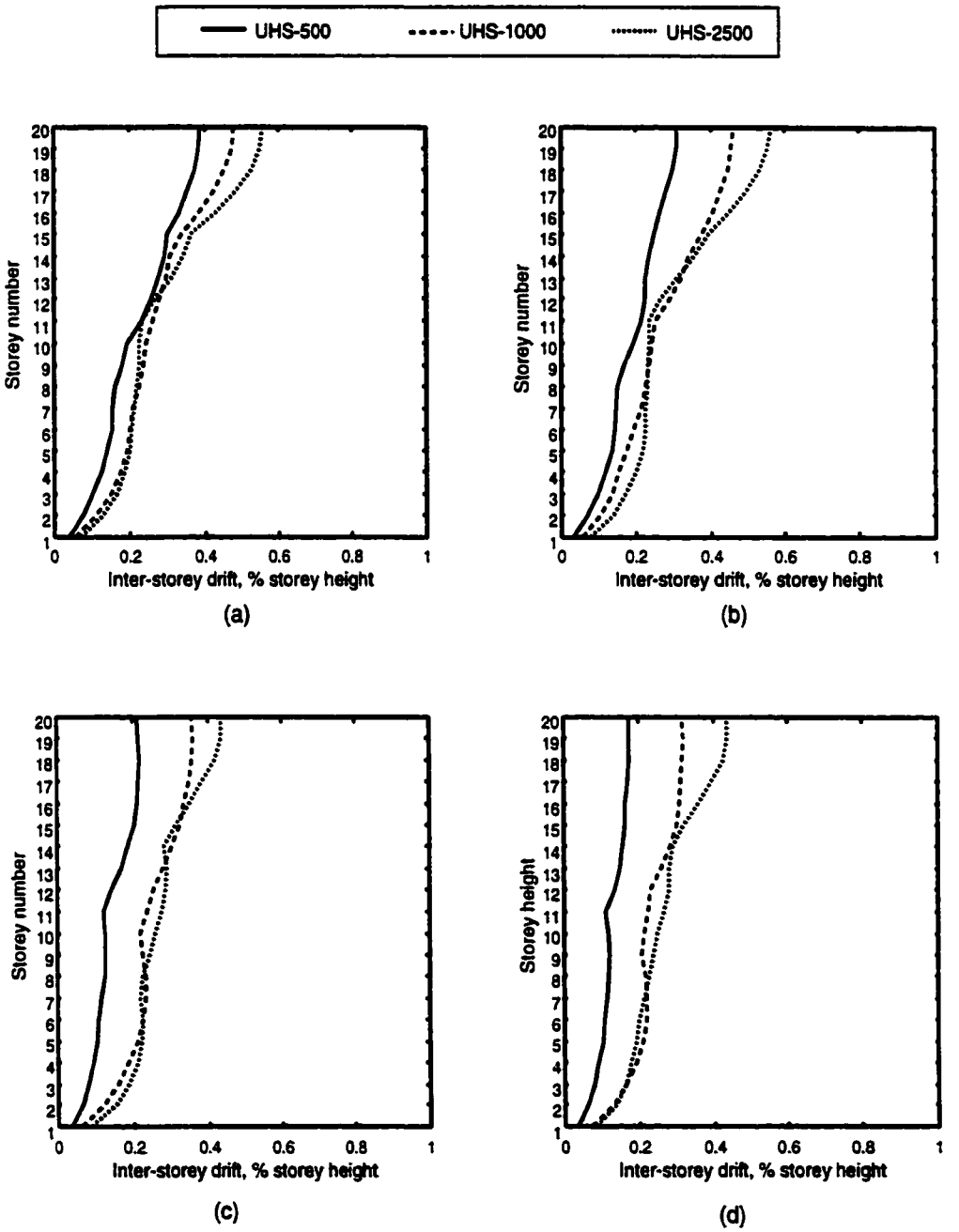


Figure 4.32: Inter-storey drift envelopes for the twenty-storey shear wall building in Montreal: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

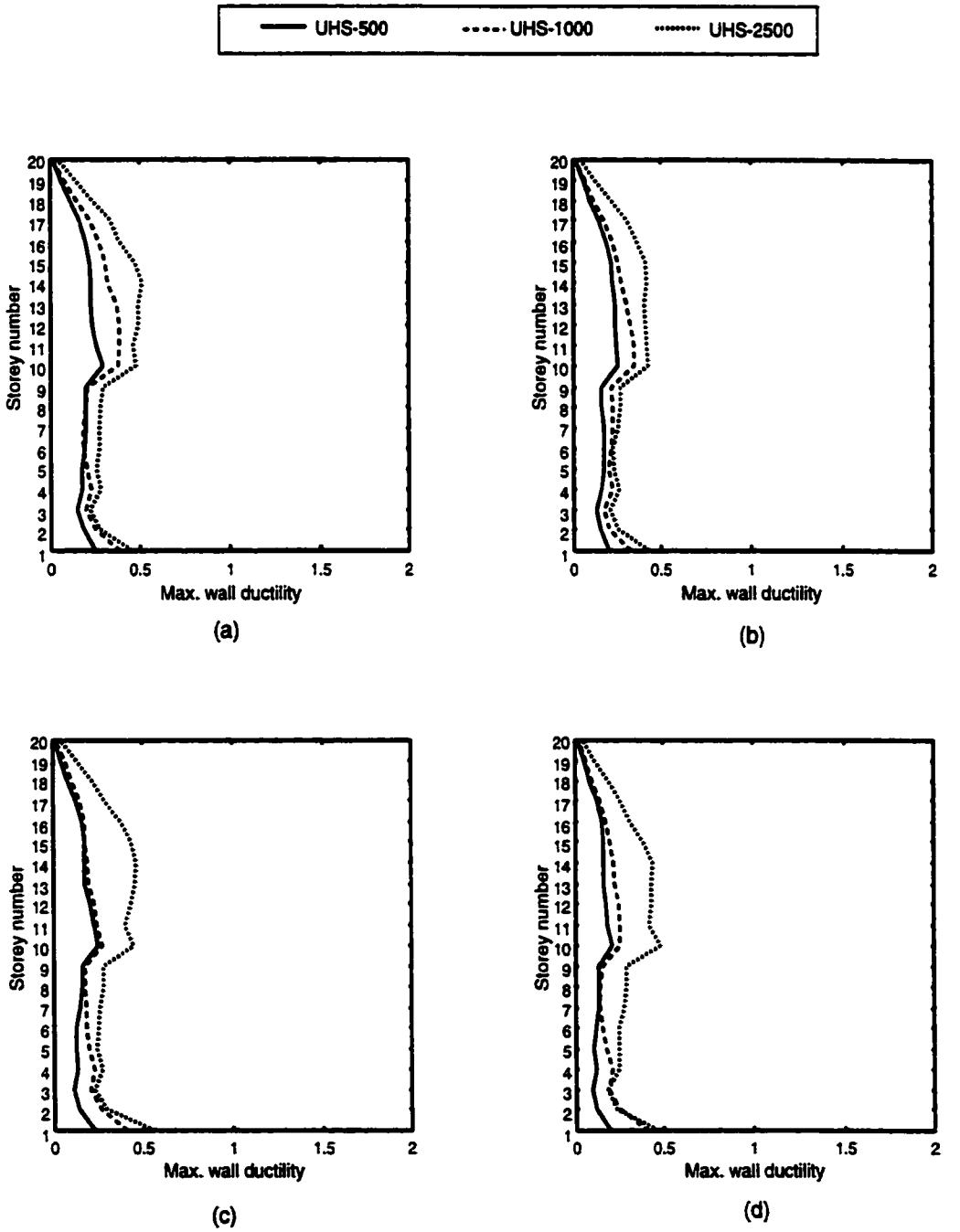


Figure 4.33: Maximum shear wall ductility demand in the twenty-storey shear wall building in Montreal: (a) wall model with factored strength, (b) wall model with nominal strength, (c) wall-frame model with factored strength, (d) wall-frame model with nominal strength

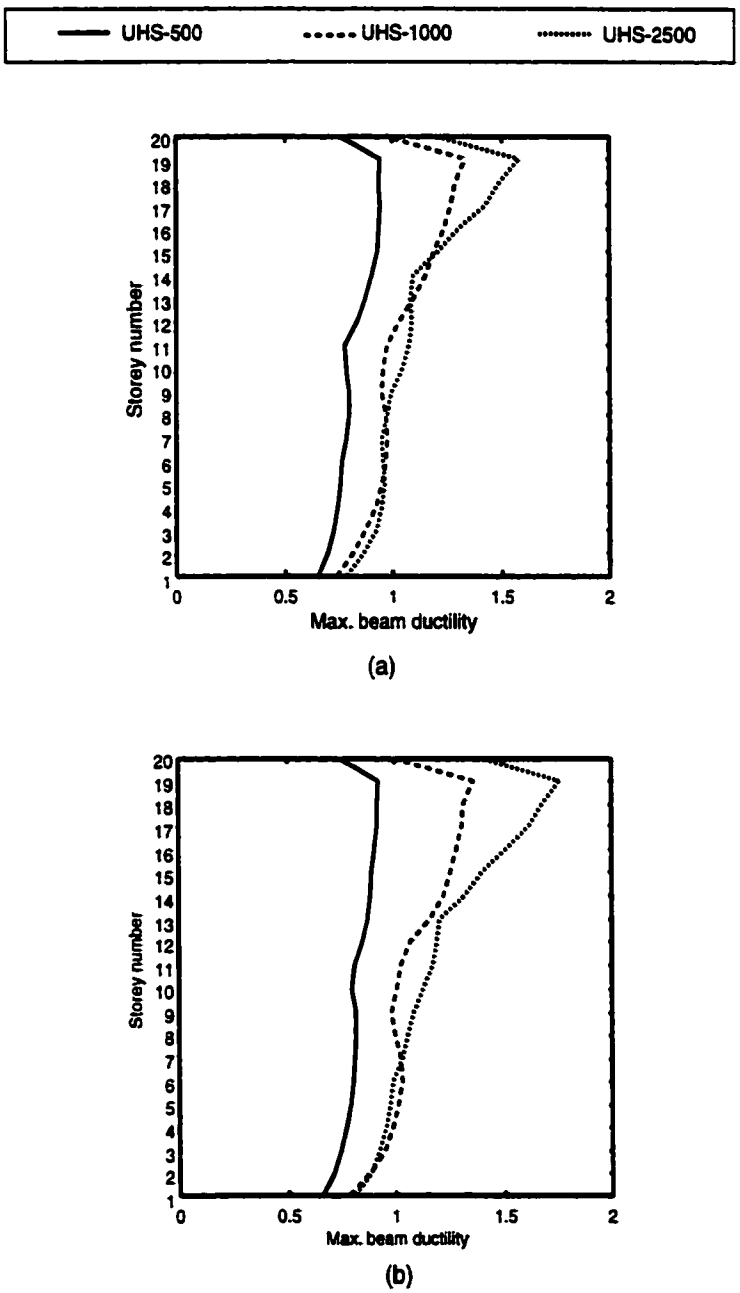


Figure 4.34: Maximum beam ductility demand in the twenty-storey shear wall building in Montreal: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength

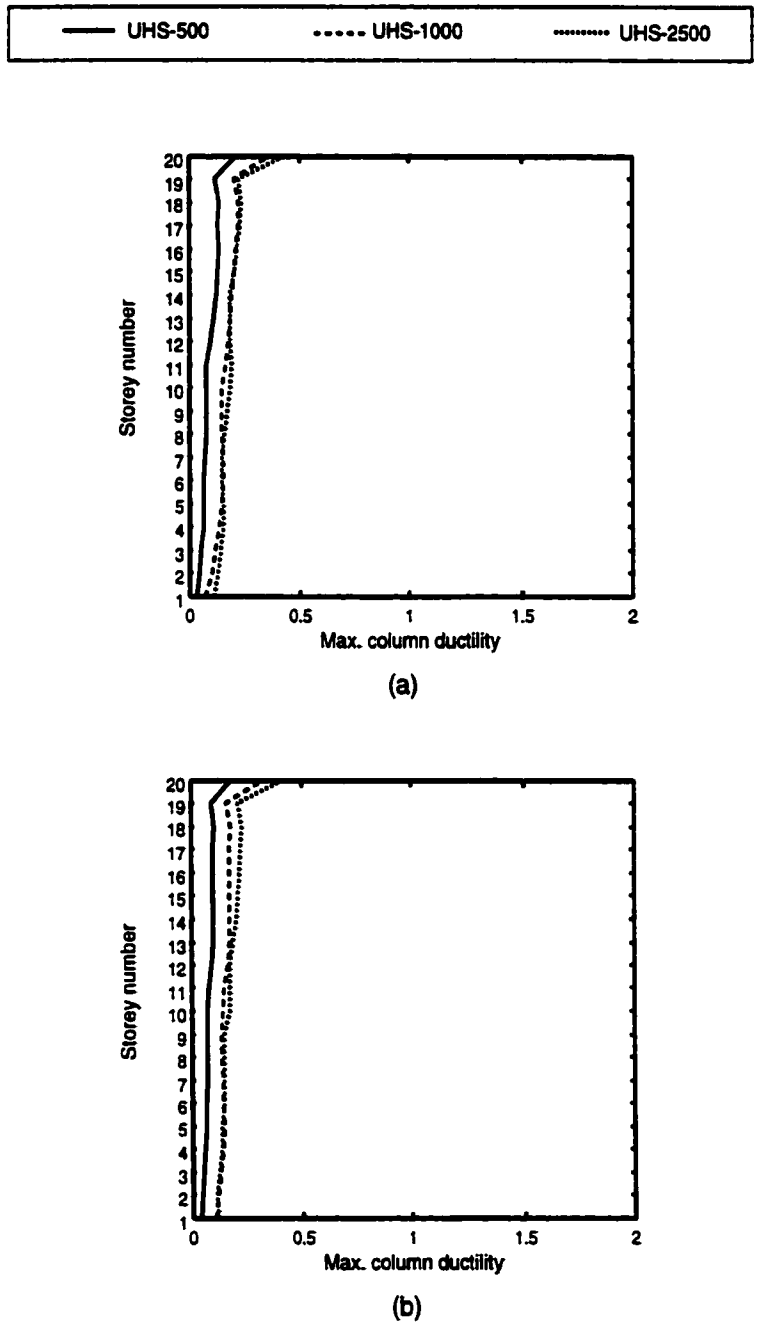


Figure 4.35: Maximum column ductility demand in the twenty-storey shear wall building in Montreal: (a) wall-frame model with factored strength, (b) wall-frame model with nominal strength

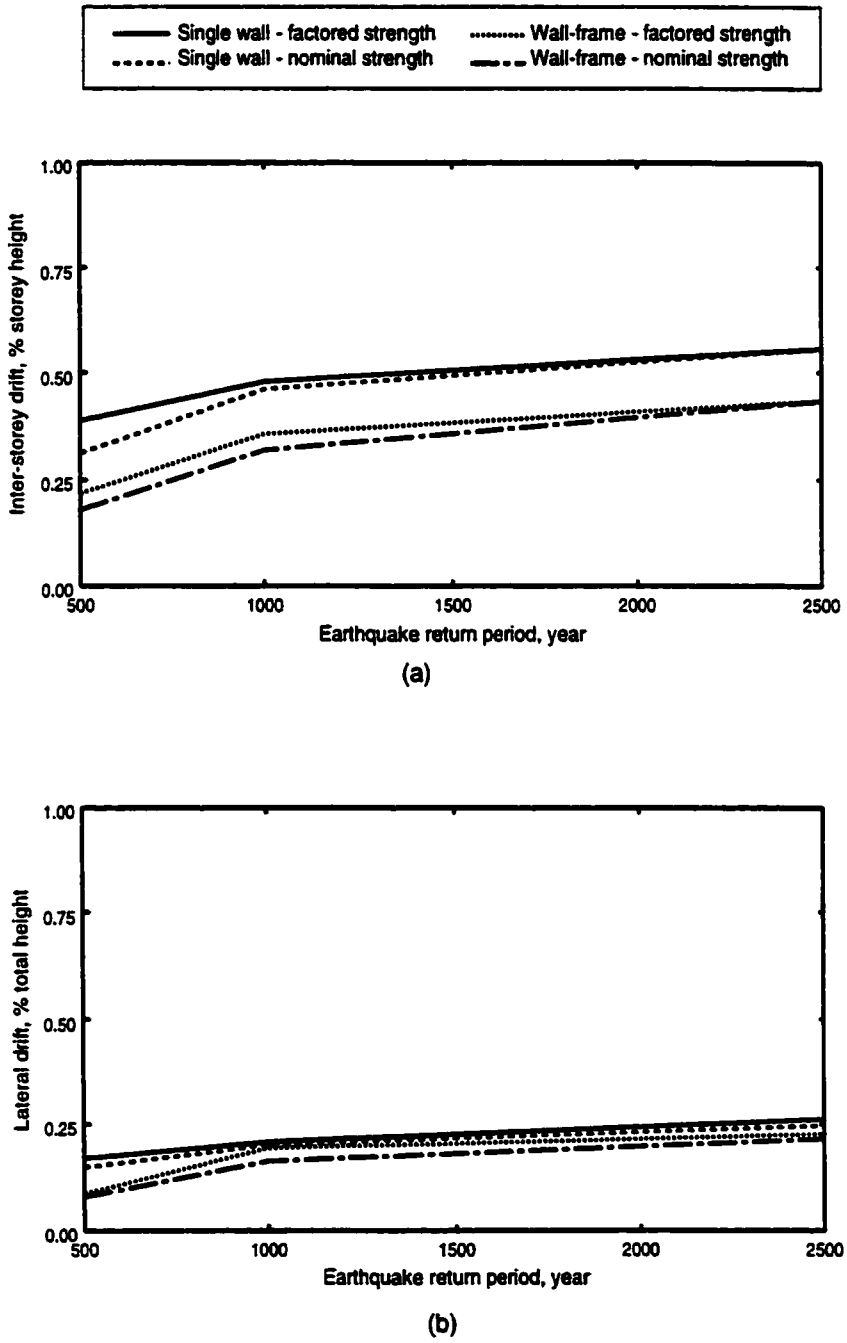


Figure 4.36: Peak displacement response in the twenty-storey shear wall building in Montreal: (a) inter-storey drift, (b) roof displacement (lateral drift)

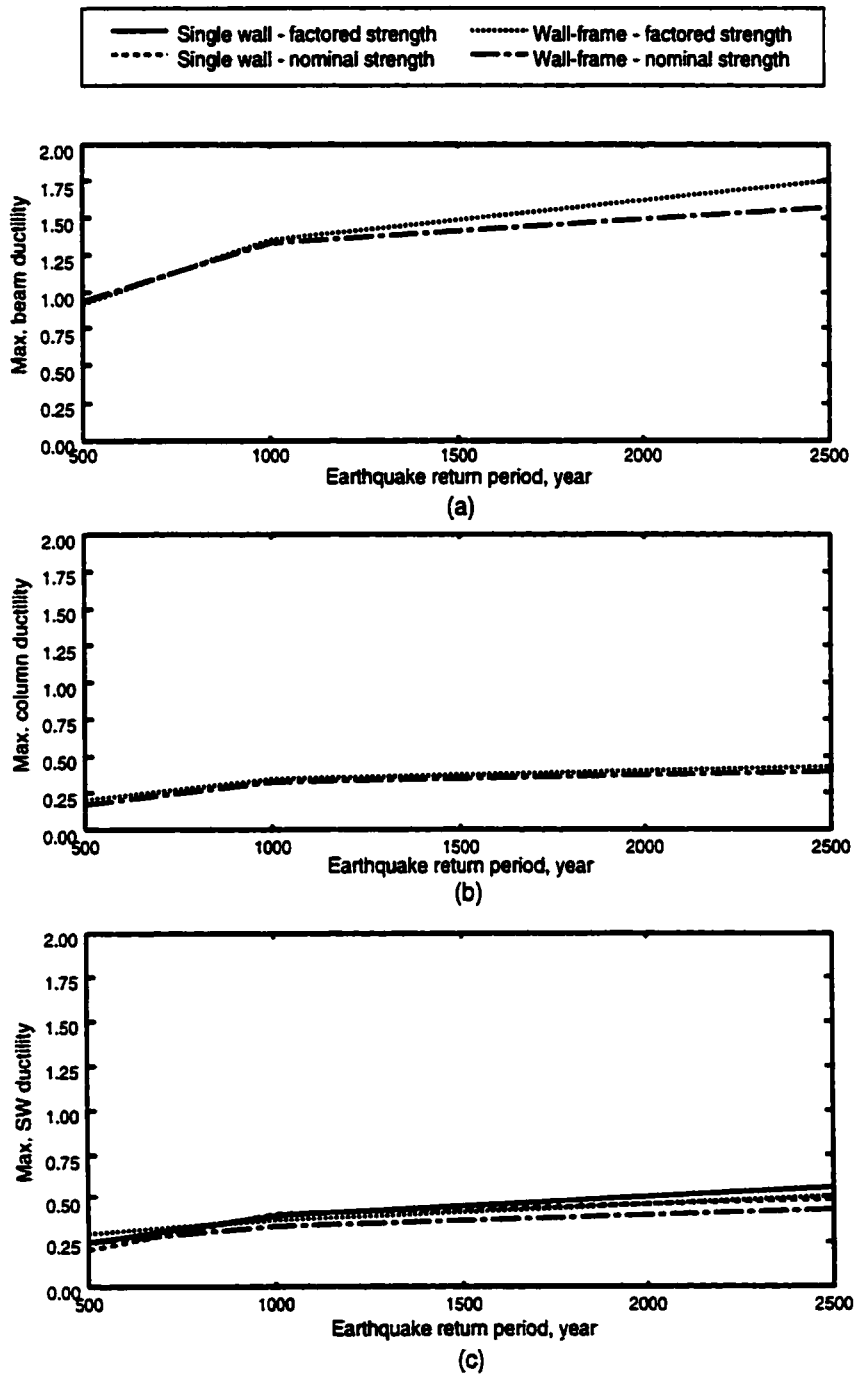


Figure 4.37: Maximum element ductility demand in the twenty-storey shear wall building in Montreal: (a) beam, (b) column, and (c) shear wall

Chapter 5

Simplified Method of Performance Evaluation

5.1 Introduction

Based on the study presented in the previous chapters, a simplified method for the evaluation of seismic performance is suggested here. This method is essentially based on a static push-over analysis of a multi-storey frame (MDOF system). An equivalent single degree of freedom system (SDOF) is derived from the push-over curve. The peak response of the single degree of freedom system is obtained from dynamic analyses, and the push-over analysis results are used for interpreting the SDOF response to get an estimate of the response of the actual structure.

In the N2 method, suggested by Fajfar *et al* (1996), an equivalent SDOF system is derived using the static displacement shape of the original structure produced by an assumed distribuition of the lateral forces. The push-over curve of the original system is idealized as a bilinear curve. The static displacement shape and bilinear push-over curve are used for reducing the system properties of the original structure to

obtain those of an equivalent SDOF system. Seismic demand for the SDOF system is then obtained using inelastic response spectra, and the seismic demand for the MDOF structure is determined by a second stage push over analysis in which the structure is pushed to the maximum displacement determined for the SDOF model. The method was applied to various structures including infilled frames and bridge structures (Fajfar *et al* 1997).

The simplified analysis approach suggested by Mazzolani *et al* (1997), is based on the trilinear representation of the load-displacement curve and elastic and rigid-plastic analysis of the equivalent SDOF system. The response of the equivalent SDOF system is directly mapped to the performance of the overall system. The method was used for evaluating the seismic performance of moment-resisting steel frames.

Based on the dynamic or push-over analyses of a building frame, relationships between the roof-displacement and other damage parameters can be established. Relationships between roof-displacement and the damage index were established by Ghobarah *et al* (1997) for the seismic performance evaluation. Such relationships could be used for interpreting the response of the SDOF structure and evaluating the performance of the building based on the SDOF response.

In the method suggested here an equivalent SDOF system is derived based on the bilinear idealization of the push-over curve of the original structure. The peak response of the SDOF system for a given hazard level can be obtained either by using inelastic response specta or performing inelastic dynamic analyses using corresponding ground motion records. The peak SDOF response is then translated to the performance of the MDOF system using the results of the push-over analysis

performed earlier.

The proposed method bears some resemblence to the N2 method mentioned above. However the procedures for deriving the equivalent SDOF system and the method of estimating the MDOF response from the SDOF response are quite different. Unlike the N2 method, the static displacement shape of the original structure is not used in the derivation of the equivalent SDOF system and a second stage push-over analysis is not required. A detailed description of the proposed method is given in the following section.

5.2 Description of the proposed method

Static push-over analysis is performed to construct the base shear - top displacement curve for the frame. A curve representing the relationship between the roof-displacement and the maximum inter-storey drift is also constructed.

The push-over curve is then idealized as a bilinear force-displacement relationship as shown in Fig. 5.1, where F_y and D_y are the effective yield force and yield displacement respectively; and F_r is the force corresponding to a reference displacement beyond yield. The idealized bilinear curve is constructed based on the approximate equal energy criteria.

The equivalent stiffness of the single degree of freedom system (SDOF) is obtained from the following expressions

$$k = F_y/D_y \tag{5.1}$$

where, k is the stiffness of the equivalent SDOF system. The period of the SDOF system is given by

$$T = 2\pi\sqrt{m/k} \tag{5.2}$$

where, m is the mass of the equivalent system. The period T is taken to be equal to the fundamental period of the structure. Equation 5.2 thus allows the determination of mass m of the equivalent system.

Seismic response of the SDOF system is obtained through a series of inelastic dynamic analyses for the UHS-based ground motion records corresponding to all three levels of seismic hazard (16 records in total). The peak displacement response could also be obtained using the inelastic response spectra for different levels of seismic hazard, if such spectra are available.

Peak displacement response of SDOF system corresponding to each level of seismic hazard is taken to be equal to the peak roof-displacement of the MDOF building frame which is then used to predict the global damage in the MDOF structure. The curve representing the relationship between the roof-displacement and the maximum inter-storey drift is used to obtain the maximum inter-storey drift in the MDOF model corresponding to the peak roof-displacement obtained for the SDOF model. The maximum value of inter-storey drift is used for determining the qualitative level of seismic performance achieved by the building.

Table 5.1: Description of the building models

Model	Description
CMRF-V6FB	Six storey bare CMRF building
CMRF-V6FP	Six storey infilled CMRF building
CMRF-V12FB	Twelve storey bare CMRF building
CMRF-V12FP	Twelve storey infilled CMRF building
CSWF-V12FW	Twelve storey CSWF building: wall model
CSWF-V12FF	Twelve storey CSWF building: wall-frame model
CSWF-V20FW	Twenty storey CSWF building: wall model
CSWF-V20FF	Twenty storey CSWF building: wall-frame model

5.3 Examples

The building models described in Table 5.1 are considered here to demonstrate the effectiveness of this method for the evaluation of the seismic performance of buildings. These models are chosen from those studied in Chapters 3 and 4. All buildings are assumed to be located in Victoria and only factored material strength is considered.

The push-over curves and corresponding idealized bilinear load-deformation curves for the building models considered here are shown in Figures 5.2 through 5.5. Details of the models are given in Table 5.2, where H is the height, W is the weight, F_y is the yield lateral load, D_y is the lateral drift at yield, F_r is the lateral load corresponding to a roof-displacement of 2%, and T_0 is the fundamental period. The equivalent SDOF models can be derived from this information. The properties of the equivalent SDOF models are given in Table 5.3.

The relationship between the roof-displacement and the maximum inter-storey drift for each model are shown in Figures 5.6 through 5.9. Peak SDOF response is

Table 5.2: Details of the building models

Model	H (m)	W (kN)	F_y/W	D_y (%)	F_r/W	T_0
CMRF-V6FB	22.80	3592	0.160	0.55	0.180	1.47
CMRF-V6FP	22.80	3592	0.310	0.45	0.330	0.77
CMRF-V12FB	44.70	7372	0.070	0.58	0.075	2.91
CMRF-V12FP	44.70	7372	0.152	0.43	0.155	1.29
CSWF-V12FW	44.70	7372	0.072	0.40	0.085	2.00
CSWF-V12FF	44.70	7372	0.135	0.55	0.160	1.80
CSWF-V20FW	73.90	12436	0.048	0.60	0.058	3.57
CSWF-V20FF	73.90	12436	0.070	0.60	0.080	2.70

Table 5.3: Details of the equivalent SDOF models

Model	Height	Stiffness, k (kN/m)	Mass, m (kN)
CMRF-V6FB	22.80	4544.5	274.0
CMRF-V6FP	22.80	10503.0	183.0
CMRF-V12FB	44.70	2199.0	507.0
CMRF-V12FP	44.70	5811.5	254.0
CSWF-V12FW	44.70	4090.0	300.0
CSWF-V12FF	44.70	6322.0	382.0
CSWF-V20FW	73.90	1346.3	394.0
CSWF-V20FF	73.90	1988.8	459.0

Table 5.4: Response under UHS-500 events

Building model	D_r (SDOF) %H	Estimated drift D_s(approx), %h	Actual MDOF drift, D_s, %h	D_s (approx)/ D_s (MDOF)
CMRF-V6FB	0.40	0.57	0.60	0.95
CMRF-V6FP	0.35	0.48	0.50	0.96
CMRF-V12FB	0.50	0.90	1.00	0.90
CMRF-V12FP	0.30	0.42	0.45	0.93
CSWF-V12FW	0.70	0.94	1.10	0.86
CSWF-V12FF	0.45	0.65	0.75	0.87
CSWF-V20FW	0.45	0.62	0.60	1.03
CSWF-V20FF	0.35	0.50	0.55	0.91

calculated for each model and the maximum inter-storey drift is obtained from these curves.

Tables 5.4 through 5.6 show the response of the SDOF models and how it compares to that of the MDOF models. In these tables the roof-displacement, D_r is expressed as a percentage of the building height, H and the inter-storey drift, D_s is expressed as a percentage of the storey height, h.

As mentioned earlier, the bilinear idealization of the push-over curve is based on the approximate equal energy criteria. The idealized bilinear curve follows the push-over curve closely. Minor variation in the bilinear curve does not change the estimated drift response of a structure drastically.

Accounting for the effects of higher modes

The simplified method presented here is based on a push-over analysis of the original structure. The distribution of lateral loads used in the push-over analysis is based on the NBCC 1995 guidelines as explained in Chapter 2. Push-over analysis

Table 5.5: Response under UHS-1000 events

Building model	D_r (SDOF) %H	Estimated drift D_s(approx), %h	Actual MDOF drift, D_s, %h	D_s (approx)/ D_s (MDOF)
CMRF-V6FB	0.45	0.65	0.70	0.93
CMRF-V6FP	0.42	0.62	0.60	1.03
CMRF-V12FB	0.70	1.25	1.20	1.04
CMRF-V12FP	0.35	0.50	0.55	0.91
CSWF-V12FW	0.80	1.10	1.20	0.92
CSWF-V12FF	0.55	0.75	0.80	0.94
CSWF-V20FW	0.55	0.70	0.70	1.00
CSWF-V20FF	0.40	0.55	0.60	0.92

Table 5.6: Response under UHS-2500 events

Building model	D_r (SDOF) %H	Estimated drift D_s(approx), %h	Actual MDOF drift, D_s, %h	D_s (approx)/ D_s (MDOF)
CMRF-V6FB	1.10	1.95	2.2	0.89
CMRF-V6FP	0.80	1.52	1.7	0.89
CMRF-V12FB	1.32	2.50	2.5	1.00
CMRF-V12FP	0.75	1.65	1.5	1.10
CSWF-V12FW	1.70	2.40	2.8	0.86
CSWF-V12FF	1.25	1.70	2.0	0.85
CSWF-V20FW	1.45	2.00	2.4	0.83
CSWF-V20FF	0.95	1.27	1.5	0.85

using the NBCC 1995 distribution of lateral loads may fail to capture the effects of higher modes and the push-over results may not provide the accurate prediction of the dynamic behaviour of the structure. Humar and Rahgozar (2000), and Humar and Mahgoub (2000) have shown that the effects of higher modes on the base-shear depends on structural types, modal periods and spectral shapes. The higher mode effect is found to be more pronounced for flexural wall systems than for shear frame structures. Also, the higher mode effect is more significant for structures located in the eastern regions of Canada than in the western regions.

To include the effects of higher modes in the present analysis, the following models are cosidered: CSWF-V12FW and CSWF-V20FW (see Table 5.1). For a building model, the contribution of each mode to the base shear is computed using the method suggested in Humar and Rahgozar (2000). Components of the base shear corresponding to the individual modes are calculated based on the modal contribution ratios. Each component is distributed along the height of the building according to the corresponding modal shape. The modal distributions of base shear components are then combined together using square root of sum squares (SRSS) technique and scaled to the design base shear. This SRSS modal distribution of lateral forces is used in the push-over analysis. The proposed simplified method is then used to evaluate the seismic response. Only first two modes are considered in these examples.

Figure 5.10 shows the design spectral acceleration curve (based on UHS-2500) for Victoria. The modal periods for CSWF-V12FW model are 2.05 s and 0.45 s. Corresponding values of effective modal weight are $0.68W$ and $0.18W$, W being the weight of the structure. The spectral acceleration corresponding to each mode is

Table 5.7: Higher mode effect on base-shear

Building model	Period s	Eff. Modal weight	Spectral accel.	Modal base-shear factor	Ratio of modal base-shear
CSWF-V12FW	2.05	0.68 W	0.19 g	0.129	0.45
	0.45	0.18 W	0.88 g	0.158	0.55
CSWF-V20FW	3.57	0.64 W	0.10 g	0.134	0.47
	0.58	0.20 W	0.75 g	0.150	0.53

obtained from Figure 5.10 (a). The values of spectral acceleration corresponding to the first mode and the second mode are $0.19g$ and $0.88g$, respectively (g is the acceleration due to gravity). On multiplying the spectral acceleration and the modal weight we get the modal base shear. These values are tabulated in Table 5.7. The ratios of modal base shear indicate that the base shear contribution of the second mode is higher than the first mode. The base shear contribution of the second mode is also quite considerable in the twenty-storey shear wall building model, CSWF-VICT20FW (Table 5.7).

The design base shear for each model is divided according to the modal contribution ratios and each component of the base shear is distributed according to the corresponding mode shapes. Figure 5.11 shows the distribution of modal base shear contributions. The SRSS combination of the modal base-shear distributions are also shown. For the purpose of comparison, the NBCC 1995 distribution and the distribution of base shear according to the first mode only are also shown. It is observed that the NBCC distribution follows the the first mode only distribution more closely than the SRSS modal distribution. To account for the higher order modes, the NBCC distribution assigns higher magnitude of lateral force at the top storey.

Table 5.8: Estimated drift by the proposed simplified method

Model	Method	UHS-500	UHS-1000	UHS-2500
CSWF-V12FW	NBCC-1995	0.94	1.10	2.40
	SRSS-modal	0.90	1.02	2.50
	Actual (MDOF)	1.10	1.20	2.80
CSWF-V20FW	NBCC-1995	0.62	0.70	1.27
	SRSS-modal	0.58	0.65	1.30
	Actual (MDOF)	0.60	0.70	1.50

Push-over curves using the SRSS modal distribution of lateral forces are shown in Figure 5.12 and the corresponding roof displacement - maximum inter-storey drift curves are shown in Figure 5.13. Snapshots of inter-storey drift at 1% roof displacement for both building models are shown in Figure 5.14. It is observed form Figures 5.13 and 5.14 that the difference between the push-over response of a building using the NBCC distribution and the SRSS modal distribution of lateral forces is not very large.

Table 5.8 shows the response of the two building models considered here. The values of inter-storey drift estimated by the simplified method using the SRSS modal distribution of the lateral forces are not significantly different from those obtained using the NBCC distribution of the lateral forces.

5.4 Discussion

From the results presented above it is observed that the accuracy of the simplified method in predicting the maximum inter-storey drift is reasonably good in some cases. There are a few cases where the results differ from the actual values (obtained

using detailed analysis of the MDOF models) by as much as 20%. In most of the cases inter-story drift calculated using the simplified method is lower than the actual values. Given the nature of approximation involved in the proposed method, it's performance is acceptable. From the examples presented by Fajfar *et al* (1997), N2 method is seem to achieve a similar level of accuracy.

Using the inter-storey drift values obtained using the proposed simplified method, the following observations are made.

- The six-storey CMRF building in Victoria achieves a performance level of *operational* under UHS-500 and UHS-1000, and *life-safe* under UHS-2500 when the bare frame model is considered. When the infill frame model is considered, its performance levels become *fully operational* under UHS-500, *operational* under UHS-1000 and *life-safe* under UHS-2500.

- The twelve-storey CMRF building in Victoria achieves a performance level of *operational* under UHS-500 and UHS-1000, and *near collapse* under UHS-2500 when bare frame model is considered. With infill panels, its performance levels become *fully operational* under UHS-500, *operational* under UHS-1000 and *life-safe* under UHS-2500.

- The twelve-storey CSWF building in Victoria achieves a performance level of *operational* under both UHS-500 and UHS-1000 and *life-safe* under UHS-2500 when the wall model is considered. With the wall-frame model the performance levels become *operational* under both UHS-500 and UHS-1000 and *life-safe* under UHS-2500.

- The twenty-storey CSWF building in Victoria achieves a performance level of *operational* under both UHS-500 and UHS-1000 and *life-safe* under UHS-2500 for both wall and wall-frame models.

In most cases the qualitative performance levels mentioned above agree with the performance levels reported in Chapter 3 and 4.

Simplified analysis as described above could form an important second step in the design of an earthquake resisting building. The designer can quickly estimate the possible performance level of the structure and modify the design based on that. This method can be very useful in the design phase, when a number of iterations is generally needed to complete the design. Once the design is finalized, a detailed analysis could be performed to evaluate the seismic performance of the structure.

The simplified method presented here has some limitations. For cases where the effect of higher order modes is significant, this technique may not be effective in estimating the maximum inter-storey drift. Push-over analysis using the SRSS modal distribution of the lateral loads does not improve the solution very much.

The simplified methods such as the one proposed here may be criticized since they are based on many approximations. Designers must be aware of the limitations of such methods. The simplified method presented here will, however be of assistance to the designers in carrying out a performance-based design.

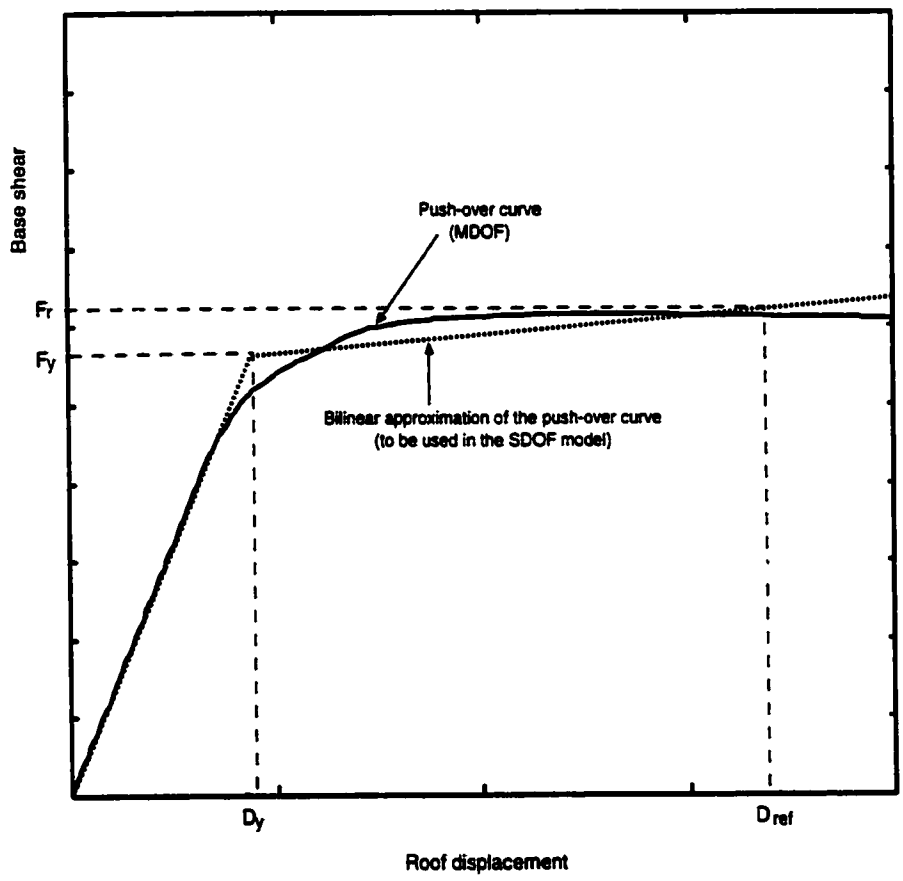


Figure 5.1: Bilinear idealization of push-over curve

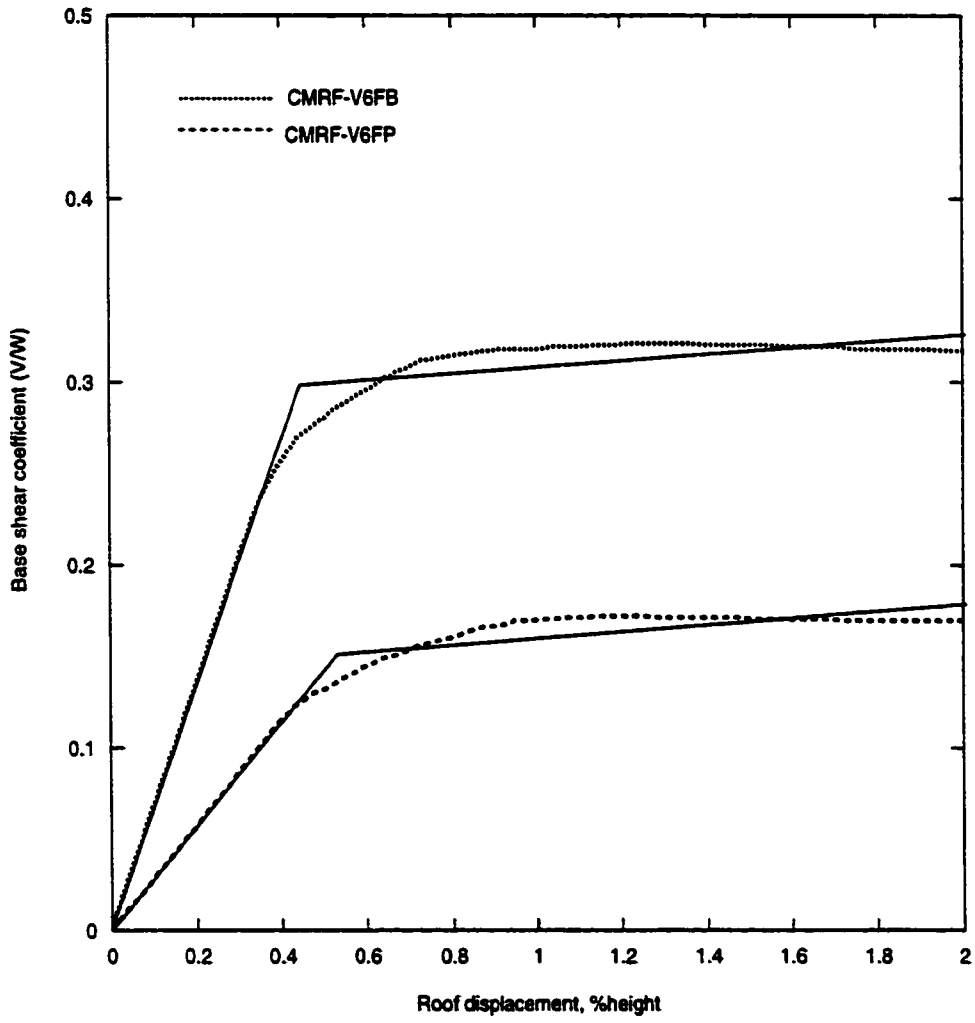


Figure 5.2: Bilinear idealization of the push-over curve: six-storey CMRF buildings in Victoria

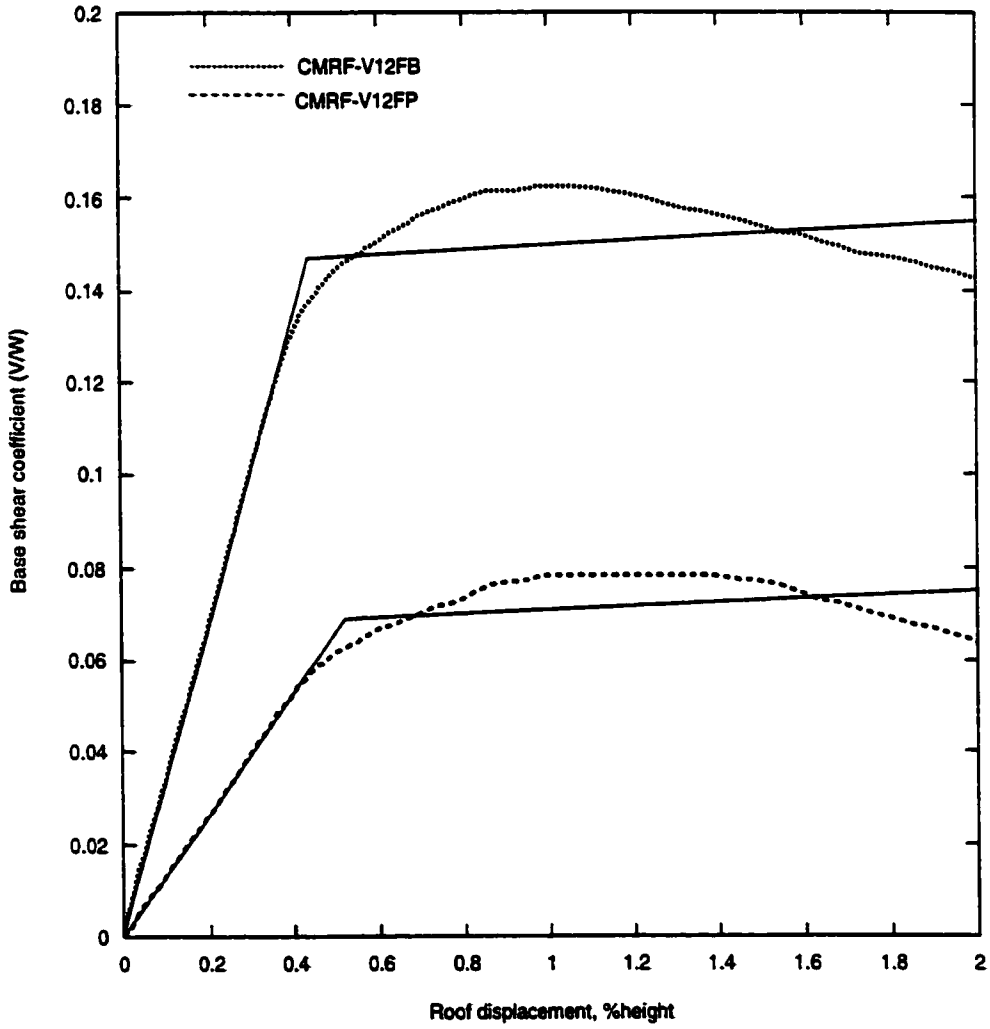


Figure 5.3: Bilinear idealization of the push-over curve: twelve-storey CMRF buildings in Victoria

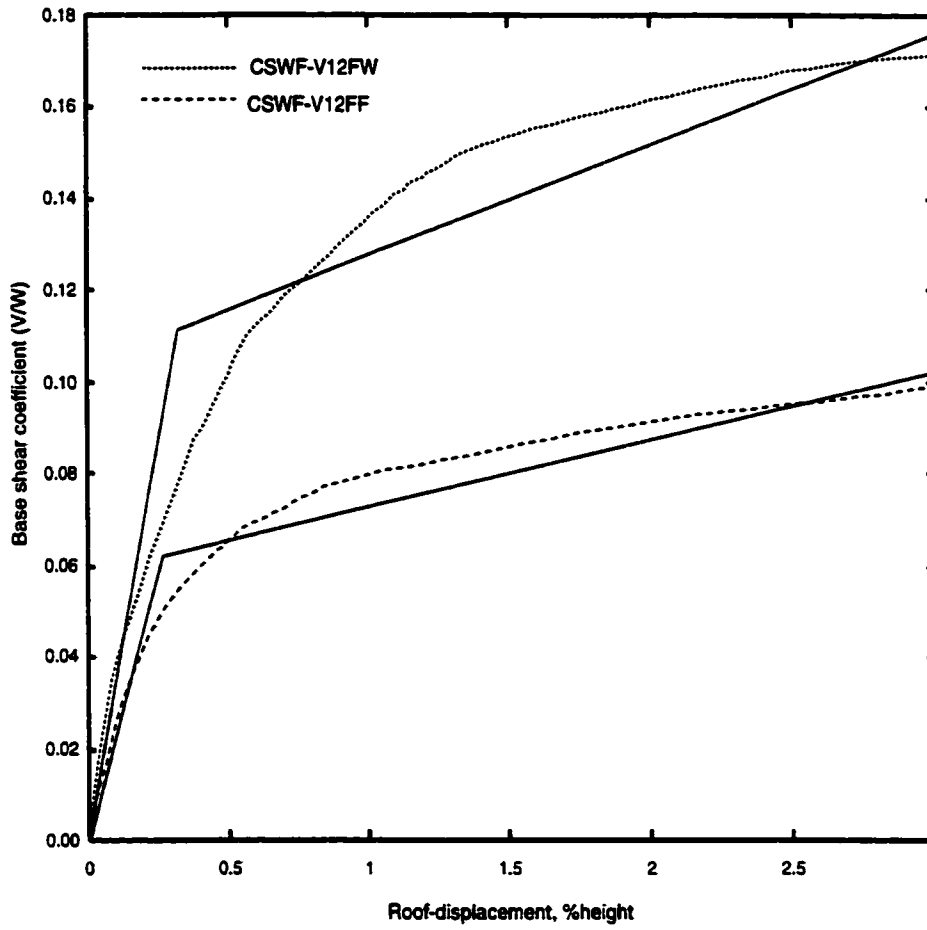


Figure 5.4: Bilinear idealization of the push-over curve: twelve-storey CSWF buildings in Victoria

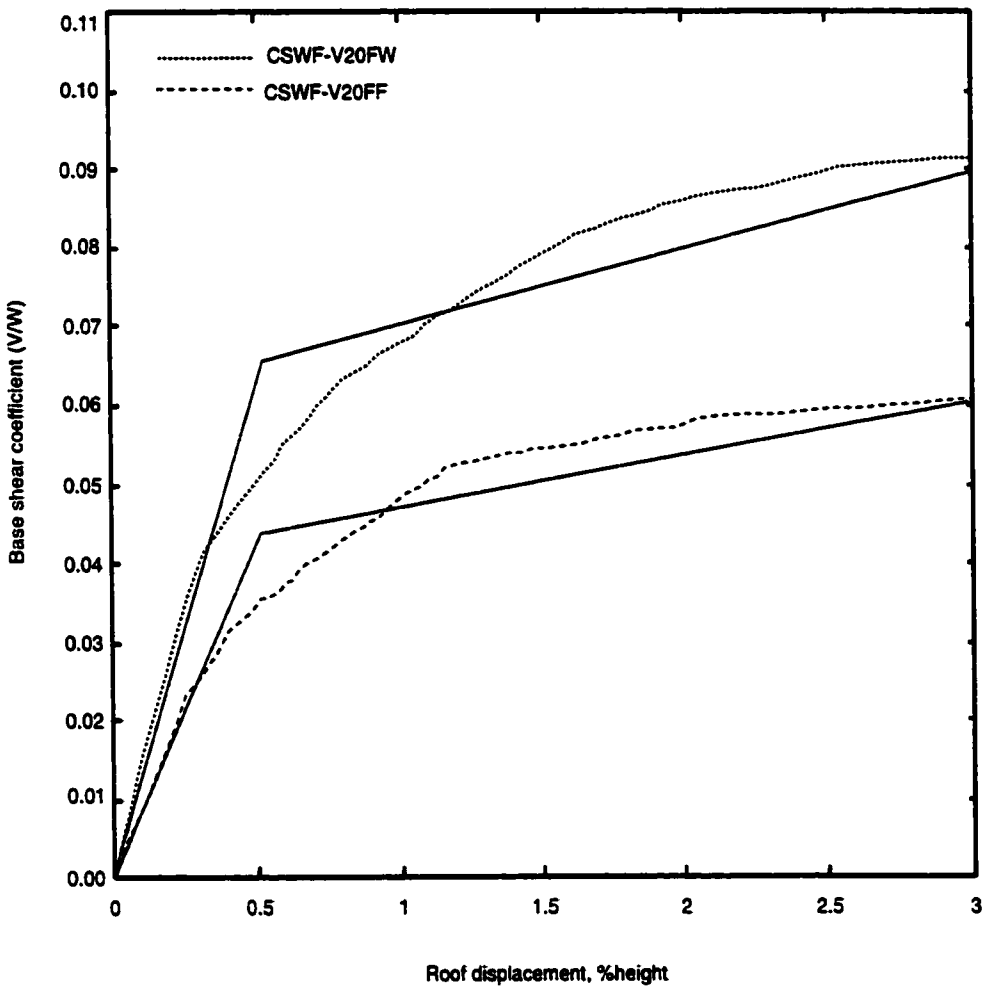
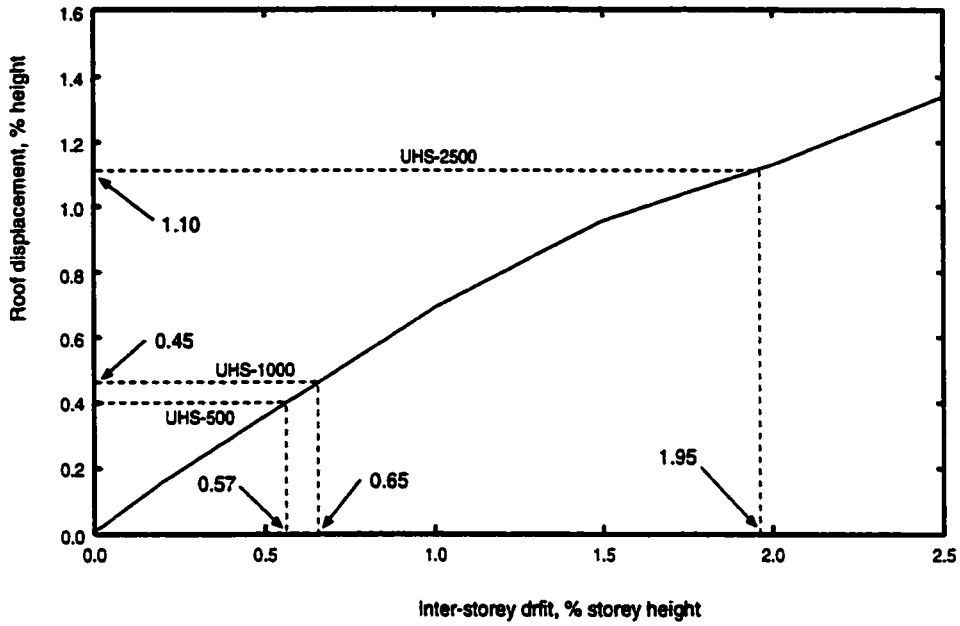
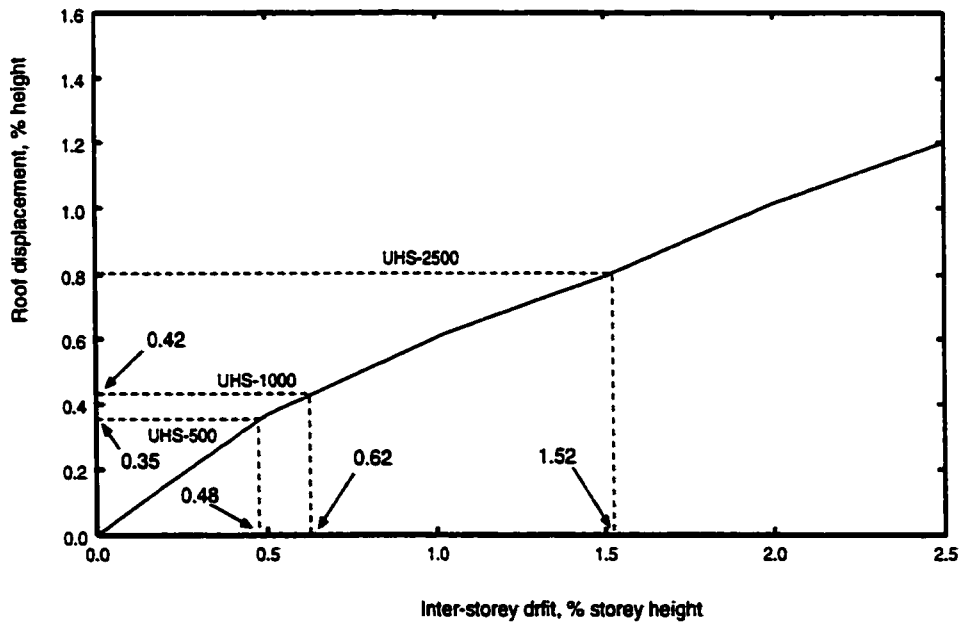


Figure 5.5: Bilinear idealization of the push-over curve: twenty-storey CSWF buildings in Victoria

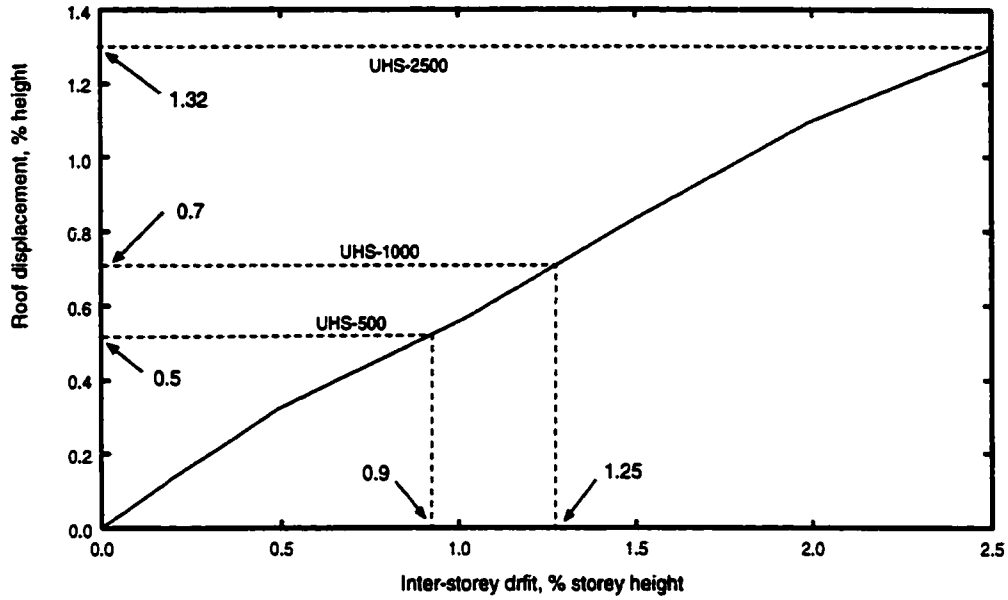


(a)

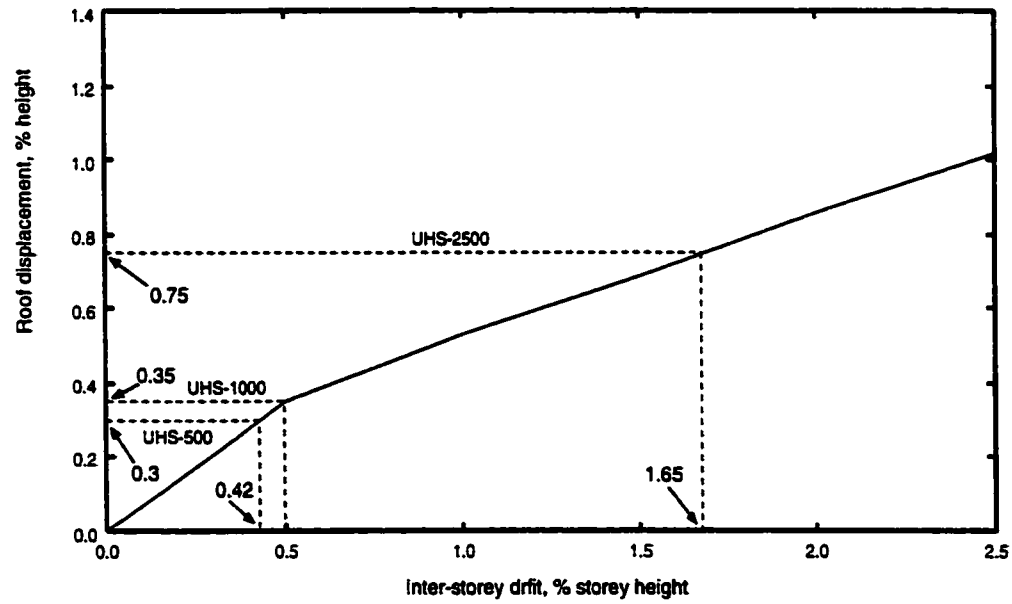


(b)

Figure 5.6: Inter-storey drift vs. roof displacement for six-storey CMRF buildings in Victoria: (a) CMRF-V6FB model, (b) CMRF-V6FP model

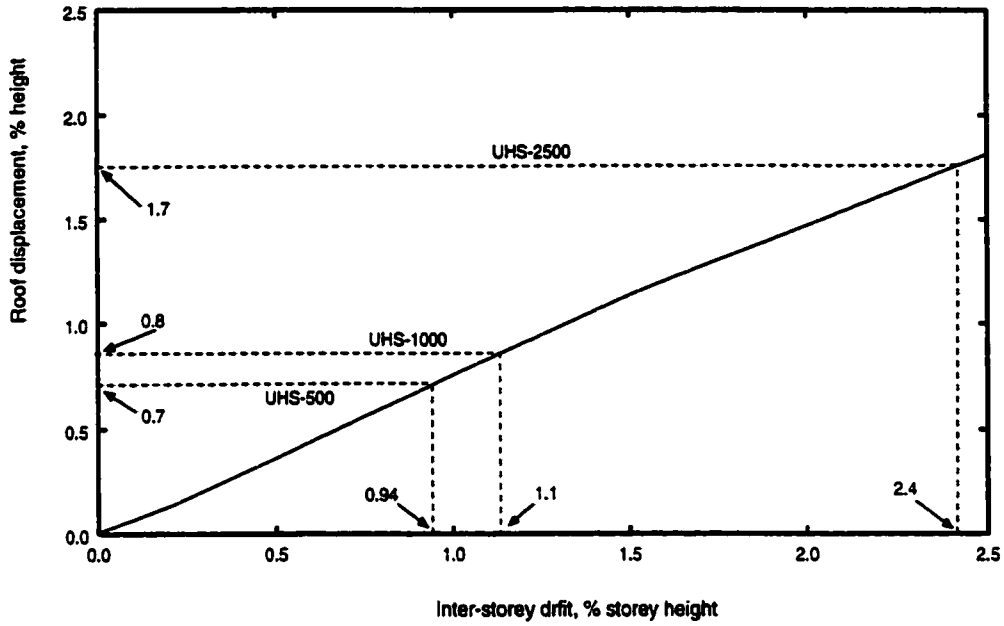


(a)

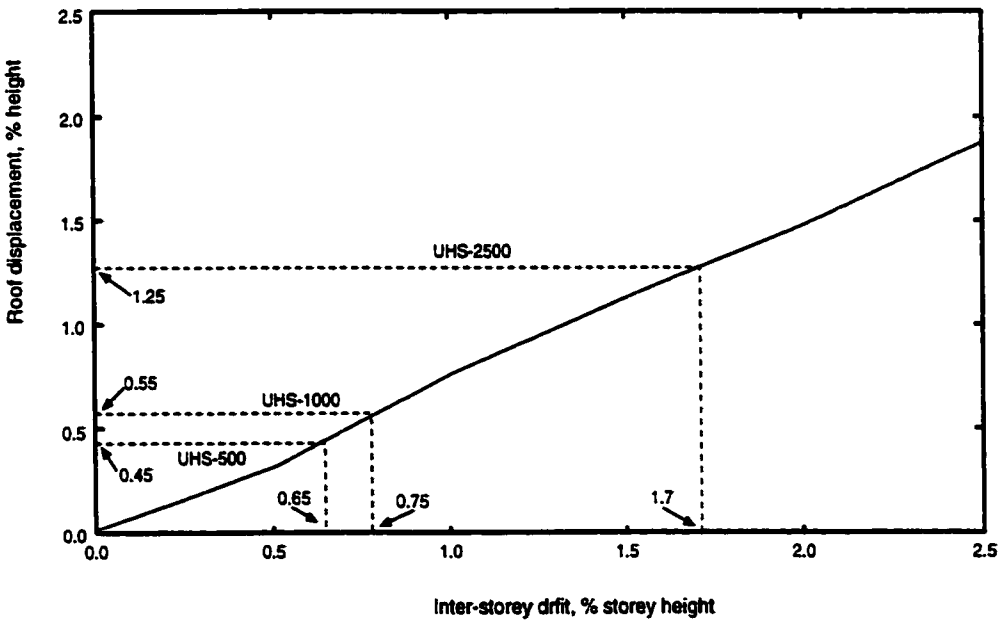


(b)

Figure 5.7: Inter-storey drift vs. roof displacement for twelve-storey CMRF buildings in Victoria: (a) CMRF-V12FB model, (b) CMRF-V12FP model

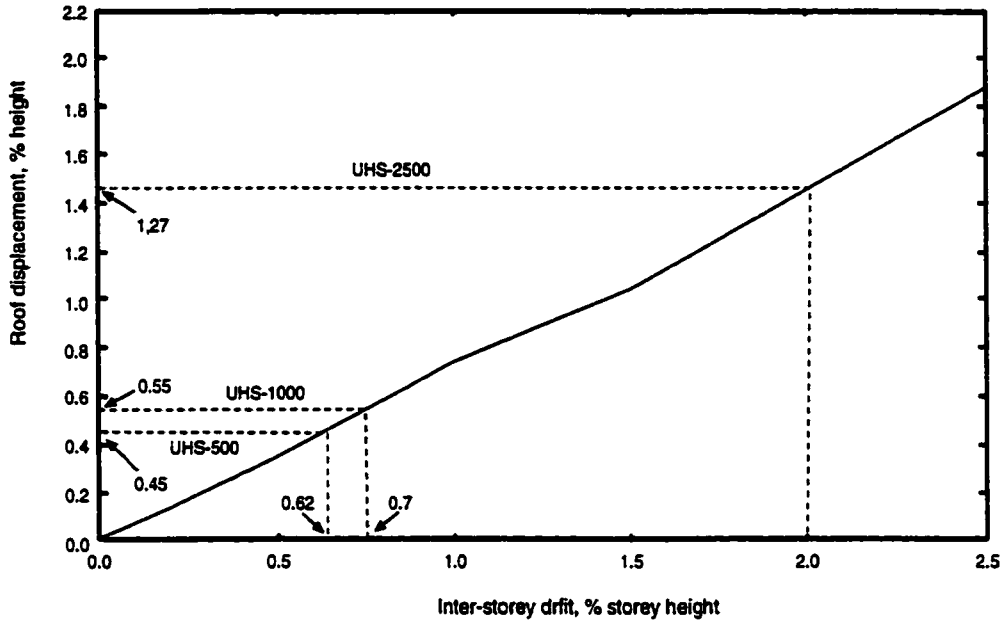


(a)

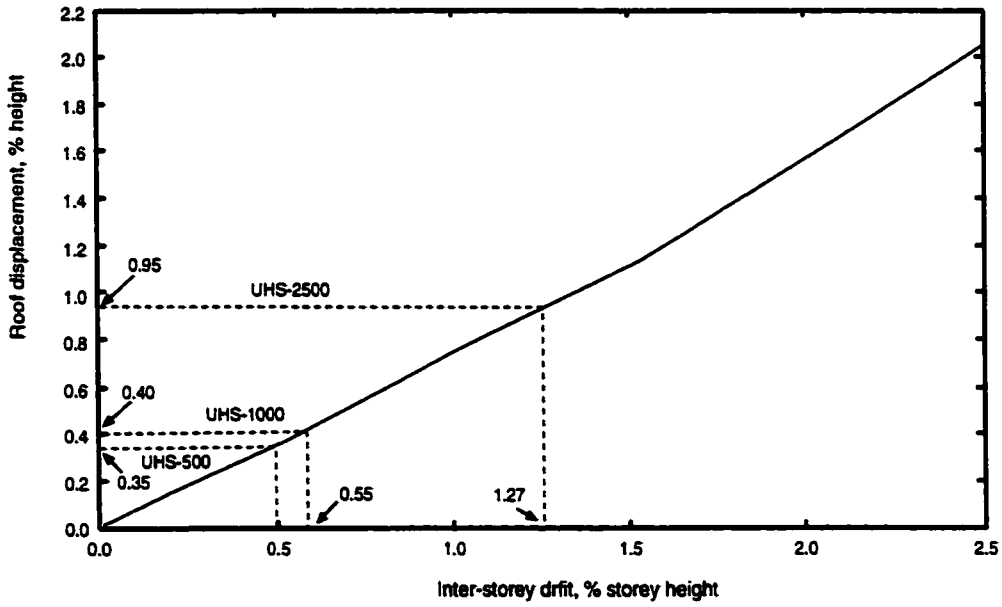


(b)

Figure 5.8: Inter-storey drift vs. roof displacement for twelve-storey CSWF buildings in Victoria: (a) CSWF-V12FB model, (b) CSWF-V12FP model



(a)



(b)

Figure 5.9: Inter-storey drift vs. roof displacement for twenty-storey CSWF buildings in Victoria: (a) CSWF-V20FB model, (b) CSWF-V20FP model

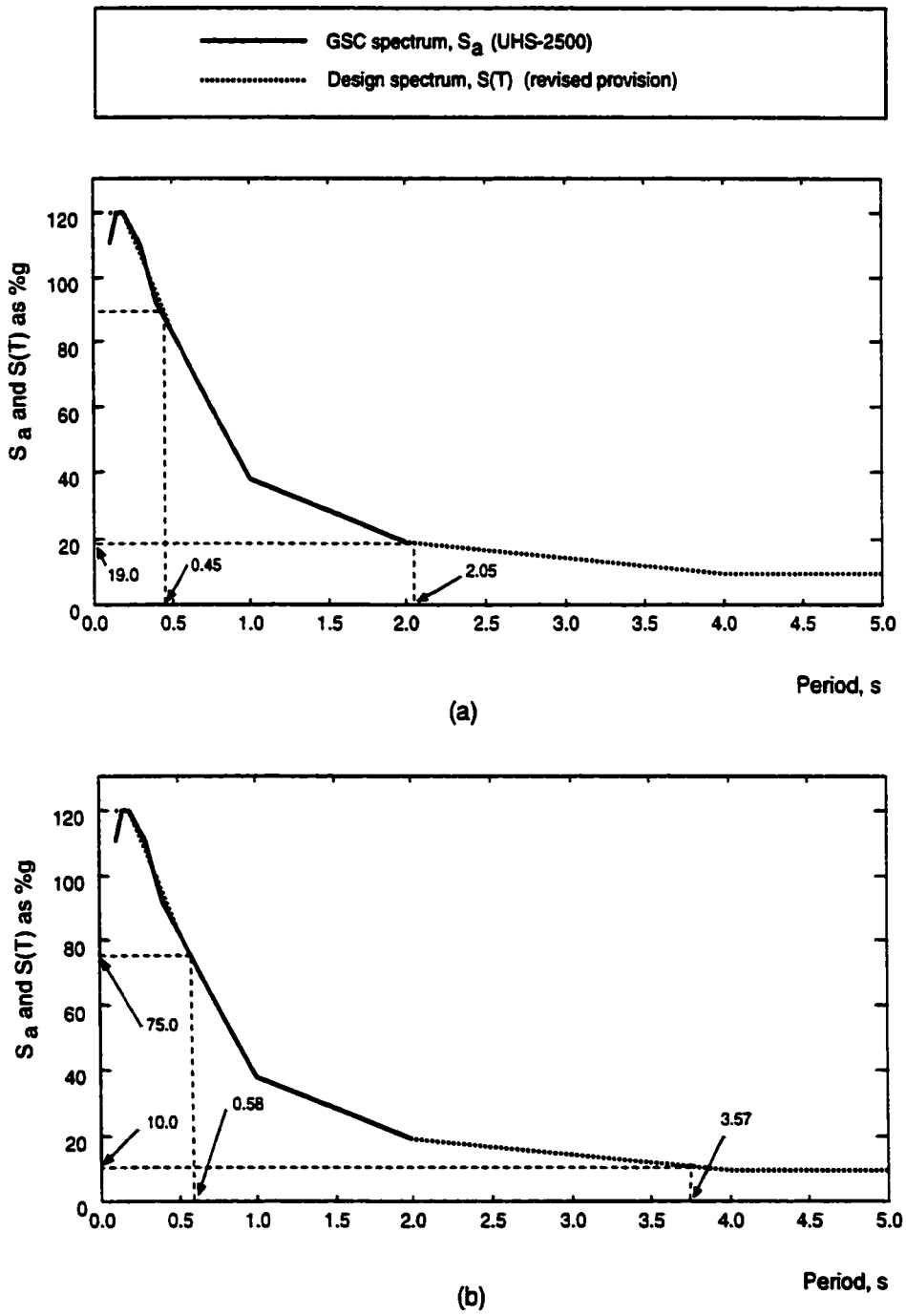


Figure 5.10: Spectral values corresponding to first two modes: (a) CSWF-V12FW model, (b) CSWF-V20FW model

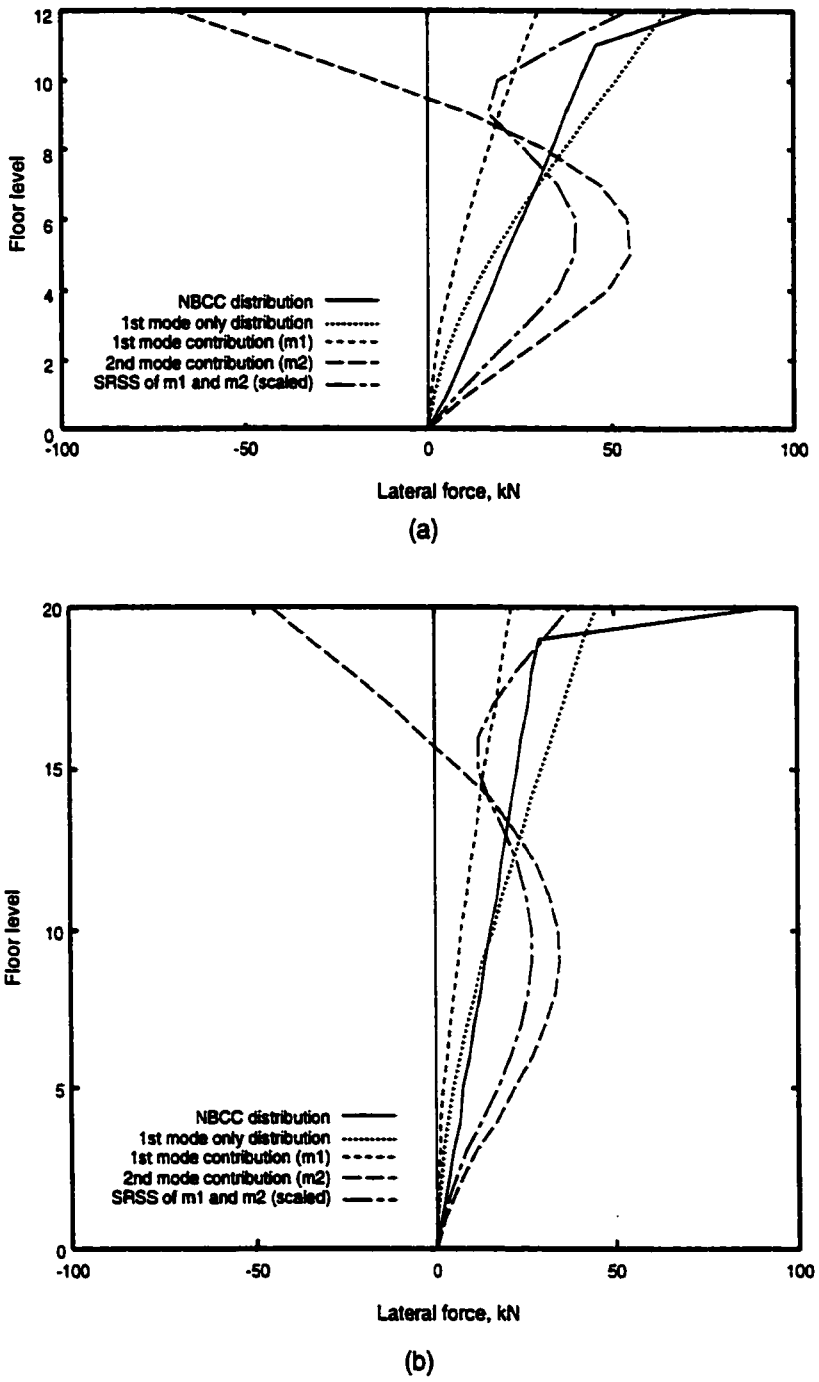
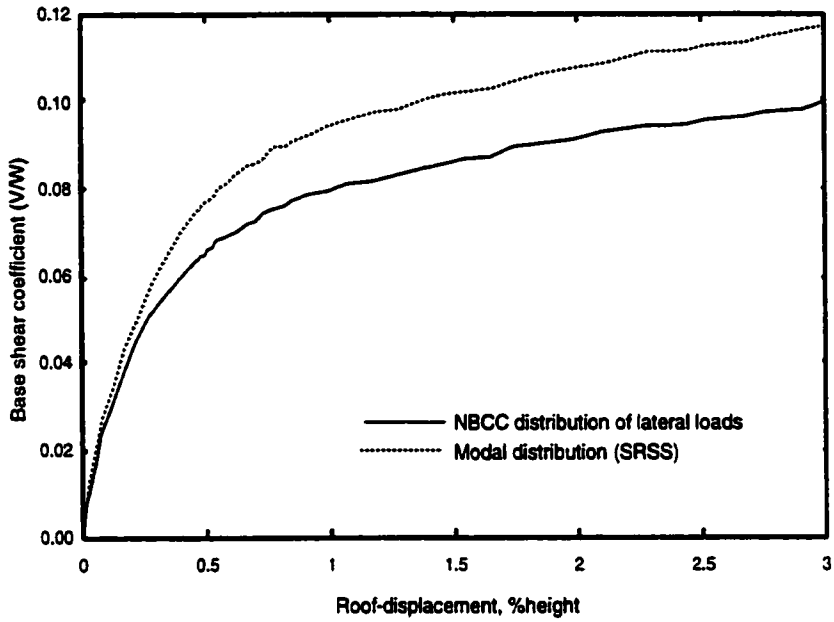
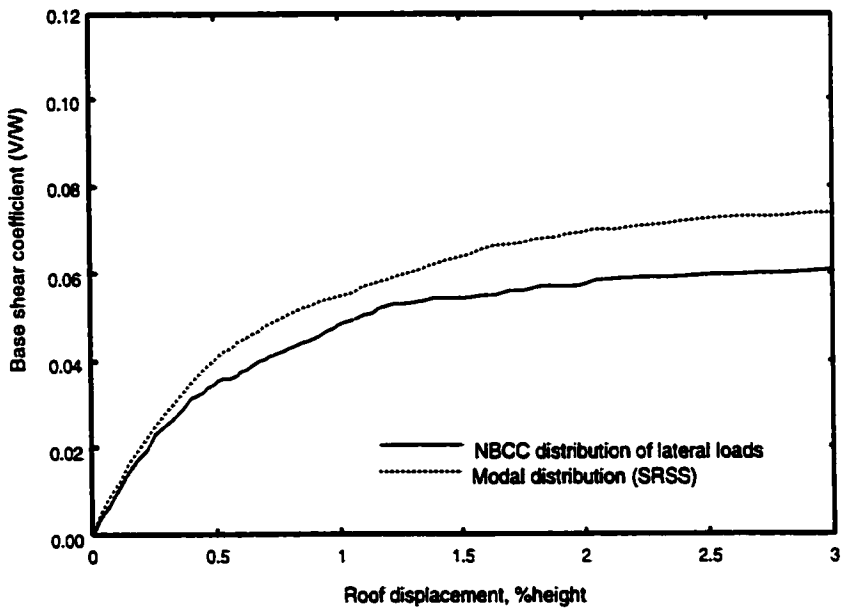


Figure 5.11: Distribution of lateral forces: (a) CSWF-V12FW model, (b) CSWF-V20FW model

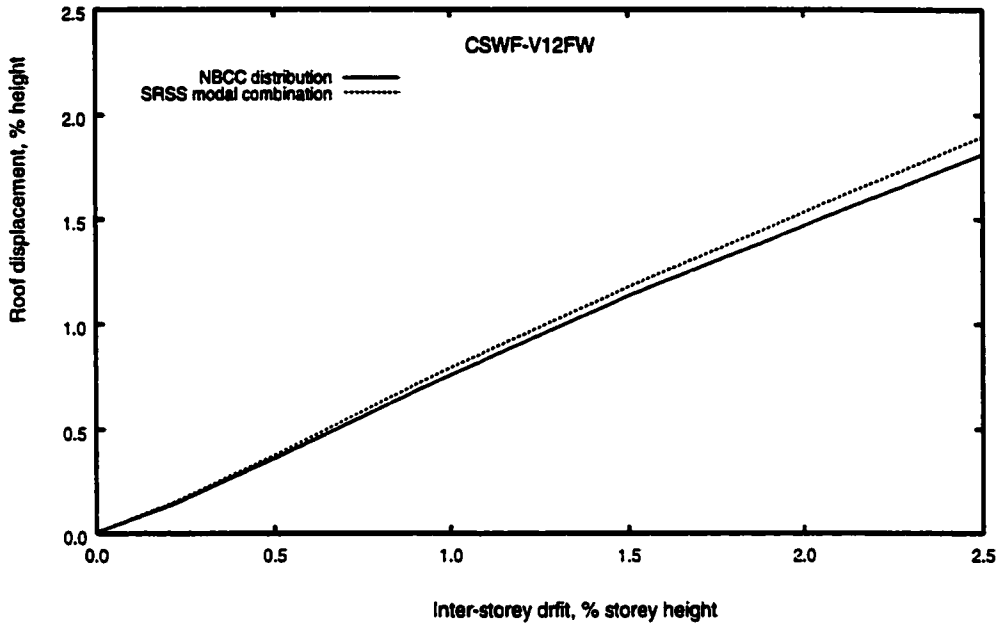


(a)

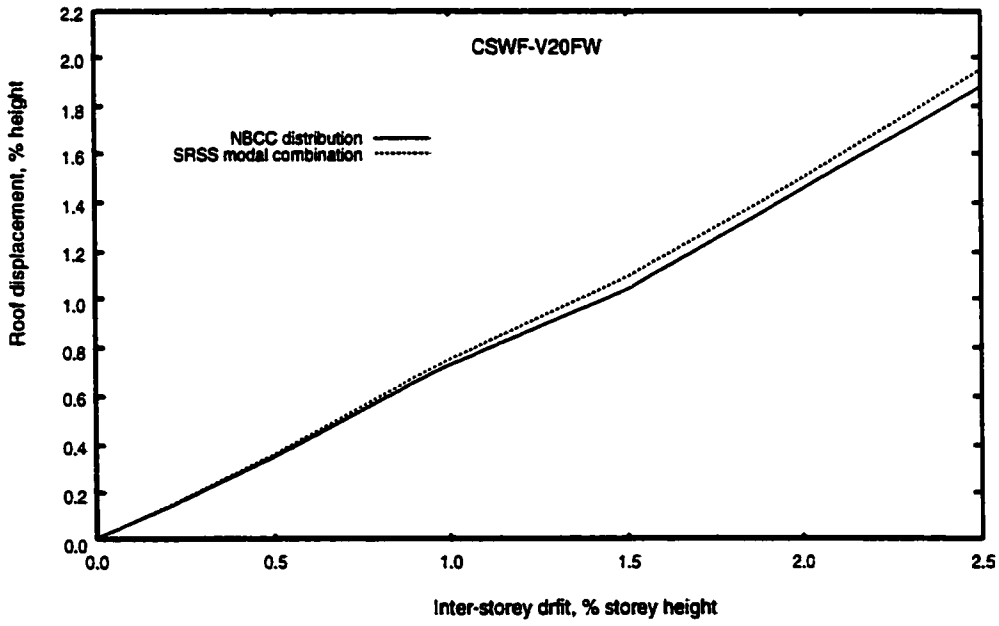


(b)

Figure 5.12: Push-over curves: (a) CSWF-V12FW model, (b) CSWF-V20FW model

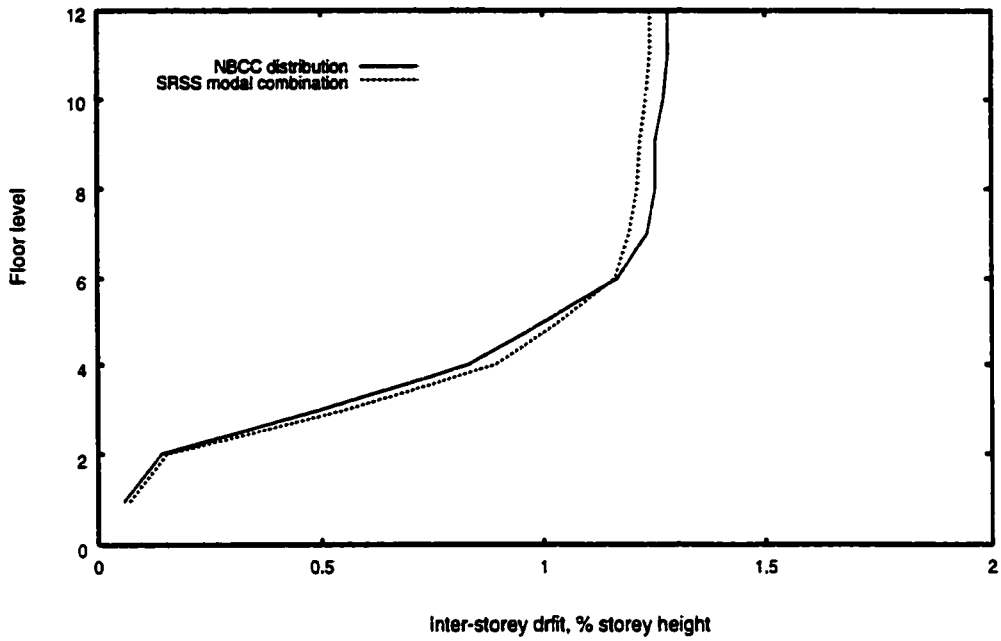


(a)

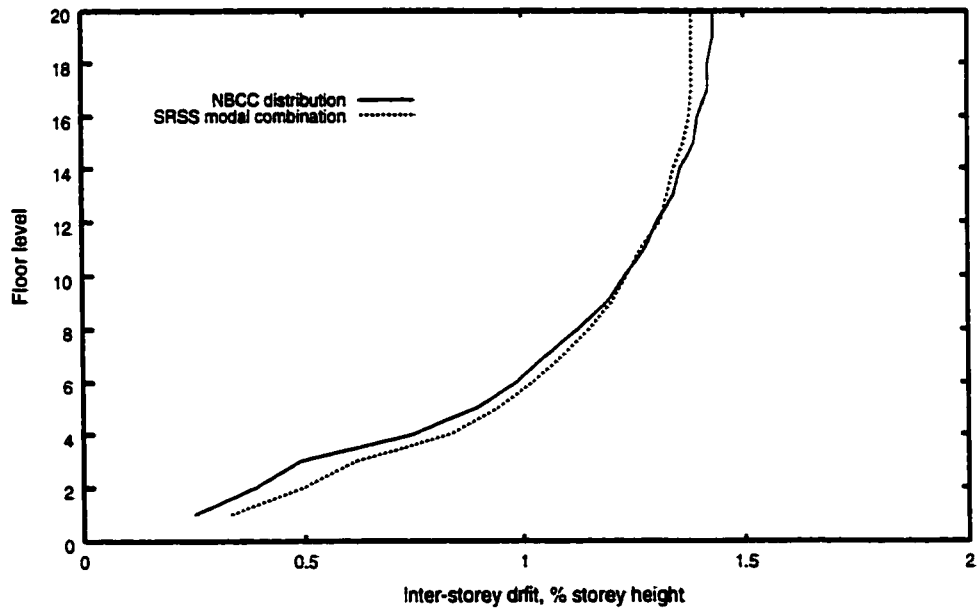


(b)

Figure 5.13: Roof-displacement vs. inter-storey drift: (a) CSWF-V12FW model, (b) CSWF-V20FW model



(a)



(b)

Figure 5.14: Inter-storey drift at 1% roof-displacement: (a) CSWF-V12FW model, (b) CSWF-V20FW model

Chapter 6

Summary and Conclusion

6.1 Summary

Lately, seismic performance of earthquake resistant buildings is getting a lot of attention and the major building code authorities around the world are stressing the need for performance based seismic design of buildings. Various performance objectives, building categories based on performance expectation, and general seismic design principles are briefly reviewed in the thesis.

Seismic design provisions of National Building Code of Canada (NBCC 1995) are briefly discussed. The Canadian National Committee on Earthquake Engineering (CANCEE) is presently working on the seismic design provisions for the future version of NBCC. In its preliminary report (1996) CANCEE has suggested a set of guidelines for calculating the design base shear based on site related uniform hazard spectrum corresponding to earthquake events with a return period of 475-years. The guidelines have subsequently been revised to use UHS values corresponding to a 2500-year earthquake (Humar 2000). The buildings considered in the present study

are designed based on the revised guidelines for seismic design proposed for the next version of NBCC.

Various methodologies for evaluation of seismic performance of buildings are discussed and key damage parameters are identified. Inter-storey drift, roof-displacement, element ductility demand, and various levels of Park-Ang damage indices are used as key damage parameters in this study.

Several computer programs that can be used for seismic performance of buildings are studied. The following programs are chosen to be used in the current work: DRAIN-2DX, DRAIN-RC and COLA. DRAIN-2DX is a program for the dynamic analysis of framed structures, while DRAIN-RC is a modified version of DRAIN-2DX is a that has the capability of modelling reinforced concrete beam-column and infill panel elements. COLA is is a column analysis program that is capable of carrying out a sectional analysis of reinforced concrete beam-column section and producing moment-curvature relationship, moment-axial force interaction curve etc.

A post-processing program is developed for analyzing the output of DRAIN-RC program. This program analyzes the dynamic analysis output data produced by DRAIN-RC for a suite of ground motion time histories and calculates inter-storey drifts, Park-Ang damage indices, as well as the envelope values of all the damage parameters considered here.

Seismic performance of concrete moment resisting frame (CMRF) and concrete shear wall frame (CSWF) buildings are studied in the thesis. The buildings are assumed to be located in Victoria and Montreal to represent western and eastern Canada, respectively.

For each location, a set of buildings consisting of a six-storey and a twelve-storey CMRF buildings, and a twelve-storey and a twenty-storey CSWF buildings are considered. The buildings are designed on the basis of revised guidelines of CANCEE (2000).

For each CMRF building, the following four models are considered in the study: (a) bare frame model with factored level of material strength, (b) bare frame model with nominal level of material strength, (c) infilled frame model with factored level of material strength, and (d) infilled frame model with nominal level of material strength.

For each CSWF building, the following four models are considered: (a) wall model with factored level of material strength, (b) wall model with nominal level of material strength, (c) wall-frame model with factored level of material strength, and (d) wall-frame model with nominal level of material strength.

Modal analysis is performed for each model to determine its fundamental period. The arrangement of infill panels in an infilled frame model is chosen such that the fundamental period of the structure is close to the value obtained using NBCC expression for period. Infill panels are modelled as strut elements. Push-over analysis is performed for each building model to determine its lateral load resisting and ductility capacities.

Inelastic dynamic analyses are carried out for each building model for UHS-compatible ground motion records corresponding to the following three levels of seismic hazard: (a) UHS-500 (475 years), (b) UHS-1000 (970 years), and (c) UHS-2500 (2500 years).

UHS-compatible ground motion time history records developed by Atkinson *et al* (1998) are used in study. A suite of sixteen such ground motion records is used, four records each corresponding to UHS-500 and UHS-100, and eight records corresponding to UHS-2500. Each building model is analyzed for all sixteen records. A total of 512 inelastic dynamic analyses are performed for the eight buildings (i.e. 32 frame models).

Based on the results of inelastic dynamic analyses of a building model, envelopes of damage parameters are calculated. Using the envelopes of damage parameters, qualitative levels of seismic performance, as defined by Vision 2000, are evaluated.

A simplified method for the evaluation of seismic performance of a building is suggested. This method is based on push-over analyses of the buildings studied here. Other simplified methods available in the literature are briefly described.

6.2 Conclusions

Performance based seismic design concept is relatively new and studies on seismic performance of buildings designed according to NBCC design provisions are very few. NBCC design provisions are undergoing major revisions and the buildings studied here are designed using the provisions suggested for the next version of NBCC (Humar 2000). This would be one of the first few studies on performance evaluation of buildings designed on the basis of the new UHS-based seismic design provisions that are being considered for adoption in the future version of NBCC.

Traditionally for analysis of building performance scaled ground motion records

of actual past earthquakes are used. But those records may not be representative of the seismicity of a given location for which a building is designed. Atkinson *et al* have developed synthesized records, called UHS-compatible ground motion records that more closely represent the seismicity of a given location. Atkinson's records corresponding to various levels of seismic hazard are used in this study.

Effect of infill panels in frame structures have been studied by various researchers. In most studies the distribution and quantity of infill panels are chosen arbitrarily. In the present study, the distribution and quantity of infill panels are chosen such that the fundamental period of the structure with infills is close to the value obtained from the empirical expressions given in NBCC 1995.

Based on the study presented here, the following conclusions are drawn.

1. Push-over analyses for CMRF buildings suggest that there is considerable over-strength in bare frame models.

2. Infilled CMRF frame models have significantly higher lateral load-resisting capacity than the corresponding bare frame models.

3. Pattern of hinge formation observed in push-over analyses of CMRF bare frame models suggest a high degree of ductility exhibiting the effect of capacity design. However, the the infill frame models do not show such predictable hinging patterns.

4. The six-storey CMRF building in Victoria achieves a performance level of *fully operational* under UHS-500, *operational* under UHS-1000, and *near collapse*

under UHS-2500, when bare frame models are considered. The level of performance under UHS-2500 improves when infill panels are considered in the analysis; in this case, the building achieves *life-safe* level of performance under UHS-2500.

5. The twelve-storey CMRF building in Victoria achieves a performance level of *operational* under both UHS-500 and UHS-1000, and *near collapse* under UHS-2500, when bare frame models are considered. The levels of performance improves when infill panels are considered in the analysis. The building with infills achieves a performance level of *fully operational* under both UHS-500 and UHS0-1000, and *life-safe* under UHS-2500.

6. CMRF Buildings in Montreal are not much vulnerable to the seismic hazard corresponding to that location. The buildings achieve a performance level of *operational* or better under the highest level of seismic hazard (UHS-2500).

7. Push-over analyses of CSWF buildings suggest that the wall and wall-frame models behave in a ductile manner.

8. The twelve-storey CSWF building in Victoria achieves a performance level of *operational* under both UHS-500 and UHS-1000, and *near collapse* under UHS-2500 when wall models are considered. When wall-frame models are considered the building achieve the following levels of performance: *operational* under both UHS-500 and UHS-1000, and *life-safe* under UHS-2500.

9. The twenty-storey CSWF building in Victoria shows similar performance behaviour as the twelve-storey building. It achieves a performance level of *oper-*

ational under both UHS-500 and UHS-1000, and *life-safe* under UHS-2500 for both wall and wall-frame models.

10. CSWF buildings in Montreal show *operational* or better performance under the highest level of seismic hazard (UHS-2500) corresponding to that location.

11. Seismic characteristics in eastern Canada is such that a lateral load-resisting building frame in this region has higher tributary mass than a corresponding frame in western Canada. Because of this a building frame in eastern Canada has higher period than an identical frame in western Canada.

12. The fundamental period of a bare frame is generally longer than the period obtained from the empirical formula given by NBCC. A bare frame model for a building in eastern Canada has longer period as compared to a corresponding frame model in western Canada. An infilled frame model in eastern Canada thus require a larger number of infill panels compared to an identical frame in western Canada in order to bring down the period of the frame close to NBCC value. This produces unrealistically high stiffness in an infilled frame in eastern Canada.

13. As the actual period of a bare frame is longer than the NBCC period, the hazard spectral value corresponding to it is less than the design value (unless both the periods are short enough to fall in the plateau region on the spectrum). In this case, the structure tends to attract less amount of dynamic lateral forces than that is assumed in the design. For a building in eastern Canada this effect is more considerable as the difference between the design period and the actual period is higher than that for a corresponding building in western Canada.

14. Buildings in western Canada are found to be more vulnerable to the seismic hazard in that area than the buildings in eastern Canada. Buildings in eastern Canada suffer only minor damage under UHS-2500 event. This happens due to the fact that the bare frame model for a building in eastern Canada attracts considerably less amount of seismic forces because of the hazard spectral shape and longer period. The infilled frame model of a building in eastern Canada is unusually stiff and consequently, its dynamic response remains very low.

15. The building models considered here are designed and studied for the seismic loadings and the associated gravity loads. Wind loads have not been accounted for. In the cases where they govern the design, the structures would have a higher strength and the levels of seismic performance achieved by the structures would expected to be better.

16. For the buildings in eastern Canada, the transeverse frames that are not designed to be ductile lateral load-resisting frames are not included in the analyses. However, in reality these frames might contribute to the overall lateral load-resisting capacity of these buildings. Modelling the inelastic behaviour of the structure including such frames could be quite complex as the non-ductile frames would participate in the lateral load-resisting system until they yield, while the ductile frames would continue to resist lateral loads beyond yielding. Considering that the ductile frames would improve the lateral load resisting capacity, a building in eastern Canada would probably achieve better levels of seismic performance than that reported here. This would further highlight the conclusion that the buildings in eastern Canada have a considerably higher

level of seismic protection for a given level of seismic hazard.

17. The simplified method suggested here can be used to obtain an approximate idea of the seismic performance of a building. It may be useful in the performance based design of a building when a number of design iterations must be performed. At the final stage of design a detailed analysis of performance should be carried out. Simplified methods have been criticized for the level of approximation implied in them. However, they may be useful in the design process as the designer can not afford to perform detailed analysis of seismic performance of a building at every iteration. Detailed analysis is very time consuming and computationally intensive.

6.3 Suggestion for future work

- The study presented here should be extended to include a variety of other buildings with various configuration and heights, and different building materials such as steel and composite construction.
- More studies need to be carried out enhance the proposed simplified method for performance evaluation of buildings.
- Use of inelastic response spectra would enhance the efficiency of the simplified method. Such response spectra corresponding to various levels of seismic hazard could be developed.

References:

1. Adams J., Weichert D., Halchuk S. and Basham P., 1996, "Trial seismic hazard maps of Canada - 1995: Final values for selected Canadian cities", Geological Survey of Canada Open File 3283, pp 97.

2. Adams J., 1997, Private communication.

3. Alsiwat J., 1993, "Effect of Anchorage Slip and Inelastic Shear on Seismic Response of Reinforced Concrete Frames", Ph.D. Thesis, Department of Civil Engineering, University of Ottawa, Ottawa, Canada.

4. Anderson J.C., Miranda E., Bertero V.V. and Kajima project research team, 1991, "Evaluation of the seismic performance of a thirty-story RC building", Report No. UCB/EERC-91/16, College of Engineering, University of California, Berkeley.

5. Anderson J.C. and Bertero V.V., 1991, "Seismic performance of an instrumented six-story steel building", Report No. UCB/EERC-91/11, College of Engineering, University of California, Berkeley.

6. Atkinson G.M and Beresnev I.A., 1998, "Compatible ground motion time-histories for new National hazard maps", *Canadian Journal of Civil Engineering*, 25, 305-318.

7. Atkinson G.M., 1997, Private communication.

8. Bagchi A., June 1999, "Seismic performance of buildings designed to National

Building Code of Canada", *8th Canadian Conference on Earthquake Engineering*, Vancouver, B.C., Canada.

9. Balendra T., Tan K., and Kong S., 1999, "Vulnerability of reinforced concrete frames in low seismic region, when designed according to BS 8110", *Earthquake Engineering and Structural Dynamics*, Vol 28, 1361-1381.

10. Bertero V.V., 1997, "Performance-based seismic engineering: A critical review of proposed guidelines", *Seismic Design Methodologies for the Next Generation of Codes*, edited by Fajfar and Krawinkler, p 1.

11. Bertero V.V., Anderson J.C. and Krawinkler H., 1994, "Performance of steel building structures during the Northridge earthquake", Report No. UCB/EERC-94/09, College of Engineering, University of California, Berkeley.

12. Bracci J.M, Kunnath K.S. and Reinhorn A.M, 1997, "Seismic performance and retrofit evaluation of reinforced concrete structures", *ASCE Journal of Structural Engineering*, Vol. 123, 1, 3-10.

13. Building Seismic Safety Council (BSSC), 1997, "Recommended Provisions for Seismic Regulations for New Buildings and Commentary", National Earthquake Hazards Reduction Program (NEHRP), Federal Emergency Management Agency, Washington, D.C.

14. Canadian national Committee on Earthquake Engineering (CANCEE), 1996, "Preliminary Formulation of Elastic Seismic Forces using Spectral Ordinates", September.

15. Canadian Portland Cement Association (CPCA), 1995, "Concrete Design Handbook", Ottawa, Ontario.

16. Canadian Standard Association (CSA), 1994, "Design of Concrete Structures", Standard A23.3-94, Rexdale (Toronto), Ontario.

17. Chaker A.A., and Cherifati A., 1999, "Influence of masonry infill panels on the vibration and stiffness characteristics of R/C frame buildings", *Earthquake Engineering and Structural Dynamics*, Vol 28, 1061-1065.

18. Drysdale R.G. and Ahmed A.H., 1994, "Masonry Structures - Behaviour and Design", Prentice Hall.

19. Fajfar P., Vidic T. and Fischinger M., 1992, "On energy demand and supply in SDOF systems", *Nonlinear seismic analysis and design of reinforced concrete buildings*, pp 41-62.

20. Fajfar P. and Gaspersic P., 1996, "The N2 method for the seismic damage analysis of RC buildings", *Earthquake Engineering and Structural Dynamics*, Vol 25, 31-46.

21. Fajfar P. and Gaspersic P., and Drobnic D., 1997, "A simplified nonlinear method for seismic damage analysis of structures", *Seismic Design Methodologies for the Next Generation of Codes*, edited by Fajfar and Krawinkler, pp 183-193.

22. Filiatrault A., Lachapelle E. and Lamontagne P., 1998a, "Seismic Performance of Ductile and Nominally Ductile Reinforced Concrete Moment Resisting Frames. I. Experimental study", *Canadian Journal of Civil Engineering*, 25, 331-341.

23. Filiatrault A., Lachapelle E. and Lamontagne P., 1998b, "Seismic Performance of Ductile and Nominally Ductile Reinforced Concrete Moment Resisting Frames. I. Analytical study", *Canadian Journal of Civil Engineering*, 25, 342-352.

24. Ghobarah A., Aly N.M., and El-Attar M., 1997, "Performance level criteria evaluation", *Seismic Design Methodologies for the Next Generation of Codes*, edited by Fajfar and Krawinkler, pp 207-215.

25. Ghobarah A., Abou-Elfath H., and Biddah A., 1999, "Response-based Damage Assessment of Structures", *Earthquake Engineering and Structural Dynamics*, 28, 79-104.

26. Hamburger R.O., 1997, "Defining performance objectives", *Seismic Design Methodologies for the Next Generation of Codes*, edited by Fajfar and Krawinkler, pp 33-42.

27. Heidebrecht A.C., 1997, "Seismic level of protection for building structures", *Canadian Journal of Civil Engineering*, Vol 24, 20-33.

28. Heidebrecht A.C. and Naumoski N., 1998, "Seismic Performance of Ductile Medium Height Reinforced Concrete Moment Resisting Frame Structures", *Sixth U.S. National Conference on Earthquake Engineering*.

29. Humar J.L., May 2000, "Future directions of seismic design provisions", *Short Course on Earthquake Resistant Design of Buildings*, Carleton University, Ottawa, Canada.

30. Humar J.L. and Bagchi A., June 2000, "Seismic level of protection in shear wall frame structures degined according to National Building Code of Canada", *28th*

Annual Conference of Canadian Society of Civil Engineering, London, Ontario, Canada.

31. Humar J.L. and Rahgozar M.A., 2000, "Application of uniform hazard spectra in seismic design of multistorey buildings", *Canadian Journal of Civil Engineering*, 27, 563-580.

32. Humar J.L. and Mahgoub M.A., 2000, "Accounting for higher modes in UHS-based design", Discussion paper, Canadian National Committee on Earthquake Engineering (CANCEE).

33. International Conference of Building Officials (ICBO), 1997, "Uniform Building Code (UBC)", Whitter, California, USA.

34. Kannan, A.E. and Powell, G.H., 1983, "DRAIN-2D: a general purpose computer program for dynamic anaysis of inelastic place structures", Report N. UCB/EERC 73-6, Earthquake Engineering Research Center, University of California, Berkeley.

35. Kongoli X., Minami T., and Sakai Y., 1999, "Effects of structural walls on the elastic-plastic earthquake responses of frame-wall buildings", *Earthquake Engineering and Structural Dynamics*, 28, 479-500.

36. Kunnath S.K., Reinhorn A.M. and Lobo R.F., 1992, "IDARC Version 3.0: A Program for Inelastic Damage Analysis of Reinforced Concrete Structures", Report No. NCEER-92-0022, National Center for Earthquake Engineering Research, State University of New York at Buffalo.

37. Linde P., 1998, "Evaluation of structural walls designed according to Eurocode 8 and SIA 160", *Earthquake Engineering and Structural Dynamics*, 27, 793-809.

38. Mazzolani F.M. and Piluso V., 1997, "A simple approach for evaluating performance levels of moment-resisting steel frames", *Seismic Design Methodologies for the Next Generation of Codes*, edited by Fajfar and Krawinkler, pp 241-251.

39. Miranda E. and Bertero V.V., 1991, "Evaluation of the seismic performance of a ten-story RC building during the Whittier Narrows earthquake", Report No. UCB/EERC-91/16, College of Engineering, University of California, Berkeley.

40. Mitchel D. and Paultre P., 1994, "Ductility and overstrength in seismic design of reinforced concrete structures", *Canadian Journal of Civil Engineering*, Vol 21, 1049-1060.

41. National Research Council of Canada (NRC), 1995,"National Building Code of Canada (NBCC)", Ottawa, Ontario.

42. Ozcebe G. and Saatcioglu M., 1989, "Hysteretic shear model for reinforced concrete members", *ASCE Journal of the Structural Engineering*, Vol 15, No. 1, January, pp 132-148.

43. Panagiotakos T.B. and Fardis M.N., 1999, "Estimation of inelastic deformation demands in multistorey RC frame buildings", *Earthquake Engineering and Structural Dynamics*, 28, 501-528.

44. Park Y.J and Ang A.H.S., 1985a, "Mechanistic seismic damage model for reinforced concrete", *ASCE Journal of Structural Engineering*, Vol 111, 722-739.

45. Park Y.J, Ang A.H.S. and Wen Y.K., 1985b, "Seismic Damage Analysis of Reinforced Concrete Buildings", *ASCE Journal of Structural Engineering*, Vol 111, 740-757.

46. Pauley T. and Priestley M.J.N., 1992, "Seismic design of reinforced concrete and masonry buildings", John Wiley and Sons, New York.

47. Paultre P. and Mitchel D., 1987, "Evaluation of seismic performance on concrete framestructures in Canada", Structural Engineering Series Report No. 87-4, McGill University, Montreal.

48. Prakash V., Powell G.H and Campbell S., 1993, "DRAIN-2DX Base Program Description and User Guide: Version 1.10", Report No. UCB/SEMM-93/17, Department of Civil Engineering, University of California, Berkeley.

49. Saatcioglu M. and Ozcebe G., 1987, "Effect of bar slip on hysteretic behaviour of concrete columns", *Proc. 5th Canadian conference on earthquake engineering*, Ottawa, Ontario, pp 833-839.

50. Shoostari A., 1998, "Seismic Drift Demand of Reinforced Concrete Buildings", Ph.D. Thesis, Department of Civil Engineering, University of Ottawa, Ottawa, Canada.

51. Stafford-Smith B., 1966, "Behaviour of Square Infilled Frames", *ASCE Journal of the Structural Engineering Division*, Vol 91, No. ST1, 381-403.

52. Stonehouse B., Heidebrecht A.C., and Kianoush M.R., 1999, "Evaluation of the level of seismic protection afforded to reinforced concrete shear wall systems", *Canadian Journal of Civil Engineering*, 26, 572-589.

53. Teran-Gilmore A. and Bertero V.V., 1992, "Performance of Tall buildings during the 1985 Mexico earthquakes", Report No. UCB/EERC-92/17, College of Engineering, University of California, Berkeley.

54. Teran-Gilmore A. and Bertero V.V., 1993, "Seismic performance of a 30-storey building located on soft soil and designed according to UBC 1991", Report No. UCB/EERC-93/04, College of Engineering, University of California, Berkeley.

55. Valles R.E., Reinhorn A.M., Kunnath S.K. and Madan A., 1996, "IDARC2D Version 4.0: A Computer Program for the Inelastic Damage Analysis of Buildings", Technical Report NCEER-96-0010, National Center for Earthquake Engineering Research, State University of New York at Buffalo.

56. Vision 2000 Committee, 1995, "Performance Based Seismic Engineering of Buildings", Sructural Engineers Association of California (SEAOC), Sacramento, California.

57. Whittaker A., Constantinou M., and Tsopleas, P., 1998, *ASCE Journal of Structural Engineering*, August, pp 905-912.

58. Yalcin C., 1996, "COLA, a Program for Sectional Analysis of Reinforced Concrete Members", University of Ottawa, Canada.