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DUCTILITY AND STRENGTH DEMANDS IN BUILDING STRUCTURES SUBJECTED TO EARTHQUAKE FORCES

by

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Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

DOCTOR OF PHILOSOPHY

in

Engineering*

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**DUCTILITY AND STRENGTH DEMANDS
IN BUILDING STRUCTURES
SUBJECTED TO EARTHQUAKE FORCES**

submitted by Mohammad Ali Rahgozar
in partial fulfillment of the requirements of
Doctor of Philosophy in Engineering

External Examiner

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ABSTRACT

The objective of earthquake resistance design is to provide structural capacities that exceed the demands imposed by severe earthquakes with a sufficient margin of safety. Current code based seismic designs are based on the use of empirical coefficients that obscure the design process and may at times lead to inconsistent designs with unknown levels of protection. The need exists for the future codes to incorporate explicitly the capacity/demand concepts to make the design process transparent and permit designs with a well defined and consistent level of protection for the given limit states. This research is concerned with the assessment of seismic demand parameters that are needed to implement a capacity / demand based seismic design approach. The objectives of the study are (a) to assess the importance of different demand parameters, (b) to evaluate patterns in demand parameters that will improve our understanding of the physical phenomena involved in seismic response of structures, (c) to provide statistical information on demand parameters that can be utilized to assess the performance of structures designed according to existing codes, and (d) to implement the findings in this study into a proposed capacity / demand based base shear format for seismic design of building structures.

A comprehensive statistical evaluation of strength demands is performed for bilinear single-degree-of-freedom (SDOF) systems. This study is conducted for three ensembles of ground motions with different frequency content, each containing 15 records. Strength demands are represented in terms of constant-ductility spectra of inelastic strength demands or strength reduction factors. An empirical method is developed that relates the *inelastic* design response spectra for a given target ductility to the *elastic* Uniform Hazard Spectra (UHS) for a given site. The method uses two spectral values obtained from the hazard maps of Canada, the peak spectral acceleration for the site and the spectral acceleration corresponding to a period of 0.5 s.

Some possible sources of structural overstrength are outlined and it is reasoned

that a more rational basis for design would be to account for such sources in assessing the capacity rather than in reducing the design loads. As an exception, one possible source of reserve strength, the redistribution of internal forces, may be used in scaling down the design forces. This is because such scaling allows the determination of design forces through an elastic analysis rather than a limit analysis. The reserve strength attributable to redistribution is evaluated for steel ductile moment-resisting frames, steel ductile concentrically braced frames, concrete ductile moment-resisting frames, and concrete ductile wall structures. The evaluation is based on static nonlinear push-over analysis in which the gravity loads are held constant while the earthquake forces are gradually increased until a mechanism forms or the specified limit on inter-storey drift is exceeded.

A comprehensive statistical study is performed on the ductility and strength demands in multi-storey buildings, having three distinct types of structural systems: moment-resisting frames, concentrically braced frames, and flexural walls. The multi-degree-of-freedom (MDOF) models, used in this study, are subjected to UHS-compatible acceleration time histories corresponding to earthquakes with a return period of 2500 years developed for selected cities in western as well as eastern Canada. The objective of this part of the study is to determine the modification that must be applied to demand parameters derived from SDOF models in order to account for multi-mode effects on the demand parameters in the real structures.

A new base shear format is proposed for seismic design of building structures, on the basis of the findings in this study. This formulation for design base shear is intended to account for the capacity/demand concepts. The basic information as well as a step-by-step implementation of the proposed approach for calculation of design base shear is also provided.

Dedicated to:

my teachers, with gratitude,

my parents, with affection,

my wife, and my daughter Sameen, with love

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List of Notations

a, a_{max}, PGA	Peak ground acceleration
A	Cross-section area, empirical model parameter
A_b	Effective cross-section area of beam
A_c	Effective cross-section area of column
A_{gb}	Gross cross-section area of beam
A_{gc}	Gross cross-section area of column
b	Width of the bracing member
B	Empirical model parameter
$B(\lambda)$	Parameter for compressive resistance
c	Damping coefficient
C	Seismic response factor, base shear coefficient
C_H	Base shear coefficient corresponding to High a/v records
C_L	Base shear coefficient corresponding to Low a/v records
C_r	Factored compressive resistance
C'_r	Factored compressive buckling resistance
C_f	Total gravity load
D	Dead load
$DCBF_s$	Ductile concentrically braced frames
E	Young's modulus, earthquake load
F	Foundation factor
F_y	Yield strength of steel
g	Ground acceleration
GM	Geometric mean value
HSS	Hollow structural section
H	Storey height
I	Importance factor
I_b	Effective moment of inertia of beam

I_c	Effective moment of inertia of column
I_{gb}	Gross moment of inertia of beam
I_{gc}	Gross moment of inertia of column
I_w	Effective moment of inertia of wall
I_{gw}	Gross moment of inertia of wall
IRS	Inelastic response spectra
ID_t	Maximum interstorey drift
ID_y	Yield storey drift
J_{NBCC}	NBCC base overturning moment reduction factor
J_V	The ratio of MDOF base shear strength to the maximum base shear developed in the structure
k	Initial stiffness of a SDOF system, effective length factor
l	Unsupported length
L	Live load
m	Mass of a SDOF system
$MDOF$	Multi-degree of freedom system
M, M_M	MDOF base shear strength modification factor
M_V	The ratio of maximum base shear developed in the MDOF system to the base shear strength of the associated SDOF system
M_{nb}	Nominal moment resistance of beam
M_{rb}	Factored moment resistance of beam
M_{rc}	Factored moment resistance of column
M_r^-	Negative moment resistance of beam
M_r^+	Positive moment resistance of beam
n	Parameter for compressive resistance, number of statistical samples
N	Number of storeys
$NBCC$	National building code of Canada
$NEHRP$	National earthquake hazard response protection
P	Gravity load
r	Radius of gyration

R	Strength reduction factor
R_d	Reserve strength ratio
R_f	Reduction factor due to $P - \Delta$ effect
R_w	Reduction factor
S	Seismic response factor
$SDOF$	Single degree of freedom
S_a	SDOF response acceleration
$S_a(\mu)$	SDOF response acceleration for a target ductility of μ
$S_a(T)$	SDOF response acceleration at period T
$S_a(\mu, T)$	SDOF response acceleration at period T and target ductility of μ
S_m	Maximum SDOF response acceleration
S_{aL}	Geometric spectral mean of the two long period records
S_{aS}	Geometric spectral mean of the two short period records
t	Thickness
T	Structural period, first mode period
T_g	The predominant period of a ground motion
T_r	Factored tensile resistance
$u(t)$	Relative displacement
$u_g(t)$	Ground displacement
$\bar{U}$	Calibration factor
UBC	Uniform building code
UHS	Uniform hazard spectra
v	Zonal velocity
V	Design base shear, the base shear at formation of the first yield
$V(t)$	Restoring force in a SDOF system
V_u	The reduced base shear strength, V_e/R , for ultimate limit state
$\bar{V}$	Storey shear strength
V_f	The lateral force the produces a storey shear of V_{fd}
V_{fd}	Storey shear strength when the compression brace buckles
V_{fy}	Storey shear strength when the tension brace yields

V_{maxM}	The maximum base shear experienced by the MDOF system
V_{yM}	The base shear strength of the MDOF system
V_{yS}	The base shear strength of the SDOF system
w	Uniformly distributed load
W	Seismic weight
x_i	The i th statistical sample
Z	Seismic zone factor
α	Empirical model parameter
Δ	Storey drift
$\Delta_e, \Delta_{elastic}$	Elastic storey displacement
$\Delta_i, \Delta_{inelastic}$	Inelastic storey displacement
Δ_{i1}	The first storey inelastic displacement
Δ_p	Plastic storey displacement. $\Delta_i - \Delta_y$
Δ_y	Storey yield displacement
Δ_{y1}	The first storey yield displacement
η	Normalized strength
ϕ	Resistance factor for steel
γ	Empirical model parameter
λ	Non-dimensional slenderness parameter
θ	The angle between the bracing member and the horizontal axis
μ	Ductility ratio
μ_t	Target ductility ratio
μ_1	The first storey ductility ratio
ω	Natural circular frequency
ξ	Damping ratio

Chapter 1

INTRODUCTION

1.1 GENERAL

In structural design, engineers have been and will be faced with the apparent conflict between safety and economy, practicality and creativity, functionality and elegance, simplicity and comprehensiveness of design. The aim of design codes is to guide the engineer in such important decision making and, therefore, codes need to be transparent and reflective of important parameters affecting the design of structures.

In concept, earthquake protection is a simple *capacity/demand* issue with the objective of providing capacities that exceed the imposed demands by a sufficient margin of safety. *Seismic demands* describe the severity of the effects imposed by the earthquakes. In this context a demand parameter is defined as a quantity that relates seismic input (ground motion) to structural response. Thus, it is a response quantity obtained by filtering the ground motion through a linear or nonlinear structural filter. A simple example of a demand parameter is the acceleration response spectrum, which identifies the strength demand for an elastic single degree

of freedom (SDOF) system. The peak ground acceleration (PGA) is not a demand parameter; it is merely a ground motion parameter independent of the structural response. *Seismic capacities* describe the ability of the structure to sustain the imposed seismic demands. An example is the ductility capacity which is a measure of the maximum deformation capacity of a critical element or of the structure as a whole.

Current code based seismic designs are based on the use of empirical coefficients that obscure the design process and lead to inconsistent designs with unknown levels of protection. Even though present codes have served the profession well, the need exists for improvements in their design approach. These improvements should explicitly incorporate capacity/ demand concepts which are transparent to the design process and provide consistent levels of protection for the given limit states and still be simple enough for routine design. Such a design approach could have the format given in Chapter 5 of this thesis. This capacity/demand based design approach has formed the motivation for this study, which is intended to provide basic information needed for its implementation.

1.2 STATEMENT OF THE PROBLEMS

The generally accepted objectives in the seismic design of a building are to ensure that (1) the life safety of the users and the general public is preserved in the event of the maximum credible earthquake that the building may experience.

and (2) that the building suffers no structural damage under a less severe but more frequent earthquake. For special structures additional objectives may be defined. For the seismic design of normal buildings, most codes, in fact, specify only a single design earthquake which the building and its components are required to sustain without collapse. The building is expected to undergo some structural and non-structural distress during the design earthquake. It is assumed that the building designed in this manner will automatically meet the goal of no damage in a moderate earthquake.

In keeping with the foregoing principle, the codes require that the building be designed to resist a base shear obtained from an idealized response spectrum which is representative of the design earthquake and hence of the seismicity of the site. The elastic base shear is reduced to account for inelasticity in the structure. The amount of such reduction is dependent on the capacity of the structure to undergo inelastic deformation without collapse, often referred to as the ductility capacity. An additional reduction is permitted in recognition of the observed fact that structures designed according to the provisions of the code possess considerable reserve of strength beyond that required by the design forces. For the sake of discussion, consider two North American seismic codes, the National Building Code of Canada (NBCC) (Canadian, 1995) and the Uniform Building Code (UBC) (International, 1994).

In Canada, NBCC defines the design earthquake as one that has a 10% likeli-

hood of being exceeded during a period of 50 years. The corresponding elastic base shear V_e is obtained from

$$V_e = vSIFW \quad (1.1)$$

where v is the zonal velocity ratio representing the seismicity of the site. S is the seismic response factor. I is the importance factor. F is the foundation factor and W is the gravity load contributing to inertia forces. The seismic response factor S is a function of the fundamental natural period of the structure. Thus, vS represents the spectral acceleration in units of g . The elastic base shear is reduced as follows to obtain the design base shear.

$$V = V_e \frac{U}{R} \quad (1.2)$$

where the reduction factor R is a measure of the ductility capacity of the structure. and the calibration factor U which may be interpreted as a factor that accounts for the reserve strength or overstrength. In NBCC, $U = 0.6$ and R varies from 1 to 4. depending on the lateral load resistance system of the building structure.

In the United States, UBC stipulates that the building be designed for a base shear given by

$$V = \frac{ZIC}{R_W} W \quad (1.3)$$

where Z is the seismic zone factor, I is the importance factor, C is the seismic response factor, W is the seismic weight, and R_W is a reduction factor. Factor ZC represents the elastic spectral acceleration in units of g . The magnitude of the reduction factor R_W varies from 4 to 12. In UBC the design earthquake forces are

supposed to be used in association with the calculated strength at working stress level. For the purpose of comparison with NBCC, it is useful to convert the design forces to correspond to yield level. This can be done by applying a load factor of 1.5 to the forces or by changing the reduction factor to $R = R_W/1.5$. The maximum value of the modified reduction factor thus works out to 8. Such a large value can not be justified purely on the basis of the ductility of the structure. Evidently the reduction factor of UBC is a composite factor accounting for both ductility and overstrength.

There are issues concerning some of the above mentioned parameters used by the seismic codes and whether or not these parameters actually reflect the true behaviour of a structure during the maximum credible earthquake it experiences. These issues can be classified into three main categories:

- 1) the derivation of design base shear from the uniform hazard elastic spectra (UHS) representing the seismicity of the site.
- 2) the validity of overstrength factor and.
- 3) the incorporation of the ductility and strength demands in MDOF systems.

The present work addresses these issues and develops new methodologies to obtain more realistic values for the parameters used in the codes. Individual issues are described below in brief.

1.2.1 Issues Regarding The Derivation of Base Shear From UHS

In the current practice of seismic design, the earthquake forces that the building is required to resist without collapse are calculated by an equivalent lateral load analysis procedure. In this procedure, the elastic design base shear is obtained from an idealized response spectrum for a multi-degree-of-freedom system with 5% damping. The elastic response spectrum is derived by amplifying the ground motion parameters: the peak ground acceleration, and the peak ground velocity for the seismic zone in which the building is located. In the NBCC and UBC, S and ZC , in Eqs. 1.1 and 1.3, are, in fact, both representative of the elastic response acceleration. The elastic base shear calculated as above is, then, reduced by a constant force reduction factor to account for inelasticity in the structure. There are two difficulties related to this procedure.

- 1) It is now recognized that the idealized spectrum derived by amplifying the peak ground motion bounds may deviate significantly from the true spectrum for a site (Atkinson, 1991). Spectral shape for a site is governed by the magnitudes and source distances of earthquakes that contribute most significantly to the hazard. If for a given site these parameters are different from those used in deriving the standard spectral shapes, the site specific spectra deviate considerably from the standard spectra.

Since the mid 1970s, methodologies have become available for deriving linear elastic spectra for a given site and for a given hazard level, say 10% probability

of exceedance in 50 years. Such spectra are called Uniform Hazard Spectra (UHS). For a given site, the UHS provide the envelope of the spectral acceleration responses, at specified values of the period, of an elastic SDOF system subjected to the earthquakes that contribute the most in the seismicity of that site. Because UHS provide response parameters that can be used directly in estimating the design earthquake forces, they are preferable to the spectra derived indirectly from peak ground motion bounds. However, methodologies must be developed to derive the design base shear from UHS.

- 2) In the interest of economy, most buildings are designed to have a strength that is a fraction of the strength required to resist the forces derived from an elastic response spectrum. This implies that a building structure is expected to undergo inelastic deformation during the design earthquake. For a given structure and earthquake motion, the amount of inelastic deformation depends on the ratio of the strength of the structure to the strength required to keep its response within elastic limit. In general, the smaller is this ratio, the greater is the inelastic deformation. Inelastic deformation causes damage to the structure. The level of damage should be kept within the capacity of the structure to sustain such damage without collapse. Various measures have been adopted to assess the damage level and the damage capacity. The simplest of these measures is the so called ductility, defined as the ratio of the total deformation to the yield deformation and denoted by μ .

In the current practice of seismic design (e.g., in NBCC), a force reduction factor, R is applied to the elastic base shear to obtain the design shear. Factor R is somehow related to ductility capacity, although engineering judgment has perhaps played a key role in the selection of the specified values. The recommended values of R are independent of the period of the structure. However, it is shown in this study as well as in studies by other researchers (Bertero, et al., 1991; Nassar and Krawinkler, 1991; Tso and Naumoski, 1991; Miranda, 1993) that R is strongly dependent on the period, and for a given μ , in the short period range it generally decreases with a decrease of period. The preceding observations reveal that there is considerable uncertainty associated with the selection and use of R for an inelastic structure. Therefore, it is preferable to obtain the design strength directly from inelastic response spectra produced for a range of values of the ductility μ . These spectra provide the relationship between strength and period of the structure for a specified value of ductility. Their use avoids the uncertainties associated with the use of R in calculating the base shear.

1.2.2 Issues Regarding The Overstrength Factor

Observations of structural performance under many past earthquakes have led to the conclusion that code designed buildings must possess significant overstrength in order for them to have survived without damage earthquake forces considerably larger than those considered in design. The additional strength of the structure

beyond what the code recommends is referred to as overstrength. Structural overstrength can result from many different sources. Table 3.1 lists some of these sources (Fishingier and Fajfar, 1990; Miranda and Bertero, 1989; Mitchell and Paultre, 1994; Nassar and Krawinkler, 1991; Osteraas and Krawinkler, 1989; Tso and Naumoski, 1991; Zhu et al, 1991; Uang, 1992).

As discussed earlier, it is apparent that both the US and the Canadian Code provisions rely on the presence of significant overstrength in the structure. The recognition of the overstrength is explicit in NBCC, with the provision of factor U , while it is implied in UBC with the use of composite reduction factor R representing both ductility and overstrength.

There are a number of questions related to the reliance on overstrength. First, it is necessary that the sources of reserve strength be clearly recognized so as to determine whether the contribution to reserve strength from a particular source can, in fact, be relied upon. Second, in principle, the overstrength should not be counted upon in designing new building structures, and if there is a reserve strength, it should be used in assessing the capacity of the structure and not in scaling the design forces.

1.2.3 Issues Regarding the Seismic Demands of MDOF Systems

Real structures are mostly MDOF systems whose response is governed by several translational and torsional modes. According to most seismic code provisions, the strength that the MDOF structure must possess in order to resist without collapse

the design earthquake is calculated by an equivalent static lateral load analysis procedure. In this procedure, the elastic design base shear is obtained from an idealized response spectrum with 5% damping. This response spectrum is, in fact, derived from the elastic response of SDOF systems, which is modified in long period range in order to provide adequate protection for MDOF systems.

There are several issues that relate to the procedure of calculating the strength demand in MDOF systems. Although the design response spectra utilized by the codes are raised in the long period range in order to provide more protection for MDOF systems, recent studies (Nassar and Krawinkler, 1991) have shown that for steel moment resisting frames such raising still does not provide sufficient strength for MDOF systems. The situation may be even worse for other structural types such as concrete flexural wall type structures. Further study is required to determine representative MDOF seismic demands in different structural types.

As discussed earlier, it is now recognized that it is preferable to derive the elastic design response spectra directly from the uniform hazard spectra (UHS) rather than by amplifying the ground motion bounds. Therefore, it is quite likely that the UHS will form the basis of the future seismic codes. The elastic strength spectra derived from UHS are applicable only to elastic SDOF systems, hence, these strength spectra need to be modified before they can be applied to a MDOF system.

In deriving the spectra for use with MDOF systems, it is useful to compare the strength requirement for a multi-storey building with that for an associated

SDOF system. The associated SDOF system is defined as one having, in its linear range, the same period and damping as the multi-storey building. The weight of the associated SDOF system is equal to the total weight of the multi-storey building. The ductility demand in a multistorey building having the same strength (base shear capacity) as the associated SDOF system can be quite different from that in the SDOF system (Nassar and Krawinkler, 1991). It is apparent that if the ductility demand in a multistorey building is to be limited to that in the associated SDOF system, the design strength has to be suitably adjusted. The ratio of the base shear strength of MDOF system to that of the SDOF system for identical ductility demands is referred to as the MDOF strength ratio.

Since the MDOF strength ratio is dependent on the ductility capacity, it must also be dependent on the lateral load resistance system (LLRS) used in the building. This is because different LLRSs have different ductility capacities. There is no clear provision in the current codes to recognize the latter. In fact, regardless of the structural type, the codes (e.g., NBCC 95) use the same response spectrum. It might be argued that the design spectra given in the code cover the envelope of the responses of different LLRSs. However, there is no complete and comprehensive study on the nonlinear seismic ductility and strength demands of different MDOF LLRSs to support such argument.

Nassar and Krawinkler (1991) studied the ductility and strength demands in MDOF systems. However, their results are limited to simplified moment resisting

frames of just steel. Seismic demands of other structural types and/or LLRSs is expected to be different from those of steel moment resisting frames. This is because of the inherent differences in the nonlinear dynamic behaviour of such structures.

The above discussion highlights the need for a comprehensive study on the seismic demands of different MDOF LLRSs.

1.3 REVIEW OF LITERATURE

The following is an overview of some existing literature on the preceding issues.

Literature on the SDOF response spectrum and UHS

The derivation of a spectral shape from peak ground motion bounds is based on studies carried out by Newmark and Hall (1982). In these studies, the spectral shapes were obtained by averaging the spectral curves for ground motion records from a few earthquakes. Most of these earthquakes were in the magnitude range of 6 to 7 and had a distance to the source of about 20 km.

Bertero, et al., 1991; Nassar and Krawinkler, 1991; Miranda, 1993 studied the strength reduction factor, R , from the inelastic response of SDOF systems subjected to a large number of earthquakes with diverse frequency content. They showed that R is strongly dependent on the period, and for a given μ , in short period range it generally decreases with a decrease of the period.

Bertero et al. (1991) report that the period range in which R increases with

the period is actually bounded by the predominant period of the ground motions T_g . Therefore, a constant reduction factor of $R = \mu$ appears to be too high for structure with period $T \leq T_g$, and may lead to an unsafe design.

Bazzurro and Cornell (1994); and Krawinkler and Nassar (1990) have studied the effect of magnitude and the source distance of different earthquake events on the strength reduction factor R . They have shown that the mean value of R , although dependent on the period, is insensitive to the magnitude and source distance of the earthquake.

Atkinson (1991) studied the uniform hazard spectra (UHS) in comparison with the traditional scaled spectra derived by amplifying the peak ground motion bounds. That study covered the seismicity of both western and eastern North America. One of the conclusions in that study is that a scaled spectrum may overestimate the elastic response for intermediate frequencies for some types of earthquakes, sometimes by as much as 300%. In the same study it was recommended that the future seismic hazard mapping should be based on uniform hazard linear response spectra. It is worth noting that UHS for a site is governed by the magnitudes and source distances of earthquakes that contribute most significantly to the hazard.

Adams et al. (1996) have summarized the methods used by them to derive the new seismic hazard maps for Canada. They have provided both the 50th and 84th percentile hazard values for major cities in Canada. The hazard maps incorporate new geological and tectonic information, as well as earthquake data collected since

1985 when the last such maps were produced. The maps will provide spectral accelerations for a 5% damped elastic SDOF system at several values of the period and for a uniform probability of exceedance of 10% in 50 years. The new hazard maps may form the basis of the earthquake design provisions of NBCC 2000.

Studies on the issue of overstrength

Mitchell and Paultre (1994) investigated different sources of overstrength in reinforced concrete structures. They proposed a design approach that accounts for structural overstrength due to three different sources: hierarchy of strength, design forces controlled by other load cases (e.g., wind), and provisions of some minimum requirements set by the material codes.

Uang (1992) studied the force reduction and displacement amplification factors applicable to seismic design of real structures. He developed expressions for these two factors that explicitly include an overstrength factor. The overstrength factor in this study was defined as being the ratio of the ultimate yield strength to the strength at the first significant local yielding. In derivation of the ultimate strength, the effect of different sources of overstrength including multiple load combinations and code minimum requirements was also considered.

Nassar and Krawinkler (1991) studied the reserve strength present in structures designed according to the US codes. They defined two different levels of overstrength factor. One is the ratio of the base shear at failure of the weakest member, E_l , to the design base shear, E . This is referred to as local overstrength

factor. The other overstrength factor, called the global overstrength factor, is defined as the ratio of the lateral strength capacity, E_g , to the design base shear, E . In derivation of these factors, they included the effect of different sources of overstrength such as code minimum requirements and discrete member sizes etc.. They provided these overstrength factors for moment resisting frames of both steel and concrete as well as steel braced frames. The braced frames possessed the lowest values of overstrength factors in the vicinity of 2.0 for both local and global levels, reflecting the lack of ductility capacity in these structures. The low rise steel moment-resisting frames possessed a global overstrength factor of up to 7.0 while the local value was in the range of 3.0. The overstrength factors in concrete moment-resisting frames was lower than those in the steel counterparts. The values ranged up to 5.0 for the global and 3.0 for the local overstrength factors.

Osteraas and Krawinkler (1989) studied the behaviour of steel frame structures in Mexico City during the 1985 Mexico earthquake and found an overstrength factor which ranged from 2.0 for long period structures to about 13 for very short period structures. Similar investigation for low rise reinforced concrete frame structures in Mexico City has been carried out by Miranda and Bertero (1989) who found an overstrength factor greater than 2.5 for a 4-storey building and greater than 5.0 for a 2-storey building.

Fishinger and Fajfar (1990) have summarized the results from various investigations of a 7-storey reinforced concrete frame-wall structure. They have shown

that the overstrength factor could be up to 4.0.

Following nonlinear static push-over analyses of two reinforced concrete frames of 10 and 18 storeys, Zhu et. al. (1991) concluded that while $P-\Delta$ has a detrimental effect, strain hardening has a beneficial impact on structural overstrength.

Literature on ductility and strength demands in MDOF systems

The number of studies on ductility and strength demands in MDOF systems are few. The scope of the existing literature in this regard is usually limited to simplified structural models and use of earthquake motions from certain sites. Such studies have recently been carried out by Nassar and Krawinkler (1991). They performed inelastic analyses of simplified multistorey steel moment-resisting frames subjected to a series of ground motions from Whittier Narrows Earthquake, October 1, 1987. The results obtained by the two researchers show that the design base shear in the MDOF systems needs to be modified in order to limit the storey ductility demands at the base to the prescribed target ductility ratio, which is also the ductility ratio in the associated SDOF system. The ratio of the base shear of MDOF system to that of the SDOF system for identical ductility demands is referred to as the strength ratio. The two researchers studied the variation of MDOF strength ratio for weak-beam and weak-column models. They considered four different target ductilities of $\mu_t = 2, 3, 4$ and 8, and two strain hardening ratios of 0 and 10%. They concluded that storey ductility demand is not only different from that in the associated SDOF system, but also varies across the height of the multistorey

building. In general, storey ductility demand is higher in the lowest and uppermost storeys and usually is most critical in the first storey. They also concluded that the strength ratio increases with period as well as with target ductility. In general, the strength demand is higher for a weak column model than for a weak beam model. Also, strain hardening has a beneficial effect, reducing the amount of extra strength required.

In the same study, Nassar and Krawinkler (1991), indicated that although UBC raises the higher period range of the design response spectrum, the strength demand in steel moment resisting frames is still underestimated, especially in the long period range.

Additional existing literature that is relevant to the issues outlined in the previous sections is referred to, and briefly reviewed where appropriate, in the chapters in which the individual components of the study are presented.

1.4 OBJECTIVE AND SCOPE OF THE THESIS

The main objective of this study is to provide information for the development of a transparent capacity / demand approach, for seismic design of building structures. The capacity issues are not addressed in this study, but it is assumed that ductility capacity is the basic design parameter. For reasons discussed in Section 1.2, inelastic strength and ductility demands then become the relevant demand parameters. The objectives of the study in this thesis are:

- (a) to assess the importance of different demand parameters.
- (b) to evaluate patterns in demand parameters with a view to improving our understanding of the physical phenomena involved in seismic response of structures.
- (c) to provide statistical information on demand parameters that can be utilized to assess the performance of structures designed according to existing codes
- (d) to implement the findings in this study into a proposed capacity / demand based base shear format for seismic design of building structures.

The objectives listed here will be achieved through studies that have three main components. Chapter 2 contains the first of these: the study of inelastic SDOF constant ductility response spectra for three different ensembles of ground motions each containing 15 records. These inelastic spectra, in fact, provide the strength demand for a SDOF system for a number of different target ductility ratios. Then, these SDOF strength demand spectra as well as the uniform hazard spectra UHS are used for the development of a proposed empirical model for producing *inelastic* uniform hazard spectra for seismic design.

The second component of the study is dealt with in Chapter 3. This chapter focuses on the issues related to overstrength. Some different sources of overstrength mentioned in literature are examined. It is then reasoned that while some of these sources are uncertain, others, although reliable, are best taken into account in design in assessing the capacity of the structure rather than in scaling down the design

forces. One exception is the reserve strength owing to the redistribution of internal forces. Scaling down of design forces to account for this source of reserve strength simplifies the analysis and is therefore useful. A complete and comprehensive study is conducted to assess the extent of reserve strength attributable to redistribution in different building structures: steel moment-resisting frames, steel concentrically braced frames, concrete moment-resisting frames and concrete flexural wall frames are analyzed for their response to lateral loads.

A complete and comprehensive evaluation of the ductility and strength demands in multi-storey building structures forms the third component of the study in this thesis. Chapter 4 covers this study. Different structural types with a relatively wide range of first mode period are studied to gain an insight into the pattern of their inelastic responses to ground motions with diversified frequency contents. The site-specific UHS compatible acceleration time histories developed by Atkinson et al., (1998) for some selected cities in the west and the east of Canada are used as the input ground motions. One set of records containing four time histories that simulate the seismicity in two cities of Vancouver and Victoria in the west is used. Another set of four records is considered for three cities of Montreal, Ottawa and Quebec City in the east. The structural types studied are: moment-resisting frames, concentrically braced frames, and flexural walls.

The results obtained from the studies in Chapters 2, 3, and 4 are then logically assembled in Chapter 5 in order to form the basis of a proposed format/approach

for calculating the seismic base shear required for the design of multi-storey building structures.

Chapter 6 summarizes this work and provides an overview of the conclusions made throughout the study. A list of suggestions for future work is also included in this chapter.

Chapter 2

APPLICATION OF INELASTIC UNIFORM HAZARD SPECTRA IN SEISMIC DESIGN

2.1 INTRODUCTION

The objective of the earthquake-resistant design requirements of National Building Code of Canada (NBCC) (Canadian Commission, 1995) and of the US Uniform Building Code (UBC) (International, 1994) is to ensure that buildings that are designed to satisfy the requirements do not collapse when subjected to the design earthquake. The design earthquake is defined as an event that has 10% probability of exceedance in 50 years. The forces that the building is required to resist without collapse are calculated by an equivalent lateral load analysis procedure. In this procedure, the elastic design base shear is obtained from an idealized response spectrum for a multi-degree-of-freedom system with 5% damping. In both NBCC and UBC this spectrum is derived from two ground motion parameters: the peak ground acceleration, and the peak ground velocity for the seismic zone in which the building is located. The spectral shape is, in effect, derived by applying suitable amplification factors to the ground motion bounds.

The derivation of a spectral shape from peak ground motion bounds is based on studies carried out by Newmark and Hall (1982). In these studies, the spectral shapes were obtained by averaging the spectral curves for ground motion records from a few earthquakes. Most of these earthquakes were in the magnitude range of 6 to 7 and had a distance to the source of about 20 km. It is now recognized that the idealized spectrum derived as above may deviate significantly from the true spectrum for a site, and the error can be as large as 300% (Atkinson, 1991). Spectral shape for a site is governed by the magnitudes and source distances of earthquakes that contribute most significantly to the hazard. If for a given site these parameters are different from those used in deriving the standard spectral shapes, the site specific spectra deviate considerably from the standard spectra.

Since the mid 1970s, methodologies have become available for deriving linear elastic spectra for a given site and for a given hazard level, say 10% probability of exceedance in 50 years. Such spectra are called Uniform Hazard Spectra (UHS). They provide spectral accelerations at specified values of the period of an elastic single-degree-of-freedom (SDOF) system. Because UHS provide response parameters that can be used directly in estimating the design earthquake forces, they are preferable to the spectra derived indirectly from peak ground motion bounds.

The Geological Survey of Canada is producing a suite of new seismic hazard maps for Canada, to be released for trial use (Adams et al. 1996). The hazard maps incorporate new geological and tectonic information, as well as earthquake data

collected since 1985 when the last such maps were produced. The maps will provide spectral accelerations for a 5% damped elastic SDOF system at several values of the period and for a uniform probability of exceedance of 10% in 50 years. The new hazard maps may form the basis of the earthquake design provisions of NBCC 2000. However, before the UHS can be used in design, the site specific response values obtained from such spectra must first be incorporated into a seismic zoning map for the country. The UHS typically provide spectral ordinates for a number of different vibration periods. On the other hand, zoning maps for use in design should be based on a limited number of parameters, perhaps no more than 2.

In United States, the recent version of NEHRP recommended provisions for seismic design (Building Seismic Safety Council, 1994) provides zoning maps of spectral acceleration values at periods of 0.3 and 1.0 s for trial use in design.

An essential step in the use of UHS in design is the development of a methodology to obtain the design elastic base shear from the information contained in the zoning maps, and the adjustment of elastic base shear to account for the expected inelastic response of the building structure. A methodology for achieving these objectives is proposed here.

Current thinking in both the NBCC code committees and NEHRP favours the use of a 2500 year earthquake corresponding to a 2% probability of exceedance in 50 years rather than the 450 year earthquake which has 10% probability of exceedance in 50 years for defining the seismic hazard. However, conceptually the

methodology described here does not change with the recurrence interval used in defining the hazard.

2.2 UNIFORM HAZARD SPECTRA

Preliminary seismic hazard spectra have been prepared by the Geological Survey of Canada for 22 western and eastern cities of Canada. As stated earlier, the uniform hazard spectra (UHS) provide spectral acceleration ordinates for a 5% damped SDOF system for selected values of the period. Tables 2.1a and 2.1b list the UHS values corresponding to an 84% confidence level, for Eastern and western Canada respectively (Adams et al. 1996). The values shown are for firm ground conditions, including soft to firm rock, stiff cohesive soils and dense granular soils with shear wave velocities in the range of 360 to 750 m/s. The spectral values have a 10% chance of exceedance in 50 years, implying an annual probability of exceedance of 0.0021. These values are used in the design methodology presented here.

2.3 INELASTIC RESPONSE SPECTRA

In the interest of economy, most buildings are designed to have a strength that is a fraction of the strength required to resist the forces derived from an elastic response spectrum. This implies that a building structure is expected to undergo inelastic deformation during the design earthquake.

For a given structure and earthquake motion, the amount of inelastic deformation depends on the ratio of the strength of the structure to the strength required to keep its response within elastic limit. In general, the smaller is this ratio, the greater is the inelastic deformation. Figure 2.1 shows the relationship between the displacement of a SDOF structure and the lateral force acting on it during an earthquake. If the structure is elastic, the maximum value of the lateral force or base shear is V_e and the corresponding displacement is Δ_e . The minimum strength required to keep the structure elastic is thus V_e . Assume that the structural strength provided is V_y , with $V_y \leq V_e$. Then if the lateral force-displacement relationship for the structure is elasto-plastic in nature, the structure will yield at a base shear of V_y and will undergo further displacement at a constant value of shear equal to V_y . Let the maximum displacement be denoted by Δ_i . This displacement is comprised of two parts, yield displacement Δ_y , and plastic or inelastic displacement Δ_p . As stated earlier, the smaller is the ratio V_y/V_e , the greater is the inelastic deformation, Δ_p .

Inelastic deformation causes damage to the structure. The level of damage should be kept within the capacity of the structure to sustain such damage without collapse. Various measures have been adopted to assess the damage level and the damage capacity. The simplest of these measures is the so called ductility, defined as the ratio of the total deformation to the yield deformation and denoted by μ

$$\mu = \frac{\Delta_i}{\Delta_y} \quad (2.1)$$

The objective of the earthquake design is to determine the value of strength V_y that will limit the ductility demand to within the ductility capacity. In the 1995 NBCC, strength V_y is determined from V_e , the elastic base shear, by applying a reduction factor R to the latter, so that

$$V_y = \frac{V_e}{R} \quad (2.2)$$

The design yield strength is, in fact, taken as $V = U V_y$, where $U = 0.6$ is a reduction factor to account for the reserve strength observed in structures designed according to the code. The assumption is that a structure designed to have a yield strength V will, in fact, possess a yield strength of V_y .

Equation 2.2 permits the use of elastic response spectra for determining the design strength of an inelastic structure. In the current NBCC, the R values are somehow related to ductility capacity, although engineering judgment has perhaps played a key role in the selection of the specified values. The recommended values of R are independent of the period of the structure. However, it is shown in this study and supported by other researchers (Bertero, et al., 1991; Nassar and Krawinkler, 1991; Miranda, 1993) that R is strongly dependent on the period, and for a given μ , in short period range it generally decreases with a decrease of the period. This means that the use of a constant (period independent) R that is adequate for long period structures will lead to an unsafe design for short period structures.

Since the use of elastic spectra with reduction factor R to obtain the design strength for an inelastic structure involves considerable uncertainty, it is preferable to obtain the design strength directly from an inelastic response spectrum for the appropriate value of the ductility μ . Such a spectrum provides the value of strength required for an SDOF system of given period in order to limit its ductility demand to a target value. The method used in the present study to derive the design inelastic response spectra is described in the following paragraphs.

2.3.1 Earthquake Ground Motions

The methodology for the use of inelastic spectra in design, presented here, uses a set of existing ground motion records. The basis for the selection of such records is described in the following paragraphs.

Since the recording of first ground excitation during the Long Beach earthquake of 1933 in California, thousands of strong motion records have been obtained from many countries including Canada, United States, Japan, and Mexico. These records are different in many respects. The intensity, duration of strong vibration and frequency content of an earthquake record depend on a number of factors, including the magnitude, the epicentral distance, the local geology and the soil condition at the site. These differences among records can lead to significant differences in structural responses, and in turn in the design implications.

One major factor that has significant effect on structural responses is the fre-

quency content of the earthquake ground motion. The ratio of peak ground acceleration (expressed in units of g) to peak ground velocity (expressed in units of m/s) has been used as a measure of the frequency content of a ground motion. Records with high a/v ratios are normally associated with moderate earthquakes at short epicentral distance, while records with low a/v ratio are normally associated with large earthquakes at large epicentral distances. In this study, three ensembles of ground motions with different a/v ratios are selected from the MUSE Database of McMaster University (Naumoski et al., 1988). The first ensemble contains 15 records with high a/v ratio ($a/v \geq 1.2$). The second ensemble contains 15 records with intermediate a/v ratio ($1.2 > a/v \geq 0.8$). The third ensemble includes 15 records with low a/v ratio ($a/v \leq 0.6$). All of the selected records were recorded on rock or stiff soil sites. Out of these 45 records, 29 are from the United States, 7 from Japan, 5 from Mexico, 3 from Yugoslavia and 1 from Canada. Complete details of these earthquake records including their elastic response spectra are given in the report from MUSE Database of McMaster University (Naumoski et al., 1988).

2.3.2 Method of Analysis

The response of a damped SDOF system when subjected to earthquake ground motions is given by

$$m\ddot{u}(t) + c\dot{u}(t) + V(t) = -m\ddot{u}_g(t) \quad (2.3)$$

where m , c , and $V(t)$ are respectively the mass, damping coefficient, and restoring force of the system; $u(t)$ = the displacement relative to the ground; $u_g(t)$ = the ground displacement; and an overdot represents derivative with respect to time.

On dividing through by $m\Delta_y$, Eq. 2.3 can be normalized as follows:

$$\ddot{\mu}(t) + 2\omega\xi\dot{\mu}(t) + \omega^2\frac{V(t)}{V_y} = -\frac{\omega^2}{\eta}\frac{\ddot{u}_g(t)}{(\ddot{u}_g)_{max}} \quad (2.4)$$

where μ = the displacement ductility ratio; V_y = the system's yield resistance; ω = the natural circular frequency; ξ = the damping ratio; and η = the nondimensional strength of the system. The last three quantities are defined as

$$\omega = \left(\frac{k}{m}\right)^{1/2} \quad (2.5)$$

$$\xi = \frac{c}{2m\omega} \quad (2.6)$$

$$\eta = \frac{V_y}{m(\ddot{u}_g)_{max}} \quad (2.7)$$

where k = the initial stiffness of the system.

By keeping the nondimensional strength factor, η , constant one can solve Eq. 2.4 to derive the so-called constant strength spectra. However, since the objective of the earthquake design is to determine the strength demand that will limit the ductility

demand to within the ductility capacity, computation of the so-called constant ductility strength demand spectra is of importance. A constant displacement ductility inelastic response spectrum (IRS) is a plot against period of the yield strength of an SDOF system required to limit the displacement ductility to specified target displacement ductility ratio, μ_t . Sometimes, this type of spectrum is also referred to as strength demand spectrum (Krawinkler and Nassar 1990). In this study, constant displacement ductility IRS are computed by iteration on the system's nondimensional strength η until the ductility computed with Eq. 2.4 is, within a certain tolerance, the same as the target ductility. The tolerance is chosen such that η is considered satisfactory if the computed ductility is within 1% of the target ductility.

The following values of target ductilities are selected for this investigation: 1 (elastic behaviour), 2, 3, 4, 5, and 6. For each earthquake record and each target ductility, the IRS are computed for a set of 42 periods. Considering the large number of records, ductilities, and periods of vibration and the large computational effort involved in calculating constant displacement ductility IRS through iteration, this study is limited to SDOF systems that have a bilinear hysteretic behaviour with a post-yield stiffness of 3% of the elastic stiffness and a damping ratio of 5%. Previous studies such as those by Iwan (1980), Al-Sulaimani and Rossett (1985) and Susicuglu et al. (1994) have shown that the strength demand obtained by using a stiffness degrading hysteretic model does not differ appreciably from that obtained by using a bilinear hysteretic model. The nonlinear dynamic time history analyses

are performed a total of 11340 times, a number obtained by the combination of 45 earthquake records, 6 target ductilities, 1 strain hardening, 1 hysteretic model, and 42 discrete periods ranging from 0.075 to 3.0 s.

It has been suggested that constant displacement ductility IRS can be computed by interpolation from constant strength IRS or constant yield displacement IRS (Elghadamsi et al. 1987; Mahin et al. 1983). While such a procedure is conceptually correct and can result in a significant saving in the computational effort, it has been shown by Miranda (1993) and also noted in the current study that the procedure can lead to significant errors in the required lateral strength of a given system. As a matter of fact, it is found that at times more than one strength demand may be obtained corresponding to a given target ductility. For seismic design, only the root with the largest strength is of interest. This strength corresponds to the minimum strength required by the structure (i.e., strength that needs to be supplied) to limit the ductility demand to the target ductility.

The results of IRS analyses are presented in the following sections.

2.3.3 Inelastic Strength Demand Spectra

The normalized strength demand spectra, η , are derived for all of the 45 records. The mean value and the coefficient of variation (COV) of the normalized strength demands are computed for each of the three sets of 15 records having high, intermediate, and low a/v ratios for ductility ratios varying from 1 to 6. Figures 2.2a.

b, and c show the mean normalized strength demand spectra for each ensemble. It can be seen that the peak normalized strength demand is roughly the same for the three ensembles and occurs at $T = 0.2, 0.3$ and 0.5 s for the high, intermediate and low a/v ratio ensembles, respectively.

Figure 2.2 shows that the magnitude and shape of the elastic and inelastic strength demand spectra differ significantly for different a/v ratio records. Also, regardless of the a/v ratio of the records, for ductilities greater than 4 the strength demand decreases monotonically with increasing period.

Figure 2.3 shows the effect of the a/v ratio of the ground motion on both elastic and inelastic spectra. Figure 2.3a presents a comparison of the elastic response spectra for the three levels of a/v ratios. Figure 2.3b shows the inelastic spectra corresponding to $\mu = 4$. It can be seen that the strength spectra strongly depend on the a/v ratio of the ground excitation. The normalized strength demand for low a/v records is generally the highest throughout the period range for $\mu = 4$, while for the elastic case it dominates the intermediate to long period range.

Figure 2.4 shows the spectra of COV of strength demands for the three ensembles. The COV which is defined as the ratio of the standard deviation to the mean value is a measure of the dispersion of the strength demands. It is seen that generally the dispersion tends to increase with period, but is almost independent of the level of inelasticity (i.e., μ), except for intermediate a/v records that it is higher for lower ductilities in the vicinity of a period of 1.0 s.

2.3.4 Inelastic Displacement Spectra

In the past most of the effort in improving earthquake resistance design codes has been focused on designing for strength only, without proper consideration of the effect of deformation. For any structure that is responding in inelastic range under practically a constant strength, the degree of damage depends on the amount of the plastic deformation. Thus, to control damage, it is necessary to control deformations. Hence, proper consideration of both strength and deformation capacities is necessary for seismic design of structures.

In practice, it is commonly assumed that maximum inelastic displacement $\Delta_{inelastic}$ is almost the same as the elastic displacement $\Delta_{elastic}$. The results of this study show that this argument does not hold true for the entire period range. As a matter of fact, for short to intermediate period ranges, the maximum inelastic deformations could be significantly higher than the maximum elastic deformations depending on the a/v ratio of the motion and the ductility ratio. This observation has also been made by Miranda (1993). Figure 2.5 shows the spectra of the mean value of the ratio $\Delta_{inelastic} / \Delta_{elastic}$ for three ensembles of earthquake records and three ductility ratios: 2, 4, and 6. In the short to intermediate period range, the ratio $\Delta_{inelastic} / \Delta_{elastic}$ increases with increase in ductility and decrease in period. For the remaining period range, the maximum elastic displacement provides a reasonable estimate of the inelastic one.

2.3.5 Strength Reduction Factor Spectra

For a given period, the ratio of spectral strength value corresponding to a ductility of 1.0 to that for a specified value of μ greater than one provides the reduction factor R . The mean values of reduction factors for the three sets of earthquake records, and five values of μ are plotted in Fig. 2.6 as functions of the period T . It is clear that R is dependent on the a/v ratio of the record, ductility capacity, as well as the period. Factor R increases with period in short period range and then becomes constant and almost equal to μ . Bertero et al. (1991) report that the period range in which R increases with the period is actually bounded by the predominant period of the ground motions T_g . Therefore, a constant reduction factor of $R = \mu$ appears to be too high for structure with period $T \leq T_g$, and may lead to an unsafe design. It has been argued that structures designed according to the current codes possess a considerable overstrength which compensates for this drawback of the constant R factor. It is shown in this study that the entire issue of overstrength requires careful consideration. Chapter 3 of this thesis deals with the concept of overstrength in detail.

The following observation can be made from Fig. 2.6.

- 1) The reduction factor approaches 1.0 as the period tends to zero.
- 2) For higher periods, R is constant and approaches the ductility ratio, μ .
- 3) The nature of variation of R with T is significantly different for earthquakes with

different a/v ratios. For high a/v ratio earthquakes, R increases from 1.0 to the ductility value as the period increases from 0.0 to about 0.3 s. For intermediate a/v ratio ground motions, the increase in R is slower, with R becoming equal to μ at periods near to 0.8 s. This supports the argument made by Bertero et al. (1991) that this period in fact corresponds to the predominant period of the ground motions, since the low or intermediate a/v records have inherently longer predominant period than the high a/v records.

In order to evaluate the sensitivity of strength reduction factors to a/v ratio of the ground motions, the spectra of dispersion (COV) of the R factor versus period are presented in Fig. 2.7. The following conclusions can be drawn from these results. First, despite the increase of dispersion in normalized strength with period, the dispersion in the R factor remains almost constant with period. Second, even though dispersion of normalized strength demands is almost independent of the ductility, the dispersion in the R factor increases slightly with ductility. Finally, for all values of ductility, the dispersion in R factor is almost independent of the a/v ratio of the earthquake record. Figure 2.8 provides support for the last conclusion through a comparison of the COV-spectra of R factors for intermediate and high a/v ratio ensembles of records and three ductility ratios of 2, 4, and 6.

2.4 PROPOSED METHODOLOGY FOR DERIVING INELASTIC HAZARD SPECTRA

Since the use of elastic spectra with reduction factor R to obtain the design strength for an inelastic structure involves considerable uncertainty, it is preferable to obtain the design strength directly from an inelastic response spectrum for the appropriate value of the ductility μ . It is expected that, in the future, ground motion relations will become available for inelastic spectral values. However, inelastic response spectra can also be derived from the corresponding elastic spectra. Factor R may be considered as the ratio of elastic spectral acceleration for a given period to the corresponding inelastic spectral acceleration associated with a specified inelastic damage level. In the present work ductility μ has been used as the measure of such damage, but other measures such as hysteretic energy absorption, accumulated ductility level etc. can equally well be used. Previous studies (Krawinkler and Nassar, 1990; Bazzurro and Cornell; 1994) have shown that the mean value of R , although dependent on the period, is insensitive to the magnitude of earthquake event and distance to the source. This implies that if an ensemble of earthquake records provides an elastic spectrum that matches the UHS for a particular site, the same ensemble can be used to derive the inelastic response spectra for that site.

It should be noted that a UHS is the envelope of responses produced by several different earthquakes with varying magnitude and source distance. Thus the maximum short period spectral values may result from short distance earthquakes.

while the long period values may be contributed by more distant earthquakes. In the present study the response spectra produced by two different sets of earthquake records have been used: high a/v and intermediate a/v ensembles. High a/v ratio records are normally associated with moderate earthquakes at short epicentral distance, while records with intermediate a/v ratio are normally associated with large earthquakes at relatively large epicentral distances. The details of the records used in the present study are given in previous section.

The elastic and inelastic normalized strength demand spectra η for these two sets of records were presented in the previous section. They can be used to compute the corresponding elastic and inelastic acceleration response spectra, $S_a(\mu)$. This is achieved simply by multiplying the η -spectrum for each record by its own maximum ground acceleration a_{max} . This is because, η is defined as $mS_a(\mu)/ma_{max}$.

The elastic spectral acceleration curve for each record in the high a/v series is normalized by its own maximum spectral value, S_{mj} . The mean of the high a/v spectra is further scaled so that its maximum is equal to the maximum uniform hazard spectral value for the site, S_m . The same composite scaling factor, S_m/S_{mj} , is later used to scale the inelastic response spectrum for individual records. For the intermediate a/v records, the elastic spectral acceleration curve for each record is normalized by the spectral value at 0.5 s, $S_j(0.5)$. The mean spectrum is then scaled so that the spectral value at 0.5 s is equal to the 0.5 s uniform hazard spectral value for the site, $S_a(0.5)$. The same composite scaling factor, $S_a(0.5)/S_j(0.5)$, is

later used to scale the inelastic spectrum for each individual record. The envelope of the two mean spectrum curves provides the elastic response spectrum for the site under consideration. It is found that this envelope response spectrum matches reasonably with the UHS for all 22 locations considered. As an example, the elastic spectrum envelopes derived as above are compared with UHS curves for Vancouver and Montreal respectively in Figs. 2.9a and b. In each case, the match is quite good.

Considering that the suites of records used in the study provide a good representation of the UHS, the same suites are used to derive the inelastic response spectra for six different values of the ductility capacity μ , namely 1, 2, 3, 4, 5 and 6. In deriving the mean inelastic spectra, individual records are scaled by the composite scaling factors described in the previous paragraph. For use in the design code, curves are fitted to the inelastic spectra derived as above. Using empirical equations for these curves, the inelastic design base shear coefficient can be represented as

$$V_y = CW \quad (2.8)$$

$$C = \text{Max}\{C_H S_m, C_L S_a(0.5)\} \leq \gamma S_m \quad \text{for} \quad T < 0.5\text{s} \quad (2.9a)$$

$$C = C_L S_a(0.5) \quad \text{for} \quad T \geq 0.5\text{s} \quad (2.9b)$$

where

$$C_H = A - BT \quad (2.10a)$$

$$C_L = \frac{\alpha}{T^{2/3}} \quad (2.10b)$$

and parameters γ , A , B and α have the following values

μ	γ	A	B	α
1	1.000	1.460	2.280	0.630
2	0.500	0.716	1.155	0.308
3	0.366	0.457	0.731	0.219
4	0.321	0.369	0.620	0.165
5	0.295	0.339	0.647	0.143
6	0.281	0.322	0.651	0.127

The spectral curves obtained from Eq. 2.8 are compared in Fig. 2.10 with the response spectrum envelopes for Vancouver for ductility values, $\mu = 1$ and $\mu = 4$ respectively. The match is quite good in each case.

Figures 2.11 and 2.12 show the elastic and inelastic design spectra obtained from Eq. 2.8 for two locations, Vancouver and Montreal. In parts (a) of the figures, elastic spectra for $\mu = 1$ are compared with the corresponding UHS as well as the provisions of NBCC. Parts (b) and (c) of the figures show the inelastic spectra for ductility values of 2, 3, 4, 5 and 6.

2.4.1 Comparison with Inelastic Spectra Derived by using R

In the current NBCC, the elastic seismic forces that have been reduced by R to account for inelasticity are further reduced by U , often referred to as the overstrength factor (Nassar and Krawinkler, 1991; Tso and Naumoski, 1993). It is implied in this two stage reduction that R provides a measure of the ductility capacity. Thus μ in the new provision can be taken as R of the current NBCC.

It would be of interest to compare the inelastic spectra obtained by the methodology presented here and those obtained by applying the factor R to an elastic spectrum. This has been done for Vancouver and Montreal and the results are shown in Figs. 2.11 and 2.12. For a low ductility value of 2, the two set of spectra are almost identical. For higher ductilities and short periods the inelastic spectra obtained by the method presented here are significantly higher than those derived indirectly by using the modification factor R .

2.4.2 Comparison of Design Base Shears

It is evident that the seismic design forces obtained by the procedures outlined here will be significantly different from those implied in the present NBCC. These differences arise primarily because of the use of new Uniform Hazard Spectra instead of the old spectral acceleration curves based on the ground motion bounds.

In order to assess the difference in design base shears obtained respectively from NBCC and UHS, base shear coefficients are derived for three multistorey

buildings, 3, 6 and 12 storeys high, located in four Canadian cities, Vancouver and Prince George in the West, and Montreal and Fredericton in the East. The four cities represent widely different levels of seismicity. Montreal and Vancouver are selected because they are large population centres. Prince George and Fredericton are chosen because in each case the current provisions of NBCC and the UHS yield significantly different values for the spectral accelerations, S_a . In Fredericton, UHS gives a spectral acceleration that is significantly higher (up to 107% higher) than that obtained from NBCC; for Prince George the reverse is true. The spectral values for these four cities are presented in Fig. 2.13. For the purpose of comparison, the spectra derived from NBCC are also shown. For the purpose of these calculations the fundamental period of each building is taken as being equal to $0.1N$, where N is the number of storeys.

As stated earlier, in NBCC the design base shear is obtained from

$$V = V_e \frac{U}{R} \quad (2.11)$$

where, V_e is the elastic base shear. The elastic base shear is thus reduced by two factors, R and U , to obtain the design base shear. The reduction factor R is a measure of the ductility capacity of the structure, and the calibration factor U accounts for the reserve strength or overstrength. In NBCC, $U = 0.6$ and R varies from 1 to 4.

The base shear coefficient derived from UHS according to the methodology proposed here is the product of Eq. 2.8 and the reserve strength factor U . It has

been assumed in the present discussion that factor U will remain the same as the NBCC value, that is 0.6. This factor and the concept of overstrength needs much additional study. Such a study is to be done in the present work for a variety of structural types.

The base shear coefficients for the three buildings and for each of the four cities are obtained from NBCC assuming $R = 4$. The shear coefficients obtained from inelastic UHS correspond to $\mu = 4$. The results are presented in Table 2.2. The large difference in the seismic risk implied by NBCC and that according to the new UHS is evident from the results.

2.5 IMPACT OF EARTHQUAKE FORCES IN DESIGN OF STEEL BUILDINGS

To study the impact of revisions in the design earthquake forces implied by the new UHS, the multistorey steel buildings referred to in the preceding section are designed for a combination of earthquake and gravity forces. Figure 2.14 shows a plan view of the buildings as well as the elevation of a typical multi-storey building. The bay spans are 8 m in both directions and all storeys have a height of 3.5 m. It is assumed that the lateral load resistance is provided by moment-resisting frames. In conformity with NBCC 95, the following load combinations are used

$$1.25D + 1.50L \quad (2.12a)$$

$$1.0D + 0.5L + 1.0E \quad (2.12b)$$

where D represents dead load, L is the live load, and E is the earthquake load. The dead load is assumed to be 3.40 kN/m^2 and the live load is taken as 2.4 kN/m^2 . A uniform reduction factor of 0.691 is applied to the live load for the design of both the beams and the columns.

In the design of low-rise buildings, if every frame in the N-S direction is designed to be moment-resisting, the earthquake forces in a frame will be comparatively small and design will be governed by the combination of dead and live load. In such a situation it is not necessary to design every frame to be moment-resisting and some of the frames could be of simple construction. The number of moment-resisting frames is chosen so that the combination of gravity and earthquake forces starts becoming critical.

The ratios of the total number of frames in the building to the number of frames that need to be moment-resisting are presented in Table 2.3. A ratio of say, 5.0 means that one moment-resisting frame has enough capacity to resist a combination of gravity loads tributary to it and lateral earthquake forces arising from the mass in five bays. Thus, only one out of five frames is, in this case, moment-resisting the other four being of simple construction. A ratio of 1 implies that all frames need to be designed to take the lateral forces due to earthquake and are therefore moment-resisting.

In designing the moment-resisting frames, $P - \Delta$ effect is included. Also, a strong-column-weak-beam principle is followed, so that at any joint the sum of

the column moment strengths is at least 1.2 times the sum of the beam moment strengths. A drift limit of 0.005 the storey height under the specified earthquake forces is enforced. Steel section design conforms to the provisions of CSA S16.1, 1994. The column and beam sections required to resist the calculated design moments, shears and axial forces are selected from the list of available sections given in a steel handbook.

The frames of simple construction are designed for the single combination of factored dead and live loads. The interior columns are assumed to carry only axial loads, while the exterior columns are designed for a combination of axial load and moment produced by the eccentricity of the beam column connection.

Figure 2.15 shows the moment resisting frames for a six storey building located in Vancouver. Two alternative designs are presented, one according to the NBCC and the other based on the inelastic UHS. Also shown is a typical frame of simple construction. Table 2.4 shows the average steel consumed per meter length of a building. In calculating the steel weights only the frames running in the N-S direction are included. Both simple and moment-resisting frames are accounted for, but no special allowance is made for the weight of the connections.

A comparison of the results presented in Tables 2.2 and 2.4 shows that even when the difference in base shear coefficients is quite high (up to 107%), the difference in the weight of the steel consumed is not that significant. This can be attributed to the fact that the gravity loads play an important role in determining

the final member sizes. Nevertheless, the use of the proposed UHS based design spectra may result in an increase in the steel consumption of up to 13% for a six storey building located in Vancouver. For a similar building located in Prince George, the use of UHS may lead to a decrease in steel quantity of the order of 8%.

2.6 SUMMARY AND CONCLUSION

The current Canadian and US practice in seismic design is to obtain the earthquake base shear from an elastic spectrum whose shape is related to the peak ground motion bounds for the site under consideration. Realizing that the determination of spectral shape by amplifying the peak ground motion bounds is subject to considerable error, new methodologies have been developed by other researchers that allow direct determination of the linear elastic spectra for a given site and a given probability of exceedance. Using the new methodology and additional geological evidence and records of ground motion collected since 1985, when the last seismic zoning maps of Canada were produced, the Geological Survey of Canada is developing new seismic hazard maps for Canada. These maps will provide spectral accelerations for a 5% damped elastic SDOF systems at several values of the period. This will allow the construction of elastic acceleration spectra commonly referred to as the Uniform Hazard Spectra.

A design methodology that allows the determination of design base shear from uniform hazard spectral values is presented in this chapter. The first step in the

development of the proposed methodology is the determination of elastic spectral curves for two sets of selected earthquake records having different a/v ratios, high and intermediate. The records with high a/v ratios are scaled to the maximum spectral acceleration at the site, while the intermediate a/v ratio records are scaled to the site spectral acceleration at a period of 0.5 s. The envelope of the two spectra is shown to closely match the UHS. The same suites of earthquakes are then used to produce the inelastic response spectra for different ductilities. Empirical expressions are developed to represent both the elastic and the inelastic spectra. The proposed expressions are related to just two ground motion parameters, the maximum spectral acceleration and the spectral acceleration at a period of 0.5 s. The empirically obtained elastic spectrum curves are compared with the UHS as well as the current NBCC seismic coefficient curves. A comparison is also made between the inelastic spectra derived by the methodology presented here and those derived by using the reduction factor. As would be expected, in the short period range, the methodology based on the use of inelastic spectra gives significantly higher design forces than those obtained by using a period independent value of R . The results presented here show that the use of inelastic spectral curves in place of the elastic curves along with a modification factor provides a more rational method of obtaining the design forces.

A series of multistorey steel buildings located in four cities across Canada are designed for combination of gravity and earthquake forces. The earthquake forces

determined according to NBCC are compared with those obtained from inelastic UHS. For the four cities studied, the difference between the two sets of forces varies from -71% to +107%. However, in the final design, the difference in the amount of steel consumed is not that significant, varying between -8% to +13%.

Table 2.1a : Uniform Hazard Spectral Accelerations in Units of g For Eastern Locations (Firm Ground),
84 th Percentile Values (Adams et al. 1996).

Location	Period						
	0.10 s	0.15 s	0.20 s	0.30 s	0.40 s	0.50 s	1.00 s
St. John's	0.086	0.120	0.160	0.170	0.160	0.150	0.084
Halifax	0.110	0.140	0.170	0.180	0.170	0.150	0.083
Moncton	0.170	0.220	0.240	0.220	0.200	0.180	0.095
Fredricton	0.210	0.270	0.280	0.260	0.250	0.220	0.110
La Malbaie	1.500	1.700	1.700	1.500	1.300	1.100	0.600
Quebec	0.320	0.360	0.400	0.390	0.360	0.320	0.170
Trois-Rivieres	0.410	0.450	0.480	0.410	0.360	0.320	0.160
Montreal	0.430	0.490	0.500	0.440	0.380	0.340	0.170
Ottawa	0.370	0.430	0.460	0.400	0.360	0.310	0.160
Niagara Falls	0.250	0.310	0.310	0.260	0.210	0.170	0.081
Toronto	0.160	0.120	0.210	0.180	0.150	0.130	0.063
Windsor	0.090	0.110	0.120	0.100	0.096	0.086	0.042

Table 2.1b : Uniform Hazard Spectral Accelerations in Units of g For Western Locations (Firm Ground),
84 th Percentile Values (Adams et al. 1996).

Location	Period									
	0.10 s	0.15 s	0.20 s	0.30 s	0.40 s	0.50 s	1.00 s	2.00 s		
Calgary	0.087	0.129	0.138	0.124	0.105	0.081	0.040	0.023		
Kelowna	0.203	0.280	0.314	0.258	0.216	0.171	0.098	0.055		
Kamloops	0.198	0.276	0.310	0.254	0.212	0.170	0.108	0.061		
Prince George	0.073	0.112	0.119	0.109	0.089	0.072	0.042	0.029		
Vancouver	0.685	0.866	1.044	0.759	0.633	0.531	0.298	0.151		
Victoria	0.854	1.025	1.219	0.830	0.681	0.564	0.310	0.151		
Tofino	0.389	0.517	0.575	0.503	0.436	0.373	0.211	0.120		
Prince Rupert	0.231	0.333	0.353	0.328	0.288	0.253	0.167	0.102		
Queen Charlotte	0.527	0.738	0.820	0.800	0.732	0.667	0.458	0.252		
Inuvik	0.063	0.101	0.111	0.106	0.091	0.077	0.045	0.030		

Table 2.2: Base shear coefficients derived from NBCC and the proposed UHS

No. of storeys	Fredricton			Montreal			Prince George			Vancouver		
	C_{NBCC}	C_{UHS}	Diff.	C_{NBCC}	C_{UHS}	Diff.	C_{NBCC}	C_{UHS}	Diff.	C_{NBCC}	C_{UHS}	Diff.
3 St.	0.028	0.049	75%	0.057	0.074	30%	0.032	0.016	-50%	0.085	0.117	37%
6 St.	0.015	0.031	107%	0.029	0.047	62%	0.029	0.010	-66%	0.058	0.074	28%
12 St.	0.010	0.020	100%	0.021	0.029	38%	0.021	0.006	-71%	0.041	0.047	15%

Table 2.3: Number of frames supported by one moment-resisting frame designed to resist the lateral forces of earthquake

No. of Storeys	Fredricton		Montreal		Prince George		Vancouver	
	NBCC	UHS	NBCC	UHS	NBCC	UHS	NBCC	UHS
3 St.	5.0	2.8	2.4	1.9	4.5	8.5	1.6	1.2
6 St.	3.5	1.8	1.9	1.15	1.9	5.5	1.0	1.0
12 St.	2.2	1.1	1.1	1.0	1.1	3.7	1.0	1.0

Table 2.4: The average weight of steel consumed in N-S frames per unit length of the buildings in E-W direction

No. of Storeys	Fredricton			Montreal			Prince George			Vancouver		
	NBCC	UHS	Diff.	NBCC	UHS	Diff.	NBCC	UHS	Diff.	NBCC	UHS	Diff.
3 St.	8.04	8.55	6.4%	8.74	9.10	4.1%	8.11	7.77	-4.2%	9.42	10.11	7.2%
6 St.	18.42	19.64	6.6%	19.51	21.06	8.0%	19.51	17.95	-8.0%	21.65	24.50	13.2%
12 St.	44.07	46.35	5.2%	46.35	47.93	3.4%	46.35	43.15	-6.9%	56.82	60.09	5.8%

(a)

(b)

Figure 2.1: Lateral force-displacement relationship for a single story frame

Fig. 2.2 Mean normalized strength demand spectra for different target ductilities and ground excitations with different a/v ratios

Fig. 2.3 Effect of a/v ratio of the records on the elastic and inelastic normalized strength spectra

Fig. 2.4 Coefficient of variation (COV) of normalized strength demands for different target ductilities and ground motions with different a/v ratios

Fig. 2.5 Mean displacement ratios for different target ductilities and ground excitations with different a/v ratios

Fig. 2.6 Mean strength reduction factors for different target ductilities and ground excitations with different a/v ratios

Figure 2.7: Effect of ductility ratio on the dispersion of strength reduction factors

Figure 2.8: Effect of a/v ratio of the records on the dispersion of strength reduction factors

Figure 2.9: Comparison of scaled mean elastic spectrum curves of high and intermediate a/v ratio records with Uniform Hazard spectra for (a) Vancouver and (b) Montreal

Fig. 2.10: Comparison of the proposed spectral curves with the response spectrum envelopes for Vancouver for two ductilities, (a) $\mu=1$ and (b) $\mu=4$.

Figure 2.11: Elastic and inelastic spectra for Vancouver

Figure 2.12: Elastic and inelastic spectra for Montreal

Figure 2.13: Comparison of base shear coefficient spectra for two cities of (a) Fredricton, and (b) Prince George

Figure 2.13: Comparison of base shear coefficient spectra for two cities of
(c) Montreal, and (d) Vancouver

Fig. 2.14: (a) Plan view of the buildings studied
(b) Elevation of a typical multi-storey frame

(a) : NBCC

(b) : UHS

(c) : Simple construction

All columns are W310

Fig. 2.15 : Comparison of the member sizes for a six storey building in Vancouver designed according to (a) earthquake and gravity, NBCC, (b) earthquake and gravity, the proposed UHS, and (c) gravity alone, simple construction

Chapter 3

ACCOUNTING FOR OVERSTRENGTH IN SEISMIC DESIGN

3.1 INTRODUCTION

Observations of structural performance under many past earthquakes have led to the conclusion that code designed buildings must possess significant overstrength in order for them to have survived without damage earthquake forces considerably larger than those considered in design. Many researchers have attempted to identify the factors that may have contributed to the observed overstrength (Fishingier and Fajfar, 1990; Miranda and Bertero, 1989; Mitchell and Paultre, 1994; Nassar and Krawinkler, 1991; Osteraas and Krawinkler, 1989; Tso and Naumoski, 1991; Zhu et al, 1991). Table 3.1 lists some of the sources of overstrength mentioned in the literature. These attempts have been useful in understanding the phenomenon of overstrength but may have also led to the belief that the identified sources of overstrength can be counted upon in designing new building structures. This belief is not justified in all cases and yet researchers are attempting to quantify the overstrength and are developing recommendations for its use in reducing the design seismic forces (Uang, 1992; Mitchell and Paultre, 1994). A critical examination

of the factors that contribute to the reserve strength (Table 3.1) is necessary to understand when and if a particular source of overstrength can be relied upon. For this purpose, it is useful to divide the contributing factors into several categories as outlined below.

3.2 FACTORS THAT AFFECT RESERVE STRENGTH

3.2.1 Factors that involve uncertainty

The factors included in this category may have been, at least in part, responsible for the observed overstrength and satisfactory performance during past earthquakes of buildings designed to meet the requirements of earthquake codes. However, the amount of contribution, if any, from these factors is not certain and can not be relied upon in the design of new building structures. The following is a partial list of factors that belong to this category

- (a) The difference between the actual strength of the material used in construction and the strength used in calculating the capacity.

Most material standards specify an acceptance criterion to ensure that the probability of the material strength falling below its nominal value used in design is reasonably small. The mean strength is in general higher than the nominal strength. Furthermore, in estimating the capacity of the structural member, codes using a limit states approach to design stipulate that a resistance

factor less than 1 be applied to the specified nominal strength. The magnitude of this factor depends on the variability of the material and the quality of control. The underlying philosophy is to minimize the risk of the member capacity falling below its estimated value. Evidently, in a majority of cases the actual capacity will be larger than its estimated value. This may be cited as one reason why structures have been able to sustain seismic loads significantly larger than those used in design. However, the excess capacity can not be used in design, for that will increase the risk of the capacity falling below that estimated and will be contrary to the philosophy of limit states design. If it is argued that a greater risk is acceptable in designing for earthquake forces than for say wind forces, the right thing to do would be to make the specified nominal value of strength closer to the mean and/or to use a resistance factor nearer to 1.

- (b) Effect of using discrete member sizes, for example, selection of members from a discrete list of available sections, and the use of limited bar sizes and arrangement in concrete structures.

It is true that the use of discrete member sizes will lead to a capacity that is higher than required. However, the amount of overstrength is quite uncertain and can not be relied upon in design.

3.2.2 Factors that can not be accounted for because of lack of knowledge

There are a number of factors that are known to contribute to strength but

which are difficult to quantify because of the complexity of the behaviour and/or lack of knowledge. The following can be cited as examples.

- (a) Use of conservative models for predicting member capacities.
- (b) Effect of nonstructural elements, such as for example, infill walls.
- (c) Effect of structural elements that are not included in the prediction of lateral load capacity, for example, contribution of reinforced concrete slabs, contribution of columns in flat plate structures with shear walls, increased resistance due to concrete confinement, and reduced stiffness due to concrete cracking.

3.2.3 Factors that can be but are not commonly accounted for in calculating the capacity.

In many cases the contribution from an identified source of additional strength can be estimated at the time of design. The correct approach in such a case would be to design for realistic loads instead of the reduced loads and at the same time to account for the contributing factors in estimating the capacity. The alternative procedure to ignore the contribution from the source of strength and to reduce the design load by an arbitrary factor which may have no relation to the magnitude of the strength contribution is not rational. The following are examples of factors that belong to this category.

- (a) Effect of minimum requirements prescribed by the code.

It is not necessary to reduce the design loads and then to implicitly rely on

the extra capacity resulting from the use of member sizes, reinforcement etc. dictated by the minimum requirements of the code. The use of realistic loads should not result in any change in member design as long as the capacity of the member is calculated on the basis of actual size and reinforcement provided.

- (b) Architectural considerations that dictate provision of extra or larger structural members, for example, shear walls.

Again, use of realistic loads instead of the reduced load will not alter the design as long as all structural members are included in calculating the capacity.

- (c) Control of design by other loading cases, for example, wind.

In this case too, the use of realistic earthquake forces will not change design.

3.2.4 Factors related to simplification in design procedure

Often, simplifying assumptions are made in design that lead to overestimation of strength demand or underestimation of capacity. This approach may be necessary for routine design where the extra effort required in obtaining more accurate estimates of demand and capacity is not justified. The following are some examples.

- (a) Use of single degree-of-freedom spectra along with assumed load distribution to estimate the demand on multi-degree-of-freedom systems.

While this may be a source of overstrength in some cases, in others it may lead to underestimation of strength demand. Additional research is needed to

address this issue.

(b) Redistribution of internal forces in the inelastic range.

In seismic design the capacity of the structure is usually determined at the first yield. This allows the use of analysis procedures that are applicable to elastic structures. Redistribution allows the structure to resist forces that are significantly higher than those causing first yield. Thus the lateral force at which a mechanism will form in a frame structure is often considerably higher than that at which the first plastic hinge will form. In a similar manner, the redistribution of lateral force from a compression brace to a tension brace allows a braced structure to carry significantly higher lateral load than at compression brace buckling.

3.3 ACCOUNTING FOR OVERSTRENGTH IN DESIGN

The discussion in the previous section indicates that in a rational method of design only the reliable sources of extra strength should be taken into account and that the contribution to strength from such sources should be used in estimating the capacity rather in reducing the expected demand. An argument for reducing the design load may perhaps be justified in accounting for the reserve strength attributable to redistribution of internal forces. Even there a more logical procedure would be to use the concept of limit design in which the load corresponding to the development of a mechanism or the attainment of a specified limit on drift is

explicitly calculated. Limit design, however, involves considerable complexity and may not be a practical procedure for the design of normal structures. Assessment of the difference between limit strength and the strength at first yield is thus of considerable interest.

As a simple example of the reserve strength attributable to redistribution of internal forces, consider the single storey moment frame shown in Fig. 3.1a. The frame is designed so as to remain elastic under the application of a lateral load V and a gravity load w per unit length on the beam. The columns are designed to have a strength that is higher than that of the beam, so that plastic hinges form only in the beam. It is also assumed that the moment-rotation relationship for a hinge is perfectly elasto-plastic. Now apply a lateral load to the frame increasing from zero as the gravity load remains constant. The relationship between the lateral force and the lateral displacement is plotted in Fig. 3.1b. For lateral force less than V , the structure is elastic. When it reaches V , a plastic hinge is formed at the right hand end of the beam. The slope of the force-displacement relation becomes flatter. When the lateral force reaches V_y , a second plastic hinge is formed in the beam. Depending on the ratio of lateral to gravity load, this hinge may form either at the left hand end of the beam or along the span. Here we will assume that the hinge forms at the left hand end. The frame now becomes a mechanism and can not take any additional lateral load. The ratio $R_d = V_y/V$ represents the reserve strength due to redistribution.

Assume now that if the frame were to have sufficient capacity so as to remain elastic, the design earthquake will induce in it a base shear V_e . Relying on the ductility of the frame, it is to be designed to have a reduced base shear strength $V_u = V_e/R$. Thus the design requirement is

$$V_y \geq V_u = \frac{V_e}{R} \quad (3.1)$$

If the reserve strength ratio, R_d is known or can be determined, the design requirement can be stated in the alternative form

$$V \geq \frac{V_u}{R_d} = \frac{V_e}{RR_d} \quad (3.2)$$

The use of Eq. 3.2 offers the advantage that only an elastic analysis needs to be carried out to determine the design forces in members of the frame. On the other hand, use of Eq. 3.1 requires a limit analysis.

3.4 ANALYTICAL STUDIES ON OVERSTRENGTH DUE TO REDISTRIBUTION

Reserve strength values associated with redistribution of internal forces strongly depend on the structural type, the load combinations, material properties, as well as the criteria used to determine when the ultimate lateral strength is reached. The ultimate strength criterion could be a storey drift limit, structural mechanism, or

local (member) ductility capacity. In this study the reserve strength due to redistribution is obtained analytically for four types of structural framing: (1) ductile steel moment-resisting frames, (2) ductile steel concentrically braced frames, (3) ductile concrete moment-resisting frames, and (4) ductile concrete flexural wall frames. These frames have 3 bays. It is expected that the results obtained for 3 bay frames will also be applicable for frames with a larger number of bays. The evaluation of reserve strength ratio R_d is based on static nonlinear push-over analysis in which the gravity loads are held constant while the earthquake forces are gradually increased until a mechanism forms or the specified limit on inter-storey drift is exceeded.

3.5 STEEL MOMENT-RESISTING FRAMES

In order to assess the range of reserve strength associated with redistribution that may be present in buildings with moment-resisting frames, a series of such buildings is analyzed for a combination of gravity and earthquake forces. Figure 3.2a shows a plan view of the buildings studied. Buildings with a height of 2, 5, 7, 10, 15, 20, and 30 storeys are analyzed. The lateral load resistance is provided by moment-resisting frames of steel. Design for gravity and earthquake forces in the North-South direction is carried out according to the provisions of NBCC. The value of zonal velocity ratio, v , is assumed to be 0.2. Load combinations given by Eq. 2.12 are used. The dead load is assumed to be 3.40 kN/m^2 and the live load is taken as 2.4 kN/m^2 . A uniform reduction factor of 0.691 is applied to the live

load for the design of both the beams and the columns. In the design of low-rise buildings, if every frame in the N-S direction is assumed to be moment-resisting, the earthquake forces will be small and design will be governed by the combination of dead and live load. In such a situation it is not necessary to design every frame to be moment-resisting and some of the frames could be of simple construction. The number of moment-resisting frames is chosen so that the combination of gravity and earthquake forces starts becoming critical. Thus, for a two storey frame only 1 in every 3 frames is designed to be moment-resisting; in a five storey frame 2 out of 3 frames are moment-resisting.

In designing the building frames a strong-column-weak-beam principle is used so that in a nonlinear analysis plastic hinges do not form in the columns, except at the base. Also, an elastic drift limit of 0.5% of the storey height under the specified earthquake forces is enforced. It may be noted that according to NBCC 95 the elastic storey drift limit is defined as $0.02/R$ where R is 4.0 for ductile moment-resisting frames, and 0.02 is the ultimate allowable inelastic storey drift. The column and beam sections required to resist the calculated design moments shears and axial forces are selected from the list of available sections given in a steel handbook. In order to discount the extra strength obtained by selection from a list of discrete member sizes, the values of the section strengths used in the subsequent analysis are assumed to be equal to the design moments, rather than the actual resisting moment of the section selected.

A typical frame of each building is now subjected to an incremental static push-over analysis for the gravity and earthquake forces tributary to it. The gravity loads are held constant at their full value implied in load combination given by Eq. 2.12b. The lateral earthquake forces are assumed to be distributed along the height according to the provisions of NBCC. The lateral forces are now increased in suitable increments until a mechanism forms, or an interstorey displacement goes past the design limit of 0.02 of the storey height. In the analysis it is assumed that the plastic hinges form at only the ends of the members. The moment-rotation relationship for a potential hinge is taken to be bilinear or elasto-plastic. The computer program used in the study is DRAIN-2DX (Prakash et al. 1993).

In the push-over analysis, the first set of plastic hinges forms at the design earthquake forces. If the frame under consideration is not subject to any gravity loads, these plastic hinges would be enough to create a mechanism, and there would be no reserve strength beyond the first yield. However, because the frames have been designed for a combination of gravity and earthquake forces and the beam and column sections are uniform along the member lengths, the first set of hinges is not enough to form a mechanism. The frame thus continues to resist additional lateral forces.

The complete base shear versus top floor lateral deflection for a 10 storey frame is shown in Fig. 3.3b. The base shear value at the point where any one of the interstorey drifts exceeds 0.02 of the storey height is also shown. This may be taken

as the ultimate strength of the frame. The ratio of the ultimate base shear to the shear at first yield is 1.75 in this case. The order in which the various plastic hinges are formed is indicated in Fig. 3.3a. It should be noted that hinges form also at the bases of the first storey columns. This is to be expected. A few hinges form also at other locations in the columns, even though the sum of the strengths of columns meeting at a joint are designed to be larger than the sum of the strengths of beams meeting the same joint. For the ten storey frame shown in Fig. 3.3a, the column hinges other than those at the base form after the ultimate shear has been reached and hence do not influence the reserve strength value calculated. Figures 3.3a and b clearly show the reserve strength beyond first yield that can be attributed to redistribution.

The calculated reserve strengths for all of the frames studied are shown in Fig. 3.4. The horizontal axis in the figure shows the number of storeys, or equivalently the period, taken to be equal to $0.1N$ as per the NBCC. The true calculated period is, in all cases, larger than this value. The ordinate of the graphs in Fig. 3.4 gives the ratio of the ultimate base shear strength V_y to the base shear at first yield V .

Four different curves are presented in Fig. 3.4, with and without $P - \Delta$ effect, and with and without strain hardening. The reserve strength becomes smaller when the $P - \Delta$ effect is taken into account. Similarly the strength is reduced when the effect of strain hardening is ignored and the moment-rotation relationship for plastic hinges is considered to be elasto-plastic.

In general, the reserve strength is higher for low rise buildings. However, the increase is not as significant as reported in the literature. This is so first because in the present study the reserve strength due only to redistribution is accounted for, and second because not all of the frames are designed to be effective in resisting the lateral forces.

For purpose of comparison, the reserve strength factor used in NBCC, $1/U = 1.67$ is also shown in Fig. 3.4. For the type of frames studied, this factor may be considered to provide a reasonable estimate of the reserve strength attributable to redistribution, except in the case of very tall buildings without any strain-hardening in the moment-rotation relationship.

As stated earlier, the magnitude of reserve strength depends on the relative values of the gravity and earthquake loads. The reserve strength decreases with an increase in the ratio of the earthquake base shear to the total gravity load. In the limit, if the lateral force resisting frame carries no gravity loads, the reserve strength is zero. The variation of reserve strength of a ten-storey frame with the ratio of earthquake base shear V to the total gravity load W used in the design load combination, Eq. 2.12b, is shown in Fig. 3.5. As would be expected, the reserve strength decreases with an increase in the ratio V/W .

3.6 CONCENTRICALLY BRACED FRAMES OF STEEL

Two important factors that control internal force redistribution in structures

are (i) the structural redundancy and (ii) local (or member) ductility capacities. Structures with sufficient redundancy and ductility capacity in their critical members permit redistribution of forces and possess desirable ductility. When compared to moment-resisting frames, concentrically braced frames have low redundancy. To ensure that they have adequate ductility capacity, the Canadian code for the design of steel structures, CAN/CSA-S16.1, clause 27 (CSA 1994), specifies that the members of braced frames satisfy the following requirements: (i) the slenderness ratio of bracing members, kl/r , where k is the effective length factor, l is the unsupported length, and r is the radius of gyration, should be less than or equal to $1900/\sqrt{F_y}$, where F_y is the yield strength of steel; (ii) the width to thickness ratio of bracing members, b/t , should be less than or equal to $330/\sqrt{F_y}$ (for hollow structural sections (HSS)); (iii) both the tension and compression braces should be able to carry at least 30% of shear in the storey; and (iv) the columns should not yield or buckle before all the braces do, i.e., the columns should never fail.

Redistribution in ductile concentrically braced frames (DCBFs) is achieved through buckling of the compression braces and yielding of the tension braces. An analysis of such frames therefore requires definition of the load-displacement relationship for a bracing member loaded in compression or tension. Following an extensive series of experimental tests, Jain (1978), and Jain and Goel (1978) introduced a model to simulate the buckling behaviour of a bracing member under cyclic axial loading. They proposed the hysteresis model shown in Fig. 3.6. According to

this model, after the first cycle of loading, the compressive buckling strength is reduced and becomes constant. CSA S16.1 Clause 27 accounts for this phenomenon. by specifying a reduction factor to be applied to compressive strength C_r to compute the buckling strength after the first cycle C'_r . During an earthquake, a braced member will undergo more than one cycle of deformation. In a push-over analysis it is therefore reasonable to assume that the design buckling strength is C'_r .

The reserve strength in DCBFs can be defined as the difference between the strength corresponding to the first buckling of any compression brace and the ultimate lateral strength of the structure. If the other sources of overstrength are ignored, the strength at the first brace buckling must coincide with the design strength of the structure specified by the design code. The usual criterion that determines the ultimate strength is either the formation of structural mechanism or the attainment of the specified storey drift limit (2% of storey height based on NBCC 95).

Two types of DCBFs are commonly used in buildings designed to resist seismic forces. These two types are shown in Figs. 3.7a and b, and will henceforth be referred to as DCBF Type I and DCBF Type II. In the Type I frame, shown in Fig. 3.7a, the braces are connected to the beam-column joints in each storey, hence the gravity loads are not directly carried by the braces. However, in the Type II frame, shown in Fig. 3.7b, the braces are connected to the middle of the beam at every other floor and thus some of the beam load goes directly to the braces. This

changes the characteristics of the structure in terms of redistribution of internal forces. It is shown in this study that the type of bracing has a significant effect on the redistribution of internal forces and hence on the reserve strength of the structure.

3.7 CONCENTRICALLY BRACED FRAMES TYPE I

As mentioned in the preceding section, the criterion that determines the ultimate strength is the occurrence of a structural mechanism or the attainment of storey drift limit, whichever comes first. In braced frames, storey drift limit usually does not control the ultimate strength because braced frames possess a relatively high lateral stiffness which prevents large lateral deformations in the structure.

For DCBFs the redistribution of internal forces in one storey does not affect the distribution of shear in other storeys. As soon as the tension brace in a storey yields, a mechanism is formed in that storey, it cannot take any further load and the structure reaches its ultimate capacity. Consequently, the formation of mechanism in one storey controls the global ultimate strength of the structure.

Figure 3.7a shows a single-bay, single-storey ductile concentrically braced frame. The tension and compression braces both have the same stiffness. Since the objective is to estimate the reserve strength that exists in buildings designed according to the code, member capacities are calculated as specified in the code. According to CSA, S16.1 the tensile yield strength T_r and compressive yield strength C_r are

given by the following expressions.

$$T_r = \phi A F_y \quad (3.3)$$

$$C_r = \phi A F_y B(\lambda) \quad (3.4)$$

where $\phi = 0.9$ is the resistance factor. A is the cross-sectional area of the brace. F_y is the yield strength of steel, and

$$B(\lambda) = (1 + \lambda^{2n})^{-\frac{1}{n}} \quad (3.5)$$

and

$$\lambda = \frac{kl}{r} \sqrt{\frac{F_y}{\pi^2 E}} \quad (3.6)$$

Also, $n = 1.34$ for W shapes, fabricated I-shapes, fabricated box shapes, and hollow structural sections manufactured according to CSA Standard G40.20 Class C (cold-formed non-stress-relieved); and $n = 2.24$ for WWF shapes, and hollow structural sections manufactured according to CSA Standard G40.20 Class H (hot-formed or cold formed and stress-relieved). In this study brace sections are assumed to belong to class H with $n = 2.24$. The buckling strength C'_r is derived by applying a reduction factor to C_r , so that

$$C'_r = \frac{C_r}{1 + 0.35\lambda} \quad (3.7)$$

Let the frame in Fig. 3.7a be subjected to a lateral force V , increasing monotonically from zero. A stage is reached where the compression brace buckles. At this stage the internal forces in both the tension and compression braces are equal to C'_r and the total storey shear strength is given by

$$V = 2C'_r \cos \theta \quad (3.8)$$

The ultimate strength is reached when the tension brace has yielded. The storey ultimate shear strength is given by

$$V_y = (C'_r + T_r) \cos \theta \quad (3.9)$$

The frame now develops a mechanism and cannot take any further load. The variation of storey shear with the storey drift is similar to that shown in Fig. 3.1b.

The storey reserve strength ratio R_d is given by

$$R_d = \frac{V_y}{V} = \frac{C'_r + T_r}{2C'_r} \quad (3.10)$$

On substituting for C'_r and T_r and canceling the common terms,

$$R_d = \frac{1 + 0.35\lambda + B(\lambda)}{2B(\lambda)} \quad (3.11)$$

According to Eq. 3.11 reserve strength ratio is independent of the angle of braces, the level of earthquake shear and the gravity load.

In deriving Eq. 3.11 the $P-\Delta$ effect has been ignored. The procedure for derivation of R_d when $P-\Delta$ effect is included is as follows. The effect of the movement of gravity loads through the storey drift caused by lateral loads can be represented by an extra shear in the storey. Since the storey shear strength is constant, this extra shear produced by $P-\Delta$ effect limits the lateral load carrying capacity of the frame. Let the storey shear strength be denoted by $\bar{V}$, total gravity load by C_f , storey height by H and the storey drift by Δ . The lateral force that produces a storey shear equal to $\bar{V}$ is then given by

$$V_f = \bar{V} - \frac{C_f \Delta}{H} \quad (3.12)$$

Let the frame in Fig. 3.7a be subjected to a total gravity load of C_f and let the lateral load V increase gradually from zero. At the stage when the compression brace buckles the lateral load carrying capacity is V_{fd} . At ultimate, corresponding to the yielding of tension brace, the lateral load is V_{fy} . The two are given by

$$V_{fd} = V - \frac{C_f \Delta}{H} \quad (3.13)$$

$$V_{fy} = V_y - \frac{C_f \Delta_y}{H} \quad (3.14)$$

where

$$\Delta = \frac{2C'_r H}{E A \sin(2\theta)} \quad (3.15)$$

$$\Delta_y = \frac{2T_r H}{E A \sin(2\theta)} \quad (3.16)$$

The reserve strength ratio R_d can now be derived as follows.

$$R_d = \frac{V_{fy}}{V_{fd}} \quad (3.17)$$

On using Eqs. 3.8, 3.9, 3.13, 3.14 and 3.17, R_d is given by

$$R_d = \frac{(C'_r + T_r) \cos \theta - \frac{C_f \Delta_y}{H}}{2C'_r \cos \theta - \frac{C_f \Delta}{H}} \quad (3.18)$$

On substituting for C'_r , T_r , Δ_y and Δ and canceling the common terms,

$$R_d = \frac{(1 + 0.35\lambda)(1 - R_f) + B(\lambda)}{(2 - R_f)B(\lambda)} \quad (3.19)$$

where

$$R_f = \frac{C_f}{E.A \sin \theta \cos^2 \theta} \quad (3.20)$$

The factor R_f which accounts for the $P - \Delta$ effect, increases with an increase in gravity load and a decrease in stiffness of the brace. This factor is usually small and decreases rapidly with a decrease in height and/or weight of the structure. This will be apparent from the results presented in Tables 3.2 and 3.3 for the frames studied in this work. The largest value of R_f , calculated for a 30 storey frame, is still less than 0.05. Further, because R_f appears in both the numerator and denominator, it has a very minor effect on the reserve strength ratio.

Figure 3.8 shows the variation of R_d with slenderness of braces in the critical storey. The results have been obtained from Eqs. 3.11 and 3.19. For the case when $P - \Delta$ effect is included, R_f is conservatively taken as 0.05. The results presented in Tables 3.2 and 3.3 and in Fig. 3.8 show that $P - \Delta$ effect does not have an appreciable impact on the reserve strength.

According to the Eqs. 3.11 and 3.19, the key parameter that controls the reserve strength in DCBFs is the slenderness ratio of the braces and, to a lesser extent, the gravity forces. The latter indirectly bring the building height and the effect of building sway ($P - \Delta$ effect) into consideration. In order to study the effect of these parameters and the manner in which redistribution of internal forces in DCBFs takes place, two types of studies are conducted: (1) study of the effect of building height and $P - \Delta$ effect on reserve strength ratio, (2) study of the effect

of level of earthquake force and the brace slenderness ratio on the reserve strength ratio. To carry out these two studies a series of multi-storey DCBFs are designed according to the procedures described in the following sections. Push-over analyses are then carried out on these frames to determine the reserve strength.

3.7.1 Effect of building height and building sway on the reserve strength ratio

The standard procedure for the design of DCBFs is to first design the structure for applied loads using the load combinations given in Eq. 2.12. Then, if required, the member sizes are modified to meet the storey drift limits. This part of design is called *strength design*. Subsequent to this the requirements of CSA S16.1 Clause 27 are applied to provide sufficient ductility capacity in the structure. These requirements usually control the final member sizes for both columns and braces. This part of design is called *ductility design*. The design method adopted for this part of study is as follows. In designing the braces, only the minimum requirements on the slenderness and width-thickness ratios required by CSA S16.1 Clause 27 are considered. The smallest HSS section that meets these requirements is HSS203x203x11. The columns are designed to carry the larger of the gravity load combination ($1.25D + 1.5L$), and the sum of vertical components of the yielding and buckling brace forces and the gravity loads ($D + 0.5L$). The second load combination usually dominates. The beams are designed as beam columns. The design moment is that produced by gravity loads, while the axial compression arises from

unequal capacity of braces in tension and compression.

The procedure of design as outlined in the preceding paragraph yields brace sizes that are the same throughout the height of the building. Also, if every frame is provided with bracing, the individual frames generally have significantly higher strength than that required to resist the earthquake forces tributary from one bay. Conversely, it can be stated that not every frame needs to be provided with braces, and each braced frame can be designed to support earthquake forces from more than one bay. The maximum number of bays, N , that can lean on the braced frame is obtained on dividing the lateral strength provided in the the braced frame by the storey shear arising from earthquake forces tributary from one bay. When the number of lean-to bays is selected to be equal to N , the strength and ductility designs yield identical sizes for braces in the critical storey (usually the first). If the number of lean-to bays is larger than N , strength design starts to govern in the first and at least some of the upper storeys. The braces sizes must then be increased beyond the minimum required for ductility design, and these sizes may now vary across the height of the building.

The buildings considered in this study are 2, 5, 7, 10, 15, 20, and 30 storeys in height. They are assumed to be located in seismic zone 4 where $v = 0.2m/s$. The plan view and the elevation of a typical frame are shown in Fig. 3.2c and d.

The earthquake base shear tributary from *one* bay is computed and distributed along the height of the braced frame according to NBCC 95. Program DRAIN-

2DX is used to conduct a static nonlinear push-over analysis of the frames. The earthquake force is increased in steps until the first brace buckles. The first buckling of course occurs in the first storey since the brace sizes are uniform across the height. The lateral load is further increased until the tension brace in the first storey yields and ultimate strength of the building is reached.

Table 3.2 lists for each structure the magnitude of the forces corresponding to first buckling and to the formation of mechanism, normalized by the design earthquake force tributary from one bay. Results are presented for two cases – with and without $P - \Delta$ effect considered. The reserve strength ratios R_d as obtained from the push-over analysis using DRAIN-2DX and as computed from the closed form solution given by Eqs. 3.11 and 3.19 are both shown in Table 3.2. Figure 3.9 illustrates the same results in graphical form.

The results presented in Table 3.2 show that in all of the buildings studied, the base shear force, V , corresponding to the first buckling is higher than the design base shear for tributary of only one bay. For example, for a 10 storey building, the normalized value of V with $P - \Delta$ effect taken into account is 3.23. In other words, for this building one braced frame can support earthquake forces tributary from slightly over three bays.

From Fig. 3.9 it is observed that there is an almost constant difference between the results of DRAIN-2DX program and the closed form solutions. This can be attributed to the fact that in the derivation of Eqs. 3.11 and 3.19 the axial loads

produced in the braces by gravity loads are ignored while the DRAIN-2DX analysis includes them. In reality braces do participate in carrying the gravity loads. The existence of a gravity load in compression brace consumes part of the compressive capacity of the brace and causes the brace to buckle earlier than if there were no compressive gravity load in it. On the other hand, gravity loads produce a compressive force in the tension brace; as a result yielding of the tension brace is delayed. The reduction in the buckling strength capacity of the compression brace and the increase in the capacity of the tension brace increases the calculated reserve strength. In fact, when the same frames were analyzed by program DRAIN-2DX with the gravity loads removed, the calculated reserve strength values were identical to those given by Eqs. 3.11 and 3.19.

Referring to Fig. 3.9 and the results of push-over analysis, the following conclusions can be drawn.

- 1) Reserve strength ratio is independent of the height of the structure.
- 2) $P - \Delta$ effect has a negligible effect on the reserve strength ratio.
- 3) Use of the reserve strength ratios given by Eqs. 3.11 and 3.19 is conservative.

3.7.2 Effect of level of earthquake force and brace slenderness ratio on the reserve strength ratio

A 10 storey building is designed for gravity loads and three different design earthquake forces corresponding to 3, 6 and 9 times that tributary from one bay.

For the first case the ductility design requirements control the column and brace sizes, although the earthquake force is close to governing the design, specially in the first storey. In the other two cases, the strength required to resist earthquake forces controls the design except in the top storeys where these forces are low. Table 3.3 shows the results of push-over analyses for the three designs obtained both by using DRAIN-2DX and from the closed form solutions given by Eqs. 3.11 and 3.19.

Figure 3.10 shows the variation of reserve strength ratio R_d with the slenderness ratio of the braces in the critical storey which is found to be the first storey. Since in the three 10 storey frames under consideration, the earthquake forces govern the design of braces at least in the first story, the base shear and the slenderness ratio of the braces in the first story are interrelated. In other words, a higher base shear results in a smaller slenderness ratio. The earthquake base shear normalized by the weight of the structure ($D + 0.5L$) is also shown on the horizontal axis in Fig. 3.10. The following conclusions can be drawn from these results.

- 1) The reserve strength ratio increases with an increase in brace slenderness, or correspondingly a decrease in earthquake base shear.
- 2) $P - \Delta$ does not have any significant effect on the reserve strength ratio. In fact, in Fig. 3.10, the plots for the two sets of values, with and without $P - \Delta$ effect, are almost indistinguishable.
- 3) When the participation of braces in carrying the gravity loads is discounted, a simple closed form solution gives the true reserve strength. Even when such

participation is accounted for, the closed form solution provides a reasonable and conservative estimate of the reserve strength.

3.8 CONCENTRICALLY BRACED FRAMES OF TYPE II

Figure 3.7b shows a two storey frame of Type II. The lateral loads shown in that figure produce tension in braces 1 and 4 and compression in braces 2 and 3. It is assumed that these four braces have the same stiffnesses. Let this frame be subjected to lateral forces at each floor. A portion of the gravity load, W , acts at the point where the braces intersect the beam in the first storey. Let the two lateral loads increase in the same proportion monotonically from zero while the gravity loads remain constant. First, brace 2 buckles under compression. At this stage the structure becomes determinate. As the lateral load is increased further, one of braces 1 and 4 yields in tension. This leads immediately to the formation of a mechanism and the frame cannot take any further load. The lateral load-displacement relationship for this two-storey frame is also trilinear, and is similar to that in Fig. 3.1b. The base shear at the instant when brace 2 buckles is V while the base shear when a mechanism is formed is V_y .

Three 10 storey buildings are designed using bracing of type II. For a meaningful comparison of the computed reserve strength ratios with those in Type I frames, presented in the preceding section, the following design criterion is adopted. Since the slenderness ratio of the braces is a factor that controls the reserve strength of

the structure, and since it is expected that the reserve strength in a critical storey represents also the global reserve strength, the bracing members in each storey of the 10 storey frames are selected to have the same slenderness ratios as those of the three 10 storey Type I frames studied in the previous section. The criteria used in the design of beams and columns are similar to the those for Type I frames. This procedure permits one to study the effect of the type of bracing on the manner in which the redistribution takes place.

Table 3.4 shows the results of the push-over analyses for the three 10 storey frames of Type II. The results include the base shears corresponding to first buckling and to the formation of mechanism, as well as the reserve strength ratios for two cases – with and without $P - \Delta$ effect included. Program DRAIN-2DX is used to perform the analyses. The base shears corresponding to first buckling for these frames are lower in comparison to those for Type I frames shown in Table 3.3. It should be noted that while the slenderness ratio is the same in both types of frames, the brace length in Type II frames is shorter, hence a smaller cross-section area needs to be provided. This leads to lower buckling and tension yielding resistances for the braces and, therefore, to lower storey shear resistances. Figure 3.11 shows complete results of a push-over analysis for two 10 storey buildings one with frames of Type I and the second with frames of Type II. The 10 storey Type I frame was designed to carry earthquake forces from 6 bays; the Type II frame had braces with the same slenderness ratios as Type I frame. Figure 3.11a illustrates the sequence

in which buckling and yielding of braces occurs in the frame of type I. Figure 3.11b shows similar results for the frame of type II. Figure 3.11c shows the variation of the total base shear with the roof displacement for the two frames. Type I and II.

In Fig. 3.12 the reserve strength ratios are plotted against the slenderness ratio of the first storey braces in the 10 storey frames of types I and II. For a given slenderness ratio, the reserve strength ratio for Type II frames is higher than that for Type I frame. This is mainly because of the gravity load W that acts at the intersection of the braces and the beams. This load advances the buckling of the compression brace and delays the yielding of the tension brace. For both type of frames, the reserve strength ratio increases with slenderness ratio, and the effect of $P - \Delta$ is negligible.

3.9 CHARACTERISTICS OF THE CONCRETE STRUCTURES STUDIED

In this study the reserve strength due to redistribution is obtained analytically for two types of structural framing: (1) ductile concrete moment-resisting frames, and (2) ductile concrete flexural wall frames. A series of buildings of the two types are studied. Figures 3.13a and b show plan views of the buildings. Figure 3.13c shows a typical moment-resisting frame, while Fig. 3.13d illustrates a typical wall-frame. The buildings are designed for force levels given in the provisions of NBCC 95. The design and detailing of concrete sections follow the provisions of CSA

Standard A23.3-94 (CSA A23.3) (Canadian Standards, 1994) corresponding to the level of ductility desired. The buildings are assumed to be located in Zone 4, a region of active seismicity. For this region, the specified peak horizontal ground velocity, having a probability of exceedance of 10% in 50 years, is 0.2 m/s.

The specified yield strength of steel is taken as 400 MPa and the specified 28-day concrete strength as 30 MPa. For the design of concrete sections, no redistribution is assumed to occur in the bending moments caused by gravity loads acting alone. The floor slabs are taken as being 100 mm in thickness and the two secondary beams supporting the floors are 300 mm wide and 350 mm deep.

3.10 CONCRETE DUCTILE MOMENT-RESISTING FRAMES

In order to assess the range of reserve strength associated with redistribution that may be present in buildings with moment-resisting frames, a series of such buildings is analyzed for a combination of gravity and earthquake forces. Buildings with heights of 2, 5, 7, 10, 15, 20, and 30 storeys are studied.

3.10.1 Design Process

Design for gravity and earthquake forces in the North-South direction is carried out according to the provisions of NBCC 95 which require that the load combinations given in Eq. 2.12 be considered. The dead load is assumed to be 5.72 kN/m^2 , and the live load is taken as 2.4 kN/m^2 . A uniform reduction factor of 0.691 is

applied to the live load for the design of both the beams and the columns. In the design of low-rise buildings, if every frame in the N-S direction is assumed to be moment-resisting, the earthquake forces will be small and design will be governed by the combination of dead and live load. In such a situation, it is not necessary to design every frame to be moment-resisting and some of the frames could be as lean-to frames. The number of moment-resisting frames is chosen so that the combination of gravity and earthquake forces starts becoming critical. Thus, for a two storey building, only 1 in every 2 frames is designed to be moment-resisting. For buildings of other heights, each frame is designed to be moment-resisting.

To account for the effect of cracking in the calculation of load effects, the sectional properties for beams and columns are adjusted in accordance with the provisions of CSA A23.3. The adjusted values are: $A_c = A_{gc}$, $A_b = A_{gb}$, $I_c = 0.70I_{gc}$, $I_b = 0.35I_{gb}$, where A_c is the effective cross-section area of a column, A_b that of a beam, I_c is the effective moment of inertia of a column, I_b that of a beam, and subscript g indicates gross value. In modelling the structure, a rigid zone of width 300 mm is assumed at each end of both the beams and columns except at the base of the columns in the first storey. Using some trial sizes and the load effects computed, the steel reinforcement required for the elements is determined according to CSA A23.3. Table 3.5 presents the final sizes for beams and columns.

The NBCC 95 requires that the extra moments and shears generated by the $P - \Delta$ effect be computed corresponding to the inelastic storey drifts. The NBCC 95

also specifies that the inelastic drifts can be estimated by taking the product of the elastic drifts and the force modification factor R , which for concrete moment-resisting frames is 4.0. On the other hand, the design forces must be derived using an elastic analysis. One possible way to meet these requirements is to reduce the stiffness of the frame by R (applying a factor of $1/R$ to the stiffnesses of all the beams and columns) and to then perform a second order analysis under the load combination given by Eq. 2.12b. This procedure is used in the current study.

To check the storey drifts under the specified earthquake loads, as required by NBCC, a first order analysis would be sufficient. The specified drift limit is in this case $0.02/R$ times the storey height.

The column moment capacities must be re-examined to check whether the strong column weak beam condition required by CSA A23.3 is satisfied. CSA A23.3 specifies that at every joint

$$\sum M_{rc} \geq 1.1 \sum M_{nb} \quad (3.21)$$

where M_{rc} is the factored moment resistance of the columns framing into the joint and M_{nb} is the nominal moment resistance of the beams framing into the same joint. As an approximation, CSA A23.3 allows that the nominal moment capacity may be assessed from the factored moment capacity by using the equation

$$M_{nb} \approx 1.2M_{rb} \quad (3.22)$$

When required, the reinforcement and/or the size of the columns are modified to satisfy the condition given in Eq. 3.21.

3.10.2 Push-over analysis

In the push-over analysis, plastic hinges are assumed to form at the ends of the members. The moment-rotation relationship for the hinges is taken as being bilinear with a post-yield stiffness equal to 5% of the initial elastic stiffness.

To discount the effect of sources of overstrength other than the internal force redistribution, the negative moment capacities of the beams are set to be equal to the moments used in their design even though the actual capacities may be higher on account of the minimum requirements and discrete reinforcement sizes. As specified by CSA A23.3, the positive moment capacity of a beam is designed to be 50% of its negative moment capacity.

Formation of the plastic hinges at the base of the columns is inevitable and thus is accepted by CSA. However, necessary measures must be taken to ensure the required ductility for these regions. The P-M interaction curves for the columns are derived using the design charts given by CSA A23.3.

The frames are now subjected to the gravity loads produced by $D + 0.5L$. Lateral earthquake forces, distributed along the height according to NBCC 95, are

then applied and increased monotonically from zero. A second order analysis is carried out so that the $P - \Delta$ effect is accounted for.

The first hinging is expected to happen at a lateral base shear that is slightly higher than the code specified design shear. This is because the design forces have been amplified corresponding to a drift of 4 times the elastic drift. The lateral loads are increased until the ultimate strength is reached. The ultimate strength corresponds to either the attainment of drift limit in any storey of 0.02 or the formation of a structural mechanism, whichever happens first. The R_d factor which is a measure of the available reserve strength in the structure due to redistribution, is the ratio of the ultimate base shear to the base shear at the formation of the first plastic hinge.

The results of push over analyses are presented in Table 3.6. The lateral shears are normalized with respect to design earthquake shear tributary to one frame. According to the results presented in Table 3.6, structural mechanism controls the ultimate capacity of tall buildings (15 storey and higher). In low rise buildings, (10 storey and lower), however, the drift limit determines the ultimate strength of the structure.

The calculated reserve strengths are also shown in Fig. 3.14. The horizontal axis in the figure shows the number of storeys, or equivalently the period, taken to be equal to $0.1V$ as per the NBCC. The true calculated period is, in all cases, larger than this value. The ordinate of the graphs in Fig. 3.14 gives the ratio of

the ultimate base shear strength V_y to the base shear at first yield V . In general, the reserve strength is higher for low rise buildings. However, the increase is not as significant as reported in the literature. This is so because in the present study the reserve strength due only to redistribution is accounted for.

For purpose of comparison, the reserve strength factor used in NBCC, $1/\bar{U} = 1.67$ is also shown in Fig. 3.14. For the type of frames studied, this factor may be considered to provide a reasonable estimate of the reserve strength attributable to redistribution, except in the case of very tall buildings. It should be noted that for moment-resisting frames the reserve strength ratios due to redistribution could be even less than the values presented here if advantage is taken in the design of the 20% redistribution of moments in continuous beams permitted by the CSA A23.3.

As indicated in the preceding section and shown in Table 3.6, in the 2 storey building only 1 out of every 2 frames is moment-resisting, and the others are just lean-to frames. However, for structures with higher number of storeys every frame must be moment-resisting. This, of course, depends on the relative magnitudes of earthquake to gravity loads, that is on, the seismic zone in which the buildings are located. In this study, Zone 4 is used which is relatively active seismic zone. The peak ground velocity v in this zone is 0.2 m/s. If the ground velocity used for design were smaller, the number of lean-to frames relative to the lateral load resisting frames would have increased.

In concrete building structures the lean-to frames that are designed for gravity

loads alone may also contribute to the resistance against the seismic loads. This is because the gravity loads used in combination with earthquake force (Eq. 2.12b) are less than those used in the design of beams and columns of the lean-to frames (Eq. 2.12a). As mentioned earlier, the resistance provided by the lean-to frames will depend on their number relative to the number of lateral load resisting frames. In this study, for the seismic load level used, there are no lean-to frames except for the 2 storey building and therefore the calculated reserve strength ratios are purely due to redistribution of the internal forces in the moment-resisting frames.

3.11 DUCTILE FLEXURAL WALL STRUCTURES

A pure flexural wall subjected to distributed lateral loads tends to act as a cantilever, in which the maximum moment occurs at the base. Therefore, in a design based on CSA A23.3, the first plastic hinge is expected to form at the base. Internal load redistribution in such a structure does not occur because as soon as the first hinge is formed the wall fails to carry any extra lateral load, except when post-yield strain hardening is considered in the plastic hinges. The effect of the latter is not usually very significant. In order to benefit from the structural action of the wall even after first yield, structures are built by combining the walls with moment resisting frames, either as lean-to frames or in the form of beam and column elements in the frame containing the wall.

In the wall-frame structures, it is mainly the flexural wall that resists the seismic

forces and dissipates energy through flexural yielding at one or more plastic hinges. The beams and columns in the lean-to frames and in the same frame as the wall are designed to resist only the factored gravity loads given by Eq. 2.12a and to have sufficient ductility to sustain the deformations of the structure during an earthquake.

As indicated in the foregoing section, since the gravity loads used in combination with earthquake forces (Eq. 2.12b) are smaller than those used in the design of beams and columns in lean-to frames (Eq. 2.12a), these elements have spare capacity when the building is subjected to a load combination that includes earthquake forces. The flexural wall building structures have a relatively large number of lean-to frames. Because of their spare capacity, the lean to frames will contribute to the reserve strength in the building. Therefore, the assessment of a unique and representative reserve strength factor for concrete buildings with wall structures is not possible.

To illustrate the foregoing statement, a 5 storey wall structure with 0, 3, 6, and 12 lean-to frames is analyzed in this study. The reserve strength ratio here is defined as the ratio of the ultimate base shear to the design base shear corresponding to the formation of first plastic hinge in the wall. The ultimate strength corresponding to failure is conservatively taken to occur when a plastic hinge has formed in any beam or column following the yielding of wall. No further redistribution is assumed possible, because the beams and columns have been designed to possess

only nominal ductility.

3.11.1 Design process

The web thickness of the wall, b_w , is taken as 350 mm (1/10 of the wall height, a minimum requirement by CSA A23.3) and assumed to be uniform across the height and width of the wall. The $P - \Delta$ effect is not considered in the design. This is appropriate because the lateral stiffness of the wall is relatively large, and the $P - \Delta$ effect is negligible.

In modelling the structure for analysis and design, a rigid zone of 300 mm is considered at the ends of all beams, columns and walls, except at the bases of the columns and walls in the first storey. To account for concrete cracking, a reduced moment of inertia, as specified by CSA A23.3, is used for the beams, columns, and the wall. Thus $I_b = 0.35I_{gb}$, $I_c = 0.70I_{gc}$, and $I_w = 0.70I_{gw}$, where I_w is the moment of inertia of a wall. The effect of elastic shear deformations in the wall is included in the analysis and design.

3.11.2 Push-over analysis

For the purpose of push-over analysis, the moment capacity for plastic hinges at the ends of beams and columns are taken to be equal to the moments imposed by gravity load combination in Eq. 2.12a. The P-M interaction relationship for the wall is taken according to the provisions of CSA A23.3.

Gravity loads specified in load combination given by Eq. 2.12b. ($D + 0.5L$), are first applied and then kept constant throughout the push over analysis. Lateral earthquake loads distributed in accordance with NBCC 95 are then applied. Their magnitude is increased monotonically from zero until the ultimate strength is reached. The ultimate strength is equal to the base shear corresponding either to the attainment of a storey drift limit or to the formation of a plastic hinge in any beam or column following yielding of the wall. The reserve strength ratio is defined as the ratio of the ultimate base shear to the design base shear which corresponds to the formation of first plastic hinge in the wall.

Table 3.7 shows the results of push-over analyses of 5-storey flexural wall frame structures with different number of lean-to frames. For the wall structure with 0 and 3 lean-to frames the first yield in the wall does not coincide with the design earthquake loads because of the minimum requirements specified in the design of the wall. However for the wall structure with 6 and 12 lean-to frames the design load almost equals the base shear at the first yield in the wall. Figure 3.15 also presents the reserve strength ratio for different number of lean-to frames. Two main conclusions can be drawn: 1) reserve strength ratio is strongly dependent on the number of lean-to frames; and 2) that as the number of lean-to frames increases the reserve strength ratio decreases.

3.12 SUMMARY AND CONCLUSIONS

Experience of satisfactory performance during past earthquakes shows that buildings designed according to current seismic codes must possess considerable reserve strength. Some of the sources of such overstrength are uncertain, others, although reliable, are best taken into account in design in assessing the capacity of the structure rather than in scaling down the design forces. One exception is the reserve strength owing to the redistribution of internal forces. Scaling down of design forces to account for this source of reserve strength simplifies the analysis and is therefore useful. In order to assess the extent of reserve strength attributable to redistribution, different building structures: steel ductile moment-resisting frames, steel ductile concentrically braced frames, concrete ductile moment-resisting frames and concrete ductile flexural wall frames are analyzed for their response to lateral loads.

For buildings with moment-resisting frames of both steel and concrete, reserve strength attributable to redistribution may vary considerably depending on the methodology used in design, the ratio of earthquake to gravity loads affecting design, the number of storeys, and the $P - \Delta$ effect. However, the reserve strength factor of 1.67 implied in the NBCC may be considered as being a reasonable estimate for the purpose of design.

Two types of steel concentrically braced frames are studied. The main parameter that controls the reserve strength in these frames is the slenderness ratio of

the bracing members. The reserve strength of the critical storey is also the global reserve strength of the frame. The reserve strength is independent of the height of the frame and the effect of building sway. The reserve strength in frames having concentric braces which intersect the beams at mid span may be higher than in frames that have concentric braces connected only to the beam-column joints. Reserve strength increases with an increase in the brace slenderness ratio or a decrease in the design earthquake load.

The effect of lean-to frames on the reserve strength in concrete buildings with both moment-resisting and wall structures are discussed. The contribution of the lean-to frames depends on their number relative to the lateral load resisting frames. For moment-resisting frame buildings in high seismic zones, the number of lean-to frames is small and almost all of the frames are designed to carry the lateral loads due to earthquake. In wall structures, however, the number of lean-to frames may be large. Hence the estimation of a unique and representative reserve strength ratio is impractical. The NBCC implied value of 1.67 for reserve strength ratio may be conservative for this type of structures.

The estimates of reserve strength presented in this study are applicable only to the type of structures studied. Care should be taken in extrapolating the results to other cases. Additional studies are needed to assess the reserve strength in structural systems that are different.

Table 3.1 : Some sources of overstrength given in the literature

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1. Effects of discrete member sizes
 2. Effects of underestimating member strength capacities in the design process (e.g., conservative models for predicting member strength, actual vs. nominal material strength properties)
 3. Effects of code minimum sizes and requirements
 4. Effects of stiffness (drift) requirements on member strength
 5. Effects of desired uniformity of members for constructability
 6. Architectural considerations (e.g., excess shear wall area)
 7. Effects of structural elements that are not considered as part of the lateral load resisting system (e.g., columns in flat plate structures with shear walls)
 8. Effects of non-structural elements
 9. Code-calculated period and related base shear
 10. Importance of building
 11. Reduced stiffness due to cracking
 12. Assumed lateral load distribution
 13. Slab participation in reinforced concrete structures
 14. Design controlled by other loading case (e.g., wind)
 15. Effects of capacity design philosophy
 16. Effects of other loads in load combinations (e.g., effects of gravity loads on the required member strength)
 17. Redistribution of internal forces in the inelastic range (e.g., strength difference between the formation of the first plastic hinge and a mechanism in frame structures, difference between strength of braces in tension and in compression)
 18. Increased resistance due to confinement in reinforced concrete elements
-

Table 3.2: The reserve strength ratios in concentrically braced frames Type I, obtained from a push-over analysis using program DRAIN-2DX and from a closed form solution

No. of Storeys	DRAIN-2DX							Closed Form Solution						
	P-Δ included				P-Δ ignored			kl/r	EA (MN)	C _f (MN)	R _f	R _d		
	V	V _y	R _d	V	V _y	R _d	P-Δ included					P-Δ ignored		
30	1.830	3.781	2.07	1.899	4.006	2.11	109.7	1646	24.36	0.044	1.970	1.992		
20	2.272	4.744	2.09	2.312	4.907	2.12	109.7	1646	16.24	0.029	1.978	1.992		
15	2.628	5.551	2.11	2.662	5.666	2.13	109.7	1646	12.18	0.022	1.981	1.992		
10	3.226	6.860	2.13	3.253	6.939	2.13	109.7	1646	8.121	0.015	1.985	1.992		
7	3.888	8.225	2.12	3.911	8.294	2.12	109.7	1646	5.684	0.010	1.987	1.992		
5	4.651	9.852	2.12	4.670	9.913	2.12	109.7	1646	4.060	0.007	1.989	1.992		
2	8.134	17.30	2.13	8.147	17.35	2.13	109.7	1646	1.624	0.003	1.991	1.992		

Table 3.3: The reserve strength ratios for three 10-storey buildings with concentric braces (Type I) computed using program DRAIN-2DX and Eqs. 3.11 and 3.19.

Earthquake load tributary of	DRAIN-2DX						Closed Form Solution				
	P- Δ included			P- Δ ignored			Brace				
	V	V_y	R_d	V	V_y	R_d	section in 1st st.	kl/r	R_f	P- Δ included	P- Δ ignored
3 bays	3.226	6.860	2.13	3.253	6.939	2.13	HSS203x11	109.7	0.015	1.985	1.992
6 bays	7.100	11.07	1.56	7.144	11.13	1.56	HSS254x13	87.19	0.010	1.511	1.514
9 bays	9.665	14.69	1.52	9.726	14.73	1.52	HSS305x13	72.12	0.008	1.305	1.306

Table 3.4: The reserve strength ratios for three 10-storey building with concentric braces (Type II) computed using program DRAIN-2DX.

Design case	P- Δ included			P- Δ ignored			Brace section in 1st st.	kl/r
	V	V_y	R_d	V	V_y	R_d		
I	0.647	2.285	3.54	0.656	2.354	3.59	HSS127x8	107.5
II	1.448	3.167	2.19	1.465	3.241	2.21	HSS152x8	88.36
III	2.392	4.464	1.87	2.427	4.570	1.88	HSS178x9.5	75.88

Table 3.5 : Member sizes for 2, 5, 7, 10, 15, 20, and 30 storey concrete moment-resisting frames

Storey #	Beams (mm x mm)	Columns (mm x mm)
1 - 6	750 x 400	700 x 700
7 - 12	700 x 400	650 x 650
13 - 18	650 x 400	600 x 600
19 - 24	600 x 400	550 x 550
25 - 30	600 x 400	500 x 500

Table 3.6 : Results of Push-over analyses and the reserve strength ratios for concrete moment-resisting frames (5% strain hardening and P - Δ effect included)

Number of storeys	$V_{1st\ yield}$	$V_{ultimate}$	Type of failure	R_d
2	2.07	4.65	Storey drift	2.25
5	1.09	2.25	Storey drift	2.06
7	1.10	2.04	Storey drift	1.86
10	1.10	1.79	Storey drift	1.63
15	1.14	1.69	Mechanism	1.48
20	1.15	1.69	Mechanism	1.47
30	1.17	1.67	Mechanism	1.42

Table 3.7 : Normalized base shears corresponding to the first yielding in the wall and formation of the first hinge in a beam

	Number of lean-to frames			
	0	3	6	12
A_s^* (mm ²)	5.85×10^3	5.85×10^3	7.31×10^3	1.55×10^4
width of edge column (m)	0.35	0.35	0.35	0.85
Base shear at first yield in wall	4.89	1.46	1.04	1.06
Base shear at first yield in a beam	18.40	5.97	4.05	2.96
R_d	3.76	4.09	3.89	2.79

* A_s is the area of reinforcement in each edge column.

Figure 3.1 : Load-displacement relationship for a single storey frame

Figure 3.2 : Multi-storey steel structures (a) plan view for moment resisting frames, (b) elevation of a typical moment-resisting frame, (c) plan view for braced frames, and (d) elevation of a typical braced frame

Fig. 3.3: Results of push-over analysis for a 10 storey frame, P- Δ included, strain hardening modulus = 0; (a) sequence of formation of hinges; (b) base shear versus roof displacement

Figure 3.4 : Reserve strength values for buildings of different heights

Figure 3.5 : Effect of ratio of earthquake force to gravity force on the reserve strength of a 10 storey building, P- Δ effect included, 0% strain hardening

Figure 3.6 : Hysteretic behaviour of bracing members of steel based on a model proposed by Jain and Goel (1978)

Figure 3.7 : (a) Single storey braced frame Type I, (b) two storey braced frame Type II

Figure 3.8 : Closed form solution for variation of reserve strength ratio with the brace slenderness ratio

Figure 3.9 : Variation of reserve strength ratio with the height of the structure in DCBFs

Figure 3.10 : Variation of reserve strength ratio for a 10 storey braced frame Type I with the slenderness ratio of the brace in the first storey or the level of earthquake design force

Figure 3.11 : Results of push-over analyses for two 10 storey frames, Type I and II. P- Δ effect included, (a) sequence in buckling and yielding of the braces in frame Type I, (b) sequence in buckling and yielding of the braces in frame Type II, (c) base shear (normalized with design shear for one frame) against roof displacement

Figure 3.12 : Effect of the type of bracing on the reserve strength ratio for a 10 storey building (slenderness ratio in this figure corresponds to the braces in the first storey)

Figure 3.13 : (a) Plan view of a building with concrete moment-resisting frames, (b) plan view of a building with concrete wall frames, (c) elevation of a typical concrete moment-resisting frame (d) elevation of a typical concrete wall frame structure

Figure 3.14 : Reserve strength ratios for concrete moment-resisting frames

Figure 3.15 : Reserve strength ratios for a 5 storey concrete flexural wall structure with different number of lean-to frames

Chapter 4

DUCTILITY AND STRENGTH DEMANDS IN MULTI-STOREY BUILDINGS

4.1 INTRODUCTION

The issue of seismic demand for an inelastic SDOF system has been dealt with in Chapter 2. That information is relevant for an assessment of the damage potential of a ground motion, but needs to be modified to become of direct use in the design of real structures, which are mostly multi-degree-of-freedom (MDOF) systems. For *elastic* MDOF systems, the combination of modal responses using SRSS, CQC, or other approaches, provides reasonable estimates of peak dynamic response characteristics. For *inelastic* MDOF systems, modal superposition cannot be applied. Therefore, other techniques have to be employed in order to predict the strength and ductility demands for use in design. An approach for estimating the strength and ductility demands of inelastic MDOF systems is presented in this chapter.

Storey ductility ratio, defined as the ratio of maximum dynamic interstorey displacement to the interstorey yield displacement, is used here as the deformation parameter. The strength demand associated with a specific storey ductility ratio,

therefore, becomes the basic design parameter. The objective of the proposed approach is to design structures with sufficient strength to limit the storey ductility ratios to a target value.

In studying the seismic demand parameters of MDOF systems, it is useful to compare the strength requirement for a multistorey building with that for an associated SDOF system. The associated SDOF system is defined as one having, in its linear range, the same period and damping as the multistorey building. The weight of the associated SDOF system is equal to the total weight of the multistorey building. The ductility demand in a multistorey building having the same strength (base shear capacity) as the associated SDOF system can be quite different from that in the SDOF system (Nassar and Krawinkler, 1991). In order to limit the ductility demand in the MDOF system to that of its associated SDOF system (target ductility) the strength demand for SDOF systems must be modified before it can be used for an MDOF system. The ratio of the strength demand of an MDOF system to that of its associated SDOF system for identical target ductility is referred to as the strength ratio or strength modification factor.

The inelastic strength demands for SDOF systems can be evaluated by the procedure described in Chapter 2. The question to be answered is how do the ductility demands of MDOF systems compare to the SDOF target ductility ratio μ_t , if the MDOF base shear capacities are identical to the inelastic strength demands of their associated SDOF systems. Or more relevant for design, the question becomes:

how different should the base shear strength of the MDOF system be (assuming the code prescribed seismic load pattern over the height of the structure) compared to that of the corresponding SDOF system, in order to limit the maximum storey ductility ratio in the MDOF system to the prescribed target ductility ratio μ_t (see Fig. 4.1).

The ductility and strength demands in inelastic multi-storey models must, in principle, strongly depend on the characteristics of the lateral load resistance system, the characteristics of the earthquake ground motion, the lateral distribution of the storey stiffnesses and strengths, structural system redundancy, modes of failure of structural elements, etc. In the present work, multi-storey building frames with a number of structural systems are studied to arrive at representative values of the strength modification factors.

4.2 UHS COMPATIBLE GROUND MOTIONS USED IN THIS STUDY

Atkinson and Beresnev (1998) have provided simulated ground motion time histories that are compatible with uniform hazard spectra (UHS) produced by the Geological Survey of Canada. These simulated records provide a realistic representation of the ground motion for the earthquake magnitudes and distances that contribute most strongly to hazard at the selected cities and probability level.

The Canadian code committees are currently considering the adoption of earth-

quakes with a recurrence interval of 2500 years as the collapse-level design earthquake. The UHS to be used in design should therefore be derived for an earthquake hazard corresponding to a 2500 year earthquake (having an annual exceedance probability of $1/2500$ or 2% in 50 years).

In the current Canadian building code the design earthquake has a recurrence interval of 500 years or 10% probability of exceedance in 50 years. This rather large probability is justified by the observation that structures designed for such an earthquake in fact possess considerable reserve strength and are therefore able to sustain a significantly higher level of earthquake. The magnitude of this higher earthquake, however, depends on the profile of the seismic hazard.

In Canada, the hazard profile is quite different between the East and the West. This will be clear from the plots of peak spectral acceleration (PSA) versus recurrence interval of earthquake shown in Fig. 4.2 (Adam, 1997). The plots in Fig. 4.2 are for an SDOF elastic system having a period of 0.2 s and a damping that is 5% of critical. Two curves are shown, one for Montreal, representing the eastern locations, and the other for Vancouver, typical for the Western part of the country. It will be observed that for eastern locations the PSA, and hence the required strength, varies significantly more rapidly with the recurrence interval than for western locations. This implies that, of the two structures, one in the West and the other in the East, both designed according to the code for a 500 year earthquake and having the same reserve strength, the one in the West will be able to sustain an earthquake with

considerably longer recurrence interval than the one in the East. For example, if the actual strength of a structure is assumed to be 1.67 times the design strength, a structure in the West designed for an earthquake with recurrence interval of 500 years will in fact be able to sustain an earthquake with a recurrence interval of 2500 years. On the other hand, a similar structure located in the East will sustain an earthquake with recurrence interval of about 1200 years. If a consistent level of safety is to be ensured, the structures in both the East and the West should be designed for an earthquake with a recurrence interval of say 2500 years. Then, if the reserve strength is to be accounted for in design, the design shear obtained from the 2500 year earthquake may be reduced by the reserve strength factor.

In this study, the acceleration time histories produced by Atkinson and Beresnev (1998) are used. These records simulate the elastic UHS (with a 2500 year recurrence earthquake hazard level) for selected cities in both the West and the East. One set of records for Vancouver and Victoria in the West and another set of records for Montreal, Ottawa, and Quebec city in the East are considered. In each set there are four records, two long predominant period earthquakes to cover the long period range of UHS, and two short predominant period earthquakes to cover the short period range of the UHS. Table 4.1 shows the characteristics of the records used. Figures 4.3 and 4.4 depict variation of the ground acceleration with time for these records for the West and the East. Figures 4.5 and 4.6 show the inelastic SDOF response spectra for these records for four ductilities of 1, 2, 3 and

4. for the West and the East, respectively. The hysteresis model used to generate these spectra is elasto-plastic with no strain hardening. Figures 4.7 and 4.8 show the elastic SDOF response spectra of the records for two sets of locations along with the corresponding UHS2500.

It can be seen from the Figs. 4.5 to 4.8 that the long predominant period earthquakes tend to dominate the long period range of the UHS while the short predominant period earthquakes cover the short period range of the UHS. It is worthwhile noting that a short period earthquake usually has a short source distance to the site with a small magnitude while a long period earthquake occurs far away from the site and has a moderate to large magnitude.

4.3 MULTI-STOREY MODELS STUDIED

The research presented in this section is intended to provide some of the answers needed to assess the ductility and strength demands in inelastic multi-storey models and to compare these values to those for the associated SDOF systems. The focus is on evaluation of regular structural systems. Realizing that no two structures are the same, and that the dynamic behaviour of real structures depends on so many parameters, it was decided to focus on simplified multistorey models in order to gain an insight into basic inelastic dynamic behaviour patterns. Therefore, regular 2-dimensional single-bay frames with widely spaced elastic modal periods are utilized. The response of symmetrical one-bay frames is considered to provide satisfactory

approximation to the response of actual multi-bay frames subjected to dynamic or static loads (Council on Tall Buildings and Urban Habitat, 1979). The torsional effects of three-dimensional structures are neglected.

A comprehensive study on the ductility and strength demands in different multi-storey structural models is carried out in the present work. Three distinct structural models are considered: namely, moment-resisting frames, flexural walls, and concentrically braced frames. Figure 4.9 illustrates, for each structural model studied, a typical multi-storey frame.

4.4 GENERAL ASSUMPTIONS MADE IN DESIGN AND ANALYSES OF THE STRUCTURAL MODELS

The following characteristics are assumed for the multi-storey building models studied.

- (1) The storey height is the same and is equal to 3.5 m.
- (2) The bay size is equal to 8.0 m.
- (3) For each structural model, four structural heights, namely, 5, 10, 20, and 30 storeys, are considered.
- (4) The mass at each floor is the same. This mass is computed based on a tributary area of 8x8 m. A dead load of 3.4 kN/m^2 with a live load of 2.4 kN/m^2 , and a live load reduction factor of 0.691 are used. The load combination given in

Eq. 2.12b is utilized to derive the weight at each floor, and the mass is calculated from that weight.

- (5) Even though the NBCC 95 uses different equations for estimating the natural periods of different structures, period is calculated here from equation, $T = 0.1N$ sec, for all the models studied. N is the number of storeys.
- (6) The distribution of earthquake lateral loads along the height of the structure in a static analysis is based on NBCC 95.
- (7) In a dynamic analysis, a Rayleigh damping is used and the damping value is selected to be 5% of critical for the first two modes.
- (8) Axial deformations in the beams and columns in the moment-resisting frames, and in flexural walls are neglected.
- (9) Local (member level) inelastic deformations, in moment-resisting frames as well as flexural walls, are concentrated in plastic hinges at the ends of the elements.
- (10) The plastic hinges, in moment-resisting frames and flexural walls, have an elasto-plastic force displacement or moment-rotation relationship with zero strain hardening.
- (11) The effect of gravity loads is not considered in assessing the ductility demands.
- (12) The $P - \Delta$ effect is not considered.

The last three assumptions are justified by the observation that the contribution

of these factors to the final design has already been considered in deriving the reserve strength factor, R_d , which was studied in Chapter 3 for different structural types. Here, the main focus is on a study of the effects of the higher structural modes on the ductility and strength demands in MDOF systems with different structural types.

4.5 CHARACTERISTICS OF MOMENT-RESISTING FRAMES

Moment-resisting frames could be of steel or reinforced concrete. In practice, the principal steps in the design of these two types of the moment-resisting frames are basically the same. The design loads are derived using the load combinations given by NBCC. Then, the requirements of the relevant material code are satisfied. Finally, the storey drifts are calculated and if they exceed the limits set by NBCC, the design is appropriately modified.

In general, the moments produced at the ends of the beam, by the applied loads, control the design. If only the lateral loads are present, negative and positive moments of equal magnitudes are produced at each end of the beams, and the support sections of the beam are designed to have equal positive and negative moment capacities. However, if gravity loads are also present, as is usually the case, these two applied moments are quite different, the negative moment being generally larger than the positive moment. In a steel moment-resisting frame, a uniform section is generally used throughout the span of the beam. Thus, the negative

and positive moment capacities are the same even when the applied moments are different. In a reinforced concrete structure, even when a uniform section is used for the beam, its capacity can be adjusted by varying the amount of reinforcement. Thus, the positive moment capacity provided is generally less than the negative moment capacity. Since a reduction in the positive moment depends on the presence of gravity moment and the magnitude of such loads is variable, the code specifies that a minimum positive moment capacity of 50% of the negative moment must be provided.

In the present study, the gravity loads are ignored for the reasons stated in the preceding section. Thus the positive and negative applied moments and hence the moment capacities in the positive and negative moment regions are taken to be the same, irrespective of whether the structure is of steel or concrete. The same set of analytical studies for determining the strength demands are thus applicable to multi-storey frames of both steel and concrete.

4.5.1 Member Stiffnesses

The elastic member stiffnesses in each storey of the frames studied are selected so that, under the 1995 NBCC equivalent static load pattern, the interstorey drift in every storey is identical. The ratio of the beam stiffness to the sum of the column stiffnesses in each storey is set to be $1/8$. For structural geometries used here, this implies a frame with columns that are stiffer than the beams, typical of

earthquake-resistant construction. The condition of identical storey drift and the selected value of the beam to column stiffness ratio allow the determination of the relative stiffnesses of the elements. The absolute values of the stiffnesses are now adjusted so that the first mode period of the structure is equal to that given by the NBCC recommended expression $T = 0.1N$ seconds, where N is the number of storeys. Because of the way in which the stiffnesses are selected, the first mode shape is close to a straight line. The periods and modal masses for the first five modes of various multi-storey frames are presented in Table 4.2a.

4.5.2 Member Strengths

The method used for selecting the member strengths is the same as that used in the study by Nassar and Krawinkler (1991). The models are designed to develop the *structural mechanism* shown in Fig. 4.10a under the 1995 NBCC equivalent static lateral load pattern. In other words, member strengths are tuned so that all of the shown plastic hinges develop simultaneously under this specific lateral load pattern. The moment capacities at plastic hinges are thus selected to be equal to the design moments at the corresponding member ends. The concept of strong-column-weak-beam is followed so that plastic hinges will form in beams only (as well as at base supports). A bilinear moment-rotation ($M - \theta$) hysteresis model with zero strain hardening ratio is assumed for each plastic hinge. Therefore, under an incremental lateral load pattern, each structure has a bilinear response

similar to that of a bilinear SDOF system. At lower values of the lateral loads, the relationships between the applied load and storey displacements are linear. As the lateral load is increased, the storey displacements approach yield level, and at a certain value of the load all storeys yield simultaneously. The storey displacements at this stage are considered as being the yield displacements.

4.6 CHARACTERISTICS OF FLEXURAL WALLS

4.6.1 Member Stiffnesses

The flexural walls used here have a uniform stiffness along the height. Once the mass at each floor is known, the stiffness of the wall can be determined by setting the first mode period to $T = 0.1.N$ seconds, where N is the number of storeys. The modal periods and modal masses can now be calculated. They are presented in Table 4.2b for the first five modes of different multi-storey walls studied.

4.6.2 Member Strengths

The member strengths are selected in a manner similar to that used for the moment-resisting frames. The models are designed to develop *structural mechanism* shown in Fig. 4.10b under the 1995 NBCC equivalent static lateral load pattern. In other words, member strengths are tuned so that all of the shown plastic hinges develop simultaneously under this specific lateral load pattern. The moment capacities at plastic hinges are thus determined as being equal to the design

moments at the corresponding member ends. A bilinear moment-rotation ($M - \theta$) hysteresis model with zero strain hardening ratio is assumed at each plastic hinge location. Thus for each storey, the yield displacement is the displacement when this simultaneous yielding takes place in all the storeys.

4.7 CHARACTERISTICS OF BRACED FRAMES

4.7.1 Member Stiffnesses

In this study a concentrically braced frame is modelled as a simple truss. First, for a given base shear the storey shears are computed using the lateral load distribution specified by NBCC 95. Then, using these storey shears, the internal forces in the braces can be determined by assuming a rigid body movement at the floors and equal stiffnesses for compression and tension braces in a storey. The sectional area of the braces for any given storey is now calculated by setting the storey drift to $0.02/R$ (as per NBCC 95). For this part of analysis, the columns are taken as being axially rigid. It was found in this study that the axial deformation of the columns does not have a significant effect on the lateral storey drifts, most of which is provided by the deformation of braces. Following the derivation of the brace forces, the axial forces in the columns can also be computed by solving the truss starting from the top storey. The columns are designed for these forces by using Eqs. 3.4, 3.5 and 3.6. Since the floor masses are known, the first mode period can now be calculated. The preliminary member stiffnesses are modified (increased) so

that the first mode period becomes $T = 0.1V$. The modal characteristics for the first five modes of the braced frames studied are presented in Table 4.2c.

4.7.2 Member Strengths

As in the case of moment-resisting frames and flexural walls, the member strengths are selected so that all of the braces either buckle in compression or yield in tension simultaneously when subjected to the 1995 NBCC equivalent static lateral load pattern (see *structural mechanism* shown in Fig. 4.10c). In other words, the axial buckling strength and tensile yielding strength of the braces are selected so that all the braces buckle in compression or yield in tension under this specific lateral load pattern. This means that in all storeys each brace is assumed to have equal buckling strength C_r and tensile strength T_r . This assumption provides simplification in the analyses and can be justified as follows. As reasoned in the previous sections, any factor that is already accounted for in the derivation of reserve strength ratio studied in Chapter 3 may be discounted here. This is because the intent here is to capture the effect of higher structural modes on the inelastic demand parameters, and the contribution due to the difference between C_r and T_r in final design base shear is already accounted for in R_d factor.

The hysteresis model developed by Jain and Goel, previously discussed in Chapter 3 and shown in Fig. 3.6, is adopted in the dynamic analyses. According to that model, following one cycle of motion, the buckling strength reduces to C'_r given in

Eq. 3.7. Since the earthquakes usually impose more than one cycle of movement in the buildings, the compressive strength of the brace at each storey is considered to be equal to this reduced buckling strength, C'_r . Because of the method of tuning strength so that there is a simultaneous buckling and tensile yielding of all the braces under the 1995 NBCC equivalent static lateral load pattern, C'_r for each brace is taken to be equal to T_r and equal to the design axial load in the brace. The yield displacement for each storey is thus the displacement at the instant all of the braces are either buckling or yielding in tension.

4.7.3 Storey Target Ductility

One of the main purposes of this study is to evaluate the strength that must be provided in a multi-storey model so as to limit the storey ductility demands throughout the height to a target value. In general, a single storey frame, with a natural period of T , that has a strength equal to that provided in the associated SDOF system corresponding to a target ductility of μ_t when subjected to a given earthquake motion, must yield a storey ductility demand equal to μ_t when subjected to the same motion. However, this is not the case for a single storey braced frame. Although, such a frame in its elastic range is identical to a SDOF system, the storey ductility demand in it is usually different from the target value. There are two reasons for this difference. First, while the hysteresis model used to derive the SDOF response spectra given in Figs. 4.5 and 4.6 is elasto-plastic (bilinear),

the hysteresis model used for a bracing member, as shown in Fig. 3.6, is totally different from a bilinear model. Second, although in its elastic range a single storey braced frame behaves as a SDOF system, its behaviour changes when it goes in inelastic range. This is because the frame has two braces, one in compression and the other in tension, and the hysteretic behaviour of the two is quite different as seen in Fig. 3.6. The behaviour of the single storey frame is a composite of the behaviour of the two braces and is even more complex. Such a problem is not encountered in walls or moment resisting frames, first, because the only inelastic deformation allowed is in moment plastic hinges formed at the end of the members, and second, because the hysteretic model at the moment plastic hinges is bilinear, similar to that of the associated SDOF.

Two different strategies can be used to address the anomaly associated with a braced frame.

- (1) Create a new set of constant-ductility SDOF response spectra using the hysteretic model that simulates inelastic response of a single storey braced frame, for all the ground motions used in this study.
- (2) Using the inelastic SDOF response spectra already derived for an elasto-plastic (bilinear) SDOF systems for the specified target ductility ratio, compute the storey ductility for a single storey braced frame. This storey ductility can now be considered as the storey target ductility for the braced frames. This storey target ductility would, of course, be different for different periods, different

earthquake records, and different target ductilities of the associated bilinear SDOF systems.

The advantage of the first option is that the storey ductility is actually limited to a prescribed constant target value, for example, $\mu_t = 3.0$. The disadvantage is that a constant ductility spectra derived in this way may not yield a strength reduction factor R equal to μ_t , an assumption implied in the provisions of most modern seismic design codes. When the second option is used, a bilinear SDOF system does in fact satisfy this requirement at least approximately (refer to Chapter 2). The second option is therefore selected for this study, even though it may be inconvenient to deal with a variable storey target ductility.

As implied in the second option, the target ductility for a braced frame as derived from the analysis of an associated single storey frame will change depending on the period, earthquake record, and the target ductility ratio for the associated SDOF system having a bilinear force-displacement relationship. Tables 4.3 and 4.4 illustrate the storey target ductility ratios derived for different periods and records, for two associated bilinear SDOF target ductility of $\mu_t = 2.0$ and 3.0 , respectively. According to the results obtained in this study (Tables 4.3 and 4.4), there is no consistent relationship between the ductility demand computed for such a single storey braced frame and the target ductility of its associated SDOF system having a bilinear hysteretic behaviour.

4.8 STOREY DUCTILITY DEMANDS

If an MDOF system were to behave exactly like its associated SDOF system when the two systems are subjected to the same ground motion, all resulting interstorey ductility ratios in MDOF system would be equal to the SDOF target ductility ratio μ_t . However, due to the participation of higher modes in MDOF systems, this is usually not the case. This section addresses the question of how do the storey ductility ratios in an MDOF system having the same strength as its associated SDOF system compare to the target ductility ratio of the associated SDOF system. The storey ductility ratios along the height of a multi-storey frame subjected to a given earthquake motion are computed as follows. For the given period of T and structural type, the member stiffnesses of the MDOF system are derived following the simplified methods outlined in the previous sections. Next, the spectral acceleration S_a , corresponding to a given target ductility, produced in the associated SDOF system is calculated. The product of the total mass of the multi-storey frame (MDOF system) and this S_a provides the base shear strength of multi-storey frame. This base shear is used to determine the strengths of those members of the MDOF system that are expected to yield, following the method described earlier. A dynamic time history analysis of the MDOF system is then carried out for the selected ground motion to determine the maximum interstorey drifts. These drifts are considered as being the maximum inelastic storey drifts. To calculate the storey ductility, the yield storey displacements are also required. For

each storey, the storey yield displacement can be determined by the same analysis that provided the member strengths. The analysis procedure, in fact, consists of a static analysis of the structure for the lateral loads obtained by distributing the base shear according to the code provisions. Because of the manner in which the member strengths are selected, all storeys yield at the same time. The storey ductility experienced during the earthquake motion then becomes the ratio of the maximum dynamic inelastic storey displacement to the yield storey displacement computed as above.

4.8.1 Storey ductility demands in moment-resisting frames

To assess the storey ductility demands in moment-resisting frames, a total of 64 nonlinear dynamic time history analyses are performed for the following combinations: 2 locations, West and East; a total of 4 ground motion records for each location; 4 structure models of 5, 10, 20 and 30 storeys; 2 SDOF target ductility ratios of $\mu_t = 2.0$ and 4.0. A target ductility of 4.0 represents a ductile behaviour for moment-resisting frames and is in conformance with its NBCC 95 strength reduction factor of $R = 4.0$. A target ductility of 2.0 represents a nominally ductile behaviour for moment-resisting frames and corresponds to $R = 2.0$. Program DRAIN-2DX (Prakash et al., 1993) is used to carry out the dynamic analysis.

As illustrations of the data obtained, Figs. 4.11a and b show the height wise variation of storey ductility demands for a 20 storey moment-resisting frame when

subjected to the long period Trial 1 record for the West, for the target ductility ratios of $\mu_t = 2.0$ and 4.0 , respectively. As can be seen from Fig. 4.11, the first storey has the highest ductility demand. Also, relative to other storeys, the storey ductility tends to be higher in the upper storeys (although not as high as that in the lower storeys). This can be attributed to the effects of higher modes. One of the reasons why the ductility demand in the first storey is so different from that of the other storeys may be as follows. NBCC 95 and many other codes specify in their method of lateral load distribution the application of a portion of base shear, V , as a concentrated force F_t at the top floor in excess of the height wise triangular distribution of $(V - F_t)$. This force F_t is equal to $0.07TV$ and could go up to 25% of the total base shear. Application of this force causes the higher storeys to have a reserve of strength compared to the first storey. This is because application of F_t does not change the strength of the first storey, while increases significantly that of the upper storeys.

Figures 4.12a and b show the variation of the first storey ductility demands plotted against the first mode period for a target ductility of $\mu_t = 4.0$ for the records in the West and the East, respectively. The horizontal dashed lines represent the SDOF target ductility ratio for each case. Figure 4.13a and b illustrates the same results for a target ductility of $\mu_t = 2.0$. The results presented in Figs. 4.11, 4.12 and 4.13 are discussed in Section 4.8.4.

4.8.2 Storey ductility demands in flexural walls

To study the storey ductility demands in flexural walls a total of 64 nonlinear dynamic time history analyses are performed for the following combinations: 2 locations, West and East; a total of 4 ground motion records for each location; 4 structure models of 5, 10, 20 and 30 storeys; 2 SDOF target ductility of $\mu_t = 2.0$ and 4.0. Target ductility of 4.0 is used to represent the behaviour of ductile flexural walls and 2.0 to study the response of the nominally ductile flexural walls. NBCC 95 recommends strength reduction factors of $R = 4.0$ and 2.0 respectively, for these two types of structure. Program DRAIN-2DX (Prakash et al., 1993) is used to carry out the dynamic analysis.

Figures 4.14a and b show the height wise variation of storey ductility demands for a 20 storey flexural wall when subjected to the long period Trial 1 record for the West, for the two target ductility ratios of $\mu_t = 2.0$ and 4.0, respectively. The results show that, as in the case of the moment-resisting frames, the lower storeys have the highest ductility demand. However, unlike the moment-resisting frames, the storey ductility tends to decrease monotonically with height.

Figures 4.15a and b show the variation of the first storey ductility demands plotted against the first mode period for a target ductility of $\mu_t = 4.0$ and for the records in the West and the East, respectively. The horizontal dashed lines represent the SDOF target ductility ratio for each case. Figure 4.16a and b illustrates the same results, this time for a target ductility of $\mu_t = 2.0$. A complete discussion of

the results presented in Figs. 4.14, 4.15 and 4.16 is provided in Section 4.8.4.

4.8.3 Storey ductility demands in concentrically braced frames

The height wise variation of the storey ductility demands in braced frames are examined, on the basis of the results obtained from a total of 64 nonlinear dynamic time history analyses for the following combinations: 2 locations, West and East; a total of 4 ground motion records for each location; 4 structure models of 5, 10, 20 and 30 storeys; 2 different strength levels corresponding to an associated bilinear SDOF system with $\mu_t = 2.0$ and 3.0. The target ductility ratios for a single storey braced frame that correspond to the target ductilities of $\mu_t = 2.0$ and 3.0 in the associated bilinear SDOF system are given in Tables 4.3 and 4.4 for a range periods and different earthquake ground motions. A target ductility of 3.0 in the bilinear system is used to represent the behaviour of ductile concentrically braced frames for which the NBCC 95 specifies a strength reduction factor of $R = 3.0$, while, $\mu_t = R = 2.0$ is considered to represent the response of the nominally ductile braced frames.

Program SNAP-2D (Firmansjah, J.; Goel, S. C.; and Rai, D. C. : 1996) is used to carry out the dynamic analysis. This program is equipped with the brace hysteresis model described in Chapter 3. It may be noted that DRAIN-2DX does not provide such a hysteresis model.

As an example, Figs. 4.17a and b show the height wise variation of storey

ductility demands for a 20 storey braced frame when subjected to the long period Trial 1 record for the West. The storey target ductility ratios for these two particular cases are 3.0 and 1.92, respectively. It should be noted that the strengths supplied for these two frames are in fact those of the associated SDOF systems having target ductilities of $\mu_t = 3.0$ and 2.0, respectively. As can be seen from Fig. 4.17 the first storey has the highest ductility demand as was the case for moment-resisting and flexural wall models. However, unlike the moment-resisting frames where the storey ductility tends to be increased in the upper storeys, it now decreases monotonically with height. This behaviour is similar to that for flexural walls.

Figures 4.18a and b show the variation of the first storey ductility demands with the first mode period. The ground motions used are simulated records for the West and the East. The data obtained is for the case where the associated bilinear SDOF system has a target ductility of $\mu_t = 3.0$. Figures 4.19a and b illustrates the same results, this time for an associated bilinear SDOF system having a target ductility of $\mu_t = 2.0$. The results are further discussed in the next section.

4.8.4 Discussion on the storey ductility demands

The following conclusions can be drawn from the results presented in Figs. 4.11, 4.14, 4.17 and Table 4.2.

- 1) Distribution of base shear strength over the height in conformance with the NBCC 95, does not lead to equal ductility demand in all storeys.

- 2) The ductility demand in the lower storeys is the highest among all storeys.

Because of this fact the global ductility of the structure is taken to be equal to the first storey ductility demand in the present study.

- 3) Figure 4.11 shows that in moment-resisting frames the storey ductility demand tends to increase in the upper storeys. This phenomenon has been attributed to the effect of higher structural modes (Nassar and Krawinkler, 1991). However, it is not observed in flexural walls or braced frames (Figs. 4.14 and 4.17). It is apparent that factors other than the effect of higher modes contribute to the variation of storey ductility along the height. These factors are the level of redundancy in the structure, and the manner in which the loads are redistributed in the inelastic range. If the predominant factor were only the effect of higher structural modes, both the flexural walls and the braced frames should show a higher second mode contribution than a moment-resisting frame.

The foregoing observation can be justified as follows. During an earthquake, while members in the lower storeys of a moment-resisting frame are experiencing yielding, the high level of redundancy allows a redistribution of the internal loads or the earthquake energy throughout the height of the structure including the upper storeys. In a braced frame or a flexural wall, the redundancy is low, and as soon as the members (particularly those in the first storey) experience yielding, the distribution of the internal loads becomes significantly limited and the loads experienced by the upper storeys do not increase beyond those

existing prior to the occurrence of yielding in the lower storeys. Consequently, even though the effect of the higher elastic modes is greater in these two types of structures, the lack of redistribution of internal loads does not allow the higher modes to contribute to the upper storey shears and displacements.

The following conclusions can be drawn from the results presented in Figs. 4.12, 4.13, 4.15, 4.16, 4.18, and 4.19.

- 4) With a few exceptions, the storey ductility increases monotonically with an increase in the first mode period.
- 5) By comparing Figs. 4.13 with 4.12, Figs. 4.16 with 4.15 and Figs. 4.19 with 4.18, it can be seen that the lower the strength provided (or the higher the target ductility) is, the higher the storey ductility becomes.
- 6) The variation of storey ductility strongly depends on the seismicity of the site in which the building is located. Usually in the east of Canada storey ductility demands are higher than those in the West. This difference tends to increase in the higher period range.
- 7) In the West, as expected, long period records control the maximum ductility demands in the long period range and the short period records dominate the short period range (e.g., in the vicinity of $T = 0.5s$). However, in the East, this is not the case. Almost always, the ductility demand in the entire period range is dominated by the short period records.

- 8) The horizontal dashed line in the results presented for moment-resisting frames and flexural walls represents the target ductility. For the cases where the ductility demands are below this line the strength modification factor will become less than unity, whereas for the cases where the ductility demands are above this line the strength modification factor will be more than one. Storey target ductility ratios for braced frames for all the cases studied here are given in Tables 4.3 and 4.4.

4.9 BASE SHEAR DEMANDS

The results presented so far, clearly show that the ductility demands for MDOF systems differ significantly from those of the associated SDOF systems. The implication is that the base shear strength capacities of the MDOF systems must be modified compared to the inelastic strength demands of the equivalent SDOF systems, in order to limit the storey ductility demand to the desired target ductility values (see Fig. 4.1). The following discussion provides a systematic approach to achieving this objective.

It may be possible to achieve the target ductilities by redistribution of the associated SDOF strength across the height of the multi-storey building, i.e., by modifying the strength of selected storeys, most likely each by a different factor. Such a procedure will, however, be quite complex given that the ductility demands are influenced by many different parameters. The alternative is to increase the design base shear by an appropriate value, keeping the distribution of design strength

unchanged. The second alternative is adopted in this study.

Computation of the MDOF strength modification factor, M , which should be applied to the strength of the associated SDOF system to obtain the strength of MDOF system, requires a complex iterative procedure. Let the modification factor M and the first storey ductility ratio be defined as:

$$M = \frac{V_{y,M}}{V_{y,S}} \quad (4.1)$$

$$\mu_1 = \frac{\Delta_{i1}}{\Delta_{y1}} \quad (4.2)$$

where $V_{y,M}$ is the base shear strength of the MDOF system, $V_{y,S}$ is the shear strength of the associated SDOF system, Δ_{i1} is the maximum interstorey drift in the first storey derived by performing a dynamic nonlinear time history analysis of the structures subjected to the given ground motion, Δ_{y1} is the yield interstorey drift derived from static analysis of the structure subjected to the associated SDOF strength demand (base shear) for the same ground motion distributed along the height according to NBCC 95. As discussed in preceding sections, on account of the way in which the member strengths have been selected, the relationship between the lateral load (base shear) and any storey drift (or even overall drift) is bilinear. When the static base shear reaches the associated SDOF strength demand, all the resisting elements in the structure yield (plastic hinges in walls and moment-resisting frames yield and braces in a braced frame yield or buckle) simultaneously, leading to a structural mechanism.

The iterative procedure used to determine the M factor is as follows:

Step 1:

The SDOF strength demand (base shear) corresponding to the design earthquake is distributed along the height of the structure according to the provisions of NBCC. Then the internal end forces (end moments for the moment-resisting frames and walls, and axial forces in the bracing members) are computed.

Step 2:

The strength capacities of the resisting members are set to be equal to the internal forces determined as above. Nonlinear dynamic analysis of the structure subjected to the design ground motion is then performed to calculate the maximum interstorey drifts, Δ_i , for all storeys across the height of the structure.

Step 3:

The storey ductility demand is computed for all the storeys across the height (Eq. 4.2). If the ductility demand in the first storey, μ_1 , is equal to the target ductility μ_t , end of iterations has been reached and the strength modification factor M is derived from Eq. 4.1 (if first iteration, $M = 1.0$). If μ_1 is not equal to μ_t , the strength supplied for MDOF must be modified. In fact, if the computed ductility demand is higher than the target value, the strength supplied must be increased and if it is less than the target value, the strength supplied must be decreased.

Step 4:

Having updated the strength, the process is repeated beginning with step 1 with the new strength (base shear). Note that any modification made to the base shear in step 3, will automatically affect the strength capacities of the resisting elements in the structure as well as the storey yield displacements. This procedure is continued until the storey ductility in the first storey, computed in step 3, becomes equal to the target ductility. At this stage, M is derived from Eq. 4.1. This value is the actual strength modification factor for the given first mode period, structural type, earthquake record and target ductility ratio.

As an example, Figs. 4.20a, b, and c illustrate the results of such an iteration process for a 30-storey building, with three different structural types, subjected to the short period Trial 2 ground motion for the East, for an associated SDOF target ductility ratio of $\mu_t = 2.0$. These figures show the relationship between the strength modification factor and the first storey ductility demand. As would be expected, the storey ductility demand decreases monotonically with an increase in the base shear strength.

Figures 4.21a and b show the variation of the strength modification factor with the first mode period, for moment-resisting frames and a target ductility $\mu_t = 4.0$, for the West and the East, respectively. Figures 4.22a and b show the variation of the strength modification factor with the first mode period, for moment-resisting frames and a target ductility $\mu_t = 2.0$, for the West and the East, respectively.

Figures 4.23a and b show the variation of the strength modification factor with the first mode period, for flexural walls and a target ductility $\mu_t = 4.0$, for the West and the East, respectively. Figures 4.24a and b show the variation of the strength modification factor with the first mode period, for flexural walls and a target ductility $\mu_t = 2.0$, for the West and the East, respectively. Figures 4.25a and b show the variation of the strength modification factor with the first mode period, for braced frames and a target ductility $\mu_t = 3.0$, for the West and the East, respectively. Figures 4.26a and b show the variation of the strength modification factor with the first mode period, for braced frames and a target ductility $\mu_t = 2.0$, for the West and the East, respectively.

The following observations can be made from the results presented in Figs. 4.21 to 4.26.

- 1) In general, the strength modification factor M is period dependent and tends to increase with an increase in the period.
- 2) The strength modification factor M strongly depends on the type of ground motion. Structures subjected to the records in the East usually require a larger strength in order to limit their ductility to the target value, and therefore have a larger modification factor.

In the West, usually the long period records give a larger value of M factor in the long period range. Similarly the short period records govern the strength demand of the structures in the short period range (e.g., in proximity of $T =$

0.5s). However, for the East the short period records govern in the entire period range. Similar type of behaviour was also observed in the variation of storey ductility demands for the East and the West. The implication of the values obtained for M factor in seismic design is discussed in detail in Section 4.11.

- 3) By comparing the Figs. 4.21 with 4.22, Figs. 4.23 with 4.24, and Figs. 4.25 with 4.26, it can be concluded that the strength modification factor, M increases with an increase in the target ductility, i.e., a structure with a large storey ductility demand requires a rather large strength modification factor and vice versa.
- 4) An M factor of less than one means that the storey ductility demand is less than the target value.
- 5) In moment-resisting frames, in the West, M factor is up to 1.6 for $\mu_t = 4.0$ and 1.4 for $\mu_t = 2.0$. However, in the East M factor could be up to 4.3 for $\mu_t = 4.0$ and 3.3 for $\mu_t = 2.0$. It is to be noted that these two cases in the East are for a first mode period of 3.0 s and correspond to the short period records.
- 6) In flexural walls, in the West, M factor is up to 1.3 for $\mu_t = 4.0$ and 1.2 for $\mu_t = 2.0$. However, in the East M factor could be up to 2.3 for $\mu_t = 4.0$ and 1.7 for $\mu_t = 2.0$. It is to be noted that these two cases in the East are for a period of 3.0 s and correspond to the short the period records. The values for the West are for a period of 0.5 s and correspond to the long period records.

- 7) In braced frames, in the West, M factor is up to 2.5 for $\mu_t = 4.0$ and 2.2 for $\mu_t = 2.0$. However, in the East M factor could be up to 6.8 for $\mu_t = 3.0$ and 4.9 for $\mu_t = 2.0$. It is noteworthy that all of the values mentioned here, for both the East and the West, are for a period of 3.0 s and correspond to the short period records.
- 8) Among the three structural types studied, for a given period, ground motion, and target ductility, generally the flexural walls require the least amount of strength magnification to limit their storey ductility ratio to the target value, while the braced frames demand the highest, and the moment-resisting frames are in between.

4.10 MAXIMUM BASE SHEAR DEVELOPED IN THE STRUCTURES

Figure 4.1 illustrates the procedure leading to the derivation of base shear strength $V_{y,M}$ that limits the storey ductility throughout the height of the structure to a prescribed value μ_t . According to Eq. 4.1, this base shear strength is given by:

$$V_{y,M} = M V_{y,S} \quad (4.3)$$

where $V_{y,S}$ is the strength of the associated SDOF system for a ductility of μ_t , and M factor is the strength modification factor to ensure that the storey ductility ratios in the MDOF system are also limited to μ_t . Relating to the use of M factor

in seismic design a number of questions arise that need to be addressed. Those questions are as follows:

- 1) Is $V_{y,M}$ the maximum base shear that actually develops in the structure designed to have this base shear strength?
- 2) In case the maximum base shear is different from $V_{y,M}$, what type of relation exists between the two?
- 3) If the maximum base shear experienced in the structure $V_{max,M}$ is greater than $V_{y,M}$, does that mean the structure is going to suffer inelastic deformations beyond what it is designed for?

This section addresses these questions.

The results obtained in this study show that, at the end of an iterative procedure in which the base shear strength is progressively adjusted until the maximum storey ductility is equal to the target ductility, the maximum base shear developed in the structure $V_{max,M}$ is greater than or equal to $V_{y,M}$. Let a factor J_V be defined as

$$J_V = \frac{V_{y,M}}{V_{max,M}} \quad (4.4)$$

The results of this study show that the J_V factor strongly depends on the structural type. For moment-resisting frames and flexural walls, J_V is less than one, i.e., $V_{max,M} > V_{y,M}$. In braced frames, however, the J_V factor is 1.0, meaning that the maximum base shear developed in the structure is exactly equal to the base

shear strength that is enough to limit the storey ductilities to the target value or $V_{max,M} = V_{y,M}$. That behaviour for the braced frames is related to the fact that the maximum base shear cannot exceed the sum of the horizontal projections of the tensile yield strength and the compressive buckling strength of the braces in the first storey. This projected strength for the first storey braces is, of course, exactly equal to $V_{y,M}$.

Figures 4.27a and b illustrate the variation of the J_V factor for moment-resisting frames for a target ductility of $\mu_t = 4.0$ and different records in the East and the West, respectively. Figures 4.28a and b show similar results for a target ductility $\mu_t = 2.0$. Figures 4.29a and b illustrate the variation of the J_V factor for flexural walls for a target ductility of $\mu_t = 4.0$ and different records in the East and the West, respectively. Figures 4.30a and b present for a target ductility of $\mu_t = 2.0$.

The following conclusions can be drawn from the results presented in Figs. 4.27 to 4.30.

Moment-resisting frames:

- 1) J_V factor is almost independent of the first mode period of the structure.
- 2) J_V factor is almost independent of the target ductility.
- 3) For eastern records, J_V factor is slightly less than that for the Western counterparts. In the West, it varies between 0.9 and 1.0, while in the East it ranges from 0.85 to 0.95.

It should be noted that the J_V factor calculated for moment-resisting frames is different from the J factor in NBCC (referred to here as J_{NBCC}). In moment-resisting frames, ductility values are controlled by the beam end moments. The base shear corresponding to a set of given values for these moments as obtained from a dynamic analysis is slightly different from the NBCC distributed base shear required to produce the same end moments. The ratio of these two base shears is J_V . On the other hand, the NBCC factor J_{NBCC} is related to overturning moments and influences the axial forces in the columns.

Flexural walls:

- 1) J_V factor is strongly dependent on the first mode period; decreasing with an increase in the period.
- 2) J_V factor tends to slightly decrease with an increase in the target ductility.
- 3) J_V factor produced by long period records is usually greater than that for the short period records, meaning that the difference between the maximum base shear and $V_{y,M}$ is larger for short period records than for long period records.
- 4) J_V factor for western records is as low as 0.23 for $\mu_t = 4.0$ and 0.3 for $\mu_t = 2.0$. In the East it could be as low as 0.12 for $\mu_t = 4.0$ and 0.18 for $\mu_t = 2.0$. These particular numbers are for a period of 3.0 s and correspond to short period records. The implications of the results, presented in this section, for J_V factor in seismic design are discussed in detail in Section 4.11.

The following provides the justification for the differences observed for the J_V factor values for different structural types, namely: moment-resisting frames and flexural walls. In other words an explanation is provided for the fact that the maximum base shear developed in flexural walls, $V_{max,M}$, is so much greater than the base shear strength, $V_{y,M}$, required to limit the storey ductility to the target value. It should be noted that the member moment capacities for both moment-resisting frames and flexural walls are determined from a static analysis for the lateral loads obtained by distributing $V_{y,M}$ according to NBCC 95.

Consider Fig. 4.31a which shows a typical flexural wall subjected to an equivalent static base shear $V_{y,M}$ distributed, according to NBCC 95, along the height. $V_{y,M}$ is thus the base shear strength that determines the moment capacities at the member ends. In other words, the design moment profile along the height of the wall (Fig.4.31b), is controlled by $V_{y,M}$. The distribution of base shear according to NBCC 95 along the height, is based on participation of only the first mode. Therefore, the profile of the moment and the moment capacities at the member ends are also affected by the participation of only the first mode. However, as is seen from Table 4.2b, in flexural walls the second mode contribution to the overall response of the structure is substantial. This means that in a dynamic analysis a significantly larger value of maximum base shear $V_{max,M}$ must be reached before the moment capacity determined on the basis of the first mode distribution of $V_{y,M}$ is reached. In fact, this is the reason why NBCC 95 specifies a reduction factor J_{NBCC} to be

applied to the overturning moment calculated at the base of the structure. The variation of J_{NBCC} with period is given in Fig. 4.31c.

The implication of the NBCC provisions is that the design base shear is the maximum base shear that is developed in the structure, $V_{max,M}$, and not the shear that corresponds to the moment capacities or limits the storey ductility in the structure, $V_{y,M}$. However, NBCC 95 requires that this maximum shear be reduced by J_{NBCC} to obtain the overturning moment. This way the storey ductilities remain unchanged in a wall structure (i.e., the ductility in the first storey will remain matched to the target ductility). In addition, such a structure neither suffers the inelastic deformations beyond what it is designed for, nor is the design uneconomical because $V_{max,M}$ is used instead of $V_{y,M}$ for determination of the end member moment capacities.

It should be noted that for flexural walls J_V has an identical meaning to J_{NBCC} . This is because overturning moment directly affects the flexural moment in wall structures. As stated earlier, this is not the case for moment-resisting or braced frames where the overturning moment primarily affects the axial forces in the columns. Therefore, in such structures the reduction factor J_{NBCC} is applied only to the axial forces in the columns calculated from distribution of the design base shear, not to the moments in the members of a moment-resisting frame or to the axial loads in the braces of a braced frame.

By comparison of the results presented in Figs. 4.29 to 4.30 and the NBCC

values for J_{NBCC} , it can be concluded that the overturning moment reduction factor specified by NBCC is on the conservative side for flexural walls (i.e., $J_{NBCC} > J_V$). For moment-resisting frames, such comparison is not directly relevant. Section 4.11 provides additional discussion regarding the preceding arguments.

4.11 MDOF STRENGTH MODIFICATION FACTORS IN A DESIGN-ORIENTED FORM

4.11.1 Reinterpretation of MDOF strength modification factors

Section 4.9 provided the MDOF strength modification factor, M , corresponding to different multi-storey structures in order to limit the storey ductilities to the target value. The assumption was that this M factor was being applied to the strength of an associated SDOF and the product gave the strength of the multi-storey structure. The M factor was found to be strongly dependent on, the structural type, the frequency content of the ground motions, and the target storey ductility. The discussion in Section 4.10, however, shows that the maximum base shear experienced by the structure is different from the base shear strength required to limit the ductilities to target values. In fact, for flexural walls, the former can be considerably greater than the latter.

The design base shear specified by the seismic codes (such as NBCC 95) is, in fact, an estimate of the *maximum base shear* that the structure is likely to experience when the building is subjected to the design earthquake. These obser-

vations point to the need for a broader definition for MDOF strength modification factor, if this factor is to be used directly in a code design base shear formula. It is apparent that, for a multi-storey building that is expected to be strained into the inelastic range, the MDOF strength modification factors can be defined in two different ways.

$$M_M = \frac{V_{y,M}}{V_{yS}} \quad (4.5)$$

$$M_V = \frac{V_{max,M}}{V_{yS}} \quad (4.6)$$

- 1) Factor M_M is identical to the M factor described in earlier sections. The product of M_M and V_{yS} provides the base shear strength $V_{y,M}$ required for deriving the member load capacities as well as limiting the storey ductilities to the target value.
- 2) Factor M_V is the shear-based MDOF modification factor. The product of M_V and V_{yS} yields the maximum base shear $V_{max,M}$ that will develop in the structure. Since the seismic code formula is concerned with the maximum base shear, the M_V factor is the one that should be used in the derivation of design base shear.

Combining Eqs. 4.4, 4.5 and 4.6, the J_V factor can also be defined as:

$$J_V = \frac{M_M}{M_V} \quad (4.7)$$

4.11.2 MDOF strength modification factors for use in seismic design

The results presented so far in this chapter are based on a series of dynamic analyses of selected structures for their response to UHS compatible ground motions. The ground motion time histories developed by Atkinson and Beresnev (1998) for selected cities in Canada are used. For each city, 4 records are provided, 2 to cover the short period range of the design SDOF response spectrum, UHS, and the other 2 to match the long period range of the UHS. The elastic response spectra of these records are illustrated in Figs. 4.7 and 4.8 along with the UHS2500, for some selected cities in the West and the East of Canada.

Atkinson and Beresnev (1998) suggest that, the geometric mean of the elastic responses of the two long period records provides a better representation of the response acceleration for the long period region of the design spectrum. Similarly, the geometric mean of the elastic responses of the two short period records provides a more realistic representation of the response acceleration for the short period range of the design spectrum. The geometric mean of samples $x_1 x_2 \dots x_n$ is defined as:

$$G.M = \sqrt[n]{\prod_{i=1}^n x_i} \quad (4.8)$$

Using Eq. 4.8, the elastic response acceleration spectra for a given location can be reduced to two components, one being the geometric spectral mean of the responses of the two long period records and the other the geometric spectral mean of the two short period records. For a given site, the composite or the envelope of these two spectra is intended to match the design SDOF response spectrum. UHS. Equations 4.9 to 4.11 illustrate this procedure in a systematic manner. For a given site and a period of T.

$$S_{aL} = \sqrt{S_{aL1} S_{aL2}} \quad (4.9)$$

$$S_{aS} = \sqrt{S_{aS1} S_{aS2}} \quad (4.10)$$

$$S_a = \max(S_{aL}, S_{aS}) \quad (4.11)$$

where

S_{aL1} = the response acceleration for the long period record Trial 1

S_{aL2} = the response acceleration for the long period record Trial 2

S_{aS1} = the response acceleration for the short period record Trial 1

S_{aS2} = the response acceleration for the short period record Trial 2

S_{aL} = the geometric mean of the response accelerations for the long period records

S_{aS} = the geometric mean of the response accelerations for the short period records

S_a = the resultant design elastic response spectrum obtained, representing the envelope of S_{aL} and S_{aS}

The aforementioned procedure is recommended for derivation of the design SDOF *elastic* response. It is reasonable to extend this procedure for the derivation of all *elastic* and *inelastic* responses of both the SDOF and MDOF systems studied here. For each city, four values have so far been obtained for each of the M_M , M_V and J_V factors corresponding to 4 records for that city. However, of these only one set of values is applicable in design. In fact, some of the results obtained for these factors may not affect the design and need not therefore be considered. As an example, consider a flexural wall with a period of 3.0 s in the East. For a target ductility $\mu_t = 4.0$, the long period record Trial 1 gives an M_M factor of 0.93 while the short period record Trial 1 requires an M_M of 1.61. Although the M_M factor is larger for the short period record Trial 1, the base shear strength demand, V_{yM} for such a structure is controlled by the long period record Trial 1. This is because V_{yM} , being the product of M_M , S_a and W , is larger for the long period record Trial 1 than that for the short period Trial 1 (refer to Table 4.5). Thus, the M_M factor for the short period record Trial 1, even though higher, does not govern the design and need not be considered.

Following the above reasoning, the strength V_{yS} of the associated SDOF for a given site, first mode period, and target ductility, is computed as follows. The

indices used in Eqs. 4.12 through 4.23 have the following meaning

$L1$ = the response to the long period record Trial 1

$L2$ = the response to the long period record Trial 2

$S1$ = the response to the short period record Trial 1

$S2$ = the response to the short period record Trial 2

L = the geometric mean of the responses to the long period records

S = the geometric mean of the responses to the short period records

$$V_{ySL} = \sqrt{V_{ySL1} V_{ySL2}} \quad (4.12)$$

$$V_{ySS} = \sqrt{V_{ySS1} V_{ySS2}} \quad (4.13)$$

thus

$$V_{yS} = \max(V_{ySL}, V_{ySS}) \quad (4.14)$$

Similarly for multi-storey structures, the design base shear strength V_{yM} required to limit the storey ductilities can be derived from

$$V_{yML} = V_{ySL} \sqrt{M_{ML1} M_{ML2}} \quad (4.15)$$

$$V_{y,MS} = V_{y,SS} \sqrt{M_{MS1} M_{MS2}} \quad (4.16)$$

Thus,

$$V_{y,M} = \max(V_{y,ML}, V_{y,MS}) \quad (4.17)$$

In a similar manner, the design maximum base shear can be obtained as follows

$$V_{max,ML} = \sqrt{V_{max,ML1} V_{max,ML2}} \quad (4.18)$$

$$V_{max,MS} = \sqrt{V_{max,MS1} V_{max,MS2}} \quad (4.19)$$

and in turn,

$$V_{max,M} = \max(V_{max,ML}, V_{max,MS}) \quad (4.20)$$

Finally, the design MDOF strength modification factors M_M and M_V can be evaluated as follows:

$$M_M = \frac{V_{y,M}}{V_{y,S}} \quad (4.21)$$

$$M_V = \frac{V_{max.M}}{V_{yS}} \quad (4.22)$$

and the design J_V factor is given by:

$$J_V = \frac{V_{y.M}}{V_{max.M}} = \frac{M_M}{M_V} \quad (4.23)$$

Tables 4.5a, b, and c present examples of the values of S_a , M_M , and $V_{max.M}$, respectively, for all of the 4 records used for the East. The values are for flexural walls with a target ductility of $\mu_t = 4.0$. Table 4.6 lists the design values for V_{yS} , $V_{y.M}$, $V_{max.M}$, M_M , M_V , and J_V factors obtained, according to the procedure described in the preceding paragraphs, from the values given in Table 4.5.

Figures 4.32 through 4.37 present M_M , M_V and J_V factors obtained as above for the three structure types, for the West and the East of Canada. Parts (a) of these figures present the M_M factor for different ductility ratios, while parts (b) show the variation of M_V factor for different ductility ratios as well as for the elastic case. Part (c) show the J_V factor for different levels of ductility. It should be noted that $J_V = M_M/M_V$. The values of M_M , M_V and J_V factors at this stage can be considered as the design values. The following observations can be made on the basis of the results presented in these figures. Although some of these observations might already have been referred to in another context they are repeated here, in brief for the sake of completeness.

On comparing parts (a) of Figs. 4.32 to 4.37 the following conclusions can be drawn in respect of M_M factor.

- (1) M_M factor increases monotonically with an increase in the period.
- (2) In moment-resisting frames and braced frames M_M factor tends to increase with an increase of ductility ratio, while in flexural walls it is not affected very much by the ductility ratio.
- (3) Except for flexural walls, M_M factor is higher for the Eastern records compared to that for the Western records.
- (4) The highest value of M_M is obtained for the braced frames, being up to 3.1 for $\mu_t = 3.0$. For flexural walls, this factor is less than 1.0 almost for the entire period range, being the lowest among the three structural types. For moment-resisting frames in the West it is 1.6 at a period of 3.0 s and $\mu_t = 4.0$. The corresponding value for the East is 1.75.

On the basis of the results presented in parts (b) of Figs. 4.32 to 4.37, the following conclusions can be drawn in respect of the factor M_V .

- (1) M_V is always greater than M_M , except for braced frames where the two are identical.
- (2) As in the case of M_M , M_V increases monotonically with increases of both the period and the ductility. The M_V values are the lowest for the elastic case.
- (3) In flexural walls, where the M_M factor was the lowest among all the three

structure types studied. M_V is in fact the highest. In the East, for a period of 3.0 s and for a ductility of $\mu_t = 4.0$, M_V is 5.8 for flexural walls while it is only 2.0 for moment-resisting frames. For braced frames $M_V = 3.1$ for a ductility of 3.0. Usually, among the three structure types, for a given period and ductility, the moment-resisting frames have the lowest values of M_V factor. This observation implies that for a given period and target ductility, the design base shear is the lowest for moment-resisting frames.

- (4) The effect of ductility on M_V is more substantial for flexural walls than for other types of structures. The reverse is true for the M_M factor.

On the basis of the results presented in parts (c) of Figs. 4.32 to 4.37, the following conclusions can be made in respect to the factor J_V .

- (1) For moment-resisting frames, J_V factor varies between 0.85 to 0.95, being slightly lower in the East than in the West. The J_V factor for $\mu_t = 2.0$ is only slightly higher than that for $\mu_t = 4.0$.
- (2) For flexural walls, the J_V factor is both period and ductility dependent. It decreases monotonically with increases in both the period and the ductility ratio. For the West, J_V factors are higher than those for the East, i.e., for two wall structures, one in the West and the other in the East, having the same design maximum base shears, the one in the West needs a relatively higher strength to limit its storey ductilities to the target value. The J_V factors computed here are less than the code value, J_{NBCC} . As a result, the ductility demand in the code

designed walls is expected to be less than the target values defined here. The J_V factor for a period of 3.0s could be as low as 0.18 in the East for a ductility of $\mu_t = 4.0$, while in the West it is 0.37 for the same period and ductility.

- (3) For braced frames, the J_V factor is constant, and is independent of the period and ductility, being always equal to 1.0.

4.12 SUMMARY AND CONCLUSION

The results of a comprehensive study of the ductility and strength demands in multi-storey structures subjected to earthquake motions have been presented in this chapter. The 2500 year UHS compatible records developed by Atkinson et al., (1998) for some selected cities in the West and the East of Canada are used in the analyses. Three different structural types: moment-resisting frames; flexural walls; and braced frames, are considered. The inelastic dynamic responses of these structures are thoroughly examined. In the study of the storey ductility demands in MDOF systems, the base shear strength of the MDOF system is set equal to that of associated SDOF system. It is concluded on the basis of the analysis of such MDOF systems that storey ductility varies across the height. It is found that the first storey ductility demand is the highest, and in fact it sets the global ductility of the structure. This global ductility demand in the MDOF systems is observed to be different from the target ductility of the associated SDOF system. To match these two ductilities, the strength of MDOF system, which was originally set as

equal to that of the associated SDOF system, is modified by a factor M_M . The values of M_M factor for different structures, periods, target ductilities, and different records in the West and the East of Canada are derived and examined. It is found that M_M factor is dependent on the structural type, target ductility and the type of ground motion. It, generally, increases monotonically with period and ductility ratio. It is also higher for the East than for the West. Nonlinear dynamic analyses are conducted on the MDOF systems whose strength had been modified as above to limit the storey ductility ratios to the target value. The maximum base shear developed in the structure, $V_{max,M}$, is obtained from these analyses and compared to the base shear strength $V_{y,M}$. It is observed that the ratio of these two base shears depends strongly on the structural type. For moment resisting frames, this ratio is found to be in the proximity of 1.0, while for flexural walls it reduces with an increase in the period and ductility. For braced frames, the two base shears are identical. The results obtained from the different trial records are synthesized to obtain the design values for M_M , M_V , and J_V factors.

Table 4.1 Characteristics of the UHS2500 compatible records used for the west and the east (Atkinson and Beresnev 1998).

Description	West				East			
	Duration (sec)	PGA %g	Source distance (kM)	Magnitude	Duration (sec)	PGA %g	Source distance (kM)	Magnitude
Long, Trial 1	24.70	21.42	50	7.5	35.52	14.92	150	7.5
Long, Trial 2	24.70	23.10	50	7.5	35.52	14.76	150	7.5
Short, Trial 1	9.20	46.85	10	6.5	7.60	37.60	20	6.0
Short, Trial 2	9.20	39.57	10	6.5	7.60	37.25	20	6.0

Table 4.2 Modal period and modal mass (in percent of total mass) for MDOF models used in this study, (a) moment-resisting frames, (b) flexural walls, and (c) braced frames

Mode No.	5-storey		10-storey		20-storey		30-storey		(a)
	T (sec)	Mass %	T (sec)	Mass %	T (sec)	Mass %	T (sec)	Mass %	
1	0.500	74	1.000	75	2.000	76	3.000	77	
2	0.177	14	0.371	11	0.746	11	1.107	10	
3	0.088	7	0.212	5	0.445	4	0.665	4	
4	0.049	4	0.137	3	0.309	2	0.469	2	
5	0.032	2	0.095	2	0.230	2	0.357	1	

Mode No.	5-storey		10-storey		20-storey		30-storey		(b)
	T (sec)	Mass %	T (sec)	Mass %	T (sec)	Mass %	T (sec)	Mass %	
1	0.500	68	1.000	64	2.000	63	3.000	62	
2	0.078	21	0.158	20	0.319	19	0.478	19	
3	0.028	7	0.056	7	0.114	7	0.171	7	
4	0.014	3	0.029	3	0.058	3	0.087	3	
5	0.010	1	0.017	2	0.035	2	0.053	2	

Mode No.	5-storey		10-storey		20-storey		30-storey		(c)
	T (sec)	Mass %	T (sec)	Mass %	T (sec)	Mass %	T (sec)	Mass %	
1	0.500	82	1.000	71	2.000	69	3.000	68	
2	0.202	12	0.305	20	0.508	21	0.741	21	
3	0.128	4	0.157	5	0.248	5	0.358	5	
4	0.093	2	0.108	2	0.168	2	0.240	2	
5	0.074	0	0.084	0	0.128	1	0.181	1	

Table 4.3 Storey target ductility ratios for braced frames having the strength of an associated SDOF system with an elasto-plastic hysteresis model and a target ductility of $\mu = 2.0$ for records in (a) the west and (b) the east

Period (sec)	Long period records		Short period records		(a)
	Trial 1	Trial 2	Trial 1	Trial 2	
0.5	1.521	1.531	1.965	2.044	
1.0	2.272	1.977	2.834	2.572	
2.0	1.920	1.969	2.080	1.706	
3.0	1.543	1.904	2.330	1.991	

Period (sec)	Long period records		Short period records		(b)
	Trial 1	Trial 2	Trial 1	Trial 2	
0.5	1.821	1.527	2.304	2.026	
1.0	2.041	1.823	1.938	2.082	
2.0	1.525	1.498	2.072	2.209	
3.0	1.619	2.077	2.034	2.017	

Table 4.4 Storey target ductility ratios for braced frames having the strength of an associated SDOF system with an elasto-plastic hysteresis model and a target ductility of $\mu = 3.0$ for records in (a) the west and (b) the east

Period (sec)	Long period records		Short period records		(a)
	Trial 1	Trial 2	Trial 1	Trial 2	
0.5	1.874	2.559	2.500	2.918	
1.0	4.052	3.293	4.864	3.169	
2.0	3.003	2.951	2.841	2.823	
3.0	1.891	4.620	3.309	2.982	

Period (sec)	Long period records		Short period records		(b)
	Trial 1	Trial 2	Trial 1	Trial 2	
0.5	2.201	2.350	3.825	2.779	
1.0	2.505	2.241	2.606	3.366	
2.0	1.819	3.409	3.551	4.077	
3.0	1.757	2.722	3.980	3.072	

Table 4.5a : Inelastic SDOF response acceleration spectra in units of g for a target ductility of 4.0 for 4 records in the east

T (sec)	Long period records		Short period records	
	Trial 1	Trial 2	Trial 1	Trial 2
0.5	0.103	0.063	0.048	0.093
1.0	0.043	0.042	0.034	0.020
2.0	0.019	0.015	0.006	0.007
3.0	0.012	0.006	0.003	0.004

Table 4.5b : MDOF strength modification factor M_M for a target ductility of 4.0 for flexural walls in the east

T (sec)	Long period records		Short period records	
	Trial 1	Trial 2	Trial 1	Trial 2
0.5	0.72	1.06	0.91	1.02
1.0	0.71	0.82	0.93	1.21
2.0	0.85	0.57	1.47	2.10
3.0	0.93	1.16	1.61	2.28

Table 4.5c : Maximum base shear $V_{\max M}$ in units of Newton developed in the flexural walls for a target ductility of 4.0 and the 4 records in the east

T (sec)	Long period records		Short period records	
	Trial 1	Trial 2	Trial 1	Trial 2
0.5	1.372E5	1.276E5	1.528E5	2.300E5
1.0	2.018E5	1.921E5	2.680E5	2.892E5
2.0	2.726E5	2.097E5	2.887E5	3.473E5
3.0	3.201E5	2.931E5	3.609E5	4.414E5

Table 4.6 The design demand parameters for flexural walls located in the east for a target ductility of 4.0

Period (sec)	V_{ys} (N)	V_{yM} (N)	V_{maxM} (N)	M_M	M_V	J_V
0.5	109017.8	95239.4	187467.3	0.874	1.720	0.508
1.0	115026.4	87767.5	278398.3	0.763	2.420	0.315
2.0	91388.4	63611.8	316647.3	0.696	3.465	0.201
3.0	68901.1	71564.3	399125.6	1.039	5.793	0.179

Figure 4.1 Modification in MDOF strength capacity to achieve equal ductility demands in SDOF and MDOF systems

Figure 4.2 : Seismic Hazard Curves, PSA at 0.2s Period, Firm Ground (Adams, 1997).

Figure 4.3 : The UHS compatible acceleration time histories used for the west of Canada

Figure 4.4 : The UHS compatible acceleration time histories used for the east of Canada

Figure 4.5 : Inelastic response spectra of the records used for the west for four target ductilities of 1.0 to 4.0
 (a) long period trial 1, (b) long period trial 2, (c) short period trial 1, and (d) short period trial 2

Figure 4.6 : Inelastic response spectra of the records used for the east for four target ductilities of 1.0 to 4.0
 (a) long period trial 1, (b) long period trial 2, (c) short period trial 1, and (d) short period trial 2

Figure 4.7 : Comparison of the elastic response spectra of the records used for the west with the corresponding UHS with return period of 2500 years

Figure 4.8 : Comparison of the elastic response spectra of the records used for the east with the corresponding UHS for Montreal with return period of 2500 s

Figure 4.9 : A typical multi-storey model of different structural types studied,
(a) moment-resisting frame, (b) braced frame, and (c) flexural wall

Figure 4.10 Structural mechanism formed under equivalent static design base shear for (a) moment-resisting frames, (b) flexural walls, and (c) braced frames

Figure 4.11 Comparison of the height wise storey ductility demands for a 20 storey moment-resisting frame subjected to long period record trial 1 in the west with associated SDOF target ductility of (a) $\mu = 4.0$ and (b) $\mu = 2.0$

Figure 4.12 : Variation of first storey ductility demand with period for moment-resisting frames having the strength of an associated SDOF system with a target ductility of 4.0 and different records in (a) the west and (b) the east

Figure 4.13 : Variation of first storey ductility demand with period for moment-resisting frames having the strength of an associated SDOF system with a target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.14 Comparison of the height wise storey ductility demands for a 20 storey flexural wall subjected to long period record trial 1 in the west with associated SDOF target ductility of (a) $\mu = 4.0$ and (b) $\mu = 2.0$

Figure 4.15 : Variation of first storey ductility demand with period for flexural walls having the strength of an associated SDOF system with a target ductility of 4.0 and different records in (a) the west and (b) the east

Figure 4.16 : Variation of first storey ductility demand with period for flexural walls having the strength of an associated SDOF system with a target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.17 Comparison of the height wise storey ductility demands for a 20 storey braced frame subjected to long period record trial 1 in the west having of an associated bilinear SDOF with a target ductility of (a) $\mu = 3.0$, and (b) $\mu = 2.0$

Figure 4.18 : Variation of first storey ductility demand with period for braced frames having the strength of an associated bilinear SDOF system with a target ductility of 3.0 and different records in (a) the west and (b) the east

Figure 4.19 : Variation of first storey ductility demand with period for braced frames having the strength of an associated bilinear SDOF system with a target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.20 : Samples of iteration process to achieve the M factor corresponding to a target ductility of 2.0 for a 30 storey for the short period record trial 1 for the east and for different structural types (a) moment-resisting, (b) flexural wall, and (c) braced frame

Figure 4.21 : Variation of strength modification factor, M with period for moment-resisting frames for a target ductility of 4.0 and different records in (a) the west and (b) the east

Figure 4.22 : Variation of strength modification factor, M with period for moment-resisting frames for a target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.23 : Variation of strength modification factor, M with period for flexural walls for a target ductility of 4.0 and different records in (a) the west and (b) the east

Figure 4.24 : Variation of strength modification factor, M with period for flexural walls for a target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.25 : Variation of strength modification factor, M with period for braced frames for an associated bilinear SDOF target ductility of 3.0 and different records in (a) the west and (b) the east

Figure 4.26 : Variation of strength modification factor, M with period for braced frames for an associated bilinear SDOF target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.27 : Variation of J_v factor with period for moment-resisting frames for a target ductility of 4.0 and different records in (a) the west and (b) the east

Figure 4.28 : Variation of J_V factor with period for moment-resisting frames for a target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.29 : Variation of J_V factor with period for flexural walls for a target ductility of 4.0 and different records in (a) the west and (b) the east

Figure 4.30 : Variation of J_V factor with period for flexural walls for a target ductility of 2.0 and different records in (a) the west and (b) the east

Figure 4.31 : (a) A typical flexural wall subjected to equivalent static lateral base shear, (b) moment profile along the height of the wall, (c) the variation of NBCC overturning moment reduction factor with period

Figure 4.32 : Variation of M_M , M_V , and J_V factors with period for different target ductility ratios for moment-resisting frames in the west

Figure 4.33 : Variation of M_M , M_V , and J_V factors with period for different target ductility ratios for moment-resisting frames in the east

Figure 4.34 : Variation of M_M , M_V , and J_V factors with period for different target ductility ratios for flexural walls in the west

Figure 4.35 : Variation of M_M , M_V , and J_V factors with period for different target ductility ratios for flexural walls in the east

Figure 4.36 : Variation of M_M , M_V , and J_V factors with period for different target ductility ratios for braced frames in the west

Figure 4.37 : Variation of M_M , M_V , and J_V factors with period for different target ductility ratios for braced frames in the east

Chapter 5

A PROPOSED BASE SHEAR FORMAT FOR SEISMIC DESIGN

5.1 INTRODUCTION

Current provisions on seismic design in most codes are based on *elastic* strength demand spectra (e.g, product of vS in NBCC or ZCW in UBC), which are scaled down to design base shear spectra by means of structural system-dependent, but period-independent, reduction factors (R in NBCC or R_w in UBC). The elastic strength demand spectra are then modified by raising the long period ordinates to account for higher mode effects and to provide additional safety for multi-storey structures. The following remarks apply to many codes, but are specially relevant to the NBCC.

While the designs based on the above cited approach appear to be satisfactory, there are several conceptual problems with the approach, some of which are explained in more detail in Chapter 1. First, it is well established that the reduction factor (R factor) depends on the system ductility ratio and period. Second, ductility capacities, whether at the global (structure) or local (member) level, are not explicitly addressed in the code. Third, the issue of *damage control (serviceability)*

during moderate earthquakes is not dealt with separately so that the design is primarily against *collapse during severe earthquakes*. Fourth, the ductility demands that code designed structures may experience during severe earthquakes are not proportional to the R factors because of the large variation in the reserve strength (overstrength) in structures.

It is desirable that seismic design codes adopt a different and more transparent design approach that would permit adaption of the design to take account of the reserve strength and ductility capacities of different structural systems and elements that control seismic behaviour. This chapter presents the summary of a proposed design approach that is based on the results of this study on the ductility and strength demands for SDOF and MDOF systems. The basic information for implementation of this approach was presented in Chapters 2, 3, and 4.

5.2 PROPOSED DESIGN METHODOLOGY

The proposed design approach has to be simple enough to be adopted in a design code. It should also permit the designer to explicitly consider demand versus capacity. The objective of design is to provide capacities that exceed seismic demands by an adequate margin of safety, taking into account the uncertainties inherent in both the capacities and demands. The serviceability limit state should be separated from the collapse limit state by using two design earthquake levels with different probability of occurrence (representing moderate and severe earthquakes).

respectively). The design methodology proposed here is not concerned with the issue of damage control (serviceability). It focuses only on design for safety against collapse during a severe earthquake.

In the design for safety against collapse, it is required that the global ductility capacity of the structure be the basis for seismic design. As discussed in Chapter 4, in a multi-storey structure, it is usually the first storey that determines the global ductility demand. However, the first storey ductility itself is a function of many structural and loading parameters, such as local (member or connection) ductility capacities, geometry of the structure, the hysteresis behaviour relating to local inelastic deformations, the gravity loads, etc.. In a sound design ductility capacity should exceed the maximum ductility demand. The present work is not, however, concerned with ductility capacities, but deals only with demands.

The present code design provisions are based on the concept of providing a base shear strength that is a fraction of the maximum base shear obtained from an elastic design spectrum. In other words, the elastic base shear is reduced by a period independent modification factor R whose value is selected on the basis of the ductility capacity of the structure. Chapter 2 of the present work presents the results of a study on the reduction factor R applicable to inelastic SDOF systems. The results show that the reduction factor R is dependent on the period. For short periods the value of R is lower than the target ductility μ_t . Factor R approached μ_t as the period increases to about 0.5 s. Other studies have reached a similar

conclusion. Many modern seismic codes thus adopt a value of R that is equal to the ductility capacity. Realizing that R is not equal to the ductility demand over the entire period range, the present work attempts to provide recommendations for modifications to the provisions of NBCC 95. In formulating these proposals, the ductility capacity for a given structural system is taken to be equal to the current NBCC strength reduction factor, R . Table 5.1 lists the R factors considered for the structural systems studied in this work.

The results and methodologies discussed in earlier chapters form the basis of a new format for seismic design base shear proposed here. The format considers, to some extent, the capacity/demand principals and yet is simple enough for use in routine design.

5.3 PROPOSED BASE SHEAR FOR SEISMIC DESIGN

A revised base shear format is proposed in this work. The aim in proposing such a base shear format is to incorporate some key structural parameters whose effects are found in this study to be critical on the seismic response of the building structures. As discussed earlier (in Chapter 1), the current codes (e.g., NBCC 95) do not account for these parameters properly in the design process and may therefore, lead to inconsistent designs with unknown levels of protection. Some of these key parameters are evaluated in Chapters 2, 3 and 4 of this research. They form the basis of the proposed base shear given by Eq. 5.1 and applicable to the

seismic design of building structures. It is simple enough for use in routine design. It is also in line with the current codes in that it uses only a single design earthquake level. Additional study is required to extend this base shear format for application to the serviceability design level.

The base shear V is obtained from.

$$V = \frac{CM_V FIW}{R_d} \quad (5.1)$$

where

W = the weight of the structure that contributes to the inertia force.

C = the UHS-based base shear coefficient corresponding to the inelastic response of an SDOF system for a given target ductility μ_t , at period T . Chapter 2 presents a proposed empirical model which provides values for this factor.

R_d = the reserve strength ratio that accounts for the redistribution of internal forces. This factor may be period dependent and is a function of the structural type. Values of this factor for different structural types are given in Chapter 3.

M_V = MDOF modification factor. This factor is period dependent and is a function of the structural type and ductility capacity, μ_t . This factor also varies with the seismicity of the site. Chapter 4 provides estimates of this factor for different cases studied.

F = the foundation factor that accounts for the effect of foundation on the design base shear. This factor is outside the scope of the present work and has not been studied here.

I = the importance factor, having the same definition and magnitude as those used by NBCC 95.

Figure 5.1 schematically shows a step-by-step implementation of the proposed design approach. The implementation is summarized as follows:

- (1) At a preliminary stage of design, some information must be provided, such as: 1) the geographic location, e.g., the West or the East of Canada, 2) the maximum spectral acceleration obtained from the UHS for the site, UHS_m and the uniform hazard spectral value for a period of 0.5 s, $UHS(0.5s)$, 3) the type of structural system and its ductility capacity μ_t ($= R$), 4) the information needed for estimation of the first mode period, T , and 5) an estimate of the total weight, W .
- (2) Using the empirical model introduced in Chapter 2 for derivation of UHS matched SDOF inelastic strength demand spectra, the inelastic base shear coefficient $C(\mu_t, T, UHS)$ and/or the associated SDOF system strength demand can be derived, for the given seismic site, period and target ductility.
- (3) The structural system dependent MDOF modification factor, M_V is then applied to the strength of the associated SDOF system. This base shear now

represents the maximum design base shear, corresponding to a collapse level earthquake, that the structure is likely to experience. This step defines the strength demand at ultimate level, where either a structural mechanism occurs or the storey inelastic drifts are on the verge of exceeding their limit, for example 2% according to NBCC 95.

- (4) A reduction factor R_d is then applied to the calculated base shear in step 3 to account for the reserve strength in the structure due to redistribution of internal forces. This step determines the strength associated with the end of elastic response where the first yield or local strength capacity is about to be reached. It permits the use of a simple elastic analysis, rather than a complex inelastic one, to determine the design forces in the members.
- (5) For estimation of member strength capacities, this base shear may be reduced by a factor J_V derived in Chapter 4. The base shear can now be distributed according to the provisions of NBCC and an elastic analysis performed for the lateral loads so obtained. This analysis provides member strength capacities that are required to limit the storey ductilities to the target value.

To obtain an optimized design, the above procedure may need to be repeated in an iterative manner as follows. After the seismic load effects have been derived in the first round, the preliminary member sizes can be updated to satisfy the relevant material code. The total weight and the first mode period are updated next. Using these updated values a better estimate may be obtained for the design forces.

5.4 TYPICAL VALUES FOR C , R_d , M_V , AND J_V FACTORS

The new parameters implemented in the proposed base shear format are C , R_d , M_V , and J_V . The empirical method represented by Eqs. 2.8 through 2.10 in Chapter 2 provides an estimate for C factor for a given period, target ductility ratio, and the site dependent UHS. Typical design values for the other three factors are discussed in detail in Chapters 3 and 4. These values are repeated here for the sake of completeness.

5.4.1 Typical values for R_d factor

Figures 3.4 and 3.14 in Chapter 3 present values of the reserve strength ratio, R_d , for ductile moment-resisting frames of both steel and concrete studied in this work. When the effect of $P - \Delta$ and 5% strain hardening are considered, the R_d values range from 1.42 to 2.25 as shown in Table 5.2. An average of 1.67 may be considered as being appropriate for design.

This study showed that the assessment of typical values of the reserve strength ratio for concrete buildings with wall structural system is not possible. R_d factor for this type of structural system depends on the number of lean-to frames, the number of storeys, the relative magnitudes of earthquake to gravity loads, etc.. Therefore the reserve strength ratio, corresponding to redistribution, for this type of structures must be estimated for each individual case separately. A push-over analysis as described in Section 3.10 may be used for derivation of R_d for this type

of structure. However, to avoid such an analysis, one may use conservatively, the NBCC implied value of 1.67 as the reserve strength factor.

This study showed that the reserve strength in ductile steel concentrically braced frames is controlled by the reserve strength in the critical storey, being usually the first storey. The reserve strength in the first storey, however, strongly depends on the slenderness ratio of the braces. The reserve strength ratio for these structures can be estimated by the closed form solutions given by Eqs. 3.11 and 3.19. The values of R_d derived using these two equations are plotted in Fig. 3.8. These equations tend to provide reserve strength ratios which are slightly on the conservative side (refer Fig. 3.9). According to the minimum slenderness ratio required by CSA S16.1-94 and the Eqs. 3.11 and 3.19, R_d factor is always less than 2.0. For a practical value of slenderness ratio, say 80, R_d , for example works out to about 1.4.

5.4.2 Typical values for M_M , M_V and J_V factors

Figures 4.32 through 4.37 present values of M_M , M_V and J_V factors applicable for design for different records, structural types and target ductility ratios including the elastic case. These values are repeated here for moment-resisting frames, flexural walls, and braced frames, in Tables 5.3, 5.4, and 5.5, respectively.

The results obtained in this study for the M_V and M_M factors can be incorporated in the proposed base shear in two different manners. The first option is

that for a given location, structural type, and first mode period, one may select for each of the factors M_V and M_M the upper bound of the values obtained for different ductility ratios. For moment-resisting frames and flexural walls this upper bound corresponds to $\mu_t = 4.0$; while for braced frames it corresponds to $\mu_t = 3.0$. If this procedure is used, then for a given location and structural type there will be only one spectrum for each of the factors M_V and M_M , regardless of the ductility capacity. This procedure simplifies the design process. However, for ductility capacities other than those of the upper bound the procedure leads to a conservative design. The alternative is to assume that the M_V and M_M factors vary not only with location, structural type and period; but also with different levels of structural ductility capacities. This may lead to economy in design, but involves a design process that is more complex. The following paragraphs provide simplified relationships for derivation of an upper bound value for M_V and M_M factors for different structural types and locations on the basis of the first option.

For moment-resisting frames M_M and M_V are close to each other. In fact, it may be assumed that $M_V = M_M$. For these structures the shear obtained by using M_M provides the design end moments for beams and columns. As a simple design provision, $M_V = M_M$ can be taken as 1.0 for periods up to 1.0 s; then increasing linearly to 1.75 for a period of 3.0 s. Identical values are proposed for the West and the East.

For braced frames in the West, $M_V = M_M$ may be taken as 1.0 at a period of

0.5 s, then increasing linearly to 1.9 at a period of 3.0 s. For the East M_V may again be taken as equal to 1.0 at 0.5 s, increasing to 3.0 at a period of 3.0 s.

For flexural walls, a procedure that is similar to the one used for moment-resisting frames may be favoured, and values of M_M and M_V corresponding to $\mu_t = 4.0$ adopted for design. In the West M_V may be taken as increasing from 1.0 at $T = 0.2$ s to 1.2 at $T = 0.5$ s and then to 2.75 at $T = 3.0$ s. For walls in the East, M_V may be taken as increasing from 1.0 at $T = 0.2$ to 1.7 at $T = 0.5$ s and then to 5.8 at $T = 3.0$ s. As a conservative provision M_M may be taken as 1.0 throughout.

Figure 5.2 plots these proposed simplified relationships for the upper bound values of M_M and M_V factors for different structural types and locations.

5.5 ADVANTAGES OF THE PROPOSED BASE SHEAR FORMAT

The base shear format proposed here has the following advantages over the current code provisions.

- (1) Capacity/demand concepts are explicitly considered.
- (2) It uses the inelastic UHS-based design spectra, leading to a more reliable estimate of the design forces.
- (3) The structural type is considered more effectively and realistically. The structural type plays a key role in the response or behaviour of the building structures. In the current NBCC the structural type is considered only in the selec-

tion of force reduction factor R and to some extent in computing the first mode period. However, according to the proposed base shear formula, the structural type affects many different parameters such as μ_t , R_d , M_V , and J_V .

- (4) The design base shear coefficient, C , is derived purely from the response of an SDOF system, i.e., the MDOF effects are now kept separate by using the factors M_V and J_V or M_V and M_M . This could be beneficial, since SDOF response spectrum can also be used for modal response analysis of MDOF systems. It should however be noted that the use of UHS in the modal analysis of a MDOF system is conservative. The UHS for a given period is the envelope of the responses due to all possible types of earthquakes that can contribute to the hazard at the site with the same probability of exceedance. UHS-based response spectrum is good for SDOF systems, but may at times, overestimate the contribution of higher modes in a MDOF system.

Table 5.1 : Lateral load resisting systems as well as their corresponding strength reduction factors for the structures studied in this work

Case	Type of Lateral-Force-Resisting System	
Steel Structures		R
1	Ductile moment-resisting frame	4.0
2	Ductile concentrically braced frame	3.0
3	Nominally ductile moment-resisting frame	2.0 ⁽¹⁾
4	Nominally ductile concentrically braced frame	2.0
5	Elastic moment-resisting frame	1.0
6	Elastic concentrically braced frame	1.0
Reinforced Concrete Structures		
7	Ductile moment-resisting frame	4.0
8	Ductile flexural wall system	4.0 ⁽²⁾
9	Nominally ductile moment-resisting frame	2.0
10	Nominally ductile flexural wall system	2.0
11	Elastic moment-resisting frame	1.0
12	Elastic flexural wall system	1.0

⁽¹⁾ NBCC 95 specifies an $R = 3.0$, however, for the benefit of comparison with other structural types, $R = 2.0$ is assumed.

⁽²⁾ NBCC 95 specifies an $R = 3.5$, however, CSA A23.3 allows the use $R = 4.0$ and also for the sake of comparison with other structural types, $R = 4.0$ is considered.

Table 5.2 : Typical reserve strength ratios for moment-resisting frames
(5% strain hardening and P - Δ effect included)

Number of storeys	First mode period (sec)	Reserve strength ratio, R_d	
		Steel structures	Concrete structures
2	0.2	2.09	2.25
5	0.5	2.20	2.06
7	0.7	2.21	1.86
10	1.0	1.89	1.63
15	1.5	1.81	1.48
20	2.0	1.73	1.47
30	3.0	1.55	1.42

Table 5.3 : Typical design values of M_V and J_V factors for moment-resisting frames in
(a) western Canada and (b) eastern Canada

(a)

Period (sec)	Elastic	$\mu_t = 2.0$			$\mu_t = 4.0$		
	M_V	M_M	M_V	J_V	M_M	M_V	J_V
0.5	0.892	0.985	1.072	0.919	0.950	1.111	0.855
1.0	0.832	1.210	1.304	0.928	1.090	1.230	0.886
2.0	0.758	1.018	1.070	0.951	1.183	1.245	0.950
3.0	0.839	1.138	1.181	0.964	1.625	1.707	0.952

(b)

Period (sec)	Elastic	$\mu_t = 2.0$			$\mu_t = 4.0$		
	M_V	M_M	M_V	J_V	M_M	M_V	J_V
0.5	0.903	0.884	1.070	0.826	0.943	1.142	0.826
1.0	0.730	1.153	1.220	0.945	0.998	1.149	0.869
2.0	0.831	1.180	1.266	0.932	1.172	1.317	0.890
3.0	0.988	1.362	1.460	0.933	1.774	1.976	0.898

Table 5.4 : Typical design values of M_V and J_V factors for flexural walls in
(a) western Canada and (b) eastern Canada

(a)

Period (sec)	Elastic	$\mu_t = 2.0$			$\mu_t = 4.0$		
	M_V	M_M	M_V	J_V	M_M	M_V	J_V
0.5	0.856	0.854	1.022	0.836	0.833	1.176	0.708
1.0	0.854	0.943	1.351	0.698	0.939	1.583	0.593
2.0	1.024	0.814	1.542	0.528	0.938	2.222	0.422
3.0	1.176	0.884	1.646	0.537	1.008	2.746	0.367

(b)

Period (sec)	Elastic	$\mu_t = 2.0$			$\mu_t = 4.0$		
	M_V	M_M	M_V	J_V	M_M	M_V	J_V
0.5	0.932	0.714	1.296	0.551	0.874	1.720	0.508
1.0	1.056	0.612	1.535	0.399	0.762	2.420	0.315
2.0	1.967	0.732	2.483	0.295	0.696	3.465	0.201
3.0	2.446	0.926	3.118	0.297	1.037	5.793	0.179

Table 5.5 : Typical design values of M_V factor for braced frames in
(a) western Canada and (b) eastern Canada

(a)

Period (sec)	Elastic	$\mu_t = 2.0$	$\mu_t = 3.0$
0.5	0.863	1.100	1.175
1.0	0.773	1.395	1.367
2.0	0.989	1.543	1.898
3.0	1.006	1.409	1.804

(b)

Period (sec)	Elastic	$\mu_t = 2.0$	$\mu_t = 3.0$
0.5	0.935	1.172	1.154
1.0	0.750	1.201	1.638
2.0	1.674	1.957	2.461
3.0	1.688	2.515	3.099

Given:

- 1) The geographic area, e.g., the west or the east of Canada,
- 2) The UHS_m and $UHS(0.5s)$,
- 3) The lateral load resistant system, i.e., structural type and strength reduction factor, $R (= \mu_t)$,
- 4) The necessary information for estimation of first mode period, T .
- 5) Some estimate of total weight, W .

Required:

- 1) The design base shear

UHS matched SDOF spectra (Chapter 2)

Accounting for reserve strength (Chapter 3)

MDOF strength modification (Chapter 4)

Design (maximum) base shear can be estimated as:

$$V = \frac{CM_V FIW}{R_d}$$

Derivation of shear for member capacities (Chapter 4)

Figure 5.1 : Implementation of the proposed seismic design procedure

Figure 5.2 : Proposed simplified relationships that provide an upper bound value for M_V and M_M factors (symbols MRF, FW, and BF refer to moment-resisting frames, flexural walls, and braced frames, respectively)

Chapter 6

SUMMARY AND CONCLUSIONS

6.1 INTRODUCTION

Current code based seismic designs are based on the use of empirical coefficients that obscure the design process and may lead to inconsistent designs with unknown levels of protection. The need exists for the future codes to incorporate explicitly the capacity/demand concepts to make the design process transparent and permit designs with a well defined and consistent level of protection for the given limit states. This study is intended to provide basic information needed for the implementation of such a seismic design approach. A capacity/demand based format for the derivation of seismic design base shear is proposed in this study. The concepts of this approach are discussed in Chapters 2, 3, and 4. A step by step implementation of this approach is outlined in Chapter 5. In the proposed design base shear format, it is postulated that the global ductility capacity of the structure is the basic design parameter, and the objective of design is to provide the structure with sufficient strength capacity so that the ductility demands in the structure are less than their allowable capacities. In the present study, the NBCC structural system-dependent values for the strength reduction factor are assumed to

represent the target ductility ratio for the corresponding structural systems. More detailed studies are needed to determine rational estimates of the global ductility capacities of different structural types. For different target ductilities the required strength (inelastic strength demand) may be estimated from SDOF systems and appropriate modifications that account for MDOF effects. Thus, implementation of this approach necessitates extensive information on system-dependent inelastic SDOF and MDOF seismic demand parameters.

A comprehensive evaluation of seismic demand parameters is performed in Chapter 2 for bilinear SDOF systems. In this study, the inelastic strength demands as well as strength reduction factors are evaluated statistically for specified target ductility ratios. Such statistical study can be attempted only for ground motions with similar frequency characteristics, such as high, intermediate, and low a/v ratio records. A design methodology that allows the determination of design base shear from uniform hazard spectral values is presented. The first step in the development of the proposed methodology is the determination of elastic spectral curves for two sets of selected earthquake records having different a/v ratios, high and intermediate. The records with high a/v ratios are scaled to the maximum spectral acceleration at the site, while the intermediate a/v ratio records are scaled to the site spectral acceleration at a period of 0.5 s. The envelope of the two spectra is shown to closely match the UHS. The same suites of earthquakes are then used to produce the inelastic response spectra for different ductilities. Empirical

expressions are developed to represent both the elastic and the inelastic spectra. The proposed expressions are related to just two ground motion parameters, the maximum spectral acceleration and the spectral acceleration at a period of 0.5 s.

The conclusions drawn from this SDOF study are summarized as follows:

- (1) The strength reduction factor, R , depends strongly on the target ductility ratio and period of the SDOF system. In the long period range it approaches the target ductility ($R = \mu_t$).
- (2) The empirically obtained elastic spectrum curves are compared with the UHS as well as the current NBCC seismic coefficient curves. A comparison is also made between the inelastic spectra derived by the methodology presented here and those derived by using the reduction factor. In the short period range, the methodology based on the use of inelastic spectra gives significantly higher design forces than those obtained by use of a period independent value of R . The results presented here show that the use of inelastic spectral curves in place of the elastic curves along with a constant modification factor, R , provides a more rational method of obtaining the design forces.
- (3) A series of multistorey steel buildings located in four cities across Canada are designed for combination of gravity and earthquake forces. The earthquake forces determined according to NBCC are compared with those obtained from inelastic UHS. For the four cities studied, the difference between the two sets of forces varies from -71% to +107%. However, in the final design, the difference

in the amount of steel consumed is not that significant, varying between -8% to +13%.

The issue of the validity of overstrength is also dealt with in this study. Some possible sources of structural overstrength are outlined and it is reasoned that a more rational basis for design would be to account for such sources in assessing the capacity rather than in reducing the design loads. As an exception, one possible source of reserve strength, the redistribution of internal forces, may be used in scaling down the design forces. This is because such scaling allows the determination of design forces through an elastic analysis rather than a limit analysis. In order to assess the extent of reserve strength attributable to redistribution, different building structures: steel ductile moment-resisting frames, steel ductile concentrically braced frames, concrete ductile moment-resisting frames and concrete ductile flexural wall frames are analyzed for their response to lateral loads. A static nonlinear push-over analysis is used in which the gravity loads are held constant while the earthquake forces are gradually increased until a mechanism forms or the specified limit on inter-storey drift is exceeded. The conclusions drawn from this study are summarized here.

- (1) For buildings with moment-resisting frames of both steel and concrete, reserve strength attributable to redistribution may vary considerably depending on the methodology used in design, the ratio of earthquake to gravity loads affecting design, the number of storeys, and the $P - \Delta$ effect. However, the reserve

strength factor of 1.67 implied in the NBCC may be considered as being a reasonable estimate for the purpose of design.

- (2) The main parameter that controls the reserve strength in steel concentrically braced frames is the slenderness ratio of the bracing members. The reserve strength of the critical storey (usually first storey) is also the global reserve strength of the frame. The reserve strength is independent of the height of the frame and the effect of building sway. The reserve strength in frames having concentric braces which intersect the beams at mid span may be higher than in frames that have concentric braces connected only to the beam-column joints. Reserve strength increases with an increase in the brace slenderness ratio or a decrease in the design earthquake load.
- (3) The effect of lean-to frames on the reserve strength in concrete buildings with both moment-resisting and wall structures are discussed. The contribution of the lean-to frames depends on their number relative to the lateral load resisting frames. For moment-resisting frame buildings in high seismic zones, the number of lean-to frames is small and almost all of the frames are designed to carry the lateral loads due to earthquake. In wall structures, however, the number of lean-to frames may be large. Hence the estimation of a unique and representative reserve strength ratio is impractical. The NBCC implied value of 1.67 for reserve strength ratio may be conservative for this type of structures.

In the study of MDOF system presented in this work, three types of structural

systems are analyzed for an evaluation of the storey ductility demand, base shear strength demand, and maximum base shear experienced by the structure. The three structural types are (1) moment-resisting frames, (2) flexural walls, and (3) braced frames. The UHS2500 compatible records developed by Atkinson et al., (1998) for some selected cities in the West and the East of Canada are used in the analysis. The main objective of the MDOF study is to estimate the modifications required to the inelastic strength demands obtained from bilinear SDOF systems, in order to limit the storey ductility demand in the first storey of the MDOF systems to a prescribed value. The main conclusions derived from the parametric study of these MDOF systems are summarized as follows.

- (1) A MDOF system having the same strength as that of an associated SDOF system experiences storey ductility ratios that vary across the height.
- (2) The first storey ductility demand is the highest, and in fact sets the global ductility of the structure. This global ductility demand in the MDOF systems is observed to be different from the target ductility of the associated SDOF system.

If the maximum ductility demand in a MDOF system is to be limited to the target ductility of the associated SDOF system, the MDOF system should have a base shear strength that is M_M times the strength of the associated SDOF system. The following conclusions apply to the M_M factor.

- (1) The MDOF strength modification factor M_M is found to be strongly dependent

on structural type, period, target ductility ratio, and the characteristics of the ground motion.

- (2) M_M factor increases monotonically with an increase in the period.
- (3) In moment-resisting frames and braced frames, M_M factor tends to increase with an increase of ductility ratio, while in flexural walls it is not affected very much by the ductility ratio.
- (4) Except for flexural walls, M_M factor is higher for the Eastern records compared to that for the Western records.
- (5) The highest value of M_M is obtained for the braced frames, being up to 3.1 for $\mu_t = 3.0$. For flexural walls, this factor is less than 1.0 almost for the entire period range, being the lowest among the three structural types. For moment-resisting frames in the West it is 1.6 at a period of 3.0 s and $\mu_t = 4.0$. The corresponding value for the East is 1.75.

It is observed that a MDOF system that is designed to have a base shear strength of M_M times that of the associated SDOF, does in fact experience a base shear that is M_V times that in the SDOF system. The following conclusions apply to the factor M_V .

- (1) M_V is always greater than M_M , except for braced frames where the two are identical.
- (2) As in the case of M_M , M_V increases monotonically with increases of both the

period and the ductility. The M_V values are the lowest for the elastic case.

- (3) In flexural walls, where the M_M factor was the lowest among all the three structure types studied, M_V is in fact the highest. In the East, for a period of 3.0 s and for a ductility of $\mu_t = 4.0$, M_V is 5.8 for flexural walls while it is only 2.0 for moment-resisting frames. For braced frames $M_V = 3.1$ for a ductility of 3.0. Usually, among the three structure types, for a given period and ductility, the moment-resisting frames have the lowest values of M_V factor. This observation implies that for a given period and target ductility, the design base shear is the lowest for moment-resisting frames.
- (4) The effect of ductility on M_V is more substantial for flexural walls than for other types of structures. The reverse is true for the M_M factor.

The ratio of base shear strength $V_{y,M}$ to maximum base shear $V_{max,M}$; or the ratio of M_M to M_V is denoted by the factor J_V . The following conclusions apply to the factor J_V .

- (1) It is observed that J_V factor strongly depends on the structural type.
- (2) For moment-resisting frames, J_V factor varies between 0.85 to 0.95, being slightly lower in the East than in the West. The J_V factor for $\mu_t = 2.0$ is only slightly higher than that for $\mu_t = 4.0$.
- (3) For flexural walls, the J_V factor is both period and ductility dependent. It decreases monotonically with increases in both the period and the ductility ratio.

For the West, J_V factors are higher than those for the East, i.e., for two wall structures, one in the West and the other in the East, having the same design maximum base shears, the one in the West needs a relatively higher strength to limit its storey ductilities to the target value. The J_V factors computed here are comparable to the code value, J_{NBCC} . In general, J_V is less than J_{NBCC} . As a result, the ductility demand in the code designed walls is expected to be less than the target values defined here. The J_V factor for a period of 3.0s is as low as 0.18 in the East for a ductility of $\mu_t = 4.0$, while in the West it is 0.37 for the same period and ductility.

- (4) For braced frames, the J_V factor is constant, and is independent of the period and ductility, being always equal to 1.0.

6.2 RECOMMENDATIONS FOR FUTURE WORK

Earthquake-resistant design of structures is a complex task with many uncertainties involved. The effect of many structural and loading parameters need to be further explored. This study focuses on only a small part of a large problem. The studies suggested for the future work related to the topic of this thesis are as follows.

- (1) Development of an empirical relationship for inelastic SDOF response spectra based on the envelope of the inelastic responses of site-specific UHS compatible time histories developed by Atkinson and Beresnev (1998) would be useful.

These spectra may also be developed for hysteresis models other than elastoplastic.

- (2) A comprehensive analytical study is needed to provide realistic estimates of structural system-dependent global (storey) ductility capacities. The global ductility capacity of different structural types is dependent on many different parameters such as material properties, member local ductilities, structural geometry, vertical loads etc. Local ductility capacity can be experimentally derived for structural members with different material properties. However, determination of the global ductility is a difficult task, realizing that no two structures have the same geometry or material properties. Also, since in the beginning the building has not been designed as yet, the designer can not have a precise estimate of the ductility capacity. However, since the design is iterative it can be evaluated and improved during the iteration procedure. For an individual structure, this could be achieved by relating the critical member (usually the first plastic hinge) local ductility demands to say the first storey ductility.
- (3) The results in this study show that the reserve strength in a structure is also dependent on the acceptable ductility demand. It may be noted that the acceptable ductility demand is currently represented by the strength reduction factor R (as per NBCC), which, in turn, governs the level of design earthquake forces. A study is necessary to explore the relationship between reserve strength ratio and the acceptable structural ductility demand. The study on

R_d factor presented in this work is limited to specific values of R factor, 4.0 for moment-resisting frames, 3.5 for flexural wall structures, and 3.0 for braced frames. Further study is necessary to determine R_d factor for structural systems with ductility capacities other than those studied in this work.

- (4) A comprehensive study is necessary to develop a height wise lateral load distribution that accounts for the effect of higher modes. In this regard, a complete study needs to be performed on the effect of the load, F_t , applied at the top floor as per NBCC 95 for structures that are to be strained into inelastic range.
- (5) A study is required on the optimization of the distribution of base shear strength in a storey-by-storey basis, so that only a modification of the strength of selected storeys is enough to limit the storey ductility throughout the height to the target value. This may have a significant economical impact to the final design.
- (6) The study on MDOF demand parameters needs to be extended to other structural types, and to UHS based ground motions for other cities.
- (7) In the study of MDOF system demand parameters presented in this work, $V_{y,M}$ was defined as the base shear that when distributed according to NBCC provides the member design moments. If the individual members are selected to have flexural strengths equal to these design moments, the ductility demands in the MDOF structure are limited to the target ductility. However, during earthquake excitations shear distribution in the structure is different from the NBCC distribution, because of the participation of higher modes. As a result

a base shear somewhat higher than $V_{y,M}$ must be reached before the member moments reach their design values. This higher shear was referred to as $V_{max,M}$. For flexural walls the design moment is determined by the overturning moment produced by $V_{y,M}$ when distributed according to NBCC. The ratio of $V_{y,M}$ to $V_{max,M}$ is thus the familiar J_{NBCC} factor. For moment-resisting frames and braced frames $V_{y,M}$ does not provide a measure of the actual overturning moment, which in this case influences the axial forces in the columns rather than the member end moments in moment-resisting frames or brace forces in the braced frames. A study is required to find a value of base shear, V_o that when distributed according to NBCC will produce an overturning moment equal to that obtained in dynamic analysis. The ratio of V_o to $V_{max,M}$ will then provide a comparison with J_{NBCC} for moment-resisting frames or braced frames.

- (8) A comprehensive study is necessary to address the issue of damage control (serviceability limit state). Design earthquakes with a probability of exceedance different from that of the collapse level design earthquakes (representing moderate but more frequent earthquakes) must be considered. Although the designs for these two limit states are different, both designs follow the same basic concept (Nassar and Krawinkler, 1991).
- (9) Further studies are required to provide more realistic values for the first mode period for different structural types. The code formulae for estimating the first mode period tend to yield a period value that is smaller than that obtained

from an analytical model. The causes for the difference between the two need further investigation in order to determine the best estimate of the period for the purpose of design.

BIBLIOGRAPHY

- Adams, J. 1997. Private Communication.
- Adams, J., Weichert, D. H., Halchuk, S., and Basham, P. W. 1996. Trial seismic hazard maps of Canada-1995: Final values for selected Canadian cities. Geological Survey of Canada Open File 3283. Ottawa, On.
- Atkinson, G. M. 1991. Use of the uniform hazard spectrum in characterizing expected levels of seismic ground shaking. 6th Canadian Conference on Earthquake Engineering, Toronto, On. 469-476.
- Atkinson, G. M. and Beresnev, I. A., 1998. Compatible ground motion time-histories for new National hazard maps. To be published in Canadian Journal of Civil Engineering, April 1998 issue.
- Bazzurro, P. and Cornell, C. A. 1994. Seismic hazard analysis of nonlinear structures I: Methodology. Journal of Structural Engineering ASCE. **120**: 3320-3344.
- Bertero, V. V., Anderson, J. C., Krawinkler, H., and Miranda, E. 1991. Design guidelines for ductility and drift limits. Report UCB/EERC-91/15. Earthquake Engineering Research Centre, University of California, Berkeley, Calif.
- Bertero, V. V. and Kamil, H. 1975. Nonlinear Seismic Design of Multistorey Frames. Canadian Journal of Civil Engineering, **2**: 494-516.
- Bertero, R.D. 1992. Tall Reinforced Buildings: Conceptual Earthquake-Resistant Design Methodology. Report UCB/EERC-92/16. Earthquake Engineering Research Centre, University of California, Berkeley, Calif.
- Building Seismic Safety Council. 1991. NEHRP recommended provisions for the development of seismic regulations for new buildings. Federal Emergency Management Agency. Earthquake Hazard Reduction Series 16. BSSC. Washington D.C.
- Canadian Commission on Building and Fire Code 1995. National Building Code of

- Canada 1995. National Research Council of Canada, Ottawa, On.
- Canadian Standards Association. 1994. A23.3-94 Design of Concrete Structures. Toronto, On.
- Chopra, A. K.. 1995. Dynamics of Structures: Theory and Applications to Earthquake Engineering. Prentice Hall, New Jersey.
- Firmansjah, J., Goel, S. C. and Rai, D. C. 1996. General purpose computer program for static and dynamic analysis of two-dimensional structures. Research Report UMCEE 96-20, Department of Civil and Environmental Engineering, The University of Michigan College of Engineering, Ann Arbor, Michigan.
- Fishinger, M. and Fajfar, P. 1990. The role of overstrength in seismic resistance of buildings. Ninth European Conference on Earthquake Engineering and Structural Dynamics, Moscow, USSR. 2. 329-338.
- Humar, J.L. and Rahgozar, M.A. 1996. Application of uniform hazard spectra in seismic design. Canadian Society of Civil Engineering. 1996 Annual Conference. 1st Structural Specialty Conference. Volume IIb pp. 811-822.
- Humar, J.L. and Rahgozar, M.A. 1996. Application of inelastic response spectra derived from seismic hazard spectral ordinates for Canada. Canadian Journal of Civil Engineering, 23: 1051-1063.
- Humar, J.L. and Rahgozar, M.A. 1996. Concept of overstrength in seismic design. 11th World Conference on Earthquake Engineering, Acapulco, Mexico. Paper No. 639.
- International Conference of Building Officials 1991. Uniform Building Code. Whittier, Ca.
- Jain, A.K. 1978. Hysteresis behaviour of bracing members and seismic response of braced frames with different proportions. Ph.D. thesis, University of Michigan. Ann Arbor, MI.

- Jain, A.K. and Goel, S.C. 1978. Hysteresis model for steel members subjected to cyclic buckling and cyclic end moments a buckling-user's guide for EL9 and EL10 of DRAIN-2D. Report No. UMEE-78R6. Department of Civil Engineering, The University of Michigan. Ann Arbor, Mich.
- Kannan, A.E., Powell, G.H., 1973. General purpose computer program for inelastic dynamic response of plane structures. UCB/EERC-73/6. Earthquake Engineering Research Centre, University of California, Berkeley.
- Krawinkler, H., Zohrei, M., Lashkari, B., Coffe, N., and Hadidi, H., 1983. Recommendations for Experimental Studies on the Seismic Behaviour of Steel Components and Materials, John A. Blume Earthquake Engineering Center, Report No. 61, Department of Civil Engineering, Stanford University.
- Mahin, S.A. and Lin, J. 1983. Construction of inelastic response spectra for single-degree-of-freedom systems – Computer program and applications. UCB/EERC-83/17. Earthquake Engineering Research Centre, University of California, Berkeley.
- Miranda, E. and Bertero, V. V. (1989). The Mexico earthquake of September 19, 1985 – Performance of low rise buildings in Mexico City. *Earthquake Spectra*, 5: 121-143.
- Miranda, E. 1993. Site-dependent strength reduction factors. *Journal of Structural Engineering*, ASCE, 119: 3503-3519.
- Mitchell, D. and Paultre, P. 1994. Ductility and overstrength in seismic design of reinforced concrete structures. *Canadian Journal of Civil Engineering*, 21, 1049-1060.
- Nassar, A. A. and Krawinkler, H. 1991. Seismic demand for SDOF and MDOF systems. Report No. CA94305-4020. John A. Blume Earthquake Engineering Centre, Stanford University, CA.
- Naumoski N., Tso W. K. and Heidebrecht A. C. 1988. A selection of representative

- strong motion records having different a/v ratios. EERG Report 88-01. McMaster University. Hamilton, On.
- Newmark, N. M. and Hall, W. J. 1982. Earthquake spectra and design. Earthquake Engineering Research Institute, Berkeley, California 103 pp.
- Osteraas, J. and Krawinkler, H. 1989. The Mexico earthquake of September 19, 1985 – Behaviour of steel buildings. *Earthquake Spectra*, **5**, 51-88.
- Prakash, V. Powell, G.H. and Campbell, S. 1993. General purpose computer program for inelastic dynamic response of plane structures. UCB/EERC-93/17. Earthquake Engineering Research Centre, University of California, Berkeley.
- Rahgozar, M.A. and Humar, J.L. 1996. Application of inelastic uniform hazard spectra in seismic design. 11th World Conference on Earthquake Engineering, Acapulco, Mexico, Paper No. 1128.
- Rahgozar, M.A. and Humar, J.L. 1998. Accounting for Overstrength in Seismic Design of Steel Structures. *Canadian Journal of Civil Engineering*, **25**, pp 1-15.
- Rahgozar, M.A. and Humar, J.L. 1997. Accounting for Overstrength in Seismic Design of Concrete Building Structures. Proceedings of the Al-Azhar Fifth International Conference, December 1997, Cairo, Egypt.
- Tso, W. K. and Naumoski, N. 1991. Period-dependent seismic forces for short period structures. *Canadian Journal of Civil Engineering*, **18**: 568-574.
- Uang, C. M. 1992. Seismic reduction and displacement amplification factors. Tenth World Conference on Earthquake Engineering, Madrid, Spain, 5875-5879.
- Zhu, T. J., Tso, W. K., and Heidebrecht, A. C. 1991. Seismic performance of high-rise reinforced concrete frame buildings located in different seismic regions. Sixth Canadian Conference on Earthquake Engineering, Toronto, On. 551-558.

IMAGE EVALUATION TEST TARGET (QA-3)

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