Seismic design and performance of steel moment resisting frame buildings designed using NBCC 2005
Yousuf, Md
ProQuest Dissertations and Theses; 2007; ProQuest
pg. n/a

SEISMIC DESIGN AND PERFORMANCE OF STEEL MOMENT RESISTING FRAME BUILDINGS DESIGNED USING NBCC 2005

Md Yousuf

A Thesis
In
The Department
Of
Building, Civil and Environmental Engineering

Presented in Partial Fulfillment of the Requirements
For the degree of Master of Applied Science (Civil Engineering) at
Concordia University
Montreal, Quebec, Canada

October 2006

© Md Yousuf 2006

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Library and Archives Canada

Published Heritage Branch

395 Wellington Street
Ottawa ON K1A 0N4
Canada

Bibliothèque et Archives Canada

Direction du Patrimoine de l'édition

395, rue Wellington
Ottawa ON K1A 0N4
Canada

Your file Votre référence
ISBN: 978-0-494-28906-8
Our file Notre référence
ISBN: 978-0-494-28906-8

NOTICE:
The author has granted a non-exclusive license allowing Library and Archives Canada to reproduce, publish, archive, preserve, conserve, communicate to the public by telecommunication or on the Internet, loan, distribute and sell theses worldwide, for commercial or non-commercial purposes, in microform, paper, electronic and/or any other formats.

The author retains copyright ownership and moral rights in this thesis. Neither the thesis nor substantial extracts from it may be printed or otherwise reproduced without the author's permission.

AVIS:
L'auteur a accordé une licence non exclusive permettant à la Bibliothèque et Archives Canada de reproduire, publier, archiver, sauvegarder, conserver, transmettre au public par télécommunication ou par l'Internet, prêter, distribuer et vendre des thèses partout dans le monde, à des fins commerciales ou autres, sur support microforme, papier, électronique et/ou autres formats.

L'auteur conserve la propriété du droit d'auteur et des droits moraux qui protège cette thèse. Ni la thèse ni des extraits substantiels de celle-ci ne doivent être imprimés ou autrement reproduits sans son autorisation.

In compliance with the Canadian Privacy Act some supporting forms may have been removed from this thesis.

While these forms may be included in the document page count, their removal does not represent any loss of content from the thesis.

Conformément à la loi canadienne sur la protection de la vie privée, quelques formulaires secondaires ont été enlevés de cette thèse.

Bien que ces formulaires aient inclus dans la pagination, il n'y aura aucun contenu manquant.

Canada

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

ABSTRACT

SEISMIC DESIGN AND PERFORMANCE OF STEEL MOMENT RESISTING FRAME BUILDINGS DESIGNED USING NBCC 2005

Md Yousuf

Currently the building code authorities in many jurisdictions including Canada feel that a performance-based design approach should be used to ensure that the structure has adequate strength and deformation capacities. Although the National Building Code of Canada (NBCC 2005) is not yet a fully performance-based code, it presents an objective-based format which incorporates some concepts of the performance-based design. It provides a force-based seismic design approach, where the displacement capacities are provided indirectly. The current research focuses on the performance of a set of five, ten, fifteen and twenty storey steel moment resisting frame buildings designed according to the seismic provisions on NBCC 2005. A series of static and dynamic analyses have been carried out to evaluate their performance. It has been observed that the assumed ductility capacity in the force-based design may not always be achievable, and the capacity of the building, under seismic force decrease, with the increase of building height. Also the non-structural element has a great effect on the performance of the structure and it decreases the drift demand. The thesis also examines a number of simple methods available for performance-based design of buildings and implemented some of them for one of the buildings considered here. It has been observed that the design base shear calculated for a target damage parameter or drift demand, varies based on the method used. Further studies are needed to develop a robust method for performance-based design within the context of the Canadian code.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Acknowledgement

It is my pleasure to express my sincere gratitude to my thesis supervisor Dr. Ashutosh Bagchi, Assistant Professor, Department of Building, Civil and Environmental Engineering, Concordia University, for his guidance and encouragement in the research work. I would be completely lost without his active technical support and proper guidance.

I would like to thanks all the faculty-members of Building, Civil and Environmental Engineering Department for their guidance and suggestion during my course work.

I also thank IITS members for their help in the Computer Laboratory of the Concordia University for their continuous support with the well-equipped computer facility during my work.

The financial assistance provided for this research through FRDP research grant by Concordia University to Dr. Bagchi is also gratefully acknowledged.

I would like to take this opportunity to thank my parents, my wife and kids for their moral support and encouragement to complete my thesis.

Finally I like to thanks my colleagues and friends for their encouragement and active help to make my life easy in the Concordia University.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

TABLE OF CONTENTS

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

LIST OF FIGURES

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

LIST OF TABLES

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

NOTATION AND ABBREVIATION

C_f	Factored axial compressive force of column.
C_r	Resistive compressive axial force of column.
D	Notation for dead load.
F_a	Acceleration-based site co-efficient.
F_v	Velocity-based site co-efficient.
H	Height of building.
h_s	Story height of the building
I_E	Earthquake importance factor of the structure
I_{reqd}	Required moment of inertia of area of beam
L	Notation for live load
M_f	Factored moment
M_{pb}	Plastic moment of beam
M_r	Resistive Bending Moment
M_v	Factor to account for higher mode effect.
m^*	Mass of equivalent single degree of freedom system
R_d	Ductility related force modification factor.
R_o	Overstrength related force modification factor
R_μ, R_y	Ductility reduction factors.
S	Notation for Snow load
S(T)	The design spectral response acceleration
S_a	Inelastic acceleration spectrum
S_{ae}	Elastic acceleration spectrum.
$S_a(T)$	The 5% damped spectral response acceleration.
S_d	Inelastic displacement spectrum
S_{de}	Elastic displacement spectrum.
T	Period of vibration seconds
T_a	The fundamental lateral period of vibration of the building.
V_f	Factored shear of beam.
Vr	Specified shear resistant of beam.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

W	Weight of the building.
W_i	Weight of the i^{th} story.
α_h, α_l	Factors used in calculation of effective width of inclined strut
δ_y, δ_u	Yield and Ultimate displacement of equivalent single-degree-of freedom system
Δ	Deflection of beam.
Δ_y	Yield Displacement.
ω	Effective width of inclined strut.
Γ	Transformation factor for Multi-Degree-of-Freedom system.
γ	Demand variability factor
γ_a	Analysis uncertainty factor
φ_R	Resistance factor for that accounts for randomness in capacity.
φ_U	Resistance factor that accounts for uncertainty in relationships between tests and actual behavior in real buildings

MDOF	Multi-Degree-Of-Freedom system
NBCC	National Building Code of Canada
RHA	Response History Analysis
SDOF	Single-Degree-Of –Freedom system
SD	Standard Deviation
UHS	Uniform Hazard Spectra

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Chapter 1

Introduction

1.1 General:

Earthquake is one of the most catastrophic events which causes huge loss of life and property. The immediate and most disastrous action of earthquake is movement of ground mass or surface motion which causes a number of hazardous actions such as severe damage to infrastructure including loss of life. Some of severe earthquakes that has happened in the period on the earth are: Northridge, U.S.A. (1994), Kobe, Japan (1995), Izmit, Turkey (1999), Bhuj, India(2001), Bam, Iran (2003) and Kashmir, Pakistan (2005). The seismic damage to structure defines the level of structural performance. The designers are always concerned about seismic damage and performance of structures. While designing in an earthquake prone region emphasize is given to control the level of damage so that a structure performs at a satisfactory level.

Performance-based design is a new state-of-art in the field of structural engineering. The idea in the design method is to ensure the expected level of performance of a structure subjected to a given level of hazard, such as earthquake. The method requires an accurate evaluation of performance of a structure at various stages in the design process, and it requires reliable analysis of structures subjected to the design loads (Vision 2000

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Committee, 1995). The performance of a building during an earthquake depends on many factors: the structure's configuration and proportions, its dynamic characteristics, the hysteretic behavior of the elements and joints, the type of nonstructural components, the quality of materials and workmanship, adequacy of maintenance, the site conditions, and the intensity and dynamic characteristics of the earthquake ground motion (Fragiacomo *et al* 2002). In the seismic design or in the evaluation of performance of buildings all the above mentioned factors should be considered. Performance-based seismic design can provide a cost-effective design by reducing the structural and non-structural damage during earthquake.

In the performance-based seismic design the performance objectives are related to seismic hazards. The seismic hazards include direct ground fault rupture, ground shaking, liquefaction, lateral spreading and land sliding (FEMA-350, 2000). In this regard seismic design and evaluation of performance of the building depends on the specific hazard of the site where the structure is or will be located. As the performance objective is related to the hazard level, in defining the performance objectives the structural performance level and corresponding hazard level should be defined together. Therefore, the development of the design earthquake hazard spectra is also an important part of the seismic design methodology. Hazard spectra depend on the site's soil condition.

In Canada the western region is considered to be more vulnerable to earthquakes than the eastern region because of the matrix of the rock in this region. The rock formation in western Canada is more fragile due to repetitive stresses imposed by past earthquakes

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

than rest of the country. In the vicinity of Vancouver Island, more than 100 earthquakes of magnitude 5 or higher have occurred during the past 70 years (NRCAN, 2006). According to Foo, *et al* (2001), the February 28, 2001 earthquake near Seattle, which rattled the buildings and the occupants in Vancouver, could be viewed as a reminder of the seismic hazard to people living in Canada's most active seismic zone, the pacific coast. It has also been reported by Foo (2001) that the earthquake occurred at Saguenay in Quebec in 1988 was the strongest event in the eastern North America within the last 50 years. But Canada has a record of suffering from stronger earthquake occurred in 1949 with magnitude 8.1. An average of 1500 earthquakes with magnitude varying from 2 to 5 (NRCAN, 2006) occurs in Canada every year. So, the design of structure with earthquake resistant capability by ensuring the required level of performance, located in different region of Canada has become the demand of time. The seismic design provisions provided in the latest edition of National building Code of Canada (NBCC 2005) is mainly force-based design (Fragiacomo *et al* 2002.) which emphasize on the design of a structure for the strength or capacity. In this design methodology a non-linear structure can be designed using linear elastic analysis by transforming the elastic demand spectrum into an inelastic design spectrum. The graph of spectral acceleration versus period used in NBCC 2005 is shown in Figure 1.1 and design values of spectra for Vancouver are shown in Table 1.2 and the spectral values in between the periods reported in Table 1.2 is calculated by linear interpolation.

The effect of earthquakes on the performance of a structure also depends on the design levels of seismic hazard. The earthquake design levels or seismic hazard can be expressed

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

in terms of the recurrence interval or a probability of exceedance (Bagchi, 2001) . Based on the Vision 2000 report (1995) and recent knowledge on seismic hazard in North America, the design levels of earthquakes are presented in Table 1.1.

Table 1.1: Design Earthquakes (SEAOC Vision 2000, 1995)

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

Table 1.2: Design Spectra of NBCC 2005 (Adams and Atkinson, 2003)

Location	Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(≥4.0)
Vancouver	0.96	0.66	0.34	0.18	0.09

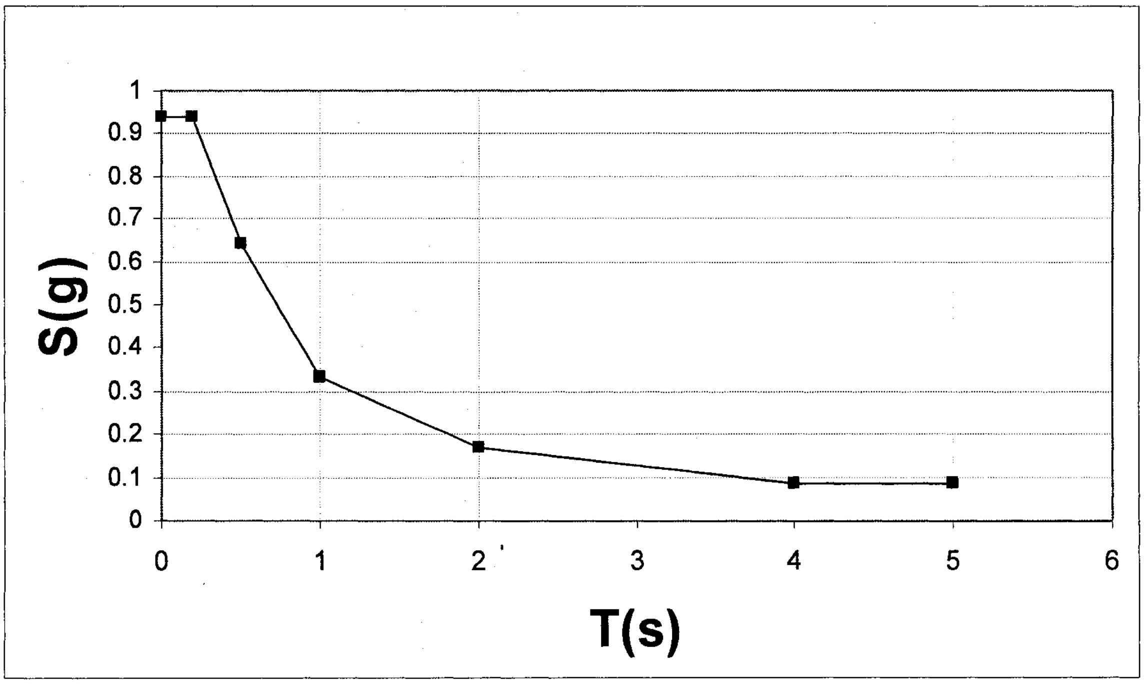

Figure 1.1: Design Spectra for Vancouver (NBCC 2005)

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The definition of the performance of a structure is a multi-objective concept. Performance objectives are statements of acceptable responses of a structure (Ghobarah, 2001). Therefore, any response parameter such as interstory drift, peak roof displacement, lateral load capacity and residual interstory drift, can be specified or targeted as the performance objectives. In FEMA-273 (1997) both the peak and residual interstory drifts are utilized in defining the performance levels as an indicator of damage. The evaluation of performance is a reliability-based probabilistic approach (FEMA-350, 2000) because of the uncertainties involved in the judgment and prediction of the characteristics of the earthquake parameters. The level of confidence comes from the knowledge of assessment of uncertainties which is very important for ensuring the level of performance. Principally the level of confidence ensures whether the structure is likely to be able to meet the desired level of performance or not. In FEMA-273 (1997) four levels of structural performance are mentioned. But only two, Immediate Occupancy *(IO)* and Collapse Prevention *(CP)* levels are mostly used in the evaluation of performance. The characteristic parameters of these two performance levels are shown in the Table 1.3.

Table 1.3: Performance Level (FEMA-273,1997)

Performance levels	Drift Limit (%)	Residual Limit drift (%)
Immediate Occupancy (IO)	0.7	-
Collapse Prevention (CP)	5.0	5.0

Four different types of structural performances has also been mentioned in the report of Vision 2000 and these are: Fully Functional, Operational, Life Safe and Near collapse. A short description of these four performance levels is presented in the Table 1.4.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The casualties of the earthquake of Northridge, California (1994) and Kobe, Japan (1995) expose the inadequacy of the code guided force-based design. During these earthquakes more than 150 steel moment resisting building (Lee and Foutch 2002) were collapsed although all those building were designed fulfilling the code requirements except evaluation of performance. The buildings were designed for static loads but their performances under dynamic loading such as ground shaking were unknown. After these earthquakes, the evaluation of the performance of buildings designed for equivalent static earthquake load became more necessary.

Table 1.4: Structural Performance Level (Vision 2000)

Performance Level	Description	Transient drift	Permanent drift
Fully Functional	No significant damage has occurred to structural and non-structural components. Building is suitable for normal intended occupancy and use.	<0.2%	Negligible
Operational	No significant damage has occurred to structure, which retains nearly all of its pre-earthquake strength and stiffness. Non-structural components are secure and most would function, if utilities available. Building may be used for intended purpose, albeit in an impaired mode.	<0.5%	Negligible
Life Safe	Significant damage to structural elements, with substantial reduction in stiffness, however, margin remains against collapse. Nonstructural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted.	< 1.5%	<0.5%
Near Collapse	Substantial structural and nonstructural damage. Structural strength and stiffness substantially degraded. Little margin against collapse. Some falling debris hazards may occur.	<2.5%	< 2.5

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Building code authorities of various countries including Canada now recognize the need for performance-based design code. The performance-based seismic design includes identification of seismic hazards, selection of the performance levels and performance design objectives, determination of suitability, conceptual design, methodologies for preliminary and final design, acceptability checks during design, design review, specification of quality assurance during the construction and monitoring of the maintenance and occupancy during the life of a building (Bertero 2002). In this type of design it is assumed that the building can resist any type of foreseeable earthquake with some damage.

In the evaluation of performance, the buildings are first designed to fulfill the regular code requirements and then the performance is evaluated through a set of rigorous analysis. The analysis can be done with appropriate computer software for inelastic static and dynamic analysis. The current building code (NBCC, 2005) is presented in an objective-based format where an acceptable solution needs to be achieved for a specified objective, rather than just satisfying the minimum requirements (Yun *et al.,* 2002). Seismic loading provisions in most existing building codes focus on the minimum lateral seismic forces for which the building must be designed (Yousuf *et al.,* 2006). But only specifying the lateral load is not enough to ensure that the building will perform at the desired level of performance. In seismic design, structures are designed to resist minor level of earthquake without any damage, moderate level of earthquake with some damage

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

in the non-structural element and major earthquake with some damage of structural or non-structural element but no collapse.

The main objective of the seismic design is that the structure will be safe from collapse due to earthquakes, the non-structural damage will be limited and there will not be any damage to human life. Though earthquake brings a huge amount casualty, it is observed that this casualty is not due to the mechanism of earthquake but due to the failure of human creation such as collapse of buildings, bridges, dams, transportation system etc. Therefore, design of safe structure is the only way to minimize earthquake effect. For this reason, the structure designed to withstand cyclic seismic forces must be properly configured with accurate continuity including adequate strength, stiffness, and deformability. The response of the building to the vibratory motions of the ground surface during an earthquake should be the main concern in the seismic design. The code-based seismic design is basically force-based design where the lateral forces are calculated from the earthquake ground motion in the form of base shear and then the structure is designed to carry the equivalent static load. In the provisions of most building codes including the NBCC 2005, the base shear induced by earthquakes is reduced as compared to that of elastic behavior. The lowering of the base shear is justified by the ductility of the structural members i.e. the capacity to deform beyond the yield point without major structural failure (NBCC 2005). The seismic design of structure is mainly capacity-based design where the elements of the structure are designed to dissipate energy under deformation caused by earthquake. In the capacity-based design, some zones of members are chosen for inelastic response and members are designed in such a

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

manner that these members will be capable to develop large plastic deformation without significant loss of its strength. The capacity of other members must be greater than the capacity of the members participated in the plastic deformation. In the capacity based seismic design, the energy dissipation must start by forming plastic hinge in beam first and than at the base of the column at any joint in a multistory building frame; but for the single story building frame the formation of the plastic hinges in column will occur at the upper end (CISC, 2004). The design and detailing of steel structures are done in accordance with design provision as specified in CSA S16-2001 and illustrated in CISC (2004).

1.2 Background of NBCC 2005.

Minimum requirements for earthquake resistant design are given in the National Building Code of Canada (NBCC 2005) to ensure an acceptable level of performance and safety. In the current version of NBCC the seismic design provisions specify the minimum lateral seismic force for which the building must be designed and it also specifies the acceptable drift limits under these forces. The minimum requirements for seismic design given in NBCC consider the site specific seismic hazard spectra, the site characteristics, the probability of occurrence of the design seismic ground motion, the type of structures and the foundation, the allowable stresses in the materials of construction, the type of soil and the amount of damage that is considered tolerable (CJCE 2003).

Before the publication of the NBCC 2005 a series of papers of different authors on the various issue of the code was published in a special issue of the Canadian Journal of Civil

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Engineering (CJCE, 2003). According to NBCC 2005 the Seismic Force Resisting System (SFRS) should be designed to resist 100% of the earthquake induced loads and their effects. Description of SFRS has also been provided in the code. The 2005 edition of the National Building Code of Canada (NBCC) addresses building performance in a broad sense and include the following issues: ground motions, site soil effects, analysis and design (De Vall, 2003).

According to NBCC (2005) the minimum lateral earthquake force V, is calculated by using the following Equation:

$$V = \frac{S(T_a)M_V I_E W}{R_d R_0} \geq \frac{S(2.0)M_V I_E W}{R_d R_0} \quad \text{.....................................} \quad 1.1$$

Where $S(T_a)$ is the spectral acceleration corresponding to the building's fundamental period T_a; M_V is the factor to account for multistory effect, I_E is the importance factor, W is the total weight of the building, R_d ductility related force modification factor, R_0 is the overstrength related force modification factor. The design acceleration values $S(T_a)$ is:

$$S(T_a) = \left.\begin{array}{ll} & F_a S_a(0.2) \text{ for } T \leq 0.2s \\ = & F_v S_a(0.5) \text{ or } F_a S_a(0.2) \text{ whichever is smaller for } T_a = 0.5 \\ = & F_v S_a(1.0) \text{ for } T_a = 1.0s \\ = & F_v S_a(2.0) \text{ for } T_a = 2.0s \\ = & F_v S_a(2.0)/2 \text{ for } T_a \geq 4.0s \end{array}\right\} \ldots\ldots 1.2$$

The total lateral seismic force V, shall be distributed in accordance with the Equation 1.3.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

$$F = \frac{(V - F_t)W_x h_x}{\sum_{i=1}^{n} W_i h_i} \text{ (NBCC, 2005)} \quad \text{..........} 1.3$$

Where x is the respective number of the story and n is the total number of the story of the building. The additional top story force *(F_t)* depends on the design period of the frame. According to NBCC 2005 the additional concentrated top story load *(F_t)* if equal to $0.07T_aV$ but need not exceed $0.25V$ and may be considered as zero where T_a does not exceed 0.7s. The seismic force at the top is equal to F_x+F_t

A major change of seismic design provision adopted in the latest edition of NBCC (2005) is that the seismic design criteria described in it is on seismic response character of the specific site. National Building Code of Canada, 2005 (NBCC, 2005) has been presented in an objective-based format where the design is achieved through the attainment of acceptable solution, rather than just satisfying the minimum requirement (CJCE, 2003). In NBCC 2005 the site-specific spectral acceleration (Humar & Mahgoub, 2003) is used to express the seismic hazard which is presented as uniform hazards spectrum (UHS). This hazard spectrum has 2% probability of exceedance in 50 years (return period 2500 years) whereas the NBCC 1995 based on 10% probability of exceedance in 50 year (return period 475). The probability of exceedance of the UHS is a function of period (Adams & Atkinson 2003) which may be constant or uniform.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The major changes of seismic design provisions that included in new NBCC (2005) are:

(a) A revised formula to calculate the base shear.

(b) Revised formulae for calculating the fundamental period of a building, for design

(c) Site specific uniform hazard spectra.

(d) New force reduction factor.

(e) Incorporation of site coefficient comes from the soil condition.

(f) Revised method to take the higher mode effect in account.

The revision of the code comes from the accumulated knowledge and experience gathered from the earthquake of last two decades. During this period the earthquakes were observed through extensive instrumentation of buildings located in moderate to high seismic zones. An updated method of analysis for the seismic forces has been adopted in the NBCC 2005. Dynamic analysis for the calculation of seismic design forces and deflection for higher seismic zone, tall buildings and building with structural irregularity of the lower height is specified in the latest edition of NBCC (2005). A description of structural irregularity is also provided in NBCC 2005.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

1.3 Objectives and Scope of the present research.

The scope and objective of this research work can be explained as follows:

1. To implement the new seismic design provision described in the latest edition of National Building Code of Canada (NBCC, 2005) using the method of equivalent static loads and dynamic analysis in the design of a series of Steel moment resisting frame buildings located in the high seismic zone in Canada, such as Vancouver.
2. To evaluate the performance of the buildings designed according to the requirements of NBCC 2005 considering both bare and infilled frames. The infilled frames will be considered to simulate the effects of non-structural elements in a building structure.
3. To determine the effect of non-structural elements on the building performance under seismic loadings.
4. To refine the design of the buildings based on their performance characteristics and devise a simple method to facilitate such refinement.
5. To evaluate the existing methods for performance-based seismic design (PBSD) procedure of buildings applied to steel moment resisting frames, and work towards developing a new PBSD method to achieve a uniform level of performance in these buildings.
6. To automate the design and analysis software tools to be used in the above mentioned tasks.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

1.4 Organization of the thesis.

The thesis has been organized into six chapters. Objective of the thesis with some introductory materials are presented in the current chapter i.e Chapter 1. A review of previous work and the ongoing work on this topic is incorporated in the Chapter 2. Design of the Steel moment resisting frame buildings considered in the research are presented in Chapter 3. Chapter 4 presents the results of the evaluation of seismic performance of the buildings. The methods of performance-based seismic design are described in Chapter 5 and a detailed evaluation of these methods has also been presented in this chapter. Summary of the thesis work including conclusions are presented in the Chapter 6. The thesis report ended with a list of reference.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Chapter 2

Literature Review

2.1 General:

Massive damage including loss of human life due to occurrence of devastating earthquakes in the past few years around the world have put the civil engineering community on the high alert and bound the structural designer to incorporate the seismic design concept with the long practiced general design procedure of the building. Because of uncertain natural phenomena the estimation of the earthquake hazards is not easy. Researchers are trying to develop specific design guidelines by accumulating the characteristics of different earthquakes occurred in different locations on the earth during the past several years. Some of these characteristics are the pattern and duration of the earthquake, peak acceleration, peak velocity, ground displacement and interval of occurrence. These are some important element used in the seismic design directly or indirectly.

Early investigation in this field attempted to calculate the building's base shear from the earthquake hazard spectra and the static design of the building is done using this base shear as lateral static force distributed along the side of the building in the form of inverted triangle or some other representative shapes. Evaluation of the performance of the building under dynamic load induced by the ground motion due to earthquake is an

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

essential step in the performance-based seismic design. There are some published works available on the evaluation of the performance of the buildings and performance-based seismic design. However, there is very limited study in that direction in the Canadian code context.

In the following sections the evaluation of the seismic performance of the steel moment resisting frame including a brief review on the seismic design concepts are presented along with a review of pervious research work. An overview of the concept of performance-based seismic design is also discussed.

2.2 Seismic design concept:

The basic principle of the seismic design of the buildings have been established a few decades ago and a formal design process is described in Newmark and Hall (1982) .As the knowledge in earthquake Engineering developed based on the research and experience from the past earthquakes the traditional force-based design approach is being modified to developed a performance-based approach. Steel Association of California (SAC, 2000) under contract to the Federal Emergency Management Agency (FEMA) prepared a seismic design guideline for new steel moment resisting frame buildings. In their report, SAC recommends some criteria for design of new steel moment resisting frame buildings including a basic design approach. The steel moment resisting frames are designed in such a manner that these frames have ability to undergo yielding and large

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

plastic deformation. This plastic deformation basically comes from the plastic rotation of the beam and it participates in the dissipation of the earthquake energy.

In the seismic design the ductility of the frame is one of the most important criteria. According to FEMA-350 (2000) this ductility of the steel-moment frame generate through the development of yielding in beam-column assemblies at the beam-column connections. In FEMA-310 (1998) the fundamental requirements for all ductile moment resisting frames are stipulated as follows:

(i) All frames should have sufficient strength to resist seismic demands,

(ii) They have sufficient stiffness to limit inter-story drift,

(iii) Beam-column joints have the ductility to sustain the rotations they are subjected to,

(iv) Elements can form plastic hinges, and

(v) Hinges should be developed in the beams before the columns at the locations distributed throughout the structure.

One important concept in the seismic design is strong column/weak beam concept or the *capacity design* concept, which means that, at a joint the beams will yield before the columns. This is enforced to prevent the brittle failure or soft-story mechanism in the building. The percent of strong column/weak beam joints in each story of each line of moment resisting frames shall be greater than 50% for Life Safety (*LS*) and 75% for Immediate Occupancy (*IO*) (FEMA-310, 1998). In designing of the steel moment resisting frames, the design of a connection is very important and sensitive. FEMA-350

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

(2000) has provided the design procedure and qualification data for various types of connections that can be used in the design of new steel moment resisting frame. Table 2.1, and Figures 2.1 and 2.2 show the pre-qualified connection details and calculation of demands at critical sections.

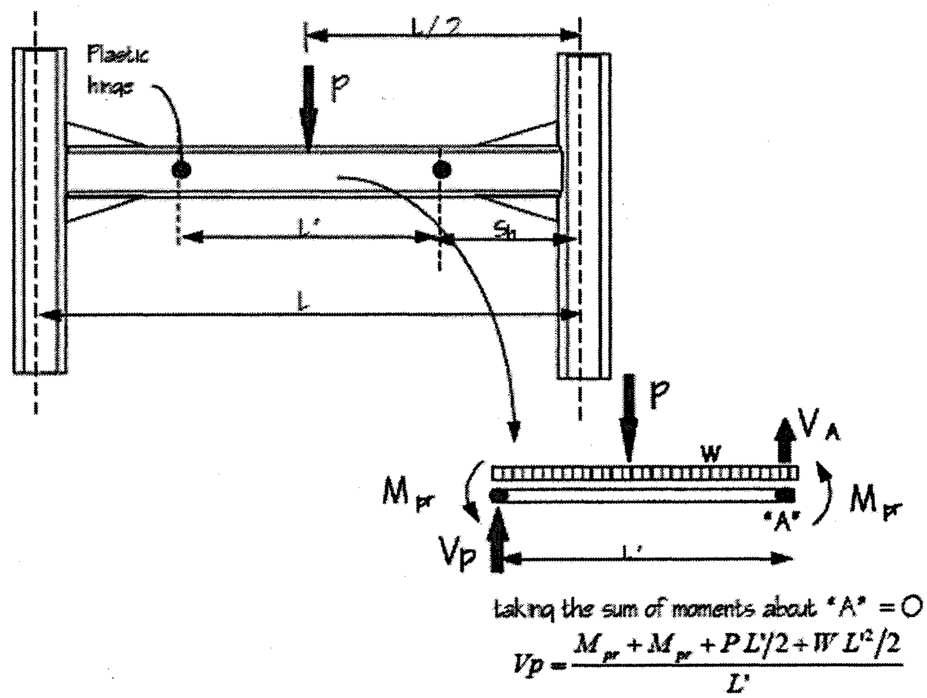

Figure 2.1 Sample Calculation of Shear at the Plastic Hinge (FEMA-350, 2000)

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

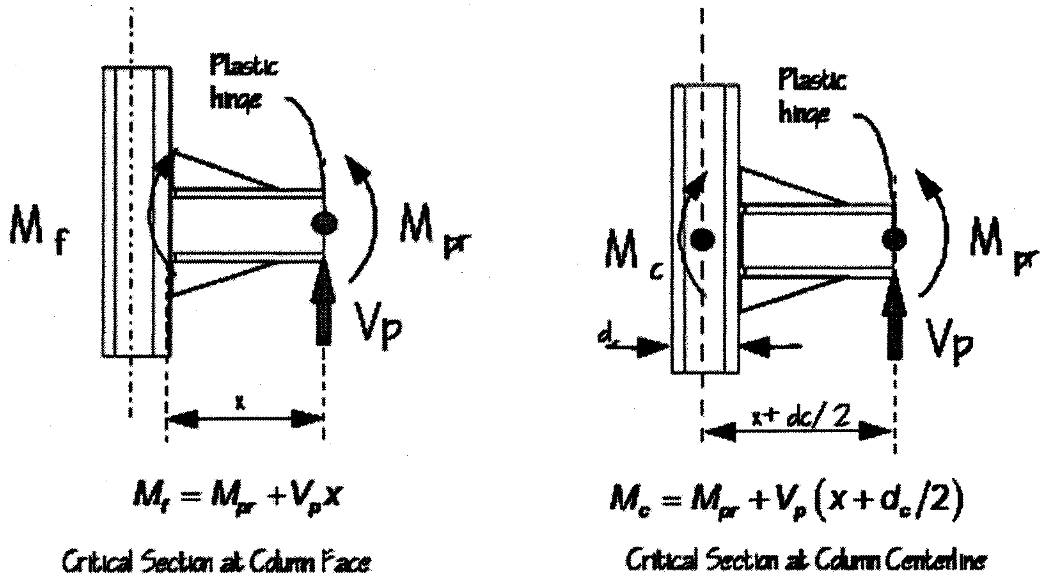

Figure 2.2 Calculation of Demands at Critical Sections (FEMA-350, 2000)

Table 2.1: Pre Qualified Connection details (FEMA-350, 2000)

Category	Connection Description	Acronym	Permissible Systems
Welded, fully restrained	Welded Unreinforced Flanges, Bolted Web	WUF-B	OMF
	Welded Unreinforced Flanges, Welded Web	WUF-W	OMF, SMF
	Free Flange	FF	OMF, SMF
	Welded Flange Plate	WFP	OMF, SMF
	Reduced Beam Section	RBS	OMF, SMF
Bolted, fully Restrained	Bolted, Unstiffened End Plate	BUEP	OMF. SMF
	Bolted, Stiffened End Plate	BSEP	OMF, SMF
	Bolted Flange Plates	BFP	OMF, SMF
Bolted, partially restrained	Double Split Tee	DST	OMF, SMF

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The connection used in the present work is in the category of welded, fully restrained joint which is welded un-reinforced flanges, welded web (WUF-W) type.

An informative discussion on the building analysis methodology defined in the National Building Code of Canada is presented by Saatcioglu and Humar (2003), where the application of the dynamic analysis in the computation of design earthquake action is discussed briefly. Different aspects of linear and non-linear analysis adopted in the NBCC 2005 are described there. Structural modeling including member modeling is also been discussed in the paper in details.

Calculation of seismic design forces by equivalent static load method according to NBCC 2005 has been presented by Humar and Mahgoub (2003). The paper contains a short discussion on the development of design spectral acceleration curve, formulation of base shear, and the effects of the higher modes on base shear and methodology of estimation of shear adjustment factors. The authors also have presented a comparative study on design features of moment-resisting frame and flexural wall. They have shown that the higher mode weights are relatively large in flexural walls compared to moment-resisting frames. The authors have also shown that the shear adjustment factor M_v to account for the multi Story effects, depends on only the modal periods and modal weights.

2.3 Evaluation of Performance and Performance-Based Seismic Design:

After the devastating Northridge (1994) and Kobe (1995) earthquakes the designers realized the importance of the evaluation of the performance of the statically designed

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

buildings for seismic load. Especially, from that point a new track in seismic design namely, the concept of Performance-based seismic design has been introduced. Evaluation of the structural and nonstructural performance is an essential part for performance-based design purpose. The main purpose of performance evaluation is to check whether the building is performing up to the desired level or not under dynamic forces induced by ground motion. In the performance design, usually a target level of performance is assumed, and the capacity of the structure is determined backwards. A performance objective consists of the specification of structural performance level and a corresponding probability that this performance level may be exceeded (Yun *et al* 2002).

A reliability based probabilistic approach has been adopted in FEMA-350 (2000) for the evaluation of the building's performance. As mentioned in FEMA-273 (1997) four levels of performance for building are defined; they are, Collapse Prevention, Life safety, Immediate Occupancy and Operational. To evaluate the performance of the structures, four distinct analytical procedures have been described in FEMA-273 (1997). These procedures are Linear Static, Linear Dynamic, Nonlinear Static and Nonlinear Dynamic Procedures. The performance of a building basically depends on the performance of both structural and non-structural elements and the level of performance is determined according to the damage of these elements. So, depending on the damage of structural and non-structural elements FEMA-350 (2000) adopted two mostly used level of structural performance, and these are Collapse Prevention (*CP*) and Immediate Occupancy (*IO*) as described in Table 2.2.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

A demand and capacity factor design (Yun *et a.,l* 2002.) concept can be used to determine the confidence level for the evaluation of the performance of the building. The confidence parameter is defined in Equation 2.1 (Yun *et a.,l* 2002).

$$\lambda = \frac{\gamma \cdot \gamma_a \cdot D}{\phi . C} \qquad \text{..} 2.1$$

Where γ is the demand variability factor, γ_a is the analysis uncertainty factor, D is the calculated demand of the structure, ϕ is the resistant factor, C is the capacity of the structure and λ is the confidence factor.

The parameters of the Equation 2.1 are described in FEMA-350 and by Yun *et al* (2002). Bertero and Bertero (2002) explained the performance-based engineering, performance-based seismic engineering and performance-based seismic design and discussed how these concepts can bring innovation in design and constructions. In that paper the performance-based design methods and the techniques to satisfy the objective of a reliable performance-based seismic design are discussed. A three-step approach for the formulation of the simple seismic code regulations is also discussed by Bertero and Bertero (2002).

Chopra and Goel (2002) have developed an important method for the performance evaluation of building, which is called Modal Pushover Analysis (MPA). MPA procedure is used for estimating seismic demands for buildings. It accounts for the higher mode contribution in the performance of the building through a series of static pushover analysis with mode compatible distribution of seismic lateral forces. In this method the

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

lateral force is applied according to the mode shape of the building and then the responses of the building is recorded. Various mode combination methods such as, the Square Root of Sum of Square (SRSS), weighted absolute sum, or absolute difference methods can be used to determine the peak modal responses. In the paper, the SRSS response is compared to the exact values obtained from the response history analysis. But in SRSS method both negative and positive responses are added up as they are squared. So, this combination may not represent the actual response of the structure and the comparison will not be perfect.

A new way of organizing the performance parameters of building including a discussion on performance chart as presented by Shustov (1999). Rating of story performance is presented in the paper and the story performance rating *(R)* can be defined as the ratio of calculated interstory *(v)* and an interstory drift at the assumed elastic limit of shear deformation *(*v_e*)*. The seismic performance charts represent the contours of equal seismic performance ratio (Shustov 1999).

A simplified method of pushover analysis for asymmetric buildings is presented by Kilar and Fajfar (1997). The method is illustrated with simple example where analysis was performed by using an event-to-event strategy and it was shown that this simple method is capable to estimate important non-linear structural behaviors including estimation of required ductility of the different macro-elements in relation to the target maximum displacement. Inverted triangular distribution pattern of load distribution was used in the analysis.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Table 2.2: Building Performance Levels (FEMA-350,2000)

	Building Performance Levels	
	Collapse Prevention Level	Immediate Occupancy Level
Overall Damage	Severe	Light
General	Little residual stiffness and strength, but gravity loads are supported. Large permanent drifts. Some exits may be blocked. Exterior cladding may be extensively damaged and some local failures may occur. Building is near collapse.	Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, ceilings, and structural elements. Elevators can be restarted. Fire protection operable.
Nonstructural Components	Extensive damage.	Equipment and contents are generally secure, but may not operate due to mechanical failure or lack of utilities.
Comparison with Performance intended By FEMA-302 for SUG-I buildings when Subjected to the Design Earthquake	Significantly more damage and greater risk.	Much less damage and lower risk
Comparison with Performance intended By FEMA-302 for SUG-I buildings when subjected to the maximum considered Earthquake	Same level of performance	Much less damage and lower risk

SUG = Seismic User Group

Lee and Foutch (2001) evaluated the performance of the new steel frame buildings to develop a probabilistic performance-based design method for such buildings. For the performance evaluation they developed a method called Incremental Dynamic Analysis (IDA). In the analysis they used a linearly varying scale factor for the whole analysis session. They used the statically designed structure as the base structure for the iterative

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

dynamic analysis. The base structure was designed to represent the post Northridge buildings, which were influenced by the 1995 Northridge earthquake.

Researches have also been done to find the effect of irregularities of building geometry on buildings performance. Tremblay and Poncet (2004) evaluated the performance of the steel frame building with mass irregularity that was designed by both equivalent static procedure and response spectrum method. To model the joints they used non-linear panel zone model. Tremblay and Poncet (2004) used the period and base shear of each dynamic analysis cycle of the Incremental Dynamic Analysis (IDA) as developed by Lee and Foutch (2001) and used that as the base shear for the next iteration. They used a confidence index parameter to determine the confidence level and the Equation 2.2 was used to calculate this index.

$$\lambda = \frac{\gamma \gamma_a D}{\phi_R \phi_U C} \qquad \text{...2.2}$$

Where D is the median estimate of the demand as obtained from structural analysis and C is the median capacity estimate, γ is the demand variability factor, γ_a is the analysis uncertainty factor, $\o_{R,,}$ & $\o_U$ are the resistant factors and λ is the confidence factor.

One of the earliest studies on the seismic performance of buildings designed according to the draft version of NBCC 2005 was reported by Bagchi (2001). The concrete moment resisting frame and shear wall buildings were considered in that study. The revised results were presented in Humar and Bagchi (2004). However, the seismic provisions in the final version of NBCC 2005 are somewhat different from those proposed in the draft version

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

considered in those studies. A similar study based on the current provisions will be useful.

A simplified method of evaluation of performance developed by Bagchi (2004) is available for evaluating the seismic performance of a multi degree of freedom system (MDOF) by converting it to single degree of freedom system (SDOF). In this method the peak response of SDOF is obtained by dynamic or response spectral analysis and a relation between the roof displacement and the maximum story drift of MDOF system is derived from the pushover analysis, and this relation will be used to interpret the response of SDOF obtained from dynamic analysis. Gupta and Krawinkler (2000) outlined a process for estimation of seismic roof and storey drift demands for the frame structure from the spectral displacement demand at the first mode period of the structure through a series of modification factors. They showed that the relation between the roof and the maximum storey drift demands depends strongly on the height of the structure.

A new methodology called the "Capacity-Demand Diagram Method" for evaluation of performance of inelastic structures is presented by Chopra and Goel (1999).In this method the performance is evaluated by determining the deformation demand from the graphical analysis of the capacity curve and the demand spectra presented in the acceleration-displacement (A-D) format as shown in Figure 2.3. The variation of viscous damping has been considered in the analysis. A simplified method was proposed, but the accuracy of the result is questionable as the variation in deformation obtained by this method and the response history analysis was found to be appreciable.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

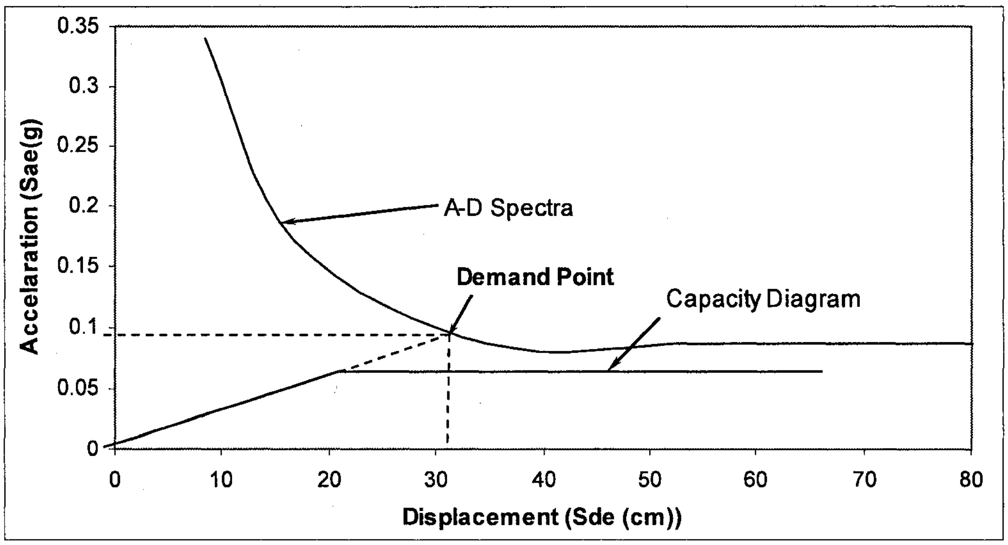

Figure 2.3: Capacity-Demand Diagram

A non-linear method of seismic analysis for performance-based seismic design was presented by Fajfar (2000). The method is called N2 method and the Acceleration-Displacement (A-D) format is used to formulate the method. It is based on the static pushover analysis of the multi-degree-of freedom (MDOF) system and the construction of an equivalent single-degree-of-freedom (SDOF). The pushover analysis of the MDOF system and the response spectrum analysis of SDOF are combined in this method and a modal participation factor is used for transformation of MDOF to SDOF. The A-D format (Similar to that in Fig.2.3 & 2.4) is used to calculate the elastic and inelastic spectra by the following Equations (Eq. 2.3 and 2.4):

$$S_{de} = \frac{T^2}{4\pi^2} S_{ae} \qquad \text{...2.3}$$

$$S_d = \mu \frac{T^2}{4\pi^2} S_a \qquad \text{...2.4}$$

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Where, S_{de} and S_{ae} are elastic displacement spectrum and acceleration spectrum and S_d & S_a are inelastic displacement and acceleration spectrum respectively, μ is the ductility and T is the period of the vibration.

In the N2 method (Fajfar, 2000) the elastic period of the structure and the characteristic period of the ground motion are used to determine the reduction factor for ductility. By using this method the elastic demand can easily be determined without constructing the inelastic demand spectra. But this method has some limitations such as the method is limited to planer analysis and also pushover analysis is approximated on a time-independent displacement shape.

A new and improved displacement-based design is introduced by Chopra and Goel, (2001). The method is called Direct Displacement-Based design; a simplified procedure is used in this method to estimate the seismic deformation of an inelastic single degree of freedom (SDF) system. Direct displacement-based design is being advocated as a more rational and relevant approach to seismic design of structures compared to traditional strength-based design (Chopra and Goel, 2001). A step-by-step procedure of the proposed direct displacement-based design using elastic and inelastic design spectra is presented is also presented by Chopra and Goel (2001).

A displacement-based seismic design has been presented by (Medhekar and Kennedy, 2000), where they show the advantage of the method over the spectral acceleration-based design method. In this method a single assumed shape of displacement is used to describe a multi degree of freedom system (MODF) as an equivalent single degree of freedom

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

(SDOF) system. The effective stiffness and effective displacement of SDOF is used to calculate the base shear of the MDOF. Estimation of the displaced shape of the structure play a vital role in this design process, so, the design process may not be suitable for all type of structure. But the advantage of the method is that there is no need for estimation of the fundamental period of the structure and the arbitrary force modification factor does not require.

Ghobarah (2001) has presented a philosophy of performance-based seismic design in his paper by reviewing methods related to performance-based seismic design. In this paper the author describes the design criteria based on the other published work on the concept of performance-based seismic design. A methodology of performance-based design is also presented in the paper.

The concept of performance-based seismic design is currently gaining momentum. In comparison to the traditional strength base seismic design the displacement-based seismic design will provide a better level of confidence by assuring the achievement of performance objective in the field. Humar and Ghorbaine-Asl (2005) introduced a new performance-based seismic design method where the roof displacement (Δ_u) is assumed to be the primary parameter and a target value is assumed according to the code (NBCC 2005) guidelines. The yield displacement (Δ_y) is calculated from the structural dimensions and material properties, and the ductility capacity (μ) is determined from the ration between Δ_u and Δ_y. The strength of the structure is then calculated from the inelastic demand spectrum corresponding to the ductility capacity. An example of the

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

inelastic demand spectrum in the Acceleration-Displacement (A-D) format is shown in Figure 2.4.

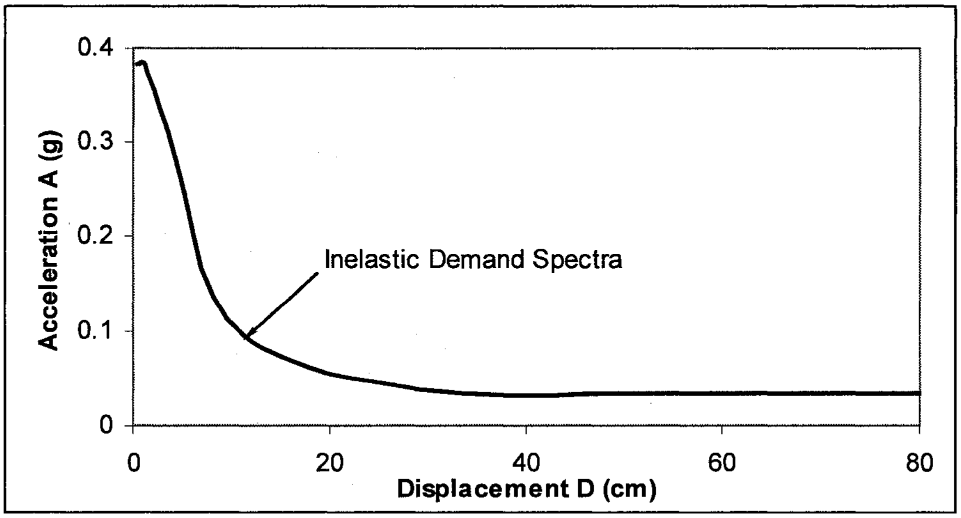

Figure 2.4: Inelastic Demand Spectra for Displacement-Based Seismic Design

The method described in Humar and Ghorbaine-Asl (2005) can be implemented in an iterative form where the initial design of the structure is performed with the assumed values of yield and the ultimate displacements. Also the multi degree of freedom system (MDOF) is converted to single degree of freedom system by assuming that the response of the structure to be predominately in the first mode of vibration. The contribution of mode of vibration does not account properly in this method.

A new form of Capacity Spectrum method for performance-based seismic design is presented using "Yield Point Spectra" by Aschheim (2004). In this method the yield displacement is determined kinematically and the yield displacement is calculated by using Equation 2.5 (Aschheim 2004).

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

$$\Delta_y = \left(\frac{T}{2\pi}\right)^2 S_a \qquad \text{..2.5}$$

Where, T is the natural period of the oscillator and S_a is the pseudo-spectral acceleration and calculated from the product of base shear co-efficient (C_y) and gravitational acceleration (g) and Δ_y is the yield displacement.

A bilinear yield point spectra is drawn for different ductility and the demand curves for the desired performance levels are superimposed in the yield point spectra (Fig.2.3). The yield point of the structure is determined from the pushover analysis of the equivalent single degree of freedom system of the structure and plotted on the spectra. If the yield point from the pushover analysis is above the superimposed curves of the performance levels than the designed will be satisfactory. But the method is not suitable for the structures in which the higher modes are prominent and this is the limitation of this simple method.

From the review of the above mentioned research work it has been observed that the performance-based seismic design is not well defined yet. The goal of all the performance-based seismic design is either checking of the force-based seismic design or evaluation of performance of the designed building. It should be noted that the performance objective has not been properly defined in the NBCC 2005. A single objective of performance is not enough to define the performance level of the structure. To evaluate the performance of a structure the multi-objective performance criteria is necessary which is defined in the Vision 2000 (1995) report.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

In evaluation of the seismic performance, the effect of non-structural element has not been accounted properly in any of the abovementioned work. Proper co-relation between evaluation of seismic performance and the performance-based seismic design has not yet been properly developed. In order to develop a performance-based seismic design methodology in the Canadian context, the interstory drift limits specified by NBCC 2005 can perhaps be used as the target performance objective in the evaluation of the performance of the buildings under seismic load. The code (NBCC 2005) specified interstory drifts are: $0.01h_s$ for post-disaster buildings, $0.02h_s$ for schools and $0.025h_s$ for all other buildings. Where, h_s is the story height of the building.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Chapter 3

Design of Building Frames

3.1 General:

For evaluation of the performance in this research work four buildings of five, ten, fifteen and twenty story height with regular geometric shape are considered. A typical floor plan and elevation of frames are shown in Figures 3.1 and 3.2 respectively. The buildings are of steel moment resisting frame type and located in Vancouver, Canada. Vancouver represents a location with higher seismicity as compared to other parts of the country. The building frames along the north-south direction have been designed. Each building consists of series of frames in the east-west (E-W) direction to resist the lateral loads and three bays in the north-south (N-S) direction. In the N-S direction two exterior bays are of 9 meters and the interior one if 6 meters and center to center spacing of the frames in the E-W direction is 6 meters. The first story height of the building is 4.85 meter and others are of 3.65 meter each. The frames are symmetrical along the vertical center line of the frames. Therefore, accidental torsion has not been considered in the design of the frames.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

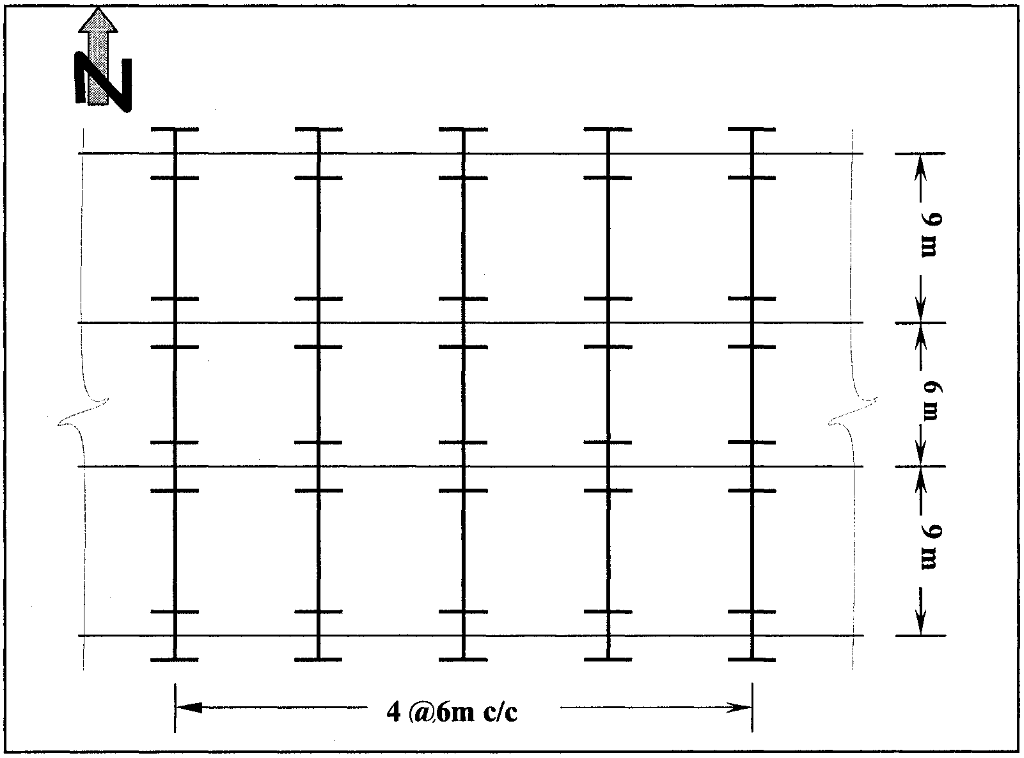

Figure 3.1: Typical Plan of the Buildings

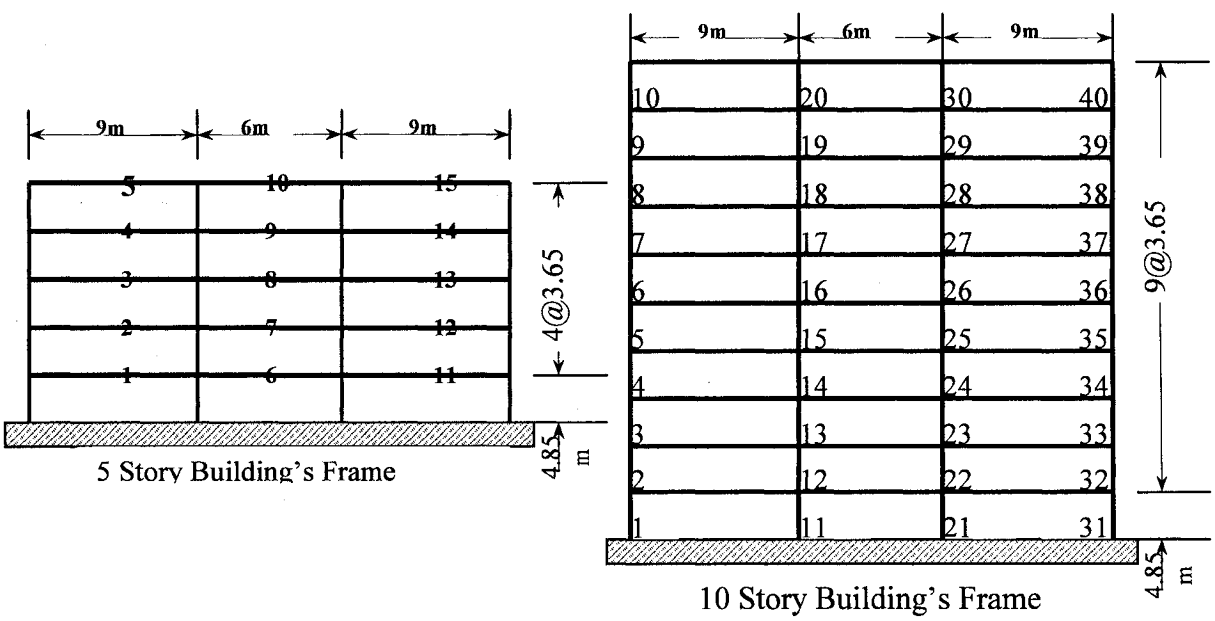

Figure 3.2 (a): Elevation of 5 & 10 Story Frames

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

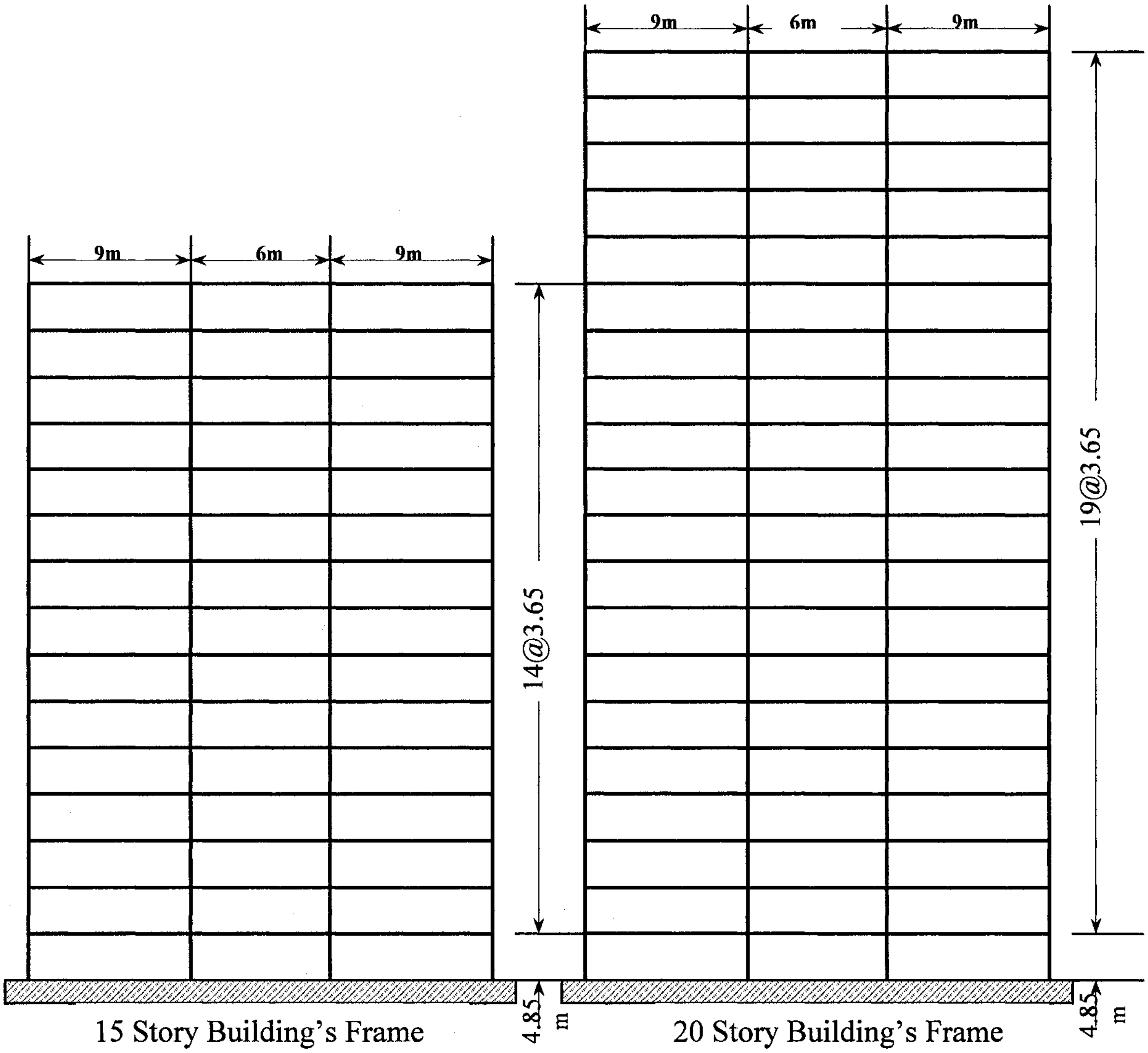

Figure 3.2(b): Elevation of 15 & 20 Story Frames.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

According to NBCC 2005 seismic provisions, the design considers the earthquake events with a probability of exceedance of 2% in 50 years (Herrera *et a.,l* 2003). Like any other structural design, the code defined force-based seismic design involves two steps: First one is the calculation of the member forces and the Second one is the design of the members according to code guideline to withstand the calculated factored forces safely. The effect of non-structural elements in a building has been accounted for by considering the infill panels in the building frame. The typical elevation of infill frame is shown in Figure 3.3.

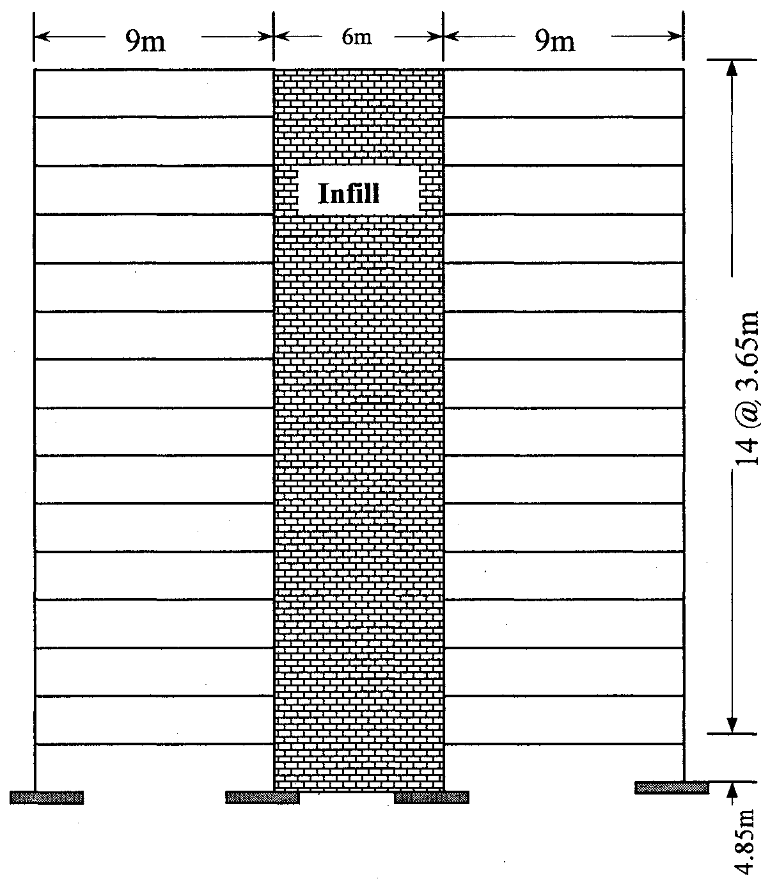

Figure 3.3: Elevation of Typical Infill Frame

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

3.2 Application of Computer Programs:

3.2.1 General:

Application of computer in Civil engineering brings a radical change in the design productivity. Large scale complicated work can be been done with outstanding power of modern computer. It expands the scope of Civil engineering design with more accuracy and flexibility than before, brings financial benefit by reducing implementation time and effort. Performance-based seismic design relies on the interactive process of performance evaluation through inelastic structural analysis, which is possible only through computer applications. In seismic design enormous computing effort and powerful software are required to conduct the detailed dynamic analysis by using a number of multiple ground motion records. Also modeling of a structure to represent in computer for the evaluation of performance of the structure is very important as the accuracy of the analysis completely depends on the model provided in the computer. The difference in modeling and software tools produce some variability in the solution and the uncertainty associated with it needs to be considered in performance evaluation.

3.2.2 Selection of computer programs:

There is a number of commercial and non-commercial software tools available for elastic and inelastic dynamic analysis of structural systems. Examples include DRAIN-2DX (Prakash *et al.*, 1993), DRAIN-BUILDING (NISEE, 2005), DRAIN-RC (DRAIN-RC, 2006), IDARC2D (IDARC2D, 2006), SAP-2000 (SAP, 2000, 2006), ETABS (NISEE 2005). Among these DRAIN-2DX is more popular to researchers because of its flexibility, the ease of use and availability. DRAIN-2DX is a general purpose computer

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

program for static and dynamic analysis of plane structures (Prakash *et al.,* 1993). All kind of analysis such as analysis for calculation of member force for static design, linear and non-linear static and dynamic analysis to evaluate the performance of the structure can easily be done with this software. It has some pre-defined functions to facilitate a number of different type of analysis necessary for seismic engineering. Some of such functions are GRAV for gravity analysis of elements and nodal loads, STAT for nonlinear analysis, ACCN analysis for ground acceleration, MODE for modal analysis of the building. In DRAIN-2DX the structure needs to be modeled in two dimension where the elements are connected at the nodes. All elements are categorized into different types and information of the elements is input according to the type. Modeling of steel structure in DRAIN-2DX is easier because of well defined element behavior (e.g. elasto-plastic or bilinear material hysteretic behavior). As a part of the present research, Yousuf *et al.* (2006) automated the input and output data handling for DRAIN-2DX and generate the graphical plots of the response quantities. El Kafrawy *et al.* (2006) developed a similar system for analyzing reinforced concrete buildings using IDARC2D.

3.2.3 Automation of DRAIN-2DX:

Like any other software DRAIN-2DX also has some limitations. Preparation of the data and interpretation of the output file one the main difficulties as there is no graphical user interface. Designing a structure by collecting data manually from the DRAIN-2DX's output files increases the probability of error. Also evaluation of performance of structure by using output file of analysis done by DRAIN-2DX is not an easy job especially when

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

a large number of repetitive analysis using multiple ground motion records are necessary. So, automation of this software becomes necessary to ensure the efficient use of this tool. For this work a set of MATLAB-based computer program have been developed to use as pre and post processors and to interpret the output of DRAIN-2DX for the determination of the maximum, mean and standard deviation of the response parameters, such as, interstory drift, maximum roof displacement and base shear from dynamic analysis. Push over graph can also be drawn to calculate the lateral load capacity by using the automated program. The programs developed here are used for generating the graphical plots of the response quantities in MATLAB format (Yousuf *et al.*, 2006). The automation of DRAIN-2DX is summarized in Figures 3.4 and 3.5.

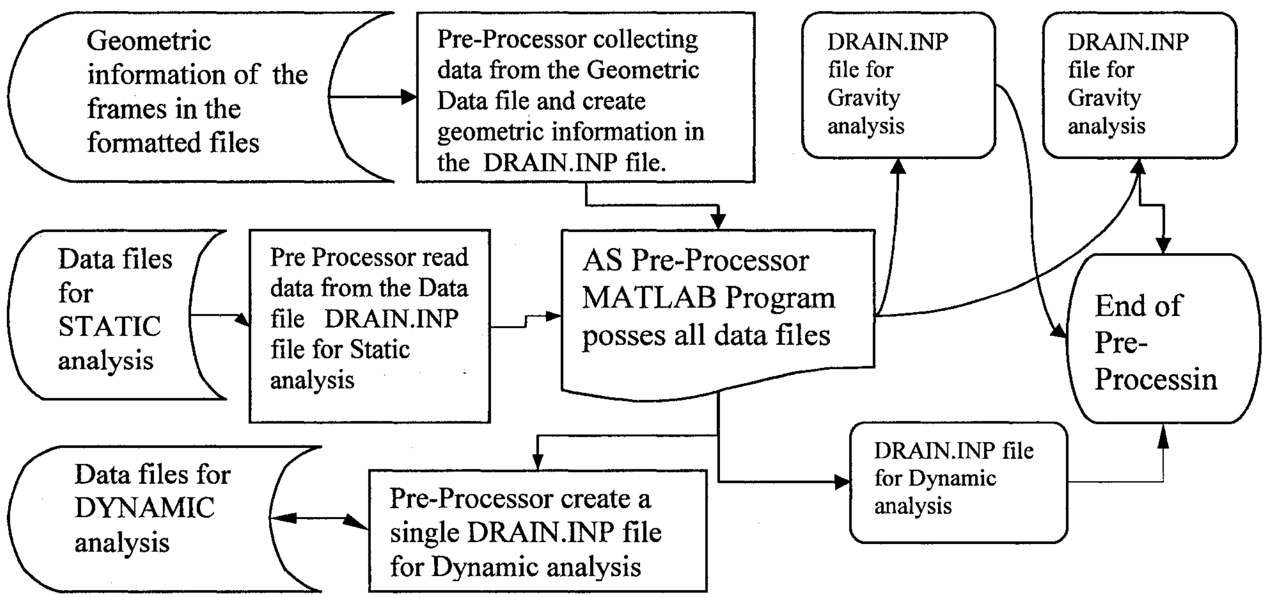

Figure 3.4: Pre- Processor of DRAIN-2DX (Yousuf *et al* 2006)

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

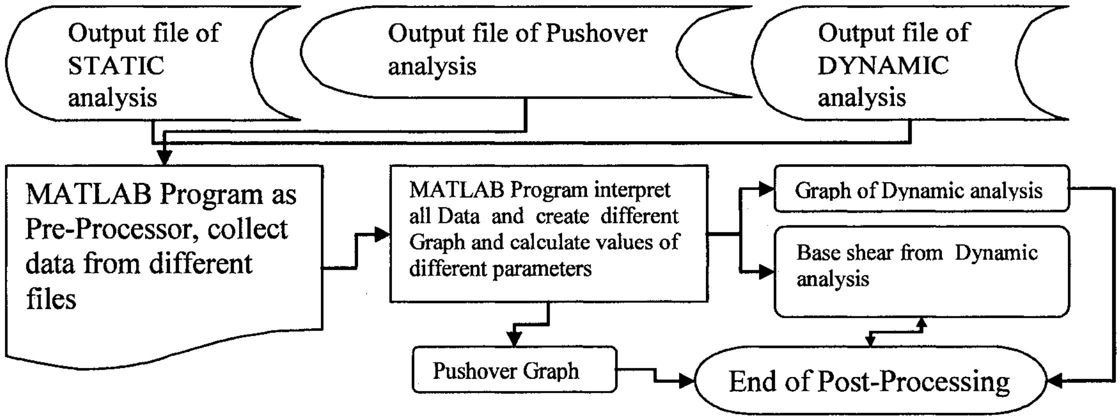

Figure 3.5:Post-Processor of DRAIN-2DX (Yousuf *et al.,* 2006)

3.3 Structural Modeling:

Ductile steel moment resisting frames (SMRF) are modeled for the analysis. For the simplicity of the analysis and design the exterior and interior frames are kept similar. Therefore, only one interior frame with mass of its tributary area for each type of building has been designed and analyzed for and used in the performance evaluation. So, the three dimensional main structure has become two dimensional in design and analysis. The beam members of the same floor level are grouped in the same section type and the column sections are changed at every sixth level i.e. columns are spliced at every fifth floor level. Column continuity represents a benefit in seismic resistance as it helps redistributing the inelastic demand along the building height (Tremblay and Poncet 2005). A simplified model of the frames has been developed by assuming 5% strain hardening ratio for steel. For capacity based design model the elements of the frames are detailed to develop ductile response under cyclic inelastic deformation due to seismic action and other elements including connections are detailed to remain elastic under gravity load and the maximum earthquake induced lateral load.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Frames of the building are modeled by using the plastic hinge beam-column element available in DRAIN-2DX (Element type-2). The type-2 element of DRAIN-2DX is an inelastic element which is necessary for modeling of the frames. P-M interaction curve and the yield surface can be defined to consider the effect of axial force on bending strength. Strain hardening can also be modeled in this element. The geometry of the element type-2 of DRAIN-2DX is shown in figure 3.6 (Prakash *et al.,* 1995). In the model the mass was lumped to the joint without modeling any diaphragm. The connections of beam to column are assumed to be rigid and chosen from the FEMA-350 (2000) predefined connection type. The category of predefined connection is welded, fully restrained. Because of capacity-based design the column and beam sections are chosen in such a way that at any joint the sum of capacity of the columns are greater than the sum capacity of beams. It is important for yielding of the beams before the columns in a joint, during earthquake.

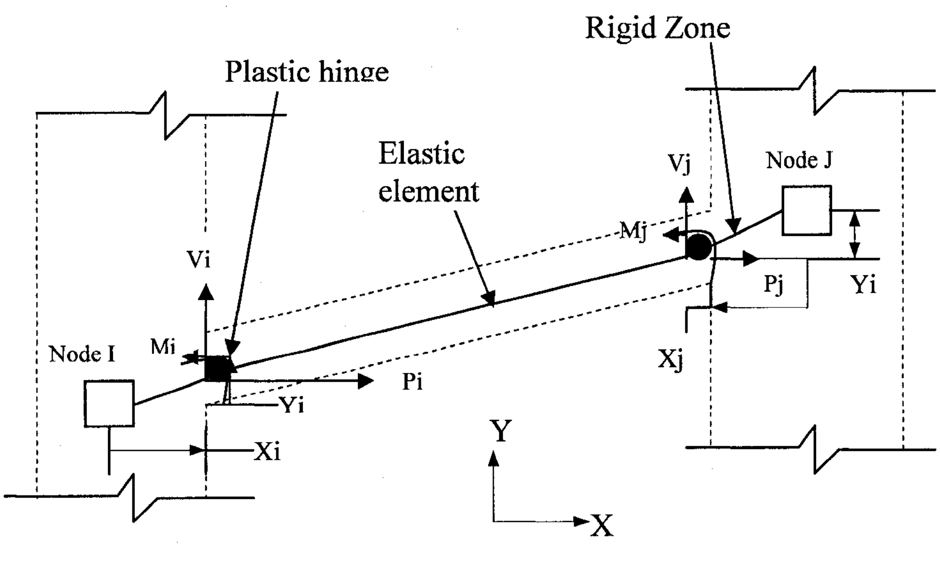

Figure 3.6: Model of Element Type-2 of DRAIN-2DX (Prakash *et al.,* 1995)

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

According to the current practice, contribution of the non-structural elements is not considered in the lateral load resisting capacity of a frame. Thus for modeling of the bare frame the non-structural element has been ignored. However, the non-structural element, in reality, would contribute to the overall performance of the structure. To simulate that the moment resisting-frames are also modeled with infill panels to study the effect of the non-structural elements on the performance of the buildings subjected to earthquakes. The infill panels are modeled as compression strut in the mid bay at each story level of buildings frames. Two inclined strut at each story level are used as shown in the figure 3.7. Two dimensional symmetric frames are modeled for each building to avoid twisting. For modeling the infill panels clay masonry of 100 mm thickness and compressive strength of f_m =8.6 MPa has been used. The effective width *(w)* is calculated from the theory of beams on elastic foundation (Drysdale *et al* 1994) and the expression is given in the Equation 3.1.

$$w = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_l^2} \qquad \text{............................} \qquad 3.1$$

Parameters α_h and α_l used in the above Equation are calculated from the following expressions (Drysdale *et al* 1994).

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

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

Where, *I, t, h* and *l* are respectively; moment of inertia of area of beam and column, thickness, height and length of the infill panel. E_m and E_f are elastic moduli of wall and

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

frame materials. The angle θ can be calculated by using the expression, $\theta = tan^{-1}(h/l)$. Also the elastic modulus of the masonry can be defined as, $E_{ma} = kf_m$, where k is a constant and for masonry $k=500$. Therefore, $E_{ma} = 4300MPa$.

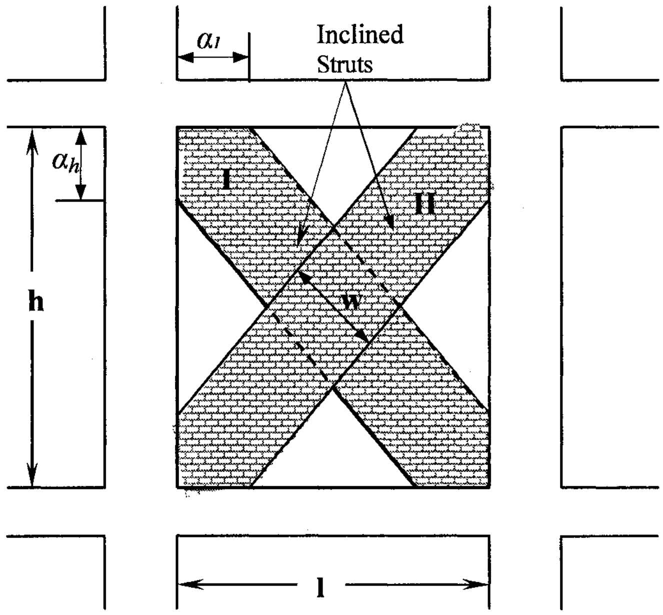

Figure 3.7: The model of infill panel

The inclined strut acts as a truss member and resists compressive force only. When load applied from left side on the frame the Strut-I goes under compression and Strut-II under goes in tension. But as clay masonry very weak in tension, almost negligible, so, Strut-II remains inactive. When the lateral load is reversed, Strut-II goes in compression, while Strut-I becomes inactive. Thus, the infill panels increase the stiffness of the frame and overall lateral load carrying capacity.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

3.4 Design of the Building Frames:

Building frames are designed to satisfy the NBCC 2005 requirements and the steel structural elements have been designed according to CSA S16-01. The equivalent static lateral load procedure for the seismic load as prescribed by NBCC 2005 has been used in designing the buildings. The following loadings have been considered in the design: gravity loads (dead load *(D)*, live load *(L)*) and seismic load *(E)*. The gravity loads from the live loads are calculated according to NBCC 2005 and values are presented in the Table 3.1. The dead loads comprise the self weight of the frame elements and other non-structural components. The total weight of the building has also been calculated same way at each iteration of the static design. Live load at the roof is mainly snow load *(S)*.

Table 3.1:Design loads.

Dead Load (kPa)		Live Load (kPa)		
Roof	Floor	Roof	Interior typical floor	Corridor
3.4	4.05	2.32	2.4	4.8

Design base shears are calculated by using Equation 1.1 taken from NBCC 2005. Linear gravity analysis of frames has been done using DRAIN-2DX to determine the member forces. The base shear is distributed along the height of the frame in the form of inverted triangle as suggested in NBCC 2005, and the force is assigned to each story level according to the weight and the story height of the respective story level. Seismic force of the specified story level is calculated by using Equation 1.3. Non-linear static analysis is used to calculate the forces of the frames elements for seismic force. The effect of P-Δ has been considered in the analysis. Different combination of the forces is used to evaluate the design force for both beam and column of the frames. The combinations of different loads are shown in the Equation 3.4 and 3.5.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

a. $1.25D+1.5L$ 3.4

b. $1.0D \pm 1.0E + (0.5L+0.25S)$ 3.5

In static design it has been ensured that the structure is safe for the combination of gravity loads only and then the combination of earthquake loads is used to check whether the structure design for gravity loads is adequate for sustaining the equivalent seismic load or not. If the structure designed for gravity load fails to withstand the seismic load, the design has been modified to satisfy the both *a* and *b* combinations of the loads. During this design process the empirical fundamental periods of the frames has been calculated by using the Equation 3.4.

$T_a = 0.085(h_n)^{3/4}$ (NBCC, 2005) 3.6

Where T_a is the empirical fundamental period and h_n is the total height of the frame. This period has been used to calculate the equivalent seismic force for the first iteration. After designing of frames by using the empirical fundamental period a detail modal analysis of frames has been done. From the modal analysis the fundamental periods of frames has been calculated and if the fundamental period has found more than the period, T_a obtained from Equation 3.6, the seismic force has been revised using the modal period or $1.5T_a$, whichever is smaller (NBCC, 2005). A summery of the periods of different frames is depicted in the Table 3.2. Modal analysis has been done for both bare and infill panel frames.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Table 3.2: Fundamental Periods of the Buildings.

Frame Height	By Empirical Equation (Eq.3.6), (T_a) sec	Modal analysis, sec		$1.5T_a$ sec
		Bare Frame	Infilled Frame	
5 story	0.787	1.412	1.077	1.181
10 Story	1.293	2.528	1.993	1.939
15 Story	1.739	3.571	2.935	2.609
20 Story	2.149	4.789	4.008	3.224

From the importance point of view the buildings are designed as normal buildings and the frames are assumed to be fully ductile. The parameters used in Eq.1.1 for calculation of equivalent seismic force are: importance factor $I_E=1.0$, factor for higher mode effect $M_v=1.0$, ductility factor $R_d=5.0$ and the force reduction factor $R_0=1.5$. A soil type- C which is very dense soil and soft rock, is considered with a site specification factor $F_v=F_a=1.0$.Therefore, the design spectral acceleration value ($S(T)$) has become equal to the spectral acceleration value($S_a(T_a)$) as provided in the code. The equivalent base shear of the four buildings calculated using the fundamental period obtained from modal analysis is presented in the Table 3.3.

Table 3.3: Base shear of different building.

Building Height	Base Shear V (kN)	
	Bare Frame	Frame with infill panel
5 story	154.70	161.91
10 Story	192.44	193.14
15 Story	293.75	294.85
20 Story	400.96	402.49

The base shear shown in the Table 3.3 is the design base shear used in the final design. The fundamental period determined by the modal analysis and $1.5T_a$ whichever is smaller is used for recalculating base shear. A sample calculation of the base shear after modal

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

analysis is shown. in Table 3.4. During selecting the design base it has been checked that the design base shear is greater than or equal to the base shear calculated for spectral acceleration S(2.0)g and should be less than 2/3 of base shear corresponding to acceleration S(0.2)g.

If there is any variation of the base shear after modal analysis the design of the buildings has been revised with the new base shear and make necessary modification in the sections of the frames. It has been observed that the modal analysis does not effect the design of the medium to high rise building (20 story and up) but it change the base shear of low rise (5 to 15 story) building from 15% to 20%.

Table 3.4: A sample calculation of base shear after modal analysis.

Modal Period	1.5 T_a	Selected Period	$S(1.0)g$	$S(2.0)g$	$S(\text{Design})g$	M_v (1.0)	M_v (2.00)
4.789	3.223	3.223	0.340	0.180	0.180	1.00	1.10
M_v (Design)	Weight, W (kN)	Factor R_d	Factor R_0	Base Shear after modal analysis, V_m (kN)		Base shear before modal analysis, V_s (kN)	
1.10	15187	1.50	5.00	400.96		400.96	

Type-D ductile frames are designed according to the clause 27 of CSA S16-01 (2001) and presented in the CISC's (2004) "Handbook of Steel Construction". The steel sections used in the design are of CSA G40.21 with yield strength, F_y=350 MPa for both beam and column. Beam-Column concept is used in designing of column to avoid yielding and buckling. The modulus of elasticity of steel *(E)* is 200x10^3 Mpa. The column strengths are checked by using the Equation 3.7, which is adapted from CISC (2004) to be applied to plane and unbraced frames.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

$$\frac{C_f}{C_r} + \frac{0.85U_{1x}M_{fx}}{M_{rx}} \leq 1.0 \quad \text{..............................3.7}$$

In the Equation 3.7 the constants U_{1x} is taken as 1.0 because of unbraced frames. The factored axial compressive force (C_f) and factored moment (M_{fx}) are obtained from the analysis. M_{rx} is the resistive moment about X axis. The resistive compressive axial force (C_r) and resistive bending moment (M_{rx}) for the respective column are taken from the CISC's (2004) handbook. Individual column is designed as beam-column element to avoid yielding and flexural buckling.

Beams are designed to fulfill the Limit States criteria of the CAN/CSA-S16-01(2001). Factored beam shear (V_r) and moment resistant (M_r) are taken from the "Handbook of Steel Construction" (CISC 2004).The factored resistant calculated here are compared with the specified factored resistance and checked against the following conditions: $Vr > V_f$ and $Mr > M_f$. Design iteration continued till these criteria satisfied. Deflection of a beam has been checked for live and dead loads to satisfy the serviceability limit state, and the deflection has been calculated by using Equation 3.8.

$$\left.\begin{array}{l} I_{reqd} = WC_d B_d \\ \Delta = \left(I_{reqd}/I\right)\Delta_m \end{array}\right\} \quad \text{.. 3.8}$$

Where I_{reqd} is the required moment of inertia of area, I is the gross moment of inertia, Δ_m is the specified maximum deflection, Δ is the calculated deflection, C_d is the value of deflection constant and B_d constant subjected to load and support. W is the total live load subjected to the beam.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

As a part of the check for capacity based design, the column and beam capacities at shake down condition have been calculated by using the following expression given in "Handbook of Steel Construction" (CISC 2004). The shake down condition can be defined as the condition when the system behaves elastically after initially yielding under cyclic loading.

$$\sum M_{rc} \geq \sum\left(1.1\,R_y\;M_{pb} + V_h\left[x + \frac{d_c}{2}\right]\right) \qquad \text{3.9}$$

$$M_{rc} = 1.18\;\phi\;M_{pc}\left(1 - \frac{C_f}{\phi C_y}\right) \leq \phi M_{pc} \qquad \text{3.10}$$

Where, M_{rc} and M_{pb} are moment of resistance of the column and the beam plastic moment respectively. Ø is resistance factor, V_h is shear acting at plastic hinge locations when plastic hinging occurs, C_f is factored axial compressive load of column, C_y is axial compressive load at yield. R_y is a factor applied to F_y to estimate the probable yield stress where F_y is the specified minimum yield stress. In seismic design this check of capacity is mandatory because after the shake down plastic hinges are developed mainly in beam at a certain specified distance from column center line and this distance depends on the type of connection of beam and column at this stage column carry all the loads. The distance of plastic hinge from the center of the column, for the connection chosen for this work is $x+d_c/2$ (Fig.2.1 and 2.2) .Where, d_c the depth of column and x is the distance of the plastic hinge from the face of the column. All joints of every frame considered here

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

have satisfied this capacity design criteria. The finalized sections for different elements of the frames are presented in the Table 3.5. A flow chart of the whole design process is presented in the Figure 3.8, which has been adapted from (Hannan, 2006) and modified.

Table 3.5 (a): Section of Beams.

Story Level	Building Height			
	5 Story	10 Story	15 Story	20 Story
Top Story	W310x79	W310x79	W310x107	W310x107
Other Story	W310x86	W310x107	W310x129	W310x129

Table 3.5(b): Sections of Columns.

Building Height	Column Row	Story 1 to 5	Story 6 to10	Story 11 to 15	Story 16 to 20
5 Story	External	W310x179			
	Internal	W310x253			
10 Story	External	W310x283	W310x158		
	Internal	W310x314	W310x202		
15 Story	External	W310x283	W310x253	W310x179	
	Internal	W360x314	W360x260	W310x283	
20 Story	External	W310x283	W310x253	W310x202	W310x179
	Internal	W360x314	W360x287	W360x262	W360x262

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

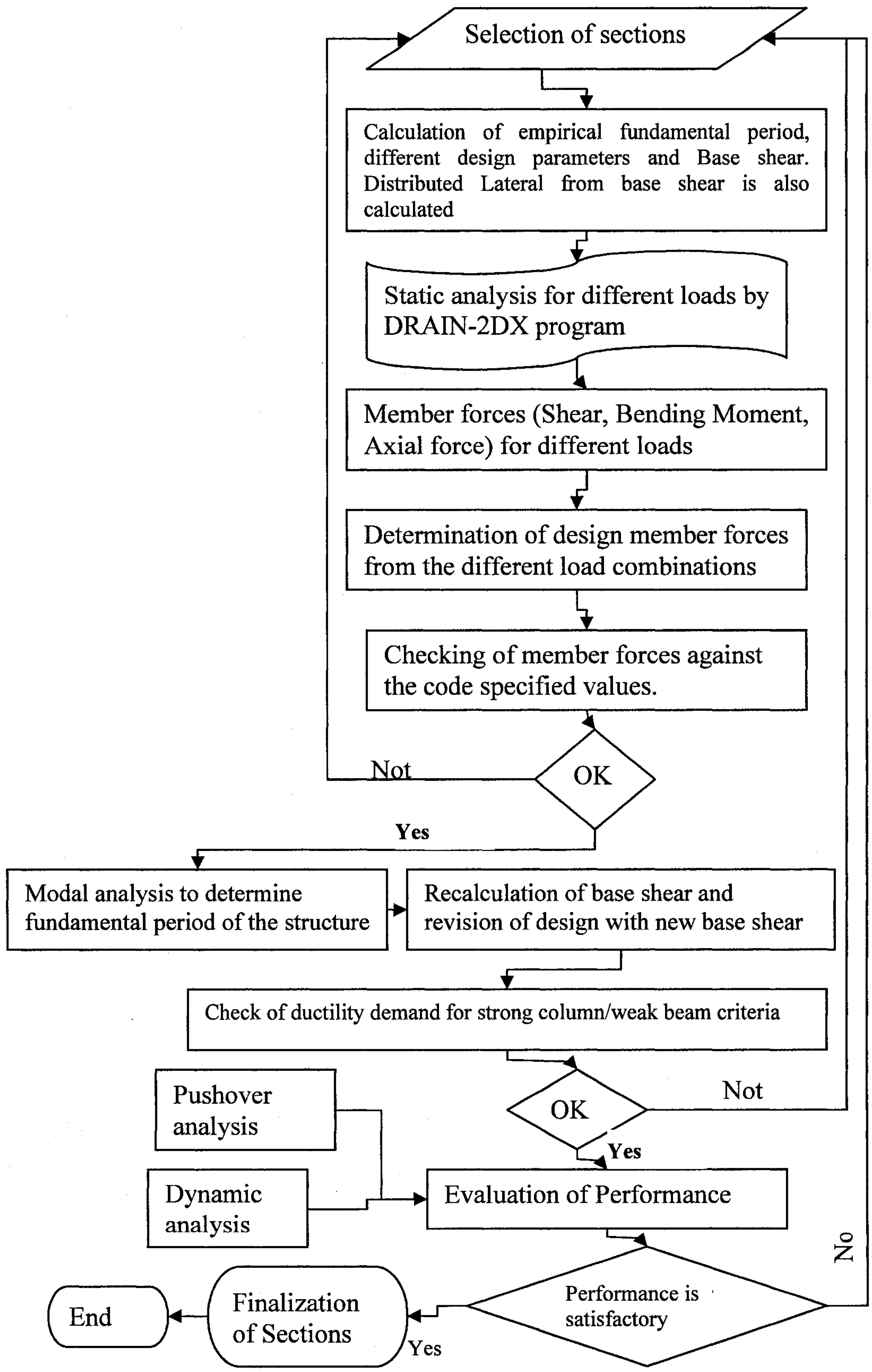

Figure 3.8: Flow chart of the design process.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Chapter 4

Evaluation of the Seismic Performance of Buildings

4.1 Introduction:

Performance of a structure can be defined as the response of the structure to an action imposed upon it. The seismic performance of a building can be defined as the response to the ground motion during earthquake and three performance objectivities as define by SEAOC (1995) are: resist minor earthquake without damage, moderate earthquake with some damage to non-structural but no structural damage and major earthquake without collapse. So, the performance of the structure is a coupling of expected performance levels with levels of seismic ground motions (Bertero and Bertero, 2002).

Lateral load resisting capacity and interstory drift are two main parameters used in quantification of the performance of a building. A building can be designed for equivalent seismic loads for pre-defined level of confidence which comes from the target interstory drift. But it may not be possible to achieve this level of confidence in reality because of the uncertainties associated with the design assumptions. These uncertainties are related to the properties of the structure, yield strength of the components or the elements, the presence of inherent damping and also the weight of the building which may not be properly defined. These uncertainties can neither overcome nor be avoided

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

because the mass of the structure, the period of fundamental vibration and the damping producing factors present in the structure have a great influence on the response of the building. Therefore, the evaluation of the seismic performance of the building under seismic action is very important. According to Gupta and Krawinkler (2000) the evaluation of performance of the structures necessitates the ability to predict global (e.g. roof), inter-mediate (e.g. story) and local (element) deformation demands. The estimation of dynamic characteristics and prediction of the building's response to the seismic ground motion is a way of evaluating the performance of a building. Modal analysis can be carried out to estimate some of the dynamic characteristics such as the periods and mode shapes.

4.2 Methodology of Performance Evaluation:

Though the methodology of the evaluation of the performance of structure is still under development but some linear and non-linear static and dynamic methods have developed and are widely used in evaluation of the seismic performance of structures like buildings. Performance evaluation methodologies can be implemented in two ways: First, by evaluating the performance of the structure designed for the equivalent static load, through some rigorous analysis and Second, by ensuring the level of performance through the performance-based seismic design. Modeling of the structure for analysis is very important in the evaluation of performance; the analytical model must simulate the behavior of the frame well. A simplified hysteresis model, elastic-perfectly-plastic type (Mazzolani *et al* 2002), is used to carryout the evaluation of performance of the steel moment resisting frame building. Non-linear time history analysis provides the maximum

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

interstory drift and roof displacement. Lateral load resisting capacity is determined through static non-linear push-over analysis.

4.2.1 Modal Analysis:

Modal analysis is performed to obtain the mode shapes of the frames and to determine natural frequencies of the frames. The modal analysis has been done by using the *MODE function in input file of DRAIN-2DX computer program. The mode shapes of different frames are shown in Figures 4.1 and 4.2. The natural period obtained from the modal analysis and the empirical period (Eq. 3.6) has been used in revised design of the structure.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

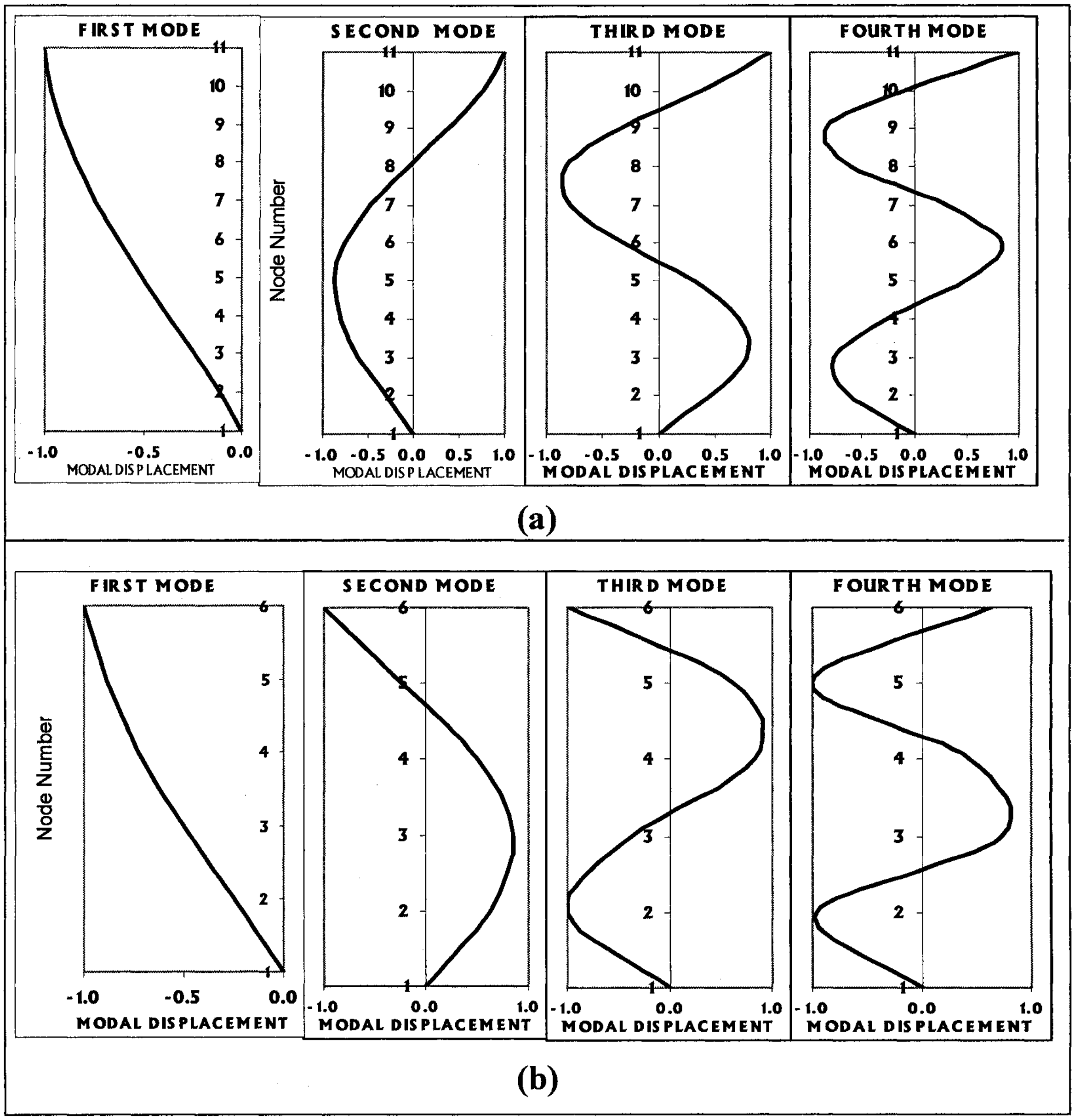

Figure 4.1: Mode Shapes of 10 and 5 Story Building Frames; (a) Mode Shapes of 10 Story Building Frame, (b) Mode Shapes of 5 Story Building Frame.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

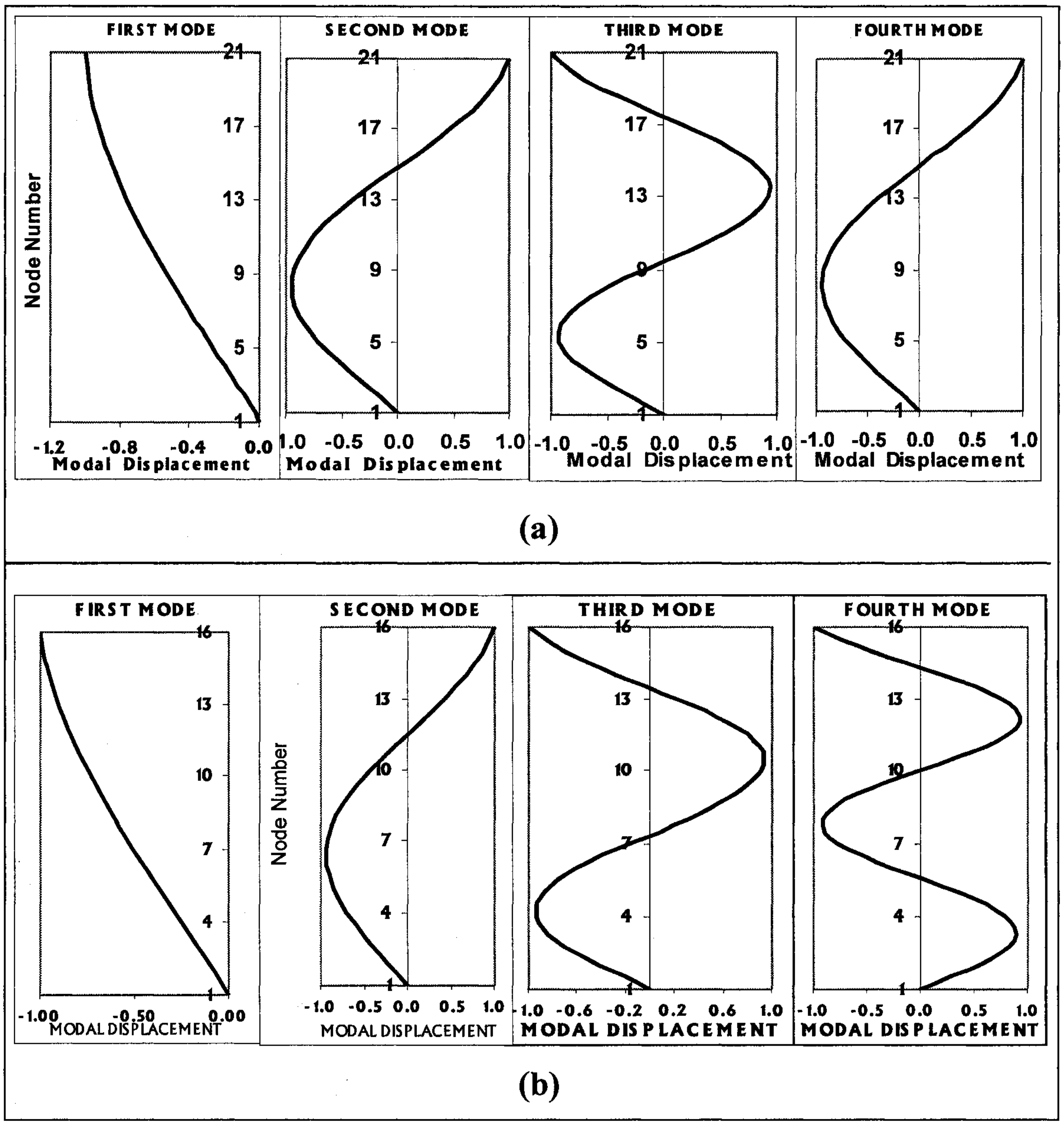

Figure 4.2: Mode Shapes of 20 and 15 Story Building Frames; (a) Mode Shapes of 20 Story Building Frame, (b) Mode Shapes of 15 Story Building Frame

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

4.2.2 Pushover Analysis:

Pushover analysis is a non-linear static analysis method to evaluate the lateral load carrying capacity of buildings. This analysis method is a very important tool for evaluation of seismic performance and performance-based earthquake-resistant-design (SEAOC Vision 2000), which concerned with the identification of the hazards, selection of performance criteria, and objectives with desired performance level. Pushover analysis is preferred for structures where higher mode effect is not significant (Chopra, 2002) to non-linear dynamic time history analysis as suggested in the FEMA-273 as a reliable way of estimation of seismic demands.

The pushover analysis of buildings designed for equivalent static force has been performed by applying estimated equivalent seismic lateral forces, which has been monotonically increased. The structures have been pushed to a predetermined target displacement or collapse level of structures and the roof displacement history has been recorded. The computer program DRAIN-2DX has been in analysis for plane two dimensional models of the frames. The pushover analysis has been performed using inverted triangular load distribution patterns for all frames. Through the pushover analysis a curve called capacity curve has been drawn for each building. A capacity curve is a plot of base shear vs. roof displacement (Akbas *el at* 2003). The base shears has been used here are the normalized base shear (Herrera *et al* 2003) or the base shear coefficient which is defined as the ratio of the seismic base shear *(V)* to the weight *(W)* tributary to the frame of the building.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

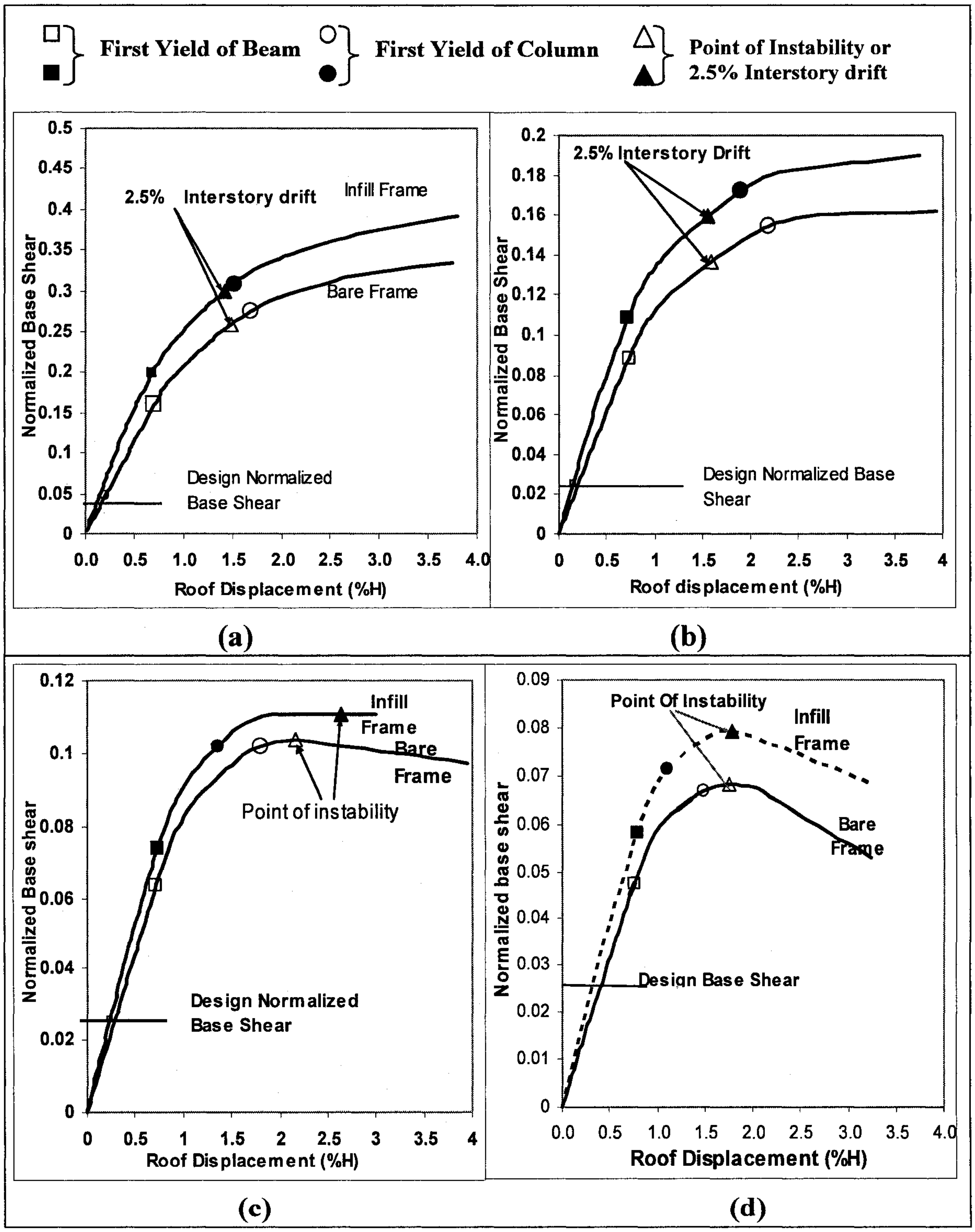

Figure 4.3: Pushover Graphs; (a) Pushover Graph of 5 Story Building (b) Pushover Graph of 10 Story Building (c) Pushover Graph of 15 Story Building (d) Pushover Graph of 20 Story Building

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The pushover graphs of different frames are shown in Figure 4.3. On the graph the point of first yielding of beam and column is shown and the point of instability of the frame is also marked on the graph.

The point of instability can be defined as the point where the slope of the pushover graph tends to be negative (i.e. the curve moves downwards with respect to the horizontal line). Some frames deformed beyond the 2.5% interstory drift at that case the point corresponding to 2.5% interstory drift are marked on the graph. The analysis is carried out by considering 5% strain hardening. P-Δ effect has been considered in the pushover analysis to account for the second order effect. By allowing the P-Δ effect the effect of large deformation has been considered. The capacity of the frames is calculated from the pushover graph by calculating the yield displacement due to seismic load. Analysis of both bare frame and frame with infill panel is done. In the analysis the gravity load *(D+0.5L)* is applied corresponding to lateral load. The pushover graph of the bare and infill frames is shown in the Figure 4.3.

The normalized base shear of 5,10,15 and 20 story buildings are 0.042, 0.0253, 0.0255 and 0.0264 for bare frames and 0.044, 0.0253, 0.0255 and 0.0264 for infill frames. The numbering sequence of beam and column is shown in the Figure 3.1. The first yielding in the 5 story frame is started from the beam no.6 at a normalized base shear of 0.159, the first yielding of the column start at the normalized base shear of 0.273 in column no.6 for bare frame and for frame with infill panel these occurred for the base shear coefficients of 0.198 at beam no.6 and 0.304 at column no.6 respectively. Plastic hinge formation in the

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

10 story building with bare frame occurs first at beam no.12 for normalized base shear of 0.0886 similarly the first yielding in a column occurs at normalized base shear of 0.154. In the infill frame hinging occurs at the normalized base shear of 0.109 at beam no.12 and 0.172 at column no.11. For 15 story building the first yielding of beam occurs at normalized base shear 0.064 in bare frame at beam no.19 and for infill frame at the normalized base shear of 0.074 at beam no.19 too. The column yielding of 15 story building at a normalized base shear of 0.10 for bare frame and 0.102 for infill frame at column no.31. For 20 story bare frame building the first yielding occurs at the normalized base shear of 0.048 in beam no.23 and at 0.066 in column no.41. For 20 story infill frame the first yielding occurs at the normalized base shear 0.059 in beam no.23 and at 0.072 in column no.41. The yield displacements of the different frames are shown in Table 4.1. The base shear coefficient at the yield point is shown in Table 4.2.

Table 4.1: Yield displacements of buildings.

Story Number	First yield displacement (Δ_{y1}) %H	
	For Bare Frame	For Infill Frames
5 story	0.709	0.751
10 story	0.729	0.703
15 story	0.722	0.735
20 story	0.760	0.790

H=height of the building.

Table 4.2: Base Shear Coefficient at the point of yield.

	Beam Yielding			Column Yielding		
Building Height	Bare Frame	Infill Frame	First Beam yield	Bare Frame	Infill Frame	First Column Yield
5 Story	0.159	0.198	No. 6	0.273	0.304	No. 6
10 Story	0.0886	0.109	No.12	0.154	0.172	No. 11
15 Story	0.064	0.100	No.19	0.074	0.102	No. 31
20 Story	0.048	0.059	No.23	0.066	0.072	No. 41

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

In pushover analysis it has also been observed that formation of plastic hinges in column occurred at the bottom of the lower story, which satisfies the requirements of the capacity based seismic design. The available capacity and the deformation demand is also calculated in the push over analysis. The 5 story building is pushed upto 3.5% of total height of the building but the pushover graph in both case bare and infill frame is still ascending. The base shear, lateral roof displacement and the interstory drifts at point of instability are also determined as percentage of total height of building from the pushover analysis and presented in Table 4.3.

Table 4.3: Displacement at point of instability or 2.5% drift of the frames.

Building's Height	Bare Frame			Infill Frame		
	Base Shear Coefficient	Roof Displacement %H	Interstory Drift %h	Base Shear Coefficient	Roof Displacement %H	Interstory Drift %h
5 Story	0.397	1.465	2.5	0.500	1.427	2.5
10 Story	0.163	1.590	2.5	0.201	1.56	2.5
15 Story	0.103	2.697	4.02	0.111	2.813	6.60
20 Story	0.063	1.755	3.43	0.079	1.789	3.49

H = height of the building, h= story height

The roof displacement form the pushover analysis has also been determined for the value of maximum (mean *(M)*+standard deviation*(SD)*) interstory drift obtained from dynamic analysis, and shown in the Table 4.4. The values of roof displacement are used to compare the dynamic analysis and the static analysis.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Table 4.4: Roof displacement (% H) at Maximum M+SD of interstory drift.

Building's Height	Roof displacement (% H)	
	Bare Frame	Infill Frame
5 Story	1.183	0.925
10 Story	1.220	0.849
15 Story	1.144	1.090
20 Story	1.483	1.253

M=Mean value, SD=Standard deviation.

4.2.3 Dynamic Analysis:

Rigorous non-linear time history analysis is necessary to evaluate the performance of a building under seismic ground motion. Nonlinear analysis allows for flexural yielding and accounts for subsequent changes in strength and stiffness (Saatcioglu & Humar 2003). Estimation of roof displacement and interstory drift of a building induced by ground excitation due to earthquake is the objective of dynamic analysis in the performance evaluation methodology. The maximum ductility demand in a member is also calculated from the output of nonlinear time history analysis. If the ductility demand is less than the ductility capacity and the deflection is within acceptable limit, the design is satisfactory (Saatcioglu and Humar, 2003).

To consider the effect of gravity load in the lateral displacement the P-Δ effect has been considered in the dynamic analysis. Non-linear time history analysis is the only way to accurately capture the magnitude of the lateral displacement caused by P-Δ effect. A two dimensional model of the frames are used to carry out the response history analysis by using the nonlinear computer program, DRAIN-2DX. The analysis has been done for 30 (thirty) ground motion records. Among these, eight records are synthesized and

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

compatible to the seismic hazard spectrum for Vancouver, Canada (Tremblay *et al.* 2001) and twenty two are real ground motion collected from the data base of Pacific Earthquake Engineering Research Center (PEER, 2006) by comparing the peak acceleration-peak velocity ratio of seismic motion (A/V) to Vancouver's A/V ratio of seismic motion. Because peak acceleration and peak velocity controls the spectral shape of the seismic motion.

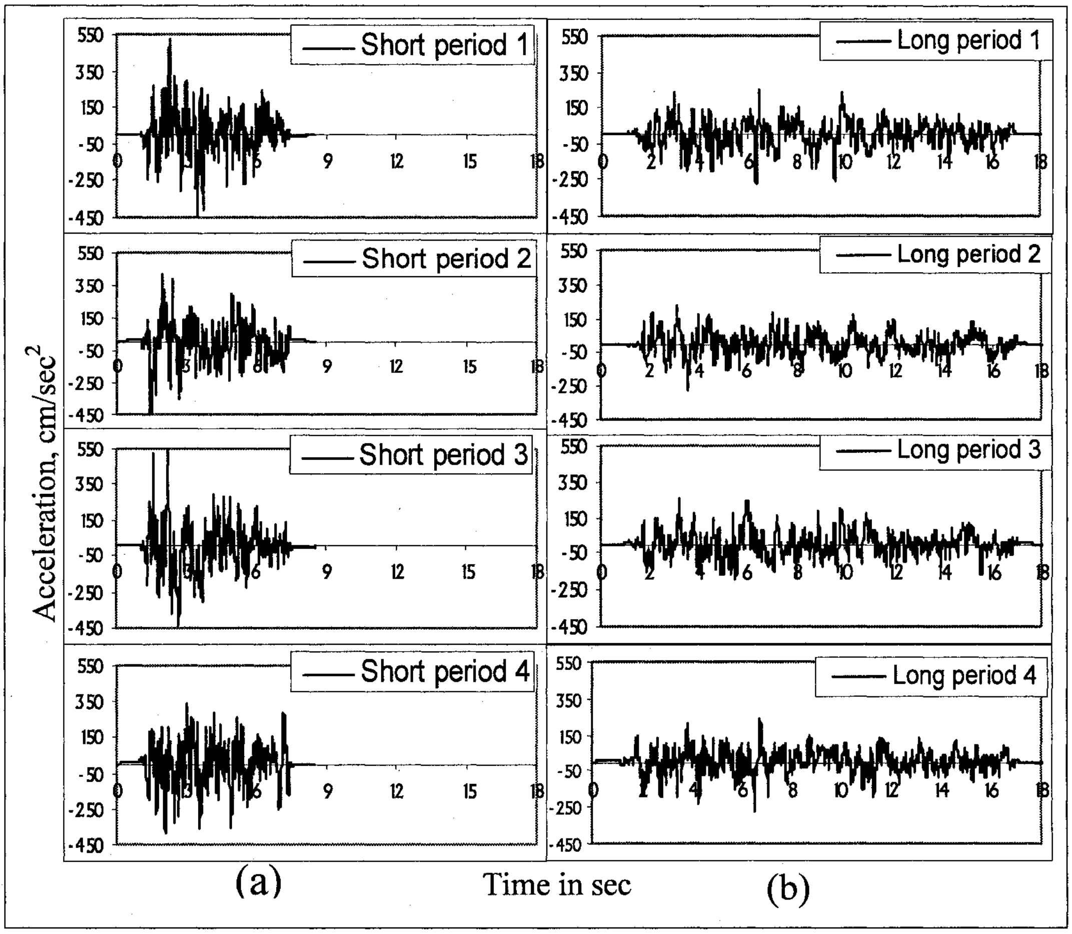

Figure 4.4: Time History of Synthesized ground motions.(a) For short period motion, (b) For long period motions.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Table 4.5: Summary of Synthesized Ground Motion.

Record No	LP1	LP2	LP3	LP4	SP1	SP2	SP3	SP4
Peak Acc.(cm/sec2)	266.2	279.4	248.6	271.7	523	527	567	380
Duration (sec)	18.24	18.24	18.24	18.24	8.55	8.55	8.55	8.55

LP = Long Period, SP = Short Period, Acc = Acceleration

Table 4.6: Summary of Real Ground Motion

Record No.	Location	Peak Acceleration (g)	Peak Velocity (m/sec)	A/V
1	Imperial Valley	0.348	0.334	1.04
2	Kern Country	0.179	0.177	1.01
3	Kern Country	0.156	0.157	0.99
4	Borrego Country	0.046	0.042	1.09
5	San Fernando	0.150	0.149	1.01
6	San Fernando	0.211	0.211	1.00
7	San Fernando	0.165	0.166	0.99
8	San Fernando	0.180	0.205	0.88
9	San Fernando	0.199	0.167	1.19
10	Record No.S-882	0.07	0.07	1.00
11	Record No.S-634	0.078	0.068	1.15
12	Monte Negro-2	0.171	0.194	0.88
13	Report Del Archivo: SUCH850919AL.T	0.105	0.112	0.94
14	Report del Archivo: VILE850919AT.T	0.123	0.105	1.17
15	Kobe, Japan	0.061	0.049	1.24
16	Kobe, Japan	0.694	0.758	0.92
17	Kobe, Japan	0.707	0.758	0.93
18	Kobe, Japan	0.144	0.150	0.96
19	Northridge, CA	0.469	0.571	0.82
20	Northridge, CA	0.510	0.493	1.03
21	Northridge, CA	0.088	0.072	1.22
22	Northridge, CA	0.080	0.082	0.98

A/V ratio is used as a selection criteria for the real ground motion records. Four of eight synthesized records are with long period and four are with short period. The spectra of the long period and short period ground motions are shown in the Figure 4.4.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The A/V (A in g. V in m/s) of Vancouver is close to 1.0 and an average A/V of 1.02 has chosen to select the seismic motion for the response history analysis (Naumoski et al 2004). The selected ground motion records required scaling to match with the design spectra of Vancouver at the specified period range. The summary of the ground motions are presented in the Table 4.5 & 4.6. The time history of synthesized ground motion is shown in Figure 4.4.

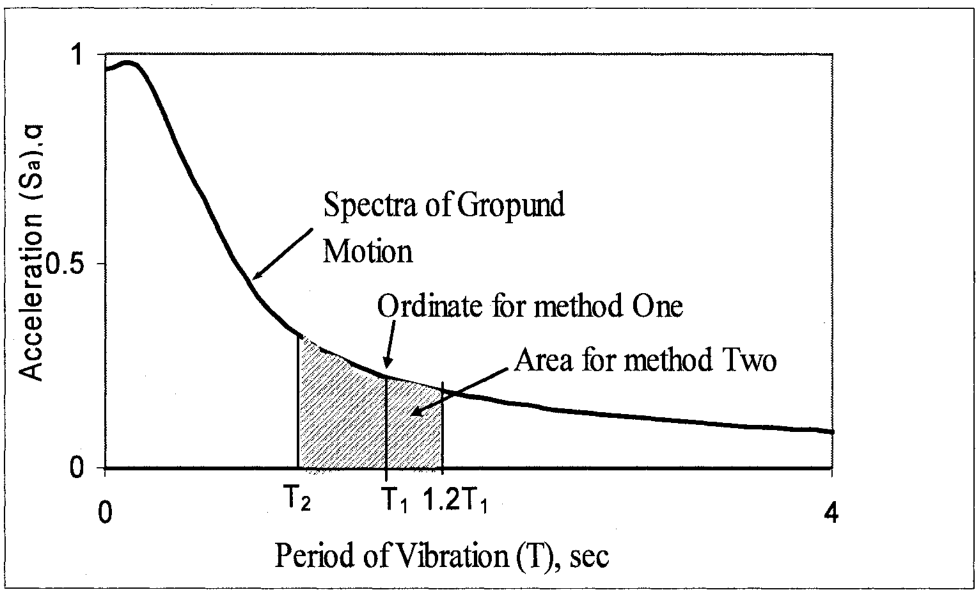

Figure 4.5: Scaling ordinate.

Before calculation of the scaling factors, the spectra of the selected motions were developed with 5% damping. The scaling has been done in two ways: (i) based on the acceleration ordinates, (ii) based on partial area under the acceleration spectrum (Naumoski *et al* 2004) as shown in the Figure 4.5. The analysis has been done for both; bare and infill frame using two types of scaled factor.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Table 4.7: Record of Scale Factor

Ground Motion Record No.	Scale Factor							
	5 Story		10 Story		15 Story		20 Story	
	Ordinate Method	P.Area Method	Ordinate Method	P.Area Method	Ordinate Method	P.Area Method	Ordinate Method	P.Area Method
1	0.015	0.011	0.009	0.010	0.015	0.015	0.030	0.018
2	0.022	0.023	0.026	0.021	0.030	0.029	0.029	0.034
3	0.022	0.023	0.034	0.024	0.034	0.035	0.023	0.039
4	0.069	0.096	0.084	0.085	0.093	0.113	0.074	0.126
5	0.023	0.032	0.040	0.030	0.033	0.039	0.033	0.048
6	0.010	0.020	0.016	0.016	0.009	0.017	0.013	0.019
7	0.022	0.029	0.017	0.022	0.019	0.026	0.015	0.027
8	0.034	0.024	0.028	0.023	0.015	0.030	0.020	0.030
9	0.023	0.027	0.017	0.022	0.012	0.023	0.020	0.025
10	0.067	0.049	0.142	0.067	0.223	0.143	0.430	0.240
11	0.026	0.049	0.101	0.044	0.157	0.072	0.287	0.138
12	0.020	0.020	0.015	0.018	0.021	0.023	0.034	0.029
13	0.036	0.029	0.052	0.033	0.055	0.048	0.068	0.061
14	0.037	0.038	0.040	0.039	0.044	0.047	0.038	0.051
15	76.718	58.844	83.588	81.243	86.001	108.967	120.523	129.968
16	29.531	51.211	50.477	40.274	71.509	55.651	140.610	81.848
17	2.881	3.585	7.241	3.681	9.294	6.342	16.302	10.002
18	5.179	8.708	10.931	7.276	18.474	11.232	34.390	17.925
19	4.431	4.819	9.420	5.247	18.760	9.244	29.731	15.859
20	6.698	7.819	12.213	7.765	14.709	11.819	23.954	16.644
21	4.642	5.943	8.523	5.746	15.907	9.962	17.749	14.451
22	29.413	37.661	53.673	35.902	59.202	55.259	95.393	79.660

In the first method the scaling factor is calculated as the ratio of the accelerogram's ordinate of the real motion to the ordinate of the design spectra for the fundamental period of the structure. In the second method the concept is that the area under the acceleration spectra of each real ground motion is same as the area under the design

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

spectra for the period range of second mode period and 1.2 times the fundamental period. The multiplying factor 1.2 is used to consider the nonlinear deformation during response (Naumoski *et al* 2004).

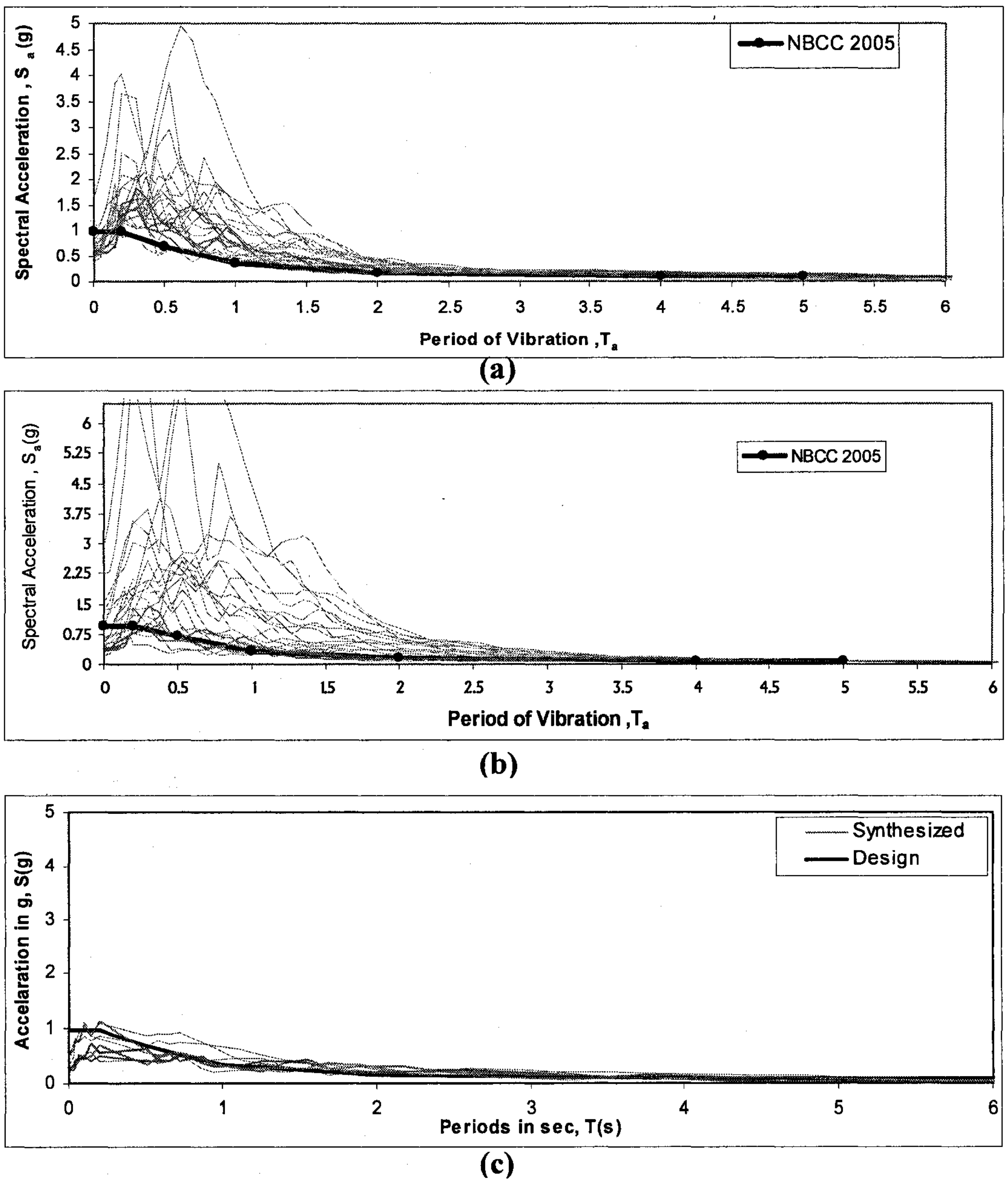

Figure 4.6: Spectrum of Ground Motions: (a) For 20 story building scaled by partial area method and (b) For 20 story building scaled by ordinate method, (c) For synthesized ground motions

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The scale factors for different ground motion are presented in the Table 4.7 and the scaled spectra are shown in Figure 4.6. From the time history analysis the maximum interstory drift of every record for each frame is calculated. The mean drift *(M)* and mean plus standard deviation *(M+SD)* of the real ground motion for each frame is calculated and checked with the code specified value. The maximum interstory drift of each synthesized record is recorded and used for evaluation. Number of synthesized records is not enough to calculate the mean and mean plus standard deviation.

Table 4.8(a): Summary of interstory Drift for Real Ground Motion

Building Height	Interstory Drifts (%hs) of Infilled Frame					
	Record Scaled By Partial Area Method			Record Scaled By Ordinate Method		
	Maximum	Max. of Mean	Max. (Mean+SD)	Maximum	Max. of Mean	Max. of (Mean+SD)
5 Story	1.70	1.25	1.5	1.78	1.06	1.32
10 Story	1.36	0.92	1.12	2.04	1.06	1.35
15 Story	1.73	1.13	1.42	2.74	1.26	1.82
20 Story	1.66	1.26	1.46	3.74	1.43	2.14

SD = Standard Deviation, h_s = Story height.

Table 4.8(b): Summary of interstory Drift for Real Ground Motion

Building Height	Interstory Drifts (%hs) of Bare Frame					
	Record Scaled By Partial Area Method			Record Scaled By Ordinate Method		
	Maximum	Max. of Mean	Max. of (Mean+SD)	Maximum	Max. of Mean	Max. of (Mean+SD)
5 Story	2.59	1.66	2.05	1.73	1.46	1.58
10 Story	2.58	1.50	1.87	2.21	1.43	1.77
15 Story	1.74	1.30	1.54	2.52	1.38	1.84
20 Story	2.44	1.59	1.99	3.98	1.67	2.48

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The base shear also calculated from the dynamic analysis. A summary of the interstory drift is presented in the Table 4.8 and 4.9. The figure 4.7 shows the story drift graph of dynamic analysis.

Table 4.9: Interstory drift for Synthesized Ground Motion

Record No. ↓	Maximum Interstory Drifts (% hs)							
	5 Story Building		10 Story Building		15 Story Building		20 Story Building	
	Infill	Bare	Infill	Bare	Infill	Bare	Infill	Bare
Long Period	1.38	2.16	1.27	1.76	1.08	1.45	1.06	1.19
Short Period	1.94	2.29	1.46	2.33	2.03	2.41	1.55	1.91

From the dynamic analysis it has been observed that the interstory drift of some records is higher than the code specified limit but the mean of all maximum interstory drift and mean plus standard deviation *(SD)* is below the code specified limit. It has also been observed that mean value and *SD* value reduce with the increase of number of ground motions. The mean value of interstory drift of some building is almost 60% below the code specified limit. Therefore, the performance achieved from the dynamic analysis is satisfactory. For some building the design becomes more conservative. It is observed that each building designed here with NBCC 2005 seismic provisions is found to be robust and has achieved a seismic level of performance equal to or better than the life safety performance level. However, the performance levels for all the buildings are not uniform. For instance the shorter buildings (e.g., the five story building) have a higher level of seismic performance as compared to the twenty story building.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

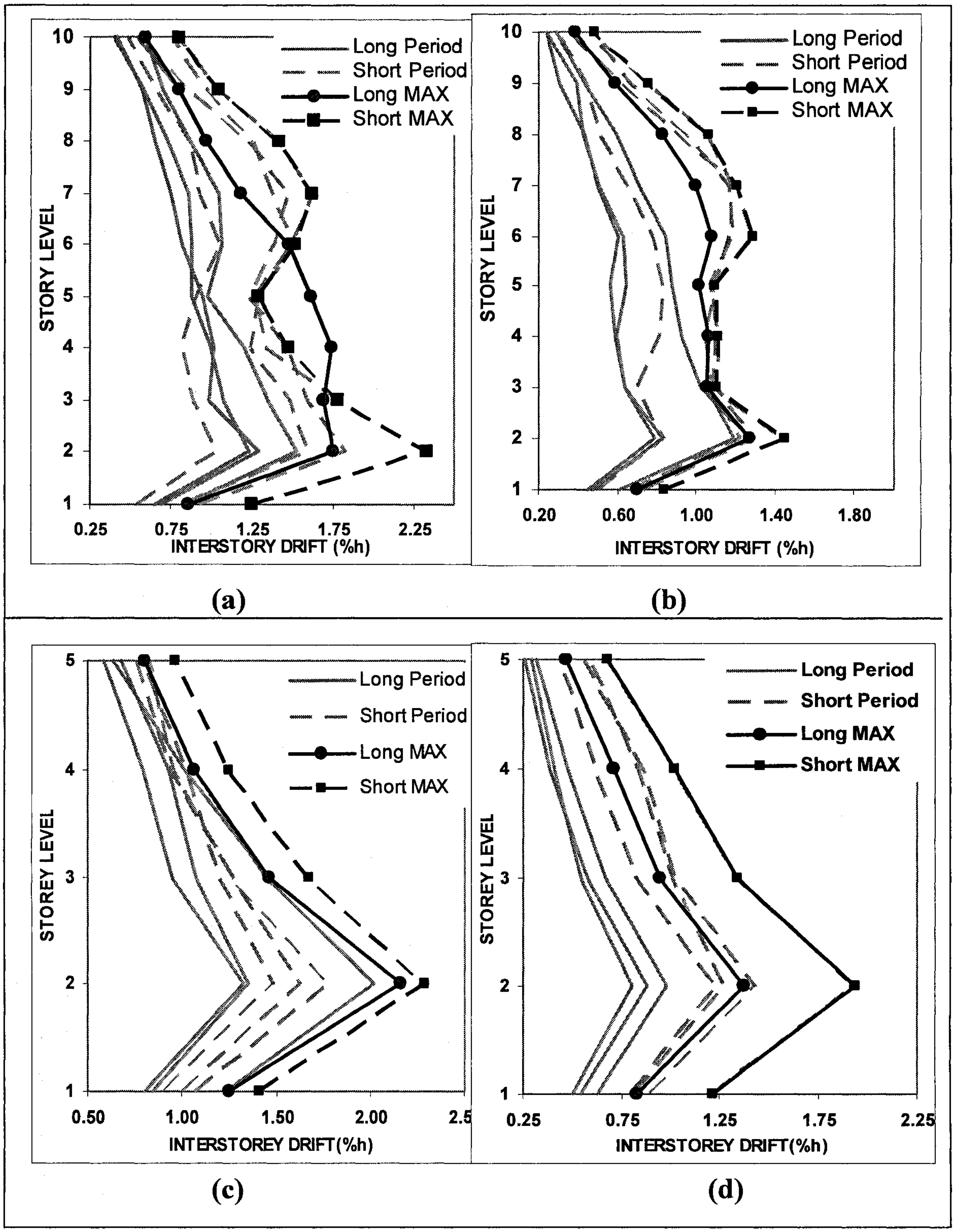

Figure 4.7(A): Graph of Dynamic Analysis for Synthesized ground motion; (a)Bare frame of 10 story building, (b) Infill frame of 10 story building, (c) Bare Frame of 5 story building, (d) Infill frame of 5 story building.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

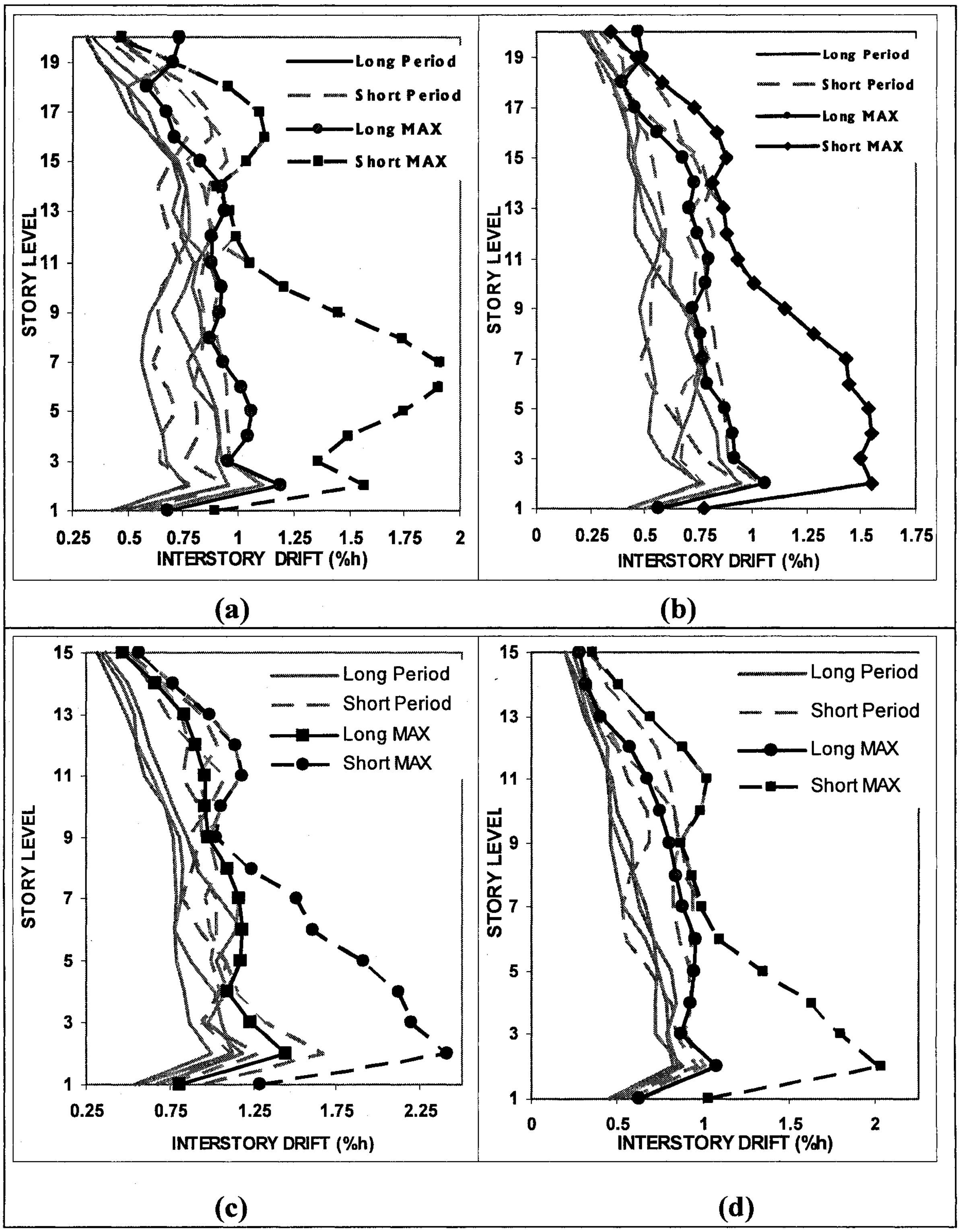

Figure 4.7(B): Graph of Dynamic Analysis for Synthesized ground motion; (a)Bare frame of 20 story building, (b) Infill frame of 20 story building, (c) Bare Frame of 15 story building, (d) Infill frame of 15 story building.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

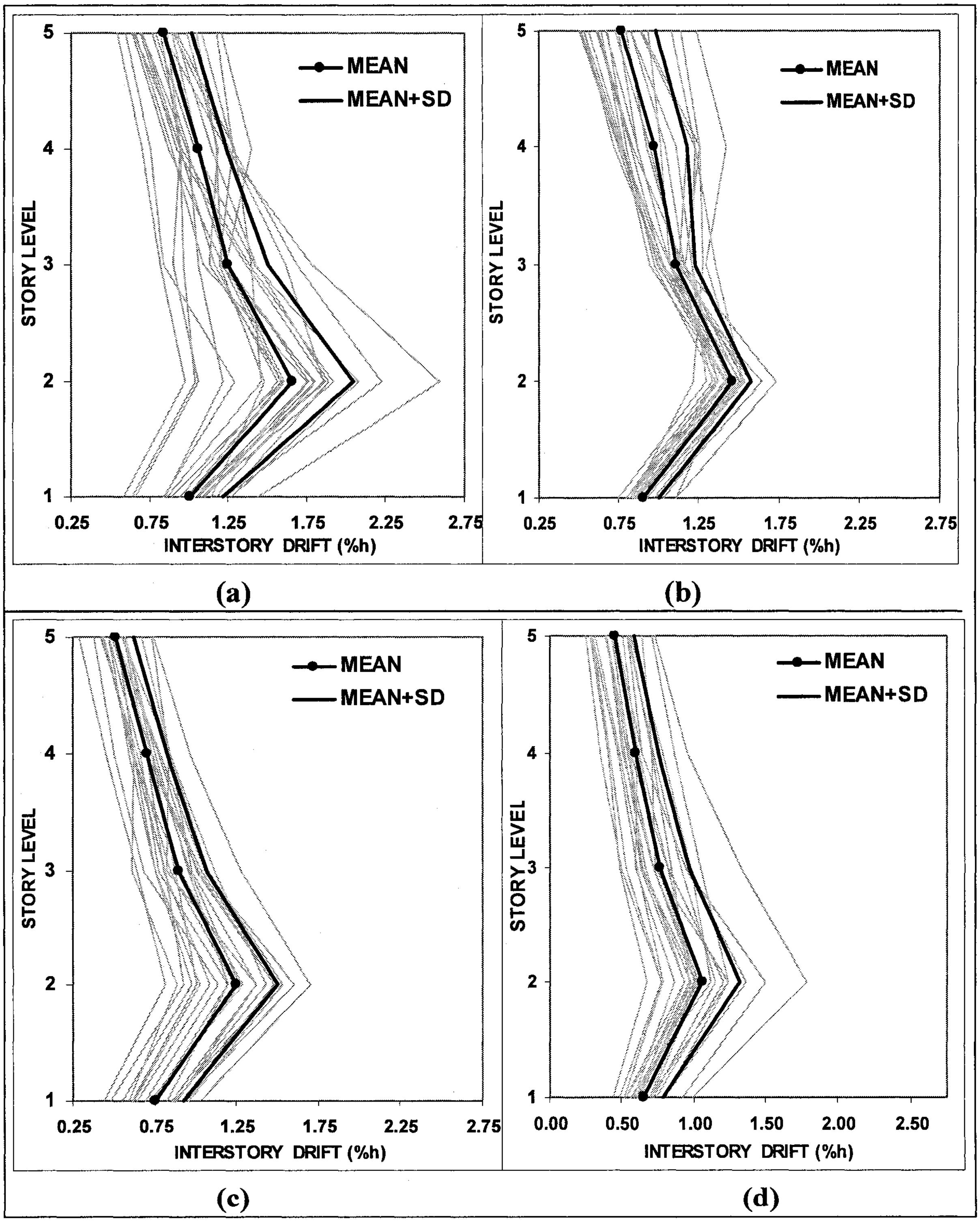

Figure 4.8 (A): Graph of Dynamic Analysis of 5 Story Building's Frame by real ground motion; (a)Bare frame scaled by partial area method, (b) Bare frame scaled by Ordinate method, (c) Infill Frame scaled by partial area method, (d) Infill Frame scaled by Ordinate method.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

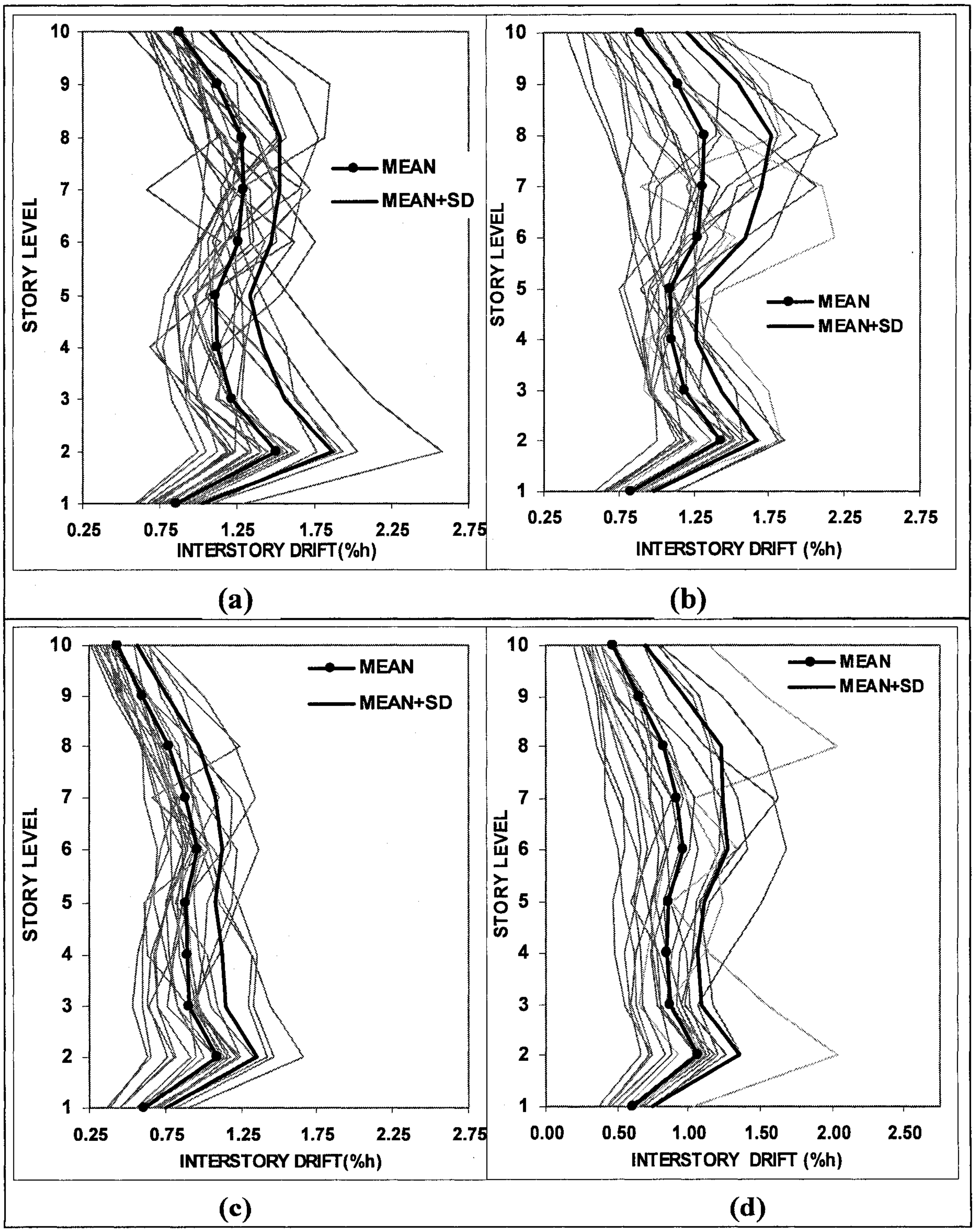

Figure 4.8 (B): Graph of Dynamic Analysis of 10 Story Building's Frame by real ground motion; (a)Bare frame scaled by partial area method, (b) Bare frame scaled by Ordinate method, (c) Infill Frame scaled by partial area method, (d) Infill Frame scaled by Ordinate method.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

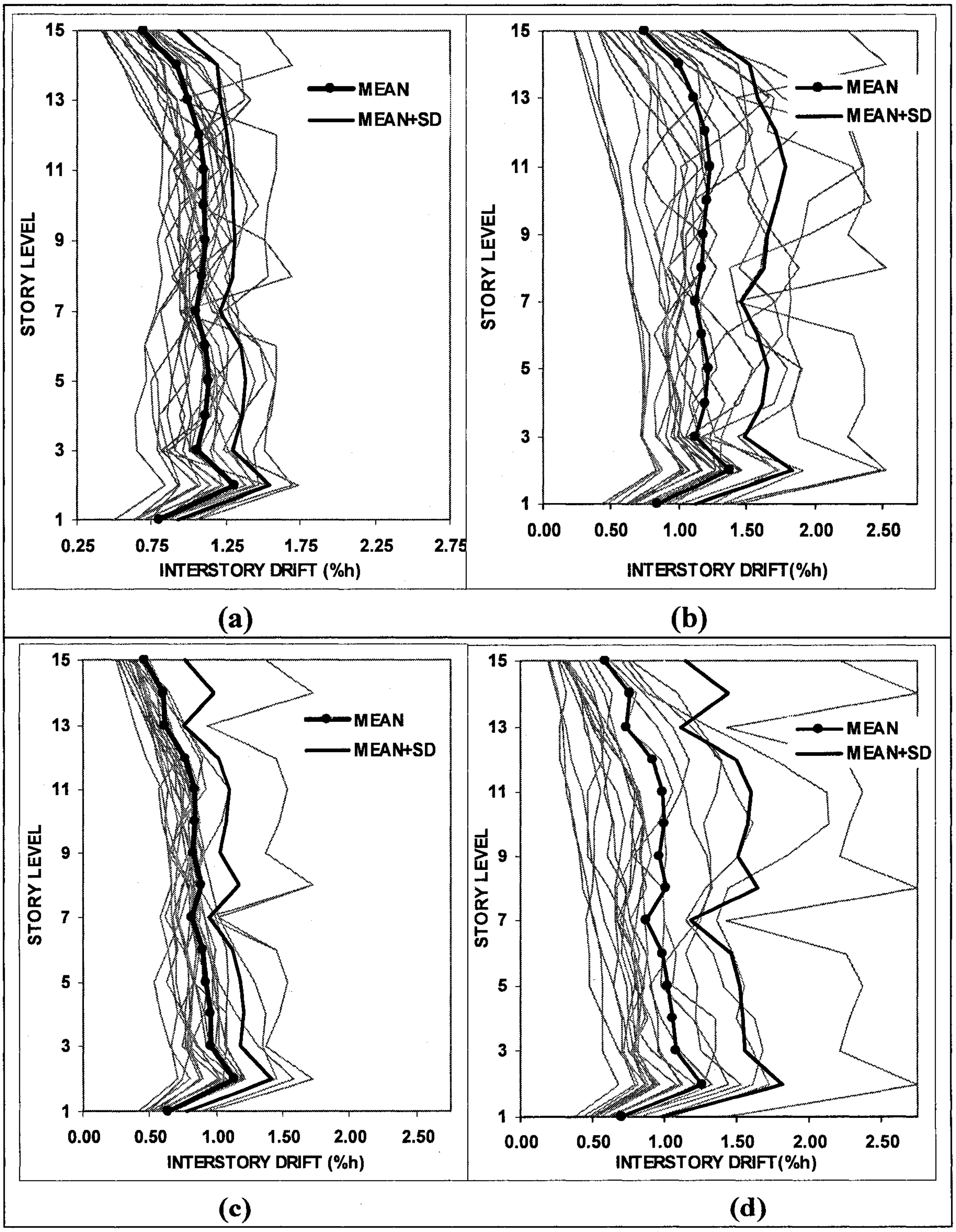

Figure 4.8(C): Graph of Dynamic Analysis of 15 Story Building's Frame by real ground motion; (a)Bare frame scaled by partial area method, (b) Bare frame scaled by Ordinate method, (c) Infill Frame scaled by partial area method, (d) Infill Frame scaled by Ordinate method.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

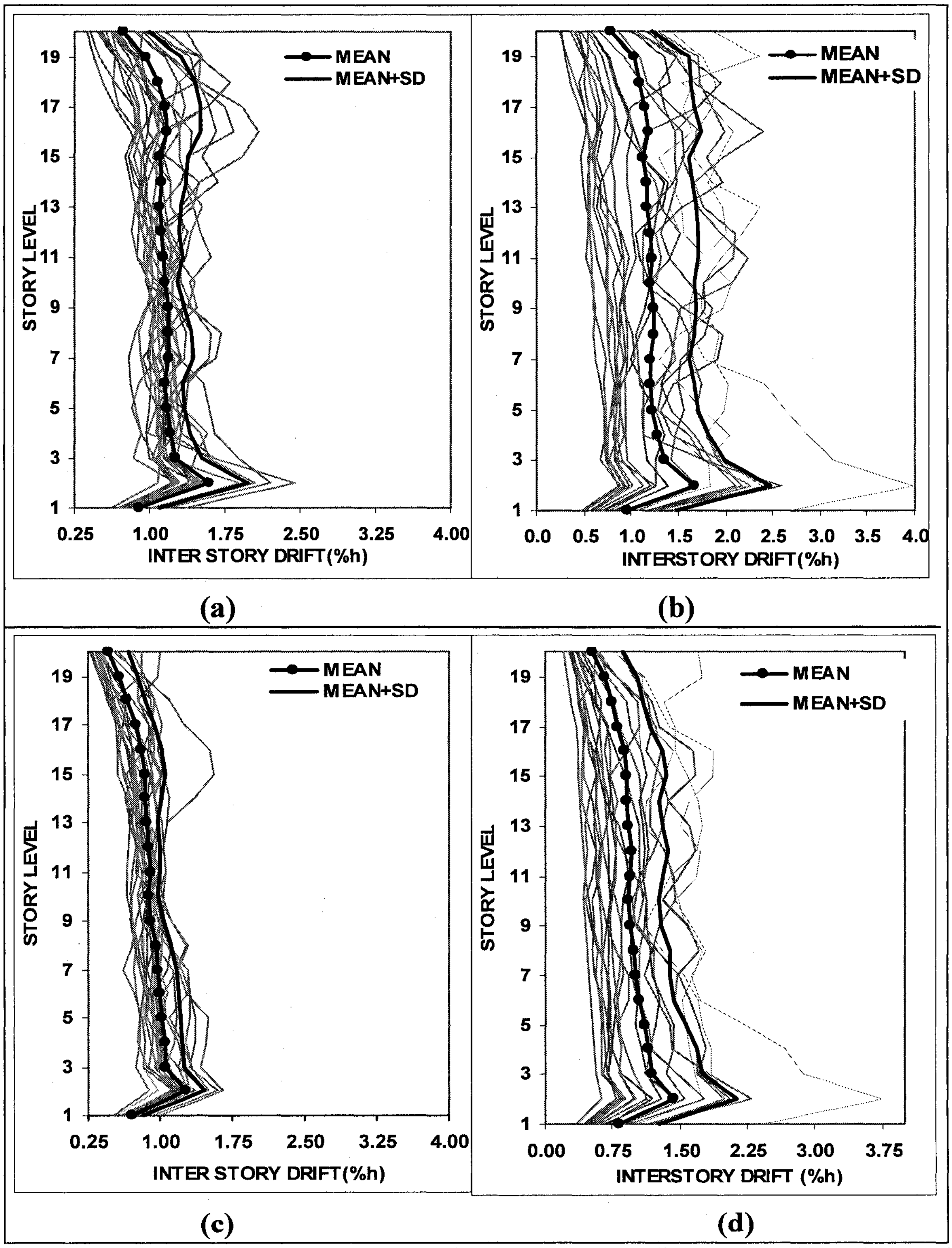

Figure 4.8(D): Graph of Dynamic Analysis of 20 Story Building's Frame by real ground motion; (a)Bare frame scaled by partial area method, (b) Bare frame scaled by Ordinate method, (c) Infill Frame scaled by partial area method, (d) Infill Frame scaled by Ordinate method.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Chapter 5

Performance-Based Seismic Design

5.1 General:

The present building codes, such as NBCC 2005 present primarily a strength-based or capacity-based design approach for seismic resistance of buildings and the performance levels are satisfied indirectly through the specified inter-story drift limit. From the results presented in Chapter 4, it has been observed that, while the NBCC 2005 produces a robust design of the lateral load resisting system of buildings, the level of performance achieved by different buildings is not uniform. An alternative to strength-based design is the performance-based seismic design approach. Step to performance-based seismic design is the progress of research of the last decade on seismic design. The performance-based design is informally in practice in the structural design for long time by designing the structures to fulfill the criteria of service limit state and ultimate limit state. That is an indirect implementation of performance-based design. In seismic design the desired performance has been ensured by evaluating the performance of the structure through static and dynamic analysis of the structure designed. But if a structure can be designed based on performance, it will be better than fixing the performance through evaluation. Knowledge of performance level expressed in terms of key response or damage parameters of a structure is very important in setting the performance-based design methodology. Three types of performance levels such as serviceability (*i.e.*, operational),

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

damage control and life safety or collapse prevention are controlled by three structural characteristics: stiffness, strength and deformation (Ghobarah, 2001). Some times the level of stress can also be treated as a performance target.

The focus of the research on the performance-based seismic design is on the development of an efficient and reliable design methodology that can easily be used in designing of the structure for the target performance level. Vision 2000 (1995) report considered the following three types of performance-based earthquake resistant design: (a) Strength-based design, (b) Displacement-based design and (c) Energy-based design. In the first and third categories of design the performance is established through the evaluation of performance and in the second one, the design is done by specifying the required level of performance in the beginning and determining the corresponding capacity of the structure. Thus, it can be said that the displacement-based seismic design is the direct performance-based seismic design. The methodology of performance-based seismic design can be described in the following steps (Kunnath, 2006).

- Step-1: Define a performance objective by incorporating the description of the hazard and the expected level of performance.
- Step-2: Selection of a trial design.
- Step-3: Through an analysis of the mathematical model of the structure the seismic demands on the system and its components is determined.
- Step-4: To verify the performance objectives of the structure as defined in Step-1, the performance of the structure is evaluated through static and dynamic non-linear analysis. If the performance level does not satisfy the performance

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

objective of step-1 the design must be revised to achieve the required performance objective.

As a completely new methodology of seismic design developing something intuitive in performance-based seismic does not belong to this research work. But as a part of seismic design a comparative study of different type of performance-based seismic design proposed by several researchers are presented in this research work. The study also includes a numerical example to verify the different performance-based seismic design method. Four different concepts of performance-based seismic design selected for this work are:

1. Nonlinear Analysis (N2) Method By Peter Fajfar (2000).
2. Displacement-Based Design Method for Building By Humar and Ghorbanie-Asl (2005).
3. Capacity-Demand-Diagram Method By A.K. Chopra and R.K.Goel (1999).
4. Yield Point Spectra: A Simple Alternative in the Capacity Spectrum Method By Mark Aschheim (2004).

To evaluate the above mentioned design methods, a 20 story building designed in the Chapter 3 is considered here. The building has been redesigned using performance-based-seismic design methodologies.

5.2 N2 Method (Fajfar 2000):

This is non linear method of seismic design. The objective of this method is to calculate the seismic demand of multi-degree-of-freedom (MDOF) system by converting it to a

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

single-degree-of-freedom (SDOF) system. Step-by-step design methodology is as follows:

- Step-I. Design of the structure for equivalent static load by force-based design method.
- Step-II. Determination of demand curve from the pushover analysis by applying the static load laterally according to assumed displacement shape.
- Step-III. Transformation of MDOF to SDOF and determination of capacity diagram for SDOF. To transform MDOF to SDOF a factor called transformation factor (Γ) is used. The following Equations (Fajfar 2000) are used in the transformation process.

  $m^* = \sum m_i \Phi_i$ 5.1

  $\Gamma = \dfrac{m^*}{\sum m_i \Phi_i^2}$ 5.2

  Where m_i is the mass of ith story, m^* is the mass of the equivalent single-degree-of –freedom system, Φ_i is the assumed displacement of the ith story and Γ is the transformation factor for multi-degree-of-freedom system.

- Step-IV. Determination of Demand Spectra in Acceleration-Demand (AD) format and superimpose of the Capacity diagram into the demand spectra. To determine the A-D spectra the following Equations can be used.

  $S_d = \dfrac{\mu}{R_\mu} \dfrac{T^2}{4\pi^2} S_a$ (Fajfar 2000) 5.3

  $R_\mu = (\mu - 1)\dfrac{T}{T_c} + 1$ when T<Tc and $R_\mu = \mu$ when T≥Tc 5.4

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Where S_d is the inelastic displacement, S_a is the inelastic acceleration, μ is the ductility of the system, R_μ is the ductility reduction factor, T is the period of vibration and T_c is the characteristic period of vibration.

- Step-V. Determination of Seismic demand for SDOF model from the capacity diagram and the acceleration-displacement spectra. The values in the graph at the intersection point of capacity-diagram and the acceleration-displacement spectra are the values of SDOF system.
- Step-VI. Determination of Global Seismic demand for MDOF model from the demand of SDOF model by multiplying the demand of SDOF with the transformation factor.
- Step-VII. Calculation of local seismic demands from the pushover analysis of MDOF system upto global demand.
- Step-VIII. Comparison of the local and global seismic demand with the capacities for the relevant performance. The demand values also can be compared with the values obtained from the dynamic analysis of the structure for the real ground motion to verify the design.

This is a modified version of capacity-spectrum method. In this method a ductility reduction factor is used to modify the elastic spectra for determination of inelastic spectra. The performance objectives are determined in the rational way and the demand quantities are determined without iteration.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

5.2.1 Design of the Building:

The performance-based seismic design is done by N2 (Fajfar 2000) method. The displacement shape of the first mode shape (Fig.4.2 (a)) is considered as the assumed displacement shape of the building. So, the displacement shape Φ is:

Φ = [0.06, 0.12, 0.19, 0.26, 0.33, 0.40, 0.47, 0.53, 0.60, 0.66, 0.71, 0.76, 0.81, 0.85, 0.89, 0.92, 0.95, 0.97, 0.99, 1.00]

These are displacements of different story of 20 story building in the X direction. The masses of multi-degree-of –freedom system at different story level is shown in the Table:5.1.

Table 5.1: Masses of Story levels

Story level	Mass (Tons)	Story level	Mass (Tons)	Story level	Mass (Tons)	Story level	Mass (Tons)
1	80.90	6	78.00	11	77.44	16	77.28
2	78.40	7	78.00	12	77.44	17	77.28
3	78.40	8	78.00	13	77.44	18	77.28
4	78.40	9	78.00	14	77.44	19	77.28
5	78.12	10	77.70	15	77.36	20	60.22

The pushover analysis is done by using the code suggested inverted triangular load pattern and the graph is shown in the Figure-5.1(a).

The equivalent mass for SDOF m^* = 950.05 (Equation.5.1) tons.

The transformation factor Γ = 1.30 (Equation.5.2). The pushover graph for MDOF is modified by the transformation factor for SDOF.

From the bilinear idealization of the pushover graph the yield strength *(F_y)* and corresponding yield displacement *(D_y)* for SDOF are calculated and the values are

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

$F_y^*=750kN$ and $D_y^*=42.0cm$. The elastic period of the idealized bilinear system is calculated by using the Equation 5.5 and the calculated elastic period is $T^* = 4.678s$.

$$T^* = 2\pi\sqrt{\frac{m^* D_y^*}{F_y}} \quad \text{(Fajfar, 2000)} \ldots\ldots 5.5$$

The capacity diagram of the single-degree-of –freedom system is obtained by dividing the F^* by equivalent mass m^* in the idealized pushover graph and shown in the Figure 5.1(b). From the capacity diagram the acceleration at the yield point *(S_{ay})* is determined and the value is 0.078g

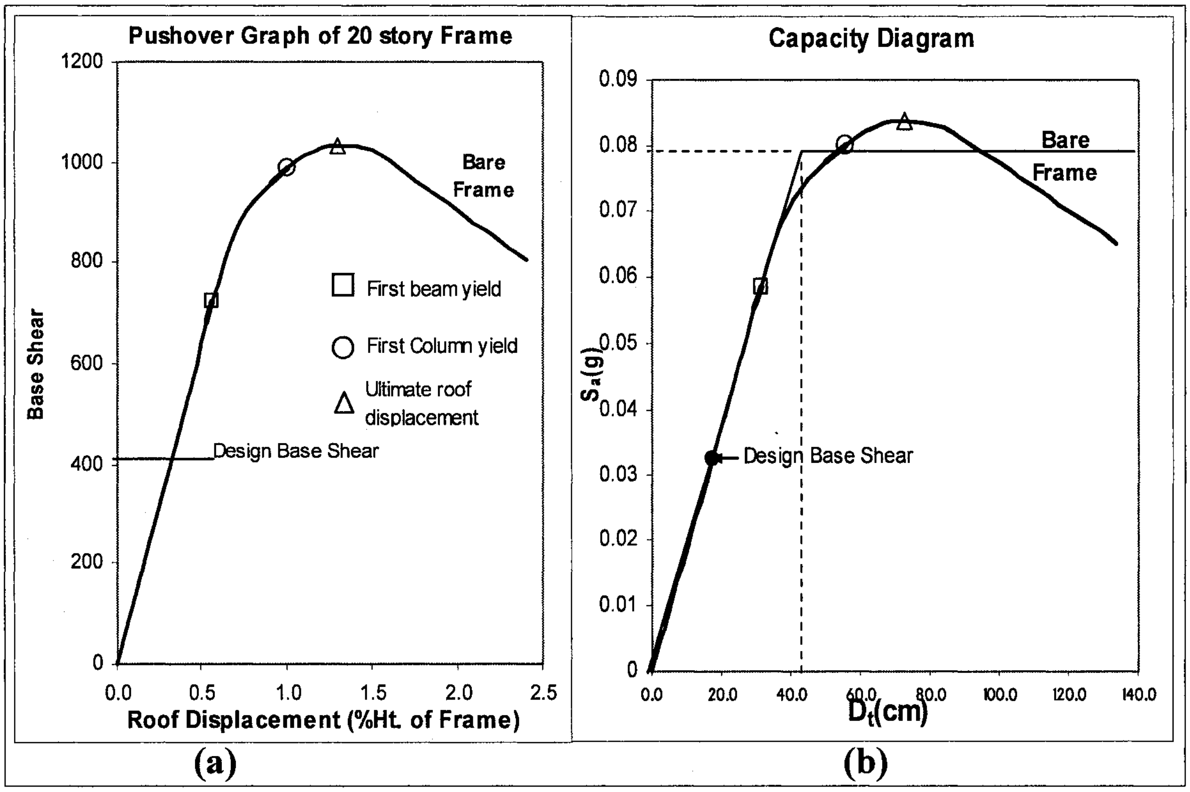

Figure 5.1: Pushover graph and Capacity diagram (a) Pushover curve for MDOF, (b) Capacity diagram of equivalent SDOF.

The demand spectra in the acceleration-displacement format for the ground motion has been determined and shown in the Figure-5.2. From the graph the value of elastic acceleration *(S_{ae})* is (0.09g) and elastic displacement *(S_{de})* is 35 cm.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The ductility reduction factor ($R_\mu = S_{ae}/S_{ay}$) is 1.33. The elastic period of system ($T^* = 4.523$s) is greater than (T_c=0.2s). Therefore, $\mu = R_\mu = 1.33$. For the period of $T^* > T_c$ the elastic displacement and the inelastic displacement become same, i.e. $S_d = S_{de}$. Therefore, the inelastic displacement demand of SDOF system is 35cm. So, the inelastic displacement demand of the MDOF is (35*1.30) 45.5 cm. and the ultimate capacity of the building is about 96 cm (Figure 5.1(a)) which means that the design is satisfactory. The performance of a building can easily be determined in this method that makes the performance-based seismic design simple.

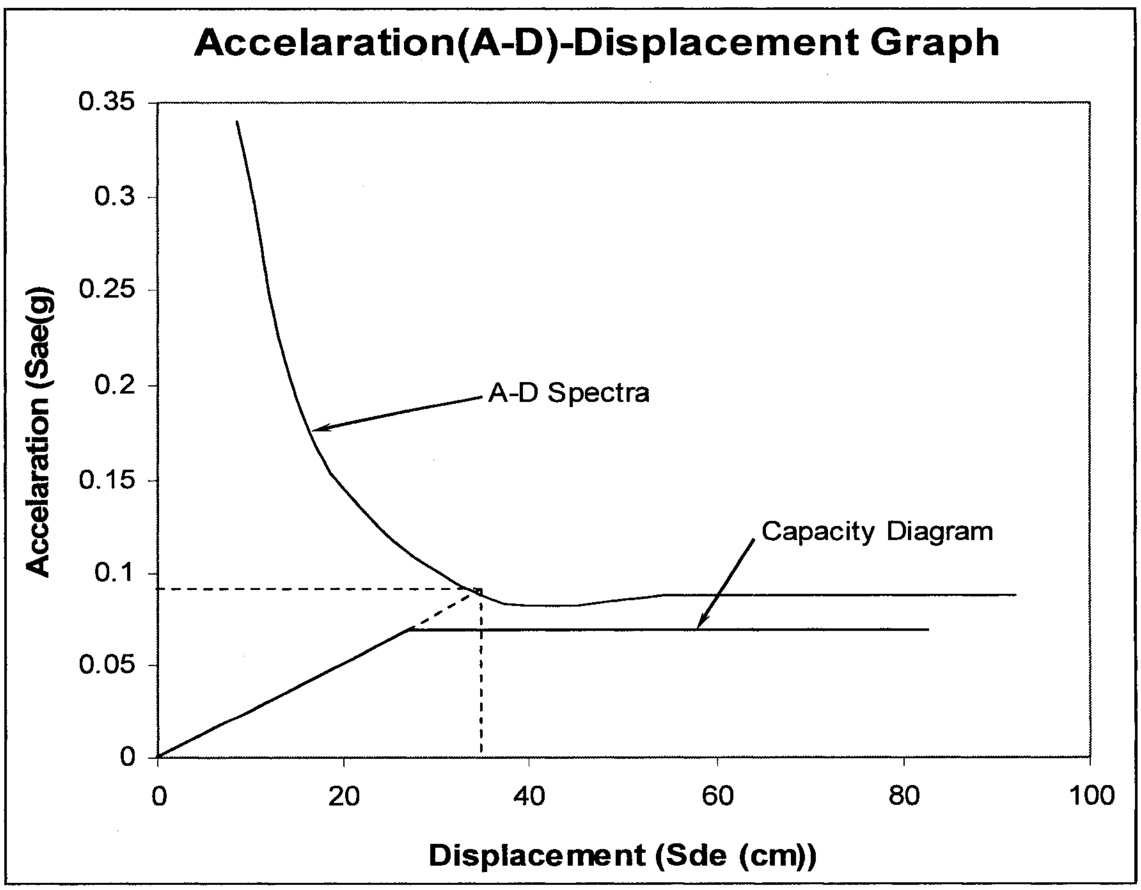

Figure 5.2: Demand Spectra for N2 method.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

From the Figure 5.2 it has been observed that the Capacity-Diagram does not intersect the Demand-Graph which indicates that the capacity of the building is less than the demand. To calculate the required capacity of the building the Capacity-Diagram needs to be raised, which has been done in the Figure by extending the Capacity-Diagram with dotted line.

5.3 Displacement-Based Design Method (Humar and Ghorbanie-Asl., 2005):

The design concept based on the target roof displacement and the design base shear is calculated from this target roof displacement. The initial target displacement should be less than the following limits:

a. Code defined maximum roof displacement.

b. Roof displacement at the P-Δ instability limit in pushover analysis.

c. Roof displacement at which the element's ductility demand exceeds its ductility capacity.

The design of the structure is done as usual by using the base shear obtained from the calculation of this method. In determination of the base shear the concept of single-degree-of-freedom (SDOF) system is used and for this the multi-degree-of-freedom (MDOF) system is transformed to SDOF by using a modification factor.

The steps followed in this design method are as follows:

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

- Step-I. Calculation of the ductility capacity (μ) from the assumed yield displacement and ultimate displacement. If the ductility capacity recommended by the code is lower than the calculated one than the code permitted ductility capacity should be used.
- Step-II. Calculation of ultimate displacement of SDOF *(δ_u)*.
- Step-III. Construction of inelastic spectrum for ductility of μ and in acceleration-displacement *(A-D)* format. The Equation 5.6 has been used to calculate the inelastic displacement.

$$D = \frac{\mu}{R_y}\left(\frac{T_n}{2\pi}\right)^2 A \quad \text{(Fajfar 2000)} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ 5.6$$

Where D is the roof displacement, μ is the ductility, R_y ductility reduction factor, T_n is the period of vibration and A is the spectral acceleration. For Krawinkler and Nasser proposed R_y-μ-T_n relation the following Equations are used

$$R_y = [C(\mu - 1) + 1]^{1/C} \quad \text{(Chopra and Goel 1999)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 5.7$$

Where R_y ductility reduction factor, μ is the ductility capacity and C is a constant as described in the Equation 5.8.

$$C(T_n, \alpha) = \frac{T_n^a}{1 + T_n^a} + \frac{b}{T_n} \quad \text{(Chopra and Goel 1999)} \ \ldots\ldots\ldots\ldots\ldots\ldots \quad 5.8$$

Where b & α are constant related to the material property, T_n is the period of vibration.

- Step-IV. Determination of inelastic acceleration from the A-D spectrum for equivalent ultimate displacement *(δ_u)*.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

- Step-V. Calculation of design base shear from the inelastic acceleration obtained from the A-D spectrum. The Equation 5.9 has been used for the calculation of design base shear.

$$V = \frac{A_y M^*}{R_0} \quad \text{(Humar and Ghorbaine-Asl., 2005)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.9$$

Where M^* is the mass of equivalent single-degree-of-freedom system and calculated by dividing the mass of multi-degree-of-freedom system with a modification facto, R_0 is the overstrength related force reduction factor A_y is the spectral acceleration.

$$\text{The modification factor } (\Gamma) = \frac{\phi^T m 1}{\phi^T m \phi} \quad \text{(Fajfar 2000)} \ldots\ldots\ldots\ldots\ldots\ldots\ldots 5.10$$

- Step-V. Design of the structure for the base shear calculated in step-V need to be done by the code (NBCC 2005) mentioned procedure.
- Step-VI. Pushover analysis of the designed structure is required for the refined value of yield and ultimate displacement. Steps I to V will have to be repeated until the design base shear converges.

5.3.1 Design of Building:

The same 20 story building as designed before will be designed by this method. The pushover analysis of the statically design building is done and the yield displacement is determined from the pushover curve. The displacement shape of the first mode shape is assumed as the displacement shape of the building in converting to SDOF. The initial

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

design base shear obtains by using NBCC 2005 provision is 400.96 kN. The assumed yield displacement (Δ_y) is 0.57% of total height of frame (422.94 cm).

The roof displacement 1.4% of total height of frame is obtained from the initial pushover analysis and this value is used as the ultimate displacement for the first iteration. Therefore, assumed ultimate displacement (Δ_u) is 1038.8 cm (1.4% of total height of frame.)

The ductility capacity ($\mu=\Delta_u/\Delta_y$) is 2.5 but the code permitted ductility for a fully ductile moment resisting frame is 5. So, the calculated ductility 2.5 is used in the design. The conversion factor (Γ) for MDOF is 1.3 (Equation.5.10). Therefore, the yield (δ_y) and ultimate displacements (δ_u) of the SDOF are:

$\delta_y = (\Delta_y/\Gamma) = 325.34$ mm, and $\delta_u = (\Delta_u/\Gamma) = 799.08$ mm

The equivalent SDOF mass $M^* = 951.85$ kN. The hazard spectrum as specified by NBCC 2500 is used for this design. The demand spectrum for ductility $\mu = 2.5$ and in acceleration-displacement (A-D) format is shown in the Figure 5.3. and the calculation details is shown in the Table 5.2.

Values of α & b in the Equation for elasto-plastic force-deformation behavior are 1 and 0.42 (Chopra and Goel 1999).

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Table 5.2: Calculation of different factors for A-D spectra

A_y,g	μ	T_n, s	c	b	R_y	D, cm	$A=A_y/\mu$,g
0.96	2.5	0.1	4.2909	0.42	1.5961	0.3736	0.384
0.96	2.5	0.2	2.2667	0.42	1.9225	1.2408	0.384
0.69	2.5	0.5	1.1733	0.42	2.3756	4.5109	0.276
0.34	2.5	1	0.92	0.42	2.5664	8.2301	0.136
0.17	2.5	2	0.8767	0.42	2.6052	16.215	0.068
0.0875	2.5	4	0.905	0.42	2.5796	33.715	0.035
0.0875	2.5	5	0.9173	0.42	2.5687	52.903	0.035
0.0875	2.5	6	0.9271	0.42	2.5602	76.434	0.035
0.0875	2.5	6.5	0.9313	0.42	2.5566	89.829	0.035
0.0875	2.5	7	0.935	0.42	2.5534	104.31	0.035
0.0875	2.5	7.5	0.9384	0.42	2.5506	119.88	0.035
0.0875	2.5	8	0.9414	0.42	2.548	136.53	0.035
0.0875	2.5	8.5	0.9441	0.42	2.5457	154.27	0.035

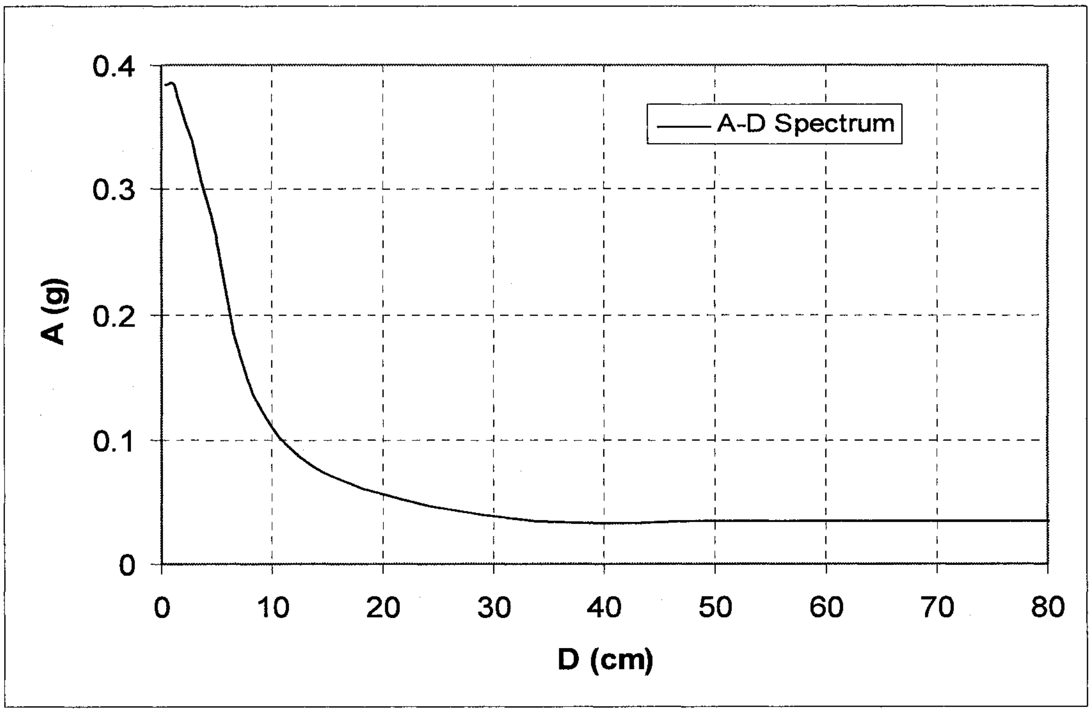

Figure-5.3: Acceleration-Displacement Spectra for Displacement-Based Design

The inelastic demand acceleration (A) is calculated form the A-D spectrum for ductility 2.5 and the value is 0.035g. The new design base shear V= 217.89 kN (Eq.4).The base

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

shear obtained from the displacement-based design is smaller than the one used in the design of the structure, therefore the design is satisfactory.

5.4 Yield Point Spectra Method (Aschheim M., 2004):

5.4.1 General:

Yield point Spectra (YPS) is a simplified method of Capacity Spectrum method of performance-based seismic design. When the performance objectives are stated in terms of ductility and peak displacement of structure than this method can easily be used to interpret the performance of the structure. In this case a graphical procedure is followed to evaluate the performance of the structure. YPS method can also be used to determine the vibration properties for the target displacement and the ductility demands where as other methods can only be used to determine the displacement properties from the given vibration properties. In this design method the graph of base shear verses yield displacement is plotted either by exact or approximate method depending on the information available. The yield point is constituted by the yield strength (V_y) and yield displacement (Δ_y). The following Equations are used to determine the yield point spectra.

$$\Delta_y = \mu \left(\frac{T}{2\pi} \right)^2 S_a \quad \text{(Aschheim 2004)} \quad \ldots\ldots 5.11$$

$$S_a = \frac{S_{ae}}{R_\mu} \quad \text{(Fajfar 2000)} \quad \ldots\ldots 5.12$$

Where S_a is the inelastic spectral acceleration, S_{ae} is the elastic spectral acceleration, μ is the ductility capacity of the system, R_μ is the ductility reduction factor, T is the period of vibration and Δ_y is the yield displacement. In the above equations Miranda and Bertero

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

(1994) discussed R-μ-T relationship has been used. Equation 5.13 is used for calculation of R_μ values.

$$R_\mu = \mu + (1-\mu)\exp\left(\frac{-16T}{\mu}\right) \quad \text{(Miranda and Bertero 1994) ....5.13}$$

Where μ is the ductility capacity of the system, R_μ is the ductility reduction factor, T is the period of vibration .This evaluation technique is also based on single-degree-of-freedom system.

5.4.2 Design of Building:

For design of the building the hazard spectra described in NBCC 2005 is selected as the design spectra and the assumed level of performance is shown in the Table 5.3.

Table 5.3: Performance objectives

	Building Performance Level	
	Immediate Occupancy	Life Safety
Peak Transient drift	1% of total building height	2% of total building height
Ductility of the System	2	8

The yield point spectra is determined from NBCC 2005 defined design spectra and presented in the Figure 5.4 and the spreadsheet of the calculation is shown in Table 5.4. Equations 5.7 and 5.8 are used to implement R-μ-T relationship. The coefficients of the equations are taken from the Chopra and Goel (1999) prescribed values. The demand curves for two levels of performance have been determined from the Yield Point Spectra using the technique discussed by Aschheim (2004) and shown in the in the Figure 5.5.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Table 5.4: Calculation for YPS (μ =8).

Δ_y, cm	T, s	μ	R_μ	S_{ae}	C_y	S_a,g
0.84	0.1	8	2.27	0.960	0.423	0.423
2.31	0.2	8	3.31	0.960	0.290	0.290
6.32	0.5	8	5.42	0.690	0.127	0.127
9.58	1.0	8	7.05	0.340	0.048	0.048
17.17	2.0	8	7.87	0.170	0.022	0.022
34.79	4.0	8	8.00	0.088	0.011	0.011
54.34	5.0	8	8.00	0.088	0.011	0.011
78.25	6.0	8	8.00	0.088	0.011	0.011
91.84	6.5	8	8.00	0.088	0.011	0.011
106.51	7.0	8	8.00	0.088	0.011	0.011
122.27	7.5	8	8.00	0.088	0.011	0.011

For the selected 20 story building the maximum mean interstory drift obtained from the Response History Analysis (RHA) is 1.67% of story height i.e.16.1 cm. Gupta and Krawinkler (2000) shows that for low to medium rise building the story drift to roof drift ratio varies from 1.2 to 2.0. In this design an average of 1.6 is considered. Therefore, the roof displacement corresponding to maximum of mean story drift is 3.82 (6.1/1.6) cm. For the first modal displacement the transformation factor for SDOF is 1.3. So, the maximum roof displacement of SDOF is 2.93 cm. The design base shear coefficient of the 20 story building is 0.048. The point corresponding to maximum roof displacement and design base shear is lies below the performance levels curve as shown in the graph. So, according to YPS design methodology the design of the steel moment resisting frame is not satisfactory

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

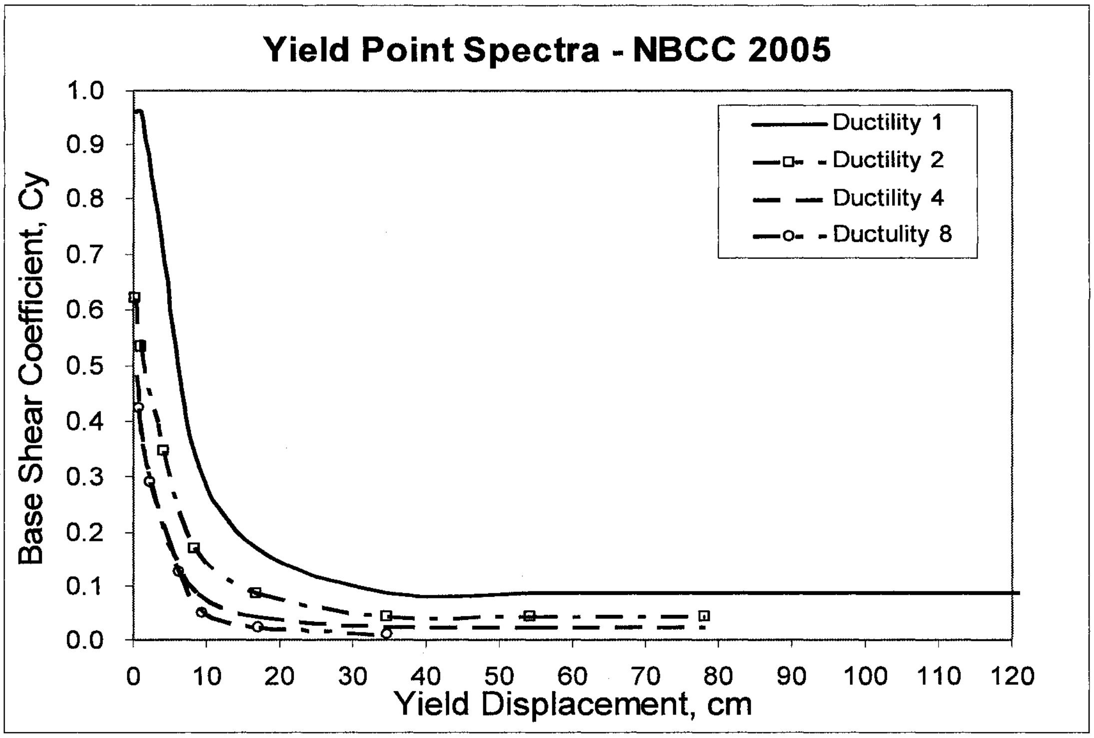

Figure 5.4: Yield Point Spectra for NBCC 2005

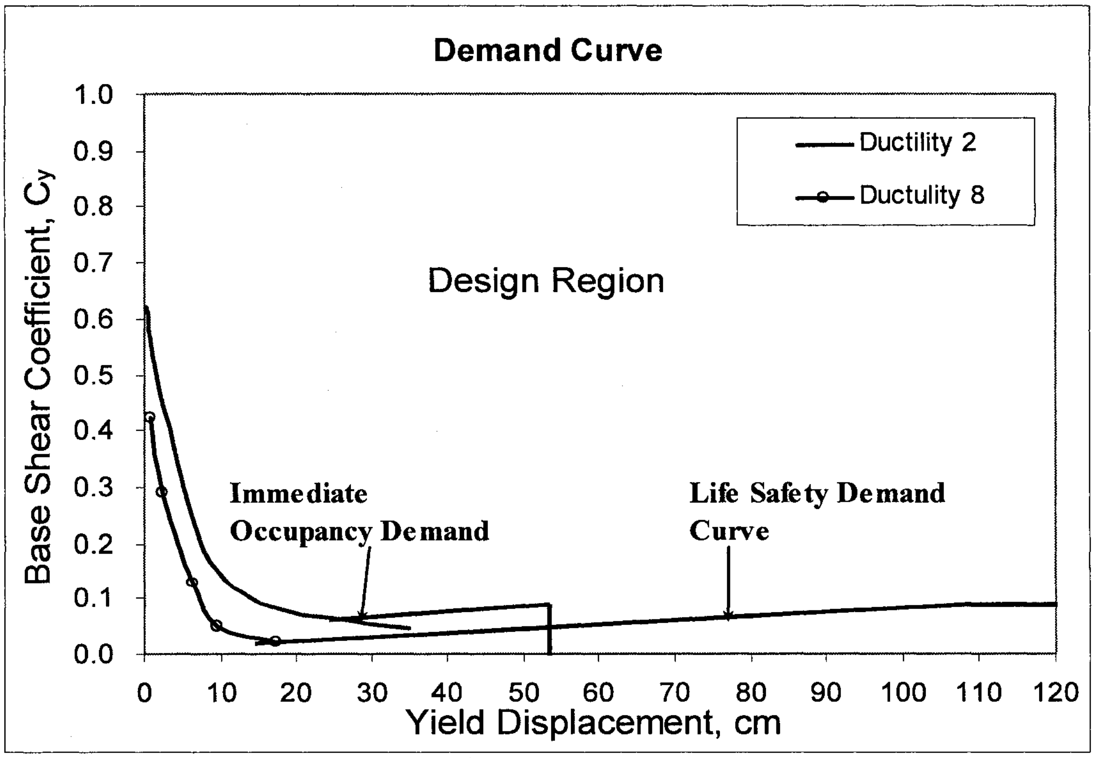

Figure 5.5: Demand Curves of Yield-Point-Spectra Method

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

5.5 Capacity-Demand Diagram Method (Chopra and Goel,1999):

5.5.1 General:

This is a simplified nonlinear realistic analysis procedure of predicting earthquake demands of structure. This is an approximate method and the method based on the capacity and demand diagram. In the development of capacity diagram the concept of single-degree-of-freedom (SDOF) system is used and the multi-degree-of-freedom system is transformed to SDOF with a transforming factor. The factor is determined from the assumed deformed shape. The steps followed in this design method are as follows:

- Step-I. Development of Pushover curve from base shear and roof displacement relationship.
- Step-II. Development of Capacity diagram from the Pushover curve.
- Step-III. Development of response spectrum in acceleration and displacement (A-D) format. Where acceleration A is the pseudo acceleration and displacement D is the deformation. In calculation of A-D spectrum the Equation 5.6 is used and in the Equation Nasser and Krawinkler prescribed R_y-μ-T relation as discussed by Chopra and Goel (1999) is used. Equation 5.7 and 5.8 are used for to evaluate the R_y-μ-T relationship.
- Step-IV. The response spectrum and capacity diagram are plotted together and the displacement demand of SDOF is determined from this plot. This demand is then transformed to the demand of MDOF by the transformation factor.
- Step-V. The demanded displacement is then compared with the specified performance target.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

5.5.2 Design of Building:

A 20 story steel moment resisting frame building is selected to determine the performance level with the Capacity-Demand-Diagram methodology of the performance based design. The force-based design of the building has described earlier in this report. The first modal deformation shape has been selected as the deformed shape of the structure to determine the capacity diagram. From this assumption the transformation factor (Γ) is calculated and the value is 1.3. Also the pushover analysis has been done according to code suggested inverted triangular force distribution method. In this method of pushover analysis the total equivalent earthquake force has been considered. The hazard spectra presented in NBCC 2005 has been selected as the design spectra to determine demand diagram. The capacity diagram and the demand diagram are shown in the figure 5.6 and the calculation of demand diagram is presented in the Table 5.5.

Table 5.5: Spread sheet calculation of demand diagram.

A, g	μ	T_n, s	c	R_y	D, cm
0.960	5	0.1	4.291	1.965	0.606849
0.960	5	0.2	2.267	2.77	1.721962
0.690	5	0.5	1.173	4.404	4.865347
0.340	5	1.0	0.92	5.352	7.891051
0.170	5	2.0	0.877	5.568	15.16987
0.088	5	4.0	0.905	5.425	32.05534
0.088	5	5.0	0.917	5.367	50.62774
0.088	5	6.0	0.927	5.319	73.56185
0.088	5	6.5	0.931	5.3	86.6425
0.088	5	7.0	0.935	5.282	100.8272

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

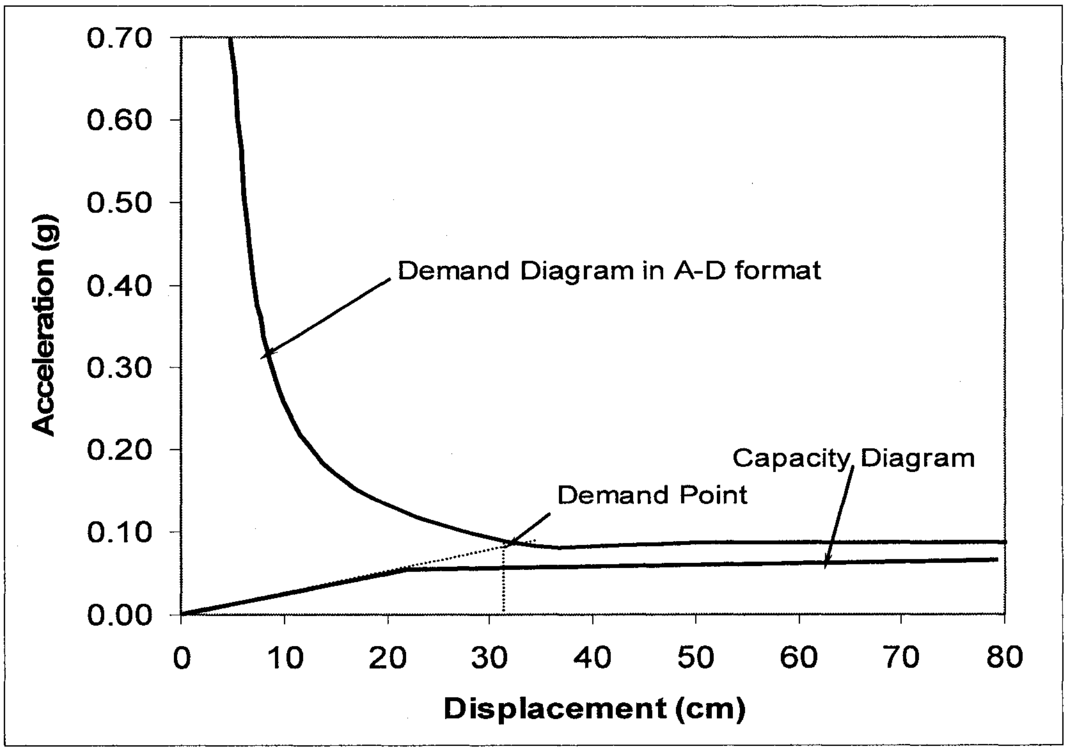

Figure 5.6: Capacity-Demand Diagram of Capacity-Demand-Diagram Method

From the intersection of Capacity diagram and Demand diagram the demanded displacement value of SDOF system is 32 cm. So, for MDOF system the displacement demand is (32*1.3) 41.6 cm. From the response history analysis (RHA) the maximum mean value of interstory drift is 6.1 cm. An average ratio of story drift to roof drift (Gupta and Krawinkler, 2000) has been chosen to convert the story drift to roof drift. So, the roof drift from RHA is (6.1/1.6) 3.9cm which is less than the value obtained by the Capacity-Demand Diagram design method. The converted interstory drift obtained from Capacity-Demand diagram method is 0.9% of story height. Now if the performance of the building is compared to the Vision 2000 prescribed level of performances than it will observed that the Capacity-Demand Diagram method satisfies both (Immediate

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Occupancy and Life Safety) levels of performance but the RHA satisfies only Life Safety performance objective.

5.6 Discussion

From the analysis of the results of the performance-based seismic design it has been observed except in the direct displacement-based seismic design method, the displacement demand is calculated and compared to the displacement of static analysis or the response history analysis. So, these methods mainly compare the seismic performance of the building and the design concept on the based of performance has not been perfectly reflected in the above discussed methodologies of seismic design of the structure. In the direct displacement-based seismic design the base shear of the building has been calculated for seismic force and then compared to the equivalent static base shear of the force-based design method and then seismic performance of the structure has been evaluated for this new base shear. So, for seismic design of structure the direct displacement-based design of performance-based seismic design is more suitable than other methods.

The advantage of all the performance-based seismic design methods is that these methods are simple and the performance objective can easily be determined without rigorous analysis or calculation. The main disadvantage of these methods is the accuracy is not high because all of them are based on many assumptions. As in conversion of MDOF to

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

SDOF the displacement shape is assumed. So, in the SDOF system the total mass of the MDOF system may not be accounted for properly. Also there is a huge variation of results obtained from various performance-based seismic design methodologies and from dynamic or static analysis. Clearly the methods need to be further developed in order for them to be used in the design practice.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Chapter 6

Discussion and Conclusion

6.1 Discussion:

The work presented in this thesis is the evaluation of seismic performance of moment resisting steel frame buildings. The design provision prescribed in the National Building Code of Canada (NBCC 2005) has been followed in the design of the steel frames. According to NBCC 2005 the seismic force has been applied in the buildings as equivalent static load and calculated from the seismic action by using empirical Equations. The design base shear coefficients of all the buildings except 5 story building are almost same and it is close to 0.02 and for 5 story building it is approximately 0.04 After finalization of column and beam sections from the force-based seismic design, the performance of the buildings has been evaluated through rigorous static and dynamic nonlinear analysis. Performance of both bare frame and infill frame has been evaluated. Nonlinear static Pushover analysis has been done to evaluate the ductility capacity under seismic action. In this method the frame has been pushed to a targeted roof displacement by applying the seismic force as lateral force in the inverted triangular shape and the yield displacement and ultimate displacement has been calculated to calculate the ductility capacity.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The calculated ductility capacities are 2.42, 3.11, 7.4 and more than 15 for twenty, fifteen, ten and five story bare frames, and 2.27, 3.67, 9.09 and more than 12 for twenty, fifteen, ten and five story infill frames, respectively. So, it has been observed that the ductile capacity of the buildings under seismic force decrease with the increase of building height. Also the normalized base shear of all buildings is less than the base shear at yielding. It has been observed that in pushover analysis the roof displacement corresponding to instability or 2.5% interstory drift is higher then the roof displacement corresponding to maximum of M+SD. Where, M & SD are the mean and standard deviation of the interstory obtained from Response History Analysis. Nonlinear response history analysis (RHA) has been done to evaluate the seismic demand of the buildings with both bare and infill frame. The synthesized and real ground motions are used in the RHA and the real ground motions are scaled to make those compatible to the specified location of the buildings.

Two types of scaling procedure are used to verify the effect of scaling method in the performance evaluation. The mean value (M) and mean plus standard deviation (M+SD) of the interstory drift of bare and infill frames are calculated from the RHA of real ground motions. Since the synthesized ground motions are not enough to calculate mean and standard deviation, the maximum interstory drift of each record of motion has been determined to evaluate the performance of the buildings. The maximum value of interstory drift for long period synthesized records varies from 1.19% to 2.16% of height of the story for the bare frame and 1.06% to 1.38% of height of the story for the infill frame. The maximum value of interstory drift for short period records varies from 1.91%

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

of height of the story to 2.33% of height of the story for bare frame and 1.46% of height of the story to 2.03% of total height of frame for infill frame. The value of mean plus standard deviation of the interstory drift for real ground motion varies from 1.54% of height of building story to 2.48% of total height of building story for bare frames and 1.12% to 2.14% for infill frames. This also satisfies the code specified level of performance.

The interstory drift demand is found to be less than the NBCC 2005 specified limit of interstory drift of 2.5% of story height. The NBCC 2005 specified only one performance objective which is collapse limit. But FEMA define different level of performance with different performance objectives, which gives the designer opportunity to play with performance level. In NBCC 2005 there is no alternative to select performance level and also the drift is too high for some performance level such as Immediate Occupancy (IO) as define in FEMA-273 (1997) and Vision 2000 (1995). FEMA-273 (1997) define interstory drift limit for Immediate Occupancy performance level is less than 2%. That means the buildings performance is not satisfactory for IO level. But for NBCC 2005 define performance level the evaluated performance of the buildings is satisfactory. For NBCC 2005 performance objective buildings are perhaps slightly overdesigned and there is a scope for optimization if performance-based design approach is followed. The confidence level of the dynamic analysis can be increased by increasing the number of the ground motion record. The mean(M) plus standard deviation (SD) value (representing 84% confidence level) of interstory drift has been compared to the code suggested limit of interstory drift. Some of these results are presented in Yousuf *et al.* (2007).

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

The thesis work also contains the review of the different methodologies for performance-based seismic design. To validate these methods the performance of a 20 story building has been re-evaluated by using different methodology of performance-based seismic design and check with the level of performance obtain from the RHA. It has been observed that level of seismic performance for which the building has been designed by the performance-based seismic design methodology is almost same as the level of seismic performance of the structure obtained from RHA.

6.2 Conclusions:

Based on the work on study presented in the thesis the following conclusions are made:-

- The code described force-based seismic design is not sufficient to ensure the performance of the building under seismic action.
- To ensure the desired level of performance of the buildings the evaluation of performance should be carried out through a set of linear or nonlinear analysis. Simplified methods may be used for simple buildings which respond mainly in first mode of vibration.
- The NBCC 2005 prescribed one level of performance, which is not sufficient for describing the level of seismic performance of all type of buildings. Because all type of buildings may not perform up to the same level of performance as expected in the code. Therefore, NBCC 2005 defined single level of performance should be redefined into multi level of performance.
- The ductility capacity assumed in the force-based design may not always be achievable (usually it is much less).

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

- In the pushover analysis the interstory drift has been determined for the roof displacement obtained from the Response History Analysis (RHA) and compared to the interstory drift obtained from RHA. It has been observed that the interstory drift of pushover analysis is higher than the interstory drift obtained from the RHA for the same roof displacement. It means the static analysis is not sufficient to judge the performance of the building under seismic force.
- The non structural elements have a significant effect on the performance of the building under seismic action. It reduces the drift demand from 30% to 15%.
- The method of scaling of the ground motion also effects the evaluation of performance of the buildings. The interstory drift of the building varies from 20% to 30% for different method of scaling. The uncertainties related to scaling and modeling needs to be considered in the design.
- The methodologies available for performance-based seismic design are based on simplified methods for performance evaluation of the buildings.
- The base shear of 20 story building determined from the Direct-displacement based design (Section 5.3.1) of performance-based seismic design method is almost half of the base shear of the force-based design, which indicates that the force-based design is more conservative. The performance of the building designed by the Direct-displacement based seismic design method should be re-evaluated through RHA to ensure the level of performance. So, this is not a true performance-based seismic design.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

6.3 Scope for future work:

The work presented in this thesis report is limited to two dimensional symmetric frame building. But the real world structures are always three dimensional with geometric irregularities. The geometric irregularity brings the eccentricity of loading with torsional action. So, further work is needed to understand the seismic performance of unsymmetrical frame of the buildings. It is also necessary to evaluate the effect of three dimensional modeling of the building on the performance metrics.

In this work the non-structural elements are modeled with the infill panel of clay brick masonry in one bay of each story level but the real structure may contain different type of infill panel in the different story levels. Therefore, further work may be done with variation of infill panel arrangement. The present work has been performed for the new structures using NBCC 2005 but the performance based-seismic design methodologies can be use for performance-based seismic retrofit of existing structure. However appropriate study needs to be conducted towards that.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Reference:

Adams, J. and Atkinson G., 2003, *Development of seismic hazard maps for the proposed 2005 edition of the National Building Code of Canada*, Canadian Journal of Civil Engineering, Volume 30 255-271.

Akbas B., Shen J., Kara FéI., Tugsal U.M., 2003, Seismic *behavior and Pushover Analyses in Steel Frames*, fifth National Conference on Earthquake Engineering, 26-30 May, Istanbul, Turkey, Paper No: AT-053

Aschheim M., 2004, *Yield Point Spectra: A Simple Alternative to the Capacity Spectrum Method*, ASCE online research library http://www.ascelibrary.org (Updated on October 2005)

Bagchi, 2001, *Evaluation of the Seismic Performance of Reinforced Concrete Buildings*, Ph. D thesis, Department of Civil and Environmental Engineering, Carlton University, Ottawa, Canada

Bagchi A., 2004, *A simplified method of evaluating the seismic performance of buildings*, Earthquake Engineering and Engineering Vibration, vol.3, No.2, pp 223-236.

Bertero VV., 1977, *Strength and deformation capacities of buildings under extreme environments*. Structural Engineering and Structural Mechanics, Pister KS (ed) ; Prentice Hall: New Jersey; pp 211-215.

Bertero R.D and Bertero V.V., 2002, *Performance-based seismic engineering: the need for a reliable conceptual comprehensive approach*, Earthquake Engineering and

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Structural Dynamics, Earthquake Engng Struc. Dyn.; 31:627-652 (DOI:10.1002/eqe.146)

CISC ,2004, *Handbook of Steel Construction*, Canadian Institute of Steel Construction, Willowdale, Ontario, Canada.

Chopra A.K. and Goel R.K., 1999, *Capacity-Demand-Diagram Method for Estimating Seismic deformation of Inelastic Structures: SDF systems*, Report No.PEER-1999/02 Pacific Earthquake Engineering Research Center, University of California, Berkeley, USA

Chopra A. K . and Goel R. K., 2001, *Direct Displacement-Based Design: Use of Inelastic Design Spectra Versus Elastic Design Spectra,* Earthquake Spectra, Vol.17, No.1.

Chopra A K and Goel R K , 2002, *A modal pushover analysis procedure for estimating seismic demands for buildings,* Earthquake Engineering and Structural Dynamics, Earthquake Engng Struct. Dyn. 2002; 31:561-582 (DOI: 10:1002/eqe.144)

CJCE , 2003, Special Issue on NBCC 2005. *Canadian Journal of Civil Engineering.* 2003 Vol.30, No.4

Corte G.D., Matteis G.De, Landolofo R., Mazzolani F.M., 2002, *Seismic analysis of MR steel frames based on refined hysteretic models of Connections*, Journal of Constructional Steel Research .**58**:1331-1345

CSA ,2001, *CSA Standard S16-01*,Canadian Standards Association, Rexdale, Ontario.

DeVall R.H, 2003, *Background information for some of the proposed earthquake design provisions for the 2005 edition of the National Building Code of Canada,* Canadian Journal of Civil Engineering. **30**: 279-286,

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Drysdale R.G. and Ahmed A.H., 1994, *Masonry Structures- Behavior and Design*, Prentice Hall.

Fajfar P.,2000, *A Nonlinear Analysis Method for Performance Based Seismic Design*, Earthquake Spectra, Vol.16, No.3pp.573-592

Fragiacomo M, Amadio C. and Rajgelj S., 2002, *Evaluation of the structural response under seismic actions using non-linear static method*, Earthquake Engineering and Structural Dynamics, Volume 32, issue 2 pp 291-308

FEMA-273,1997, *NEHRP Guidelines for the Seismic Rehabilitation of the Buildings,* prepared for the Building Seismic Safety Council (BSSC), Washington,D.C.USA

FEMA- 310,1998, *Handbook for the Seismic Evaluation of Buildings – A Prestandard,* prepared by the American Society of Civil Engineers for the Federal Emergency Management Agency, Washington, D.C. U S A

FEMA-350, 2000, *Recommended Seismic Design Criteria for New Steel Moment-Frame Building,* prepared by the SAC Joint Venture for the Federal Emergency Management Agency, Report No.FEMA-350, Washington, D.C, U S A.

Foo S., Naumoski N., Saatcioglu M., 2001, *Seismic Hazard, Building Codes and Mitigation Options for Canadian Buildings*, Office of Critical Infrastructure Protection and Emergency Preparedness, 2nd Floor, Jackson Bldg. Ottawa, ON K1A 0W6, Canada

Ghobarah A., 2001, *Performance-based design in earthquake engineering: state of development*, Engineering Structures 23 : 878-884.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Goel R. K. and Chopra A.K, 2001, *Improved Direct Displacement-Based Design Procedure for Performance-Based Seismic Design of Structures,* ASCE online research library http://www.ascelibrary.org (Updated on October 2005).

Gupta A. and Krawinkler H.,2000,*Estimation of seismic drift demands for frame structures*, Earthquake Engineering and Structural Dynamics, Earthquake Engng Struct.Dyn.2000; 29:1287-1305

Hannan M., 2006, *Earthquake Resistant Design of a Twenty Story Building with Steel Moment Resisting Frames,* M.Eng. Project , Department of Building Civil and Environmental Engineering, Concordia University, Montreal, Canada

Herrera R., Ricles J. M., Sause R., Lewis B., 2003, *Seismic Performance Evaluation of Steel Moment Resisting Frames With Concrete Filled Tube Columns* ', Dept. of Civil Eng., Lehigh University, Bethlehem, Pennsylvania, http://www.ncree.org.tw

Humar J. and Ghorbanie-Asl M., 2005, *A New Displacement-Based Design Method for Building*, 33rd Annual General Conference of the Canadian Society for Civil Engineering, GC-136, Toronto, Ontario, Canada.

Humar J. and Bagchi A., 2004, *Seismic Performance of Concrete Frame/Wall Buildings Designed According to NBCC 2005* , 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, Paper ##1535 (Proceedings on CD ROM).

Humar, J. and Mahgoub M. A., 2003, *Determination of seismic design forces by equivalent static load method*, Canadian Journal of Civil Engineering, Volume 30 287-307.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

IDARC 2D. (2006).A Computer Program for Seismic Inelastic Structural Analysis, Department of Civil, Structural and Environmental Engineering, University at Buffalo, Buffalo, New York, http://civil.eng.buffalo.edu/idarc2d50/

Kilar V. and Fajfar P., 1997, *Simple Push-over Analysis of Asymmetric Buildings*, Earthquake Engineering and Structural Dynamics, Vol.26, 233-249

Kunnath S.K., 2006, *Performance-Based Seismic Design and Evaluation of Building Structures*, Earthquake Engineering for Structural Design, edited by Chen, Lui & Taylor. Francis Group, FL, U S A.

Lee k. and Foutch D.A., 2002, *Performance evaluation of new steel frame buildings for seismic loads*, Earthquake Engineering and Structural Dynamics, *Earthquake Engng Struct. Dyn.* 2002: **31**:653-670

Medhekar M.S. and Kennedy D.J.L., 2000, *Displacement-based seismic design of buildings-theory,* Engineering structures. 22: 201-209.

Natural Resources Canada (NRCAN), "Earthquakes Canada – East", http://seismo.nrcan.gc.ca/major_eq/majoreq_e.php (Updated on August 2006)

Naumoski N., Saatcioglu M., and Amiri-Hormozaki K., 2004, *Effects of Scaling of Earthquake Excitations on the Dynamic Response of Reinforced Concrete Frame Buildings*, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, Paper No. 2917.

NBCC 2005, *National Building Code of Canada, 2005*, Canadian Commission on Building and Fire Codes, National Research Council of Canada, Ottawa

Newmark N.M. and Hall W.J., 1982, *Earthquake Spectra and Design*, Earthquake Engineering Research Institute, Berkeley, CA.

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

NISEE, 2005, Software library, University of California, Berkeley, California. http://nisee.berkeley.edu/software/

El Kafrawy O, Yousuf M, and Bagchi A, "Computer Aided Design and Analysis of RC Frame Buildings Subjected to Earthquakes", ASCE/ICCCBE-XI Joint Confrerence, Montreal, June, 2006, pp 1254-1263.

PEER, 2006, *Pacific Earthquake Engineering Research Center: NGA Database*, http://peer.berkeley.edu (Updated on January 2006)

Prakash V., Powell, Graham H, Campbell, Scott D., 1993, *Drain-2DX: Static and Dynamic analysis of Inelastic Plane Structure*, Department of Civil Engineering, University of California, Berkeley.

Saatcioglu M. and Humar J., 2003, *Dynamic Analysis of buildings for earthquake-resistant design, Canadian Journal for Civil Engineers*, Can.J.Civ.Eng. **30**: 338-359.

SAC, 2000, *Recommended Seismic Design Criteria for New Steel Moment-Frame Building* by the SAC Joint Venture for the Federal Emergency Management Agency, Report No.FEMA-350, Washington, D.C, U S A.

SAP 2000. (2006), *Structural Analysis Program*, CSI Berkeley, 1995 University Ave. suite 540 , Berkeley, CA , USA, http://www.csiberkeley.com

SEAOC Vision 2000 Committee, 1995, *Performance-based seismic engineering*, Report prepared by Structural Engineers Association of California, Sacramento, CA.

Shustov V., 1999, *A New Concept of a Performance Building code for Earthquake Design*, California State University Northridge, U S A, http://www.seaint.org

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.

Tremblay, R. and Atkinson, G.M, 2001, *Comparative study of in elastic seismic demand of eastern and western Canadian sites*, Earthquake Spectra, 17(2): 333-358

Tremblay R. and Poncet L., 2005, *Seismic Performance of Concentrically Braced Steel Frames in Multistory Buildings with Mass irregularity*, Journal of Structural Engineering, Vol.131.No.9.

Yousuf M., El Kafrawy O., and Bagchi A., 2006, Automated Analysis and Performance Evaluation of Earthquake Resistant Steel Frame Building, ASCE/ICCCBE-XI Joint Conference, Montreal, Canada, pp 1274-1283.

Yousuf M., Hannan M., and Bagchi, A., 2007, "Seismic Performance of a twenty-storey steel frame building", Accepted in the 9th Canadian Conference on Earthquake Engineering (9CCEE), Ottawa, Paper# 1327.

Yun S.Y., Hamburger R.O., Cornell C. A., and Foutch D.A., 2002, *Seismic Performance Evaluation for Steel Moment Frames,* Journal of Structural Engineering, Vol. 128, No. 4, pp. 534-545

Reproduced with permission of the copyright owner. Further reproduction prohibited without permission.