Seismic Performance Evaluation of

Reinforced Concrete Shear Wall Seismic Force Resisting Systems

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A Thesis

in

The Department

Of

Building, Civil & 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

December 2008

Your file Votre référence
ISBN: 978-0-494-63212-3
Our file Notre référence
ISBN: 978-0-494-63212-3

ABSTRACT

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Building codes in various jurisdictions including Canada are moving towards performance-based design approaches where a structure is designed not only to have adequate strength, but also for the required performance attributes, such as, adequate deformability. From that point of view, performance assessment of structures in the design phase plays an important role in the implementation of the above concept. The focus of this article is to study the seismic performance and torsional sensitivity of reinforced concrete shear wall buildings designed using the seismic provisions of the National Building Code of Canada (NBCC 2005) and the Canadian standard on reinforced concrete buildings (CSA-A23.3-04). The buildings considered here are of regular plan and the height is limited to what is permitted for the use of the Equivalent Static Load (ESL) method of the building code. A set of three reinforced concrete buildings of four, eight and sixteen storey heights are designed here. The buildings are assumed to be located in Vancouver and various levels of accidental mass eccentricity up to 10% as permitted in the ESL method, are considered. After the preliminary design of the buildings using the ESL method, dynamic elastic Response Spectrum Analysis "RSA" has been performed to compare the base shear and make appropriate refinement in the design as suggested in NBCC. The buildings are then analyzed using inelastic dynamic analysis with fifteen recorded accelerograms of past earthquakes. The

earthquake records are selected such that the peak velocity to acceleration ratio of each record is compatible to the seismicity of Vancouver. The ground motion records are scaled to fit the design spectrum using two different methods. The performance parameters such as the demand to capacity ratios for storey drift, plastic rotation, and storey shear are extracted from the results of the inelastic dynamic analysis. The statistical quantities such as mean, standard deviation and the maximum values of the demand to capacity ratios are found to be well below the acceptable limits, while the storey shear, exceed the limit in all cases. It also is observed that none of the buildings are torsionally sensitive within the code specified range of eccentricity for which ESL method is applicable. The changes in the dynamic response due to the change in eccentricity are almost proportional within the range of eccentricity considered here. Another point to note here is that while results for the four and eight storey buildings are not very sensitive to the method of scaling of the ground motion records, for the sixteen storey building, it is not so.

ACKNOWLEDGEMENTS

I would like to thank my thesis supervisor, Dr. Ashutosh Bagchi, for all his patience and guidance throughout my research. Without his support and motivation, I would not have been able to write this thesis. I would also like to thank Prof. Oscar A. Pekau for his support and encouragement.

I admire the encouragements that I have always received from Dr. Pershia Samadi and Dr. Mahbod Bassir, and also their editorial comments throughout writing my thesis.

My mother whom I can never thank her enough for everything that she did for me; my father and brother who were with me when I started this journey, but are really missed now... their unconditional love will always be in my mind; also, my other brothers for their love and devotion.

And my daughter, my little angel, who has added so much to the beauty of my life.

I am also grateful for having a wonderful individual, who has always been by my side, my soul mate, my wife.

1. TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABREVIATIONS

ARS	Acceleration Response Spectrum
CCBFC	Canadian Commission on Building and Fire Codes
CM	Center of Mass
DRS	Design Response Spectrum
DRSL	Design Response Spectrum Load
EDP	Engineering Demand Parameters
ESL	Equivalent Static Load
FEMA	Federal Eemergency Management Agency
FAM	Full Area Method
IDA	Incremental Dynamic Analysis
NBCC	National Building Code of Canada
NEHRP	National Earthquake Hazards Reduction Program
OBC	Objective-based codes
OM	Ordinate Method
PAM	Partial Area Method
PBEE	Performance Based Earthquake Engineering
PBSD	Performance Based Seismic Design
PEF	Post Earthquake Fire
RC	Reinforced Concrete
RSL	Response Spectrum Load
SEAOC	Structural Engineers Association of California

SFRS	Seismic Force Resisting System
SRSL	Scaled Response Spectrum Load

LIST OF SYMBOLS

A	pick ground acceleration
A_v	area of shear reinforcement within a distance s
B	maximum of all values of Bx in both orthogonal directions
b_w	wall thikNess
c	depth of the neutral axes
D	dead load
Dnx	floors' dimension perpendicular to the direction of earthquake load at level x
d_v	effective shear depth
E_c	concrete modulus of elasticity
f'_c	specified compresive strength of concrete
F_a	acceleration-based site coefficient
F_t	portion of V to be concentrated at the top of the structure; reflecting higher modes effect
F_v	velocity-based site coefficient
F_x	lateral force applied at level x
f_y	specified yeild strength of reinforcement
h_n	buildings height above the base
h_w	vertical height of wall
h_x, h_i	heights above the base to levels x and i respectively,
I	importance factor of the building

L	live load
l_w	width of shear wall
M_f	factored moment
M_p	probable flexural resistance
M_v	factor to account for higher modes effect on base shear
N	total number of storeys
R_o, R_d	over-strength and ductility factors respectively
S	spacing of shear reinforcements
$S(T)$	design spectral response acceleration values
$S(T_a)$	design spectral response acceleration in "g"
$S_a(T)$	5% damped spectral response acceleration in "g"
T1	Building fundamental period
T2	Second period of building's vibration
T_a	fundamental period of vibration in S
V	lateral earthquake force at the base of the structure
V	pick ground velocity
Vd	lateral earthquake design force at the base of the structure
Ve	lateral earthquake elastic force at the base of the structure
V_f	factored shear force
V_r	factored shear resistance
W	dead load of the structure plus twenty five percent of the snow load; also storey weight.
β	factor accounting for shear resistance of cracked concrete

γ_w wall over strength factor

Δf deflection of the top of a wall due to the effect of factored loads

ε_{cu} maximum strain at the extreme concrete compression fibre at ultimate

θ angle of inclination of diagonal compressive stresses to the longitudenal axis of the member

θ_{id} wall inelastic rotational capacity

θ_{ic} wall inelastic rotational demand

ϕ_c resistance factor for concrete

ϕ_s resistance factor for reinforcing bars

δave average of the displacements of the extreme points of the structure at level x generated by the above forces,

δmax maximum storey displacement at the extreme points of the structure at level x in the direction of the seismic load induced by the equivalent static forces and exerted at a distance equal to $\pm 0.1 Dnx$ from the Center of Mass (CM) at each floor

CHAPTER 1

1. INTRODUCTION

1.1 Motivation

Earthquake engineering has come a long way since its confinement during 1960 and May 1963, when it started as a little unit in the Division of Planning (History of Earthquake Engineering 2008), and is growing in fast pace as we gain more experience over time. Each time an earthquake takes place, we find out something new and earthquake engineering develops from new learning. The aftermath of the 1989 Loma Prieta and 1994 Northridge earthquakes are such examples from which we learnt that sometimes an only life-safe building is not sufficient (PREPARE FOR EARTHQUAKES, 2008).

One of the fundamental goals of the building design regulatory agencies is prevention, or mitigation, of losses from hazards including earthquake. To accomplish such objective, the level of performance expected from buildings, during and after an earthquake, should be known. Current building code-specified procedures have been provided to maintain life safety in the largest earthquakes and decrease property damage and loss in the moderate ones; however, there have been dramatic financial losses due to seismic activities and the fire following them, for instance, the amount of America's financial losses in the 1990s' is estimated twenty times bigger than that of three earlier decades all together (FEMA 349, 2000).

Building owners, insurers, lending institutions and government agencies have had a fundamental misperception about the expected performance of a building that satisfies code requirements in the sense that these buildings would be earthquake proof; this is one of the reasons that has led to unexpected, even ruining, financial losses which contributes to other causes like denser population, aging buildings, incompatibility of buildings with the new improved code and standards, increasing cost of down time, or business interruption, damages to building non-structural components and its contents.

Traditionally, life safety and property loss prevention have been achieved via indirect ways by which designer has never actually had an assessment of the performance level of a building; such design may or may not satisfy the level of damage and loss protection perceived by the owner. To rectify this insufficiency, many agencies have been working toward development of better criteria. The result was formation of Performance-Based Earthquake Engineering (PBEE), a rather new but fast growing thought that is present in many recently published guidelines like Structural Engineers Association of California (SEAOC) Vision 2000 (1995), and FEMA 356 (2000).

PBSD permits engineers to design buildings with more foreseeable and particular reliable levels of performance in the event of an earthquake of a given magnitude. It also allows the owners, financially or else, to quantify the anticipated risk to their buildings; this would also allow them to choose a level of performance that fits into their needs in addition to the basic safety level.

Therefore, a building with 50 years' lifetime may be needed to undergo no damages under an occasional event, 50% in 50 years. Although suffering some damages in rare case of 10% in 50 years event, it should however be able to remain repairable, and stays stable and life-safe for 2% in 50 year extremely rare events, although, it may finally have to be torn down.

PBSD is the basis on which, in PBEE, methods can be established to quantify structural damage (beams, columns, foundations, etc.) and non-structural damage (partitions, glass panels and so on), means to approximate the number of casualties, the building contents' loss, the building downtime, the expense of rehabilitation, also price inflation assessment after a major earthquake. So, we need powerful and simplified analysis methods that will accurately analyze building structures and estimate the (distribution of) Engineering Demand Parameters (EDP) at any possible level of vibration, and in particular, the level of shaking that will make a structure to exceed a defined limit-state, therefore failing a specified performance objective.

Several methodologies have been proposed to fulfill this role, such as push over analysis, modal pushover analysis, dynamic time history analysis, and Incremental Dynamic Analysis (IDA), where general procedures in PBSD can be organized as: 1) modeling a building's design; 2) Simulate the performance of the design for various severities of earthquake records; 3) Assessing the level of damage, if any, nurtured by the structure by using the outputs from each simulation; 4) Evaluating the possible financial losses by using information obtained in stage three; 5) Adjusting design of the building and revising steps 1 through 5 until the desired magnitude of property and financial loss is projected.

Structural members are to be designed to satisfy the requirements of serviceability and safety limit states for various environmental conditions. Fire following an earthquake also represents one of the most severe undesired conditions that in first place depend on the level of performance of the building under the earthquake; when other measures for containing the fire fail, structural integrity is the last line of defence. In pursuing the above mentioned steps, adequate attention must also be given to the Post Earthquake Fire (PEF) scenarios (Mousavi et.al., 2008).

The general steps discussed earlier can be even more simplified, say for a particular group of buildings, for instance those that fit into Equivalent Static Load (ESL) method requirements. Such simplification can provide a more precise estimate of these buildings' performance level for particular groups of earthquakes that can then be further developed for wider intensity spectrum seismicity. This is the goal in this research.

1.2 National Building Code of Canada NBCC; Past and Present

Building codes, including National Building Code of Canada (NBCC) traditionally have included:

1) Specifications on components,

2) Allowable installation methodologies,

3) Minimum and maximum room and exit sizes and location,

4) Qualification of individuals or corporations doing the work,

Despite the fact that, historically, the building codes change to ensure that the problem never happens again when a problem occurs, the above requirements have been usually a combination of prescriptive requirements that spell out exactly how something is to be done, and poorly defined performance requirements (e.g. live safe) which just outline what the required level of performance is and leave it up to the designer how this is achieved.

As mentioned earlier, in recent years there has been a worldwide move among the building code authorities toward performance requirements. In Canada, in the early 1990's, the Canadian Commission on Building and Fire Codes (CCBFC), too, was faced with similar dilemma. That problem was a reflection of concerns addressed by three separate groups of Canada's code using community.

The first group – primarily stakeholders, designers, and product manufacturers- were requesting for performance-based codes, perceived to be more open to innovation.

The second group – primarily house builders – was content with the Codes' prescriptive content and worried the loss of this "formula -based approach" if performance-based codes were to be used.

The third group– primarily enforcement officials – had heard fearful reports about the outcomes of the adoption of performance-based codes in other countries and worried that the introduction of performance-based codes would cause an arbitrary atmosphere in which they would have no ground for turning down ill-considered designs and products (Bergeron et. al., 2004).

To comprehensively solve the dilemma, The CCBFC and the staff of the Canadian Codes Centre at the National Research Council of Canada looked for an answer that would satisfy the objectives and avoid the fear of all parties. Considering all the above mentioned issues, NBCC 2005 is presented in the objective-based format; where, the objectives express the aim that codes intend to achieve. Seismic performance of buildings using the draft version of NBCC 2005 (NRC, 2005), was first studied by Bagchi (Bagchi, 2001), parts of which was updated in Humar and Bagchi (2004).

A series of publications related to the development of the NBCC explaining the seismic provisions are published in a special issue of the Canadian journal of Civil Engineering (CJCE, 2003)

The objectives define the codes and give the reasoning behind the acceptable solutions. Using the bottom-up analysis of the codes and the taking advantage of the feedbacks received in the consultation on objective-based codes (OBC), the CCBFC found out the objectives of the codes as represented bellow:

- Safety
- Health
- Accessibility (NBC)
- Fire and Structural Protection of Buildings (NBC)
- Protection of Buildings and Facilities from Water and Sewage Damage (NPC)
- Fire Protection of Buildings and Facilities (NFC)

The objectives are discussed in Division A of the OBC. Sub-objectives (second-level and third-level objectives) that provide more in depth information about what the codes are intended to achieve. The NBCC "Safety" objective has 5 second-level: "Fire Safety, Structural Safety, Safety in Use, Resistance to Unwanted Entry and Safety at Construction and Demolition Sites."

Although they have many characteristics in common, OBC and performance-based codes have certain key differences. Two public consultations have showed that these differences have certainly addressed the concerns of code users and that the concept is largely backed up by all three groups of the code users

The primary idea behind the 2005 OBC in Canada is the realization that the acceptable solutions present an implicit expression of the levels of building performance that are satisfactory to those involved. "Acceptable solutions" are provisions that could be either prescriptive or performance-based that can also be seen as a point of reference against which other ways of complying with the codes' objectives and performance expectations will be evaluated or compared. In an OBC, every acceptable solution is related to at least one of the objectives and functional statements in the code.

1.3 Earthquake in Canada

1.3.1 Elements causing earthquake in Canada

The coastal region in western Canada forms part of the circum-Pacific earthquake belt also kNown as "ring of fire" that is an area of frequent earthquakes and it is in a horseshoe shape. Almost continuous series of tectonic plate movements are taking place in these regions; and about 90% of the world's earthquakes, 80% of which are the world's largest earthquakes, occur along this region. It, in addition to coastal region in western

Canada, includes seismically active regions like Alaska, California, Mexico, Nicaragua, Chile, New Zealand and Japan.

Seismic activities along the West Coast of Canada are originated by the slow movement of a string of main tectonic plates

Two of the largest existing tectonic plates, the North American Plate and the Pacific Plate in the Queen Charlotte Islands region, are sliding against each other at nearly 6 millimetres per year. The Juan de Fuca Plate, Farther in south, is forcing under the continent at about 40 millimetres per year (Earthquakes Canada, 2008).

But, eastern part of Canada rests totally within the North American Plate and is far away from the active boundaries of this plate in the mid-Atlantic in the east, and just off British Columbia in the west. The forces causing earthquakes in east part of Canada are of a diverse nature. It seems that the slow movement of the North American Plate away from the Mid-Atlantic Ridge may activate old sectors of weakNess and faults such as the St. Lawrence Valley, which would cause them to readjust and have room for the ongoing strain.

1.3.2 Earthquake's Impacts in Canada

Several big earthquakes have taken place in the short history of Canada; the first documented of which can be found in Jacques Cartier's journal, where it talks about occurrence of an important earthquake. It probably took place about 1534 near La Malbaie, about 100 kilometres downstream of the Québec city. Also, aboriginal legends in West coast allude to earthquakes verifying that earthquakes regularly take place in some parts of Canada (Earthquakes Canada, 2008).

The magnitude of these old earthquakes has been estimated based on the description of damages and ground vibration recorded in historical documents. Two of the most significant Canadian earthquakes that happened before existence of any measuring device, would probably have had a magnitude of 7.0 to 7.5 on the Richter scale. One of these two earthquakes happened in near the mouth of the Saguenay River in 1663; the other was in 1872, east of Vancouver.

In Canada, the 20th century largest earthquake (magnitude 8.1) happened in 1949 in the lightly populated Queen Charlotte Islands. In 1929, a tsunami formed by an offshore earthquake of magnitude 7.2 south of Newfoundland drowned 28 people. Also the largest earthquake in eastern North America since 1935 took place in November 1988 when an earthquake of magnitude 6 in the Saguenay region of Quebec caused tens of millions of dollars in damage. Fig. 1-1 shows the most significant earthquakes of the 20th Century.

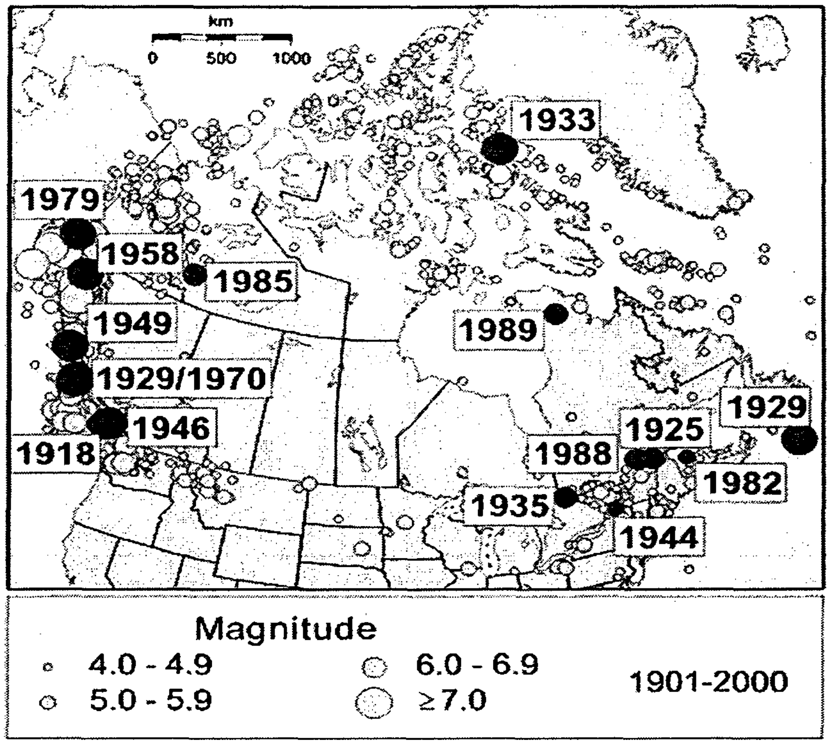


Fig. 1-1: Significant Earthquakes of the 20th Century (Earthquakes Canada, 2008)

Fig. 1-2 and Table 1-1 show the date, magnitude and location of the ten biggest earthquakes ever to be found in Canada or its territorial waters. Note that several big earthquakes taking place in neighbouring Alaska or Washington State have also had an effect on people living in western Canada. Earthquakes Magnitudes before the 20th century are less precise since they have been approximated from non-instrumental data.

Table 1-1: Top 10 earthquakes in Canada (Earthquakes Canada, 2008)		
Date	Magnitude	Location
1700	9.0	Cascadia subduction zone, British Columbia.
1949	8.1	Offshore Queen Charlotte Islands, British Columbia.
1970	7.4	South of Queen Charlotte Islands, British Columbia.
1933	7.3	Baffin Bay, Northwest Territories.
1946	7.3	Vancouver Island, British Columbia.
1929	7.2	Grand Banks south of Newfoundland.
1929	7.0	South of Queen Charlotte Islands, British Columbia.
1663	7.0	Charlevoix, Quebec.
1985	6.9	Nahanni region, Northwest Territories.
1918	6.9	Vancouver Island, British Columbia.

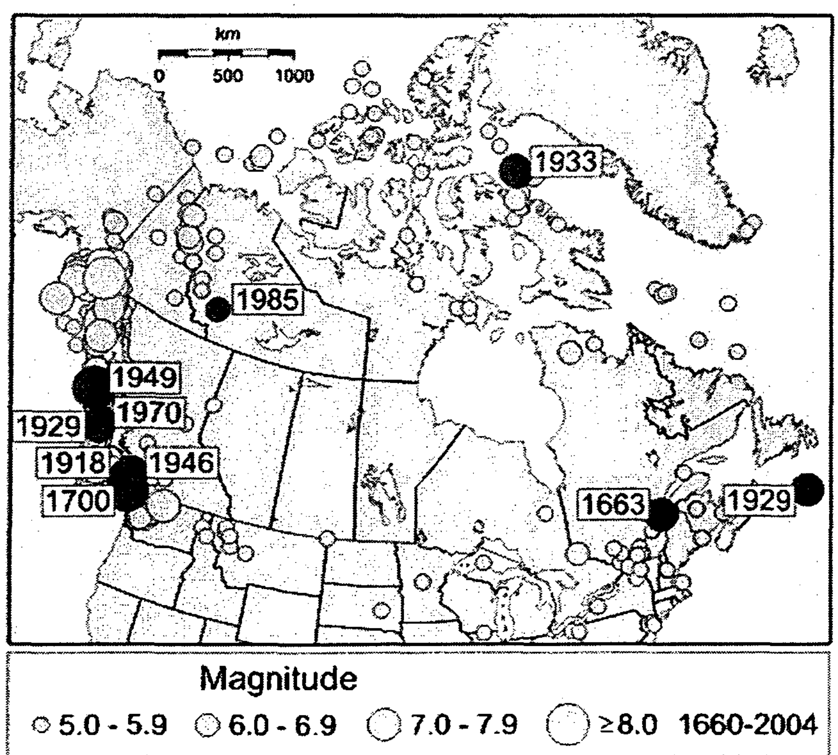


Fig. 1-2: Top 10 earthquakes in Canada (Earthquakes Canada, 2008)

1.4 **Problem Statement**

ESL method provides a simplified way for analyzing buildings that satisfy certain requirements set forward by the code, which would provide the information required for design of the buildings. However, similar to other common methods of practice such as linear response spectrum method, it does not offer a well defined level of performance.

On the other hand, methods that currently can be used in PBSD are still either much more complicated, like inelastic dynamic analysis, or their results are not very much close to that of the most accurate one, like pushover analysis method. So, developing ESL method and making its results comparable with at least methods like pushover analysis would be an appealing improvement.

1.5 **Objectives**

The objectives of this research is to study the ESL method seismic design provided in the current edition of NBCC in order to

- determine buildings level of performance and distribution and dispersion of EDP that ESL method of design yields for case of shear wall seismic force resisting systems "SFRS",
- study buildings torsional behaviour and sensitivity;
- Find possible pattern/s in the buildings' performance and develop a likely method and/or expression/s to modify and/or eliminate the possible unwanted level of behaviour that would provide more realistic, precise and consistent estimate of EDP in a simplified manner.

CHAPTER 2

2. LITRUTURE REVIEW

2.1 **Performance Based Seismic Design (PBSD)**

The origin of the development of "Tentative Provisions for the Development of Seismic Regulations for Buildings" goes back to the occurrence of San Fernando earthquake at U.S.A. in 1971. It was published by Applied Technology Council (ATC) of U.S. and referred to as ATC 3-06 (1978) document.

Building Seismic Safety Council (BSSC) studied and adapted systematically this document and then published it as NEHRP's first recommended provisions for the development of seismic regulations for new buildings (NEHRP, 1985), which constituted the basis of PBSD and later editions (Ghosh, 2004).

Performance-based design was created in the U.S. as main approach to resolve seismic design problem in the 1990's; in particular, code-based strength and ductility requirements related to he design of new building could not be virtually or consistently applied to the assessment and improvement of existing building ((FEMA 445, 2006).

To erect an economical building which is safe in predictable conditions, the selection of structural, nonstructural, and geotechnical systems and their materials and configuration, constitutes the structural design in most of the current codes.

Structural engineers applied traditionally allowable-stress design (ASD) and load-and-resistance-factor design (LRFD) based on individual structural elements and connections to guarantee that none of them will support loads or undergo deformation beyond their resistance.

Consequently, the performance capability of some of the buildings designed to these prescriptive criteria could be better than the minimum standards anticipated by the code, while the performance of others could be worse (PEER, 2008).

Performance Based Design looks for assuring that a designed building as a whole and in terms of safety and serviceability will behave in some expected manners. The First generation of PBSD procedures initiated the performance concept as discrete performance levels defined with names that meant to imply the anticipated level of damage. Such levels of damage have been classified as Collapse, Collapse Prevention, Life Safety, Immediate Occupancy, and Operational Performance. They, in addition, brought in the concept of performance linked to damage of both structural and non-structural components. Performance Objectives were worked out by relating one of these levels performances to a particular level of earthquake hazard. In brief, the first-generation of PBEE approach presume that if for instance, a particular level of ductility demand, is reached, then the designer can be reasonably ensured of an affiliated performance level.

Founded on all other earlier efforts, PEER presented and changed the assumption of earlier generations (in that if for instance, a particular level of ductility demand, is reached, then the designer can be reasonably ensured of an affiliated performance level) with more clear, probabilistic explanations of physical damage and the system's level of performance. By employing such methodology, the engineer will be able to tell a building owner that, for example: "The probability in which your building will be operational after an earthquake of such intensity is this much; and here is the probability that costs of repair will not go beyond e.g. US$ 500,000 dollars during the next 50 years.

2.2. Methods of Analysis

Advancement of the computer technology gave an opportunity for expanding the structural analysis from static to dynamic, and from linear to non-linear permitting for more realistic foreseeing on the status of structures subjected to, particularly the lateral forces. Such development in combination with experimental results, and what we have learned from real events comprising earthquakes, have driven structural analysis into a formal PBSD phase, giving a more vibrant image of the post earthquake status of buildings.

For instance, many inelastic static analyses methods, except methods implemented in Federal Emergency Management Agency (FEMA) documents, have been established and expressed in form of Acceleration-Displacement (A-D) an illustration of which is shown in Fig. 2-1. In such arrangement, the capacity of a structure is directly evaluated with the demands resulting from seismic ground motion on the structure. The graphical illustration of the concept makes it possible to have a visual interpretation of the process and also of the relations between the basic parameters affecting the seismic response. In this process the structure's capacity, which is symbolized by a force-displacement curve, is calculated from a non-linear static pushover analysis. Then the base shear forces and roof displacements are then respectively transformed into the spectral accelerations (A) and spectral displacements (D) of an equivalent single-degree-of-freedom system. These spectral values outline the capacity diagram in (A-D) format. Then the capacity curve and the demand curves are drawn together in one diagram. It is the definition of earthquake demand spectrum that signifies the primary difference between different methods. In all methods, the crossing point of the capacity curve and the demand spectrum gives an approximation of the displacement demand and inelastic acceleration (strength).

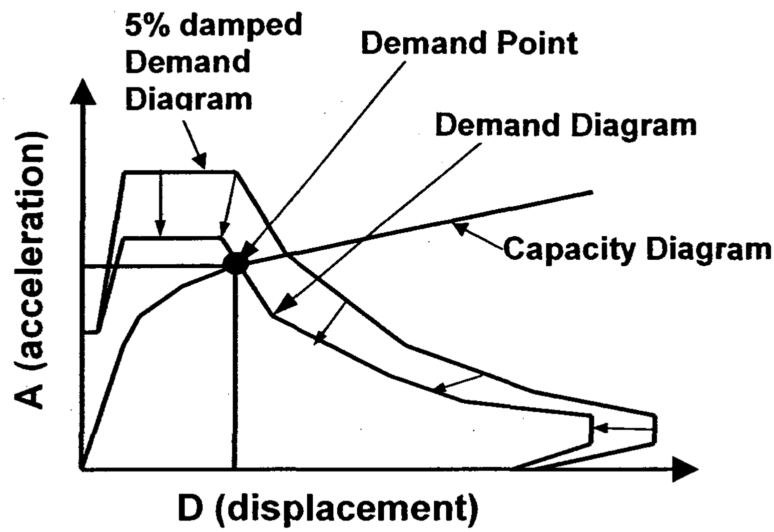


Fig. 2-1 : Capacity Spectrum Method (Chopra and Goel, 1999)

Methods developed to make PBSD happen also include modal pushover analysis, Incremental Dynamic Method (IDM), N2 method, Incremental N2 method (IN2), Displacement-Based Design Method (DBM), Yield point spectra, Direct Inelastic Earthquake Design Using Secont Stiffness.

For instance, Chopra and Goel (2001) demonstrated that pushover analysis of a one-story system gives a well prediction of utmost earthquake demands, and developed a modal pushover analysis (MPA) method for linearly elastic buildings and showed that it is equivalent to RSA method. The MPA technique was then developed into inelastic buildings.

Bagchi (2004) presented a simplified technique for seismic performance assessment of a MDOF by converting it to a SDOF system. In this technique the ultimate response of SDOF system is achieved by dynamic or response spectral analysis and a relation between the maximum story drift and the roof displacement of MDOF system is developed from the pushover analysis; this derived relation will then be utilized to interpret the response of SDOF obtained from dynamic analysis.

Incremental Dynamic Analysis or Dynamic Pushover is another method that involves a series of scaled accelerogram nonlinear dynamic analyses, where the record's intensity measures (IMs) are, preferably, selected to address the whole range from elastic to inelastic and at last to collapse of the structure. The intention is to trace Damage Measures (DMs) of the structural model at each IM level of the scaled accelerogram, the consequential response values oftentimes is plotted against the intensity level as continuous curves (Vamvatsikos and Cornell, 2002).

Direct displacement-based design technique involves a simplified procedure to approximate the deformation of an inelastic SDF system due to earthquake, correspond to the structure first (elastic) mode of vibration. This step is usually achieved by analysis of an equivalent linear system utilizing elastic design spectrums. Goel and Chopra (2001) also derived a method that is based on the concepts of inelastic design spectra (Goel, and Chopra, 2001).

Many agencies took advantage of the past researchers work on the methods of analyses for PBSD, such as earlier mentioned techniques, and then developed guidelines and pre-standards forming the First-generation of PBSD procedures. Such procedures resulted in an important enhancement over building code procedures practiced at that time in that they offered a systematic way of designing building through which a desired level of performance can be reached.

In conventional practice, seismic design has specifically been performed for just a single design event level, at which a level of performance commonly phrased "life safety" has been aimed. Such life safety performance level has been described just qualitatively and in terms of considerations that are inadequately expressed, like limiting damage to structural elements, avoiding major falling hazards, and maintaining egress for occupants. ongoing efforts at performance-based engineering are looking for reliable methods of meeting multiple performance targets through clear design procedures. In this regard, SEAOC's Vision 2000 (SEAOC, 1995) and the NEHRP Guidelines (ATC, 1996) are similarly developed systems of designating building performance that somewhat utilise different terminology, and are of the major works in providing more quantitative definitions of building performance levels. Table 2-1 summarizes the performance levels defined by these projects.

Table 2-1: Definitions of Structural performance (Hamburger, 1997)

Performance Level		Description
NEHRP Guidelines	Vision 2000	
Operational	Fully Functional	No significant damage has occurred to structural and non-structural components. Building is suitable for normal intended occupancy and use.
Immediate occupancy	Operational	No Significant damage has occurred to structure, which retains nearly all of its pre-earthquake strength and stiffness. Non-structural components are secure a most would function, if utilities available. Building may be used for intended purpose, albeit in an impaired mode.
Life Safety	Life Safe	Significant damage to structural elements, with substantial reduction in Stiffness, however, margin remains against collapse. Non-Structural elements are secured but may not function. Occupancy may be prevented until repairs can be instituted
Collapse prevention	Near Collapse	Substantial structural and non-structural damage. Structural strength and stiffness substantially damaged. Little margin against collapse. Some falling debris hazards may have occurred .

Vision 2000 (1995) emphasises on defining what represents a frequent, rare or very rare earthquake (Table 2-2), and focuses on detailed descriptions in what the performance conditions are that one wants for different types of events and structures. The Vision 2000 (1995) document suggests that buildings to be constructed based on their intended occupancies and usage to meet the performance objectives shown in Fig.2-2. In this figure a relationship is developed between the performance target, type of facility, and probability of earthquake occurrence, which is then linked to response parameters related to each performance objective. These parameters are identified and some initial estimates are quantified.

			Performance objectives			
Earthquake Classification	Return Periods	Probability of Occurrence	Fully Operational	Operational	Life Safe	Near Collapse
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				

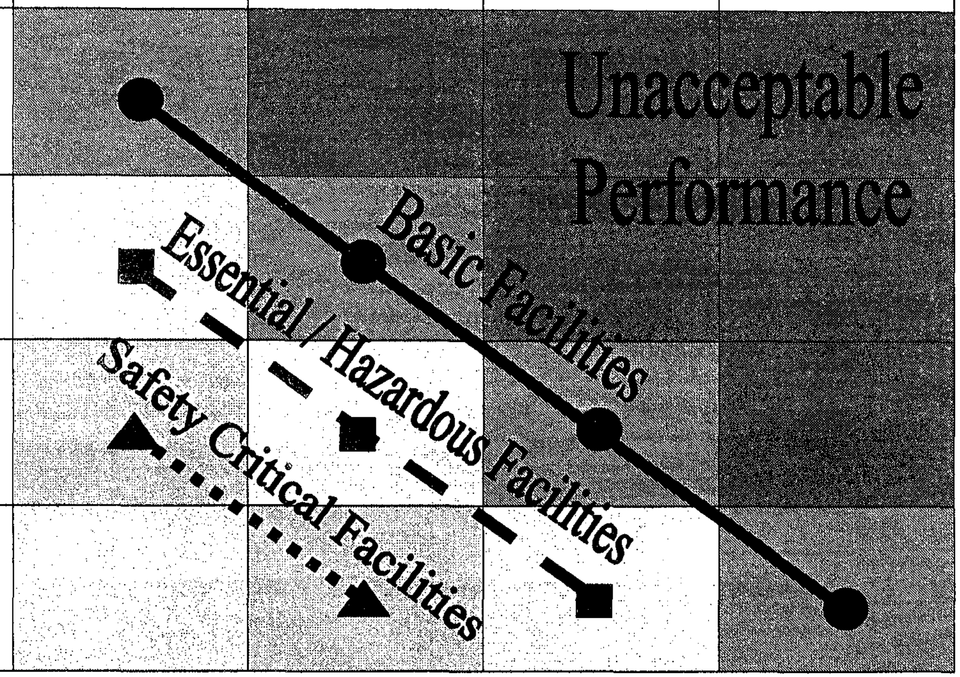


Fig2-2: Performance Objectives (SEAOC Vision 2000, 1995)

Table 2-2: Earthquake Classification (SEAOC Vision 2000, 1995, Bagchi, 2001)

Earthquake Classification	Recurrence Interval	Probability of Occurrence
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

In addition, Vision 2000 (1995) acceptance measures include engineering response parameters (e.g. drift, stress, plastic hinge rotation angle, acceleration, etc.) to be considered which are adequate for a particular performance objective such as drift limits, Table 2-3.

Table 2-3 : Vision 2000 Drift limits (PEER, 2008)

Limit State	Permissible Maximum Drift (%)	Permissible Permanent Drift (%)
Fully operational	0.2	negligible
Operational	0.5	negligible
Life Safe	1.5	0.5
Near Collapse	2.5	2.5

FEMA which is one of organizations working in the establishment of PBSD guidelines, published FEMA 273 (1997) providing a displacement based design approach. This document was followed by FEMA 356 (2000) giving an enhancement to the first-generation procedures of FEMA 273 and brought FEMA 273/274 (1997) to the pre-standard level (FEMA 445, 2006).

Furthermore, FEMA developed "ATC-55" project as guidelines for a better application of FEMA 356 (2000) and ATC-40, coefficient method and capacity-spectrum method respectively that usually provide different assessment for displacement demand for the same building. These guidelines represented FEMA 440 (2005).

PEER too brought forward its own second-generation of PBEE approach. Founded on FEMA and the ASCE pioneering methodologies, FEMA/ASCE 356 (2000) pre-standard, PEER essentially added two new features to PBEE: (1) *Damage analysis*. This is the clear probabilistic calculation of physical damage, for instance which bars have buckled, or which beams have spalling and so on; and (2) *Loss analysis*. This is the unambiguous, probabilistic calculation in order to assess the performance of the building in terms that are important for owners and stakeholders, terms such as economic loss, life loss, and loss of use on the other word in terms of dollars, deaths, and downtime.

2.5. TORSION

In the elastic range of responses, torsional motion results when a structural system's centres of rigidity do not coincide with its centres of mass. Structures with non-coincident centres of mass and rigidity are termed as asymmetric or torsionally unbalanced structures, and the torsional motion induced by symmetry or unbalance is commonly termed as natural torsion. Asymmetry may exist even in a nominally symmetric structure because of uncertainty in the evaluation of the centres of mass and stiffness, inaccuracy in the measurement of the dimensions of structural elements, or lack of precise data for material properties such as modulus of elasticity. Torsional vibrations may also be due to a ground rotational motion about vertical axis.

Torsional motion provoked by the earthquake has been reported as one of the main causes of damage in building structures, particularly in the recent earthquake events such as 1985 Mexico earthquake, the 1989 Loma Prieta earthquake, the 1994 Northridge earthquake, and the 1995 Kobe earthquake. This contributed to the development of the study of torsional response of buildings.

Elastic and inelastic torsional response of building models were widely studied in the past. Nevertheless, the results of these studies have not always been reliable; possibly due to the complexity of torsional behaviour. This resulted to extensively differing torsional provisions in different building codes (Humar and Kumar, 2004).

The effect of torsional ground motions during earthquake on buildings was first pointed out by Newmark (1969). He brought up that it must be torsion effects on buildings beyond those due to the absence of coincidence between the centers of stiffness and mass (Nathan, 1975) .

It was Newmark (1969) who demonstrated that torsional ground motions must happen during an earthquake, thus there must be torsional effects on buildings aside from those due to lack of coincidence between the centers of resistance and of mass. Then, he suggested a proposed spectrum for other displacements of a building caused by torsional input.

Newmark arrived at a proposed spectrum for the additional displacements of a building that would arise from torsional input.

He described some conclusions concerning the 'design eccentricity' which should be used to represent these effects in an otherwise symmetric building. This question has some applied importance. In this regards, Hart et al. (1975) investigated the register of the 1971 San Fernando earthquake; they concluded that torsional building response is mainly due to the torsional (or twisting) component of the ground motion.

As a result, a suitable 'design eccentricity' in codes of practice must be allowed. Mentioning the 5% of the maximum building dimension required by the Uniform Building Code (International Conference of Building Officials, 1967) and the recommendations of the Structural Engineers Association of California (1968), Newmark evaluates his finding in comparison with this recommendation.

Nevertheless, the commentary section of the latter document (p. 58) mentioned that "this is 'accidental' torsion". Similarly, the National Building Code of Canada has an analogous remark again evoking torsion arising from calculated and accidental lack of symmetry, without referring to torsional ground motion. Accordingly, it is questionable if any allowance has been intentionally made for this phenomenon in any of these codes.

Humar and Kumar (1998) have reported insufficient consideration for some of the parameters controlling the torsional response, particularly the torsional stiffness defined by the ratio of uncoupled torsional frequency to the uncoupled lateral frequency. Thus a clear provision in the building codes does not exist concerning the torsional stiffness or the frequency ratio. These authors suggest new torsion design provisions leading to some progress. The proposed provisions are easy to use and are not very different from the usual provisions of some of the standard codes. These proposals form the basis for the provisions in NBCC 2005.

In conclusion, it can be seen that a great deal of efforts has been put into establishing new, simplified, accurate and reliable methods in achieving PBSD concept. However, there is a lack of effort in bringing the existing simplified Equivalent Static load (ESL) method to the PBSD level. So that, the performance level of a building which is designed using ESL method of analysis, can be fairly narrowed down; and that level of performance can then be scaled up or down by establishing similar approaches, as an effort in filling such gap.

CHAPTER 3

3. METHODOLOGY

3.1 Introduction

In pursuing the objectives set for this research 3 sets of Reinforced Concrete (RC) buildings with shear wall SFRS are analyzed, designed and assessed. Heights of the buildings vary from low rise to high rise with a maximum value of 59.6 m that is within the limit of 60 m as specified in NBCC 2005 where ESL method can be used. Also, all other requirements for a building to be considered a regular building as defined in NBCC 2005 are satisfied. The buildings are then designed using the provisions of CSA-A23.3-04 Standard for reinforced concrete buildings. For evaluation of the seismic performance, the buildings are analyzed using dynamic time history and Response Spectrum Analysis (RSA) methods (using both Design spectrum and actual types of the spectrum).

3.2 NBCC 2005 and CSA-A23.3-04 implementation

NBCC acceptable solution requires all buildings to be designed for earthquake load (E) based on results from dynamic analysis; however, it allows the use of ESL method for regular buildings as defined in the code. The NBCC 2005 utilizes site-specific uniform hazard spectrum (UHS) corresponding to two percent probability of exceedance in fifty years, in other word a twenty five hundred years return periods (Humar and Mahgoub, 2003). The code defines the base shear as follows with the minimum and maximum boundaries outlined in Equation 3-1:

$$\left\{ \frac{S(2)M_vI}{R_oR_d}W \right\} \leq \left\{ V = \frac{S(T_a)M_vI}{R_oR_d}W \right\} \leq \left\{ \frac{2}{3} \times \frac{S(0.2)M_vI}{R_oR_d}W \right\} \tag{3-1}$$

In the above expression, V stands for base shear, $S(T_a)$ is the design spectral response acceleration in "g", T_a represents the fundamental period of vibration in terms of seconds, M_v factor reflects the higher modes effect, I stands for the importance factor of the building, R_o and R_d are over-strength and ductility factors respectively, and W is equal to the dead load (D) of the structure plus twenty five percent of the snow load (S).

To estimate structure's fundamental period of vibration T_a, the code offers an empirical formula for RC shear wall SFRS as:

$$T_a = 0.05(h_n)^{3/4} \tag{3-2}$$

where h_n stands for buildings height above the base; however, it allows use of larger values stated as other means of calculation, but limits it to twice the empirical value.

The code defines the design spectral response acceleration values $S(T)$ as a function of $S_a(T)$ which is the 5% damped spectral response acceleration in "g", acceleration based site coefficient F_a, and velocity based site coefficient F_v, as follow.

$$\begin{aligned} S(T) &= F_aS_a(0.2) \text{ for } T \leq 0.2\,\text{s} \\ &= F_vS_a(0.5) \text{ or } F_aS_a(0.2) \text{ the smallest of the two values for } T = 0.5 \\ &= F_vS_a(1.0) \text{ for } T = 1.0\,\text{s} \\ &= F_vS_a(2.0) \text{ for } T = 2.0\,\text{s} \\ &= F_vS_a(2.0)/2 \text{ for } T \geq 4.0\,\text{s} \end{aligned} \tag{3-3}$$

The base shear distribution over the height of the building is the same as in the previous code and is defined as:

$$F_x = (V - F_t)\frac{W_x h_x}{\sum_{i=1}^{n} W_i h_i} \qquad (3\text{-}4)$$

where F_x stands for the lateral force applied at level x, n reflects the total number of storeys, h_x and h_i are the heights above the base to levels x and i respectively, W is the storey weight. F_t is considered to be reflecting the higher modes effect and is exerted at the building's roof level and is defined as:

$$F_t = 0 \qquad \text{for} \quad T_a \le 0.7\text{ s}$$

$$F_t = 0.07 T_a V \le 0.25V \qquad \text{for} \quad T_a > 0.7\text{ s} \qquad (3\text{-}5)$$

Humar et al. (2003) concluded in their work, that buildings torsional sensitivity is a function of rotational to translational frequency ratio, and following that established a simplified method in determining such sensitivity. Based on their work, NBCC 2005 requires the building to be also examined for its torsional sensitivity where the ESL method is used.

The Code considers a building, with floors deemed as rigid diaphragms, to be torsionally sensitive if a ratio symbolized by B exceeds 1.7. Where, B is the maximum of all values of Bx in both orthogonal directions. Bx for each level x, and for each orthogonal independent direction would be calculated as $Bx = \delta$ max / δ ave. In this formula δmax stands for the maximum storey displacement at the extreme points of the structure at level x in the direction of the seismic load induced by the equivalent static forces and exerted at a distance equal to $\pm 0.1Dnx$ from the Center of Mass (CM) at each floor, and δave is the average of the displacements of the extreme points of the structure at level x generated by the above forces, where Dnx is the floors' dimension perpendicular to the direction of earthquake load at level x. If torsionally sensitive, then the code requires a dynamic analysis to be conducted.

On the design part, CSA-A23.3-04 (2004) requires maintaining ductility of shear walls in which the following restrictions should be satisfied; where the demand rotation should be smaller than or equal to that of capacity (Fig. 3-1).

$$\theta_{id} = \Delta f(R_d R_o - \gamma_w)/(h_w - l_w/2) \geq \; 0.004 \tag{3-6}$$

$$\theta_{ic} = (\varepsilon_{cu} l_w / 2 / c - 0.002) \leq \; 0.025 \tag{3-7}$$

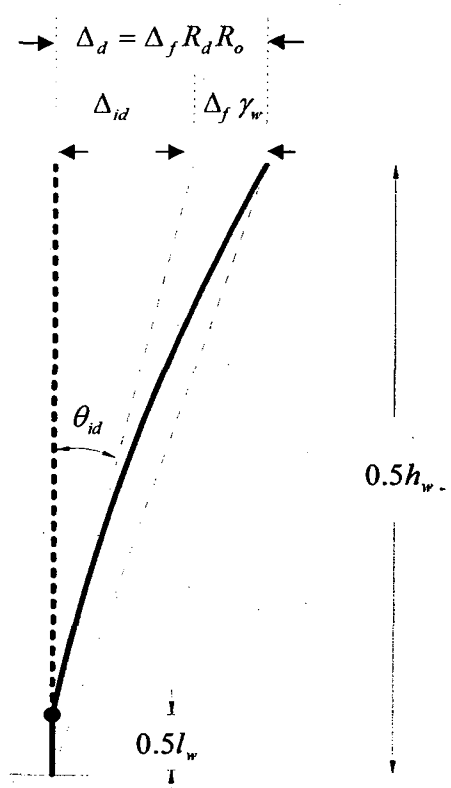


Fig. 3-1 : Inelastic rotation demand of shear walls

In addition, the Code requires the shear wall to resist the shear that corresponds to the development of plastic hinge at their base in which the design shear and shear resistance are calculated as follow

$$V = M_p / M_f \times V_f \tag{3-8}$$

$$V_r = \phi_c \beta \sqrt{f'_c} \quad b_w d_v + \phi_s A_v f_y d_v Cot\,(\theta) / s \tag{3-9}$$

3.3 Choice of computer programs used

Among the commonly used computer programs for structural analysis and design, including STAAD Pro (REI, 2008; later merged in Bentley corporation), SAP2000 (CSI, 2008), ETABS (CSI, 2008) and SAFE (CSI, 2008), the last two mentioned programs are used here and for their reliability and flexibility, their results are randomly checked against manual calculations. However, because of their incapability in nonlinear analysis of RC shear walls, they could not be used for evaluation of the designed buildings.

There are many programs that may provide inelastic structural analysis option, yet, they may vary in features like 2D or 3D analysis capability, and computer time consumptions. To evaluate buildings performance as realistic and time wise efficient as possible, different softwares were explored and PERFORM 3D (CSI COMPUTERS & STRUCTURES, INC.) is found to be the best and foremost match among a series of programs for pursuing the objectives of this research. There are programs like ANSYS that are general purpose nonlinear programs; however, they may not be practical in featuring building's structural elements, particularly in a large scale; there also are programs like IDARC 2D that just provide two dimensional analysis that will not fit into the objectives of this work. Also, there could be other programs like CANNY 2004 developed by Kangning Li; its trial version was tried but did not prove reasonable results.

In order to validate the results of PERFORM 3D and ETABS, at first, a 2D 12 storey shear wall sample adopted from Humar and Bagchi (2004) is employed. The plan view and elevation are shown in Fig.3-2, and the ETABS and PERFORM 3D geometrical models are shown in Fig. 3-3. The shear walls are modeled in both Programs and the results extracted from the dynamic analyses and pushover analyses are compared with those given in Humar and Bagchi (2004) that are modeled and analyzed using DRAIN program. The comparative results produced in the Fig. 3-4 and Fig. 3-5 are based on the nonlinear dynamic time history analysis, except for ETABS which has only linear dynamic analyses capability for shear walls.

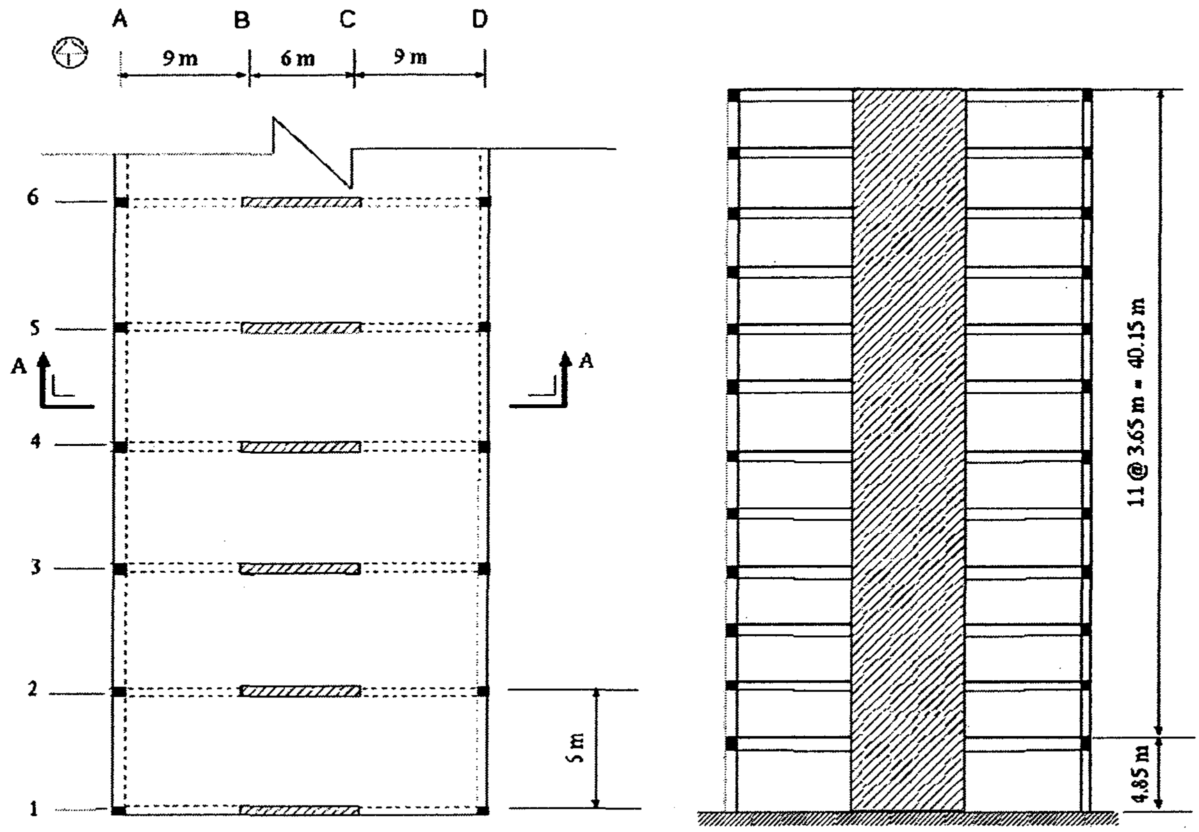


Fig. 3-2: Layout of the building with shear wall (a) plan, and (b) elevation

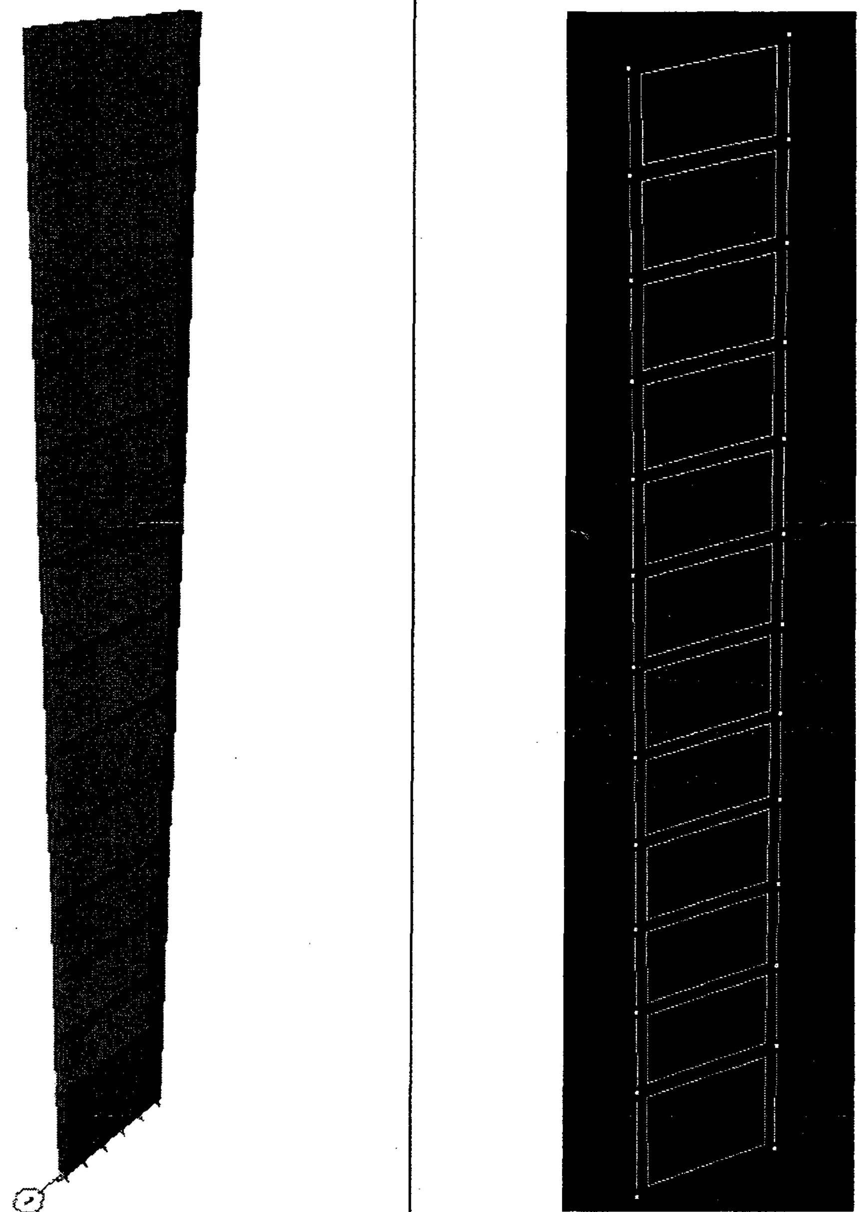


Fig. 3-3: 12 Storey shear wall model (a) ETABS, (b) PERFORM 3D

Fig. 3-4 proves that PERFORM 3D pushover analysis results in similar out come as that in Humar and Bagchi (2004). It also can be seen in Fig. 3-5 that the inter-storey drifts are in a good agreement with the results presented in Humar and Bagchi (2004).

Fig 3-5 shows that the displacements resulting from all programs, both linear and nonlinear analyses, are in good agreement with each other. These results are also in a good agreement with the Equal Displacement Rule established by Velesos and Newmark (1960); this rule states that "displacement of a structure due to a given ground motion is basically the same for both elastic and inelastic structural behavior". Also, NBCC is using the same rule in its provisions.

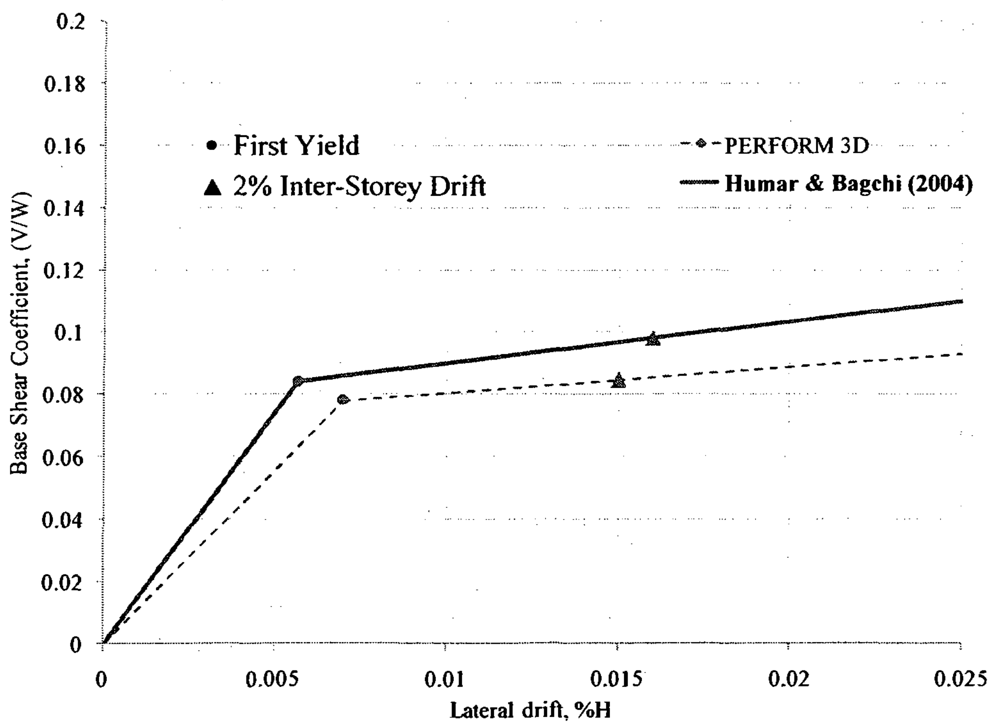


Fig. 3-4: First yield and 2% drift in 12 storey wall model extracted from PERFORM 3D that is Close to findings of Humar and Bagchi (2004)

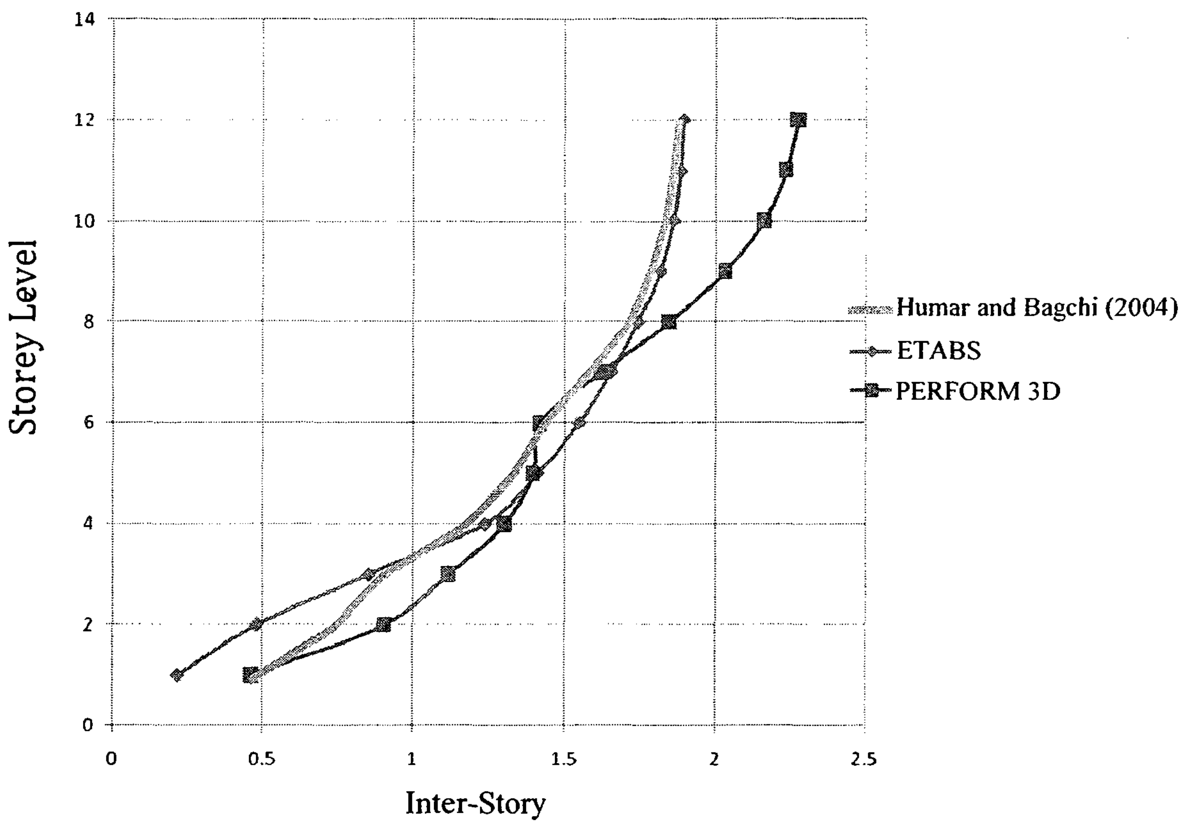


Fig. 3-5: 12 Storey Wall verification of PERFORM 3D Inter-Story Drift

It should also be noted that for designing slab, CSA-A23.3-04 (2004) considers three types of RC strips; design strip, column strip and a strip with a width equal to that of the column plus 1.5 times the thickness of the slab on each side of the column; here thereafter, we call it Core Strip.

Although, PERFORM-3D is capable of carrying nonlinear dynamic analysis in general, such analysis does not include the flooring system. Therefore, in order to simulate the possible nonlinear behavior for slabs, the slab has been replaced by equivalent strips as discussed and represented as beams in Perform 3D models. The detail of Core Strips are presented in Fig.4-12 to Fig.4-14 series.

The four, eight and sixteen story building models constructed in ETABS and PERFORM 3D are shown in Fig.3-6 to Fig.3-7. As shown in these figures, ETABS models incorporate the wall, flat slab and column components, while the PERFORM 3D models includes columns, walls and equivalent beams/strips.

In a computer program, linear analysis is typically done through specifying the properties of an element by assigning a cross section and an elastic material to the element. Generally, the element properties are then completely defined. However, it is more complex for nonlinear analysis, because more properties are required. Linear analysis requires just stiffness properties, while nonlinear analysis needs both stiffness and strength properties.

Walls in ETABS are modeled using shell element, and in PERFORM 3D they are modeled using fiber elements.

In ETABS, the Shell element is a three/ four node formulation that combines membrane and plate- bending behaviour; in this work the four joint homogenous shell formulations is used for modeling of the wall.

The membrane behaviour uses an isoperimetric formulation that includes translational in plane stiffness components and a rotational stiffness component in the direction normal to the plane of the element, where in-plane displacements are quadratic.

The homogenous plate-bending behaviour includes two-way, out-of-plane, plate rotational stiffness components and a translational stiffness component in the direction normal to the plane of the element.

A thick-plate formulation which includes the effects of transverse shearing deformation is used in the modeling, where out-of-plane displacements are cubic.

In PERFORM 3D, The important forces in a shear wall are considered as shear force and axial-bending action along the vertical direction of the wall. The transverse direction is assumed to be a secondary direction. For the primary axial-bending behaviour a fiber wall cross section must be defined. For shear the user must specify a shear material and also an effective wall thickness. A shear wall element can also bend out of plane. PERFORM 3D assumes that out-of-plane bending to be elastic and a secondary mode of behaviour. For transverse behavior, PERFORM 3D requires an effective thickness and an elastic modulus to be specified. Also for out-of-plane bending an effective plate bending thickness and a modulus must be specified.

The back bone relationship used for reinforcement is a bilinear relationship, and a trainer relationship with strength loss is used for the concrete.

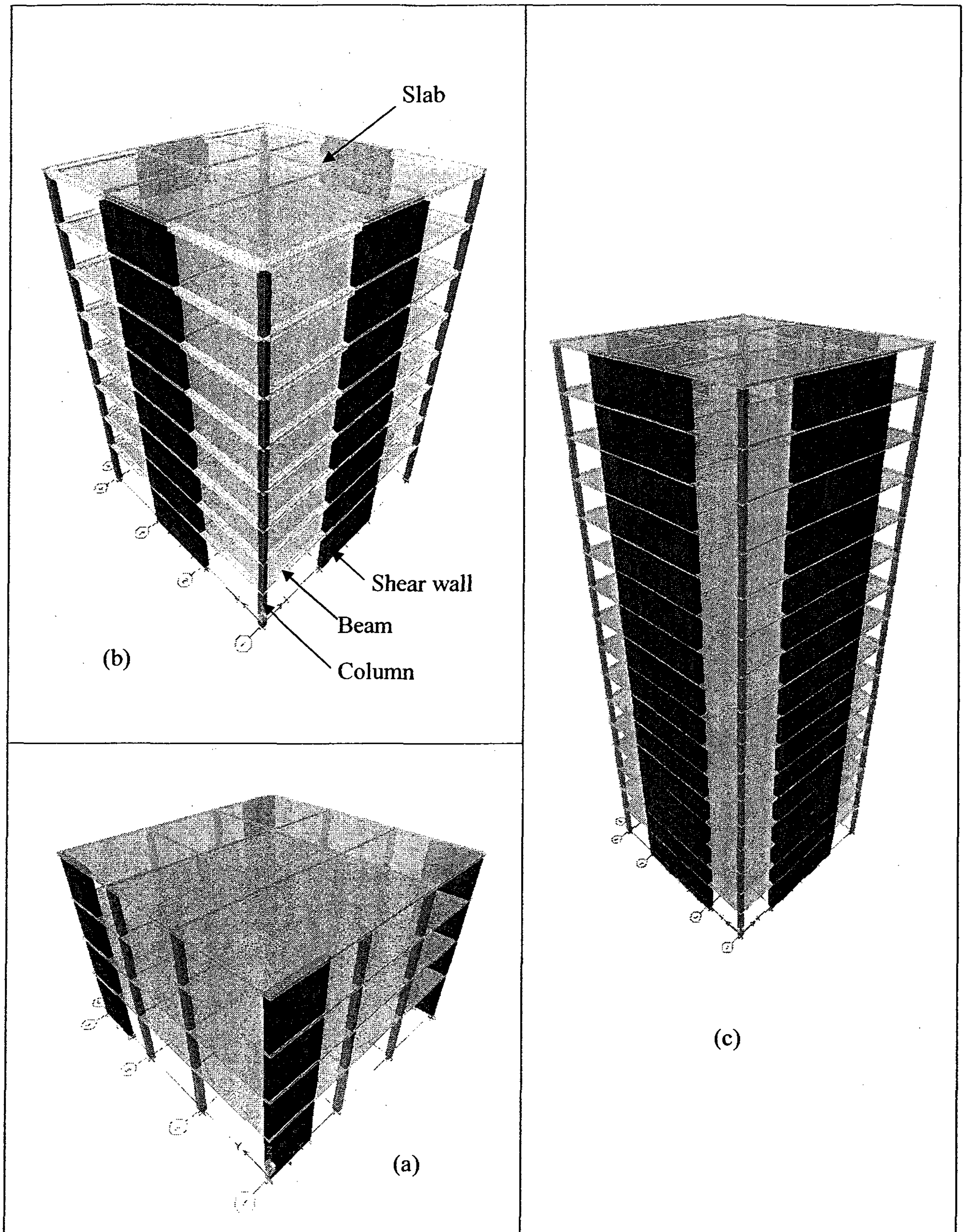
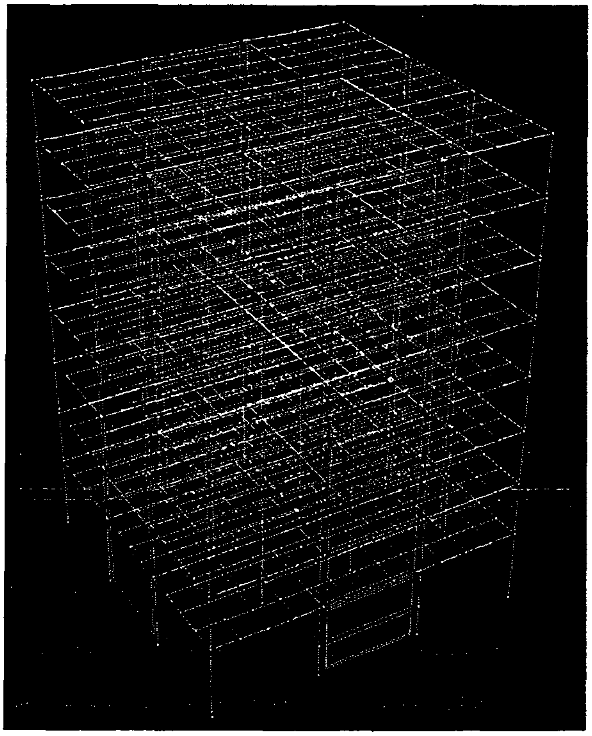
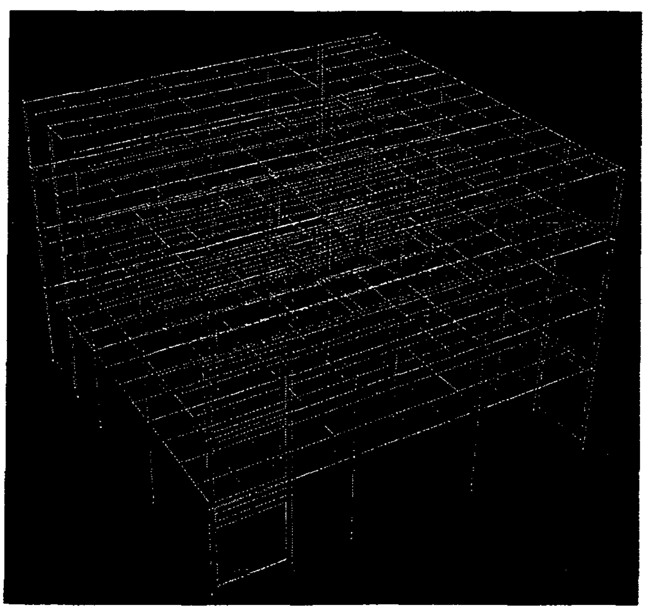


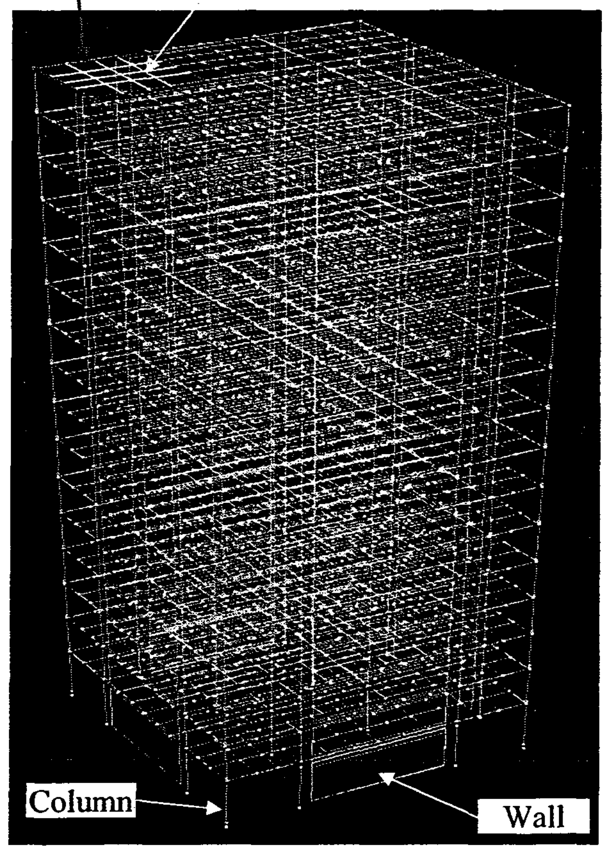
Fig. 3-6: Buildings' geometric modeling in ETABS (a) 4 Storey (b) 8 Storey (c) 16 Storey



(a)



(a)



(c)

Fig. 3-7: Buildings' geometric modeling in PERFORM 3D (a) 4 Storey (b) 8 Storey (c) 16 Storey

3.4 Seismic excitation, and selection of earthquake records

In a dynamic analysis, the most suitable ground motions would be earthquake records from the region that the building would be built in. Since, such records do not exist for Vancouver, a series of records that can provide the most similar outcomes should be used.

Characteristics of earthquake records vary very much from record to record. Key features of these records including the intensity, frequency content, and duration of strong shaking are functions of different factors such as earthquake's magnitude, local site's condition, and the epicentral distance. Difference in earthquake features can lead to essential differences in building structures' responses. Frequency content is of the fundamental factors that affects the structural responses. Peak ground acceleration to peak ground velocity ratio (A/V) is a simplified way of estimating frequency content of an earthquake record (Heidebrecht and Lu, 1988). Statistical studies have proven that records with high A/V values have high frequency content and are typically associated with moderate to strong earthquakes at small epicentral distances, and low A/V values normally have low frequency content and represent large earthquakes at large epicentral distances.

Naumoski (Naumoski et. al., 1988) presented three ensembles of recorded accelerograms with different A/V ratios: high A/V ratios (A/V > 1.2), intermediate A/V (0.8 < A/V <1.2), and low A/V ratios (A/V < 0.8), where A is in g, and V is in m/s. Each group consists of 15 accelerograms. All the selected accelerograms are recorded on rock or stiff soil sites.

Seismic zoning map of NBCC 1995 provides typical A/V ratios for different zones. This ratio for Vancouver area is about 1.0. The second (intermediate) group of accelerograms provided by Naumoski et. al. (1988) as mentioned above, have A/V varying from 0.8 to 1.2 (average of about 1 for the group) that fits into the Vancouver area; these records are presented in Table 3-1 and used in this research.

On the other hand, NBCC 2005 provides the 5% damped Acceleration Response Spectrum (ARS) values for the reference ground conditions that correspond to Uniform Hazard Spectrum with 2500 years return period (UHS-2500) as a representative of the earthquake intensity level for the areas and requires the design spectral acceleration values to be calculated based on that.

There are different methods for scaling the intensity of an earthquake record intensity to a required level, which include ordinate, partial area, and full area methods. In doing so, spectral analysis is carried out for each ground motion record, and the actual response spectrum is scaled up or down to match Vancouver's design spectrum.

Ordinate Method (OM) is based on building's fundamental period of vibration. In that case, the ground motion time history are scaled up or down by multiplying them by a ratio equal to the design spectral acceleration, S_{a1} divided by the actual spectral acceleration, S_{a2} the scaling factor (Fig. 3-8). On the other hand, in the Partial Area Method (PAM), A_2 which is the area under the actual ARS between the second period "T2" and 1.2 times the fundamental period "T1" is scaled to equal the area under the design spectral acceleration curve "A_1" between the same period range, then all values of the actual acceleration response spectrum are scaled by using A_1/A_2 as the scaling factor.

Full Area Method (FAM) is similar to PAM, but A_1 and A_2 are areas between the minimum and maximum period range, which is taken as 0.01 here.

The scaled records are used to excite the buildings along their principle axes, similar to the response spectrum, and ESL method analyses.

Table 3-1: Description and peak ground motion parameters for "Intermediate A/V Records" [0.8<A/V<1.2] (Naumoski et. al., 1988)

Rec. No.	Earthquake	Date	Magn.	Site / Duration (Sec.)	Epic. Dist. (km)	Max. Acc. A(g)	Max. Vel. V(m/s)	A/V	Soil Cond.
1	Imperial Valley California	May 18 1940	6.6	El Centro / 53.74	8	0.348	0.334	1.04	Stiff Soil
2	Kern County California	July 21 1952	7.6	Taft Lincoln School Tunnel / 54.4	56	0.179	0.177	1.01	Rock
3	Kern County California	July 21 1952	7.6	Taft Lincoln School Tunnel / 54.38	56	0.156	0.157	0.99	Rock
4	Borrego Mtn. California	April 8 1968	6.5	San Onofre SCE Power Plant / 25.28	122	0.046	0.042	1.10	Stiff Soil
5	Borrego Mtn. California	April 8 1968	6.5	San Onofre SCE Power Plant / 45.1	122	0.041	0.037	1.11	Stiff Soil
6	San Fernando California	Feb. 9 1971	6.4	3838 Lankershim Blvd., L.A. / 65.18	24	0.150	0.149	1.01	Rock
7	San Fernando California	Feb. 9 1971	6.4	Hollywood Storage P.E. Lot, L.A. / 43.62	35	0.211	0.211	1.00	Stiff Soil
8	San Fernando California	Feb. 9 1971	6.4	3407 6th Street, L.A. / 53.74	39	0.165	0.166	0.99	Stiff Soil
9	San Fernando California	Feb. 9 1971	6.4	Griffith Park Observatory, L.A. / 43	31	0.180	0.205	0.88	Rock
10	San Fernando California	Feb. 9 1971	6.4	234 Figueroa St., L.A. / 47.08	41	0.199	0.167	1.19	Stiff Soil
11	Near East Coast of Honshu	Nov. 16 1974	6.1	Kashima Harbor Works / 30	38	0.070	0.072	0.97	Stiff Soil
12	Near East Coast of Honshu	Aug. 2 1971	7.0	Kushiro Central Wharf / 60	196	0.078	0.068	1.15	Stiff Soil
13	Monte Negro Yugoslavia	Apr. 15 1979	7.0	Albatros Hotel, Ulcinj / 40.4	17	0.171	0.194	0.88	Rock
14	Mexico Earthq.	Sept. 19 1985	8.1	El Suchil, Guerrero Array / 120.02	230	0.105	0.116	0.91	Rock
15	Mexico Earthq.	Sept. 19 1985	8.1	La Villita, Guerrero Array / 128.04	44	0.123	0.105	1.17	Rock

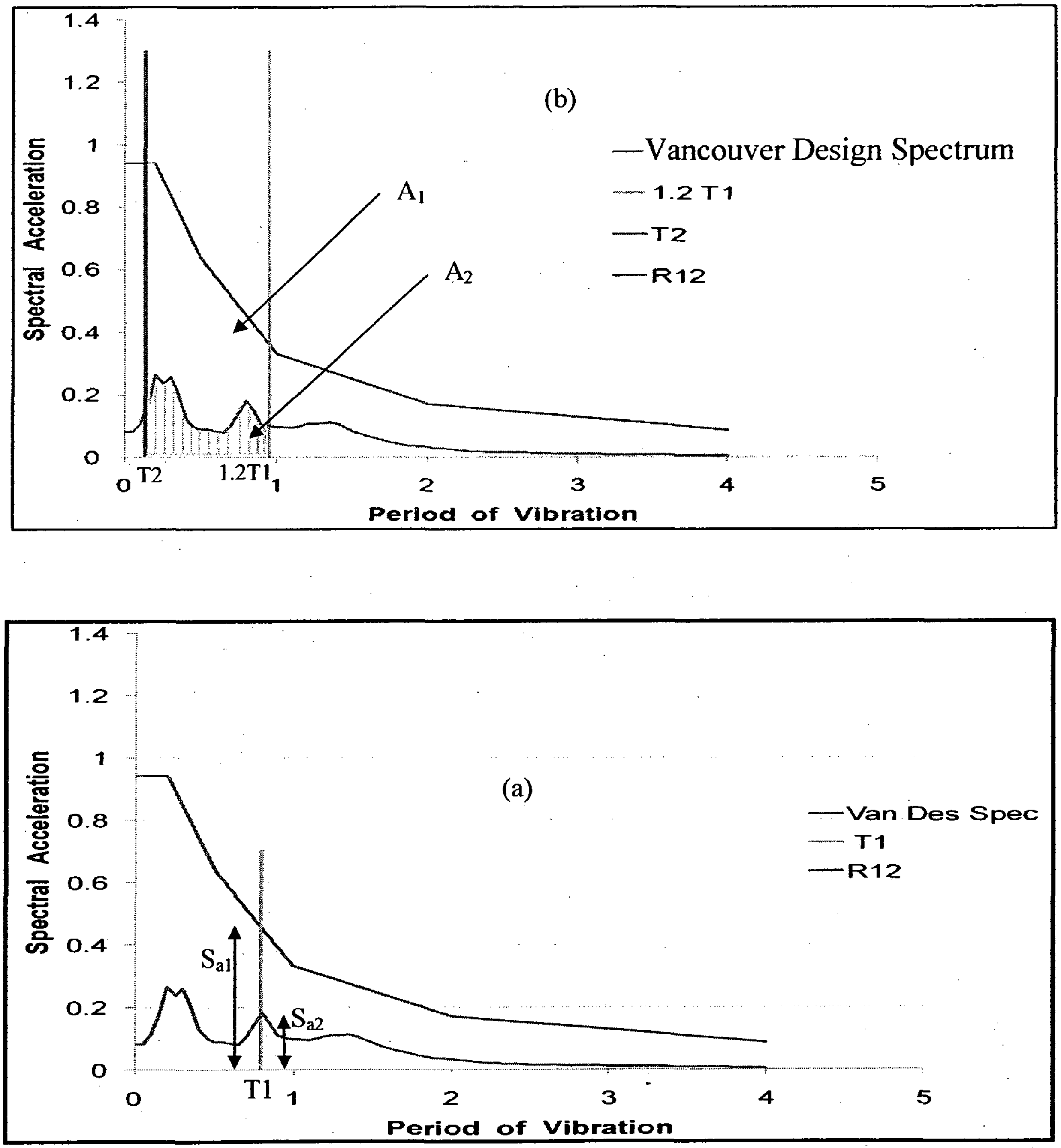


Fig. 3-8: Record Scaling Methods; (a) Ordinate Method, (b) Partial Area Method

CHAPTER 4

4. BUILDINGS DESIGN

To achieve the objectives set earlier, three Reinforced concrete (RC) Buildings with shear wall Seismic Force Resisting System (SFRS) are configured, modeled, analyzed and designed, after which the performance evaluation of the buildings are carried out.

Four, eight, and sixteen storey RC buildings with plan views, illustrated in Fig.4-5 to Fig.4-7 series, that fall into the definition of regular symmetric buildings of NBCC 2005 are analyzed and designed, regarding which a summary of the design procedure is presented in this chapter. It also should be noted that the buildings are of heights equal to 15.8, 30.4, and 59.6m respectively, which are within the limit of Equivalent Static Load (ESL) provided in NBCC 2005, as well as varying levels of the domination of flexural behavior and shear. Moreover, sensitivity of buildings' performance against torsional effect is studied here by varying the level of mass eccentricity within a range of 0 to 10% of building width as recommended in NBCC 2005 for accidental torsion.

In a region like Vancouver, not only buildings with fundamental frequency similar to that of the earthquakes are hit by the ground motions, but all buildings with all frequencies are hit with the same earthquakes. Therefore all of these buildings need to satisfy similar performance objectives as buildings that are in tune with the ground motion (A/V =1). This is the same approach used in NBCC. The code does not require the spectral acceleration and relevant parameters (for buildings with A/V = 1 frequency content) to be applied to buildings with other frequencies, but gives an spectrum from which the spectral acceleration for each period can be extracted.

In another word, studying buildings with different periods could, for instance, result in a response spectrum like solution that can be used for buildings with different periods.

4.1 Buildings' description

Three 3 bays by 3 bays 4, 8, and 16 storey office buildings with flat slab (0.25 m overhang) are analyzed and designed. The 1st floors' heights are 4.85m and the height of all other floors above the first floor are 3.65m. Buildings are located in Vancouver representing high seismic activity in Canada, and founded on class "C" soil.

The fundamental periods of the buildings, calculated using the empirical formula of the code, are found to be 0.3962, 0.6473, and 1.0725 as given in $T_a = 0.05(h_n)^{3/4}$ (Equation (3-2)) for 4, 8, and 16 storey buildings, respectively. On the other hand, the code requires that the fundamental period calculated from the model analysis does not exceed twice that obtained using the empirical equation. For the four and eight storey buildings, the fundamental period computed using ETABS are larger than twice that by the empirical formula of the code. Therefore, the values of the fundamental periods are revised to 0.7925, 1.2947, and 2.0178 respectively.

The material properties include concrete with an unconfined compressive strength f_c' equal to 30 MPa; concrete initial modulus of elasticity E_c is considered equal to 26,600 MPa and it has a normal density of 24 kN/m^3. The reinforcing steel is weldable and has a tensile specified yield strength f_y of 400 MPa.

The design live load (L) is equal to 2.4 kN/m^2 for all floors except for the first storey that is 4.8 kN/m^2. The snow load is 2.3 kN/m^2. Exterior walls dead load are 0.85 kN/m2, for partition on floors 1 kPa, 0.5 kPa for ceiling and mechanical services on all floors, and 0.5 kPa for roofing. Critical loads combinations are 1.25D + 1.5L + 0.5S and D + E + 0.5L + 0.25S.

The four and sixteen storey buildings have no beam element and their floors are of the flat plate type; however the eight storey building configuration for just flat plate resulted in large punching shear in the corner columns and proved the need of perimeter beam element presented in Fig.4-12 with the portion of slab as the effective flange width.

Premier beams in the 8^{th} storey building are designed to allow for the formation of plastic hinges at the end of beams before the columns, so that the strong column-weak beam requirement set by the standard is satisfied.

The stiffness of the members is based on the CSA-A23.3-04 (2004) requirements for seismic resisting buildings and is averaged over every few floors. For instance, the eight storey walls, sections view of which are shown in Fig. 4-8, are modeled with a flexural rigidity of $0.68EI_g$ for stories one to four, and $0.64EI_g$ for stories 5 to 8. These values of flexural rigidity are used in the static and response spectrum analyses of the building models for calculating the design moments and shear in the structural elements. However, for the detailed structural analysis involving nonlinear dynamic analysis, the effective flexural rigidities are calculated by Perform 3D using the fibre models as described earlier.

4.1.1 Structural Analysis

The analysis of structures are carried out based on dead, live, and snow load as mentioned earlier, and the earthquake load as discussed below. For ductile shear walls, R_d= 3.5 and R_o= 1.6, are considered as recommended in the code. The seismic load is assumed to be unidirectional and along one of the principal axes of the buildings.

Located in Vancouver, the buildings are assumed to be founded on a class "C" soil, the acceleration-based coefficient F_a and the velocity based coefficient F_v are both equal to 1.0, and the higher mode factor varies from one for the four storey, 1.044 for the eight storey, and 1.2 for the 16 storey buildings. The 5% damped spectral response acceleration, $S_a(T)$, and design spectral response acceleration values, $S(T)$, for Vancouver are as shown in Table 4-1 and Fig.4-1.

Table 4-1: Acceleration Response spectrum

	$T \leq 0.2$	$T = 0.5$	$T = 1$	$T = 2$	$T \geq 4$
Sa(T) , S(T)	0.94	0.64	0.33	0.17	0.085

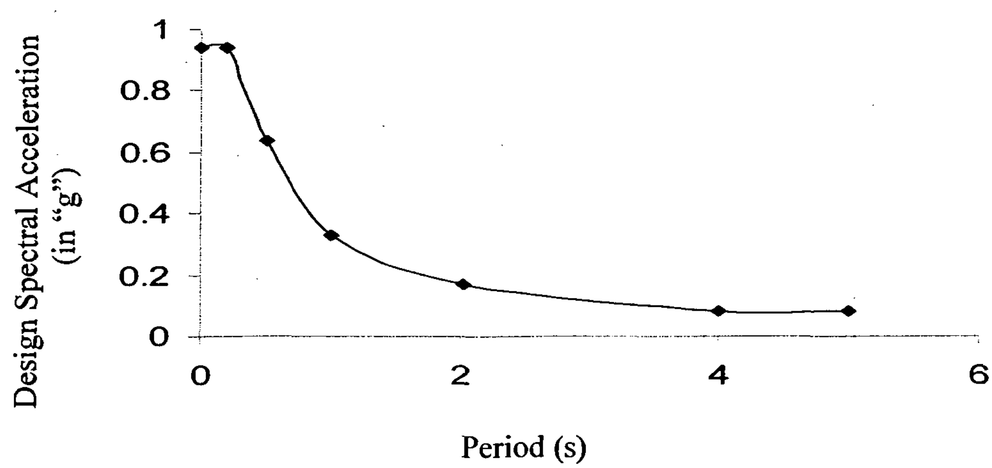


Fig. 4-1: Design spectral response acceleration

Using Equation 3-1 of the ESL method for the 8 storey building results in design base shear equal to V = 2494 kN, while the minimum and maximum limits imposed by the code are Vmin = 1145 kN and Vmax = 4221 kN

Torsional sensitivity as described in section 3.2 is determined, and the "B" parameter defined by the code equals to 1.11 that is smaller than the limit of 1.7 set by the code; therefore the building is not a torsionally sensitive building, and dynamic analysis is not a necessity; this is also true for four and sixteen storey buildings.

However response spectrum linear dynamic analyses for all the buildings are carried out; in these processes, the first 12 modes are taken into account. For instance, the 8 storey building results in cumulative modal participation mass ratios of all greater than 97.8% that is bigger than the minimum 90% required by the code. In pursuing the results from dynamic analysis, the elastic base shears from linear spectral dynamic analysis is multiplied by $I_E/(R_d\ R_o)$ to obtain the design base shear that code requires to be equal or greater than 80% of that for ESL method.

Then to obtain the design values, the forces and deflections are multiplied by V_d / V_e. Also, to obtain the realistic deformation values, the earlier design values are scaled by the factor $(R_d\ R_o)$ / I_E.

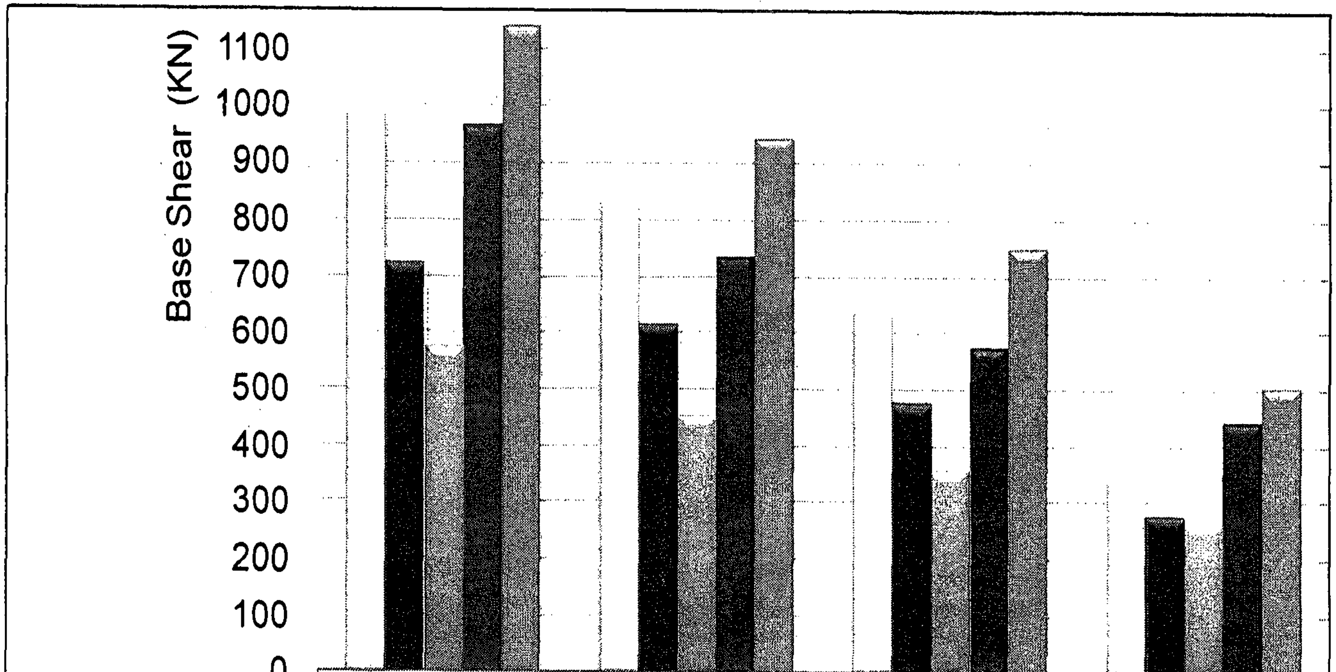
80% of the base shear "Vd" resulting from the ESL method, as given by Equation (3-1), is equal to 1995 kN; and design base shear for linear dynamic analysis equals to 2325 kN that is greater than 80% of the design base shear from ESL method, therefore the scaling factor would be equal to 0.1786.

The maximum base shear resulting from RSA, for the critical wall, is equal to 1126 kN as compared to that of revised ESL method which is 1254 kN. Similarly, the maximum inter-storey drift is equal to 0.22% that is slightly smaller than 0.21% from the response spectrum method, and both are well below the code's limit

To maintain shear wall's ductility the standard requires the inelastic rotational demand at hinge to be smaller than that of capacity; for the 8 storey ESL method θ_{id} , Equation (3-6), equals to 0.0062 which is smaller than $\theta_{ic} = 0.0071$, Equation (3-7), and satisfies the code requirement; this requirement is satisfied for the other two buildings too.

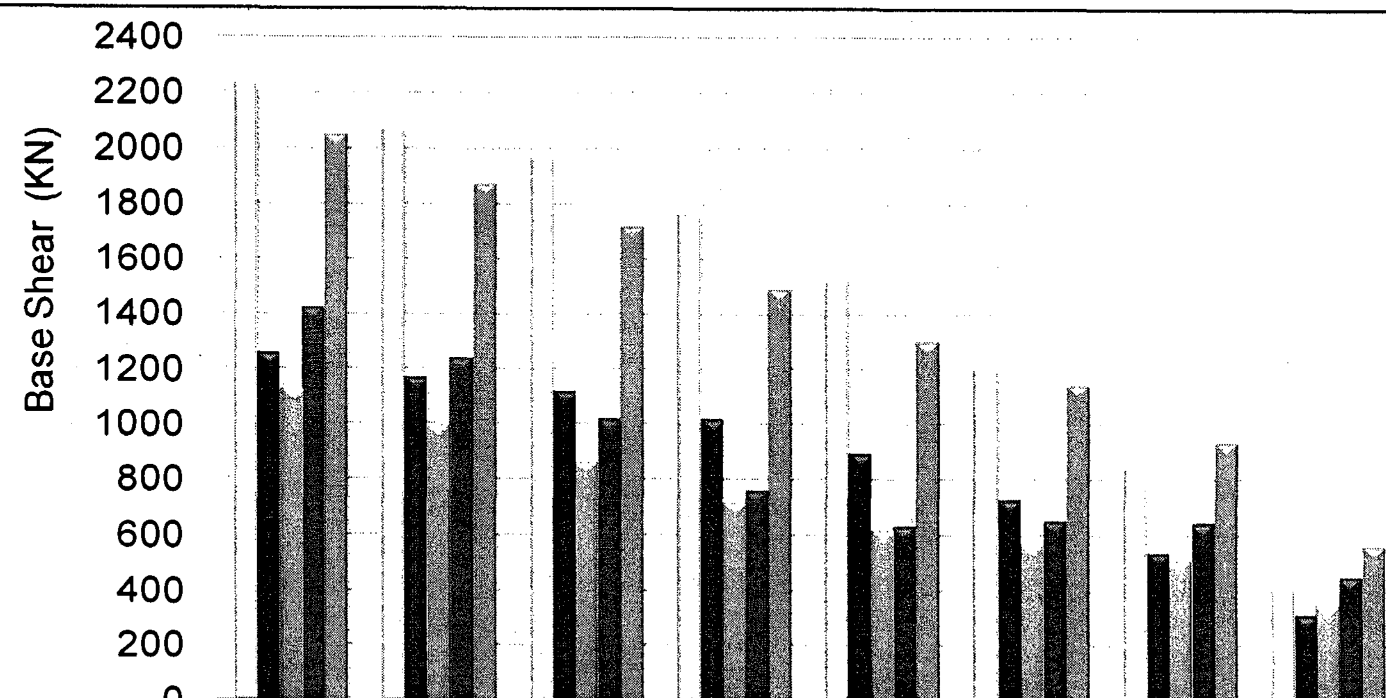
The Code also requires the modified factored shear to be smaller than the capacity as defined in Equation 3-8 and Equation 3-9 the demand value for the 8 storey building on the critical shear wall is 2285KN that is smaller than the capacity of 2924 kN; this requirement is also satisfied for the other two buildings.

For comparison, base shear demand and its distributon along the height of the critical wall for each building and for different analysis methods are shown in Fig.4-2 to Fig. 4-4 series. It can be seen that the base shear resulting from ESL methode using fundamental period is well comparable with that of actual RSA using OM and PAM of scaling.



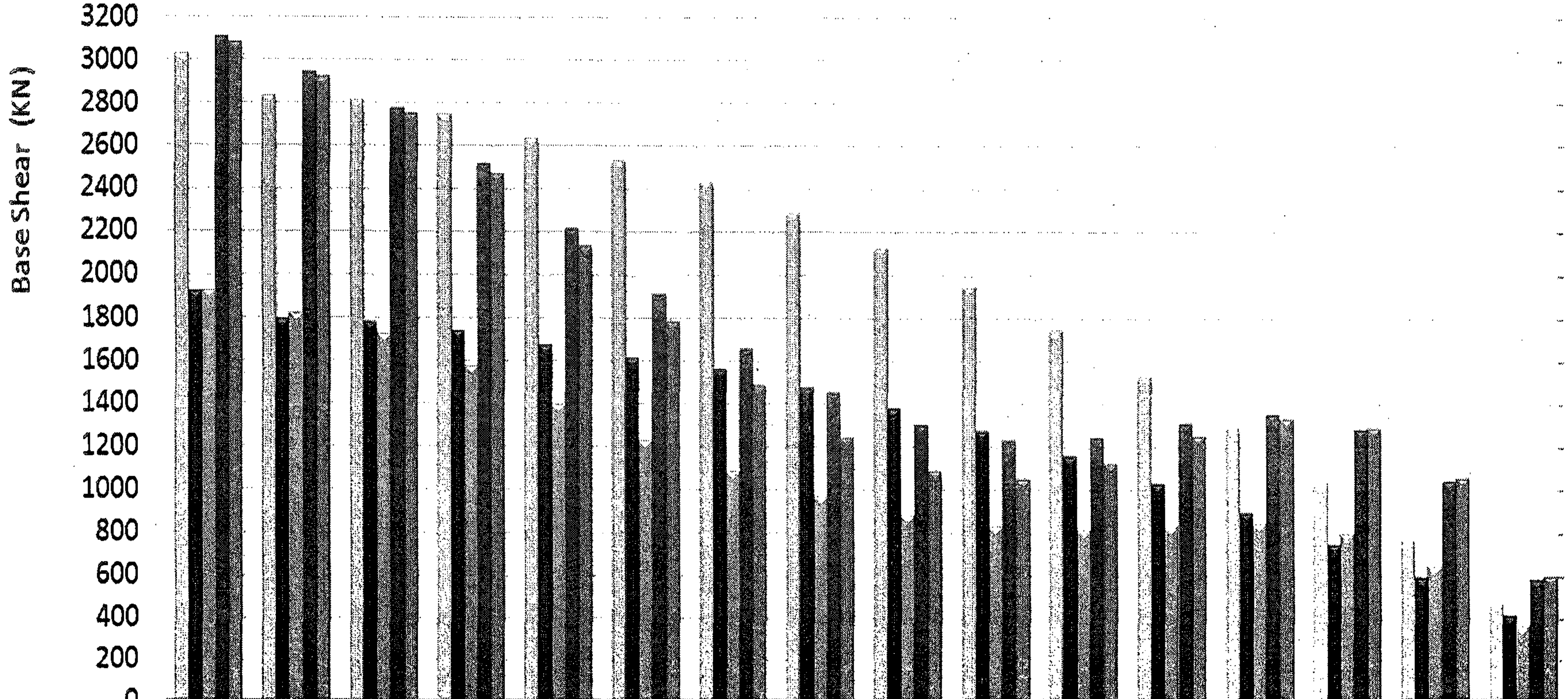
Storey Level	1	2	3	4
PAM Scaled Spec	1137.99	940.17	749.45	503.45
OM Scaled Spec	960.89	733.86	573.35	445.67
Des Resp Spec	575.3	455.6	359.8	263.3
Revised 1st Period	719.09	611.44	475.86	274.18
Emperical 1st Period	980.71	828.28	632.11	341.82

Fig. 4-2: Story Shear Distribution of Critical Wall over the Height; 4 Storey Building



Storey Level	1	2	3	4	5	6	7	8
PAM Scaled Spec	2043.	1870.	1708.	1485.	1299.	1137.	932.3	562.1
OM Scaled Spec	1417.	1236.	1012.	758.8	629.8	650.1	647.9	449.2
Des Resp Spec	1126.	991.1	860.3	715.5	618.1	565.8	508.7	344.4
Revised 1st Period	1254.	1168.	1117.	1013.	887.4	725.9	535.6	312.2
Emperical 1st Period	2222	2064.	1962	1761.	1516.	1205	837.6	407.7

Fig. 4-3: Story Shear Distribution of Critical Wall over the Height, 8 Storey Building



	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
PAM Scaled Spec	3082.	2924.	2746.	2467.	2132.	1787.	1489.	1245.	1092.	1054.	1129.	1253.	1337.	1296	1064.	601.6
OM Scaled Spec	3107.	2942	2776.	2518.	2212	1910.	1662.	1454.	1310.	1237.	1248.	1311.	1353.	1289.	1051	591.7
Des Resp Spec	1923.	1818.	1721.	1570.	1392.	1222.	1085.	969.0	881.6	825.9	812.3	829.5	839.4	791.6	641.9	360.3
Revised 1st Period	1916.	1791.	1779.	1741.	1674.	1615.	1559.	1473.	1380.	1274.	1160.	1035.	898.4	749.7	594.1	418.0
Emperica 1st Period	3029.	2836.	2812.	2746.	2634.	2532.	2431.	2285.	2125.	1943.	1746.	1530.	1294.	1039.	768.3	469.7

Fig. 4-4: Story Shear Distribution of Critical Wall over the Height, 16 Storey Building

The final results of design for the buildings are presented in graphical form in the following drawings, Fig.4-5 to Fig.4-14 series. All buildings consist of 3 bays in either orthogonal directions, and the width of the buildings in both directions is the same and equal to 21m from center to center of columns as shown in Fig.4-5 to Fig.4-7. It should be noted that the building configurations chosen here are idealized structures, the placement of the shear walls may in some cases not represent a common application where the architectural correlations would be important for the placement of these walls.

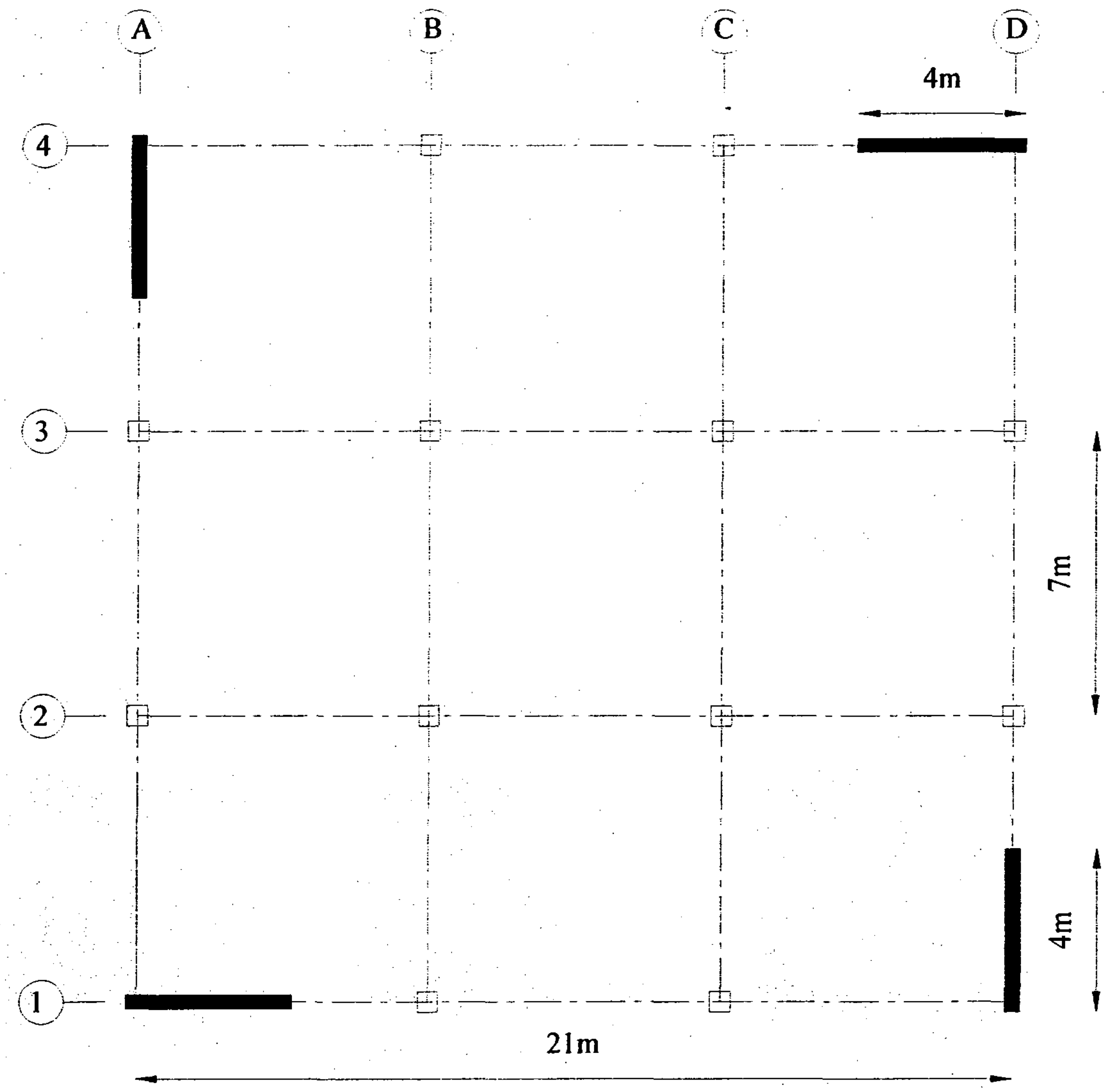


Fig. 4-5: Plan view; 4 Storey Building

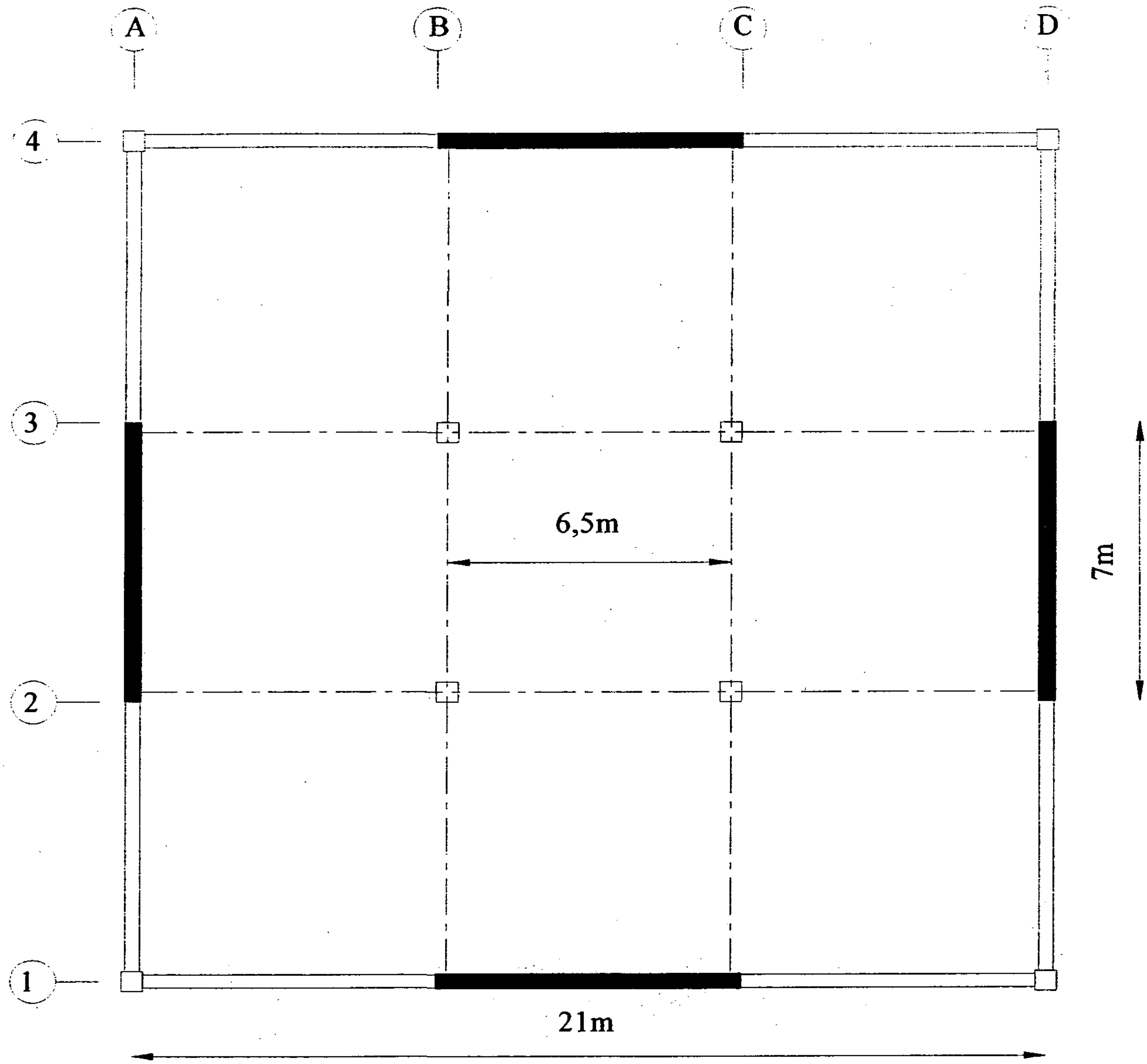


Fig. 4-6: Plan view; 8 Storey Building

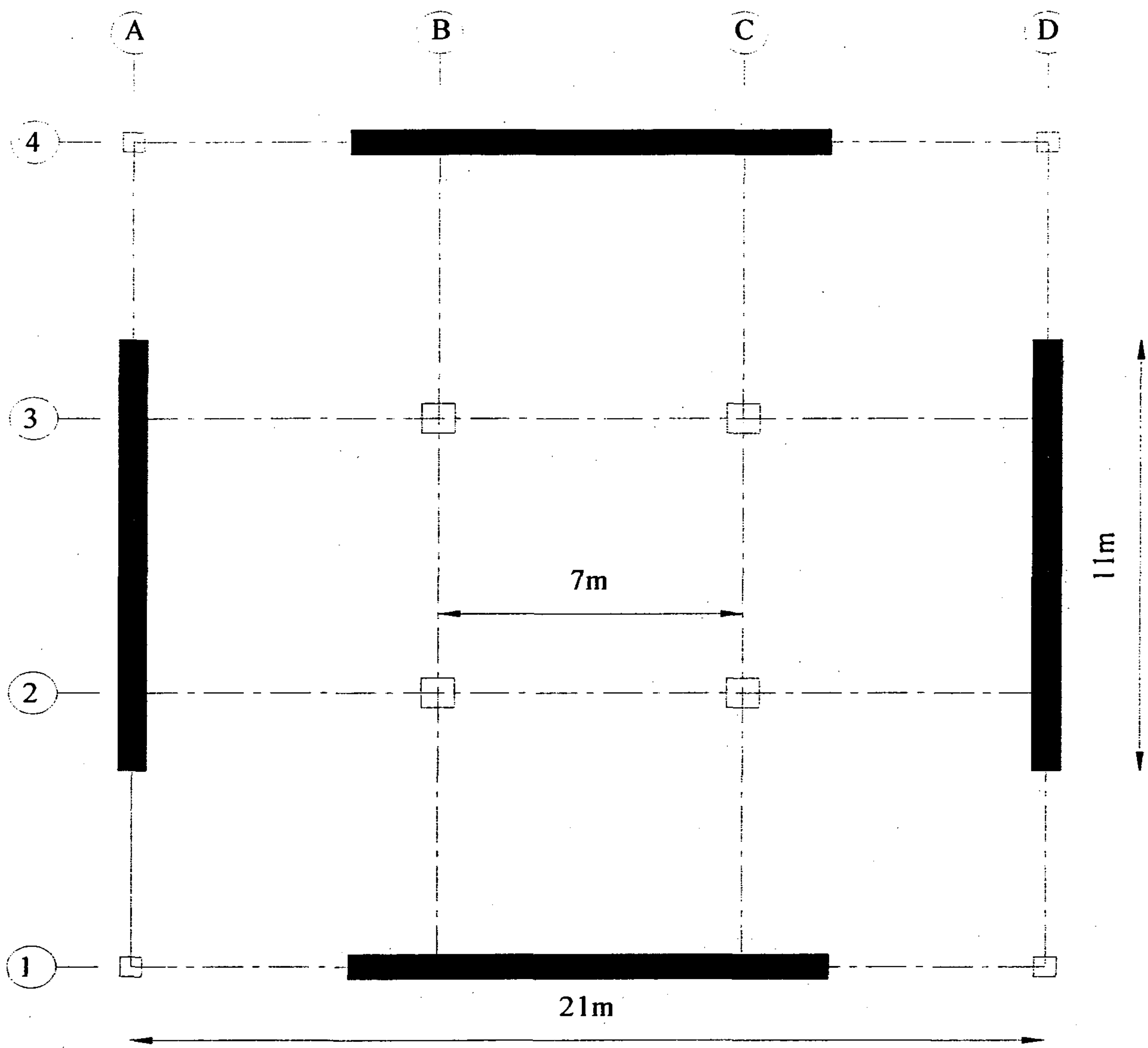


Fig. 4-7: Plan view; 16 Storey Building

The 4 storey building consists of 4m wide walls, Fig.4-8, with a constant thickness of 35 cm along the height to the roof of the building; however the reinforcement varies along the height, the concentrated bars for the first 2 stories consists of 8-25M and it turns to 8-15M for the 2^{nd} set of top stories. The concentrated confining bars near the edges of the walls vary along the height too; it includes closed ties at 100 mm plus an additional cross-tie in each direction for the first two stories, and similar arrangement for the 2 top stories but with spacing equal to 150mm; horizontal and vertical distributed bars are 10M at 220mm that would be almost equal to the minimum requirement.

Similar trend can be seen for the columns (illustrated in Fig.4-9), also for core slab strips and beams (illustrated in Fig.4-10 Fig.4-14 series) for all buildings. It should be noted that the structural configurations are set up in a way to keep all aspects of building design/construction including ESL method requirements, practical and economical features optimized.

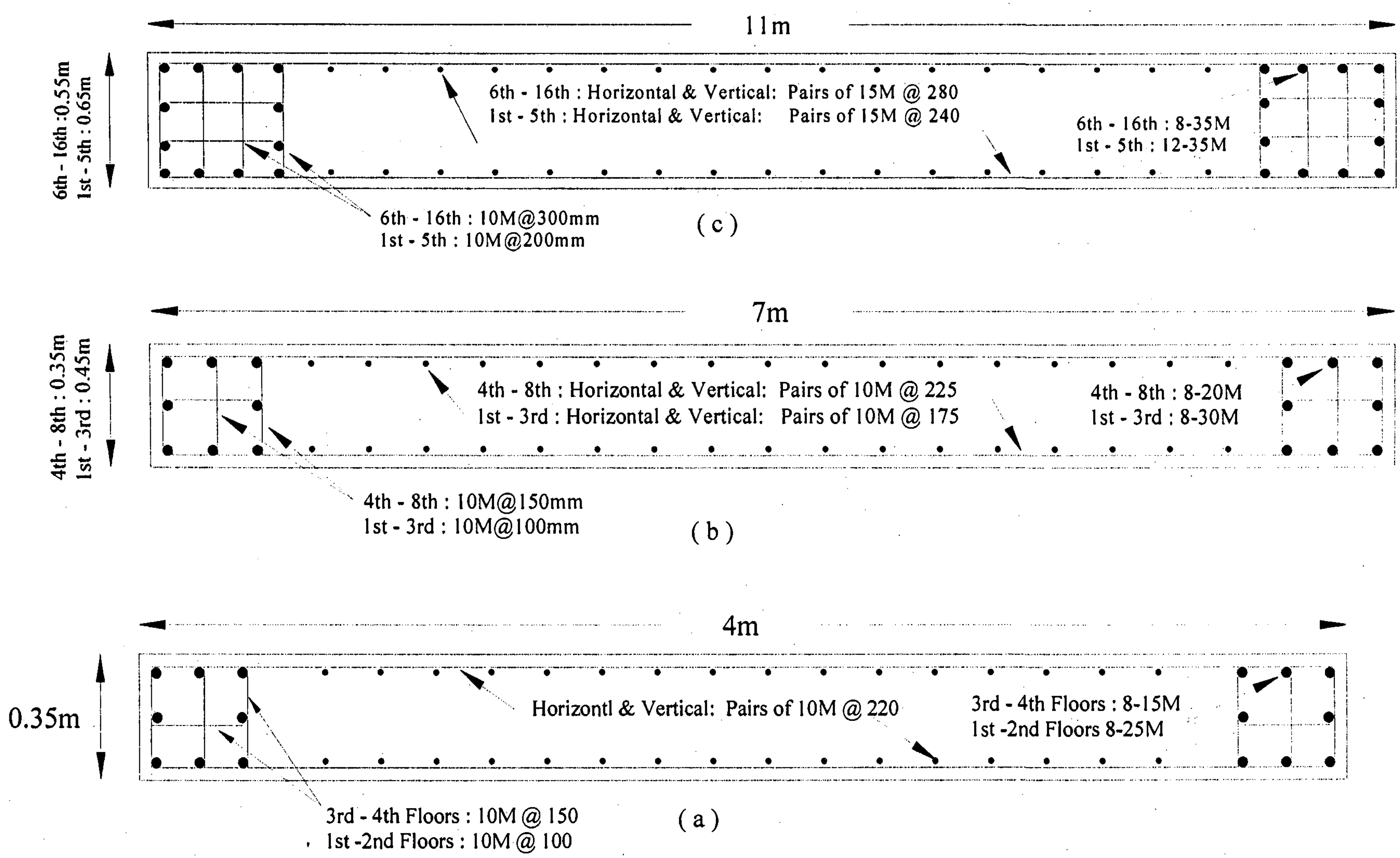


Fig. 4-8: Shear Wall Sections; (a) 4 Storey (b) 8 Storey (c) 16 Storey

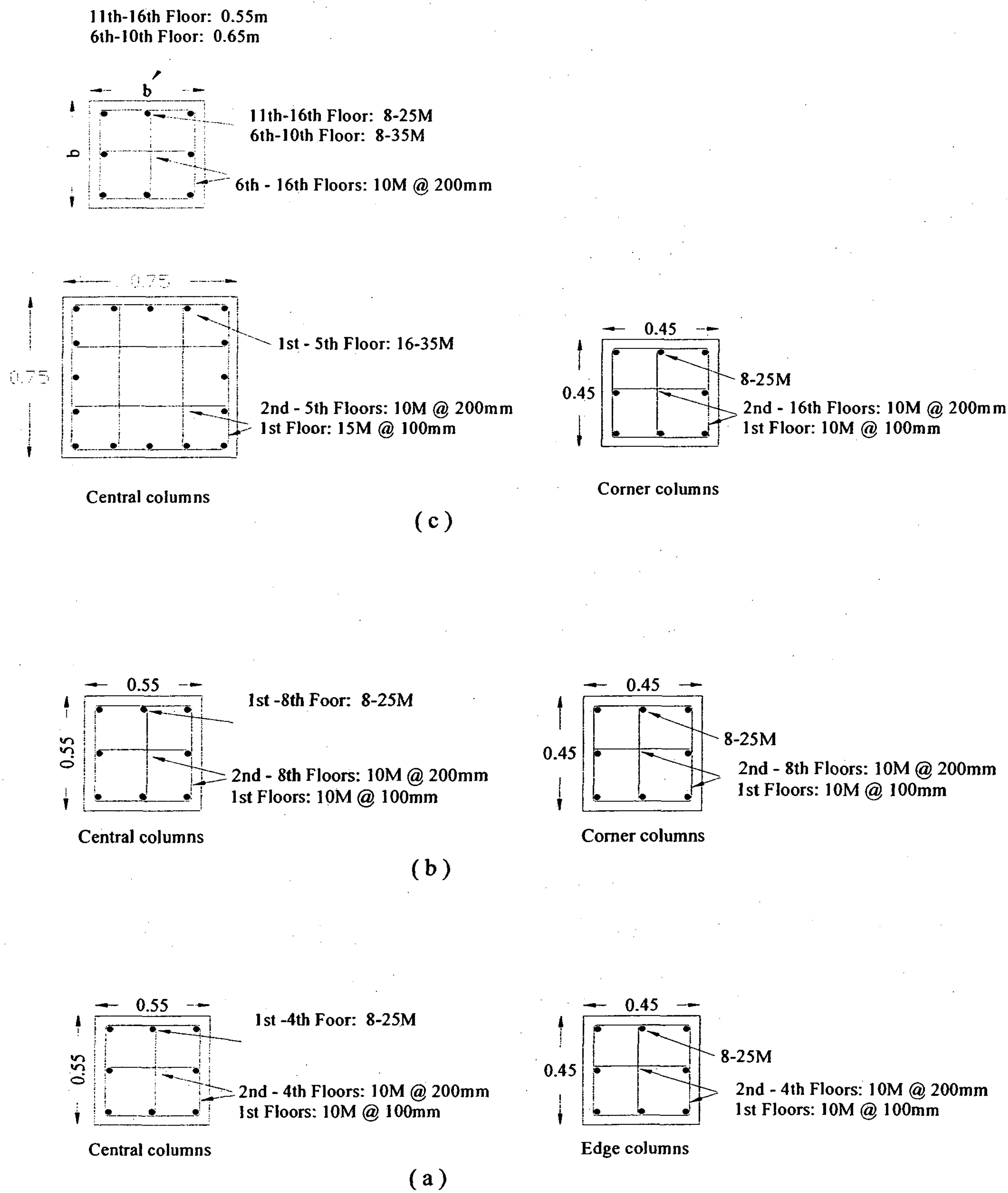
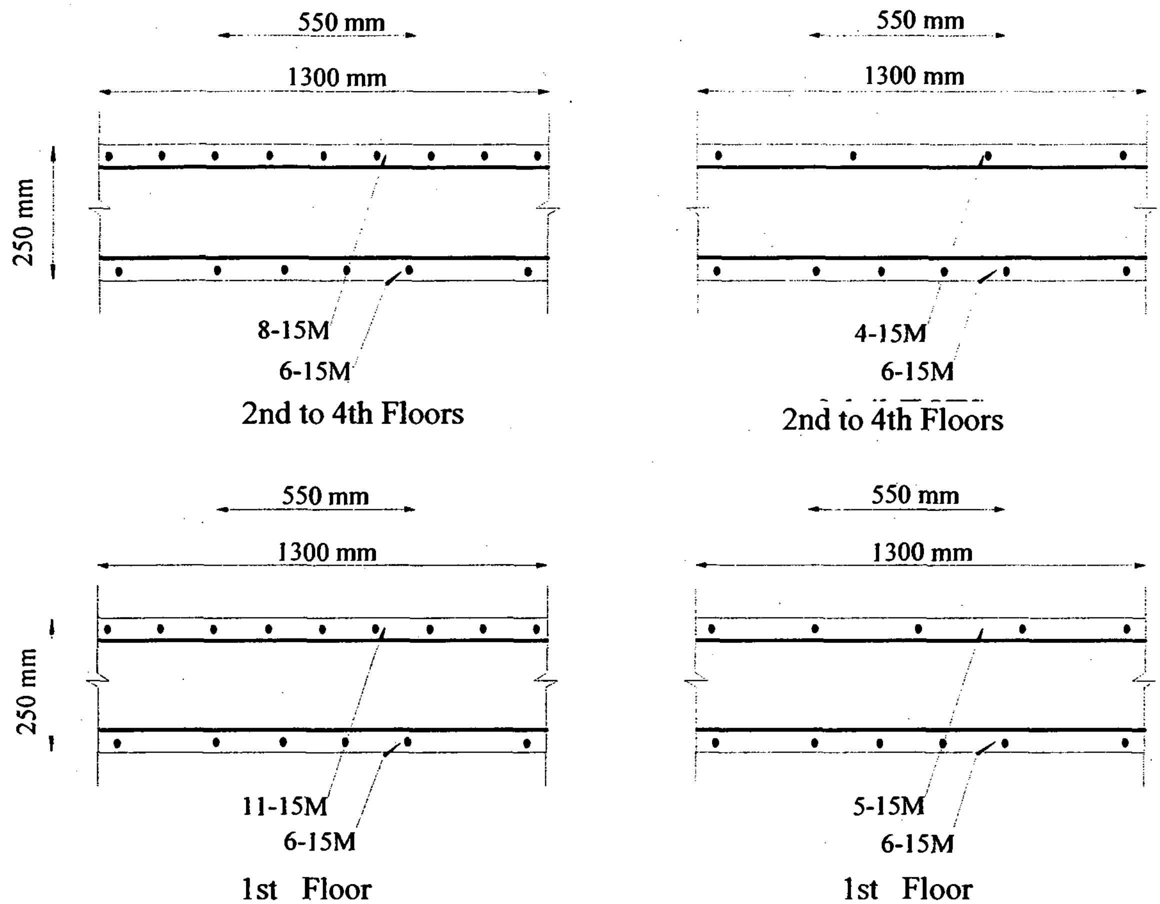


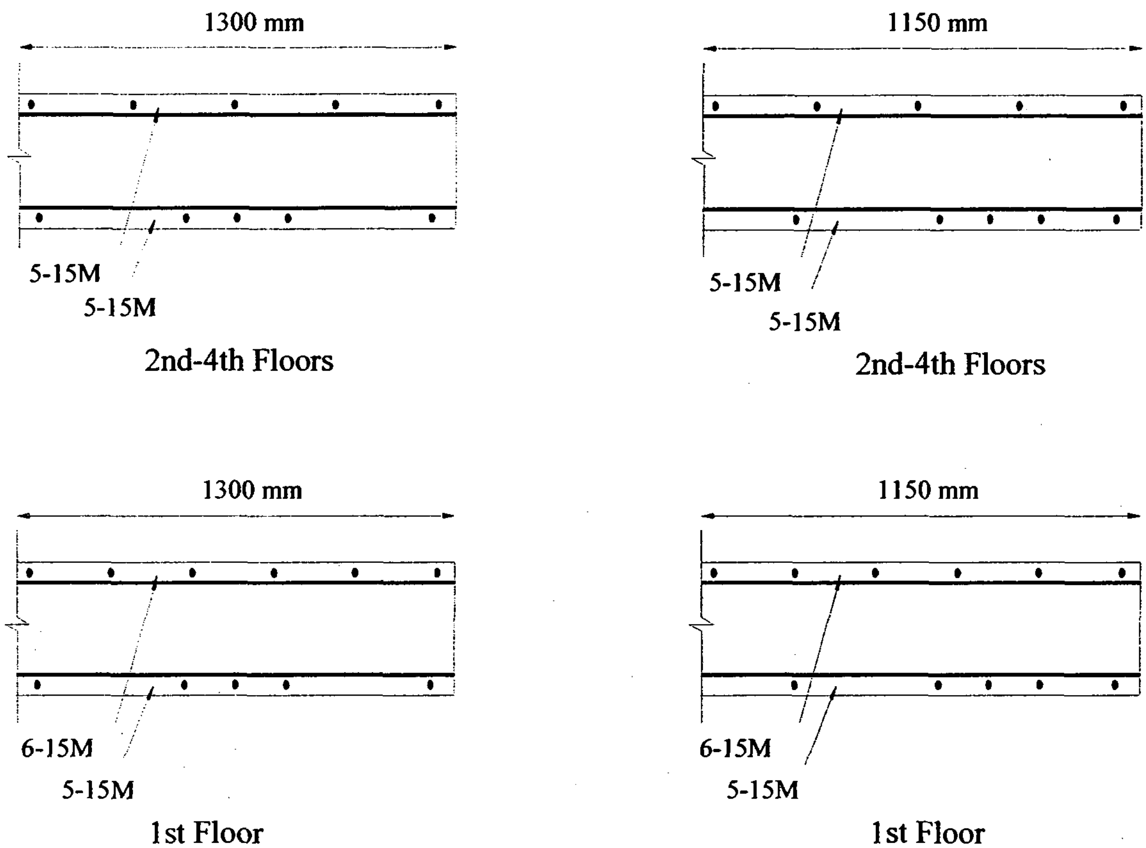
Fig. 4-9 : Column Sections; (a) 4 Storey Building (b) 8 Storey Building (c) 16 Storey Building



Slab; Column Strip along Central Axes at Supports

Slab; Column Strip along Central Axes at Mid Span

Fig. 4-10 : RC Core Strips (as defined in page 35) along Central Axes; 4 Storey Building



Slab; Column Strip along Central-Edge Axes at Support

Slab; Column Strip along Edge Axes at Support

Fig. 4-11 : RC Core Strips (as defined in page 35) along Edge Axes; 4 Storey Building

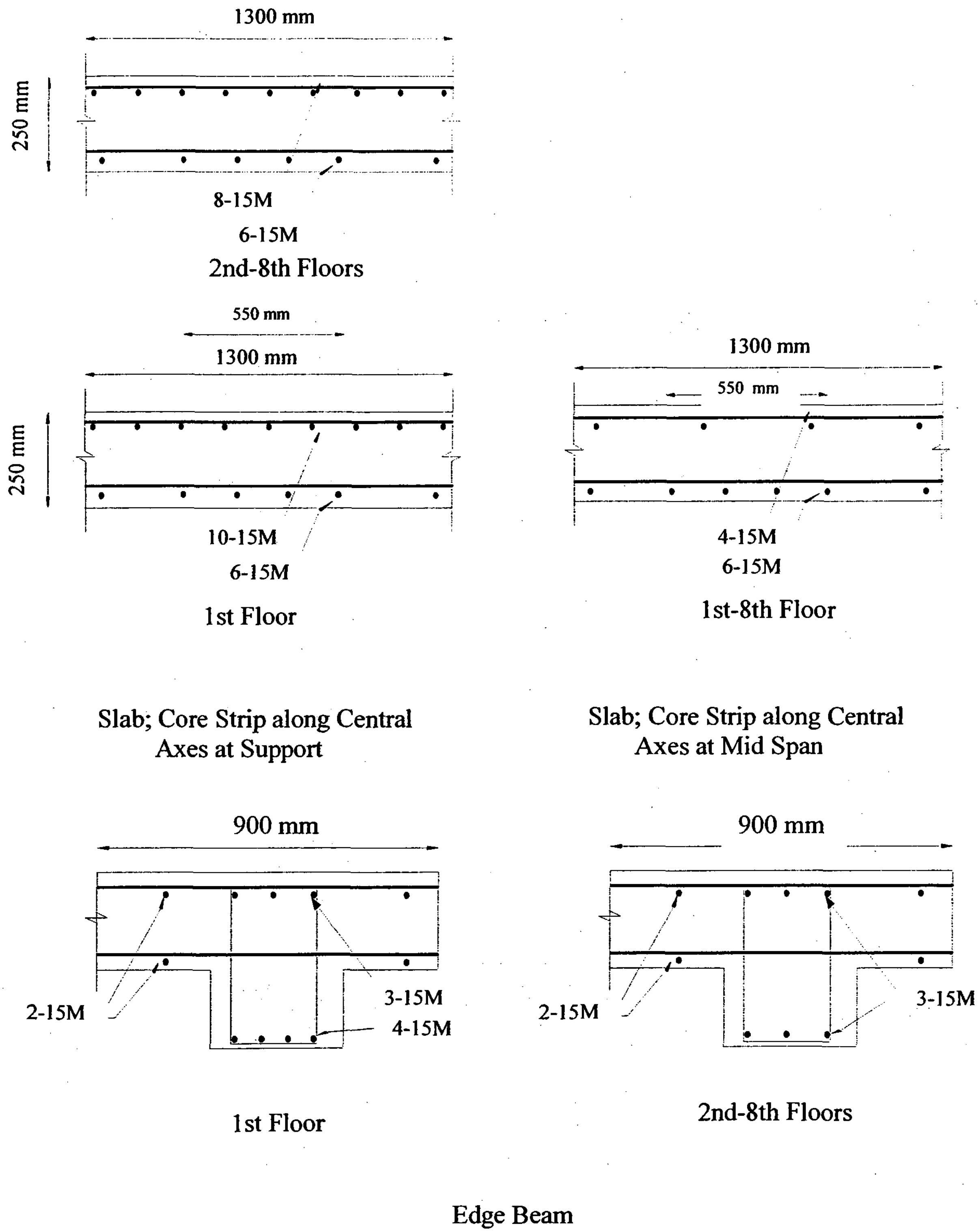
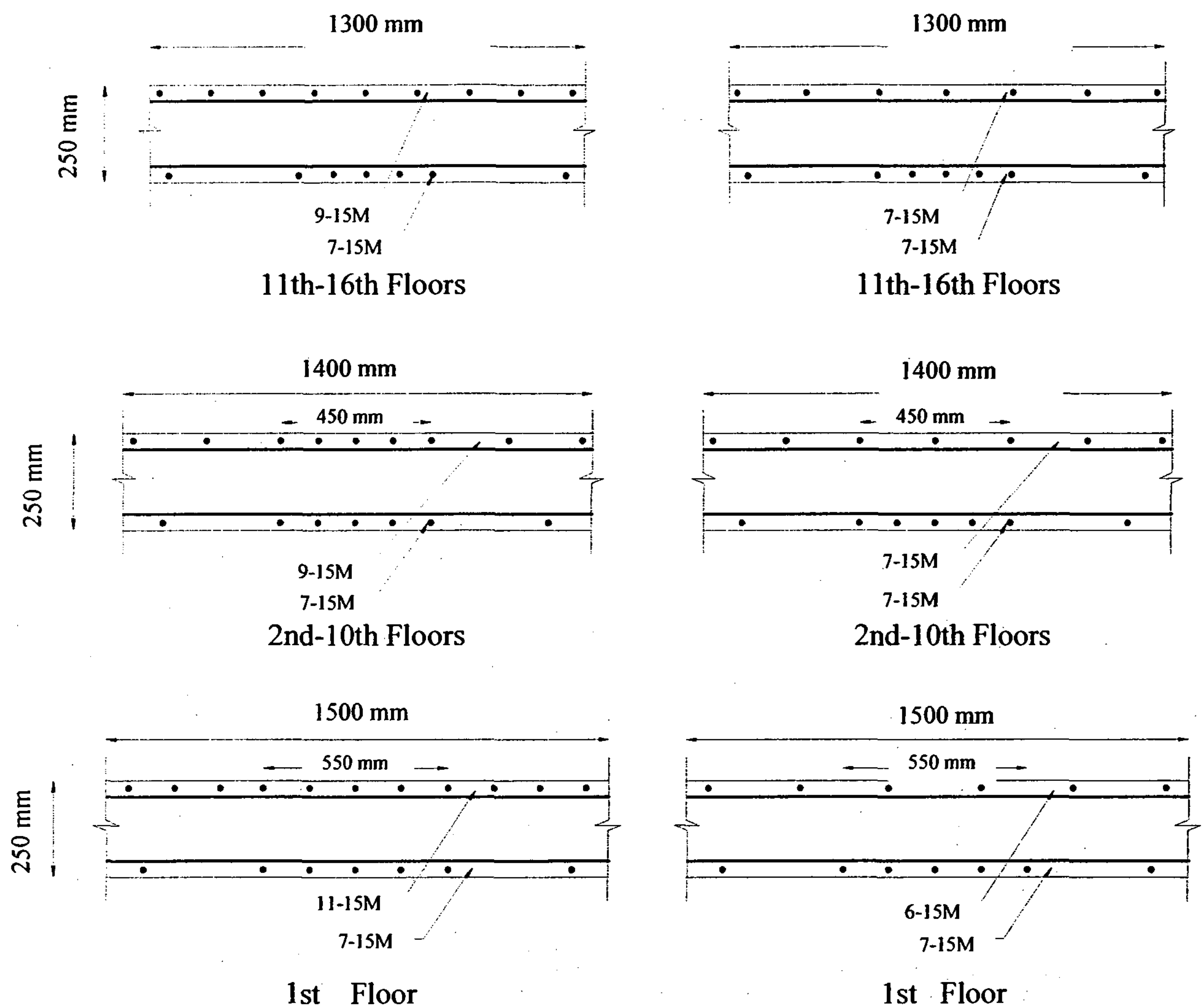


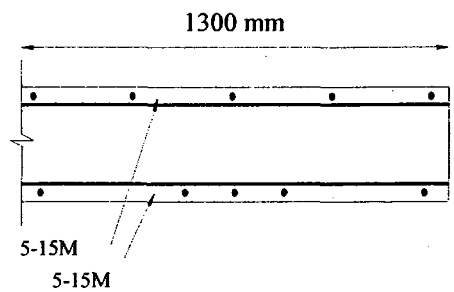
Fig. 4-12 : RC Core Strips (as defined in page 35); 8 Storey Building



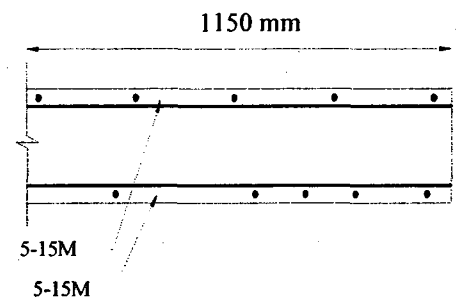
Slab; Core strip along Central Axes at Support

Slab; Core strip along Central Axes at Mid Span

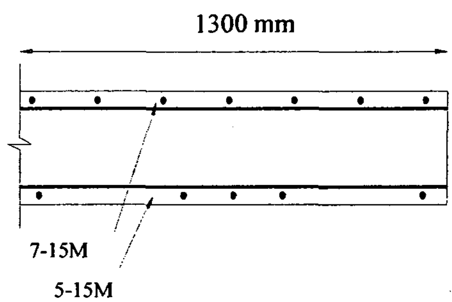
Fig. 4-13: RC Core Strips (as defined in page 35) along Central Axes; 16 Storey building



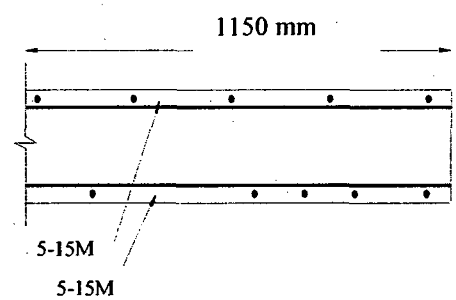
2nd-16th Floors



2nd-16th Floors



1st Floor



1st Floor

Slab; Core Strip along Corner Axes at Support

Slab; Core Strip along Corner Axes at Mid Span

Fig. 4-14: RC Core Strips (as defined in page 35) along Edge Axes; 16 Storey Building

CHAPTER 5

5. EVALUATION OF BUILDINGS' PERFORMANCE

5.1 Introduction

The best and most accurate way to evaluate performance features of a building is to run a nonlinear time history dynamic analysis. Such evaluation would require exposure of the building to a group of earthquake records that should include an adequate number of seismic accelerograms each of which has features representing the region of interest; while the EDPs can be extracted and then assessed.

Key Engineering Demand Parameters (EDPs) studied in this research include inter-storey drift, plastic hinge rotation of shear wall, shear on shear wall, tensile strain values of bars, and compressive (crushing) strain values of the concrete. It should be noted that all values in the evaluation part are nominal but the design loads.

Based on the NBCC 2005 seismic provisions' acceptable solution, buildings should be able to attain the "Collapse Prevention" performance level in the case of a UHS-2500 event, where for instance the inter-storey drift is limited to 2.5% for the buildings considered in this work.

For the purpose of dynamic analysis, a group of 15 earthquake records Table 3-1 offered by Naumoski (Naumoski et. al., 1988), are scaled to fit in to the code's requirement, as described in section 3.4. The calculated scaling factors for the three methods, FAM, PAM, and OM, are presented in Table 5-1. Although, Full Area Method "FAM" is not employed in the time history dynamic analyses, it does not reduce its importance. FAM can specially be of a bigger interest when the building enters into inelastic phase in such case building periods would increase.

The envelope curves over the 15 scaled response spectrums, resulting from the actual accelerograms, prove a well identifiable margin with some spikes above the Vancouver's design spectrum curve defined by the code; that is true for all methods and all buildings, Fig. 5-1 to Fig. 5-3.

The impact of such difference is shown in Table 5-2 and figures Fig. 4-2, Fig. 4-3, and Fig. 4-4. The results proves that base shears resulting from PAM and OM envelopes of scaled response spectrums analysis are much more in tune with the $ESL_{empirical}$ method than with the $ESL_{revised}$ or DRS method of analyses.

Table 5-1: Earthquake Records' Scaling Factors

Earthquake Records Scaling Factors for 16 Storey Building															
Earthquake Record	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13	R14	R15
Full Area Method	1.0131	2.1363	2.2689	2.5519	9.4523	2.7089	1.5411	2.2758	2.1072	1.9045	5.6994	4.8533	1.7464	1.8783	2.1345
Ordinate Method	0.8861	2.4563	3.1267	2.2914	10.7083	2.8643	2.0978	2.3272	2.1278	2.4527	10.8502	6.5015	1.6873	1.6173	2.0202
Partial Area Method	0.8622	1.8515	1.9881	2.0308	9.1698	2.6597	1.6143	2.2348	1.9740	2.0809	4.5137	4.1988	1.5915	1.4330	1.6918

Earthquake Records Scaling Factors for 8 Storey Building															
Earthquake Record	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13	R14	R15
Full Area Method	1.0052	2.1141	2.2566	2.5268	9.4509	2.6506	1.5211	2.2625	2.0910	1.8949	5.6754	4.7580	1.7375	1.8604	2.1254
Ordinate Method	1.1669	2.3599	1.9811	1.7930	7.6450	1.8381	0.9946	1.6387	2.9649	2.0244	4.9934	2.5977	1.9207	0.7797	1.4446
Partial Area Method	0.8735	1.8463	1.8109	2.1770	8.4999	2.3180	1.4778	2.3056	1.8223	1.7840	4.0593	3.7571	1.5597	1.7200	1.7882

Earthquake Records Scaling Factors for 4 Storey Building															
Earthquake Record	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10	R11	R12	R13	R14	R15
Full Area Method	0.9922	2.1097	2.2331	2.4772	9.3794	2.6720	1.5155	2.2101	2.0497	1.8955	5.5119	4.7344	1.7257	1.8369	2.0708
Ordinate Method	0.8277	1.6522	1.5082	2.5637	9.4082	2.0398	1.5865	2.6314	2.1044	1.9881	2.8702	2.6254	2.4222	1.7874	1.4586
Partial Area Method	0.8365	1.7473	1.7444	2.8367	8.0957	2.1290	1.5275	2.2964	1.7007	1.6738	3.7229	3.9451	1.5027	2.8021	2.3819

Maximum among 3 methods   Medium among 3 methods   Minimum among 3 methods

It is also observed that the spectral accelerations corresponding to each method of scaling varies well around the design spectral acceleration values defined by the code, except for the exact value of T1 in OM of scaling to which spectral values for all the 15 earthquake records are equal. The ARSs for the 15 accelerograms are shown with Series1 to Series15 and with solid lines; and the envelop spectrum over the 15 records are shown with dotted line, Fig.5-1 and Fig.5-2. It can be seen that if a building enters to inelastic phase, the building's period and spectral acceleration will both increase (the thick lines in Fig.5-3); the rate of such increasing declines as the buildings height increases. That is despite the case of design ARS defined by the code in that the response spectrum will decrease as the period increases.

After a building yields, its fundamental period will increase. Then, the increased fundamental period will be within the range of periods considered in Full Area Method (FAM), and or Partial Area Method (PAM). However, this increased fundamental period is different from that of Ordinate Method (OM) of scaling. Therefore, in such case, PAM and FAM scaling approaches can be used as preferable methods compared to OM. Then, an overall envelope spectrum over the 3 earlier envelopes (i.e., the envelop for the 15 accelerograms for each of the 3 methods of scaling), also a synthesized accelerogram representing this overall response spectrum can be created. These spectra, and the synthesized accelerogram can be used for response spectrum and time history dynamic analyses respectively instead of 45 run for each case.

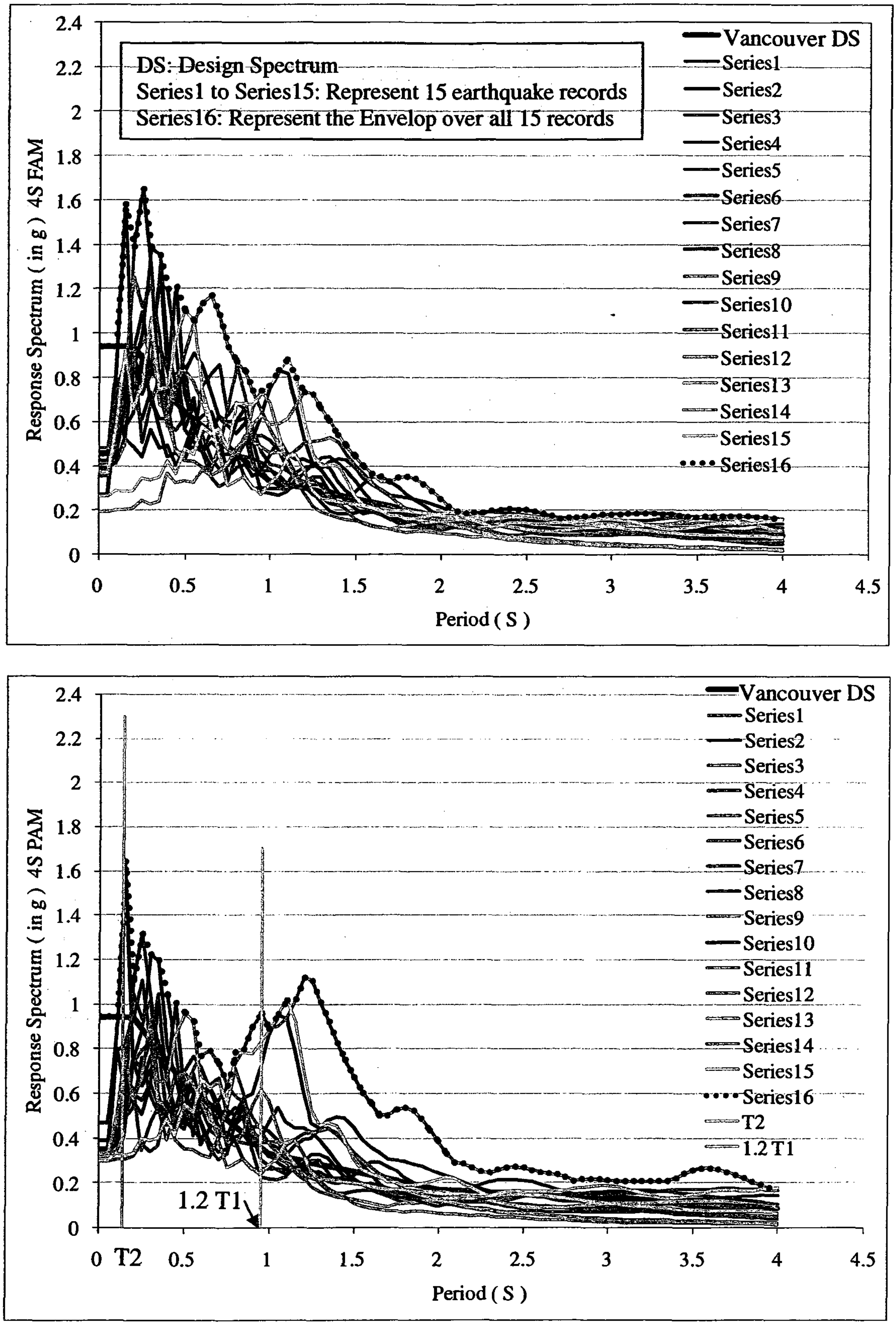
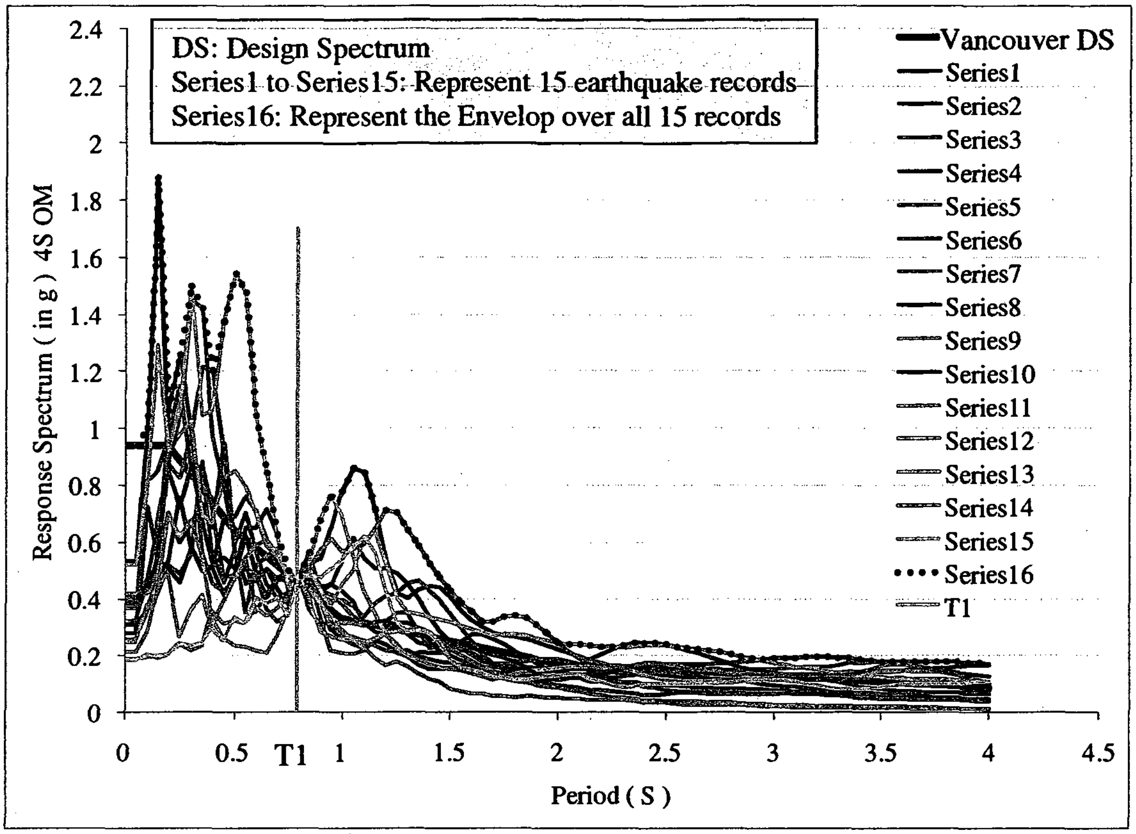


Fig. 5-1: Magnified Records and their Envelop, 4 Storey Building; (a) PAM (b) FAM



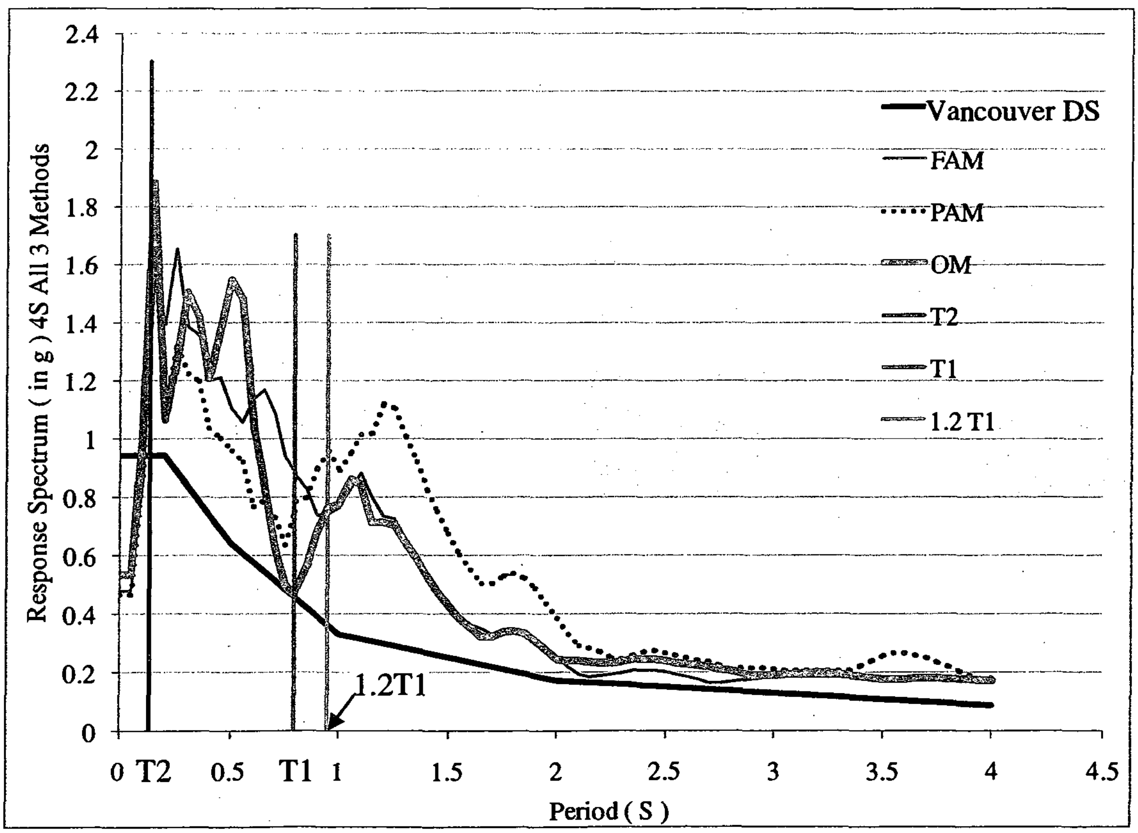
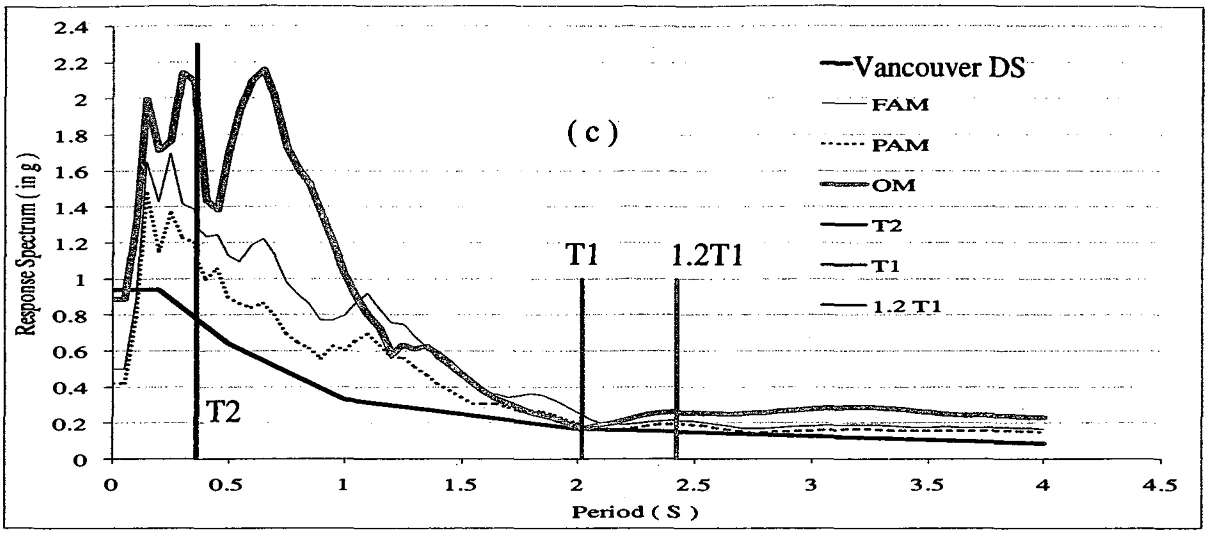
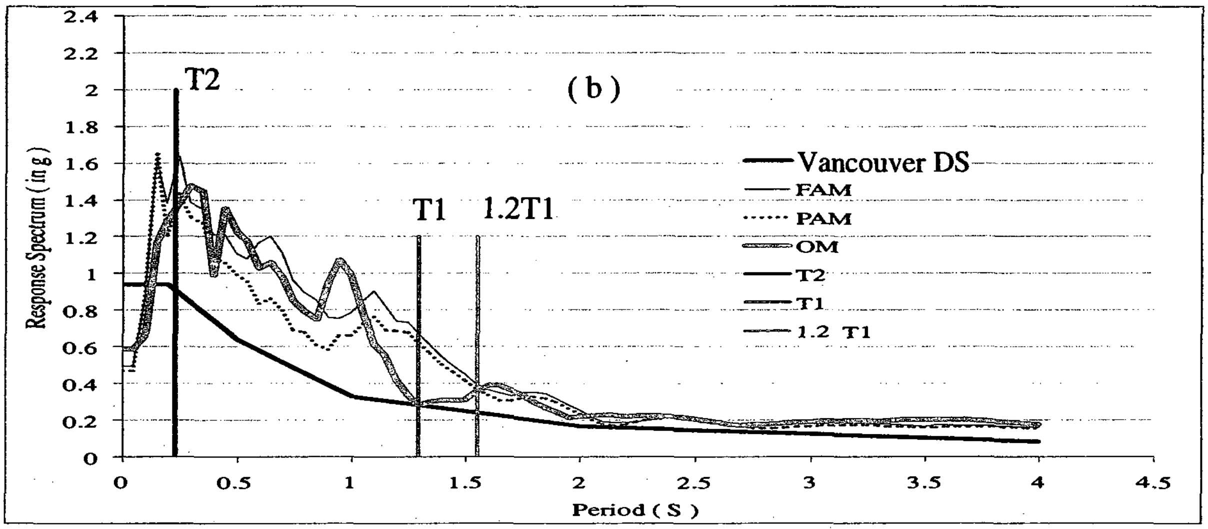


Fig. 5-2:Magnified Records and their Envelop, 4 Storey Building; (a) OM (b) Envelop of all Methods





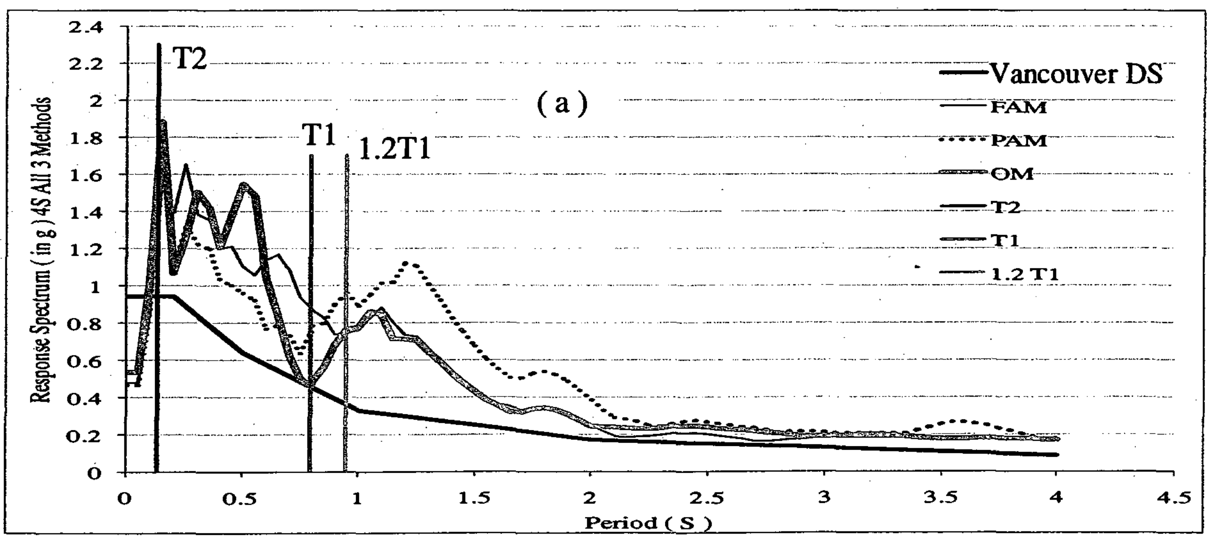


Fig. 5-3 : Envelop over all the 15 records for each method of scaling;
(a) 4 Storey (b) 8 Storey (c) 16 Storey

A comparison of results from different methods of analyses as shown in Table 5-2 indicates shows that "EDP /$EDP_{Revised}$" ratio for M (moment at the base of wall) and for Δf (maximum displacement at the roof level) for each method analysis are very close for all three buildings. "EDP /$EDP_{Revised}$" ratio for M and Δf decreases as the building hight increases. However, an irregularity is observed in the case of 8 storey building that can be because of perimeter beams in the 8 Story building. "EDP /$EDP_{Revised}$" ratio for "V" (base shear in wall) follows similar pattern but in the reverse order (in an ascending order), and with similar irregularity in the 8 story building.

Also, EDP /$EDP_{Revised}$ for (V) divided by that of M or Δf increases as the buildings' height increases. This can be due to higher mode effects and is in a good agreement with the N21.6.9.1 explanatory note on CSA standard A23.3-04 (Cement Association of Canada, 2006) that states "the inelastic effects of higher mode result in a need to increase the shear capacity in wall, but there is not any simplified method to incorporate that yet". This also is true for ESL methods for the other 2 buildings; however, the ratio for base shear (V) resulting from actual response spectrum analyses gets larger than those for M and Δf as the buildings' heights increases. This similar to the irregularity mentioned earlier for the 8 storey building.

Table 5-2: EDP from different methods and comparative calculated data for all the buildings

Method	16 Storey				EDP / EDP Revised			Shear D/C (PERF 3D)	S9 / Max[S1-S8]
	T (Sec)	V (kN)	M (kN.m)	Δf (m)	V (kN)	M (kN.m)	Δf (m)		
ESL ($T_{Empirical}$)	1.0725	3029	120898	0.1343	**1.58**	1.53	1.50		
ESL ($T_{Revised}$)	2.0178	1916	78936	0.0895	1.00	1.00	1.00		
DRSL($T_{Computational}$)	2.0178	1923	46768	0.0478	1.004	0.59	0.55		
SRSL($T_{Computational}$) OM	2.0178	3107	69908	0.0751	**1.62**	0.89	0.89	**1.39**	**1.6**
SRSL($T_{Computational}$) PAM	2.0178	3082	59207	0.0577	**1.61**	0.75	0.68	**1.18**	**2.15**

Method	8 Storey				EDP / EDP Revised			Shear D/C (PERF 3D)	S9 / Max[S1-S8]
	T (Sec)	V (kN)	M (kN.m)	Δf (m)	V (kN)	M (kN.m)	Δf (m)		
ESL ($T_{Empirical}$)	0.6473	2222	46505	0.0834	**1.77**	1.71	1.67		
ESL ($T_{Revised}$)	1.2947	1255	27185	0.0498	1.00	1.00	1.00		
DRSL($T_{Computational}$)	1.2981	1126	17982	0.0319	0.90	0.66	0.64		
SRSL($T_{Computational}$) OM	1.2981	1418	19047	0.0323	**1.13**	0.70	0.65	**1.53**	**2.19**
SRSL($T_{Computational}$) PAM	1.2981	2043	38323	0.0695	**1.63**	1.41	1.40	**1.55**	**2.23**

Method	4 Storey				EDP / EDP Revised			Shear D/C (PERF 3D)	S9 / Max[S1-S8]
	T (Sec)	V (kN)	M (kN.m)	Δf (m)	V (kN)	M (kN.m)	Δf (m)		
ESL ($T_{Empirical}$)	0.3962	981	10950	0.0376	**1.36**	1.35	1.32		
ESL ($T_{Revised}$)	0.7925	719	8084	0.0284	1.00	1.00	1.00		
DRSL($T_{Computational}$)	0.8943	575	6054	0.0204	0.80	0.75	0.72		
SRSL($T_{Computational}$) OM	0.8943	961	9557	0.0319	**1.32**	1.18	1.12	**1.30**	**1.82**
SRSL($T_{Computational}$) PAM	0.8943	1138	12572	0.0426	**1.57**	1.56	1.50	**1.29**	**1.29**

After employing the scaled accelerograms in the inelastic dynamic analysis process, the results are extracted and depicted in terms of D/C ratios in Fig. 5-4 to Fig 5-11.

D/C ratio of nine EDP are illustrated in these figures. Levels of the D/C ratio for shear in wall are shown with short horizontal line segments (Fig. 5-4); in order to avoid a jerky view such short line segments are not shown for any other EDP. In order to be able to easily follow the trend in change of D/C for each EDP, and to make one EDP well distinguishable from the others, D/C values for each of the 15 records are connected to each other by line segments (hereafter called tracing line). Therefore, the level of D/C ratio for EDPs other than shear in walls can be recognized only as the pivot points, the point in which the 2 tracing lines converge. Such illustrations are provided for two scaling methods, OM and PAM. Then, the mean, mean plus standard deviation and the maximum statistical values over the 15 records and for each method of scaling (OM , PAM) are depicted with horizontal line segments. This type of graph is given for 0% , 5% and 10% mass eccentricities. Then, in a 2nd type of graph such as Fig. 5-5 (lower graph), only the statistical values of D/C (horizontal line segments) for each mass eccentricity (0% 5% 10%), and under each scaling method (OM , PAM) are ilustrated together for each building. In this type of graph, each horizontal line segment is connected to that of preceeding or succeeding one just to provide a step-like look to visualize the change magnitude in corresponding quantities.

To have just the statistical values of the EDPs for all buildings in one view, the second type of the grapghs are put together in Fig. 5-10 and Fig. 5-11 5-11. Fig. 5-11 illustrates the statistical D/C values only for shear in the wall as the chief EDP; this is similar to the second type of graphs described above.

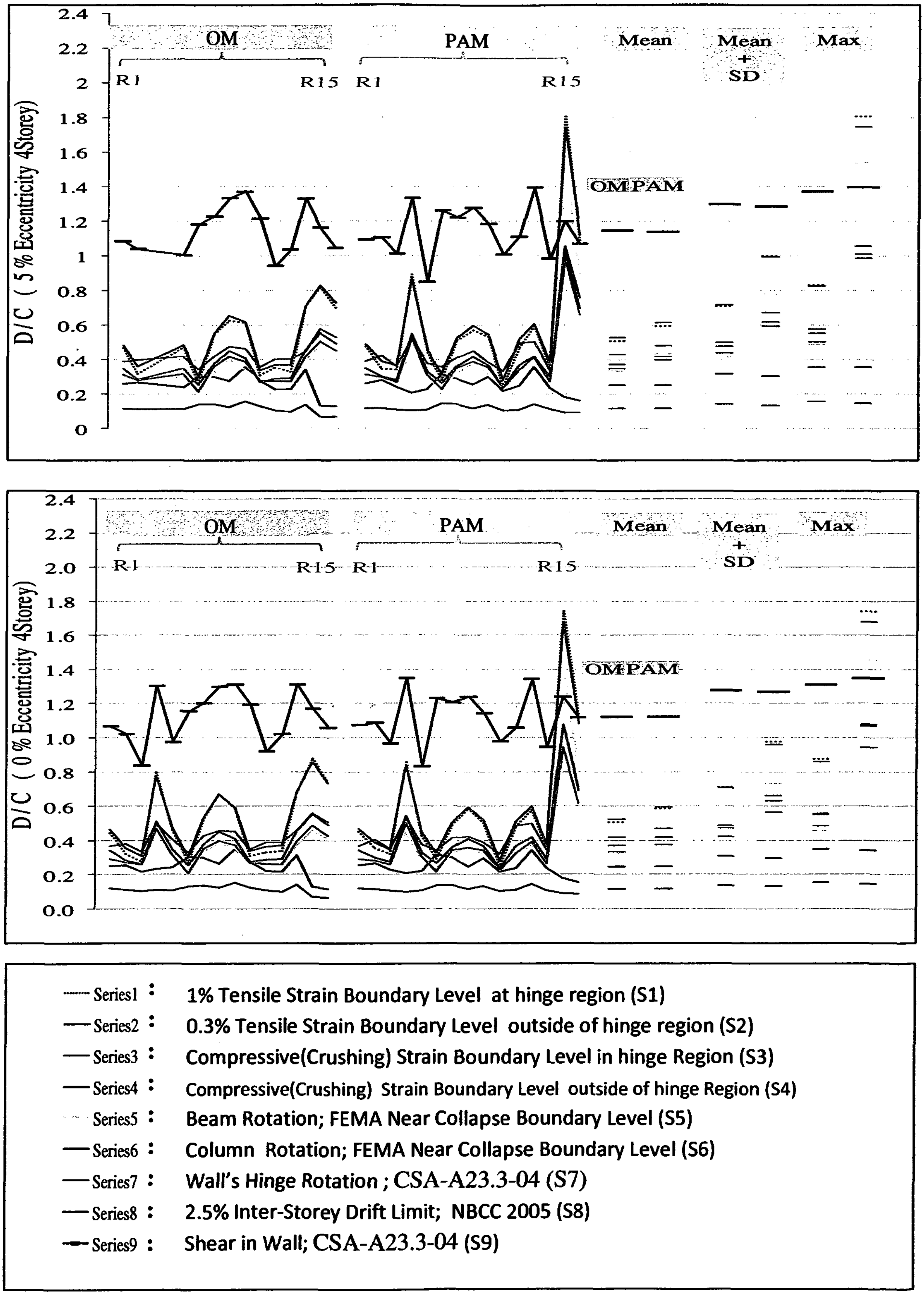
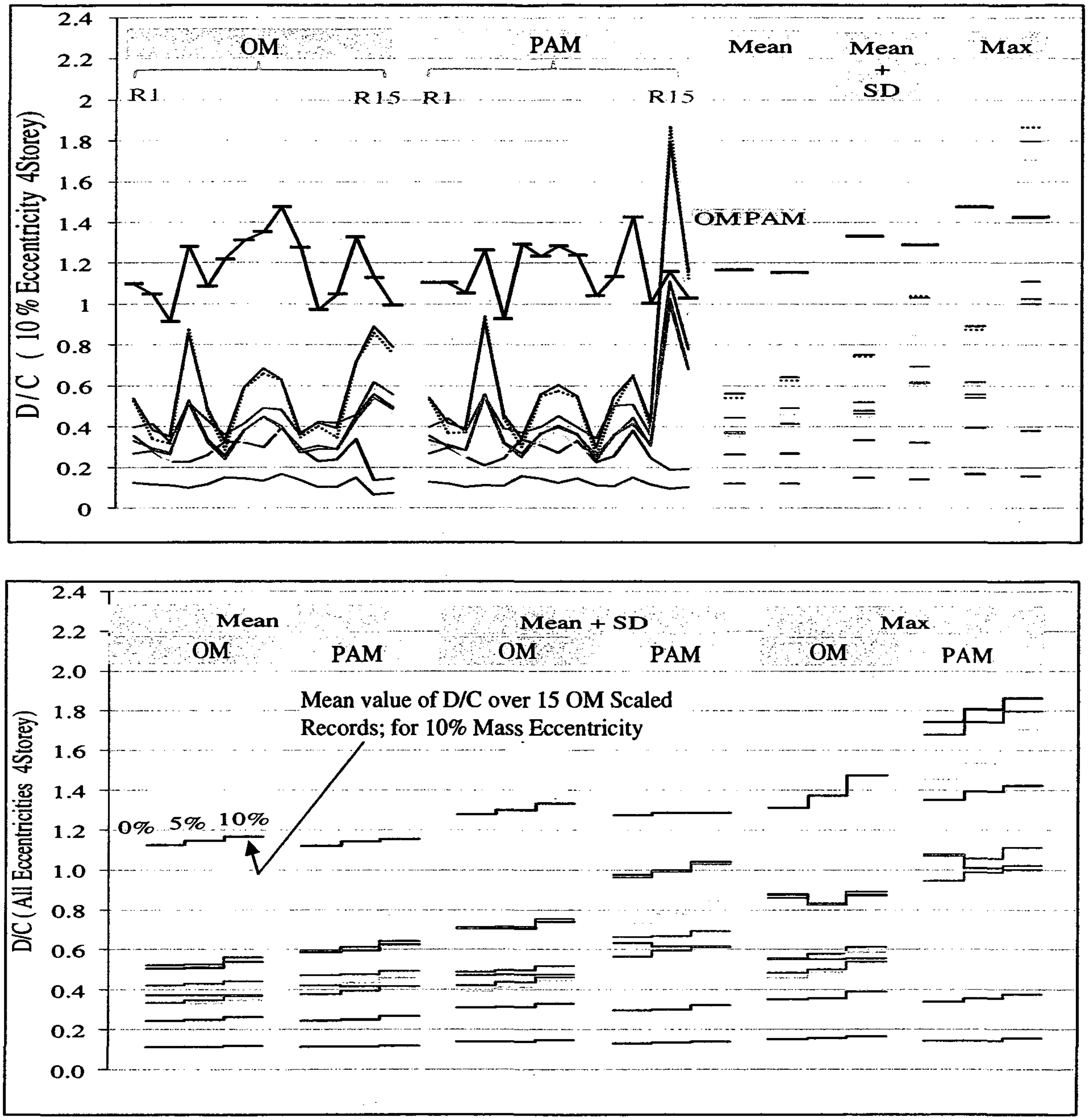
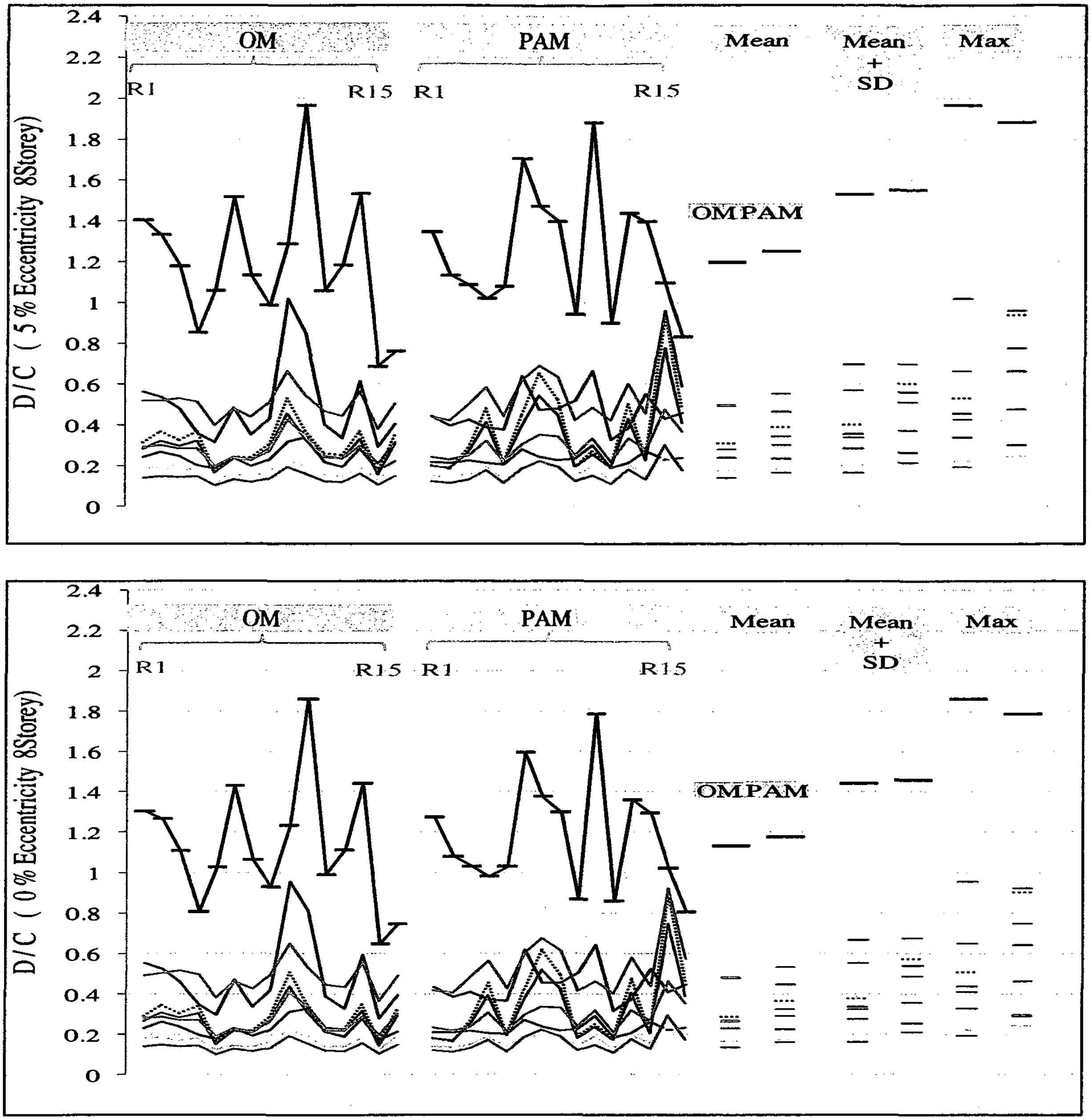


Fig. 5-4: Demand to Capacity/Boundary Levels; 4 Storey Building



- Series1 : 1% Tensile Strain Boundary Level at hinge region (S1)
- Series2 : 0.3% Tensile Strain Boundary Level outside of hinge region (S2)
- Series3 : Compressive(Crushing) Strain Boundary Level in hinge Region (S3)
- Series4 : Compressive(Crushing) Strain Boundary Level outside of hinge Region (S4)
- Series5 : Beam Rotation; FEMA Near Collapse Boundary Level (S5)
- Series6 : Column Rotation; FEMA Near Collapse Boundary Level (S6)
- Series7 : Wall's Hinge Rotation ; CSA-A23.3-04 (S7)
- Series8 : 2.5% Inter-Storey Drift Limit; NBCC 2005 (S8)
- Series9 : Shear in Wall; CSA-A23.3-04 (S9)

Fig. 5-5: Demand to Capacity-Boundary Levels; 4 Storey Building



Series1 : 1% Tensile Strain Boundary Level at hinge region (S1)
Series2 : 0.3% Tensile Strain Boundary Level outside of hinge region (S2)
Series3 : Compressive(Crushing) Strain Boundary Level in hinge Region (S3)
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Series8 : 2.5% Inter-Storey Drift Limit; NBCC 2005 (S8)
Series9 : Shear in Wall; CSA-A23.3-04 (S9)

Fig. 5-6: Demand to Capacity-Boundary Levels; 8 Storey Building

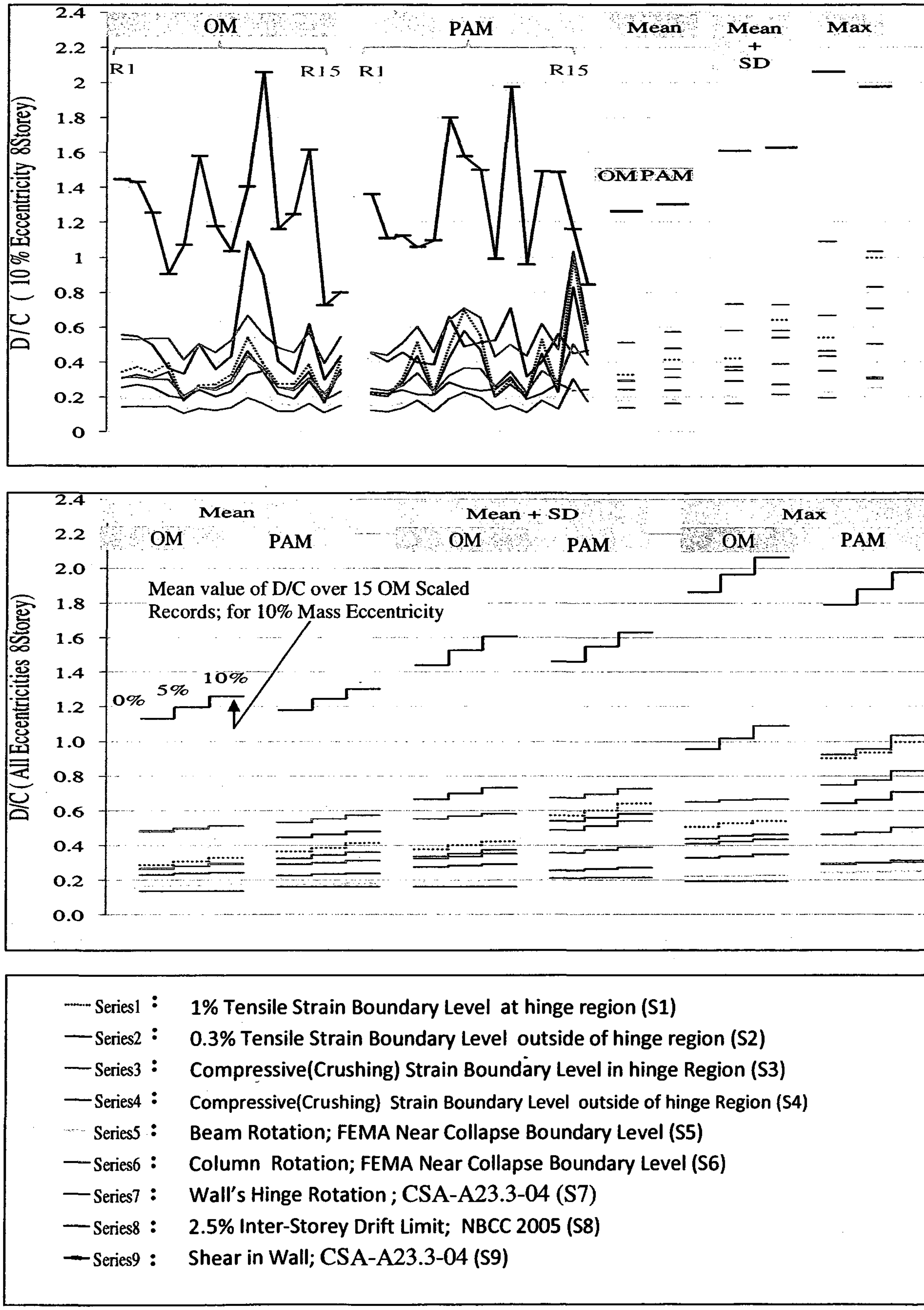
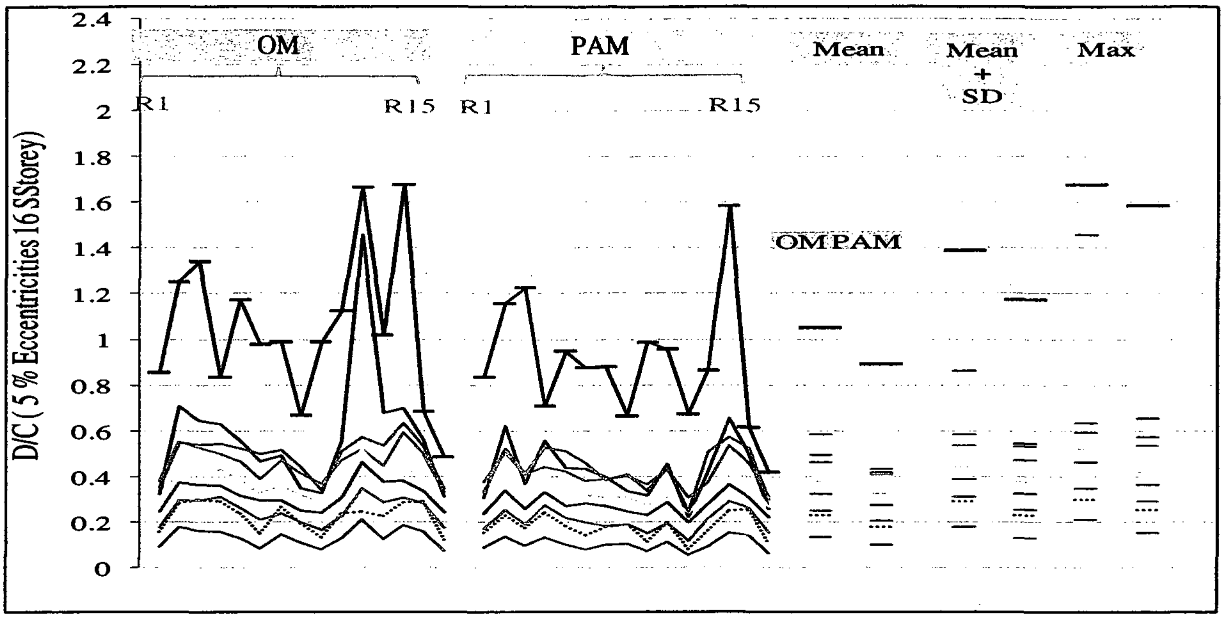
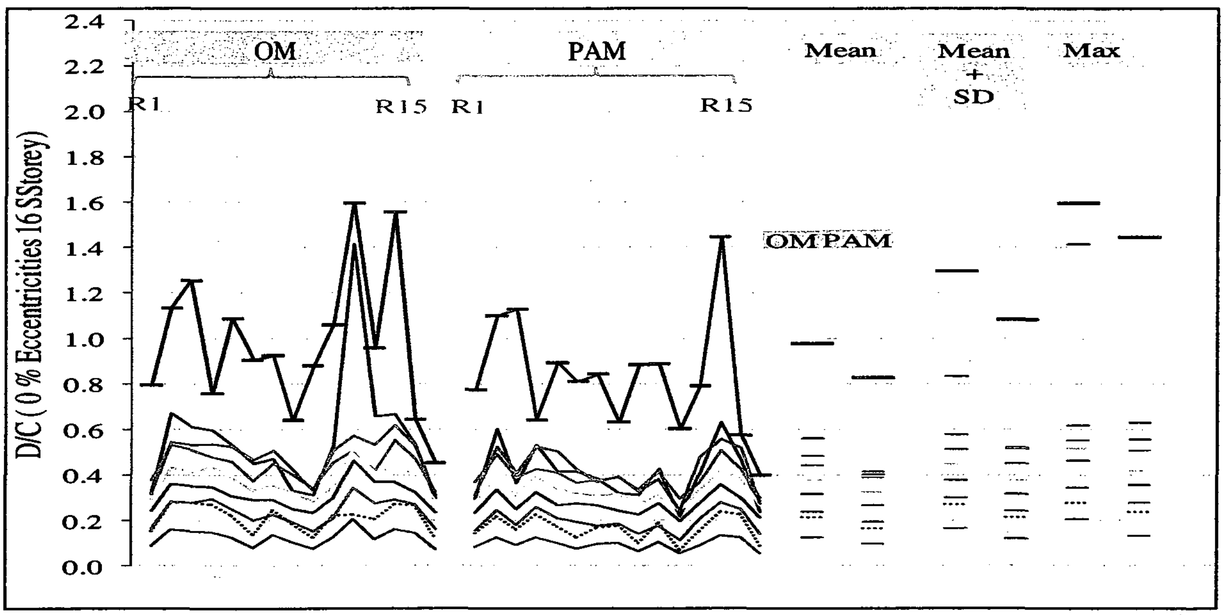


Fig. 5-7: Demand to Capacity-Boundary Levels; 8 Storey Building





Series1 : 1% Tensile Strain Boundary Level at hinge region (S1)
Series2 : 0.3% Tensile Strain Boundary Level outside of hinge region (S2)
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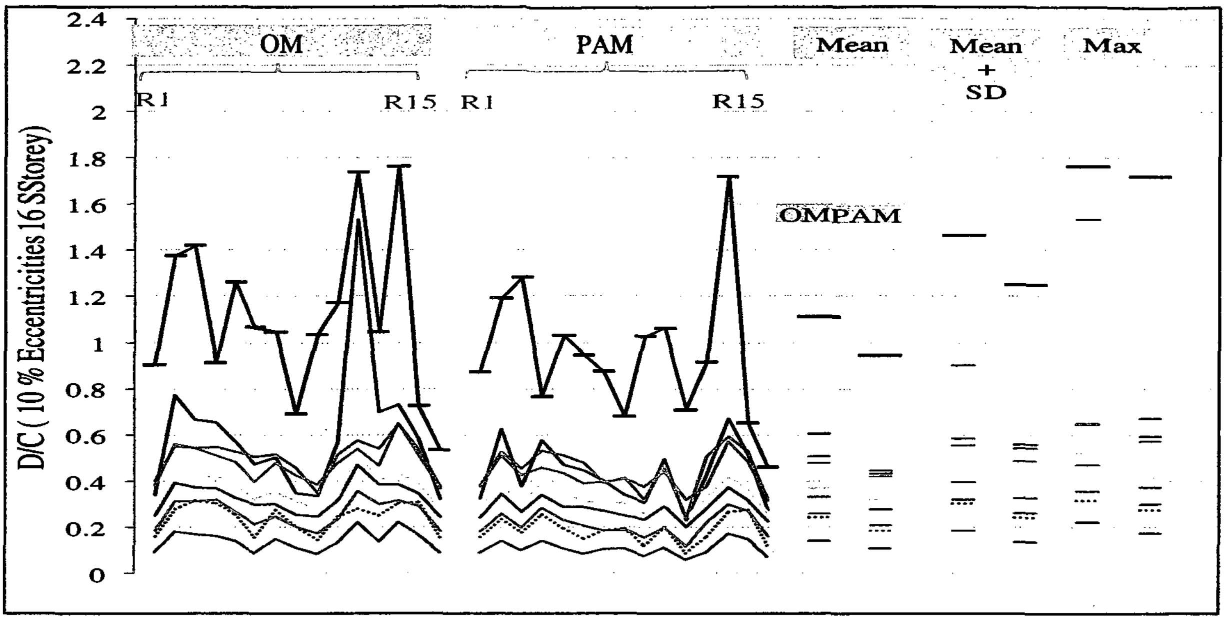
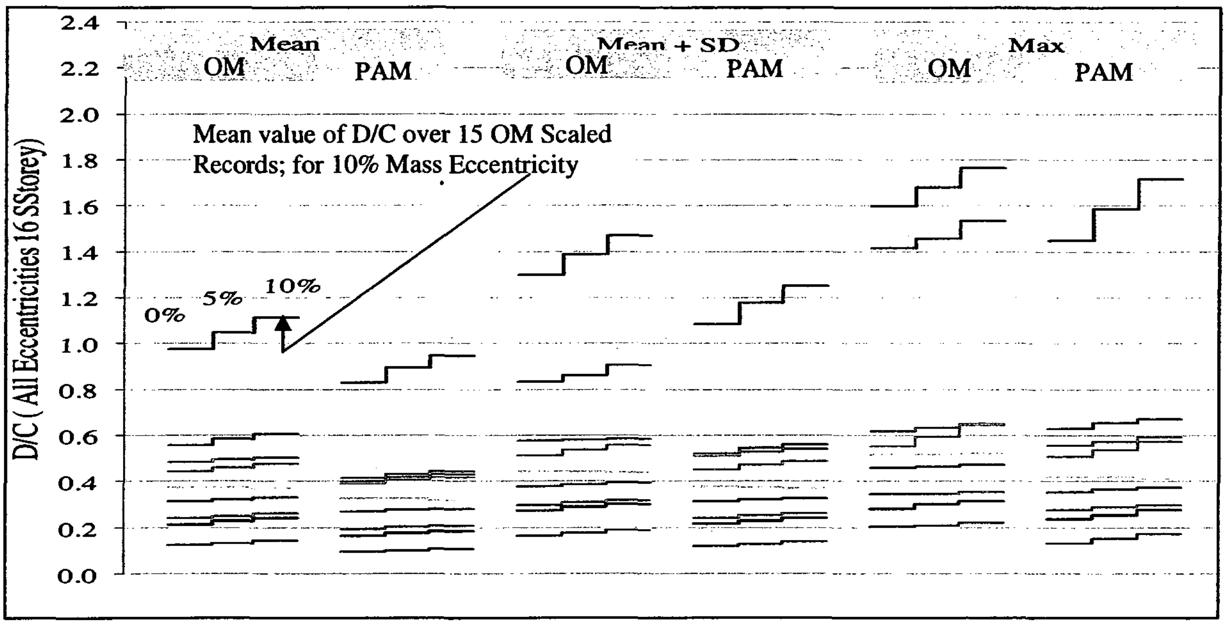
Series6 : Column Rotation; FEMA Near Collapse Boundary Level (S6)
Series7 : Wall's Hinge Rotation ; CSA-A23.3-04 (S7)
Series8 : 2.5% Inter-Storey Drift Limit; NBCC 2005 (S8)
Series9 : Shear in Wall; CSA-A23.3-04 (S9)

Fig. 5-8: Demand to Capacity-Boundary Levels; 16 Storey Building





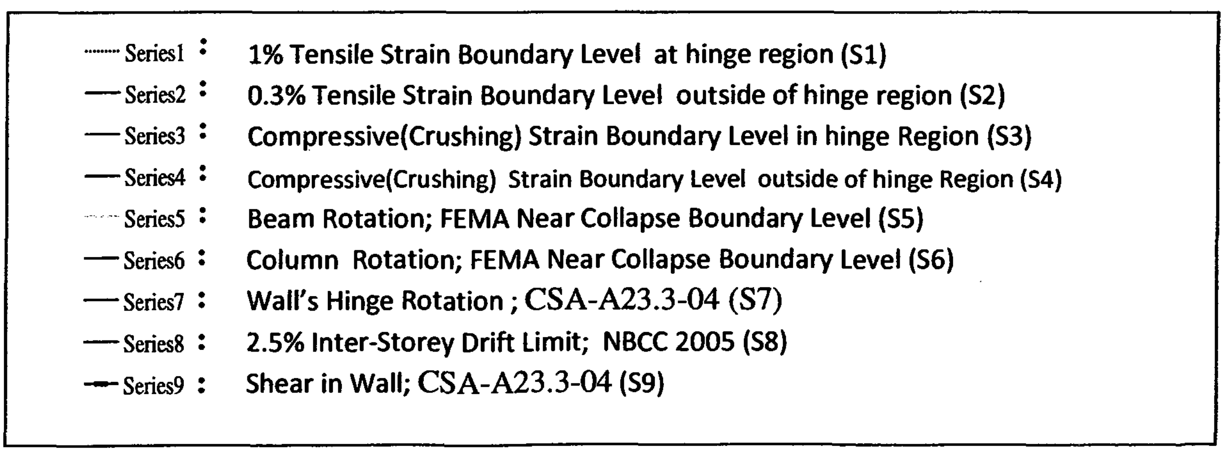


Fig. 5-9: Demand to Capacity-Boundary Levels; 16 Storey Building

As it can be seen, in Fig. 5-4 to Fig. 5-9, D/C values for shear in wall are well above one, also there is a clear gap between D/C values for shear in wall and D/C values for all other EDPs. Clearly, the shear capacity of the walls are inadequate for the selected ensemble of the ground motion records.

Since the evaluation phase is based on the nominal strength of materials, to have an adequate margine of safety that the standard requires through empoying factored values, D/C of shear in wall should be well bellow "one", about the same level as the other EDPs.

Variation of EDPs, depicted in the graphical illustration of D/C statistical values, takes place at very small and almost equal steps, which implies D/C values are not torsionally sensitive for zero to 10% mass eccentrisities studied in this research. Moreover, "shear in wall" D/C statistical values (Fig. 5-11 5-11) show that OM and PAM give almost equal values for the four and the eight storey buildings, while there is a noticeable difference in the case of sixteen storey building. In this regard, the ratio of the partial area under the envelope of the response spectra of the 15 scaled records, A_{OM} / A_{PAM} are calculated (Table 5-3) and it indicates a possible corelation between A_{OM} / A_{PAM} ratio and the above mentioned difference in EDPs, due to possible result of higher mode effects.

Table 5-3: Ratio of areas under response spectrum curves of different scaling methods

Building's Storey	Area under the Envelop of 15 Scaled Response Spectrum Curves; Between periods T2 & 1.2 T1		A_{OM} / A_{PAM}
	A_{PAM}	A_{OM}	
16 Storey	1.0572	1.7553	1.6603
8 Storey	1.0281	1.0861	1.0564
4 Storey	0 .7978	0.8732	1.0945

It can also be seen that for the four and eight storey buildings that shear on wall D/C value increases as the building height increases, while it drops for the 16 storey building which could be a result of the higher mode effects.

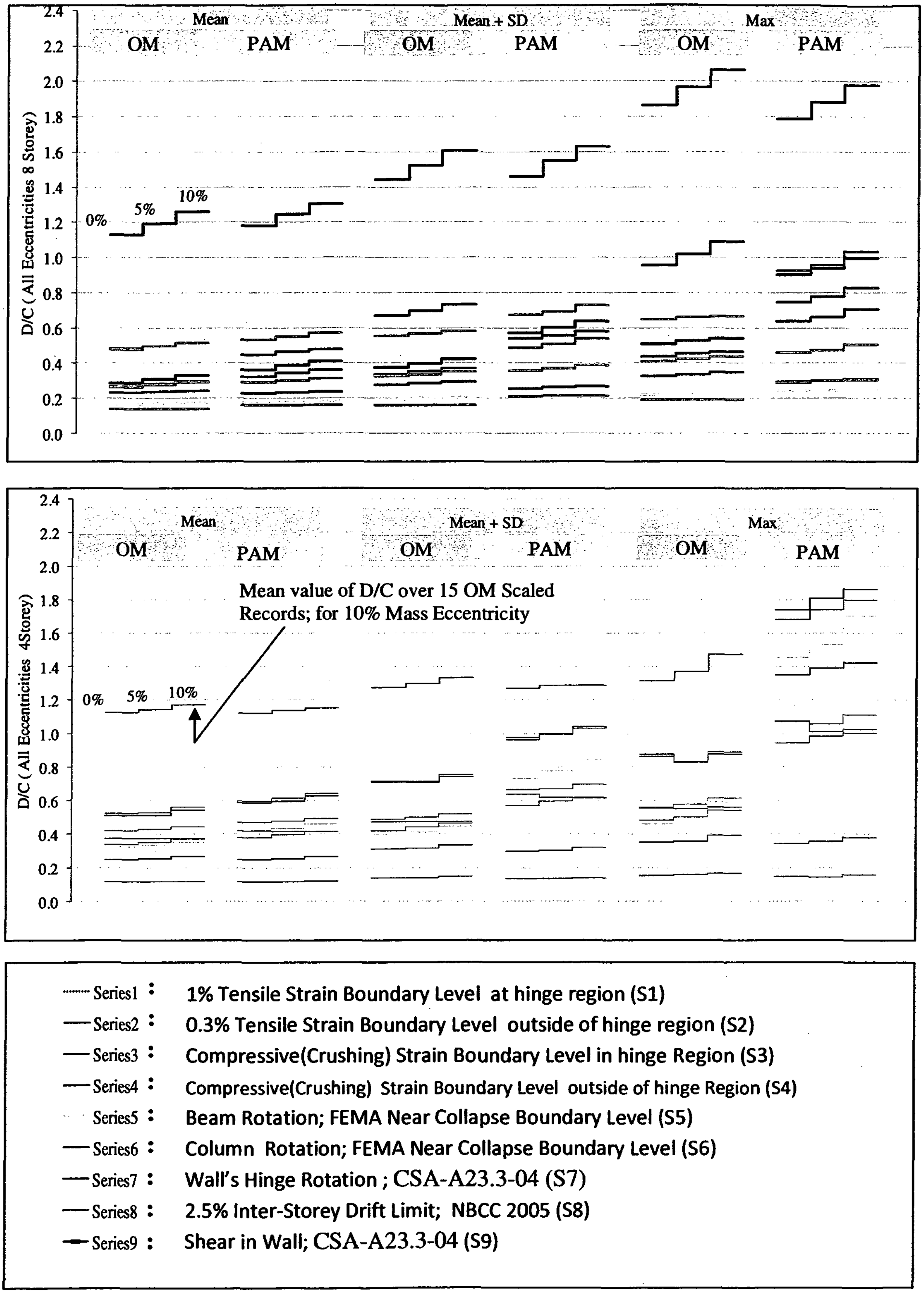
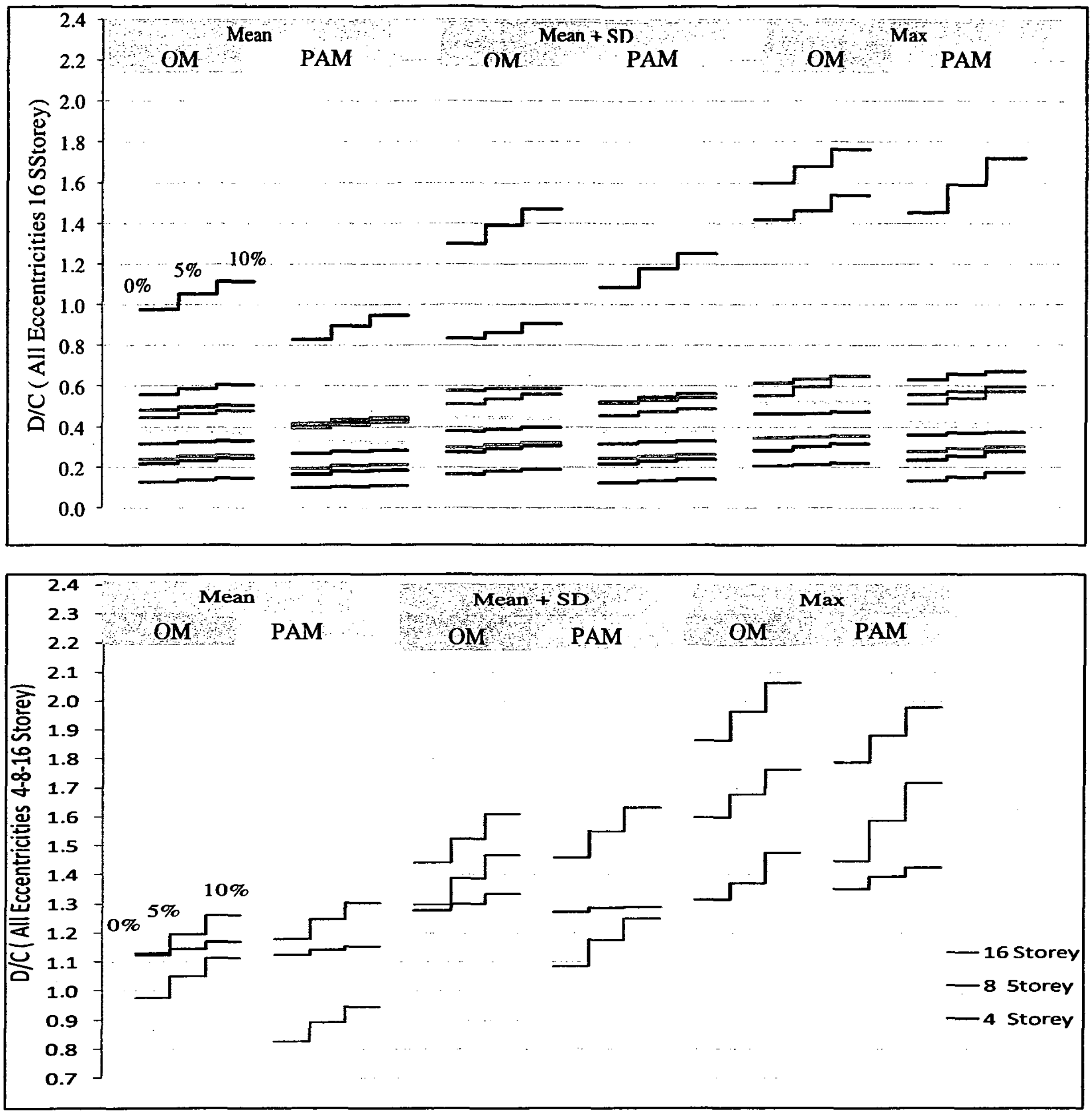


Fig. 5-10: Envelop of D/C over the 15 Scaled Records



Series1 : 1% Tensile Strain Boundary Level at hinge region (S1)

Series2 : 0.3% Tensile Strain Boundary Level outside of hinge region (S2)

Series3 : Compressive(Crushing) Strain Boundary Level in hinge Region (S3)

Series4 : Compressive(Crushing) Strain Boundary Level outside of hinge Region (S4)

Series5 : Beam Rotation; FEMA Near Collapse Boundary Level (S5)

Series6 : Column Rotation; FEMA Near Collapse Boundary Level (S6)

Series7 : Wall's Hinge Rotation ; CSA-A23.3-04 (S7)

Series8 : 2.5% Inter-Storey Drift Limit; NBCC 2005 (S8)

Series9 : Shear in Wall; CSA-A23.3-04 (S9)

Fig. 5-11: Envelop of D/C over the 15 Scaled Records

It is evident from Table 5-4 to Table 5-6 that buildings torsional stiffness reduces as the building's height increases; for instance in case of 0%-5% deviation in mass eccentricity, the "Mean + SD" of D/C values for OM in "shear in wall" increases from 1.77% for 4 storey, to 5.8% for 8 storey, and to 7% for 16 storey building.

The results point out to a need for magnifying the walls' shear capacity. The key values in Table 5-2 are shown in red bold font. The ratio of shear in wall demand resulting from ESL ($T_{Empirical}$), PAM Scaled Response Spectrum Load (RSL ($T_{Computational}$)), OM Scaled RSL ($T_{Computational}$) to that from ESL ($T_{Revised}$) show fairly close values.

V_{ESL} (Empirical) to V_{ESL} (Revised) ratio, still the most simplified method of calculation, could be taken as a reasonable coefficient to bring D/C ratio of shear on wall below one. However, in order to bring that to a level with reasonable margin of safety, a modification value such as the ratio of shear D/C to maximum of all other D/C "S9 / Max[S1-S8]" as shown in Tables 5-5 to 5-7 would be more reasonable.

These values show that a modification factor ranging from 1.3 to 1.55 for the hinge region could be used to scale up the wall's shear capacity in the hinge region which is the critical area that could undergo excessive shear and fail if not modified.

Table 5-4: Comparisons of D/C variation over the 15 scaled records; 4 storey building

	[Mean + SD] of D/C for OM Eccentricity			Variation (in %) of Eccentricity	
	0%	5%	10%	5% from 0%	10% from 5%
S1	0.7124	0.7084	0.7431	-0.5574	4.9036
S2	0.3107	0.3165	0.3340	1.8637	5.5547
S3	0.4881	0.4996	0.5218	2.3424	4.4483
S4	0.1396	0.1413	0.1476	1.2404	4.4365
S5	0.3976	0.4131	0.4495	3.8828	8.8087
S6	0.4737	0.4755	0.4765	0.3707	0.2103
S7	0.7090	0.7151	0.7535	0.8556	5.3670
S8	0.4222	0.4405	0.4657	4.3385	5.7080
S9	1.2774	1.3000	1.3339	1.7745	2.6021
Min of S1 to S8	0.1396	0.1413	0.1476		
Max of S1 to S8	0.7124	0.7151	0.7535		
S9 / Max[S1-S8]	1.7931	1.8180	1.7703		

	[Mean + SD] of D/C for PAM Eccentricity			Variation (in %) of Eccentricity	
	0%	5%	10%	5% from 0%	10% from 5%
S1	0.9781	0.9989	1.0395	2.1199	4.0669
S2	0.2996	0.3028	0.3225	1.0667	6.4873
S3	0.6652	0.6709	0.6955	0.8561	3.6730
S4	0.1334	0.1340	0.1404	0.4373	4.7369
S5	0.7342	0.7783	0.8470	5.9966	8.8331
S6	0.6360	0.6193	0.6142	-2.6328	-0.8250
S7	0.9625	0.9943	1.0323	3.3067	3.8229
S8	0.5679	0.5949	0.6195	4.7556	4.1357
S9	1.2729	1.2868	1.2885	1.0864	0.1382
Min of S1 to S8	0.1334	0.1340	0.1404		
Max of S1 to S8	0.9781	0.9989	1.0395		
S9 / Max[S1-S8]	1.3014	1.2882	1.2396		

Series1	:	1% Tensile Strain Boundary Level at hinge region (S1)
Series2	:	0.3% Tensile Strain Boundary Level outside of hinge region (S2)
Series3	:	Compressive(Crushing) Strain Boundary Level in hinge Region (S3)
Series4	:	Compressive(Crushing) Strain Boundary Level outside of hinge Region (S4)
Series5	:	Beam Rotation; FEMA Near Collapse Boundary Level (S5)
Series6	:	Column Rotation; FEMA Near Collapse Boundary Level (S6)
Series7	:	Wall's Hinge Rotation ; CSA-A23.3-04 (S7)
Series8	:	2.5% Inter-Storey Drift Limit; NBCC 2005 (S8)
Series9	:	Shear in Wall; CSA-A23.3-04 (S9)

Table 5-5: Comparisons of D/C variation over the 15 scaled records; 8 storey building

	[Mean + SD] of D/C for OM Eccentricity			Variation (in %) of Eccentricity	
	0%	5%	10%	5% from 0%	10% from 5%
S1	0.3771	0.4001	0.4239	6.1170	5.9525
S2	0.6684	0.6978	0.7324	4.3915	4.9656
S3	0.5526	0.5677	0.5812	2.7387	2.3792
S4	0.2774	0.2848	0.2932	2.6695	2.9453
S5	0.1943	0.1995	0.2044	2.6458	2.4514
S6	0.1604	0.1619	0.1621	0.9268	0.1375
S7	0.3370	0.3546	0.3723	5.2228	4.9810
S8	0.3244	0.3374	0.3515	3.9984	4.1791
S9	1.4413	1.5256	1.6082	5.8505	5.4165
Min of S1 to S8	0.1604	0.1619	0.1621		
Max of S1 to S8	0.6684	0.6978	0.7324		
S9 / Max[S1-S8]	2.1562	2.1863	2.1957		

	[Mean + SD] of D/C for PAM Eccentricity			Variation (in %) of Eccentricity	
	0%	5%	10%	5% from 0%	10% from 5%
S1	0.5728	0.6028	0.6414	5.2385	6.3976
S2	0.5391	0.5588	0.5818	3.6526	4.1244
S3	0.6737	0.6947	0.7294	3.1153	4.9847
S4	0.2561	0.2632	0.2703	2.7841	2.6781
S5	0.2073	0.2117	0.2166	2.1008	2.3386
S6	0.2112	0.2135	0.2154	1.0918	0.8858
S7	0.4866	0.5104	0.5414	4.8834	6.0744
S8	0.3583	0.3715	0.3899	3.6784	4.9700
S9	1.4599	1.5494	1.6307	6.1303	5.2478
Min of S1 to S8	0.2073	0.2117	0.2154		
Max of S1 to S8	0.6737	0.6947	0.7294		
S9 / Max[S1-S8]	2.1669	2.2302	2.2358		

Series1 : 1% Tensile Strain Boundary Level at hinge region (S1)
Series2 : 0.3% Tensile Strain Boundary Level outside of hinge region (S2)
Series3 : Compressive(Crushing) Strain Boundary Level in hinge Region (S3)
Series4 : Compressive(Crushing) Strain Boundary Level outside of hinge Region (S4)
Series5 : Beam Rotation; FEMA Near Collapse Boundary Level (S5)
Series6 : Column Rotation; FEMA Near Collapse Boundary Level (S6)
Series7 : Wall's Hinge Rotation ; CSA-A23.3-04 (S7)
Series8 : 2.5% Inter-Storey Drift Limit; NBCC 2005 (S8)
Series9 : Shear in Wall; CSA-A23.3-04 (S9)

Table 5-6: Comparisons of D/C variation over the 15 scaled records; 16 storey building

	[Mean + SD] of D/C for OM Eccentricity			Variation (in %) of Eccentricity	
	0%	5%	10%	5% from 0%	10% from 5%
S1	0.2732	0.2914	0.3045	6.6513	4.5229
S2	0.8317	0.8635	0.9042	3.8203	4.7162
S3	0.5125	0.5360	0.5575	4.5688	4.0197
S4	0.3791	0.3881	0.3977	2.3604	2.4772
S5	0.4446	0.4438	0.4393	-0.1720	-1.0186
S6	0.5779	0.5839	0.5868	1.0290	0.4976
S7	0.1643	0.1761	0.1875	7.1638	6.4983
S8	0.2987	0.3101	0.3201	3.8245	3.2236
S9	1.2977	1.3886	1.4689	6.9988	5.7870
Min of S1 to S8	0.1643	0.1761	0.1875		
Max of S1 to S8	0.8317	0.8635	0.9042		
S9 / Max[S1-S8]	1.5603	1.6080	1.6245		

	[Mean + SD] of D/C for PAM Eccentricity			Variation (in %) of Eccentricity	
	0%	5%	10%	5% from 0%	10% from 5%
S1	0.2162	0.2305	0.2409	6.6016	4.4897
S2	0.5215	0.5448	0.5619	4.4661	3.1491
S3	0.4511	0.4710	0.4878	4.4224	3.5655
S4	0.3143	0.3232	0.3289	2.8103	1.7635
S5	0.3778	0.3764	0.3706	-0.3758	-1.5236
S6	0.5150	0.5306	0.5426	3.0263	2.2616
S7	0.1202	0.1304	0.1387	8.4610	6.3872
S8	0.2427	0.2539	0.2619	4.5864	3.1614
S9	1.0860	1.1755	1.2494	8.2474	6.2864
Min of S1 to S8	0.1202	0.1304	0.1387		
Max of S1 to S8	0.5215	0.5448	0.5619		
S9 / Max[S1-S8]	2.0825	2.1579	2.2235		

Series1 : 1% Tensile Strain Boundary Level at hinge region (S1)
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Series8 : 2.5% Inter-Storey Drift Limit; NBCC 2005 (S8)
Series9 : Shear in Wall; CSA-A23.3-04 (S9)

CHAPTER 6

6. SUMMARY, CONCLUSION, AND FUTURE WORK

6.1 Summary

One of the primary objectives of the building design regulatory authorities is avoidance, or reduction of losses from hazards including earthquake. To achieve such goals, the performance level expected from buildings, during and following an earthquake, should be known. However, current building code procedures have been presented to sustain life safety in the major earthquakes and decrease property damage and loss in the moderate ones.

Traditionally, life safety and property loss avoidance have been accomplished by indirect ways through which designer has never really had an evaluation of the performance level of a building. This kind of design methodology may or may not assure the level of damage and loss protection recognized by the owner. To resolve this deficiency, many building code authorities around the world have been working toward establishing a better criterion. The result was creation of Performance-Based Earthquake Engineering that includes Performance-Based Seismic Design (PBSD).

PBSD allows engineers to design buildings with more predictable and reliable levels of performance in the occasion of a seismic activity of a given degree. It seeks to ensure that a building as a whole will perform in some predictable way, in terms of safety and functionality. Progress of computers gave the chance of broadening analysis from static to dynamic, and from linear to non-linear, allowing a more realistic envisions on the condition of structures exposed to, in particular, lateral forces.

In conclusion, The literature survey in this research shows that an immense effort made in order to find new, simplified, accurate and reliable methods to accomplish PBSD of structures. Yet, it seems there is an insufficiency of endeavour in improving and bringing performance level of the existing simplified method to the PBSD level, where the performance level of a building, designed using the simplified method, can be addressed; so that performance level can be scaled up or down to achieve a particular performance objective.

In pursuing to fill such a gap, three sets of RC buildings, four, eight, and sixteen storeys, with shear wall SFRS, and using NBCC 2005 the Equivalent Static Load (ESL) method provisions are modeled, analyzed, and designed. These buildings are also analyzed using the NBCC 2005 dynamic elastic response spectrum method for comparison with the Equivalent Static Load. The buildings are also analyzed by using the inelastic dynamic approach for performance evaluation.

Fifteen actual earthquake records which are scaled and fitted into the design response spectrum defined by the code are used here for dynamic analyses of the buildings.

The Engineering Demand Parameters (EDPs) considered here include inter-storey drift, plastic hinge rotation of shear wall, shear on shear wall, tensile strain values of bars, and compressive (crushing) strain values of the concrete. All values in the assessment part are nominal.

Buildings are designed to attain the "Collapse Prevention" performance level with 2% probability of exceedance in 50 years, or UHS-2500, where inter-storey drift as one of the main damage controlling boundaries is limited to 2.5% for the buildings considered in this work. The results are then assessed.

In the evaluation process, the demand to capacity ratio (D/C values) for Engineering Demand Parameters (EDPs) are illustrated for each individual actual accelerogram that has been scaled up or down to fit into Vancouver's design spectrum. The scaling methods used in this process are Ordinate Method (OM) and Partial Area Method (PAM) as discussed earlier. Mean, mean plus standard deviation, and maximum of D/C values for the selected EDPs over each ensemble of 15 accelerograms are then calculated. Assessment of the processed data shows that demand to capacity ratios for shear in wall is well above one and an unreasonable gap between D/C values for shear in wall and all that of the others exist.

6.2 Conclusions

The present study focuses on the evaluation of buildings designed based on the ESL method. Response spectrum, and nonlinear dynamic analysis have been employed in the design and evaluation process. The factored loads and nominal material strength values are used in the evaluation of the seismic performance. The main conclusion of this study is that the D/C value for shear in wall is well above one and fallen apart from D/C for all other EDPs. This seems to be in agreement with the observation made in the N21.6.9.1 explanatory note on CSA standard A23.3-04 (Cement Association of Canada, 2006). This explanatory note states that there is a need for magnifying the shear strength in wall due to inelastic effects of higher modes. Other findings are as follow:

- Demand to capacity ratio of shear on wall is well above "one" for all buildings studied in this research
- All demand to capacity (or boundary limit) ratios other than "shear on wall" are virtually well bellow "one"; where that provides a clear margin of safety as such boundary of safety is intended to be maintained by using different safety factors in the design process
- There is a clear gap between D/C ratio of "shear on wall" and all of the others; and that is true for all of the buildings and both scaling methods

- Over the range of zero to 10% mass eccentricities, D/C ratios for all the buildings and in all cases vary almost linearly. This would lead to the conclusion that the buildings, within the specified ranges are not torsionally sensitive
- For the 4 and the 8 storey buildings, both ordinate and partial area method of scaling for ground motion records result in nearly equal values, while that is not true for the 16 storey building; such difference could be due to higher mode effects
- The ratio of area beneath the response spectrum curve scaled by OM to that of PAM proves a similar trend as the above mentioned item
- The shear D/C varies in an increasing pace for the 4 and the 8 storey buildings, while it is otherwise for the 16 storey building; and such change could also be the result of higher mode effects
- The shear D/C variation for different mass eccentricities (variation of $D/C_{0\%\ Ecc}$ from that of 5% , or variation of $D/C_{10\%\ Ecc}$ from that of 5%) increases as the building's height increases; that implies that the rotasional stiffness of the building reduces with the increase of building's height
- The base shear resulting from linear dynamic RSA (OM and PAM) are well in tune with that of ESL ($T_{Empirical}$)method
- V_{ESL} ($T_{Empirical}$) to V_{ESL} (Revised) ratio is fairly well close to shear D/C ratios resulting from non-linear dynamic analysis; using this ratio as "wall's shear capacity" modification factor will bring this ratio down close to "one"
- Using magnification factors ranging from 1.3 to 1.55 for the hinge region, in which the critical section is located, reduces the value of D/C to a level below one.
- Such modification as the above is even a cost wise rational modification, since it is equal to a very small portion of the whole expenditure

6.3 Recommendations for future work

This research has been carried out for 3 buildings that provide reasonably assuring results, within this scope of this work.

However, more buildings are required to undergo similar process in order to ascertain a more reliable pattern of correlation of different EDPs in such buildings. Then, a well defined level of performance for buildings that are designed based on ESL method can be achieved. This will allow a modification of the ESL method such that a desired level of seismic performance can be achieved.

7. REFERENCES

ATC 3-06, Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC Publication ATC 3-06, Applied Technology Council, NBS Special Publication 510, NSF Publication 78-8, U.S. Government Printing Office, Washington, DC, 1978.

Bagchi A., 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.

Bergeron, D., Desserud, R.J., Haysom, J.C., 2004, The Origin and development of Canada's objective-based codes concept, NRCC-47034.

Chopra A. K. and Goel R. K., 2001, A Modal Pushover Analysis Procedure to Estimate Seismic Demands for Buildings: Theory and Preliminary Evaluation. PEER, Pacific Earthquake Engineering Research Center.

Chopra A. K. and Goel R. K., 1999, Capacity-Demand-Diagram methods for estimating seismic deformation of inelastic structures: SDF systems, Report No. PEER-1999/02.

Cement Association of Canada, 2006, Explanatory Notes on CSA Standard A23.3-04

CSI, COMPUTER & STRUCTURES INC, 2008

Earthquakes Canada, 2008, Citing online sources: Natural Resources Canada. Available from http://earthquakescanada.nrcan.gc.ca/historic_eq/20th/signif_e.php [Cited Aug 24, 2008].

FEMA 349, 2000, Action plan for performance based seismic, Federal Emergency Management Agency, Washington D.C., USA.

FEMA 440, 2005, Improvement of Nonlinear Static Seismic Analysis Procedures, Federal Emergency Management Agency, Washington D.C., USA.

FEMA 445, 2006, Next-Generation Performance-Based Seismic Design Guidelines, Federal Emergency Management Agency, Washington D.C., USA.

Ghosh S. K., 2004, Update on the NEHRP provisions: The resource document for seismic design, PCI JOURNAL.

Goel R. K, and Chopra A. K., 2001, Improved direct displacement-based design procedure for performance-based seismic design of structures, Conference Proceeding Paper, Part of Structures — A Structural Engineering Odyssey, section 22, chapter 4, ASCE.

Hamburger R. O., 1997, A framework for performance-based earthquake resistive design. Citing online sources: nisee; National Information Service for Earthquake Engineering; University of California, Berkeley, Available from http://nisee.berkeley.edu/lessons/hamburger.html [Cited Oct 11, 2008].

Heidebrecht A. C. and Lu C.Y., 1988, Evaluation of the seismic response factor, Canadian Journal of Civil Engineering, Vol. 16, pp. 382-338.

History of Earthquake Engineering, 2008, Citing online sources: Well come to California. Available from http://wwwoandm.water.ca.gov/earthquake/about/history_index.cfm [Cited Nov 20, 2008].

Humar J.L. and Kumar P., 2004, Review of code provisions to account for earthquake induced torsion, 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada.

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

Humar J.L. and Kumar P., 1998, [1] Torsional motion of buildings during earthquakes. I. Elastic response; [2] Torsional motion of buildings during earthquakes. II. Inelastic response, Canadian Journal of Civil Engineering.

Humar J., Yavari, S. and Murat Saatcioglu, M., 2003, Design for forces induced by seismic torsion. NRC Canada.

Mousavi S., Bagchi A., and Kodur V. K.R., 2008, Review of post-earthquake fire hazard to building structures, Canadian Journal of Civil Engineering, Volume 35, pp. 689-698.

NEHRP, 1985, Recommended Provisions for the Development of Seismic Regulations for New Buildings, Building Seismic Safety Council, Washington, DC.

Nathan N. D., 1975, Rotational components of earthquake motion, Department of Civil Engineering, University of British Columbia, Vancouver, British Columbia, Canada.

Naumoski, N., Tso, W.K. and Heidebrecht, A.C., 1988, A selection of representative strong motion earthquake records having different A/V ratios, Earthquake Engineering Research Group, McMaster University, Canada

PEER, 2008, Citing online sources: Pacific Earthquake Engineering Research Center, Available from http://74.125.95.132/search?q=cache:RZ232ihY8ZgJ:www.peertestbeds.net/+peer+structural +engineers+used+allowable-stress+design+(ASD)+and+load-and-resistance-factor+design+(LRFD),+which+focus+on&hl=en&ct=clnk&cd=1&gl=ca [Cited Nov 15, 2008].

PEER, 2008, Citing online sources: Moving Toward Performance-based Engineering. Available from*http://peer.berkeley.edu/course_modules/eqrd/index.htm?c227top.htm&227cont.htm&DesPhil/desphil5.html [Cited Nov 18, 2008].

PREPARE FOR EARTHQUAKES, 2008, Citing online sources: PREPARE FOR EARTHQUAKES Available from http://www.prepareforearthquakes.com/ [Cited Nov 15, 2008].

SEAOC Vision 2000, 1995, Performance-based seismic engineering, Structural Engineers Association of California, Sacramento, CA.

Vamvatsikos D. and Cornell C. A., 2002, Incremental dynamic analysis, Earthquake Engineering & Structural Dynamics, Volume 31, Issue 3, pp. 491-514.

Veletsos, A.S. and Newmark, N.M. (1960). "Effect of Inelastic Behavior on the Response of Simple Systems to Earthquake Motions", Proceedings of the Second World Conference on Earthquake Engineering, Tokyo, Japan, Vol. II, pp. 859–912.