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OVERSTRENGTH FACTOR AND DUCTILITY FACTOR

FOR SEISMIC DESIGN OF RC BUILDINGS

By

Sagun Kandel

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The undersigned certify that they have read, and recommended to the Institute of Engineering for acceptance, a thesis entitled "**Overstrength Factor and Ductility Factor for Seismic Design of RC Buildings**" submitted by Mr. Sagun Kandel (075/MSStE/017) in partial fulfillment of requirement for the degree of Master of Science in Structural Engineering.

Supervisor, Assoc. Prof. Dr. Rajan Suwal

Department of Civil Engineering

Institute of Engineering

Pulchowk Campus

External Examiner, Dr. Suman Narsingh Rajbhandari

Program Coordinator, Prof. Dr. Kamal Bahadur Thapa

Department of Civil Engineering

Institute of Engineering

Pulchowk Campus

Date:

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ABSTRACT

It is of utmost importance to design a structure such that they are economical and also has adequate strength to resist the loads applied on them. Due to this reason, the design lateral strength in most of the design codes including NBC 105: 2020 is lowered from the required elastic lateral strength by the combination of overstrength factor (Ω_u) and ductility factor (R_μ) resulting in a smaller member section. The structural member sizes govern the time period and drift of the structure on which the overstrength factor (Ω_u) and ductility factor (R_μ) is dependent. The total number of 36 configurations of low-rise building configurations most common in Nepal is selected and each building is analysed with two different structural member sizes. The NBC 105:2020 is selected for the seismic design of RC buildings and non-linear analysis is performed using a provision in FEMA 356:2000. The results indicated that the change in building configuration and structural member sizes affects the overstrength factor (Ω_u) and ductility factor (R_μ).

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LIST OF ABBREVIATIONS

2D	Two Dimensional
3D	Three Dimensional
ADB	Asian Development Bank
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
C/B	Column Beam Capacity Ratio
CLPIU	Central Level Project Implementation Unit
CQC	Complete Quadratic Combination
DUDBC	Department of Urban Development and Building Construction
EEAP	Earthquake Emergency Assistance Project
ETABS	Extended Three-Dimensional Analysis of Building System
FEMA	Federal Emergency Management Agency
HYSD	High Yielding Strength Deformed Steel
IS	Indian Standard
MoUD	Ministry of Urban Development
NBC	Nepal Building Code
NEHRP	National Earthquake Hazards Reduction Program
OMRF	Ordinary Moment Resisting Frame
RC	Reinforced Concrete
SAP	Structural Analysis and Design
SC	Local Soil Condition
SMRF	Special Moment Resisting Frame
SRSS	Square Root of Sum of Square
SSI	Soil Structure Interaction
TMT	Thermo Mechanically Treated

LIST OF SYMBOLS

μ	Displacement Ductility Ratio
B	Number of Bays
B	Width of Member
BL	Bay Length
C	Basic Seismic Coefficient for Fundamental Translational Period
$C(T_1)$	Elastic Site Spectra
C_d	Design Horizontal Seismic Force Coefficient
$C_d(T_1)$	Horizontal Base Shear Coefficient
$C_h(T)$	Spectral shape factor
CP	Collapse Prevention
$C_s(T1)$	Serviceability Elastic Site Spectra
D	Depth of Member
d	Effective Depth
DL	Dead Load
d_u	Maximum Displacement
d_y	Yield Displacement
E	Earthquake Load
H	Height of the Building
I	Importance Factor
IO	Immediate Occupancy
K	Structural Performance Factor
LL	Live Load
LS	Life Safety
ϕ	Function of Soil Condition

R	Response Reduction Factor
R_μ	Ductility Reduction Factor / Ductility Factor for Ultimate Limit State
R_r	Redundancy Factor
R_s	Ductility Factor for Serviceability Limit State
R_ζ	Viscous Damping Factor
S	Number of Storeys
T	Time Period of Vibration
T_g	Predominant Period of Ground Motion
V_d	Design Lateral Force
V_e	Maximum Elastic Lateral Force
V_y	Yield Base Shear
W	Seismic Weight
Z	Seismic Zoning Factor
λ	Live Load Participation Factor
Ω	Overstrength Factor
Ω_s	Overstrength factor for Serviceability Limit State
Ω_u	Overstrength factor for Ultimate Limit State

CHAPTER-1 INTRODUCTION

1.1 Background

Reinforced concrete (RC), is a composite building material of structural concrete reinforced with steel bars / rebars. Concrete is relatively strong in compression but has low tensile strength and ductility which is compensated for by the addition of reinforcement. RC buildings are structures in which members resisting horizontal and vertical forces are made up of reinforced concrete. According to latest census data of Nepal out of total 54,23,297 buildings, 5,39,004 of them are RC structure which accounts for around 9.94%. This value is increasing rapidly as now people are aware of the benefit of RC structure in resisting earthquake over load-bearing masonry structure. The RC buildings have been used for a variety of purposes such as commercial buildings, residential buildings, storage facilities, factories, and many more.

Earthquake is the natural phenomena caused by release of seismic wave from the earth surface from a faint tremor to a wild motion due to sudden release of energy stored in the rocks beneath the earth surface. It's as old as earths' history itself, however, our understanding and interpretations about their behavior & ways to reduce damages due to them is recent. Most of the earthquake are minor and go un-noticed but the major ones, though occasional are responsible for huge loss of life and property.

The theoretical development of earthquake forces in structure reveals that the maximum elastic response acceleration during earthquake (range for which structure is designed) would be several times larger than the design acceleration i.e., the seismic coefficient specified in most of the codes. This is opposite to the fact that design loads in codes are usually higher than the actual anticipated load. It is based on the probability that the occurrence of large earthquakes is quite rare and the capacity of the structure to absorb energy.

It is very important to design a structure such that they are economical and also has an adequate strength to resist the loads applied on them. Most of the design codes makes use of the design philosophy that total safety and no damage, even in an earthquake with a reasonable probability of occurrence, cannot be attained. Allowing some nonstructural as well as structural damage can still have a high level of life safety making the structure economical. Due to this reason, the design lateral strength in most seismic codes is lower than the required elastic lateral strength. Utilization of inelastic behavior of the structure helps in reducing the lateral force to be resisted by the structure hence reducing the member sizes and finally reducing the cost of construction. A term referred to as Force reduction factor or Response reduction factor is used in most of the seismic design codes to reduce the elastic lateral force to a design lateral force.

1.2 Need of Research

The force reduction factor in most of the seismic design codes make use of a constant pre-determined value mostly depending just upon structural type and the detailing procedure for seismic analysis and design. The NBC 105:2020 has a single value for overstrength factor and ductility factor on behalf of response reduction, both of which are governed by the type of structural system. This may not be justified as it has been found that it depends upon various parameter such as building configuration, number of storey and also on structure member sizes. Very few researches account for their effect on overstrength factor and ductility factor in RC buildings. Therefore, it is essential to study the real behavior of RC buildings through non-linear analysis to assess the value of these factors considering different structure member sizes and geometry of the structure in the context of Nepal.

1.3 Objective of Research

The work presented in this thesis intends to contribute to the development of revised force- based design guidelines for the next generation of seismic design codes in Nepal. The main aim of this study is to evaluate the overstrength factor and ductility factor for RC buildings carrying out non-linear static analysis to assess the reduction factor based on the outcomes given by past studies and code provisions.

Specific objectives of the study are:

1. To identify the effect on the overstrength factor and ductility factor for various configuration of structures.

2. To evaluate the overstrength factor and ductility factor for structures with different structural member sizes.
3. To propose an empirical formula to obtain overstrength factor and ductility factor.

1.4 Methodology

The methodology worked out to achieve the above-mentioned objectives is as follows:

1. Definition of the problem related to the RC framed structures.
2. Need of the study presents the problems with the current methods adopted for design of structures.
3. The research objectives are set to address the problems as stated in the need for study.
4. Review of existing literatures and seismic design code provision for designing buildings are studied.
5. Selection of different type of RC framed buildings. The parameters of interest will include number of storeys, number of bays, and different structure member sizes.
6. Modeling of frames and using ETABS2020v19. The structure will be designed based on seismic design code NBC 105: 2020.
7. Nonlinear Static analysis will be done to obtain the force - displacement curve. A bilinear curve will be prepared from the obtained force - displacement curve and this will be used to obtain the ductility factor and overstrength factor.
8. Conclusion, discussion and recommendation are made based upon the results.

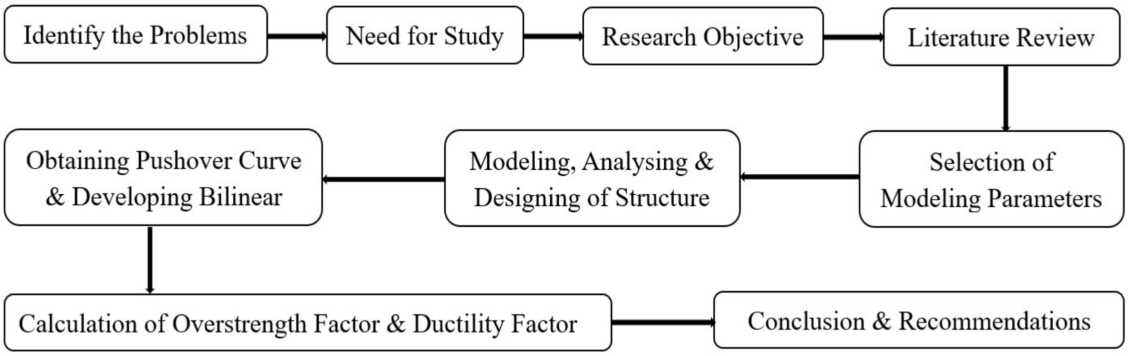

Figure 1-1: Methodology Flowchart

1.5 Thesis Organization

This thesis work has been mainly organized in six chapters.

Chapter 1 is an introductory chapter which gives brief introduction about the force reduction factors and RC buildings. Also, the need of study, objective of research and methodology to be followed to fulfill the research objective is presented.

Chapter 2 presents an extensive literature review on overstrength factor and ductility factor. This also include the review of code provisions for force reduction factor namely overstrength factor and ductility factor in different seismic codes.

Chapter 3 describes the theoretical background of response reduction factor and its components. This chapter also includes the theoretical formulation given by different researchers and one that is used in this study. Structural analysis procedures mainly focusing on nonlinear static analysis and steps for bilinearization of pushover curve is also discussed in detail.

Chapter 4 contains the different structural models used in this study along with their designated nomenclature. This chapter also present the materials and member's section used in the structural models and the assumptions made in the study.

Chapter 5 presents the results obtained from the nonlinear pushover analysis of the building models. The results are evaluated and compared to find the influence of different parameters on overstrength factor and ductility factor. It also presents a sample calculation of the results.

Chapter 6 includes the conclusions drawn from this research. This chapter also discusses about the scope for future work.

CHAPTER-2 REVIEW OF LITERATURE AND CODE PROVISIONS

2.1 Overview

Many research articles related to the explanation and variation of overstrength factor and ductility factor are studied. This chapter presents a brief summary of the literature review.

2.2 Literature Review

Literature review is carried out as a part of this study to gain insight on the works to be carried out and to act as a guide for successful completion of this thesis work. The problems related with this work was identified and necessary reference were taken from the literatures shown below:

Subarna Lal Dangol (2020) assessed the ductility and overstrength factor for mid-rise RC Buildings. The design was carried out by using NBC 105-1994. The buildings with different plan irregularities, storey height and bay length were considered. The results indicated that ductility reduction factor was more for regular plan building with larger floor height but decreased with the increase with grid size. Also, the over strength factor increased with smaller floor height and larger grid size, while there was no significant effect with plan irregularities

Tekkan Pandit (2020) evaluated response reduction factor of existing masonry infilled RC buildings common to Pokhara. Those real field buildings were selected on the basis of changes in construction pattern before and after Gorkha earthquake. The results indicated changes in response reduction factor due to the presence of infill masonry wall and creating opening in the infill masonry wall decreased the overstrength factor. The natural time period was higher in the bare frame model than in the model with infill walls. But, with respect to base shear the result was inversed i.e., higher in the model with infill walls than in the bare frame model by 1.2 to 2.2 times.

Prayush Rajbhandari (2020) carried out nonlinear static analysis in steel framed 2-D building considering infill wall with various configuration of number of bays, number of storeys, and material of infill wall and the study was conducted on these buildings only. The overstrength factor increased and ductility factor decreased with the increase in infill strength. Also, number of storeys and number of bays affected the overstrength factor and ductility factor significantly. An empirical formula has been proposed to obtain the overstrength factor and ductility factor considering the infill wall.

Priyana Rajbhandari (2019) carried out nonlinear static pushover analysis of RC building with various configuration. The bay size, number of storeys, and number of bays were the varying parameters considered and all the other parameters were kept same. The infill walls were not considered during the analysis. The result indicated that the factors do not show significant changed with number of bays but dependency of the factors on the span is not same for buildings of all storey. The ductility factors did not show much variation but the over strength factor varied significantly while changing building configuration. At the end an empirical formula has been proposed to obtain the overstrength factor and ductility factor. Indian Standard code has been used for the design of building and calculation of response reduction factor. Same and large column and beam sizes has been used for every model from 6 storey to 3 storey which over predicts the overstrength factor but reduces the value for ductility factor.

Nishanth et al. (2017) assessed the seismic response reduction factor for moment resisting RC frames. The nonlinear static pushover analysis of 2D framed structures of both ductile and ordinary moment resisting frames was evaluated to the actual values of the response reduction factor. A total of 5 different number of storey buildings were taken varying from G+3 to G+15 and also the zone factors listed in IS code 1893:2016 were considered i.e., zone II to zone V. The effect of geometric non-linearity of the structure was also considered in the analysis. They found that the time period of the structure highly affected the values of over-strength, ductility and response reduction factors but the effect of seismic zone was only seen on overstrength factor and the results also indicated that the ductile buildings had higher overstrength values than ordinary moment resisting frames.

Brahmavrathan and Arunkumar (2016) carried out nonlinear static pushover analysis to evaluate the response reduction factor of irregular reinforced concrete framed structures. The 3D OMRF and SMRF buildings having varying number of storeys i.e., 3, 6 and 9 storeys were analysed using finite element analysis software

SAP 2000. The results indicated that the actual value of R obtained after the analysis was less than that assumed during the design process and also the value of response reduction factor decreased with the increase in the number of storeys.

H. Chaulagain et al. (2015) published a paper on seismic response of RC buildings in Kathmandu valley. The house survey was done in 10 districts and a total of 300 houses were surveyed out of which 200 houses were taken for the study and were classified under various topics. Some of the conclusions were that engineered structures has higher strength and lower deformation where as non-engineered buildings in Nepal exhibited high vulnerability with low ductility.

Hemchandra Chaulagain (2010) assessed the response reduction factor of RC buildings using non-linear pushover analysis. The 12 engineered designed representing a majority of RC buildings in Kathmandu were taken for the analysis. He concluded that most of the buildings has R value less than 5 and also response reduction factor changes significantly with changes in C/B ratio.

Kadid and A. Boumrkik (2008) evaluated the performance of RC buildings under future expected earthquake. A non-linear pushover analysis was conducted with buildings of stories 5, 8, and 12. He concluded that the pushover analysis is a faster and simple way to get the insight about the nonlinear behaviour of the building. He also added that the quality of material and failure to meet the strong column weak beam criteria caused the failure of RC buildings in Algeria by the Boumerdes earthquake.

Mahmoudi (2003) investigated the relationship between overstrength and member ductility of RC moment resisting frames. The building having one, two, three, four, five, six, eight, ten and fifteen stories with three spans were analysed. The results indicated that the overstrength is influenced by member ductility greatly and higher buildings have the lower overstrength. He also added that the effect of column ductility factors on overstrength factor is higher than that of beam ductility factor

Elnashai et al. (2002) published a paper to addresses the issue of overstrength in modern code designed RC buildings. The nonlinear static pushover analysis and time history analysis for twelve buildings of various characteristics were carried out. He concluded that the nonstructural elements contribute to produce higher overstrength in the building. He also stated that buildings designed for low seismic intensity level has higher overstrength factor and the actual values of overstrength factor during the earthquake should be higher than the values obtained from inelastic pushover analysis.

Barakat et al. (1997) carried out seismic nonlinear time-history analysis three-dimensional G+3, G+5, and G+7 storey RC buildings. These buildings had shear walls in both orthogonal directions. The number of bays and bay sizes also differed in both orthogonal directions. The code of practice for design was the Jordanian Seismic Code and seismic zones were varied from zones 4, 3, 2, to 1. The El Centro (N-S) earthquake record of May 1940 as an actual earthquake excitation was used for time-history analysis. It was observed that the seismic zoning has a slight effect on the ductility reduction factor for different buildings and the value of ductility reduction factor was almost same as displacement ductility ratio. The overstrength factor was found to vary with number of stories, seismic zones, and design gravity loads. However, seismic zones affected the overstrength most. The overstrength decreased as the number of storeys increased. The variation in response reduction factor has a significant implications for the seismic design codes which currently does not account for it.

Humar J. l. and Rahgozar M. A. (1996) published a paper to establish a concept of overstrength in seismic design. A static nonlinear pushover analysis was carried in the moment resisting steel building frames from G+1 to G+29. The result concluded that the building designed using a current seismic code possess considerable reserve strength. He also highlighted the sources contributing to the reserve strength in the buildings i.e., serviceability criteria, actual vs nominal material strength, discrete member sizes, code-based strength, presence of non-structural members etc.

Jain and Navin (1995) assessed the seismic overstrength of multi-storey reinforced concrete frames building. The buildings selected had four bays each and storeys were varied as G+2, G+5, and G+8 storey and designed for seismic zones I to V as per Indian codes. Their result suggested that the design of low-rise buildings and buildings in lower seismic zones are more conservative. A very large variation was seen in the overstrength of RC frame buildings, and this has important implications for the seismic design codes. He also stated that significant research efforts must be carried out with the ultimate aim to account for the variation in the overstrength in an explicit manner for the evaluation of design seismic force on buildings.

Eduardo Miranda (1993) evaluated the site dependent strength reduction factor that are used to reduce elastic design spectra to account for the hysteretic energy dissipation of the structure. A total of 124 earthquake ground motion which were recorded on 3 different soil conditions i.e., rock soil sites (38 records), alluvium soil sites (62 records), and soft soil sites (24 records). After carrying out the regression analysis the equation for ductility reduction factor (R_μ) was computed assuming 5% critical damping. The findings also included that soil condition also greatly affects the mean strength reduction factor. The total of 3 different equations were proposed for 3 different soil conditions which depend upon displacement ductility ratio (μ) and period of vibration (T).

Chia-Ming Uang (1991) published a paper to establish response modification factor (R) and the displacement modification factor (C_d) used in National Earthquake Hazards Reduction Program (NEHRP) recommended provisions. These expressions depend on the structural overstrength and structural ductility factors. Result indicated that the value of overstrength factor can be higher for building structure with less than four storeys and also for buildings lying in low seismic zones because gravity loads are more likely to govern the design.

Newmark and Hall (1983) carried out a study to determine the ductility reduction factor in which the expression depending upon the displacement ductility ratio and the dominant vibration period (T) of the structure was proposed. They stated that for a relatively long period structure, T greater than 1.0 seconds, the value of ductility reduction factor equals to displacement ductility ratio, as the peak displacement from the inertia force obtained from an elastic system and reduced inertia force obtained from inelastic system are the same. For a structure of natural period less than 0.03 seconds, the ductility does not help in reducing the response of the structure as the structure becomes very rigid and therefore, no ductility reduction factor should be used in such case. For moderate period structure, T in between 0.12 and 0.5 seconds, equal energy concept can be applied i.e., the energy that can be stored by the elastic system at maximum displacement is the same as that stored by an inelastic system.

2.3 Research Gap

All the previous research done, were carried out using code other than NBC 105:2020 for the seismic design of RC buildings. The NBC 105:2020 is an updated version of NBC 105:1994. The updating of NBC 105:1994 was initiated after the 2015 Gorkha earthquake by the Department of Urban Development and Building Construction (DUDBC) under the initiative of Central Level Project Implementation Unit (CLPIU) of the ADB financed Earthquake Emergency Assistance Project (EEAP) under Ministry of Urban Development (MoUD). The final version of NBC 105:2020 was published recently in 2020, so not much research has been done on it.

Most of the research has been carried out on the mid-rise buildings (7 – 12 storeys) or higher which are very less in number in Nepal compared to low-rise buildings (1 – 6 storeys). The building configurations were selected randomly without proper representation. The residential buildings of Nepal are mostly low-rise buildings which this research is focused on. Also, some of the previous researches, for simplicity has adopted same column and beam sizes for all the models, making the model unrealistic to the real field. The changes in the overstrength factor and ductility factor brought upon by the changes in structural member sizes and building configuration has also not been thoroughly researched in the past.

2.4 Review of Codes

Each seismic design code of building for different countries has their own recommendation for force reduction factor. Here an overview of Nepal National Building Code NBC 105: 1994 and NBC 105: 2020 for seismic load calculation is summarized.

NBC 105: 1994 provides an equation to calculate the design horizontal seismic force coefficient, C_d for the seismic design of buildings in Nepal as

$$C_d = CZIK \tag{2.1}$$

Where,

K = is the structural performance factor whose value depends on minimum detailing requirement for given structure type. Code specifies the value of K in the range of 1 to 4.

Z = Seismic Zoning Factor

I = Importance Factor

C = Basic seismic coefficient for fundamental translational period in the direction under consideration.

Table 2-1: Structural Performance Factor Prescribed by NBC 105:1994

Item	Structural Type	Structural Performance Factor K
1.(a)	Ductile Moment Resisting Frame	1.0
1.(b)	Frame as in 1(a) with reinforced concrete shear walls	1.0[1]
2.(a)	Frame as in 1(a) with either steel bracing members detailed for ductility or reinforced concrete infill panels	1.5[1,2]
2.(b)	Frame as in 1(a) with masonry infills	2.0[1,2]
3.	Diagonally-braced steel frame with ductile bracing acting in tension only	2.0
4.	Cable-stayed chimneys	3.0
5.	Structural of minimal ductility including reinforced concrete frames not covered by 1 and 2 above and masonry bearing wall structures	4.0

Notes:

1. These factors shall apply only if the steel bracing members, shear walls and / or the infill panels are taken into consideration in both the stiffness and lateral strength calculations.
2. These factors shall apply only if the frame acting alone is capable of resisting at least 25 percent of the design seismic forces.

The structural performance factor (K) as stated in NBC 105:1994 depends upon the structural ductility. It magnifies the design horizontal seismic force coefficient (C_d) but has a similar meaning to the force reduction factor in a sense that higher ductile structure has lower value of structural performance factor (K) and lower ductile structure has higher value of structural performance factor (K).

The NBC 105: 2020 provides the set of equations to calculate horizontal base shear coefficient which is quite similar to NBC 105: 1994 however it removes the structural performance factor and replaces it with the overstrength factor (Ω) and ductility factor (R_μ). The NBC 105: 2020 has two different states to calculate the horizontal base shear coefficient, $C_d(T_1)$. One of them is an ultimate limit state while other is a serviceability limit state.

The horizontal base shear coefficient, $C_d(T_1)$ is given by;

$$C_d(T_1) = \frac{C(T_1)}{R\mu \; x \; \Omega u} \quad \text{(For Ultimate Limit State)} \tag{2.2}$$

$$C_d(T_1) = \frac{Cs(T_1)}{\Omega s} \quad \text{(For Serviceability Limit State)} \tag{2.3}$$

Where,

$C(T_1)$ = Elastic Site Spectra given by $C_h(T)ZI$

Z = Seismic Zoning Factor

I = Importance Factor

R_μ = Ductility Factor

Ω_u = Overstrength factor for Ultimate Limit State

Ω_s = Overstrength factor for Serviceability Limit State

Table 2-2: Ductility and Overstrength Factors Prescribed by NBC 105:2020

S.No.	Structural System	R_μ	Ω_u	Ω_s
Moment Resisting Frame System				
1.	Steel Moment Resisting Frame	4	1.5	1.25
2.	RC Moment Resisting Frame	4	1.5	1.25
3.	Steel + RC Moment Resisting Frame	4	1.5	1.25
4.	Steel Eccentrically Braced Frame	4	1.5	1.25
5.	Steel + RC Eccentrically Braced Frame	4	1.5	1.25

6.	Steel Concentric Braced Frame	3	1.3	1.15
7.	Steel + RC Concentric Braced Frame	3	1.3	1.15
8.	Steel Buckling Restraint Braces	4	1.5	1.25
	Structural Wall Systems			
9.	RC Shear Wall	3	1.3	1.15
10.	Steel + RC Composite Shear Wall	3	1.3	1.15
11.	Reinforced Masonry Shear Wall	2.5	1.2	1.1
12.	Confined Masonry Wall	2.5	1.2	1.1
13.	Unreinforced Masonry Wall Buildings	2.0	1.2	1.1
	Dual Systems			
14.	Steel Eccentrically Braced Frame	4	1.5	1.25
15.	Steel + RC Composite Eccentrically Braced Frame	4	1.5	1.25
16.	Steel Concentric Braced Frame	3.5	1.4	1.2
17.	Steel + RC Composite Concentric Braced Frame	3.5	1.4	1.2
18.	Steel Buckling Restraint Braces	4	1.5	1.25
19.	RC Shear Wall	3.5	1.4	1.2
20.	Steel + RC Composite Shear Wall	3.5	1.4	1.2
21.	Reinforced Masonry Shear Wall	2.5	1.2	1.1

The Ductility factor (R_s) for serviceability limit state is taken as 1.

The NBC 105:2020 is an improvement over the NBC 105:1994 as it has a provision for both the overstrength factor and ductility factor and has provided those values for more than 20 structural types but has omitted the provision for the frames with masonry infill.

Here is a table of force reduction factors as recommended by the seismic design code of buildings of different countries.

Table 2-3: Provisions for Seismic Coefficient and Force Reduction Factor

Country	Code	Seismic Coefficient	Reduction Factor	Dependency
Nepal	NBC 105:2020	Ultimate Limit State $C_d(T_1) = \frac{C(T_1)}{R_\mu\ \Omega_\mu}$	Overstrength Factor (Ω) and	Ω, μ

		Serviceability Limit State $C_d(T_1) = \frac{C(T_1)}{\Omega_s}$	Ductility Factor (μ)	
Nepal	NBC 105:1994	$C_d = CZIK$	Structural Performance Factor (K)	μ
India	IS 1893:2016 (Part 1)	$A_h = \frac{\left(\frac{S_a}{g}\right)\left(\frac{Z}{2}\right)}{\left(\frac{R}{I}\right)}$	Response Reduction Factor (R)	Ω, μ, ζ
Europe	BS EN 1998-1:2004	$S_d(T_1) = a_g S \frac{2.5}{q}$	Behavior Factor (q)	Ω, μ, ζ
New Zealand	NZS 1170.5:2004	$C_d(T_1) = \frac{C(T_1)S_p}{k_\mu}$	Inelastic Spectrum Scaling Factor (k_μ) and Structural Performance Factor (S_p)	Ω, μ, ζ
Bangladesh	BNBC-2015	$S_a = \frac{2}{3}\frac{ZC_s}{\left(\frac{R}{I}\right)}$	Response Reduction Factor (R)	Ω, μ, ζ
Pakistan	Building Code of Pakistan	$\frac{C_v I}{RT}$	Numerical Coefficient (R)	Ω, μ, ζ
U.S.A	IBC 2015, ASCE-7 2016	$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$	Response Modification Factor (R)	Ω, μ, ζ

CHAPTER-3 THEORETICAL FORMULATION

3.1 Response Reduction Factor

Response reduction factor is a ratio of maximum lateral force, (V_e) which would develop in a structure if it were to remain entirely linear elastic under the specified ground motion to the design lateral force, (V_d) which it has been designed to withstand. The structures are designed for the much less seismic forces as it uses the concept of nonlinear response of a structure to reduce the design force acting on the structure.

$$R= \frac{V_e}{V_d} \tag{3.1}$$

In the mid-1980s, research in University of California in Berkely described a response reduction factor, (R) as the product of three factors that included reserve strength, ductility and added viscous damping. The overstrength factor, (Ω) accounts for the reserve strength, ductility factor, (R_μ) takes into consideration of ductility of the structure while viscous damping factor, (R_ζ) considers the added viscous damping. So, mathematically (R) can be expressed as:

$$R = \Omega * R_\mu * R_\zeta \tag{3.2}$$

Where, the equation (3.2) can be expressed in term of equation (3.1) as:

$$R = \frac{V_y}{V_d} * \frac{V_e}{V_y} * 1 \tag{3.3}$$

In above equation, the overstrength factor, (Ω) is the ratio of maximum base shear at the yield level, (V_y). to the design base shear, (V_d) and ductility factor, (R_μ) is the ratio of maximum lateral force, (V_e) which would develop in a structure if it were to remain entirely linear elastic to the yield base shear, (V_y). The damping factor, (R_ζ) is usually taken as 1 unless the structures have added damping devices.

Later, Applied Technology Council re-defined R and expressed it as the product of 3 main factors i.e.,

$$R = \Omega * R_\mu * R_r \tag{3.4}$$

Where, damping factor (R_ζ) was excluded because viscous damping factor may be used to reduce displacements in nonlinear framing system, but cannot be used to proportionally reduce force demands. Since the seismic design code is force based and uses response reduction factor (R), the damping factor was not considered going forward. Redundancy factor (R_r) was introduced to accounts for the reliability of seismic framing systems which should be composed of multiple vertical lines of seismic framing in each principal direction of a building.

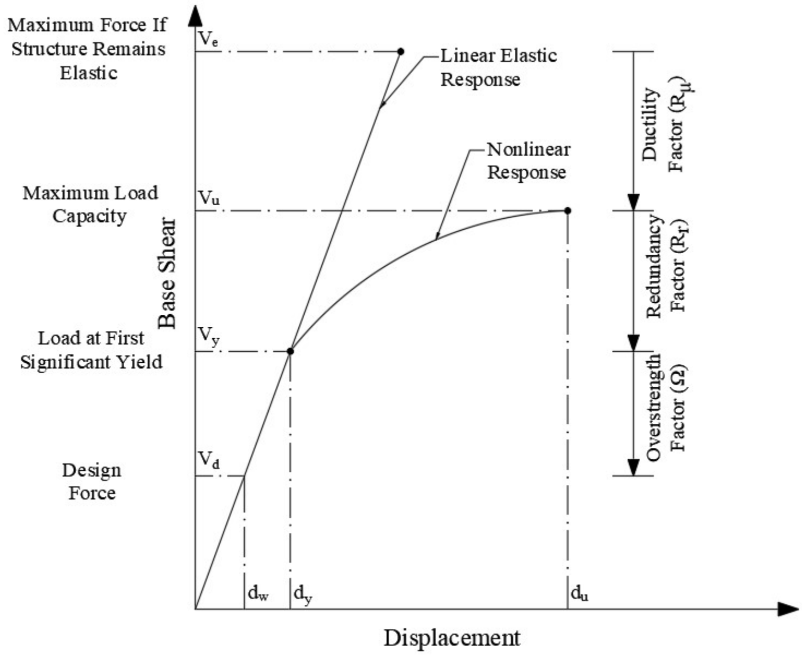

Figure 3-1: Response Reduction Factor and its Co-Factors

3.2 Overstrength Factor

The structures are able to endure larger earthquake than those they were designed for without considerably damage. This can be explained by the presence of significant additional strength beyond the design strength. Overstrength factor is denoted by Ω and is the ratio of V_y and V_d.

$$\Omega = \frac{V_y}{V_d} \tag{3.5}$$

Where V_y is the significant yield strength which is a point on the capacity curve where significant change in slope occurs and is not the point where first yielding occurs. Rather it is defined as the stage of complete plastification of at least the most critical region of the structure. The point in the capacity curve where the change in slope occurs can be located by idealizing the capacity curve to bilinear curve. There are many reasons that accounts for the higher strength in the structures than what it is designed for. But some factor can actually reduce the structural strength. Some of the factors that affect the structural overstrength are listed below.

1. Code prescribed minimum requirement: While designing for the structural member sizes, code-based design may result in smaller member sizes than the minimum requirement suggested by the code. For low rise residential building in lower seismic hazard zone this is the main contribution of higher strength in the structure.
2. Actual material strength: The design material strengths are reduced by a certain safety factor during the code-based design process but in reality, the strengths are much higher.
3. Serviceability criteria: The deflection control criteria suggested by the code may govern the selection of member sizes. Even though the structure members have sufficient strength the resist the load applied, the structure might still deflect higher than that allowed by the code. In this case, the higher structural member sizes are required.
4. Discrete member sizes: The available member sizes in the market compels the use of higher section sizes which affects the local over strength. This is majorly seen during the selection of reinforcement diameter, as only the certain sizes of reinforcement are available in the market.
5. Non-structural member: The addition of non-structural members such as walls contribute to the lateral stiffness of the structure which in most case is not included in the design process.

Some factors unintentionally contribute to reducing the strength of the structure. Deterioration, short column, soft storey, disturbance in load path, poor workmanship, etc. can reduce the structural strength.

3.3 Ductility Factor

Ductility is a property of structural members or structure to undergo large inelastic deformation without significant loss of strength prior to failure. The ductility of the structure determines its ability to withstand large lateral displacement imposed by serve earthquake, as during serve earthquake the structures cross their elastic limit and reach the inelastic region. The ductility reduction factor (R_μ) can be defined as the ratio of maximum lateral force, (V_e) which would develop in a structure if it were to remain entirely linear elastic ($\mu = 1$) under the specified ground motion to the idealized yield strength (V_y) of the structure.

$$R_\mu = \frac{V_e}{V_y} \tag{3.6}$$

The displacement ductility ratio (μ) mainly governs the ductility factor (R_μ) and are used in conjunction which is the ratio of maximum absolute displacement (d_u) to the yield displacement (d_y) and measures the level of inelastic deformation.

$$\mu = \frac{d_u}{d_y} \tag{3.7}$$

The ductility factor (R_μ) gets affected mainly by displacement ductility ratio (μ) but other factors also influence it i.e., period of vibration (T) and local soil condition (SC). So, ductility reduction factor (R_μ) can be written as a function of displacement ductility ratio (μ), period of vibration (T), and local soil condition (SC).

$$R_\mu = f\,(\mu, T, SC) \tag{3.8}$$

Also, some of the other condition of ductility factor (R_μ) are as follows: -

$$\lim_{T \to 0} f(\mu, T, SC) = 1 \tag{3.9}$$

$$\lim_{T \to \infty} f(\mu, T, SC) = \mu \tag{3.10}$$

$$R_\mu = f(\mu, T, SC) = 1;\ \mu \le 1 \tag{3.11}$$

3.3.1 Past Studies on Ductility Factor

The study on ductility factor (R_μ) has been going on since 19th century by various researchers. Few of these past researches on ductility factor (R_μ) are discussed in this section.

3.3.1.1 Newmark and Hall (1982)

In 1982, Newmark and Hall carried out a study to determine the ductility reduction factor in which the expression depending upon the displacement ductility ratio and the dominant vibration period (T) of the structure was proposed. They stated that for a relatively long period structure, T greater than 1.0 seconds, the value of ductility reduction factor equals to displacement ductility ratio, as the peak displacement from the inertia force obtained from an elastic system and reduced inertia force obtained from inelastic system are the same. For a structure of natural period less than 0.03 seconds, the ductility does not help in reducing the response of the structure as the structure becomes very rigid and therefore, no ductility reduction factor should be used in such case. For moderate period structure, T in between 0.12 and 0.5 seconds, equal energy concept can be applied i.e., the energy that can be stored by the elastic system at maximum displacement is the same as that stored by an inelastic system.

Based on the findings, Newmark and Hall proposed a relationship to find ductility reduction factor and categorized them with respect to dominant vibration period (T) of the structure.

For periods below 0.03 seconds ($T < 0.03$)

$$R_\mu = 1 \tag{3.12}$$

For periods between 0.12 seconds and 0.5 seconds

$$R_\mu = \sqrt{2\mu - 1} \tag{3.13}$$

For periods above 1 seconds ($T > 1$)

$$R_\mu = \mu \tag{3.14}$$

Where,

μ = displacement ductility ratio

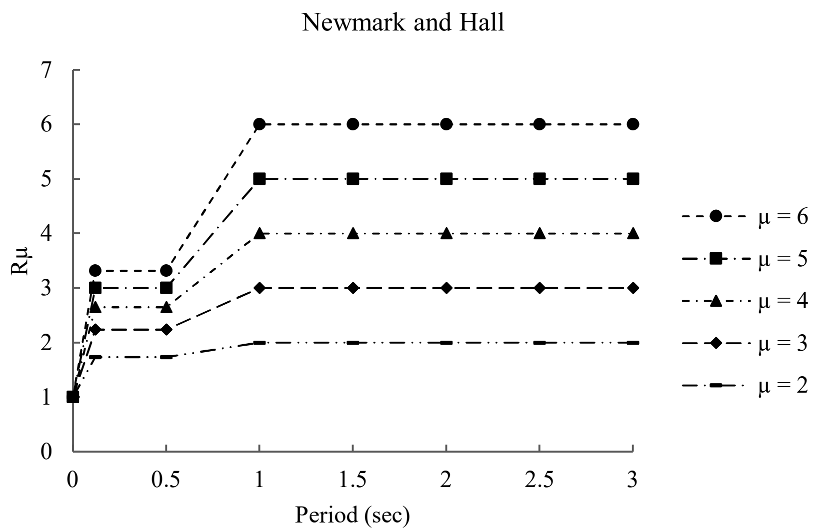

Figure 3-2: Ductility Factor as per Newmark and Hall

3.3.1.2 Riddell, Hidalgo and Cruz (1989)

This study considered 4 sets of earthquakes grouped as A, B, C, and D. A total of 9, 6, 1, and 7 earthquakes were considered in group A, B, C, and D respectively. The elastic and inelastic response spectra for various sets of earthquake data have been computed to study the response reduction factor to account for the energy dissipation capacity of the structure. A damping of 5% has been considered and an expression has been proposed to obtain the ductility factor.

For $0 \leq T < T^*$

$$R_\mu = 1 + \frac{R^* - 1}{T^*} T \tag{3.15}$$

For $T \geq T^*$

$$R_\mu = R^* \tag{3.16}$$

Table 3-1: Values of Parameters R^* and T^*

Parameter	μ=2	μ=3	μ=4	μ=5	μ=6	μ=7	μ=8
R^*	2.0	3.0	4.0	5.0	5.6	6.2	6.8
T^*	0.1	0.2	0.3	0.4	0.4	0.4	0.4

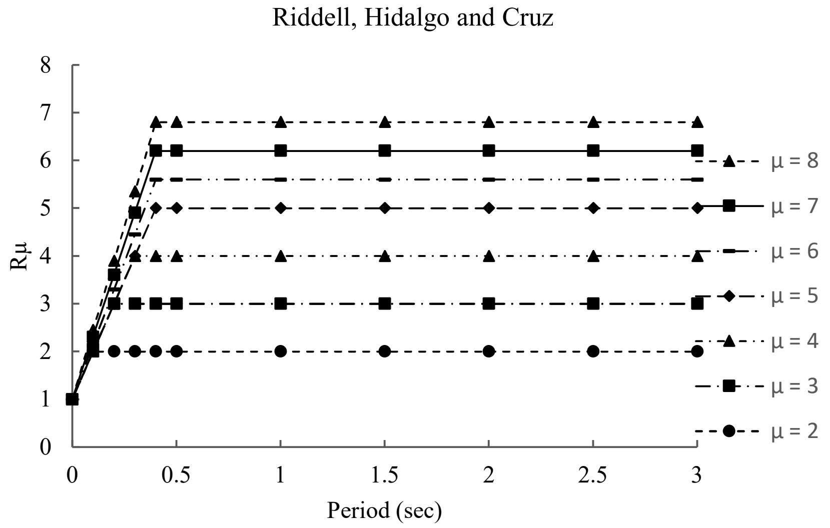

Figure 3-3: Ductility Factor as per Riddell, Hidalgo and Cruz

3.3.1.3 Eduardo Miranda (1993)

In 1993, Eduardo Miranda considered a total of 124 earthquake ground motion which were recorded on 3 different soil conditions i.e., rock soil sites (38 records), alluvium soil sites (62 records), and soft soil sites (24 records). After carrying out the regression analysis the equation for ductility factor (R_μ) was computed assuming 5% critical damping. The findings also included that soil condition also greatly affects the mean strength reduction factor. The total of 3 different equations were proposed for 3 different soil conditions which depend upon displacement ductility ratio (μ) and period of vibration (T).

The equations are as follow: -

$$R_\mu = \frac{\mu - 1}{\phi} + 1 \geq 1 \tag{3.17}$$

For rock soil sites

$$\phi = 1 + \frac{1}{10T - \mu T} - \frac{1}{2T} \exp\left[-\frac{3}{2}\left(\ln T - \frac{3}{5}\right)^2\right] \tag{3.18}$$

For alluvium soil sites

$$\phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} \exp\left[-2\left(\ln T - \frac{1}{5}\right)^2\right] \tag{3.19}$$

For soft soil sites

$$\phi = 1 + \frac{T_g}{3T} - \frac{3T_g}{4T} \exp\left[-3\left(\ln\frac{T}{T_g} - \frac{1}{4}\right)^2\right] \tag{3.20}$$

Where,

ϕ = function necessary to compute approximate strength reduction factor

T = period of vibration

T_g = predominant period of ground motion

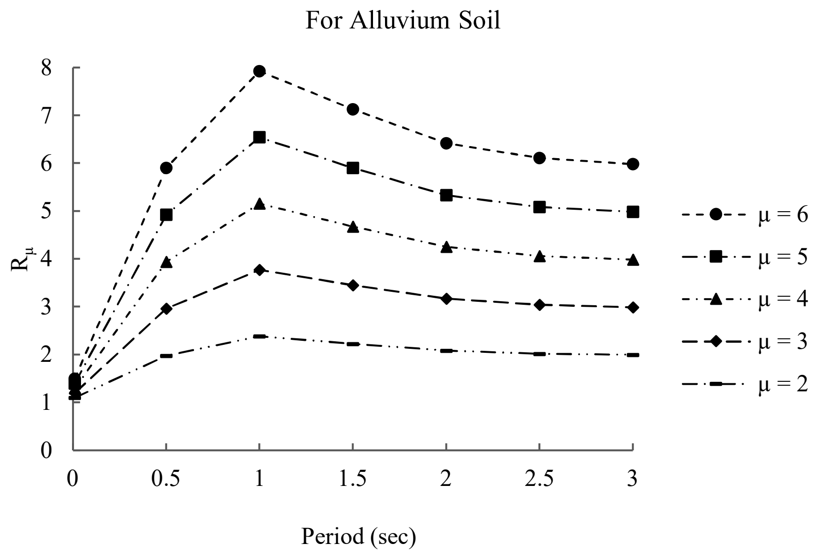

Figure 3-4: Ductility Factor for Alluvium Soil as per Miranda

3.4 Material Stress and Strain Relationship

For concrete model Mander's stress strain backbone curve is selected. The positive side is shown for compression while tension is shown on the negative side. The maximum value is shown for compression at 25 MPa for stress while corresponding strain is 0.002 for M25 concrete. While at the tension side the stress is shown as -3.11 Mpa having -0.000125 strain.

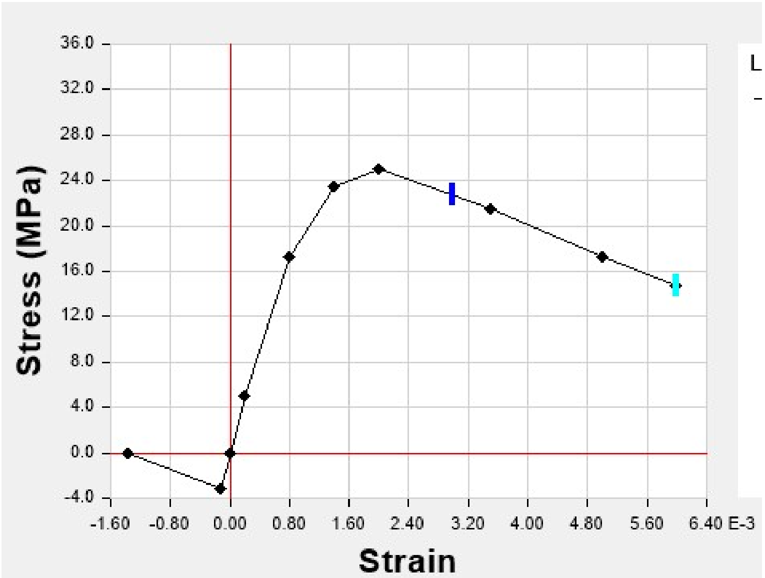

Figure 3-5: Mander’s Concrete Stress Strain Curve

Similarly for rebar of Fe500, Park’s stress strain curve is used where expected yield strength is 550 MPa and with strain hardening it reaches upto 599.5 MPa.

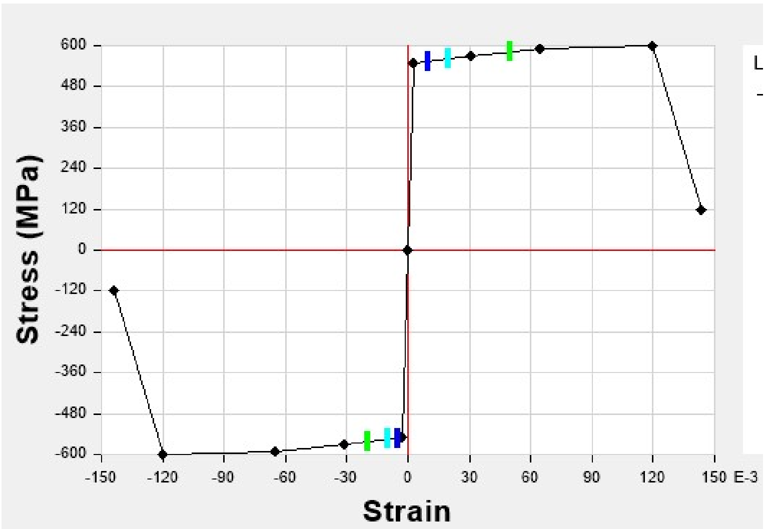

Figure 3-6: Park's Stress Strain Curve for Rebar

3.5 Analysis Method

Analysis of the structures are done to get the responses of the structures which can be inter storey drift, displacement, time period, base shear etc. The selection of the analysis method to be carried out is based on the structure and code provision. A single or multiple analysis can be carried in conjunction as per the required output. The analysis methods can be categorized as: -

1. Static Analysis
    a) Linear Static Analysis
    b) Nonlinear Static Analysis
2. Dynamic Analysis
    a) Linear Dynamic Analysis
    b) Nonlinear Dynamic Analysis

Nonlinear static analysis is discussed in detail as it is the main analysis method adopted in this research work.

3.5.1 Linear Static Analysis

Linear static analysis also known as equivalent static method is one the simplest method and requires less computational effort among the four-analysis method. The design base shear is calculated which is then distributed along the height of the building. The lateral force at each floor levels are again distributed to individual lateral load resisting elements. According to NBC 105: 2020, the procedure of obtaining the base shear and its distribution at each floor levels is described in detail in Section 4.6.2.

3.5.2 Linear Dynamic Analysis

Linear dynamic analysis also known as response spectrum analysis, is a method to get a maximum response of a structure by combining the contribution of each natural mode of vibration. The modal combination method can be done by either SRSS (Square Root of Sum of Square) or CQC (Complete Quadratic Combination). This method measures the response of the structures subjected to transient dynamic loading. It provides insight into dynamic behaviour as a function of structural period for a given time history and level of damping by measuring pseudo spectral acceleration, velocity and displacements.

3.5.3 Non-linear Dynamic Analysis

Non-linear dynamic analysis also called as non-linear time history analysis is a complex method to evaluate a performance of the structure. It takes into account both the non-linearity of the structure and the dynamic loading to evaluate the seismic response of the structures. It is a step-by-step analysis of dynamic response of the structure to a specified loading that vary with the time. As this method is computationally demanding and consumes more time, therefore it is carried out only for the important and significant structures.

3.6 Nonlinear Static Analysis

The nonlinear static analysis also called pushover analysis is an important and simple performance evaluation tool to evaluate the seismic performance of existing or new structures. The pushover analysis gives an idea about the nonlinear behaviour of the structure without the added complexities of nonlinear dynamic analysis. Geometrical as well as material non linearity is considered during the analysis method. At first the gravity loads are applied on the structure and is allowed to deform and at the end of this, monotonically increasing lateral load is applied to determine the seismic demand. The stresses and deformations from the previous step are carried over to the next step of the loading. By doing this, the potential week points on the structures are identified by analyzing the state of hinge formation on the structural members.

The load is applied on the model till the target displacement is reached or to the point of structure failure. There are several methods to estimate the target displacement, the two of which are coefficient method and capacity method. But most of the analysis software uses 4 % of total building height as the default target displacement.

At the end of the analysis, for each successive increment of load, the curve between the base shear with respect to displacement of the control point is obtained. It is the main output required from the pushover analysis for obtaining the overstrength factor and ductility factor.

3.6.1 Pushover Analysis Procedure

The nonlinear static pushover analysis can be carried out using ETABS to obtain the capacity curve for the three-dimensional models. The gravity loads are first run as a force-controlled while the lateral displacement is applied as the displacement-controlled. The force-controlled method is preferred when the load is known and structure is expected to sustain the load. The displacement-controlled method is used when the magnitude of applied load is not known and specified drifts are required and structure is expected to lose strength. The procedure done during the pushover analysis is: -

1. The three-dimensional building is modelled and all the necessary loads are applied.
2. Hinges are assigned at the point where the member is expected to fail. In this study it is assigned at 0.5*d where d is the effective depth of the member.
3. At first, a force-controlled method is initiated for gravity loads which is the sum of all the dead loads and 30% of live loads.
4. At the end of force-controlled method, the displacement-controlled method is carried out until the target displacement is reached or the structure reaches its maximum base shear.
5. The force-displacement curve is obtained which is then idealized to obtain yield displacement (d_y), ultimate displacement (d_u), yield base shear (V_y).

3.6.2 Idealization of Capacity Curve

The pushover curve must be idealized first to obtain the data required for the calculation going forward. The idealization is done by developing the bilinear curve which is the plot containing the two straight lines. The start of the first line segment starts from the origin and intersects at (V_y, u_y) with second line segment. The second line segment start from the intersected point and ends at the point in the curve having maximum force (V_u). The yield base shear is symbolized by V_y and its corresponding displacement is symbolized by u_y.

The first line segment signifies the elastic region. The intersecting point is the point on the plot that represents the start of nonlinearity. So, the second line segments represent the elasto-plastic region.

FEMA 356:2000 provides a procedure for bilinearization based on equal energy concept i.e., area under the capacity curve is equal to area under the idealized force displacement curve. The process is based on the iterative method that approximately balances the area below and above the curve. The two main points considered in FEMA 356:2000 for bilinearization of pushover curve is as follows: -

1. The area under the two curves must be equal i.e., area under the pushover curve before idealization and area under the idealized bilinear curve must be equal.
2. The first line segment of the bilinear curve must intersect the original pushover curve at 60% of significant yield strength.

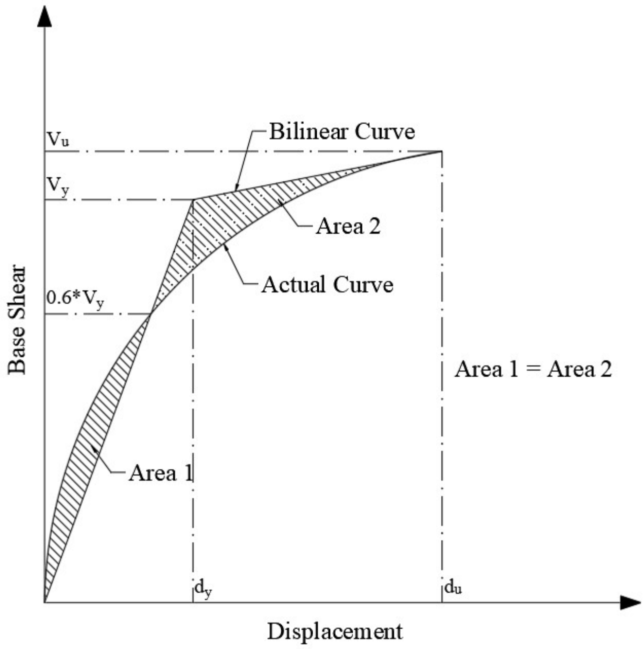

Figure 3-7: Bilinear Idealization of Generic Pushover Curve

Where,

V_y	=	Yield base shear
d_y	=	Yield displacement
V_u	=	Ultimate base shear
d_u	=	Ultimate displacement

3.6.3 Force Displacement Relationship

The plastic hinges are assigned to the frame elements (columns and beam) where the elements are expected to fail which is around the joint location of column and beam. These hinges represent the localized force-displacement relation of a member through its elastic and inelastic phases under seismic loads.

It is important to model the force displacement relationship for carrying out the non-linear analysis. This force displacement relationship affects the yielding and post yielding behaviour of the elements. The force displacement behavior is described by properties that are provided based on ASCE 41 - 13. The uncoupled moment M3 hinges are assigned to beams as the axial load effects are ignored due to the rigid floor diaphragm effect and the P-M2-M3 hinges are assigned to columns because of its coupled axial and biaxial bending behaviour.

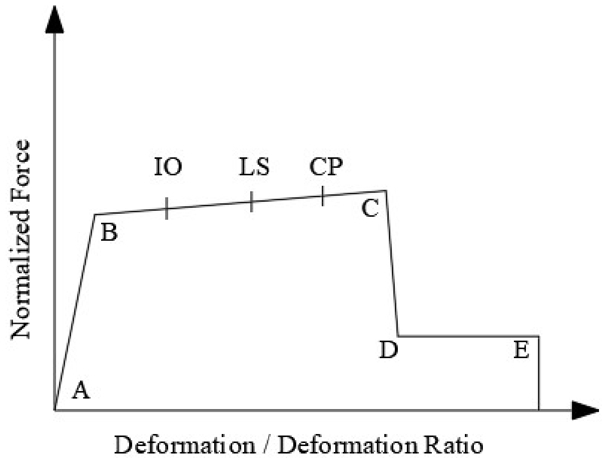

Figure 3-8: Generalized Force Displacement Relation

The A, B, C, D, and E in Figure 3-8 is a point that defines a behaviour for the force deformation relation. Within the ductile range of B and C, there are 3 acceptance criteria IO, LS, and CP which stands for immediate occupancy, life safety, and collapse prevention respectively. The following in detail describes all the points and paths in the force displacement curve: -

1. Point A is origin and denotes the unloaded condition.
2. The portion A to B represents linear response, where point B is the effective yield point from where the yielding of the elements occurs. There is no deformation till point B regardless of the deformation values specified for the point B.
3. Again, from point B to C, there is a linear response at reduced stiffness which represents the strain hardening phenomenon.
4. The ultimate capacity of the element during a pushover analysis is at point C. Then there is a sudden reduction in seismic force resistance till point D.

At point D, there still exists some residual strength which allows the element to sustain the gravity loads till point E. The point E represents the total failure.

5. Beyond point E, even the gravity load cannot be sustained and the strength of the element drops to 0 and the hinges will drop the load on the horizontal axis.

3.7 Formulation Used in this Study

From all the above-mentioned formulas concluded in different researches, the following equations were used to determine the overstrength factor and ductility factor. They were selected because they were the most recently published among all the other formulas.

Overstrength Factor (Ω)

$$\Omega = \frac{V_y}{V_d} \tag{3.21}$$

Ductility Factor(R_μ)

$$R_\mu = \frac{\mu - 1}{\phi} + 1 \geq 1 \tag{3.22}$$

Where,

Displacement Ductility Factor (μ)

$$\mu = \frac{d_u}{d_y} \tag{3.23}$$

For Alluvium Soil (ϕ)

$$\phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} \exp\left[-2\left(\ln T - \frac{1}{5}\right)^2\right] \tag{3.24}$$

CHAPTER-4 BUILDING DESCRIPTION AND MODELLING

4.1 General

For modeling, analyzing (linear and non-linear), and designing of all the models, finite element analysis software ETABS v19.0.0 is used. The mathematical model is created in the software which closely represents the real model. 3D models of the structures are created where beams and columns are modelled as the frame elements and slabs as shell elements that are interconnected at nodes.

4.2 Structural Configuration

Most of the RC buildings in Nepal are of low-rise residential type. The National Census data of Nepal only have limited information on the buildings mainly focused on construction type, building use, types of foundation, types of walls, and type of roofing. The research paper named "Seismic response of current RC Buildings in Kathmandu Valley", has provided the statistical information of RC buildings in Nepal. Some of this information include a number of storey, inter-storey height, beam span length, and plinth area. The 300 building drawings were collected from 10 district headquarters, out of which 200 were taken for statistical analysis. These 10 districts were selected based on them having the highest concentration of RC buildings in Nepal. So, to incorporate the maximum number of different types of RC buildings of Nepal, following modeling parameters are selected.

1. Low-rise buildings having 2, 3, 4, and 5 number of storeys with a regular storey height of 2.9m are modeled.
2. The number of bays is taken as 2, 3, and 4 having bay lengths of 3m, 3.5m, and 4m which make these buildings having the plinth area from 36m^2 to 256m^2.
3. The buildings having regular plan and elevation with an equal number of bays in both the horizontal directions are considered in this study.

From the graphical figures below, the different modeling parameters for buildings are considered. These statistical analysis figures are taken from the research article mentioned above.

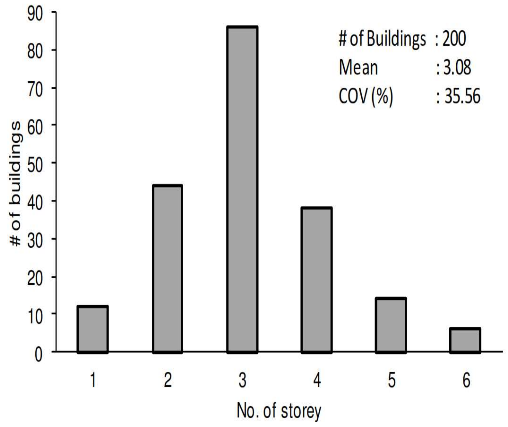

Figure 4-1: Distribution of Buildings According to Number of Storey

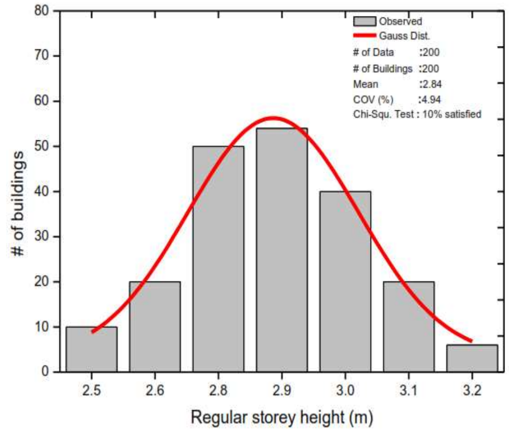

Figure 4-2: Distribution of Buildings According to Storey Height

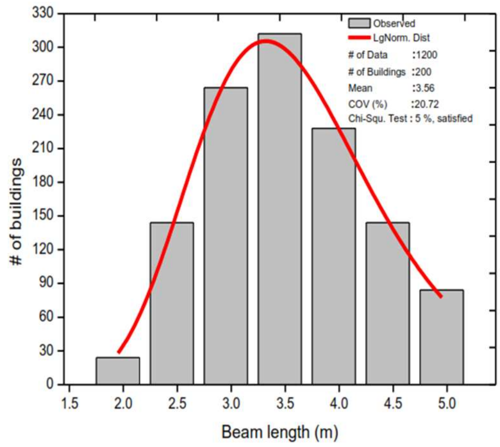

Figure 4-3: Distribution of Buildings According to Beam Length

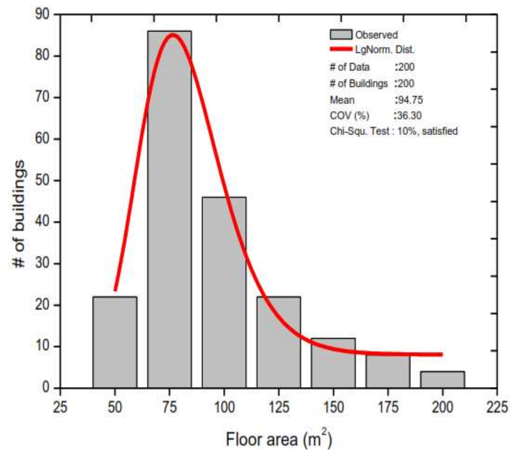

Figure 4-4: Distribution of Buildings According to Plinth Floor Area

(Source: Chaulagain et al. (2015), Seismic response of current RC Buildings in Kathmandu Valley)

4.3 Structural Modelling Parameters

The varying parameters considered in this study are number of storeys, numbers of bays, bay length and members' sizes. There are 4 numbers of storeys i.e., 2, 3, 4, and 5. Also, these models have 3 variations of bays i.e., 2, 3, and 4 in which each bays have 3 different bay lengths i.e., 3m, 3.5m, and 4m. Again, these 36 different configurations of building models, are each modelled with 2 different structural member sizes making a total of 72 models.

The 36 different configurations of the building having 4 different storeys i.e., 2, 3, 4, and 5 are modelled. At first, for these 36 models, sizes of column and beam differ according to number of storeys and later a larger but same cross-sectional sizes' column and beam are provided irrespective of the number of storeys for all these 36 models. However, all the columns and beams in a particular building model are of the same size.

4.4 Structural Members

A total of 72 models has been analyzed in this study in which there are 36 unique configuration of buildings and each building is model with 2 different structural members' sizes i.e., small and large cross-sectional size. A total of 5 different structural members' sizes are used during the analysis of the models. The slab of thickness 5" is used for all the models. The different structural member's sizes with their assigned codes (for nomenclature) are tabulated below:

Table 4-1: Structural Member Sizes Assigned to the Models

S. No.	Number of Storey	Column Dimension (D X B)	Beam Dimension (D X B)
1	2	12" X 12"	14" X 9"
2	3	13" X 13"	14" X 9"
3	4	14" X 14"	14" X 10"
4	5	15" X 15"	16" X 10"
5	2, 3, 4, 5	16" X 16"	16" X 12"

While carrying out the analysis of the above-mentioned model, column beam capacity ratio as suggested by NBC 105:2020 of 1.2 was checked. Also, an inter-storey drift limit of 0.025 for the ultimate limit state and 0.006 for the serviceability limit state was satisfied. Finally, the dimensions of the columns, beams, and slabs is finalized.

4.5 Materials Properties

The concrete having characteristics compressive strength of 25 N/mm^2 (M25) and reinforcement of grade HYSD500 TMT are assigned to analyze and design of all the structural members i.e., column and beam. The modulus of elasticity, unit weight, and poisson's ratio of concrete used is 25,000 MPa, 25 kN/m^3, and 0.2 respectively. Similarly, modulus of elasticity, unit weight, and poisson's ratio of steel used is 200,000 MPa, 76.97 kN/m^3, and 0.3 respectively.

4.6 Loads and Load Combinations

The unit weight of material is obtained from NBC 102: 1994 which is applied as super imposed dead load and NBC 103: 1994 gives the value for occupancy load (imposed load). The NBC 105: 2020 (Seismic Design of Buildings in Nepal) is used to calculate the design lateral load.

4.6.1 Gravity Loads

The self-weight of the structure is calculated by the software, while other gravity loads are given manually as follows: -

1. Live Load
   a) 2kN/m^2 on general floors
   b) 1.5kN/m^2 on roof
2. Floor Finish 1kN/m^2
3. Wall Load
   a) 7.5kN/m for 9” external walls
   b) 4kN/m for 5” internal walls

4.6.2 Lateral Loads

The design lateral load is calculated as per NBC 105: 2020 where it is obtained from the product of seismic weight and horizontal base shear coefficient. The horizontal base shear coefficient is further classified into the ultimate limit state and serviceability limit state.

The total seismic weight of the structure (W) is the sum of the dead loads and factor of the live loads, i.e.

$$W = DL + \lambda * LL \tag{4.1}$$

Where,

W = Total seismic weight of the structure

DL = Total dead load of the structure which is the sum of the total self-weight of the structure and applied dead loads such as floor finish, wall loads

LL = Total live load of the structure

λ = Live load participation factor. It is taken as 0.3 in this study.

The horizontal base shear can be classified as: -

Ultimate Limit State	Serviceability Limit State
$C_d(T_1) = \dfrac{C(T_1)}{R_\mu \ \Omega_u}$ Where, C(T1) = Elastic site spectra R_μ = Ductility factor which is taken as 4 for this study Ω_u = Overstrength factor which is taken as 1.5 for this study	$C_d(T_1) = \dfrac{Cs(T_1)}{\Omega_s}$ Where, C_s(T1) = Elastic site spectra Ω_s = Overstrength factor which is taken as 1.25 for this study

Where,

The elastic site spectra C(T) is given as: -

$$C(T) = C_h(T)\, Z\, I \tag{4.2}$$

Where,

$C_h(T)$ = Spectral shape factor

Z = Seismic zoning factor which is taken as 0.4 in this study

I = Importance factor which is taken as 1 in this study

The spectral shape factor $C_h(T)$ depends upon the soil type and the buildings' fundamental time period. The soil type is taken as B for this study.

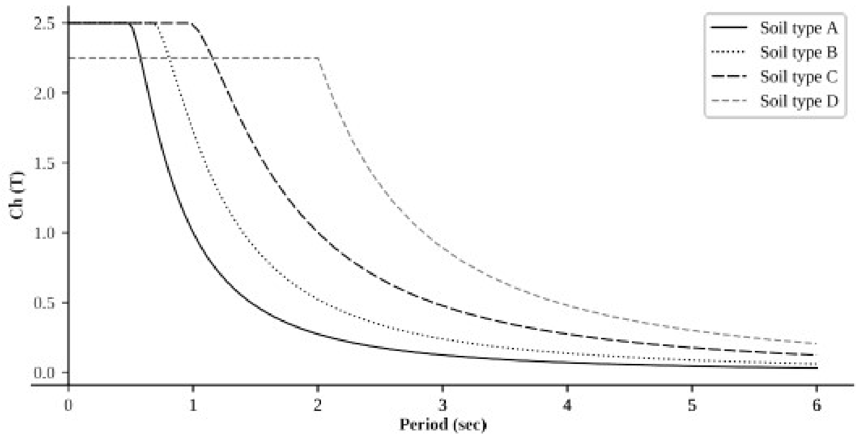

Figure 4-5: Spectral Shape Factor for Equivalent Static Method

The approximate fundamental time period of the building (T_1) is calculated as specified in NBC 105: 2020 for moment resisting concrete frame as: -

$$T_1 = 0.075\, H^{0.75} \tag{4.3}$$

Where,

H = Height of the building from foundation or top of rigid foundation

Again, the approximate fundamental time period of the building (T_1) is increased by a factor of 1.25.

4.6.3 Load Combinations

For the design of structures, seismic load effect is combined with other load effects. The following load combination is adopted.

1.2DL + 1.5 LL

DL + 0.3LL + E

DL + 0.3LL – E

4.7 Building Nomenclature

By the combination of all the parameters, a total of 72 buildings are considered in this study. So, a proper short naming rule is required to identify the models. xSyBzBLMn represents building with x number of storeys with y number of bays where each bays have a length of z meters and the structural member sizes assigned to that model is Mn. The model named 4S3B3.5BLM4 denotes 4 storied building with 3 bays where each bays have 3.5m bay length and column and beam dimension of 14” X 14” and 14” X 10” respectively.

Table 4-2: Code Assigned to the Structural Member

S. No.	No. of Models	Column Dimension (D X B)	Beam Dimension (D X B)	Code Assigned (Mn)
1	9	12” X 12”	14” X 9”	M1
2	9	13” X 13”	14” X 9”	M2
3	9	14” X 14”	14” X 10”	M3
4	9	15” X 15”	16” X 10”	M4
5	36	16” X 16”	16” X 12”	M5

Where,

Mn represents the column and beam sizes used in the particular model. The code assigned to these column and beam sizes are used for nomenclature.

So, from the combination of building configuration and structural member sizes, following is name assigned to all the models used in this study.

Table 4-3: Nomenclature of All the Models

<table>
<tr><th>S.No.</th><th>Number of Storey</th><th>Number of Bay</th><th>Bay Length (m)</th><th colspan="2">Nomenclature</th></tr>
<tr><td>1</td><td rowspan="9">2</td><td rowspan="3">2</td><td>3</td><td>2S2B3BLM1</td><td>2S2B3BLM5</td></tr>
<tr><td>2</td><td>3.5</td><td>2S2B3.5LM1</td><td>2S2B3.5LM5</td></tr>
<tr><td>3</td><td>4</td><td>2S2B4BLM1</td><td>2S2B4BLM5</td></tr>
<tr><td>4</td><td rowspan="3">3</td><td>3</td><td>2S3B3BLM1</td><td>2S3B3BLM5</td></tr>
<tr><td>5</td><td>3.5</td><td>2S3B3.5LM1</td><td>2S3B3.5LM5</td></tr>
<tr><td>6</td><td>4</td><td>2S3B4BLM1</td><td>2S3B4BLM5</td></tr>
<tr><td>7</td><td rowspan="3">4</td><td>3</td><td>2S4B3BLM1</td><td>2S4B3BLM5</td></tr>
<tr><td>8</td><td>3.5</td><td>2S4B3.5LM1</td><td>2S4B3.5LM5</td></tr>
<tr><td>9</td><td>4</td><td>2S4B4BLM1</td><td>2S4B4BLM5</td></tr>
<tr><td>10</td><td rowspan="3">3</td><td rowspan="3">2</td><td>3</td><td>3S2B3BLM2</td><td>3S2B3BLM5</td></tr>
<tr><td>11</td><td>3.5</td><td>3S2B3.5LM2</td><td>3S2B3.5LM5</td></tr>
<tr><td>12</td><td>4</td><td>3S2B4BLM2</td><td>3S2B4BLM5</td></tr>
</table>

13		3	3	3S3B3BLM2	3S3B3BLM5
14			3.5	3S3B3.5LM2	3S3B3.5LM5
15			4	3S3B4BLM2	3S3B4BLM5
16		4	3	3S4B3BLM2	3S4B3BLM5
17			3.5	3S4B3.5LM2	3S4B3.5LM5
18			4	3S4B4BLM2	3S4B4BLM5
19	4	2	3	4S2B3BLM3	4S2B3BLM5
20			3.5	4S2B3.5LM3	4S2B3.5LM5
21			4	4S2B4BLM3	4S2B4BLM5
22		3	3	4S3B3BLM3	4S3B3BLM5
23			3.5	4S3B3.5LM3	4S3B3.5LM5
24			4	4S3B4BLM3	4S3B4BLM5
25		4	3	4S4B3BLM3	4S4B3BLM5
26			3.5	4S4B3.5LM3	4S4B3.5LM5
27			4	4S4B4BLM3	4S4B4BLM5
28	5	2	3	5S2B3BLM4	5S2B3BLM5
29			3.5	5S2B3.5LM4	5S2B3.5LM5
30			4	5S2B4BLM4	5S2B4BLM5
31		3	3	5S3B3BLM4	5S3B3BLM5
32			3.5	5S3B3.5LM4	5S3B3.5LM5
33			4	5S3B4BLM4	5S3B4BLM5
34		4	3	5S4B3BLM4	5S4B3BLM5
35			3.5	5S4B3.5LM4	5S4B3.5LM5
36			4	5S4B4BLM4	5S4B4BLM5

Following is the elevation view of the models having different storeys and bays but the variation in bay length and structural members' sizes are not shown.

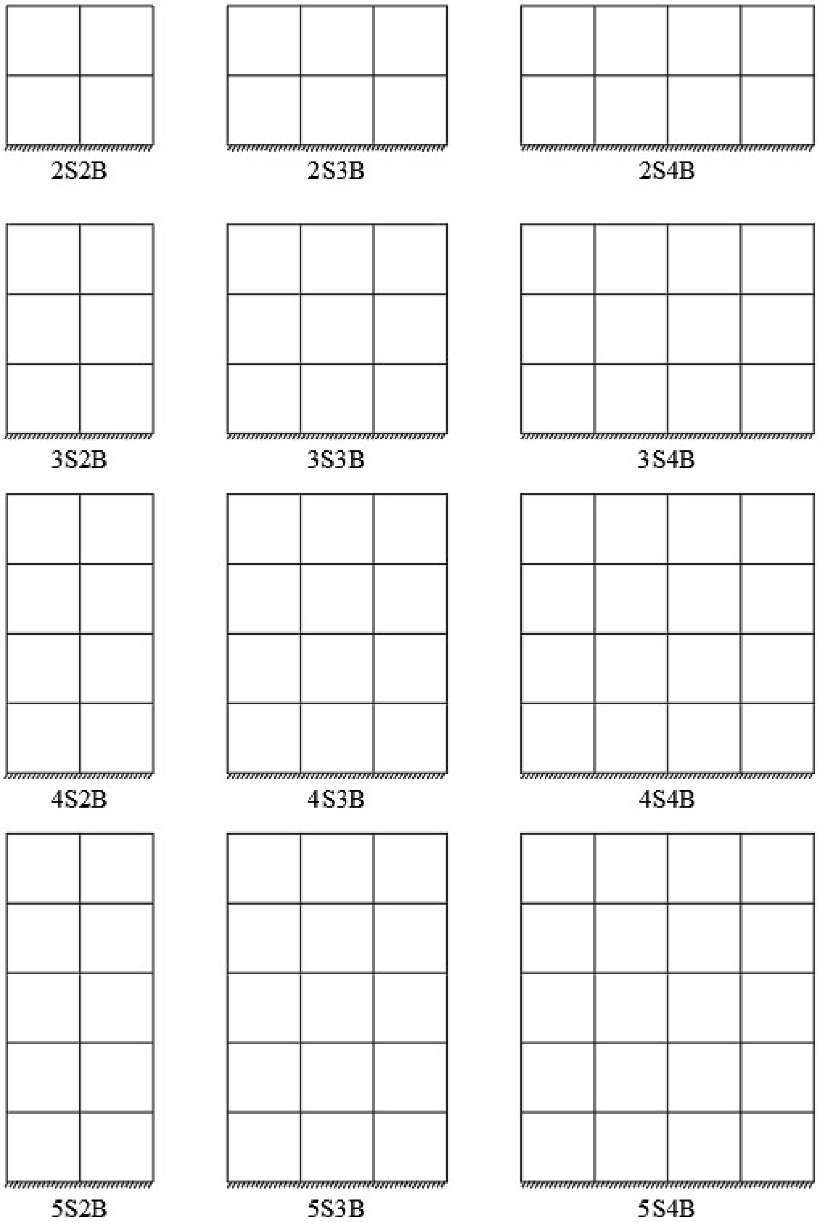

Figure 4-6: Typical Elevation View of Building Models

CHAPTER-5 RESULTS AND DISCUSSIONS

5.1 Sample Analysis Evaluation

For the sample calculation, the 3 storied 3 bay building with bay length 4m having two different structural members' sizes (designated nomenclature 3S3B4BLM1 and 3S3B4BLM3) are shown. The structure is first designed by an equivalent static method using ETABS and all the members are checked to see if they are capable of resisting the applied loads mentioned in section 3.5. These members are then checked for percentage of longitudinal reinforcement to conform with NBC 105: 2020. The fundamental time period along with the design base shear is obtained and later is used for the calculation of the cofactors of response reduction factors.

The material non-linearities are considered by assigning frame hinge properties near to the column beam joints which represents post yield behavior. The default hinges are assigned for both the beams and columns in which force - displacement behavior is described by properties that are provided based on ASCE 41 - 13. The uncoupled moment M3 hinges are assigned to beams as the axial load effects are ignored due to the rigid floor diaphragm effect and the P-M2-M3 hinges are assigned to columns because of its coupled axial and biaxial bending behaviour. A non-linear gravity case is applied which incorporates total dead load plus 30% of live load which is a force-controlled load. The pushover load case is continued at the end of the gravity case until the displacement reaches an assigned value or the structure becomes unstable due to the formation of a plastic hinges.

The pushover curve is obtained which is then idealized to form a bilinear curve using the equal area concept given in FEMA 356:2020. After the idealization, yield base shear, yield displacement, and ultimate displacement are obtained which is further used for calculations of the overstrength factor and the ductility factor.

A detailed calculation of the sample buildings with same building configuration but different member sizes (3S3B4BLM1 and 3S3B4BLM3) are shown along with the pushover curve and idealized bilinear curve.

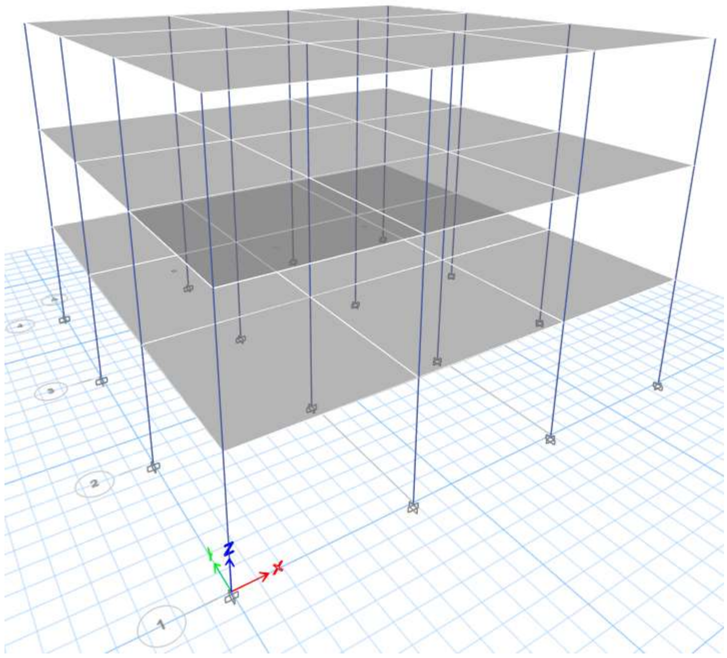

Figure 5-1: Finite Element Modelling of 3 Storey 3 Bay 4m Bay Length (3S3B4BL) Model

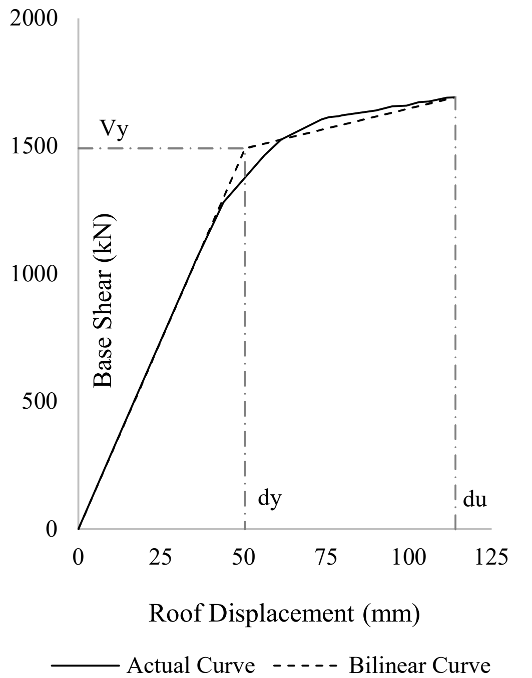

Figure 5-2: Pushover Curve and Idealized Bilinear Curve of Model 3S3B4BLM2

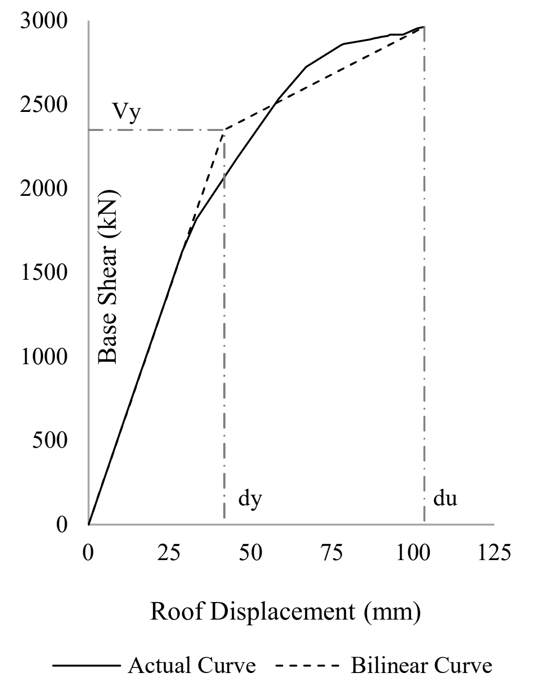

Figure 5-3: Pushover Curve and Idealized Bilinear Curve of Model 3S3B4BLM5

Fundamental time period, (T) = 0.71 sec	Fundamental time period, (T) = 0.54 sec
Design lateral strength, (V_d) = 746 kN	Design lateral strength, (V_d) = 817 kN

Yield strength, (V_y) = 1490 kN	Yield strength, (V_y) = 2349 kN
Yield displacement, (d_y) = 50 mm	Yield displacement, (d_y) = 41 mm
Ultimate displacement, (d_u) = 114 mm	Ultimate displacement, (d_u) = 103 mm
Ultimate strength, (V_u) = 1690 kN	Ultimate strength, (V_u) = 2962 kN
Ductility factor, (R_μ) = 2.517	Ductility factor, (R_μ) = 2.472
Overstrength factor, (Ω_u) = 1.997	Overstrength factor, (Ω_u) = 2.874

The detail calculation for both the models are shown below. For model 3S3B4BLM3, following are the data obtained from analysis and bilinear curve.

Fundamental time period, (T) = 0.71 sec

Design lateral strength, (V_d) = 746.285 kN

Yield strength, (V_y) = 1490.761 kN

Yield displacement, (d_y) = 50.418 mm

Ultimate strength, (V_u) = 1690.956 kN

Ultimate displacement, (d_u) = 114.071 mm

Now from the calculations, all the required values are calculated.

Displacement ductility ratio, (μ) $= \frac{d_u}{d_y} = \frac{103.460}{41.841} = 2.262$

For alluvium soil, (ϕ) $= 1 + \frac{1}{12T - \mu T} - \frac{2}{5T}\exp\left[-2\left(\ln T - \frac{1}{5}\right)^2\right] = 0.8319$

Ductility factor, $\left(R_\mu\right)$ $= \frac{\mu - 1}{\phi} + 1 = 2.517$

Overstrength factor, (Ω_u) $= \frac{V_y}{V_d} = 1.997$

Again, following are the data obtained from analysis and bilinear curve for model 3S3B4BLM1, which has same building configuration as 3S3B4BLM3 but larger structural member sizes.

Fundamental time period, (T) = 0.539 sec

Design lateral strength, (V_d) = 817.440 kN

Yield strength, (V_y) = 2349.365 kN

Yield displacement, (d_y) = 41.841 mm

Ultimate strength, (V_u) = 2962.156 kN

Ultimate displacement, (d_u) = 103.460 mm

Now from the calculations, all the required values are calculated.

Displacement ductility ratio, (μ) $= \frac{d_u}{d_y} = \frac{103.460}{41.841} = 2.473$

For alluvium soil, (ϕ) $= 1+\frac{1}{12T-\mu T}-\frac{2}{5T}\exp\left[-2\left(\ln T-\frac{1}{5}\right)^2\right] = 1.0001$

Ductility factor, $\left(R_\mu\right)$ $= \frac{\mu-1}{\phi}+1 = 2.473$

Overstrength factor, (Ω_u) $= \frac{V_y}{V_d} = 2.874$

For all the models, a similar process is followed for calculation of the overstrength factor and the ductility factor and their calculated values are shown in the appendix.

5.2 Effect on Overstrength Factor

Overstrength factor (Ω_u) is examined for every combination of building configuration and different structural member sizes.

5.2.1 Due to Building Configurations

The overstrength factor decreases while increasing the number of storey. While increasing the number of storey, both the design base shear and the yield strength increase but the yield strength increases at a lower rate than the design base shear which eventually decreases the overstrength factor. While increasing the number of bay does not affect the overstrength factor, as both the design base shear and the yield strength increases at almost the same rate. So, the overstrength factor varies only slightly. Also, from Table 5-1 it can be seen that for 2 storey model, there is a % difference of 7.1% in between 2 bays and 4 bays model but for 5 storey model % difference decreases to

2.12% in between 2 bays and 4 bays model. So, it can be said that the effect of number of bays further decreases with an increase in the number of storey.

Table 5-1: Overstrength Factor for 4m Bay Length Model Varying Number of Storey and Bay

Model Name	Design Lateral Strength, V_d (kN)	Yield Strength, V_y (kN)	Overstrength Factor, Ω_u
2S2B4BLM1	244.093	540.563	2.215
2S3B4BLM1	488.548	1039.637	2.128
2S4B4BLM1	815.169	1677.408	2.058
3S2B4BLM2	373.539	752.886	2.016
3S3B4BLM2	746.285	1490.761	1.998
3S4B4BLM2	1244.113	2395.162	1.925
4S2B4BLM3	513.358	1004.989	1.958
4S3B4BLM3	1023.880	1970.640	1.925
4S4B4BLM3	1705.412	3257.308	1.910
5S2B4BLM4	664.066	1279.369	1.927
5S3B4BLM4	1322.272	2503.376	1.893
5S4B4BLM4	2200.556	4150.298	1.886

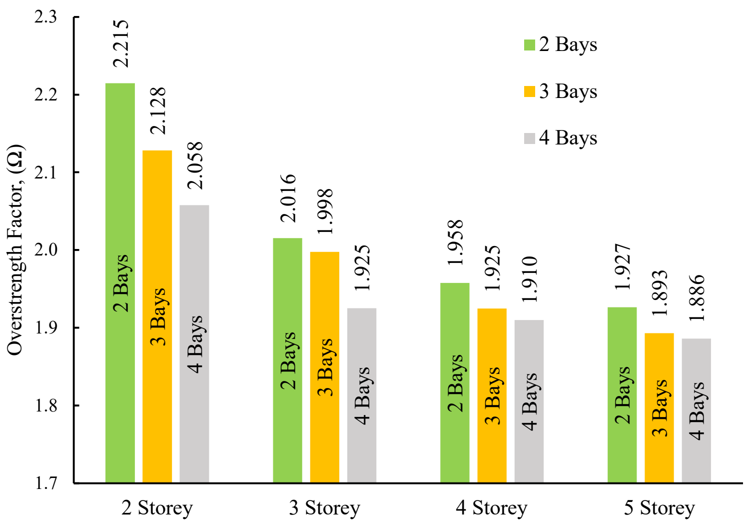

Figure 5-4: Overstrength Factor for 4m Bay Length Model Varying Number of Storey and Bay

For every storey, increasing the bay length, decreases the overstrength factor. This can be explained as increasing the bay length only increases the seismic weight / design base shear but does not increase the lateral stiffness. The bay length is the major factor affecting the overstrength factor. While increasing the bay length by just 1m, there is a huge decrease in overstrength factor by 21.19% and 10.58% for 2 storey and 5 storey respectively.

The overstrength factor varied from the highest 2.700 for the model having 2 storey, 3 bays and 3m bay span to the lowest 1.893 for the model having 5 storey, 3 bays and 4m bay span. According to NBC 105:2020, the value of overstrength factor for RC moment resisting frame is 1.5, so for all the model the calculated value of overstrength factor was higher than that mentioned in the code.

Table 5-2: Overstrength Factor for 3 Bay Model Varying Number of Storey and Bay Length

Model Name	Design Lateral Strength, (V_d) kN	Yield Strength, (V_y) kN	Overstrength Factor, (Ω_u)
2S3B3BLM1	329.614	890.089	2.700
2S3B3.5LM1	405.557	994.391	2.452
2S3B4BLM1	488.548	1039.637	2.128
3S3B3BLM2	507.097	1243.594	2.452
3S3B3.5LM2	621.376	1448.584	2.331
3S3B4BLM2	746.285	1490.761	1.998
4S3B3BLM3	720.826	1685.837	2.339
4S3B3.5LM3	855.248	1898.202	2.219
4S3B4BLM3	1023.880	1970.640	1.925
5S3B3BLM4	911.611	1929.995	2.117
5S3B3.5LM4	1108.046	2241.832	2.023
5S3B4BLM4	1322.272	2503.376	1.893

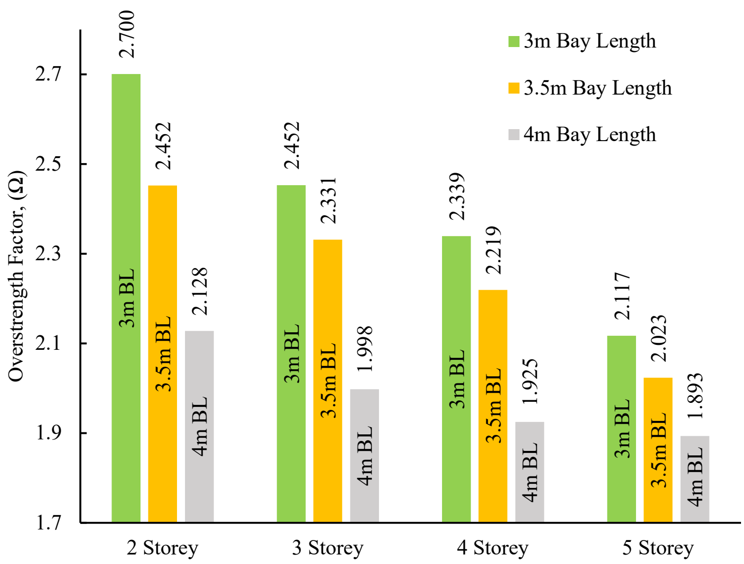

Figure 5-5: Overstrength Factor for 3 Bay Model Varying Number of Storey and Bay Length

The Figure 5-6 represents the overstrength factor for model having same length of 12m. There are two 12m model for all storied building from 2 storey to 5 storey in which one has 4 bays with 3m bay length and other has 3 bays with 4m bay length. As increasing the bay length rapidly decreases the overstrength factor, the model which has smaller bay length by having more bays i.e., model with 4 bays and 3m bay length has higher value of overstrength factor than model with 3 bays and 4m bay length. For model which has higher bay by having small bay length for a fixed 12 m total span of building, overstrength factor increased by 18.31%, 18.24%, 19.59% and 10.24% for 2 storey, 3 storey, 4 storey, and 5 storey respectively.

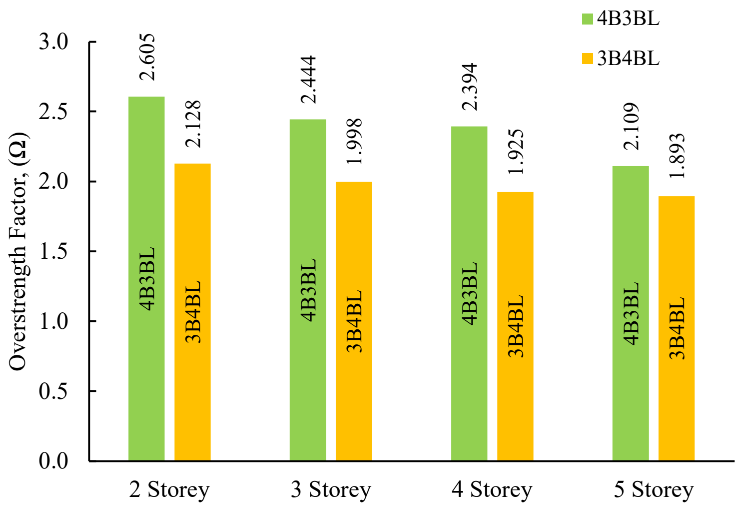

Figure 5-6: Overstrength Factor for Model having Same Length (12m)

5.2.2 Due to Structural Member Size

The models are analysed with varying column and beam sizes and its effect on the overstrength factor is observed. The overstrength factor rapidly increased by 14.35% to 128.04% when difference between the structural member size also increased. This can be attributed to the concept that increasing the member sizes increases its lateral stiffness which then gives higher yield strength. The Table 5-3 shows the member sizes used with its code assigned.

Table 5-3: Member Sizes with Assigned Codes

Column Dimension (D X B)	Beam Dimension (D X B)	Code Assigned (Mn)
16” X 16”	16” X 12”	M5
15” X 15”	16” X 10”	M4

14” X 14”	14” X 10”	M3
13” X 13”	14” X 9”	M2
12” X 12”	14” X 9”	M1

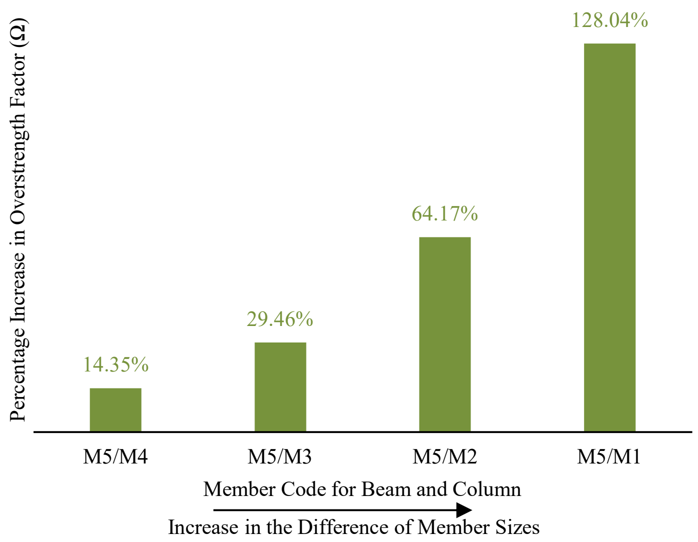

Figure 5-7: Effect on Overstrength Factor by Varying Member Sizes

Where M5/M4 represents a model with member sizes M4 which is then replaced by member sizes M5 and it is similar for all the other designation mentioned in the figure 6.

5.3 Effect on Ductility Factor

Ductility factor is calculated based on the formulation mentioned in section 2. Building configuration and structure member size are varied to study its effect on ductility factor.

5.3.1 Due to Building Configurations

The value for ductility factor is higher for 2 storey building but reduces with the increase in number of storey from 2 to 5. Even though the time period increases as the number of storey increases but the value of displacement ductility ratio (μ) decreases significantly which then reduces the ductility factor. The effect of number of bay on the ductility factor is very less showing slight decrease with increase in number of bay. This can be explained as increasing the number of bay makes the building stiff. But contrary to overstrength factor, the effect of number of bay further demises as the number of storey decreases. As it can be seen from Table 5-4 that for 2 storey model, there is a % difference of just 0.72% in between 2 bays and 4 bays model but for 5 storey model % difference slightly increases to 5.75% in between 2 bays and 4 bays model.

Table 5-4: Ductility Factor for 4m Bay Length Model Varying Number of Storey and Bay

Model Name	Fundamental Time Period, (T) sec	Displacement Ductility Ratio, (μ)	ϕ	Ductility Factor, (R_μ)
2S2B4BLM1	0.507	3.305	1.059	3.177
2S3B4BLM1	0.520	3.250	1.041	3.162
2S4B4BLM1	0.512	3.264	1.051	3.154
3S2B4BLM2	0.698	2.280	0.841	2.522
3S3B4BLM2	0.710	2.262	0.832	2.518
3S4B4BLM2	0.667	2.221	0.865	2.412
4S2B4BLM3	0.869	2.025	0.750	2.366
4S3B4BLM3	0.878	1.995	0.747	2.331
4S4B4BLM3	0.882	1.870	0.745	2.168
5S2B4BLM4	0.979	1.945	0.731	2.293
5S3B4BLM4	0.986	1.875	0.730	2.199
5S4B4BLM4	0.990	1.847	0.730	2.161

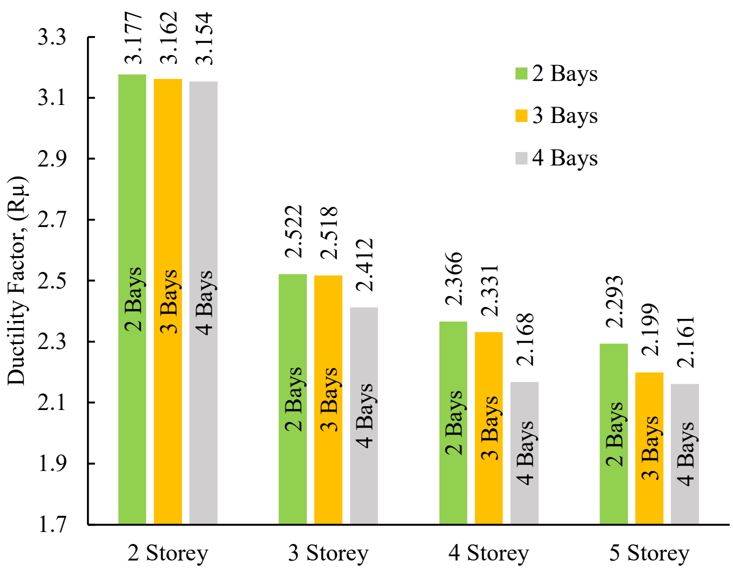

Figure 5-8: Ductility Factor for 4m Bay Length Model Varying Number of Storey and Bay

Similar to the overstrength factor, increasing the bay length decreases the ductility factor. This can be justified as seen in the Table 5-5, we can see that increasing the bay length decreases the displacement ductility ratio (μ) which then decreases the ductility factor. The bay length highly influences the ductility factor but not as much as number of storey.

Table 5-5: Ductility Factor for 3 Bay Model Varying Number of Storey and Bay Length

Model Name	Fundamental Time Period, (T) sec	Displacement Ductility Ratio, (μ)	ϕ	Ductility Factor, (R_μ)
2S3B3BLM1	0.412	3.796	1.205	3.320
2S3B3.5LM1	0.468	3.470	1.115	3.216
2S3B4BLM1	0.520	3.250	1.041	3.162
3S3B3BLM2	0.563	2.687	0.977	2.728
3S3B3.5LM2	0.636	2.387	0.895	2.550
3S3B4BLM2	0.710	2.262	0.832	2.518
4S3B3BLM3	0.734	2.325	0.816	2.624
4S3B3.5LM3	0.786	2.085	0.783	2.385
4S3B4BLM3	0.878	1.995	0.747	2.331
5S3B3BLM4	0.779	2.216	0.789	2.542
5S3B3.5LM4	0.882	2.088	0.747	2.455
5S3B4BLM4	0.986	1.875	0.730	2.199

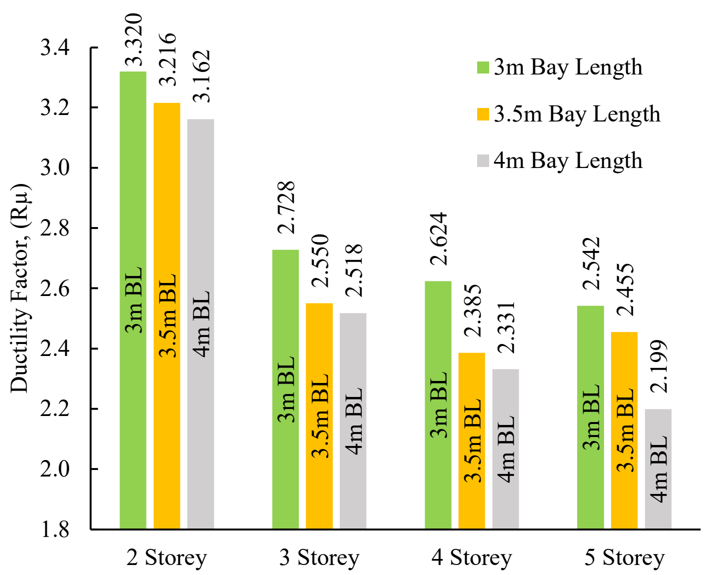

Figure 5-9: Ductility Factor for 3 Bay Model Varying Number of Storey and Bay Length

The Figure 5-10 represents the ductility factor for model having same length of 12m. There are two 12m model for all storied building from 2 storey to 5 storey in which one has 4 bays with 3m bay length and other has 3 bays with 4m bay length. As dependency on bay length is more than number of bays, the model which has smaller bay length by having a greater number of bays i.e., model with 4 bays and 3m bay length has higher value of ductility factor then model with 3 bays and 4m bay length. For model which has higher bay by having small bay length for a fixed 12 m total span of building, ductility factor increased by 6.14%, 7.22%, 6.94% and 11.4% for 2 storey, 3 storey, 4 storey, and 5 storey respectively.

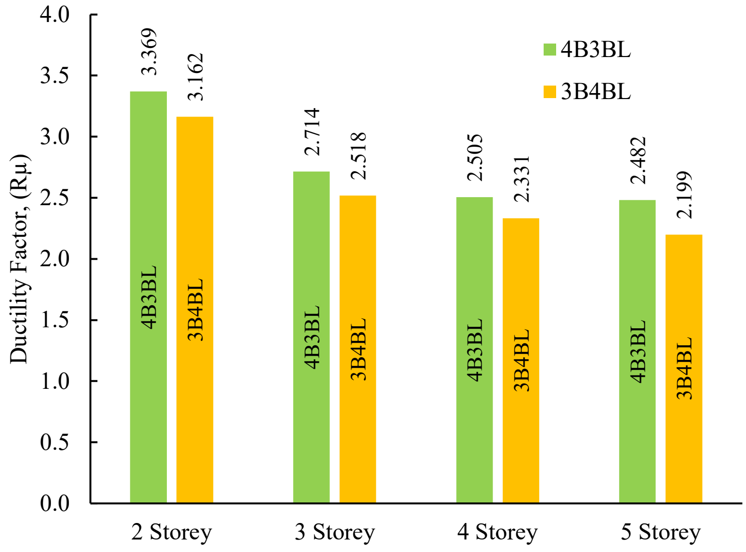

Figure 5-10: Ductility Factor for Model having Same Length (12m)

5.3.2 Due to Structural Member Size

The ductility factor decreased by the lowest 10.23% to the highest 20.93% times when difference between the structural member size assigned to them increased. This can be attributed to the concept that increasing the member sizes increases its lateral stiffness which makes the structure stiffer resulting in less plastic deformation. The Table 5-3 shows the member sizes used with its code assigned.

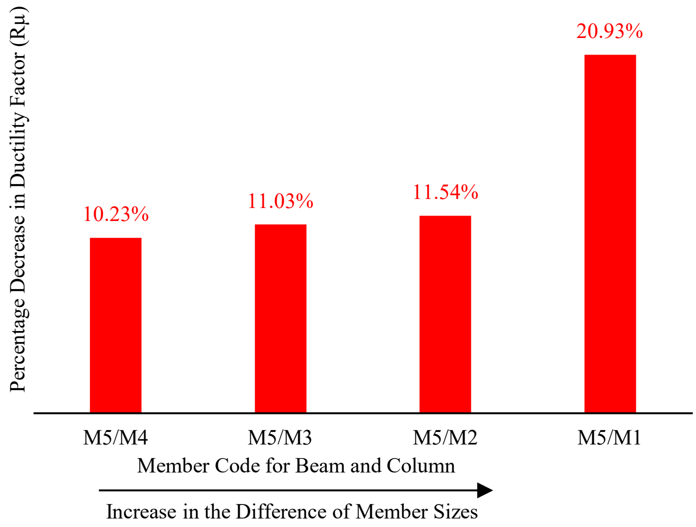

Figure 5-11: Effect on Ductility Factor by Difference in Member Sizes

Where M5/M4 represents a model with member sizes M4 which is then replaced by member sizes M5 and it is similar for all the other designation mentioned in the Figure 5-11.

5.4 Generalized Equation for Overstrength Factor and Ductility Factor

A generalized equation has been proposed to calculate the overstrength factor and ductility factor by carrying out the regression analysis. The factors considered are number of storeys, number of bays, bay length and structure member sizes. The equation is first provided for a model with base structure member sizes for each storey and later can be modified for other structure member sizes. The base structure member sizes are the smallest member sizes required for each number of storey building to satisfy the necessary design and check criteria as per NBC 105: 2020 for the applied loads on the models.

Table 5-6: Base Size of Beam and Column for a Storey

For Storey	Base Beam Dimension (D X B)	Base Column Dimension (D X B)
2	14” X 9”	12” X 12”
3	14” X 9”	13” X 13”
4	14” X 10”	14” X 14”
5	16” X 10”	15” X 15”

Where, the maximum depth and width considered in this study are 16” X 12” for beam and 16” X 16” for column respectively.

Overstrength Factor and Ductility Factor for Base Structure Member Sizes:

$$\Omega_u = 4.4668 - 0.1233S - 0.0601B - 0.4627BL \tag{5.1}$$

$$R_\mu = 4.6839 - 0.2603S - 0.0502B - 0.2719BL \tag{5.2}$$

Where,

Ω_u = Overstrength Factor

R_μ = Ductility Factor

S = Number of Storey

B = Number of Bays

BL = Bay Length

The equation 5.1 and 5.2 gives overstrength factor and ductility factor for different configuration of building with base structure member sizes in Table 5-6, but also can be modified to incorporate the effect of column and beam sizes on overstrength factor and ductility factor by introducing a new factor as Δ_Ω and Δ_R for overstrength factor and ductility factor respectively which then enable to obtain overstrength factor and ductility factor for model with different structure member size rather than just base structure member sizes.

$$\Omega_u = \mathit{\Delta}_\Omega\,(4.4668 - 0.1233S - 0.0601B - 0.4627BL) \tag{5.3}$$

$$R_\mu = \mathit{\Delta}_R\,(4.6839 - 0.2603S - 0.0502B - 0.2719BL) \tag{5.4}$$

Where,

$$\mathit{\Delta}_\Omega = (1.0444 - 0.2581\Delta_{Bb} - 0.2843\Delta_{Bd} + 0.6058\Delta_{Cbd}) \tag{5.5}$$

$$\mathit{\Delta}_R = (0.8650 + 0.1065\Delta_{Bb} + 0.0597\Delta_{Bd} - 0.1195\Delta_{Cbd}) \tag{5.6}$$

Where,

Δ_{Bb} = Increase in beam width from base size in inches

Δ_{Bd} = Increase in beam depth from base size in inches

Δ_{Cbd} = Increase in column size in anyone direction from base size in inches

Note that only the square column is considered having same width and depth.

A model of 2 storey, 2 bay and 3.5m bay length having member sizes of code M1 and M5 (mentioned in Table 4-2) is chosen for the comparison of values obtained from the formula mentioned in Section 3.7 and proposed equation mentioned in Section 5.4.

Table 5-7: Variation Coming from the Proposed Equation

	Member Assigned	From Analysis	From Proposed Equation	Variation
Overstrength Factor (Ω_u)	M1	2.526	2.481	1.78%
	M5	5.415	5.271	2.66%
Ductility Factor (R_μ)	M1	3.219	3.111	3.36%
	M5	2.673	2.570	3.85%

Since the variation between the values obtained from analysis and proposed equation is quite low, the proposed equation is acceptable.

CHAPTER-6 CONCLUSIONS

6.1 Conclusions

A total of 72 buildings models were analysed to obtain the overstrength factor (Ω_u) and ductility factor (R_μ). The force vs displacement curves were obtained by non-linear static pushover analysis. Using extensive statistical tools, an empirical equation has been proposed for overstrength factor and ductility factor.

Table 6-1: Base Size of Beam and Column for a Storey

For Storey	Base Beam Dimension (D X B)	Base Column Dimension (D X B)
2	14” X 9”	12” X 12”
3	14” X 9”	13” X 13”
4	14” X 10”	14” X 14”
5	16” X 10”	15” X 15”

Where, the maximum depth and width considered in this study are 16” X 12” for beam and 16” X 16” for column respectively.

For Base Structure Member Sizes:

$$\Omega_u = 4.4668 - 0.1233S - 0.0601B - 0.4627BL \quad (6.1)$$

$$R_\mu = 4.6839 - 0.2603S - 0.0502B - 0.2719BL \quad (6.2)$$

Where,

Ω_u is Overstrength Factor, R_μ is Ductility Factor, S is Number of Storey, B is Number of Bays and BL is Bay Length.

The equation 6.1 and 6.2 can be modified to incorporate the effect of column and beam sizes on overstrength factor and ductility factor by introducing a new factor as Δ_Ω and Δ_R for overstrength factor and ductility factor respectively.

$$\Omega_u = \Delta_\Omega\,(4.4668 - 0.1233S - 0.0601B - 0.4627BL) \quad (6.3)$$

$$R_\mu = \Delta_R\,(4.6839 - 0.2603S - 0.0502B - 0.2719BL) \quad (6.4)$$

Where,

$$\Delta_\Omega = (1.0444 - 0.2581\Delta_{Bb} - 0.2843\Delta_{Bd} + 0.6058\Delta_{Cbd}) \quad (6.5)$$

$$\Delta_R = (0.8650 + 0.1065\Delta_{Bb} + 0.0597\Delta_{Bd} - 0.1195\Delta_{Cbd}) \qquad (6.6)$$

Where,

Δ_{Bb} = Increase in beam width from base size in inches

Δ_{Bd} = Increase in beam depth from base size in inches

Δ_{Cbd} = Increase in column size in anyone direction from base size in inches

Note that only the square column is considered having same width and depth.

In addition to the formulation of empirical relation, the following conclusions has been made from the analytical study carried out by varying the building configurations and structural member sizes.

- Both the overstrength factor (Ω_u) and ductility factor (R_μ) is dependent upon many parameters such as building configuration and member sizes. Using a single value for them will introduce the unwanted uncertainty in the building.
- The dependency on bay length is more than the number of bays for both the overstrength factor and ductility factor. For the overstrength factor the effect of bay length and number of bays reduced as the number of storey increased but it is opposite in the case of ductility factor.
- According to NBC 105: 2020 for ultimate limit state, the value of overstrength factor and ductility factor are 1.5 and 4 respectively for RC moment resisting frame. The value obtained from the analysis showed the higher value (>1.5) for overstrength factor ranging from 1.886 to 2.873 while for the ductility factor the value fluctuated from 2.161 to 3.283 which were less than (<4) that specified in the code.
- For a given fixed span, if it is divided in a way that has a higher number of bays by reducing the bay length, then the value for the overstrength factor (Ω_u) and ductility factor (R_μ) increased.
- The overstrength factor increased and ductility factor decreased while providing higher sizes of column and beam than required.

These conclusions are limited to the scope of the work carried out in this research. More wider parameters need to be included to reduce the limitations of this research in order to accurately predict the overstrength factor and ductility factor.

6.2 Recommendation for Further Work

The overstrength factor and ductility factor are governed by many parameters. Among them the selected parameters in this study were number of storey, number of bays, bay length and structural member sizes. The limited parameters have been considered in this study. Some of the limitations of this study that can be incorporated in future studies are as follows: -

1. The non-linear time history analysis can be performed to obtain a more accurate value of overstrength factor and ductility factor.
2. Soil structure interaction (SSI) can be included in the analysis process to reflect the real condition in the site for these models.
3. The effect of infill walls can be considered to better match the real behaviour at the field.
4. The material properties for reinforcements and concrete can also be considered as one of the varying parameters.
5. User defined hinges can be considered in place of default hinges which is used in this study.
6. Redundancy factor can also be incorporated in the overall response reduction factor.
7. This research can be extended by considering the different seismic zone factor.

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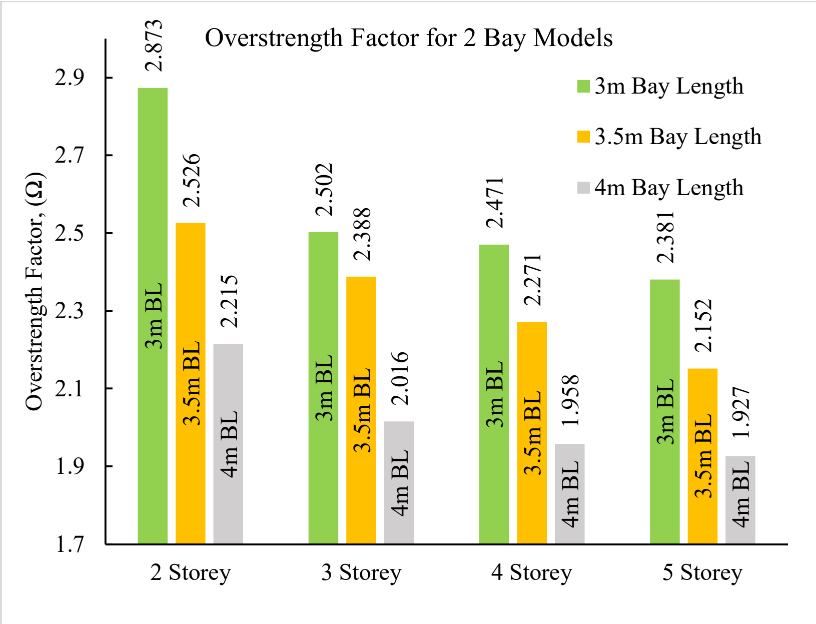
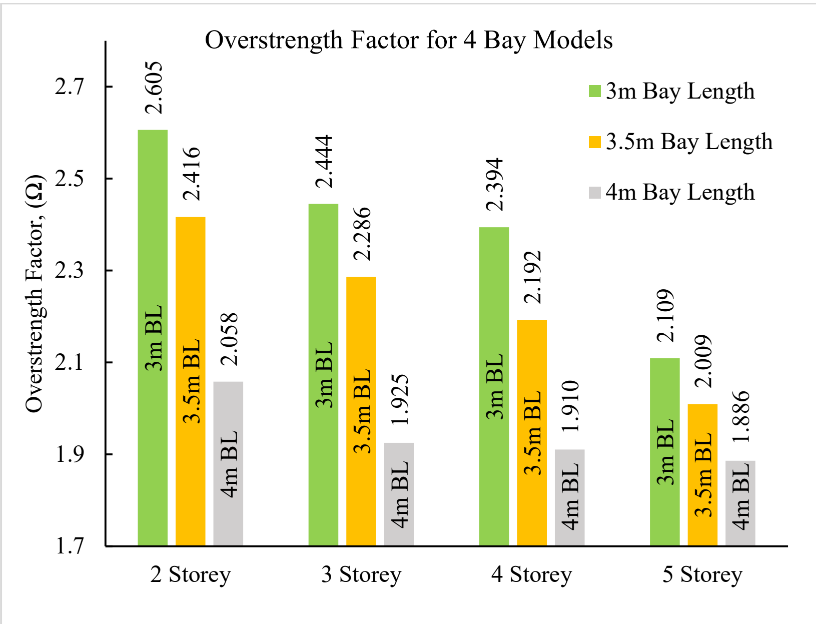
APPENDIX

Appendix 1: Calculation of Overstrength Factor and Ductility Factor

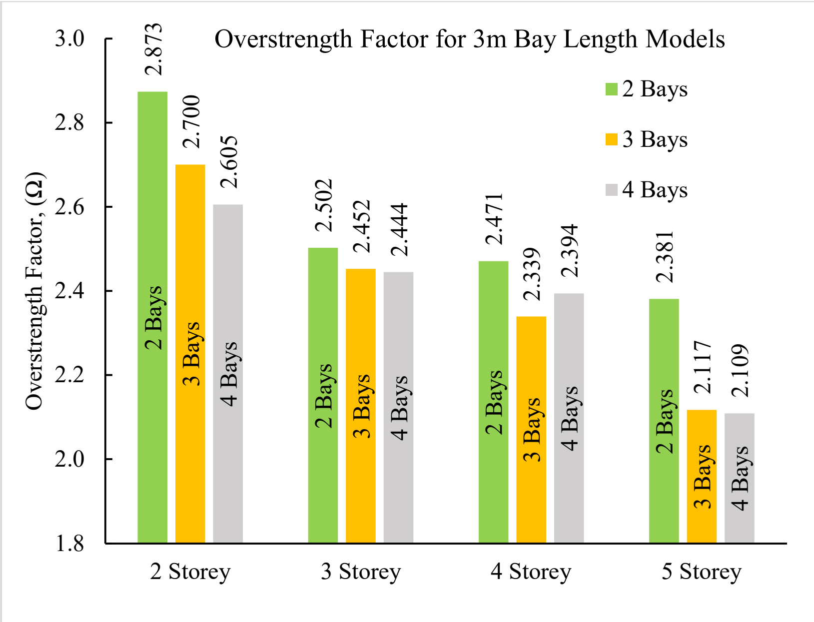
Modal Name	V_d	V_y	d_u	d_y	T	μ	φ	R_μ	Ω_u
2S2B3BLM1	167.4	481.1	85.1	22.6	0.41	3.77	1.21	3.283	2.873
2S2B3.5LM1	204.2	515.8	88.6	25.2	0.46	3.51	1.13	3.219	2.526
2S2B4BLM1	244.1	540.6	87.4	26.5	0.51	3.30	1.06	3.177	2.215
2S3B3BLM1	329.6	890.1	84.1	22.2	0.41	3.80	1.21	3.320	2.700
2S3B3.5LM1	405.6	994.4	89.3	25.7	0.47	3.47	1.11	3.216	2.452
2S3B4BLM1	488.5	1039.6	90.1	27.7	0.52	3.25	1.04	3.162	2.128
2S4B3BLM1	544.6	1419.1	83.8	21.9	0.42	3.83	1.19	3.369	2.605
2S4B3.5LM1	673.6	1627.6	90.3	26.0	0.47	3.47	1.11	3.232	2.416
2S4B4BLM1	815.2	1677.4	90.2	27.6	0.51	3.26	1.05	3.154	2.058
3S2B3BLM2	258.2	646.2	106.0	38.6	0.56	2.75	0.98	2.773	2.502
3S2B3.5LM2	313.5	748.6	113.9	47.0	0.63	2.42	0.90	2.575	2.388
3S2B4BLM2	373.5	752.9	102.7	45.1	0.70	2.28	0.84	2.522	2.016
3S3B3BLM2	507.1	1243.6	103.8	38.6	0.56	2.69	0.98	2.728	2.452
3S3B3.5LM2	621.4	1448.6	112.5	47.1	0.64	2.39	0.90	2.550	2.331
3S3B4BLM2	746.3	1490.8	114.1	50.4	0.71	2.26	0.83	2.518	1.998
3S4B3BLM2	836.9	2045.9	103.7	38.9	0.57	2.67	0.97	2.714	2.444
3S4B3.5LM2	1031.1	2357.0	111.0	46.9	0.64	2.37	0.89	2.538	2.286
3S4B4BLM2	1244.1	2395.2	109.8	49.4	0.67	2.22	0.86	2.412	1.925
4S2B3BLM3	357.5	883.3	140.2	57.0	0.69	2.46	0.85	2.719	2.471
4S2B3.5LM3	432.3	981.6	150.7	70.0	0.78	2.15	0.79	2.462	2.271
4S2B4BLM3	513.4	1005.0	149.1	73.6	0.87	2.02	0.75	2.366	1.958
4S3B3BLM3	720.8	1685.8	135.1	58.1	0.73	2.33	0.82	2.624	2.339
4S3B3.5LM3	855.2	1898.2	144.5	69.3	0.79	2.09	0.78	2.385	2.219
4S3B4BLM3	1023.9	1970.6	148.4	74.4	0.88	2.00	0.75	2.331	1.925
4S4B3BLM3	1155.6	2766.5	132.3	58.3	0.70	2.27	0.84	2.505	2.394
4S4B3.5LM3	1417.9	3108.2	140.1	68.9	0.79	2.03	0.78	2.323	2.192
4S4B4BLM3	1705.4	3257.3	140.8	75.3	0.88	1.87	0.75	2.168	1.910
5S2B3BLM4	465.9	1109.2	170.1	73.5	0.78	2.32	0.79	2.666	2.381
5S2B3.5LM4	561.0	1207.2	182.8	82.8	0.88	2.21	0.75	2.608	2.152
5S2B4BLM4	664.1	1279.4	183.2	94.2	0.98	1.95	0.73	2.293	1.927
5S3B3BLM4	911.6	1930.0	144.4	65.2	0.78	2.22	0.79	2.542	2.117
5S3B3.5LM4	1108.0	2241.8	166.9	80.0	0.88	2.09	0.75	2.455	2.023
5S3B4BLM4	1322.3	2503.4	175.8	93.7	0.99	1.88	0.73	2.199	1.893
5S4B3BLM4	1501.8	3167.4	140.4	64.8	0.78	2.17	0.79	2.482	2.109
5S4B3.5LM4	1835.4	3688.0	162.7	79.7	0.88	2.04	0.75	2.397	2.009
5S4B4BLM4	2200.6	4150.3	173.6	94.0	0.99	1.85	0.73	2.161	1.886

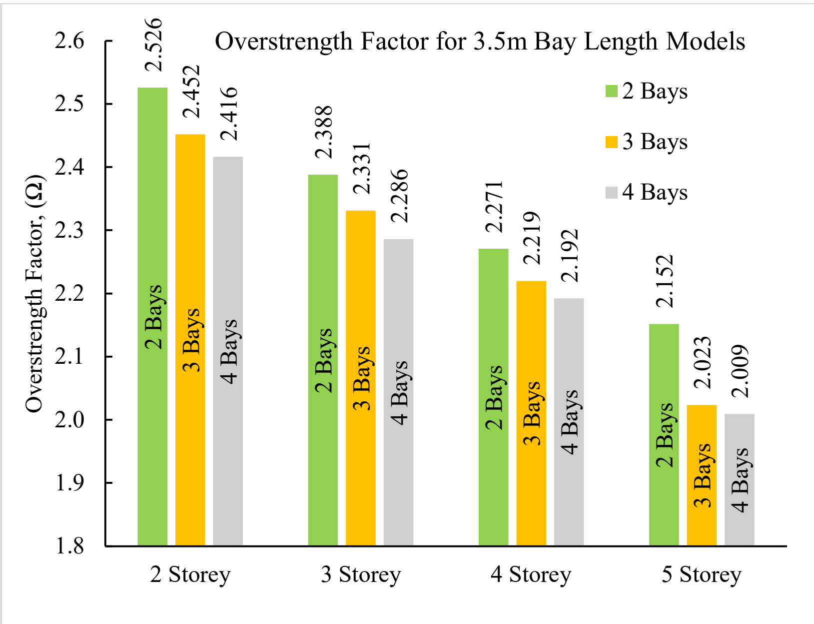
Appendix 2: Graphs of Overstrength Factor for Different Bay Models



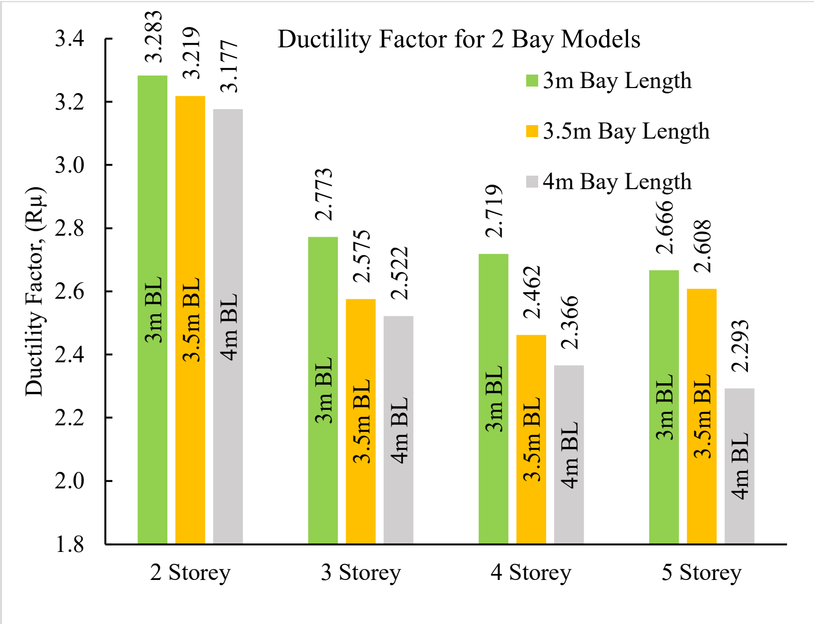


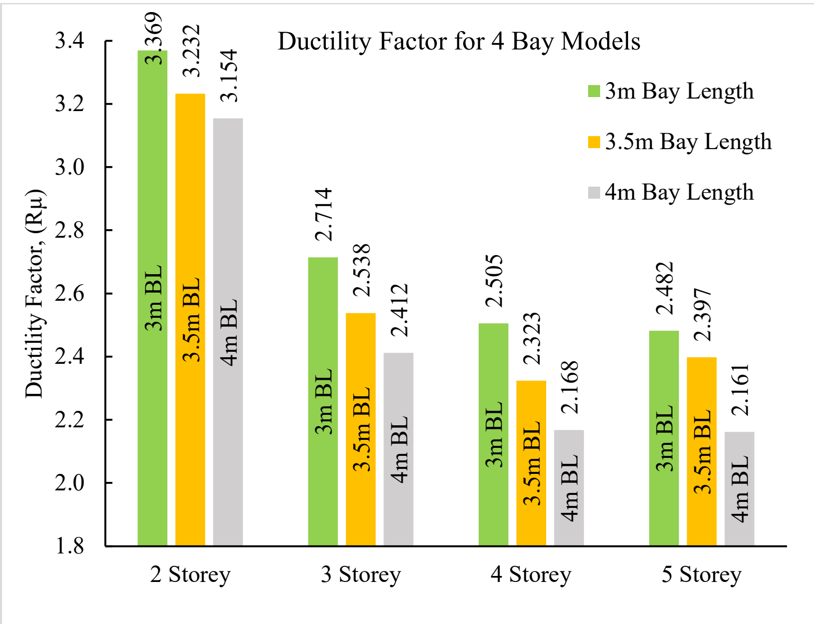
Appendix 3: Graphs of Overstrength Factor for Different Bay Length Models



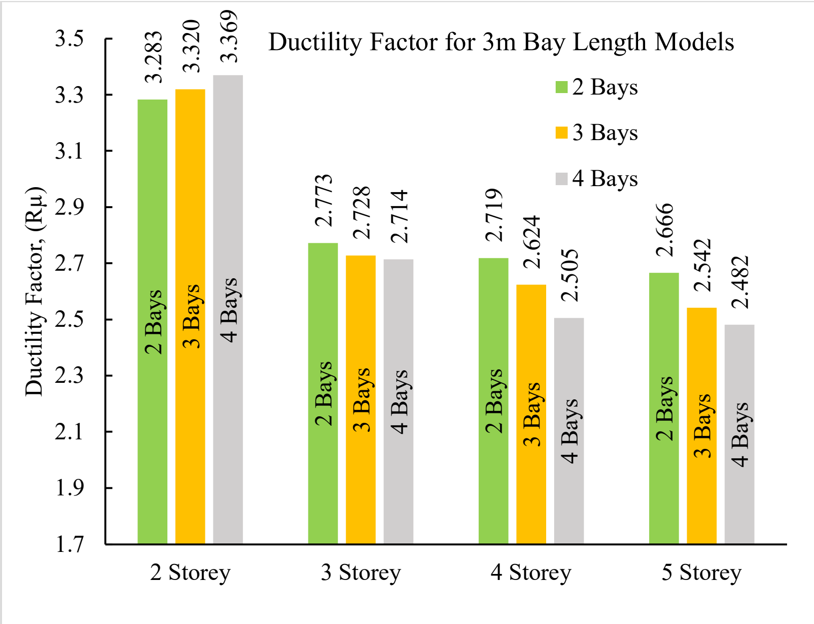


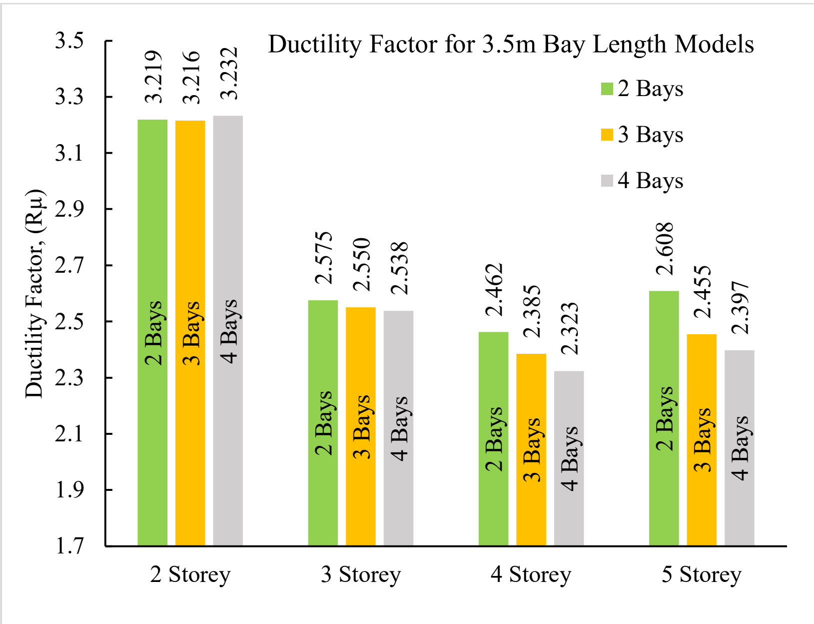
Appendix 4: Graphs of Ductility Factor for Different Bay Models





Appendix 4: Graphs of Ductility Factor for Different Bay Length Models





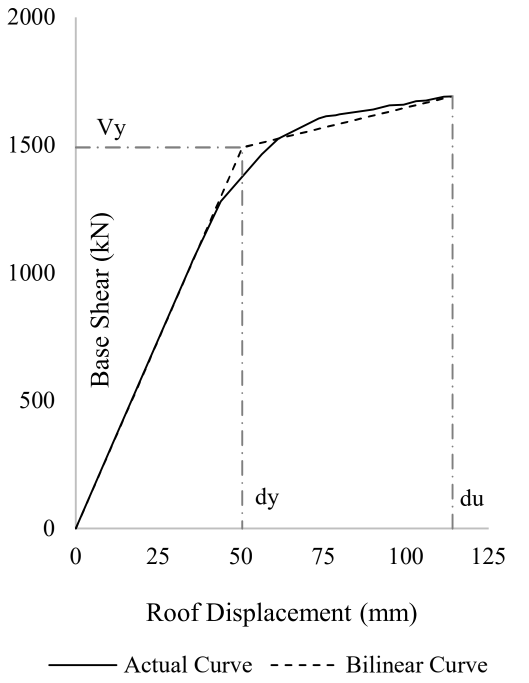
Appendix 5: Reinforcement Detailing of All the Model

Model Name	Storey	Beam Reinforcement		Column Reinforcement
		Top	Bottom	
2S2B3BLM1	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
2S2B3.5LM1	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
2S2B4BLM1	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
2S3B3BLM1	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
2S3B3.5LM1	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
2S3B4BLM1	Storey 2	3#12	3#12	8#20
	Storey 1	3#12	3#12	8#20
2S4B3BLM1	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
2S4B3.5LM1	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
2S4B4BLM1	Storey 2	3#12	3#12	8#20
	Storey 1	1#16+2#12	3#12	8#20
3S2B3BLM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
3S2B3.5LM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	1#16+2#12	3#12	4#20+4#16
	Storey 1	1#16+2#12	3#12	4#20+4#16
3S2B4BLM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	2#16+1#12	3#12	4#20+4#16
	Storey 1	2#16+1#12	3#12	4#20+4#16
3S3B3BLM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
3S3B3.5LM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	1#16+2#12	3#12	4#20+4#16
	Storey 1	1#16+2#12	3#12	4#20+4#16
3S3B4BLM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	1#16+2#12	3#12	4#20+4#16
	Storey 1	2#16+1#12	3#12	4#20+4#16
3S4B3BLM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	3#12	3#12	4#20+4#16
	Storey 1	3#12	3#12	4#20+4#16
3S4B3.5LM2	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	1#16+2#12	3#12	4#20+4#16
	Storey 1	1#16+2#12	3#12	4#20+4#16
3S4B4BLM2	Storey 3	3#12	3#12	4#20+4#16

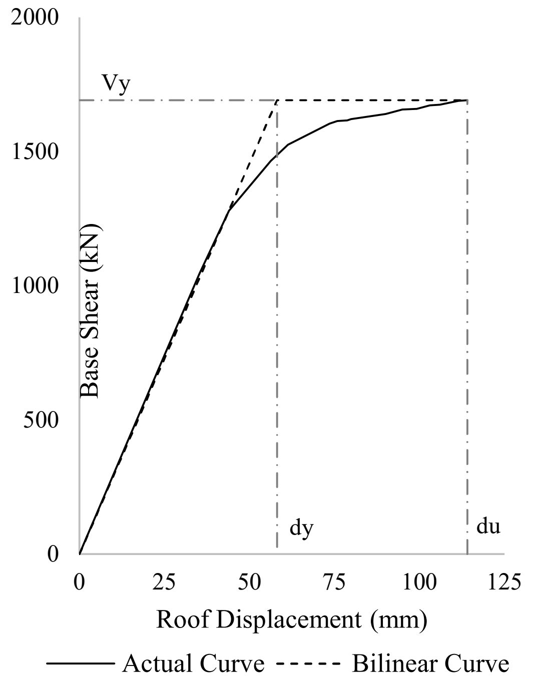
	Storey 2	2#16+1#12	3#12	4#20+4#16
	Storey 1	3#16	1#16+2#12	4#20+4#16
4S2B3BLM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	1#16+2#12	3#12	4#20+4#16
	Storey 1	1#16+2#12	3#12	4#20+4#16
4S2B3.5LM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	1#16+2#12	3#12	4#20+4#16
	Storey 2	2#16+1#12	1#16+2#12	4#20+4#16
	Storey 1	2#16+1#12	1#16+2#12	4#20+4#16
4S2B4BLM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	2#16+1#12	3#12	4#20+4#16
	Storey 2	1#20+2#16	2#16+1#12	4#20+4#16
	Storey 1	1#20+2#16	2#16+1#12	8#20
4S3B3BLM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	1#16+2#12	3#12	4#20+4#16
	Storey 1	1#16+2#12	3#12	4#20+4#16
4S3B3.5LM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	1#16+2#12	3#12	4#20+4#16
	Storey 2	2#16+1#12	1#16+2#12	4#20+4#16
	Storey 1	3#16	1#16+2#12	4#20+4#16
4S3B4BLM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	2#16+1#12	3#12	4#20+4#16
	Storey 2	1#20+2#16	2#16+1#12	4#20+4#16
	Storey 1	1#20+2#16	2#16+1#12	8#20
4S4B3BLM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	3#12	3#12	4#20+4#16
	Storey 2	1#16+2#12	3#12	4#20+4#16
	Storey 1	1#16+2#12	3#12	4#20+4#16
4S4B3.5LM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	1#16+2#12	3#12	4#20+4#16
	Storey 2	2#16+1#12	3#12	4#20+4#16
	Storey 1	3#16	1#16+2#12	4#20+4#16
4S4B4BLM3	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	2#16+1#12	3#12	4#20+4#16
	Storey 2	1#20+2#16	2#16+1#12	4#20+4#16
	Storey 1	1#20+2#16	2#16+1#12	8#20
5S2B3BLM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	1#16+2#12	3#12	4#20+4#16
	Storey 2	2#16+1#12	1#16+2#12	8#20
	Storey 1	2#16+1#12	1#16+2#12	4#25+4#20
5S2B3.5LM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	1#16+2#12	3#12	4#20+4#16
	Storey 3	3#16	1#16+2#12	4#20+4#16
	Storey 2	1#20+2#16	2#16+1#12	8#20

	Storey 1	3#16	2#16+1#12	4#25+4#20
5S2B4BLM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	2#16+1#12	3#12	4#20+4#16
	Storey 3	1#20+2#16	2#16+1#12	4#20+4#16
	Storey 2	2#20+1#16	3#16	8#20
	Storey 1	2#20+1#16	3#16	4#25+4#20
5S3B3BLM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	1#16+2#12	3#12	4#20+4#16
	Storey 2	2#16+1#12	1#16+2#12	8#20
	Storey 1	2#16+1#12	1#16+2#12	4#25+4#20
5S3B3.5LM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	1#16+2#12	3#12	4#20+4#16
	Storey 3	3#16	1#16+2#12	4#20+4#16
	Storey 2	1#20+2#16	2#16+1#12	8#20
	Storey 1	1#20+2#16	2#16+1#12	4#25+4#20
5S3B4BLM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	2#16+1#12	3#12	4#20+4#16
	Storey 3	1#20+2#16	2#16+1#12	4#20+4#16
	Storey 2	2#20+1#16	3#16	8#20
	Storey 1	2#20+1#16	3#16	4#25+4#20
5S4B3BLM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	3#12	3#12	4#20+4#16
	Storey 3	1#16+2#12	3#12	4#20+4#16
	Storey 2	2#16+1#12	1#16+2#12	8#20
	Storey 1	2#16+1#12	1#16+2#12	4#25+4#20
5S4B3.5LM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	1#16+2#12	3#12	4#20+4#16
	Storey 3	2#16+1#12	1#16+2#12	4#20+4#16
	Storey 2	1#20+2#16	2#16+1#12	8#20
	Storey 1	1#20+2#16	2#16+1#12	4#25+4#20
5S4B4BLM4	Storey 5	3#12	3#12	4#20+4#16
	Storey 4	2#16+1#12	3#12	4#20+4#16
	Storey 3	1#20+2#16	2#16+1#12	4#20+4#16
	Storey 2	2#20+1#16	3#16	8#20
	Storey 1	2#20+1#16	3#16	4#25+4#20

Appendix 6: Different Techniques of Bilinearization and its Comparison



Idealized Bilinear Curve of Model 3S3B4BLM2 by Provision in FEMA 356:2000



Idealized Bilinear Curve of Model 3S3B4BLM2 by Provision of Elasto-Plastic Response

Fundamental time period, (T) = 0.71 sec	Fundamental time period, (T) = 0.71 sec
Design lateral strength, (V_d) = 746 kN	Design lateral strength, (V_d) = 746 kN
Yield strength, (V_y) = 1490 kN	Yield strength, (V_y) = 1691 kN
Yield displacement, (d_y) = 50 mm	Yield displacement, (d_y) = 58 mm
Ultimate displacement, (d_u) = 114 mm	Ultimate displacement, (d_u) = 114 mm
Ultimate strength, (V_u) = 1690 kN	Ultimate strength, (V_u) = 1690 kN
Ductility factor, (R_μ) = 2.517	Ductility factor, (R_μ) = 2.164
Overstrength factor, (Ω_u) = 1.997	Overstrength factor, (Ω_u) = 2.266

Model Name	FEMA 356:2000		Elasto-Plastic		Percentage Change	
	R_μ	Ω_u	R_μ	Ω_u	R_μ	Ω_u
2S2B3BLM1	3.283	2.873	2.685	3.259	18.23%	-13.42%
2S2B3.5LM1	3.219	2.526	2.682	2.857	16.70%	-13.09%
2S2B4BLM1	3.177	2.215	2.677	2.519	15.73%	-13.74%
2S3B3BLM1	3.32	2.7	2.850	3.008	14.14%	-11.40%
2S3B3.5LM1	3.216	2.452	2.789	2.725	13.28%	-11.12%
2S3B4BLM1	3.162	2.128	2.795	2.364	11.61%	-11.10%

2S4B3BLM1	3.369	2.605	2.930	2.903	13.04%	-11.43%
2S4B3.5LM1	3.232	2.416	2.866	2.678	11.32%	-10.84%
2S4B4BLM1	3.154	2.058	2.805	2.276	11.06%	-10.57%
3S2B3BLM2	2.773	2.502	2.190	2.896	21.04%	-15.75%
3S2B3.5LM2	2.575	2.388	2.086	2.744	18.98%	-14.90%
3S2B4BLM2	2.522	2.016	1.999	2.307	20.74%	-14.45%
3S3B3BLM2	2.728	2.452	2.261	2.798	17.12%	-14.09%
3S3B3.5LM2	2.55	2.331	2.191	2.625	14.06%	-12.59%
3S3B4BLM2	2.518	1.998	2.164	2.266	14.05%	-13.40%
3S4B3BLM2	2.714	2.444	2.300	2.749	15.26%	-12.47%
3S4B3.5LM2	2.538	2.286	2.211	2.550	12.87%	-11.53%
3S4B4BLM2	2.412	1.925	2.063	2.167	14.47%	-12.57%
4S2B3BLM3	2.719	2.471	2.006	3.023	26.23%	-22.33%
4S2B3.5LM3	2.462	2.271	1.914	2.725	22.24%	-19.97%
4S2B4BLM3	2.366	1.958	1.790	2.385	24.37%	-21.82%
4S3B3BLM3	2.624	2.339	2.081	2.737	20.68%	-17.00%
4S3B3.5LM3	2.385	2.219	1.939	2.581	18.69%	-16.30%
4S3B4BLM3	2.331	1.925	1.866	2.252	19.96%	-17.00%
4S4B3BLM3	2.505	2.394	2.060	2.751	17.78%	-14.93%
4S4B3.5LM3	2.323	2.192	1.935	2.509	16.70%	-14.45%
4S4B4BLM3	2.168	1.91	1.833	2.155	15.43%	-12.84%
5S2B3BLM4	2.666	2.381	1.869	2.967	29.88%	-24.62%
5S2B3.5LM4	2.608	2.152	1.728	2.751	33.73%	-27.82%
5S2B4BLM4	2.293	1.927	1.659	2.380	27.67%	-23.53%
5S3B3BLM4	2.542	2.117	1.824	2.599	28.24%	-22.77%
5S3B3.5LM4	2.455	2.023	1.820	2.451	25.86%	-21.16%
5S3B4BLM4	2.199	1.893	1.679	2.278	23.66%	-20.33%
5S4B3BLM4	2.482	2.109	1.846	2.543	25.61%	-20.56%
5S4B3.5LM4	2.397	2.009	1.843	2.394	23.10%	-19.16%
5S4B4BLM4	2.161	1.886	1.708	2.228	20.98%	-18.13%