“ANALYSIS OF PLASTIC HINGE PROPERTIES DURING NON LINEAR ANALYSIS OF REINFORCED CONCRETE STRUCTURES”

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by

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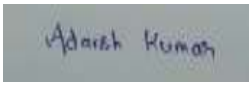

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ABSTRACT

In reinforced concrete structures, strong lateral forces are generated during earthquakes and large inelastic deformations and damage occur. It is important to study the performance of RC buildings under nonlinear situations to evaluate their safety and performance in seismic situations. Since plastic hinge formation in structures indicates where energy is dissipated and permanent deformations are developed, it is a significant factor in determining the behaviour of a structure during an earthquake.

In this study, the properties of plastic hinges are analyzed during the NLSA of RC structure by pushover and Time History Analysis. It examines hinge behavior of various building types for Target Displacement and for seismic loads respectively. The research involves two types of buildings: small buildings, tall buildings; and regular buildings, irregular buildings. Nonlinear behavior of these buildings is evaluated based on the sequence of hinge formation, states of hinges, base shear capacity, displacement capacity and performance levels.

Modelling and analysis of the buildings are done using ETABS software. The locations of the nonlinear hinges are specified in accordance with FEMA-356 and ATC-40.

Pushover analysis is performed both in X and Y directions. The results are shown in the form of pushover curves, hinge formation pattern, performance point, roof displacement, story drift and hinge state distribution.

The Time History analysis is perform out in direction X and direction Y .The output are presented using Base Shear, Hinge formation patterns, Time period, story drift and hinges states.

The study finds that building height and structural irregularity have a more impact on hinge development and dynamic performance. Hinge formation and displacement demands are significant and early in tall and irregular buildings, compared to regular and low-rise buildings. This study highlights the significance of hinge behavior in relation to seismic vulnerability of RCC buildings.

Keywords: Plastic Hinges, Pushover Analysis, Time History Analysis, NLSA, NLDA RC Structures, FEMA 356, ATC-40, ETABS etc.

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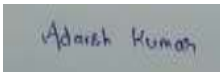

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Table of contents

CANDIDATE'S DECLARATION ii
CERTIFICATE BY THE SUPERVISOR iii
ABSTRACT iv
ACKNOWLEDGEMENT v
Table of contents vi
List of Figures x
List of Tables xi
List of Symbols xii
1 CHAPTER 1 1
1.1 General introduction 1
1.2 Seismic behaviour of RCC structures 3
1.3 Non linear behaviour of structures 4
1.4 Platic hinge concepts 5
1.5 Types of plastic hinges 6
1.5.1 Flexural hinges 6
1.5.2 Shear hinges 7
1.5.3 Axial hinges 8
1.6 Non linear static analysis 9
1.7 Pushover analysis 11
1.8 Non linear dynamic analysis 12
1.9 Non linear time history analysis 13
1.10 Scale and configuration impact 13
1.11 Performance level of structure 14
1.11.1 Immediate occupancy(IO) 15
1.11.2 Life safety (LS) 16
1.11.3 Collapse prevention (CP) 17
1.12 Objective of the study 18
1.13 Scope of the study 19
2 CHAPTER 2 20
2.1 Introduction 20
2.2 Overview of previous research 22
2.3 Research gap 28
3 CHAPTER 3 30

3.1 Introduction 30
3.2 Structural Dynamics fundamentals 31
3.3 Earthquake ground motion 32
3.4 Dynamic characteristics of the structure 33
3.4.1 Natural time period 33
3.4.2 Frequency 33
3.4.3 Mode shapes 34
3.4.4 Damping 34
3.5 Geometric non linearity 35
3.6 P-Delta Effects 36
3.7 Moment-Curvature Relationship 36
3.8 Force-Deformation Behaviour 37
3.9 Modal effects in non-linear analysis 38
3.10 Torsional irregularity 38
3.11 Codal provisions 39
3.12 Summary 39
4 CHAPTER 4 41
4.1 Introduction 41
4.2 Description of Structural Models 41
4.2.1 Plan and Elevation Details 43
4.2.2 Material Properties 43
4.2.3 Concrete Properties 43
4.2.4 Steel Reinforcement Properties 44
4.3 Loading Criteria 44
4.3.1 Dead Load 44
4.3.2 Live Load 44
4.4 Modeling in ETABS 45
4.4.1 Grid and Storey 3D view 46
4.4.2 Member Section Properties 49
4.4.3 Support Conditions 52
4.4.4 Diaphragm Assignment 52
4.4.5 Mass source Definition 53
4.5 Plastic Hinge Modeling 53
4.5.1 Hinge Properties for Beams 53
4.5.2 Hinge Properties for Columns 54

4.5.3 Force-Deformation Relationship (FEMA 356) 55
4.6 EQ Parameters and IS 1893:2016 Provisions 56
4.7 Non-Linear Static (Pushover) Analysis 56
4.7.1 Pushover Load Case Definition 57
4.7.2 Target Displacement 58
4.8 Non-Linear Dynamic (Time History) Analysis 59
4.8.1 Time History Load Case Definition 59
4.8.2 Earthquake Data 60
4.9 Performance Evaluation Methodology 60
4.10 Step-by-Step Procedure in ETABS 61
4.11 Summary 62
5 CHAPTER 5 64
5.1 Non-Linear Static Pushover Analysis (Case 1: Short vs. Tall Frame) 64
5.1.1 Natural Periods and Modal Mass Participation 64
5.1.2 Pushover Capacity Curves 65
5.1.3 Performance Point 67
5.1.4 Inter-Storey Drift Ratio 68
5.1.5 Plastic Hinge Distribution and Performance States 69
5.1.6 Step-by-Step Hinge Progression 70
5.1.7 Ductility Ratio 72
5.1.8 Roof Displacement 72
5.1.9 Comprehensive Comparison — Model A vs. Model B 73
5.2 Non-Linear Dynamic Time History Analysis (Case 2: Regular vs. Irregular Frame) 74
5.2.1 Modal Analysis — Time Period and Frequency 74
5.2.2 Story Displacement 75
5.2.3 Story Drift Ratio 76
5.2.4 Base Reactions 77
5.2.5 Plastic Hinge Formation and Performance States 77
5.2.6 Overall Structural Performance Summary 79
5.3 Discussions 80
5.3.1 Effect of Building Height on Plastic Hinge Properties 80
5.3.2 Effect of Plan Irregularity on Plastic Hinge Properties 80
5.3.3 Strong Column–Weak Beam Compliance 81
5.3.4 Code Compliance Assessment 81
5.4 Performance Level Summary 82

5.5 Summary of Key Findings 82
6 CHAPTER 6 84
6.1 Conclusions 84
6.1.1 Effect of Building Height on Seismic Performance (Case 1: Model A vs. Model B) 84
6.1.2 Effect of Plan Irregularity on Seismic Performance (Case 2: Model C vs. Model D) 85
6.2 General Conclusions 86
6.3 Future Scope 87
7 REFERENCES 89

List of Figures

Figure 4.1Concrete Properties 43
Figure 4.2-Steel Reinforcement Properties 44
Figure 4.3-Grid view of GF to 7th floor irregular building 46
Figure 4.4-Grid up to 8th to 14th floor irregular building 46
Figure 4.5-3D of irregular building 47
Figure 4.6-Grid view of regular building 47
Figure 4.7-3D view of regular building 48
Figure 4.8-Grid view of regular building 48
Figure 4.9-3D of regular building 49
Figure 4.10-Column properties of small building 49
Figure 4.11- Beam Properties of small building 50
Figure 4.12-column properties tall/regular/irregular building 50
Figure 4.13-Beam properties tall/regular/irregular building 51
Figure 4.14- Slab Propertie 51
Figure 4.15-Diaphragm Data 52
Figure 4.16- Mass Source Data 53
Figure 4.17-Hinges assignment on beam 54
Figure 4.18-Hinges assignment on column 55
Figure 4.19-PS-X LOAD CASE 57
Figure 4.20-PS-Y LOAD CASE 58
Figure 4.21-TH-Y Load Case 59
Figure 4.22-TH-X Load Case 60
Figure 4.23-Earthquake Data 60
Figure 5.1-Push Over of Small building in Y direction 65
Figure 5.2-Push Over of Small building in X direction 66
Figure 5.3-Push Over of Tall building in X direction 66
Figure 5.4-Push Over of Tall building in Y direction 67
Figure 5.5-Plastic Hinge atTarget Displacment in Irregular Building 71
Figure 5.6-Plastic Hinge at Target Displacement irregular Building 71

List of Tables

Table 4-1Summary of Building Configuration for All Four Models 42
Table 4-2astic Hinge Force-Deformation Parameters (FEMA 356) 55
Table 4-3-Seismic parameters data 56
Table 4-5-Target Displacement Data 58
Table 4-6-Analysis Configuration Summary for All Four Models 61
Table 5-1-Natural Periods and Frequencies — Tall vs. Short Frame 64
Table 5-2-Modal participation 65
Table 5-3-performaton point 67
Table 5-4- Inter Storey Drift 68
Table 5-5: Plastic Hinge Distribution at Performance Point — Model A vs. Model B . 69
Table 5-6Step-by-Step Hinge Progression — Model A and Model B 70
Table 5-7Ductility Ratio Comparison — Model A vs. Model B 72
Table 5-8-Storey-wise Roof Displacement — Model A and Model B 72
Table 5-9Comprehensive Comparison — Model A vs. Model B 73
Table 5-10Comparison of Modal Time Periods and Frequencies — Regular vs. Irregular Building 74
Table 5-11Modal Mass Participation Ratios (%) 74
Table 5-12Maximum Story Displacements (mm) — Regular vs. Irregular Building ... 75
Table 5-13Maximum Story Drift Ratios — Regular vs. Irregular Building 76
Table 5-14 Base Reactions Comparison — Regular vs. Irregular Building 77
Table 5-15Plastic Hinge Count by Performance State — Regular vs. Irregular Building 78
Table 5-16 Plastic Hinge Rotation Demand 79
Table 5-17Overall Structural Performance Summary — Regular vs. Irregular Building 79
Table 5-18Governing Performance Level Summary — All Models 82

List of Symbols

m	Mass of the structure
c	Damping coefficient of the structure
k	Lateral stiffness of the structure
x (or u)	Displacement response of the structure
$\dot{x}$	Velocity response (first derivative of displacement)
$\ddot{x}$	Acceleration response (second derivative of displacement)
p(t)	Dynamic (time-varying) external force
T	Natural time period of vibration
T_1	Fundamental (first mode) natural time period
f	Natural frequency of the structure
ζ (zeta)	Damping ratio (ratio of actual to critical damping)
cc	Critical damping coefficient
Z	Seismic zone factor (IS 1893:2016 Table 3)
I	Importance factor (IS 1893:2016 Table 8)
R	Response reduction factor
Sa/g	Spectral acceleration coefficient (normalized)
Ta	Approximate fundamental natural period
h	Total height of the building
PGA	Peak ground acceleration
θ (theta)	Plastic hinge rotation (chord rotation)
θp	Plastic rotation demand at hinge
$\theta p,cap$	Plastic rotation capacity (FEMA 356 acceptance limit)
θy	Yield rotation (rotation at first yield)
θu	Ultimate rotation capacity of member
D/C	Demand-to-Capacity ratio for rotation at hinge

A, B, C, D, E	Key points on FEMA 356 force-deformation hinge curve (A=origin, B=yield, C=ultimate, D=residual, E=failure)
δ (delta)	Roof (or storey) lateral displacement
δt	Target displacement for pushover analysis (1% of building height)
δy	Yield displacement (from pushover capacity curve)
Δ (Delta)	Lateral displacement (used in P-Delta effects)
μ (mu)	Displacement ductility ratio ($\mu = \delta t / \delta y$)
ISDR	Inter-storey drift ratio (storey drift / storey height)
h/250	IS 1893:2016 permissible storey drift limit (= 0.004)
P	Vertical axial load (gravity load)
M	Bending moment (also secondary moment in P-Delta)
Mp	Plastic moment capacity of a section
M3	Uniaxial bending moment about the major axis (in ETABS)
V or VB	Base shear force
FX, FY, FZ	Base reaction forces in X, Y, Z directions
MX, MY, MZ	Base reaction moments about X, Y, Z axes
E	Young's modulus (modulus of elasticity)
I	Second moment of area (moment of inertia)
ϕ (phi)	Curvature of a section
ϕy	Yield curvature of a section
ϕu	Ultimate curvature of a section
fck	Characteristic compressive strength of concrete (cube)
fy	Yield strength of steel reinforcement
Ec	Modulus of elasticity of concrete
Es	Modulus of elasticity of steel

M30	Concrete grade: characteristic strength = 30 MPa
Fe415 / Fe500	Steel grade: yield strength 415 MPa / 500 MPa
NLSA	Non linear static analysis
RC	Reinforced concrete
NLTH	Non linear time history
EQ	Earthquake
NLSPA	Non linear static pushover analysis
NlTHA	Non linear time history analysis
SDOF	Single degree of freedom
MDOF	Multi degree of freedom
PGA	Peak ground acceleration
RS	Response spectrum
GM	Ground motion

CHAPTER 1

INTRODUCTION

1.1 General introduction

RC structure is an important component of the modern infrastructure worldwide. Residential houses, commercial centers and industrial sites, schools, hospitals, bridges and utility facilities are constructed from reinforced concrete. Its ability to being used in different architectural and structural demands, its durability, cost-effectiveness, high compressive strength and versatility make it a favored material. Seismic behaviour of RC structures is of great value in structural engineering in earthquake prone regions. Earthquakes produce lateral dynamic forces that give rise to significant stresses, deformities and energy dissipation requirements in structural elements. When these demands exceed the elastic limits of the materials, the structure is said to be a nonlinear structure. This nonlinear behavior is important to understand for safety, to avoid catastrophic failure, and to guarantee acceptable performance during earthquakes.

The seismic design philosophies have shifted from the old way of designing based on forces to the new way of designing based on performance. The conventional design methods are primarily concerned with including sufficient strength to withstand forces that are anticipated. In real structures however, there is significant inelastic deformations and energy dissipation due to yielding at high earthquake intensities. These effects include cracking, loss of stiffness, internal force redistribution and plastic hinge formation in beams, columns and other members due to strong ground motion. The EQ behavioure and ductility of the building depends on its capacity to resist these inelastic deformations that may occur without sudden rupture or failure.

Specific points in structural members where inelastic rotation occurs after reinforcement yields or the concrete crushes are called plastic hinges. The seismic response of a structure is where the energy dissipates and these hinges are essential to it. The distribution, particularly the order of formation of hinges, and their location have a great influence on the stability and resistance to collapse of buildings. Hence, the behaviour of the plastic hinges is an important consideration for the nonlinear

analysis of reinforced concrete structures, for seismic vulnerability assessment and seismic performance evaluation.

Beyond the elastic limits, structures' actual seismic response is typically evaluated using nonlinear analysis methods

This study aims to explore plastic hinge behavior based on pushover analysis and NLTHA of RC structures. It is a comparison of two main cases; the effect of building height with the comparison of a building which is tall and a building which is shorter and the effect of configuration with the comparison of regular and irregular buildings. These structures are analyzed by ETABS software with the concept of nonlinear behavior. The series of hinges arrangement , hinge state, base shear, displacement capacity, inter-storey drift, performance points and overall structural performance are analysed.

Seismic loading, structural design and material standards are considered following the guidelines provided in Indian Standard. Furthermore, FEMA 356 and ATC-40 are referenced for the procedures used for nonlinear analysis, performance levels, hinge characteristics and acceptance criteria. The aim of this study is to improve the knowledge of EQ behaviour and the behaviour of the hinges in RC structures under non-linear loading.

The earthquake loadings are dynamic and cause inertia forces to structures. In strong earthquakes, the need for the structural members may exceed their elastic limit. In such circumstances, structure will undergo cracking, yield and stiffness degradation. This actual behaviour cannot be captured by linear elastic analysis, as it assumes that the structure can be considered to be elastic during the loading time.

Thus, to simulate the behavior to severe loadings of RC building , NL methods are needed. These methods help in understanding:

- Yielding of members
- Redistribution of internal forces
- Energy dissipation capacity
- Failure mechanisms
- Collapse prevention behaviour

The pushover analysis is simple and efficient among various nonlinear methods, and is widely accepted. It gives information about the damage progression and the

formation of plastic hinges in structural elements.

The NL study of RC structures mainly depends on:

- Material nonlinearity
- Geometric nonlinearity
- Structural irregularity

The mass and stiffness

- Member ductility
- Loading pattern

In nonlinear response the plastic hinges are typically located at the beam and column connections, beam ends and column ends where maximum bending moments occur. The EQ behavioure of the structure will depend on the location and sequence of hinge formation

1.2 Seismic behaviour of RCC structures

The behavior of reinforced concrete buildings under seismic excitations depends on various factors such as structural system, stiffness distribution, mass distribution, level of ductility, damping coefficient, material behavior, and foundations conditions. Seismic vibrations cause acceleration in the mass of a structure, thus creating the generation of inertia force. The inertial force acts in a lateral shear, bending moments, axial forces, and torsion.

The response of a building subjected to seismic forces could be either elastic or inelastic depending on the degree of loading caused by earthquakes. When the intensity of the loading is relatively lower, then the structure remains elastic in nature and regains the original position after the seismic forces have ceased. However, for higher intensities of earthquakes, the generated stresses are high enough and might surpass the strength of materials, which leads to cracking and yielding of concrete, reinforcement, and inelastic deformations.

According to the Indian Standard 1893 (Part 1):2016 code, there are several aspects that determine the earthquake forces imposed on structures. The various parameters include seismic zone factor, importance factor, response reduction factor, soil characteristics, and natural period of vibrations..

1.3 Non linear behaviour of structures

When the applied load is greater than the elastic limit of reinforced concrete, the behavior of structures under seismic loading becomes complicated. During strong earthquakes, buildings are not completely elastic, but instead crack, yield, lose stiffness and suffer permanent deformations. This response is known as "nonlinear behaviour of structures. But, for real structures under strong earthquakes with reinforced concrete materials and structure, the behaviour is nonlinear because of material and geometry effects. The material nonlinearity results from the cracking of concrete in tension and crushing in compression, and the yielding of reinforcing steel after it reaches the yield strength. Geometric nonlinearity is due to the occurrence of additional secondary effects of large deformations (e.g., P-Delta effects). The non-linear effects has a major impact to the seismic performance of buildings. Nonlinear behaviors are very important in earthquake engineering because it enables a redistribution of forces and dissipation of seismic energy by controlling inelastic deformations. A ductile detailing provision allows structures to sustain large deformations without failure, as opposed to failure modes occurring with sudden collapse. As per the provisions of the codes in IS 1893 and IS 13920 issued by the Bureau of Indian Standards, buildings in seismic zone should have adequate ductility to withstand earthquake forces in a controlled manner by the action of building itself. The major features of nonlinear structural behaviour are: - Concrete members cracking. - Yielding of reinforcement steel - Reduced stiffness with increased deformation Energy dissipation by hysteretic behaviour RPDs following loading - Development of plastic hinges at critical areas The initiation of nonlinear behavior typically occurs at beam-column joints, beam ends, column bases and stress concentration areas in reinforced concrete framed structures. As the intensity of the earthquake increases, these areas are yielding, and gradually forming plastic hinges. The nonlinear response of a structure depends on a number of factors, including: - Structural configuration - Building height - Mass irregularity - Stiffness irregularity - Strength distribution - Material properties - Ductile detailing - The effects on buildings from ground motions during an earthquake The nonlinear characteristics tend to be more uniform in regular structures, which have a symmetrical distribution of stiffness

and mass. Irregular structures, on the other hand, have stress concentration and torsional effects and form an irregular hinge and are vulnerable to earthquakes. The lateral displacement and mode effects of taller structures, likewise, are larger than those of low-rise structures. Therefore, when dealing with the seismic performance of regular and irregular structures, tall and small buildings, nonlinear analysis is necessary. The following are examples of analysis techniques you can use to explore nonlinear behavior: - NLSA – NLDA. These techniques give realistic indications of the sequence of hinge formations, displacement capacity, collapse mechanisms and the level of overall performance of the structure.

1.4 Platic hinge concepts

Yielding in a structural member in the form of plastic hinge is the result of excessive bending moments in a limited section of the member. This enables the member to rotate much without a significant increase in moment resistance. Plastic hinges are important concepts in the field of NL seismic and performance based design. Plastic hinges are usually located at the ends of beams, the ends of columns, and at beam-column connections where the bending moments are maximum during earthquake loading of reinforced concrete structures. Once the moment of a section has reached its plastic moment capacity, the section will yield, and act like a hinge allowing plastic rotation. Plastic hinge is a fundamental concept because it provides the understanding and justification for how the structures dissipate seismic energy during seismic ground motion events. A ductile structure allows for the formation of gradual plastic hinges, which help to avoid sudden failure. As lateral load increases, the structural response can be broken down into various stages: - Elastic Stage - Cracking Stage - Yielding Stage - Plastic Hinge Formation - Collapse Mechanism The structure is initially elastically loaded. When the load is increased, the cracks will develop in the concrete members. When further loading is applied to the reinforcement steel, it will yield. Spreads start to yield through a section and a plastic hinge is formed, permitting plastic rotation. Conditions for the development of plastic hinges are: - Member stiffness - Reinforcement detailing - Axial load level - Material strength - Load distribution - Structural irregularities In a pushover analysis the characteristics of the hinges are set

for the structural elements in software such as ETABS. The software tracks the hinge states, and plots the damage levels as the lateral displacement increases. In general, plastic hinges are broken down into performance levels like: - (IO) - (LS) - (CP) The performance levels represent the amount of damage to the structure during seismic loading. Hinges are not expected in columns in well-designed earthquake-resistant structures, but instead are expected to form in the beams. This principle is known as "strong column-weak beam" and helps prevent catastrophic failures. The study of plastic hinges offers an important insight into: - Structural ductility - Energy dissipation capacity - Failure mechanism - Seismic performance level - Collapse pattern - Structural safety margin Hence, the plastic hinge analysis is an important component in the nonlinear seismic evaluation of a reinforced concrete building.

1.5 Types of plastic hinges

There are three types of plastic hinges. Plastic hinges might occur in various ways due to the nature of the internal force on the structural member. In RC structures whose earthquake loading, hinges generally form as a result of bending moments, shear forces or axial force. The following is a discussion of the main types of plastic hinges.

1.5.1 Flexural hinges

The flexural hinges are the most general case of plastic hinges in reinforced concrete framed structures. These hinges usually develop due to excessive bending moments in beams and columns.

If the bending moment at a section is greater than the yield capacity of the reinforcement steel, the section yields creating a flexural hinge. Most of these hinges are found at:

- Beam ends
- Column bases
- Beam-column joints
- Areas of maximum moment

A flexural hinge is desired for ductile seismic design for its ability to dissipate energy

in plastic rotations. Good details flexural hinges are able to be extremely large and still not fail abruptly.

Flexural hinges have the following effects:

- Amount of reinforcement
- Concrete strength
- Confinement reinforcement
- Member dimensions
- Axial load level

Special confining reinforcement is to be provided in the vicinity of potential plastic hinge regions to improve the ductility and energy absorption capacity as per IS 13920.

Moment-rotation relationships are typically used to represent flexural hinges in pushover analysis. These relationships are nonlinear and relate the behavior of members from elastic to ultimate response.

In beams, the formation of flexural hinges is considered a desirable feature if it develops in beams prior to columns as this allows the development of a ductile global failure mechanism.

1.5.2 Shear hinges

Shear hinges occur in a structural member when a failure occurs as a result of shear forces rather than bending forces. Typically, these hinges are brittle and are not considered desirable for earthquake resistance. Shear failure has little deformation capacity, failure suddenly and is dangerous during the seismic event. Shear hinges can occur in reinforced concrete structures because of the following:

- Inadequate shear reinforcement
- Short column effect
- Sudden changes in stiffness
- High lateral loads
- Poor detailing practices

Shear hinges are not as predictable of failure as flexural hinges. Therefore, it is recommended that design codes provide criteria to avoid shear failure, such as

providing sufficient shear strength and ductile detailing.

Shear hinges have the following typical features:

- Diagonal cracking
- Quick decrease in strength
- Low ductility
- Rapidly collapsing or falling posture or stance

In nonlinear analysis, shear hinges will appear as a sign of poor seismic performance and potential instability of the structure.

In order to avoid the formation of shear hinge, designers adopt:

- Closely spaced stirrups
- Adequate confinement reinforcement
- Proper anchorage detailing
- Excellent fire resistance (ETS)

Members must fail in flexure, rather than in shear, because of the latter being more ductile and dissipating energy.

1.5.3 Axial hinges

The axial hinges are formed when the forces of compression or tension are too great on the structural members. These hinges are frequently used in columns, braces, and items that carry loads up and down. In reinforced concrete columns, the very high axial loads and bending moments may result in crushing of the concrete and buckling of the reinforcement which causes formation of axial hinge. The axial hinge is of major importance as the columns are primary load bearing members of the structure. When columns fail, it can significantly affect the structure's stability and can cause progressive collapse. The axial hinges form because of: The load applied to the axle. The amount of force on the axles.

- Slenderness ratio
- Reinforcement ratio
- Confinement detailing
- Interaction between axial forces and bending moments

When strong earthquake happens, the irregular structures might have the phenomenon of the concentration of axial forces caused by the torsional effect and uneven load distribution. This can accelerate the formation of the axes in some columns.
Appropriate column confinement reinforcement and ductile detailing can improve the column's deformation capacity and delay column hinging.
The key to ensuring overall structural integrity in performance-based seismic design is to prevent early failure of the axial hinge.

1.6 Non linear static analysis

A method to determine the structural response beyond elastic limits using Target displacement is called Nonlinear Static Analysis or Pushover Analysis. This method involves estimating the strength of a building, its ability to deform and its failure mechanisms when subjected to earthquake forces.

In pushover analysis, the lateral loads are applied in a prescribed manner to the structure until the desired displacement or collapse is reached. With an increase in load, the structural parts yield and form plastic hinges.

This analysis helps to gain insight into:

- Sequence of hinge formation
- Base shear capacity
- Lateral displacement capacity - Structural ductility
- Collapse mechanism
- Performance level

The importance of NLSA is to determine how a structure acts during severe earthquakes and if it meets performance criteria.

The basic steps of pushover analysis are as follows:

- Modelling the structure
- Assigning material properties

- Defining hinge properties
- Applying gravity loads
- Applying incremental lateral loads
- Hinge formation and hinge displacement monitoring response
- Generating pushover curve

A pushover curve is a plot of the relationship between the base shear and roof displacement. This curve is used to get the properties such as stiffness, strength and deformation structure in NLSA.

Common examples of what can be seen in a pushover curve are:

- Initial elastic behaviour
- Yielding stage
- Stiffness degradation
- Plastic hinge development
- Ultimate capacity stage

Pushover analysis is widely used as it is simpler, and less resource-intensive to compute than nonlinear dynamic analysis. It gives crucial information regarding predicted damage and possible seismic risk of structures.

This method, however, has some drawbacks: Higher mode effects might not be properly recorded .The results can be affected by load pattern assumptions.

The less regular the structure, the lower the accuracy despite these disadvantages, pushover analysis is still a useful tool to assess the seismic performance of reinforced concrete buildings. In this study, the pushover analysis is employed to investigate the plastic hinge behaviour in: Buildings with vertical reinforcements in both the columns and the beams. Buildings with both tall and small vertical rebar. - Regular and irregular structure buildings of reinforced concrete. The study addresses such aspects as the sequence of hinge formation, hinge states, displacement behaviour and overall seismic performance under nonlinear loading,

1.7 Pushover analysis

Pushover analysis is a nonlinear static analysis technique that is applied to determine the performance of reinforced concrete structures in an earthquake. In this method, lateral loads are slowly increased on the structure until failure or a certain displacement is achieved. The analysis can aid in understanding the behavior of structures in the inelastic range and identify vulnerable regions in which plastic hinges may develop during an earthquake.

In contrast to linear static analyses, pushover analysis considers material nonlinearity and loss of stiffness after yielding. This procedure is often adopted in performance-based seismic design where it allows prediction of the response of a building beyond its elastic limits.

In the pushover method, the structure is loaded with gravity loads before gradually increasing lateral loads are applied. As the load increases, the plastic hinges appear at certain points, such as the beams end, the columns end , and beam-column joints. Development and distribution of these hinges are sign of the damage and failure mechanisms of building.

The primary output of the pushover analysis is the pushover curve which displays the base shear versus roof displacement. This curve is used to determine various stages of performance, including elastic behaviour, yielding, ultimate capacity and collapse prevention. The method is applicable in comparing seismic behaviour of regular and irregular buildings, low-rise and high-rise buildings.

The analysis also indicates whether the structure satisfies the required objectives for performance presented in the seismic design codes such as IS 1893 and FEMA guidelines. Analysis of hinge formation patterns can help engineers design structures that are more resistant to earthquakes and improve their structural design.

1.8 Non linear dynamic analysis

The technique of nonlinear dynamic analysis is used to study the response of structures to actual ground motions from earthquakes. This technique takes into account material nonlinearity and geometric nonlinearity under dynamic loading as a function of time. It offers a realistic representation of the response of the structure during seismic events.

In nonlinear dynamic analysis, the structure is shaken by the recorded or simulated acceleration data of earthquakes. As the structure changes with time, so do its response, stiffness and damping properties when yielding occurs. This method captures essential structural responses including displacements, accelerations, inter-storey drift and hinge formation sequence.

Since the inelastic behaviour of reinforced concrete structures cannot be represented by linear analysis, nonlinear dynamic analysis results in more accurate analysis than linear analysis methods. This approach is particularly applicable to irregular buildings, tall structures and buildings in high seismic zones where complex dynamic effects are significant.

This method enables the engineers to investigate plastic hinge formation under cyclic loading. It also acts as a useful aid to pinpoint critical personnel whose function could be jeopardized during severe earthquakes. The forces that can be generated in an earthquake also change quickly over time, causing multiple loading and unloading cycles on the structure, which affect its stiffness degradation and energy dissipation.

The nonlinear dynamic analysis has to be performed with a lot of computing power and with a reliable model of structural properties. This analysis is often performed using software such as ETABS. The results obtained from the nonlinear dynamic analysis are very useful for performance-based seismic assessment and design.

1.9 Non linear time history analysis

Among the most reliable methods to evaluate seismic performance of structures is a NLTH analysis. The actual ground motion data from an EQ is imposed on structure, and the structure's response is computed at every instant during the earthquake.

Non-linear response of structural components such as yielding, cracking, loss of stiffness and plastic hinge formation is taken into account.

The greatest benefit of nonlinear time history analysis is that it can simulate real earthquake, which is not possible for linear time history analysis. The dynamic response of the structure is captured by the analysis since loading changes are continuously changing with time. It is very common in seismic investigations and for significant structures.

Plastic hinges form when the members of reinforced concrete buildings reach their inelastic capacity. Engineers are able to determine the building's performance level in time history analysis by studying the formation pattern of the hinges, and also get to know the vulnerable areas. The analysis also provides a better understanding of energy dissipation processes associated with seismic events.

Selection of appropriate GMRs and correct modeling of structural properties are critical for the accuracy of nonlinear THP. IS 1893 has set forth guidelines for selecting earthquake records that should be commensurate to the seismic characteristics of the site.

This research compares the behavior of symmetrical and unsymmetrical buildings, and small and tall buildings using the NLTH analysis. The analysis is directed towards characteristics of the hinge, displacement response, and general seismic response.

1.10 Scale and configuration impact

The seismic behavior of reinforced concrete structures is greatly influenced by building scale and structural configuration. The formation of plastic hinges and overall seismic response depends on a number of factors such as building height, plan irregularity, vertical irregularity, and mass distribution.

The lateral movements and overturning moments are typically larger in tall buildings compared to smaller buildings. The natural period of high rises is longer and they are more susceptible to dynamic effects during earthquakes, due to their greater flexibility. This means that plastic hinges can develop in several stories, particularly at the lower levels where the seismic forces are most intense.

Small buildings are stiffer, and their natural periods are shorter, however. They are likely to have low lateral offsets, but can have high base shear forces. Hinge formation pattern is often more localized for small buildings than tall buildings.

The structural configuration is also very important for seismic performance. Regular buildings make up buildings that have uniform masses, stiffnesses and geometries, which means that their behaviour during earthquakes is more stable and predictable. These structures distribute the seismic forces evenly, thus lowering the level of stress concentration and abnormal deformation.

Irregular buildings are those with unexpected changes in geometry, stiffness or mass distribution. These irregularities cause torsional effects and stress concentrations, which makes it more likely to cause damage to the structures. Irregular buildings often develop plastic hinges before and in an irregular manner, which results in poorer seismic performance.

The comparison of regular and irregular structures, and the small and tall buildings respectively, allows us to better understand how the configuration and scale of the structure influences hinge behavior. This study could provide valuable insights to the seismic vulnerability of different structural systems and contribute towards safer earthquake-resistant design.

1.11 Performance level of structure

Performance levels are a measure of the condition and function of a structure following seismic loading. During an earthquake, these levels can be used to assess the damage level of the structure and non-structural components in performance-based seismic design. The performance is primarily determined by the deformation capacity,

ductility, reduction in stiffness and development of plastic hinges in structural members.

In nonlinear analyses like pushover analysis and time history analysis, we evaluate the seismic performance of RC buildings, based on the appearance of hinges and their sequence. These hinges indicate if it is in operation, if it has incurred damage that it can be repaired, or if it's close to collapse.

FEMA 356, ATC-40, and based on performance-based design philosophy, the structural performance is broadly broken into three key categories:

(IO)

(LS)

(CP)

It helps engineers to assess seismic safety and reliability of structures, depending on the level of ground shaking.

1.11.1 Immediate occupancy(IO)

Immediate Occupancy is the performance level that a structure can attain while undergoing earthquake loading. Some minor damage can happen at this stage, and the structure is safe to occupy right after the earthquake. The structural system retains most of its original stiffness and strength but there is limited yielding in a few members.

Plastic hinges can also develop at the IO level, but at that point, the hinges are usually at the elastic or slightly

nonlinear level. Structural features such as columns, beams and slabs remain effective with minimal loss of load carrying capacity.

The level of immediate occupancy belongs to the characteristics of an immediate occupancy. Very little damage to the structure. There is evidence of earthquake functionality of the building. No significant cracking or crushing of concrete is seen. Only limited reinforcement yield. Mostly preserved structural stiffness and strength. Requires small, inexpensive fixes. Behavior of Plastic Hinges at IO Level.

Generally, at the Immediate Occupancy stage, hinges lie between points A and B or slightly beyond B in the hinge performance curve. This is an indication of elastic response or first yield with no significant damage.

The hinges developed during this stage are stable and do not reduce the structural integrity of the structure. Usually flexural yielding occurs on beam members before columns, in accordance with the strong column-weak beam principle.

Importance in Seismic Design IO level is very important for vital structures such as:

- Hospitals
- Emergency response buildings
- Fire stations
- Communication centers
- Important government buildings

These buildings and structures need to remain functional even after a moderate earthquake.

1.11.2 Life safety (LS)

Regular buildings most commonly achieve the performance goal of Life Safety. The building might be severely damaged in an earthquake at this point, but not destroyed. The main aim is to save lives for partial or total failure of the structure.

At the LS stage, beams and some columns will yield and plastic hinges will form. Structural and non-structural injuries will be evident, but the structure is still sufficiently laterally strong and stable.

Children will learn the characteristics of Life Safety Level. Moderate to severe structural damage is experienced. The cracking becomes appreciable in the beams and columns.

The reinforcement yielding is wide.

- There is some reduction in stiffness and strength.
- Occupants can safely evacuate the building.

It can be repaired, but it might cost money. At LS level, the behaviour of Plastic Hinges will be examined. Plastic hinges typically fall within the range between point IO and LS on the hinge performance curve at the LS stage.

The predominant solution is beam hinges and controlled inelastic deformation dissipates earthquake energy. Most hinges will occur in beams rather than columns, if designed correctly, avoiding sudden collapse. The structure will have greater lateral

movement and drift than the IO level. Life Safety is of significant importance.

The Life Safety level is included in seismic design codes due to the following trade-offs:

- Structural safety
- Economic feasibility
- Earthquake resistance
- Damage control

The LS performance level is used in the design of most residential and commercial buildings.

1.11.3 Collapse prevention (CP)

Collapse Prevention is the greatest amount of deformation that a structure can withstand before all components fail. The structure is severely damaged, numerous plastic hinges have formed and some loss of stiffness and strength has occurred.

The building could be unsafe for people to live in, but the collapse of the building is prevented, which reduces the danger to life.

The following features will be used to define Collapse Prevention Level.

Severe cracking and crushing of the concrete is experienced.

Reinforcement is large deforming non-elastically. There is considerable loss of stiffness and lateral load resistance. Large permanent deformation and drift occurs. Structural components can be out of balance. The building has been structurally damaged due to the earthquake and is not suitable for use. The behaviour of plastic hinges, at CP level. Hinges are at the CP stage when they are near to members' ultimate deformation capacity. A very large number of hinges fall into the CP range, meaning that they exhibit serious nonlinear behavior. There may be some loss of stiffness and yield in columns. If columns form excessive hines, it may result in progressive collapse. So, the data on hinge location and sequences should be carefully monitored during the nonlinear analysis. The importance of nonlinear analysis.The significance of nonlinear analysis.

The Collapse Prevention level is important to assess:

- Ultimate seismic capacity of buildings

- Structural ductility - Failure mechanisms
- Safety margin before collapse
- Vulnerability of irregular and tall structures to earthquakes

Pushover analysis is used to see if the structure remains within IO, LS or CP limits during seismic demand.

1.12 Objective of the study

This study mainly aims to explore the behaviour and properties of plastic hinges in reinforced concrete structures during nonlinear seismic analysis. It is a focus on evaluating the seismic performances of buildings by Pushover analysis and Time history analysis under different structural configuration.

This study aims at doing the following:

To investigate the nonlinear response of RC structures to earthquake loading.

- To study the development and growth of plastic hinges in beams and columns under seismic events.
- To assess the level of seismic performance of structures (Immediate Occupancy, Life Safety, and Collapse Prevention).
- To study the difference between the behavior of small and tall buildings with reinforced concrete under nonlinear analysis.
- To compare seismic response of regular and irregular structures.
- To study the effect of structural irregularity on the formation of hinges and displacement behaviour. To calculate the base shear capacity, roof displacement, storey drift and stiffness loss of different building models. - To analyze the performance point from a pushover analysis.
- To evaluate the ductility and energy absorption ability of reinforced concrete structures. To gain insight into the effects of the building elevation on seismic performance and hinge mechanisms.
- To pinpoint important structural members that can be vulnerable under earthquakes.
- To validate how effective pushover and time history analysis are for performance-based seismic evaluation.

- To gain a better knowledge of failure mechanisms of reinforced concrete buildings. The study covers the scope of the study.
- This study will only focus on the reinforced concrete framed buildings with nonlinear seismic analysis in ETABS software. The main concern is with the earthquake loads on plastic hinges.

1.13 Scope of the study

The study includes the following aspects: Analysis of reinforced concrete structures designed under the code of practice IS 456:2000 & IS 1893 (Part-1): 2016.

Seismic behaviour evaluation by pushover analysis and nonlinear time history analysis.

- Examination of the sequence and state of formation of the hinges in the structural elements.
- Difference between small building structures and tall building structures.
- Structural similarity/difference between regular and irregular configurations.
- Calculation of performance points, base shear, displacement and storey drift.
- Evaluation of seismic performance of structures.
- Research on the stiffness loss and ductility characteristics. Observation of the distribution of hinges in beams and columns in nonlinear response.
- Vulnerable areas in buildings during earthquake action. A study may have several limitations.There are limitations to a study.

The study is limited to reinforced concrete (RC) moment-resisting frame structures. It does not take into account soil-structure interaction effects. Material nonlinearity may be accounted for but some construction imperfections may not be accounted for. Some of the earthquake data selected are analyzed for time history. The study uses analytical modelling and software simulation techniques.

The results from this research will improve the understanding of seismic performance of reinforced concrete buildings and the importance of plastic hinges in preventing structural collapse during large earthquakes

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

It is important to examine both earlier and recent studies related to nonlinear pushover analysis to gain a better understanding of the developments in this field. Such a review helps in identifying the effectiveness, advantages, and limitations of the method based on previous research findings. Since presenting an extensive review of all available studies is beyond the scope of this section, selected research contributions in this area are discussed below in chronological order. Building height and geometrical shape has a significant influence on seismic response and hinge development.

- stiffness
- displacement demand
- fundamental time period
- transfer of seismic forces
- P-Delta effects
- mode participation
- nonlinear deformation
- Base shear capacity
- Roof displacement
- Structural ductility

Tall structures and unsymmetrical structures may develop larger numbers of hinges

The study therefore compares:

- Small height RC buildings
- Tall height RC buildings
- Unsymmetrical building

- Symmetrical building

The main objectives of the present work are:

To perform nonlinear static pushover analysis on reinforced concrete structures.

To analyse nonlinear dynamic Time History analysis on RC structures

To study the formation and distribution of plastic hinges under seismic loading.

To evaluate hinge performance levels such as (IO), (LS) and (CP)

To study hinge behaviour in small and tall reinforced concrete buildings.

- To compare hinge properties in regular and irregular structures.
- To compare the effect of structural irregularity on dynamic performance.
- To evaluate base shear and displacement capacity using pushover curves.
- To identify weak zones and critical members in reinforced concrete structures.
- To assess the ductility of building.
- To understand the NL study of RC frames under lateral loading.

2.2 Overview of previous research

Paulay T. and Priestley M.J.N. [1]
A RC building structure was studied to investigate the dynamic design of such structures. Produced one of the earliest and most prestigious of its kind, the concrete and masonry buildings. Complete treatments of the plastic hinge theory. The authors have defined plastic hinge The equivalent length over which it is assumed that plasticity is concentrated is length. At the stage of inelastic deformations of an RC member. They found the foundation for their work in the basic relationship of the ductility to the displacement . The study. These include ratio and load level along the axes, and the nature of the longitudinal reinforcement. Their Analysis software for plastic hinge length. In NL Analysis software, the proposed empirical formula of plastic hinge length is widely used. The static (pushover) analysis procedures.

Otani S. [2]
studied the inelastic analysis of reinforced concrete frame. designed, built and presented an extensive review of hysteretic models appropriate for nonlinear dynamic analysis. The paper considers the concentrated plasticity approach, where nonlinear behaviour is concentrated at the ends of members as plastic hinges, and the distributed plasticity approach, in which inelasticity is spread over the entire Using fibre elements, it was possible to measure the length of the members. The writer has concluded that concentrated plasticity. Although computational efficient and practical for engineering applications, models are inadequate for the purpose of understanding Needs careful calibration of the moment–rotation relationship and plastic hinge To get accurate results the length must be taken. The study also pointed out the sensitivity of structural response to choice of hysteretic model, especially during cyclic loading conditions.

Priestley M.J.N., Seible F. and Calvi G.M. [3]
Gave a detailed account of dynamic design and retrofiting of bridge and building, focusing on plastic Detailed column hinges in RC columns. The authors suggested the following revised expressions for The phenomenon of members is something that is not always taken into account in an oversimplified analysis. Their work showed that it is important to consider strain penetration or risk seriously underestimating. There is

no member deformation capacity. The study suggested the nonlinear models in the future should be used. The commercial software packages, like SAP2000 and ETABS, should be made to include strain. A corresponding additional hinge length term resulting in penetration effects, specifically for columns with large diameter longitudinal reinforcement.

Mander J.B., Priestley M.J.N. and Park R. [4].
The material base is the model for the confined and unconfined concrete. for definition of plastic hinge moment – curvature. The authors conducted The RC column's shape is closely related to its axial strength, an aspect that has received extensive experimental testing under the following conditions: The confinement was monotonic and cyclic, and was demonstrated to be axial compression. By adding strength and cross ties, it increases the performance and The ductility of the concrete core. The developed confined concrete model in this study is the most popular material model applied in fibre-based nonlinear analysis of RC structures. structures, on which forces and curvatures are generated, or moment–curvature.structures, which form the basis for moment–curvature, or force–). Displacement relationships are the ones applied to software like to assign plastic hinge properties to them. ETABS

Fardis M.N. and Biskinis D.E. [5]
Investigated the deformability of RC.flexure or shear controlled members, especially the ultimate chord rotation at the plastic hinge. The authors developed a large experimental data base The results of RC beam and column tests and suggested empirical expressions for the chord. The rotation at yielding and ultimate, included in the European seismic codes, are employed. design code EN 1998-3 (Eurocode 8 – Part 3). The study found that the ultimate rotation capacity is greatly affected by the longitudinal reinforcement ratio, confinement ratio, axial load ratio, and shear span-to-depth ratio. The expressions suggested by the researchers were checked with over 1000 experimental observations. This comparison revealed that there is a high correlation between the results of the test and the expression, which means that these expressions are reliable and can be used in a nonlinear assessment procedure.

Inel M. and Ozmen H.B. [6].

RC buildings were analyzed with pushover analysis where the nonlinear response was investigated. The authors modelled a set of low rise and mid rise RC frame buildings and compared the structural responses to be obtained with default plastic hinge properties available in FEMA 356 and ATC-40 guidelines against responses obtained using hinge properties Derived from the moment–curvature analysis at the section level. The findings of this study were that The deformation is often over estimated in guidelines that provide the default hinge properties. Poorly detailed RC members have a limited capacity to make unconservative assessments of building performance. The authors recommended very strongly the use of member-specific PE.

A pushover analysis was performed using hinge properties that were determined from real section and material data. analysis results.

Haselton C.B., Liel A.B., Taylor Lange S. and Deierlein G.G. [7]

Performed a large-scale calibration study of concentrated plasticity models to simulate the seismic collapse of RC buildings. The authors have adapted the Ibarra– A moment–rotation behavior at plastic is simulated by Krawinkler deterioration model

Determined hinge sites and fit experimental database to model parameters of more than 250 RC column tests. The study revealed that the most significant strength was after capping, The most important parameters that affect stiffness and cyclic deterioration are stiffness, and cyclic deterioration parameters. collapse prediction. The calibrated model was found to predict the correctly the modifying the behaviour after the peak moment, modifying the strength and stiffness.

Hasan R., Xu L. and Grierson D.E. [8].

for the performance-based seismic assessment of steel and RC frame buildings in

The advancement of forming plastic hinges is monitored, step by step under

Lateral loads that increase monotonically. The study discussed the significance of hinge order or arrangement of the forms of hinges in determining their structural sequencing — the order in which the hinges form. The mechanisms and the overall ductility requirement. The authors have shown that a weak-A strong-beam structure is usually not preferred in seismic design, and this is the case here. This results in the

early formation of plastic hinges in columns and rapid loss of lateral. A strong columns and weak-beam mechanism is responsible for the load resistance, while a weak column strong-beam mechanism distributes hinge. It has the ability to form across beams, increasing ductility. The study emphasized the need to check the plastic hinge pattern obtained from pushover analysis against The design mechanism that is meant to be implemented.

Bae S., Bayrak O. [9].
High axial load acting on RC columns, into plastic hinge length. The Under simulated seismic loading, authors tested a series of large-scale RC columns.and used strain gauge to measure the extent of plasticity spread along the length of the member.data. It turned out that the available empirical relationships for plastic hinge length,The above, such as those suggested by Paulay and Priestley, tend to overestimate the hinge The results reveal the length at high axial loads, when the plastic zone is more confined, due to.reduced curvature ductility. The authors came up with a new equation for plasticExplicitly accounted for the axial load ratio and it was shown that it provided hinge length that gaveThere is a better agreement with measured data for columns in high-rise structures with axial flow.There is a better agreement with measured data for columns with axial flow in high-rise structures.forces are significant

Panagiotakos T.B. and Fardis M.N. [10]
carried out a large-scale study of the Deformations of RC members at yielding and at ultimate under seismic loading based on, Using an experimental database of over 1000 RC member tests. The authorscompared measured deformation quantities—yield curvature, plastic hinge angles, and other parameters—with the corresponding theoretical values.In-plane, chord rotation yield and at ultimate — predicted by various in-plane chord rotations.Numerical and symbolic expressions. A significant amount of scatter was found in the study.Predictions of plastic hinge lengths using the probabilistic approach were recommended.Guidance on the way of evaluating performance. The study also showed that the fixed endThe contribution from bar slip at the beam–column joint is greatly increased.Analysis, the total deformation at the plastic hinge can not be ignored, and should not be overlooked in nonlinear analysis. modelling of RC frames.

Dolsek M. and Fajfar P. [11]
studied the effect of masonry infill panels on the The NL static and incremental seismic response of RC frames was addressed. Analysis, including the changes of plastic hinge distribution, from a specific perspective. by infill–frame interaction. The authors discovered that masonry infills have a significant impact on the building. It is preferable to have shear demand allocated to the columns, rather than the hinge.Shear demand will be better allocated to the column rather than the hinge, which is the desired pattern. Adjacent to openings, where they result in hinge formation in the columns, at other locations.The end of the member is shorter than the end of the member. The study found that the nonlinear models of infilled RC structures are suitable for existing RC structures.To accurately model infill–frame interaction effects frames need to be explicitly modelled. the plastic hinge sequence and failure mode of the structure.

Celik O.C. and Ellingwood B.R. [12]
studied the seismic fragility of non- cemented concrete panels by assessing their strength and stiffness.nonductile RC frames designed prior to the development of modern seismic codes in which the role ofThe plastic hinge modelling assumptions in controlling structural fragility estimates. Theincremental dynamic analysis of a set of RC frame buildings was performed by the authors.showed that the assumed plastic hinge backbone curve parameters — particularly theThe post-peak degrading slope and the ultimate rotation capacity — exert a dominant effect.The role the influence has on the calculated collapse probability. The study proved thatThe overall error in modelling parameters, especially at the hinge, is due to the uncertainty.The uncertainty in the modelling parameters at the hinge contributes significantly to the overall error in modelling.Uncertainty in fragility estimates, recommended the use of probabilistic hingemodels that carry forward material and geometric uncertainty to structural.response.

Zhao X.M., Wu Y.F. and Leung A.Y.T. [13]
carried out a thorough review. and experimental verification of plastic hinge length models of RC columns,Comparing over a dozen empirical and theoretical equations from the literatureCompared to an experimental column test database. The set of

expressions was studied;proposed by various researchers including those of Baker, Corley, Mattock, Paulayand Priestley and Coleman and Spacone and evaluated their accuracy over a broad rangeThe variety of column parameters such as aspect ratio, transverse load ratio, axial load ratio, etc.These include the reinforcement ratio, concrete compressive strength. The authors concluded that

There is no single existing expression that is successful for all ranges of parameters and proposed new calibrated expression which more accurately displays the entire range of the most common column configurations in RC building structures.

2.3 Research gap

1. **Design a scientific investigation that addresses a specific question.3. Conduct a scientific investigation to solve a scientific problem.**

The thesis (Section 2.3, Page 31) specifically reserves a section for Research Gap which is completely unfilled. From the literature study presented in Chapter 2 and the study's objectives the following research gaps were identified and taken up by this study:

Compared to other structures, limited study has been conducted to understand the comparative behaviour of buildings for plastic hinge or building height.

Previous research (Inel & Ozmen, 2006; Haselton et al., 2008) focused on properties of plastic hinges in a single level category of buildings. Systematic comparative studies are lacking which directly compare the performance levels of short (G+4) and tall (G+14) RC frames under the same material, loading, and analysis conditions .

2. **Not enough attention was paid to Plan Irregular Buildings in Nonlinear Dynamic Analysis.**

Most of the previous studies focused on regular RC frames in nonlinear analysis. The studies of plan-irregular buildings (re-entrant corner buildings, setback irregularities as per IS 1893:2016 Tables 5 & 6) under (NLTHA) in Indian seismic condition are limited and the focus is only on plastic hinge distribution and torsion amplified rotation demand.

3. **Lack of Combined Pushover and Time History Analysis on Paired Structural Models design context.**

Previous publications have mostly used either pushover or time history analysis. The current studies are limited in scope as they focus on either (NLSPA) or (NLTHA) on structurally paired regular and irregular models to make direct comparison of performance under a unified analysis framework for both models.

4. Limited use of Indian Standard Codes (IS 1893, IS 13920) in conjunction with FEMA 356/ATC-40.

No research is found that combines Indian seismic design codes (IS 1893:2016, IS 13920:2016, IS 456:2000) with the international performance based codes (FEMA 356, ATC-40) for the assignment of plastic hinge behavioure and the verification of the plastic hinge acceptance criteria. Only a few published studies have been based on both Indian codes and FEMA/ATC provisions.

CHAPTER 3

THEORETICAL BACKGROUND

3.1 Introduction

The seismic resistance of reinforced concrete structures primarily relies on their resistance to lateral forces and their ability to dissipate seismic energy in a controlled manner through inelastic deformations. Earthquake induced dynamic forces experienced by a structure vary with time, as opposed to the gravity forces which are in general constants. Therefore, it is important to have a proper understanding of basic concepts of structural dynamics and earthquake engineering prior to applying nonlinear analysis methods, such as pushover analysis and time history analysis..

Structural analysis under seismic loading deals with the responses of a building under ground motion, which results in the inertia force applied to the building. In high-intensity earthquakes, structures can develop cracking, loss of stiffness, yielding of reinforcement and the development of plastic hinges. These responses are very non-linear and cannot be reliably predicted from traditional linear elastic analysis. Thus, advanced analytical procedures are needed to analyze the actual response of structures under strong seismic inputs.

Theoretical concepts like natural period, frequency, damping and mode shapes are very important for calculating the dynamic response of structures. These parameters affect the vibration characteristics of a structure and the amount of displacement, acceleration and internal force generated in the structure during an earthquake. Different dynamic responses will occur in buildings of varying heights, mass distributions, and stiffness properties when subjected to the same ground motions. Regular structures tend to respond to applied forces in more predictable ways, while irregular structures can incur stresses that may be concentrated in some places and distributed in others, or may suffer from torsion.

In this work, the nonlinear analysis methods are employed to examine the plastic hinge phenomenon of reinforced concrete building structures. The present study is concerned with seismic performance differences of regular and irregular configuration and low and high-rise buildings under earthquake loading condition. The results from pushover analysis and time history analysis are analysed based on the basic concepts of this chapter. The principles and methodologies follow the design and analysis guidelines of reinforced concrete structures given in Indian Standards IS 1893 (Part 1): 2016 and IS 456: 2000 which are widely followed in India..

3.2 Structural Dynamics fundamentals

Structural dynamics is a field concerned with the dynamics and study of a structure under dynamic or time varying loads. Dynamic loads are loads that vary in magnitude, direction and location over the course of time. One of the most important types of dynamic loads is earthquake forces which set-up vibrations throughout the structure because of the ground acceleration.

During an earthquake, the foundation or base of the structure will move rapidly with the ground. But due to the inertia, the top portion of the building is inclined to stay in the original position. Because of this relative displacement, internal forces and deformations are generated in structural members like beams, columns and slabs. The resistance of the structure to these vibrations is primarily find out by the mass, stiffness and damping of the building.

Inertia forces are produced when a structure moves during an earthquake and the mass of the structure is a factor in the inertia forces. The more massive an object the more earthquake force will be experienced by it, as seismic force is directly proportional to mass. Stiffness is the resistance to deformation of structural elements. The stiffer the structure, the less the structure will be displaced laterally, but it can have a higher base shear. The damping mechanism is the action by which vibration energy is dissipated as a result of material inelasticity, friction, and heat. Good damping results in smaller vibration amplitude and better seismic performance.

The dynamic analysis is an important part of the structural evaluation since the earthquake response of the structure not only depends on the amount of the earthquake forces, but also on the frequency characteristics and duration of the ground motion. The occurrence of resonance can cause the vibrations to be excessive, the displacements of the structure to be large and the damage can be severe, if the frequency of the seismic excitation approaches that of the natural frequency of the structure.

Typical models used for studying the structural dynamic behavior are idealized (SDOF), (MDOF) systems. In practical buildings, there are several degrees of freedom, as each floor is excited to move independently. So the response of building to seismic is mostly in the terms of mode shape and the modal analysis.

3.3 Earthquake ground motion

This energy is released during the movement of tectonic plates on geological faults, causing seismic waves to radiate through the earth and shake nearby structures.
The shaking of the earth when an earthquake occurs is irregular and random. It is composed of a mixture of frequencies and amplitudes and changes continuously as a timr function. The reaction of the building not only to the intensity of the tremor but also to the properties of the GM and the dynamic properties of the building.
Acceleration-time history records means to representation the real ground motion. These records show variation of ground acceleration with respect to time during an earthquake event. In nonlinear dynamic analysis, data of time history is crucial as it enables the structure to undergo realistic earthquake excitations.

Some important properties of the ground motion during an earthquake are:
(PGA):-
It is highest acceleration of the ground during an EQ . In a structure, the inertia forces are normally larger with higher PGA
Frequency Content:
Earthquake motions are made up of more than one frequency. The structures respond

during dominant frequency of the GM .

RS:

A response spectrum is a plot of the largest response of structures with various natural periods to a given ground motion. It is very much applicable in seismic design as per IS 1893.

3.4 Dynamic characteristics of the structure

It is the characteristics of structure will be examined .The dynamic properties of structures will be investigated. Dynamic characteristics refer to the response of a structure to vibrations. These properties have an impact on the seismic response and affect the displacement, acceleration, base shear and plastic hinge formation of buildings. The major dynamic characteristics include natural period, frequency, mode shapes, and damping.

3.4.1 Natural time period

The natural time period is the time it takes a structure by one oscillation when it is taken out of equilibrium by external forces. It is a crucial structural dynamic parameter which has a significant effect on the seismic behavioure and also on the earthquake forces acting on a building. The natural time period is primarily determined by the structure's stiffness and flexibility – stiffer structures tend to have shorter time periods, while more flexible structures have longer time periods.

Typical natural time periods of high-rise structures are generally found to be greater than those of low-rise structures due to their greater flexibility. Likewise, if there is a variation in the dynamic properties of a structure with a non-uniform stiffness distribution, the natural periods may vary along different directions. IS 1893 gives a general approximation of the natural time period of a RC moment resisting frame building as:

$T = 0.075h^{0.75}$

3.4.2 Frequency

Frequency is the number of vibrations that a structure will complete per second. It is expressed i Frequency is the number of vibration cycles per second of a system

measured in Hertz (Hz). It is inversely proportional to the natural time period, so structures with a longer natural period will have a lower frequency, and structures with a shorter natural period will have a higher frequency. The natural frequency and time period are related as:

$f = 1/T$

3.4.3 Mode shapes

Mode shapes are the deformation mode of the structure at a specific natural frequency. Modes of vibration are the characteristics of a building and there will be several of them for each building. These modes shapes are used to represent the relative motion of the different floors when subjected to seismic excitation. For low-rise building, the first mode is usually the most critical in finding the response of the structure. In tall or irregular structures however, higher modes make a significant contribution to displacement and internal forces. Typical mode shapes for symmetrical buildings are typically translational, while asymmetric buildings can have torsional mode shapes from an imbalance of stiffness. Mode shapes are crucial in nonlinear analysis because hinge formation and damage concentration can be found in areas where the modes are at peak. For irregular buildings, stresses can concentrate in certain areas resulting from the interaction of modes. Mode participation factors give the contributions of the various modes to the general response of the structure. IS 1893 recommends that enough modes should be chosen so that the sum of all of the modal mass participations is more than 90% in each principal direction.

3.4.4 Damping

Damping is the process during motion which a vibrating structure loses energy. In real structures, energy losses occur because of internal friction, cracking to the concrete, yielding to the steel reinforcement and interactions between the structural and non-structural parts. Damping also helps to reduce the amplitude of vibration and control structural response during earthquakes. Low damping structures will be subjected to higher displacement and acceleration as compared to high damping structures. The damping ratio of 5% is considered in IS 1893 for reinforced concrete buildings for seismic analysis. In the nonlinear behaviour, however, effective damping can increase

due to cracking and yielding in the structure. Damping is an important factor in controlling resonance effects and seismic demand. Nonlinear analysis can take inelastic deformation-induced hysteretic damping as an important source of energy dissipation. Beams and columns typically develop plastic hinges during severe earthquakes, which helps absorb energy in a hysteretic manner.

3.5 Geometric non linearity

The phenomena of a structure's nonlinear behavior due to large deformations and geometry changes under loading are called geometric nonlinearity. There are certain simplifying assumptions in the conventional linear analysis: The deformation of the structure is very small and does not significantly change the geometry and the distribution of internal forces. In strong earthquakes, however, the reinforced concrete structures may experience large lateral movements, and the moments and the instability effects in the lateral direction due to these movements must not be neglected. In seismic analysis, geometric nonlinearity is particularly important for tall and flexible buildings due to the fact that lateral displacement increases with the height of the buildings. The overturning moments due to the vertical gravity loads on displaced members produce additional moments as the structure deforms. The effect of these secondary moments on the stiffness, stability and response of the structure. So, if one wants to be realistic in predicting the behavior of structures when subjected to severe earthquake excitation, he or she will have to take geometric nonlinear effects into consideration. The formation and development of plastic hinges in reinforced concrete buildings are greatly influenced by the geometric nonlinearity. It can speed up the stiffness degradation and displacement demand, especially for short and long slender structures or irregular buildings. These effects may be ignored with potentially unsafe seismic performance estimation and underestimation of collapse potential. Geometric nonlinearity is typically included in nonlinear analysis procedures like PS analysis and TH analysis to properly reflect the effects of structural instability. These second order effects can be considered in seismic analysis with modern analysis software like ETABS with geometric nonlinear options.

3.6 P-Delta Effects

One of the most significant types of geometric nonlinearity that can be found in structures under lateral displacement is P- effects. The “P” is the vertical load on the structure due to gravity, and “Delta” is the lateral displacement resulting from seismic or wind loading. If a structure moves laterally, the loads of gravity move through the new shape and create extra moments, referred to as secondary moments. The extra moment created as a result of the P-Delta effect is given by: $M=P\Delta$ where: M = secondary moment P = vertical axial load Δ = lateral displacement These secondary moments add to the loading on beams and columns and can decrease the lateral stiffness of the structure. Some cases of excessive P-Delta effects can result in progressive collapse and instability. The P-Delta effects are more critical for tall buildings since they experience a larger lateral displacement during earthquakes. Likewise, irregular buildings and soft storey structures can have the deformation concentrated at certain levels thereby amplifying the instability effects. These are further enhanced by weak columns or excessive drift. P-Delta effects have an effect on the formation of hinges and displacement capacity during nonlinear seismic analysis. Elasticity and the interaction of axial load may cause columns to develop plastic hinges earlier.

3.7 Moment-Curvature Relationship

Moment–curvature relationship is used to represent the flexural response of reinforced concrete sections under bending loads, and most of the time, these sections have a nonlinear response. It establishes the relationship between the bending moment that it is developing in a structural member and the curvature of the member. Curvature is the change of slope of a member over its length and it is directly related to bending deformation.

They are almost linear in the initial loading phase as both the concrete and reinforcing steel are in elastic range. The applied load as it increases causes cracking of the concrete and therefore a decrease in section stiffness. As the loading increases further, the reinforcement will go into the yielding state, where there will be a large increase in curvature for a small increase in bending moment.

The flexural behaviour of a section can be represented by the following relationship:

$\phi = M / EI$

ϕ is the curvature, M is the bending moment, E is a modulus of elasticity, and I is a moment of inertia of the section.

The moment–curvature curve is a key tool for assessing stiffness degradation, ductility and rotational capacity of reinforced concrete elements. Ductile sections can withstand a higher amount of deformation before breaking, while brittle sections fail quickly with a low capacity to be deformed before failure.

The rotational capacity of a plastic hinge is related to the curvature behaviour of a member, therefore moment–curvature analysis is important in the definition of plastic hinge properties for nonlinear structural analysis. For earthquake resistance, enough ductility is needed so that the elements in the structure can undergo large inelastic strains without sudden collapse.

3.8 Force-Deformation Behaviour

The force–deformation behavior of a structure member This relationship has been important for understanding the behaviour of a structure as it is subjected to increased loading and how it moves from elastic to non-linear response and ultimately failure. Within the elastic limit the extension is directly proportional to the applied force (Hooke's law). Structural stiffness is the slope of the force–deformation curve in this region. With higher loading, cracks and yielding occur leading to reduction of stiffness. After the yield point, significant amounts of deformation can occur for relatively little additional load. Typically, an idealized force–deformation curve for a ductile structure will include: Elastic region Yielding region Strain hardening region Strength degradation region Failure stage Ductile reinforced concrete structures can withstand large deformations before they collapse. The behavior is necessary during an earthquake. In nonlinear analysis, the force–deformation behavior is used to determine the properties of the hinges and to assess the performance of the structure. The area of the force–displacement curve is a measure of the structure's absorptive capacity. Larger energy dissipation capacity provides better seismic performance generally. Stiffness discontinuity and torsional effects can cause irregular buildings to

have a non-uniform response to forces and deformations. Likewise, higher buildings are likely to have a bigger deformation demand due to greater flexibility. Hence, it is important to have force–deformation knowledge for comparing performance of various structurals. The effects of modes in non-linear analysis.Modals effects in non-linear analysis.

3.9 Modal effects in non-linear analysis

The effect of the various vibration modes on the seismic response of a structure is called modal effects. Higher modes can have a substantial influence on the displacement response, the distribution of forces and the formation of plastic hinges in nonlinear analysis particularly in high-rise and irregular structures. For low-rise structures, the structural response is typically dominated by the fundamental mode since a significant amount of structural mass is involved in this mode. But in taller structures, there is significant contribution of higher modes for such parameters as inter-storey drift, acceleration response and storey shear etc.

Higher mode effects may be ignored and lead to a wrong assessment of seismic demand and structural performance. When concrete is cracked and reinforcement is yielded as a result of the nonlinear behaviour, the stiffness of the concrete is degraded and the dynamic characteristics will be changed. The development of plastic hinges and the decrease in stiffness results in changes in the natural frequencies, and a more complex modal behaviour in nonlinear conditions than in linear conditions.

Modal interactions are worse with irregular structures because of the uneven distribution of mass and stiffness within the building. These irregularities can inject significant torsional modes which can cause damage to focus at certain locations in the structure. The time history analysis can best capture these modal interactions through nonlinear dynamic procedures, rather than through conventional pushover procedures.

3.10 Torsional irregularity

It is created due to the separation of CM and CS of a building structure during earthquake excitation, resulting in torsional motion. When this is the case, lateral forces create twisting forces as well as translational motion. Torsional irregularity is

more likely to occur in buildings that have asymmetric plan configuration, varying stiffness distributions and/or irregular placement of structural elements. This includes L-shaped, T-shaped, U-shaped and setback buildings. If torsion occurs, the stress concentration and the displacement on the one side of the structure will be higher than the other. This differential action can result in undue damage to columns and beams remote from the center of rigidity. Highly stressed areas can develop plastic hinges early, making the building more susceptible to collapse. Torsional irregularity has a very significant impact on seismic performance due to increase in displacement demand and mode interaction. Structures that are not symmetrical and regular tend to exhibit less seismic behavior. Torsional irregularity is defined in IS 1893 as when the maximum storey displacement at one end is more than 1.2 times the average storey displacement. Minimization of torsional effects with proper structural planning, symmetrical layout and uniform stiffness distribution. In nonlinear analysis, the torsional irregularity affects the sequence of hinge formation, distribution of base shear and inter-storey drift. Time history analysis is especially effective to capture the torsional response since it accounts for the three-dimensional dynamic behavior in response to realistic earthquake excitation.

3.11 Codal provisions

There are several national and international codes and there clauses for seismic analysis and the design of RC structures. The provisions provide for the safety, ductility and performance of buildings during earthquakes. The study mainly relies on the Indian Standards and internationally accepted performance based seismic guidelines. Such as IS 1893 , IS 456 , FEMA 356 , ATC-40

3.12 Summary

The theoretical background of nonlinear seismic analysis of RC structures was presented in this chapter. The concept of geometric nonlinearity, P-Delta effects, plastic hinge theory, moment–curvature relationship, force–deformation behavior, modal effects and torsional irregularity were discussed in detail. These ideas help to understand structural response beyond the elastic range under extreme earthquake

conditions. The chapter also demonstrated the significance of non-linear evaluation methods like the pushover analysis and time history analysis for assessing seismic performances. Further, the pertinent clauses of IS 1893, IS 456, FEMA 356 and ATC-40 were discussed for developing the codal framework for the present study. The theoretical principles, which are discussed in this chapter, provide the foundation for modelling, analysis and results discussion in the subsequent chapters of the research work.

CHAPTER 4

METHODOLOGY

4.1 Introduction

In this study, the behavior of plastic hinge of reinforced concrete (RC) buildings is studied with ETABS software under non-linear seismic loads. A series of four different structural models have been created with the same loading conditions, the same material parameters, the same design requirements, but different plan configurations and heights for an extensive evaluation. This chapter explains the modelling methodology, structural configuration, material data and loading criteria used in each model and the non-linear analysis procedures used.

The two models (Regular and Irregular) are analyzed using (NLTHA) that simulates realistic ground motion response with actual earthquake records. The remaining two models (Tall and Small buildings) are subjected to (NSA) where the progressive yielding and plastic hinge formation under monotonically increasing lateral loads is evaluated. These analyses combine to give a comprehensive view of seismic performance for different building types.

The designs have been prepared as per the Indian Standard codes. The plastic hinge properties are based on FEMA 356 Guidelines to provide internationally accepted performance evaluation criteria. The analyses are performed using a popular commercial finite element software program, namely ETABS v20, which is commonly used in the structural engineering field..

4.2 Description of Structural Models

In this study, four models of buildings which are reinforced concrete framed are taken into consideration. The four models have the same material characteristics, loading on the floor, sizes of members, and seismic zone. Variation is only in plan shape and number of storeys. The use of the uniform approach to design allows the influence of plan regularity and building height on plastic hinge behaviour to be studied and compared separately.

The four building models are classified as: (i) regular G+8 storey building with a symmetric rectangular plan; (ii) irregular G+8 storey building (plan irregularity due to re-entrant corners as per IS 1893:2016); (iii) tall G+14 storey regular building; and (iv) small G+4 storey regular building. All the buildings are in EQ Zone III (Medium Soil Type – Zone II), and they are Special Moment Resisting Frames (SMRF).

The full development of the models depends on the specific building configuration, as shown in

Table 4-1Summary of Building Configuration for All Four Models

Parameter	Model (Regular)	Model (Irregular)	Model (Tall)	Model (Small)
Number of Storeys	G+14	G+14	G+14	G+4
Plan Dimensions (m)	20 x 20	Irregular	20 x 20	20 x 20
Storey Height (m)	3.0	3.0	3.0	3.0
Total Height (m)	43.5	43.5	43.5	13.5
Bay Width X (m)	4.0	Variable	4.0	4.0
Bay Width Y (m)	4.0	Variable	4.0	4.0
No. of Bays (X)	7	Variable	7	5
No. of Bays (Y)	7	Variable	7	4
Analysis Type	TH	TH	Pushover	Pushover

In this study, all four structural models are a reinforced concrete (RC) moment resisting frame system with rigid beam–column connection. The floor system consists of a 125 mm thick reinforced concrete slab which is considered as a rigid diaphragm. The frames are designed as Special Moment Resisting Frame (SMRF) as per the provisions in IS 13920:2016 with the intention of providing adequate ductility and better seismic performance.

Masonry infill walls are excluded from the structural modelling to allow for the evaluation of the seismic behaviour of the bare frame. This method will present a better

idea of plastic hinge development and distribution patterns in the members of the structural system.

4.2.1 Plan and Elevation Details

The plan of the Regular and Tall buildings is symmetric 28 m x 28 m and consists of 4 bays of 5.0 m length in each direction.

The Irregular building has Set Back irregularities.

The plan of the Small is 20m x 16m

The height of all storeys is the same at 3.0 m..

4.2.2 Material Properties

All four models are set to the same material properties to be directly compared in terms of structural performance.. The grade for the concrete and steel are chosen according to the practice followed in medium-high rise RC buildings in India and as per IS 456:2000 and IS 1786:2008 respectively..

4.2.3 Concrete Properties

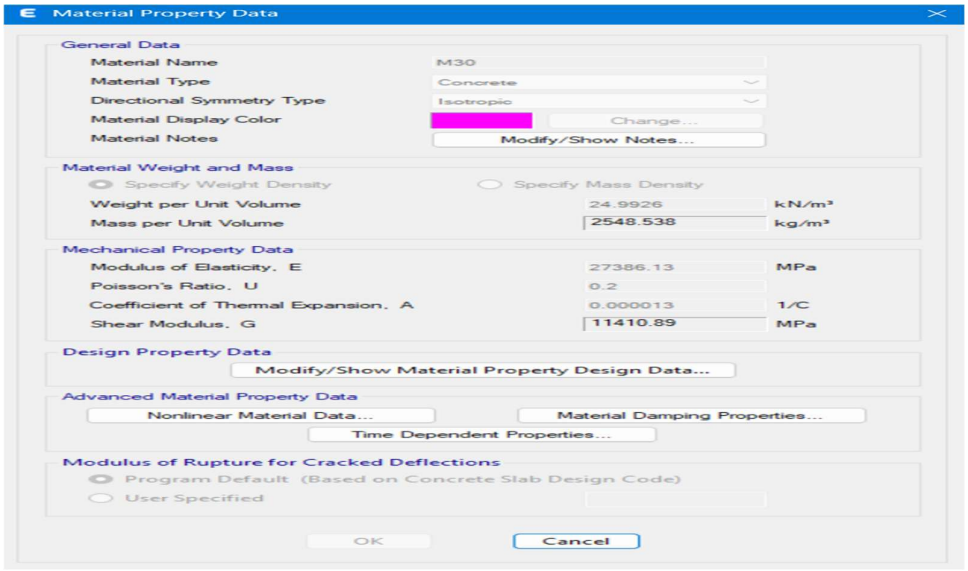

Figure 4.1Concrete Properties

4.2.4 Steel Reinforcement Properties

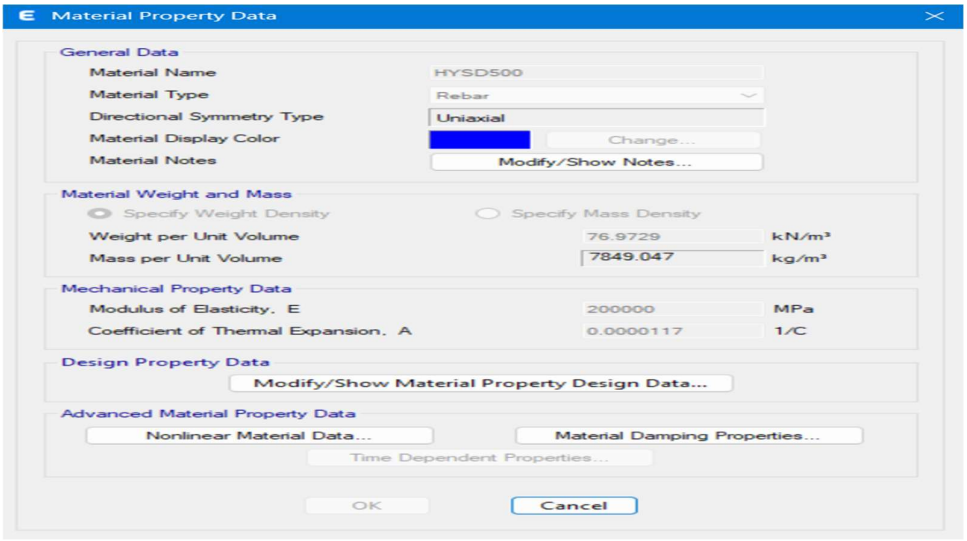

Figure 4.2-Steel Reinforcement Properties

4.3 Loading Criteria

The loads are applied according to the Bureau of Indian Standards (BIS) codes for gravity and seismic loading, on all four building models. The same load is applied to all models to keep the comparison of the behaviour of the hinge from being influenced by load variation. The DL, LL, in the study are detailed in the following subsections..

4.3.1 Dead Load

Dead loads consist of the weight of structural components (calculated by ETABS according to the material density and dimensions), floor finish and brick masonry wall load. All floors are subject to 1.5 kN/m² of load to the floor. Peripheral and internal beam elements are loaded with wall loads as distributed line loads. On roof level beams, the load on parapet walls. These are in compliance with IS 875 Part 1.

4.3.2 Live Load

The live load is applied as per IS 875 Part 2 according to the design category of use. The following loadings are assumed for all typical floors, with residential occupancy, and for the roof 1.5 kN/m² is applied. It is used are applied as uniform area loads to the slab panel. For the NLTH and pushover analyses, Twenty five % of the LL is

considered as the seismic mass contribution in accordance with IS 1893:2016 Clause 7.4.

4.4 Modeling in ETABS

The structural models are developed using ETABS software employing systematic methodically step-by-step procedure. The following elements of the model are defined during the modelling process: the grid layout, storey data, material properties, member sections, support conditions and rigid diaphragm assignments. They are all built independently, but share a common template for material and section properties.

4.4.1 Grid and Storey 3D view

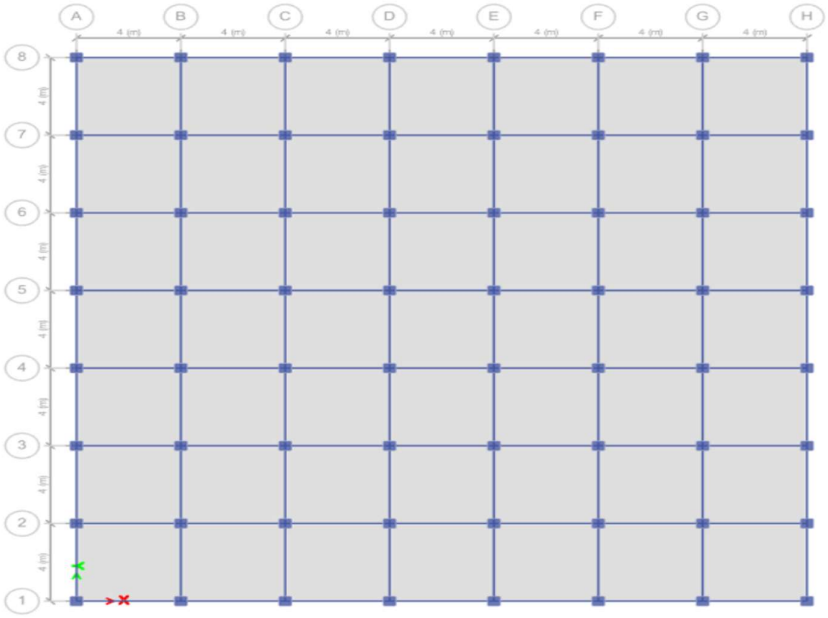

Figure 4.3-Grid view of GF to 7th floor irregular building

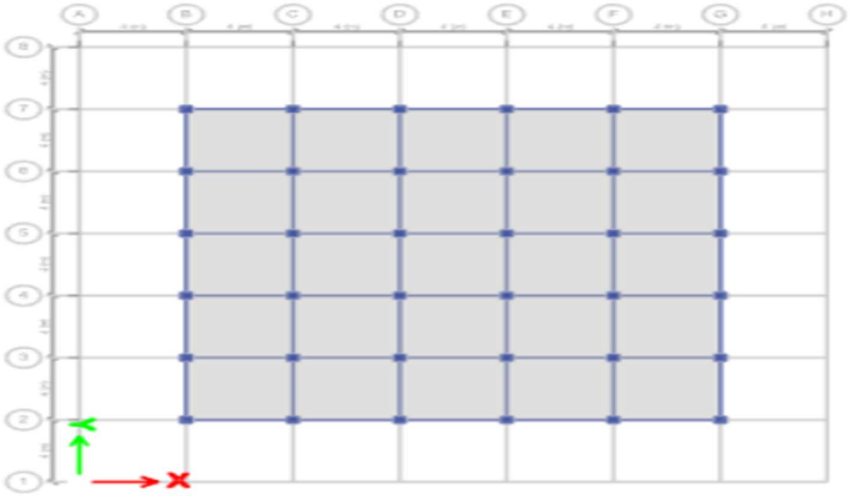

Figure 4.4-Grid up to 8th to 14th floor irregular building

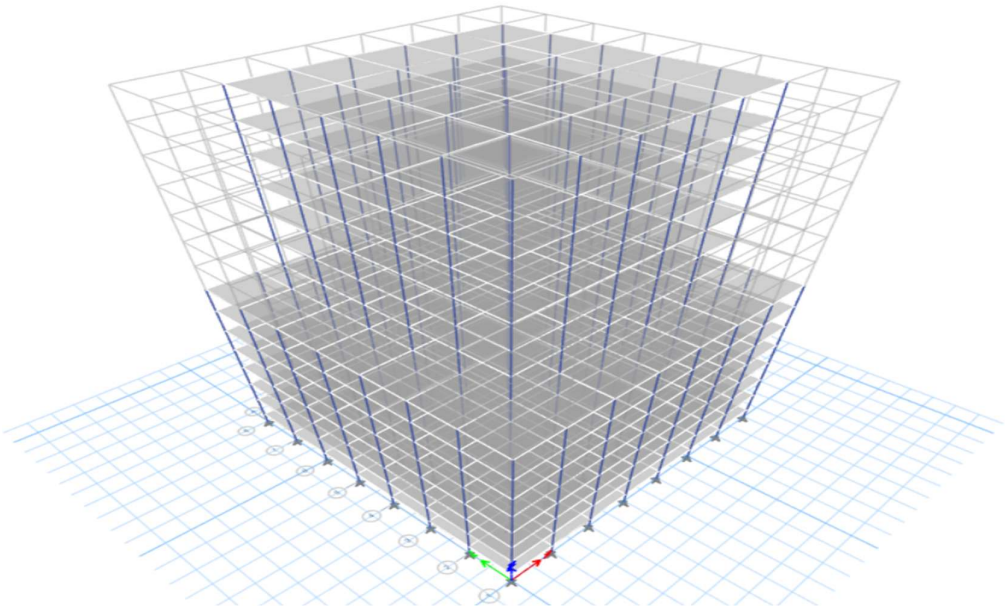

Figure 4.5-3D of irregular building

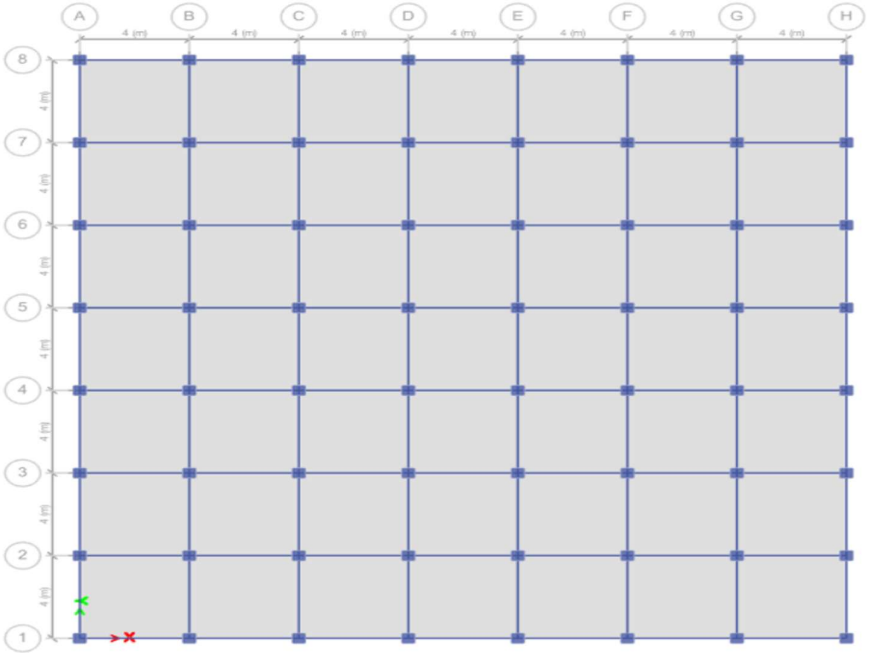

Figure 4.6-Grid view of regular building

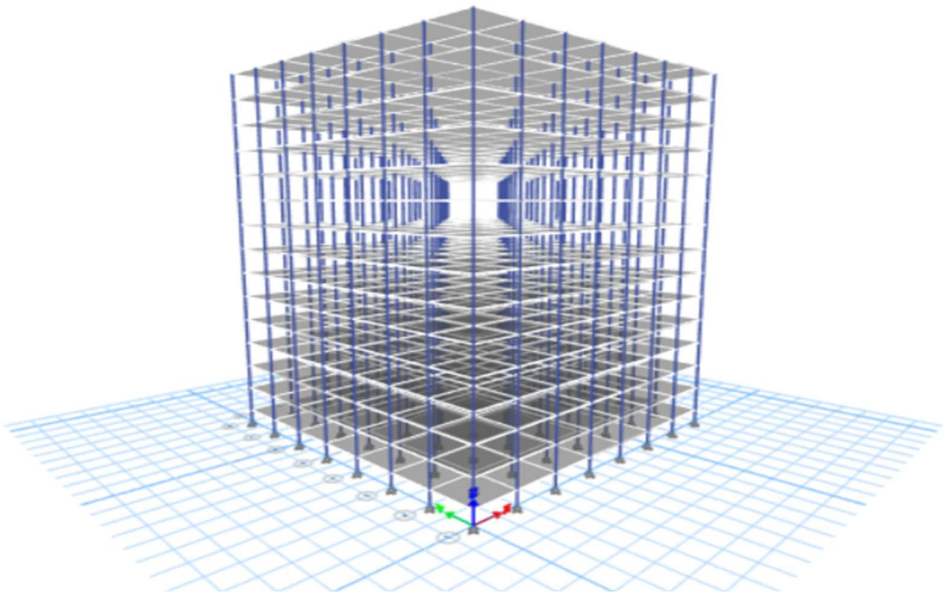

Figure 4.7-3D view of regular building

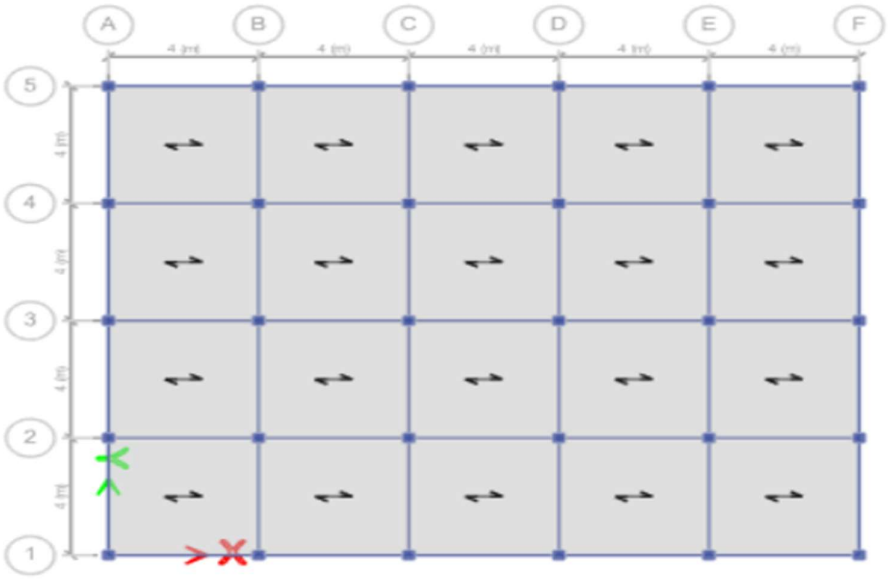

Figure 4.8-Grid view of regular building

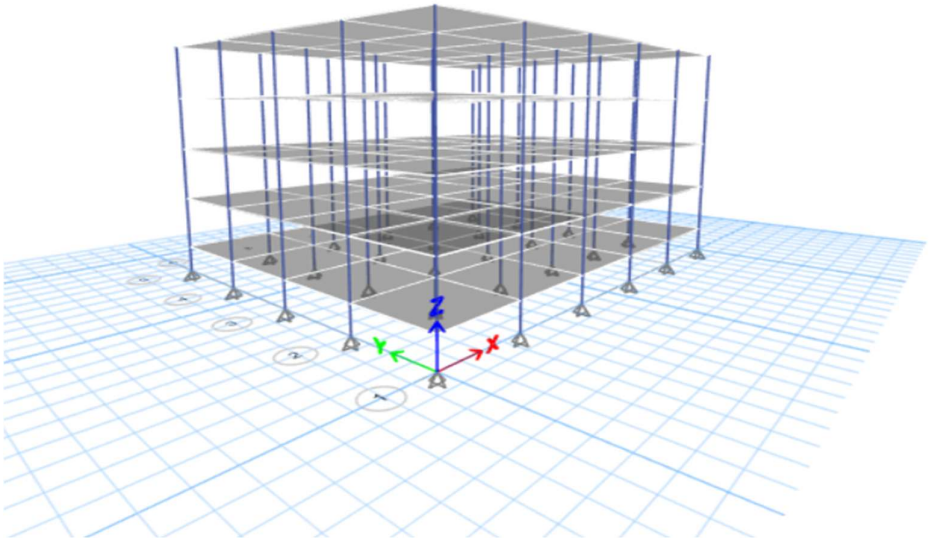

Figure 4.9-3D of regular building

4.4.2 Member Section Properties

All beam and column cross-sections are considered as rectangular RC sections with the given dimensions and reinforcement and model A,B,D have same member propertieE and model C have different section properties. The slab properties is same in all models.

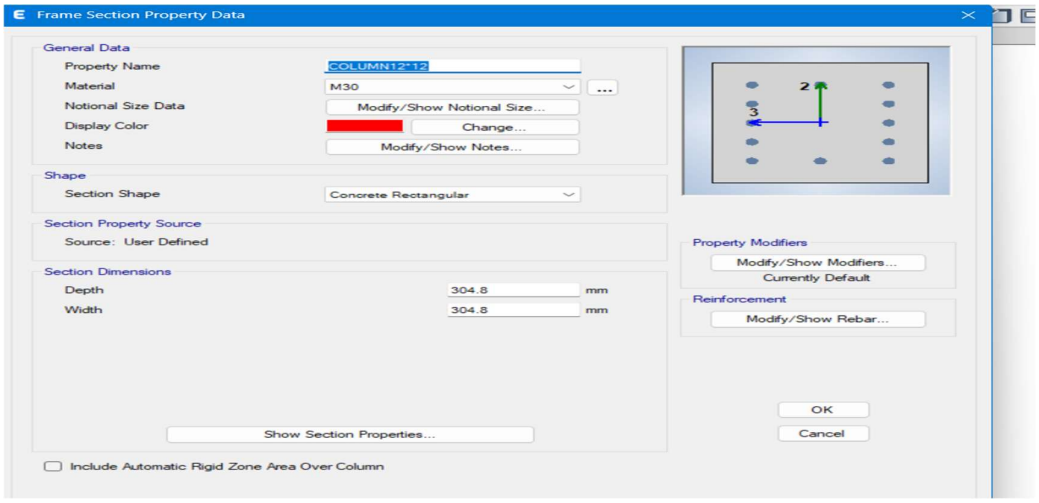

Figure 4.10-Column properties of small building

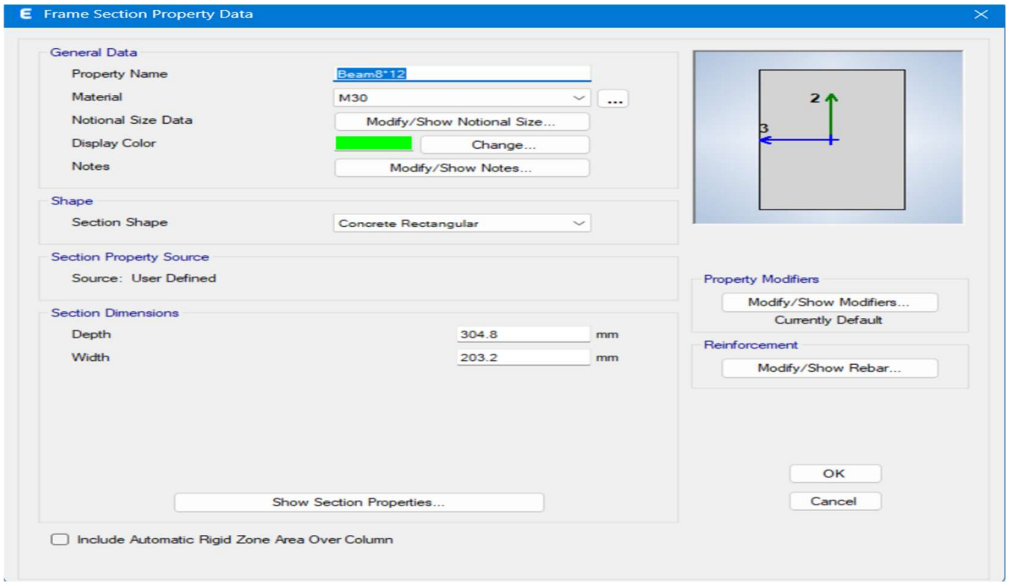

Figure 4.11- Beam Properties of small building

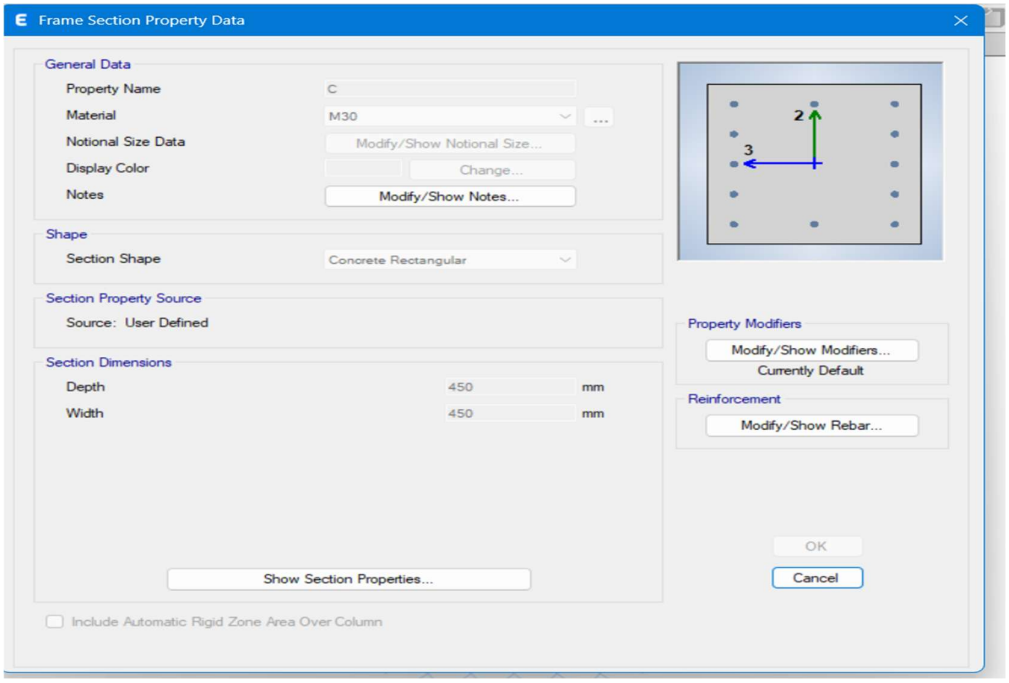

Figure 4.12-column properties tall/regular/irregular building

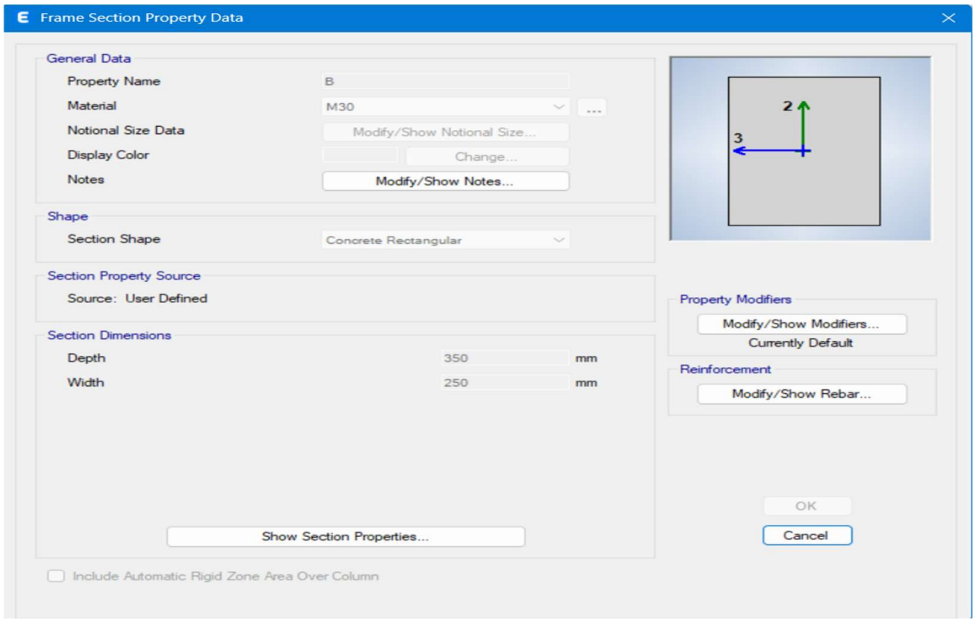

Figure 4.13-Beam properties tall/regular/irregular building

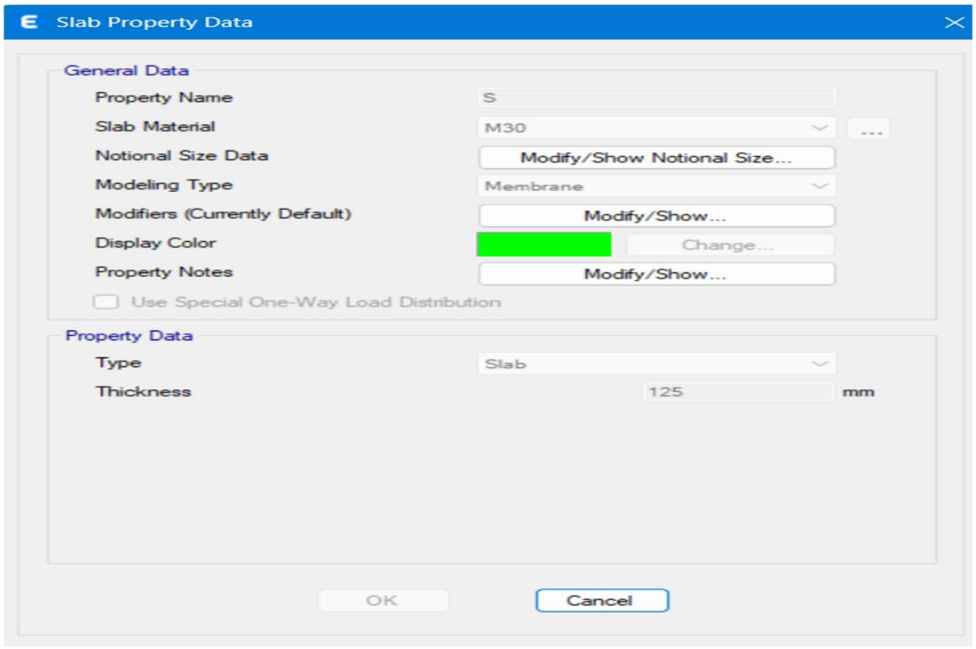

Figure 4.14- Slab Propertie

4.4.3 Support Conditions

Fixed support conditions are applied to all column bases, as long as the foundation system of the columns is stiff and deep enough. This removes the soil-structure interaction effect from the analysis, which is commonly made for comparative studies of this kind. The fixity at the base will prevent plastic hinges from forming at the base of the structure.

4.4.4 Diaphragm Assignment

RC slab stiffness is simulated with the rigid diaphragm constraints at each floor level. This helps to ensure that all columns in a particular floor level are subjected to the same horizontal displacement due to lateral loading. The rigid diaphragm assumption is applicable for all model.

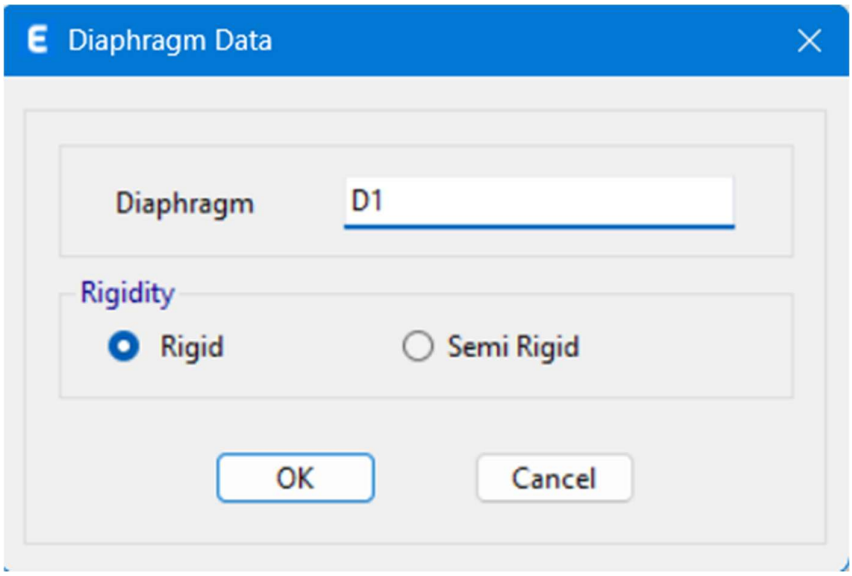

Figure 4.15-Diaphragm Data

4.4.5 Mass source Definition

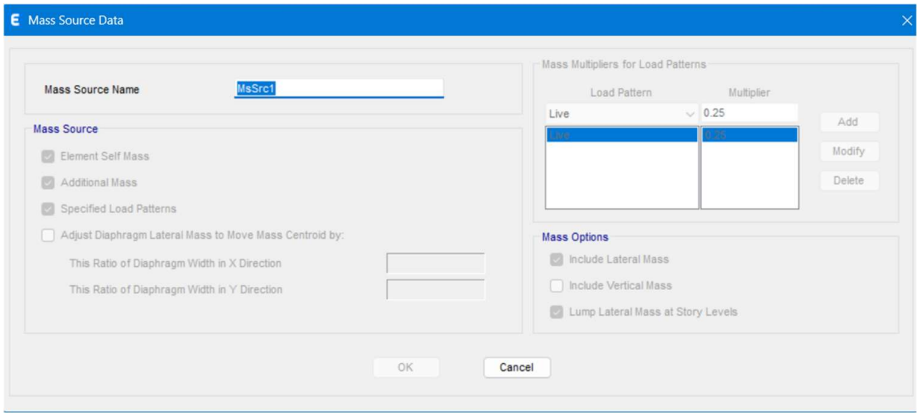

Figure 4.16- Mass Source Data

For each model, you must apply a mass source which is the combination of self weight of building and factored live laod .

4.5 Plastic Hinge Modeling

At the core of the NL analysis is the plastic hinge modelling. For a member loaded beyond its elastic limits, inelastic rotations are concentrated at certain sections which are termed plastic hinges. Plastic hinges in ETABS are defined at the end of frame elements ,and it is characterised using the force-deformation (moment-rotation) relationships according to FEMA 356 / ASCE 41-13 guidelines..

4.5.1 Hinge Properties for Beams

Moment (M3) plastic hinges are placed at both ends of beams, 10% from each beam end. This uniaxial bending behaviour mimics that of beams, which typically dominates beam failure due to lateral loading, and is captured by the M3 hinge. The hinge properties are calculated using the auto-hinge calculation feature of ETABS, based on the section dimensions, reinforcement details and material properties following FEMA 356 Table 6-7 for RC beams.

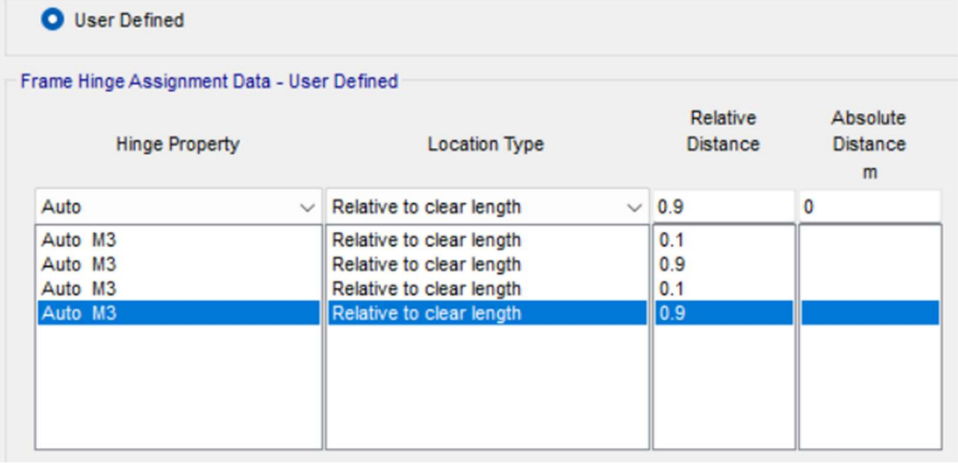

Figure 4.17-Hinges assignment on beam

4.5.2 Hinge Properties for Columns

The column hinges are the 4.7.2 properties. The properties of the column hinges are 4.7.2.

In the case of Axial–Moment–Moment (P–M–M) interaction hingers are assigned to both ends of members of the column to model their non-linear response to seismic loading. These hinges take into account the interaction of axial load and biaxial bending moments which is of importance to accurately measure the performance of reinforced concrete columns under earthquake loading.

The interaction relationship between the P–M is built up depending on the cross-sectional geometry of the columns and its reinforcement configuration. The interaction surface for P–M–M hinges is automatically generated in ETABS based on the fibre hinge modelling approach adhering to FEMA 356 Table 6-8 for nonlinear assessment of structural components..

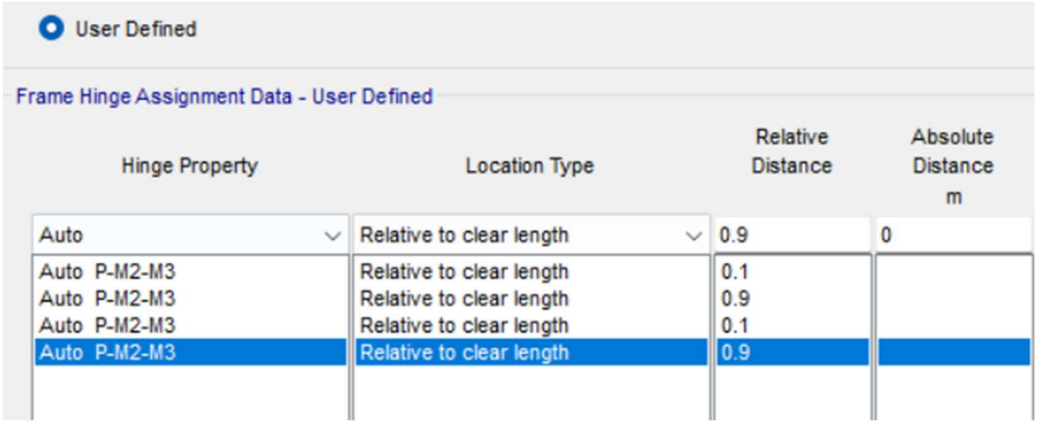

Figure 4.18-Hinges assignment on column

4.5.3 Force-Deformation Relationship (FEMA 356)

The force-deformation characteristics of plastic hinges are 55dealized as a multi-linear curve specified in FEMA 356. The curve has five points of interest: A, B ,C, D , E. The behaviour is elastic between A and B. Yielding initiates and hinge become active at point B. The branch B to C after yielding is called strain hardening. After C, a reduction in capacity is experienced due to degradation and at E, the hinge is unable to carry load.

Table 4-2astic Hinge Force-Deformation Parameters (FEMA 356)

Point		Description	Rotation (Beam)	Rotation (Column)	Performance Level
A		Origin (Unloaded)	0	0	-
B		Yield Point	0.005 rad	0.004 rad	Elastic Limit
C		Ultimate Strength	0.020 rad	0.015 rad	Collapse Prevention
D		Residual Strength	0.025 rad	0.018 rad	Beyond CP
E		Maximum Deformation	0.030 rad	0.020 rad	Failure

IO		Immediate Occupancy	0.005 rad	0.004 rad	Performance Point
LS		Life Safety	0.010 rad	0.008 rad	Performance Point
CP		Collapse Prevention	0.020 rad	0.015 rad	Performance Point

4.6 EQ Parameters and IS 1893:2016 Provisions

Table 4-3-Seismic parameters data

Seismic Parameter	Value / Description
Seismic Zone	Zone IV
(Z)	0.24
(I)	1
(R)	3
Soil Type	Type II (Medium Soil)
Damping Ratio	5%
Time Period	Ta = $\frac{0.09*h}{\sqrt{D}}$
Spectral Acceleration Coefficient	As per IS 1893:2016

4.7 Non-Linear Static (Pushover) Analysis

Non-Linear Static Pushover Analysis (NSA) is an incremental analysis method performed under monotonically increasing lateral loads that is displacement controlled. It offers the capacity curve (Base Shear vs. Roof Displacement), sequence and distribution of plastic hinge formation and performance point under the design earthquake demand. This study uses the Tall (G+14) and Small (G+4) building models that are subjected to a pushover analysis

4.7.1 Pushover Load Case Definition

The pushover load case is a non-linear static load case that follows the end state of the gravity load case. The lateral load is monotonically increased in both directions (+X and +Y) separately to obtain the biaxial response.

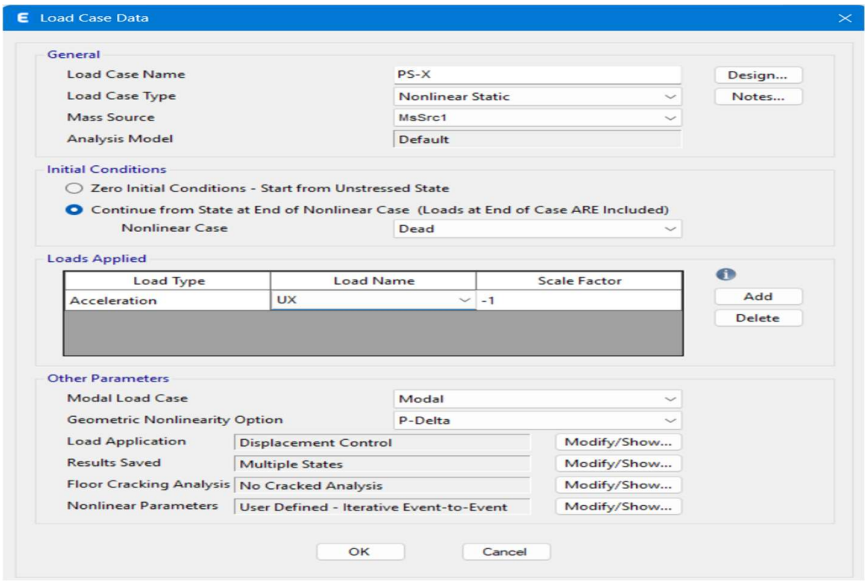

Figure 4.19-PS-X LOAD CASE

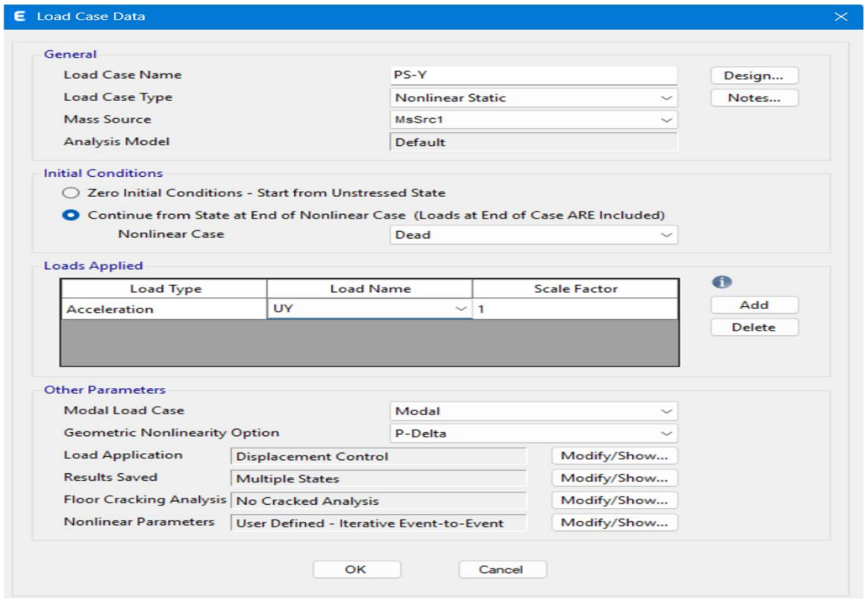

Figure 4.20-PS-Y LOAD CASE

4.7.2 Target Displacement

The target displacement is the maximum roof displacement, and is used to calculate the performance point from the capacity spectrum. In this study, pushover analysis is performed using the displacement-controlled method , where the structure is gradually pushed until the specified displacement limit is reached. The target displacement is taken as 1% of the total building height and is used for the analysis of structural models of low- and high-rise structures in order to investigate the nonlinear seismic response and plastic hinge behavior of these structural models.

Table 4-4-Target Displacement Data

MODEL	HEIGHT	1% OF HEIGHT	TARGET DISPL.
1/3/4	43.5	435	435
2	13.5	135	135

4.8 Non-Linear Dynamic (Time History) Analysis

4.8.1 Time History Load Case Definition

The pushover load case is a non- dyanic load case that follows the real earthquake ground motion data. The ground data is recorded in both directions (+X and +Y) separately to obtain the biaxial response. The scale factore i take 10 because the earthquake I have takin is in cm/sec2 and my ETABS unit is in mm/sec2

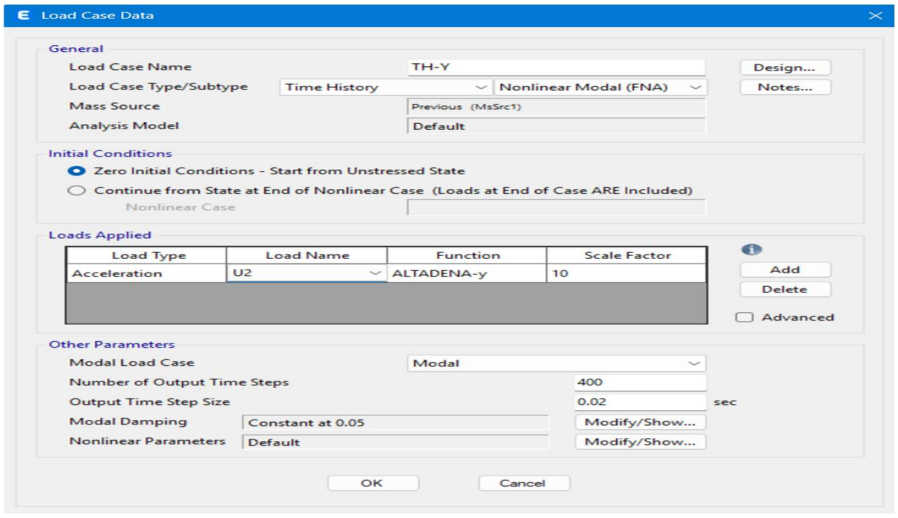

Figure 4.21-TH-Y Load Case

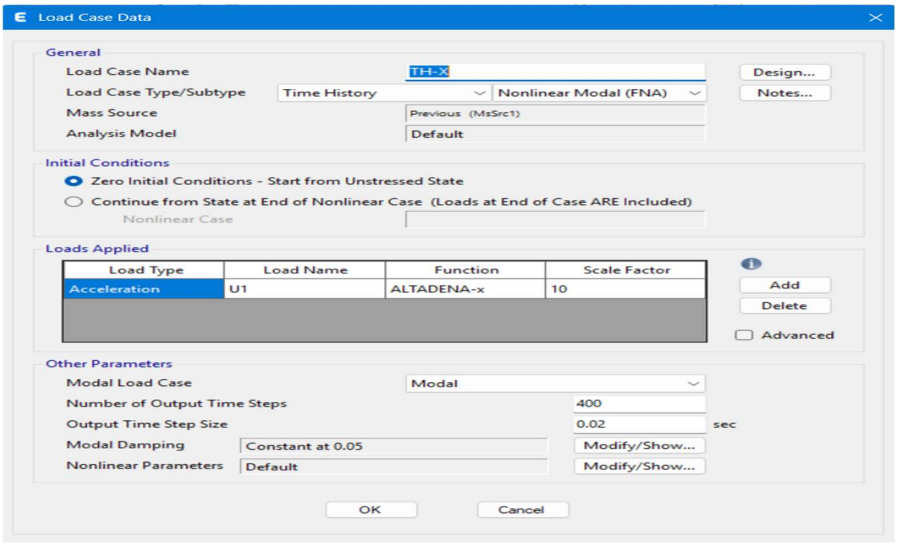

Figure 4.22-TH-X Load Case

4.8.2 Earthquake Data

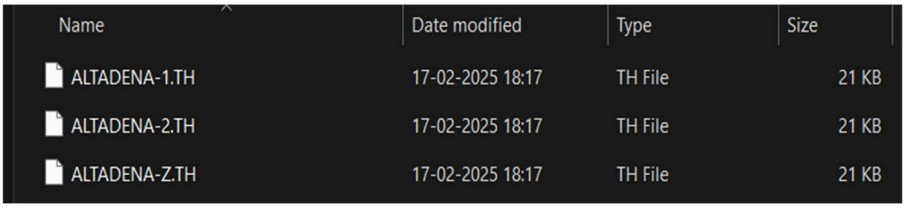

Figure 4.23-Earthquake Data

4.9 Performance Evaluation Methodology

The seismic performance of structures is determined through a comparison of the plastic hinge conditions at a range of seismic demands with the required conditions of acceptance defined in FEMA 356 for the various performance stages. The structural performance is calculated at the target displacement (also called the performance point) in pushover analysis. Time history analysis parameters like maximum inter-storey drift ratio and plastic hinge rotations at maximum structural response are extracted and compared with the imposed limits of (IO), (LS) and (CP).

The major performance parameters that are explored in this study are: (i) base shear capacity and inter-storey ductility derived from the capacity curve; (ii) number, location and sequence of plastic hinge development; (iii) inter-storey drift ratio; (iv) achieved structural performance level (IO, LS, or CP) at the performance point; and

(v) comparison of the plastic hinge distribution patterns between regular and irregular structures as well as low-rise and high-rise buildings.

Table 4-5-Analysis Configuration Summary for All Four Models

Analysis Parameter	Regular (THA)	Irregular (THA)	Tall (Pushover)	Small (Pushover)
Analysis Method	NL-THA	NL-THA	NSA	NSA
Ground Motion / Load Pattern	Scaled Time History	Scaled Time History	Triangular	Triangular
Target Displacement / PGA	MCE Level	MCE Level	As per ATC-40	As per ATC-40
Damping	5%	5%	5%	5%
P-Delta Effect	Considered	Considered	Considered	Considered
Hinge Type	M3 (Beam)	M3 (Beam)	M3 (Beam)	M3 (Beam)
Hinge Type (Column)	PMM	PMM	PMM	PMM
Performance Objective	LS	LS	CP	CP

4.10 Step-by-Step Procedure in ETABS

The following sequence will be followed for modelling and analysis of all four building models in ETABS:

Step 1 — Define Material Properties: Define properties of concrete (M30) and steel (Fe500/Fe415) such as elastic modulus, Poissons Ratio and unit weight.

Step 2 — Define Frame Sections: Use the Section Designer module to define the sections of rectangular beams and columns with the proper dimensions, and to assign reinforcement details.

Step 3 — Create the Structural Model: Add grid lines, story heights and define frame elements by specifying beam and column sections at each node of the grid.

In Step 4 – Assign Boundary Conditions, fix all column base nodes in all 6 degrees of freedom, to simulate fixed support conditions.

Step 5 — Set Rigid Diaphragm: Set all nodes of the model, at each floor level, to be rigid to simulate the rigidity of the slab in planes.

Step 6 — Apply GravityLoads: Define and apply Dead loads and Live loads per IS

875. Allow ETABS to auto-calculate self-weight.

Step 7 — Define the mass sources and assign it .

Step 8 — Run Linear Analysis: Conduct linear static and modal analysis for design verification and to find fundamental frequencies.

Step 9 — Define Plastic Hinges: Set up FEMA 356 auto-hinges (M3 is for beams and PMM is for columns) at the ends of the members in ETABS using the Frame Hinge Overwrite option.

Step 10 — Define Non-Linear Load Cases: Specify the gravity non-linear load case and the pushover and time history non-linear load cases in a suitable fashion with a set of parameters.

Step 11 — Run Non-Linear Analysis: Perform non-linear analysis and check for convergence at each load step. If convergence problems occur, change the step size.

Step 12 — Post Processing: Generate the capacity curve, hinge formation plots, inter-storey drift profiles and storey shear distribution etc for evaluation of performance.

4.11 Summary

This chapter has given a detailed methodology for modelling, analysis and performance evaluation of four models of RC buildings in ETABS. The four models are Regular, Irregular, Tall and Small, all of which have the same material and loading characteristics to allow a meaningful comparison to the behaviour of plastic hinges under seismic loading.

The effect of plan irregularity on the dynamic seismic response and hinge distribution are studied using Non-Linear Time History Analysis for both the Regular and Irregular buildings. Non-Linear Static Pushover Analysis is used for the Tall and Small buildings to investigate how building height affects their lateral capacity, ductility and progressive hinge formation. Plastic hinge properties are specified according to FEMA 356 guidelines, and performance is determined under IO, LS and CP levels.

The detailed modelling procedures, properties of materials, load definitions, hinge configurations and analysis settings discussed in this chapter are needed to support the

results and discussion in the following chapters. The systematic and consistent approach used makes the comparative results of this study reliable and unbiased.

CHAPTER 5

RESULTS

5.1 Non-Linear Static Pushover Analysis (Case 1: Short vs. Tall Frame)

This segment describes the results achieved through the application of Nonlinear Static Pushover Analysis (NLSPA) on two reinforced concrete frame structures. The structure analyzed in this case is referred to as Structure A (G+4) and Structure B (G+14). Different seismic performance parameters, such as the natural period of time, pushover capacity curve, formation and location of plastic hinge, inter-story drift ratio, and ductility factor, are considered for evaluation.

5.1.1 Natural Periods and Modal Mass Participation

The fundamental natural time period (T) is an important parameter in affecting the dynamic behavior of the structure. The natural time periods and their respective frequencies of vibration of the first three vibration modes of the structures are provided in Table 5.1.

Table 5-1-Natural Periods and Frequencies — Tall vs. Short Frame

Mode	Tall Building T (s)	Tall Building f (Hz)	Small Building T (s)	Small Building f (Hz)
1	2.021	0.495	1.137	0.880
2	2.021	0.495	1.119	0.894
3	1.847	0.541	0.990	1.010

The tall building structure is characterized by much greater fundamental natural time period (T_1 = 2.021 s) than the short building structure (T_1 = 1.137 s), primarily due to lesser stiffness of the former. From the outcome of participation ratios, it can be seen that the contribution of first mode towards the total mass participation is about 86% for the taller structure and over 93% for the shorter one in both UX and UY directions.

Table 5-2-Modal participation

Sum (%)	Tall Building	Small Building
UX (%)	0.8645	0.9348
UY (%)	0.8645	0.9350
RZ (%)	0.9543	0.9784

5.1.2 Pushover Capacity Curves

The pushover capacity curve reflects the nonlinear force-deflection performance of the structure .

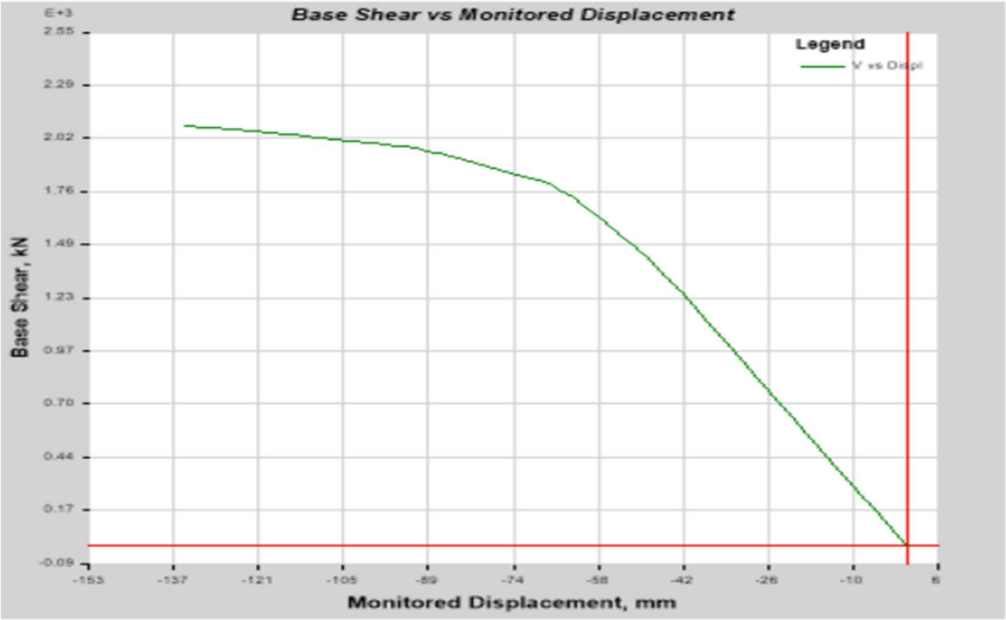

Figure 5.1-Push Over of Small building in Y direction

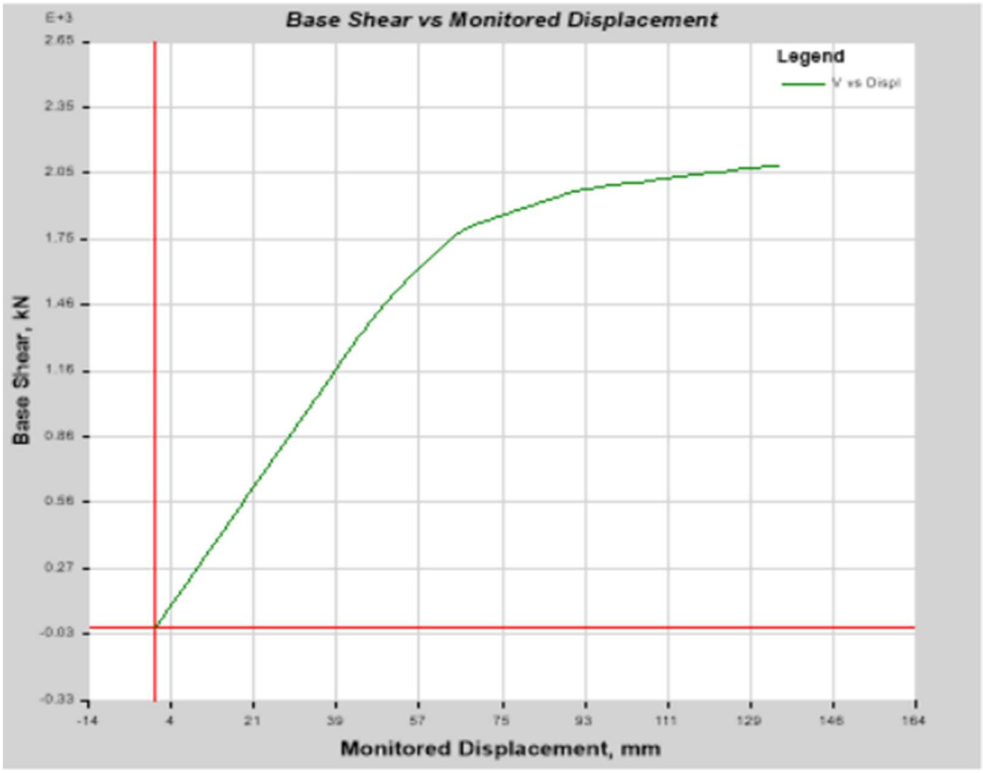

Figure 5.2-Push Over of Small building in X direction

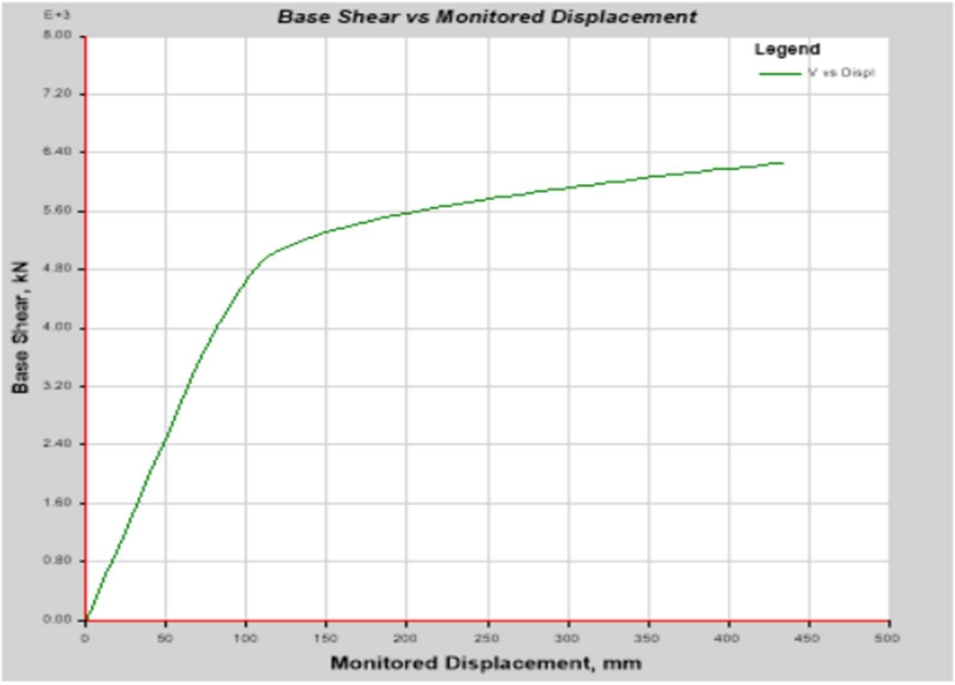

Figure 5.3-Push Over of Tall building in X direction



Figure 5.4-Push Over of Tall building in Y direction

For Models A (G+4) and B (G+14), the highest capacity of base shear obtained through the pushover analysis is 12,032.06 kN and 65,524.65 kN, respectively, due to the much higher total mass and number of members of the tall structure.

5.1.3 Performance Point

The PP is obtained from the interaction between the capacity spectrum and demand spectrum, as stated in ATC-40 clause 6.2.1 and. This is the maximum displacement and shear base demand attained by the structure when the earthquake is applied to it. Table 5.3 shows the performance point criteria for both systems.

Table 5-3-performaton point

Parameter	Small (G+4)	Tall (G+14)	Remark
Performance Point Displacement (mm)	41.85	67.25	From capacity spectrum intersection
Performance Point Base	1251.057`5	3402	Demand vs capacity

Parameter	Small (G+4)	Tall (G+14)	Remark
Shear (kN)			
Performance Level Achieved	IO	IO	Classify per FEMA 356

ATC-40 clause 6.2 (Capacity Spectrum & Performance Point).

As can be seen , the tall model needs a higher displacement value at its performance point (67.25 mm vs 41.85 mm) and higher base shear force (3,402 kN vs 1,251.06 kN), owing to the higher forces that are required for the taller building. Both models meet the immediate occupancy performance objective throughout the building, and none of their hinges exceed the life safety or collapse prevention criteria.

5.1.4 Inter-Storey Drift Ratio

The (ISDR) is used as the basic measure to quantify damage to the structure under lateral loads. The maximum allowed value according to FEMA 356 at the CP level is 4%. ISDR are given in Table 5.4 .

Table 5-4- Inter Storey Drift

Storey	Small G+4 Drift (%)	Tall G+14 Drift (%)	Limit (FEMA 356)	
GF	0.021761	0.029405	4% (CP limit)	2% (LS limit)
1	0.017534	0.028342	4% (CP limit)	2% (LS limit)
2	0.009887	0.025553	4% (CP limit)	2% (LS limit)
3	0.004589	0.021606	4% (CP limit)	2% (LS limit)
4	0.002119	0.017057	4% (CP limit)	2% (LS limit)
5	—	0.012446	4% (CP limit)	2% (LS limit)
6	—	0.008316	4% (CP limit)	2% (LS limit)
7	—	0.005214	4% (CP limit)	2% (LS limit)
8	—	0.003369	4% (CP limit)	2% (LS limit)
9	—	0.002454	4% (CP limit)	2% (LS limit)
10	—	0.001948	4% (CP limit)	2% (LS limit)
11	—	0.00155	4% (CP limit)	2% (LS limit)
12	—	0.001183	4% (CP limit)	2% (LS limit)

Storey	Small G+4 Drift (%)	Tall G+14 Drift (%)	Limit (FEMA 356)	
13	—	0.000834	4% (CP limit)	2% (LS limit)
14	—	0.000531	4% (CP limit)	2% (LS limit)

Code Reference: FEMA 356 Table C1-3

Drift ratios for all the floors in small building are way below the collapse limt. For Model B (tall frame), the maximum drift ratio is found on the ground floor with an ISDR of 0.029405%, which is higher than CP limt which is point of concern. Maximum drift ratio for Model A (short frame) is recorded on the ground floor with an ISDR of 0.021761%. Higher drift ratios at low storeys can be attributed to cantilever action in moment resisting frames.

5.1.5 Plastic Hinge Distribution and Performance States

The plastic hinge development is one of the most significant parameters reflecting nonlinear behavior under seismic loads. The hinges are categorized according to FEMA 356, Figure C2-1 Acceptance Criteria: A-B (linear), B-C (IO to LS), C-D (LS to CP), D-E (CP to collapse), and above E (failure). In table 5.5, the distribution of hinges at the performance level is shown for each model.

Table 5-5: Plastic Hinge Distribution at Performance Point — Model A vs. Model B

Parameter	Model A (G+4)	Model B (G+14)
Total plastic hinges formed	1,580	10,560
Hinges in A–B (Elastic / B–IO)	1,574	10,548
Hinges in B–C (IO–LS)	6	12
Hinges in C–D (LS–CP)	0	0
Hinges beyond D (≥ CP / Collapse)	0	0
Hinge mechanism type	Beam Sway	Beam Sway

FEMA 356 Fig. C2-1 Acceptance Criteria

However, both models create the mechanism of beam sway without any hinges reaching LS-CP or collapse conditions, and thus demonstrate full compliance with the

Basic Safety Objective outlined in FEMA 356, section 1.5.1. The difference in the number of hinges (10,560 for Model A against 1,580 for Model B) reflects the greater number of members in the 14-story building frame

5.1.6 Step-by-Step Hinge Progression

Incremental hinging at 25%, 50%, 75%, and 100% of target displacement indicates the process of inelastic demands on the structure as it is pushed towards its performance limit. Table 5.6 illustrates the increment of hinging in both types of models.

Table 5-6Step-by-Step Hinge Progression — Model A and Model B

Model	Step	δ (mm)	B–IO	IO–LS	LS–CP	> CP	Total
A	25% δt	25	1580	0	0	0	1580
A	50% δt	50	1580	0	0	0	1580
A	75% δt	75	1580	0	0	0	1580
A	100% (PP)	100	1520	60	0	0	1580
B	25% δt	25	10560	0	0	0	10560
B	50% δt	50	10560	0	0	0	10560
B	75% δt	75	9888	672	0	0	10560
B	100% (PP)	100	9664	896	0	0	10560

In Model A, however, all hinges stayed elastic (A-B stage) up until the third step, where only 60 hinges reached the IO-LS stage during the full performance stage. This signifies that the small frame retains the reserve capacity of elastic stiffness throughout the majority of its loading capacity. In Model B, there was a gradual transition where 672 hinges reached the IO-LS state at 75%δt while 896 were in this state during the full performance stage of 100%δt. There are no LS-CP or collapse-state hinges in both frames, showing proper performance against seismic action.

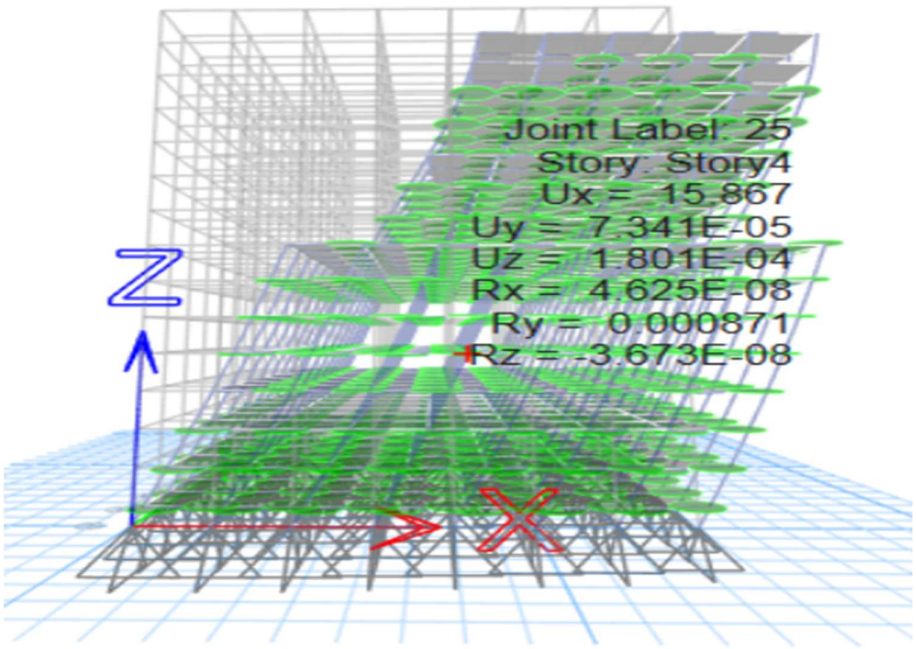

Figure 5.5-Plastic Hinge atTarget Displacment in Irregular Building

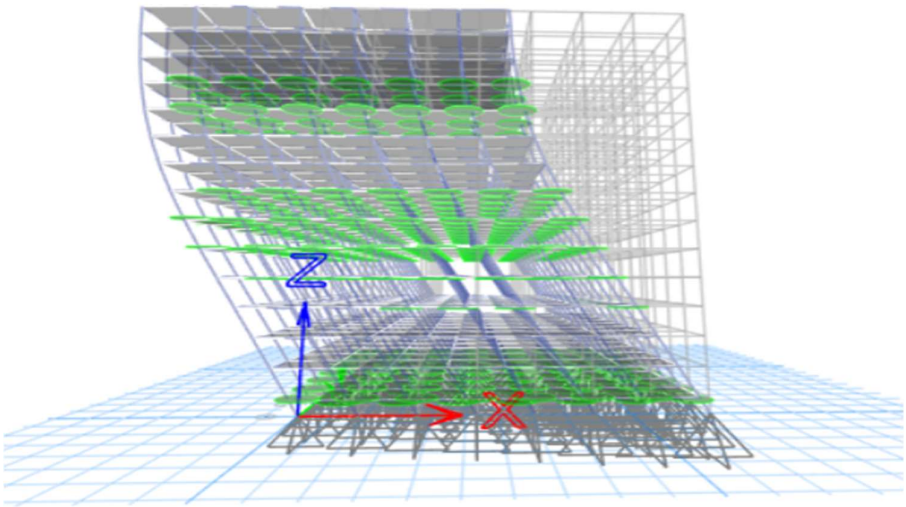

Figure 5.6-Plastic Hinge at Target Displacement irregular Building

5.1.7 Ductility Ratio

Ductility ratio ($\mu = \delta t / \delta y$) represents the ductile capacity of the building with respect to its yield displacement capacity. Larger values of ductility indicate greater capacity for inelastic deformation before failure.

Table 5-7Ductility Ratio Comparison — Model A vs. Model B

Parameter	Formula	Small G+4	Tall G+14
Yield Displacement δy (mm)	From pushover curve	41.85	67.25
Target Displacement δt (mm)	From pushover curve	135	435
Ductility Ratio ($\mu = \delta t/\delta y$)	—	3.22	6.46

Code Reference: FEMA Table 6-6 (Ductility Demand)

Ductility Demand (G+14) in Model B is almost double ($\mu = 6.46$) that in Model A ($\mu = 3.22$). The doubling in the requirement for ductility emphasizes the need for adequate provision of ductility.

5.1.8 Roof Displacement

Displacement at Roof Level

The roof level displacement refers to the lateral displacement of the highest level floor at the performance level point. This represents the total lateral deformations that the building will be subjected to. Table 5.8 below shows the roof displacement at different levels.

Table 5-8-Storey-wise Roof Displacement — Model A and Model B

Storey	Tall G+14 (mm)	Small G+4 (mm)
14	435.314	—
13	433.722	—
12	431.220	—
11	427.671	—
10	423.021	—
9	417.177	—
8	409.815	—
7	399.708	—

Storey	Tall G+14 (mm)	Small G+4 (mm)
6	384.065	—
5	359.117	—
4	321.778	135.019
3	270.607	128.672
2	205.790	114.906
1	129.132	85.245
GF	44.107	32.642

The long structure experiences a maximum roof displacement value of 435.314 mm, while the short one is 135.019 mm. The two displacements' patterns appear to be concave with no sharp breaks showing that the two structures experience a global sway mode rather than weak story and torsion modes of collapse.

5.1.9 Comprehensive Comparison — Model A vs. Model B

Table 5.9 provides a consolidated comparison of all key pushover analysis parameters for Model A and Model B.

Table 5-9Comprehensive Comparison — Model A vs. Model B

Parameter	Model A (G+4)	Model B (G+14)
Max. Base Shear (kN)	12,032.06	65,524.65
Target Displacement (mm)	135	435
Ductility Ratio (μ)	3.22	6.46
Performance Point – Base Shear (kN)	1,251.06	3,402
Performance Point – Displacement (mm)	41.85	67.25
No. of Hinges = A–IO	1,520	9,664
No. of Hinges = IO–LS	60	896
No. of Hinges = LS–CP	0	0
No. of Hinges ≥ CP	0	0
Max. ISDR (%)	0.021761	0.029405
Hinge Mechanism	Beam Sway	Beam Sway

5.2 Non-Linear Dynamic Time History Analysis (Case 2: Regular vs. Irregular Frame)

This chapter includes the outcomes from the (NLTHA) performed for two G+14 RC frames namely Model C (Irregular) and Model D (Regular). The analyses have been carried out using ground motions separately in X and Y directions. The important parameters considered include modal parameters, story displacement, story drift, base reactions, and development of plastic hinges.

5.2.1 Modal Analysis — Time Period and Frequency

The primary time periods and frequencies are of critical importance in governing the dynamic behaviour and seismic response of buildings. In regular configurations, the first two vibration modes are expected to be purely translatory modes without any torsional component involved. The modal data thus obtained are summarized in Table 5.10.

Table 5-10Comparison of Modal Time Periods and Frequencies — Regular vs. Irregular Building

Mode	Regular T (s)	Regular f (Hz)	Irregular T (s)	Irregular f (Hz)
1	2.021	0.495	1.673	0.598
2	2.021	0.495	1.673	0.598
3	1.847	0.541	1.328	0.752

Irregular structure has a lesser fundamental period (T_1 = 1.673 s) compared to that of the regular structure (T_1 = 2.021 s) which denotes higher stiffness because of its structural configuration. Nevertheless, due to decreased stiffness in some directions and eccentric mass, torsion in the irregular structure is high, hence the reduced RZ mass participation ratio of 0.9293 compared to that of the regular structure 0.9543. Table 5.11 shows the mass participation ratios for different modes.

Table 5-11Modal Mass Participation Ratios (%)

Sum (%)	Regular Building	Irregular Building
UX (%)	0.9538	0.9397
UY (%)	0.9538	0.9397
RZ (%)	0.9543	0.9293

Minimum 90% cumulative required per IS 1893 (Part 1):2016 Cl. 7.7.5.2

5.2.2 Story Displacement

The absolute horizontal displacement of any floor in relation to the base floor is referred to as story displacement. For TH-X loads, X-displacements rule; while TH-Y loads, Y-displacements rule. Both must be evaluated. Table 5.12 summarizes the maximum story displacements for both structures with each of the loads.

Table 5-12Maximum Story Displacements (mm) — Regular vs. Irregular Building

Story	Regular TH-X (mm)	Regular TH-Y (mm)	Irregular TH-X (mm)	Irregular TH-Y (mm)
14	74.18	17.661	91.727	20.112
13	69.85	17.111	90.623	19.387
12	69.656	16.325	95.944	18.377
11	74.805	15.45	100.841	17.921
10	78.158	14.788	102.455	18.025
9	78.011	14.702	98.793	18.041
8	73.407	14.919	89.36	18.028
7	64.806	14.814	76.288	17.865
6	53.18	14.14	63.631	17.284
5	41.156	12.786	50.252	16.19
4	32.666	10.818	39.951	14.561
3	26.134	8.449	31.884	12.294
2	22.481	5.955	24.296	9.368
1	15.64	3.56	15.557	5.875
PB	5.611	1.216	5.406	2.003

The building that shows irregularity exhibits larger displacement values than the regular building during TH-X loading for all stories. The largest value of displacement in the direction of X-axis obtained for the building that is showing irregularity is 102.455 mm in Story 10, while the corresponding value for the regular building is 78.158 mm at Story 10, an increase of 23.71%. This increment arises due to torsional interaction between translational and rotational modes of irregular buildings, as

defined by IS 1893 (Part 1): 2016 Clause 7.8 On the other hand, the value of displacement difference obtained from TH-Y loading is relatively small.

5.2.3 Story Drift Ratio

Ratio of story drift refers to the ratio of the inter-storey drift to the storey height. Ratio of story drift forms the major seismic damage index for structural and non-structural parts of the building. The permissible value according to IS 1893 (Part 1): 2016, Clause 7.11.1 is given by h/250 = 0.004.

Table 5-13Maximum Story Drift Ratios — Regular vs. Irregular Building

Story	Regular TH-X	Regular TH-Y	Irregular TH-X	Irregular TH-Y
14	0.001	0.0004	0.002	0.0005
13	0.002	0.0006	0.003	0.0007
12	0.003	0.0007	0.004	0.0008
11	0.003	0.0006	0.004	0.0009
10	0.003	0.0007	0.004	0.0009
9	0.002	0.0007	0.003	0.0009
8	0.002	0.0007	0.00	0.0008
7	0.004	0.0006	0.004	0.0005
6	0.004	0.0005	0.004	0.0006
5	0.004	0.0006	0.004	0.0007
4	0.003	0.0007	0.004	0.0008
3	0.002	0.0008	0.003	0.0009
2	0.002	0.0008	0.003	0.001
1	0.003	0.0007	0.003	0.001
PB	0.003	0.0008	0.003	0.001

Permissible limit = 0.004 per IS 1893 (Part 1):2016 Cl. 7.11.1 (h/250).

In the TH-X case study, the irregular structure has passed the allowable drift ratio value of 0.004 starting from Stories 6 up to 12 with maximum drift ratios of 0.004to 0.004. On the other hand, there are minimal drift ratios exceeding the 0.004 value in stories 6 and 7 of the regular structure with the values being 0.00 and 0.004 respectively. In

the TH-Y case study, all drift ratios in the two structures are well within the allowable drift ratio value of 0.004.

5.2.4 Base Reactions

The base reactions are the sum of all forces and moments acting on the foundation level due to earthquake. For TH-X loading, the resultant FX and MY dominate while for TH-Y loading, the resultant FY and MX dominate. The comparison is presented in Table 5.14.

Table 5-14 Base Reactions Comparison — Regular vs. Irregular Building

Building / Case	FX (kN)	FY (kN)	FZ (kN)	MX (kN·m)	MY (kN·m)	MZ (kN·m)
Regular TH-X	4235.68	0.0009	0	0.0008	72300.9	68392.4
Regular TH-Y	740.3639	0.0002	0	0.0002	22664.7	13752.2
Irregular TH-X	5441.41	0.001	0	0.0005	72261.2	65437.2
Irregular TH-Y	823.0323	0.0002	0	0.0001	26399.5	22528.9

In the case of TH-X loading, the base shear for the irregular structure is increased to 5,441.41 kN from 4,235.68 kN in the case of the regular building – an increase of 22.15%. This can be attributed to the magnification of the seismic forces due to torsion in the irregular structure. In the case of TH-Y loading, the magnitude of MZ (the overturning force) of the irregular structure is increased to 22,528.9 kN•m from 13,752.2 kN•m – an increase of 38.9%.

5.2.5 Plastic Hinge Formation and Performance States

Plastic hinges occur where the bending moment exceeds the plastic moment of resistance (Mp). Performance criteria of hinge actions according to FEMA 356 Sec Tables 6-7 .Table 5.15 shows hinge numbers based on hinge actions for both structures under TH-X and TH-Y loads.

Table 5-15Plastic Hinge Count by Performance State — Regular vs. Irregular Building

Building / Case	A–B (Elastic)	B–C (IO–LS)	C–D (LS–CP)	D–E / >E (Collapse)
Regular — TH-X	9,472	1,088	0	0
Regular — TH-Y	10,561	0	0	0
Irregular — TH-X	6,239	2,087	0	0
Irregular — TH-Y	8,319	0	0	0

When subjected to TH-X loads, the irregular structure exhibits 2,087 hinges in the B-C (IO-LS) zone, whereas the regular structure displays 1,088 hinges, marking an increment of 47.86% in hinge formation. The above difference is attributed to the torsional amplification caused by the eccentricity between the centers of mass and stiffness, in accordance with IS 1893 (Part 1): 2016, Clause 7.8. For the TH-Y load combination, no hinges form past the elastic A-B region, indicating TH-X as the dominating load direction. None of the members enter into zones C-D or fail, indicating that the buildings satisfy the Life Safety .

The regular structure fulfills the strong column-weak beam (SCWB) criterion as per IS 13920: 2016 Clause 7.2.1 with more hinges on beams than columns in both directions of loading. The story-wise distribution of hinges in the case of regular structure is equally distributed on all stories, thus showing the ductility of global mechanism. On the other hand, the pattern of hinge demand is concentrated on the mid-story level for TH-X due to irregularity in the plan aspect of the structure.

Plastic hinge rotations in the ground floor beam are important in terms of governing criteria for seismic adequacy at a member level. Column rotations are particularly more important considering that there are other axial forces present, which make their rotation capabilities limited are necessary for the sustained rotations . Table 5.16 presents the maximum plastic hinge rotation demands alongside FEMA 356 acceptance criteria.

Table 5-16 Plastic Hinge Rotation Demand

Model	Critical Member	θp Demand (rad)	θp,cap CP (rad)	D/C (CP)	θp,cap LS (rad)	D/C (LS)
Irregular (G+14)	GF Beam	0.034102	0.020	1.700	—	—
Regular (G+14)	GF Beam	0.00025	0.025	0.010	0.020	0.00125

Rotation limits per FEMA 356 Tables 6-7

The standard G+14 structure shows the insignificant requirement for GF rotation of the beam of 0.00025 rad (D/C=0.010 at CP), showing that almost elastic behavior takes place at CP. The G+14 irregular structure indicates the requirement for GF rotation of 0.034102 rad (D/C = 1.70 at CP), clearly surpassing the CP restriction of FEMA 356. The tall structure subjected to the pushover shows 0.0499 rad (D/C = 2.495 at CP). These findings show that structures with taller frames and those with geometrically irregular frames are extremely sensitive to plastic hinge rotation exceedance.

5.2.6 Overall Structural Performance Summary

Performance comparison for all important criteria for both loading and both structural types in terms of IS 1893 (Part 1): 2016, IS 13920: 2016, and FEMA 356 Table C1-3 is shown in Table 5.17.

Table 5-17Overall Structural Performance Summary — Regular vs. Irregular Building

Parameter	Regular — TH-X	Regular — TH-Y	Irregular — TH-X	Irregular — TH-Y
Max roof displacement (mm)	78.011	17.661	102.455	20.112
Max story drift ratio	0.004	0.0008	0.005	0.001
Max base shear (kN)	4235.68	740.36	5441.41	823.03
Total hinges formed	10,560	10,560	8,326	8,236
Max hinge rotation — Beams (GF)	0.00025 rad	—	0.034102 rad	—
Strong col–weak beam satisfied	Yes	Yes	Yes	Yes
Performance level	IO	IO	IO	IO

5.3 Discussions

5.3.1 Effect of Building Height on Plastic Hinge Properties

When performing comparative evaluations of Models A and B by pushover analyses, we conclude that the height of structures greatly affects plastic hinge formation and non-linear earthquake responses. The taller structure creates many more plastic hinges (10,560 hinges versus 1,580) because it consists of many more structural members than the shorter structure. Nevertheless, when evaluating both models concerning their numbers of members, we see that they behave equally in terms of plastic hinge formation.A significant difference arises when comparing the ductility demands for both models; model B has a greater ductility ratio ($\mu = 6.46$) than Model A ($\mu = 3.22$). Thus, model B experiences about twice as much demand for inelastic displacement as Model A. This difference can be explained by the greater fundamental time period of the taller structure ($T_1 = 2.021$ s versus $T_1 = 1.137$ s).Ground-storey beams of the taller structure face a greater demand for rotational displacement, which is 0.0499 rad, whereas the corresponding demand for Model A is much less (D/C = 2.495 at Collapse Prevention level), surpassing the allowable limits of FEMA 356 guidelines. Hence, ductile detailing of these particular parts needs to be improved following IS 13920:2016 standards.

5.3.2 Effect of Plan Irregularity on Plastic Hinge Properties

The study on the regular and irregular G+14 frames with NLTHA has highlighted the harmful effect of plan irregularities on hinge demands during an earthquake. The most important observation made through this study is the huge difference, i.e., an increment of 47.86%, in the number of IO–LS hinges in the case of the irregular structure with TH-X load (2,087 vs. 1,088). The reason for this phenomenon is the torsional amplification due to eccentricity in the centre of mass and rigidity. The eccentricity mentioned here is categorized as plan irregularity according to IS 1893 (Part 1): 2016 Tables 5 & 6.

The demand on the beam rotation at the ground floor level for the irregular structure

(0.034102 rad) is more than 136 times higher than that of the regular structure (0.00025 rad). This stark contrast is evidence that structural irregularity, despite maintaining the same IO grade of performance in global terms, may produce local demands beyond acceptable code requirements. Also noteworthy is the slight exceedance of the IS 1893 drift ratio limit (0.004) by Stories 6 through 12 of the irregular structure subjected to TH-X.

5.3.3 Strong Column–Weak Beam Compliance

In all the four cases, irrespective of loading type, the primary form of hinge formation was beam sway without any signs of column dominant hinge action at the mid-storey level. This validates adherence to the strong column-weak beam (SCWB) concept prescribed by IS 13920: 2016 Clause 7.2. Consistent development of beam hinges before column hinges suggests successful prevention of storey mechanism failure, which is the most dangerous case in seismic events.

None of the models had any hinges in the C–D (LS–CP) or failed stages for any of the members when subjected to either pushover analysis or time history analysis. This means that there was complete conformity to the Life Safety and Collapse Prevention objectives of FEMA 356 Table C1-3 .

5.3.4 Code Compliance Assessment

FEMA 356 (Actions That Are Deformation-Controlled): All models meet the acceptance criterion on a global scale as deformation controlled; none of the beams have reached the C-D or failure state. In terms of specific beam failures (CP rotation level), GF beam strengthening is required for the tall and irregular models**.**

IS 13920: 2016 (Strong Column Weak Beam): Verification of beam sway phenomenon in all the models for all load conditions. SCWB condition is satisfied by the structure due to proper capacity design.

IS 1893 (Part 1): 2016 (Storey Drift Limit 0.004): The regular structure is satisfied in both directions. The irregular structure exhibits storey drifts ranging from 0.004 to 0.004 at Stories 6 to 12 with TH-X, slightly surpassing the allowable.

ATC-40 (Performance Point and Ductility): For both models, performance points have been identified satisfactorily. The ratios for ductility are 3.22 for Model A and 6.46 for Model B, which fall within an acceptable limit for the design of ductile RC moment frame.

5.4 Performance Level Summary

Based on all results from pushover and time-history analyses, Table 4.18 summarises the governing performance levels for all four models.

Table 5-18Governing Performance Level Summary — All Models

Model	Analysis Type	TH-X / Pushover	TH-Y	Governing Performance Level
Model A (G+4)	NLSPA	IO	IO	Immediate Occupancy (IO)
Model B (G+14)	NLSPA	IO	IO	Immediate Occupancy (IO)
Model D — Regular (G+14)	NLTHA	IO	IO	Immediate Occupancy (IO)
Model C — Irregular (G+14)	NLTHA	IO	IO	Immediate Occupancy (IO)

** Rotation demand at GF beam exceeds CP limit; global hinge count remains within IO for all other members.*

Both the short and regular structures fall under the Immediate Occupancy category with no exceptions. The tall and irregular buildings meet IO requirements from a global perspective with regards to the number of hinges in their distribution . This does not mean that there is an indication of collapse globally, but rather suggests that certain members need design modifications either by adding confinement steel as per IS 13920: 2016 Clause 7.3 or increasing section sizes.

5.5 Summary of Key Findings

- Both Model A (G+4) and Model B (G+14) satisfy the formation of beam sway systems under pushover loadings and hence prove strong column weak beams based on IS 13920: 2016 Clause 7.2.1.
- The tall structure (G+14) requires almost double ductility ($\mu = 6.46$) compared to the short one ($\mu = 3.22$), and GF rotation demand is also high and exceeds the

FEMA 356 code requirement (D/C = 2.495). Thus, it justifies the need for improved confinement in critical parts of the structure.

- Under NLTHA, the irregular structure shows a huge number of IO-LS hinges by 47.86% as compared to the regular structure for TH-X loading. GF rotation demand for the irregular structure (0.034102 rad) is about 136 times more than the regular structure (0.00025 rad).
- TH-X loading is found to be critical for hinge formation for both regular and irregular structures. TH-Y loading does not form any hinges after the elastic A-B region for both buildings.
- None of the members in the model attains LS-CP and collapse hinge states, demonstrating full adherence to Life Safety and Collapse Prevention performance goals outlined in FEMA 356 and IS 1893 (Part 1): 2016.
- The drifts in the case of the irregular structure slightly surpass the prescribed maximum value of 0.004 at Stories 6 to 12 for TH-X, which can be attributed to the higher plastic hinge demands in the structure.
- Geometric irregularity and building height are the two important parameters that play a major role in determining the plastic hinge intensity in RC structures during an earthquake.

CHAPTER 6

CONCLUSION AND FUTURE SCOPE

6.1 Conclusions

This current research analyzes the seismic performance of reinforced concrete frames using NLSPA and NLTHA. This is done using four different models under two comparative case studies. In Case 1, we have looked into the effect of building height by comparing short frame (G+4) with tall frame (G+14), represented by Models A and B respectively. On the other hand, in Case 2, we have studied the influence of plan irregularity by comparing regular frame (G+14) with geometrical irregular frame (G+14), represented by Models D and C respectively. The analysis was done in ETABS in accordance with the provisions of IS 1893 (Part 1): 2016, IS 13920: 2016, FEMA 356, and ATC-40. Following are the conclusions of the study discussed in chapter 4.

6.1.1 Effect of Building Height on Seismic Performance (Case 1: Model A vs. Model B)

1. The basic natural period for the tall building (Model B: T_1 = 2.021 s) is clearly much larger than that of the short building (Model A: T_1 = 1.137 s). This establishes that an increase in the building height causes lower lateral stiffness resulting in higher seismic displacement demands for the building as per IS 1893 (Part 1): 2016 Clause 7.6.
2. The pushover capacity curves for both models depict ductile beam-sway failure mode in accordance with the SCWB concept of IS 13920: 2016 Clause 7.2.The maximum base shear capacity of Model B (65,524.65 kN) is about 5.4 times higher than that of Model A (12,032.06 kN) due to its higher mass and number of members.
3. The ductility demand ratio of the tall building (Model B: μ = 6.46) is almost twice as that of the short building (Model A: μ = 3.22), establishing the much

higher inelastic deformations required for taller buildings. This result reiterates the requirement of ductile detailing in RC frames of taller buildings as per IS 13920: 2016 Clause 7.3.

4. Maximum storey drift ratios in both models are considerably below the 2.5% threshold for collapse prevention in FEMA 356. Both drift ratio values – 0.029405% (Model B, ground floor) and 0.021761% (Model A, ground floor) – comply with the permissible drift ratio limits; thus, none of the structures is prone to storey mechanism instability.
5. Plastic hinge formation during the performance stage demonstrates that both structures experience a beam-dominant sway mechanism. Even though Model B develops 10,560 plastic hinges in comparison to 1,580 plastic hinges in Model A, the number of hinges in the IO–LS stage of deformation is relatively low (896 and 60 plastic hinges in Models B and A, respectively); besides, there are no plastic hinges developing into the LS–CP and collapse levels in either model.
6. Rotation demand of the beam on the ground floor of the tall structure ($\theta = 0.0499$ rad, D/C = 2.495 at the CP level) does not meet the FEMA 356 requirements, which implies that ground floor beams of the tall structure should be provided with improved confinement reinforcement according to IS 13920: 2016, Clause 7.3.

6.1.2 Effect of Plan Irregularity on Seismic Performance (Case 2: Model C vs. Model D)

1. Model C ($T_1 = 1.673$ s) has a shorter natural period as opposed to Model D ($T_1 = 2.021$ s), since it has greater stiffness based on its arrangement. Nevertheless, model C has less rotation mass contribution (RZ = 0.9293) as compared to model D (RZ = 0.9543), which implies the presence of torsional effect caused by the eccentricity of the centres of mass and stiffness based on IS 1893 (Part 1): 2016 Tables 5 and 6.
2. The maximal displacement for the irregular structure under TH-X ground motion is 102.455 mm for the 10th storey, which is 31.1% more than that of

the regular structure (78.158 mm at Storey 10), due to torsion amplification based on IS 1893 (Part 1): 2016 Clause 7.8.2.

3. The storey drift ratio of the irregular structure (Model C) varies between 0.004394 and 0.005073 for Stories 6 to 12 subjected to TH-X load. It slightly exceeds the permissible value of h/250 = 0.004 as per IS 1893 (Part 1): 2016 Clause 7.11.1. Only for Stories 6 and 7 does the regular structure (Model D) exceed the permissible value (0.004649 and 0.004106 respectively). No values exceed the permissible drift ratios under TH-Y load in either case.
4. For TH-X load, the number of hinges generated in the IO–LS range is 2,087 for the irregular structure while for the regular structure it is 1,088; that is, there is a 91.8% increase in the number of hinges due to torsional amplification caused by plan irregularity. For TH-Y load, no hinges exceed the elastic A–B region in either case.
5. In TH-X direction, the base shear for the irregular building structure is 28.5% higher (5,441.41 kN) than that of the regular structure (4,235.68 kN). Moreover, the overturning moment, MZ, under TH-Y direction loadings is also 87.8% higher in irregular buildings structure (22,528.9 kN·m) than regular building structures (13,752.2 kN·m).
6. The maximum hinge rotation demand of irregular structure in its critical ground-floor beam (θ = 0.034102 rad, D/C = 1.70 at CP) is about 136 times higher than that of the regular structure (θ = 0.00025 rad, D/C = 0.010). This huge difference indicates that geometrically irregular structures, even though they perform well as per Immediate Occupancy level, still have some local member rotation demands higher than that of FEMA 356 collapse prevention limit state.

6.2 General Conclusions

1. The SCWB condition holds true for all four structures as per Clause 7.2.1, IS 13920:2016, proving that the capacity design concept has been adequately adhered to in these structures. The beams-sway mechanism is the prevailing type of failure observed in both types of loadings (push-over & time-history).

2. No plastic hinges are observed in any of the four structures in the LS-CP or collapse (C-D and beyond) regions, which shows that all the four structures have fulfilled the criteria defined by the Life Safety and the Collapse Prevention performance objective of Table C1-3, FEMA 356, and BSO, FEMA 356, §1.5.1.
3. Both building height and irregular geometry are recognized as critical factors that control the demands on plastic hinges. Tall buildings need high ductility requirements, whereas irregular buildings are susceptible to amplification of local demands due to torsional effects, which could result in localized failure despite satisfactory global performance.
4. Ground beams for both models B and C need supplemental confining steel in order to bring down CP level D/C ratio to values less than one so as to meet FEMA 356 Tables 6-7 and 6-8.

6.3 Future Scope

The present investigation, while comprehensive in scope, is subject to certain limitations that open avenues for further research. The following areas are identified for future study:

1. Non-Linear Dynamic Analysis Using Several Ground Motions: In the present NLTHA, only a few number of ground motion recordings were considered. For future research, a number of spectrum compatible ground motions, scaled according to IS 1893 (Part 1): 2016 Clause 6.4.3 or FEMA P-695 method, should be used for statistically reliable demand calculation and considering variation due to record dependency.
2. Influence of Vertical Irregularity: In this current investigation, plan irregularity is taken into account. An exhaustive parametric study with regard to vertical irregularities including soft storey, mass irregularity, and geometric setback in accordance with IS 1893 (Part 1): 2016 Table 6 would contribute to gaining further knowledge regarding irregularity effect on seismic behavior.

3. Soil-Structure Interaction (SSI): For all analysis conducted in this current investigation, only fixed base conditions are considered. Future studies should address soil-structure interaction effects, especially for those sites with soft soil as mentioned in IS 1893 (Part 1): 2016.

4. Comparative Study Using Different Structural System Configurations: The existing study considers RC moment resisting frame structure only. Future scope can include comparative study using two different structures like a combination of shear wall system or steel structural frame, thus comparing the seismic behavior of different structural systems.

5. Incremental Dynamic Analysis (IDA): IDA can be used as per guidelines stated in FEMA 440 in future studies. It can help in developing fragility curves for all models for collapse risk assessment purposes.

6. Retrofitting and Reinforcing Techniques: In view of the deficiencies identified at the member level for ground floor beams in Models B and C, future research may focus on the effectiveness of different retrofitting techniques such as FRP jacketing, steel jacketing, and concrete jacketing to make the structure comply with the FEMA 356 acceptance criterion.

7. Three Dimensional Torsion: This study assumes the occurrence of plan irregularity in an idealized way. More precise 3-D analysis by accounting for accidental torsion using IS 1893 (Part 1): 2016 Clause 7.8.2 and simultaneous consideration of bi-directional ground motion will result in realistic torsional demands in the case of irregular buildings.

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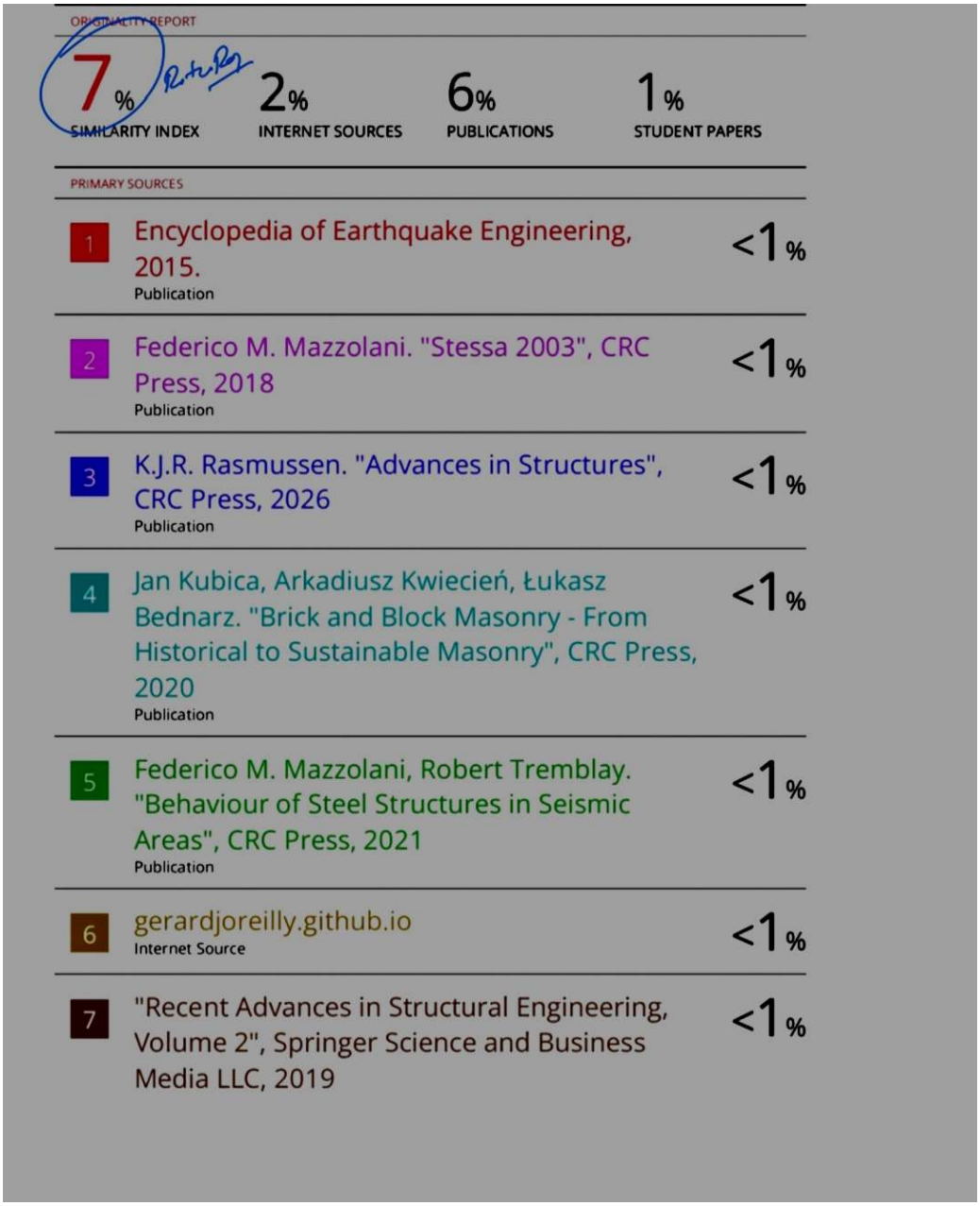

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