A

PROJECT REPORT

ON

# ANALYSIS OF HIGH RISE BUILDING WITH SOFT STOREY HEIGHTS AND APPROACHING METHODOLOGY

AWARD OF

**BACHELORS OF ENGINEERING**

IN

**CIVIL ENGINEERING**

**MUMBAI UNIVERSITY, MUMBAI**

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**(AFILIATED TO MUMBAI UNIVERSITY)**

2020-2021

# CERTIFICATE

*This is to certify that the thesis entitled,* **ANALYSIS OF HIGH RISE BUILDING WITH SOFT STOREY HEIGHTS AND APPROACHING METHODOLOGY** *submitted partial fulfilment of the requirements for the award of* ***Bachelors of Engineering*** *degree in* ***Civil Engineering*** *during 2019-2020 session at the Anjuman I Islam's Kalsekar Technical Campus, New-Panvel is an authentic work carried out by her under my supervision and guidance.*

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# PROJECT APPROVAL SHEET

The report entitled **ANALYSIS OF HIGH RISE BUILDING WITH SOFT STOREY HEIGHTS AND APPROACHING METHODOLOGY** submitted by *Sawaid Anjum (15CE37),Shadab Ansari (15CE11),*Shaikh Faisal (15CE45), Sajid Shamsudeen (15CE35) after presenting his thesis work in form of PowerPoint presentation is hereby approved in partial fulfillment for the award of Degree of Bachelors in Civil Engineering of Mumbai University, Mumbai.

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# ACKNOWLEDGEMENT

It gives us immense pleasure that we would like to express our most sincere heartfelt gratitude to our guide, ***Prof Firoz Nadaf*** & Co-guide ***Prof sayyedazara jahangirdar*** Assistant Professor, Dept. of Civil Engineering, Anjuman-I-Islam's Kalsekar Technical Campus, New-Panvel for her mentoring and research support throughout our studies.

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We would like to thank ***our dearest parents*** for making us believe in my dreams and for constantly supporting us to achieve them.

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We declare that this written submission represents ideas in our own words and where others ideas or words have been included, we have adequately cited and referenced the original sources. We also declare that; we have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea/data/fact/source in our submission. We understand that any violation of the above will be cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

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**Place: New-Panvel.**

A

PROJECT REPORT

ON

# STRUCTURAL RESPONSE OF HIGH RISE BUILDINGS FOR DIFFERENT SOFT STOREY HEIGHTS AND APPROACHING METHODOLOGY

SUBMITTED

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## ABBREVIATIONS

DL = Dead Load

EL = Earthquake Load

Es = Modulus of Elasticity of Steel

$E_{Qx}$ = Earthquake Load in X Direction

$E_{Qy}$ = Earthquake Load in Y Direction

$f_{ck}$ = Characteristic Cube Compressive Strength of Concrete

$f_y$ = Characteristic Strength of Steel

h = Height of Building in meter

I = Importance Factor

LL = Live Load or Imposed Load

OMRF = Ordinary Moment Resisting Frame

$P_u$ = Axial load on Member

R = Response Reduction Factor

SMRF = Special Moment Resisting Frame

Sa/g = Average Response Acceleration Coefficient

$T_a$ = Time Period

$V_B$ = Base Shear

$V_u$ = Shear Force due to Design Loads

W = Total Load

Z = Zone Factor

SCM = Seismic Coefficient Method

RSM = Response Spectrum Method

$F_m$ = Compressive Strength

K = Modulus of Elasticity in Compression Initial Tangent

P = Parking

# ABSTRACT

Earthquakes are natural hazards under which disasters are mainly caused by damage or collapse of buildings and other man-made structures. Due to accommodation of vehicles and their movements at ground levels infill walls are generally avoided, which creates soft storey effect. It should be noted that 70 to 80 % of buildings of urban areas in India fall under the classification of soft storey. This soft storey is also called as Open ground storey or Weak storey. It is a typical feature in the modern multi-storey constructions. Such features are highly undesirable in buildings built in seismically active areas; this has been verified in numerous experiences of strong shaking during the past earthquakes. The majority of buildings that failed during the Bhuj earthquake (2001) and Gujarat earthquake were of the open ground storey type. The collapse mechanism of such type of building is predominantly due to the formation of soft-storey.

As per Indian Standard IS 1893: 2002, the Columns and Beams of the open ground storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads of bare frames. This Multiplication Factor value however does not account for number of storeys, number of bays, type and number of infill walls present, etc, and hence it is independent of all of the above factors. The multiplication factor of 2.5 is not realistic for low rise buildings. This calls for an assessment and review of the code recommended multiplication factor for low rise open ground storey buildings. Therefore, the objective of this study is defined as to check the applicability of the multiplication factor of 2.5.

This study includes analysis of (G+7) RCC Framed building analysed using Seismic Coefficient Method (SCM) as per IS 1893: 2002. In modelling the masonry infill panels, Equivalent diagonal Strut method is used. This study basically includes Four models namely, Frame without masonary infill effect (Bare frame), Masonary Infill frame, Frame with Tie-beam (Tie-beamed frame) and Frame with Bracings (Braced frame) which are analysed for Soil type I (Hard) and Soil type III (Soft) considering time period for seismic analysis as per Program calculated and as per Codal provision. The response of columns in Open ground storey are discussed and conclusions are made in this study analysed on ETABS software.

# Chapter One
# INTRODUCTION

# CHAPTER ONE
# INTRODUCTION

## 1.1 GENERAL INTRODUCTION

Many buildings in India are arising with open ground storey for architectural and functional purposes. Parking is one of the most important purposes to build open ground storey as shown in figure1.1. This may be due to land limitations. Parking is not only the purpose to build soft storey but also restaurants, hotels, retail shopping, and multipurpose halls. To fulfill these requirements, the term is introduced and it is called "Soft storey/Open ground storey". Soft storey is the one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above. Weak storey is the one in which the storey lateral strength is less than 80 percent of that in the storey above. Weak storey is related to lateral strength.

**Figure 1.1: Open Ground Storey**

## 1.2 SOFT STOREY EFFECT

It is reported that many buildings with vertical stiffness irregularities such as buildings with first storey, not infilled with masonry walls, as done in the upper storeys, suffered extensive damage during Bhuj (India) earthquake (26 January 2001). The first storey is made wall-free to accommodate the parking in the building owing to high cost of land as shown in Figure 1.2. Such considerable decrease in lateral stiffness of masonry infill compared to the lateral stiffness of adjoining storey, leads to "Soft storey effect". The consequence of the presence of a soft storey may lead to a dangerous sway mechanism due to formation of plastic hinges at the top and bottom end of columns. These columns are subjected to large lateral forces, hence relatively large cyclic deformations and apparently severe stresses are induced in it.

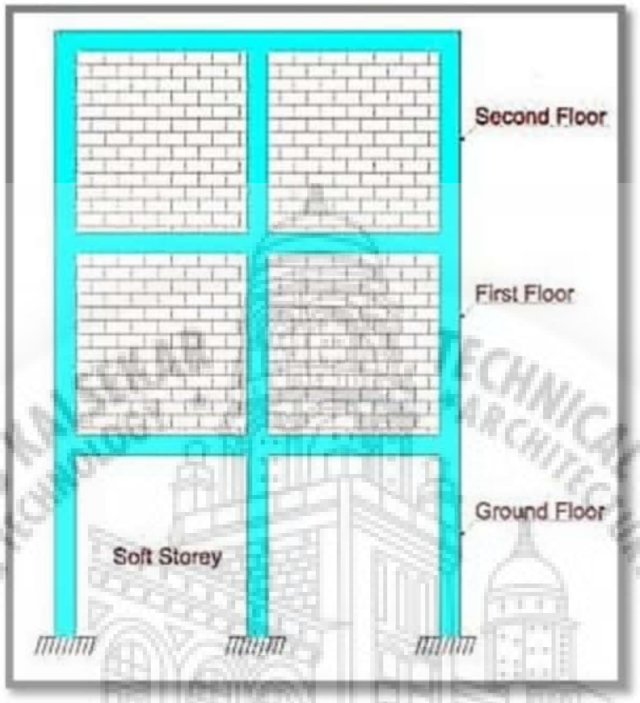



**Figure 1.2: Soft Storey with Infill Effect**

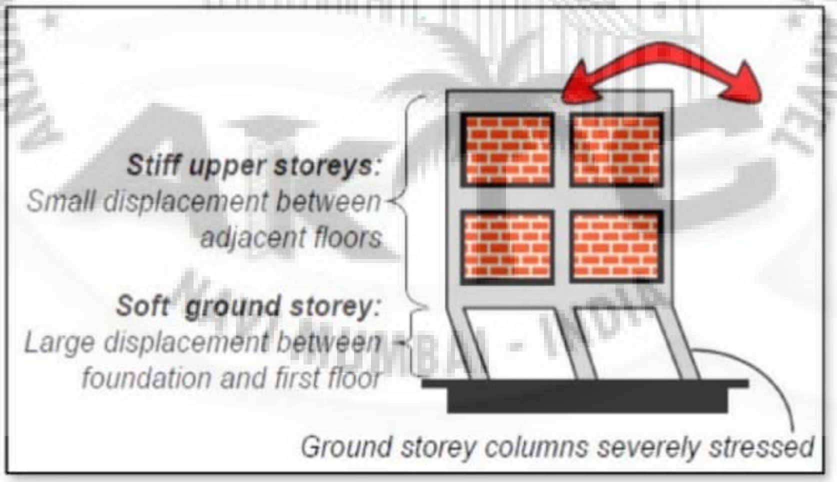



**Figure 1.3: Large Displacement in Ground Storey and Foundation**

As shown in Figure 1.3, greater displacement occurs in between ground storey column and foundation and small displacement occur in upper storey. Hence, ground storey columns are severely stressed. To overcome these soft storey effect IS1893 (Part-I):2002 clause no.7.10

recommended various analytical approaches and solutions for it. So by using these clauses soft storey behavior can be improved and it will show better performance under lateral loading.

## 1.3 AIMS OF PRESENT WORK

1. To know the structural response of soft storey under lateral loading and to overcome the setbacks of soft storey by introducing Masonary infill walls, Tie-beam and Bracings in Soft-Storey.
2. To know which approach of soft storey analysis and design as specified in IS1893 (Part-I):2002 is most convenient optimum and easily applicable.

## 1.4 THESIS OUTLINE

Chapter one of General introduction focuses on the background of this dissertation. It shows that detailed investigation and study has to be done for soft storey behavior and soft storey design.

Chapter two of Literature review deals with the summary of the technical papers published till date and the data regarding the dissertation in the same. It also focuses on the extensive research significances carried out up till now regarding the dissertation as well as the scope for further studies.

Chapter three of Theoretical formulation with different loads and their combinations has been defined along with some different seismic parameters using software ETABS 9.7 for Structural analysis of Frame without effect of masonary infill (Bare frame), Masonary Infill frame, Frame with Tie-beam and Frame with Bracings including strength and stiffness effect.

Chapter four of Parametric investigation shows the variation of seismic parameters, models with and without infill effect, Frame with Tie-beam and Frame with Bracings that are studied for soft storey behavior. It includes detailed tables and Figures showing the variation of ratios of axial forces and bending moment in X and Y direction that is acting in-plane or out-of-plane for different soft storey heights. The detailed observations and findings based on the results obtained during the analysis of frames is carried out and various solutions are obtained for soft storey effect.

Chapter five of Conclusion and scope for future work deals with summarized observations. It also deals with one of the approach of soft storey analysis which is more beneficial.

# Chapter Two
# LITERATURE REVIEW

# CHAPTER TWO
# LITERATURE REVIEW

## 2.1 PRELIMINERY REMARKS

The various international papers have been searched and few of them are represented here. In these papers, soft storey behavior is studied by Seismic coefficient method, Response spectrum method, Push over analysis. Detailed descriptions of papers have been studied in section 2.2. In section 2.3, the concluding remarks on papers and the present study is illustrated.

## 2.2 LITERATURE REVIEW

Robin et.al studied the comparison in the pushover analysis for the two buildings. Behavior of infilled frames explained in it. Infill walls tend to interact with the frame when the structure is subjected to lateral loads, and also exhibit energy-dissipation characteristics under seismic loading. Masonry walls contribute to the stiffness of the infill under the action of lateral load. The term "Infilled frame" is used to denote a composite structure formed by the combination of a moment resisting plane frame and infill walls. The infill may be integral or non-integral depending on the connectivity of the infill to the frame. In case of buildings under consideration, integral connection is assumed. The composite behavior of an infilled frame imparts lateral stiffness and strength to the building. The typical building of an infilled frame subjected to lateral load is illustrated in Figures 2.1(a), 2.1(b), 2.1(c).

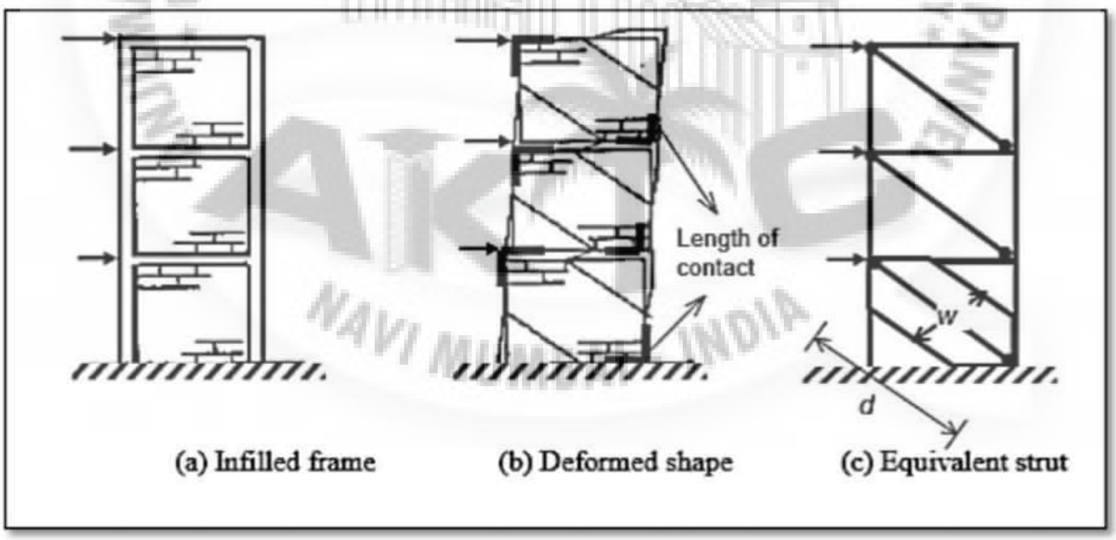



**Figure 2.1: Behavior of Infilled Frames**

The two buildings was modeled and analyzed for static, response spectrum and pushover analyses, using the finite element package SAP2000. Pushover analysis (non-linear static method) was also carried out. The pushover analysis provides an insight into the structural aspects, which control the performance during earthquakes. The analytical models of the buildings include all components that influence the mass, strength and stiffness. This paper concluded that the lateral load resisting mechanism of the masonry infilled frame essentially different from the bare frame. The bare frame acts primarily as a moment resisting frame with the formation of plastic hinges at the joints under lateral loads. In contrast, the infill frame behaves like a braced frame resisted by a truss mechanism formed by the compression in the masonry infill panel and tension in the column. The plastic hinges are confined with the joint in contact with the infill panel. It is seen that the existing buildings with open ground storey are deficient and in need of retrofit.

Sujatha et.al studied the effect of masonry infill in upper floors, on the beams and columns in open ground storey by analyzing a four storied building using SAP 2000. The finite element modeling performed using SAP 2000. Three conditions considered for the study (i) bare frame model (ii) frame with infill walls on all floors except ground floor (iii) frame with infill walls on all floors except ground floor and with some openings in the upper floor. The building configuration used in this study was simple and regular as four storey plane frame was considered. This paper concluded that the ratio of the shear force and bending moments of the base for the case where the strength and stiffness of masonry infill is considered to that of the bare frame model ranges from 2.18 to 2.69. As per the design criteria given by IS 1893:2002, the columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads for bare frames.

Sachin Patel and Sumant Patel studied A four storyed building without infill panel and with infill panel using STAAD software. This paper concluded that by considering infill wall panel the stiffness of building increase by 30.388% in x-direction and in z-direction 20.81%. From their comparison, for seismic analysis of building has to take effect of infill stiffness and strength.

Khandave highlighted the importance of explicitly recognizing the presence of the open storey in the analysis of the building. Various analytical models on STAAD were analysed and checked results for soft storey at various levels. He concluded that soft storey shows poor performance during earthquake excitation by comparing drift, base shear, drift.

Tamboli and Karadi had done comparison of bare frame, infill frame, and open first storey frame. In this paper seismic analysis of RC frame models has been studied that includes bare frame, infilled frame, and open first storey frame and concluded that the seismic analysis of RC frames should be done by considering the infill walls in the analysis, For modeling the infill wall the equivalent diagonal strut method can be effectively used, the storey drift of first storey of open first storey frame is very large than the upper storeys, this may probably cause the collapse of structure, The presence of infill wall can affect the seismic behavior of frame structure to large extent, and the infill wall increases the strength and stiffness of the structure.

Misam and Mangulkar have studied the effect of adding shear wall to the building in different arrangement in order to reduce soft storey effect on structural seismic response. A (G+14) storied regular building consisting of one bare RC frame and three with different arrangement of shear walls along RC frame was considered. It was found that location and numbering of shear wall plays an important factor for the soft storey structures to displace during earthquake. Considerable reduction in shear force, bending moment etc. is observed in dual type structural system as compared with frame system.

Kasnale and Jamkar had done investigation on the behavior of RC frames with various arrangement of infill when subjected to dynamic earthquake loading. The result of bare frame, frame with infill, soft ground flour and soft basement are compared and observed that, providing infill below plinth improves earthquake resistant behavior of the structure when compared to soft basement. They had done comparison for time period for various models such as bare frame, frame with infill, soft ground flour and soft basement by both Seismic coefficient method and Response spectrum method. They concluded that provision of infill wall enhances the performance in terms of displacement control, storey drift and lateral stiffness. This paper concluded that IS code method describing very insufficient guidelines about infill wall design procedures.

Saurabh Singh, Saleem Akhtar and Geeta Batham, the multiplication factor of 2.5 is not realistic for low rise buildings. This calls for an assessment and review of the code recommended multiplication factor for low rise open ground storey buildings. The objective of their study is defined as to check the applicability of the multiplication factor of 2.5 and to study the effect of infill strength and stiffness in the seismic analysis of open first storey building.

Krushna B. Avhad objective of present study is the study of comparative performance of OGS buildings designed according to various MFs using nonlinear analysis. As the more realistic performance of the OGS building requires the modelling the stiffness and strength of the infill walls, the stiffness and strength of the infill walls also considered. The two extreme cases of infill walls, strong and weak are considered in the study.

Miss Jewalikar Gauri Anantrao studied a comparative study is carried out considering different analytical models for soft storey behavior, and also the detailed study of provisions of soft storey as specified in IS1893(Part-I):2002 is carried out. Unreinforced masonry infill is modeled by using Equivalent Diagonal Strut method approach.

## 2.3 CONCLUDING REMARKS

With reference to above literature it has been observed that major work has been done for behavior of soft storey. Study of soft storey for different frames having stiffness irregularity by both Seismic coefficient method is not observed here. To fill the gap between past works, the objectives of present study are as follows

1) Analysis is carried out by grouping the columns to study which bottom storey columns suffers more during earthquake.
2) The frames are analysed by introducing Masonary infill, Tie-beam and Bracings at periphery in Soft-Storey.
3) Analysis is carried for both time period considered for seismic analysis as per Program Calculated and as per Codal Provisions.
4) Analysis is carried for different soil conditions that is Hard and Soft soil.
5) Analysis is carried out for different Soft-Storey (bottom storey) heights varying from 4m to 5m height (with an interval of 0.2m each).

# Chapter Three
# THEORETICAL FORMULATION

# CHAPTER THREE
# THEORETICAL FORMULATION

## 3.1 INTRODUCTION

An open ground storey configuration is arising in buildings due to many reasons such as parking, restaurants, etc.as shown in figure 3.1 Open ground storeys as being built in the countries have two main characteristics. Firstly, the lateral stiffness of the ground storey is much smaller than those of storey above. This leads to excessive deformation in ground storey. The above genesis of the name is called, **"Soft storey building."** Soft storey failure is considered as one of the most drastic failure for instance Bhuj earthquake (2001), The San Francisco Earthquake (1906), The Bingol and Turkey Earthquake of 1 of May 2003.


**Figure 3.1: Open Ground Storey**

The lateral strength of the ground storey is much smaller than those of storey above. This sudden drop in strength in the ground storey is irrational. Based on strength considerations, ground storeys of such buildings are also called "**weak storeys**".

**Soft storey** is one in which the lateral stiffness is less than 70 percent of that in the storey above or less than 80 percent of the average lateral stiffness of the three storeys above. It can be at any level.

**Weak storey** is one in which the storey lateral strength is less than 80 percent of that in the storey above, the storey lateral strength is the total strength of all seismic force resisting elements sharing the storey shear in the considered direction. Weak storey is when, $F_i<0.8F_{i+1}$.as shown in figure 3.2

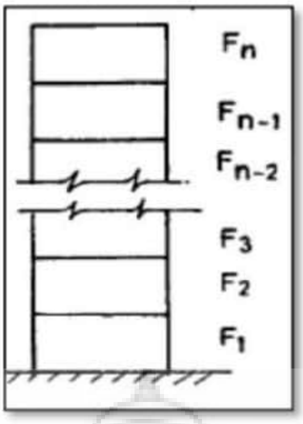



**Figure 3.2: Weak Storey**

The study aims at finding that which approach of soft storey is optimum. In proposed work, Static analysis of Multistoried Building (P+7) has been analyzed and designed including strength and stiffness effects of Infill by using ETABS 9.7 software and by referring IS1893 (Part-I):2002, IS13920.

Bare frame analysis of Multistoried Buildings has been analyzed and then columns of the soft storey are designed for 2.5 times the storey shears and moments calculated under seismic loads by using ETABS 9.7 software and referring IS1893(Part I):2002.

Analysis of Space frame with (Special moment resisting frame) having Masonary Infill, Ti-beam and Bracings along the periphery in bottom has been studied here.

## 3.2 VARIOUS TYPES OF IRREGULARITIES AND THEIR DESCRIPTION

Open ground storey is an example of poor vertical configuration. The failure of soft storey comes under two categories first one is stiffness irregularity and second mass irregularity.

### 3.2.1 Stiffness Irregularity

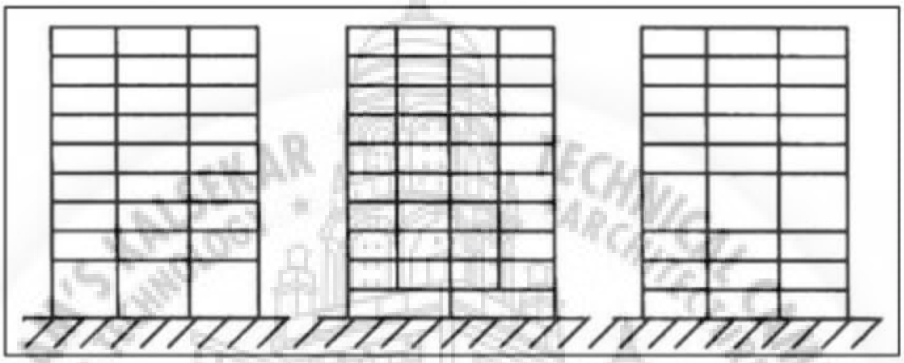
a) Soft storey, in which the lateral stiffness is less than 70% the lateral stiffness of the storey above or less than 80% of the average lateral stiffness of three storey above.

b) Extreme soft storey, in which the lateral stiffness is less than 60% the lateral stiffness of the storey above or less than 70% of the average lateral stiffness of the three storeys above.

Stiffness irregularities in the vertical direction are due to following,

a) Increasing the storey height at the first storey or any intermediate storey which creates the difference in lateral stiffness between adjacent storeys as shown in Figure 3.3 a, 3.3 b.

b) Provision of floating columns as shown in Figure 3.3(c) which leads to the difference in lateral stiffness between adjacent storeys and also to cause interference in the stress path.



(a) (b) (c)

**Figure 3.3: Various Types of Stiffness Irregularities.**

### 3.2.2 Mass Irregularity

Mass irregularity shall be considered to exist where the seismic weight of any storey is more than 200 percent of that of its adjacent storeys. The irregularity need not be considered in case of roofs. Mass irregularities in vertical direction are due to the use of a particular storey in a residential building for stacking goods or heavy equipment's or parking of vehicle. Mass concentration at various levels is as shown in Figure 3.4.

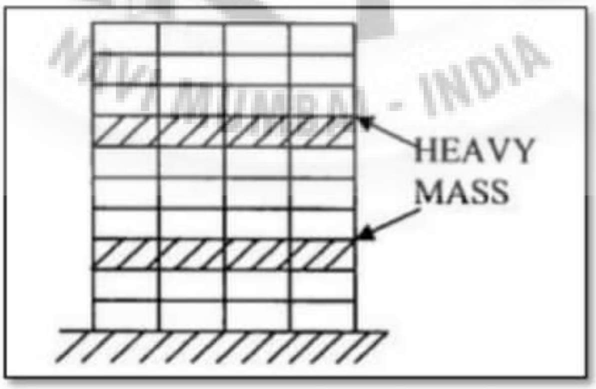



**Figure 3.4: Frame Showing Mass irregularity**

### 3.2.3 Vertical Geometric Irregularity

It shall be considered to exist where the horizontal dimension of the lateral force- resisting system in any storey more than 150% of that in its adjacent storey as shown in Figure 3.5.

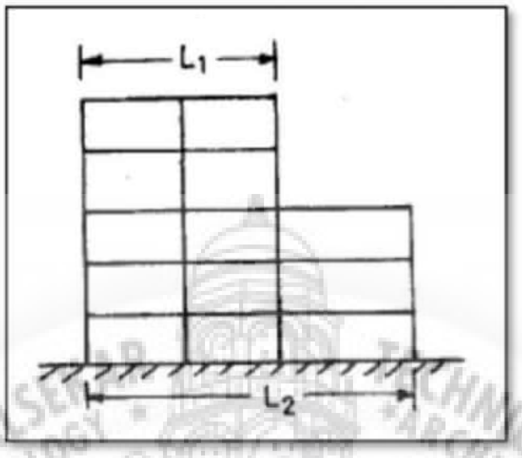



**Figure 3.5: Frame Showing Vertical Geometric Irregularity**

### 3.2.4 Discontinuity in Capacity (Weak Storey)

A weak storey is the one in which the storey lateral strength is less than 80% of that in storey above. The storey lateral strength is the total strength of all seismic force-resisting elements sharing the storey shear in considered direction.

## 3.3 MOST DRASTIC FAILURE OF OPEN GROUND STOREY

Some famous earthquakes in which destruction was due to Soft Storey problem as shown in below figures from 3.6 – 3.10

1. The San Francisco Earthquake, 1906
2. The Bingol, Turkey Earthquake of the 1 of May 2003
3. 26th Jan 2001, Bhuj earthquake, India
4. 17thJan 1994, Northridge earthquake

These are some pictures showing failures of soft storey.

Figure 3.6: 1971 San Fernando Earthquake

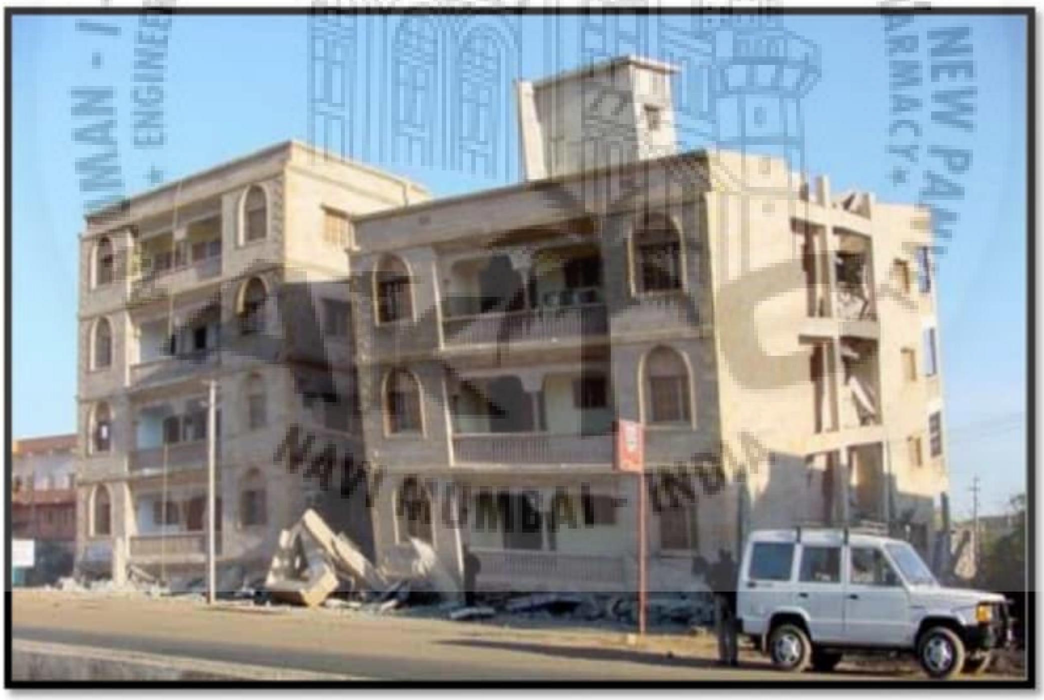


Figure 3.7: 2001 Bhuj Earthquake

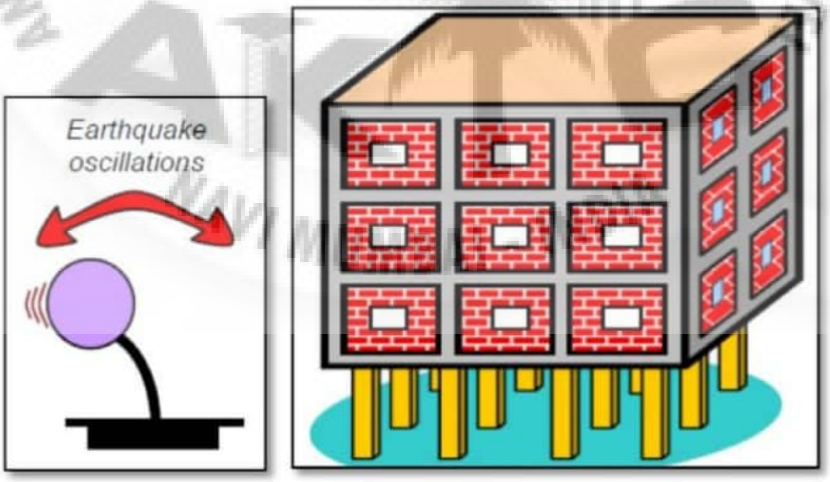
Figure: 3.8 Total collapse of Soft Storeyed at Open Garage at the Ground Floor of the apartment Building in Northridge Earthquake (cars parked at the garage level seen to be crushed)

Figure 3.9: Soft Storey (Open Plinth), Vertical Split between two blocks (Bhuj)

Figure 3.10: Collapse of Soft Middle Storey in a Building at Bhuj

## 3.4 STRUCTURAL IDEALIZATION OF AN OPEN GROUND STOREY AS AN INVERTED PENDULUM

Building with Open ground storey is like inverted pendulum [Figure 3.11(a)], with large mass concentration at higher elevations and weak supports at lower elevations. It is clear how weak the columns are compared to the structure above as shown in Figure 3.11 (b).




(a) (b)

Figure 3.11: Structural Idealization of Open Ground Storey as an Inverted Pendulum

### 3.5 PLASTIC HINGE FORMATION IN OPEN GROUND STOREY COLUMNS

Because of stiffness irregularity, plastic hinge forms in open ground storey column and it leads to excessive deformation. Afterwards it actually cuts from its base as shown in Figure 3.12. The stiffness discontinuity in the soft storey leads to large stress concentration at the joints accompanied by large deformations. The plastic hinges are formed at these locations leading to mechanism and subsequent collapse of structure. The consequence of presence of soft storey either in the ground storey or in the upper storey may lead to dangerous sway mechanism in the soft storey due to formation of plastic hinges at the top and bottom end of columns, as these columns are subjected to cyclic deformations.

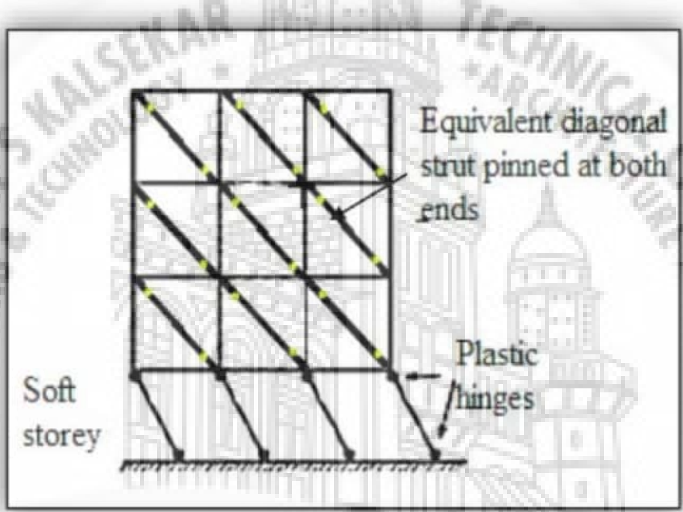



**Figure 3.12: Plastic Hinge Formations in Columns after Earthquake**

### 3.6 BEHAVIOR OF RCC FRAMED STRUCTURES WITH MASONRY INFILLS

#### 3.6.1 Introduction to Infill Behavior

A large number of Reinforced concrete buildings are constructed with masonry infills. Masonry infills are often used to fill the void between the vertical and horizontal resisting elements of the buildings frames with the assumption that these infills will not take part in resisting lateral load. Hence, its significance in the analysis of frame is generally neglected. It has been recognized that frames with infills have more strength and rigidity of the structure in comparison to the bared frames and their ignorance has become the cause of failure of many of the multi-storied buildings.

### 3.6.2 Failure Mechanism of Infilled Frame

The two most common modes of masonry failure may be called **out of plane failure** and **in plane failure.**

**3.6.2.1 Out of Plane Failure:** The structural walls perpendicular to seismic motion are subjected to out of plane bending results in out of plane featuring vertical cracks at the corners and in the middle of the walls. Out of plane loading is as shown in Figure 3.13.

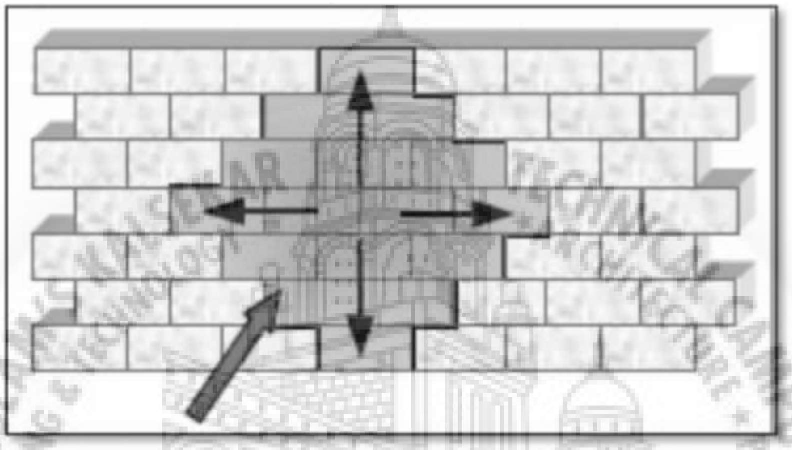


**Figure 3.13: Out of Plane Loading**

Due to inadequate anchorage of the wall into the roof diaphragm and limited tensile strength and mortar unitedly causes out of plane failure of wall in **un-reinforced** masonry building, which are most vulnerable. The resulting flexural stress apparently exceeds the tensile strength of masonry leading to rupture followed by collapse. Moreover long span diaphragms causes excessive horizontal flexure. Out of plane wall movement has been characterized as shown in Figure 3.14.

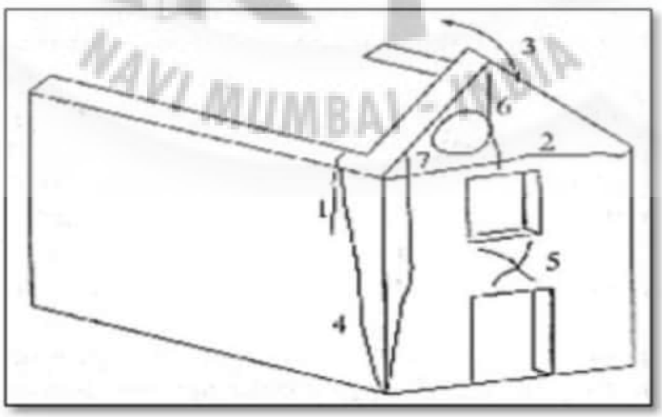



**Figure 3.14: Out of Plane Failure**

1. Vertical cracks in the corner and/ or T walls
2. Horizontal cracks along the façade
3. Partial collapse of an exterior wall
4. Wythe separation
5. Cracks at lintel and top of slender piers
6. Cracks at the level of roof
7. Masonry ejection

### 3.6.2.2 In Plane Failure

The structural walls parallel to seismic motion are subjected to in plane forces or bending and shear causes horizontal and diagonal cracks in the wall respectively. In plane loading is shown in Figure 3.15.

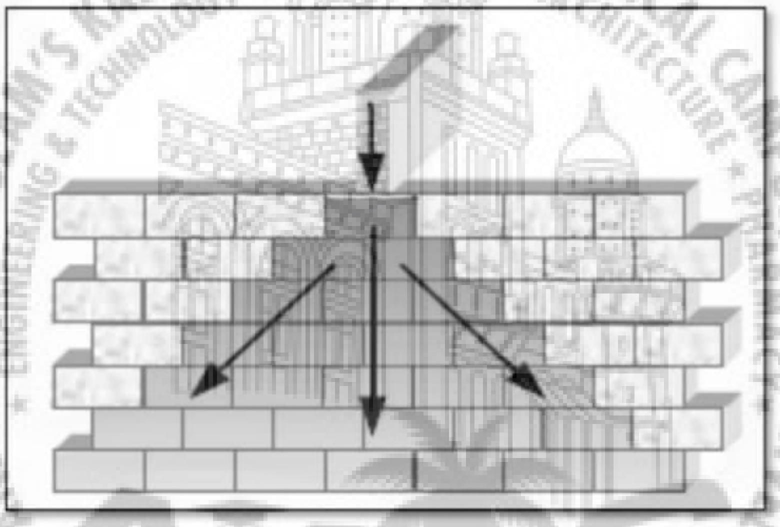


**Figure 3.15: In Plane Loading**

The other types of masonry failure are diaphragm failure, pounding, connection failure and failure of nonstructural components.

In plane failure of walls in un-reinforced masonry structures due to excessive bending or shear are most common as in evident from double diagonal (X) a hear cracking. This cracking pattern frequently found in cyclic loading indicates that the planes of principle tensile stress in the walls frequently found in cyclic loading indicates that the planes of principle tensile stress in the walls remain in capable of withstanding repeated load reversals leading to total collapse. "X" cracks occurs mainly in short piers, rocking (top and bottom) in slender piers. These cracks happen to be worse at lower storey. In plane failures are characterized as in Figure 3.16.

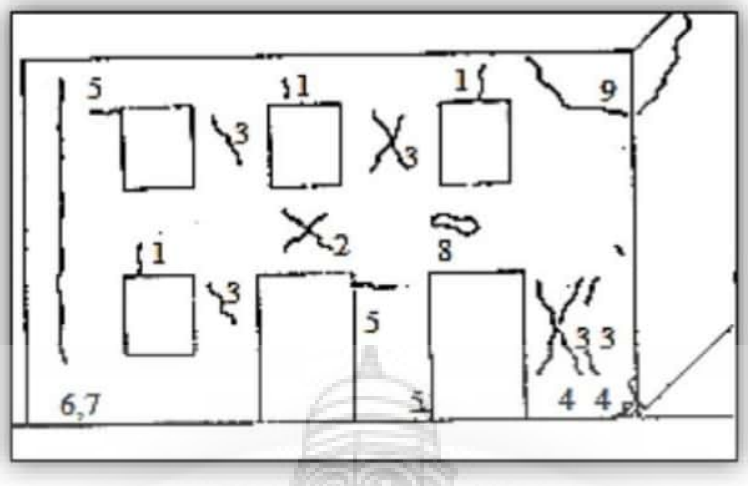



**Figure 3.16: In Plane Failure**

1. Vertical cracks on openings
2. Diagonal shear cracks on parapets and in doors and window lintels
3. Diagonal shear cracks in the masonry piers between openings
4. Crushing of corners of walls due to excess of compressive stress
5. Horizontal flexure cracks on top and or base of masonry piers
6. Vertical cracks at wall intersections
7. Passing through vertical cracks at wall intersections
8. Spalling of material at the location of floor beam due to pounding
9. Separation and expulsion of the intersection zone of two corners

### 3.6.3 Behavior of Bare Frame and Infill Frame

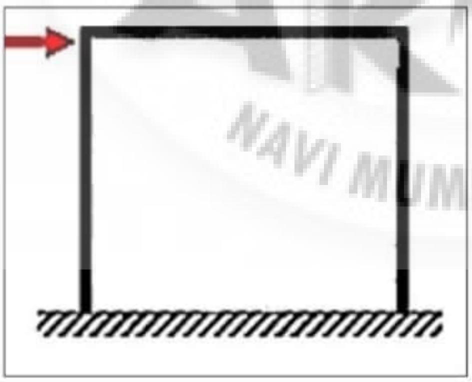


**Figure 3.17: RCC Frame without Brick Infill**

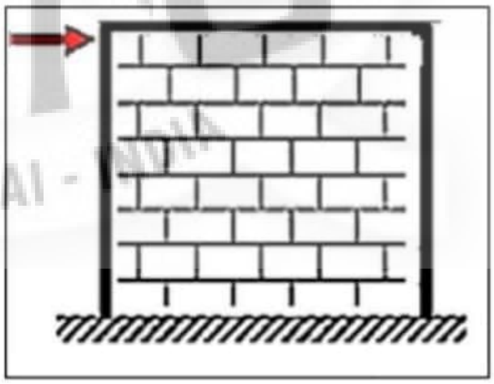


**Figure 3.18: RCC Frame with Brick Infill**

The in-plane stiffness of masonry infill wall is not taken into account in bare frame. Bare frame will deflect under horizontal loads by bending in columns and beams Non-structural components such as masonry that are subjected to seismic forces are not normally within the design scope of the structural engineer, whose responsibility is to provide the seismic safety of the building. In addition, non-structural components such as partition walls are often added after the initial building design, and the original architect is often not involved as shown in figure 3.17 and 3.18. Finally, non-structural components remain uninvolved in the building design and become the source of damage.

Infill walls are probably the most important non-structural element in the context of seismic design. Due to their significant in-plane stiffness and strength, infill walls modify the anticipated seismic performance of a building. However, if infilled, the composite structure has a completely different structural behavior which is dominated by masonry infill. Rather than the lateral loads being resisted by bending in columns and beam a diagonal compression strut forms within the plane of the infill, effectively acting as a compression bracing member. Simultaneously, a diagonal tension crack opens up between the other two corners of the frame. This crack occurs because of the tensile elongation along the diagonal and the low tensile strength of masonry. Under reversed cycles of the earthquake load, the familiar pattern of "**X**" cracking occurs.

### 3.6.4 The Infill Effect in Soft Storey Analysis and Design

Open ground storey buildings are inherently poor systems with sudden drop in stiffness and strength in the ground storey. In the current practice, stiff masonry walls (Figure 3.19a) are neglected and only bare frames are considered in design calculations (Figure 3.19b). Thus, the inverted pendulum effect is not captured in analysis and design.

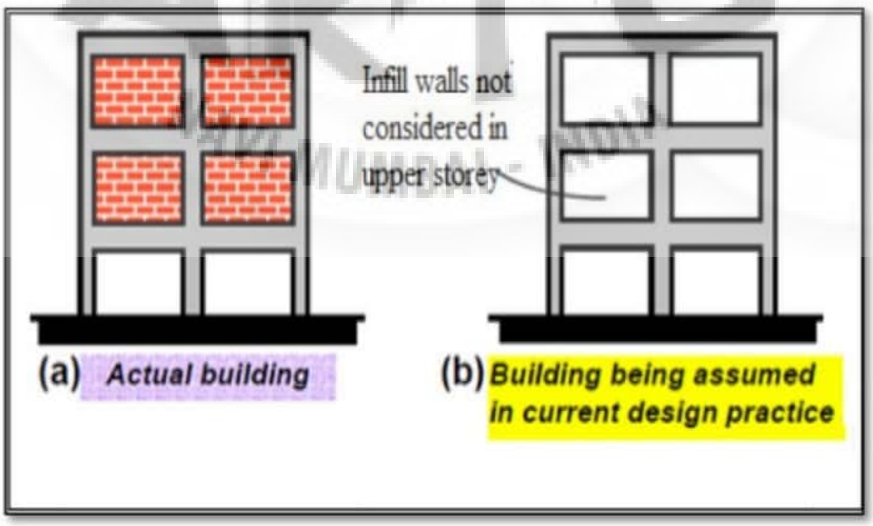



**Figure 3.19: Difference between Common Practice and Actual Building**

Infill effect

The masonry infill in the frame has the following significant effects under the lateral loadings

The infill forms a strut along one diagonal and tie along the other diagonal. The strut resists the applied seismic forces through axial compression along the strut and dissipates the seismic energy as it cracks in the tie because of lack of tensile resistance of the masonry infill. The masonry infill alters the plane stiffness distribution in plan and elevation due to provision of an irregular arrangement of infill panels leading to a soft storey and/or a magnified torsional effect. The contribution of masonry infill increases the stiffness of the frame and decreases the natural period of the structure, resulting in the increased seismic forces than the bare frame (stiffness contribution of infill neglected). The increased stiffness of the frame due to masonry infill may also alter the relative positions of the Centre of mass hence the behavior of structure under torsion.

It is recommended to isolate masonry infill from the RC frames so that they can be treated as non-structural components. The isolation helps to prevent the problems associated with the shortening of natural period.

### 3.6.5 FEMA 356 Pre-standard Approach

Federal Emergency Management Agency (FEMA 356— Prestandard and Commentary for the Seismic Rehabilitation of Buildings), This FEMA approach recommends that Stiffness of cracked unreinforced masonry infill panels shall be represented with equivalent struts. The equivalent compression strut analogy shall be used to represent the elastic stiffness of a perforated unreinforced masonry infill panel.

Stiffness: In-plane lateral stiffness of an infilled frame system is not the same as the sum of the frame and infill stiffnesses because of the interaction of the infill with the surrounding frame. Experiments have shown that under lateral forces, the frame tends to separate from the infill near windward lower and leeward upper corners of the infill panels, causing compressive contact stresses to develop between the frame and the infill at the other diagonally opposite corners. Recognizing this behavior, the stiffness contribution of the infill is represented with an **equivalent compression strut** connecting windward upper and leeward lower corners of the infilled frame. In such an analytical model, if the thickness and modulus of elasticity of the strut are assumed to be the same as those of the infill, the problem is reduced to determining the effective width of the compression strut. Solidly infilled frames may be modeled with a single compression strut in this fashion. The equivalent diagonal strut shall have the same thickness and modulus of elasticity as the infill panel it represents.

### 3.6.6 History of Equivalent Diagonal Strut

Several researchers have studied the interaction between frames and masonry and have reported significant findings regarding the panel's greater mechanical strength, lower displacement and higher ductility. Thus, the formulations adopted by various researchers have also varied greatly. Consequently, there is a wide variety of analytical techniques to evaluate the stiffness and the strength of infill frames. Polyakov (1956) introduced the concept of equivalent diagonal struts and Holmes (1961) improved it. Subsequently, the method of calculation was refined by Stafford-Smith (1962, 1966, 1967a, 1967b), Stafford-Smith & Carter (1969), Mainstone (1971), and Liauw & Lee (1977). In this method, the frame with masonry infill walls is modeled with a compressed diagonal strut replacing the masonry panel. Holmes (1961) proposed that the thickness and modulus of deformation of the diagonal strut should be considered equal to that of the masonry panel and the contact length (D in Figure 3.20) equal to one third of its length.

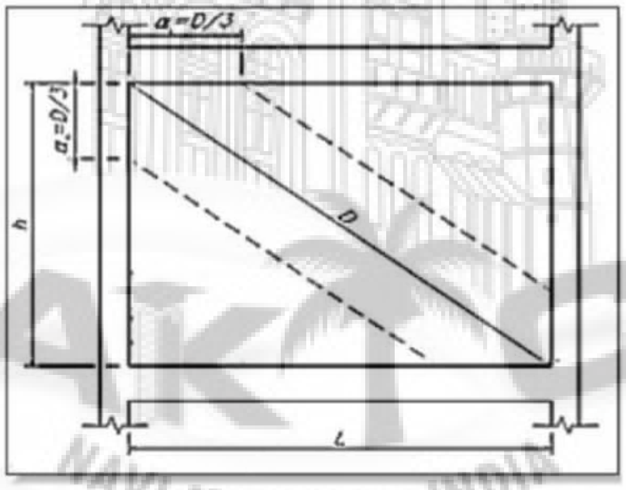



**Figure 3.20: Equivalent Diagonal Strut According to Holmes (1961)**

Smith (1962) argued that the width of the diagonal strut depends on several factors, as illustrated in Figure 2. Based on the analytical and experimental studies of Kadir (1974), Hendry (1981) proposed a semi-empirical relationship for the width of the panel, which corresponds to half the width proposed by Smith (1962). Hendry's model was adopted by the Canadian standard (CSA S304.1-04). Demir and sivri's (2002) proposed modified formula of Mainstone (1971).

### 3.6.7 Methods of Analysis for Infill Effect

Most of the work is carried out in the field of solid unreinforced infilled frames with or without opening. Available analysis and design methods for the strength and stiffness of infilled frames can be classified as -

1. Method based on the concept of elementary strength of materials treating the wall to act composite with the frame,
2. **Method based on concept of equivalent diagonal strut,**
3. Method based on equivalent frame method,
4. Method based on classical theory,
5. Method based on the finite element analysis,
6. Method based on experimental investigation and
7. Method based on plasticity and collapse design approach.
8. Method based on concept of equivalent diagonal strut

As per FEMA 356, the equivalent diagonal strut is the compression only axial member with pinned ends and it is very effective. This model is computationally attractive but is theoretically weak as identifying the equivalent non-linier stiffness of the infill masonry using diagonal strut.

### 3.6.8 Method Based on Concept of Equivalent Diagonal Strut

The equivalent width of diagonal strut as indicated in Figure 3.21 is computed as

$$W_{ef} = 0.175(\lambda_h H)^{-0.4}\sqrt{H^2+L^2} \quad \text{.......... (3.1)}$$

$$\lambda h = h_i 4\sqrt{\frac{E_i t \sin 2\theta}{4E_c I_c H_i}} \quad \text{.......... (3.2)}$$

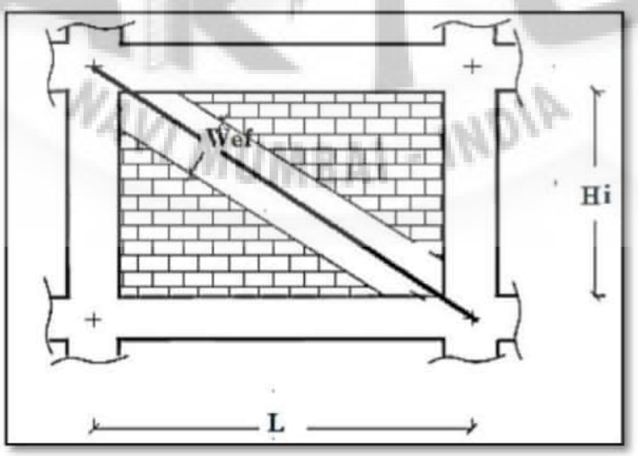



**Figure 3.21: Equivalent Diagonal Strut Method by Mainstones**

Where,

$W_{ef}$ = width of diagonal strut,

H, L =height and length of the frame,

Ec, Ei = elastic moduli of the column and of the infill panel,

T = thickness of the infill panel,

$\theta$ = angle defining diagonal strut,

$I_c$ = modulus of inertia of the column,

$H_i$ = height of the infill panel.

### 3.6.9 Evaluation of Modulus of Elasticity for Brick Masonry ($E_i$)

Modulus of Elasticity in compression initial tangent MPa and compressive strength of Table mounted brick for Maharashtra state, Pune region is 550 and 5.2 subsequently as shown in figure 3.22.

$$E_i = K\, f_m \quad \text{......................................(3.3)}$$

$$Ei = 550\sqrt{5.2}$$

$$\boxed{Ei = 1255\ \text{MPa}}$$

Where,

K = Modulus of elasticity in compression initial tangent

$f_m$ = Compressive strength

#### 3.6.9.1 Brick Masonry Properties in ETABS Software

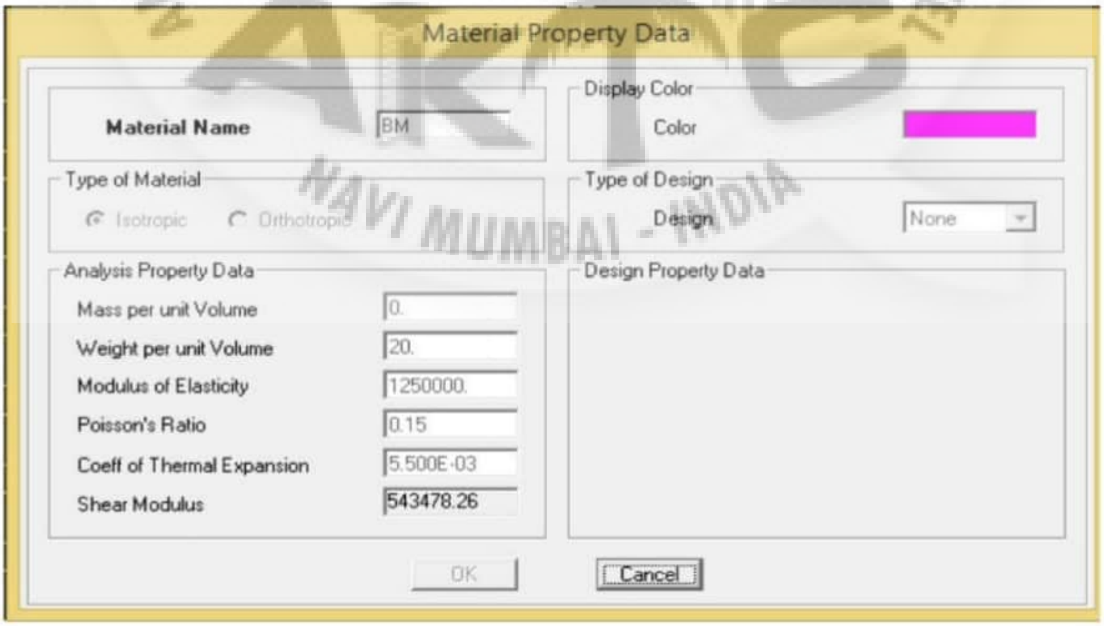



**Figure 3.22: Brick Masonry Properties**

### 3.7 MOMENT RESISTING FRAME

Moment resisting frame is a space frame designed to carry all vertical and horizontal loads, by developing bending moments in the members and at the joints.

Moment resisting frames have rigidly joined beams and columns. Loads are resisted mainly by bending and shear in beams and columns. Due to the overturning moment acting on the frames, the columns experience compression and tension forces in addition to their gravity loads. For frames to perform well during strong ground shaking columns must be stronger than beams. Elevation of moment resisting frame is as shown in Figure 3.23.Three main goals of moment resisting frames are:

1. To achieve strong column-weak beam design.
2. To avoid shear failure.
3. To provide details that enable ductile flexural response in yielding regions.

Moment resisting frames may be Ordinary Moment Resisting Frames (OMRF) or Special Moment Resisting Frames (SMRF). Ordinary RC moment resisting frames are those designed and detailed as per IS 456: 2002 only, these do not possess adequate ductility to resist strong earthquake shaking. Special RC moment resisting frames are required in the higher seismic zones and are provided ductile behavior and comply with the requirements given in IS 4326: 1993 or IS 13920: 1993 or SP 6. Values of response reduction factor are 3 and 5 for ordinary moment resisting frame and special moment resisting frame respectively.

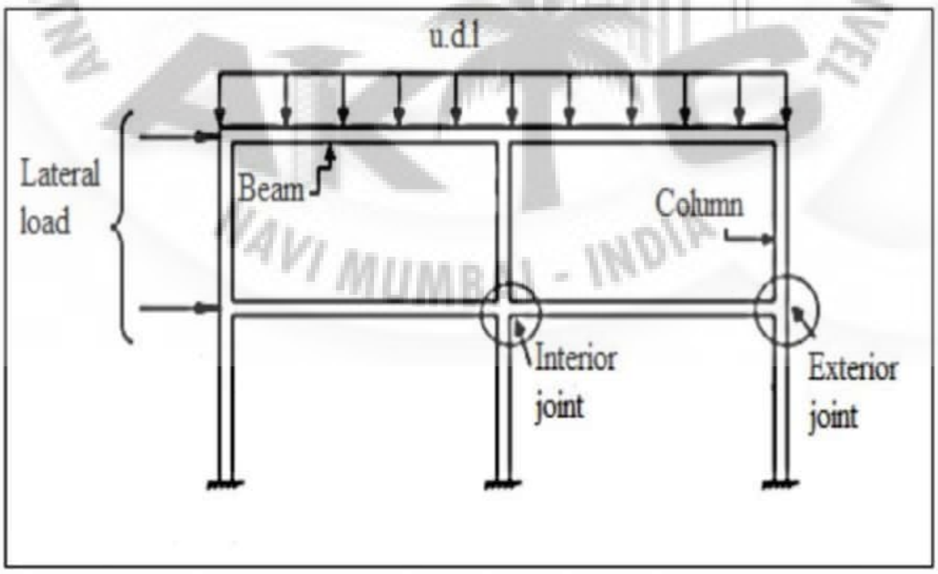



**Figure 3.23: Elevation of Moment Resisting Frame**

### 3.7.1 Special Moment Resisting Frame

Special moment resisting frames resist the lateral seismic forces predominantly by flexure action of its structural members. The structural detailing in such moment resisting frames must ensure the flexural yielding (ductile failure) of frame members prior to shear failure. The sum of design flexural strength of columns at a joint is to be greater (at least 20% greater) than the sum of design flexure strength of beams to ensure strong column weak beam philosophy of SMRF's.

## 3.8 EARTHQUAKE DESIGN PHILOSOPHY

The earthquake shaking intensities and design philosophy shown in Figure 3.24 are summarized as follows:

a) Under minor but frequent shaking, the main members of the building that carry vertical and horizontal forces should not be damage, however building parts that do not carry load may sustain repairable damage.

b) Under moderate but occasional shaking, the main members may sustain repairable damage, while the other parts of the building may be damaged such that they may even have to be replaced after the earthquake and

c) Under strong but rare shaking, the main members may sustain, severe (even irreparable) damage, but the building should not collapse.

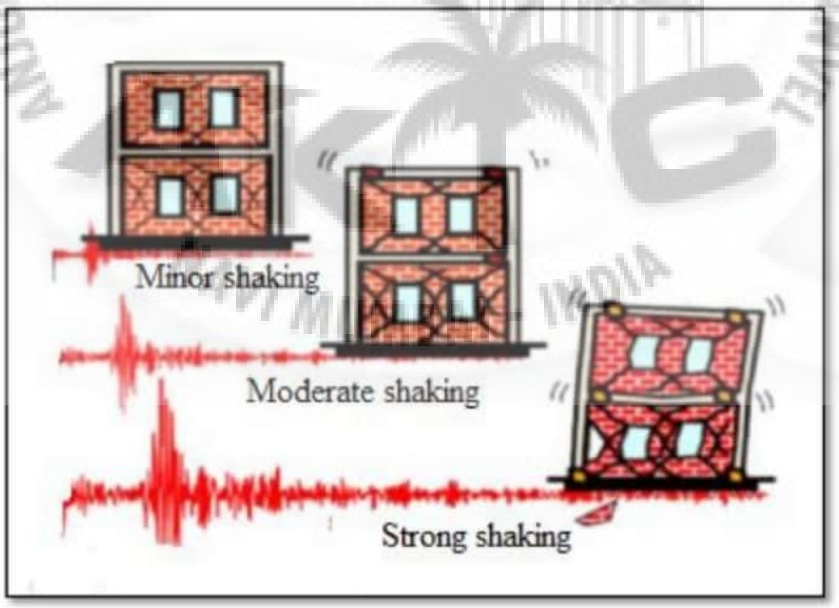



**Figure 3.24: Earthquake Shaking Intensity**

Thus, after minor shaking, the building will be fully operational within a short time and the repair costs will be small, and after moderate shaking, the building will be operational once the repair and strengthening of the damaged main members is completed. But after a strong earthquake, the building may become dysfunctional for further use, but will stand so that people can be evacuated and property recovered. The consequences of damage have to the kept in view in the design philosophy. For example, important buildings like hospitals and fire stations play a critical role in post-earthquake activities and must remain functional immediately after the earthquake. These structures must sustain very little damage and should be designed for a higher level of earthquake protection.

## 3.9 EARTHQUAKE RESISTANCE DESIGN CRITERIA

### 3.9.1 Seismic Zones in India

The varying geology at different locations in the country implies that the like hood of damaging earthquakes taking place at different locations is different. Thus, a seismic zone map is required so that buildings and other structures located in different regions can be designed to withstand different level of ground shaking. The seismic zone map of 1984 subdivided India into five zones – I, II, III, IV and V. Parts of Himalayan boundary in the north and north east, and the Kachha area in the west were classified as zone V.

The seismic zone maps are revised from time to time as more understanding is gained on the geology. The Killari (Latur) earthquake of 1993 occurred in zone I. The current Indian

Seismic zone map places this area in zone III. The zone map now has only four seismic zones – II, III, IV, and V. The areas falling in seismic zone I was merged with those of seismic zone II. Zone II and Zone III are major zones covering more percentage of land area in India. Eastern India has higher seismic intensity. It falls under zone V. North-East India falls under zone IV. Geographical statistics of India show that almost 54% of the land is vulnerable to earthquakes. Figure 3.25 and Table 3.1 shows the Map for various seismic zones and their zone factors respectively.

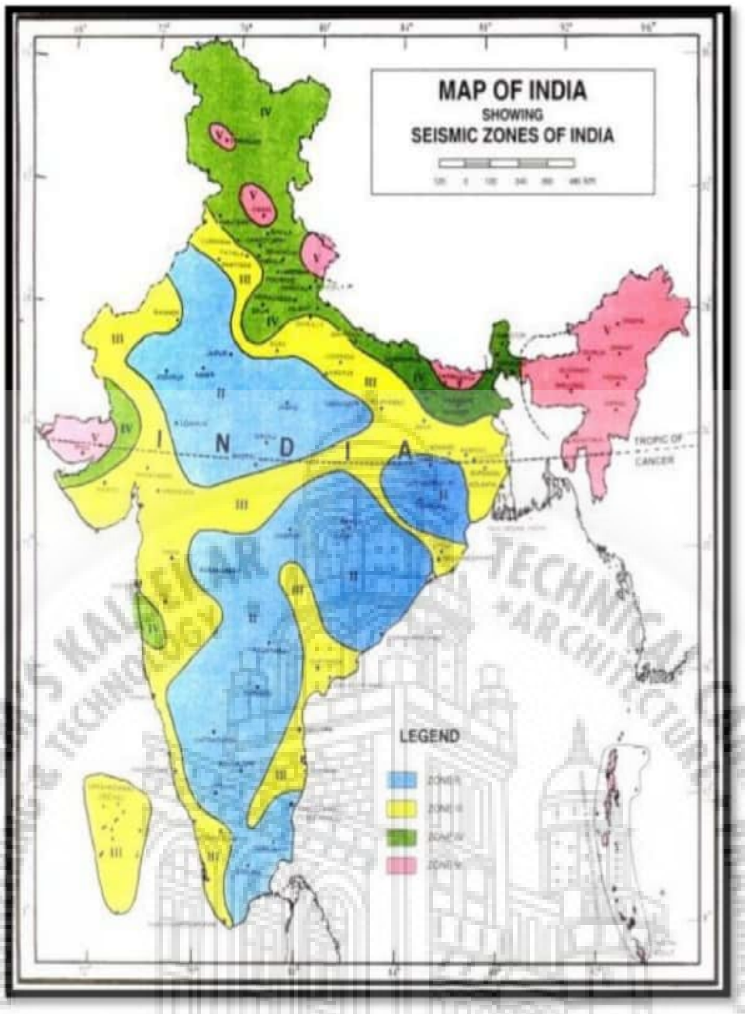



**Figure 3.25: Seismic Map of India**

### 3.9.2 Zone Factor

Seismic zoning assesses the maximum severity of ground shaking that is anticipated in a particular region. The zone factor (Z), thus is defined as a factor to obtain the design spectrum depending on the perceived seismic hazard in the zone in which the structure is located. The basis zone factors included in the code are reasonable estimate of effective peak ground acceleration. Zone factors as per IS 1893 (Part-I): 2002 are given in Table 3.1.

**Table 3.1: Zone Factor (Z) as per the zone of the building**

| **Seismic zone** | II | III | IV | V |
|---|---|---|---|---|
| **Seismic Intensity** | Low | Moderate | Severe | Very Severe |
| **Zone Factor** | 0.10 | 0.16 | 0.24 | 0.36 |

### 3.9.3 Importance Factor

The importance factor is a factor used to obtain the design seismic force depending upon the functional use of the structure. It is customary to recognize that certain categories of building use should be designed for greater levels design forces. It is shown in table no 3.2 Such categories are:

- Buildings which are essential after an earthquake – hospitals, fire stations, etc.
- Places of assembly – schools, theatres, etc.
- Structures the collapse of which may endanger lives – nuclear plants, dams, etc.

**Table 3.2: Importance factor (I) as per the functional use of building (IS-1893)**

| Structure | Importance Factor (I) |
|---|---|
| Important service and community buildings, which as hospitals, schools; monumental structure; emergency buildings like telephone exchange, television stations, radio stations, railway stations, fire stations, buildings; large community halls like cinemas, assembly halls; and subway stations, power stations. | 1.5 |
| All other buildings. | 1.0 |

### 3.9.4 Response Reduction Factor

The basic principle of designing a structure for strong ground motion is that the structure should not collapse but damage to the structural elements is permitted. Since a structure is allowed to be damaged in case of severe shaking, the structure should be designed for seismic force much less than what is expected under strong shaking. Response reduction factor for building systems are given below table no 3.3 as per IS 1893(Part-I): 2002.

**Table 3.3: Response reduction factors (R)**

| Lateral Load-Resisting System | Response Reduction Factor (R) |
|---|---|
| Ordinary RCC moment-resisting frame (OMRF) | 3.0 |
| Special RCC moment-resisting frame (SMRF) | 5.0 |

### 3.9.5 Fundamental Natural Period

The fundamental natural period is the first (longest) model time period of vibration of the structure. Because the design loading depend on the building period, and the building period cannot be calculated until a design has been prepared, IS 1893 (Part I): 2002 provides formulas from which fundamental natural period (T) may be calculated for a moments resisting frame building without brick infill panels, T may be estimated by the empirical expressions.

$$T = 0.075h^{0.75} \quad \text{For RC frame building} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.4)$$

For all other buildings including moment-resisting frame building with brick infill panels, $T_a$ may be estimated by the empirical expression.

$$T_a = \frac{0.09h}{\sqrt{d}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3.5)$$

Where, h is height of building in meters (this excludes the basement stories, where basement walls are connected with the ground floor deck or fitted between the building columns. But it includes the basement stories, when they are not so connected), and d is the base dimension of the building at the plinth level, in meter, along the considered direction of the lateral force.

## 3.10 METHODS OF ELASTIC ANALYSIS

Seismic Engineering is a sub discipline of the broader category of Structural engineering. Its main objectives are

1. To understand interaction of structures with the shaky ground.
2. To foresee the consequences of possible earthquakes.
3. To design, construct and maintain structures to perform at earthquake exposure up to the expectations and in compliance with building codes.

The methodologies available so far for the evaluation of existing buildings can be divided into two categories-(i) Qualitative method (ii) Analytical method. In the same realm, seismic analysis is a subset of structural analysis and is the calculation of the response of a structure to earthquakes. It is part of the process of structural design, earthquake engineering or structural assessment and retrofit in regions where earthquakes are prevalent.

**Structural analysis methods can be divided into the following categories**

- Equivalent Static Analysis
- Response Spectrum method

- Time history method
- Linear Dynamic Analysis

### 3.10.1 Equivalent Static Method

Seismic analysis of most structures is still carried out on the assumption that the lateral (horizontal) force is equivalent to the actual (dynamic) loading. This method requires less effort because, except for the fundamental period, the periods and shapes of higher natural modes of vibration are not required. The base shear which is the total horizontal force on the structure is calculated on the basis of the structure's mass, its fundamental period of vibration, and corresponding shape. The base shear is distributed along the height of the structure, in terms of lateral force, according to the code formula. Planar models appropriate for each of the two orthogonal lateral directions are analyzed separately, the results of the two analyses and the various effects, including those due to torsional motions of the structure, are combined. This method is usually conservative for low to medium-height buildings with a regular configuration.

### 3.10.2 Response Spectrum Method

This method is also known as Modal Method or Mode Super-Position Method. This method is applicable to those structures where modes other than the fundamental one significantly affect the response of structures. Generally, this method is applicable to analysis of the dynamic response of structures, which are asymmetrical or geometrical discontinuity or irregularity, in their linear range of behavior. This method is based on the fact that, for certain forms of damping – which are reasonable models for many buildings-the response in each natural mode of vibration can be computed independently of the others, and the modal responses can be combined to determine the total response. Each mode responds with its own particular pattern of deformation (mode shape), with its own frequency (the modal frequency), and with its own modal damping. First few mode shapes of structure are as shown in Figure 3.26.

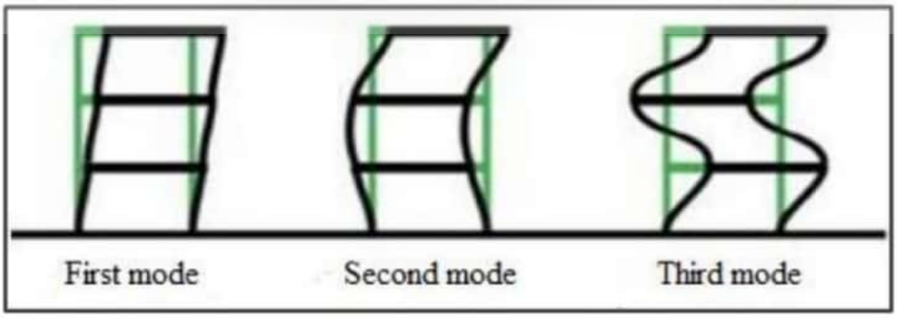



**Figure 3.26: Modes of Vibration**

The time history of each modal response can be computed by analysis of a SDOF oscillator with properties chosen to be representative of the particular mode and the degree to which it is excited by the earthquake motion. In general, the responses need to be determined only in the first few modes, because response to earthquake in primarily due to lower modes of vibration.

A complete modal analysis provides the history of response – forces, displacements, and deformation of a structure to a specified ground acceleration history. However, the complete response history is rarely needed for design; the maximum values of response over the duration of the earthquake usually suffice. Because the response in each vibration mode can be modeled by the response of a SDOF oscillator, the maximum response in the mode can be directly computed from the earthquake response spectrum. Procedures for combining the modal maxima to obtain estimates (but not the exact value) of the maximum of total response are available. In its most general from, the modal method for linear response analysis is applicable to arbitrary three-dimensional structural systems. For Planar models each of two orthogonal lateral directions are analyzed separately, and the result of the two analyses and the effects of torsional motions to the structures are combined.

### 3.10.3 Time History Method

A linear time history analysis overcomes all the disadvantages of a Modal Response Spectrum Analysis provided non-linear behavior is not involved. This method requires greater computational efforts for calculating the response at discrete times. One interesting advantage of such a procedure is that the relative signs of response quantities are preserved in the response histories. This is important when interaction effects are considered among stress resultants.

## 3.11 PROCEDURES OF SEISMIC METHODS

### 3.11.1 Equivalent Lateral Force Procedure

This method of finding design lateral forces is also known as Pseudo static analysis or the equivalent static analysis or the seismic coefficient method. This procedure does not require dynamic analysis, however, it accounts for the dynamics of building in an approximate manner. The static method is the simplest one, it requires less computational effort and is based on formulae given in IS1893 (Part I):2002. First, the design base shear is computed for the whole building, and it is then distributed along the height of the building. The lateral forces at each floor level thus obtained are distributed to individual lateral load resisting elements.

### 3.11.2 Seismic Base Shear

According to IS 1893 (Part-I): 2002, Clause 7.5.3 the total design lateral force or design seismic base shear ($V_B$) along any principal direction is determined by

$$V_B = A_h W \quad \text{……………………………………………………………(3.6)}$$

Where Ah is the design horizontal acceleration spectrum value, using the fundamental natural period T, in the considered direction of vibration and W is the seismic weight of the building.

#### 3.11.2.1 Design Horizontal Acceleration Spectrum Value (Ah)

According to IS 1893(Part I) 2002, Clause 6.4.2 the design horizontal seismic coefficient (Ah) for a structure is determined by the expression.

$$A_h = \frac{Z}{2}\frac{I}{R}\frac{Sa}{g} \quad \text{…………………………………………………(3.7)}$$

Where,

Z = Zone factor seismic intensity, it is a factor to obtain the design spectrum depending on the perceived maximum seismic risk characterized by Maximum Considered Earthquake (MCE) in the zone in which the structure is located.

I = Importance factor, depending upon functional use of structure. It is a factor used to obtain the design seismic force depending on the functional use of the structure, characterized by hazardous consequences of its failure, its post-earthquake functional need, historic value or economic importance.

R = Response reduction factor depending on the perceived seismic damage performance of the structure characterized by, As per IS 1893, Table 7. The value of R is considered from Ordinary moment resisting frame or special moment resisting frame.

Sa/g = Average response acceleration coefficient (dimensionless value). It is a factor denoting the acceleration response spectrum of the structure subjected to earthquake ground vibrations, and depends on natural period of vibration and damping of the structure. For 5% (RCC structures) damping three different curves are recommended in IS 1893(Part I):2002 for different stiffness of supporting media – rock, medium soil and soft soil. The classification of soil is based on average shear wave velocity for top 30 m of rock/soil layers or based on average Standard Penetration Test (SPT) values for top 30 m (Table 1, IS 1893(Part I):2002)

Class I – Rock or Hard soil : Well graded gravel and sand gravel mixture with or without clay binder having corrected Standard Penetration Value N > 30

Class II – Medium soil : All soils with N between 10 and 30 or gravelly sand with little or no fines (classification SP) with N > 15

Class III – Soft soil : All soils other than SP with N < 10

Where, N = Standard Penetration value

### 3.11.2.2 Seismic Weight

The seismic weight of the whole building is the sum of the seismic weights of all the floors. The seismic weight of each floor is its full dead load plus the appropriate amount of imposed load, the latter being that part of the imposed loads that may reasonably be expected to be attached to the structure at the time of earthquake shaking. It includes the weight of permanent and movable partitions, permanent equipment, a part of the live load, etc. While computing the seismic weight of each floor, the weight of columns and walls in any storey should be equally distributed to the floors above and below the storey. Any weight supported in between storey's should be distributed to the floors above and below in inverse proportion to its distance from the floors.

As per IS 1893(Part I):2002, the percentage of imposed load as given in Table 3.4 should be used. For calculating the design seismic forces of the structure, the imposed load on the roof need not be considered.

**Table 3.4: For Percentage of imposed load to be considered in seismic weight calculation**

| Imposed uniformly distributed floor load($KN/m^2$) | Percentage of Imposed load |
|---|---|
| Up to and including 3.0 | 25 |
| Above 3.0 | 50 |

### 3.11.2.3 Distribution of Design Force

Buildings and their elements should be designed and constructed to resist the effects of design lateral force. The design lateral force is first computed for the building as a whole and then distributed to various floor levels. The overall design seismic force thus obtained at each floor level is then distributed to individual lateral load-resisting elements, depending on the floor diaphragm action.

The design base shear (VB) is distributed along the height of the building as per the following expression:

$$Q_{i=V_B} \frac{W_i h_i^2}{\sum_{j=1}^{n} W_j h_j^2} \qquad \text{.........................................(3.8)}$$

Where, Qi is the design lateral force at floor i, Wi is the seismic weight of floor i, hi is the height of floor i measured from the base, and n is the number of stories in the building i.e., the number of levels at which the masses are located.

## 3.12 TYPES OF LOADS AND THEIR COMBINATIONS

Different types of loads and there combinations are as shown in below

All loads listed below, shall be considered in design of structure.

(a) **DL:** Dead Load

(b) **LL:** Live Load

(c) **EQ:** Earthquake load

(d) **$EQ_x$:** Earthquake load in X-direction.

(e) **$EQ_y$:** Earthquake load in Y-direction.

Buildings, structures, foundations and all structural components are designed for the following load combination and checked for most critical combinations.

### 3.12.1 Load Combinations

1) EQx
2) EQy
3) DL
4) LL
5) 1.5(DL+LL)
6) 1.5(DL+EQx)
7) 1.5(DL-EQx)
8) 1.5(DL+EQy)
9) 1.5(DL-EQy)
10) 1.2(DL+LL+EQx)
11) 1.2(DL+LL-EQx)
12) 1.2(DL+LL+EQy)
13) 1.2(DL+LL-EQy)
14) 0.9DL+1.5EQx
15) 0.9DL-1.5EQx
16) 0.9DL+1.5EQy
17) 0.9DL-1.5EQy

Analysis is performed using above load combination.

### 3.13 IS 1893 (PART I):2002 CODAL PROVISIONS FOR SOFT STOREY

In case buildings with a flexible storey, such as the ground storey consisting of open spaces for parking, special arrangement needs to be made to increase the lateral strength and stiffness of the soft/open ground storey.

Dynamic analysis of building is carried out including the strength and stiffness effects of infills and inelastic deformations in the members, particularly, those in the soft storey and the members designed accordingly.

The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads.

Shear wall is one of the most important solutions for soft storey effect.

**3.13.1 Design Approach 1:** Dynamic analysis of building is carried out including the strength and stiffness effects of infills and inelastic deformations in the members, particularly, those in the soft storey and the members designed accordingly.

Dynamic analysis may be performed either by the Time History Method or by the Response Spectrum Method. However, in either method, the design base shear ($V_B$) shall be compared with a base shear ($V_B$) calculated using a fundamental period $T_a$, where $T_a$, is as per IS1893 (Part-I):2002 clause no. 7.6. Where $V_B$ is less than $V_B$, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) shall be multiplied by $V_B$ / $V_B$.

**3.13.2 Design Approach 2:** The columns and beams of the soft storey are to be designed for 2.5 times the storey shears and moments calculated under seismic loads. This 2.5 times storey shear distributed accordingly as per IS1893 (Part-I):2002.

### 3.14 SUMMARY

This chapter focuses on concept of all seismic parameters required for analysis procedure by seismic coefficient method, Masonary infill effect, soft storey mechanism, modelling of infill using an equivalent diagonal strut method, soft storey failures during earthquake and also IS-1893 codal provisions of soft storey.

# Chapter Four
# PARAMETRIC INVESTIGATION

# CHAPTER 4
# PARAMETRIC INVESTIGATION

## 4.1 INTRODUCTION

A detailed parametric investigation on Soft storey behavior and also design approaches of Soft storey as per IS1893 (Part-I):2002 is illustrated in this chapter. Parametric investigation is carried out to know the soft storey behavior under lateral loading and to overcome the soft storey effect by introducing the Masonary Infill Walls, Tie-beam and Bracings at periphery in soft storey by introducing variation of the soft storey height from 4-6 m (with interval of 0.2m). Also the study is carried out on different soil conditions I and III (i.e Hard and Soft) by time period considered for seismic analysis as per Program Calculated and as per Codal Provision using Static Analysis (Seismic Co-efficient Method). Inorder to study the effect of earthquake, columns are grouped so that behavior of columns can be easily studied. And also it is carried out to know the most convenient, economical and applicable approach of soft storey design.

## 4.2 PARAMETRIC VARIATIONS

The space frame for **Soft storey** analysis is considered as shown in following Table 4.1.

**Table.4.1: Parametric Cases**

<table>
<tr><td rowspan="2">Sr. No</td><td colspan="2">Case 1 Program Calculated Space Frame (P+7)</td><td rowspan="2">Seismic Zone</td></tr>
<tr><td>Soft storey behavior</td><td>Soft storey Design</td></tr>
<tr><td>1</td><td>Soil condition (I and III)</td><td rowspan="2">Rectangular column size</td><td rowspan="6">V</td></tr>
<tr><td>2</td><td>Height Variation</td></tr>
<tr><td rowspan="2">Sr. No</td><td colspan="2">Case 2 Codal Provision Space Frame (P+7)</td></tr>
<tr><td>Soft storey behavior</td><td>Soft storey Design</td></tr>
<tr><td>1</td><td>Soil condition (I and III)</td><td rowspan="2">Square column size</td></tr>
<tr><td>2</td><td>Height Variation</td></tr>
</table>

### 4.3 PROBLEM FORMULATION

**4.3.1 Study of Behavior of soft storey in space frame:** To check Plan irregularity, space frame is considered for studying behavior of soft storey. To study the behavior of bottom storey columns Bare frame, Frame with Brick Masonry Infill, Frame with Tie-beam and Frame with Bracings these models are considered for analysis and it is described in Section 4. (P+7) RCC multi-storied building and Plan of size (5 x 3) bay is considered for space frame behavior of Open ground storey in Zone V. The spacing of bay in X-direction is 4.5m and that in Y-direction is 4m. Grouping of the columns is also shown in further section

**(A) Structural Descprition and Analysing Data used in Time period considered for Seismic Analysis as per Program Calculated**

The geometry of the models considered for analysis is as shown in fig 4.1. Rectangular columns are used for analysis and analysing data is given in table no 4.2

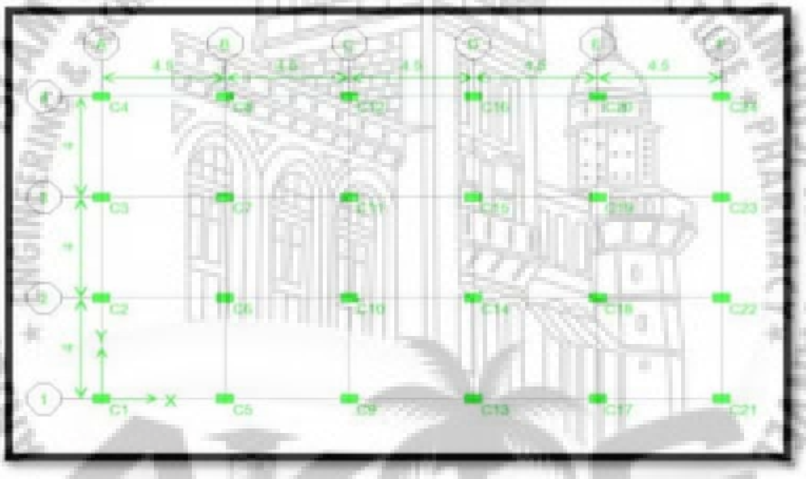


**Figure 4.1: Plan of (P+7) RCC Building (Program Calculated)**

**Table 4.2: Analysis Data of (P+7) RCC Building for Program Calculated**

| Sr. No. | Data summary for Models | |
|---|---|---|
| 1 | Grade of Concrete | M25 |
| 2 | Main Steel Reinforcement | Fe 500 |
| 3 | Yield stress of Stirrups and Links | Fe 415 |
| 4 | Number of Storey | (P+7) |
| 5 | Plan Size | 5x3 bay |
| 6 | Spacing<br>In X-direction<br>In Y-direction | 4.5m<br>4.0m |

| | | |
|---|---|---|
| 7 | Floor to Floor Height | 3.2m |
| 8 | Bottom Storey Height | 4-6m ( interval 0.2m) |
| 9 | Density of Concrete | 25 $KN/m^3$ |
| 10 | Modulus of Elasticity of Concrete | 25000$N/mm^2$ |
| 11 | Poisson Ratio for Concrete | 0.2 |
| 12 | Damping | 5% |
| 14 | Importance Factor | 1 |
| 15 | Response Reduction Factor | 5 |
| 16 | Foundation | Hard & Soft soil |
| 17 | Beam Size (mm) for Tie-Beam and Bracing | 230 x 450 |
| 18 | Column Size (mm)<br>Bottom<br>Middle<br>Top | <br>350 x 750<br>300 x 680<br>300 x 600 |
| 19 | Slab Thickness | 150mm |
| 20 | Density of Brick Masonry | 20$KN/m^3$ |
| 21 | Modulus of Elasticity of Brick Masonry | 1255 $N/mm^2$ |
| 22 | Thickness of wall | 150 mm |
| 23 | Poisson Ratio for Brick Masonry | 0.15 |

**(B) Structural Descprition and Analysing Data used in Time period considered for Seismic Analysis as per Codal Provisions**

The geometry of the models considered for analysis is as shown in fig 4.2. Sqauare columns are used for analysis and analysing data is given in table no 4.3

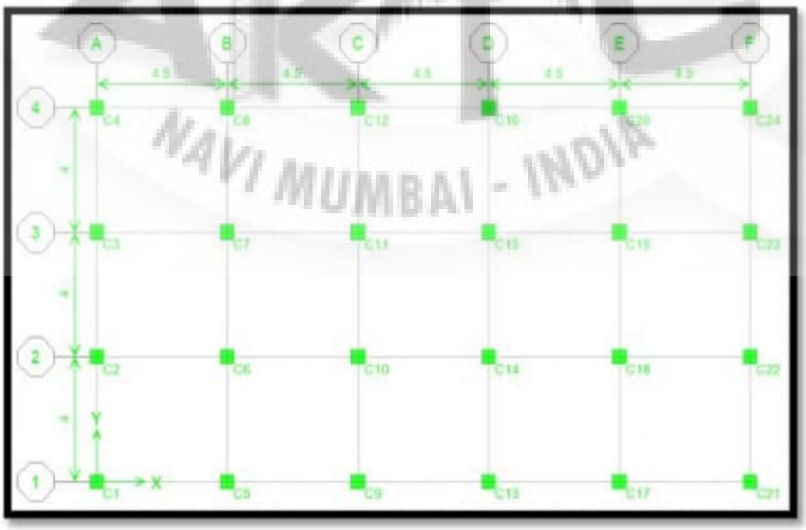



**Figure 4.2: Plan of (P+7) RCC Building (Codal Provision)**

**Table 4.3: Analysis Data of (P+7) RCC Building for Codal Provisions**

| Sr. No. | Data summary for Models | |
|---|---|---|
| 1 | Grade of Concrete | M25 |
| 2 | Main Steel Reinforcement | Fe 500 |
| 3 | Yield stress of Stirrups and Links | Fe 415 |
| 4 | Number of Storey | (P+7) |
| 5 | Plan Size | 5x3 bay |
| 6 | Spacing<br>In X-direction<br>In Y-direction | 4.5m<br>4.0m |
| 7 | Floor to Floor Height | 3.2m |
| 8 | Bottom Storey Height | 4-6m<br>( interval 0.2m) |
| 9 | Density of Concrete | 25 KN/m$^3$ |
| 10 | Modulus of Elasticity of Concrete | 25000N/mm$^2$ |
| 11 | Poisson Ratio for Concrete | 0.2 |
| 12 | Damping | 5% |
| 14 | Importance Factor | 1 |
| 15 | Response Reduction Factor | 5 |
| 16 | Foundation | Hard & Soft soil |
| 17 | Beam Size (mm) for<br>Tie-Beam and Bracing | 230 x 450 |
| 18 | Column Size (mm)<br>Bottom<br>Middle<br>Top | 600 x 600<br>500 x 500<br>450 x 450 |
| 19 | Slab Thickness | 150mm |
| 20 | Density of Brick Masonry | 20KN/m$^3$ |
| 21 | Modulus of Elasticity of Brick Masonry | 1255 N/mm$^2$ |
| 22 | Thickness of wall | 150 mm |
| 23 | Poisson Ratio for Brick Masonry | 0.15 |

**(C) Grouping of columns considered during analysis**

Inorder to study the behavior of bottom storey columns and to know the columns which are severely affected during earthquake groups of columns were made which are represented in following figures from 4.3- 4.6. The groups formed are illustrated as below

Group 1: Corner exterior columns.
Group 2: Longer direction peripheral columns.
Group 3: Shorter direction peripheral columns.
Group 4: Interior columns.
The grouping of columns gives clear idea of in-plane and out-of-plane action during earthquake
GROUPING OF COLUMNS:

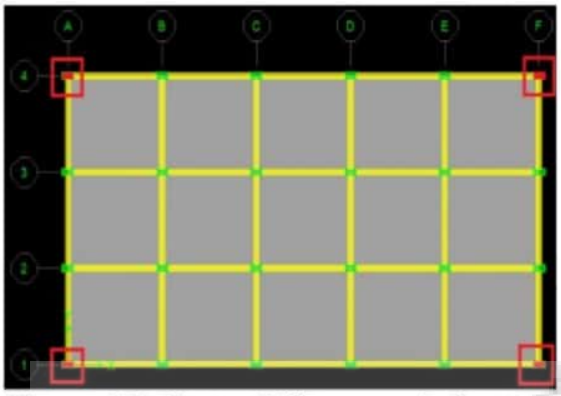


Figure 4.3: Group 1(Corner exterior columns)

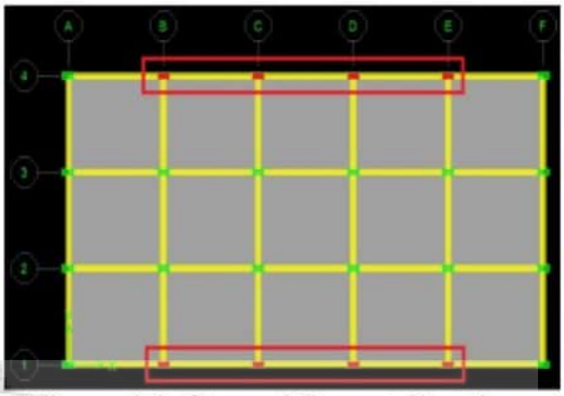


Figure 4.4: Group 2(Longerdirection peripheral columns)

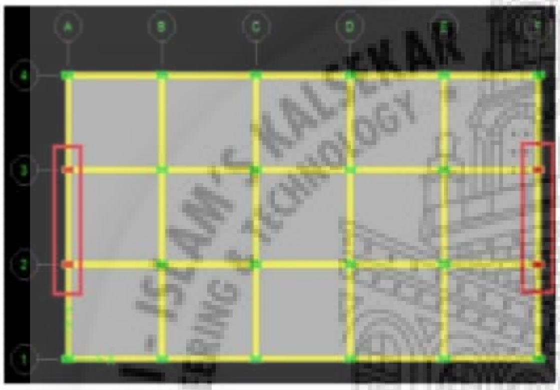


Figure 4.5: Group 3(Shorter direction peripheral columns)

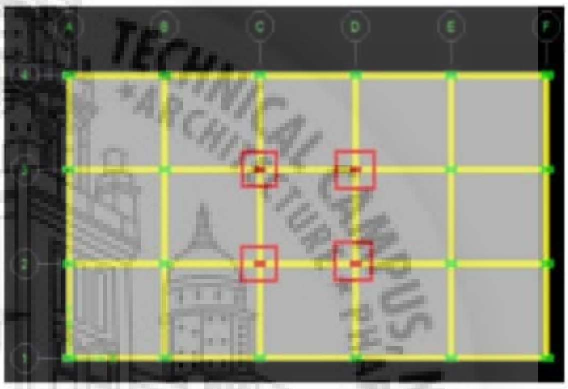


Figure 4.6: Group 4 (Interior columns)

## 4.4 METHODOLOGY

Following models were considered for analysis purpose and remained same through-out the analyasis irrespective of soil conditions and time period considered for seismic analysis as per Program Calculated and as per Codal Provision. The models analysed are shown as below from fig 4.7- 4.10. The masonary infill, tie-beam and bracings are provided only in bottom storey through-out the peripheryof the frames. It is shown in table no 4.4

**Table 4.4: Types of Models used for Analysis**

| Model No. | Name of Model |
|---|---|
| Model I | Frame without masonary infill effect (Bare Frame) |
| Model II | Frame with Masonary Infill effect |
| Model III | Frame with Tie-beam |
| Model IV | Frame with Bracings |

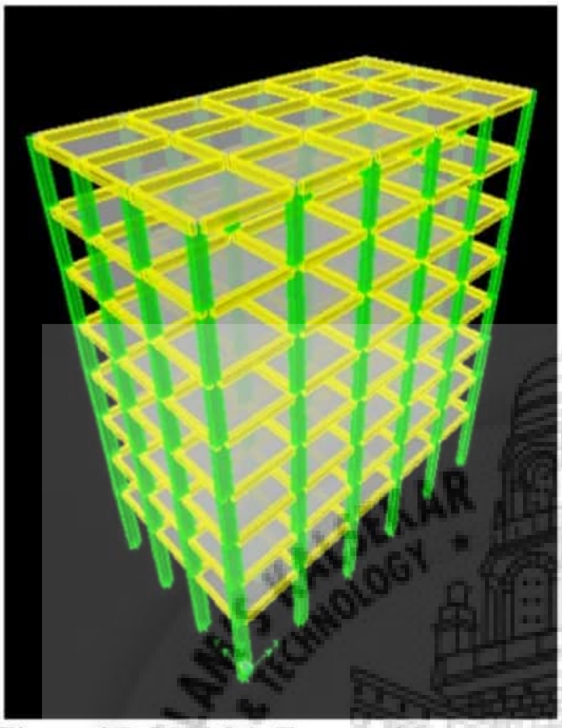


Figure 4.7: Model 1- Frame without masonary Infill effect(Bare frame)

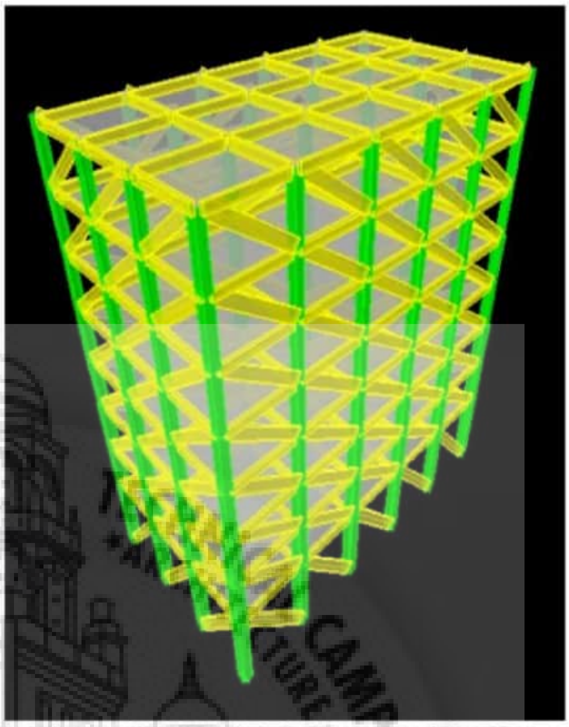


Figure 4.8: Model 2- Frame with effect of masonary infill (Infill frame)

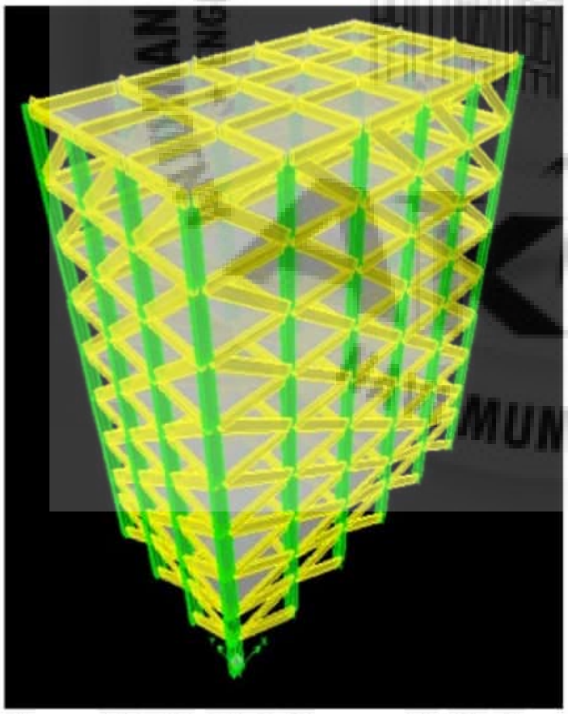


Figure 4.9: Model 3- Frame with Tie-beam

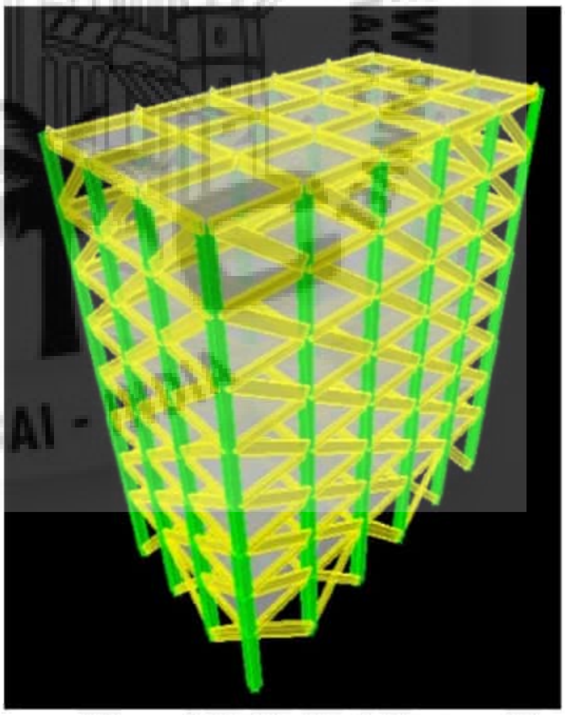


Figure 4.10: Model 4- Frame with Bracings

## 4.5 LOAD CALCULATION

**Gravity Loads**

**Dead Loads (D.L)**

(a) Slab load (D.L)

Intensity of slab load = 0.15x1x25=3.75KN/m

Load transfer = Wlx/3 = 3.75x4/3 =5KN/m

(b) Wall load on beams = (3-0.45)x0.15x20=7.65KN/m

(c) Floor finish (F.F)

Intensity of Floor finish load = 1.0x1 = 1.0KN/m

Load transfer = $Wl_x/3$ = 1.0x4/3 = 1.33 KN/m

**Live Load (L.L)**

Live load on floor = 4KN/$m^2$ …………………… As per IS 875 Part-II (Public Building)

Intensity of live load = 4x1 = 4KN/m

Load transfer = $Wl_x/3$ = 4x4/3 =5.33KN/m

### 4.5.1 Masonary Infill Validation

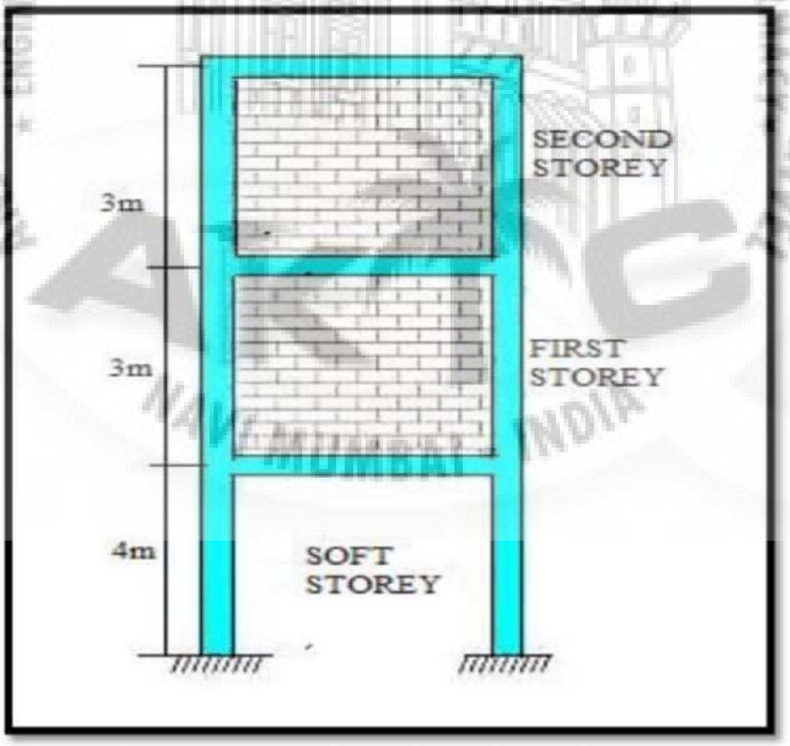



**Figure No 4.10 (G+2) Plane Frame**

**Frame properties**

$E_f = 5000\sqrt{(f_{ck})}$

$F_{ck} = 20$ MPa

$I_c = I_b = 0.002278125\ m^4$

h= height of infill wall= 2.55 m

l= length of infill wall = 4.55 m

$I_b$ Moment of inertia of the beam

$I_c$ = Moment of inertia of the column

Width of beam and column (b) = 0.30 m

Depth of beam and column (d) = 0.45 m

**Infill properties**

$E_m$ = elastic modulus of masonary wall =13800 MPa

t= thickness of the infill wall = 0.23 m

(a) Calculating the width of equivalent diagonal strut

Infill in second and third stories are modeled as equivalent diagonal struts and its equivalent width of strut is given by

$$W = \frac{1}{2}\sqrt{\alpha_h^2 + \alpha_L^2}$$

Where,

$$\alpha_h = \pi\left[\frac{E_f I_b L}{2E_m\, t\sin 2\theta}\right]^{1/4}$$

$\theta = \tan^{-1}(h/L)$

$\theta = \tan^{-1}(2.55/4.55) = 29.267^\circ$

$L_d$ , $A_d$ = Length and area of equivalent strut

$\alpha_h = 0.618$ m

$\alpha_l = 1.7$ m

W = 0.904 m

$L_d = \sqrt{h^2 + L^2} = 5.218$ m

$A_d = t \times W = 0.2079\ m^2$

(a) Analysis of frame with strut

(b) The frame has been analysed with diagonal pin jointed strut using a plane frame computer program as shown in figure 4.10. Stiffness is calculated by assuming that the support are fixed and load is applied at the floor level. Horizontal deflection is measured at the floor level and lateral stiffness is calculated by dividing horizontal deflection to unit load. The stiffness of all three stories are presented.It is shown in figure 4.11 and table no 4.5

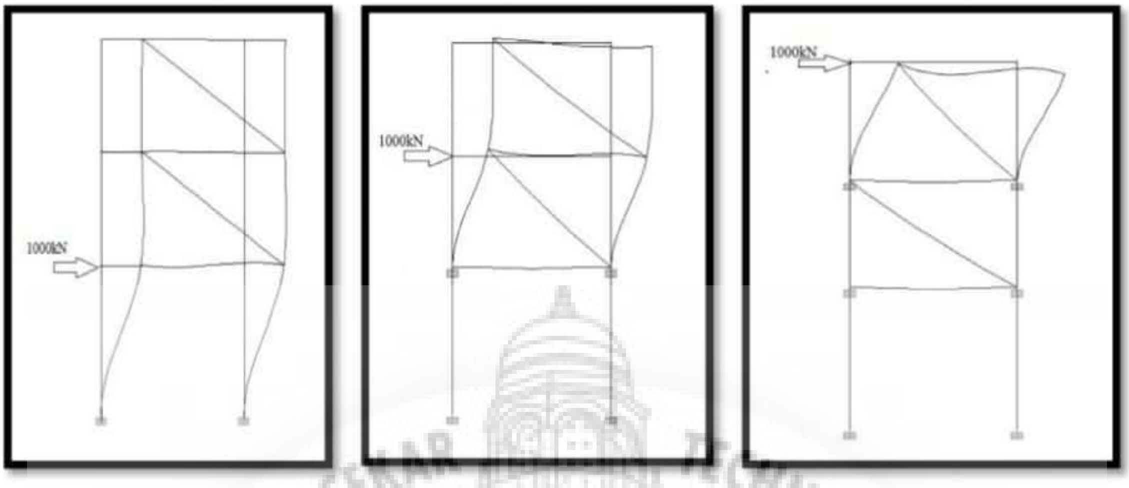



**Figure No 4.11 Distribution of Force**

The ratio of stiffness of first story without infill to second story with infill is

$$\frac{\text{K without infill}}{\text{K with infill}} = 0.042$$

The ratio of stiffness of first story without infill to second story with infill is

$$\frac{\text{K without infill}}{\text{K with infill}} = 0.042$$

### 4.5.2 Base shear Validation

**Table 4.5: Analysis Data for Base shear Validation**

| Sr. No. | Data summary for Models | |
|---|---|---|
| 1 | Type of structure | Special RC moment resisting frame |
| 2 | Seismic zone | IV ( Table 2,IS 1893 (Part 1) :2002) |
| 3 | Numbers of stories | Four |
| 4 | Floor height | 3.5m |
| 5 | Infill wall | 250mm thick including plaster in longitudinal and 150mm in transverse direction |
| 6 | Imposed load | 3.5 $KN/m^2$ |
| 7 | materials | Concrete (M-20) and reinforcement (Fe-415) |
| 8 | Size of columns | 250mm x 450mm |

| | | |
|---|---|---|
| 9 | Size of beams | 250mm x 400mm in longitudinal and 250mm x 350mm in transverse direction |
| 10 | Depth of slab | 100mm thick |
| 11 | Specific weight of RCC | 25kN/m$^3$ |
| 12 | Specific weight of infill | 20kN/m$^3$ |
| 14 | Type of soil | Rock |
| 15 | Response spectra | As per IS 1893 (Part 1):2002 |
| 16 | Time history | Compatible to IS 1893(Part 1):2002 spectra at rocky site for 5%damping |

**SOLUTION**

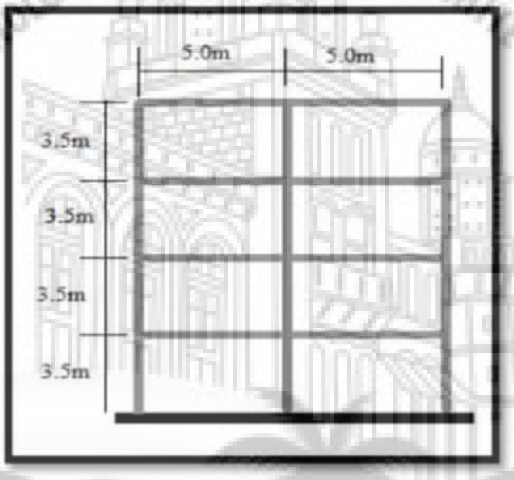



**Figure No 4.13 (G+3) MODEL**

**1] Evaluation of seismic wt. of given structure:**

**Roof**

Mass of infill + Mass of columns + Mass of beams in longitudinal and Transverse direction of that floor + Mass of slab + imposed load of that floor

= {((0.25x10x (3.5/2) +0.15x15x (3.5/2)) 20} + {(0.25x10x0.40+0.25x15x0.35)25} + {0.10x5x10x25} + {(0.25x0.45x (3.5/2) x3) x25} + 0$^*$

= 363.82 KN (Weight) = 37.087 ton (mass)

3$^{rd}$, 2$^{nd}$, 1$^{st}$ Floor

= {((0.25x10x3.50) + (0.15x15x3.5x3x25)) 20} + {(0.25x10x0.40 + 0.25x15x0.35)25} + {0.10x5x10x25} + {0.25x0.45x3.5x3x25} + {5x10x3.5x0.5$^{**}$}

= 632.43 KN (Weight) = 64.45 ton (mass)

*Imposed load on roof not considered.

**50% of imposed load, if imposed load is greater than 3kN/m$^2$

**Seismic weight of building**

= Seismic weight of all floors = $M_1 + M_2 + M_3 + M_4$

= 64.45 + 64.45 + 64.45 + 37.08

= 230.43 ton

**2] Determination of fundamental natural period**

The approximate fundamental natural period of a vibration (Ta), in seconds, of a moment Resisting frame building without brick infill panels may be estimated by the emerical Expression. Frame as shown in figure 4.13

$$Ta = 0.075 \text{ X } h^{0.75}$$

$$= 0.075 \text{ X } 14^{0.75}$$

$$= 0.5423 \text{ sec}$$

Where h is the height of the building, in meter.

**3] Determination of design base shear**

Design seismic base shear, $V_B = A_h \text{ x } W$

$$Ah = \frac{Z}{2} X \frac{I}{R} X \frac{Sa}{g}$$

$$Ah = \frac{0.24}{2} X \frac{1}{5} X 1.842$$

$$= 0.0443$$

For Ta = 0.5423, (Sa / g) = (1/Ta) = 1.842, for rock site from Figure 2 of IS 1893 (Part 1):2002

**4] Evaluation of base shear (VB) –**

Design seismic base shear,

$$V_B = 0.0443 \text{ X } (230.43 \text{ X } 9.81)$$

$$= 99.933KN$$

**5] Storey Shear Distribution**

$$Qi = VB \frac{Wihi^2}{\sum_{i=1}^{n} Wihi^2}$$

$$= 99.93 \left[\frac{632.25X3.5^2}{632.25 \text{ X } 3.5^2 + 632.25 \text{ X } 7^2 + 632.25 \text{ X} 10.5^2 + 632.25 \text{ X } 14^2}\right]$$

$$= 4.306kN$$

Similarly,

$$Q_2 = 0.1724 \text{ X } 99.933 = 17.224kN$$

$$Q_3 = 0.3872 \text{ X } 99.933 = 38.733\text{kN}$$

$$Q_4 = 0.3967 \text{ X } 99.933 = 39.66\text{kN}$$

Lateral force distribution at various floor levels as shown in figure 4.14. and Compared in table No.4.6

**Analysis of Frame From Etabs**

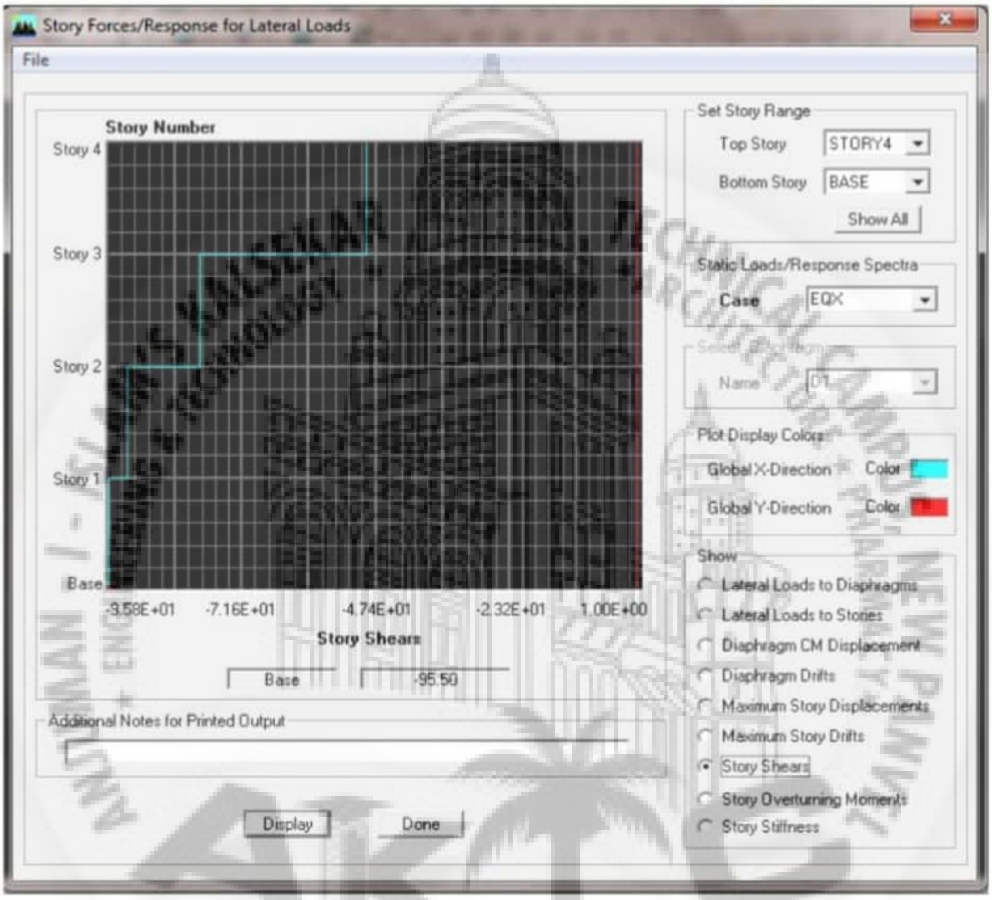



**Figure No 4.14 Base Shear Distribution from E-Tabs**

**Table 4.6: Comparison of Base shear**

| | |
|---|---|
| 1) Manually Calculated | 99.933 KN |
| 2) E-tabs Software | 95.50 KN |

## 4.6 PARAMETRIC STUDY

The shear force and bending moment demand are severly higher for ground storey columns with respect to first storey column. Therefore ratios are developed in regard to the shear force and bending moment of soft storey columns it is represented in figure no 4.15

$$RPu = \frac{\text{Axial force of bottom storey column for frame with (Infill/Tie-Beam/Braced Frame)}}{\text{Axial force of bottom storey column for Bare Frame}}$$

$$RMu_2 = \frac{\text{Bending moment of bottom storey column for frame with(Infill/Tie-Beam/Braced Frame in X-direction)}}{\text{Bending moment of bottom storey column for Bare Frame}}$$

$$RMu_3 = \frac{\text{Bending moment of bottom storey column for frame with (Infill/Tie-Beam/Braced Frame in Y-direction)}}{\text{Bending moment of bottom storey column for Bare Frame}}$$

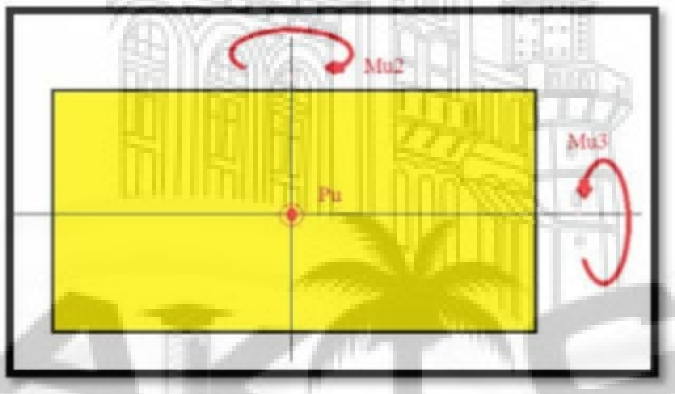


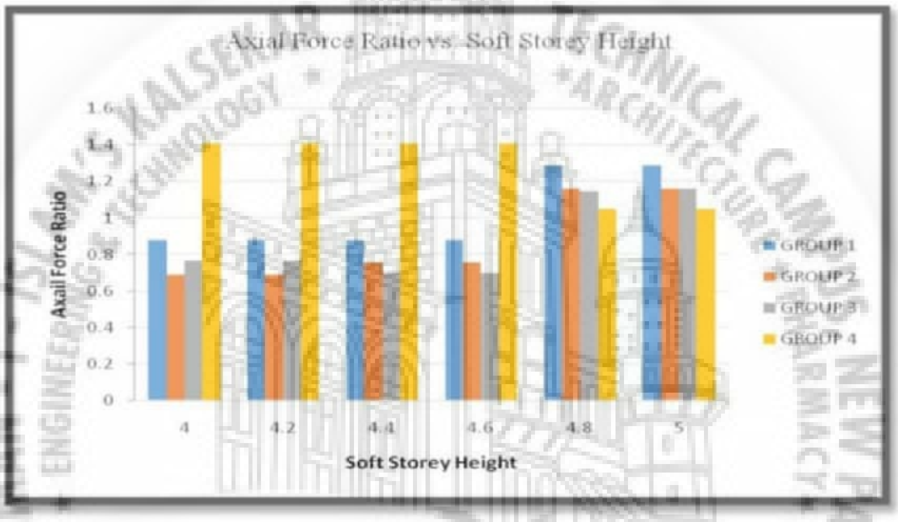

**Figure 4.15: Schematic Representaion of Directions ( Axial comp. force and Bending moments)**

Above re-presented data shows the formulations of the ratios to be developed. Depending on these formulations the study below represents the observed ratios of different soil conditions and time considered for seismic analysis as per Program Calculated and as per Codal Provision for different soft storey heights. The ratios are observed as per the grouping of columns made which is explained in section 4.3.1.(C).The ratios observed are represented schematically in Tabular and Graphical format firstly for time period considered for seismic analysis as per Program Calculated and its varying soil conditions (Hard and Soft) for different soft storey heights. And secondly for time period considered for seismic analysis as per Codal Provisions and its varying soil conditions (Hard and Soft) for different soft storey heights and as per the groups of bottom storey columns. Observations are listed based upon these ratios developed. The ratios represented in the tabular are as shown below from table no 4.7 – 4.18 . Also the graphs showing the variation of ratios developed for axial forces (RPu) and that for bending moment ($RMu_2$ and $RMu_3$) are as shown in the following graphs from 4.1-4.36

## A) CONSIDERING TIME PERIOD FOR SEISMIC ANALYSIS USING SOFTWARE (PROGRAM CALCULATED) FOR SOIL TYPE I

**Table 4.7: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Masonry Infill.**

| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ |
| 4 | 0.88 | 0.26 | 0.88 | 0.69 | 0.36 | 0.69 | 0.77 | 0.26 | 0.77 | 1.41 | 0.34 | 0.19 |
| 4.2 | 0.88 | 0.26 | 0.88 | 0.69 | 0.29 | 0.69 | 0.77 | 0.3 | 0.77 | 1.41 | 0.32 | 0.19 |
| 4.4 | 0.88 | 0.27 | 0.88 | 0.76 | 0.26 | 0.76 | 0.7 | 0.35 | 0.7 | 1.41 | 0.3 | 0.21 |
| 4.6 | 0.88 | 0.27 | 0.88 | 0.76 | 0.25 | 0.76 | 0.7 | 0.36 | 0.7 | 1.41 | 0.29 | 0.21 |
| 4.8 | 1.29 | 1.11 | 1.29 | 1.16 | 1.07 | 1.16 | 1.15 | 1.05 | 1.15 | 1.05 | 1.03 | 1.05 |
| 5 | 1.29 | 1.11 | 1.29 | 1.16 | 1.07 | 1.16 | 1.16 | 1.05 | 1.16 | 1.05 | 1.03 | 1.05 |




**Graph 4.1: Variations of Axial Force for different soft storey heights and column groups**

### Observations and Discussions

Axial compressive force ratio at bottom storey is increasing in group 1, 2 & 3 as bottom storey height increases.

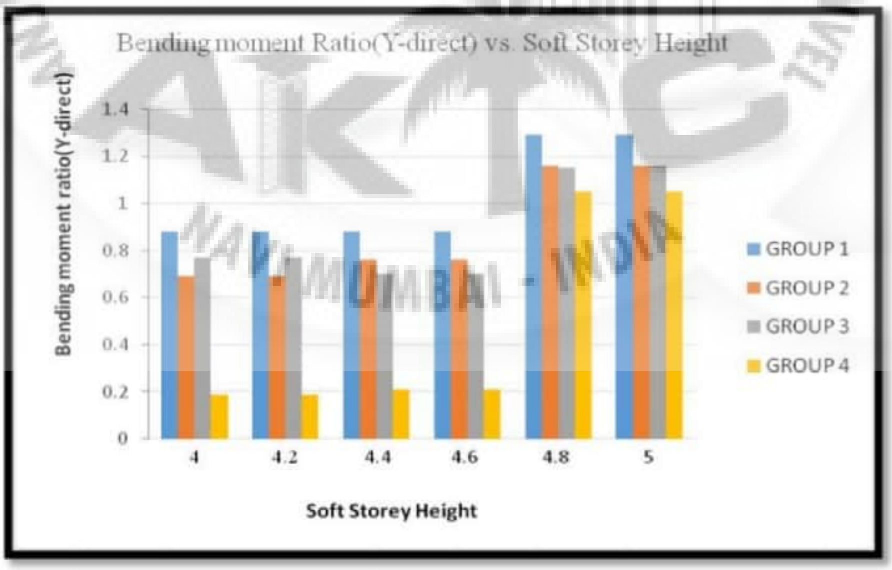
Axial compressive force ratio is constant for group 4 up to 4.6m storey height i.e 1.41. Which is observed to be maximum and then decreases for 4.8m and 5m storey height.


**Graph 4.2: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

Bending moment ratio in X-direction is in increasing trend for group 1 and 3, where as there is decreasing trend for group 2 for all bottom storey height. For group 4 it decreases up to 4.6m and increases for 4.8m and 5m.

In group 1, the ratio of Bending moment in X-direction is observed to be maximum for soft storey height (4.8-5m) i.e. 1.11.



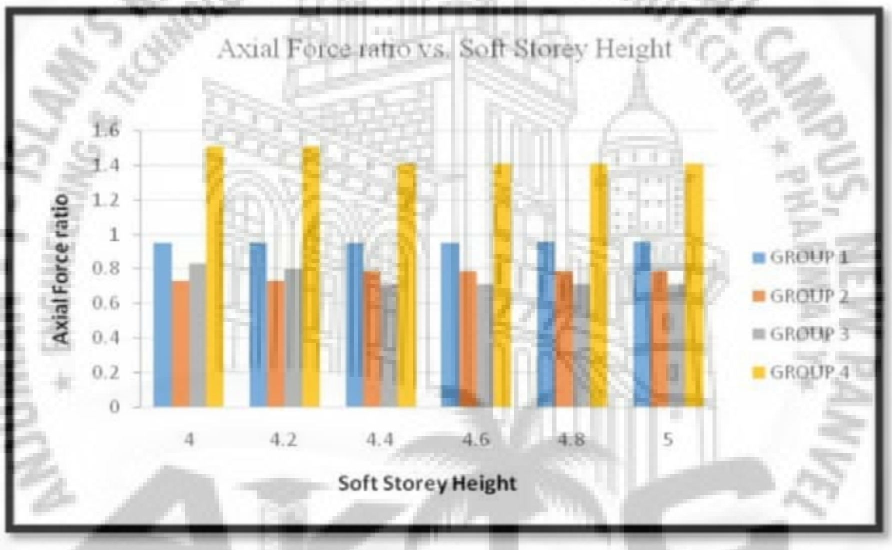

**Graph 4.3: Variations of Bending moment (Y-direct.) for different soft storey heights and column groups**

## Observations and Discussions

Bending moment ratio in Y-direction for all groups is observed to be in increasing trend. And It is maximum for soft storey height (4.8-5m) in group 1 i.e. 1.29.

**Table 4.8: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Tie-Beam.**

| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ |
| 4 | 0.95 | 0.69 | 0.92 | 0.73 | 0.26 | 0.69 | 0.83 | 0.05 | 0.78 | 1.51 | 0.372 | 0.147 |
| 4.2 | 0.95 | 0.68 | 0.9 | 0.73 | 0.04 | 0.69 | 0.8 | 0.05 | 0.8 | 1.51 | 0.35 | 0.147 |
| 4.4 | 0.95 | 0.66 | 0.87 | 0.79 | 0.04 | 0.72 | 0.71 | 0.27 | 0.71 | 1.41 | 0.3 | 0.11 |
| 4.6 | 0.95 | 0.62 | 0.86 | 0.79 | 0.04 | 0.71 | 0.71 | 0.26 | 0.71 | 1.41 | 0.29 | 0.11 |
| 4.8 | 0.96 | 0.6 | 0.85 | 0.79 | 0.06 | 0.7 | 0.71 | 0.24 | 0.71 | 1.41 | 0.27 | 0.11 |
| 5 | 0.96 | 0.57 | 0.85 | 0.79 | 0.06 | 0.79 | 0.71 | 0.23 | 0.71 | 1.41 | 0.26 | 0.11 |




**Graph.4.4: Variations of Axial Force for different soft storey heights and column groups**

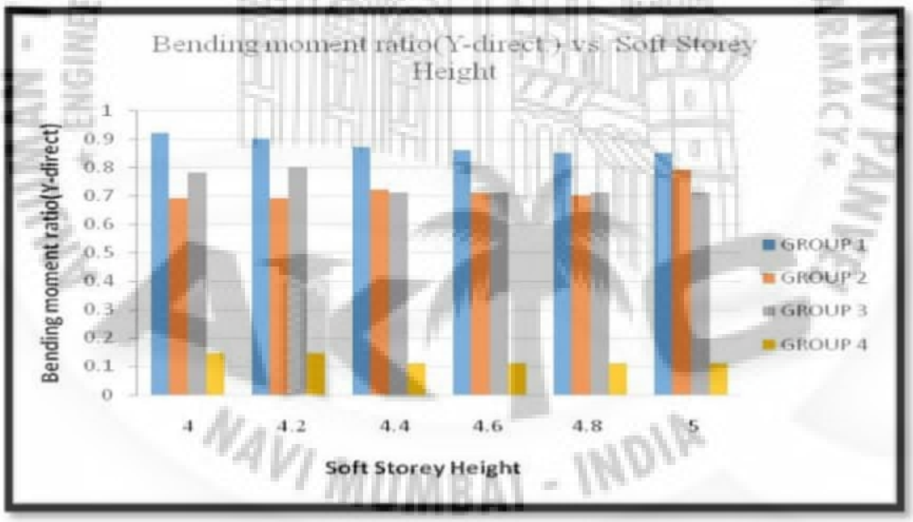
## Observations and Discussions

Axial Compressive Force ratio in group 1and 2 increases as bottom storey height increases, whereas it is observed to be decreasing for group 3 and 4. Axial Compressive Force ratio is maximum in group 4 at 4m height i.e1.51.


**Graph 4.5: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

## Observations and Discussions

Bending Moment ratio in X-direction for group 1and 4 is observed to be decreasing and it is maximum for group 1 at 4m storey height i.e 0.69




**Graph 4.6: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

## Observations and Discussions

Bending Moment ratio in Y-direction, intensively reduces for group 4 as compared to all other groups. And it is observed to be maximum in group 4 at 4m soft storey height i.e 0.92

**Table 4.9: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Bracings.**

| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ |
| 4 | 0.87 | 0.062 | 0.87 | 0.8 | 0.01 | 0.8 | 0.715 | 0.006 | 0.715 | 1.41 | 0.34 | 0.001 |
| 4.2 | 0.87 | 0.063 | 0.87 | 0.8 | 0.01 | 0.8 | 0.715 | 0.006 | 0.715 | 1.41 | 0.32 | 0.001 |
| 4.4 | 0.87 | 0.04 | 0.87 | 0.7 | 0.05 | 0.7 | 0.81 | 0.04 | 0.81 | 1.41 | 0.3 | 0.001 |
| 4.6 | 0.87 | 0.035 | 0.87 | 0.69 | 0.04 | 0.69 | 0.81 | 0.04 | 0.81 | 1.4 | 0.28 | 0.002 |
| 4.8 | 0.87 | 0.034 | 0.87 | 0.69 | 0.03 | 0.69 | 0.81 | 0.03 | 0.81 | 1.4 | 0.27 | 0.002 |
| 5 | 0.87 | 0.022 | 0.87 | 0.68 | 0.03 | 0.68 | 0.81 | 0.03 | 0.81 | 1.4 | 0.26 | 0.003 |


**Graph 4.7: Variations of Axial Force for different soft storey heights and column groups**

**Observations and Discussions**

Axial Compressive Force ratio in group 4 remains constant for bottom storey heights 4 to 4.4 m i.e. 1.41 and it decreases forremaining height from (4.6-5m) i.e 1.4.


**Graph 4.8: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

Bending Moment ratios in X-direction for group 4 is maximum for soft storey height of 4m and it is observed to be 0.34 and for rest soft storey height (4.2-5m) the trend is decreasing in the range (0.34 - 0.26).


**Graph 4.9: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

In group 1, Bending Moment ratios in Y-direction remains same for all soft storey heights it is observed to be 0.87 In group 2, the tend is observed to be decreasing as soft storey height increases in the range (0.8 – 0.68).In group 3 and group 4 there is an increasing trend observed as soft storey height increases in the range (0.715 – 0.81) and (0.001 – 0.003). Thereby we can say that group 4 columns are not affected.

The above data is represented for time period considered for seismic analysis as per Program Calculated for Soil type I (Hard soil).

## B) CONSIDERING TIME PERIOD FOR SEISMIC ANALYSIS AS PER CODAL PROVISIONS FOR SOIL TYPE I

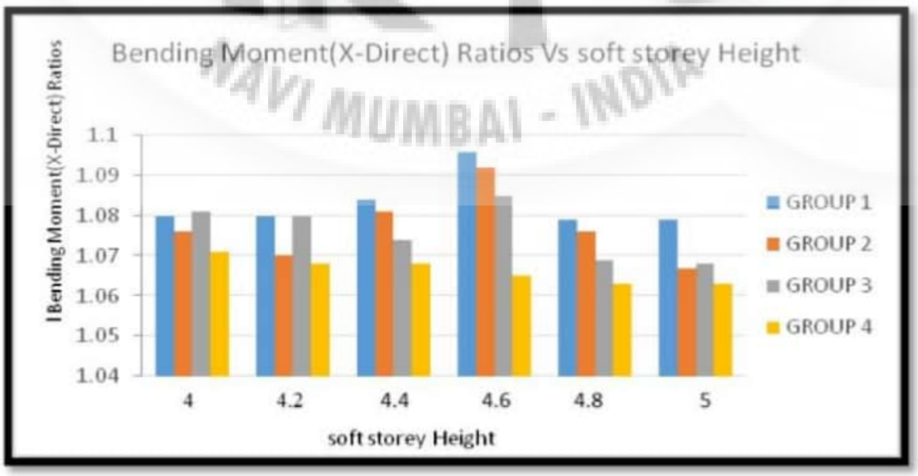
**Table 4.10: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Masonry Infill.**

| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ |
| 4 | 1.31 | 1.08 | 1.31 | 1.212 | 1.076 | 1.2 | 1.21 | 1.081 | 1.21 | 1.096 | 1.071 | 1.096 |
| 4.2 | 1.31 | 1.08 | 1.318 | 1.212 | 1.07 | 1.2 | 1.21 | 1.08 | 1.21 | 1.096 | 1.068 | 1.096 |
| 4.4 | 1.32 | 1.084 | 1.32 | 1.2 | 1.081 | 1.26 | 1.222 | 1.074 | 1.222 | 1.096 | 1.068 | 1.095 |
| 4.6 | 1.32 | 1.096 | 1.345 | 1.203 | 1.092 | 1.22 | 1.235 | 1.085 | 1.253 | 1.096 | 1.065 | 1.111 |
| 4.8 | 1.32 | 1.079 | 1.324 | 1.204 | 1.076 | 1.187 | 1.247 | 1.069 | 1.249 | 1.094 | 1.063 | 1.094 |
| 5 | 1.32 | 1.079 | 1.32 | 1.218 | 1.067 | 1.182 | 1.237 | 1.068 | 1.237 | 1.094 | 1.063 | 1.094 |


**Graph 4.10: Variations of Axial Force for different soft storey heights and column groups**

### Observations and Discussions

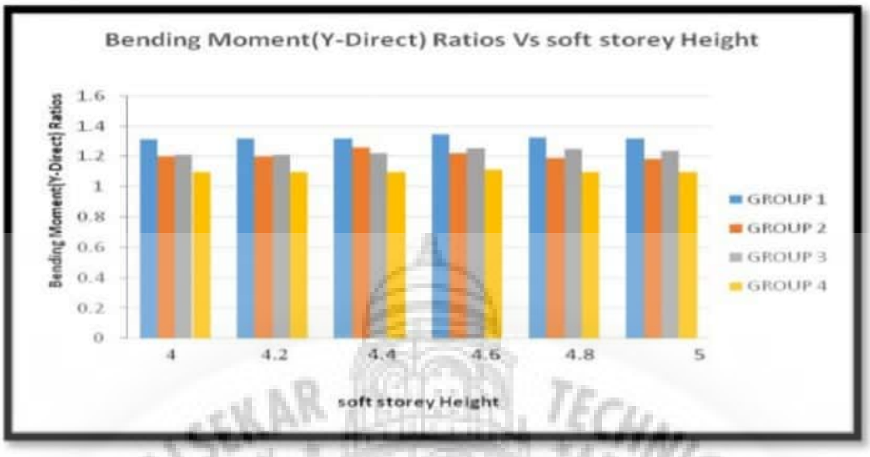
Axial compressive force ratio For group 1 and group 4, there is increasing and decreasing trend respectively in the range (1.31 – 1.32) and (1.096 – 1.094). For group 3 there is increasing trend (1.21 – 1.237). RPu is maximum for group 1 i.e 1.32.




**Graph 4.11: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

For group 1 Bending moment ratios in X-direction increases from (4 – 4.6m) and decreases there after. The maximum observed value is 1.096.



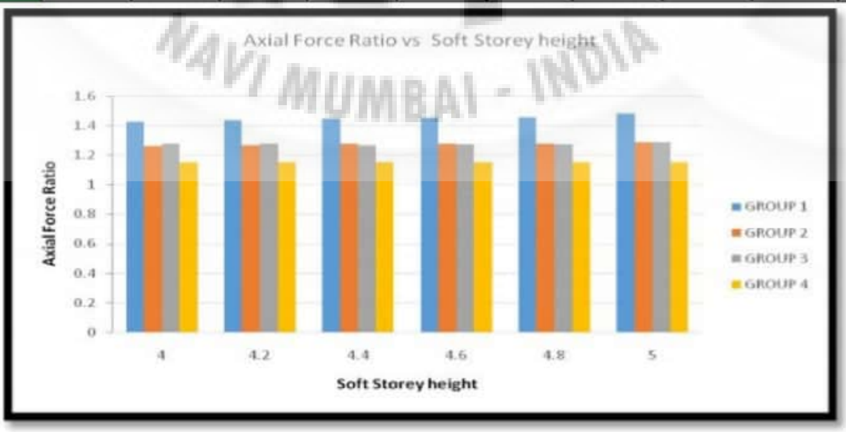

**Graph 4.12: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

Bending moment ratios in Y-direction For group 1 Bending moment ratios in Y-direction increases from (4 – 4.6m) and decreases there after. The maximum observed value is 1.345.

**Table 4.11: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Tie-Beam.**

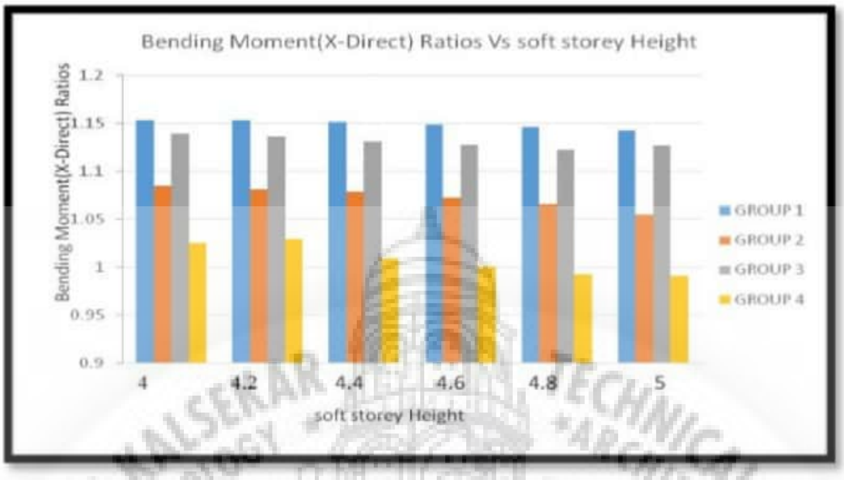
| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ |
| 4 | 1.43 | 1.153 | 1.32 | 1.266 | 1.085 | 1.172 | 1.282 | 1.139 | 1.18 | 1.154 | 1.025 | 1.064 |
| 4.2 | 1.44 | 1.153 | 1.315 | 1.268 | 1.081 | 1.15 | 1.282 | 1.137 | 1.17 | 1.153 | 1.029 | 1.064 |
| 4.4 | 1.448 | 1.152 | 1.3 | 1.282 | 1.079 | 1.139 | 1.271 | 1.131 | 1.143 | 1.153 | 1.009 | 1.054 |
| 4.6 | 1.455 | 1.149 | 1.29 | 1.282 | 1.072 | 1.137 | 1.274 | 1.128 | 1.13 | 1.153 | 1 | 1.059 |
| 4.8 | 1.462 | 1.146 | 1.27 | 1.282 | 1.065 | 1.12 | 1.276 | 1.123 | 1.116 | 1.153 | 0.992 | 1.044 |
| 5 | 1.485 | 1.143 | 1.27 | 1.29 | 1.055 | 1.114 | 1.29 | 1.127 | 1.113 | 1.153 | 0.991 | 1.03 |




**Graph 4.13: Variations of Axial Force for different soft storey heights and column groups**

**Observations and Discussions**

Axial compressive force ratio in group 1 increases from soft-storey height ( 4 – 5m) and is maximum for 5m i.e 1.485




**Graph 4.14: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

Bending moment ratios in X-direction is in decreasing trend in all groups and for all bottom storey heights and it is maximum in group 1 at 4m height which is observed to be 1.153.


**Graph 4.15: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

Bending moment ratios in Y-direction is decreasing trend for all groups and for all bottom storey heights and it is maximum in group 1 at 4m height which is observed to be 1.32.

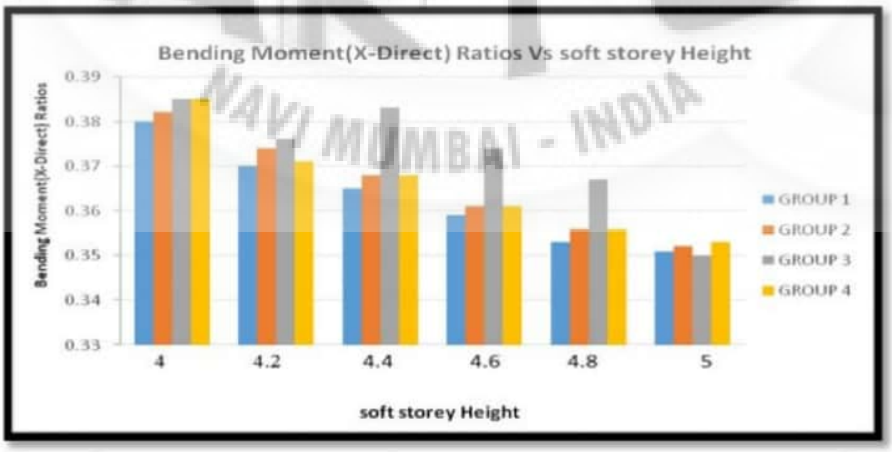
**Table 4.12: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Bracings.**

| STOREY | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| HEIGHT | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ | RPu | $RMu_2$ | $RMu_3$ |
| 4 | 1.259 | 0.38 | 1.25 | 1.59 | 0.382 | 1.59 | 1.601 | 0.385 | 1.601 | 1.092 | 0.385 | 1.09 |
| 4.2 | 1.255 | 0.37 | 1.25 | 1.604 | 0.374 | 1.604 | 1.616 | 0.376 | 1.616 | 1.091 | 0.371 | 1.09 |
| 4.4 | 1.252 | 0.365 | 1.25 | 1.081 | 0.368 | 1.081 | 1.626 | 0.383 | 1.626 | 1.09 | 0.368 | 1.09 |
| 4.6 | 1.249 | 0.359 | 1.249 | 1.072 | 0.361 | 1.072 | 1.634 | 0.374 | 1.635 | 1.09 | 0.361 | 1.096 |
| 4.8 | 1.247 | 0.353 | 1.247 | 1.072 | 0.356 | 1.064 | 1.643 | 0.367 | 1.643 | 1.089 | 0.356 | 1.089 |
| 5 | 1.247 | 0.351 | 1.247 | 1.049 | 0.352 | 1.049 | 1.64 | 0.35 | 1.64 | 1.088 | 0.353 | 1.088 |


**Graph 4.16: Variations of Axial Force for different soft storey heights and column groups**

**Observations and Discussions**

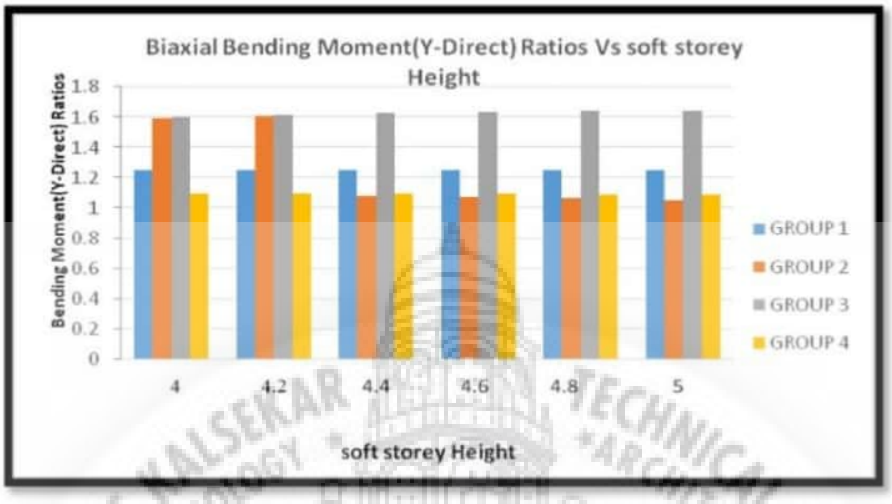
Axial Compressive Force ratios in group 1, group 2 and group 4, there is decreasing trend and for group 3 it is increasing in the range (1.601 – 1.64) maximum value is observed for group 3 at 4.8m height i.e 1.643




**Graph 4.17: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

Bending Moment ratios in X direction is observed maximum for group 3 at 4m and 4.8m i.e 0.385




**Graph 4.18: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

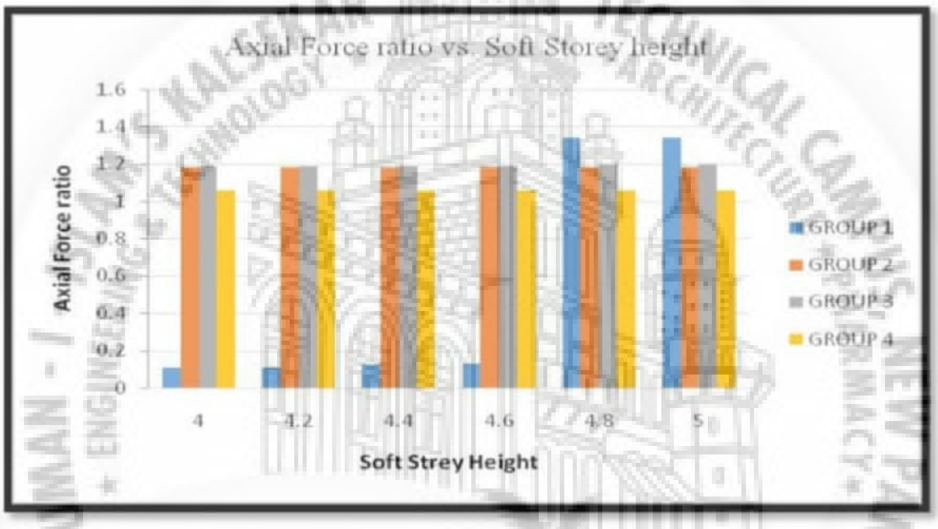
**Observations and Discussions**

Bending Moment ratios in Y-direction is maximum is observed maximum for group 3 at 4m and 4.8m i.e 1.643

## C) CONSIDERING TIME PERIOD FOR SEISMIC ANALYSIS USING SOFTWARE (PROGRAM CALCULATED) FOR SOIL TYPE III

**Table 4.13: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with brick Masonry Infill.**

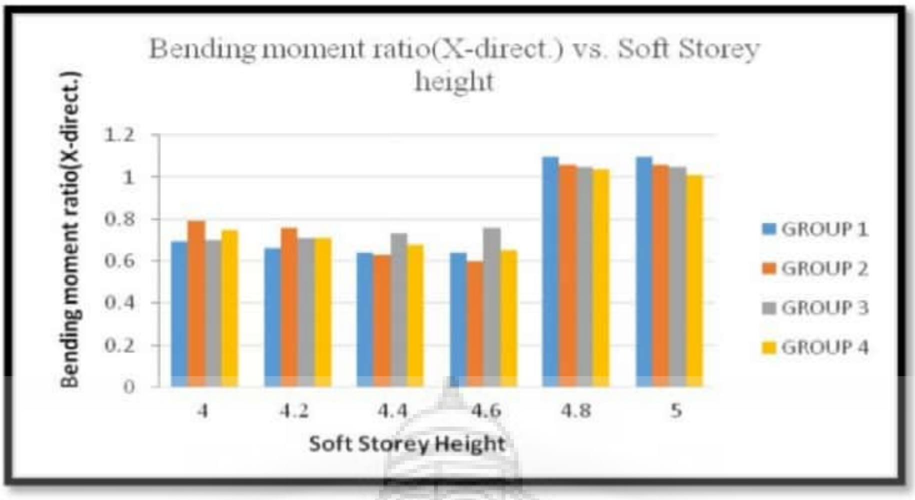
| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 |
| 4 | 0.105 | 0.7 | 0.202 | 1.18 | 0.79 | 1.18 | 1.19 | 0.7 | 1.19 | 1.06 | 0.75 | 1.06 |
| 4.2 | 0.11 | 0.66 | 0.203 | 1.18 | 0.76 | 1.18 | 1.19 | 0.71 | 1.19 | 1.06 | 0.71 | 1.06 |
| 4.4 | 0.126 | 0.64 | 0.204 | 1.18 | 0.63 | 1.18 | 1.19 | 0.73 | 1.19 | 1.06 | 0.68 | 1.06 |
| 4.6 | 0.13 | 0.64 | 0.204 | 1.18 | 0.6 | 1.18 | 1.19 | 0.76 | 1.19 | 1.06 | 0.65 | 1.06 |
| 4.8 | 1.34 | 1.1 | 1.34 | 1.18 | 1.06 | 1.18 | 1.2 | 1.05 | 1.2 | 1.06 | 1.04 | 1.06 |
| 5 | 1.34 | 1.1 | 1.34 | 1.18 | 1.06 | 1.18 | 1.2 | 1.05 | 1.2 | 1.06 | 1.01 | 1.06 |




**Graph 4.19: Variations of Axial Force for different soft storey heights and column groups**

### Observations and Discussions

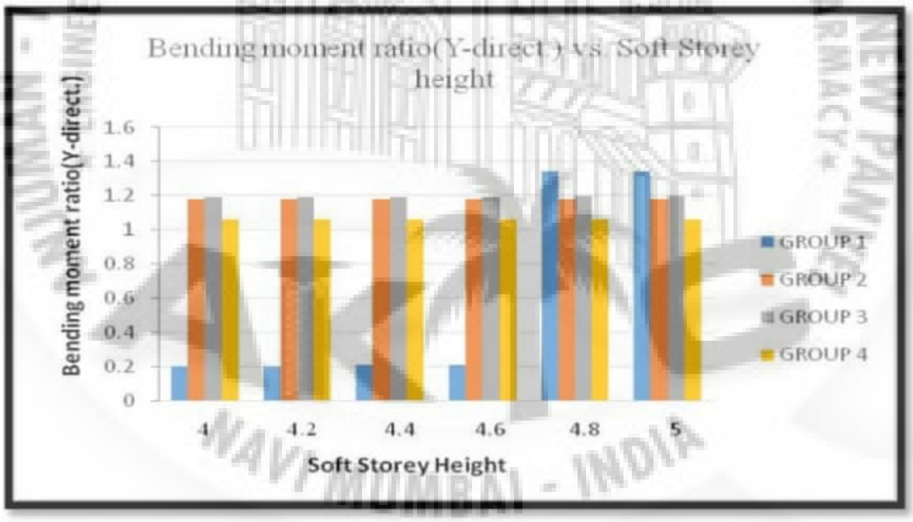
In group 1for soft storey height(4 –4.6) it is observed that the Axial Force ratios increases in the range (0.105 – 0.13) and suddenly increases for soft storey height (4.8-5m) in the range 1.34,it remains constant for group 2 and group 4 as 1.18 and 1.06.




**Graph 4.20: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

### Observations and Discussions

For group 1,2 and 4 it is observed that for soft storey height (4- 4.6) the Bending Moment in X-direction decreases in the range (0.7 – 0.64), (0.79 -0.6) and (0.75-0.65) and for soft storey (4.8-5) it increases suddenly in all groups. It is maximum for group 1 i.e 1.1.



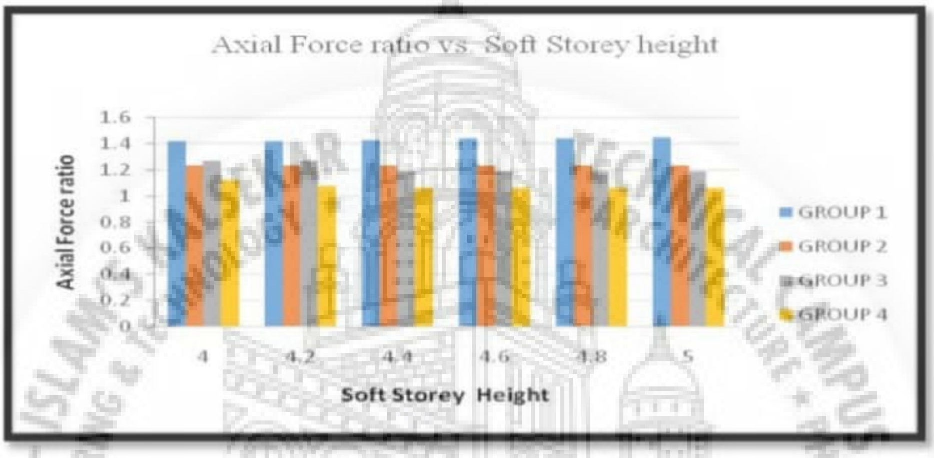

**Graph 4.21: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

### Observations and Discussions

For group 1 and 3 it is observed that as soft storey height (4 – 4.6) the Bending Moment ratios in Y-direction increases in the range (0.202 – 1.34) and (1.19 – 1.2) and it is constant for group 2 and 4 as 1.18 and 1.06.it is maximum for group 1 i.e 1.34.

**Table 4.14: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Tie-Beam.**

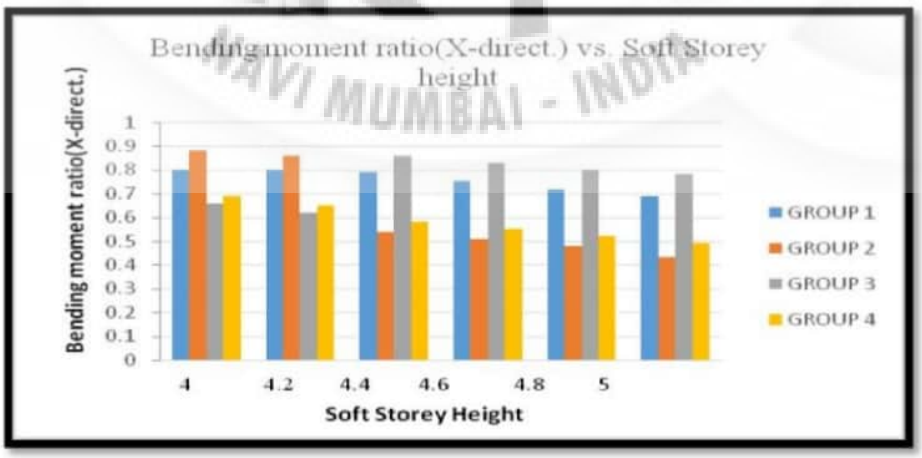
| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 |
| 4 | 1.42 | 0.8 | 1.51 | 1.23 | 0.88 | 1.15 | 1.27 | 0.66 | 1.19 | 1.12 | 0.69 | 1.08 |
| 4.2 | 1.42 | 0.8 | 1.5 | 1.23 | 0.86 | 1.14 | 1.27 | 0.62 | 1.19 | 1.08 | 0.65 | 1.08 |
| 4.4 | 1.43 | 0.79 | 1.49 | 1.23 | 0.54 | 1.13 | 1.19 | 0.86 | 1.19 | 1.06 | 0.58 | 1.06 |
| 4.6 | 1.44 | 0.75 | 1.3 | 1.23 | 0.51 | 1.11 | 1.19 | 0.83 | 1.19 | 1.06 | 0.55 | 1.06 |
| 4.8 | 1.44 | 0.72 | 1.29 | 1.23 | 0.48 | 1.09 | 1.19 | 0.8 | 1.19 | 1.06 | 0.52 | 1.06 |
| 5 | 1.45 | 0.69 | 1.28 | 1.23 | 0.43 | 1.09 | 1.19 | 0.78 | 1.2 | 1.06 | 0.49 | 1.06 |




**Graph 4.22: Variations of Axial Force for different soft storey heights and column groups**

**Observations and Discussions**

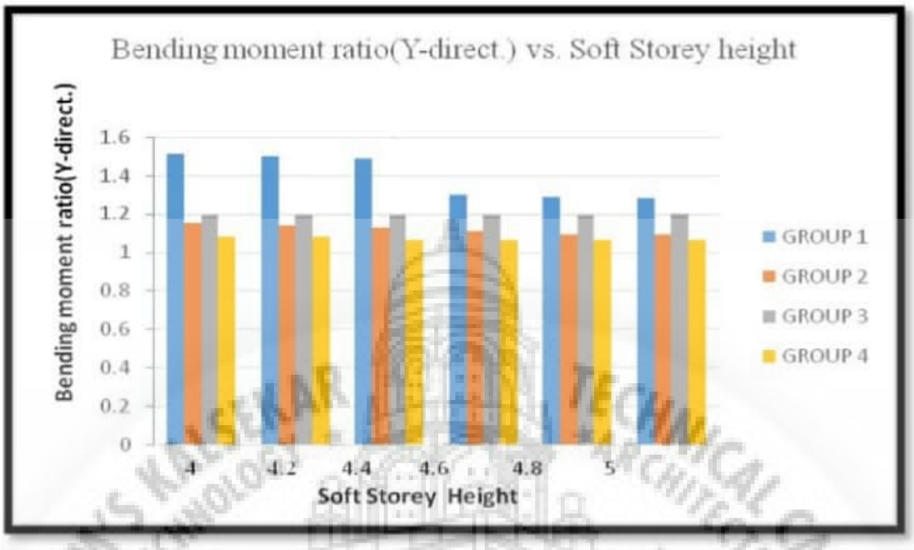
For group 3 and group 4,it is observed that the Axial Force ratios decreases as the soft storey height increases in the range (1.27 – 1.19) and (1.12 – 1.06) for group 2 it remains constant as 1.23, for group 1 it increases in the range (1.42 – 1.45).Axial Force ratio is maximum for group 1 i.e 1.45.




**Graph 4.23: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

### Observations and Discussions

For group 1,2 and 4 it is observed that as soft storey height increases the Bending Moment ratio in X-direction decreases in the range (0.8 – 0.69), (0.88 -0.43) and (0.69-0.49).



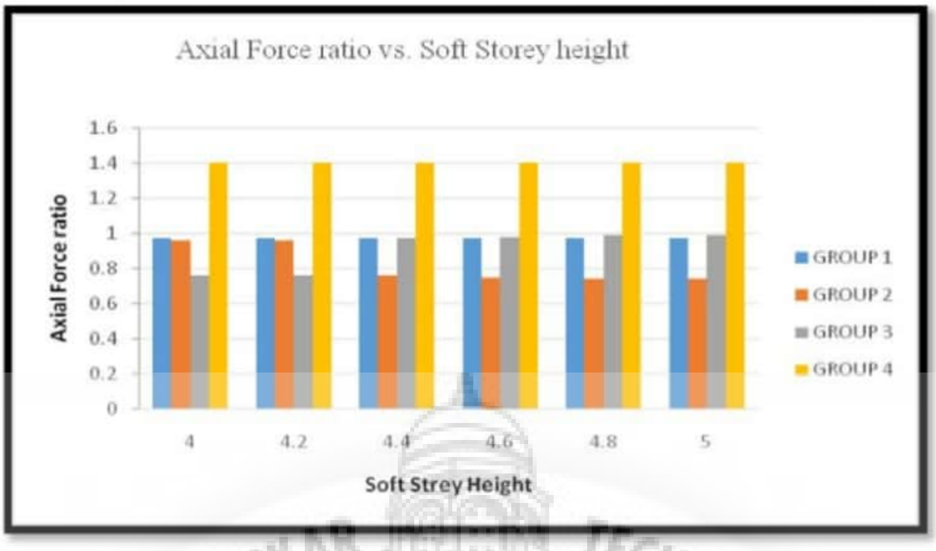

**Graph 4.24: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

### Observations and Discussions

For group 1,2 and 4 it is observed that as soft storey height increases the Bending Moment ratio in Y-direction decreases in the range (1.51 – 1.28), (1.15 -1.09) and (1.08 – 1.06). It is maximum for group 1 i.e 1.51.

**Table 4.15: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Bracings.**

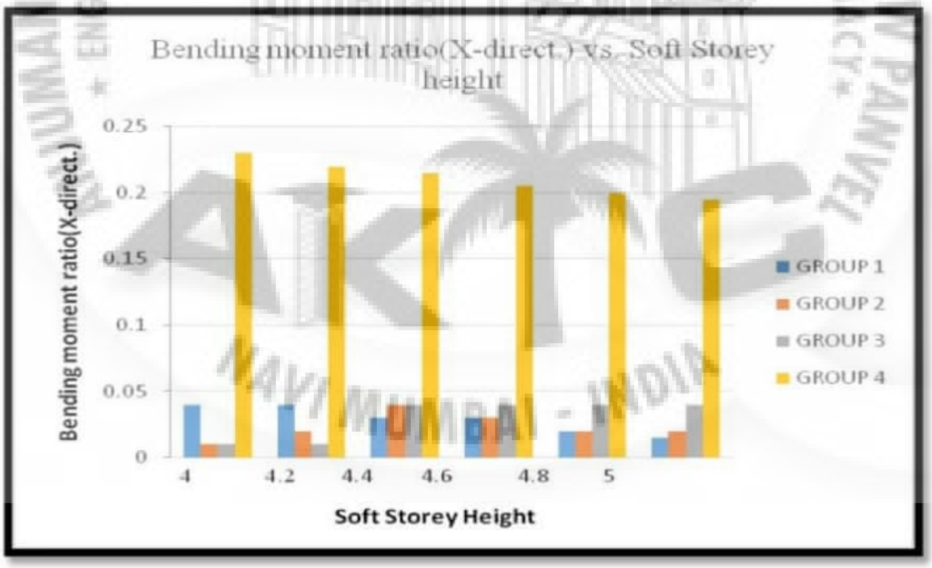
| STOREY | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| HEIGHT | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 |
| 4 | 0.97 | 0.04 | 0.97 | 0.96 | 0.01 | 0.95 | 0.76 | 0.01 | 0.77 | 1.4 | 0.23 | 0.001 |
| 4.2 | 0.97 | 0.04 | 0.97 | 0.96 | 0.02 | 0.96 | 0.76 | 0.01 | 0.77 | 1.4 | 0.22 | 0.001 |
| 4.4 | 0.97 | 0.03 | 0.97 | 0.76 | 0.04 | 0.76 | 0.97 | 0.04 | 0.97 | 1.4 | 0.215 | 0.001 |
| 4.6 | 0.97 | 0.03 | 0.97 | 0.75 | 0.03 | 0.75 | 0.98 | 0.04 | 0.98 | 1.4 | 0.205 | 0.002 |
| 4.8 | 0.97 | 0.02 | 0.97 | 0.74 | 0.02 | 0.74 | 0.99 | 0.04 | 0.99 | 1.4 | 0.2 | 0.002 |
| 5 | 0.97 | 0.015 | 0.97 | 0.74 | 0.02 | 0.74 | 0.99 | 0.04 | 0.99 | 1.4 | 0.195 | 0.003 |




**Graph 4.25: Variations of Axial Force for different soft storey heights and column groups**

## Observations and Discussions

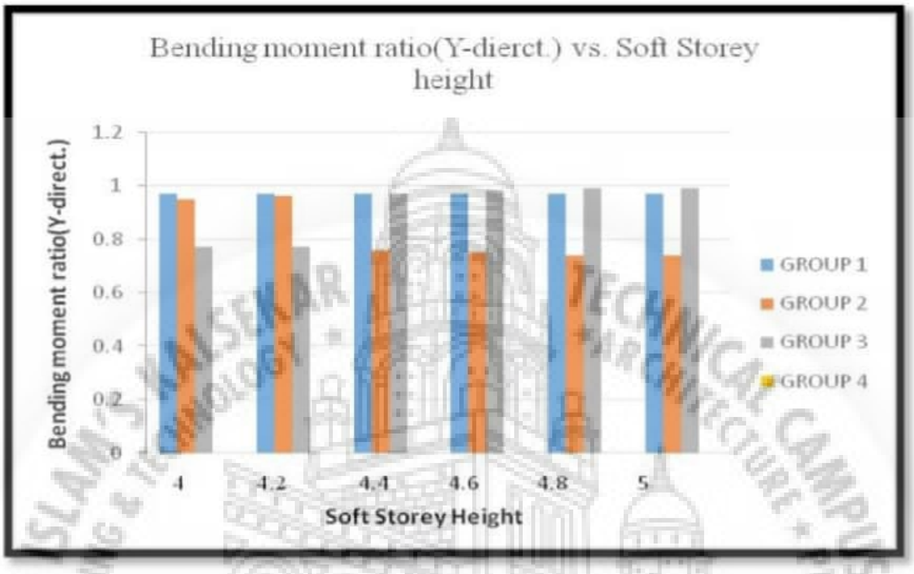
For group 1 and group 4,It is observed that Axial Force ratio remains constant i.e (0.97 and 1.4), For group 2 it is in decreasing trend from (0.96 – 0.74).For group 3 it is in increasing trend from (0.76 – 0.99), it is maximum for group 4 i.e 1.4




**Graph 4.26: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

For group 1 and group 4,It is observed that Bending Moment ratio in X-direction decreases as soft storey height increases (0.04 - 0.015) and (0.23 - 0.195),For group 2 and group 3 it is in increasing trend from (0.01– 0.02) and (0.01 – 0.04) it maximum for group 4 i.e,0.23




**Graph 4.27: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

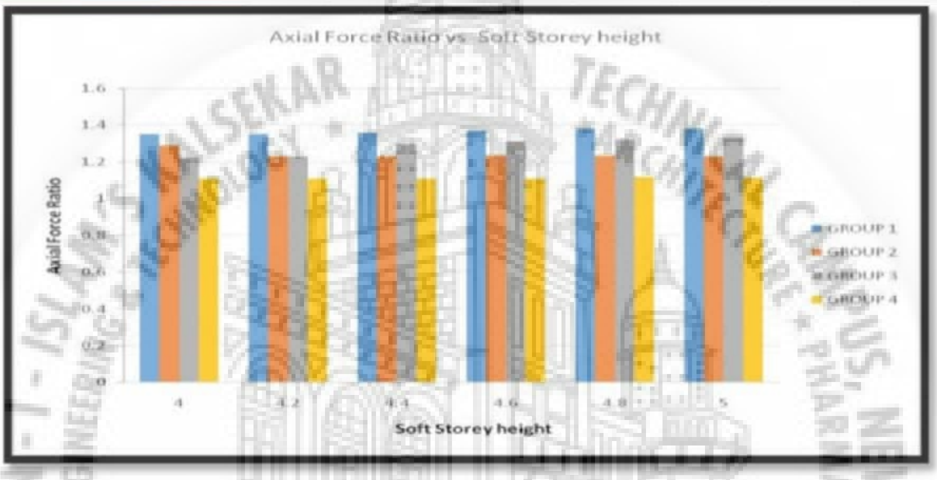
**Observations and Discussions**

For group 1Bending Moment ratio in Y-direction is constant i.e 0.97, for group 2 there is decreasing trend (0.95 – 0.74) for group 3 and group 4 there is increasing trend (0.77 – 0.99) and (0.001 – 0.003), it is maximum for group 1 i.e 0.97

### D) CONSIDERING TIME PERIOD FOR SEISMIC ANALYSIS AS PER CODAL PROVISIONS FOR SOIL TYPE III

**Table 4.16: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Masonry Infill.**

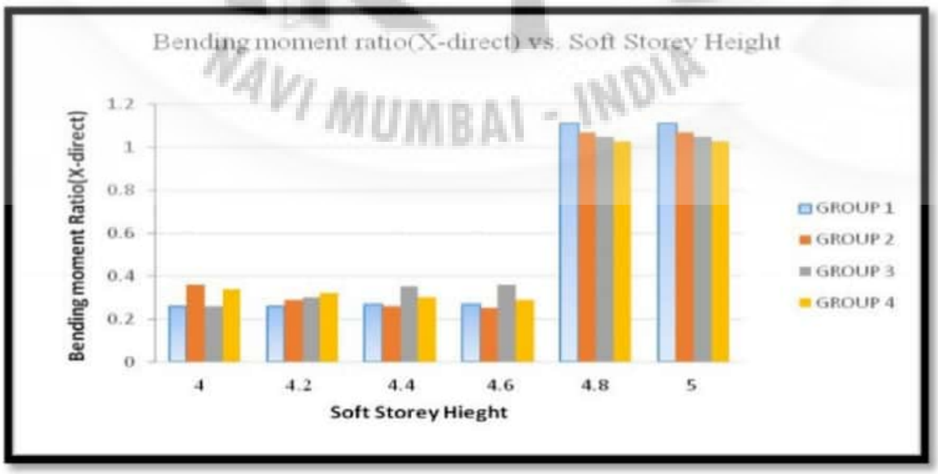
| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 |
| 4 | 1.35 | 1.05 | 1.35 | 1.289 | 1.04 | 1.272 | 1.22 | 1.053 | 1.22 | 1.11 | 1.043 | 1.11 |
| 4.2 | 1.35 | 1.058 | 1.35 | 1.23 | 1.047 | 1.262 | 1.228 | 1.053 | 1.228 | 1.11 | 1.043 | 1.11 |
| 4.4 | 1.359 | 1.065 | 1.359 | 1.23 | 1.062 | 1.238 | 1.299 | 1.054 | 1.3 | 1.11 | 1.049 | 1.114 |
| 4.6 | 1.369 | 1.071 | 1.369 | 1.232 | 1.068 | 1.232 | 1.309 | 1.061 | 1.309 | 1.11 | 1.055 | 1.114 |
| 4.8 | 1.378 | 1.077 | 1.378 | 1.233 | 1.071 | 1.222 | 1.32 | 1.067 | 1.32 | 1.114 | 1.061 | 1.114 |
| 5 | 1.38 | 1.079 | 1.38 | 1.23 | 1.074 | 1.219 | 1.328 | 1.063 | 1.328 | 1.113 | 1.057 | 1.113 |




**Graph 4.28: Variations of Axial Force for different soft storey heights and column groups**

**Observations and Discussions**

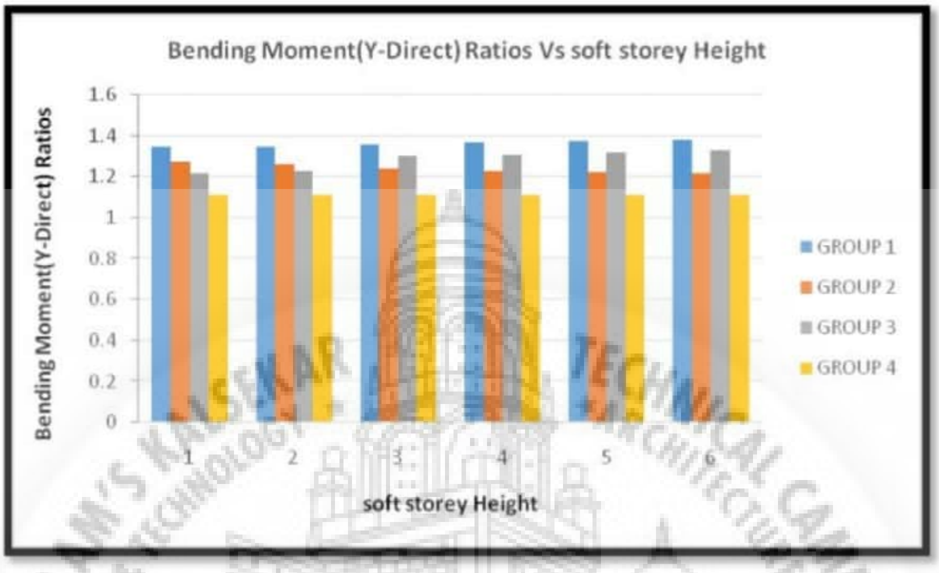
For group 1 Axial Force is observed to be maximum at 5m height i.e 1.38.and For group 2 Axial Force is 1.29 at 4m and constant as 1.32 for remaining height (4.2 – 5m).




**Graph 4.29: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

For group 1 Bending Moment in X-direction is observed to be maximum at 5m height i.e,1.079.



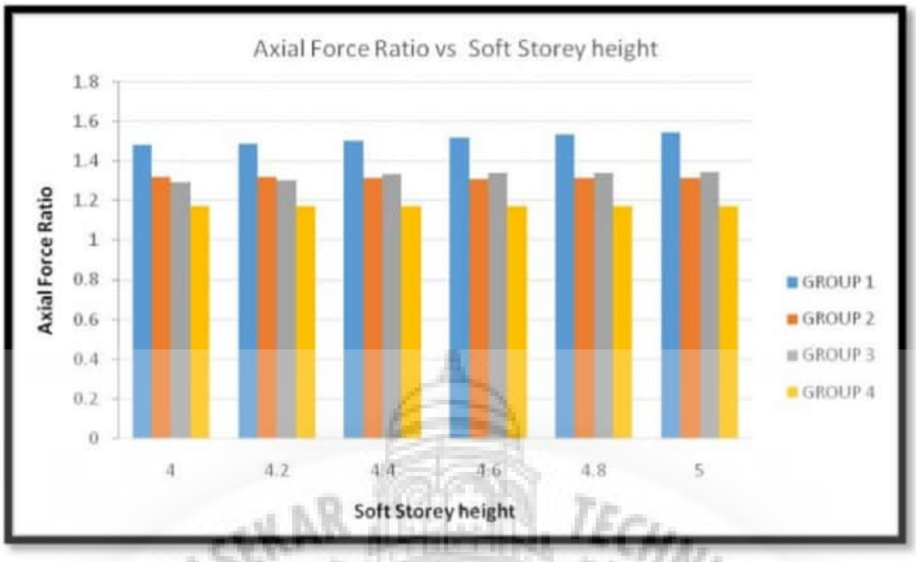

**Graph 4.30: Variations of Bending moment (Y-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

For group 1 Bending Moment in Y-direction is observed to be maximum at 5m height i.e,1.38.

**Table 4.17:Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Tie-Beam.**

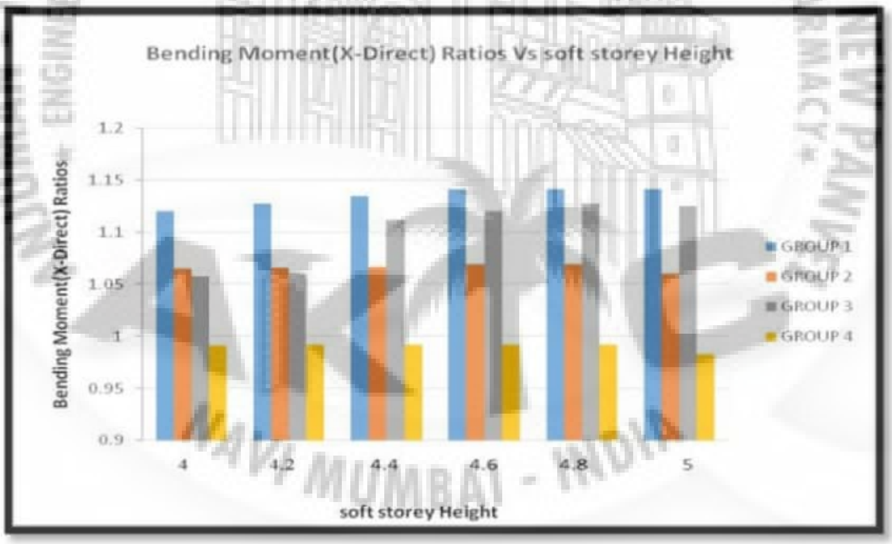
| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 |
| 4 | 1.482 | 1.12 | 1.35 | 1.32 | 1.065 | 1.184 | 1.29 | 1.058 | 1.29 | 1.17 | 0.991 | 1.096 |
| 4.2 | 1.484 | 1.128 | 1.35 | 1.32 | 1.066 | 1.182 | 1.301 | 1.06 | 1.2 | 1.17 | 0.992 | 1.094 |
| 4.4 | 1.5 | 1.135 | 1.349 | 1.315 | 1.067 | 1.174 | 1.332 | 1.112 | 1.199 | 1.173 | 0.992 | 1.093 |
| 4.6 | 1.517 | 1.142 | 1.345 | 1.31 | 1.069 | 1.162 | 1.337 | 1.12 | 1.186 | 1.173 | 0.992 | 1.078 |
| 4.8 | 1.532 | 1.142 | 1.34 | 1.313 | 1.069 | 1.149 | 1.341 | 1.128 | 1.173 | 1.173 | 0.992 | 1.063 |
| 5 | 1.541 | 1.142 | 1.329 | 1.312 | 1.06 | 1.131 | 1.344 | 1.125 | 1.159 | 1.173 | 0.983 | 1.048 |




**Graph 4.31: Variations of Axial Force for different soft storey heights and column groups**

**Observations and Discussions**

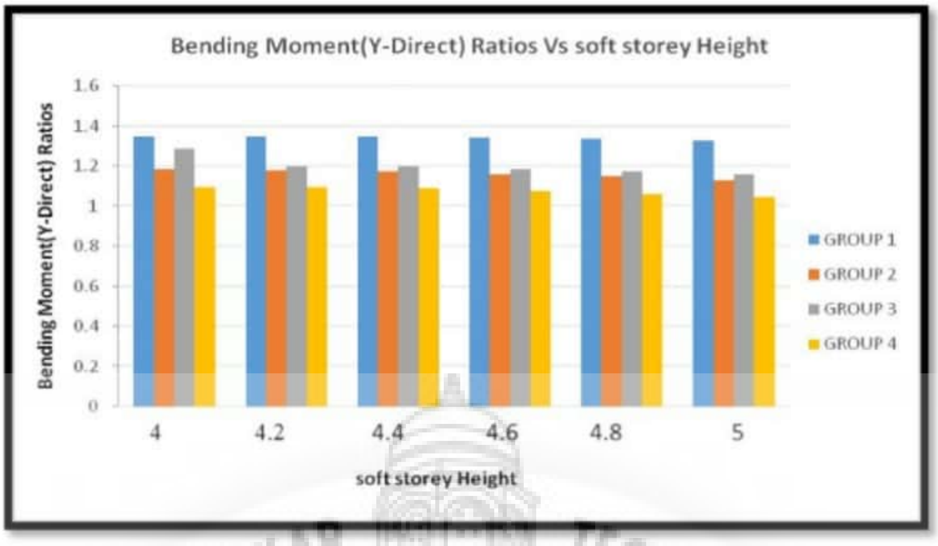
For group 1Axial Force ratio is observed to be maximum at 5m height i.e,1.541




**Graph 4.32: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

**Observations and Discussions**

For group 1 Bending Moment in X-direction is observed to be maximum at 5m height i.e,1.142



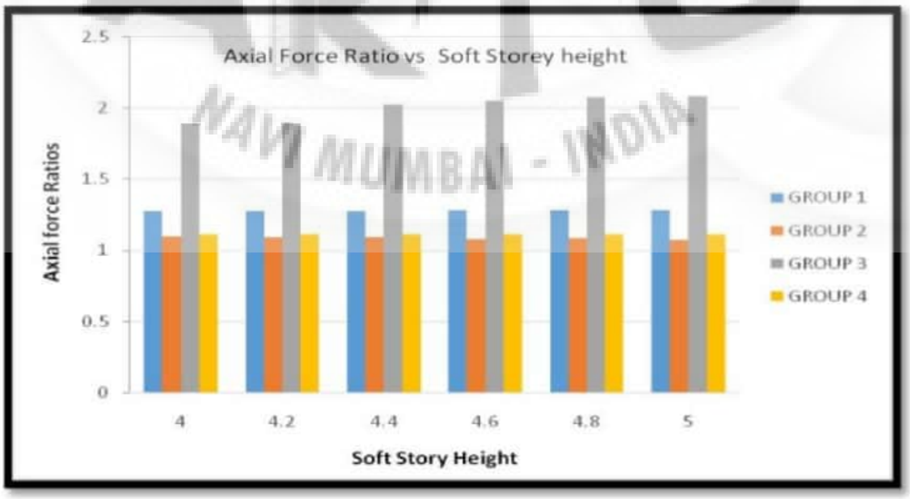

**Graph 4.33: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

## Observations and Discussions

For group 1Bending Moment in Y-direction is observed to be maximum at 4m height i.e 1.35.

**Table 4.18: Ratios obtained for different Groups and Soft Storey heights are summarized for Frame with Bracings.**

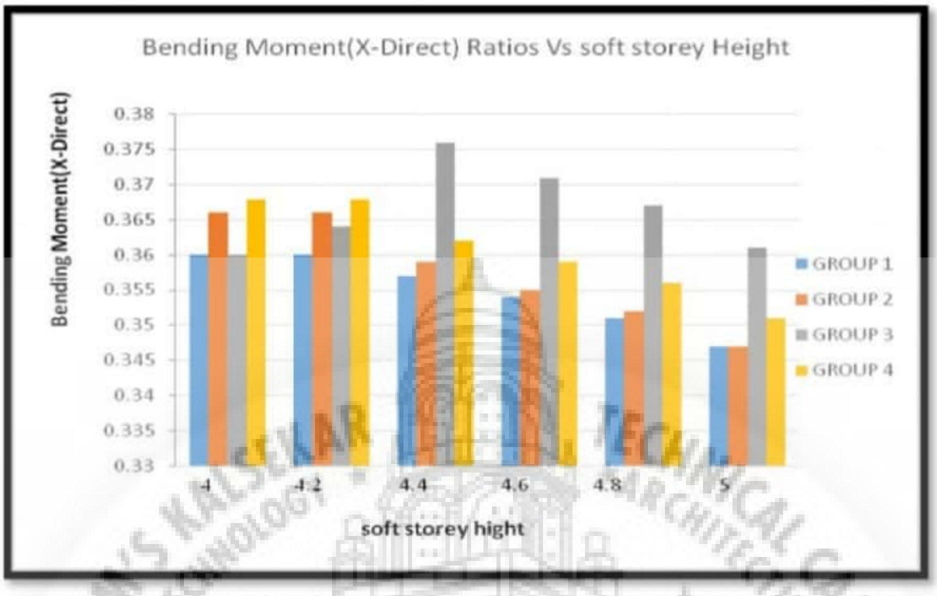
| STOREY HEIGHT | GROUP 1 | | | GROUP 2 | | | GROUP 3 | | | GROUP 4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 | Rpu | RMu2 | RMu3 |
| 4 | 1.272 | 0.36 | 1.27 | 1.09 | 0.366 | 1.096 | 1.89 | 0.36 | 1.89 | 1.11 | 0.368 | 1.109 |
| 4.2 | 1.272 | 0.36 | 1.27 | 1.09 | 0.366 | 1.095 | 1.89 | 0.364 | 1.89 | 1.11 | 0.368 | 1.109 |
| 4.4 | 1.272 | 0.36 | 1.27 | 1.09 | 0.359 | 1.092 | 2.02 | 0.376 | 2.024 | 1.11 | 0.362 | 1.108 |
| 4.6 | 1.277 | 0.35 | 1.28 | 1.08 | 0.355 | 1.078 | 2.05 | 0.371 | 2.05 | 1.11 | 0.359 | 1.107 |
| 4.8 | 1.281 | 0.35 | 1.28 | 1.08 | 0.352 | 1.08 | 2.08 | 0.367 | 2.07 | 1.11 | 0.356 | 1.107 |
| 5 | 1.281 | 0.35 | 1.29 | 1.07 | 0.347 | 1.071 | 2.08 | 0.361 | 2.08 | 1.11 | 0.351 | 1.106 |




**Graph 4.34: Variations of Axial Force for different soft storey heights and column groups**

### Observations and Discussions

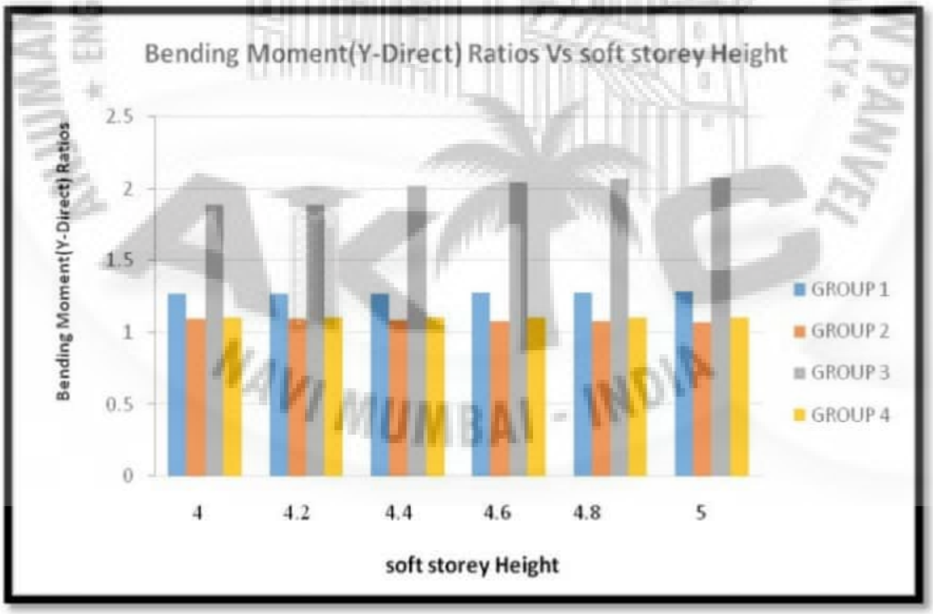
For group 3 Axial Force is observed to be maximum at 5m height i.e 2.08.




**Graph 4.35: Variations of Bending moment(X-direct.) for different soft storey heights and column groups**

### Observations and Discussions

For group 3 Bending Moment in X-direction is observed to be maximum at 4.4m height i.e 0.376.




**Graph 4.36: Variations of Bending moment(Y-direct.) for different soft storey heights and column groups**

### Observations and Discussion

For group 3 Bending Moment in Y-direction is observed to be maximum at 5m height i.e, 2.08

# Chapter Five
# CONCLUSIONS AND SCOPE FOR FUTURE WORK

# CHAPTER FIVE

# CONCLUSION AND SCOPE FOR FUTURE WORK

## 5.1 CONCLUSIONS

The seismic analysis of RC frames is done by considering strength and stiffness effect of infill walls. The equivalent diagonal strut method is used for this purpose. Following prominent conclusions are drawn from parametric investigations.

1. Few masonry infill walls are provided along periphery in the ground storey to reduce the soft storey effects shows and it shows better performance as compared to full open ground storey.
2. Seismic coefficient method using fundamental natural period as specified in IS 1893(Part-I):2002 gives insufficient guidelines for infill effect. As the same empirical relationship is used for infilled frame, frame with Tie-beam and frame with Bracings.
3. It can be concluded that fundamental natural period of bare frame not only depends on building height but also on span length and the stiffness of building which are not quantified in the codal expressions.
4. Based on extensive parametric investigation of space frame, square column is more effective than rectangular column as far as soft storey effect is considered.
5. The Ratio of maximum bending moments and Shear force of the columns for the case of Infilled frame, considered to that of bare frame model varies from column to column. As the multiplication factor 2.5 as suggested by IS 1893(Part-I):2002 is not constant for all soft storey columns. Therefore it is recommended to use the dynamic analysis approach as specified in IS1893 (Part-I):2002.
6. Also this 2.5 multiplication factor is approximate, as it is not distributed in proper manner to the soft storey columns.
7. Out of all models that is Frame with Infill, Frame with Tie-beam and Frame with Bracings the most economical is Braced frame. Bracings are more efficient in carrying moments because the ratio observed is minimum for bracings.
8. To synchronize in order providing Tie-beam is more effective than only infill whereas providing Bracings is most effective than providing Masonary infill and Tie-beam.
9. The multiplication factor observed in the study is represented in below

| Soil Type | Multiplication Factor Observed for Time Period considered for Seismic Analysis as per | |
|---|---|---|
| | PROGRAM CALCULATED | CODAL PROVISIONS |
| I | 1.51 | 1.643 |
| III | 1.53 | 2.05 |

## 5.2 SCOPE FOR FUTURE WORK

a) Dual system can also be used for example by placing shear wall centrally in certain bays will also reduce the soft storey effect.

b) Here, the infill is modeled using Equivalent diagonal strut method. Finite element approach can be used for idealization of infilled frame.

c) In this dissertation, effects of openings are not considered. Soft storey behavior can be checked with openings.

d) Here only two soil conditions are taken into consideration that is Hard and Soft,it can be analysed for Medium soil condition also.

e) Only Static analysis is carried out here. Dynamic analysis can also be carried out.

## REFERENCES

1. Analysis of lateral stiffness for infilled frame with opening, International Journal of advancedtechnology in civil engineering, ISSN:2231-5721 Vol I Issue 3. by A.S. Kasnale& Sanjay Jamkar (2012),
2. Analysis Of Rc Frame With And Without Masonry Infill Wall With Different Stiffness With Outer Central Opening by Mohammad H. Jinya1, V. R. Patel2
3. Earthquake Analysis Of High Rise Building With And Without In Filled Walls by Wakchaure M.R, Ped S. P
4. Earthquake analyais of high rise building with and without infilled walls, IJEIT ISSN:2277-3754 Vol II Issue 2. by Wakchaure M.R. ,Ped S.P (Aug 2012),
5. Earthquake Resistant design of building structures, ISBN No. 978-81-203-2892-1Publisher PHI Learning Private Limited, Textbook.by PankajAgarwal& Manish Shrikhande
6. Effect Of Infill Stiffness On Seismic Performance Of Multi-Storey Rc Framed Buildings In India by Robin Davis1, Praseetha Krishnan1, Devdas Menon2, A. Meher Prasad2
7. Effect Of Brick Infill Panel In Design Of High Rise Building by Sachin R Patel, Sumant B Patel ,p.G.Student, Structural Engineering Department Gujarat Technological University Birla VishvakarmaMahavidhyalaya Engineering College V V Nagar, Gujarat
8. Evaluation Of Seismic Behavior For MultistoriedRc Moment Resisting Frame With Open First Storey by SaurabhSingha*, SaleemAkhtara And GeetaBathama
9. Evaluation Of Magnification Factors For Open Ground Storey Buildings Using Nonlinear Analyses ,The 14thWorld Conference on Earthquake Engineering12-17, Beijing, China by R. Davis1 , D. Menon2 , A. M. Prasad2 ( Oct 2008)
10. Indian Standard Criteria for Earthquake Resistant Design of Structures, Bureau of Indian Standards, . IS : 1893 (2002), New Delhi, India
11. Seismic Response Of Rc Frame Buildings With Soft Storeys by Amit V. KhandveDepartment Of Civil Engineering, Rtm Nagpur University, Nagpur -440009 (Mh)
12. Seismic Analysis Of Rc Frame Structure With And Without Masonry Infill Walls by Haroon Rasheed Tamboli* And Umesh.N.Karadi Department Of Civil