SEISMIC PERFORMANCE ASSESSMENT OF MULTISTOREYED RC SPECIAL MOMENT RESISTING FRAMES BY PUSHOVER ANALYSIS

A Thesis Submitted
in Partial Fulfillment of the Requirements
for the Degree
of

MASTER OF TECHNOLOGY

In

Structural Engineering

By

Aman Ahmed
(University Roll No. 1170444001)

Under the Guidance of

Mr. Mohammad Afaque Khan
(Assistant Prof.)

BABU BANARASI DAS UNIVERSITY
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CERTIFICATE

Certified that **Aman Ahmed** (Roll No. 1170444001)has carried out the thesis work presented in this project entitled **"Seismic Performance Assessment of Multistoreyed RC Special Moment Resisting Frames By Pushover Analysis"** for the award of the degree of **Master of Technology (Structural Engineering)** from Babu Banarasi Das University , Lucknow under my supervision . The project body results of original work and studies are carried out by the student himself and the context of the project do not form the basis for the award of any other degree to the candidate or to anybody else from this or any other university/institution.

(Signature)	**(Signature)**
Mr. Anupam Mehrotra	Mr. Mohammad Afaque Khan
(Head of Department)	(Assistant Professor)
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Date:-

DECLARATION

I hereby declare that, I am the sole author of this thesis. This thesis contains no material that has been submitted previously, in whole or in part, for the award of any other academic degree or diploma. Except where otherwise indicated, this thesis is my own work

The work was done under the guidance of Mohammad Afaque Khan (Assistant Professor), at the Babu Banarasi Das University, Lucknow , Uttar Pradesh, India.

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In my capacity as supervisor of the candidate's thesis, I certify that the above statements are true to the best of my knowledge.

Mr. Mohammad Afaque Khan

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ABSTRACT

In this paper study about the seismic analysis of special and ordinary moment resisting frame by the pushover analysis with the help of the SAP2000 software which is product of the Computer and Structure &Inc. The code used for seismic analysis IS CODE 1893 part1:2016. The method used in this analysis is Nonlinear static Analysis in which static analysis represents the Response Spectrum method. The main aims of this paper to study about the plastic hinges which produce after the collapse of the structure and also comparative study about the ordinary and special moment resisting frame that which one is perform better in the push over analysis. The hinges apply at the all beam and column to study about the plastic hinges in the structure. The main purpose to choose special moment resisting frame is that frame which resist the strong ground motion during the earthquake. The ordinary moment resisting frame is that frame which resists the low ground motion as compared to the special moment resisting frame. After analysis we can say that which frame produce little plastic hinges as compared to the other frame. The designing criteria of the Special Moment Resisting Frame and Ordinary Moment Resisting Frame are given in the Indian Standard Code 1893 part1:2016.

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CHAPTER 1

INTRODUCTION

1.1 General

Reinforced concrete special moment frames are used as part of seismic force-resisting systems in buildings that are designed to resist earthquakes. Beams, columns, and beam- column joints in moment frames are proportioned and detailed to resist flexural, axial, and shearing actions that result as a building sways through multiple displacement cycles during strong earthquake ground shaking. Special proportioning and detailing requirements result in a frame capable of resisting strong earthquake shaking without significant loss of stiffness or strength. These moment-resisting frames are called "Special Moment Resisting Frames" because of these additional requirements, which improve the seismic resistance in comparison with less stringently detailed Intermediate and Ordinary Moment Resisting Frames.

1.2 Historical development

Concrete frame buildings, especially older, non-ductile frames, have frequently experienced significant structural damage in earthquakes. Reinforced concrete special moment frame concepts were introduced in the U.S. starting around 1960. Their use at that time was essentially at the discretion of the designer, as it was not until 1973 that the Uniform Building Code (ICBO 1973) first required use of the special frame details in regions of highest seismicity. In India the use of Special Moment Resisting Frames started by around 1993.The proportioning and detailing of SMRF in India is according to IS 13920(1993), which later got reaffirmed in the year 2002. The earliest detailing requirements are remarkably similar to those in place today.

1.3 When to use SMRF

Moment frames are generally selected as the seismic force-resisting system when architectural space planning flexibility is desired. When concrete moment frames are selected for buildings assigned to Seismic Design Categories III, IV or V, they are required to be detailed as special reinforced concrete moment frames. Proportioning and detailing requirements for a special moment frame will enable the frame to safely undergo extensive inelastic deformations that are anticipated in these seismic design categories. Special moment frames may be used in Seismic Design Categories I or II, though this may not lead to the most economical design. Both strength and stiffness need to be considered in the design of special moment frames. According to IS 13920(2002), special moment frames are allowed to be designed for a force reduction factor of R= 5. That is, they are allowed to be designed for a base shear equal to one-fifth of the value obtained from an elastic response analysis. Moment frames are generally flexible lateral systems; therefore, strength requirements may be controlled by the minimum base shear equations of the code.

1.4 Principles of design for special moment resisting frames

The design base shear equations of current building codes incorporate a seismic force- reduction factor R, that reflects the degree of inelastic response expected for design-level ground motions, as well as the ductility capacity of the framing system. A special moment resisting frame should be expected to sustain multiple cycles of inelastic response if it experiences design-level ground motion. The proportioning and detailing requirements for special moment frames are intended to ensure that inelastic response is ductile. Three main goals are: (1) to achieve a strong- column/weak-beam design that spreads inelastic response over several stories; (2) to avoid shear failure; and (3) to provide details that enable ductile flexural response in yielding regions.

1.4.1 Strong column weak beam concept

When a building sways during an earthquake, the distribution of damage over height depends on the distribution of lateral drift. If the building has weak columns, drift tends to concentrate in one or a few stories (Fig 1-1a), and may exceed the drift capacity of the columns. On the other hand, if columns provide a stiff and strong spine over the building height, drift will be more uniformly distributed (Fig 1-1c), and localized damage will be reduced. The kind of failure that is shown in Fig 1-1c is known as Beam Mechanism or Sway Mechanism. Additionally, it is important to recognize that the columns in a given story support the weight of the entire building above those columns, whereas the beams only support the gravity loads of the floor of which they form a part; therefore, failure of a column is of greater consequence than failure of a beam. Recognizing this behaviour, building codes specify that columns be stronger than the beams that frame into them. This strong-column/weak-beam principle is fundamental to achieving safe behaviour of frames during strong earthquake ground shaking. It is a design principle that must be strictly followed while designing Special Moment Resisting Frames. Structural Designers adopts the strong-column/weak-beam principle by requiring that the sum of column strengths exceed the sum of beam strengths at each beam-column connection of a special moment frame.

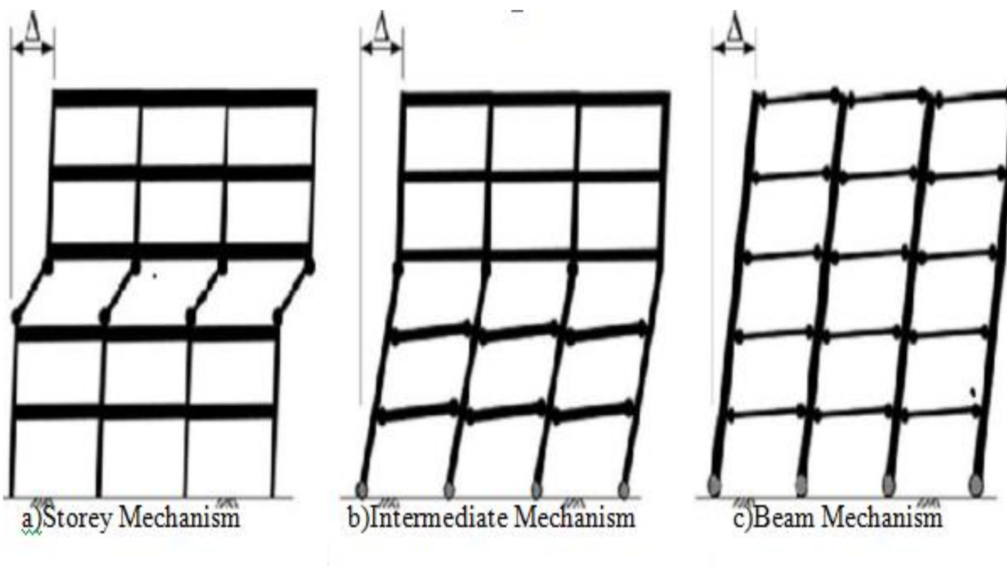

Fig 1.1 Different failure mechanisms

1.5 Avoidance of shear failure

Ductile response requires that members yield in flexure, and that shear failure be avoided. Shear failure, especially in columns, is relatively brittle and can lead to rapid loss of lateral strength and axial load-carrying capacity (Figure 3). Column shear failure is the most frequently cited cause of concrete building failure and collapse in earthquakes.Shear failure is avoided through use of a capacity-design approach. The general approach is to identify flexural yielding regions, design those regions for code-required moment strengths, and then calculate design shears based on equilibrium assuming the flexural yielding regions develop probable moment strengths. The probable moment strength is calculated using procedures that produce a high estimate of the moment strength of the designed cross section.

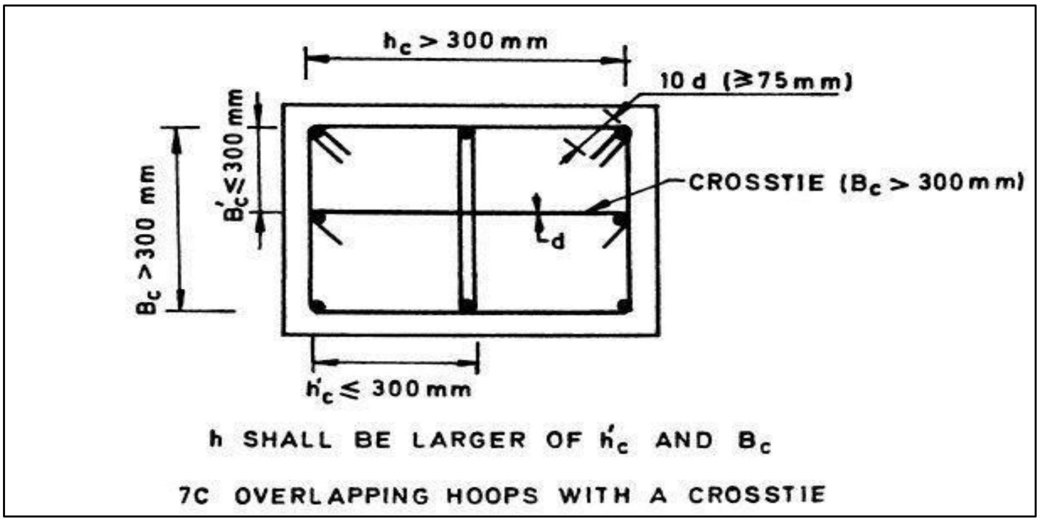

Fig 1.2 Shear Reinforcement in beams as per IS 13920 (2016)

1.6 Detailing for ductile behaviour

For achieving a ductile nature, importance must be given for the detailing in reinforcement. The various factors that should be taken care of is discussed below. The ductile nature of the building is heavily dependent on the detailing pattern and improper detailing can result in failure of the building without enough warning.

1.7 Confinement for heavily loaded sections

Plain concrete has relatively small usable compressive strain capacity (around 0.003), and this might limit the deformability of beams and columns of special moment frames. Strain capacity can be increased ten-fold by confining the concrete with reinforcing spirals or closed hoops. The hoops act to restrain dilation of the core concrete as it is loaded in compression, and this confining action leads to increased strength and strain capacity. Hoops typically are provided at the ends of columns, as well as through beam-column joints, and at the ends of beams. To be effective, the hoops must enclose the entire cross section except the cover concrete, which should be as small as allowable, and must be closed by 135° hooks embedded in the core concrete; this prevents the hoops from opening if the concrete cover spalls off. Crossties should engage longitudinal reinforcement around the perimeter to improve confinement effectiveness. The hoops should be closely spaced along the longitudinal axis of the member, both to confine the concrete and restrain buckling of longitudinal reinforcement. Crossties, which typically have 90° and 135° hooks to facilitate construction, must have their 90° and 135° hooks alternated along the length of the member to improve confinement effectiveness.

1.8 Sample shear reinforcement

Shear strength degrades in members subjected to multiple inelastic deformation reversals, especially if axial loads are low. In such members it is required that the contribution of concrete to shear resistance be ignored, that is, $V_C = 0$. Therefore, shear reinforcement is required to resist the entire shear force.

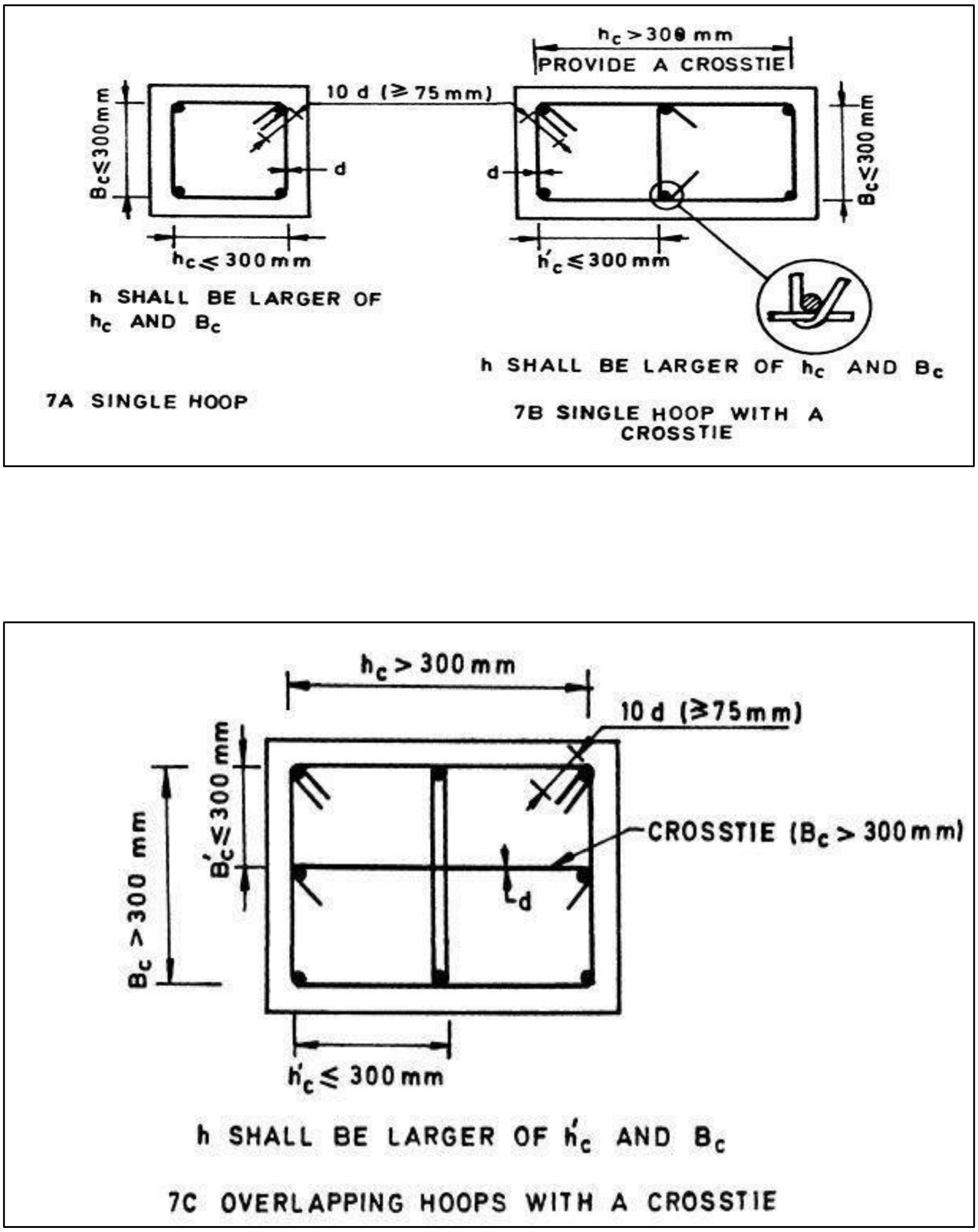

Fig 1.3 Transverse Reinforcement in columns as per IS 13920(2016)

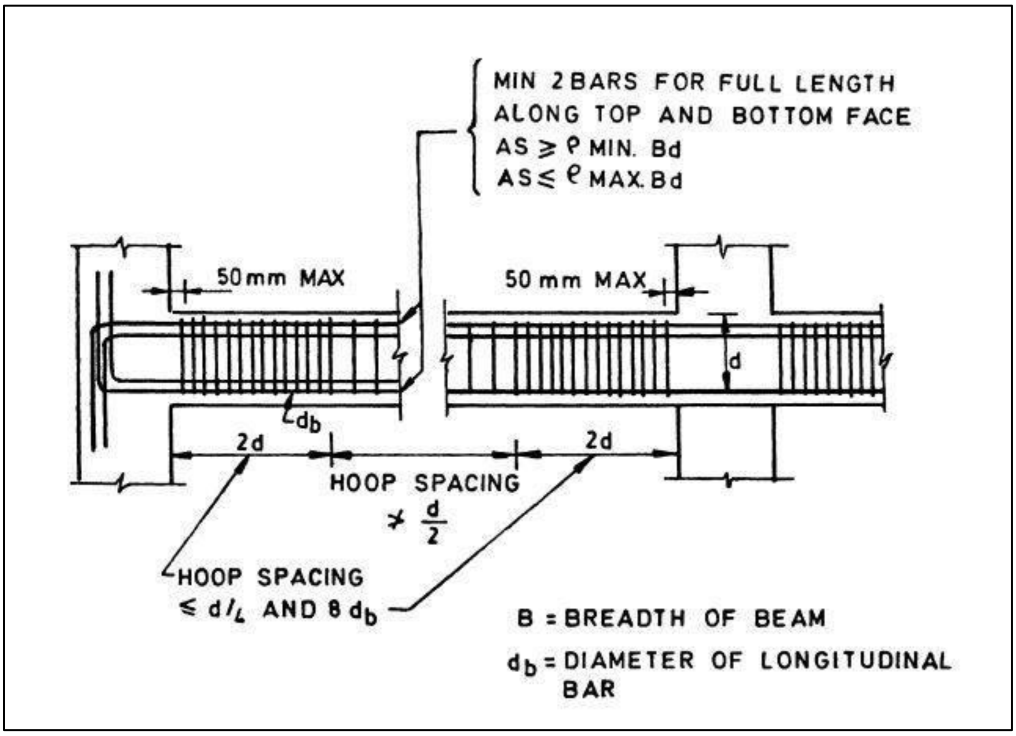

Fig 1.4 Beam Reinforcement as per IS 13920(2016)

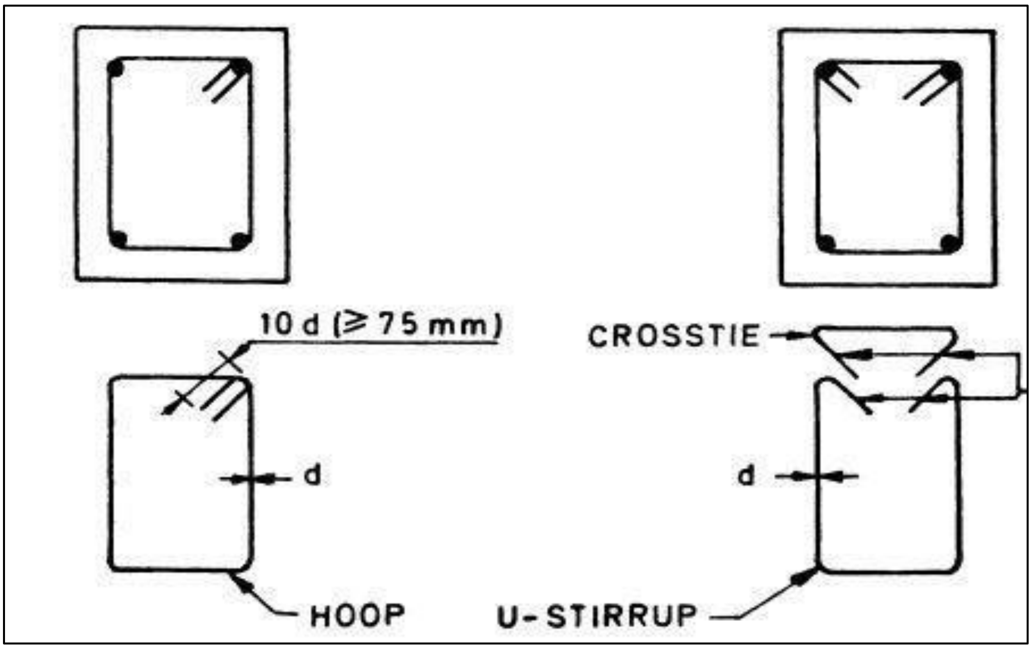

Fig 1.5 Beam Web Reinforcement as per IS 13920(2016)

1.9 Avoidance of anchorage or splice failure

Severe seismic loading can result in loss of concrete cover, which will reduce development and lap-splice strength of longitudinal reinforcement. Lap splices, if used, must be located away from sections of maximum moment (that is, away from ends of beams and columns) and must have closed hoops to confine the splice in the event of cover spalling. Bars passing through a beam-column joint can create severe bond stress demands on the joint. Bars anchored in exterior joints must develop yield strength (fy) using hooks located at the far side of the joint. Finally, mechanical splices located where yielding is likely must be splices capable of developing at least the specified tensile strength of the bar.

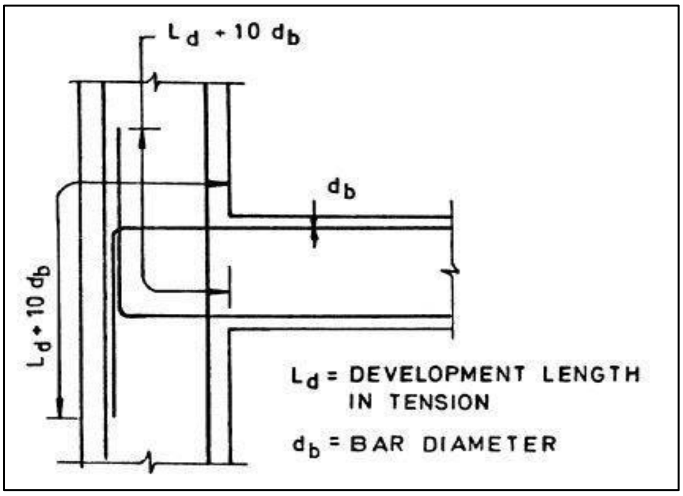

Fig 1.6 Anchorage of Beam Bars in an External Joint, IS 13920(2002)

1.10 Objectives of the thesis

Present study focus on various aspects related to the performance of SMRF buildings. The main objective of present study is the study of comparative performance of SMRF and OMRF frames, designed as per IS codes, using nonlinear analysis. The more realistic performance of the OMRF and SMRF building requires modeling the stiffness and strength of the infill walls. The variations in the type of the infill walls using in Indian constructions are significant. Depending on the modulus of elasticity and the strength, it can be classified as strong or weak. The two extreme cases of infill walls, strong and weak are considered by modelling the stiffness and strength of infill walls as accurately as possible in the present study. The behaviour of buildings depends on the type of foundations and soils also. Depending on the foundations resting on soft or hard soils, the displacement boundary conditions at the bottom of foundations can be considered as hinged or fixed. As the modelling of soils is not in the scope of the study, two boundary conditions, fixed and hinged, that represent two extreme conditions are considered.

The objectives of the present study can be identified as follows:

- To study the behavior of OMRF and SMRF buildings designed as per IS codes.
- To study the effect of type of infill walls in the performance of the SMRF buildings
- To study the effect of support conditions on the performance of OMRF and SMRF

CHAPTER 2

LITERATURE REVIEW

2.1 General

An extensive literature review is done for carrying out the project. The details of the various references and the inference from those references are discussed in this chapter.

2.2 Special Moment Resisting Frames and Pushover Analyses

Under lateral loading, the frame and the infill wall stay intact initially. As the lateral load increases, the infill wall gets separated from the surrounding frame at the unloaded (tension) corner. However at the compression corners the infill walls are still intact. The length over which the infill wall and the frame are intact is called the length of contact. Load transfer occurs through an imaginary diagonal which acts like a compression strut. Due to this behaviour of infill wall, they can be modelled as an equivalent diagonal strut connecting the two compressive corners diagonally. The stiffness property should be such that the strut is active only when subjected to compression. Thus, under lateral loading only one diagonal will be operational at a time. This concept was first put forward by Holmes (1961).

2.3 Rao et al. (1982) conducted theoretical and experimental studies on infill frames with opening strengthened by lintel beams. It was concluded that the lintel over the opening does not have any influence on the lateral stiffness of an infill frame.

2.4 Rutenberg (1992) pointed out that the research works considering single element models could not yield the ductility demand parameter properly, because they have considered distribution of strength in same proportion as their elastic stiffness distribution. Considering these drawbacks of the equivalent single element model, many investigations in this field adopted a generalized type of structural model which had a rigid deck supported by different numbers of lateral load-resisting elements representing frames or walls having strength and stiffness in their planes only. The effect of different parameters such as plan aspect ratio, relative stiffness, and number of bays on the behaviour of infill frame was studied by Riddington and Smith (1997).

2.5 Deodhar and Patel (1998) pointed out that even though the brick masonry in infill frame are intended to be non-structural, they can have considerable influence on the lateral response of the building.

2.6 Helmut Krawinkler et al., (1998) studied the pros and cons of Pushover analysis and suggested that element behaviour cannot be evaluated in the context of presently employed global system quality factors such as the R and R_W factors used in present US seismic codes. They also suggested that a carefully performed pushover analysis will provide insight into structural aspects that control performance during severe earthquakes. For structures that vibrate primarily in the fundamental mode, the pushover analysis will very likely provide good estimates of global, as well as local inelastic, deformation demands. This analysis will also expose design weaknesses that may remain hidden in an elastic analysis. Such weaknesses include story mechanisms, excessive deformation demands, strength irregularities and overloads on potentially brittle elements such as columns and connections.

2.7 Foley CM et al., (2002) studied a review of current state-of-the-art seismic performance- based design procedures and presented the vision for the development of PBD optimization. It is recognized that there is a pressing need for developing optimized PBD procedures for seismic engineering of structures.

2.8 R. Hasan and D.E. Grierson (2002), conducted a simple computer-based push-over analysis technique for performance-based design of building frameworks subject to earthquake loading. And found that rigidity-factor for elastic analysis of semi-rigid frames, and the stiffness properties for semi-rigid analysis are directly adopted for push-over analysis.

2.9 B.Akbas. et al., (2003), conducted a pushover analysis on steel frames to estimate the seismic demands at different performance levels, which requires the consideration of inelastic behaviour of the structure.

2.10 Das and Murthy (2004) concluded that infill walls, when present in a structure, generally bring down the damage suffered by the RC framed members of a fully infilled frame during earthquake shaking. The columns, beams and infill walls of lower stories are more vulnerable to damage than those in upper stories.

2.11 Oğuz, Sermin (2005), ascertained the effects and the accuracy of invariant lateral load patterns utilized in pushover analysis to predict the behaviour imposed on the structure due to randomly Selected individual ground motions causing elastic deformation by studying various levels of Nonlinear response. For this purpose, pushover analyses using various invariant lateral load patterns and Modal Pushover Analysis were performed on reinforced concrete and steel moment resisting frames covering a broad range of fundamental periods. The accuracy of approximate Procedures utilized to estimate target displacement was also studied on frame structures. Pushover analyses were performed by both DRAIN-2DX and SAP2000. The primary observations from the study showed that the accuracy of the pushover results depended strongly On the load path, the characteristics of the ground motion and the properties of the structure.

2.12 X.-K. Zou et al., (2005) presented an effective technique that incorporates Pushover Analysis together with numerical optimization procedures to automate the Pushover drift performance design of reinforced concrete buildings. PBD using nonlinear pushover analysis, which generally involves tedious computational effort, is highly iterative process needed to meet code requirements.

2.13 Kircil et al., (2006) designed 3, 5 and 7 story buildings according to Turkish Design codes and found that the fragility curve has considerable variations depending on the height of the building.

2.14 Asokan (2006) studied how the presence of masonry infill walls in the frames of a building changes the lateral stiffness and strength of the structure. This research proposed a plastic hinge model for infill wall to be used in nonlinear performance based analysis of a building and concludes that the ultimate load approach along with the proposed hinge property provides a better estimate of the inelastic drift of the building.

2.15 Mehmet et al. (2006), explained that due to its simplicity of Pushover analysis, the structural engineering profession has been using the nonlinear static procedure or pushover analysis. Pushover analysis is carried out for different nonlinear hinge properties available in some programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length (Lp) has considerable effects on the displacement capacity of the frames. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties (Programmed Default).

2.16 Girgin et al., (2007) Pushover analysis has been the preferred method for seismic performance evaluation of structures by the major rehabilitation guidelines and codes because it is computationally and conceptually simple. Pushover analysis allows tracing the sequence of yielding and failure on member and structural level as well as the progress of overall capacity curve of the structure. programs based on the FEMA-356 and ATC-40 guidelines and he pointed out that Plastic hinge length (Lp) has considerable effects on the displacement capacity of the frames. The orientation and the axial load level of the columns cannot be taken into account properly by the default-hinge properties (Programmed Default).

2.17 Shuraim et al., (2007) the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a new reinforced concrete frame. Potential structural deficiencies in reinforced concrete frame, when subjected to a moderate seismic loading, were estimated by the pushover approaches. In this method the design was evaluated by redesigning under selected seismic combination in order to show which members would require additional reinforcement. Most columns required significant additional reinforcement, indicating their vulnerability when subjected to seismic forces. The nonlinear pushover procedure shows that the frame is capable of withstanding the presumed seismic force with some significant yielding at all beams and one column.

2.18 A. Shuraim et al., (2007) summarized the nonlinear static analytical procedure (Pushover) as introduced by ATC-40 has been utilized for the evaluation of existing design of a new reinforced concrete frame, in order to examine its applicability. Potential structural deficiencies in RC frame, when subjected to a moderate seismic loading, were estimated by the code seismic-resistant design and pushover approaches. In the first method the design was evaluated by redesigning under one selected seismic combination in order to show which members would require additional reinforcement. It was shown that most columns required significant additional reinforcement, indicating their vulnerability if subjected to seismic forces. On the other hand, the nonlinear pushover procedure shows that the frame is capable of withstanding the presumed seismic force with some significant yielding at all beams and one column. Vulnerability locations from the two procedures are significantly different. The paper has discussed the reasons behind the apparent discrepancy which is mainly due to the default assumptions of the method as implemented by the software versus the code assumptions regarding reduction factors and maximum permissible limits. In new building design, the code always maintains certain factor of safety that comes from load factors, materials reduction factors, and ignoring some post yielding characteristics (hardening). In the modeling assumptions of ATC-40, reduction factor is assumed to be one, and hardening is to be taken into consideration. Hence, the paper suggests that engineering judgment should be exercised prudently when using the pushover analysis and that engineer should follow the code limits when designing new buildings and impose certain reductions and limits in case of existing buildings depending on their conditions. In short software should not substitute for code provisions and engineering judgment.

2.19 A. Whittaker , Y. N. Huang et al (2007) summarize the next (second) generation tools and procedures for performance-based earthquake engineering in the United States. The methodology, which is described in detail in the draft Guidelines for the Seismic Performance Assessment of Buildings, builds on the first generation deterministic procedures, which were developed in the ATC-33 project in the mid 1990s and in ASCE Standard: ASCE/SEI 41-06 Seismic Rehabilitation of Existing Buildings. The procedures and methodologies described in these guidelines include an explicit treatment of the large uncertainties in the prediction of losses due to earthquakes. This formal treatment of uncertainty and randomness represents a substantial advance in performance based engineering and a significant departure from the first generation deterministic procedures.

2.20 Konuralp et al., (2007) explained that structural frames are often filled with infilled walls serving as partitions. Although the infills usually are not considered in the structural analysis and design, their influence on the seismic behaviour of the infilled frame structures is considerable. In this study, a parametric study of certain infilled frames, using the strut model to capture the global effects of the infills was carried out. Three concrete planar frames of five-stories and three-bays are considered which have been designed in accordance with Turkish Codes. Pushover analysis is adopted for the evaluation of the seismic response of the frames. Each frame is subjected to four different loading cases.

The results of the cases are briefly presented and compared. The effect of infill walls on seismic behaviour of two sample frames with different infill arrangements was investigated. The results yield that it is essential to consider the effect of masonry infills for the seismic evaluation of moment-resisting RC frames, especially for the prediction of its ultimate state, infills having no irregularity in elevation have beneficial effect on buildings and infills appear to have a significant effect on the reduction of global lateral displacements.

Infills have been generally considered as non-structural elements, although there are codes such as the Eurocode-8 that include rather detailed procedures for designing infilled R/C frames, presence of infills has been ignored in most of the current seismic codes except their weight. However, even though they are considered non-structural elements the presence of infills in the reinforced concrete frames can substantially change the seismic response of buildings in certain cases producing undesirable effects (torsional effects, dangerous collapse mechanisms, soft storey, variations in the vibration period, etc.) or favourable effects of increasing the seismic resistance capacity of the building. The pushover analysis can be considered as a series of incremental static analyses carried out to examine the non-linear behaviour of structure, including the deformation and damage pattern. The procedure consists of two parts. First, a target displacement for the structure is established. The target displacement is an estimate of the seismic top displacement of the building, when it is exposed to the design earthquake excitation. Then, a pushover analysis is carried out on the structure until the displacement at the top of the building reaches the target displacement. The extent of damage experienced by the building at the target displacement is considered to be representative of the damage experienced by the building when subjected to design level ground shaking. A judgment is formed as to the acceptability of the structural behavior for the design of the new building, or the level of damage of an existing building for evaluation purposes. In the conclusion he states that the effect of infill walls on seismic behavior of a two sample frames with different infill arrangements was investigated. The results yields the following conclusions.

- It is essential to consider the effect of masonry infills for the seismic evaluation of moment resisting RC frames, especially for the prediction of its ultimate state.
- Infills having no irregularity in elevation have beneficial effect on buildings. In infilled frames with irregularities, such as soft story, damage was found to concentrate in the levels where the discontinuity occurs.
- Since infills increases lateral resistance and initial stiffness of the frames they appear to have a significant effect on the reduction of the global lateral displacement.
- Arrangement of infills may effect the post yield behavior and has an influence on distribution and sequence of damage formation. To generalize this, more infill arrangements should be investigated.
- A carefully performed pushover analysis can provide insight into structural aspects that control performance of the structure during a severe earthquake.
- The choice of the static load distribution used in pushover analysis can affect the accuracy of the response estimates.

2.21 A.Kadid and A. Boumrkik (2008), proposed use of Pushover Analysis as a viable method to assess damage vulnerability of a building designed according to Algerian code. Pushover analysis was a Series of incremental static analysis carried out to develop a capacity curve for the building. Based on capacity curve, a target displacement which was an estimate of the displacement that the design earthquake would produce on the building was determined. The extent of damage Experienced by the structure at this target displacement is considered representative of the Damage experienced by the building when subjected to design level ground shaking. Since the Behavior of reinforced concrete structures might be highly inelastic under seismic loads, the global inelastic performance of RC structures would be dominated by plastic yielding effects and consequently the accuracy of the pushover analysis would be influenced by the ability of the Analytical models to capture these effects.

2.22 Athanassiadou (2008) analyzed two ten-storeyed two-dimensional plane stepped frames and one ten-storeyed regular frame designed, as per Euro code 8 (2004) for the high and medium ductility classes. This research validates the design methodology requiring linear dynamic analysis recommended in Euro code 8 for irregular buildings. The stepped buildings, designed to Euro code 8 (2004) were found to behave satisfactorily under the design basis earthquake and also under the maximum considered earthquake (involving ground motion twice as strong as the design basis earthquake). Inter-storey drift ratios of irregular frames were found to remain quite low even in the case of the „collapse prevention" earthquake. This fact, combined with the limited plastic hinge formation in columns, exclude the possibility of formation of a collapse mechanism at the neighbourhood of the irregularities. Plastic hinge formation in columns is seen to be very limited during the design basis earthquake, taking place only at locations not prohibited by the code, i.e. at the building base and top. It has been concluded that the capacity design procedure provided by Euro code 8 is completely successful and can be characterized by conservatism, mainly in the case of the design of high- ductility columns. The over-strength of the irregular frames is found to be similar to that of the regular ones, with the over-strength ratio values being 1.50 to 2.00 for medium – high ductility levels. The author presented the results of pushover analysis using „uniform" load pattern as well as a „modal" load pattern that account the results of multimodal elastic analysis.

2.23 Karavasilis et al., (2008) presented a parametric study of the inelastic seismic response of plane steel moment resisting frames with steps and setbacks. A family of 120 such frames, designed according to the European seismic and structural codes, was subjected to 30 earthquake ground motions, scaled to different intensities. The main findings of this paper are as follows. Inelastic deformation and geometrical configuration play an important role on the height-wise distribution of deformation demands. In general, the maximum deformation demands are concentrated in the tower-base junction in the case of setback frame and in all the step locations in the case of stepped frames. This concentration of forces at the locations of height discontinuity, however, is not observed in the elastic range of the seismic response.

2.24 Tena-Colunga et al., (2008) conducted a study on 22 regular mid rise RC-SMRF buildings to fulfill the requirements of MFDC(Mexico Federal District code) and concluded that usage of secondary beams to reduce the slab thickness will result in increase in seismic behaviour in SMRF.

2.25 J.P. Moehle (2008) presented a performance based seismic design of tall buildings in the U.S. He presented that the building codes in the United States contain prescriptive requirements for seismic design as well as an option for use of alternative provisions. Increasingly these alternative provisions are being applied for the performance-based seismic design of tall buildings. Application of performance-based procedures requires: An understanding of the relation between performance and nonlinear response; selection and manipulation of ground motions appropriate to the seismic hazard; selection of appropriate nonlinear models and analysis procedures; interpretation of results to determine design quantities based on nonlinear dynamic analysis procedures; appropriate structural details; and peer review by independent qualified experts to help assure the building official that the proposed materials and system are acceptable. Both practice- and research-oriented aspects of performance-based seismic design of tall buildings are presented. He said that the west coast of the United States, a highly seismic region, is seeing a resurgence in the design and construction of tall buildings (defined here as buildings 240 feet (73 meters) or taller). Many of these buildings use high-performance materials and framing systems that are not commonly used for building construction or that fall outside the height limits of current buildings codes. In many cases, prescriptive provisions of governing building codes are found to be overly restrictive, leading to designs that are outside the limits of the code prescriptive provisions. This is allowable through the alternative provisions clause of building codes. When the alternative provisions clause is invoked, this normally leads to a performance-based design involving development of a design-specific criteria, site- specific seismic hazard analysis, selection and modification of ground motions, development of a nonlinear computer analysis model of the building, performance verification analyses, development of building-specific details, and peer review by tall buildings design experts. His views about the new generation of tall buildings in the western U.S. is that Urban regions along the west coast of the United States are seeing a boom in tall building construction. To meet functional and economic requirements, many of the new buildings are using specialized materials and lateral-force-resisting systems that do not meet the prescriptive definitions and requirements of current building codes. According to Moehle's a design criteria document generally is developed by the designer to clearly and concisely communicate to the design team, the building official, and the peer reviewers the intent and the process of the building structural design. A well prepared document will likely include data and discussion regarding the building and its location; the seismic and wind force-resisting systems; sample conceptual drawings; codes and references that the design incorporates in part or full; exceptions to aforementioned code prescriptive provisions; performance objectives; gravity, seismic, and wind loading criteria; load combinations; materials; methods of analysis including software and modeling procedures; acceptance criteria; and test data to support use of new components. The document is prepared early for approval by the building official and peer reviewers, and may be modified as the design advances and the building is better understood. The design criteria document must define how the design is intended to meet or exceed the performance expectations inherent in the building code. Performance-based seismic analysis of tall buildings in the U.S. increasingly uses nonlinear analysis of a three-dimensional model of the building. Lateral- force-resisting components of the building are modeled as discrete elements with lumped plasticity or fiber models that represent material nonlinearity and integrate it across the component section and length. Gravity framing elements increasingly are being included in the nonlinear models so that effects of building deformations on the gravity framing as well as effects of the gravity framing on the seismic system Because the behavior is nonlinear, behavior at one hazard level cannot be scaled from nonlinear results at another hazard level. Furthermore, conventional capacity design approaches can underestimate internal forces in some structural systems (and overestimate them in others) because lateral force profiles and deformation patterns change as the intensity of ground shaking increases (Kabeyasawa, Eberhard et al., 1993). Results of non-linear dynamic analysis are sensitive to modelling assumptions. A significant percentage of recent high-rise building construction in the western U.S. has been for residential and mixed-use occupancies. Thus, much of it has been of reinforced concrete, and the majority of those have used reinforced concrete core walls. Some concrete and steel framing, and some steel walls, also are used. Under design-level earthquake ground motions, the core wall may undergo inelastic deformations near the base (and elsewhere) in the presence of high shear. Ductile performance requires an effectively continuous tension chord, adequately confined compression zone, and adequate proportions and details for shear resistance. In locations where yielding is anticipated, splices (either mechanical or lapped) must be capable of developing forces approaching the bar strength. Furthermore, longitudinal reinforcement is to be extended a distance 0.8lw past the point where it is no longer required for flexure based on conventional section flexural analysis, where lw is the (horizontal) wall length. Walls generally are fully confined at the base and extending into subterranean levels. Confinement above the base may be reduced (perhaps by half) where analysis shows reduced strains, though strains calculated by nonlinear analysis software generally should be viewed skeptically as they are strongly dependent on modeling assumptions (modeling procedures should be validated by the engineer of record against strains measured in laboratory tests). The reduced confinement usually continues up the wall height until calculated demands under maximum expected loadings are well below spalling levels. Transverse reinforcement for wall shear generally is developed to the far face of the confined boundary zone; otherwise, the full length of the wall is not effective in resisting shear. Coupled core walls require ductile link beams that can undergo large inelastic rotations. Away from the core walls, gravity loads commonly supported by post-tensioned floor slabs supported by columns. Slab-column connections are designed considering the effect of lateral drifts on the shear punching tendency of the connection. For post-tensioned slabs, which are most common, at least two of the strands in each direction must pass through the column cage to provide post-punching resistance. He concluded that Performance-based earthquake engineering increasingly is being used as an approach to the design of tall buildings in the U.S. Available software, research results, and experience gained through real building applications are providing a basis for effective application of nonlinear analysis procedures. Important considerations include definition of performance objectives, selection of input ground motions, construction of an appropriate nonlinear analysis model, and judicious interpretation of the results. Implemented properly, nonlinear dynamic analysis specific to the structural system and seismic environment is the best way to identify nonlinear dynamic response characteristics, including yielding mechanisms, associated internal forces, deformation demands, and detailing requirements. Proportions and details superior to those obtained using the prescriptive requirements of the building code can be determined by such analysis, leading to greater confidence in building performance characteristics including serviceability and safety. Although performance-based designs already are under way and are leading to improved designs, several research needs have been identified, the study of which can further improve design practices.

2.26 Taewan K et al., (2009) designed a building as per IBC 2003 and showed that the building satisfied the inelastic behaviour intended in the code and satisfied the design drift limit.

2.27 Oscar Moller et al., (2009) explained the following conclusions that can be offered as suggestions for further research:

- Performance-based design in earthquake engineering implies consideration of the uncertainties in the structural demands and capacities, in order to evaluate the reliability associated with each of the required performance levels. These reliabilities must satisfy minimum target values for each level.
- Calculation of the structural responses for the formulation of the limit states equations requires a nonlinear dynamic analysis, and these responses cannot be given in an explicit relationship in terms of the intervening random

variables. Discrete data can be obtained for chosen combinations of these variables, and the results can be expressed in terms of response surfaces or neural networks. In this work the latter approach has been followed, providing flexibility and adaptability.

- The major computational demand in this approach is the construction of the discrete database, executing the nonlinear dynamic analysis for a number of variable combinations representative of the variable ranges. For a fixed combination within a sub-set of the variables, the analysis is carried out for another sub-set which groups variables including different ground motions. For each combination, and over the set of grouped variables, the mean and the standard deviation of each response of interest are obtained. These statistics are then represented by neural networks, and are utilized in representing the responses in a probabilistic manner.
- The utilization of neural networks' representation for the response demands makes feasible the calculation of the probability of non-performance via standard Monte Carlo simulation. The reliability associated with each performance level can thus be estimated for different combinations of design parameters, and these reliabilities can themselves be represented by neural networks.
- The optimization in performance-based design implies the minimization of an objective function (here the total structural cost was used) subject to the achievement of minimum target reliabilities at each performance level. This work has shown the implementation of an optimization scheme based on a search without calculation of gradients. This scheme is efficient, whether the intermediate reliability constraints are evaluated by simulation at each step, or they are implemented using the reliability neural networks.
- The optimization scheme for minimum total cost has been applied to a multi-storey, multi- bay reinforced concrete frame, with the design parameters being the depths of beams and columns, and three steel reinforcement ratios. The results show good agreement between the two ways of implementing the calculation of the reliability constraints, and that somewhat different optimum design parameters may correspond to minor differences in the total cost. In particular, the results have shown that it is important the consideration of

damage repair costs, as they influence the optimum solution.

- This work has shown that neural networks offer a very useful tool to represent the relationship between structural responses and the intervening random variables, and between achieved reliabilities and the design parameters. The first application make feasible the use of Monte Carlo simulation to estimate reliabilities or probabilities of non-performance, while the second improves the efficiency of the optimization algorithm when intermediate reliabilities need to be evaluated.
- The approach presented introduced a general scheme for reliability estimation and performance-based design optimization in earthquake engineering. It introduced required concepts like a relationship between damage level and repair cost – a relationship that still needs further general development and should be the objective of continuing research.
- Continuing research should also be focused on damage parameters and their relationship to calculated quantities like strains and displacements. Here a well known damage index was used for the purpose of the application, but further research should be focused on how damage accumulates over time as a result of the applied strains or displacement history.

2.28 Sattar and Abbie (2010) in their study concluded that the pushover analysis showed an increase in initial stiffness, strength, and energy dissipation of the infill frame, compared to the bare frame, despite the wall"s brittle failure modes. Likewise, dynamic analysis results indicated that fully-infill frame has the lowest collapse risk and the bare frames were found to be the most vulnerable to earthquake-induced collapse. The better collapse performance of fully-infill frames was associated with the larger strength and energy dissipation of the system, associated with the added walls.

2.29 P.Poluraju and P.V.S.N.Rao (2011), has studied the behaviour of framed building by conducting Pushover Analysis, most of buildings collapsed were found deficient to meet out the requirements of the present day codes. Then

G+3 building was modelled and analyzed, results obtained from the study shows that properly designed frame will perform well under seismic loads.

2.30 Dhileep. M et al., (2011) explained the practical difficulties associated with the non linear direct numerical integration of the equations of motion leads to the use of non linear static pushover analysis of structures. Pushover analysis is getting popular due to its simplicity. High frequency modes and non linear effects may play an important role in stiff and irregular structures. The contribution of higher modes in pushover analysis is not fully developed. The behavior of high frequency model responses in non linear seismic analysis of structures is not known. In this paper an attempt is made to study the behavior of high frequency model responses in non linear seismic analysis of structures.

Non linear static pushover analysis used as an approximation to non linear time history analysis is becoming a standard tool among the engineers, researches and professionals worldwide. High frequency modes may contribute significantly in the seismic analysis of irregular and stiff structures. In order to take the contribution of higher modes structural engineers may include high frequency modes in the non linear static pushover analysis. The behavior of high frequency modes in non linear static pushover analysis of irregular structures is studied. At high frequencies, the responses of non linear dynamic analysis converge to the non linear static pushover analysis. Therefore non linear response of high frequency modes can be evaluated using a non linear static push over analysis with an

Implemental force pattern given by their modal mass contribution times zero period acceleration. The higher modes with rigid content as a major contributing factor exhibit a better accuracy in non linear pushover analysis of structures when compared to the damped periodic modes.

CHAPTER 3
METHODOLOGY

3.1 Software

- The SAP name has been synonymous with state-of-the-art analytical methods since its introduction over 40 years ago. SAP2000 follows in the same tradition featuring a very sophisticated, intuitive and versatile user interface powered by an unmatched analysis engine and design tools for engineers working on transportation, industrial, public works, sports, and other facilities.

- From its 3D object based graphical modeling environment to the wide variety of analysis and design options completely integrated across one powerful user interface, SAP2000 has proven to be the most integrated, productive and practical general purpose structural program on the market today. This intuitive interface allows you to create structural models rapidly and intuitively without long learning curve delays. Now you can harness the power of SAP2000 for all of your analysis and design tasks, including small day-to-day problems.

- Complex Models can be generated and meshed with powerful built in templates. Integrated design code features can automatically generate wind, wave, bridge, and seismic loads with comprehensive automatic steel and concrete design code checks per US, Canadian and international design standards.

- Advanced analytical techniques allow for step-by-step large deformation analysis, Eigen and Ritz analyses based on stiffness of nonlinear cases, catenary cable analysis, material nonlinear analysis with fiber hinges, multi-layered nonlinear shell element, buckling analysis, progressive collapse analysis, energy methods for drift control, velocity-dependent dampers, base isolators, support plasticity and nonlinear segmental construction analysis. Nonlinear analyses can be static and/or time history, with options for FNA nonlinear time history dynamic analysis and direct integration.

➢ From a simple small 2D static frame analysis to a large complex 3D nonlinear dynamic analysis, SAP2000 is the easiest, most productive solution for your structural analysis and design needs.

3.2 Pushover Analysis

Performance assessment of the designed frames is carried out using nonlinear static pushover analysis. The modeling of the designed frames for nonlinear analysis is done in the Program SAP2000 Nonlinear. Pushover analysis is a static, nonlinear procedure to analysis a building where loading is incrementally increased with a certain predefined pattern (i.e., inverted triangular or uniform). Local non-linear effects are modeled and the structure is pushed until a collapse mechanism is developed. With the increase in the magnitude of loads, weak links and failure modes of the building are found. At each step, structure is pushed until enough hinges form to develop a curve between base shear of the building and their corresponding roof displacement and this curve known as pushover curve. At each step, the total base shear and the top displacement are plotted to get this pushover curve at various phases. It+ gives an idea of the maximum base shear that the structure is capable of resisting and the corresponding inelastic drift. For regular buildings, it also gives an estimate of the global stiffness and strength in terms of force and displacement of the building. A typical building frame and the a typical pushover curve diagram is shown in fig 3.1 below:

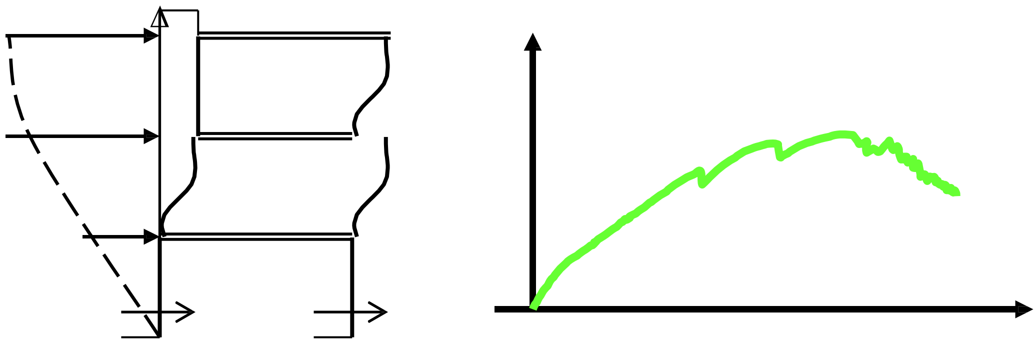

Fig 3.1 Typical Pushover Curve.

3.3 Pushover Methodology

A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic non-linear force displacement relationship can be determined. The purpose of the pushover analysis is to evaluate the expected performance of a structural system by estimating its strength and deformation demands in design earthquakes by means of a static inelastic analysis, and comparing these demands to available capacities at the performance levels of interest. The evaluation is based on an assessment of important performance parameters, including global drift, inter storey drift, inelastic element deformations (either absolute or normalized with respect to a yield value), deformations between elements, and element and connection forces (for elements and connections that cannot sustain inelastic deformations). The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring when .the structure is subjected to inertia forces that no longer can be resisted within the elastic range of structural behaviour. The pushover is expected to provide information on many response characteristics that cannot be obtained from an elastic static or dynamic analysis. The following are examples of such response characteristics:

- The realistic force demands on potentially brittle elements, such as axial force demands on columns, force demands on brace connections, moment demands on beam- to-column connections, shear force demands in deep reinforced concrete spandrel beams, shear force demands in unreinforced masonry wall piers, etc.
- Estimates of the deformation demands for elements that have to deform in elastically in order to dissipate the energy imparted to the structure by ground motions.
- Consequences of the strength deterioration of individual elements on the behaviour of the structural system.
- Identification of the critical regions in which the deformation demands are

expected to be high and that have to become the focus of thorough detailing.

- Identification of the strength discontinuities in plan or elevation that will lead to changes in the dynamic characteristics in the inelastic range.
- Estimates of the inter storey drifts that account for strength or stiffness discontinuities and that may be used to control damage and to evaluate P-delta effects.
- Verification of the completeness and adequacy of load path, considering all the elements of the structural system, all the connections, the stiff non-structural elements of significant strength, and the foundation system.

3.4 IS CODE 1893 part1:2016

This code is used for the Earthquake Resistant Design of Structure, where in this code provide the parameter and condition of the type of the seismic analysis.

3.5 Response Spectrum Analysis

Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

Response-spectrum analysis is useful for design decision-making because it relates structural type-selection to dynamic performance. Structures of shorter period experience greater acceleration, whereas those of longer period experience greater displacement. Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis.

SRSS (Square Root of Sum of Squares): This is one of the most frequently used modal combination methods. According to this rule the maximum response in terms of a given parameter (displacements, velocities, accelerations or even internal forces) may be estimated through the square root of the sum of the modal response squares, contributing to the global response. This method usually gives good results if the modal frequencies of the modes contributing for the global response are sufficiently separated to each other. Otherwise another method, such as the one following, will be more adequate.

3.5.1 CQC (Complete Quadratic Combination)

The reason why this method is more effective in evaluating the maximum response when the modal frequencies are close to each other is due to the fact that it considers the correlation between modal responses, whereas the SRSS method considers these to be independent. In fact if two vibration modes have close frequencies their contribution to the global response is not independent.

3.6 Loading Pattern

Apart from the self-weight, the building is subjected to various type of loading. The major loads acting on the building are given below:-

3.6.1 Dead Load (DL)

The dead load, include self-weight of the structure itself, and immovable fixtures such as infill walls, plasterboard or carpet. Dead loads are also known as permanent loads. The dead load of the beams and columns are automatically considered by the model. The loads from the slabs are distributed as triangular or trapezoidal line loads on the supporting beam as per IS 456:2000.

3.6.2 Live Load (LL) or Imposed Load (IL)

Live loads, or imposed loads are temporary, of short duration, or moving. These dynamic loads involve considerations such as impact, momentum, vibration, fatigue, etc. Apart from the self-weight, the building is subjected to live loads. The load distribution pattern of the live load from the slabs to the supporting beams is similar as that in case of the DL.

3.6.3 Seismic Loading

Seismic loading is one of the basic concepts of earthquake engineering which means application of an earthquake-generated agitation to the structure. It happens at contact surfaces of a structure either with the ground, or with adjacent structures, or with gravity waves from tsunami. The seismic load is calculated as per the provisions given in IS: 1893 (Part l)-2016

3.7 Load Combination

According to Indian Standard Code 1893 part1:2016 following load combination is given below:-

Table 3.1 Load Combination

A.1.5(DL+LL)	B.1.2(DL+LL+EX)	C.1.2(DL+LL-EX)
D.1.2(DL+LL+EY)	E.1.2(DL+LL-EY)	F.1.5(DL+EX)
G.1.5(DL-EX)	H.1.5(DL+EY)	I.1.5(DL-EY)
J.0.9DL+1.5EX	K.0.9DL-1.5EX	L.0.9DL+1.5EY
M.0.9DL-1.5EY		

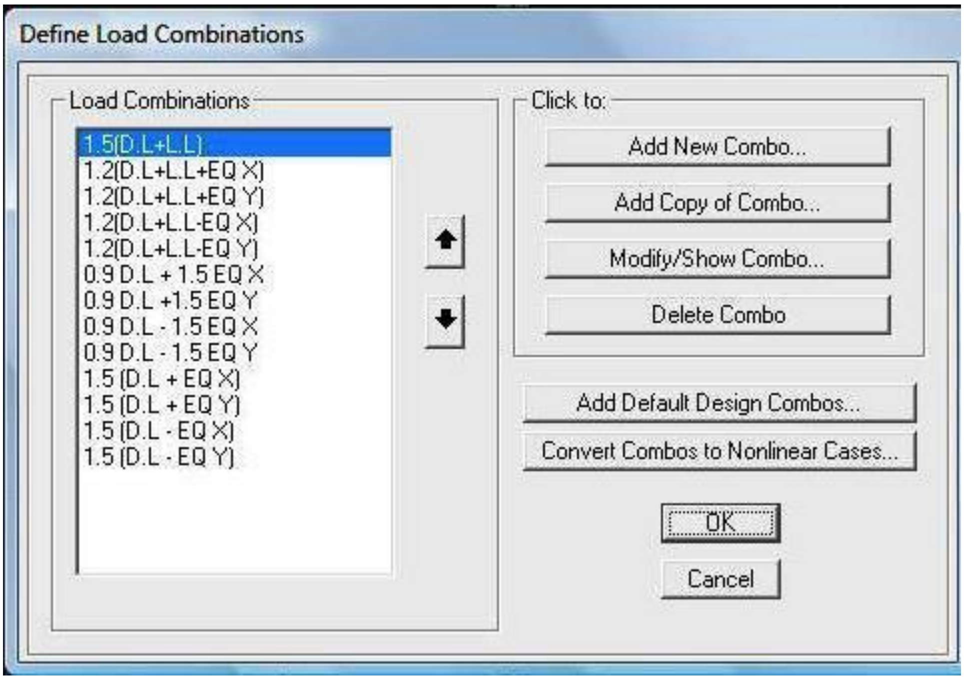

Fig 3.2 Load Combination.

3.8 Details of Models

Here we will study of the model in details.

3.8.1 Material Property

The following material property is provided in the structure, which is given in the table:-

Table 3.2 Material Property

Material Name	**Value**
Concrete	M25
Rebar	HYSD415, Mild250

3.8.2 Section and Seismic Parameter

Table 3.3 Section and Seismic Parameter

Beam	500mmX400mm
Column	600mmX400mm
Slab	150mm
Seismic Zone factor	0.36
SMRF	5.0
OMRF	3.0
Importance Factor	1.0
Soil Type	2^{nd} (Medium soil)

3.8.3 Load Parameter

Table 3.4 Load Parameter

Dead	Auto Defined
Live	$3KN/m^2$
Finishing Load	$1\ KN/m^2$

Roof	2 KN/m^2
Wall Load	15KN/m
Parapet Wall Load	7.5KN/m
EX	1893 part1:2016 (X-Direction)
EY	1893 part1:2016 (Y-Direction)

3.9 Different View of Model

The model for the SMRF and OMRF is same only value of the response reduction factor is 5 and 3 respectively

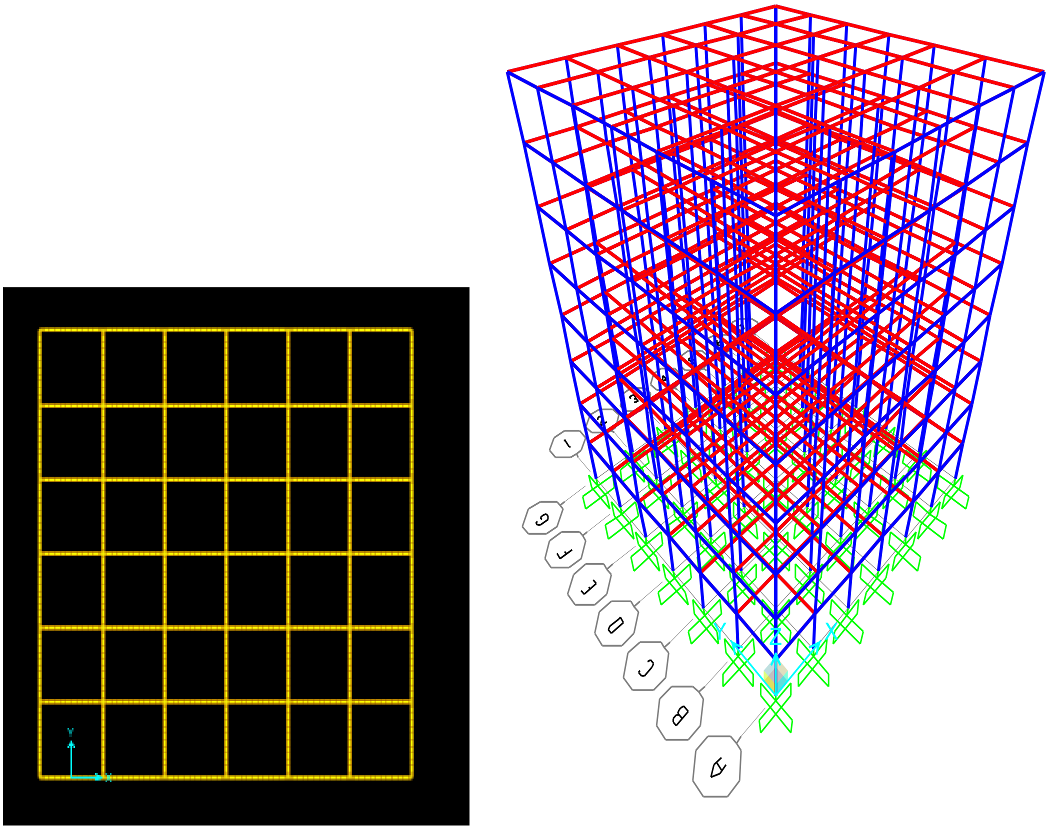

Fig 3.3 Plan and 3D View

CHAPTER 4

RESULT AND DISCUSSION

After analysis the model of SMRF and OMRF following results are given below:-

4.1 Modal Period and Frequency

The modal period and frequency of the both Special Moment Resisting Frame (SMRF) and Ordinary Moment Resisting Frame (OMRF) is same

Table 4.1 Modal Period and Frequency

Mode	Period (sec)	Frequency (cyc/sec)	CircFreq (rad/sec)	Eigenvalue (red^2/sec^2)
Mode1	0.644814	1.550835394	9.744186164	94.949164
Mode2	0.533867	1.873124305	11.76918711	138.5137652
Mode3	0.53248	1.878003432	11.79984357	139.2363082
Mode4	0.212995	4.694935292	29.49914844	870.1997589
Mode5	0.175383	5.701802045	35.82547883	1283.464933
Mode6	0.17328	5.771002298	36.26027684	1314.807677
Mode7	0.12474	8.016700066	50.37041207	2537.178412
Mode8	0.102318	9.773496603	61.40869026	3771.027239
Mode9	0.098745	10.12707928	63.63031573	4048.81708
Mode10	0.088379	11.31494737	71.09391107	5054.344191
Mode11	0.071359	14.01359763	88.05003072	7752.80791
Mode12	0.068665	14.5634917	91.50511707	8373.186451

4.2 Pushover Curve of SMRF

After analysis the special moment resisting frame, the details is given below in the table as well as graph form due to apply pushover analysis in X-direction

Table 4.2 Pushover of SMRF

Step	Displacement	Base Force	A to B	B to IO	IO to LS	LS to CP	CP to C	C to D	D to E	Beyond E	Total
	(M)	(KN)									
0	0.000085	0	0	0	0	0	0	0	0	0	0
1	0.002128	200.2	0	0	0	0	0	0	0	0	0
2	0.007542	400.3	7	0	0	0	0	0	0	0	7
3	0.007741	501.7	46	0	0	0	0	0	0	0	46
4	0.008436	518.6	57	0	0	0	0	0	0	0	57
5	0.016194	575.5	76	0	0	0	0	0	0	0	76
6	0.01634	585.3	79	0	0	0	0	0	0	0	79
7	0.016734	590.1	82	0	0	0	0	0	0	0	82
8	0.024679	591.3	82	0	0	0	0	0	0	0	82
9	0.024681	591.5	82	0	0	0	0	0	0	0	82
10	0.02818	593.4	82	0	0	0	0	0	0	0	82
11	0.03001	594.1	82	0	0	0	0	0	0	0	82
12	0.041833	595.05	82	0	0	0	0	0	0	0	82

The graph of pushover for Special Moment Resisting Frame is given below:-

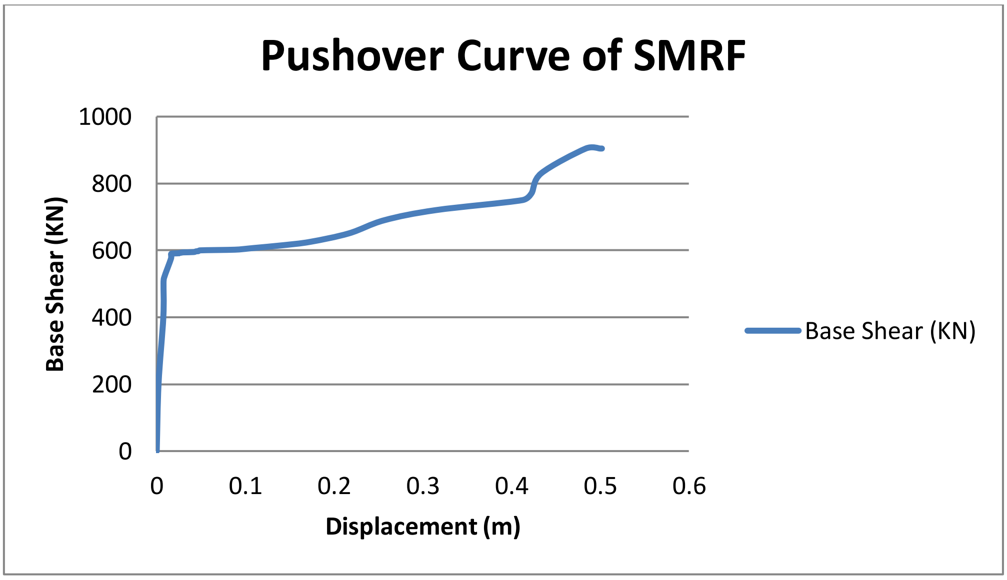

Fig 4.1 Pushover Curve of SMRF.

4.3 Pushover Curve of OMRF due to Pushover in X-Direction

4.3.1 Resultant Base Shear Vs. Monitored Displacement

After analysis the Ordinary moment resisting frame, the details is given below in the graph of resultant base shear vs. monitored displacement due to apply the pushover analysis in X-direction:-

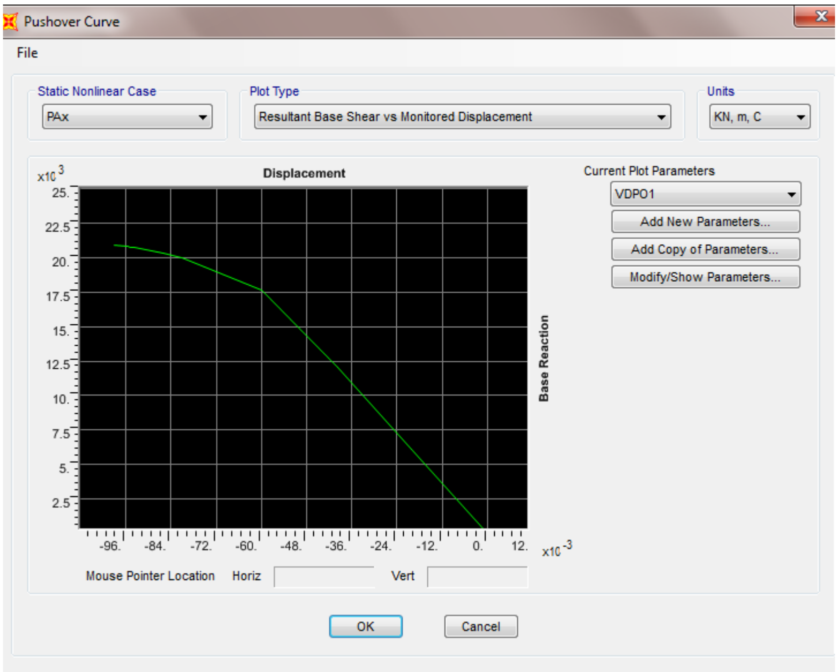

Fig 4.2 Resultant Base Shear Vs. Monitored Displacement in X

4.3.2 FEMA 356 Coefficient Method

After analysis the Ordinary moment resisting frame, the details is given below in the graph of FEMA 356 Coefficient method due to apply the pushover analysis in X-direction:-

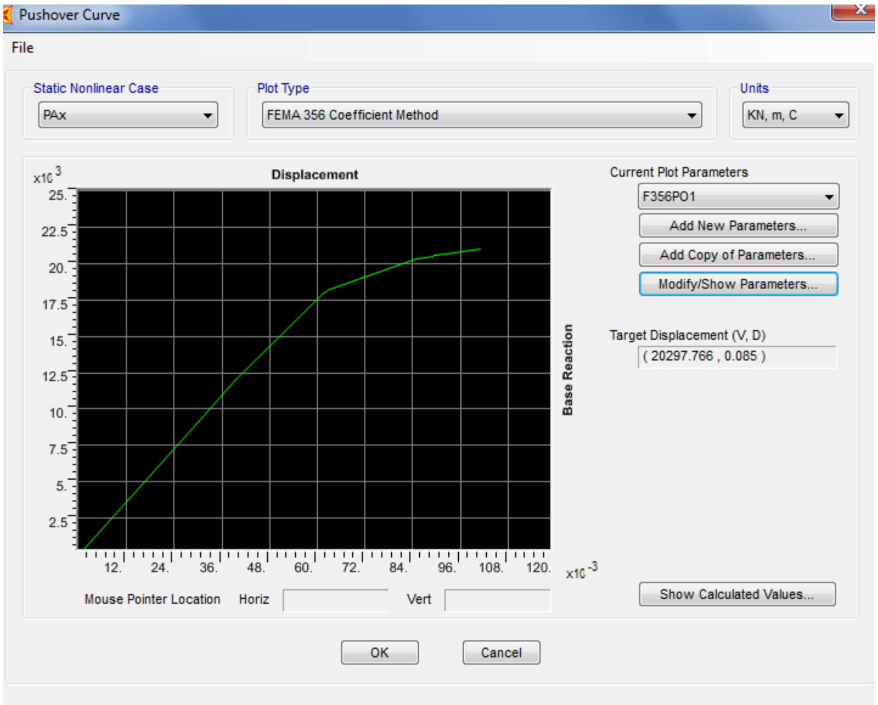

Fig 4.3 FEMA 356 Coefficient Method in X

4.4 Pushover Curve of OMRF due to Pushover in Y-direction

4.4.1 Resultant Base Shear Vs. Monitored Displacement

After analysis the Ordinary moment resisting frame, the details is given below in the graph of resultant base shear vs. monitored displacement due to apply the pushover analysis in Y- direction:-

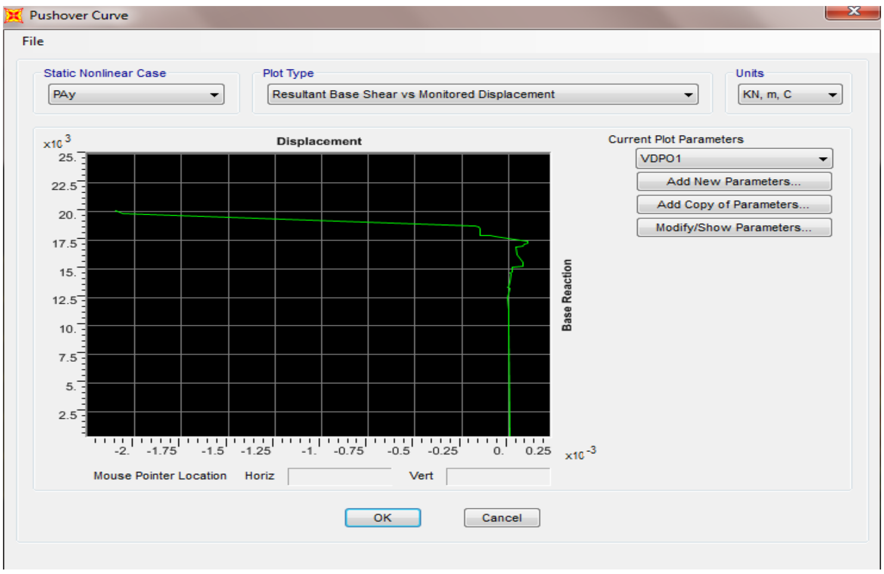

Fig 4.4 Resultant Base Shear Vs. Monitored Displacement in Y

4.4.2 FEMA 356 Coefficient method

After analysis the Ordinary moment resisting frame, the details is given below in the graph of FEMA 356 Coefficient method due to apply the pushover analysis in Y-direction:-

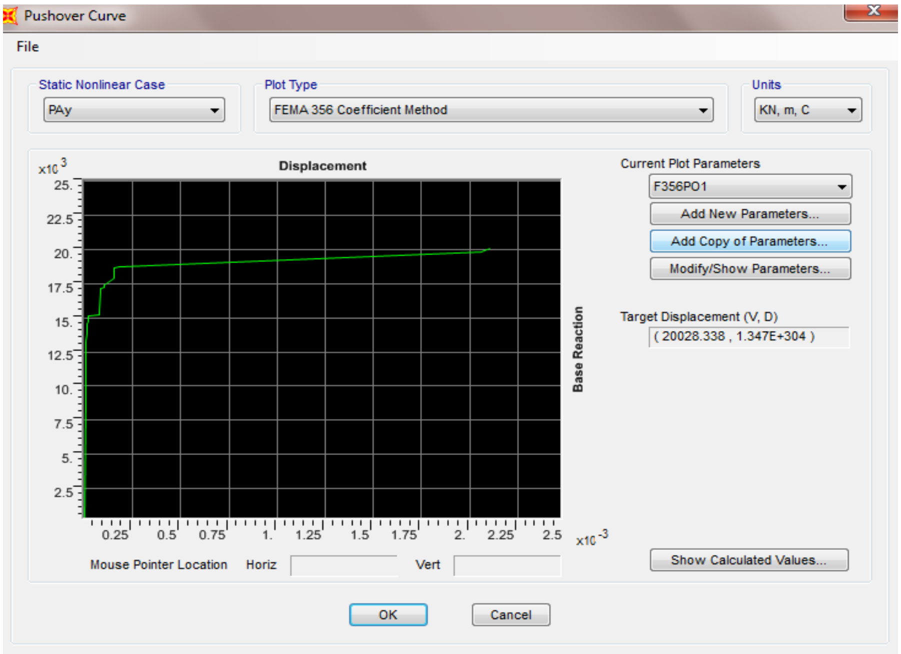

Fig 4.5 FEMA 356 Coefficient method in Y

4.5 Step By Step Hinges Formation in SMRF

In the Special Moment Resisting Frame, the hinges formation step by step due to apply the pushover analysis in the X-direction:-

4.5.1. Hinges at step 0

Due to pushover analysis in the X-direction at the step 0, there is no hinges formation that means the structure is safe at step 0, which figure is given below

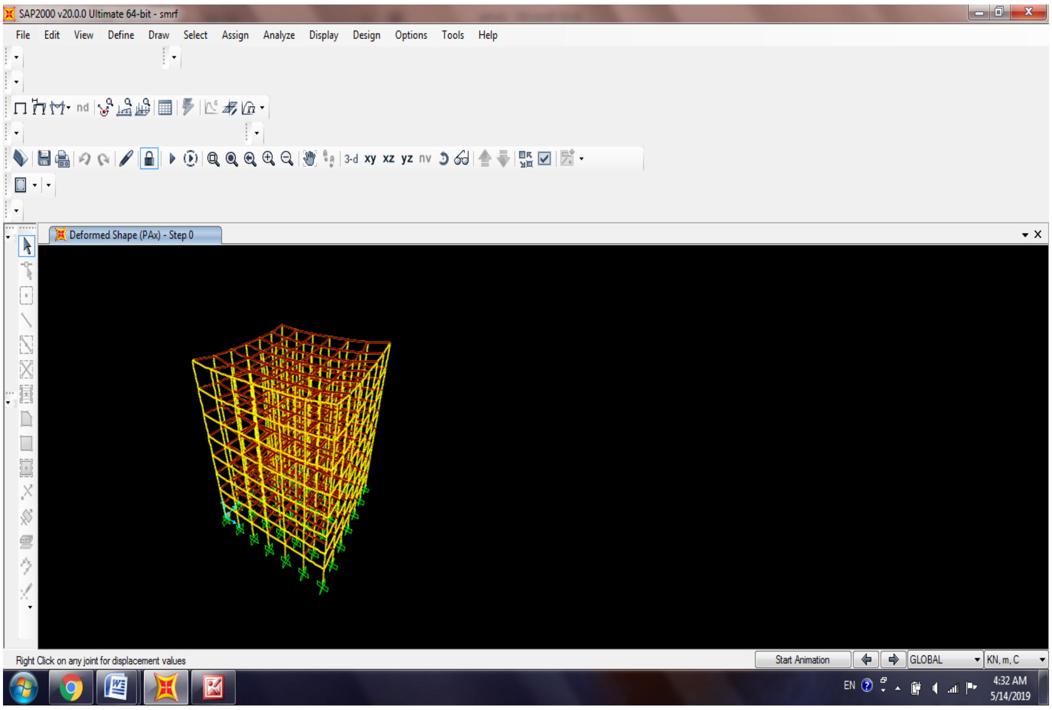

Fig 4.6 Hinges at Step-0

4.5.2 Hinges at step 1

Due to pushover analysis in the X-direction at the step 1, there is no hinges formation that means the structure is safe at step 0, which figure is given below

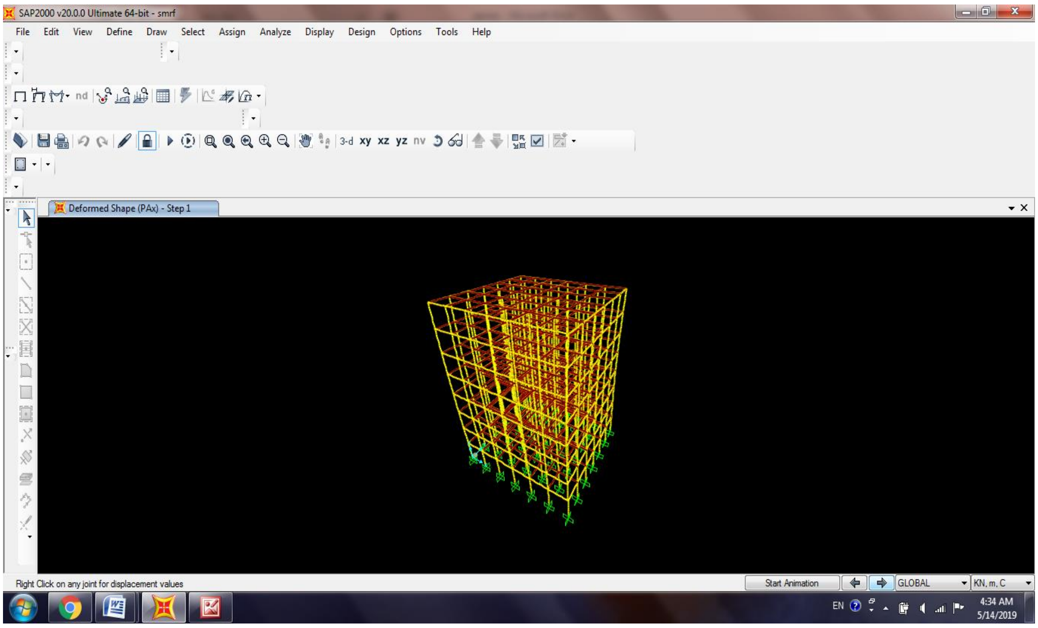

Fig 4.7 Hinges at Step-01

4.5.3 Hinges at Step 2

Due to pushover analysis in the X-direction at the step 2, there are seven hinges formed that hinged showing the yielding the support of the structure, which figure is given below

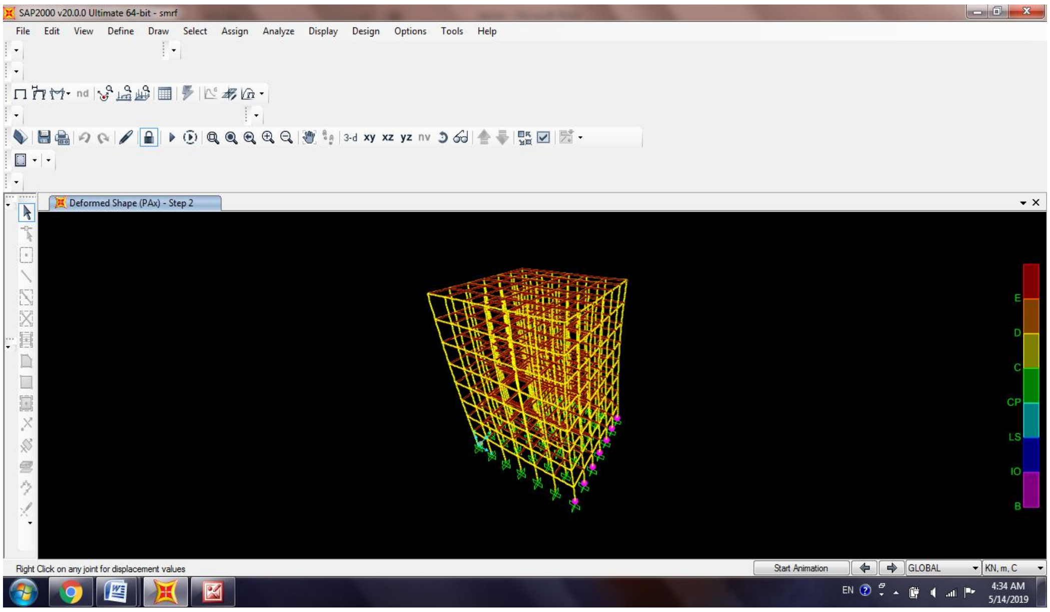

Fig 4.8 Hinges at Step-02

4.5.4 Hinges at Step 3

Due to pushover analysis in the X-direction at the step 3, there are 46 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

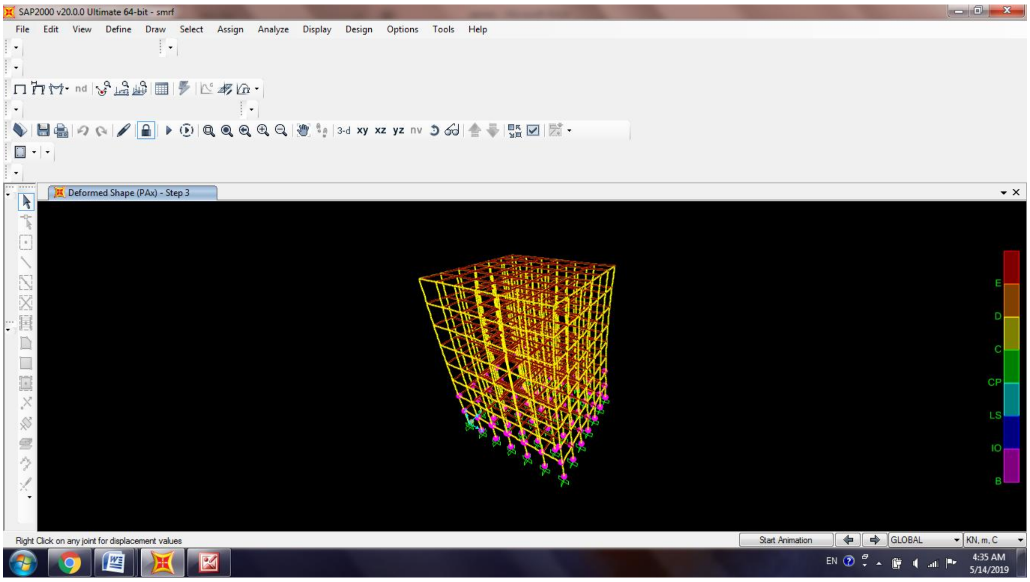

Fig 4.9 Hinges at Step-03

4.5.5 Hinges at Step 4

Due to pushover analysis in the X-direction at the step 4, there are 57 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

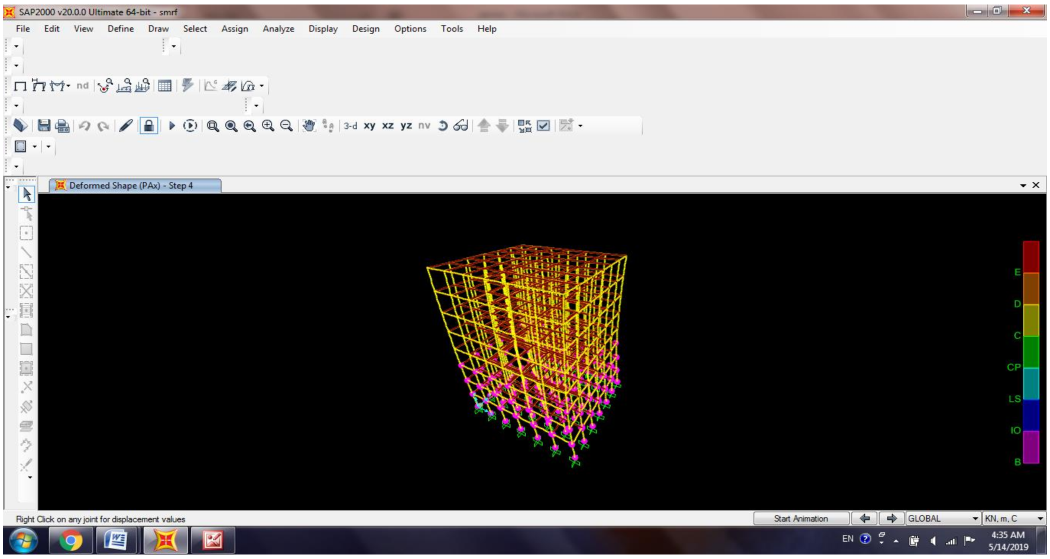

Fig 4.10 Hinges at Step-04

4.5.6 Hinges at Step 5

Due to pushover analysis in the X-direction at the step 5, there are 76 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

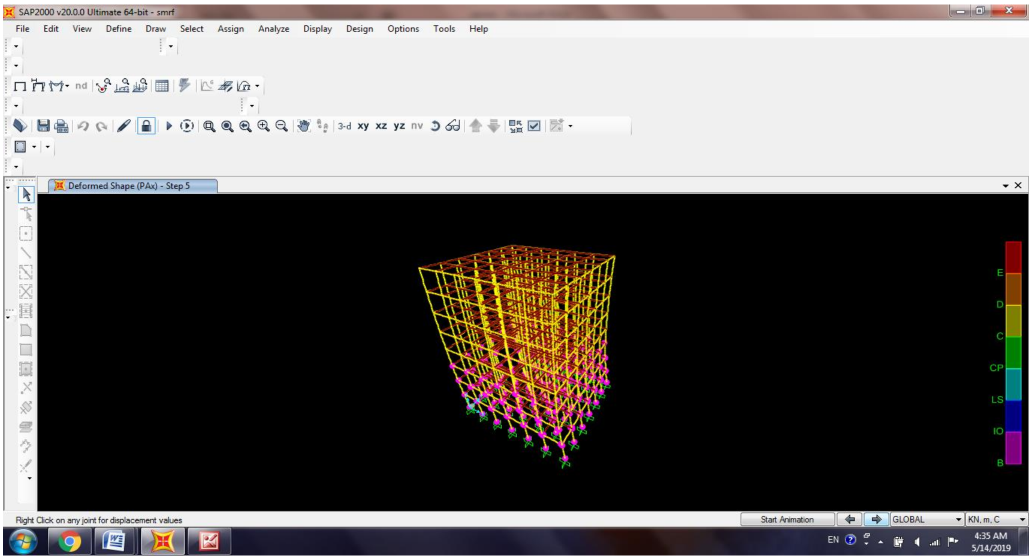

Fig 4.11 Hinges at Step-05

4.5.7 Hinges at Step 6

Due to pushover analysis in the X-direction at the step 6, there are 79 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

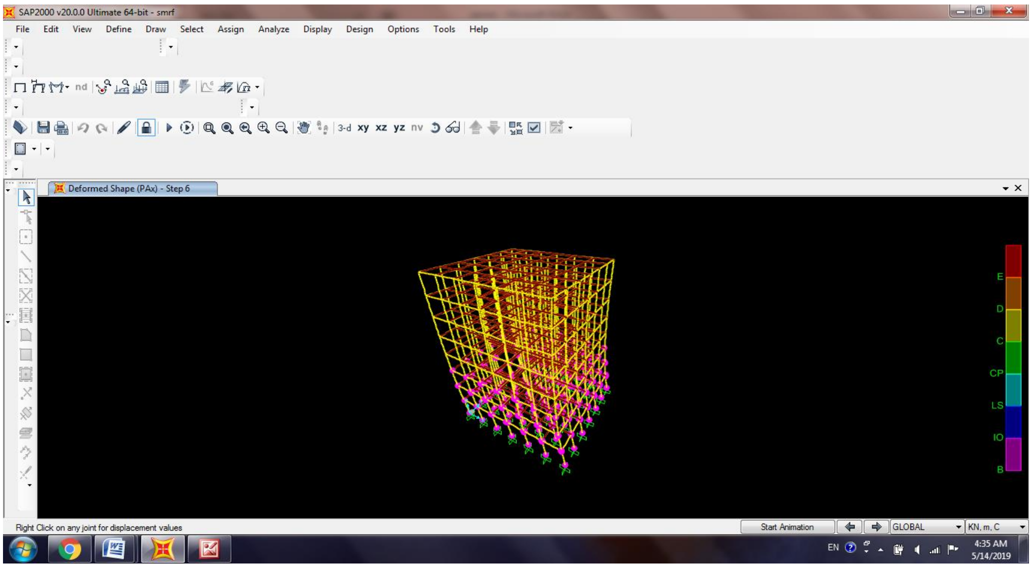

Fig 4.12 Hinges at Step-06

4.5.8 Hinges at step 7

Due to pushover analysis in the X-direction at the step 7, there are 82 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

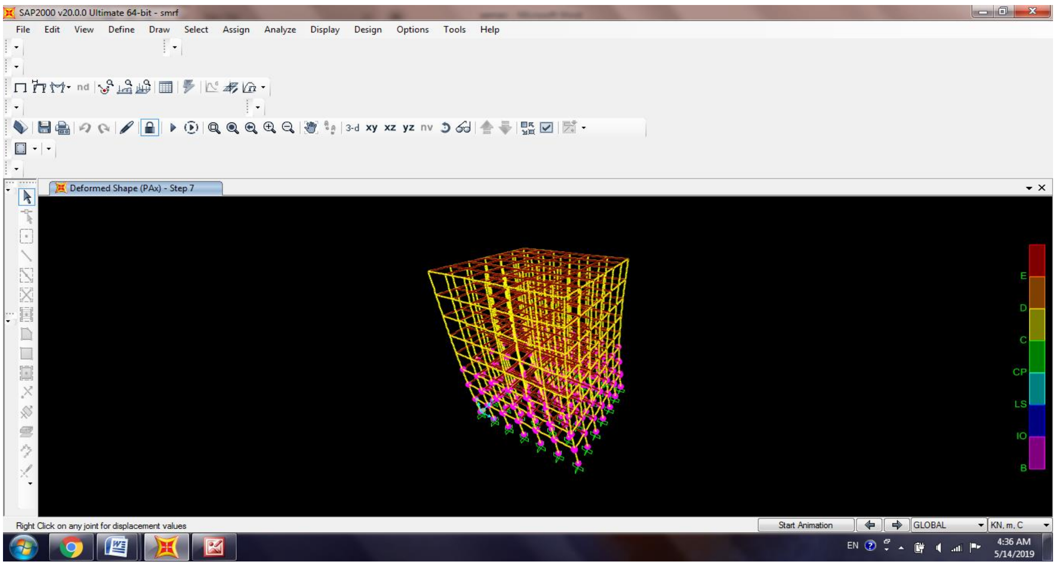

Fig 4.13 Hinges at Step-07

4.5.9 Hinges at step 8

Due to pushover analysis in the X-direction at the step 8, there are 82 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

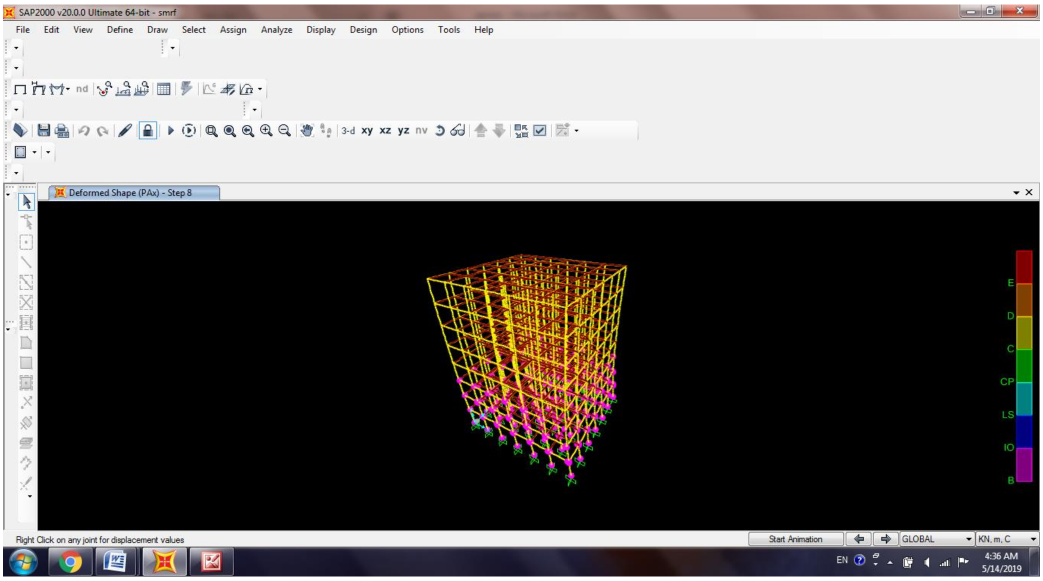

Fig 4.14 Hinges at Step-08

4.5.10 Hinges at step 9

Due to pushover analysis in the X-direction at the step 9, there are 82 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

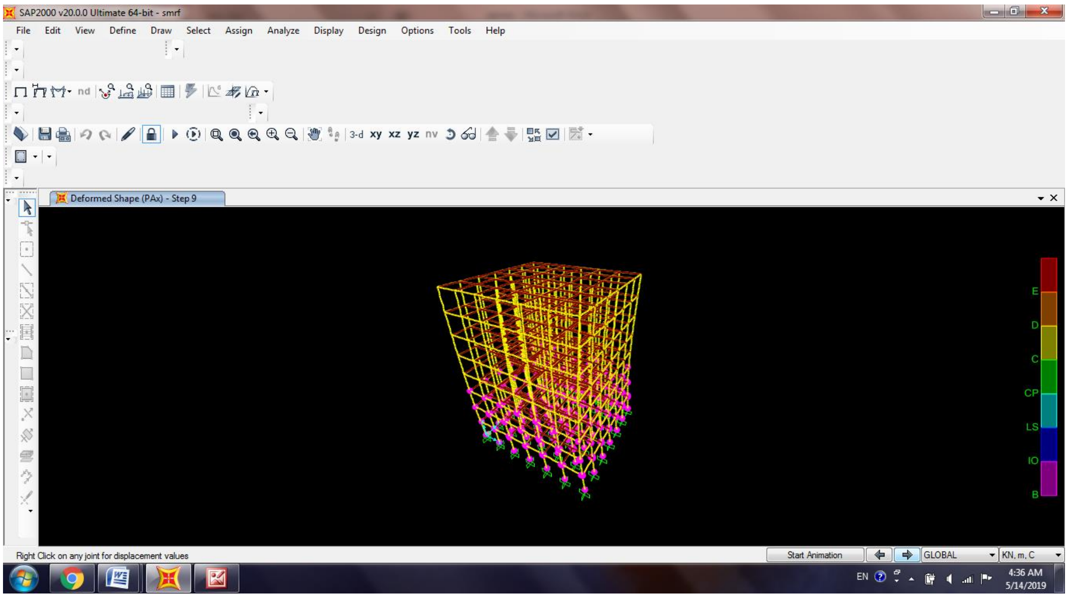

Fig 4.15 Hinges at Step-09

4.5.11 Hinges at step 10

Due to pushover analysis in the X-direction at the step 10, there are 82 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

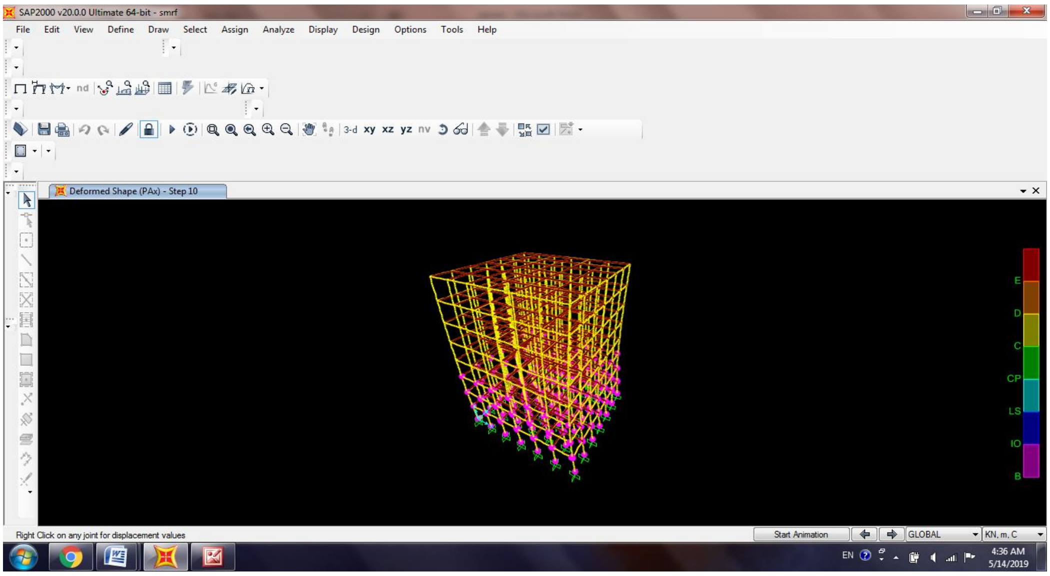

Fig 4.16 Hinges at Step-10

4.5.12 Hinges at step 11

Due to pushover analysis in the X-direction at the step 11, there are 82 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

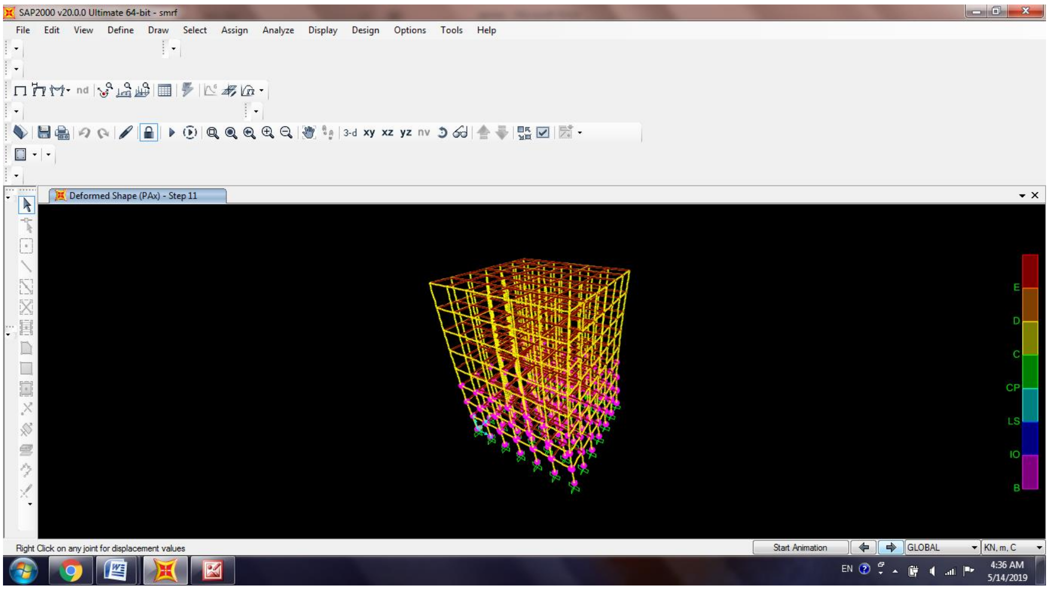

Fig 4.17 Hinges at Step-11

4.5.13 Hinges at step 12

Due to pushover analysis in the X-direction at the step 12, there are 82 hinges formed that hinged showing the yielding the support of the structure, which figure is given below

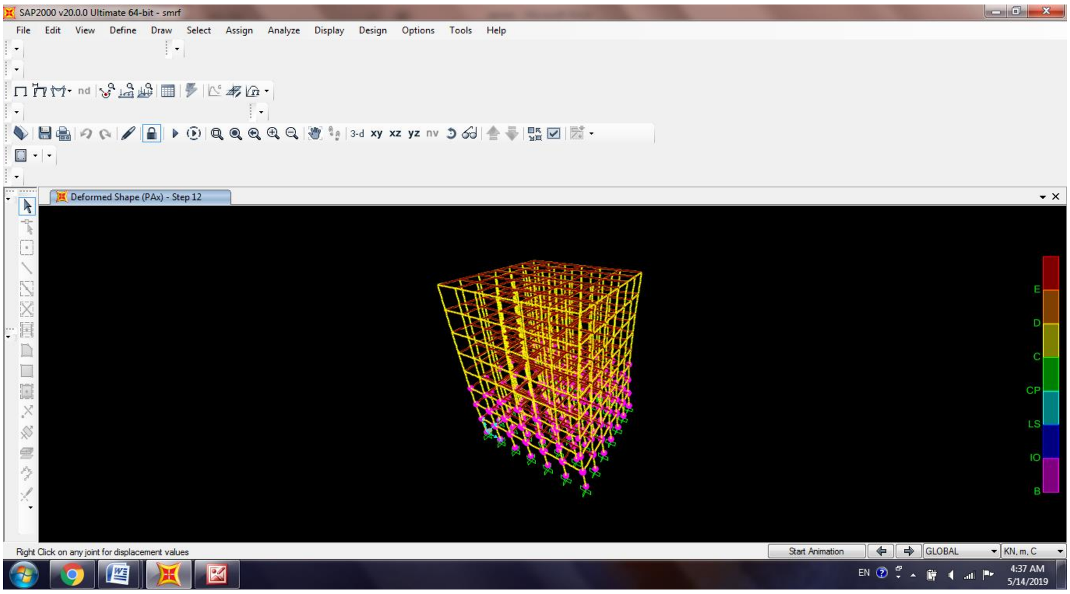

Fig 4.18 Hinges at Step-12

4.6 Displacement At Joint In Special Moment Resisting Frame at step-1

The joint displacement due to apply pushover analysis in the X-direction in the special Moment Resisting Frame (SMRF) at step-1 is given below:-

Table 4.3 Displacement in SMRF

Joint No	U1(m)	U2 (m)	U3 (m)
1	-0.000001093	-9.207E-07	-0.000189
2	-0.000001022	-6.851E-07	-0.000235
3	-0.000001013	-3.471E-07	-0.000246
4	-0.000001011	0.000005256	-0.000247
5	-0.000001013	0.000025	-0.000246

6	-0.000001022	0.000048	-0.000235
7	-0.000001093	0.000063	-0.000189
8	-8.286E-07	-8.161E-07	-0.00023
9	-8.143E-07	-6.454E-07	-0.000289
10	-8.069E-07	-3.326E-07	-0.000303

4.7 Displacement At Joint In Ordinary Moment Resisting Frame at step-1

The joint displacement due to apply pushover analysis in the X-direction in the Ordinary Moment Resisting Frame (OMRF) at step-1 is given below:-

Table 4.4 Displacement in OMRF

Joint No	U1(m)	U2 (m)	U3 (m)
1	-0.004342	-0.000011	-0.000554
2	-0.004355	-0.000008147	-0.000616
3	-0.00436	-0.00000422	-0.000629
4	-0.004362	-2.422E-14	-0.000631
5	-0.00436	0.00000422	-0.000629
6	-0.004355	0.000008147	-0.000616
7	-0.004342	0.000011	-0.000554
8	-0.004352	-0.000004004	-0.000267
9	-0.00436	-0.000002984	-0.000328
10	-0.004366	-0.000001757	-0.000342

The graph of displacement joint due to Pushover in X-direction at step-1 for SMRF and OMRF is given below:-

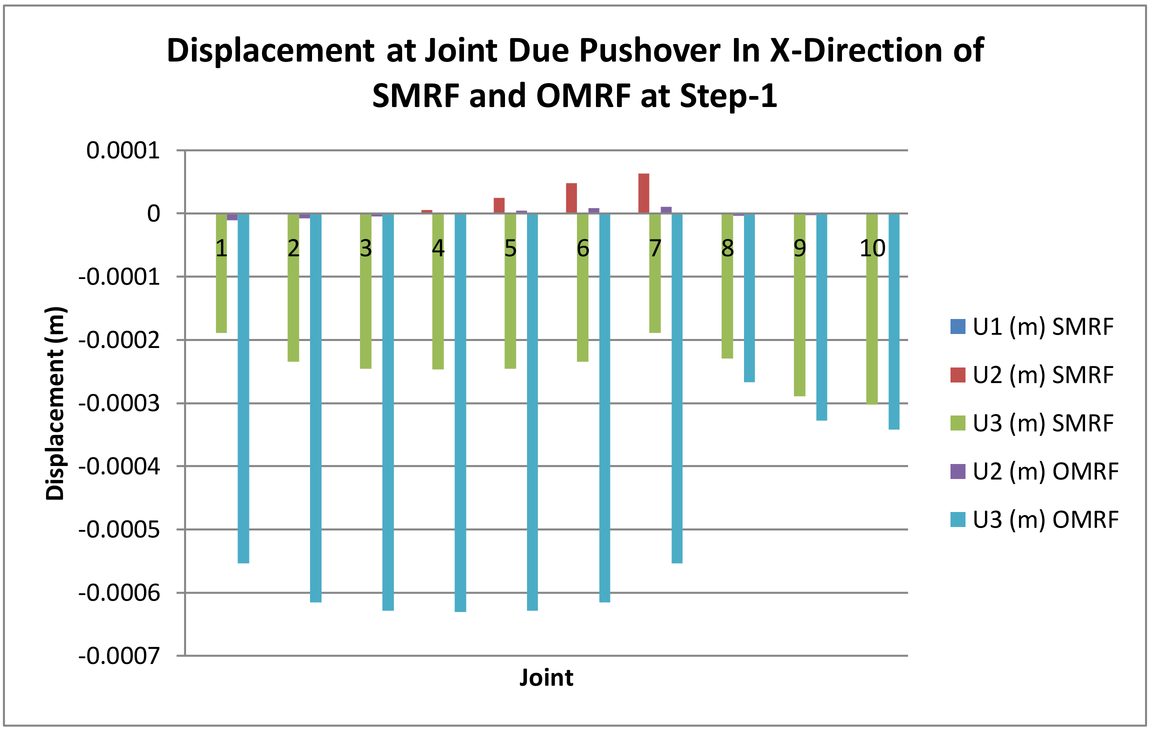

Fig-4.19: Displacement at Joint Due Pushover In X-Direction of SMRF and OMRF at Step-1

CHAPTER 5
CONCLUSIONS

After analyzing the above two model which are Special Moment Resisting Frame and Ordinary Moment Resisting Frame by the pushover analysis with respect to the response spectrum method then following conclusions are obtained which is given below:-

- In the model of Special Moment Resisting Frame and Ordinary Moment Resisting Frame the value of the modal time period and frequency is almost same.
- In this model, there is no plastic hinges formed but yielding point is formed in the both Special Moment Resisting Frame and Ordinary Moment Resisting Frame. The Number of point of the yielding in the Special Moment Resisting Frame is low as compared to the Ordinary Moment Resisting Frame.
- The value of the joint displacement increasing from lower step number to higher step number in the both Special Moment Resisting Frame and Ordinary Moment Resisting Frame. This is representing that in the building the chances of the plastic hinges increase at the higher step number.
- In the Ordinary Moment Resisting Frame we found that in the local direction of the x-axis, the value of the displacement i.e. (U1) maximum as compared to the all joint displacement.
- In the both Special Moment Resisting Frame and Ordinary Moment Resisting Frame, there is only yielding point found which is mostly below the top second floor of the building.
- The value of the displacement due to resultant base shear vs. monitored displacement is always negative but the value of the displacement due to FEMA-356 coefficient method is always positive

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ANNEXURE

PERFORMANCE ASSESSMENT OF MULTI-STOREYED REINFORCED CONCRETE SMRF AND OMRF BUILDINGS USING SAP2000: A REVIEW

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Abstract - To resist earthquakes, Reinforced concrete special moment frames are utilized as part of seismic force-resisting structures in buildings. Columns, Beams, and beam-column joints in moment frames are balanced & detailed to resist flexural, axial, & shearing movements. The main purpose of current investigation is the study of comparative performance of SMRF and OMRF frames, designed as per IS codes, via nonlinear analysis. Software program is utilized to design & model the structures. A performance of SMRF structure & OMRF structure with no infill & fixed support conditions result states that the base shear capacity of OMRF structures is 20 to 40% additional than that of SMRF structures. The behavior of SMRF structure & OMRF structure with no infill & hinged support condition result states that OMRF structures resist 2040% additional base shear than that be resisted by SMRF structures. The behavior of SMRF building with fixed & hinged support conditions states that an act of SMRF structures under fixed & hinged support condition is an identical. The SMRF structures with similar no. of bays and diverse no. of storeys experiment states that all the SMRF structures deliberated has exactly the similar amount of initial slope in the push over curve. The SMRF structures with similar no. of storeys & diverse no. of bays experiment gives the result that the no. of bays play huge part in the immovability of the structures measured for the current investigation.

Key Words: **SMRF, OMRF, Base Shear, Fixed Support, Hinged Support, Nonlinear Analysis, Infill, SAP 2000 etc.**

1. INTRODUCTION

SMRF introduced in India about 1993. IS 13920(1993) was utilized for proportioning and detailing of SMRF in India, which later was written in 2002. To resist earthquakes, Reinforced concrete special moment frames are utilized as part of seismic force-resisting structures in buildings. Columns, Beams, and beam-column joints in moment frames are balanced & detailed to resist flexural, axial, & shearing movements. Due to these forces structure sways over many displacement phases throughout strong earthquake ground shaking. Moment frames are mostly chosen as the seismic force-resisting arrangement when architectural space planning tractability is vital. Concrete moment frames are chosen for Seismic Zone III, IV or V, these are desired to be detailed as special RC moment frames. Balancing & detailing necessities for a special moment frame will allow the frame to securely go through wide inelastic deformations which are predictable in these seismic zones. It can be utilized in Seismic Zone I or II, though it will not be the best inexpensive design. It is essential to consider strength and stiffness both in the design of special moment frames. The design base shear eqn. of present building codes integrate a seismic force reduction factor R that shows the degree of inelastic response predictable for design-level ground motions, as well as the ductility capacity of the framing system. A SMRF should be predictable to retain multiple cycles of inelastic response if it experiences design level ground motion. When a structure sways during an earthquake, the spreading of damage over height depends on the spreading of lateral drift. If the structure has weak columns, drift tends to focus in one or a few stories, and may go beyond the drift capacity of the columns. On the other side, if columns deliver a stiff and strong spine over the structure height, drift will be more equivalently spread, and confined loss will be decreased. These type of failure is known as Beam Mechanism or Sway Mechanism. It is a design standard that should be firmly involved though designing SMRF. Structural Designers implements the strong-column/weak-beam standard by requiring that the addition of column strengths exceed the addition of beam strengths at each beam-column link of a special moment frame. Ductile response needs that members yield in flexure, and that shear failure be ignored. Shear failure, exclusively in columns, is comparatively brittle and can lead to quick loss of lateral strength and axial load-carrying capacity. Column shear failure is the maximum frequently mentioned reason of concrete structure failure and collapse in earthquakes. Shear failure is ignored by using of a capacity-design methodology. The common methodology is to classify flexural yielding regions, design those regions for code required moment strengths, and then determine design shears based on equilibrium supposing the flexural yielding regions form possible moment strengths.

The possible moment strength is estimated using processes that develop a higher estimation of the moment strength of the designed cross-section. Mostly hoops are provided at the ends of beams and columns, also at beam-column joints. It needs to be effective, hooks should be closed by 135° rooted in the concrete, and it avoids hooks to be opened if the cover of concrete removed. Cross-ties should involve longitudinal reinforcement around the perimeter to increase confinement efficiency. Hoops need to be closely distributed lengthwise of longitudinal axis of the member, both to restrain the concrete and confine buckling of longitudinal reinforcement. Cross-ties, which generally have 90° and 135° hooks to ease construction, must have their 90° and 135° hooks alternated along the length of the member to raise confinement efficiency. Especially if axial loads are low than shear strength reduces in members subjected to multiple inelastic deformation reversals. In these types of members it is needed that the involvement of concrete to shear resistance be ignored, that is, Vc= 0. So, shear reinforcement is essential to resist the whole shear force. Loss of concrete cover due to severe seismic loading can outcome as decrease development and lap-splice strength of longitudinal reinforcement. Lap splices should be provided away from maximum moment sections and must have locked hoops to restrain the splice in the event of cover spalling. Current study shows on several characteristics associated to the performance of SMRF buildings. The main purpose of current investigation is the study of comparative performance of SMRF and OMRF frames, designed as per IS codes, via nonlinear analysis. The more genuine performance of the OMRF and SMRF building needs modelling the stiffness and strength of the infill walls. The differences in the sort of the infill walls utilizing in Indian constructions are substantial. On the basis of modulus of elasticity and the strength, it may be categorized as strong or weak. SMRF buildings are generally built in earthquake prone nations like India since they offer much greater ductility. Failures perceived in previous earthquakes illustrate that the collapse of such buildings is primarily due to the development of soft-storey mechanism in the ground storey columns.

1.1 MOMENT RESISTING FRAMES

It is a frame which are formed by Beams and columns with a rigidly jointed connection. It's basically resist the flexure.

1.2 SPECIAL MOMENT-RESISTING FRAME

SMRF is designed and detailed as per IS 13920 code which delivers additional ductility requirements to the frame.

1.3 ORDINARY MOMENT RESITING FRAME

As per IS 456, a frame is designed is an ordinary moment resisting frame. Special ductility provisions as per IS 13920 is not considered.

1.4 OVERVIEW OF SAP2000

SAP2000 is a user friendly software to perform:
Modeling, Analysis, Design, and Reporting. SAP2000 has a wide selection of templates for quickly starting a new model. The frame element uses a general, three-dimensional, beam column formulation which includes the effects of biaxial bending, torsion, axial deformation, and biaxial shear deformations. SAP2000 has a built-in library of standard concrete, steel and composite section properties of both US and International Standard sections.

- Accuracy of the solution,
- Confirmation with the Indian Standard Codes,
- Resourceful nature of solving any type of problem,
- User friendly interface.

2. LITERATURE REVIEW

Some research has already been done on special moment resisting frames and ordinary moment resisting frames.

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Concluded that with increase in the number of bays Redundancy factor is also increases and Response reduction factor shows an increasing trend for all frames. Hence the frames with more number of bays possess higher redundancy. With number of bays in x directions ductility factor is increasing but in y direction it looks like there is no flow for that. It is revealed that value of Response reduction factor acquired is critical in the direction with less number of bays. Response reduction values should be taken as the least from both directions during design purposes with ductility and redundancy also to be considered.

Mukesh Rai & Prof. M.C. Paliwal found that the special moment resisting frame is more efficient than ordinary moment resisting braced type frame and SMRF reduces moments means reduces area of steel and also concluded that the special moment resisting frame is more efficient than ordinary moment resisting types frame and SMRF reduces nodal displacement means reduction in size of section.

Jay Prakash Kadali, M.K.M.V.Ratnam, Dr. U Ranga Raju found that the buildings designed as SMRF perform much better compared to the OMRF building. The ductility of SMRF buildings is almost 10 to 33% more than the OMRF buildings in all cases, the reason being the heavy confinement of concrete due to splicing and usage of more number of stirrups as ductile reinforcement. It is also found that the base shear capacity of OMRF buildings is 7 to 28% more than that of SMRF building.
The SMRF buildings with same number of bays and different number of storeys are compared. The pushover curve is plotted and it is found that the ductility and the magnitude of base shear that can be resisted, increases with increase in the number of storeys. It is observed that all the SMRF buildings considered has almost the same value of initial slope in the push over curve.

G.V.S.SivaPrasad, S. Adisesh studied both system of analysis results of OMRF & SMRF, and found that the storey drift is within permissible limit as per IS (1893 part1,clause no 7.11.1), but when compared with OMRF the SMRF structure having less story drift so the structure can resists the seismic loads more than the OMRF.

3. CONCLUSIONS

• This may be established that the SMRF structures with stronger infill consume base shear capacity of around 1.5 to 2.5 times additional than that of SMRF structures with weaker infill.

• This is instituted that all the SMRF structures deliberated has exactly the similar amount of initial slope in the push over curve.

• The behavior of SMRF building with fixed & hinged support conditions are compared. This is instituted that an act of SMRF structures under fixed & hinged support condition is an identical. So it is decided that hinged & fixed condition do not play big part in investigation.

• A performance of SMRF structure & OMRF structure with no infill & fixed support conditions are carried in comparison. This is instituted that the structures designed as SMRF execute ample superior related to the OMRF structure. Ductility of SMRF structures is nearly 75% to 200% additional than the OMRF structures in all circumstances, the object being the heavy limitation of concrete due to splicing & utilization of additional no. of rings as ductile reinforcement. This is also instituted that the base shear capacity of OMRF structures is 20 to 40% additional than that of SMRF structures.

ACKNOWLEDGEMENT

I wish to express my deepest gratitude and indebtedness to my supervisors, Mr. Mohammad Afaque Khan for his stimulating ideas, numerous constructive suggestions and guidance, continuous encouragement and invaluable support throughout this study.

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BIOGRAPHIES

Mr. Aman Ahmed was born in year 1994 in Lucknow city of Uttar Pradesh. He received his Bachelor of technology degree in Civil Engineering from Integral University, Lucknow in 2016. He is Pursuing his Master of Technology in Structural Engineering (Babu Banarasi Das University) Lucknow.

Mr. Mohd. Afaque Khan was born in 1982 in Gonda city.He received his bachelor of technology degree in Civil Engineering from ZHCET, AMU, Aligarh in 2009. In 2012, he received his Master's degree in Structural Engineering from ZHCET, AMU, Aligarh. He joined Babu Banarasi Das University, Lucknow in 2012 as a faculty. He worked as head of the department in Babu Banarasi Das Engineering College, Lucknow and is now working as an assistant professor in Babu Banarasi Das University, Lucknow (Department of Civil Engineering).

Seismic Performance Assessment of Multi-storeyed RC Special Moment Resisting Frames By Pushover Analysis

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Abstract - *In this paper study about the seismic analysis of special and ordinary moment resisting frame by the pushover analysis with the help of the SAP2000 software which is product of the Computer and Structure &Inc. The code used for seismic analysis IS CODE 1893 part1:2016. The method used in this analysis is Nonlinear static Analysis in which static analysis represent the Response Spectrum method. The main aims of this paper to study about the plastic hinges which produce after the collapse of the structure and also comparative study about the ordinary and special moment resisting frame that which one is perform better in the push over analysis. The hinges apply at the all beam and column to study about the plastic hinges in the structure. The main purpose to choose special moment resisting frame is that frame which resist the strong ground motion during the earthquake. The ordinary moment resisting frame is that frame which resists the low ground motion as compared to the special moment resisting frame. After analysis we can say that which frame produce little plastic hinges as compared to the other frame. The designing criteria of the Special Moment Resisting Frame and Ordinary Moment Resisting Frame are given in the Indian Standard Code 1893 part1:2016.*

***Key Words*:** SAP2000, Response Spectrum Analysis, SMRF, OMRF, Pushover Analysis, Plastic Hinges.

1. INTRODUCTION

According to Indian standards Code 1893 part1:2016, moment resisting frames are classified as Ordinary Moment Resisting Frames (OMRF) and Special Moment Resisting Frames (SMRF) with response reduction factors 3 and 5 respectively. Moment-resisting frames are commonly used in urban areas worldwide as the dominant mode of building construction. However, documented poor performance of ordinary moment frames in past earthquakes warned the international community that this structural system required special design and detailing in order to warrant a ductile behavior when subjected to the action of strong earthquake. Current design provisions assigned the highest R factor to SMRF. The elastic forces are reduced by a response reduction factor to calculate the seismic design base shear. . Present study is an attempt to evaluate the response reduction factors of SMRF and OMRF frames and to check the adequacy of R factors used by IS code containing objectives as,

(i) To find Earthquake response of frames designed as SMRF and OMRF according to IS 1893 (2016) using Pushover analysis.
(ii) To determine the Performance level of SMRF and OMRF frames using Pushover analysis.

2. Modelling

In the modeling we write the details about the model which was analyzed in SAP2000. Such as the material parameter, Section parameter, load parameter, and seismic parameter.

2.1.Material Parameter

Table-2.1: Material Parameter

Material Name	Value
Concrete	M25
Rebar	HYSD415, Mild250

2.2.Section and Seismic Parameter

Table-2.2: Section and Seismic Parameter

Beam	500mmX40mm
Column	600mmX400mm
Slab	150mm
Seismic Zone factor	0.36
SMRF	5.0
OMRF	3.0
Importance Factor	1.0
Soil Type	2nd (Medium soil)

2.3.Load Parameter

Table-2.3: Load Parameter

Dead	Auto Defined
Live	3KN/m^2
Finishing Load	1 KN/m^2
Roof	2 KN/m^2
Wall Load	15KN/m
Parapet Wall Load	7.5KN/m
EX	1893 part1:2016 (X-Direction)
EY	1893 part1:2016 (Y-Direction)

2.4.Different View of Model

The model for the SMRF and OMRF is same only value of the response reduction factor is 5 and 3 respectively

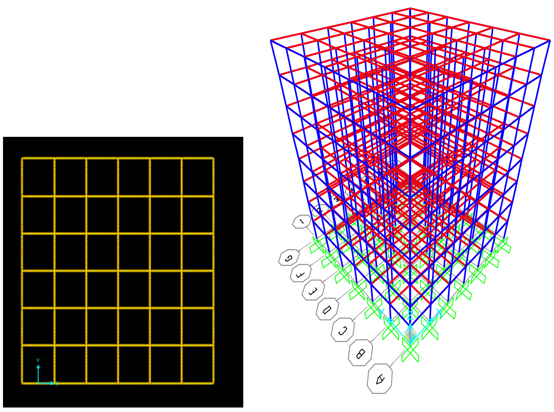

Fig-2.4: Plan and 3D View

3. Methodology

3.1.Response Spectrum Method

Response-spectrum analysis (RSA) is a linear-dynamic statistical analysis method which measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure. Response-spectrum analysis provides insight into dynamic behavior by measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a given time history and level of damping. It is practical to envelope response spectra such that a smooth curve represents the peak response for each realization of structural period.

Response-spectrum analysis is useful for design decision-making because it relates structural type-selection to dynamic performance. Structures of shorter period experience greater acceleration, whereas those of longer period experience greater displacement. Structural performance objectives should be taken into account during preliminary design and response-spectrum analysis.

3.2. Pushover Analysis

Pushover analysis is a static, nonlinear procedure to analyze the seismic performance of a building where the computer model of the structure is laterally pushed until a specified displacement is attained or a collapse mechanism has occurred as shown in Fig-3.2.The loading is increased in increments with a specific predefined pattern such as uniform or inverted triangular pattern. The gravity load is kept as a constant during the analysis. The structure is pushed until sufficient hinges are formed such that a curve of base shear versus corresponding roof displacement can be developed and this curve known as pushover curve. A typical Pushover curve is shown in Fig-3.2.The maximum base shear the structure can resist and its corresponding lateral drift can be found out from the Pushover curve.

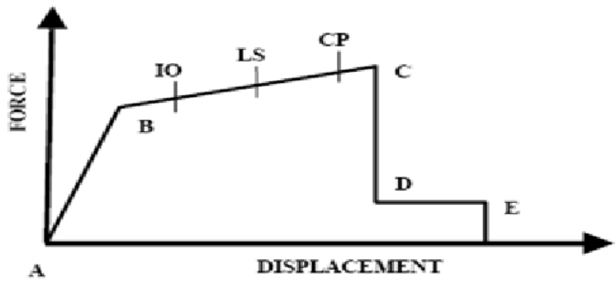

Fig-3.2: Pushover Curve

A = Original State (OL) of the structure.
B = Yielding. No deformation occur up to point B.
C = represent ultimate capacity/limit for pushover analysis.
D = Represent residual strength limit in the structure. After this limit structure initialized collapsing.
E = Represent total failure of structure. After this point hinges break down

4. Result and Discussion

After analysis the model of SMRF and OMRF following results are given below:-

4.1. Modal Period and Frequency

The modal period and frequency of the both Special Moment Resisting Frame (SMRF) and Ordinary Moment Resisting Frame (OMRF) is same

Mode	Period (sec)	Frequency (cyc/sec)
Mode1	0.644814	1.550835394
Mode2	0.533867	1.873124305
Mode3	0.53248	1.878003432
Mode4	0.212995	4.694935292
Mode5	0.175383	5.701802045
Mode6	0.17328	5.771002298
Mode7	0.12474	8.016700066
Mode8	0.102318	9.773496604
Mode9	0.098745	10.12707928
Mode10	0.088379	11.31494737
Mode11	0.071359	14.01359763
Mode12	0.068665	14.5634917

4.2. Axial force and Bending Moment at Hinges of Special Moment Resisting Frame (SMRF) due to Pushover Analysis in X-direction (Model1)

Hinges	Axial Force (P) (KN)	Moment in Local Axis Y Direction (M2) KN-m	Moment in Local Axis Z Direction (M3) KN-ms
1016H1	-47..1119	-5.283	5.8838
1016H3	-30.9167	6.6193	-8.3109
1017H1	-60.3464	-6.6567	1.0029
1017H3	-44.1511	8.2088	-1.2131
1018H1	-57.2674	-6.3205	4.625
1018H3	-41.0722	7.7999	-5.7436
1019H1	-60.7137	-6.7144	$2.484X10^{-15}$
1019H3	-44.5185	8.28	$1.698X10^{-13}$
1020H1	-60.3464	-6.6567	-1.009
1020H3	-44.1511	8.2088	1.2131
1021H1	-57.2674	-6.3205	-4.625
1021H3	-41.0722	7.7999	5.7436

4.3.Axial Force and Bending Moment At hinges of the Ordinary Moment Resisting Frame (OMRF) due to Pushover Analysis in X-direction (Model2)

Hinges	Axial Force (P) (KN)	Moment in Local Axis Y Direction (M2) KN-m	Moment in Local Axis Z Direction (M3) KN-ms
1016H1	-61.0178	-8.034	7.9012
1016H3	-43.190	10.0015	-11.0795
1017H1	-77.8944	-9.8703	8.2306
1017H3	-53.8034	13.7013	-5.0048
1018H1	-59.7309	-11.9804	7.9801
1018H3	-53.6901	9.8107	-9.1364
1019H1	-68.6408	-10.3400	1.5690
1019H3	-51.4098	13.7311	0.9452
1020H1	-73.7106	-9.4508	-5.3100
1020H3	-55.0659	13.1056	4.9736
1021H1	-68.7603	-9.4588	1.3470
1021H3	-54.9003	9.8960	8.3701

The graph of the axial forces at the selected hinges of the SMRF and OMRF is given below

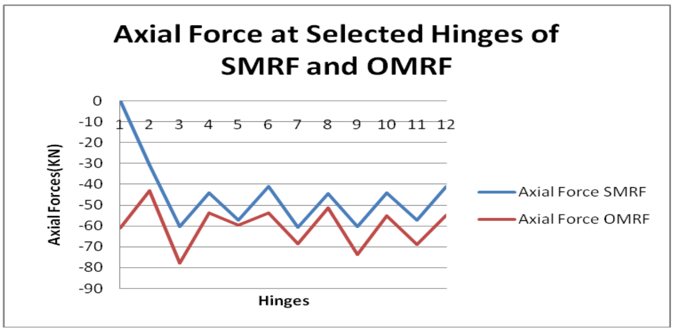

Chart-4.3: Axial Force at Selected Hinges of the SMRF and OMRF

4.4.Displacement At Joint In Special Moment Resisting Frame at step-1

The joint displacement due to apply pushover analysis in the X-direction in the special Moment Resisting Frame (SMRF) at step-1 is given below:-

Table-4.4: Displacement in SMRF

Joint No	U1(m)	U2 (m)	U3 (m)
1	-0.000001093	-9.207E-07	-0.000189
2	-0.000001022	-6.851E-07	-0.000235
3	-0.000001013	-3.471E-07	-0.000246
4	-0.000001011	0.000005256	-0.000247
5	-0.000001013	0.000025	-0.000246
6	-0.000001022	0.000048	-0.000235
7	-0.000001093	0.000063	-0.000189
8	-8.286E-07	-8.161E-07	-0.00023
9	-8.143E-07	-6.454E-07	-0.000289
10	-8.069E-07	-3.326E-07	-0.000303

4.5.Displacement At Joint In Ordinary Moment Resisting Frame at step-1

The joint displacement due to apply pushover analysis in the X-direction in the Ordinary Moment Resisting Frame (OMRF) at step-1 is given below:-

Table-4.5: Displacement in OMRF

Joint No	U1(m)	U2 (m)	U3 (m)
1	-0.004342	-0.000011	-0.000554
2	-0.004355	-0.000008147	-0.000616
3	-0.00436	-0.00000422	-0.000629
4	-0.004362	-2.422E-14	-0.000631
5	-0.00436	0.00000422	-0.000629
6	-0.004355	0.000008147	-0.000616
7	-0.004342	0.000011	-0.000554
8	-0.004352	-0.000004004	-0.000267
9	-0.00436	-0.000002984	-0.000328
10	-0.004366	-0.000001757	-0.000342

The graph of displacement joint due to Pushover in X-direction at step-1 for SMRF and OMRF is given below:-

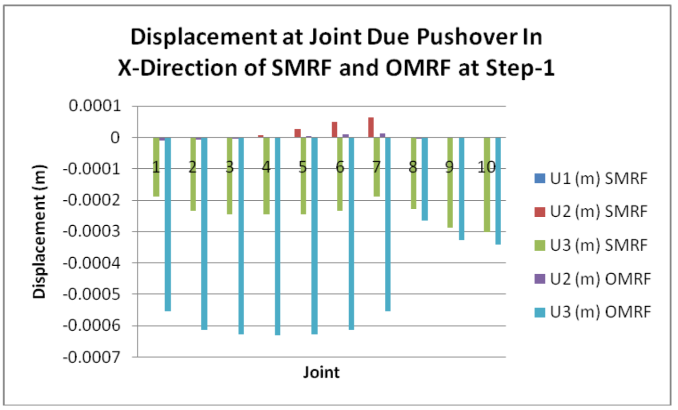

Chart-4.4: Comparative of Displacement between SMRF and OMRF

The displacement in the OMRF due U1 is maximum as compared to the other dsplacement which given above.

4.6.Plastic hinges Due to Pushover analysis in X direction in Special Moment Resisting Frame (SMRF)

Due to apply pushover analysis in special moment resisting frame in the X-direction at the step-2, there is no plastic hinges are formed but at the fixed support the number of yielding is 7 formed in the building which represent the building cannot collapse due to applies all load pattern. The figure is given below which represent the yielding at the fixed support:-

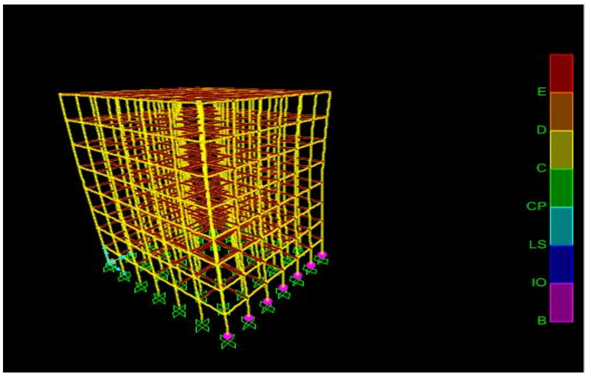

Fig-4.6: Yielding the Fixed Support In SMRF in X-direction at step-1

The pink color looking at the fixed support representing that that support is in the yielding condition.

4.7.Plastic hinges Due to Pushover analysis in Y direction in Special Moment Resisting Frame (SMRF)

Due to apply pushover analysis in special moment resisting frame in the Y-direction at the step-2, there is no plastic hinges are formed but at the fixed support the number of yielding is 28 formed in the building which represent yielding in pushover analysis in X-direction more than the apply pushover analysis in the X-direction which represent the building cannot collapse due to applies all load pattern. The figure is given below which represent the yielding at the fixed support:-

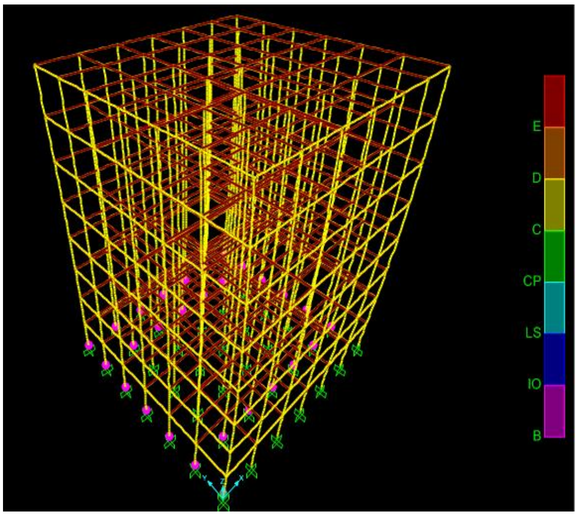

Fig-4.7: Yielding the Fixed Support In SMRF in X-direction at step-2

The pink color looking at the fixed support representing that that support is in the yielding condition.

5. Conclusions

After analyzing the above two model which are Special Moment Resisting Frame and Ordinary Moment Resisting Frame by the pushover analysis with respect to the response spectrum method then following conclusions are obtained which is given below:-

- In the model of Special Moment Resisting Frame and Ordinary Moment Resisting Frame the value of the modal time period and frequency is almost same.
- In this model, there is no plastic hinges formed but yielding point is formed in the both Special Moment Resisting Frame and Ordinary Moment Resisting Frame. The point of the yielding in the Special Moment Resisting Frame is low as compared to the Ordinary Moment Resisting Frame.
- The value of the joint displacement increasing from lower step number to higher step number in the both Special Moment Resisting Frame and Ordinary Moment Resisting Frame. This is representing that in the building the chances of the plastic hinges increase at the higher step number.
- In the Ordinary Moment Resisting Frame we found that in the local direction of the x-axis, the value of the displacement i.e. (U1) maximum as compared to the all joint displacement.
- In the both Special Moment Resisting Frame and Ordinary Moment Resisting Frame, there is only yielding point found which is mostly below the top second floor of the building.

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BIOGRAPHIES

Mr. Aman Ahmed was born in year 1994 in Lucknow city of Uttar Pradesh. He received his Bachelor of technology degree in Civil Engineering from Integral University, Lucknow in 2016. In 2017 he got enrolled in Babu Banarasi Das University Lucknow and is Pursuing his Master of Technology in Structural Engineering .

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