PERFORMANCE ASSESSMENT OF MULTISTOREYED RC SPECIAL MOMENT RESISTING FRAMES

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A thesis submitted in partial fulfillment
of the requirements for the degree of

Master of Technology

in

Structural Engineering

by

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CERTIFICATE

This is to certify that the thesis entitled, ***"PERFORMANCE ASSESSMENT OF MULTISTOREYED RC SPECIAL MOMENT RESISTING FRAMES"*** *submitted by* ***RAHUL V NAIR*** *bearing Roll No.* ***211CE2028*** *in partial fulfilment of the requirements for the award of* ***Master of Technology*** *Degree in* ***Civil Engineering*** *with specialization in* ***"Structural Engineering"*** *during 2011-13 session at National Institute of Technology, Rourkela is an authentic work carried out by him under our supervision and guidance.*

To the best of our knowledge, the matter embodied in the thesis has not been submitted to any other University/Institute for the award of any Degree or Diploma.

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ABSTRACT

KEYWORDS: *Moment resisting frames, SMRF, OMRF, Pushover analysis, Static Non-linear analysis, plastic hinges, SAP2000, ductility factor, earthquake engineering, response reduction factor.*

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.

The design criteria for SMRF buildings are given in IS 13920 (2002). In this study, the buildings are designed both as SMRF and OMRF, and their performance is compared. For this, the buildings are modelled and pushover analysis is performed in SAP2000. The pushover curves are plotted from the analysis results and the behaviour of buildings is studied for various support conditions and infill conditions. The behaviour parameters are also found for each building using the values obtained from pushover curve and is investigated.

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RAHUL V NAIR

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NOTATIONS

A_{st}	Area of steel
d	effective depth
d’	effective cover
D	overall depth of the beam
ρ	reinforcing ratio
ρ_{max}	maximum reinforcing ratio
φ	diameter of the reinforcement
f_y	yield stress of the reinforcement bar
L	span length of the beam
P_u	ultimate load
λ	load enhancement ratio
Δ	deflection
Δ_{max}	maximum deflection
μ	ductility factor

ACRONYMS AND ABBREVATIONS

ACI	American Concrete Institute
IS	Indian Standard
PSC	Portland Slag Cement
RC	Reinforced Concrete
SMRF	Special Moment resisting Frames
OMRF	Ordinary Moment Resisting Frames
FEMA	Federal Emergency Management Agency

CHAPTER 1

INTRODUCTION

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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.

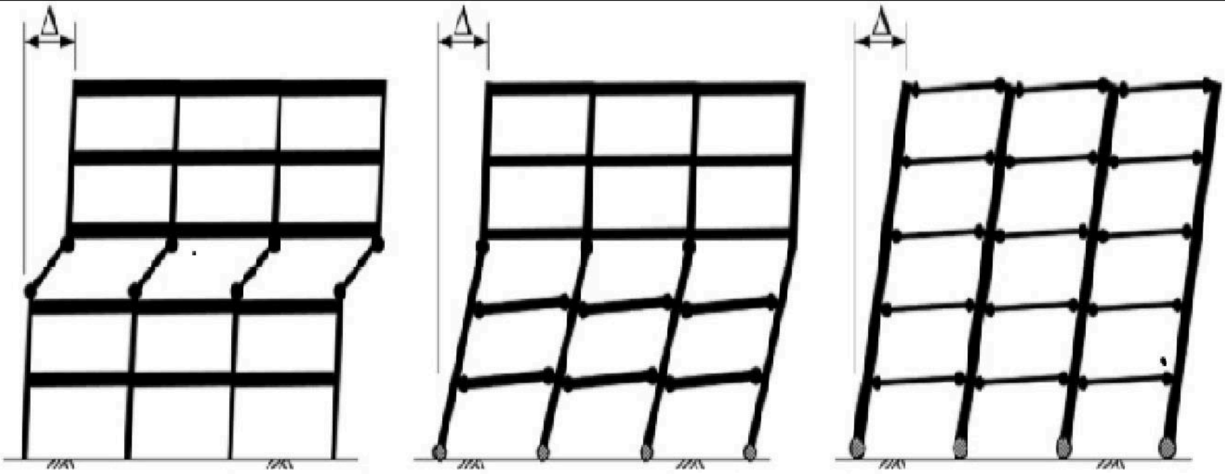
The proportioning and detailing requirements for special moment frames are intended to ensure that inelastic response is ductile. Three main goals are: (1) to achieve a strong-column/weak-beam design that spreads inelastic response over several stories; (2) to avoid shear failure; and (3) to provide 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.



a)Storey Mechanism b)Intermediate Mechanism c)Beam Mechanism

Fig. 1.1 Different failure mechanisms

1.4.2 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.

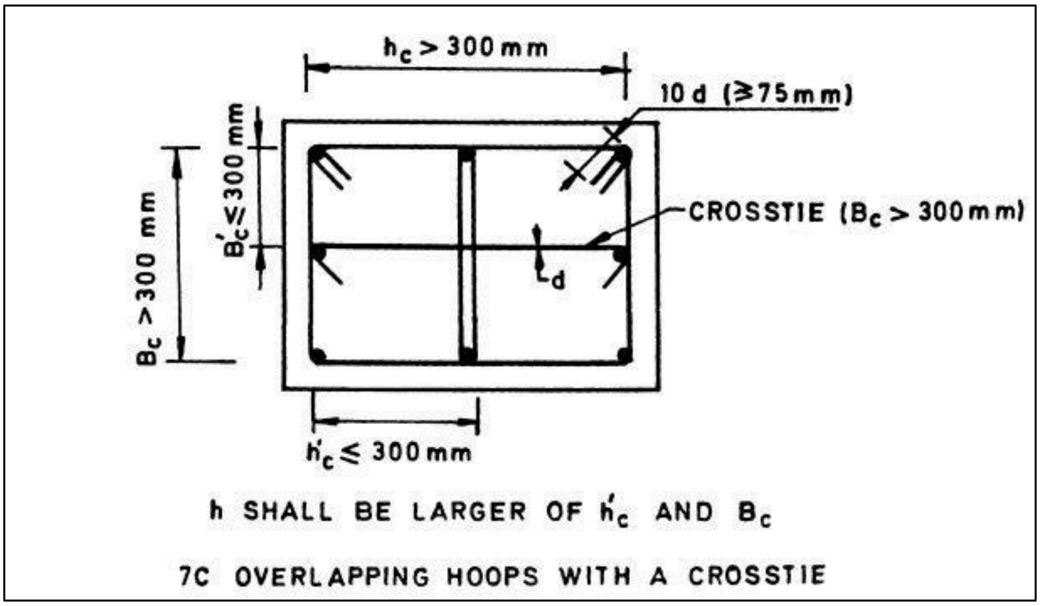


Fig 1.2 – Shear Reinforcement in beams as per IS 13920 (2002)

1.4.3 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.

1.4.3.1 CONFINEMENT FOR HEAVILY LOADED SECTIONS

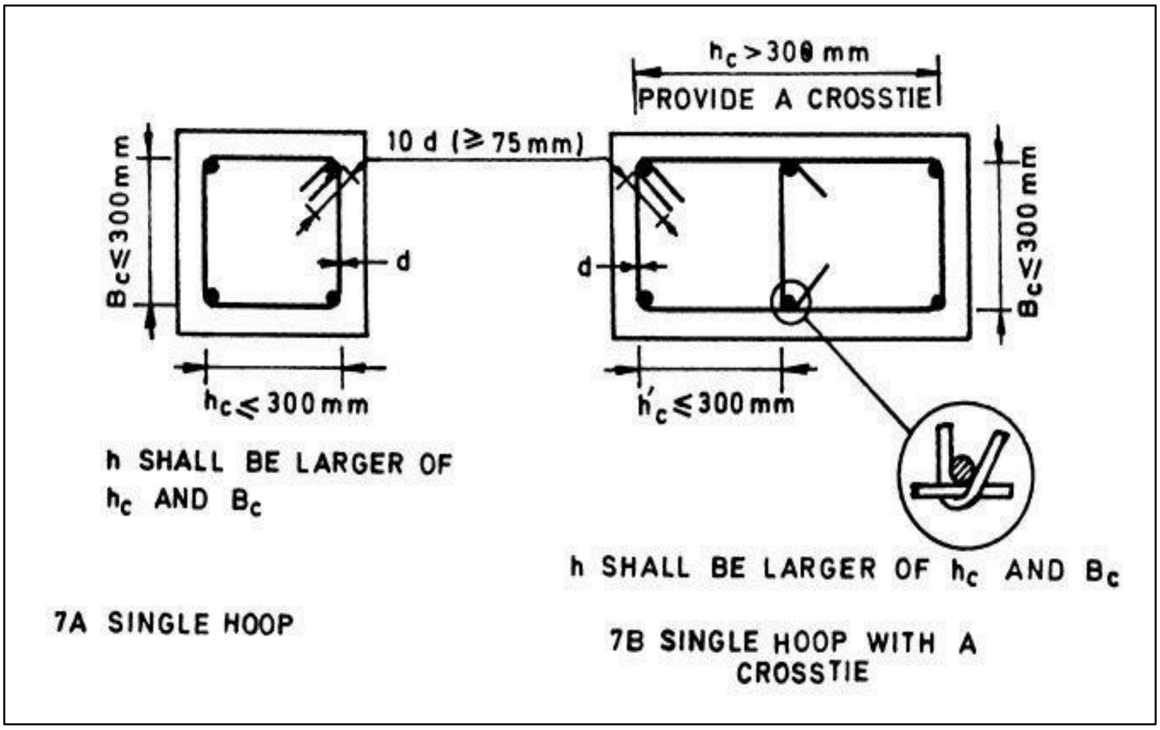
Plain concrete has relatively small usable compressive strain capacity (around 0.003), and this might limit the deformability of beams and columns of special moment frames. Strain capacity can be increased ten-fold by confining the concrete with reinforcing spirals or closed hoops. The hoops act to restrain 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.

1.4.3.2 AMPLE 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.



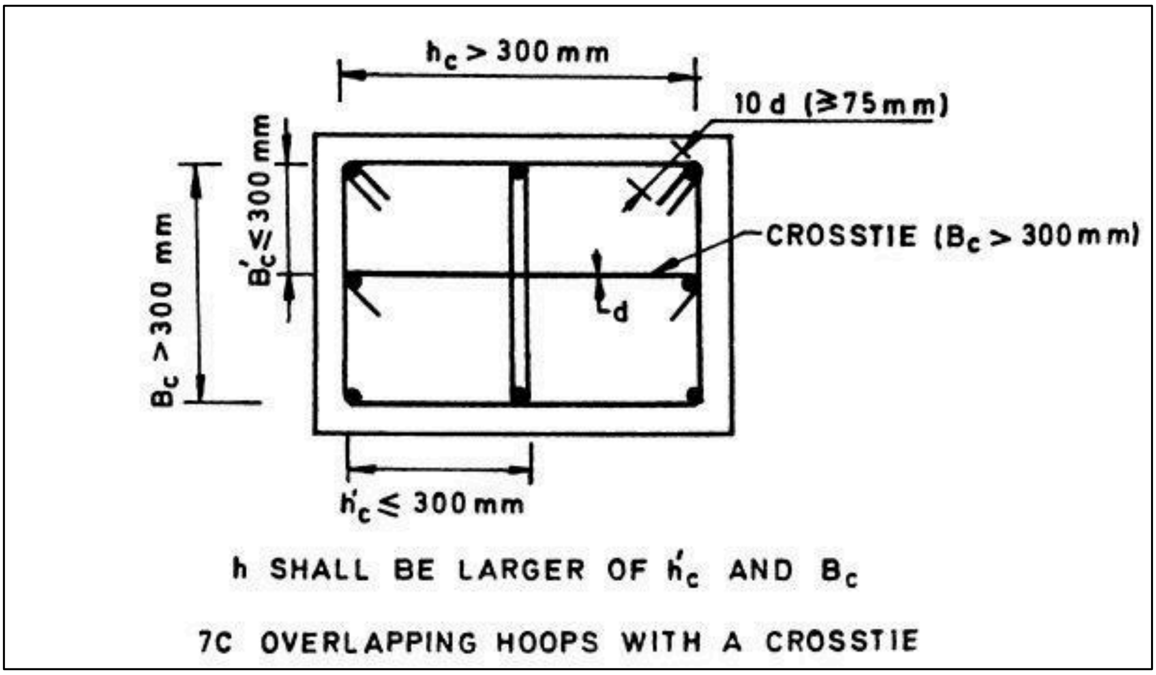


Fig 1.3 Transverse Reinforcement in columns as per IS 13920(2002)

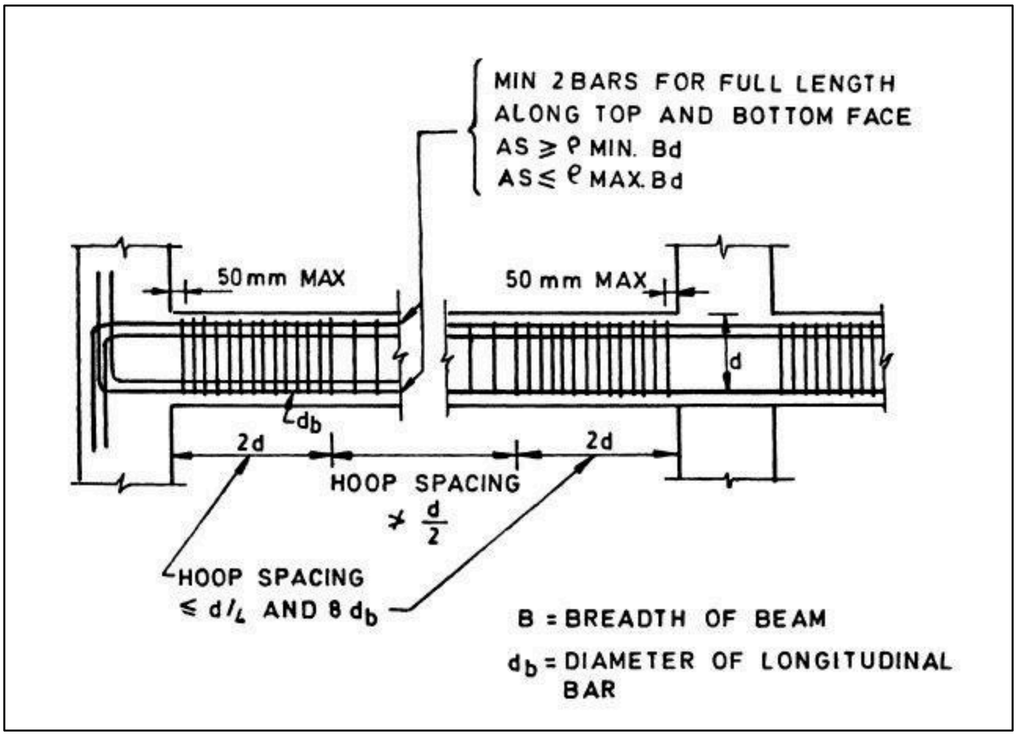


Fig 1.4 Beam Reinforcement as per IS 13920(2002)

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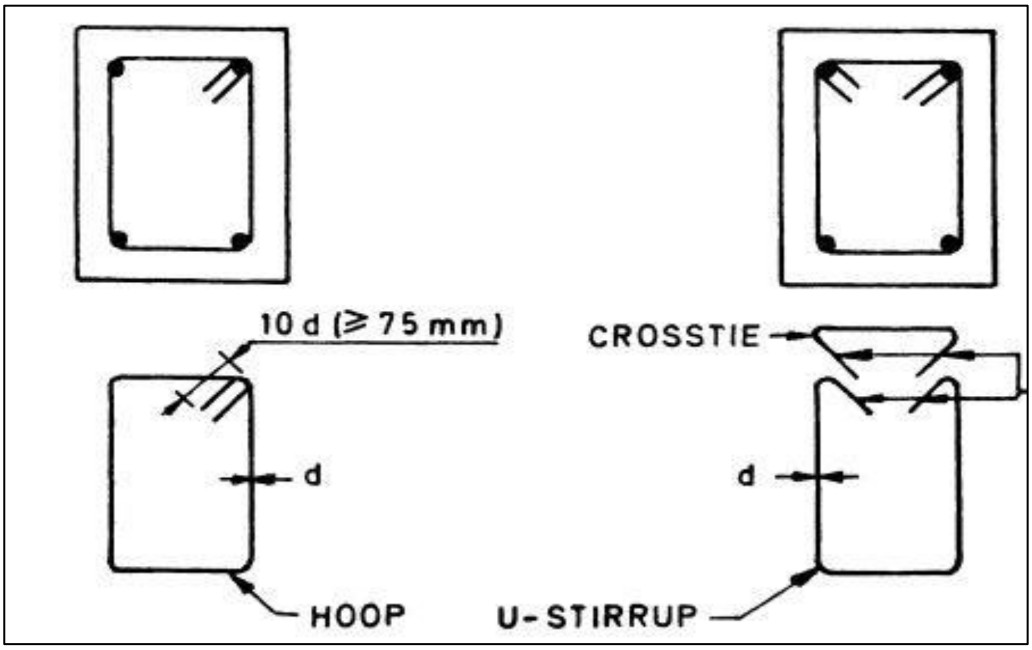

Fig 1.5 Beam Web Reinforcement as per IS 13920(2002)

1.4.3.3AVOIDANCE 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 (f_y) 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.

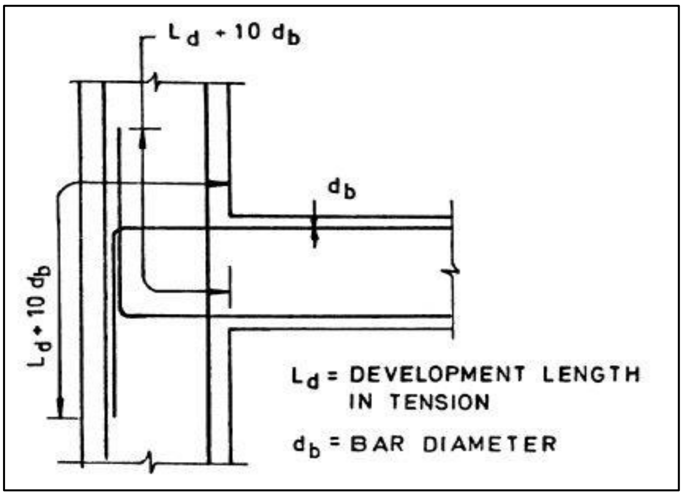

Fig 1.6 Anchorage of Beam Bars in an External Joint, IS 13920(2002)

1.5 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 modelling 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 behaviour 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

1.6 SCOPE OF THE STUDY

SMRF buildings are commonly constructed in earthquake prone countries like India since they provide much higher ductility. Failures observed in past earthquakes show that the collapse of such buildings is predominantly due to the formation of soft-storey mechanism in the ground storey columns.

The following can be considered as the scope of the study,

a) The present study deals with RC framed Buildings, regular in plan

b) This study deals with two different types of support conditions commonly used in analysis and design i.e., fixed and hinged support condition. All other types of support conditions are not considered in this project.

c) The base of the columns is assumed to be fixed.

d) Soil-structure interaction is ignored for the present study.

e) The presence of openings in infill walls is not included in the present study.

f) Concentrated plasticity based flexural hinges is considered for modelling the frame elements and it is assumed no shear failure will precede the flexural failure.

g) Nonlinear static (pushover) analysis of all building models are conducted for performance assessment, although nonlinear dynamic analysis is a superior analysis procedures, it is kept outside the scope of the present study due to time limitation.

h) Asymmetric arrangement of infill walls and Out of plane action of masonry walls is not considered

1.7 METHODOLOGY

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

(i) Review the existing literature and Indian design code provision for designing OMRF and SMRF building

(ii) Select an existing building plan for the case study.

(iii) Model the selected building with and without considering infill strength/ stiffness. Models need to consider two types of end support conditions as mentioned above.

(iv) Nonlinear analysis of the selected building model and a comparative study on the results obtained from the analyses.

(v) Observations of results and discussions

(vi) Conclusion and further recommendation keeping the scope of this study in mind.

1.8 ORGANIZATION OF THE THESIS

Chapter 1 is an introductory chapter which gives a brief introduction to the importance of the seismic evaluation and usage of SMRF buildings and the reason why they are adopted by the designers. The need, objectives and scope of the proposed research work are identified along with the methodology that is followed to carry out the work.

Chapter 2 presents the literature survey on behavior of OMRF and SMRF buildings and infill walls during earthquake.

Chapter 3 explains the description of the selected building and the structural modelling parameters and modelling of infill walls. This chapter also describes the procedures and important parameters to model the nonlinear point plastic hinges.

Nonlinear analysis is an important tool to correctly evaluate the seismic performance of a building. Nonlinear static (pushover) analysis of the selected building model is carried out as part of this project and the corresponding results are presented in **Chapter 4**. Results obtained from nonlinear analyses of the building model considering various cases are presented in the same chapter. This chapter critically evaluates the nonlinear analysis results to compare the building responses with and without considering infill strength/stiffness.

Finally, in **Chapter 5**, the conclusions derived from the entire project are given. The scope for future work is also discussed.

CHAPTER 2

LITERATURE REVIEW

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).

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.

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).

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.

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.

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.

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.

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.

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.

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.

X.-K. Zou *et al*., (2005) presented an effective technique that incorporates Pushover Analysis together with numerical optimisation 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.

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.

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.

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 (L_p) 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 (Programme Default).

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.

A. 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.

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 Behaviour 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

Athanassiadou (2008) analysed 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.

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, were 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.

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.

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.

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.

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.

Devrim.O ***et al.***, (2012) studies three 10 story steel SMRF with different spans were designed as per Turkish Codes and were analyzed using OPENSEES 15 using simulated ground motion records and model frame with span length to story height ratio of approximately 2 seems to maintain both performance and economy, while the ratio higher than 2.5 can result in relatively high deflections and high element plastic rotations in lower stories under infrequent earthquake loads.

Duan.H ***et al.,***(2012) designed a five story RC frame building according to Chinese Seismic codes and investigated the seismic performance of the same by pushover analysis and found the potential for a soft story mechanism under significant seismic loads

Mohammed.A ***et al.,*** (2012) investigated the seismic design factors for three RC-SMRF buildings with 4, 16 and 32 stories in Dubai, utilizing nonlinear analysis and found that a trend of poorer performance is detected as the building height increases.

Haroon Rasheed Tamboli and Umesh N. Karadi (2012),performed seismic analysis usingEquivalent Lateral Force Method for different reinforced concrete (RC) frame building models that included bare frame, in filled frame and open first story frame. In modelling of the masonry Infill panels the Equivalent diagonal Strut method was used and the software ETABS was used for the analysis of all the frame models. In filled frames should be preferred in seismic regions than the open first story frame, because the story drift of first story of open first story frame is Very large than the upper stories, which might probably cause the collapse of structure. The infill Wall increases the strength and stiffness of the structure. The seismic analysis of RC (Bare frame) structure lead to under estimation of base shear. Therefore other response quantities such as time period, natural frequency, and story drift were not significant. The under estimation of base shear might lead to the collapse of structure during earthquake shaking.

Narender Bodige and Pradeep Kumar Ramancharla (2012), modelled a 1 x 1 bay 2D four storied building using AEM (applied element method). AEM is a discrete method in which the elements are connected by pair of normal and shear springs which are distributed around the elements edges and each pair of springs totally represents stresses and deformation and plastic hinges location are formed automatically. Gravity loads and laterals loads as per IS 1893-2002 were applied on the structure and designed using IS 456 and IS 13920. Displacement control pushover analysis was carried out in both cases and the pushover curves were compared. As an observation it was found that AEM gave good representation capacity curve. From the case studies it was found that capacity of the building significantly increased when ductile detailing was adopted. Also, it was found that effect on concrete grade and steel were not highly significant.

2.3 SUMMARY

This chapter dealt with the numerous numbers of papers and journals that has been found helpful for carrying out the work. An extensive literature review is done and the inference is noted down. It is well established from various studies that ductile detailing is necessary to resist earthquakes. Many works have been done regarding buildings with ductile detailing, but there were very less number of works comparing the performance of SMRF and OMRF. Moreover, no works has been done in the past based on IS codes.

The next chapter deal with the details of the design of buildings and the type of analysis which has to be carried out. A detailed description about pushover analysis and plastic hinges is also discussed in the next chapter

CHAPTER 3

BUILDING DETAILS AND MODELLING FOR NONLINEAR ANALYSIS

CHAPTER 3

BUILDING DETAILS AND MODELLING FOR ANALYSIS

3.1 INTRODUCTION

This Chapter deals with the selection and design of building frames as per the design code procedures. The designed frames are modelled for nonlinear analysis. It is necessary to develop a computational model to perform any kind of analysis. The parameters defining the building models, the basic assumptions and the geometry of the selected buildings for the study is discussed. This includes the development of concentrated plasticity hinges at the critical sections of beams and columns.

3.2 BUILDING CONFIGURATIONS AND DESIGN DETAILS

A total of 12 frames are selected by varying number of storeys, number of bays, infill wall configurations, and design methodology with regard to response reduction factors and confinement detailing. A detailed description of all the frames considered is presented in Table 3.1. The storey height is 3.5m and bay width is 3m, which is same for all frames. Each frame is designed as OMRF and SMRF considering response reduction factors such as 3 and 5. IS code suggests a response reduction factor of 3 for OMRF and 5 for SMRF. The design of the frames is carried out by conducting linear static analysis of bare frames and accounting for all the load combinations suggested by IS 1893(2002). Two extreme situations such as hinged and fixed support conditions are reflected in the study. For convenient presentation of results, a suitable naming convention is followed. 4S7B-SMRF-B-F represents a bare frame, designed as SMRF with fixed support conditions. 4S7B-SMRF-I-H is an infill walled frame, designed as SMRF with hinged support conditions. A building can be treated as a bare frame if the infill frames are constructed with a clear gap between the walls and columns so that the infill walls do not take part in lateral loads. The building frame with infill walls provided in all storeys is considered as a fully infill frame.

Table 3.1 Details of all the fixed support bare frames

Sl No	Frame Name	Frame type	No. of storey	No. of bays	R	Frame Type	Support conditions
1	4S7B-SMRF-B-F	Bare	4	7	5	SMRF	Fixed
2	8S7B-SMRF-B-F	Bare	8	7	5	SMRF	Fixed
3	10S7B-SMRF-B-F	Bare	10	7	5	SMRF	Fixed
4	6S2B-SMRF-B-F	Bare	6	2	5	SMRF	Fixed
5	6S4B-SMRF-B-F	Bare	6	4	5	SMRF	Fixed
6	6S6B-SMRF-B-F	Bare	6	6	5	SMRF	Fixed
7	4S7B-OMRF-B-F	Bare	4	7	3	OMRF	Fixed
8	8S7B-OMRF-B-F	Bare	8	7	3	OMRF	Fixed
9	10S7B-OMRF-B-F	Bare	10	7	3	OMRF	Fixed
10	6S2B-OMRF-B-F	Bare	6	2	3	OMRF	Fixed
11	6S4B-OMRF-B-F	Bare	6	4	3	OMRF	Fixed
12	6S6B-OMRF-B-F	Bare	6	6	3	OMRF	Fixed

Table 3.2 shows the details of all the bare frames with hinged support

Table 3.2 Details of all the hinged support bare frames

Sl No	Frame Name	Frame type	No. of storeys	No. of bays	R	Frame Type	Support conditions
1	4S7B-SMRF-B-H	Bare	4	7	5	SMRF	Hinged
2	8S7B-SMRF-B-H	Bare	8	7	5	SMRF	Hinged
3	10S7B-SMRF-B-H	Bare	10	7	5	SMRF	Hinged
4	6S2B-SMRF-B-H	Bare	6	2	5	SMRF	Hinged
5	6S4B-SMRF-B-H	Bare	6	4	5	SMRF	Hinged
6	6S6B-SMRF-B-H	Bare	6	6	5	SMRF	Hinged
7	4S7B-OMRF-B-H	Bare	4	7	3	OMRF	Hinged
8	8S7B-OMRF-B-H	Bare	8	7	3	OMRF	Hinged
9	10S7B-OMRF-B-H	Bare	10	7	3	OMRF	Hinged
10	6S2B-OMRF-B-H	Bare	6	2	3	OMRF	Hinged
11	6S4B-OMRF-B-H	Bare	6	4	3	OMRF	Hinged
12	6S6B-OMRF-B-H	Bare	6	6	3	OMRF	Hinged

The variation of strength and stiffness properties of brick infill walls available in India is relatively very high. Krishnakedar (2004) reports that the modulus of elasticity of strong and weak infill walls are about 5000MPa and 350MPa, respectively. The same variation also can be seen in the strength also. All the infill frames considered in the present study is assumed to have both strong and weak types of infill walls to simulate the behaviour of infill framed buildings for extreme situations

Table 3.3 shows the details of all buildings with strong infill and fixed support condition

Table 3.3 Details of all the fixed support frames with strong infill

Sl No	Frame Name	Frame type	No. of storey	No. of bays	R	Infill Type	Frame Type	Support conditions
1	4S7B-SMRF-I-S-F	Infill	4	7	5	Strong	SMRF	Fixed
2	8S7B-SMRF-I-S-F	Infill	8	7	5	Strong	SMRF	Fixed
3	10S7B-SMRF-I-S-F	Infill	10	7	5	Strong	SMRF	Fixed
4	6S2B-SMRF-I-S-F	Infill	6	2	5	Strong	SMRF	Fixed
5	6S4B-SMRF-I-S-F	Infill	6	4	5	Strong	SMRF	Fixed
6	6S6B-SMRF-I-S-F	Infill	6	6	5	Strong	SMRF	Fixed
7	4S7B-OMRF-I-S-F	Infill	4	7	3	Strong	OMRF	Fixed
8	8S7B-OMRF-I-S-F	Infill	8	7	3	Strong	OMRF	Fixed
9	10S7B-OMRF-I-S-F	Infill	10	7	3	Strong	OMRF	Fixed
10	6S2B-OMRF-I-S-F	Infill	6	2	3	Strong	OMRF	Fixed
11	6S4B-OMRF-I-S-F	Infill	6	4	3	Strong	OMRF	Fixed
12	6S6B-OMRF-I-S-F	Infill	6	6	3	Strong	OMRF	Fixed

Table 3.4 shows all the buildings with weak infill and fixed support condition.

Table 3.4 Details of all the fixed support frames with weak infill

Sl No	Frame Name	Frame type	No. of storey	No. of bays	R	Infill Type	Frame Type	Support conditions
1	4S7B-SMRF-I-W-F	Infill	4	7	5	Weak	SMRF	Fixed
2	8S7B-SMRF-I-W-F	Infill	8	7	5	Weak	SMRF	Fixed
3	10S7B-SMRF-I-W-F	Infill	10	7	5	Weak	SMRF	Fixed
4	6S2B-SMRF-I-W-F	Infill	6	2	5	Weak	SMRF	Fixed
5	6S4B-SMRF-I-W-F	Infill	6	4	5	Weak	SMRF	Fixed
6	6S6B-SMRF-I-W-F	Infill	6	6	5	Weak	SMRF	Fixed
7	4S7B-OMRF-I-W-F	Infill	4	7	3	Weak	OMRF	Fixed
8	8S7B-OMRF-I-W-F	Infill	8	7	3	Weak	OMRF	Fixed
9	10S7B-OMRF-I-W-F	Infill	10	7	3	Weak	OMRF	Fixed
10	6S2B-OMRF-I-W-F	Infill	6	2	3	Weak	OMRF	Fixed
11	6S4B-OMRF-I-W-F	Infill	6	4	3	Weak	OMRF	Fixed
12	6S6B-OMRF-I-W-F	Infill	6	6	3	Weak	OMRF	Fixed

The material properties and the geometric parameters considered in the study are listed in Table 3.6. M25 concrete is used at the design stages, along with Fe415 steel. The detailed description is given in the Table 3.5.

Table 3.5 Material properties and Geometric parameters assumed

Sl No.	Design Parameter	Value
1	Unit weight of concrete	25 kN/m^3
2	Unit weight of Infill walls	18kN/m^3
3	Characteristic Strength of concrete	25 MPa
4	Characteristic Strength of concrete	415 MPa
5	Compressive strength of strong masonry (E_m)	5000MPa
6	Compressive strength of weak masonry (E_m)	350MPa
7	Modulus of elasticity of Masonry Infill walls (E_m)	$750f'_m$
8	Damping ratio	5%
9	Modulus of elasticity of steel	2e5 MPa
10	Slab thickness	150 mm
11	Wall thickness	230 mm

The seismic design data assumed for SMRF buildings is shown in the Table 3.6, and for OMRF buildings in Table 3.7

Table 3.6 Seismic Design Data assumed for Special Moment Resisting Frames

Sl No.	Design Parameter	Value
1	Seismic Zone	V
2	Zone factor (Z)	0.36
3	Response reduction factor (R)	5
4	Importance factor (I)	1
5	Soil type	Medium soil
6	Damping ratio	5%
7	Frame Type	Special Moment Resisting Frame

Table 3.7 Seismic Design Data assumed for Ordinary Moment Resisting Frames

Sl No.	Design Parameter	Value
1	Seismic Zone	V
2	Zone factor (Z)	0.36
3	Response reduction factor (R)	3
4	Importance factor (I)	1
5	Soil type	Medium soil
6	Damping ratio	5%
7	Frame Type	Ordinary Moment Resisting Frame

The loads considered for designing the frames are given in Table 3.8. The loads are calculated using the material properties and the element dimensions.

Table 3.8 Loads considered for designing buildings

Sl	Load Type	Value
1	Self-weight of beams and columns	As per dimensions.
2	Weight of slab	11.25 KN/m
3	Infill weight	11.8 KN/m
4	Parapet weight	2.5 KN/m
5	Floor finish	2.5 KN/m^2
6	Live load	3.0 KN/m^2

3.3 PUSHOVER ANALYSIS

Performance assessment of the designed frames is carried out using nonlinear static pushover analysis. The modelling 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 modelled 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:

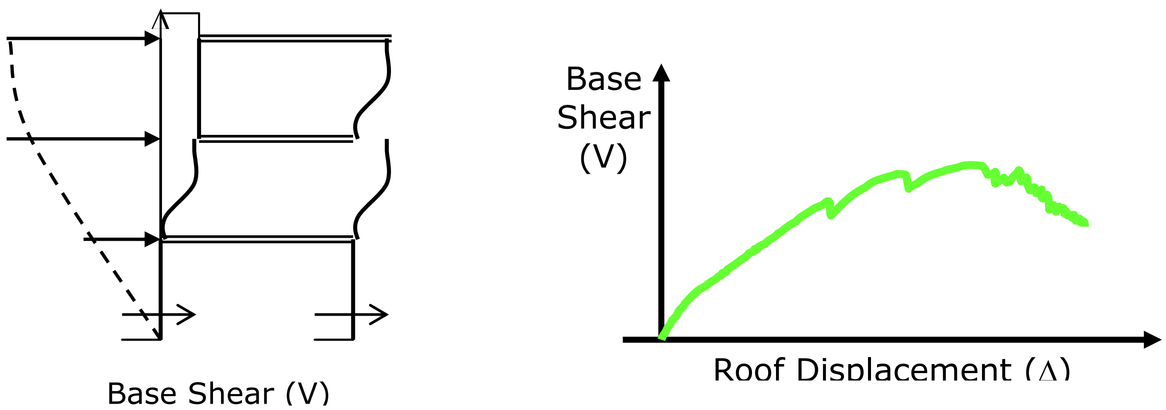

Fig.3.1 Typical Pushover Curve

3.4 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, interstorey 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 inelastically 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 interstorey 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.

3.5 MODELLING OF STRUCTURAL ELEMENTS

Beams and columns were modelled as frame elements available in in SAP 2000, with the central lines joined at nodes. Beam-column joints are considered as rigid beam-column joints and these are modelled by giving end offsets at the joints. A rigid zone factor of 1.0 is assumed to replicate the rigidity at the joints. The floor slabs are assumed to act as diaphragms, which ensure integral action of all the vertical lateral load-resisting elements.

The weight of the slab was distributed as triangular and trapezoidal load to the surrounding beams as per IS 456:2000.

3.6 NONLINEAR BEHAVIOUR OF BEAMS AND COLUMNS

In pushover analysis, it is necessary to model the non-linear load versus deformation behaviour of each element. The beams and columns are modelled as frame elements and the infill walls are modelled as equivalent struts by truss elements. Since the deformations are expected to go beyond the elastic range in a pushover analysis, it is necessary to model the non-linear load versus deformation behaviour of the members. The non-linear behaviour is incorporated in the load versus deformation property of a concentrated hinge attached to the member. A beam is assigned with a moment versus rotation curve for a section where a hinge is expected to form. In addition to that a shear force versus shear deformation curve is defined to model the possible shear failure at a section. Similarly, a column is also assigned with flexural and shear hinges. For equivalent strut, the hinge is placed at the middle of the strut with an assigned axial load versus deformation curve.

3.7 PERFORMANCE LEVELS OF BEAMS AND COLUMNS

The performance of any building frame is a combination of the performance of all its structural and non-structural components. The performance levels are discrete damage states identified from a continuous spectrum of possible damage states. The structural performance levels based on the roof drifts are as follows (FEMA 356, 2000).

i) Immediate occupancy (IO)

ii) Life safety (LS)

iii) Collapse prevention (CP)

The nonlinear procedures of FEMA require definition of the nonlinear load-deformation relation. Such a curve showing a typical load – deformation relation and target performance levels curve is shown in Fig 3.2.

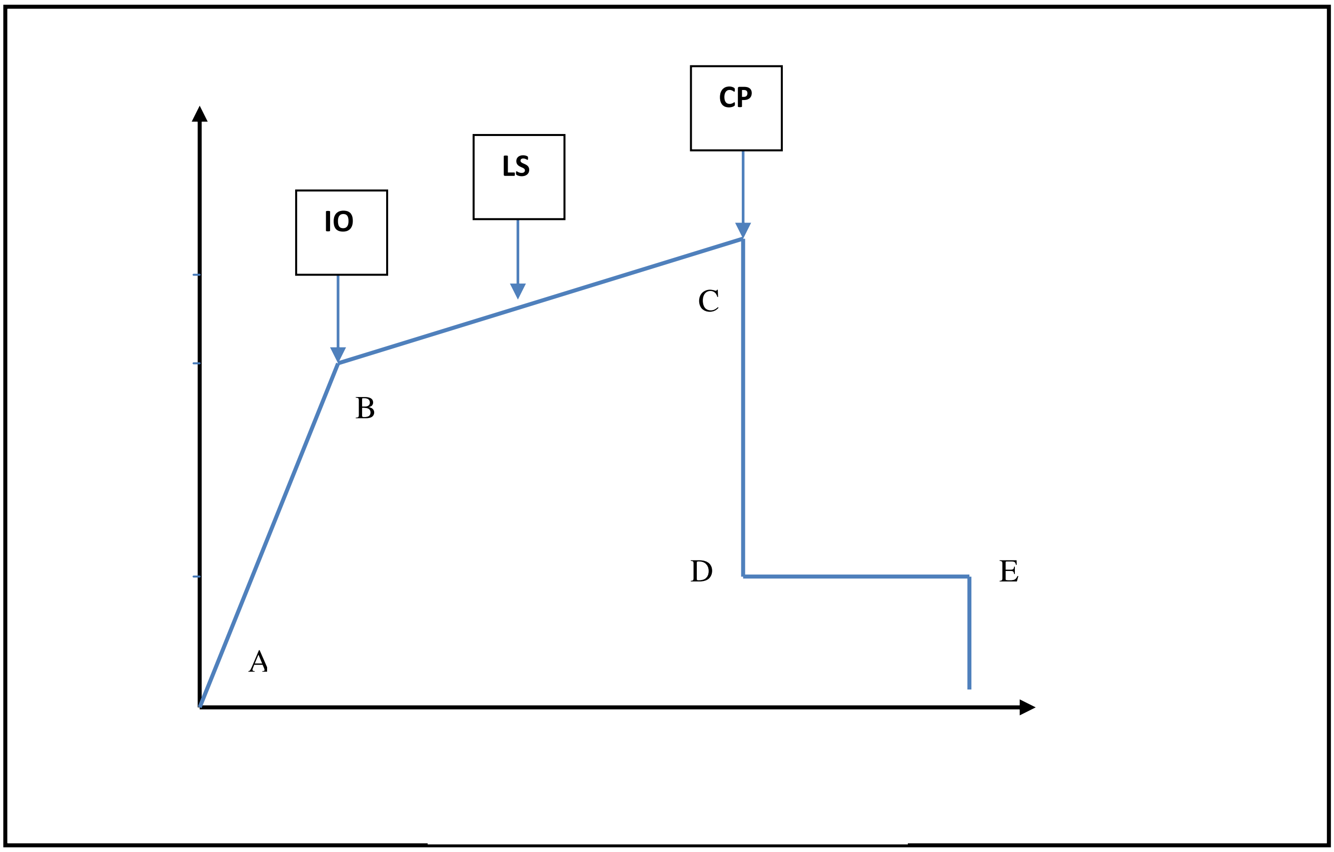

Fig.3.2 Typical load – deformation relation and target performance levels

The three levels are arranged according to decreasing performance of the lateral load resisting systems. The element performance levels are defined by values of the deformation of a structural element. Three performance levels are defined in the load versus deformation curve for the hinges of the element. An idealized load versus deformation curve is shown in Fig.3.2. It is a piece-wise linear curve defined by five points as explained above.

(i) Point 'A' corresponds to no load condition.

(ii) Point 'B' corresponds to the start of yielding.

(iii) Point 'C' corresponds to the ultimate strength.

(iv) Point 'D' corresponds to the residual strength. For computational stability, it is recommended to specify non-zero residual strength beyond C. In absence of the modelling of the descending branch of a load versus deformation curve, the residual strength can be assumed to be 20% of the yield strength.

(v) Point 'E' corresponds to the maximum deformation capacity with the residual strength. To maintain computational stability, a high value of deformation capacity is assumed.

3.8 NON-LINEAR HINGE PROPERTIES OF BEAMS AND COLUMNS

The force-deformation curves in flexure and shear are obtained from the reinforcement details obtained from the design and are assigned in all the columns and primary beams. The flexural hinges (M3) are assigned for the beams at two ends. Flexural hinges (PMM) and shear hinges (V2 and V3) were also given for all the columns at upper and lower ends. Shear hinges are not considered since all the sections are designed to fail in flexure.

3.9 MODELLING OF INFILL WALL

Infill walls are two dimensional elements that can be modelled with orthotropic plate element for linear analysis of buildings with infill wall. But the nonlinear modelling of a two dimensional plate element is not understood well. Therefore infill wall has to be modelled with a one-dimensional line element for nonlinear analysis of the buildings. All buildings with infill walls modelled as one-dimensional line element is used in the present study for nonlinear analysis. Infill walls are modelled here as equivalent diagonal strut elements.

In a linear structural analysis, the required properties of an equivalent strut are the effective width, thickness, length and elastic modulus. The thickness (*t*) is assumed to be same as that of the infill wall. The length (*d*) is the diagonal length of the frame. The remaining properties to be determined are the effective width (*w*) and elastic modulus (*Es*) of the equivalent strut. The strength of the equivalent strut is required to check its capacity with the axial load demand in the strut. The simplest form *w* and *Es*are taken equal to *d*/4 and *Em*(modulus of masonry), respectively.

3.9.1 ELASTIC MODULUS OF EQUIVALENT STRUT

The elastic modulus of the equivalent strut $E_s$can be equated to E_m, the elastic modulus of the masonry. Krishnakedar (2004) conducted a series of experiments on masonry prisms on various types of bricks in India. Following range of values for E_m were obtained.E_m= 350 to 800 MPa for table moulded bricks, E_m= 2500 to 5000 MPa for wire cut bricks

3.9.2 NON-LINEAR HINGE PROPERTY FOR EQUIVALENT STRUT

The nonlinear hinge property for the infill walls is studied by various researchers for many years and a recent study by Asokan(2006) reviewed the state of the art, combined all the previous experimental data and recommended the following simplified piece-wise linear plastic hinge property, including many parameters. The parameters considered are wall panel dimensions, grade of concrete, yield moments of the adjacent beam and column, size of the adjoining columns, wall thickness, compressive strength, shear strength, coefficient of friction between brick and mortar, interface coefficient of friction between frame and infill wall etc. A typical hinge property for the equivalent strut suggested by Asokan (2006) is as shown in Fig 3.3 below

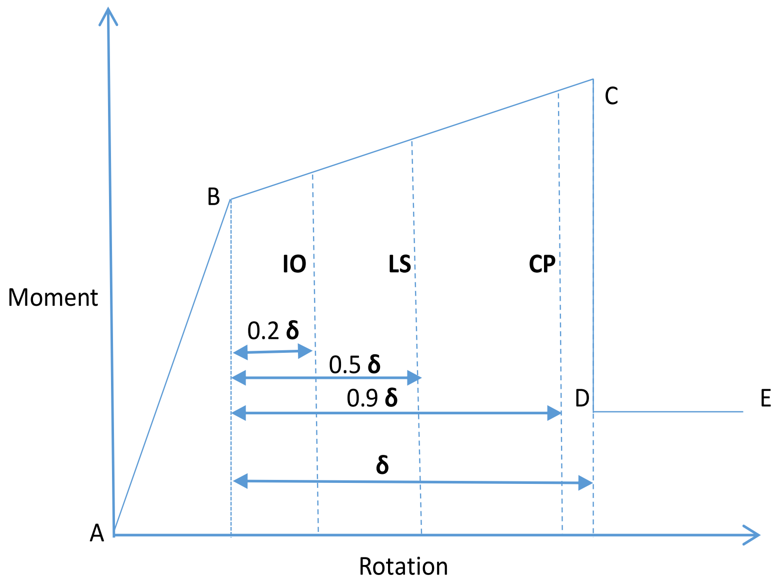

Fig. 3.3 showing nonlinear hinged property of strut

All the frames are modelled as discussed above and pushover analysis is conducted. The building is pushed (called gravity PUSH) for the vertical loads equal to self-weight plus 50% if live load. The pushover analysis for the lateral load, named as PUSHX (towards +X direction), is followed from the last stage of pushover analysis for vertical loads (gravity PUSH). The sequence of hinge formation and the base shear versus roof displacement curves called pushover curves are the quantities of interest in the present for the performance assessment of each buildings.

3.10 BEHAVIOR PARAMETERS OF THE BUILDING

In force-based seismic design procedures, behaviour factor, R (EC8), or R_w, also referred to by other terms, including response modification factor (FEMA 1997, UBC 1997), is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra (Maheri and Akbari,2011). In other words, behaviour factor is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure. The behaviour factor R, therefore accounts for the inherent ductility and overstrength of a structure and the difference in the level of stresses considered in its design. It is generally expressed in the following form taking into account the above three components,

$$R = R_\mu \times R_s \times Y \qquad (3.1)$$

where, R_μ is the ductility dependent component also known as the ductility reduction factor, R_S is the over strength factor and Y is termed the allowable stress factor. With reference to Figure 1, in which the actual force–displacement response curve is idealised by a bilinear elastic–perfectly plastic response curve, the behaviour factor parameters may be defined as:

$$R_\mu = \frac{Ve}{Vy} \qquad (3.2)$$

$$R_s = \frac{Vy}{Vs} \qquad (3.3)$$

$$Y = \frac{Vs}{Vw} \qquad (3.4)$$

and the behaviour factor, R is redefined as:

$$R\ (R_w) = \left(\frac{Ve}{Vy}\right) \times \left(\frac{Vy}{Vs}\right) \times \left(\frac{Vs}{Vw}\right) \qquad (3.5)$$

where, V_e, V_y, V_s and V_w correspond to the structure's elastic response strength, the idealized yield strength, the first significant yield strength and the allowable stress design strength, respectively.

For structures designed using an ultimate strength method, the allowable stress factor, Y, becomes unity and the behaviour factor is reduced to:

$$R = R_{\mu} \times R_s = \frac{Ve}{Vs} \qquad (3.6)$$

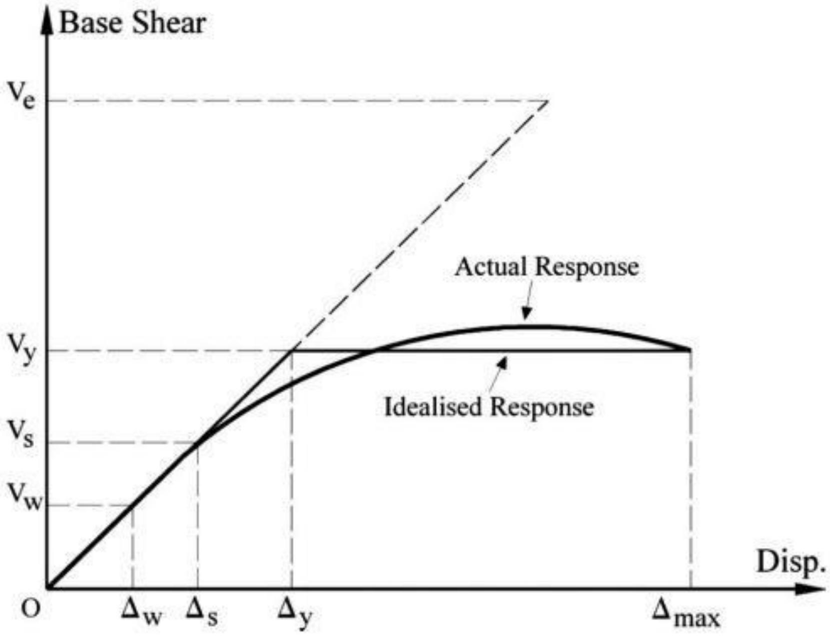

Fig 3.4 Typical pushover graph for evaluation of behaviour factor,(Maheri and Akbari, 2003)

The structure ductility, **μ**, is defined in terms of maximum structural drift (Δ_{max}) and the displacement corresponding to the idealized yield strength (Δ_y) as:

$$\mu = \Delta_{max} / \Delta_y \qquad (3.7)$$

3.11 SUMMARY

All the buildings designed and their nomenclature is presented in a tabulated format. The seismic design data used while designing the buildings, as per the IS codes, has been discussed. A detailed discussion regarding Pushover analysis is discussed. The procedure for modelling structural elements and the equivalent struts for infill walls is also discussed in the chapter in a detailed manner.

All the buildings are modelled in SAP2000 Nonlinear using the design data. The next chapter deals with the performance assessment of the buildings taken into consideration and also, the behaviour parameters of the buildings.

CHAPTER 4

PERFORMANCE ASSESSMENT OF DESIGNED BUILDINGS

CHAPTER 4

PERFORMANCE ASSESSMENT OF DESIGNED FRAMES

4.1 INTRODUCTION

This chapter deals with the performance assessment of the designed buildings. The buildings are modelled in SAP2000 for nonlinear analysis. The pushover analyses of all the frames discussed in the previous sections is conducted. The base shear versus roof displacement at each analysis step is obtained. The pushover curves are presented in each case. A comparison study is carried out to observe the difference in behaviour of buildings.

4.2 COMPARISON OF SMRF AND OMRF: BARE FRAME, FIXED SUPPORT

In this comparison, the performance of ordinary moment resisting frames and special moment resisting frames with fixed support conditions are considered. The base shear versus roof displacement at each analysis step is obtained. The pushover curves are presented in each case.

Figure 4.1 shows pushover curves of 4S7B bare frames designed as both OMRF and SMRF, with fixed support conditions. Initially the base shear increases linearly with the roof displacement. After reaching a certain base shear the building yields. The 4S7B frame designed as OMRF exhibit a higher capacity of base shear than the 4S7B SMRF frame. However, the 4S7B frame designed as SMRF undergoes a higher value of displacement as compared to the 4S7B OMRF frame. Similar behaviour is observed for the pushover curves plotted for 6S2B, 6S4B, 6S6B, 8S7B and 10S7B buildings in Fig 4.2, Fig 4.3, Fig 4.4, Fig 4.6, and Fig 4.6 respectively. This shows that the ductility of the building designed as SMRF is more than OMRF building and they perform better compared to OMRF building. In Fig 4.1, the base shear capacity of 4S7B OMRF is about 40% more than that of a 4S7B SMRF building. But the displacement capacity of 4S7B SMRF is about 3.5 times than that of a 4S7B OMRF

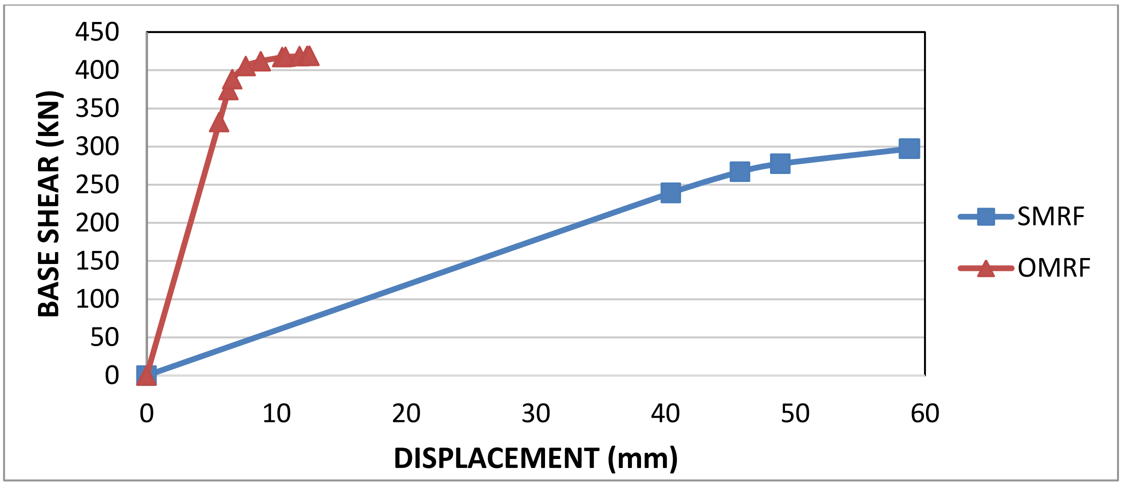

Fig 4.1 shows the pushover curves of 4S7B OMRF AND 4S7B SMRF with Fixed support condition and no infill.

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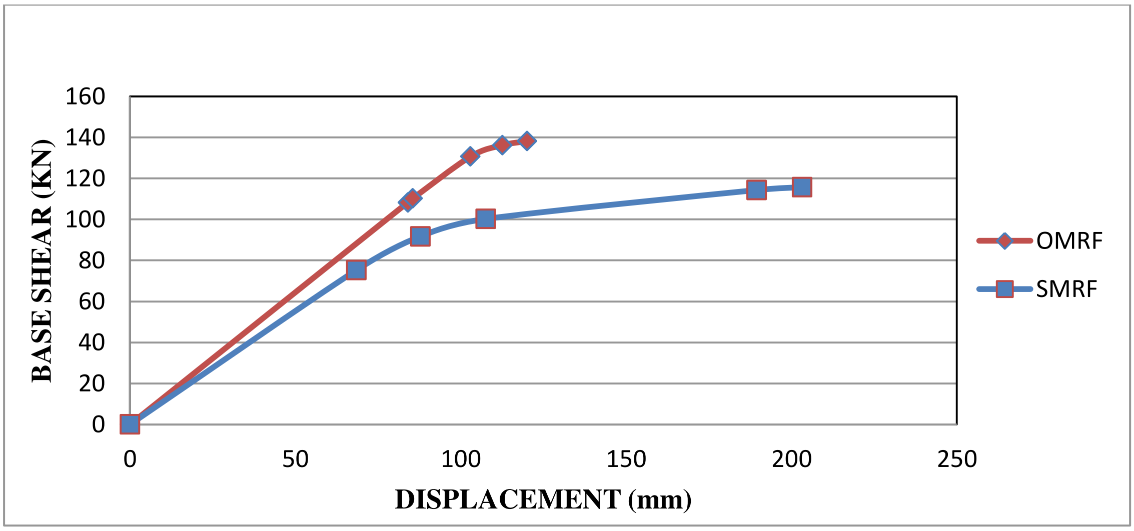

Fig 4.2 shows the pushover curves of 6S2B OMRF AND 6S2B SMRF with Fixed support condition and no infill.

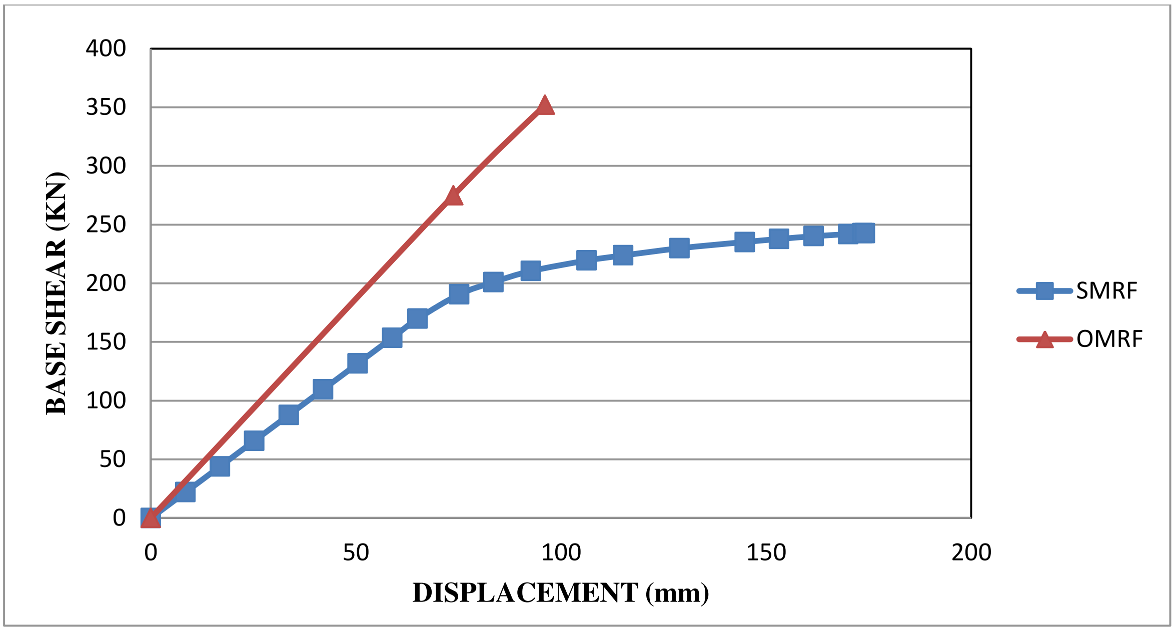

Fig 4.3 shows the pushover curves of 6S4B OMRF AND 6S4B SMRF with Fixed support condition and no infill.

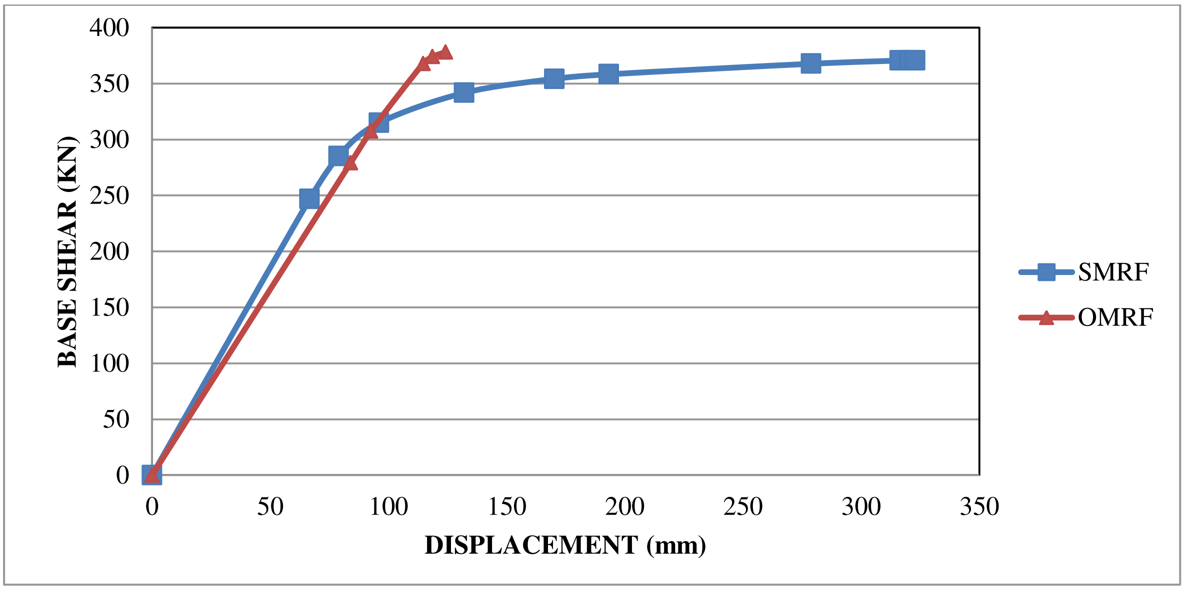

Fig 4.4 shows the pushover curves of 6S6B OMRF AND 6S6B SMRF with Fixed support condition and no infill.

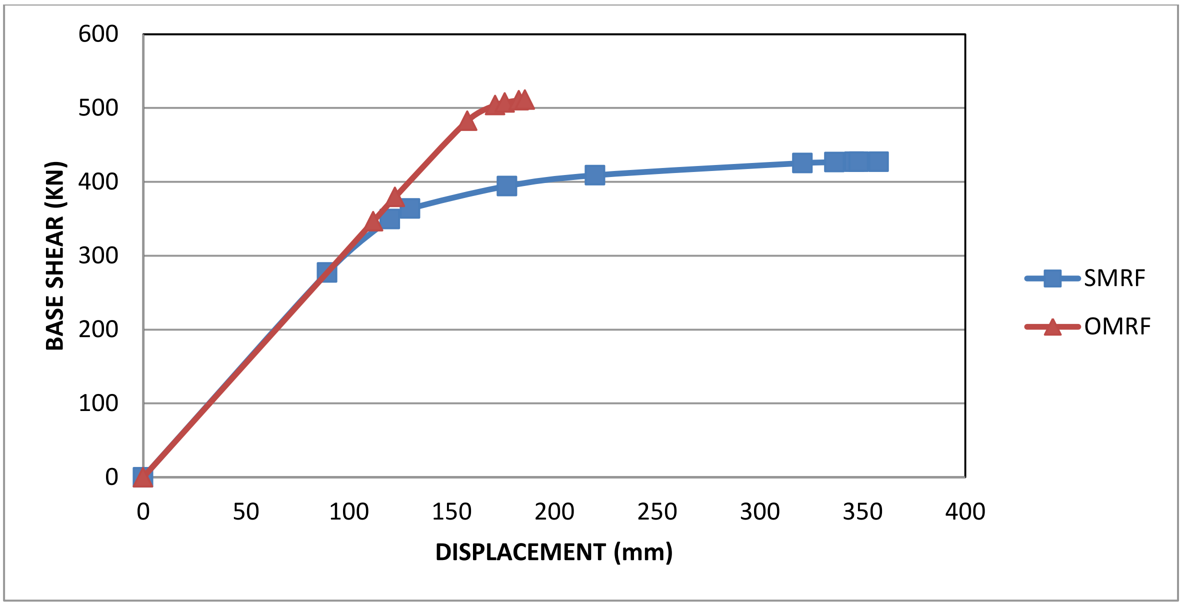

Fig 4.5 shows the pushover curves of 8S7B OMRF AND 8S7B SMRF with Fixed support condition and no infill.

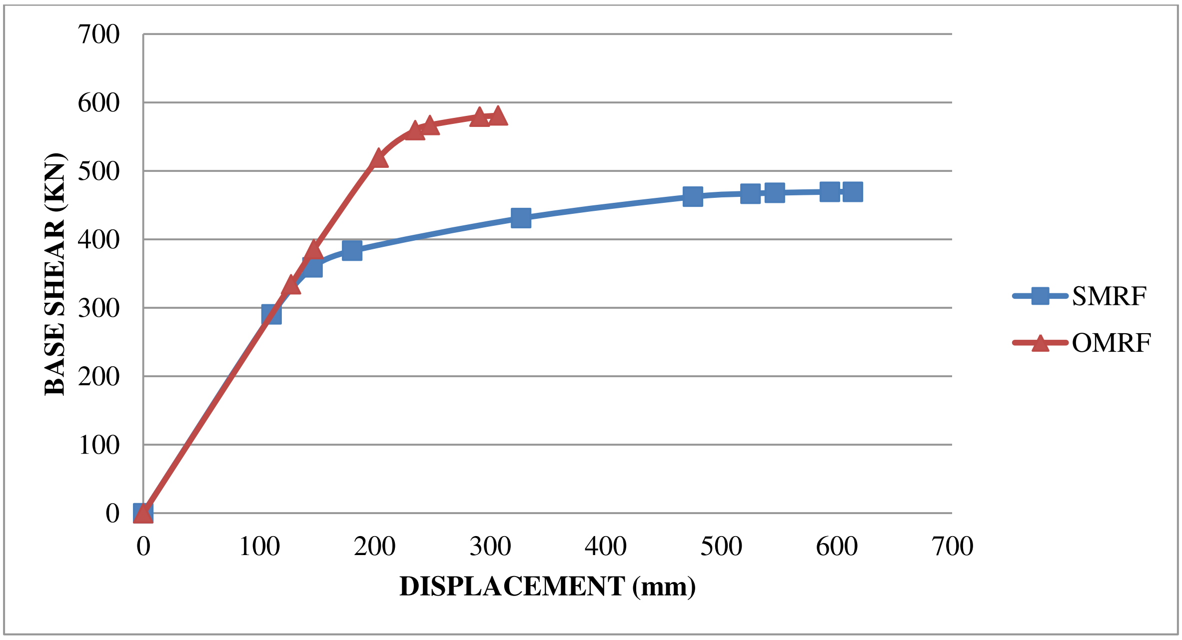

Fig 4.6 shows the pushover curves of 10S7B OMRF AND 10S7B SMRF with Fixed support condition and no infill.

Table 4.1 shows performance comparison regarding the ability of OMRF and SMRF frames to resist base shear and also, the maximum amount of displacement it can undergo. It is observed that ductility is more for SMRF configuration, in all cases, while OMRF performs better in its ability to resist base shear.

Table 4.1 Performance comparison of OMRF and SMRF buildings with Fixed Support

Building Configuration	BASE SHEAR (KN)		% Increase in Base Shear for OMRF	ROOF DISPLACEMENT (mm)		% Increase in Displacement for SMRF
	OMRF	SMRF		OMRF	SMRF	
4S7B	425	300	41.6 %	14	60	328%
6S2B	140	115	21.6%	120	220	83.3%
6S4B	350	250	40%	100	175	75%
6S6B	375	360	4.16%	110	320	199%
8S7B	520	420	23.8%	175	375	114%
10S7B	580	470	23.4%	320	625	96%

It can be seen from Table 4.1 that OMRF buildings has about 20-40% more capacity to resist base shear, while SMRF buildings has about 75-200% more deflection than OMRF buildings

4.3 COMPARISON OF SMRF AND OMRF: BARE FRAME, HINGED SUPPORT

In this comparison, the performance of ordinary moment resisting frames and special moment resisting frames with hinged support conditions are considered. The pushover curves for various configurations of buildings are plotted and the building response is observed.

The pushover analysis of all the frames discussed in the previous sections is conducted. The base shear versus roof displacement at each analysis step is obtained. The pushover curves are presented in each case.

Figure 4.7 shows pushover curves of 4S7B bare frames designed as both OMRF and SMRF, with hinged support conditions. Initially the base shear increases linearly with the roof displacement. After reaching a certain base shear the building yields. The 4S7B frame designed as OMRF exhibit a higher capacity of base shear than the 4S7B SMRF frame. However, the 4S7B frame designed as SMRF undergoes a higher value of displacement as compared to the 4S7B OMRF frame. This shows that the ductility of the frame designed as SMRF is more compared to that of OMRF. Similar behaviour is observed for the pushover curves plotted for 6S2B, 6S4B, 6S6B, 8S7B and 10S7B buildings in Fig 4.8, Fig 4.9, Fig 4.10, Fig 4.11, and Fig 4.12 respectively. It can be seen that ductility is more for SMRF configuration, in all cases and it shows that shows SMRF buildings perform better than ordinary moment resisting frames.

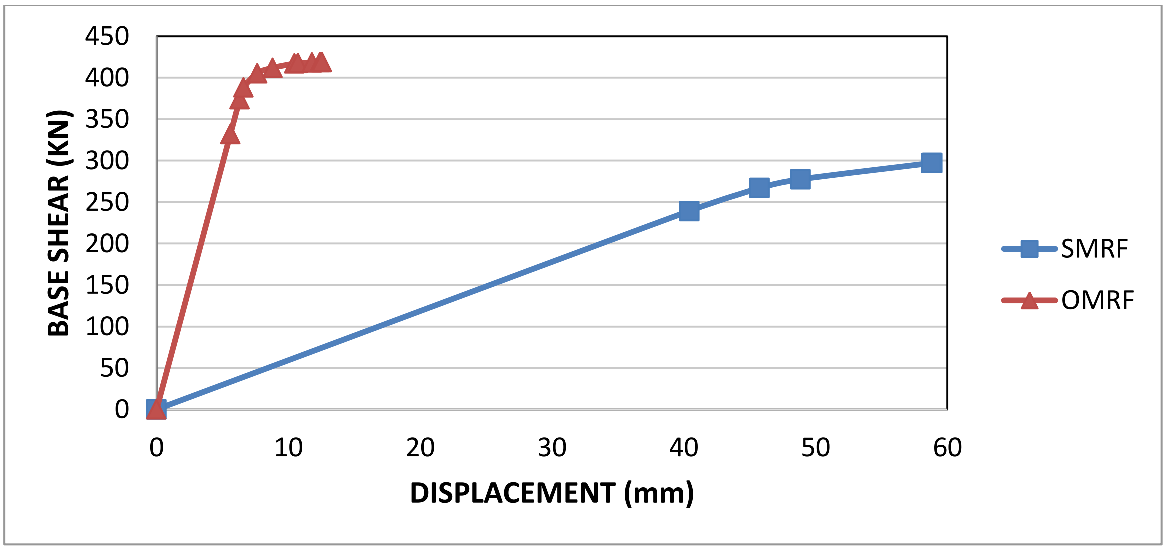

Fig 4.7 shows the pushover curves of 4S7B OMRF AND 4S7B SMRF with Hinged Support condition and no infill.

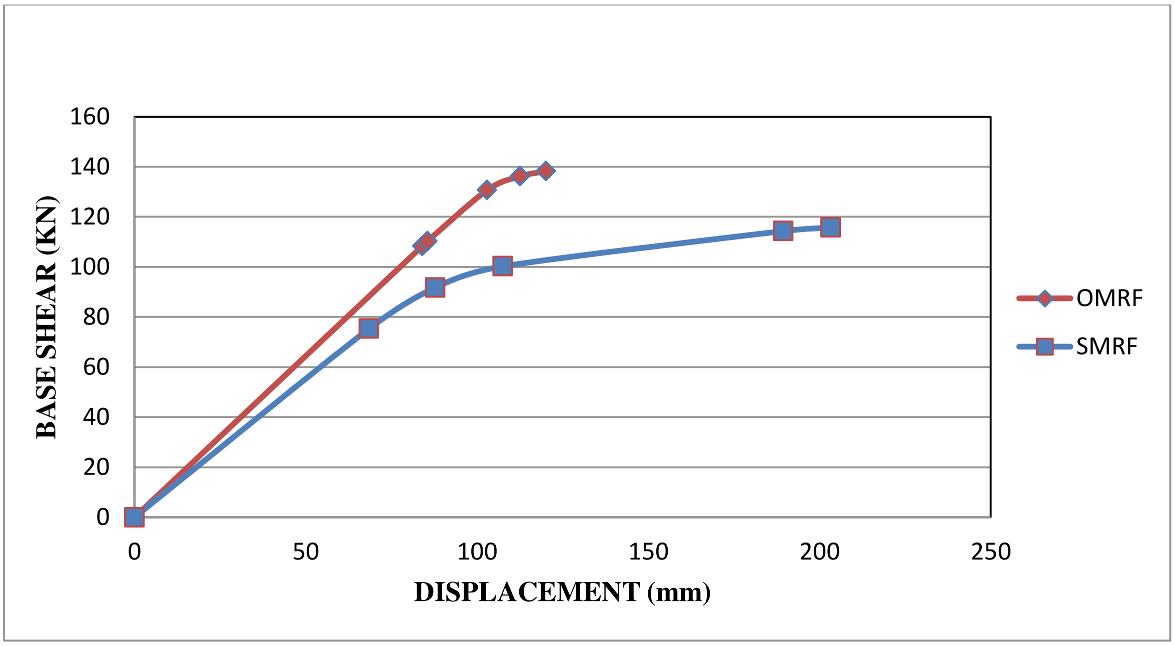

Fig 4.8 shows the pushover curves of 6S2B OMRF AND 6S2B SMRF with Hinged support condition and no infill.

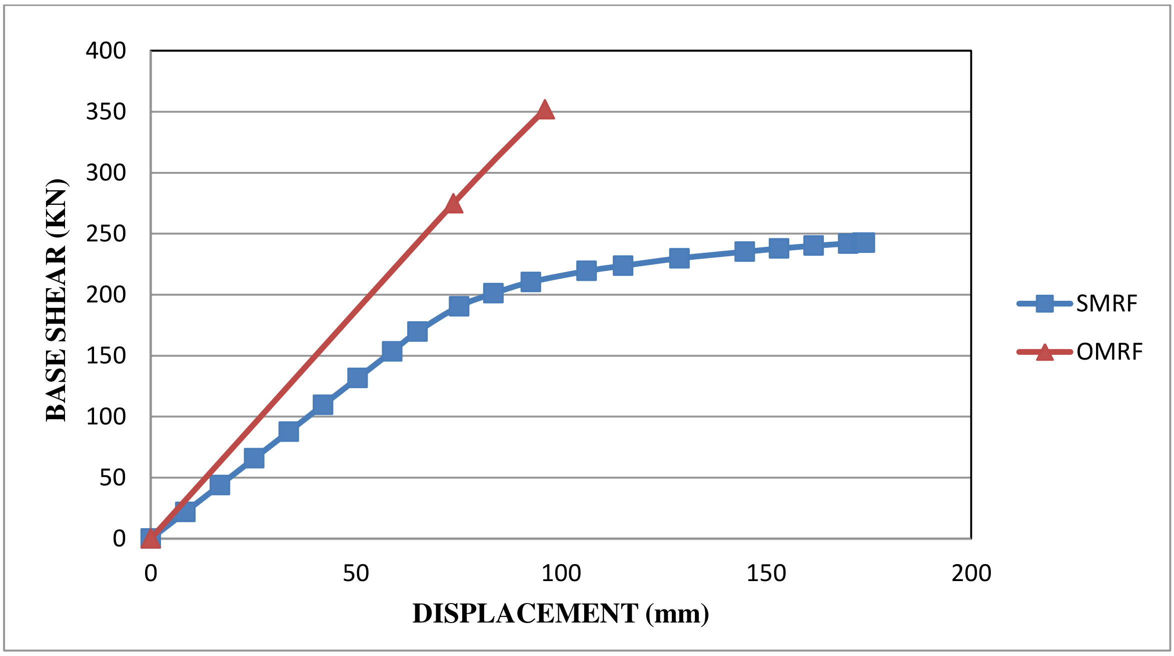

Fig 4.9 shows the pushover curves of 6S4B OMRF AND 6S4B SMRF with Hinged support condition and no infill.

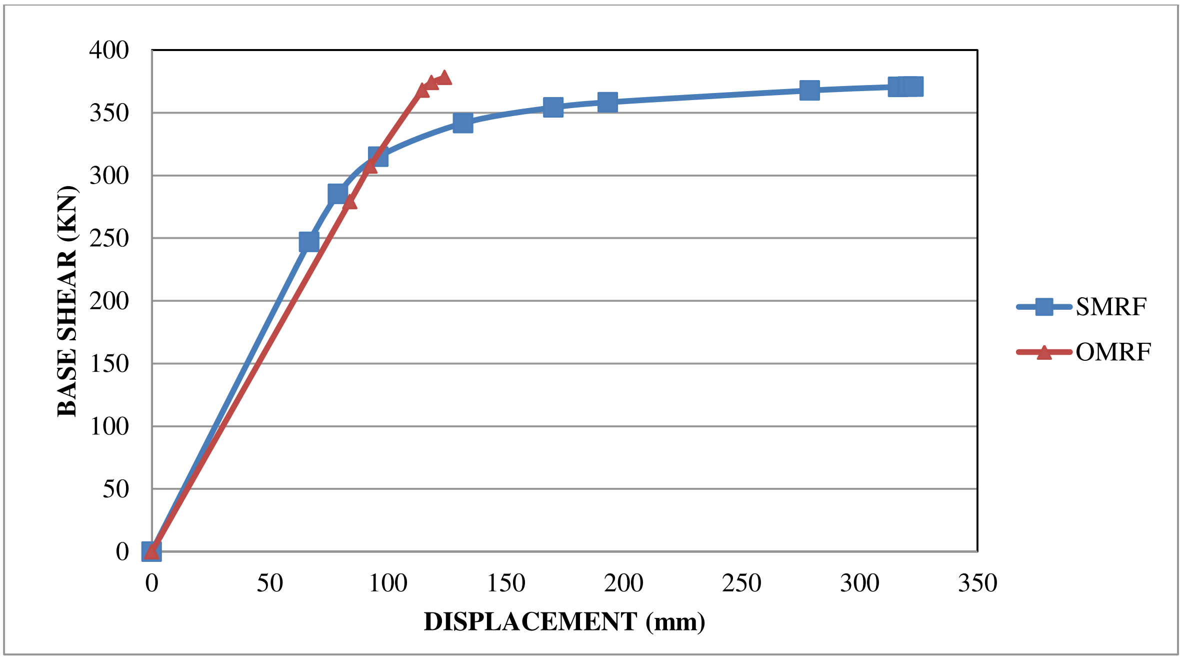

Fig 4.10 shows the pushover curves of 6S6B OMRF AND 6S6B SMRF with Hinged support condition and no infill.

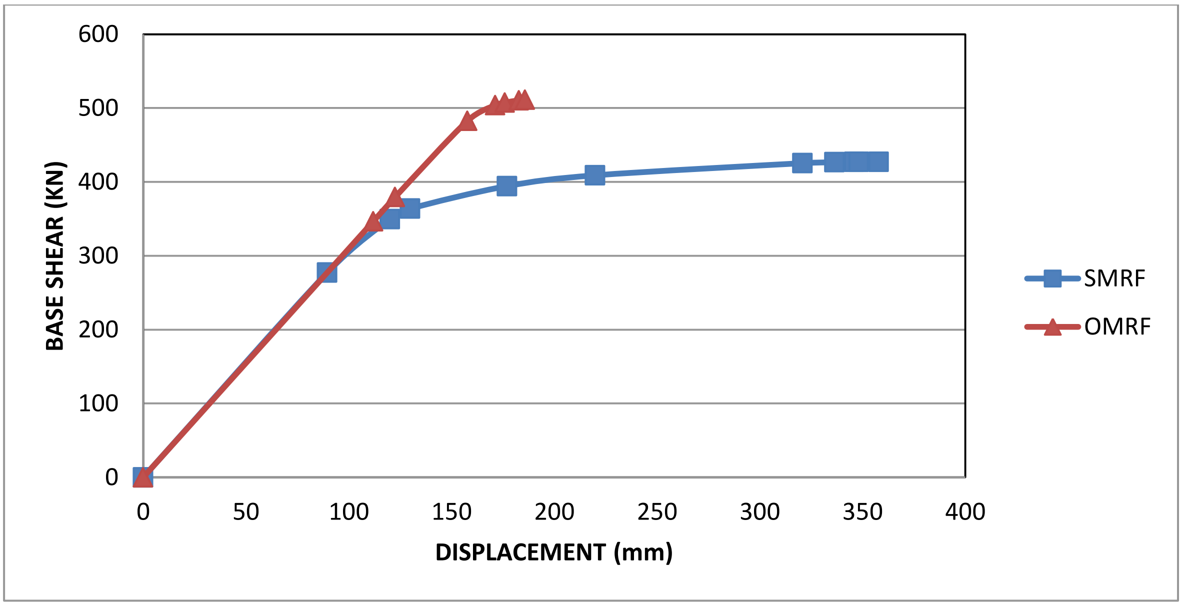

Fig 4.11 shows the pushover curves of 8S7B OMRF AND 8S7B SMRF with Hinged support condition and no infill.

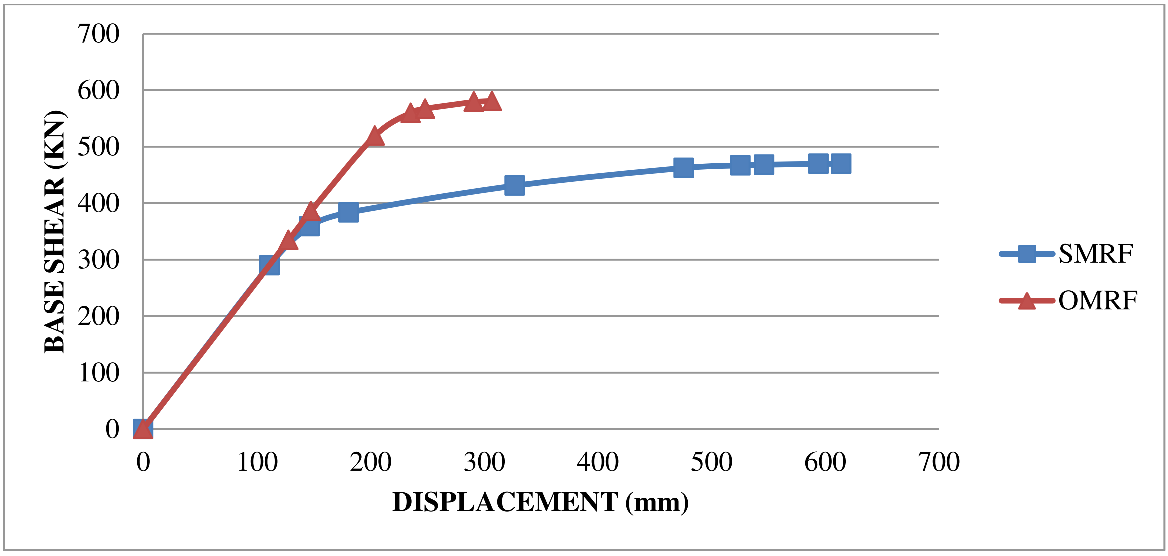

Fig 4.12 shows the pushover curves of 10S7B OMRF AND 10S7B SMRF with Hinged support condition and no infill.

Table 4.2 shows performance comparison regarding the ability of OMRF and SMRF frames to resist base shear and also, the maximum amount of displacement it can undergo. It is observed that ductility is more for SMRF configuration, in all cases, while OMRF performs better in its ability to resist base shear.

Table 4.2 Performance comparison of OMRF and SMRF building with Hinged Support

Building Configuration	BASE SHEAR (KN)		% Increase in Base Shear for OMRF	ROOF DISPLACEMENT (mm)		% Increase in Displacement for SMRF
	OMRF	SMRF		OMRF	SMRF	
4S7B	425	300	41.6 %	14	60	328%
6S2B	140	115	21.6%	120	220	83.3%
6S4B	350	250	40%	100	175	75%
6S6B	375	360	4.16%	110	320	199%
8S7B	520	420	23.8%	175	375	114%
10S7B	580	470	23.4%	320	625	96%

4.4 COMPARISON OF SPECIAL MOMENT RESISTING FRAMES WITH FIXED AND HINGED SUPPORTS.

The pushover curve of SMRF frames with hinged and fixed support condition is plotted and the results are observed. The pushover curve of 6S4B SMRF-B-F and 6S4B OMRF-B-F is plotted in Fig 4.13.

For 6 storeyed bare frame building it can be concluded from Fig 4.13, that the pushover curves for fixed and hinged support condition are same. The pushover curves overlap with each other. They exhibit the same performance in the same loading condition.

The amount of displacement and the ductility ratio of the building is predicted to be same. Hence as like in the case of 6 storey, the same thing can be concluded for other configurations, that the performance of the SMRF buildings under present study is independent of the support condition.

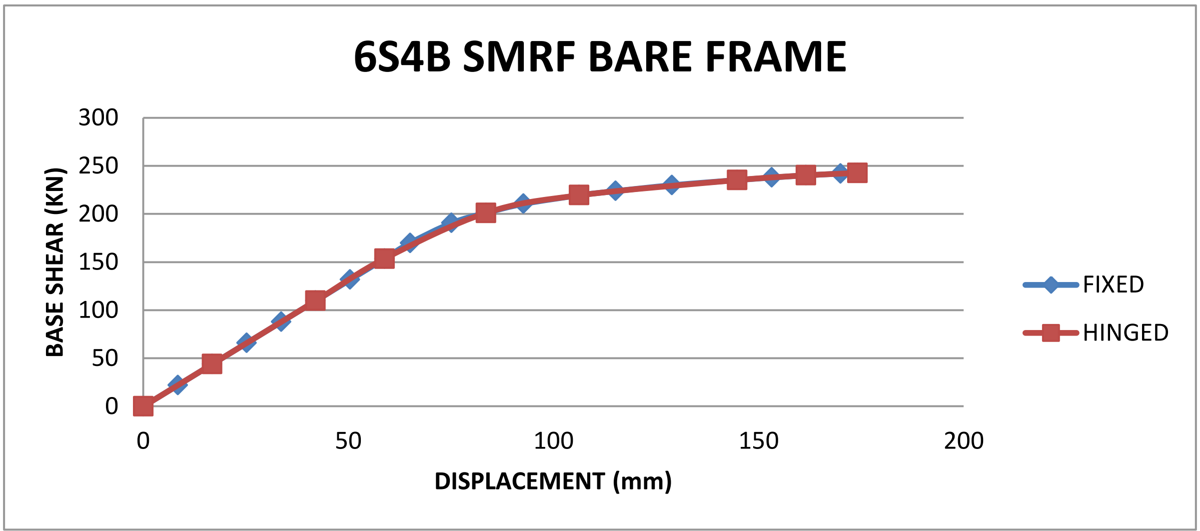

Fig 4.13 shows the pushover curves of 6S4B SMRF with both fixed and hinged support condition and no infill.

4.5 BEAHAVIOUR PARAMETERS OF THE BUILDINGS

A number of performance parameters may govern the capacity of a structure. In order to carry out an inelastic pushover analysis, one or a number of these parameters should be considered for determination of the displacement limit state (Δ_{max}). In a comparative study conducted by Mwafy and Elnashai (2002) on different classes of buildings, a number of global collapse criteria, including interstorey drift limit, column hinging mechanism, limit on drop in the overall lateral resistance and stability index limit, were considered. They concluded that the interstorey drift is the collapse parameter that controls the response of buildings designed to modern seismic codes. The R factor parameters for each system were extracted from the respective pushover response curve. The ductility dependent component, $R\mu$, is calculated using Equations (3.1)–(3.5) and ductility factor, μ, is determined from Equation (3.7). The behaviour parameters of the bare frame buildings considered is tabulated in Table 4.3.

Table 4.3 Behaviour parameters of Buildings considered.

Building Configuration	**Base Shear (KN)**	R_{des}	R_{μ}	R_s	μ
6S6B SMRF-B-F	94.2	5	11.5	1.5	5.5
6S6B OMRF-B-F	155.0	3	3.9	1.3	1.8
6S4B SMRF-B-F	63.6	5	12	1.3	10.5
6S4B OMRF-B-F	106.2	3	4.5	1.2	8.4
4S7B SMRF-B-F	94.4	5	16.4	1.3	28.1
4S7B OMRF-B-F	157.3	3	4	1.2	1.7
6S2B SMRF-B-F	33.4	5	5.77	1.6	2
6S2B OMRF-B-F	54.35	3	2.9	1.3	1.1
8S7B SMRF-B-F	120	5	24.57	2.1	7
8S7B OMRF-B-F	199.5	3	17.57	2	4.5
10S7B SMRF-B-F	129.6	5	6.5	1.3	2.14
10S7B OMRF-B-F	215.5	3	3.3	1.3	1.3

4.6 STOREY WISE COMPARISON OF SMRF BUILDINGS

The buildings with the same number of bays are considered in this comparative study. The buildings considered are 4S7B SMRF-, 8S7B SMRF AND 10S7B SMRF, all having 7 bays. The pushover curves are plotted. In Fig, 4.14, it is observed that 8S7B SMRF AND 10S7B SMRF reflect excellent ductility when compared to 4S7B SMRF. The graphs show that the 10 storey and 8 storey buildings can withstand a higher magnitude of base shear compared to the 4 storey building. But it can be seen that the slope of the curve for all buildings is almost same. Even though the magnitude of base shear that these buildings withstand is less compared to that, which can be withstood by Ordinary Moment Resisting frames, this comparison again shows that fact that Special Moment Resisting Frame buildings possess excellent ductility when compared to Ordinary Moment Resisting Frame buildings.

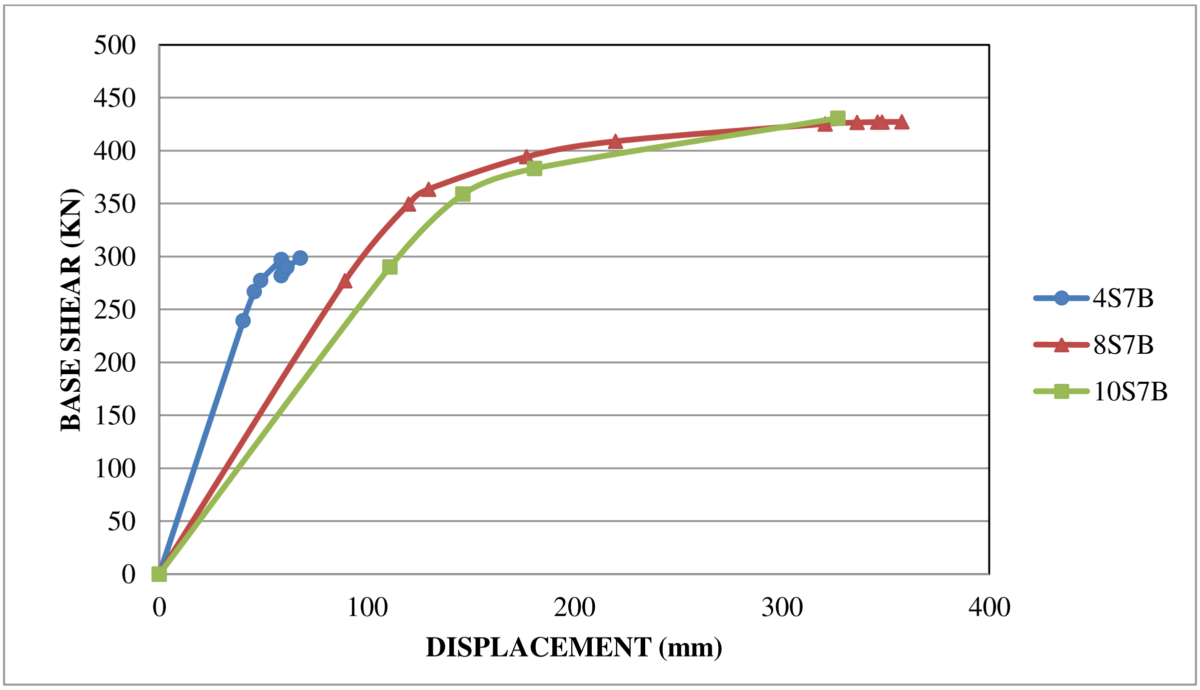

Fig.4.14 showing the storey wise comparison of SMRF buildings with fixed support conditions and no infill.

4.7 BAY WISE COMPARISON OF SMRF BUILDINGS

The buildings with the same number of storeys are considered in this comparative study. The buildings considered are 6S2B SMRF, 6S4B SMRF AND 6S6B SMRF, all having 6 storeys. The pushover analysis is performed and Base shear versus Displacement graphs are plotted and it is observed that 6S4B SMRF AND 6S6B SMRF reflect excellent ductility when compared to 6S2B SMRF. In Fig 4.15, it is observed that 6S6B SMRF can withstand a base shear of 370 KN, 6S4B SMRF can withstand a base shear of 250 KN and 6S2B SMRF can withstand a base shear of 120 KN. This shows that as the number of bays increases from 2 to 4, the base shear capacity will increase by 2 times. And when it increases from 2 bays to 6 bays, the magnitude of the base shear the building can withstand increase by 3 times It can be proposed that the number of bays play a major role in the stability of a building.

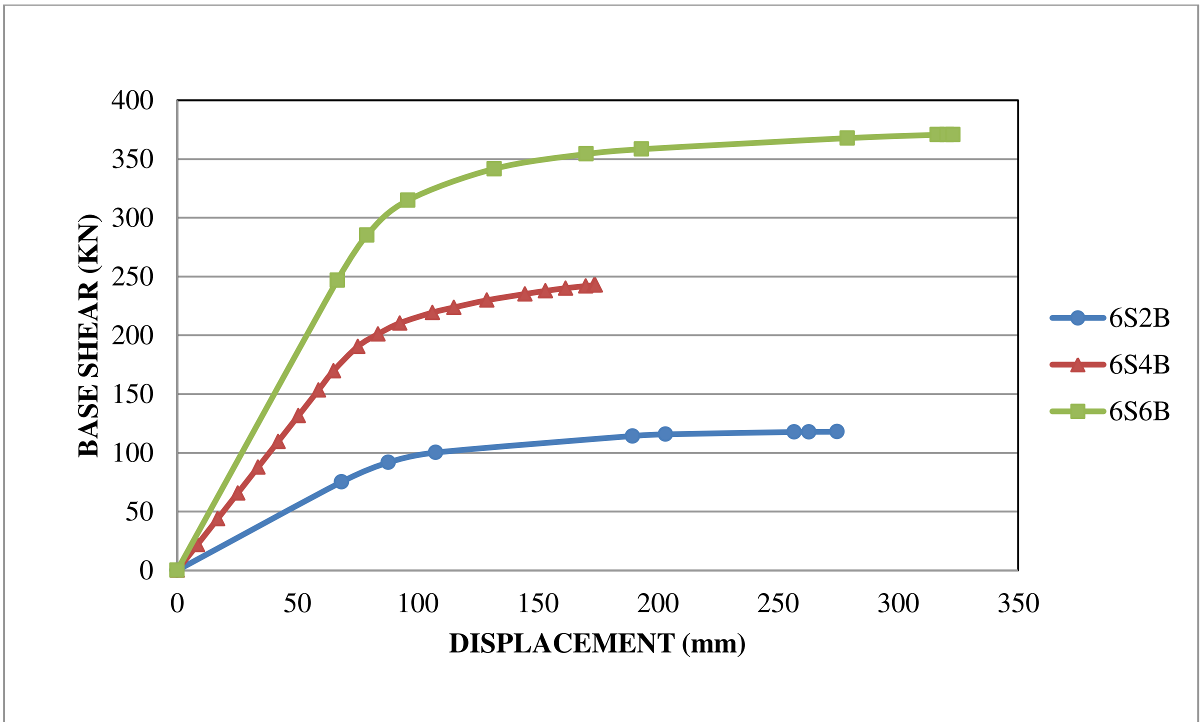

Fig.4.15 showing the bay wise comparison of SMRF buildings with fixed support conditions and no infill.

4.8 COMPARISON OF SMRF BUILDINGS WITH STRONG AND WEAK INFILL: FIXED SUPPORT CONDITION.

In this study, the performance of SMRF buildings with strong and weak infill is compared. For strong infill condition the value of modulus of elasticity of brick is taken as 5000 MPa whereas for weak infill it is taken as 350 MPa. In Fig 4.16, the static pushover curve of 6S4B SMRF building with strong and weak infill is shown. In Fig 4.16,shows the case of 6S4B SMRF buildings and it is observed that the building with strong infill can withstand a base shear of 1650 KN while the building with weak infill can resist a base shear of 700 KN. Similar behaviour is observed for 6S2B SMRF and 10S7B SMRF buildings in Fig 4.17 and Fig 4.18. It can concluded from Fig 4.16, Fig 4.17 and Fig 4.18 that, the SMRF buildings with stronger infill have base shear capacity of about 1.5 to 2.5 times more than that of SMRF buildings with weak infill. Moreover, the pushover curves for buildings modelled with weak infill are performing in a linear manner compared to those buildings which are modelled with strong infill. This suggests that SMRF buildings with strong infill perform better than those with weak infill.

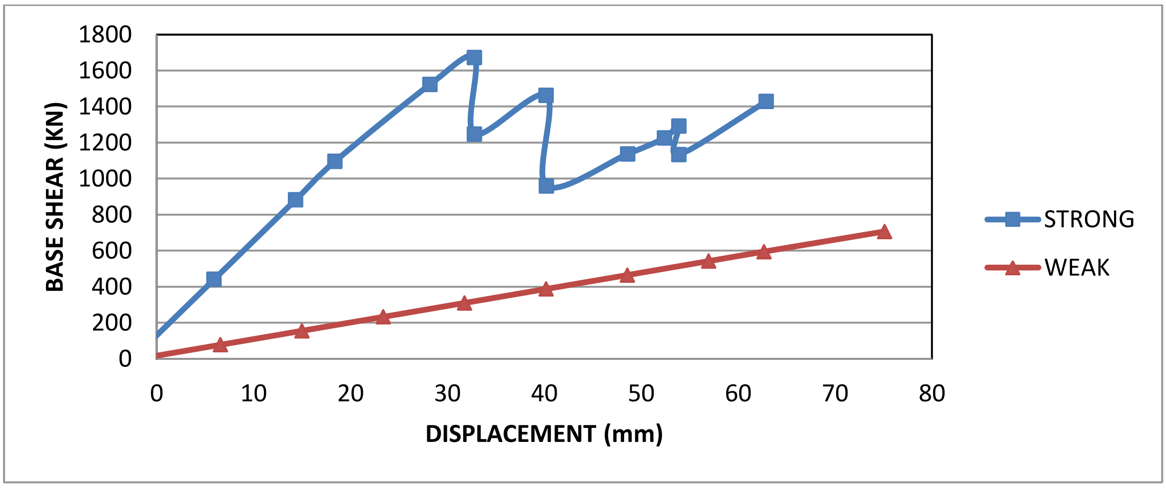

Fig.4.16 showing the comparison of 6S4B SMRF building with strong and weak infill and fixed support conditions.

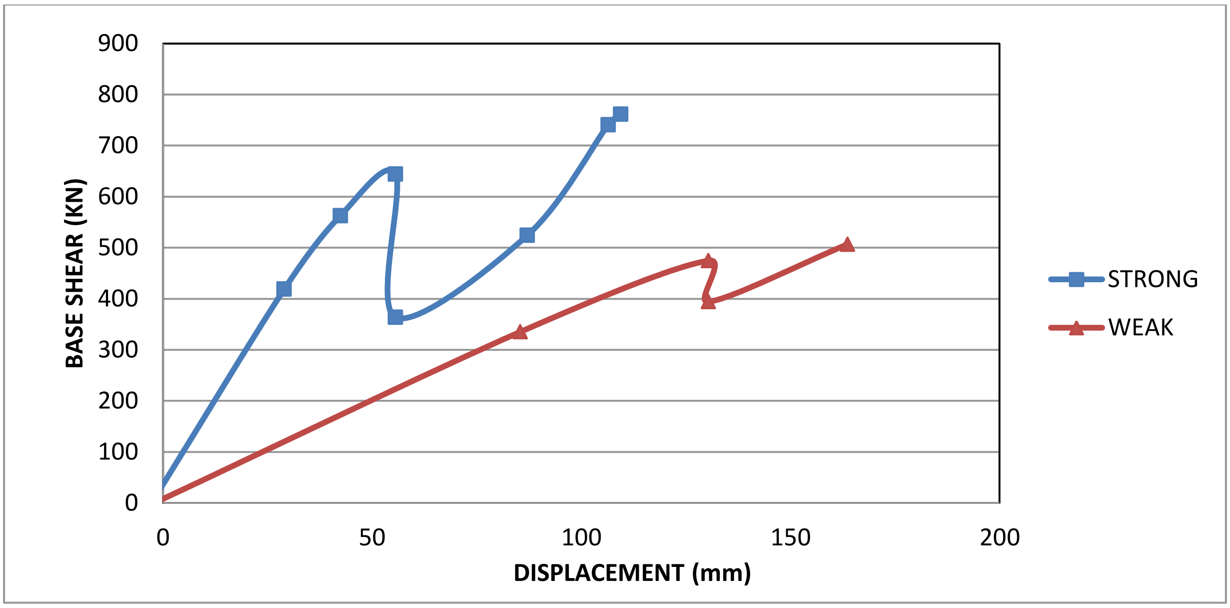

Fig.4.17 showing the comparison of 6S2B SMRF building with strong and weak infill and fixed support conditions.

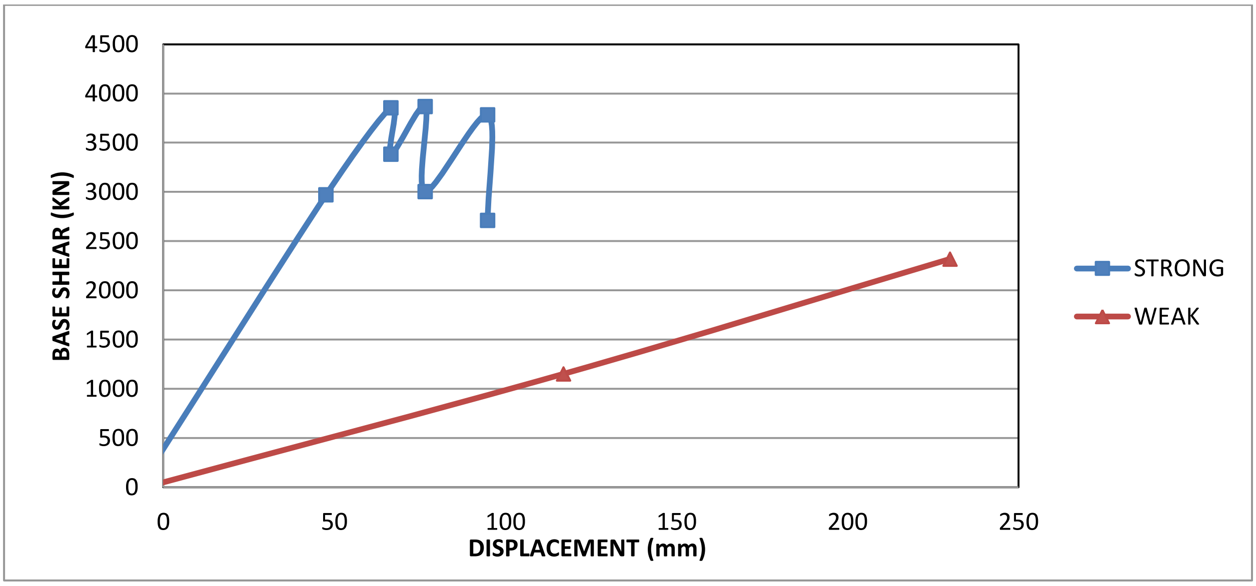

Fig.4.18 showing the comparison of 10S7B SMRF building with strong and weak infill and fixed support conditions.

4.9 SUMMARY

Static nonlinear pushover analysis is carried out on all buildings under consideration. Their response is monitored and pushover curves are plotted, comprising of Base Shear versus Roof Displacement values.

The pushover curves of SMRF buildings and OMRF buildings are compared, for both fixed and hinged support conditions. It is found that the base shear capacity of OMRF is about 20-40% more than that of a SMRF building. But the displacement capacity of SMRF is about 75-200% more than that OMRF. This concludes that SMRF buildings are more ductile than OMRF.

The SMRF buildings with fixed and hinged support conditions are also compared and it is found that the performance is almost the same.

The building behaviour parameters are also calculated from the values obtained from the pushover curve and the results are tabulated. It is found that the value of ductility factors are more for SMRF buildings, reinstating the fact that SMRF buildings are more ductile.

A comparative study on the basis of number of storeys is done for SMRF buildings and it is found that the ductility and the magnitude of base shear that can be resisted, increases slightly with increase in the number of storeys. The slope of the pushover curve for all buildings is almost the same.

A comparative study and number of bays is also carried out for the SMRF buildings and it is found that the magnitude of base shear that can be resisted increases with increase in the number of bays. As the number of bays increases from 2 to 4, the base shear capacity will increase by 2 times. And when it increases from 2 bays to 6 bays, the magnitude of the base shear the building can withstand increase by 3 times It can be proposed that the number of bays play a major role in the stability of a building.

The pushover curves of SMRF buildings with strong infill and weak infill is also compared and it is concluded that the SMRF buildings with stronger infill have base shear capacity of about 1.5 to 2.5 times more than that of SMRF buildings with weak infill.

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CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 CONCLUSIONS

The performance assessment of buildings designed as Special Moment Resisting Frame (SMRF) and Ordinary Moment Resisting Frame (OMRF) is studied for different building configurations, infill conditions and support conditions. The buildings are designed and modelled using computational software. Nonlinear analysis is performed on these buildings and the response is monitored. A pushover curve comprising of Base Shear versus Roof Displacement is plotted for each frame using the analysis data. Several comparative studies are carried out to study the behaviour of SMRF and OMRF.

- ❖ The behaviour of SMRF building and OMRF building with no infill and fixed support conditions are compared. It is found that the buildings designed as SMRF perform much better compared to the OMRF building. The ductility of SMRF buildings is almost 75% to 200% 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 20 to 40% more than that of SMRF buildings.
- ❖ The behaviour of SMRF building and OMRF building with no infill and hinged support conditions are compared. It is found that the buildings designed as SMRF perform much better compared to the OMRF building. The ductility of SMRF is more in all cases which goes about 75-200% than that of OMRF buildings. But OMRF buildings resist 20-40% more base shear than that be resisted by SMRF buildings.

- The behaviour of SMRF building with fixed and hinged support conditions are compared. It is found that performance of SMRF buildings under fixed and hinged support condition is the same. It is concluded that the support conditions doesn't have a major role in the current study.
- The building beahaviour parameters such as the ductility reduction factor R_μ , the overstrength factor R_S, and the ductility factor μ , are calculated from the pushover curve of each building. The behaviour parameters give an idea about the performance of the building and from the values of R_μ and μ obtained, it can be concluded that SMRF buildings possess higher ductility than OMRF buildings. The overstrength factor R_s, is also having a value greater than 1 in all cases depicting the fact that the buildings designed for current study can withstand more loads than what they are designed for.
- 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.
- The SMRF buildings with same number of storeys and different number of bays are compared. The pushover curve is plotted and it is found that the magnitude of base shear that can be resisted increases with increase in the number of bays. As the number of bays increases from 2 to 4, the base shear capacity will increase by 2 times. And when it increases from 2 bays to 6 bays, the magnitude of the base shear the building can withstand increase by 3 times It can be proposed that the number of bays play a major role in the stability of the buildings considered for the present study.

- ❖ The SMRF buildings with strong and weak infill are compared and it is found that the buildings with strong infill can withstand a higher magnitude of base shear when compared to those with weak infill. It can be concluded that the SMRF buildings with stronger infill have base shear capacity of about 1.5 to 2.5 times more than that of SMRF buildings with weak infill. Although, a precise conclusion cannot be drawn out for ductility, it can be suggested that weak infill are not preferred due to their linear nature in the pushover curve.

Although pushover analyses gives an insight about nonlinear behaviour imposed on structure by seismic action, pushover analyses were not able to reasonably capture neither the exact sequence of hinging nor their locations in some cases. Therefore, design and seismic evaluation process should be performed by keeping in mind that some amount of variation always exists in seismic demand prediction of pushover analysis.

Finally, more systematic and complete parametric studies, considering different periods, strength ratios, and earthquake ground motions, however, will be required to establish definite criteria for efficient design of reinforced concrete special moment resisting frame system.

CHAPTER 6

REFERENCES

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REFERENCES

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