North Asian International Research Journal of Sciences, Engineering & I.T.

ISSN: 2454-7514 Vol. 8, Issue-5 May-2022

IRJIF. I.F. : 3.821

Index Copernicus Value: 52.88 Indian Citation Index Thomson Reuters ID: S-8304-2016

NAIRJC A Peer Reviewed Refereed Journal

DOI: 10.55627/nairjsei.2022.9.7.7

PERFORMANCE ASSESSMENT OF MULTISTOREYED RC SPECIAL MOMENT RESISTING FRAMES

[1] RASHID NAZIR MIR & [2]BRAHAMJEET SINGH

[1]M.tec Scholar, Department Of Civil Engineering, RIMT University, Mandi Gobindgarh, Punjab, India

[2]Assistant Professor, Department Of Civil Engineering, RIMT University, Mandi Gobindgarh, Punjab, India

ABSTRACT

Reinforced concrete special moment frames are used as part of seismic force-resisting systems inbuildings 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 groundshaking. 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.

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

INTRODUCTION

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 wasessentially at the discretion of the designer, as it was not until 1973 that the Uniform BuildingCode (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.

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.

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 itexperiences 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.

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 MomentResisting Frames.

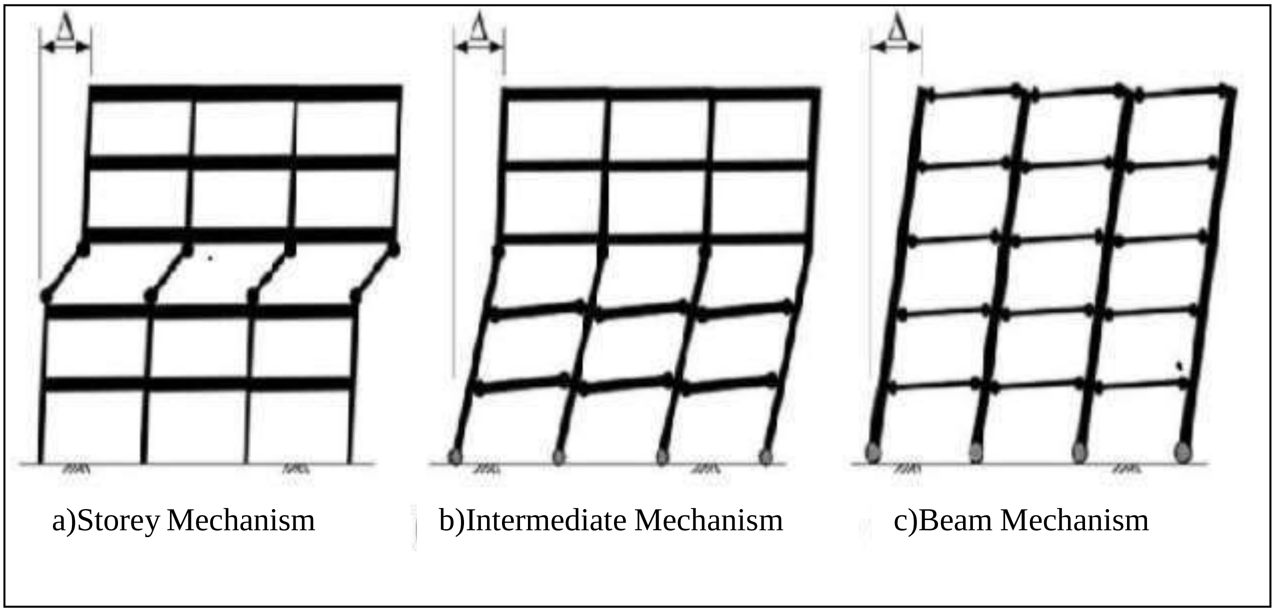

Fig. 1.1 Different failure mechanisms

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.

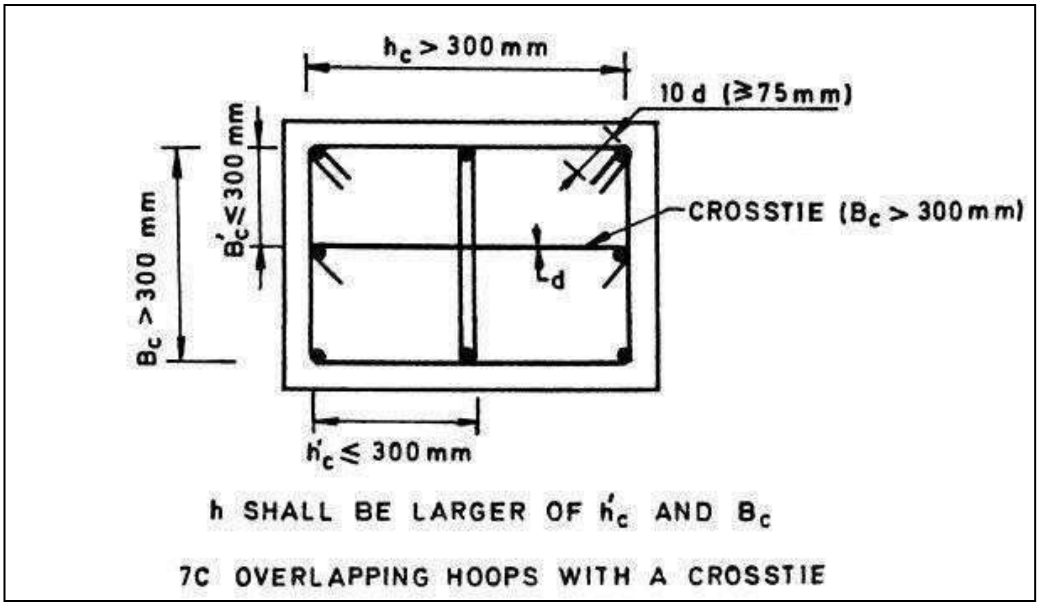

Fig 1.2 – **Shear Reinforcement in beams as per IS 13920 (2002)**

OBJECTIVES

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

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.

LITERATURE REVIEW

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 analysistechnique 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 inelasticbehaviour 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 vulnerableto damage than those in upper stories.

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 inSMRF.

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.

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 inall 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 conditi
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 3.5 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 (Em)	5000MPa

6	Compressive strength of weak masonry (*Em*)	350MPa
7	Modulus of elasticity of Masonry Infill walls (*Em*)	750*f'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 forOMRF 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

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 ideaof 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 thea typical pushover curve diagram is shown in fig 3.1 below:

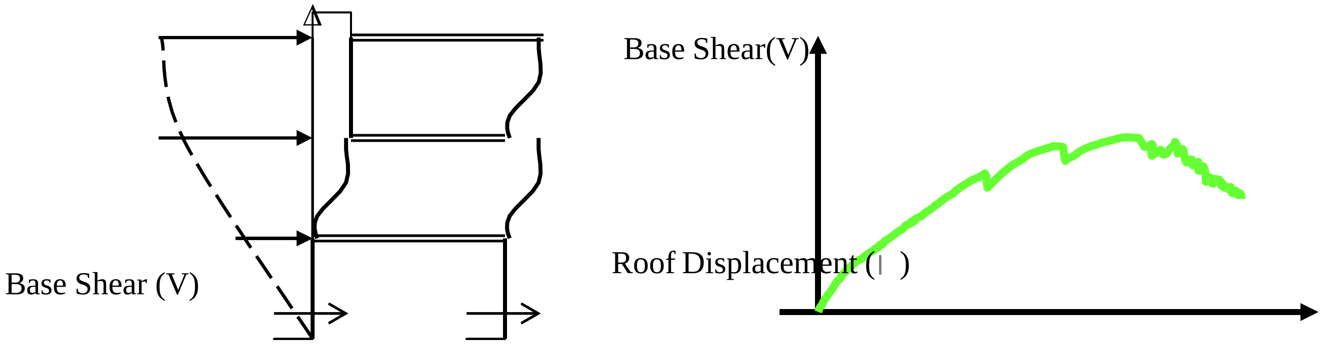

Fig.3.1 Typical Pushover Curve

PERFORMANCE ASSESSMENT OF DESIGNED FRAMES

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 4S7BOMRF

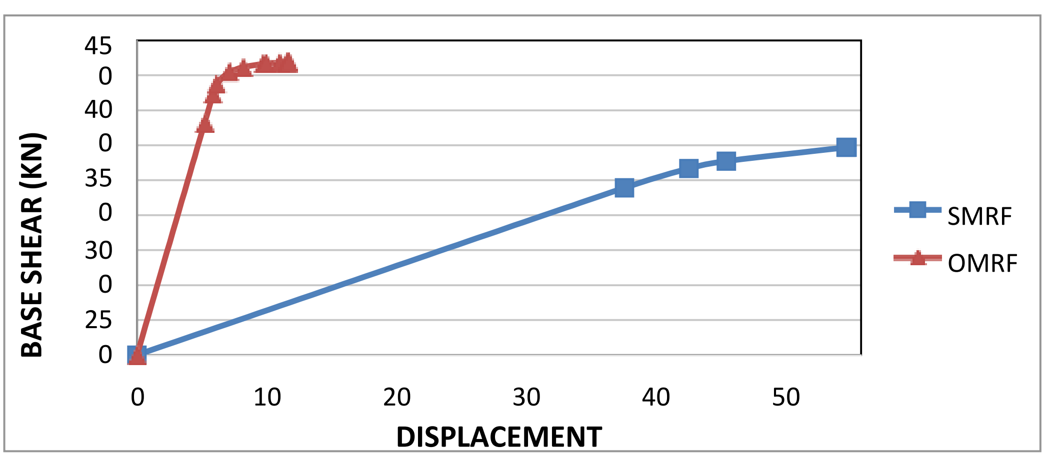

Fig 4.1 shows the pushover curves of 4S7B OMRF AND 4S7B SMRF with Fixed supportcondition and no infill.

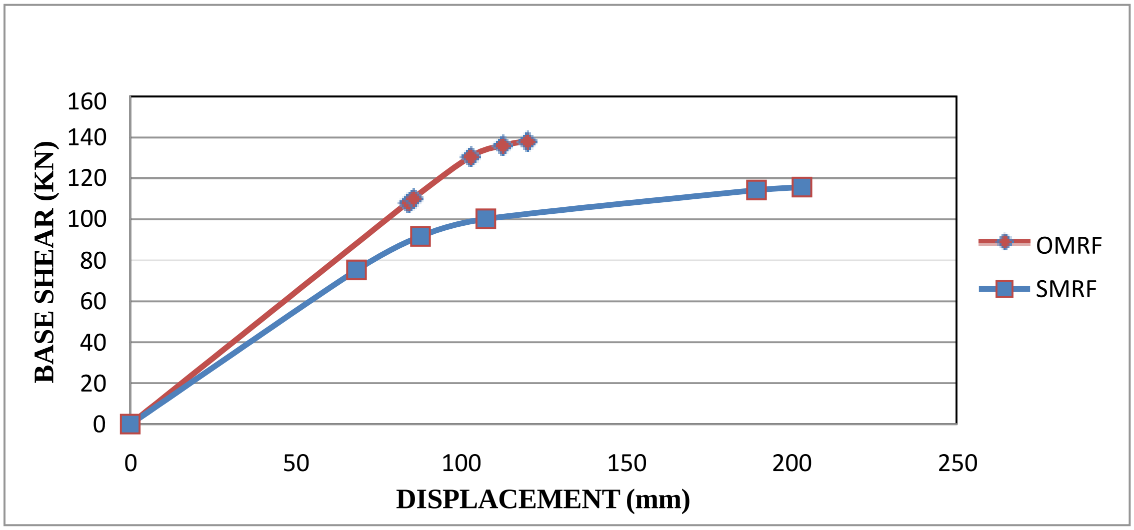

Fig 4.2 shows the pushover curves of 6S2B OMRF AND 6S2B SMRF with Fixed supportcondition and no infill.

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 andmodelled 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 supportconditions 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 stirrupsas 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 morein 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 RS, 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 $R\mu$ and μ obtained, it can be concluded that SMRF buildings possess higher ductility than OMRF buildings. The overstrength factor Rs, 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.

REFERENCES

1. ACI 318, (2005) "Building code requirement for reinforced concrete and commentary", ACI 318-05/ACI 318R-05, American Concrete Institute.
2. Achyutha, H., R. Jagadish and S. S. Rahman (1982)," Effect of contact between infill and frame on the behaviour of infilled multi-storey frames", Proceedings of the 6th International Brick Masonry Conference. Rome.
3. Asokan, A., (2006) Modelling of Masonry Infill Walls for Nonlinear Static Analysis of Buildings under Seismic Loads. M. S. Thesis, Indian Institute of Technology Madras, Chennai
4. A. Shuraim, A. Charif, (2007) Performance of pushover procedure in evaluating the seismic adequacy of reinforced concrete frames. King Saud University
5. ATC 40, (1996), "Seismic Evaluation and Retrofit of Concrete Buildings", Applied Technology Council, USA.
6. Akbas, B., Kara, F.I., and Tugsal, U.M. (2003), "Comparison of Pushover Analysis and Nonlinear Dynamic Time History Analysis on Low-, Medium-, and High-Rise Steel Frames", Project No. 02-A-02-01-03, Scientific Research Project Fund, Gebze Institute of Technology
7. Alhamaydeh, M., Abdullah, S., Hamid, A., & Mustapha, A. (2011). Seismic design factors for RC special moment resisting frames in Dubai , UAE, 10(4), 495–506.
8. Athanassiadou CJ. (2008), Seismic performance of R/C plane frames irregular inelevation. Engineering Structures 2008;30:1250
9. Chandler, A. M. Duan, (1986), "Building Damage in Mexico City Earthquake", Nature, 320(6062), 497-50.1