International Journal of Research in Engineering, Science and Management
Volume-2, Issue-11, November-2019
www.ijresm.com | ISSN (Online): 2581-5792

Performance of Multistoreyed RC Special Moment Resisting Frames

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***Abstract*: Reinforced concrete special moment frames area unit used as part of unstable seismic force-resisting systems in buildings that area unit designed to resist earthquakes. Beams, columns and beam-columns joints in moment frames area unit proportioned and elaborated to resist flexural, axial and shear sections that result as a building sway through multiple displacements cycles throughout sturdy earthquake ground shaking. Special proportioning and particularization needs to leads to a frame capable of resisting sturdy earthquake shaking while not vital loss of stiffness or strength. These moments resisting frames area unit referred to as "Special Moment Resisting Frames" as a result of these further needs that improve the unstable resistance compared with less strictly elaborated Intermediate and normal Moment Resisting frames. The design criteria for SMRF building area unit given in IS13920 (2002), during this study, the building area unit designed each as SMRF and OMRF, and their performance is compared. For this the building area unit modeled and pushover analysis is performed in ETABS. The pushover curves area unit is premeditated from the analysis result and therefore the behavior of building is studied for varied support condition and infill conditions. The behavior parameters are found for every building victimization the values obtained from pushover curves and are investigated.**

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

1. Introduction

Reinforced concrete special moment frames area unit used as part of unstable seismic force-resisting systems in buildings that area unit designed to resist earthquakes. Beams, columns and beam-columns joints in moment frames area unit proportioned and elaborated to resist flexural, axial and shear sections that result as a building sway through multiple displacements cycles throughout sturdy earthquake ground shaking. Special proportioning and particularization needs to leads to a frame capable of resisting sturdy earthquake shaking while not vital loss of stiffness or strength. These moments resisting frames area unit referred to as "Special Moment Resisting Frames"

Moment frames are usually because the seismic force-resisting system once discipline area designing flexibility is desired. Once concrete moment frames are chosen for buildings appointed to Seismic Design Categories III, IV or V, they're needed to be detailed as special concrete moment frames. Proportioning and particularization needs for a special moment frame can be alter the frame to soundly bear in depth inelastic deformations that are anticipated in these seismic design categories. Special moment frames are also utilized in Seismic Design Categories I or II, although this might not cause the foremost economical design. Each strength and stiffness have to be compelled to be thought of within the design of special moment frames. In 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 associate elastic response analysis. Moment frames are usually versatile lateral systems; thus, strength needs are also controlled by the minimum base shear equations of the code.

A. Principles of Design for Special Moment Resisting Frames

The proportioning and particularization necessities for special moment frames are meant to confirm that inelastic response is ductile. 3 main goals are: (1) to realize a strong-column/weak-beam design that spreads inelastic response over many stories; (2) to avoid shear failure; and (3) to produce details that alter ductile flexural response in yielding regions.

B. Strong Column Weak Beam Concept

When a building sways throughout an earthquake, the distribution of damage over height depends on the distribution of lateral drift. If the building was weak columns, drift tends to concentrate in one or a number of stories (Fig 1-1a), and will exceed the drift capability of the columns. On the opposite hand, if columns offer a stiff and powerful spine over the building height, drift are going to be a lot of uniformly distributed (Fig. 1 (c)), and localized damage are going to be reduced. The type of failure that's shown in Fig. 1 (c) is thought as Beam Mechanism or Sway Mechanism. To boost it's vital to acknowledge, that the columns in a given story support the load of the complete building on top of those columns, whereas the beams solely support the gravity loads of the floor of that kind a part; thus, failure of a column is of larger consequence than failure of a beam. Recognizing this behavior, building codes specify that columns be stronger than the beams that frame into them. This strong-column/weak-beam principle is key to

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Volume-2, Issue-11, November-2019
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achieving safe behavior of frames throughout strong earthquake ground shaking. It is a design principle that has to 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 every beam-column association on of a special moment frame.

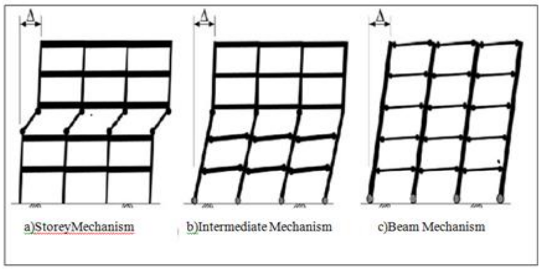

Fig. 1. Different failure mechanism

C. *Avoidance of Shear Failure*

Ductile response needs that members yield in flexure, which shear failure be avoided. Shear failure, particularly in columns, is comparatively brittle and might loss fast loss of lateral strength and axial load-carrying capacity. Column shear failure is that the most often cited explanation for concrete building failure and collapse in earthquakes Shear failure is avoided through use of a capacity-design approach. The overall approach is to spot flexural yielding regions, design those regions for code-required moment strengths, so calculate design shears based on equilibrium presumptions the flexural yielding regions develop probable moment strengths. The probable moment strength is calculated victimization that manufactures a high estimate of the instant moment strength of the designed cross section.

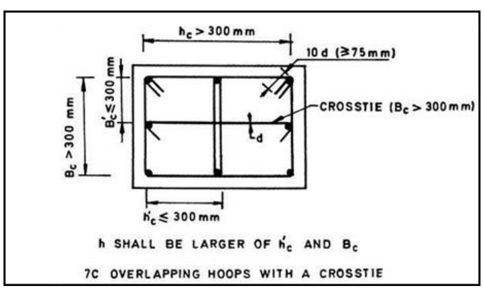

Fig. 2. Shear reinforcement in beams as per IS13920 (2002)

D. *Detailing for Ductile Behavior*

For achieving a ductile nature, importance should lean for the detailing in reinforcement. The assorted factor that ought to be taken care of is mentioned below. The ductile nature of the building is heavily addicted on the detailing pattern and improper detailing may end up in failure of the building while not enough warning.

1) *Confinement for Heavily Loaded Sections*

Plain concrete has comparatively small usable compressive strain capacity (around 0.003), and this would possibly limit the deformability of beams and columns of special moment frames. Strain capacity can be inflated ten-fold by confining the concrete with reinforcing spirals or closed hoops. The hoops act to restrain dilation of the core concrete because it is loaded in compression, and this confining action results in inflated strength and strain capability. Hoops generally area unit provided at the ends of columns, moreover as through beam-column joints, and at the ends of beams. To be effective, the hoops must enclose the complete cross section except the quilt concrete, that ought to be as tiny as allowable, and must be closed by 135° hooks embedded within the core concrete; this prevents the hoops from gap if the concrete cowl spalls off. Crossties ought to interact longitudinal reinforcement round the perimeter to enhance confinement effectiveness. The hoops ought to be closely spaced on the longitudinal axis of the member, each to confine the concrete and restrain buckling of longitudinal reinforcement. Crossties, which usually have 90° and 135° hooks to facilitate construction, should have their 90° and 135° hooks alternated on the length of the member to enhance confinement effectiveness.

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.

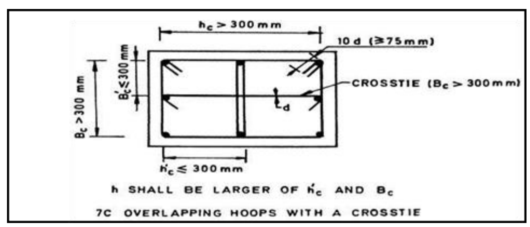

Fig. 3. Transverse reinforcement in columns as per IS13920 (2002)

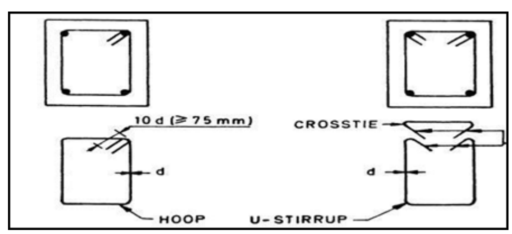

Fig. 4. Beam web reinforcement as per IS13920 (2002)

2. Objectives

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

3. Literature review

Holmes [1] (1961): Underneath lateral loading, the frame and therefore the infill wall keep intact initially. Because lateral load increases, the infill wall gets separated from the encompassing frame at the unloaded (tension) corner. However, at the compression corners the infill walls are still intact. The length over which the infill walls and therefore the frame measure intact is named the length of contact. Load transfer happens through an unreal diagonal that acts sort of compression strut. Because of this behavior of infill wall, they can be modeled as an equivalent diagonal strut connecting the 2 compressive corners diagonally. The stiffness property ought to be such that the strut is active only if subjected to compression. Thus, underneath lateral loading only 1 diagonal will be operational at a time.

Rao et. al. [2] (1982): conducted theoretical and experimental studies on infill frames with gap strong by lintel beams. It absolutely was ended the lintel over the gap doesn't have any influence on the lateral stiffness of an infill frame.

Rutenberg [3] (1992): Distinguished that the analytical works considering single element models couldn't yield the ductility demand parameter properly, as a result need they thought-about distribution of strength in same proportion as their elastic stiffness distribution. Considering these drawbacks of the equivalent single element model, several investigations during this field adopted a generalized type of structural model which had a rigid deck supported by totally different numbers of lateral load-resisting elements representing frames or walls having strength and stiffness in their planes lonely.

Helmut Krawinkler [4] (1998): studied the professionals and cons of Pushover analysis and steered that component behavior can't be evaluated in the context of presently used employed world system quality factors such as the R and R_w factors used in present US seismic codes. They additionally steered that a carefully performed pushover analysis can give insight into structural aspects that manage performance throughout severe earthquakes. For structures that vibrate primarily within the basic mode, the pushover analysis can terribly give smart estimates of world, as well as local nonresilient, deformation demands. This associate in analysis will also expose design weaknesses that could stay hidden in an elastic analysis. Such weaknesses include story mechanisms, excessive deformation demands, strength irregularities and overloads on probably brittle elements such as columns and connections.

Foley CM et. al., [5] (2002): studied a review of current progressive seismic performance based mostly design procedures and represented the vision for the event of PBD optimization. Its recognized that there's a pressing want for developing optimized PBD procedures for seismic engineering of structures.

R. Hasan and D.E. Grierson [6] (2002): conducted an easy computer-based push-over analysis technique for performance-based design of building frameworks subject to earthquake loading. And located that rigidity-factor for elastic analysis of semi-rigid frames, and therefore the stiffness properties for semi-rigid analysis are directly adopted for push-over analysis.

4. Methodology

A. Problem statement

Table 1
Details of all fixed support bare frames

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

Table 2
Materials and geometric properties 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

Table 3
Seismic data assumed for SMRF

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 4
Seismic data assumed for SMRF

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

Table 5
Loads considered for design building

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

5. Results

Comparison of SMRF and OMRF: BARE FRAME, FIXED SUPPORT:

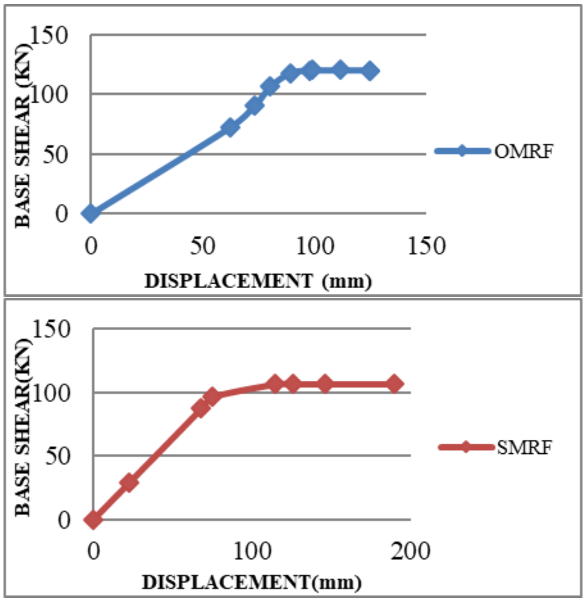

Fig. 5. Shows the pushover curve of 6S2B of SMRF and OMRF with fixed support and no infill

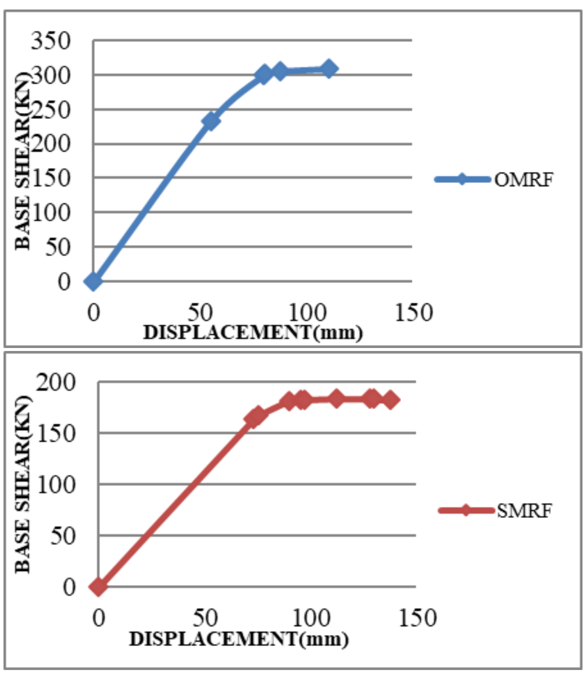

Fig. 6. Shows the pushover curve of 6S4B of SMRF and OMRF with fixed support and no infill

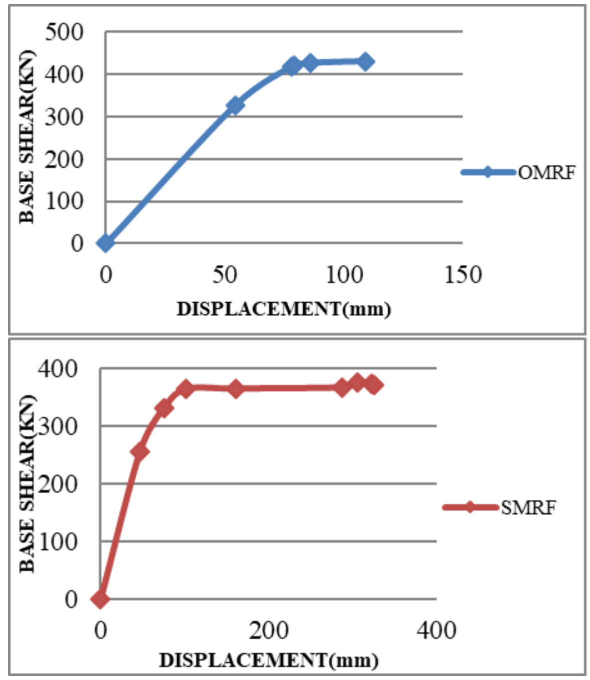

Fig. 7. Shows the pushover curve of 6S6B of SMRF and OMRF with fixed support and no infill

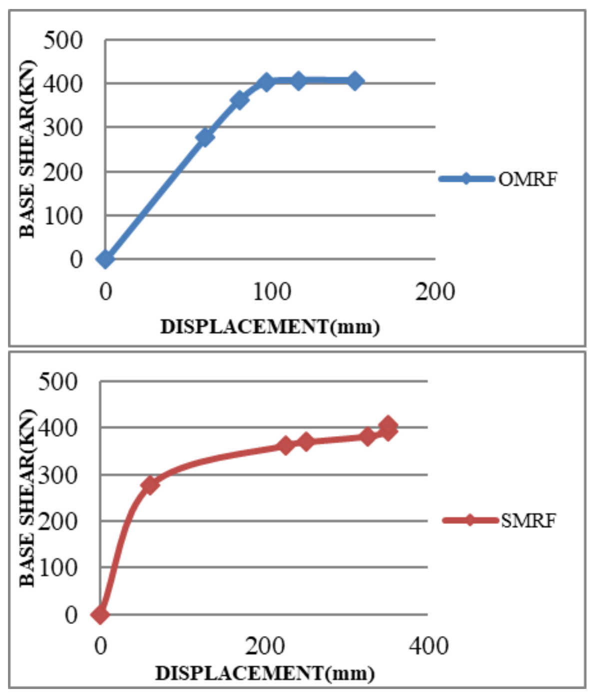

Fig. 8. Shows the pushover curve of 8S7B of SMRF and OMRF with fixed support and no infill

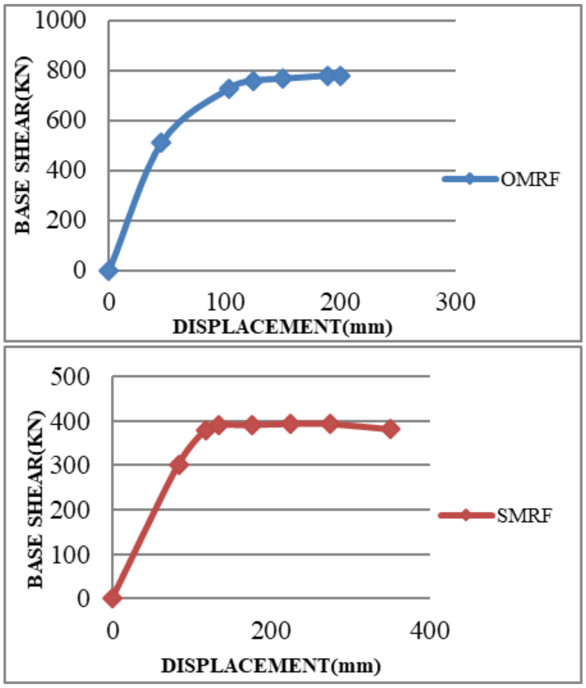

Fig. 9. Shows the pushover curve of 10S7B of SMRF and OMRF with fixed support and no infill.

Table 6 shows the 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 6
Performance comparison of SMRF and OMRF 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	
6S2B	121	109	11.06%	125	190	52.2%
6S4B	309	182	69.7%	110	138	25.45%
6S6B	430	372	15.59%	110	372	238%
8S7B	407	410	7.8%	152	351	130%
10S7B	778	381	48.4%	201	351	74%

6. Conclusion

The behavior of SMRF and OMRF building with no infill and glued support conditions area unit compared. It's found that the building designed as SMRF perform far better compared to the OMRF building. The ductility of SMRF building is nearly 50% to 240% over the OMRF building altogether cases, the rationale being the significant confinement of concrete because of conjunction and usage of additional range of stirrups as ductile reinforcement. It's found that the bottom shear capability of OMRF building is 11% to 70% over that of SMRF building.

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