EVALUATION OF RESPONSE REDUCTION FACTOR FOR REINFORCED CONCRETE FRAMES WITH SHEAR WALL.

[1]Prashant R. Barbude, [2]Amol S. Jadhav, [3]Dr.T.N.Boob

[1]Assistant Professor, [2]Under Graduate Student, [3]Principal
[1]Department of Civil Engineering,
[1]DATTA MEGHE COLLEGE OF ENGINEERING, AIROLI, NAVI MUMBAI.
[3]Dr.N.P. HIRANI INSTITUTE OF POLYTECHNIC,PUSAD,YAVATMAL.

Abstract : Response reduction factor is invariably used by most seismic design codes to include the non-linear response of structure. The non-linear behaviour of the structure is taken into account due to this factor which permits the designer to use a linear elastic force-based design. Using non-linear static pushover analysis the estimation of seismic response reduction factor for a dual system of reinforced concrete special moment resisting frame(SMRF) and shear walls has been focused on in these study. The design of the frames is done using the Indian standards for seismic and RC design and are subjected to two different lateral load patterns. Based on the push over curve which is obtained between base shear and roof displacement the actual values of Response reduction factor have been worked out. After analysis it was observed that the response reduction factor depends on four major factors, they are Strength factor, Ductility factor, Redundancy factor and damping factor affect and hence this four factors must be taken into consideration while determining the appropriate response reduction factor to be used while designing a building for earthquake resistance.

Index Terms - Response reduction factor, Base shear, Pushover analysis.

I. INTRODUCTION

Lateral strength along with ductility capacity and deformability of the structure with a limited damage and no collapse play a major role in the design of the earthquake resistant building. A building should be designed in such a manner that, after a earthquake strikes the building the structure may get physical damaged but it should not collapse even under action of a high intensity earthquake. Hence it becomes mandatory for a structural designer to provide sufficient ductility to the building so that it is able to resist high intensity earthquakes that it may probably face during its lifetime.

IS 1893(Part 1):2002 is primarily used for design of earthquake resistant buildings and takes into account the non-linear response in selected components and elements when subjected to an earthquake of design intensity level. But the design code does not clearly mention the inelastic response of a structure in methodology of designing. The non-linear behaviour of the structure is taken into account by the use of response reduction factor. The factor is dependent upon the perceived seismic damage performance of the structure and characteristics of ductile or brittle deformation. By using the response reduction factor the liner elastic design

spectrum can be reduced and the energy dissipation capacity of the structure can be taken into account.

In using this approach, the design base shear (Vd) is derived by dividing the elastic base shear demand (Ve), which is obtained using an elastic analysis considering the elastic pseudo-acceleration response spectrum (for 5% damping, Sa,5) by a factor R.

$$V_d = \frac{V_e}{R} = \frac{S_{a,5}}{R} W \tag{1}$$

Where W is the seismic weight of the structure. In an attempt to consider a structures inelastic characteristics in linear analysis method we incorporate the response reduction factor. By considering the vertical load carrying capacity, a limiting inelastic yeilding must be allowed in the structure.

The structural designer assigns the value of R on the basis of the past responses of similar buildings in the vicinity. The response reduction factor for RC special moment resisting frame with ductile shear wall is given as 5 in the Indian Standard. But the factors governing the response reduction factor on basis of which they are allocated to different types of frame are not clearly mentioned in this code. Hence, the objective of this study is to obtain the values of response reduction factor for special moment resisting reinforced concrete regular frame structure with shear walls designed and detailed as per IS456, IS1893 and IS 13920.

2.Components of R

The response reduction factor is given by the following formula:

$$R = R_s \times R_\mu \times R_\xi \times R_R \tag{2}$$

Where Rs is the strength factor, R_μ is the ductility factor, R_ξ is the damping factor and R_R is the redundancy factor. The strength factor (R_S) is a measure of the built-in over strength in the structural system and is obtained by dividing the ultimate base shear (V_u) by the design base shear (V_d).

$$R_S = \frac{V_u}{V_d} \tag{3}$$

The ductility factor is a measure of the global nonlinear response of a structural system in terms of its plastic deformation capacity.

In present study, the relationship between R_μ and ductility level (μ) developed by T. Paulay and M. J. N. Priestley is used. As per T. Paulay and M. J. N. Priestley, the relationship is given by,

$$R\mu = 1 + (\mu - 1)\, T/0.70 \tag{4}$$

Where, μ is the ductility capacity which is the ratio of the ultimate displacement to the yield displacement and, T is the fundamental time period of the structure. In order to take effect of added viscous damping the damping factor R_ξ is used which is primarily applicable for structures provided with supplemental energy dissipating devices. When such devices are not used, the damping factor is usually assigned a value equal to 1.

Redundant structural system are categorized into RC structural system with multiple lines of lateral load resisting frames. The redundancy factor for redundant structures is taken as 1.

Values of R for RC framed structures, as per IS 1893(Table 1).

Structural system	R
Ordinary moment resisting frame (OMRF)	3.0
Special moment resisting frame (SMRF)	5.0
Ductile shear wall with SMRF	5.0

The SMRF needs to follow the ductile detailing requirements of IS 13920. For a dual system of SMRF and ductile shear walls IS 1893 suggests the value of response reduction factor as 5.0. It does not explicitly segregate the components of R in terms of ductility and over-strength. Also it does not specify any reduction in the response reduction factor on account of any irregularity (vertical or plan-irregularity) in the framing system.

3. Structural performance limits

Response reduction factor is integrated to selected performance limit state of structure. The limit state to which the values of R corresponds is not specified in IS1893. But it can be safely assumed that the values are based on the Ultimate limit state, based on the design philosophy outlined in the initial sections of the seismic design code. A detailed look into the selection and definition of a Performance limit state to obtain the value of R is required since similar specifications are provided in the new design standards and guidelines around the world.

In the past 10-15 years there has been a induction of performance based seismic design(PBSD) into the Earthquake engineering field. Clear definitions are provided by PBSD guidelines for multiple performance limit state of various types. According to PBSD terminology, the limit states are typically known as structural 'performance levels' which in combination with seismic 'hazard levels' define the 'performance objective' for a structure. Based on the structural type and its intended function the performance levels are defined.

The Commonly followed guidelines for the performance assessment of RC structures are ATC-40(ATC-1996), FEMA356(FEMA-1997) and FEMA273(FEMA-1997). Few of the performance guidelines specified in the this codes are: Operational, Immediate occupancy, Life safety, Collapse prevention etc. The performance limits can be broadly grouped into two categories:

1. Global/Structural limits: They include requirements for the vertical load capacity, lateral load resistance and lateral drift.
2. Local/Elemental/Component limits: The local performance levels are typically defined based on the displacement, rotation or acceleration responses of different elements (beams, columns, shear walls, floors, etc.).

4. Description of structural systems.

Three, seven and eleven storied RC frames symmetric in plan and provided with shear walls, intended for a office building, have been considered for the study. The building is assumed to be situated in Zone III as per IS1893(2000). The structures are assumed to be constructed on medium soil as per IS1893. The design dead loads and live loads are calculated from IS875 Part 1 and IS875 Part 2 respectively. The design base shear for a building is derived as:

$$V_d = \frac{Z}{2} . \frac{I}{R} . \frac{S_a}{g} . W \tag{5}$$

where, Z denotes the zone factor (0.16 for zone III),I is the structure's importance factor (1for the buildings) ,R=5.0 for ductile or 'special' moment resisting frames (SMRF) with shear walls, Sa is the design spectral acceleration for a specific damping and site soil condition, and W is the seismic weight of the structure.

Same plan arrangement with three number of bays (5 m each in both directions) as shown in figure 2 have been used for the study. Same height of 3.5m has been provided for all the storey's with the depth of foundation equal to 2m. Fig 3 shows the typical elevations for these frames. To indicate 'short', 'medium', 'long' period structures moment resisting frames as mentioned above have been used. The fundamental period of the three structures is calculated as per the IS 1893 formulation for RC structures without brick infill,

$$T = 0.075 h^{0.75} \tag{6}$$

Table 2, gives the further details regarding the planar frames. The fundamental periods are calculated based on the empirical formula recommended in IS1893 for RC structures without brick infill and the seismic weight calculation is done as per IS1893. Based on the common practice of the engineers the models for the study have been selected. Same beam sections have been used throughout the structure, whereas the column section have been varied as per the height of the buildings as is done in common practice. The RC design is based on IS456(2000) and the ductile designing is based on IS13920(2003). Tables 3 and 4 give reinforcement details of all the sections that are used in the three models.

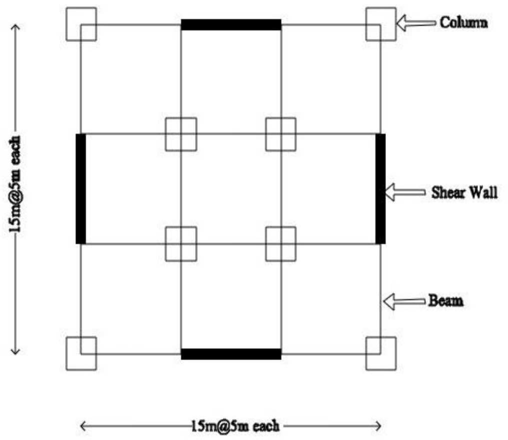


Fig.2 : Typical Plan of the study frames

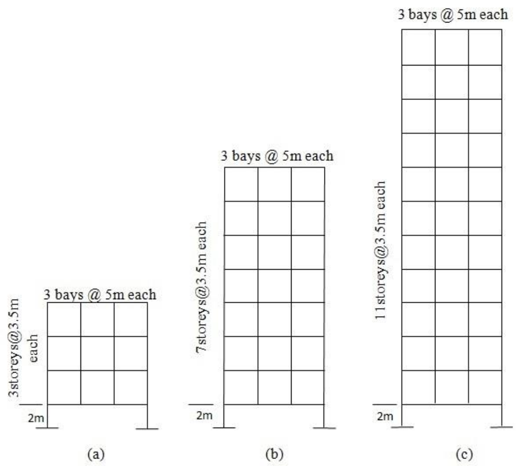


Fig.3 : Elevations of the study frames

Details of the RC frames considered for the study(Table 2)

Frame	Height (m)	S_a/g	T_d (s)	W (kN)	A_h	$V_d = A_h.W$ (kN)
3-storey	12.5	2.50	0.498	9654.50	0.04	387
7-storey	26.5	1.55	0.876	21682.54	0.025	543
11-storey	40.5	1.13	1.204	35830.94	0.018	645

Reinforcement details for shear walls(Table 3)

Frame	Members	Floors	Width (mm)	Length (mm)	Reinforcement details	
					Web	Boundary
3 storey	Shear walls	1-3	150	5000	8 mmΦ@130c/c	10mmΦ@200c/c
7 storey	Shear walls	1-7	200	5000	8 mmΦ@100c/c	12mmΦ@200c/c
11 storey	Shear walls	1-11	200	5000	10 mmΦ@130c/c	12mmΦ@150c/c

5.Modelling for RC Members

The better the non-linear behaviour of the frames the better will be the estimation o R values. The main focus in modelling scheme employed in this work is to capture non-linear static behaviour of RC structures. The nonlinear behaviour of the frame depends primarily on the moment-rotation behaviour of its members, which in turn depends on the moment-curvature characteristics of the plastic hinge section and the length of the plastic hinge. The above mentioned parameters also define the 'Component' level performance limit in terms of the plastic rotation capacity which is also described in the work.

5.1 Moment-Curvature characteristics of RC sections

For a structure subjected to seismic load, provision of ductility becomes a very important factor. To take this into account IS13920 specifies the use of transverse reinforcement or stirrups in structural members like beams and columns.

RC section details for the study frames(Table 4)

Frame	Members	Floors	Width (mm)	Depth (mm)	Reinforcement details
3storey	Beams	1-3	230	400	3-20Φ(top)+ 3-20Φ(bottom)
	Columns	1-3	300	300	10-20Φ(uniformly distributed)
7storey	Beams	1-7	250	400	6-20Φ(top)+ 3-25Φ(bottom)
	Columns	1-4	450	450	8-20Φ(uniformly distributed)
	Columns	5-7	400	400	8-16Φ(uniformly distributed)
11storey	Beams	1-11	300	450	4-25Φ(top)+ 3-20Φ(bottom)
	Columns	1-5	500	500	10-25Φ(uniformly distributed)
	Columns	6-8	400	400	10-20Φ(uniformly distributed)
	Columns	9-11	300	300	8-25Φ(uniformly distributed)

The confinement provided completely affects the magnitude of stress strain curve of concrete which leads to an increase in compressive forces of concrete. The more consistent the stress-strain model, the more consistent will be the assessment of strength and deformation behaviour of concrete members. The behaviour of concrete differs in confined and unconfined states. If the concrete is in the confined state it shows greater strength as well as greater ductility when compared to unconfined concrete. Hence it becomes necessary to use in appropriate stress-strain model which distinguishes the behaviour of confined and unconfined concrete.

The moment-curvature (M-ϕ) characteristics of the various RC sections are developed using the widely used modified Kent and Park model, which considers the confinement effect of the transverse reinforcements. Various other analytical stress-strain models referred to in the literature, are those proposed by Mander et al., Saatcioglu and Razvi, IS 456(2000). Based on the results of experiments conducted on a large number of beam-column joints of different dimensions, Sharma et al. concluded that response estimations using the modified Kent and Park model closely matched the experimental results in the Indian scenario.

A typical M-ϕ curve for a RC beam and column section for the 7-storey frame is shown in figures 4 and 5.Considering the presence of rigid floor diaphragms, the effects of axial force on the beam's M-ϕ curve are disregarded. However these effects are included while obtaining the M-ϕ curve for column sections. It is observed that there is a drop in the M-ϕ curves for both beam and column sections after the peak moment capacity is reached. This is on account of the spalling of the concrete cover when the strain in concrete in that region exceeds the ultimate strain for unconfined concrete.

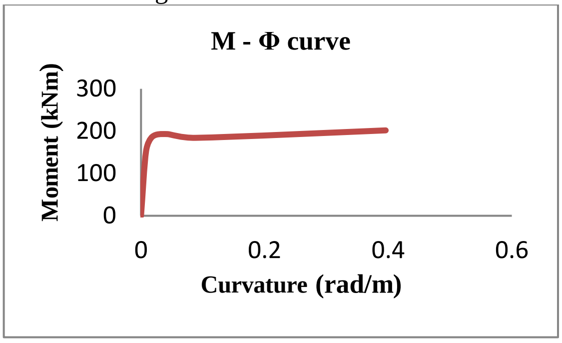


Fig. 4 : A typical M – Φ curve for a beam

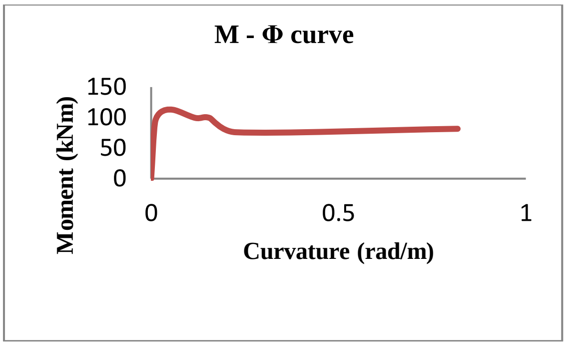


Fig. 5:A typical M – Φ curve for a column

5.2. Plastic hinge characteristics

In a reinforced concrete member the plastic rotation capacity (θ_p) depends on the ultimate curvature ($\emptyset_u$) and the yield curvature ($\emptyset_y$) of the section and the length of the plastic hinge region (L_p):

$$\theta_p = (\emptyset_u - \emptyset_y)\, L_p \tag{7}$$

Priestley proposed a model for L_p which is the most widely used and is given by:

$$L_p = 0.08L + 0.022 f_{ya} d_{bl} \tag{8}$$

Where L is the distance from the critical section to the point of contra flexure, f_{ya} is the yield strength in MPa of longitudinal bars having a diameter d_{bl}. For a moment resisting frame, where lateral loads are predominant the point of contra flexure typically occurs close to the midspan of a member. Using the above equations the plastic rotation capacities of framed structures of the three frames in study has been studied, assuming that the points of contra flexure are at the midspan of the members. The capacities have been computed for purely flexural conditions, without the effects of any axial load. Table 6 shows the plastic rotation capacities of the RC members of the 11-storeyed frame.

Plastic rotation capacities of the frame sections of 11-storey framed structure(Table 6)

Member	Size (mm)	Clear span (mm)	L_p (mm)	Φ_y (rad/mm)	Φ_u (rad/mm)	Θ_p (rad)
Beam	300x450	4700	417	6.9E-06	6.86E-05	0.0257
Column	500x500	4500	409	6.9E-06	3.57E-05	0.0118
Column	400x400	4600	366	9.1E-06	4.19E-05	0.012
Column	300x300	4700	370	1.42E-05	4.59E-05	0.0118

6. Lateral load patterns

For computing the values of R for each frame, over strength and global ductility capacity are required, for which Non Linear static pushover analyses of the three study frames are performed. The two types of lateral force distribution used in this work are as follows :

1. The equivalent lateral force distribution suggested in IS 1893 :

$$Q_i = V_d \frac{W_i h_i^2}{\sum_{i=1}^{n} W_i h_i^2} \tag{9}$$

where,
Q_i = equivalent lateral force on the ith floor ;
W_i = seismic weight of the ith floor ;
h_i = height upto the ith floor ;
n = total number of storeys.

As the equation of the lateral force is of degree two, this type of lateral load distribution is referred to as 'quadratic' load pattern in this study.

2. The buildings are subjected to a lateral load distributed across the height ofthe building based on the following formula specified in FEMA-356:

$$F_x = \frac{W_x . h_x^{\,k}}{\sum W_i . h_i^{\,k}} . V \tag{10}$$

In the above expression,
$F_x$is the applied lateral force at level '*x*',
W is the story weight,

his the story height and
Vis the design base shear.

This results in an 'inverted triangular' distribution of the lateral load when the period-dependent power *k* is set equal to unity. Figures 6 and 7 shows both the quadratic and inverted triangular load patterns as applied on the 11-storeyed frame.

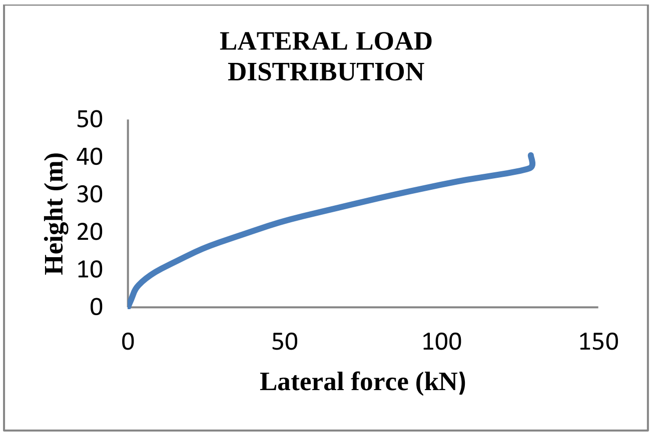


Fig. 6: Quadratic lateral load applied on the 11-storey frame

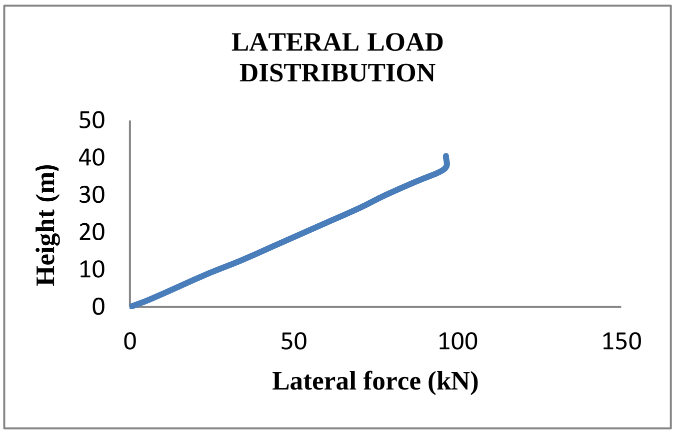


Fig. 7: Inverted triangular lateral load applied on the 11-storey frame

7. Non linear static pushover analysis

Using SAP2000, the non-linear static pushover analysis has been carried out on the three dimensional frames. All the beams and columns have been modelled using the two noded frame element and shear walls are modelled using shell area element available in SAP2000. The axial load effects are neglected in the case of beams, by using the rigid diaphragm effect. In case of columns, the effect of axial loads on plastic hinges are considered using P-M-M interaction diagram for each different RC section. No shear hinge formation is considered in these analysis, as various design and detailing provisions specified in IS 13920 eliminate the possibility of such a failure.

Pushover curve are generally used to present the output of non-linear static analysis. Pushover Curve is nothing but a plot of base shear versus roof displacement plot. Figures 8,9,10 represent the pushover curve obtained from the non-linear analysis of the three study frames.

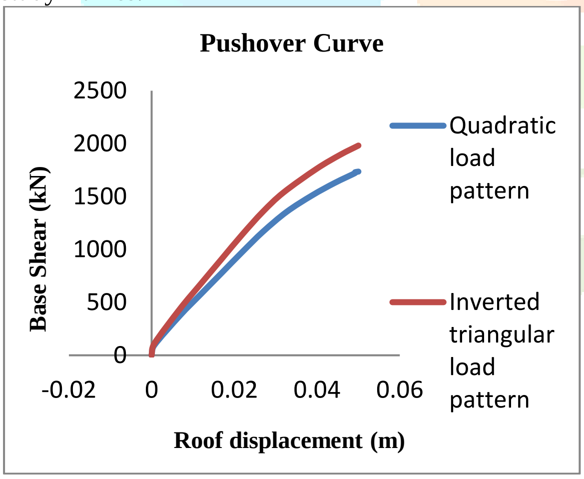


Fig. 8: Pushover curve for 3-storeyed frame

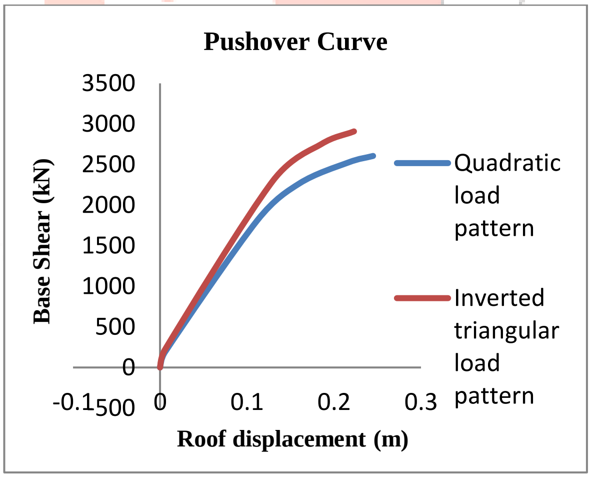


Fig. 9: Pushover curve for 7-storeyed frame

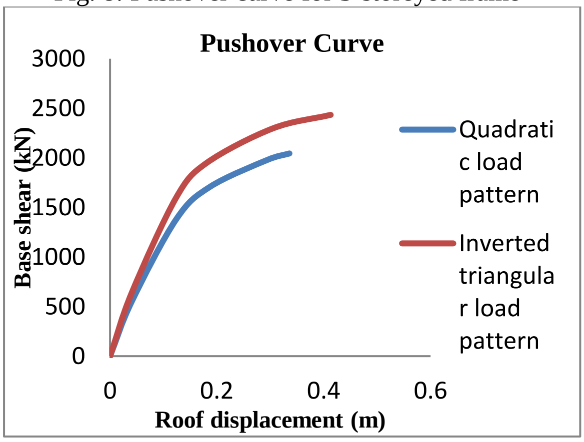


Fig 10: Pushover curve for 11-storeyed building.

8. Computation of 'R'

Base shear is created when a structure is subjected to seismic loads. This base shear is prominently higher than the actual structural response created. Thus it possesses a significant of reserve strength or over strength. When the maximum lateral strength of the structure exceeds its design strength, Over-strength is developed. Once it enters the inelastic phase the structure is capable of resisting and absorbing large amount of seismic energy. Taking benefit of the over-strength and ductility a structure possess the Seismic codes introduce a reduction in design codes. This force reduction factor is called as Response reduction factor 'R'

From the pushover curves various parameters such as the ultimate and yield base shear and the corresponding ultimate and yield roof displacements are obtained. The final values of response reduction factor are obtained from the computed values of Over-strength and the ductility capacity of the structures obtained from the parameters described above. Tables 7 and 8 present the pushover parameters and the component-wise calculation of the 'R' factor respectively for the building frames subjected to the quadratic load pattern.

Pushover parameters for the frames subjected to quadratic load pattern(Table 7)

Frame	Δ_u	Δ_y	V_u	V_y	$\mu = \frac{\Delta_u}{\Delta_y}$	$\Omega = \frac{V_u}{V_y}$
3-storey	0.05	0.026	1736	1246	1.86	4.48
7-storey	0.246	0.13	2606	2094	1.9	4.8
11-storey	0.335	0.13	2044	1551	2.57	3.17

Components of the Response reduction factor for frames subjected to quadratic load pattern(Table 8)

Frame	R_s	R_μ	R_R	R_ξ	R
3-storey	4.48	1.6	1.0	1.0	7.17
7-storey	4.8	1.9	1.0	1.0	9.12
11-storey	3.17	2.57	1.0	1.0	8.15

The pushover parameters and the components of the response reduction factor for the building frames subjected to the inverted triangular load pattern are presented in Table 9and Table10 respectively.

Pushover parameters for the frames subjected to inverted triangular load pattern(Table 9)

Frame	Δ_u	Δ_y	V_u	V_y	$\mu = \frac{\Delta_u}{\Delta_y}$	$\Omega = \frac{V_u}{V_y}$
3-storey	0.05	0.026	1982	1428	1.6	5.12
7-storey	0.23	0.13	2907	2447	1.8	5.4
11-storey	0.413	0.14	2434	1926	2.8	3.7

Components of the Response reduction factor for frames subjected to inverted triangular load pattern(Table 10)

Frame	R_s	R_μ	R_R	R_ξ	R
3-storey	5.12	1.6	1.0	1.0	8.2
7-storey	5.4	1.8	1.0	1.0	9.72
11-storey	3.7	2.8	1.0	1.0	10.4

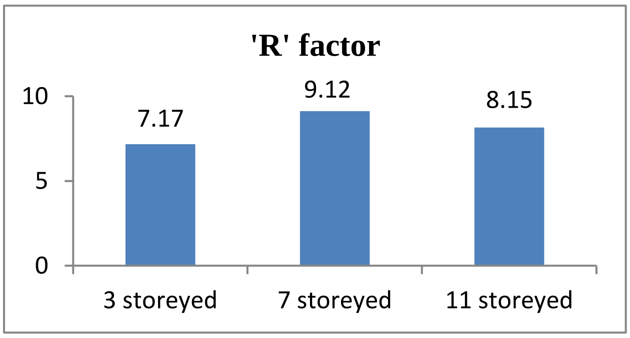
The target value of Response reduction factor '5.0' is not achieved by any of the three frames. In each case, the value exceeds or is lesser than the value '5.0'. The computed values of R are greater than '5.0'as seen in the tables 8 and 10. Higher value of 'R' means lower value of the design base shear and hence the designed building will not be heavy.

9. Effect of lateral load pattern on the seismic response.

The pushover curves on comparison show that the ultimate base shear values are on a higher side for the inverted triangular load pattern as compared to that for the quadratic load pattern. A 15% increase is observed in base shear capacity for the inverted triangular pattern. This means higher over-strength values for the frames subjected to inverted triangular load pattern and hence a higher value of 'R'. Thus, the FEMA-356 inverted triangular load pattern over estimates the value of R which leads to a potentially dangerous underestimation of the design base shear. For the two different load patterns, there is not much difference in the ultimate displacement.

10. Effect of number of storeys on the value of 'R'

The response reduction factor has been calculated for all the three frames. With the increase in height the quadratic load pattern does not show any specific trend in case of quadratic load pattern. In case of SMRF with ductile shear walls, the 'R' values obtained are in the range of 7 to 9 which is greater than the value provided in IS code for such frames. For the frames subjected to triangular loads the value of R is greater than 5 and show and increasing trend with increase in number of stories. Figures 11 and 12show the variation in the 'R' factor with increase in the number of storeys.



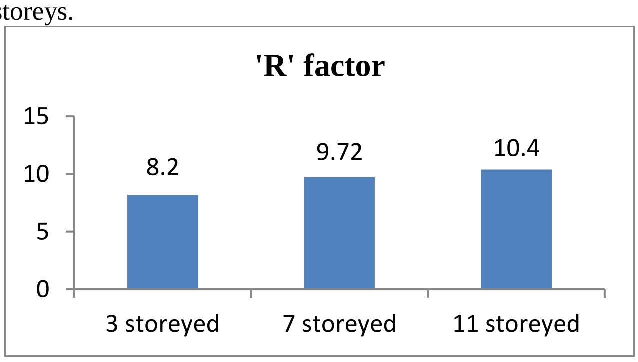


Fig. 11: Variation of 'R' factor for Quadratic load pattern

Fig. 12: Variation of 'R' factor for Inverted triangular load pattern

11. Variation in the strength factor.

The excess strength of the structure because of load factors in various design load combinations and reduced design strength of materials by factor of safety is taken into account by the strength factor. As such no variation is seen in the strength factor with the increase in the height of the building. However, it is seen that the shorter frames show higher over-strength values as compared to the taller frame. Figure 13 and 14 show the graphical representation of the variation of strength factor values with the number of storeys for all the three frames subjected to two different load patterns.

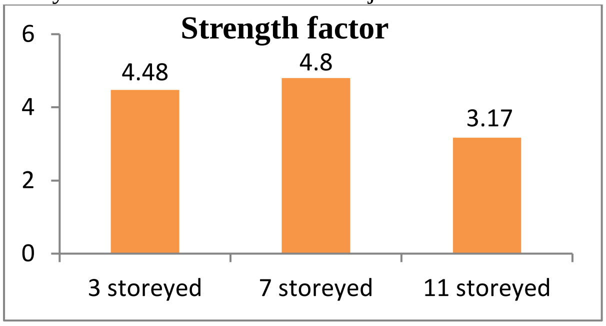


Fig. 13: Variation in the strength factor for Quadratic load pattern

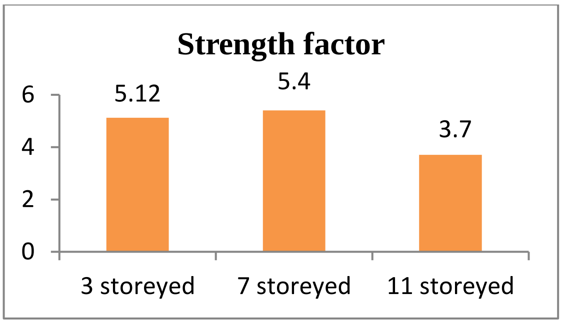


Fig. 14: Variation in the strength factor for Inverted triangular load pattern

12. Variation in the ductility factor

Figures15 and 16 show the variation in the ductility factor, R_μ with number of storeys for all the three frames subjected to both the quadratic and inverted triangular load pattern respectively. It is observed that the ductility factor for the 11-storey building exceeded the value for 7-storey building by 35.26% and that for 7-storey building it is exceeds by 18.75% when compared with 3-storey building. In case of buildings subjected to inverted triangular load it is observed that the ductility factor for the 11-atorey building exceeded that of 7-storey building by over 55.55% and that of the 7-storey exceeded the 3-storey by 12.5%.

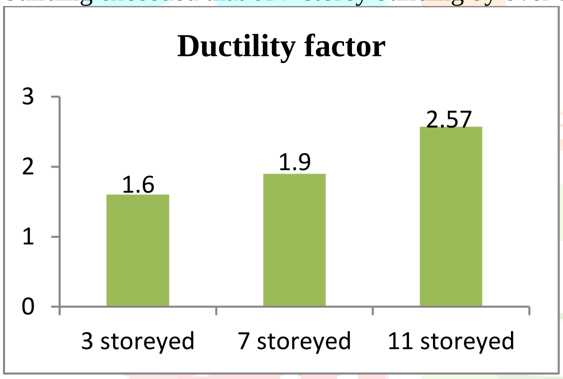


Fig.15: Variation in the ductility factor for Quadratic load pattern

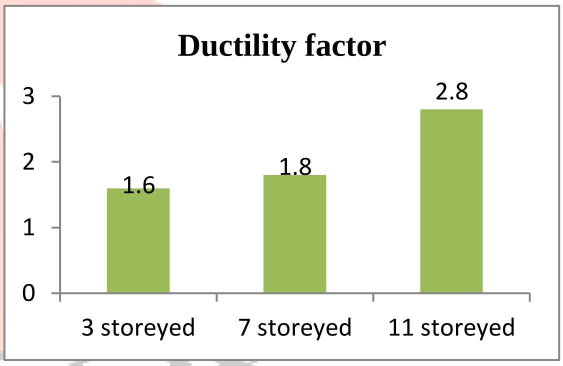


Fig.16: Variation in the ductility factor for Inverted triangular load pattern

13. Performance level of the structure

A performance check is obtained for the structure once the pushover curve is obtained. A performance check is helpful in determining whether if the structural and non-structural components are not damaged beyond the acceptable limits. A performance objective specifies the desired seismic performance of the building. It is a desired level of seismic performance of the building that is a limiting damage state within the building, the threat to life of the building's occupants due to the damage, and the post earthquake serviceability of the building; generally described by specifying the maximum allowable structural and non-structural damage, for a specified level of seismic hazard. Performance level is basically the post-earthquake damage state of the building.

As per the ATC-40, the lateral displacement at the performance point are to be checked against the displacement limits as specified in table 4.8.The damage occurred in a building is categorized into 4 performance levels namely immediate occupancy, damage control, life safety and structural stability. The maximum total drift of the structure is the governing factor of these parameters. Maximum total drift is defined as the inter story drift at the performance point displacement. . For Structural stability, the maximum total drift in story i at the performance point should not exceed the quantity $0.33V_i/P_i$, where V_i the total calculated shear force in story and P_i is the total gravity load (i.e. dead plus likely live load) at story i. For example, a structure in the Immediate occupancy performance level means less physical damage has occurred in the structure as compared to the structure in the Structural stability performance level. Higher the performance means less physical damage to the structure after an earthquake. The performance of all the three frames has been computed for the two lateral loads which have been acted upon them. For checking the ADRS of the buildings SAP2000 has been used and the effective time period of the structure has been obtained. . Hence the interstory drift of the frame is considered in order to check with the displacement limits corresponding to the effective time period and the performance level is thus obtained.

Figures show the ADRS spectra for the 11-storeyed frame subjected to both the quadratic and inverted triangular lateral load patterns respectively. The performance level of the structures are tabulated in Tables 11 and 12.

Performance levels of frames subjected to quadratic lateral load distribution(Table 11)

Frame	Teff (sec)	Interstorey drift (m)	Performance Level
3-storey	0.708	0.011	Damage control
7-storey	1.905	0.28	Structural stability
11-storey	3.231	0.19	Structural stability

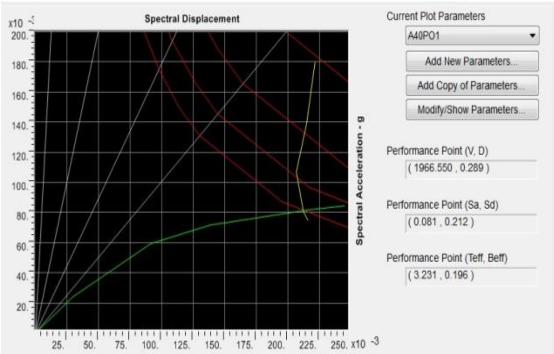


Fig. 17: ADRS spectra for frame subjected to . Quadratic load pattern

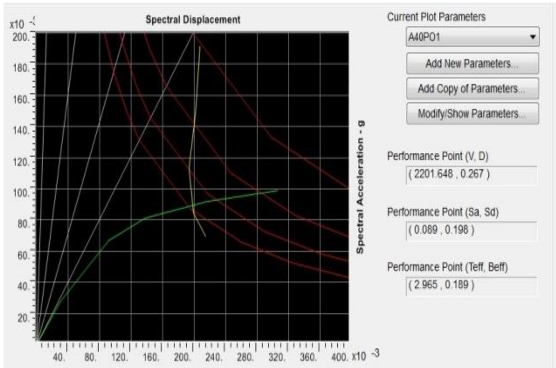


Fig. 18: ADRS spectra for frame subjected to inverted triangular load pattern.

Performance levels of frames subjected to inverted triangular load pattern(Table 12)

Frame	Teff (sec)	Interstorey drift (m)	Performance Level
3-storey	0.654	0.012	Damage control
7-storey	1.778	0.29	Structural stability
11-storey	2.965	0.20	Structural stability

14. Conclusion

1.The Indian design code does not specify any mathematical basis for the response reduction factor. A single value of R for all the buildings of a given frame type, irrespective of the plan and vertical geometry cannot be justified.
2.There is a significant difference on the values of response reduction specified in the codes and that obtained from analysis.
3.The computed value of R exceed approximately up to 70% than that specified in IS1893(PART1):2002 for the frames subjected to both the Quadratic and inverted triangular lateral load pattern.
4.The ultimate base shear capacity of the structures increased by 15 % when subjected to the inverted triangular load pattern as compared to the quadratic load pattern. There is not much difference in the ultimate displacement values for both the load patterns.
5.As compared to taller frames the shorter frames shows higher over strength values.
6.As the height of the structure increases the ductility factor also shows a increasing trend.
7.The actual value of R are subjected to be different in real life than what is computed here, because of various reasons like irregularity in dimensions leading to torsional effects, lack of quality control and poor workmanship during construction, not following the ductile tensile requirements as specified in the codes, etc.
8.An accurate estimation of the fundamental period is necessary for estimating a realistic R value for a structure.
9.The conditions of the present study are limited by the facts that only a single plan configuration (without asymmetry) in one single seismic zone has been considered. The different parameters used in the work presented are considered to be deterministic, although in reality their statistical variations are significant enough requiring reliability based framework for study.

References

1. Kent, D. and Park, R., ‘Flexural mechanics with confined concrete’, Struct Div. ASCE 1971; 97(ST7):1969-90.
2. Miranda,E. and Bertero, V. V. (1994), ‘Evaluation of strength reduction factors for earthquake-resistant design’, *EarthquakeSpectra*, 10(2), pp. 357-379.
3. ATC (1996), ATC - 40 : Seismic Evaluation and Retrofit of concrete buildings , Vol. 1 , Applied Technology Council , Redwood City , USA.
4. Priestley, M. (1997), ‘Displacement-based seismic assessment of reinforced concrete buildings,J. Earthq. Eng 1997;1(1):157-92.
5. Whittaker,A., Hart,G. and Rojahn,C. (1999), ‘Seismic Response Modification Factors’, *Journal of Structural Engineering*, 438-444.
6. BIS (2000), IS 456: Plain and Reinforced Concrete – Code of Practice, Bureau of Indian Standards, New Delhi, India.
7. Borzi,B. and Elnashai, A. (2000), ‘Refined Force reduction factors for seismic design’, Engineering Structures 22 (10), 1244-1260.
8. BIS (2002), IS 1893: Criteria for Earthquake Resistant Design of Structures,Part I, Bureau of Indian Standards, New Delhi, India.

9. Elnashai,A. and Mwafy,A. (2002), 'Overstrength and Force Reduction Factors of Multistorey Reinforced concrete Buildings', The Structural design of Tall buildings, 329-351.
10. BIS (2003), IS 13920: Ductile Detailing of Reinforced Concrete Structures subjected to seismic forces – Code of Practice, Bureau of Indian Standards, New Delhi, India.
11. Agarwal, P. and Shrikhande, M. (2010), 'Earthquake Resistant Design of Structures', PHI learning Pvt. Ltd., New Delhi.
12. Patel,B. and Shah,D. (2010), 'Formulation of Response Reduction Factor for RCC Framed Staging of Elevated Water tank using static pushover analysis, *Proceedings of the World Congress on Engineering*, Vol. III, London, U.K.
13. Mondal, A., Ghosh, S. and Reddy, G. (2013), 'Performance-based evaluation of the response reduction factor for ductile RC frames', *Engineering Structures* 56(2013), 1808-1819.
14. Tande,S. and Ambekar,R.V. (2013), 'An investigation of seismic response reduction factor for earthquake resistant design', *International Journal of Latest Trends in Engineering and Technology*, Vol.2, Issue 4 July 2013.
15. A.S.Khatavkar , A.P.Gadhi, P.R. Brabude (2015)'Component-wise Evaluation of Response Reduction Factor for Reinforced Concrete Frame', *International Journal of Computer applications*(0975-8887).