2017 Scienceline Publication

Journal of Civil Engineering and Urbanism

Volume 7, Issue 6: 87-94; Nov 25, 2017

ISSN-2252-0430

Evaluation of Seismic Response Modification Factor for Asymmetric Structures

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ABSTRACT: The research revealed that three major factors, such as reserved strength, ductility and structural redundancy affect the actual value of response modification factor (R). Those must be taken into consideration while determining appropriate 'R' for symmetric and asymmetric structures. The evaluation of 'R' is done by static-nonlinear analysis using ETABS. Also, ETABS is used to get the sequence and mechanism of plastic hinge formation. The procedure is validated by comparing results with Indian standard codal provisions for symmetrical structures and then those are evaluated for irregular structures. The 'R' calculated for symmetrical structure confirms evaluation procedure. Current Indian seismic design code never mentions about redundancy in structures. While irregularities in structural layout are punished, providing redundancy must be encouraged by the code. The values of 'R' for irregular structure varies. Hence a single value of R for all buildings of a given framing type, irrespective of plan and vertical geometry, cannot be justified.

Keywords: Response Modification Factor, Static Nonlinear Pushover Analysis, Regular And Irregular Structure, Plan Irregularity, Elevation Irregularity.

ORIGINAL ARTICLE
PII: S225204301700013-7
Received 10 Oct. 2017
Accepted 20 Nov. 2017

INTRODUCTION

Experiences during past seismic events have demonstrated that typical traditional methods of building design and construction lack in developing resistance to lateral forces in general and earthquake forces in particular. That's why the concept of earthquake resistant design came forward to enhance the behaviour of structure during lateral loads. The basic approach of earthquake resistant design should be to increase the lateral strength, deformability and ductility capacity of structure with limited damage but no collapse (Agarwal et al., 2010). This can be achieved by designing and detailing structure to get adequate toughness and ductility. This will lead to withstand the earthquake of any size and type, which is likely to experience during its life time (Applied Technology Council, 1995; Osteraas et al., 1990).

Moreover, response of symmetrical structures during earthquake events is far better than asymmetric structures (I.S. 13920-1993, 1993). It has well recognised that asymmetric buildings are especially vulnerable to earthquakes due to coupled lateral and torsional motions. The effect of such coupling and how well these effects are represented in seismic codes has been the subject of numerous investigations (Sunagar, 2012). The effects of coupling between lateral and torsional motions in the code designed systems were generally evaluated by comparing element ductility demands in asymmetric plan and the corresponding symmetric plan buildings. These investigations generally concluded that elements on the stiff side of in the code designed asymmetric plan buildings are likely to suffer more damage, whereas elements on the flexible side are expected to suffer less damage (Wood, 1991).

In present study, efforts have been made in estimating the actual value of R of reinforced concrete medium rise special moment resisting frame (SRMF) having irregularity (asymmetry) in elevation and as well as in plan by using non-linear static pushover analysis and compare it with codal values (Andrew et al., 1999; Balendra, 2003). The frame has been designed as per guidelines of IS 456:2000. The lateral loads acting on the frames were obtained from the guidelines of IS 1893: 2002 (part1). The non-linear pushover analysis is performed to calculate the R factor by evaluation of ductility reduction factor, over-strength factor and response modification factor (MagarPatil et al., 2015a;b).

MATERIALS AND METHODS

Computational model

In present study, finite element method based software ETABS-2015 has been used to model reinforced concrete ductile frames to evaluate R. Frame is designed as per provision of I.S 456:2000, I.S 875:1987, I.S:1893:2000 (Part 1) and I.S 13920:1993. The building frame is with 5 bays and 9 story with story height 3 m and bay width 4 m located in seismic zone V in India on hard rock soil type (I.S. 1893-2002, 2002; I.S. 456-2000, 2000). Nine models were selected (one regular building, five having irregularity in elevation & three having irregularity in plan) having different percentage of irregularity in elevation & as well as in the plan. General information regarding buildings & preliminary design consideration are tabulated in Table 1.

Table 1. Description of modelled building used for study

Sr. No.	Contents	Description
1	Type of structure	Multi Storied RC Rigid jointed Plane Frame(Special moment resisting frame) regular in shape
2	Number of story	G + 9
3	Floor Height	3 m
4	Materials	Concrete (M30); Steel Reinforcement (Fe415)
5	Size of Beams	300 X 500 mm
6	Size of Column	525 X 525 mm
7	Depth of slab	200 mm
8	Specific weight of infill	20 KN/m3 (150 mm width)
9	Specific weight of RCC	25 KN/m^3
10	Type of soil	Medium Soil
11	Impose load	3 KN/m^2
12	Importance factor	1
13	Seismic zone	V
14	Zone Factor	0.36

The study considers two types of frame models; Irregular buildings and regular building. To validate the software generated results of regular building frame, those are compared with that of regular building as given in IS 1893(Part 1)-2002. As per IS 1893 (Part 1) – 2002. A structure is defined to be irregular if the ratio of one of the quantities (such as mass, stiffness or strength) between adjacent stories exceeds a minimum prescribed value. However, in the recent version of IS 1893 (Part 1)-2002 (BIS, 2002), irregular configuration of buildings has been defined explicitly. Five types of vertical irregularity have been listed. They are: vertical geometric irregularity, re-entrant corner irregularity, stiffness irregularity (soft story), in-plane discontinuity in lateral-force-resisting vertical elements, and discontinuity in capacity. In this study we focus on first three irregularities.

A regular building is the one which possess four attributes like: simple and regular configuration, adequate lateral strength, stiffness and ductility. Buildings having simple regular geometry and uniformly distributed mass and stiffness in plan as well as in elevation, suffer much less damage than buildings with irregular configurations (FEMA, 1995; FEMA, 2004). All study structures have the same plan arrangement with 5 numbers of bays in both directions as shown in Figures 1, 2 and 3. The model frames on which the studies made may include: regular reinforced cement concrete building, vertically irregular building and re-entrant corner irregular building.

Modelling a building involves the modelling and assemblage of its various load-carrying elements. The model ideally represents the mass distribution, strength, stiffness and deformability. The beam-column joints are modelled by giving end-offsets to the frame elements, to obtain the bending moments and forces at the beam and column faces. The beam-column joints are assumed to be rigid (Lai et al., 1980; Lin et al., 2003).

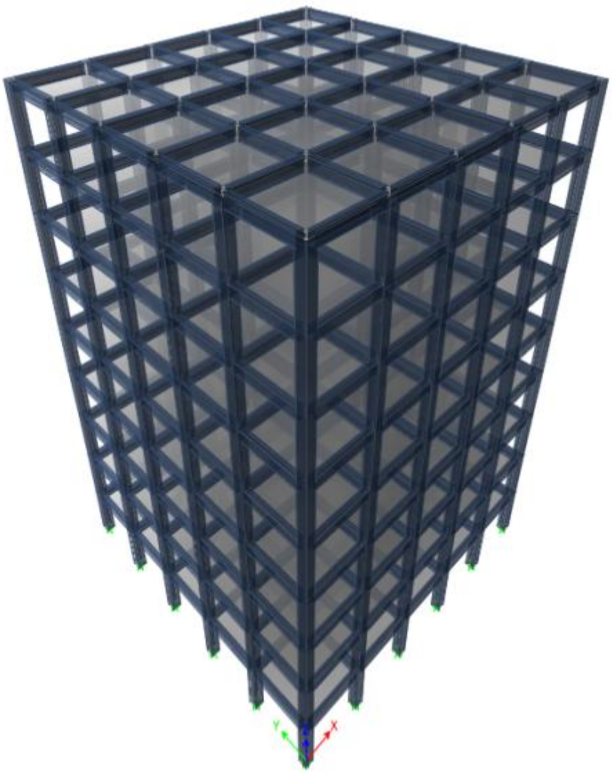

Figure 1. Three dimensional view of regular model in plan and elevation

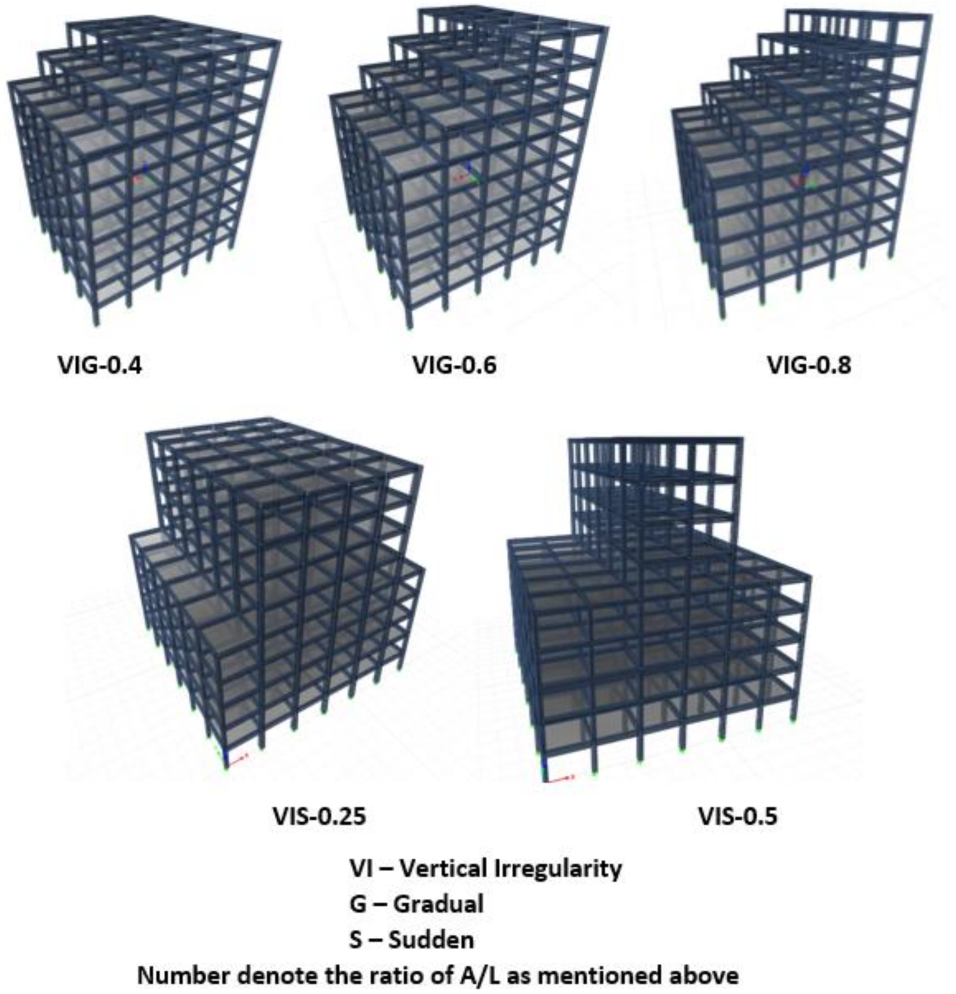

Figure 2. Three dimensional view of models having irregularity in elevation

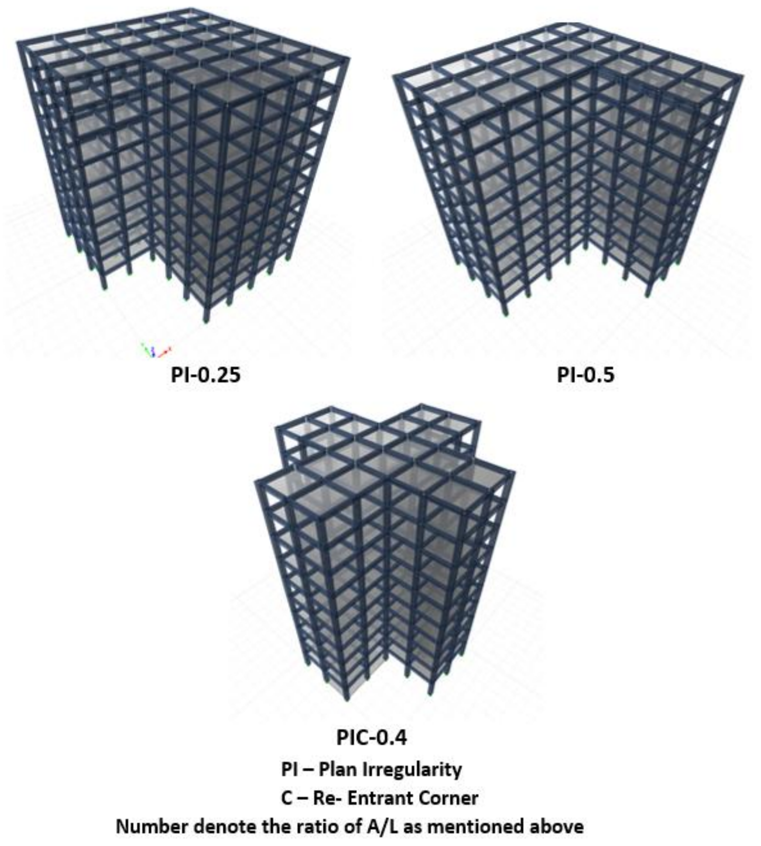

Figure 3. Three dimensional view of models having irregularity in plan

Pushover analysis

The concept of push over analysis can be utilized for estimating the dynamic needs imposed on a structure by earthquake ground motions and the probable locations of the failure zones in a building can be ascertained by observing the type of hinge formations. The strength capacity of the weak zones in the post-elastic range can then be increased by retrofitting.

Nonlinear static pushover analyses of the nine study frames are performed to estimate their over strength and global ductility capacity, which are required for computing 'R' for each frame. For this analysis nonlinear plastic hinges have been assigned to all of the primary elements. First moment hinges (M3-hinges) have been assigned to beam elements and then axial-moment 2-moment3 hinges (PMM-hinges) have been assigned to column elements. The floors have been assigned as rigid diaphragms by assigning diaphragm constraint.

In the study, the height from the base to centre of gravity of container is 27m and hence the target displacement is set to 108 mm. The displacement is applied step-by-step to the structure in an incremental manner. The base shear and roof displacement is recorded at every step. Due to plan symmetry of structure, the pushover analysis is carried out in X direction only. Hence, earthquake loads of tank full condition is given in X-direction only. The state of hinge formation using the push over curve is shown in Figure 4.

Thus from static pushover curve, maximum base shear is 6315.86 kN and yield drift is 54.57mm. The assigned hinges start yielding at a displacement value of 48.2mm. There is no indication of strength degradation at any displacement value within the range of target displacement. Even till step 10, mechanism is not formed. However, here the limiting target displacement which is 108 mm is achieved.

Similar results of maximum base shear and yield drift have been obtained from static pushover curve for remaining irregular models are shown in the Figure 5 to Figure 12.

Performance based analysis

The modern approach of performance-based engineering offers a rational design framework for making design decisions by assessing the appropriate risks and meeting various performance objectives of the engineered facilities that are subjected to natural hazards. Performance-based seismic design and assessment guidelines for new buildings and other structures have been proposed by several FEMA programs (FEMA-350; FEMA-P695; FEMA-P752).

With the scale and complexity of modern tall buildings, seismic performance based design requires extensive computational resources and effort. Performance based design optimization is the combination of state-of-the-art performance based engineering and a computational design optimization technique into an automated and synthesized design platform that aims to minimize the structural or life-cycle cost for buildings subjected to natural hazards such as severe earthquakes and extreme windstorms.

Maximum story displacement and maximum story drift obtained from the analysis are shown in Figure 13 and Figure 14, respectively, for regular building frame.

Performance based analysis is an analysis which includes modulation of formation of plastic hinges within the elastic limit and evaluation of the structure to be in the ductile state. The performance analysis shows the formation of hinges starts at the step 8 which falls in the range of immediate occupancy and life safety.

Here, the Figure 15 is the image which shows the displacement of the symmetric building at step 8 which shows that only 9 hinges are formed at this stage. The monitored displacement is recorded as 79.6 mm which is even less then the target displacement 108 mm. Also at this state the hinges formed are all in the immediate occupancy range which is safe.

Performance based analysis is a simplified, static-nonlinear analysis under a predefined pattern of permanent vertical loads and gradually increasing lateral loads. Typically, the first pushover load case is used to apply gravity load and then subsequent lateral pushover load cases are specified to start from the final conditions of the gravity pushover. Typically, a gravity load pushover is force controlled and lateral pushovers are displacement controlled. Load is applied incrementally to frame works until a collapse mechanism is reached. Thus it enables determination of collapse load and ductility capacity on a building frame.

The Pushover Analysis included 30 steps, which means that one subsequent push to building, hinges started forming in beams first. Initially hinges were in IO-LS stage and subsequently proceeding to LS-CP stage. Out of 1728 hinges only 9 hinges in IO-LS stage at step 8. Out of 1728 hinges 275 hinges are in IO-LS stage and 6 are in LS-CP stage at step 30. Overall performance of building is said to be IO-LS stage which shows it is safe (Mondal et al., 2013).

Analytical calculation of design base shear

The concept of push over analysis can be utilized for estimating the dynamic needs imposed on a structure by earthquake ground motions and the probable locations of the failure zones in a building can be ascertained by observing the type of hinge formations. The strength capacity of the weak zones in the post-elastic range can then be increased by retrofitting. Being the basic step of pushover analysis, the base shear is calculated as per the codal provisions and compared it with software values for regular RCC building. Further base shear for rest irregular buildings is obtained. The total design lateral force or design base shear along any principal direction shall be determined by the following equation 1, Clause 7.5 of IS 1893 (Part 1): 2002.

$$V_b = A_h \times W \qquad [1]$$

where, Ah = Design horizontal seismic coefficient for a structure

$$= \left[\frac{Z}{2} \times \frac{I}{R} \times \frac{S_a}{g}\right]$$

W = Seismic weight of building

Z = Zone factor

I = Importance factor

R = Response reduction factor

$\frac{Sa}{g}$ is the average response acceleration coefficient for rock and soil sites as given by IS

1893 (Part 1): 2002 which depends on fundamental natural period.

The results of design base shear calculated manually and by using software, are as shown in Table 2.

Response modification factor

Calculation of response modification factor. Calculation of response modification factor includes evaluation of three main factors which are over strength factor R_s, ductility factor R_μ and structural redundancy factor R_R. Thus response modification factor for regular building can be formulated as shown in equation 2.

$$R = R_s \times R_\mu \times R_R \qquad [2]$$

According to applied technology council (ATC) 19, structural redundancy factor is taken as 1, whereas other factors are calculated as follows.

Estimation of over strength factor (R_s). Using equation 3 for over strength factor given in ATC – 19 is calculated as shown below.

$$R_s = \frac{V_0}{V_d} \qquad [3]$$

Here, maximum base shear (V_o) is 6315.86 kN (From pushover curve shown in Figure 4) and design base shear, V_d = 2076.46 kN (from Table 2)

Hence, R_s = 3.04

Estimation of ductility factor (R_μ). The ductility factor is calculated by using equations 4, 5, 6 for ductility factor, derived by Miranda and Bertero (1994) (Miranda et al., 1994):

$$R_\mu = \left\{ \frac{(\mu - 1)}{\Phi} + 1 \right\} \qquad [4]$$

where, $\mu = \frac{\Delta m}{\Delta y} = 1.98$ [5]

$$\Phi = 1 + \frac{1}{12T - \mu T} - \frac{2}{5T} exp[-2(lnT - 0.2)^2] \quad \because \text{for alluvium site} \qquad [6]$$

Maximum drift capacity Δ_m = 108 mm (0.004 H)

Yield drift, Δ_y = 54.567 mm (from pushover curve shown in Figure 4)

T = 0.925 seconds (From ETABS model)

Φ = 0.737

$R\mu$ = 1.63

Hence, the value of response modification factor is evaluated as 4.96 for regular RCC building by equation 2. Now, calculation of response modification factor for vertically irregular building and building irregular in plan are shown in the Table 3. In this table, the values of the maximum base shear and maximum drift capacity obtained from the pushover curves displayed in the Figure 5 to Figure 12.

Table 2. Design base shear for regular building, vertical and plan irregular building by manual calculations and ETABS

Sr. No.	Name of Structure	Type of structure	Time period in sec	Design base shear V_d in kN (Manual calculation)	Design base shear V_d in kN(ETABS)
1	SYM	Symmetric	0.888	2070.52	2076.46
2	VIG-0.4	Vertical Irregularity	0.888	2173.15	2176.73
3	VIG-0.6	Vertical Irregularity	0.888	1998.11	1991.43
4	VIG-0.8	Vertical Irregularity	0.888	1754.25	1755.13
5	VIS-0.4	Vertical Irregularity	0.888	2804.53	2807.88
6	VIS-0.6	Vertical Irregularity	0.888	2513.54	2516.99
7	PI-0.25	Plan Irregularity	0.961	2819.46	2818.38
8	PI-0.5	Plan Irregularity	0.961	2292.04	2292.16
9	PIC-0.4	Plan Irregularity	0.961	1805.17	1804.56

Table 3. Evaluation of response modification factor R for all structures

Sr. No.	Name of Structure	Type of structure	Over strength factor (R_s)	Ductility factor ($R\mu$)	Redundancy factor (R_R)	Response Modification Factor (R)
1	SYM	Symmetric	3.04	1.63	1	4.96
2	VIG-0.4	Vertical Irregularity	2.03	2.28	1	4.62
3	VIG-0.6	Vertical Irregularity	2.21	2.10	1	4.64
4	VIG-0.8	Vertical Irregularity	2.31	2.05	1	4.74
5	VIS-0.4	Vertical Irregularity	2.08	1.98	1	4.12
6	VIS-0.6	Vertical Irregularity	1.97	1.89	1	3.72
7	PI-0.25	Plan Irregularity	2.13	2.09	1	4.45
8	PI-0.5	Plan Irregularity	2.19	2.18	1	4.77
9	PIC-0.4	Plan Irregularity	2.09	2.37	1	4.95

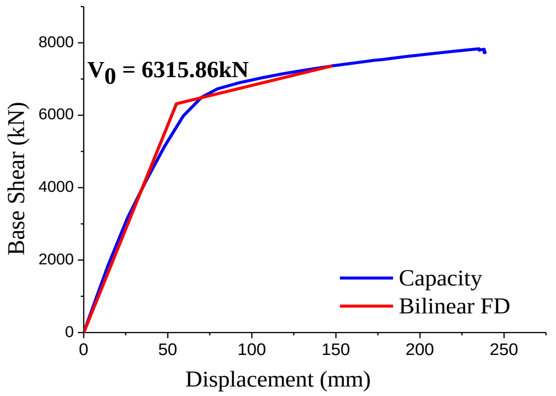

Figure 4. Pushover curve for modeled regular building

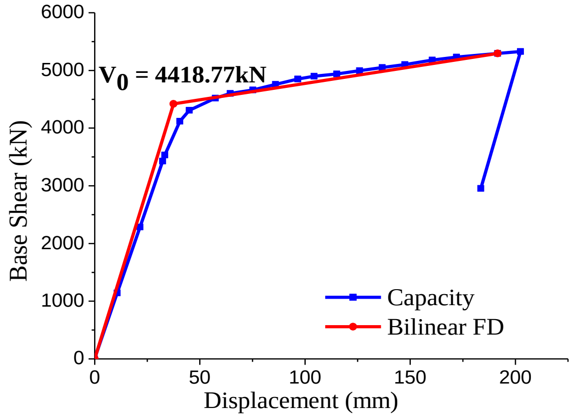

Figure 5. Pushover curve for irregular structure, VIG-0.4

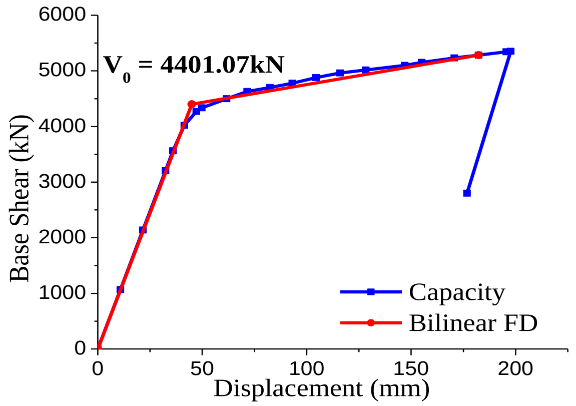

Figure 6. Pushover curve for irregular structure, VIG-0.6

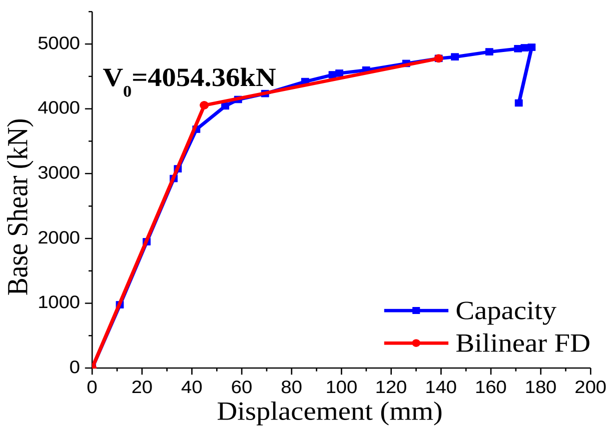

Figure 7. Pushover curve for irregular structure , VIG-0.8

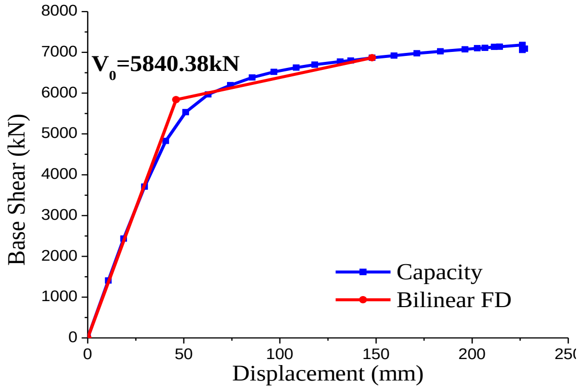

Figure 8. Pushover curve for irregular structure, VIS-0.4

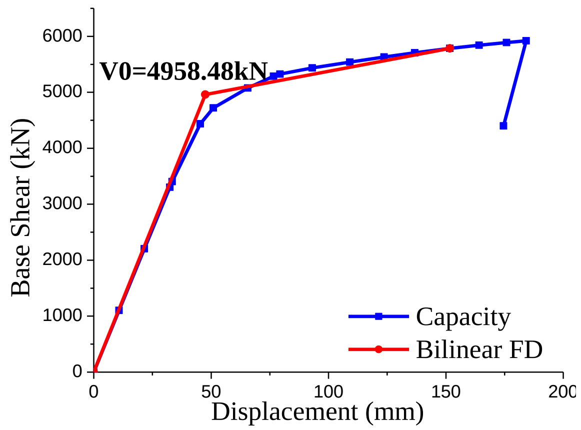

Figure 9. Pushover curve for irregular structure, VIS-0.6

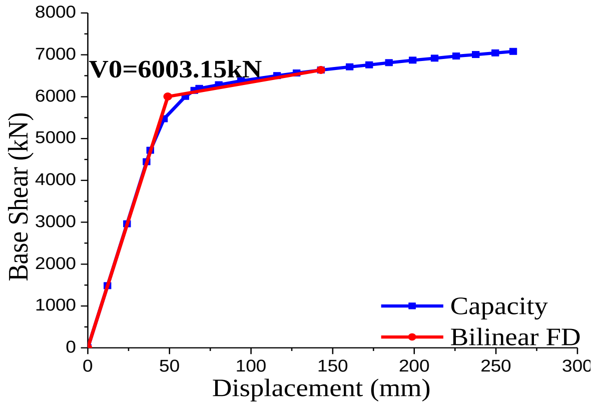

Figure 10. Pushover curve for irregular structure, PI-0.25

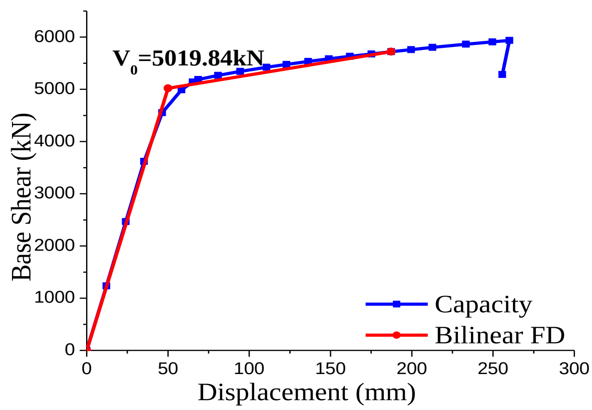

Figure 11. Pushover curve for irregular structure, PI-0.5

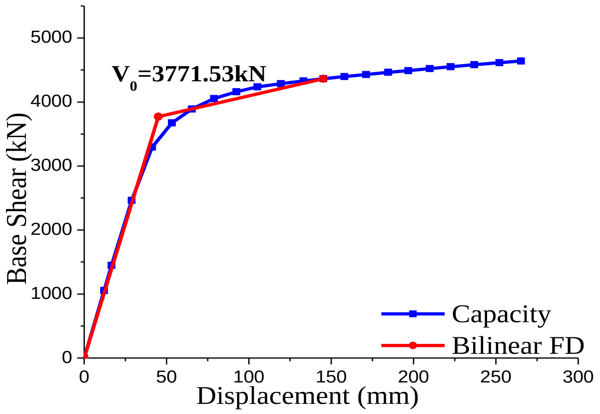

Figure 12. Pushover curve for irregular structure, PIC-0.4

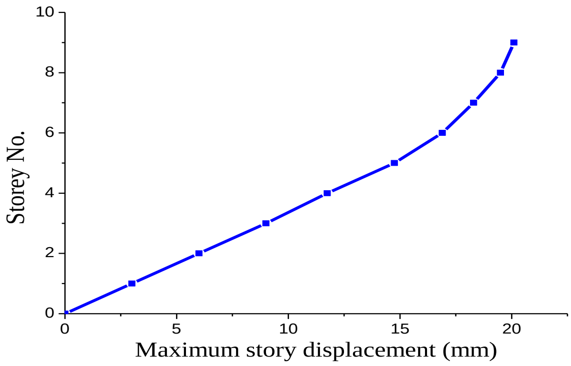

Figure. 13. Maximum story displacement for symmetric building

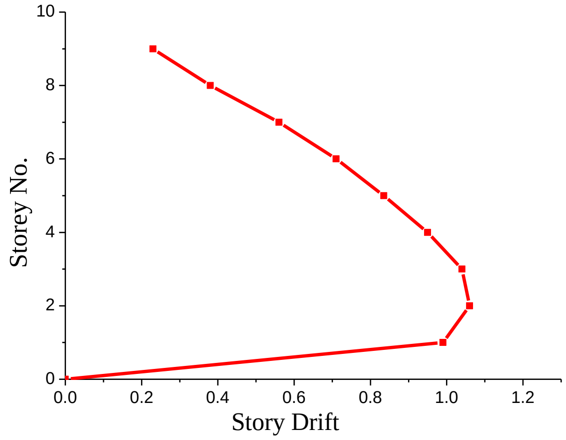

Figure 14. Maximum story drift for symmetric building

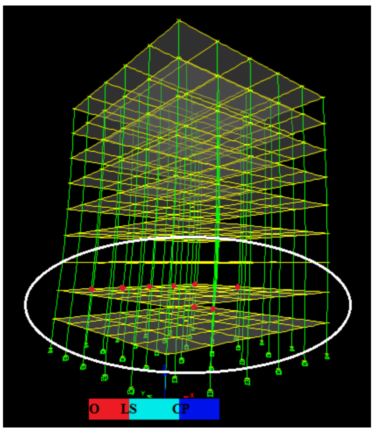

Figure 15. Displacement of symmetric building at step 8

CONCLUSION

Based on the results obtained from the pushover analysis carried out in ETABS, there developed a pushover curve which gave rise to the evaluation and calculation of many values. Using the significance of these values which include maximum base shear and maximum drift capacity, the response modification factor for regular building is evaluated and verified it with IS-1893 (Part 1) 2002.

On similar notes, evaluation of response modification factor for 5 vertically irregular buildings and 3 irregular buildings in plan was also possible, which can be concluded as:

1. The value of response modification factor for regular RCC building is evaluated as 4.96, which is verified with the Indian design codes which gives 5 for symmetric structures. This shows that the process and the calculations carried out to evaluate the response modification factor is correct which can be used further to evaluate the values of R for irregular buildings.
2. The values of ‘R’ for irregular structure are less than the response modification factor value for regular structure. Hence, a single value of R for all buildings of a given framing type, irrespective of plan and vertical geometry, cannot be justified.
3. The pushover analysis included 30 steps. Hinges started forming in beams. Initially hinges were in IO-LS stage and subsequently proceeding to LS-CP stage. Out of 1728 hinges only 9 hinges in IO-LS stage at step 8. Out of 1728 hinges 275 hinges are in IO-LS stage and 6 are in LS-CP stage at step 30. Overall performance of building is said to be IO-LS stage which shows it’s safe.

DECLARATIONS

Author’s contribution

Dr. H. R. MagarPatil has done complete analysis and drawn conclusions according to the results. He wrote the manuscript submitted to this journal.

Competing interests

The authors declare that they have no competing interests.

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