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SEISMIC DAMAGE ASSESSMENT OF LOW-RISE REINFORCED CONCRETE STRUCTURE WITH PERFORMANCE-BASED SEISMIC DESIGN APPROACH

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Abstract

Due to the failure of structures caused by earthquakes in India and elsewhere during the last 20 to 25 years and even more, attention is now being focused on earthquake unable to be affected constructions. The concept of performance-based seismic design provides a mechanism for determining the danger to life, disruption of residency, and financial loss that may have to face as a result of seismic occurrences. The purpose of this research is to assess the performance in seismic events of a R. C. structure subjected to lateral stresses. For this displacement-controlled pushover study, a gravity-based designed moment resisting frame positioned in high

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seismic area, and pushover curves were derived. The time and effort required to assess the performance of a moment resistant structure is connected to nonlinear modelling of R.C. members. Various characteristics, such as displacement from roof, base shear, and interstorey drift ratio, are acquired in order to estimate the various segments of the response minimization factor (R).

1. Introduction

To understand the behaviour and damage incurred during earthquakes, seismic design and estimate of reinforced concrete (RC) structures are necessary. The term earthquake is used to narrate any kind of seismic event which occurs naturally, or simulated by humans that generates seismic waves. Natural earthquakes are generally generated by the rupture of geological faults; however, they can induce by other incident such as shaking activities inside earth crust, mine explosions, nuclear testing and landslides. An earth tremor(earthquake) is defined as a sudden release of strain energy in the form of elastic waves on the earth's crust. (Murthy et al., 2012). This highlights the necessity for the development of dependable design and assessment techniques capable of quantifying the damage to both structural and non-structural parts, as well as minimising any loss of life. The work carried out takes a rational approach in determining the performance of a gravity-based design RC structure subjected to various lateral load patterns. For this purpose, displacement-based POA has been carried out using soft computing tool SAP 2000 structural software.

2. Problem Statement

Post-earthquake behavior of RC structures demonstrates the inadequacy of the ERD procedure described in present seismic codes to account for inelastic behavior and cyclic loading effects. This investigation aims to direct the important issues by estimating the execution of example MRFs developed for gravity loads. The strategy is to prepare nonlinear model of this structure by assigning plastic hinges to beams and columns. As suggested in literature plastic hinges were located at both ends of RC member as well as user-defined location. These frames were subjected to POA for various lateral load patterns.

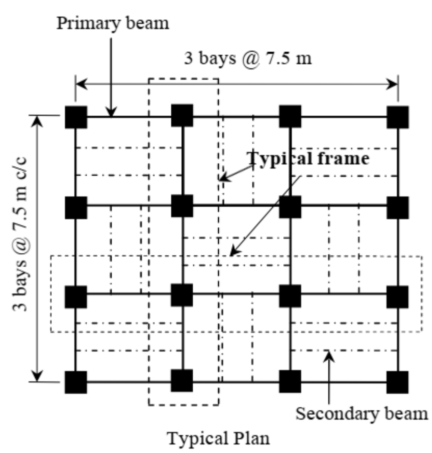
3. Example Moment Resisting Frame

The MRF in this example is a medium-rise $G + 5$ framed structure. This MRF was created in accordance with the requirements outlined in IS 456: 2000 (Rev), IS 1893: 2002 (Part 1), and IS 13920: 1996. Following Figure shows the configuration of Model MRF. This MRF represents a R. C. Structure located in the high-damage risk zone (IV), according to Indian Standard 1893, on a type of medium grade. The structures important factor (I) is considered to be 1.5 and response modification factor (R) equals to 5. The height of a ground floor storey is 4.1m and other floor heights are 5m, and the beam spans 7.5 m. There is 7.5 m space between the frames. For the analysis of selected MRF, Dead Weights, imposed loads, and earthquake forces all taken into account in accordance with IS 875 (Parts 1 and 2, 1987) and IS 1893 (2002), respectively. The example MRF is subjected to an average dead load (inclusive of finishes) (as described in Figure 1) and an imposed load of intensity 4 kN/m2 to particular floor, and 1.5 kN/m^2 for top slab.

Load combination $= DL + LL$

$U1(4.4 + 1.9)kN/m; U2 = (4.4 + 5)kN/m;$

$W1 = (42.2 + 14.3)kN;$



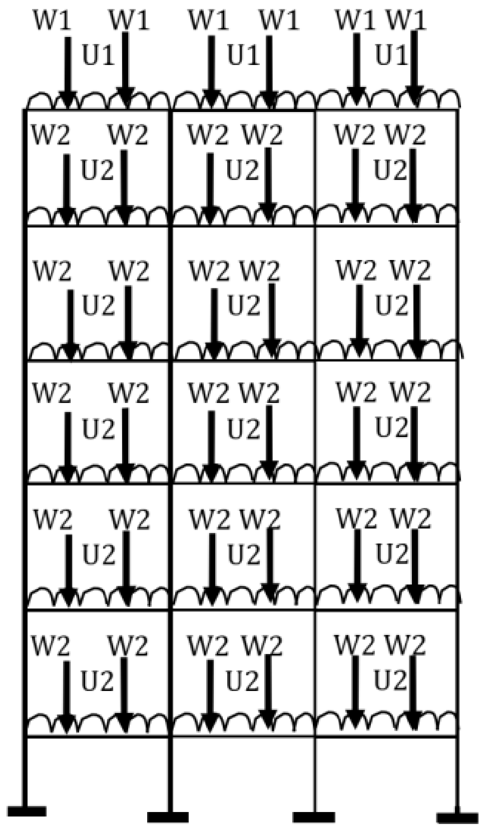


Figure 1. Plan and Elevation of MRF.

The sizes of the RC members were chosen in accordance with a commonly used procedure.

4. Nonlinear Modeling of Reinforced Concrete Members

Estimation of nonlinear response of the example MRF is the degree to which the nonlinear behaviour of the frame is reflected in the damage estimation has a substantial impact. This complete behavior of RC member is described in the terms of plastic hinges based on properties like moment-curvature, axisymmetric force-moment interaction, and shearing force deformation characteristics. The frame's nonlinear behaviour is principally determined by the actions of moment-rotation of its structural elements, which is determined by the moment-curvature properties of the flexible(plastic) hinge section and the length of plastic hinges. These two factors, in terms of plastic rotation capacity also specify the "component level" shortcomings (Mondal, 2013). The FEMA 356 requirements for modelling parameters and approval criteria were followed.

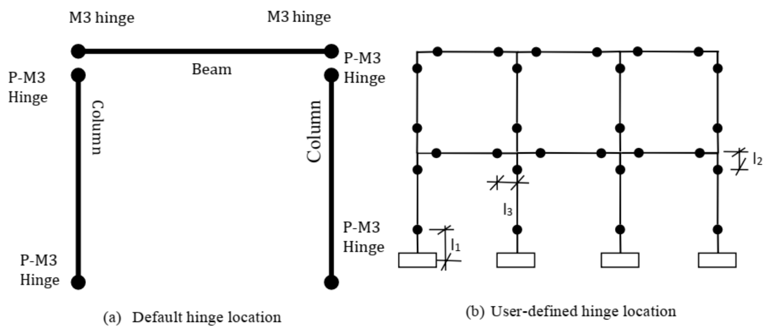


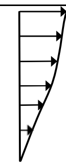
Figure 2. Location of plastic hinges.

Table 1. Plastic rotation limitations for flexure-controlled RC beams (FEMA 356, 2000).

Circumstance			Modeling criterion			Confirmation Standards				
			Flexible rotation angle (radians)		Persisting strength ratio	Angle of rotation of plastic (radians)				
						Stages of Performance				
						IO	Type of Component			
$\frac{\rho-\rho}{\rho_{bal}}$	Trans. Reinf.	$\frac{V}{b_w d\sqrt{f_c}}$					Primary			Secondary
			A	b	c		Lf. Sf.	Co. Pr	Lf. Sf.	Co. Pr
≤ 0	C meets the criteria for ductile detailing	≤ 3.0	0.025	0.05	0.2	0.010	0.02	0.02	0.02	0.05
≤ 0		≥ 6.0	0.020	0.04	0.2	0.005	0.01	0.02	0.02	0.04
≤ 0.5		≤ 3.0	0.020	0.03	0.2	0.005	0.01	0.02	0.02	0.03
≤ 0.5		≥ 6.0	0.015	0.02	0.2	0.005	0.005	0.01	0.01	0.02

P be the actual design axial load, V represents the design shear force, f_c known for concrete compressive strength, d_w stands for beam width, d represents effective depth of the flexural component, C represents validation of the plastic region, is the tension reinforcement ratio, ρ' is the compression reinforcement ratio, and ρ bal is representing the reinforcement proportion producing a balanced section.

Table 2. Modal assessment and horizontal force profile results for MRF.

Story level	Story Height (m)	Story weight (kN)	$W_i h_i^2$	$Q_i \frac{W_i h_i^2}{\Sigma W_i h_i^2}$	Obtained from SAP 2000	Loading diagram
Roof	29.1	580.23	491340.33	77.61	78.55	
5th floor	24.1	747.20	433981.23	68.55	69.38	
4th floor	19.1	747.20	272586.03	43.06	43.58	
3rd floor	14.1	747.20	148550.83	23.46	23.74	
2nd floor	9.1	747.20	61875.63	9.77	9.90	
1st floor	4.1	735.95	12371.32	1.95	1.98	
Total		4304.98	1420705.38	224.41	227.12	

The lateral loads were applied in a consistent manner by gradual increments in Stage 2. Because the lateral force profile in POA effects structural response, a set of lateral load situations was considered. One of these combinations included consistent weights, out of which top and bottom bound values of inertia forces are anticipated to be produced, while the second set included a primarily horizontal force configuration, as defined in the Standard Codes, or an elastic mode-1 pattern (Ghobarah, 2000). In this investigation, the following horizontal force combinations were most commonly used: IS 1893 horizontal force configuration (a) and Elastic First-Mode Lateral Load Pattern (b). For each load combination employed in this research, P- geometric nonlinearity effects were assessed.

5. Results and Discussion (Performance Assessment)

The measurements of strength and deformation limits are the estimations of loads and displacements estimated at the achieved deformation level. The crack development, yielding and failure succession and Capacity curves can be used to trace the history of structural deformation (Ghobarah, 2000). The fundamental goal of this research is to create a simple and effective performance indicator for quantifying structural damage utilising nonlinear responses acquired from pushover output. The assumptions were made in MRF's performance evaluation.

1. Different PBSD performance indicators are given in terms of structural and non-structural component damage sustained during a seismic event. The acceptance criteria for nonlinear behavior are defined in terms of plastic rotation and drift, which reflects the deformation capabilities of the structure. The collapse mechanism resulting from pushover analysis is used to obtain various nonlinear responses.

2. Attainment of an associated damage condition is represented by the moment at which the first element exceeds the permitted deformation parameter of a set performance level. The sequence of such yields is represented by the first hinge formation in particular performance levels. Figure 2 (a) and (b) represent plastic hinge patterns yielded at collapse for different push load case.

3. The emergence of the earliest (first) hinge inside the operational level (OP) is utilised to calculate the structure's initial stiffness (intact stiffness).

4. The highest value of a nonlinear reaction at the operational level(OP) is employed as a measure of value from yield.

5. The collapse state of the structure corresponds to the ultimate value of a nonlinear response.

6. The ATC 40 and FEMA 356 confirmation standards for plastic hinges is the shear or flexibility action have been utilised to characterise the nonlinear behaviour of structural components.

7. The frames used for analysis are uncovered (bare) frames (no failure is taken into account), moreover all the points are rigid(stiff).

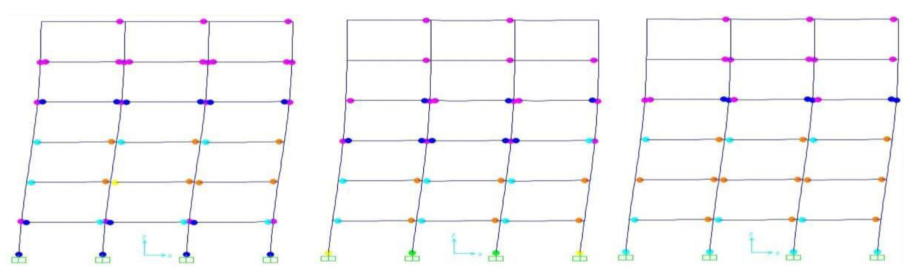


Figure 3. Hinge mechanisms at the collapse stage for different push load cases (Push 1, 2 and 3) employed in example MRFs.

Table 3. Collapse mechanism for example MRF subjected.

Step	Displ. (m)	Bs. Sh. (kN)	A - B	B - IO	IO- LS	LS - CP	CP - C	C -D	D - E	> E	To.	Remarks
1	0.012	16.35	48	36	0	0	0	0	0	0	84	Introductory hinge formed
13	0.140	186.32	48	36	0	0	0	0	0	0	84	Yield value
14	0.159	196.94	48	32	4	0	0	0	0	0	84	Introductory hinge in IO
31	0.346	233.53	44	21	18	1	0	0	0	0	84	Introductory hinge in LS-CP range
41	0.463	225.76	44	13	21	5	0	1	0	0	84	CP-C
67	0.586	162.43	36	17	13	10	0	1	7	0	84	Collapse

5.1. Formation of collapse zone

Table 04 shows the specified and determined values of drift values. The drift measured at the operational level is 0.53 (Max), which found to be less as compared to code needed value and contradictory. likewise, (trend)drift obtained at immediate occupancy(IO) is 0.58 (Max), life safety 1.19 (Max), collapse prevention 1.59 (Max) and 2.01 (Max) which are lower than prescribed value of relevant codes the explanation for this disparity might be related to constraints on plastic rotation in structural elements. The performance stages are calculated based on plastic rotations of (flexible) plastic hinges, and the fulfillment of plastic hinge limitation has been resulted in discrepancies between drift in the example model and codal specified values the explanation for this disparity might be related to constraints on plastic rotation in members.

Table 4. Drift values at various performance levels.

Performance levels	Default hinge (%)			User-defined hinge (%)			Prescribed drift limits (%)
	POA 1	POA 2	POA 3	POA 1	POA 2	POA 3	
OP	0.48	0.43	0.44	0.53	0.47	0.44	> 0.7
IO	0.55	0.45	0.51	0.58	0.53	0.50	1
LS	1.19	1.10	1.14	1.19	1.03	1.07	2

CP	1.59	1.40	1.48	1.49	1.31	1.36	4
C	2.01	1.67	1.83	1.80	1.54	1.69	< 4

The stiffness has a decline curve at separate performance stages that reach the (rigid) inelastic phase, as can be seen in figure. This graph will make it simpler to categorized different performance stages based on structural (resistant) strength. Table 04 demonstrates variance in structural stiffness.

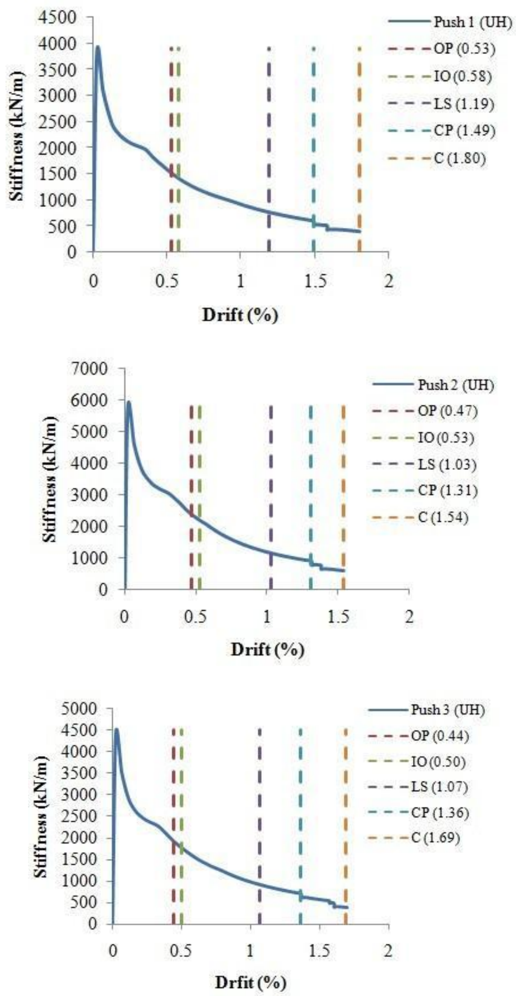


Figure 4. Pushover curve for example MRFs exposed to different horizontal forces combinations ([Push]POA 1, 2 and 3).

7. Conclusions

Using three distinct lateral load patterns, we investigated several PBSE approaches on gravity-based design structures. The following finding was made as a result of this research:

1. Push 2 load case show upper bound values for both cases, that is, location of plastic hinges at both ends and user defined location. Push 3 load case show lower bound values. Whereas Push 1 load case results to median value.

2. The failure technique of the frame in terms of flexible hinges demonstrates that in the Push 2 load scenario, failure has been focused in the bottom storey regardless of where the plastic hinges are located.

3. On comparing the stiffness, it was discovered that the case structure for Push 2 load condition was bears higher values compared to Push 1 and Push 3 load conditions. In push 2 case structure exhibits the brittle behavior, while in Push 1 and Push 3 remains in the elastic range.

4. The entire interpretation of storey (drift) displacements provides the results in which it is concluded Push(POA) 2 load case underestimates the storey displacement as building height increases.

5. The interstorey drift is consistent throughout the structure's height, but the observed distribution of the interstorey drift ratio along the height of building was non-uniform along the height. This may be attributed towards adopted lateral load pattern.

6. PBSE protocols have established numerous drift limitations to determine a structure's performance level. The drift obtained at operational level is 0.53 (Max) which is lower than the code prescribed value, non-consistent to each other. Similarly, drift obtained at immediate occupancy is 0.58 (Max), life safety 1.19 (Max), collapse prevention 1.59 (Max) and 2.01 (Max) which are lower than prescribed value of relevant codes the reason behind this inconsistency may be due to restrictions of plastic rotation in members. The performance levels are determined based on plastic rotations of plastic hinge, the fulfillment of restriction of plastic hinges has caused inconsistencies between the drift in the model and code prescribed values.

7. The primary concern of PBSD is to communicate risk to life, property and associated downtime to stakeholders, but in present state performance objectives are incapable to do so. One way to integrate the performance level and damage value is to associate them with some performance indicator.

8. Performance indicator defines in present study follows the degradation of stiffness of structure at various performance levels. When these values were associated with various drift measure results in the collapse zone. This collapse zone may help in the design optimization of RC sections.

9. The strength requirements for a ductile frame must be related to the maximum storey displacement ductility. In this research, we analyzed this relationship for different performance stages for a prescribed drift limit. The R values were computed at three performance levels, IO, LS, and CP. NLSP was used to acquire various characteristics such as roof displacements and base shear relevant to both the yield and ultimate states of a structure in order to determine the different components of R. The results show that the recommended value of R in IS 1893 is found more than the actual, which results in underestimate of design base shear. The study is limited by the fact that only a single plan configuration (without plan-symmetry) in only one single seismic zone has been considered. The overall interpretation of storey displacements yields the finding that the Push 2 load scenario underestimates storey displacement as building height increases.

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