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Seismic Vulnerability Assessment of Regular and Irregular RC Buildings using Pushover Analysis

Shaikh Ahrajfarhan Abdul Riyaz, PG Student, Department of Civil Engineering, MGM's College of Engineering, Nanded, Maharashtra, India. shaikhahrajfarhan0@gmail.com

Mohd. Zameeruddin, Associate Professor, Department of Civil Engineering, Nanded, Maharashtra, India. md_zameeruddin@mgmcen.ac.in

Abstract - With the increasing demand of urbanization in developing countries has raised the need for the construction of medium-rise and high-rise buildings. Safety of both structure and life during a seismic event has become a prime focus of structural engineers. Preliminary seismic risk assessment tools are often used to screen the new and existing structures against the potential seismic hazards. The geometric profile of a structure and the applied loading patterns influences the performance of structures during a seismic event. Among these vertical irregularities arise due to the irregular distributions in mass, stiffness and strength along the height of a structure has gained major importance. Soft story (Open floor) is a common irregular building configuration in practice in India and represents a significant source of serious seismic damage, when subjected to severe earthquakes. The Present study focuses on evaluation of performance of medium-rise building with soft storey using pushover analysis. From the results of pushover analysis a vulnerability index has been introduced to evaluate damage to the structure. An attempt is made to scale the damage value with the attainment of the performance level defined in performance-based seismic design codes. This vulnerability index can be used as a preliminary risk assessment tool.

Keywords — Structural Irregularities, Pushover analysis, Vulnerability index, Example MRFs, Seismic assessment

I. INTRODUCTION

Earthquake is one of the most unpredictable and devastating natural hazards. The force-based design methods allows to design the structures which are capable of sustaining minor damages during minor or moderate earthquakes, but they were get collapsed when subjected to severe earthquakes. This questions the adequacy of the available seismic codes to provide the safety to both life and structures. The structural engineers and associated stakeholder who are engaged in the earthquake resistant design has a prime concern towards the performance evaluation and damage assessment of reinforced concrete (RC) structures subjected to seismic hazards. The structural engineers and associated stakeholder who are engaged in the earthquake resistant design has a prime concern towards the performance evaluation and damage assessment of reinforced concrete (RC) structures subjected to seismic hazards [1].

During seismic event the RC structures are subjected to inelastic incursions, which demands effective analysis procedures to evaluate its multiple performance levels. Performance-based seismic design (PBSD) has put forth various building performance levels based on the damages sustained by structural and non-structural components. Figure 1 shows various building performance levels and ranges described in PBSD [2-6]. PBSD has also provided four analysis procedures; Viz. linear static and dynamic and nonlinear static and dynamic. Among these first two are forced-based. Three and four are displacement-based. The nonlinear methods have been used to evaluate performance of RC structures under seismic loads. Nonlinear dynamic method accounts for changes in structural parameters during cyclic loading, but involves complex procedures. While nonlinear static procedures are simple and their results are closer to that of dynamic procedures. PBSD document has proposed various performance evaluation procedures based on nonlinear static procedures (pushover analysis, POA), namely the capacity spectrum method and displacement coefficient method [7].

In capacity spectrum method, the structure is subjected to predefined lateral load pattern with monotonically incremental steps till a target displacement is reached. The response of structure is plotted for rooftop displacement and base shear known as capacity spectrum. The inelastic demand spectrum is obtained for the stated time period and damping coefficient. The intersection of this curve provides performance point which defines the level of seismic performance. Figure 2 describes the CSM procedure [1-3]. While in the displacement coefficient method (DCM) is the simplest method of obtaining target displacement. The method does not involve the conversion of capacity curve into corresponding spectral coordinates. The linearization

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of capacity curve is done to obtain performance point. Figure 3 illustrated DCM method.

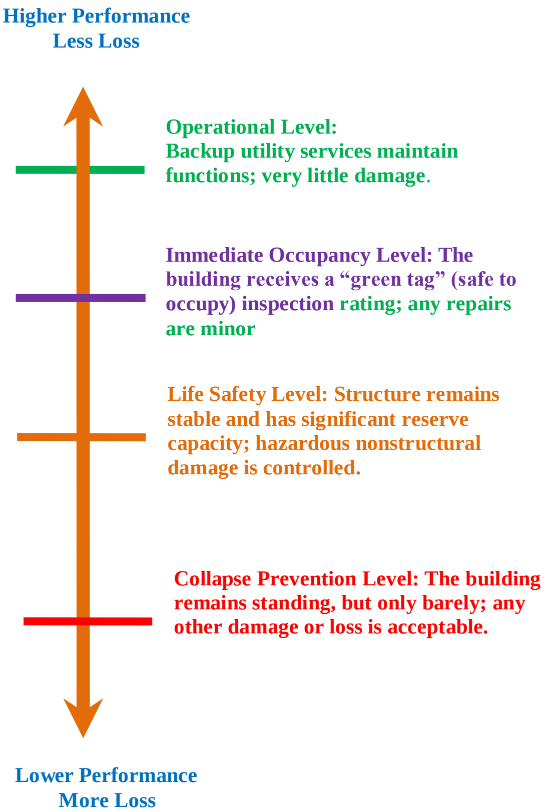

Fig. 1: Building performance levels and Ranges [2-4]

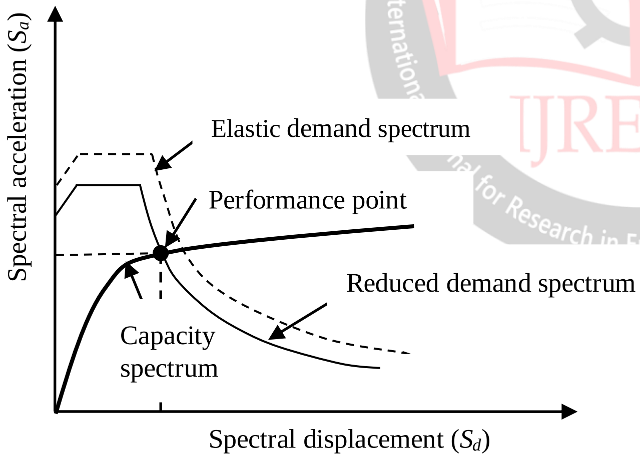

Fig. 2: Capacity Spectrum Method [6]

These procedures provide the information about nonlinear responses using the collapse mechanism and transfer of plastic hinges from on performance levels to another, but fail to provide any associated damage values. In this study, we had attempted to associate the building performance level at global and local level with a damage state using the vulnerability index. The vulnerability index is defined in term of the engineering demand parameters obtain from the results of POA. Such integration will provide a rational approach to designers to predict damage state level at the iterative stage of the design process.

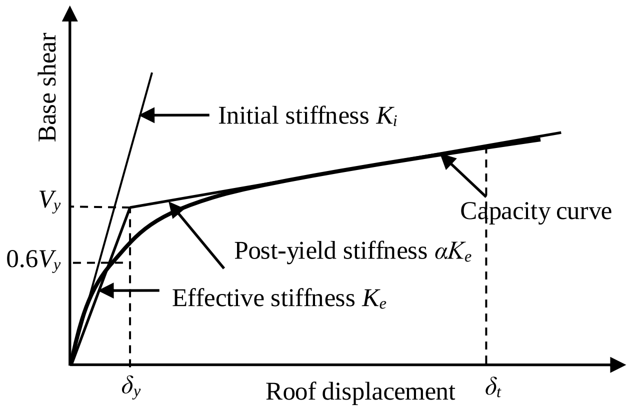

Fig. 3: Displacement Coefficient Method [6]

II. METHEDOLOGY

The development of PBSD framework shows that, nonlinear static procedures (POA) are a viable method to assess damage vulnerability of new or existing building [15]. In POA a series of incremental static analysis, which are carried on the structure to develop a capacity curve. The target displacement which is an estimate of the displacement for predefined seismic demands is estimated using the seismic capacity of the structure. The extent of damage experience of the structure at this target displacement is used to evaluate multiple performance of the structure as described in figure 1 [16].

POA has been in practice from last 50 years and more, with minor variation in computational procedures. Based on such variations, computing tools such as DRAIN 2D, IDARC, ETABS and SAP were developed and they found to be common in design practice. In the present study, POA has been performed on the 3-D reinforced concrete (RC) bare frames with soft storey. The example frame is located in zone V (severest zone as per IS 1893) and is subjected to the lateral load pattern described in the IS1893 [18]. The engineering demand parameter resulted in the output of POA were used to assess the collapse mechanism of the example frame. The assessment of the damage state correlates with the performance levels stated in PBSD by following the collapse mechanism represented by formation of plastic hinges.

The example 3-D RC frame typical plan and typical elevation are shown in figure 4. The planar geometry of the frame has a bay width of 3m and storey height of 3m, for all storey's except the bottom storey where irregularity has been introduced (soft storey). There are various possible configurations of introducing soft stories used in practice. In present study, we have considered soft storey located at first storey. The modeling parameter considered is stiffness of storey. Soft storey stiffness for first storey is kept as 70 percentage of adjacent storey. Table 1 describes the first

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storey height, associated soft storey equivalence and stiffness of all example frames used in study.

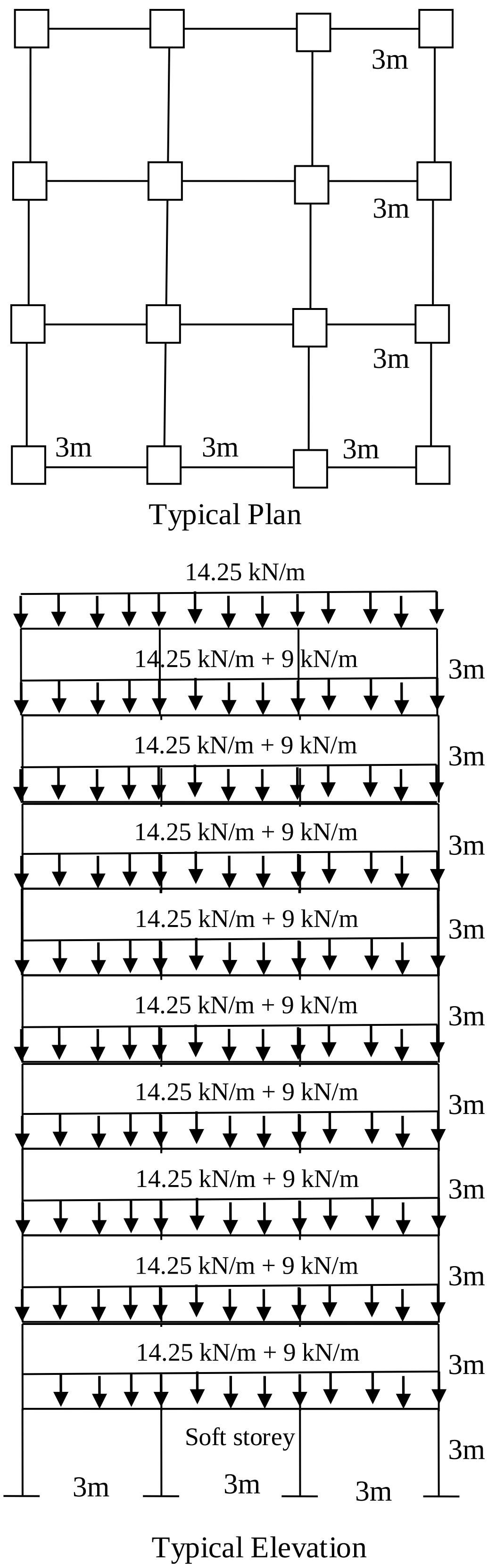

Fig. 4: Typical layout of example MRF

For gravity design, dead loads contributing from the slab, beams, columns and masonry infills (including finishes loads) measuring 14.25 kN/m were used. Live load of intensity 3 kN/m^2 was applied on the slabs. The seismic design loads combination includes100 percent dead load and 25 percent of live load contribution on a floor. The lateral loads applied on example frames are; (1) IS 1893-2002

Table 1: First storey height and soft storey ratio equivalence

Example MRFs	First storey height (m)	Soft-storey ratio equivalence	Soft-storey Stiffness (kN/m)
S10B3IR0	3.00	1.00	960.000
S10B3IR17	3.5	1.58	604.54000
S10B3IR33	4.00	2.37	405000
S10B3IR50	4.5	3.37	284.44000
S10B3IR67	5.00	4.63	207.36000
S10B3IR83	5.5	6.16	155.79000
S10B3IR100	6.00	8.00	120.000

Where IR represents percentage of irregularity

The designs of RC members are done as per the guidelines of IS 456 and IS 13920 [18-19]. Table 2 provides all parametric details of example frames used for analysis and design purpose.

Table 4: Assumed preliminary data required for analysis of frame

Sr. No	Particulars	Assumptions
1	Type of structure	Multi-storied rigid frame (moment resisting frame)
2	Seismic zone	V (table 2 I.S. 1893:2002)
3	No. of stories	Ten storied (G+9)
4	Floor height	3m
5	Tributary width	3m
6	Imposed load	3 kN/m^2
7	Materials	**Concrete:** a. Weight per unit volume 25 kN/m^3 b. Mass per unit volume 2.5485 Kg/m^3 c. Modulus of elasticity (E_c)= 5000$\sqrt{f_{ck}}$ = 25000 kNm d. Poisson ratio (µ) 0.20 e. Coefficient of thermal expansion (α) = 5.50 E-06 f. Shear modulus (G) 1041667 kN/m^2 g. Characteristic strength (f_{ck}) = 25000 kN/m^2 **Reinforcement:** a. Weight per unit volume 76.9729 kN/m^3 b. Mass per unit volume 7.849 Kg/m^3 c. Modulus of elasticity(E_s) = 2E+08 kNm d. Poisson ratio (µ) 0.30 e. Coefficient of thermal expansion (α) 1.17 E-05 f. Shear modulus (G) 76923077 kN/m^2 g. Yield strength (f_y) 41500 kN/m^2 h. Minimum tensile stress (f_u) 485000 kN/m^2 i. Expected yield strength (f_e) 456500 kN/m^2

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		j. Expected tensile stress (f_{ue}) 533500 kN/m²
8	Size of columns	(obtained from gravity analysis) Floors Size of columns Main bars(Tor) Shear bars (Tor) 01-03 600 mm x 600 mm 6No-20 mm 8 mm@ 150 mm c/c 04-06 530 mm x 530 mm 8No-20 mm 8mm@ 150 mm c/c 07-10 450 mm x 450 mm 6No-20 mm 8mm@ 150 mm c/c
9	Size of beams	Both longitudinal and lateral (obtained from gravity analysis) Floors Size of Beams Top bars(Tor) Bottom bars (Tor) Shear bars (Tor) 01-10 300 mm x 530 mm 705 mm² 625 mm² 8mm@ 110 mm c/c
10	Depth of slab	150 mm thick
12	Type of soil	Soft soil
13	Response spectra	As per I.S. 1893:2002(part1) compatible for 5 % damping

The example MRFs fundamental period of vibrations does not exceeds 1.0 seconds, so as to ensure that the first mode contribution dominates. Table 3 provides the modal analysis details of example MRFs. The adopted limits for global response parameters are storey drift not exceeding 4 percent of the total height and inter-storey drift not exceeding 2 percent [2-5].

Table 3: Modal analysis results of example MRFs

Example MRFs	Natural Time Period (Sec)	Seismic Weight (kN)
S10B3IR0	0.961	1385.55
S10B3IR17	0.976	4812.44
S10B3IR33	0.985	4959.39
S10B3IR50	0.997	4156.33
S10B3IR67	1.00	4704.68
S10B3IR83	1.020	4721.4
S10B3IR100	1.032	5540.85

Nonlinear modellings of RC members were done by assigning hinges of type P-M2-M3 for the columns and M3 for beams at both ends. The default properties for moment-curvature and stress-strain distribution defined in software ETABS V17 [14] are used. P-Δ effects are considered. The lateral loads applied include; (a) IS 1893 trivial load pattern –Push 1, (b) uniformly distributed load pattern - Push 2 and (c) Fist-mode lateral load pattern –Push 3. Table 4, provides the lateral load distribution for different push load case applied on S10B3IR0 MRF. POA performed on example MRFs includes two step procedures. In first stage the example frame is subjected to gravity loads and in second stage different lateral loads are applied.

Table 4: Modal analysis results of S10B3IR0 MRF

Storey Height (m)	IS 1893 Lateral Loads (kN)	Uniform Lateral Loads (kN)	First Mode Lateral Loads (kN)
3	0.422	1.43	0.30849
6	1.68	5.30	1.667
9	3.80	11.8	4.36
12	6.75	20.7	8.469
15	10.55	32.4	13.99
18	15.14	46.3	20.151
21	20.71	59.8	26.52
24	27.02	71.0	33.277
27	34.20	89.8	38.158
30	42.22	91.7	42.65

III. Results and Discussions

From POA analysis, the capacity curve of example MRFS are plotted in terms of base shear and rooftop displacement for every incremental increase of applied lateral loads. The capacity spectrum resulting from POA is compared with inelastic demand spectrum to obtain the performance point. Figure 4, shows the pushover curves obtained for the entire example MRFs. Table 5-6, provides the values of base shear and displacement at performance point for different PBSD Procedures applied on example MRFs.

Table 5: Base shear and displacement at performance point for CSM PBSD Procedures applied on example MRFS

Example MRFs	ASCE 41 (CSM)		FEMA 440(CSM)	
	V_p (kN)	d_p (m)	V_p (kN)	d_p (m)
S10B3IR0	1596.04	0.237	1714.44	0.301
S10B3IR17	2647.16	0.239	3057.34	0.239
S10B3IR33	2193.44	0.222	2235.95	0.255
S10B3IR50	1765.73	0.293	1788.67	0.337
S10B3IR67	1458.31	0.321	1507.64	0.412
S10B3IR83	1458.31	0.321	1507.64	0.412
S10B3IR100	1926.89	0.267	1971.37	0.309

Table 6: Base shear and displacement at performance point for CSM PBSD Procedures applied on example MRFS

E xample MRFs	ASCE 41 (DCM)		FEMA 440(DCM)	
	V_p (kN)	d_p (m)	V_p (kN)	d_p (m)
S10B3IR0	2099.17	0.183	2253.64	0.235
S10B3IR17	2647.16	0.239	1122.42	0.317
S10B3IR33	2849.41	0.253	3468.09	0.406
S10B3IR50	1766.51	0.253	865.88	0.374
S10B3IR67	775.64	0.328	781.85	0.382
S10B3IR83	12332.7	0.129	12252.8	0.028
S10B3IR100	12332.	0.129	955.26	0.353

The collapse mechanism resulting from pushover analysis shows the formation of plastic hinges. With each incremental step of lateral loading there is transformation of plastic hinge from one performance level to other

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performance level. This describes the state of damages to structural components before complete failure. Figure 5 shows the plastic hinge mechanism of all example frames at collapse state. The collapse mechanism is capable to illustrate the onset of collapse stage, but do not provide any damage value.

With the intention to correlate the collapse mechanism with the associated damage state of the example MRFs a vulnerability index has been put forth. The proposed formulation is a possible extension of the vulnerability index defined by N. Lakshman [21].

The vulnerability index of a building defined by N. Lakshman is;

$$VI_{bldg} = \frac{1.5\sum N_i^c x_i + \sum N_i^h x_i}{\sum N_i^c + \sum N_i^h}$$

Where, N_i^C is number of hinges in columns and N_i^h is number of hinges in columns and beams at "i[th]" performance levels. A weight-age factor x_i has been used to consider the changes in nonlinear characteristics of RC sections falling throughout the inelastic incursion. Table 7 provides the details of weight-age factor x_i used in calculation. An importance factor of 1.5 has been assigned to columns for ensuring safety during global performance.

Table 7: weight-age factor x_i for various performance levels [20]

El	weight-age factor x_i	Drift Limits (%)
Operational (A-B)	0	< 0.2
Immediate Occupancy (B-IO)	0.125	< 0.5
Life safety Range (IO-LS)	0.375	< 1.5
Collapse Prevention (LS-CP)	0.625	< 2.5
Near to Collapse (CP-C)	0.875	< 3.5
Collapse (C-D; D-E and >E)	1	4

To trace the effect at storey level the vulnerability index defined by N. Lakshman [21] is;

$$VI_{storey} = \frac{\sum N_i^c x_i}{\sum N_i^c}$$

Where, N_i^C is number of hinges in columns and x_i is weight-age factor x_i.

In present study we performed POA on example MRFs with soft storey effects (changes in stiffness). The effects of change in storey stiffness have been observed at global levels (overall building and soft storey levels). In POA incremental lateral loads are applied against the target displacements. The attainment of performance levels is traced with reference to permissible drift limits. The permissible limits for drift at various performance levels defined in FEMA 440 are illustrated in Table 7.

In order to associate the vulnerability index with respective performance level, the fall of hinge from one performance level to other is sequenced and categorization of vulnerability index is done in various heads as: Performance Indicator Level 1 (PL1) and Performance Indicator Level 2 (PL2). In PL1 counts for plastic hinges appearing at operational level, immediate occupancy level and life safety range (one level before the appearance of plastic hinge in collapse prevention range) and PL2 accounts for plastic hinges count after attainment of collapse prevention range. Evaluation PL1 and PL2 helps in identification of zones where losses in terms of strength drift and stiffness can be easily traced. The modified formulations are;

(a) Vulnerability index for overall structure in PL1 range;

$$VI_{bldg,PL1} = \frac{1.5\sum(N_{OP}^c x_{OP} + N_{IO}^c x_{IO} + N_{LS}^C x_{LS}) + \sum(N_{OP}^c x_{OP} + N_{IO}^c x_{IO} + N_{LS}^C x_{LS})}{\sum N_{PL1}^c + \sum N_{PL2}^h}$$

Where, N_i^C is number of hinges in columns and N_i^h is number of hinges in columns and beams at OP, IO and LS performance levels and x_i is weight-age factor x_i.

(b) Vulnerability index for overall structure in PL2 range;

$$VI_{bldg,PL2} = \frac{1.5\sum(N_{CP}^c x_{CP} + N_C^c x_C + N_D^C x_D) + \sum(N_{CP}^c x_{CP} + N_C^c x_C + N_D^C x_D)}{\sum N_{PL1}^c + \sum N_{PL2}^h}$$

Where, N_i^C is number of hinges in columns and N_i^h is number of hinges in columns and beams at CP, C, D. and >E performance levels and x_i is weight-age factor x_i.

(c) Vulnerability index for soft storey level in PL1 range;

$$VI_{storey,PL1} = \frac{\sum(N_{CP}^c x_{CP} + N_C^c x_C + N_D^C x_D)}{\sum N_i^c}$$

Where, N_i^C is number of hinges in columns and N_i^h is number of hinges in columns and beams at OP, IO and LS performance levels and x_i is weight-age factor x_i.

(d) Vulnerability index for soft storey level in PL2 range;

$$VI_{storey,PL2} = \frac{\sum(N_{CP}^c x_{CP} + N_C^c x_C + N_D^C x_D)}{\sum N_i^c}$$

Where, N_i^C is number of hinges in columns and N_i^h is number of hinges in columns and beams at CP, C, D. and >E performance levels and x_i is weight-age factor x_i.

The collapse mechanism resulted from POA performed on example MRFs are shown in Table 8-9. Table 10 provides the vulnerability index values for building and at storey levels for PL1 and PL2. Modified formulation of vulnerability index provides the margins for improvement both within the collapse prevention range and the state of collapse.

III. CONCLUSION

Parametric studies have been carried out to understand the behavior of medium rise and high rise buildings. A

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vulnerability index has been used to identify the performance of a structure subjected to seismic hazard. The engineering demand parameters used to evaluate the damage of example structures were obtained from the POA performed on regular and irregular configurations of structures. The irregularity introduced related to stiffness irregularity wherein bottom storey columns height were varied.

Table 8: Collapse mechanism form POA (Columns)

Example MRFs	Total Hinge Count	OP Hinge Count	IO Hinge Count	LS Hinge Count	CP Hinge Count	C/D/E Hinge Count
S10B3IR0	666	490	166	2	7	01
S10B3IR17	836	620	191	6	15	4
S10B3IR33	876	620	227	13	14	2
S10B3IR50	825	620	188	4	5	8
S10B3IR67	971	620	171	171	9	0
S10B3IR83	767	620	137	8	2	0
S10B3IR100	825	620	191	8	2	4

Table 9: Collapse mechanism form POA (Beams)

Example MRFs	Total Hinge Count	OP Hinge Count	IO Hinge Count	LS Hinge Count	CP Hinge Count	C/D/E Hinge Count
S10B3IR0	702	490	202	2	7	01
S10B3IR17	865	620	220	6	15	04
S10B3IR33	1026	620	377	13	14	02
S10B3IR50	834	620	197	4	5	8
S10B3IR67	980	620	180	171	9	0
S10B3IR83	823	620	192	8	2	1
S10B3IR100	833	620	193	8	8	4

Table 10: Collapse mechanism form POA (Columns)

Example MRFs	VI_{bldg}	VI_{Storey}
S10B3IR0	0.058	0.038
S10B3IR17	0.64	0.042
S10B3IR33	0.072	0.048
S10B3IR50	0.05163	0.0344
S10B3IR67	0.046	0.030
S10B3IR83	0.040	0.027
S10B3IR100	0.051	0.034

In PBSD building performance levels are defined as; Operational (OP) level, Immediate Occupancy level (IO), Life safety Range (LS), Cpllapse Prevention (CP) and Collapse (C). All there performance levels are limited based on the drift limits related to local and global state of damages. We have categorized these performance levels in two performance indicatory ranges; (i) PL1 and (ii) PL2. In PL1 inelastic incursion of OP, IO and LS are accounted. Whereas; in PL2 inelastic incursion at CP, C and other higher levels [21]. The plastic hinge mechanism resulting in POA shows the inelastic drift attainment at identified locations. The number of hinges formation in particular performance range is used to define the damage state using vulnerability index proposed in the study.This proposed methodology was found to be a rational approach of scaling damage of structure which was not clearly defined by PBSD documents. The proposed methodology if adopted will be a quick tool to evaluate damage state in concern to predefined structural and modeling parameters.

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