and Eurocode 8 for the assessment of Indian code conforming buildings via nonlinear static analysis. The first part of the paper presents the modeling issues. The models must consider the nonlinear behaviour of structure/elements. Such a model requires the determination of the nonlinear properties of each component in the structure that are quantified by strength and deformation capacities. The deformation capacity of RC components, are modeled in the form of plastic hinges using FEMA 356, ATC 40 and Eurocode 8 and analysis procedure is based on [11,14-15]. The ultimate deformation capacity of a component is assumed to depend on the ultimate rotation and plastic hinge length. Several empirical expressions for plastic hinge length has been proposed in the literature, some of them are adopted and implemented in SAP2000 for the analysis. Five different empirical expressions are considered for the estimation of plastic hinge length and incorportaed the same in the analysis. In the present study, user defined plastic hinge properties of beams and columns are modeled using analytical expressions developed based on Eurocode 8 and incorporated the same in analysis. The analysis is carried out for load patterns proportional to fundamental mode. The building performances are assessed with the capacity curve generated in each case. Performance levels are used to describe the limiting damage condition, which may be considered satisfactory for a building under specific earthquake. The performance levels are expressed in terms of target displacement, defined by limiting values of roof drift, as well as deformation of structural elements. The three performance levels considered in the present study are immediate occupancy, life safety and collapse prevention. The vulnerability index, which is a measure of damage is estimated for the two designed cases, each case has been modeled for five different expressions of plastic hinges. The vulnerability index, defined as a scaled linear combination (weighted average) of performance measures of the hinges in the components, is calculated from the performance levels of the components at the performance point or at the point of termination of the nonlinear static analysis.

2 Description of Structures

Two framed structures are considered to represent low- and medium- rise RC buildings for the study. These consists of two typical beam-column RC frame buildings with no shear walls, located in high and medium seismicity regions of India. 4- and 6-storey buildings are designed according to the code (IS:456 and IS:1893), considering both gravity and seismic loads design ground acceleration of 0.36g and 0.16g with medium soil are assumed. Both the buildings are designed for two cases, such as ordinary moment resisting frame (OMRF) and special moment resisting frame (SMRF). Material properties are assumed to be 25MPa for the concrete compressive strength and 415MPa for the yield strength of longitudinal and transverse reinforcements. The OMRF buildings are designed with transverse reinforcement spacing of 250mm and SMRF buildings are with 100mm. The column and beam dimensions and the details of arrangement of longitudinal reinforcement are shown in Fig.1.

3 Building Performance Levels

The performance levels are discrete damage states identified from a continuous spectrum of possible damage states. A building performance level is a combination of the performance levels of the structure and non-structural components. The desired-structural performance levels to be found are Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP). These levels are based on the condition of the building under gradually increased lateral loads. Three levels in a base shear versus roof displacement curve for a building with adequate ductility is discussed in the following sections. Similar to the structural performance levels, the member performance levels are discrete, damage states in the load versus deformation behaviour of each member, as shown in Fig.2. For the beams and columns of a lateral load resisting frame, the following curves relating the loads and deformations are necessary.

1. Moment versus rotation
2. Shear force versus shear deformation

For a column, the moment versus rotation curve is calculated in presence of the axial load. In a nonlinear analysis [20], for each member, the respective curve is assigned at the location where the deformation is expected to be largest. In the case of existing RC buildings with low concrete strength and an insufficient amount of transverse steel, the shear failure of members need to be considered, which is irrelevant in the present study. For RC members, the moment versus rotation curves are calculated based on conventional analysis of sections [10].

4 Performance Based Objective

The objective of a performance based approach is to target a building performance level under a specified earthquake level. The selection of the levels

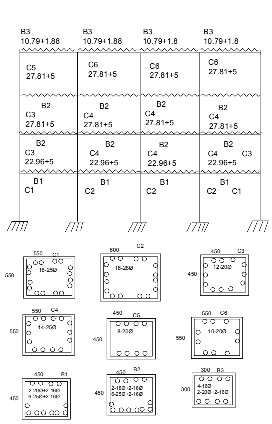

Fig.1(a) Four Storey-OMRF Frame with reinforcement details

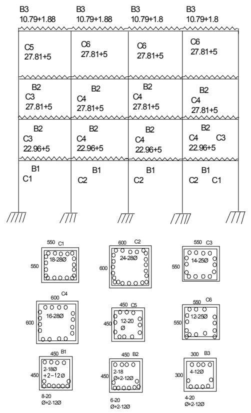

Fig.1(b) Four storey –SMRF Frame reinforcement details

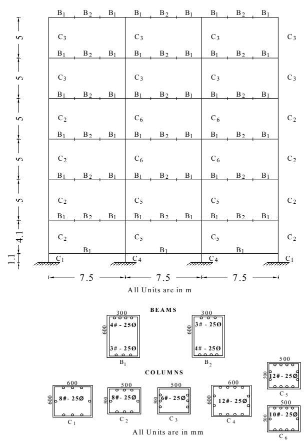

Fig.1(c) Six Storey-OMRF Frame with reinforcement details

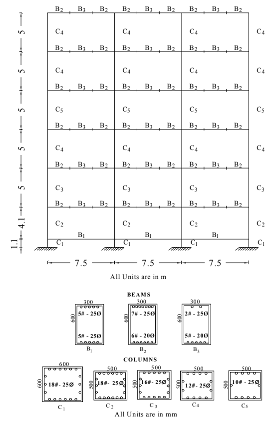

Fig.1(d) Six Storey-SMRF Frame with reinforcement details

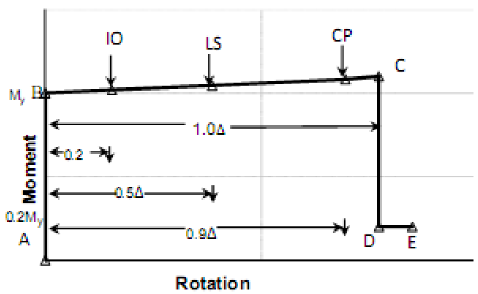

Fig.2 Typical Moment vs. Rotation curves

is based on recommended guidelines for the type of building, economic considerations and engineering judgment.

Severe earthquakes have an extremely low probability of occurrence during the life of a structure. Designing of structures to remain elastic under very severe earthquake ground motion is very difficult and economically infeasible. The most common design approach is to design the buildings based on the two-level seismic concept.

1. Buildings should resist moderate earthquakes, i.e. design basis earthquake (DBE) with essentially no structural damage (elastic behaviour).
2. Building should resist catastrophic earthquake, i.e. maximum considered earthquake (MCE) with some structural damage, but without collapse and major injuries of loss of life. (inelastic response within acceptable level)

From the safety point of view the seismic resistant design of moment resisting building frames are classified as Ordinary Moment Resisting Frames, (OMRF), Intermediate Moment Resisting Frames, (IMRF) and Special Moment Resisting Frames, (SMRF) as referred [4,21]. The yield mechanisms adopted in earthquake resistant design are (i) strong column and weak beam, (ii) flexural yielding in beams, (iii) prevent shear failure or yielding in beams and columns and flexural yielding at base of beams. The performance based design which ensures safety under a specified earthquake by estimating the capacity against the demand, is better approach than conventional code based design. This paper aims to study the behaviour of modern code-conforming OMRF and SMRF under designed earthquake condition for low and medium rise buildings.

5 Nonlinear Static Analysis

The understanding of structural behaviour is greatly facilitated by a study of the static load-deformation responses that identify the elastic and inelastic behaviour characteristics of the structures. The nonlinear static analysis (pushover analysis) is gaining popularity for this purpose. In the pushover analysis, non-linear finite element model of a given structure (eg. a building frame) subjected to gravity loads, is laterally loaded until either a predefined target displacement is met, or the model collapses. The reliable post-yield material model and inelastic member deformations are extremely important in nonlinear analysis. The evaluation is based on an assessment of important parameters, including global drift, inter-storey drift, inelastic element deformations (either absolute or normalized with respect to yield value), deformations between elements, and element and connection forces (for elements and connections that cannot sustain inelastic deformations). The inelastic static pushover analysis can be viewed as a method for predicting seismic force and deformation demands, which accounts in an approximate manner for the redistribution of internal forces occurring due to inertia forces that no longer can be resisted within the elastic range of structural behavior. The two key steps in applying this method, i.e. lateral force distribution and target displacement are based on the assumption that the structural response is mainly from the fundamental mode, and that the mode shapes remain unchanged after structure gets into the inelastic region. The nonlinear static analysis provides accurate estimate of seismic demand for low- and medium-rise moment resisting frames. In the present study, the pushover analysis is carried out for load patterns proportional to fundamental mode. A 100% dead load plus 50% live load is applied prior to the lateral load on the structure.

6 Development of user-defined hinge properties and nonlinear static analysis

The analyses had performed using “SAP2000”, adopting a member-by-member modelling approach. Inelastic beam and column members are modelled as elastic elements with plastic hinges at their ends, the effective rigidity of beams is taken equal to 40% of the gross section rigidity (EI_g) while for columns as 80% [3]. The moment rotation characteristics of the plastic hinges are estimated from section analysis using appropriate non-linear constitute laws for concrete and steel. Generally the deformations are quantified and expressed in terms of chord rotations. The lumped plasticity approach is commonly used in SAP2000 for modelling deformation capacity estimates. The various parameters which are directly related with these deformations are i) steel ductility, ii) bar pullout from the anchorage zone, iii) axial load ratio, iv) shear-span ratio and v) concrete strength. An analytical procedure based on Eurocode-8 is used to study the deformation capacity of beams and columns in terms of yield, plastic and ultimate rotations (θ_u, $\theta_{pl,}$ θ_y) and it defines the state of damage in the structure through three limit states of the NEHRP Guidelines (1997) and FEMA 356 (2000), namely i) Limit State “Near Collapse” (NC) level, corresponding to the “Collapse prevention”(CP) level ii) Limit State of “Significant Damage” (SD) level , corresponding to the “ Life Safety”(LS) level and to the single performance level for which new structures are designed according to current Indian seismic design code iii) Limit State of Damage Limitation (DL) level, corresponding to the “ Immediate Occupancy” (IO) level. The drift or chord rotation of a member over the shear span (L_s) is a primary parameter which captures the macroscopic behaviour of the member. FEMA guidelines imply values of yield rotation approximately equal to 0.005 rad for RC beams and columns, or to 0.003 rad for walls, to be added to plastic hinge rotations for conversion into total rotations, which are approximately equal to the chord rotation θ or drift of the shear span. According to these codes chord rotation θ is the summation of yield rotation

(θ_y) plus plastic rotation (θ_P). Acceptable limiting values of these plastic rotations have been specified for primary or secondary components of the structural system under collapse prevention earthquake as a function of the type of reinforcement, axial and shear force levels and detailing of RC members. For primary components acceptable chord rotations or drifts for collapse prevention earthquake are taken as 1.5 times lower than the ultimate drifts or rotations. For life safety earthquake, the acceptable chord rotations or drifts for primary and secondary components are taken as about 1.5 or 2 times lower than the ultimate rotations or drifts. The yield, plastic and ultimate rotation capacities in terms of non-dimensional numbers is estimated. User defined P-M-M (P-M-M hinges are assigned at the ends of column members which are subjected to axial force and bending moments) and M3 (M- hinges are assigned at the ends of beam members which are subjected to bending moments) curves are developed using the rotation capacities of members/elements. The default-hinge option in SAP2000 assumes average values of hinge properties instead of carrying out detailed calculation for each member. The default-hinge model assumes the same deformation capacity for all columns regardless of their axial load and their weak and strong axis orientation. Hence nonlinear static analyses are carried out using user- defined plastic hinge properties. Definition of user- defined hinge properties requires moment rotation characteristics of each element. Panagiotakos and Fardis, 2001 defined the yield curvature φ_y as the point that marks onset of nonlinearity in the moment-curvature diagram (owing to either yielding of tension reinforcement or nonlinearity in concrete- for compressive strains exceeding 90% of the strain at peak stress of uni-axially loaded concrete):

$$\varphi_y = min\{\frac{f_y}{E_s(1-k_y)d}; \qquad \frac{1.8f_c^{'}}{E_c k_y d}\} \tag{1}$$

The compression zone depth at yield k_y (normalized to d) is $k_y = (n^2A^2 + 2nB)^{1/2} - nA$, in which $n=E_s/E_c$ and A, B are given by Eq. (2) or (3), depending on whether yielding is controlled by the yielding of tension steel or by nonlinearity in the compression zone;

$$A = \rho + \rho^{'} + \rho_v + \frac{N}{bdf_y}$$

$$B = \rho + \rho^{'}\delta^{'} + 0.5\rho_v(1+\delta^{'}) + \frac{N}{bdf_y} \tag{2}$$

$$A = \rho + \rho^{'} + \rho_v - \frac{N}{\varepsilon_c E_s bd} \approx \rho + \rho^{'} + \rho_v - \frac{N}{1.8nbdf_c^{'}}$$

$$B = \rho + \rho^{'}\delta^{'} + 0.5\rho_v(1+\delta^{'}) \tag{3}$$

Considering the lower yield curvature, the yield moment is computed as

$$\frac{M_y}{bd^3} = \phi_y\left\{E_c\frac{k_y^{\,2}}{2}\left(0.5(1+\delta^{'}) - \frac{k_y}{3}\right) + \frac{E_s}{2}\left[(1-k_y)\rho + (k_y-\delta^{'})\rho^{'} + \frac{\rho_v}{6}(1-\delta^{'})\right](1-\delta^{'})\right\} \tag{4}$$

The deformation corresponding to chord rotation at yield, plastic and ultimate rotations are

$$\theta_y = \phi_y\left(\frac{L_v + \alpha_v z}{3}\right) + 0.00135\left(1 + 1.5\frac{h}{L_v}\right) + \frac{\varepsilon_y}{d-d^{'}}\frac{d_b f_y}{6\sqrt{f_c}} \tag{5}$$

$$\theta_{um}^{pl} = \frac{1}{\gamma_{el}}0.0145\left(0.25^{\nu}\right)\left[\frac{max(0.01;\omega^{'})}{max(0.01;\omega)}\right]^{0.3} f_c^{0.2}\left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.275^{100\rho_d}\right) \tag{6}$$

$$\theta_{um} = \frac{1}{\gamma_{el}}0.016\left(0.3^{\nu}\right)\left[\frac{max(0.01;\omega^{'})}{max(0.01;\omega)}f_c\right]^{0.225}\left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)}\left(1.25^{100\rho_d}\right) \tag{7}$$

The confinement effectiveness factor is

$$\alpha = \left(1 - \frac{s_h}{2b_c}\right)\left(1 - \frac{s_h}{2h_c}\right)\left(1 - \frac{\sum b_i^{\,2}}{6b_c h_c}\right) \tag{8}$$

The moment-rotation analysis are carried out considering section properties and a constant axial load on the structural element. In the development of user-defined hinges for beams, axial forces are assumed to be zero and for the columns they are assumed to be equal to maximum load due to several possible combinations considered while designing. Following, the calculation of the ultimate rotation capacity of an element, acceptance criteria are defined and labeled as IO, LS and CP as shown in Fig 2 .The typical user defined M3 and P-M-M hinge used for the analysis are shown in Fig 3. This study defines these three points as 0.2Δ, 0.5Δ and 0.9Δ. Where, Δ is the length of plastic hinge plateau.

The acceptance criteria for performance with in the damage control performance range are obtained by interpolating the acceptance criteria provided for the IO and the LS structural performance levels. Acceptance Criteria for performance with in the limited safety structural performance range are obtained by interpolating the acceptance criteria provided for the life safety and the collapse prevention structural performance levels. A target performance is defined by a typical value of roof drift, as well as limiting values of deformation of the structural elements. To determine whether a building meets performance objectives, response quantities from the pushover analysis are considered with each of the performance levels.

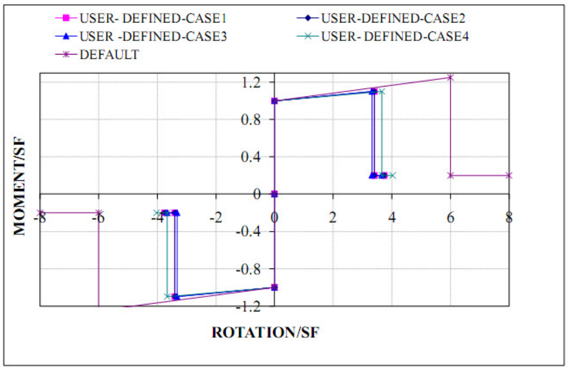

Fig. 3 Typical User- Defined moment-rotation hinge properties
Note:SF is scale w.r.t yield point

7. Evaluation of Seismic Performance of Buildings

The seismic performance of a building is measured by the state of damage under a certain level of seismic hazard. The state of damage is quantified by the drift of the roof and the displacement of the structural elements. Pushover analysis is a nonlinear static analysis in which the magnitude of the lateral load is gradually increased, maintaining a predefined distribution pattern along the height of the building. At each step, the base shear and the roof displacement relationship are plotted to generate the pushover/capacity curve. It gives an insight into the maximum base shear that the structure is capable of resisting. Building performance level is a combination of the performance levels of the structure and the non-structural components. The performance level describes a limiting damage condition which may be considered satisfactory for a given building with specific ground motion. The three global performance levels (FEMA356) considered are as follows. i) Immediate Occupancy: Transient drift is about 1% or negligible permanent drift, ii) Life Safety: Transient drift is about 2% or 1% permanent drift, iii) Collapse Prevention: 4% transient drift or permanent drift.

8 Plastic Hinge Length

Plastic hinges form at the maximum moment regions of RC members. The accurate assessment of plastic hinge length is important in relating the structural level response to member level response. The length of plastic hinge depends on many factors. The following is a list of important factors that influence the length of a plastic hinge 1) level of axial load 2) moment gradient 3) level of shear stress in the plastic hinge region 4) mechanical properties of longitudinal and transverse reinforcement 5) concrete strength and 6) level of confinement and its effectiveness in the potential hinge region. From the literature the following expressions are adopted for the present study

$$L_p = 0.18L_s + 0.025a_{sl}\,d_b\,f_y \quad (9)$$

$$L_p = 0.8h + 0.025a_{sl}\,d_b\,f_y \quad (10)$$

$$L_p = 0.5h \quad (11)$$

$$L_p = 0.08L + 0.022\,f_y\,d_{bl} \geq 0.044\,f_y\,d_{bl} \quad (12)$$

$$L_{pl} = 0.1L_V + 0.17h + \frac{0.24d_{bl}f_y}{\sqrt{f_c}} \quad (13)$$

The nonlinear static analyses are carried out for two designed cases of low and medium rise buildings, in each case separate analyses were carried out by varying the plastic hinge length estimated through the above mentioned expressions and thus totally five cases are studied. They are namely case1, case2, case3, case4 and case5 corresponding to Eq.9-13. The capacity curves observed in each case are shown in Fig.4-7.

The roof displacement obtained in this study obviously show that the demands of 4-storey buildings are higher than those of 6-storey ones. Therefore, it is difficult to precisely estimate which building group is more vulnerable during a seismic event. However SMRF building shows higher capacity compared to OMRF. The study also reveals that the amount of transverse reinforcement plays an important role in seismic performance of buildings, as the amount of transverse reinforcement increases the sustained damage decreases. A profound variation in capacity and displacement are brought out by varying the plastic hinge length and designing the building as OMRF and SMRF. Table 1 shows the inelastic response displacements of the frame. It is observed that inelastic displacement of all the frames are within collapse prevention.

Table.1 Inelastic response displacements (storey drifts in meter)

Details	IO	LS	CP
4-storey-OMRF	0.012	0.023	0.046
4-storey-SMRF	0.019	0.038	0.023
6-Storey-OMRF	0.003	0.005	0.010
6-Storey-SMRF	0.004	0.009	0.017

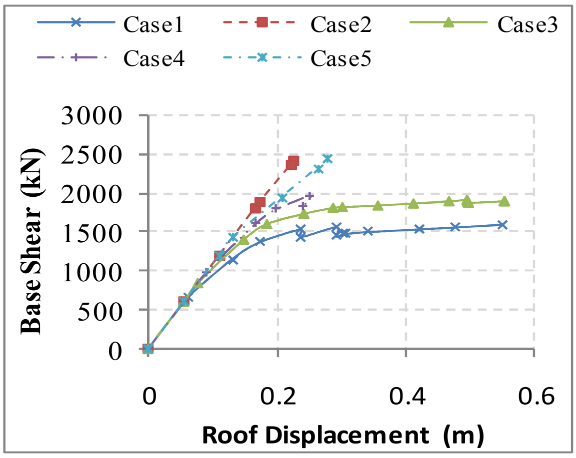

Fig.4 Capacity curves of four storey –OMRF

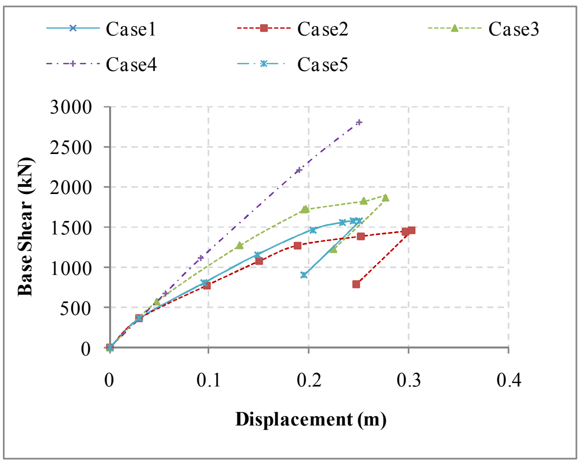

Fig.5 Capacity curves of four storey- SMRF

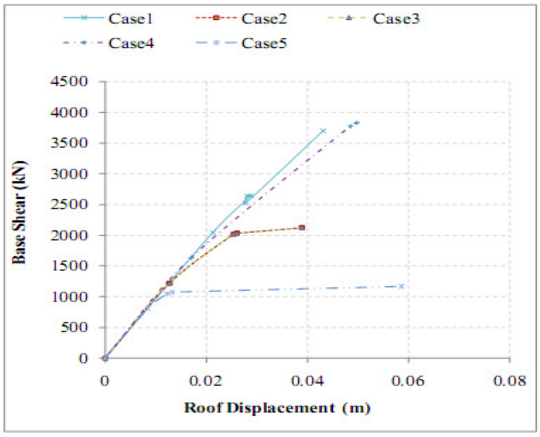

Fig.6 Capacity curves of six storey –OMRF

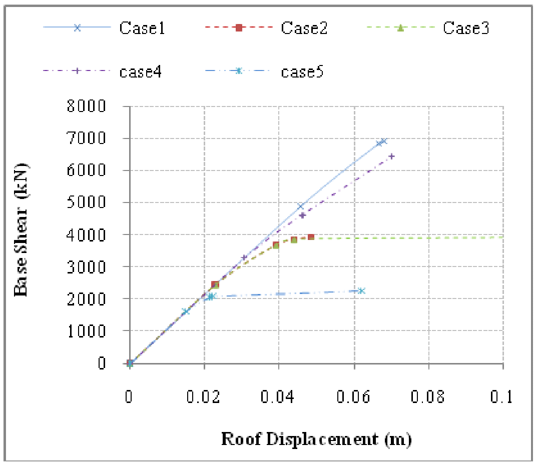

Fig.7 Capacity curves of six storey-SMRF

9 Vulnerability Analysis

The vulnerability index is a measure of the damage in a building [11] obtained from the pushover analysis. It is defined as a scaled linear combination (weighted average) of performance measures of the hinges in the components, and is calculated from the performance levels of the components at the performance point or at the point of termination of the pushover analysis. The vulnerability index of a building is assessed with the expression as follows

$$VI_{bldg} = \frac{1.5\,\Sigma_i^c N_c^i\, x_i + \Sigma_j^b N_b^j\, x_j}{\Sigma_c^i N + \Sigma_b^j N} \qquad (14)$$

Where N_c^i and N_b^j are the numbers of hinges in colunmns and beams, respectively, for the i^{th} and j^{th} performance range. A weightage factor (x_i) is assigned for columns and (x_j) is assigned for beams to each performance range, the weightage factor is shown in Table.2 .

VI_{bldg} is a measure of the overall vulnerability of the building. A high value of VI_{bldg} reflects poor performance of the building. However, this index may not reflect a soft storey mechanism.

Table.2 Weightage Factors for Performance Range

Serial Number	Performance Range (i)	Weightage Factor (xi)
1	<B	0
2	B-IO	0.125
3	IO-LS	0.375
4	LS-CP	0.625
5	CP-C	0.875
6	C-D,D-E, and >E	1.00

A soft storey mechanism is difficult to trace with this method. A storey vulnerability index (VI_{storey}) defined to quantify the possibility of a soft/weak storey with the formation of flexural hinges. For each storey VI_{storey} is defined as

$$VI_{storey} = \frac{\sum N_c^i x_i}{\sum_i^c N_c^i} \qquad (15)$$

Where N_c^i is the number of column hinges in the storey under investigation for a particular performance range. In a given building, the presence of soft/weak storey is reflected by a relatively high value of VI_{storey} for that storey, in relation to the other storeys. The vulnerability Index of the buildings studied is shown in the Table. 3. The vulnerability index of storey (VI_{storey}) is observed to be almost very neglibible in the case of four storey building. Where as it is considerable in the case of 6-storey OMRF building, where column damages are observed in the ground floor itself. From the study it is apparent that, the OMRF framed buildings are more vulnerable than SMRF and storey vulnerability index of zero indicate that most of the hinges are formed in beams rather than in columns.

Table 3 Vulnerability Index based on Nonlinear Static Analysis

Details	4- storey OMRF	4-storey SMRF	6- storey OMRF	6-storey SMRF
Case1	0.354	0.304	0.1897	0.0011
Case2	0.013	0.003	0.0357	0.017
Case3	0.301	0.263	0.0357	0.017
Case4	0.202	0.127	0.0513	0.0513
Case5	0.016	0.188	0.0513	0.054

9 Conclusions

This study has illustrated the nonlinear static analysis responses of OMRF and SMRF building frames under designed ground motions. The capacity against demand is observed significantly higher for SMRF building frames compared to OMRF. The user defined hinge definition and development methodology is also described. The user- defined hinges takes into account the orientation and axial load level of the columns compared to the default hinge. The influence of plastic hinge on capacity curve is brought out by deploying five cases of plastic hinge length. The study reveals that plastic hinge length has considerable effects on the displacement capacity of frames. Based on the analysis results it is observed that inelastic displacement of the modern code-conforming building frames are within collapse prevention level. The vulnerability index which is a measure of damage is estimated for both SMRF and OMRF are presented for 4- and 6-storey buildings. From the study it is apparent that, the OMRF framed buildings are more vulnerable than SMRF. The vulnerability index of the building quantitatively express the vulnerability of the building as such, where as storey vulnerability index assist to locate the columns in the particular storey in which significant, slight or moderate level of damages have taken place.

Acknowledgement

The paper is published with kind permission of Director, CSIR-Structural Engineering Research Centre, Chennai, India.

Notations

a_{sl} is a coefficient = 1 if slippage of longitudinal steel from its anchorage zone beyond the end section is possible , otherwise it is zero

b_0 and h_0 is the dimension of confined core to the centerline of the hoop,

b_i is the centerline spacing of longitudinal (indexed by i) laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section.

d depth of the cross- section

d_b is diameter of the tension reinforcement

d_{bl} is diameter of longitudinal reinforcement

f_c and f_{yw} are the concrete compressive strength (MPa) and the stirrup yield (MPa) strength respectively

f_c' uniaxial (cylindrical) concrete strength (MPa)

f_y is steel yield stress (MPa)

h is the depth of cross-section

Ec Young's modulus of the reinforced concrete

Es Young's modulus of the steel

K_y compressive zone depth

L_p is the length of plastic hinge

L_v is M/V , the distance from the critical section of the plastic hinge to the point of contra flexure

N is axial force

V is N/bhf_c (b width of compression zone, N axial force positive for compression)

α is the confinement effectiveness factor

$\alpha_V z$ is the tension shift of the bending moment diagram

γ_{el} is equal to 1.5 for primary seismic elements and to 1.0 for secondary seismic elements

δ' d'/d where d' is the distance of the centre of the compression reinforcement from the extreme compression fibre

Θ_y is Rotation at yield in radians

Θ_p is plastic rotation in radians

Θ_{um} is ultimate rotation in radians

ρ_{sx} is $A_{sx}/b_w s_h$, ratio of transverse steel parallel to the direction x of loading (s_h=stirrup spacing)

ρ_d is the steel ratio of diagonal reinforcement

Φ_y is the yield curvature of the end section

ω, ω' is the mechanical reinforcement ratio of the tension (including the web reinforcement) and compression, respectively, longitudinal reinforcement,

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