Seismic Performance and Vulnerability Analysis of Code - Conforming
RC Buildings

A. CINITHA.A\textsuperscript{1*}, P.K. UMESHA\textsuperscript{2}, NAGESH R. IYER\textsuperscript{3}

CSIR- Structural Engineering Research Centre
CSIR Campus, Taramani, Chennai – 600113, INDIA
Email: 1cinitha@sercm.org, 2pku@sercm.org, 3nriyer@sercm.org

Abstract: This paper investigates seismic performance and vulnerability analysis of 4-storey and 6-storey code-conforming (IS: 456-2000, Indian standard for plain and reinforced concrete code and IS: 1893-2002, Indian standard criteria for earthquake resistant design of structures) reinforced concrete (RC) buildings. The buildings are designed for two different cases such as ordinary moment resisting frame (OMRF) and special moment resisting frame (SMRF). The nonlinear static analysis (pushover analysis) is used to capture initial yielding and gradual progressive plastic behaviour of elements and overall building response under seismic excitations. The deformation characteristics of structural elements are essential to simulate the plastic hinge formation in the process of generation of capacity curve during the pushover analysis. An analytical procedure is developed to evaluate the yield, plastic and ultimate rotation capacities of beams and columns along with different plastic hinge lengths. 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 pushover analysis using SAP2000. 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 in the pushover analysis. The building performances are assessed with the capacity curve generated. 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 of the buildings is estimated in terms of vulnerability index to assess the performance of the building.

Key-words: Seismic performance, target displacement, roof drift, pushover analysis, Vulnerability analysis

1 INTRODUCTION

Pushover analysis, to evaluate the seismic performance of buildings, represents the current trend in structural engineering and promises a reasonable prediction of structural behaviour. The analysis provides adequate information on seismic demands imposed by the design ground motion on the structural system and its components. The method there by evaluates the seismic performance of the structure and quantifies its characteristic behaviour (strength, stiffness and deformation capacity) under design ground motion. This information can be used to check the specified performance criteria [1-10] and [14-17]. Modelling the inelastic behaviour of the structural elements for different levels of performance is an important step towards performance evaluation of building. The nonlinear static analysis procedures to estimate the seismic performances of structures are described in National Earthquake Hazards Reduction Program (NEHRP) guidelines for the seismic rehabilitation of buildings [6-8]. It require realistic values of the effective cracked stiffness of reinforced concrete (RC) members up to yielding for reliable estimation of the seismic force and deformation demands. [9, 12, 13 and 18] have shown that linear elastic analysis with 5% damping can satisfactorily approximate inelastic seismic deformation demands. The present paper aims to compare the influence of the different assumptions of ATC 40, FEMA 356 and Eurocode 8 for the assessment of Indian code conforming buildings via pushover 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 incorporated 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 pushover analysis. The pushover 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 pushover 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 structural performance levels 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, 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 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
1. Buildings should resist moderate earthquakes, i.e. design basis earthquake (DBE) difficult and economically infeasible. The most common design approach is to design the buildings based on the two-level seismic concept.

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 [4,20]. The yield mechanisms adopted in earthquake resistant design are (i) strong column and weak beam, (ii) flexural yielding in beams, (iii) prevent shear fail-

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

**Fig.1(b)** Four Storey –SMRF Frame with 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
5 PUSHOVER 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 (e.g., 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 MODELING APPROACH

Analysis has been performed using SAP2000, which is general purpose structural analysis software for static and dynamic analysis. In this study, SAP2000 nonlinear version 11 is used.

A two-dimensional model of each structure is created in SAP 2000 to carry out nonlinear static analysis. Beams and column elements are modeled as nonlinear frame elements with lumped plasticity by defining plastic hinge at both ends of beams and columns.

The definition of user-defined hinge properties requires moment-rotation relationship of each element. The modified Mander’s material model for confined concrete and typical steel stress-strain model with strain hardening for steel are implemented in SAP2000. The inelastic deformation capacities of RC elements are modeled with plastic hinges in the form of yield, plastic and ultimate rotations based on [10], as follows:

\[
an_{um} = \frac{1}{7_{um}} 0.016 \left(\frac{\text{max} \left(0.1k_c, c\right)}{\text{max} \left(0.1k_c, w\right)}\right)^{0.223} \left(\frac{L_p}{h_p}\right)^{0.67} \left(\frac{W_{w,c}}{h_p}\right)^{0.25} \left(\frac{L_p}{h_p}\right)^{0.25}
\]

\[
\alpha = \left(1 - \frac{s_h}{2b_0}\right) \left(1 - \frac{s_h}{2h_0}\right) \left(1 - \frac{\sum b_i^2}{6h_0b_0}\right)
\]

\[
\theta_{um} = \theta_{um} - \theta_y
\]

\[
\theta_y = \phi_y \frac{L_y + \alpha \sqrt{\gamma}}{3} + 0.0015 \left(1 + 1.5 \frac{h}{L_y}\right) + 0.13 \phi_y \frac{d_b f_y}{\sqrt{\gamma}}
\]

The inelastic capacity of members are modeled by defining the performance levels corresponding to the acceptance criteria. For the acceptable limit values of IO, LS and CP, [3] suggests the following relationships: for performance within the damage control performance range. \(\text{IO}=0.2 \Delta, \text{LS}=0.5 \Delta, \text{CP}=0.9 \Delta\), where, \(\Delta\) is length of plastic hinge. Once the structure is modeled with section properties, steel content and loads on it, flexural moment hinges (M3) are assigned to the both ends of the beams, while the axial-moment hinges (P-M-M) are assigned to the both ends of columns. The approximate component initial effective stiffness values are considered according to [3]: 0.5EI and 0.7EI for beams and columns respectively.
7 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

\[ L_p = 0.18 A_a + 0.025 a_d b_f y \] (4)

\[ L_p = 0.8 h + 0.025 a_d b_f y \] (5)

\[ L_p = 0.5 h \] (6)

\[ L_p = 0.08 L + 0.022 f_y d_b l \geq 0.044 f_y d_b l \] (7)

\[ L_{pl} = 0.1 L_v + 0.17 h + \frac{0.24 d_b l f_y}{\sqrt{f_c}} \] (8)

The pushover 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.4-8. The capacity curves observed in each case are shown in Fig.3-6.

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>
<thead>
<tr>
<th>Details</th>
<th>IO</th>
<th>LS</th>
<th>CP</th>
</tr>
</thead>
<tbody>
<tr>
<td>4-storey-OMRF</td>
<td>0.012</td>
<td>0.023</td>
<td>0.046</td>
</tr>
<tr>
<td>4-storey-SMRF</td>
<td>0.019</td>
<td>0.038</td>
<td>0.023</td>
</tr>
<tr>
<td>6-storey-OMRF</td>
<td>0.003</td>
<td>0.005</td>
<td>0.010</td>
</tr>
<tr>
<td>6-storey-SMRF</td>
<td>0.004</td>
<td>0.009</td>
<td>0.017</td>
</tr>
</tbody>
</table>

Table 1 Inelastic response displacements (storey drifts in meter)

Fig.3 Capacity curves of four storey –OMRF

Fig.4 Capacity curves of four storey- SMRF
8 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

$$\text{VI}_{\text{bldg}} = \frac{1.5 \sum_{i} N_{c}^{i} x_{i} + \sum_{j} N_{b}^{j} x_{j}}{\Sigma_{c} N_{c} + \Sigma_{b} N_{b}}$$  \hspace{1cm} (9)$$

Where $N_{c}^{i}$ and $N_{b}^{j}$ are the numbers of hinges in columns 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.

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

<table>
<thead>
<tr>
<th>Serial Number</th>
<th>Performance Range (i)</th>
<th>Weightage Factor ($x_{i}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>&lt;B</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>B-IO</td>
<td>0.125</td>
</tr>
<tr>
<td>3</td>
<td>IO-LS</td>
<td>0.375</td>
</tr>
<tr>
<td>4</td>
<td>LS-CP</td>
<td>0.625</td>
</tr>
<tr>
<td>5</td>
<td>CP-C</td>
<td>0.875</td>
</tr>
<tr>
<td>6</td>
<td>C-D,D-E, and &gt;E</td>
<td>1.00</td>
</tr>
</tbody>
</table>

Table.2 Weightage Factors for Performance Range

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

$$\text{VI}_{\text{storey}} = \frac{\sum_{i} N_{c}^{i} x_{i}}{\sum_{i} N_{c}^{i}}$$  \hspace{1cm} (10)$$

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 $\text{VI}_{\text{storey}}$ 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 ($\text{VI}_{\text{storey}}$) is observed to be almost very negligible 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 Push-over Analysis

<table>
<thead>
<tr>
<th>Details</th>
<th>4-storey OMRF</th>
<th>4-storey SMRF</th>
<th>6-storey OMRF</th>
<th>6-storey SMRF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>0.354</td>
<td>0.304</td>
<td>0.1897</td>
<td>0.0011</td>
</tr>
<tr>
<td>Case 2</td>
<td>0.013</td>
<td>0.003</td>
<td>0.0357</td>
<td>0.017</td>
</tr>
<tr>
<td>Case 3</td>
<td>0.301</td>
<td>0.263</td>
<td>0.0357</td>
<td>0.017</td>
</tr>
<tr>
<td>Case 4</td>
<td>0.202</td>
<td>0.127</td>
<td>0.0513</td>
<td>0.0513</td>
</tr>
<tr>
<td>Case 5</td>
<td>0.016</td>
<td>0.188</td>
<td>0.0513</td>
<td>0.054</td>
</tr>
</tbody>
</table>

9 CONCLUSIONS

This study has illustrated the inelastic 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 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. Except 6-storey OMRF frame all other cases shows storey vulnerability index as zero, which indicates that most of the hinges are formed in beams rather than in columns.

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Notations

- $a_{sl}$ is a coefficient
- $b_0$ and $h_0$ is the dimension of confined core to the centerline of the hoop,
- $b_i$ is the centerline spacing of longitudinal, laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section.
- $d_b$ is diameter of the tension reinforcement
- $d_{sl}$ is diameter of longitudinal reinforcement
- $f_c$ and $f_y$ are the concrete compressive strength (MPa) and the stirrup yield (MPa) strength respectively
- $f_y$ is steel yield stress (MPa)
- $h$ is the depth of cross-section
- $L_p$ is the length of plastic hinge
- $L_v$ is distance from the critical section of the plastic hinge to the point of contra flexure
- $V$ is $N/bh_f$ ( $b$ width of compression zone, $N$ axial force positive for compression).
- $\alpha$ is the confinement effectiveness factor
- $\alpha V_z$ is the tension shift of the bending moment diagram
- $\gamma_{cf}$ is a constant
- $\Theta_y$ is rotation at yield in radians
- $\Theta_p$ is plastic rotation in radians
- $\Theta_{um}$ is ultimate rotation in radians
- $\rho_d$ is the steel ratio of diagonal reinforcement
- $\Phi_y$ is the yield curvature of the end section
- $\omega_{me}$ is the mechanical reinforcement ratio of the tension (including the web reinforcement) and compression, respectively, longitudinal reinforcement.

References


