Adaptive Pushover Analysis for RC Buildings Considering Generic Frames

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SUMMARY:
Recently, pushover analysis has been the most popular tool for seismic performance evaluation of buildings in engineering practice because of its simplicity and superiority in comparison with other analysis procedures. Adaptive pushover analysis is a technique in which, the effects of higher modes and change of dynamic properties are considered. Standard software packages like SAP 2000, ETABS, MIDAS etc. does not have built in option for performing adaptive pushover analysis as of now. This study is an attempt to utilise the SAP 2000 package for performing adaptive pushover analysis for two dimensional reinforced concrete buildings. A procedure to utilise SAP2000 package for performing adaptive pushover analysis is developed. However, the pushover curve of the conventional analysis and the adaptive analysis shows a close resemblance; which implies that there is no apparent and appreciable advantage of the adaptive approach over the conventional pushover analysis, using multimode lateral load, for buildings with regular configuration.

Keywords: Adaptive pushover analysis, Analysis, RC Generic Frames, SAP2000, IS 1893:2002

1. INTRODUCTION
Multistoreyed structures are analysed and designed using either Seismic Coefficient method or Response Spectrum analysis [IS 1893: 2002]. While estimating the seismic demands, especially at low performance levels, consideration of inelastic behaviour of the structure is necessary. Nonlinear time history (NTH) analysis is a very powerful but also a complex tool for the study of structural seismic response [Krawinkler and Seneviratna 1998]. Since NTH analysis of building structures is not feasible for the practical day to day applications in the structural design consultancy field, a simpler method that is rational and would achieve a reasonable balance between required predictability and applicability for everyday use has become the need for the day, the answer to which was found in the pushover analysis (PoA). Due to its simplicity and superiority in comparison with other analysis procedures, structural engineers have been using this method, described in FEMA-356 and ATC-40. It is widely accepted that, when PoA is used carefully, it provides useful information that cannot be obtained by linear static or dynamic analysis procedures [Krawinkler and Seneviratna 1998].In conventional PoA technique, seismic demands are computed by nonlinear static analysis of the structure subjected to monotonically increasing lateral loads with an invariant vertical distribution until a predetermined state is reached [Elnashai 2001, Antoniou and Pinho2004]. This state is either the Target Displacement [FEMA-356] or a Performance Point [ATC-40]. The PoA method is based on the assumption that the response is controlled by the fundamental mode and that its changes after the structure starts yielding can be ignored. The conventional PoA procedure exhibits a number of limitations, related to its inability to account for the progressive stiffness degradation, the change in modal characteristics and the period elongation for each mode as the structure yields. In adaptive
PoA, higher mode effects and change of dynamic properties are considered [Kaan and Irtem 2004; Kunnath 2004].

2. PUSHER METHODOLOGY

In the implementation of PoA, the nonlinear behaviour of structure is incorporated by the nonlinear properties of each component of the structure in bending and shear which are applied as hinges inserted in the members. Lumped plasticity idealization is a commonly used approach for hinges [Mehmet and Hayri 2006]. The ultimate deformation capacity of a component depends on the ultimate curvature and plastic hinge length. Using PoA, a characteristic nonlinear force versus deformation can be determined which gives the capacity curve, while the response spectrum curve modified at each step of lateral loading increment for the instantaneous damping estimated, gives the set of demand curves. Superimposing them gives the point of intersection of the two as the Performance Point of the structure. In this paper, PoA is done with a lateral load pattern proportional to a combination of mode shapes. The adaptive PoA is run in stages and at the end of each stage, the hinge stiffnesses are determined; modal analysis is performed with the instantaneous hinge stiffness. The changes in mode shapes are determined, based on which the lateral load pattern is modified and the analysis is continued from that point.

3. DESCRIPTION OF THE FRAME STRUCTURES

Three structures representing low, medium and high rise reinforced concrete framed buildings are considered in present study. The selected structures are idealized as two dimensional generic frames of three different heights ie., 4, 8 and 20 storeyed frames (Fig. 3.1). Typical floor to floor height is 3.6 m. The dynamic characteristics of the three models are shown in Table 3.1. The cumulative mass participation factor for the first four modes for the 4, 8 and 20 storeyed frame models are: 92.48 %, 94.47 % and 95.29 % respectively. As this is greater than 90 % (a requirement as per IS 1893: 2002), only the first four modes are considered for the analysis.

<table>
<thead>
<tr>
<th>Building</th>
<th>Mode</th>
<th>Period</th>
<th>Mass Participation Factor (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 Storey</td>
<td>1</td>
<td>0.7890</td>
<td>78.76</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.2402</td>
<td>10.49</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.1273</td>
<td>2.59</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.0864</td>
<td>0.64</td>
</tr>
<tr>
<td>8 Storey</td>
<td>1</td>
<td>1.3357</td>
<td>79.13</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.4519</td>
<td>9.40</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.2321</td>
<td>4.03</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.1547</td>
<td>1.91</td>
</tr>
<tr>
<td>20 Storey</td>
<td>1</td>
<td>1.1117</td>
<td>82.15</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.3854</td>
<td>9.50</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.2369</td>
<td>2.22</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.1670</td>
<td>1.42</td>
</tr>
</tbody>
</table>
4. MODELLING APPROACH

Analyses have been performed using the structural analysis package of SAP 2000. In the three models created, beams and columns are modelled with frame elements, and plastic hinges defined are inserted into them. PMM (axial force - biaxial moments) hinge, which yields based on the interaction of axial force and bending moment have been used for the columns and M3 hinges (uniaxial moment) for the beams (Fig. 4.1). The dead and imposed loads are applied to the models. The mode shape coefficients $\Phi_{ik}$ (modal displacement at floor $i$ in mode $k$) is obtained from modal analysis, and the modal participation factor $\Gamma_k$, which is a measure of the degree to which the $k^{th}$ mode participates in the response is calculated [Chopra 2009]. The peak storey lateral force at storey $i$, due to all the modes considered is obtained by combining those due to each mode, using the Square Root of the Sum of Squares (SRSS) method given in Eq. 4.1 as.

$$Q_i = \text{SRSS} \left( \sum_{k=1}^{n} \Gamma_k \Phi_{ik} \left( \frac{S_a}{g} \right)_k \right)^{\frac{1}{2}} \times W_i$$  \hspace{1cm} (4.1)
In conventional PoA, the model is laterally pushed in a single stage and the pushover curve is obtained. In adaptive PoA, the push is applied to the model in different stages. When the model is subjected to a push to a particular displacement, the hinge states in structure may change. For each hinge formed at the end of a load stage, the moment $M$ and plastic rotation $\theta$ are noted (Fig. 4.2), from which the instantaneous hinge stiffness $K_h$ is calculated as $M/\theta$. These calculated stiffnesses of all hinges are applied on a similar structure modelled in the structural analysis package, STAAD.Pro.(Fig. 4.3). In this model, short segments separated from the ends of beams and columns are considered as hinges (Fig. 4.4 (a)) whose flexural rigidity, being equal to $EI$ ($E$ – modulus of elasticity, $I$ – moment of inertia), can be varied by adjusting the value of $E$ appropriately, to match the respective calculated $K_h$. The modal analysis is then done on the model. It is for the convenience of using the command prompt for input while modifying $E$ values that STAAD.Pro is chosen for the modal analysis. For the segment representing the unyielded hinge (Fig. 4.3 (b) and Fig.4.4 (a)) the stiffness as given in Eq.4.2 is

$$K_s = \frac{EI}{l} = \frac{M}{\theta} \quad \therefore E = \frac{K_s l}{I} \quad (4.2)$$

where, $\theta$ is the rotation of the hinge in radians, $K_s$ is stiffness, $l$ is length and $I$ is the Moment of Inertia of the hinge segment. For the yielded hinge, stiffness is estimated in the form of Eq.4.3 as,

$$\frac{1}{K_e} = \frac{1}{2K_s} + \frac{1}{K_h} + \frac{1}{2K_s} \quad \therefore K_e = \frac{K_s K_h}{K_h + 2K_s}, \quad E_e = \frac{K_e l}{I} \quad (4.3)$$

where, $E_e$ is the effective Modulus of Elasticity, $K_e$ is equivalent stiffness of the segment, $2K_s$ is the stiffness corresponding to the segments on either side of the plastic hinge and $K_h$ is the instantaneous secant stiffness of the plastic hinge spring (Fig. 4.4 (b)).
From the modal analysis done on the STAAD.Pro model, the mode shape coefficients are obtained. The new design lateral force obtained from the modal analysis are then assigned to the frame model in SAP2000. The pushover analysis is continued from the end of the previous step. The next changed state of the hinges will be considered for calculating the new set of $E_e$’s and the procedure is repeated.

(a) Before formation of Hinge  
(b) After the formation of Hinge

![Figure 4.4. Idealization of Hinge segment](image)

5. RESULTS AND DISCUSSIONS

For 4 and 8 storey frames, the lateral push was given with an increment ($\Delta n$) of 0.05 m per stage and for the 20 storey frame, an increment of 0.1 m per stage was given. The stages were fixed based on the performance point obtained during the conventional PoA. Plastic hinge formations for the three models were obtained at different displacements levels. The ‘hinging’ patterns for the three frames are shown in Fig. 5.1. It can be observed that the plastic hinge formation starts with beams and columns of lower stories. In Fig. 5.1, the hinge stages at different steps are indicated with different colour codes.

5.1 Variation in Mode Shapes

The scaled mode shape coefficients for different modes (obtained by multiplying $\{\Phi_k\}$ with $\Gamma_k$,) for the initial stage and for the final stage for the first four modes in the adaptive PoA for the three buildings are shown in the Fig. 5.2. The difference in the mode shapes in the initial and final stages are due to the hinge formation as the loading progresses.

5.2 Pushover Curve

The comparison of the pushover curves of the conventional and adaptive cases for the three models are shown in Fig. 5.3. It can be seen that there is no significant difference between the pushover curves of conventional and adaptive PoA methods. Fig. 5.3(c) shows the different stages of the conventional and adaptive pushover curves for the 20 storey case. Since each stage comprises of 0.1m push, the lateral load distributions were updated at 0.1, 0.2, 0.3, 0.4 and 0.5m. They were updated to be compliant with the instantaneous mode shapes at the beginning of each stage. The distribution of base shear for each stage with respect to the base shear at the end of the final stage are 46.8%, 42.9%, 6.21%, 2.52% and 1.53% for stages 1 to 5 respectively. It is a fact that the changes in the mode shapes become prominent only in the later stages, where as the increase in base shear in those stages are very small percentages with respect to the total cumulative base shear at the end of each stage. This is the reason why there is no appreciable difference between the pushover curves of the adaptive and conventional cases of analyses. But one can notice a variation between the initial mode shapes (which
remains constant in case of conventional) and the final mode shape (i.e., mode shape in the final stage) of adaptive PoA.

Figure 5.1. Hinge states at different steps in Adaptive PoA
Figure 5.2. Variation in the first four mode shapes at initial and final stages for all frames
5.3 Lateral Load Distribution

Figure 5.3. Pushover curve for all frame models

Figure 5.4. Lateral load distribution (last stage alone) for 20 storied model (A – adaptive PoA, C – conventional PoA)

Figure 5.5. Final stage cumulative lateral load distribution for 20 storied model (A – adaptive PoA, C – conventional PoA)
Although the variation between the lateral load distribution (Fig. 5.4) for the two analyses cases are significant, there is no apparent difference when it is taken cumulatively (Fig. 5.5). Fig. 5.6 shows the percentage variation of lateral load distribution at the final stage of adaptive pushover analysis from the conventional pushover analysis. Here also, it can be seen that the difference is minimal while considering the cumulative effect of lateral loads, accumulated over different stages.

### 5.4 Inter-storey Drift

The comparison of the inter-storey drifts of the conventional (considering the higher mode effects) and adaptive cases for the three buildings are shown in Fig. 5.7. It can be seen that, there is no significant change in the inter-storey drifts for all the frames considered.

![Graph showing inter-storey drift for different frame models](image_url)

(a) Four storey frame model  (b) Eight storey frame model  (c) Twenty storey frame model

**Figure 5.7.** Inter-storey drift for all frame models
6. CONCLUSION

In this paper a comparison of the adaptive and conventional PoA’s is presented. The selected structures are RC frames of three different heights; 4, 8 and 20 storied structures, analyzed using the analysis software SAP2000. The changes in the mode shapes at various stages obtained during the adaptive PoA are also presented.

From the study, it is observed that for regular structures, the variation between the results obtained from the conventional and the adaptive methods are negligible. The changes in mode shapes in the adaptive approach occur due to stiffness degradation, as the plastic hinge formation progresses. These changes are significant only during the final stages of the analysis. Hence, the contribution of changes in the vertical distribution of lateral force is negligible. This is found to be the same for all the test cases, viz., 4, 8 and 20 storeyed frame models.

REFERENCES


