Technical Note

PUSHOVER ANALYSIS OF REINFORCED CONCRETE FRAME STRUCTURES

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Abstract

The Boumerdes 2003 earthquake which has devastated a large part of the north of Algeria has raised questions about the adequacy of framed structures to resist strong motions, since many buildings suffered great damage or collapsed. To evaluate the performance of framed buildings under future expected earthquakes, a non linear static pushover analysis has been conducted. To achieve this objective, three framed buildings with 5, 8 and 12 stories respectively were analyzed. The results obtained from this study show that properly designed frames will perform well under seismic loads.

Keywords: Nonlinear static procedure; nonlinear hinge properties; pushover analysis; reinforced concrete frames

1. Introduction

The recent earthquakes including the last Algerian earthquake in which many concrete structures have been severely damaged or collapsed, have indicated the need for evaluating the seismic adequacy of existing buildings. In particular, the seismic rehabilitation of older concrete structures in high seismicity areas is a matter of growing concern, since structures venerable to damage must be identified and an acceptable level of safety must be determined. To make such assessment, simplified linear-elastic methods are not adequate. Thus, the structural engineering community has developed a new generation of design and seismic procedures that incorporate performance based structures and is moving away from simplified linear elastic methods and towards a more non linear technique. Recent interests in the development of performance based codes for the design or rehabilitation of buildings in seismic active areas show that an inelastic procedure commonly referred to as the pushover analysis is a viable method to assess damage vulnerability of buildings. Basically, a pushover analysis is a series of incremental static analysis carried out to develop a capacity curve for the building. Based on the capacity curve, a target displacement which is an

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estimate of the displacement that the design earthquake will produce on the building is determined. The extent of damage experienced by the structure at this target displacement is considered representative of the damage experienced by the building when subjected to design level ground shaking. Many methods were presented to apply the nonlinear static pushover (NSP) to structures. These methods can be listed as: (1) the capacity spectrum method (CSM) (ATC, [1]), (2) the displacement coefficient method (DCM) (FEMA-356 [2], (3) modal pushover analysis (MPA), [3]. The approach has been developed by many researchers [4,5] with minor variation in computation procedure. Since the behaviour of reinforced concrete structures may be highly inelastic under seismic loads, the global inelastic performance of RC structures will be dominated by plastic yielding effects and consequently the accuracy of the pushover analysis will be influenced by the ability of the analytical models to capture these effects. In general, analytical models for the pushover analysis of frame structures may be divided into two main types: (1) distributed plasticity (plastic zone) and (2) concentrated plasticity (plastic hinge). Although the plastic hinge approach is simpler than the plastic zone, this method is limited to its incapacity to capture the more complex member behaviour that involve severe yielding under the combined actions of compression and bi-axial bending and buckling effects. In this paper, are presented the results of pushover analysis of reinforced concrete frames designed according to the Algerian code.

2. Pushover Methodology

A pushover analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral loads, representing the inertial forces which would be experienced by the structure when subjected to ground shaking. Under incrementally increasing loads various structural elements may yield sequentially. Consequently, at each event, the structure experiences a loss in stiffness. Using a pushover analysis, a characteristic non linear force-displacement relationship can be determined.

2.1 Key elements of the pushover analysis

- Definition of plastic hinges
  In SAP2000 [6], nonlinear behaviour is assumed to occur within frame elements at concentrated plastic hinges. The default types include an uncoupled moment hinges, an uncoupled axial hinges, an uncoupled shear hinges and a coupled axial force and biaxial bending moment hinges.

- Definition of the control node: control node is the node used to monitor displacements of the structure. Its displacement versus the base-shear forms the capacity (pushover) curve of the structure.

- Developing the pushover curve which includes the evaluation of the force distributions. To have a displacement similar or close to the actual displacement due to earthquake, it is important to consider a force displacement equivalent to the expected distribution of the inertial forces. Different forces distributions can be used to represent the earthquake load intensity.
- Estimation of the displacement demand: this is a crucial step when using pushover analysis. The control is pushed to reach the demand displacement which represents the maximum expected displacement resulting from the earthquake intensity under consideration.
- Evaluation of the performance level: performance evaluation is the main objective of a performance based design. A component or action is considered satisfactory if it meets a prescribed performance.

The main output of a pushover analysis is in terms of response demand versus capacity. If the demand curve intersects the capacity envelope near the elastic range, Figure 1a, then the structure has a good resistance. If the demand curve intersects the capacity curve with little reserve of strength and deformation capacity, Figure 1b, then it can be concluded that the structure will behave poorly during the imposed seismic excitation and need to be retrofitted to avoid future major damage or collapse.

3. Description of the Frame Structures

Three structures representing low, medium and high rise reinforced concrete framed buildings are considered in this study. These structures are designed according to the Algerian code RPA2003 and are located in high seismicity region with a peak ground acceleration of 0.32g. It is worth noting that in RPA2003 [8], the number of stories in high seismicity areas is limited to 2 but for study purposes, we have chosen structures with 5, 8 and 12 stories. Material properties are assumed to be 25 MPa for the concrete compressive strength and 400 MPa for the yield strength of the longitudinal and transverse reinforcement.

The three buildings are 24 m by 12 m in plan, Figure 1. Typical floor to floor height is 3.06m. The dimensions of the beams and columns for the three reinforced concrete frames are shown in Figure 1.a and the reinforcement details for the beams and columns are shown in Table 1.

The dynamic characteristics of the three buildings are symmetrical in the x and y directions respectively as shown in Table 2. Since the mass participating factor in the first mode is approximately equal to 70% which means that the dynamic response will be dominated by the first mode, it is expected that the pushover analysis will yield realistic results.
Table 1. Reinforcement details

<table>
<thead>
<tr>
<th>Building</th>
<th>Beams cm×cm</th>
<th>Level</th>
<th>Columns cm×cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 story</td>
<td>30×50</td>
<td>1-3</td>
<td>50×50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4-5</td>
<td>40×40</td>
</tr>
<tr>
<td>8 story</td>
<td>30×50</td>
<td>1-5</td>
<td>50×50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6-8</td>
<td>40×40</td>
</tr>
<tr>
<td>12 story</td>
<td>30×50</td>
<td>1-7</td>
<td>70×70</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8-9</td>
<td>50×50</td>
</tr>
</tbody>
</table>

Table 2. Dynamic characteristics of the buildings

<table>
<thead>
<tr>
<th>Building</th>
<th>Period (s)</th>
<th>Mass participating factor X</th>
<th>Mass participating factor Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>5 Story</td>
<td>1 mode</td>
<td>0.82</td>
<td>0.785</td>
</tr>
<tr>
<td></td>
<td>2 mode</td>
<td>0.82</td>
<td>0.0109</td>
</tr>
<tr>
<td></td>
<td>3 mode</td>
<td>0.271</td>
<td>0.0468</td>
</tr>
<tr>
<td></td>
<td>4 mode</td>
<td>0.271</td>
<td>0.0707</td>
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<tr>
<td></td>
<td>1 mode</td>
<td>1.117</td>
<td>0.69</td>
</tr>
<tr>
<td></td>
<td>2 mode</td>
<td>1.117</td>
<td>0.10</td>
</tr>
<tr>
<td>8 Story</td>
<td>3 mode</td>
<td>0.394</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>4 mode</td>
<td>0.394</td>
<td>0.02</td>
</tr>
<tr>
<td></td>
<td>1 mode</td>
<td>1.51</td>
<td>0.77</td>
</tr>
<tr>
<td></td>
<td>2 mode</td>
<td>1.51</td>
<td>0.001</td>
</tr>
<tr>
<td>12 Story</td>
<td>3 mode</td>
<td>0.51</td>
<td>0.09</td>
</tr>
<tr>
<td></td>
<td>4 mode</td>
<td>0.51</td>
<td>0.10</td>
</tr>
</tbody>
</table>
4. Modeling Approach

The general finite element package SAP 2000 has been used for the analyses. A three dimensional model of each structure has been created to undertake the non linear analysis. Beams and columns are modeled as nonlinear frame elements with lumped plasticity at the start and the end of each element. SAP 2000 provides default-hinge properties and recommends PMM hinges for columns and M3 hinges for beams as described in FEMA-356.

5. Pushover Analysis

After designing and detailing the reinforced concrete frame structures, a nonlinear pushover analysis is carried out for evaluating the structural seismic response. The pushover analysis consists of the application of gravity loads and a representative lateral load pattern. The lateral loads were applied monotonically in a step-by-step nonlinear static analysis. The applied lateral loads were accelerations in the x direction representing the forces that would be experienced by the structures when subjected to ground shaking. Under incrementally increasing loads some elements may yield sequentially. Consequently, at each event, the structures experiences a stiffness change as shown in Figure 3, where IO, LS and CP stand for immediate occupancy, life safety and collapse prevention respectively.

6. Results and Discussions

6.1 Capacity curve

The three resulting capacity curves for the three buildings are shown in Figure 4. The three curves show similar features. They are initially linear but start to deviate from linearity as the beams and the columns undergo inelastic actions. When the buildings are pushed well into the inelastic range, the curves become linear again but with a smaller slope. The three curves could be approximated by a bilinear relationship. At target displacement of 0.28 m for the 5 story building, the base shear of the whole structure was 9835 KN equivalent to 1.72 times that of
the structure under elastic seismic design. For the 8 story building, the base shear was 11198 KN for a target displacement of 0.40 m which represents 1.53 times the elastic base shear and for the 12 story building, the base shear is 13367 KN for a target displacement of 0.50 m and represents 1.50 times the elastic base shear. The three curves show no decrease in the load carrying capacity of the buildings suggesting good structural behaviour.

![Capacity curve](image)

**Figure 4. Capacity curve**

From figure 5 it is obvious that the demand curve tend to intersect the capacity curve near the event point B, which means an elastic response and the security margin is greatly enhanced. Therefore, it can be concluded that the margin safety against collapse is high and there are sufficient strength and displacement reserves.

![Graphs](image)

a) b)
6.2 Plastic hinges mechanism

Plastic hinge formation for the three building mechanisms have been obtained at different displacements levels. The hinging patterns are plotted in figures 6, 7 and 8. Comparison of the figures 6, 7 and 8 reveals that the patterns for the three building are quite similar. Plastic hinges formation starts with beam ends and base columns of lower stories, then propagates to upper stories and continue with yielding of interior intermediate columns in the upper stories. But since yielding occurs at events B, IO and LS respectively, the amount of damage in the three buildings will be limited.
7. Conclusions

The performance of reinforced concrete frames was investigated using the pushover analysis. These are the conclusions drawn from the analyses:

- The pushover analysis is a relatively simple way to explore the non linear behaviour of buildings.

- The behaviour of properly detailed reinforced concrete frame building is adequate as indicated by the intersection of the demand and capacity curves and the distribution of hinges in the beams and the columns. Most of the hinges developed in the beams and few in the columns but with limited damage.

- The causes of failure of reinforced concrete during the Boumerdes earthquake may be attributed to the quality of the materials of the used and also to the fact that most of buildings constructed in Algeria are of strong beam and weak column type and not to the intrinsic behaviour of framed structures.

- The results obtained in terms of demand, capacity and plastic hinges gave an insight into the real behaviour of structures.
It would be desirable to study more cases before reaching definite conclusions about the behaviour of reinforced concrete frame buildings.

References

7. RPA 2003, Règles Para sismiques Algériennes, 2003, DTR.