Displacement Based Design, (DBD), Nonlinear Static Pushover Analysis To Verify The Proper Collapse Mechanism Of Structures

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ABSTRACT
Under the pressure of recent developments, seismic codes have begun to explicitly require the identification of sources of inelasticity in structural response, together with the quantification of their energy absorption capacity. In the pushover procedure, a static lateral load, which is distributed approximately equivalent to seismic loads generated by an earthquake, is applied to the structure, which is then displaced (pushed over) incrementally to the level of deformation expected during the earthquake (target displacement) while keeping the applied load distribution pattern. Base shear and corresponding displacement at each stage are used to build the pushover curve, following which the seismic structural deformations and the performance level of the structure are estimated. The nonlinear load-deformation characteristics of individual components and elements of the structure are considered in the model to account for the possibility of exceeding elastic limits.

INTRODUCTION
An earthquake, which is a sudden and rapid shaking of the earth caused primarily by plate tectonics, is one of the most devastating natural hazards that cause great loss of life and property. More than 10,000 people perish each year due to earthquakes and the economic losses estimated for the period 1929-1950 are in excess of $10 billion. (A.S.Elnashi).

In the past few years, the earthquake engineering community has been reassessing its procedures, in the wake of two most damaging earthquakes which caused extensive damage, loss of life and property (Northridge, California, 17 January 1994; $20 billion and 34 dead; Hyogo-ken Nanbu, Japan, $150 billion and 6000 dead). Taking into account the short duration of earthquakes (averaging about 10-30 seconds), the amount of energy released per second must be very large compared to other forms of natural hazards. The recent earthquake-resistant design philosophies aim at producing structures that can withstand a certain level of ground shaking without excessive damage. And the last earthquakes in Pakistan, the latest reports indicate that the confirmed number of people killed in all areas hit by the earthquake is 55,000. It is estimated that that figure may rise to at least 79,000. Over 81,000 people have been injured. Extensive damage has occurred throughout Kashmir and other northern areas. Numerous towns have been severely affected and some villages completely destroyed. An estimated 2.8 million people have been made homeless. This earthquake has had an impact on Pakistan, northern India and parts of Afghanistan.

Generally, four distinct analytical procedures can be used for systematic rehabilitation of structures (FEMA-273, 1997): Linear Static, Linear Dynamic, Nonlinear Static (Pushover) and Nonlinear Dynamic Procedures (NDP). Linear-elastic procedures (linear static and linear dynamic) are the most common procedures in seismic analysis and design of structures due to their simplicity. Such procedures are efficient so long as the structure behaves within elastic limits. If the structure responds beyond the elastic limit, linear analyses may indicate the location of first yielding, but cannot predict failure mechanisms and account for redistribution of forces during progressive yielding. On the other hand, Nonlinear (static and dynamic) procedures are the solutions that can overcome this problem and show the performance level of structures at any loading level. These procedures help demonstrate how structures work by identifying modes of failure and the potential for progressive collapse.
Nonlinear procedures help engineers to understand how a structure will behave when subjected to major earthquakes. The four procedures are described in more detail in the next sections.

1. **General Procedure to Perform Pushover Analysis**

1.1 An elastic structural model is developed that includes all new and old components that have significant contributions to the weight, strength, stiffness, and/or stability of the structure and whose behavior is important in satisfying the desired level of seismic performance. The structure is loaded with gravity loads in the same load combination(s) as used in the linear procedures before proceeding with the application of lateral loads.

1.2 The structure is subjected to a set of lateral loads, using one of the load patterns (distributions) described in the (ATC-96). At least two analyses with different load patterns should be performed in each principal direction (ATC-96).

1.3 The intensity of the lateral load is increased until the weakest component reaches a deformation at which its stiffness changes significantly (usually the yield load or member strength). The stiffness properties of this “yielded” component in the structural model are modified to reflect post-yield behavior, and the modified structure is subjected to an increase in lateral loads (load control) or displacements (displacement control), using the same shape of the lateral load distribution or an updated shape as permitted in the ATC. Modification of component behavior may be in one of the following forms:

1.3.1 Placing a hinge where a flexural element has reached its bending strength; this may be at the end of a beam, column, or base of a shear wall.

1.3.2 Eliminating the shear stiffness of a shear wall that has reached its shear strength in a particular storey.

1.3.3 Eliminating a bracing element that has buckled and whose post-buckling strength decreases at a rapid rate.

1.3.4 Modifying stiffness properties if an element is capable of carrying more loads with a reduced stiffness

2 Step 3 is repeated as more and more components reach their strength. Note that although the intensity of loading is gradually increasing, the load pattern usually remains the same for all stages of the “yielded” structure, unless the user decides on the application of an adaptive load pattern (Bracci et al., 1995). At each stage, internal forces and elastic and plastic deformations of all components are calculated.

3 The forces and deformations from all previous loading stages are accumulated to obtain the total forces and deformations (elastic and plastic) of all components at all loading stages.

4 The loading process is continued until unacceptable performance is detected or a roof displacement is obtained that is larger than the maximum displacement expected in the design earthquake at the control node.

5 Note: Steps 3 through 6 can be performed systematically with a nonlinear computer analysis program using an event-by-event strategy or an incremental analysis with predetermined displacement increments in which iterations are performed to balance internal forces.
The displacement of the control node versus base shear at various loading stages is plotted as a representative nonlinear response diagram of the structure. The changes in slope of this curve are indicative of the yielding of various components.

The control node displacement versus base shear curve is used to estimate the target displacement. Note that this step may require iteration if the yield strength and stiffness of the simplified bilinear relation are sensitive to the target displacement.

Once the target displacement is known, the accumulated forces and deformations at this displacement of the control node are used to evaluate the performance of components and elements of the structure.

10.1 For deformation-controlled actions (e.g., flexure in beams), the deformation demands are compared with the maximum permissible values.
10.2 For force-controlled actions (e.g., shear in beams), the strength capacity is compared with the force demand.

If either (a) the force demand in force-controlled actions, components, or elements, or (b) the deformation demand in deformation-controlled actions, components, or elements, exceeds permissible values, then the action, component, or element is deemed to violate the performance criteria. Asymmetry of a building in the direction of lateral loading will affect the force and deformation demands in individual components. Asymmetric elements and components in a building, such as reinforced concrete shear walls with T- or L-shaped cross section, have force and deformation capacities that may vary substantially for loading in opposite directions. Accordingly, it is necessary to perform two nonlinear analyses along each axis of the building with loads applied in the positive and negative directions, unless the building is symmetric in the direction of lateral loads or the effects of asymmetry can be evaluated with confidence through judgments or auxiliary calculations.

As noted in Step 1 of the NSP, gravity loads need to be applied as initial conditions to the nonlinear procedure, and need to be maintained throughout the analysis. This is because superposition rules applicable to linear procedures do not, in general, apply to nonlinear procedures, and because the gravity loads may importantly influence the development of nonlinear response. The gravity-load combinations are the same as in the linear procedures. As noted previously, the use of more than one gravity-load combination will greatly increase the analysis effort in the NSP. It may be possible by inspection to determine that one of the two specified combinations will not be critical.

The mathematical model should be developed to be capable of identifying nonlinear action that may occur either at the component ends or along the length of the component. For example, a beam may develop a flexural plastic hinge along the span (rather than at the ends only), especially if the spans are long or the gravity loads are relatively high. In such cases, nodes should be inserted in the span of the beam to capture possible flexural yielding between the ends of the beam.

The general concepts of the Nonlinear Static Procedure method are summarized in Figure 1.

Simplified Nonlinear
The **Capacity Spectrum** methods, A, B & C reduce the elastic spectrum to intersect the capacity curve in spectral coordinates to find the performance point. The equal displacement point is a good starting point for the iterative process.

The ** Equal Displacement Approximation** assumes that the inelastic displacement is the same as that which would occur if the structure remained completely elastic.

The **Displacement Coefficient** method modifies $\delta_{\text{elastic}}$ with coefficients to calculate a target displacement, $\delta_t$.

Using the performance point or target displacement, the global response of the structure and individual component deformations are compared to limits in light of the specific performance goals for the building.
4. Research Methodology

A three dimensional eight-story building with a total height of 30.4m was modeled. As mentioned in the previous section, the structural system of the building consists of reinforced concrete ordinary moment resisting frames in both direction and shear walls only in Y direction. See Figure 1-a through 1-d.

- The service dead and live loads on the slabs in kN/m2 were assumed as follows: service dead load = 10kN/m2 and service live load = 3kN/m2.
- The seismic analysis for the assumed 3D model was constructed using four different approaches as follows:
  - Static Force Procedure: as recommended in the UBC-97 Code.
  - Response Spectrum Method using the UBC-97 design response spectrum.
  - Time History Analysis using the El-Centro earthquake record.
- Pushover Method.

The analysis was performed using the SAP2000 software for the four methods. Analysis results including fundamental period, base shear, displacement and rotation for the assumed building were compared using the four methods.

### Table 2. Horizontal distribution of base shear force

<table>
<thead>
<tr>
<th>Floor #</th>
<th>Wi (kN)</th>
<th>hi (m)</th>
<th>Wihi (kN.m)</th>
<th>Fx (kN)</th>
<th>Fx + Ft (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8</td>
<td>2550</td>
<td>30.4</td>
<td>77520</td>
<td>216.8</td>
<td>309.8</td>
</tr>
<tr>
<td>7</td>
<td>3640</td>
<td>26.6</td>
<td>96824</td>
<td>270.8</td>
<td>270.8</td>
</tr>
<tr>
<td>6</td>
<td>3640</td>
<td>22.8</td>
<td>82992</td>
<td>232.1</td>
<td>232.1</td>
</tr>
<tr>
<td>5</td>
<td>3640</td>
<td>19</td>
<td>69160</td>
<td>193.4</td>
<td>193.4</td>
</tr>
<tr>
<td>4</td>
<td>3640</td>
<td>15.2</td>
<td>55328</td>
<td>154.7</td>
<td>154.7</td>
</tr>
<tr>
<td>3</td>
<td>3640</td>
<td>11.4</td>
<td>41496</td>
<td>116.1</td>
<td>116.1</td>
</tr>
<tr>
<td>2</td>
<td>3640</td>
<td>7.6</td>
<td>27664</td>
<td>77.4</td>
<td>77.4</td>
</tr>
<tr>
<td>1</td>
<td>3640</td>
<td>3.8</td>
<td>13832</td>
<td>38.7</td>
<td>38.7</td>
</tr>
<tr>
<td>Total</td>
<td>28030</td>
<td>464816</td>
<td></td>
<td>1390</td>
<td></td>
</tr>
</tbody>
</table>

### Table 3. Displacement and rotation of joints 1 through 9 of the exterior column at support # 1, static force method

<table>
<thead>
<tr>
<th>Joint #</th>
<th>Ux (cm)</th>
<th>Uy (cm)</th>
<th>Uz (cm)</th>
<th>Rx (rad)</th>
<th>Ry (rad)</th>
<th>Rz (rad)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>0.367</td>
<td>0.019</td>
<td>-0.024</td>
<td>-1.86*10^-4</td>
<td>1.39*10^-3</td>
<td>-1.72*10^-5</td>
</tr>
<tr>
<td>3</td>
<td>1.111</td>
<td>0.058</td>
<td>-0.049</td>
<td>-2.37*10^-4</td>
<td>1.91*10^-3</td>
<td>-5.30*10^-5</td>
</tr>
<tr>
<td>4</td>
<td>2.044</td>
<td>0.106</td>
<td>-0.073</td>
<td>-2.94*10^-4</td>
<td>2.17*10^-3</td>
<td>-9.83*10^-5</td>
</tr>
<tr>
<td>5</td>
<td>3.013</td>
<td>0.158</td>
<td>-0.096</td>
<td>-3.41*10^-4</td>
<td>2.01*10^-3</td>
<td>-1.47*10^-4</td>
</tr>
<tr>
<td>6</td>
<td>3.932</td>
<td>0.207</td>
<td>-0.122</td>
<td>-4.47*10^-4</td>
<td>1.74*10^-3</td>
<td>-1.94*10^-4</td>
</tr>
<tr>
<td>7</td>
<td>4.849</td>
<td>0.254</td>
<td>-0.147</td>
<td>-6.51*10^-4</td>
<td>1.77*10^-3</td>
<td>-2.41*10^-4</td>
</tr>
<tr>
<td>8</td>
<td>5.643</td>
<td>0.294</td>
<td>-0.167</td>
<td>-7.80*10^-4</td>
<td>1.57*10^-3</td>
<td>-2.82*10^-4</td>
</tr>
<tr>
<td>9</td>
<td>6.288</td>
<td>0.324</td>
<td>-0.178</td>
<td>-1.55*10^-3</td>
<td>1.51*10^-3</td>
<td>-3.15*10^-4</td>
</tr>
</tbody>
</table>
6. Response Spectrum Analysis

7. Time History Analysis

Time history analysis is the general method used for large and complex structures, and is conducted by using numerical methods. Since ground motion records are needed for this type of analysis, the code requires that at least three pairs of records be used. These records shall reflect site characteristics and seismic hazard. These records can either be scaled from actual records, or, artificially generated (synthetic records).
7.1 Results of the Time History Analysis

Figure 6. History of displacements of joints 1-9 at support #1

Figure 7. Maximum absolute displacement of joints 1-9 at support #1 in x direction
8. Nonlinear Static Pushover Analysis

The ATC-40 and FEMA-273 documents have developed modeling procedures, acceptance criteria and analysis procedures for the pushover analysis. These documents define a force-deformation criteria for plastic hinges used in pushover analysis. As shown in Figure 21, five points labeled A, B, C, D and E are used to define the force-deflection behavior of a hinge and three points labeled IO, LS, and CP are used to define the acceptance criteria for the hinge. IO, LS and CP stand for Immediate Occupancy, Life Safety and Collapse Prevention, respectively. The values assigned to each of these points vary depending on the type of member as well as many other parameters defined in the ATC-40 and FEMA-273 documents.

![Figure 8. Generalized force-displacement relation](image)

This section presents the steps used in performing a pushover analysis of a simple three-dimensional building using SAP2000 ver 7.4 program. Steps 1 through 6 review the pushover analysis method.

8.1 Results of the Pushover Method

Using the SAP2000 software, the pushover method was carried out for the 8-story three dimensional model assumed in the study. The program displays the pushover and capacity spectrum curves where their intersection defines the performance point. Table 17 gives the results of the pushover method where for each step a point on the pushover curve of base shear vs. displacement is defined and the total number of plastic hinges in each step and the distribution of this number between performance levels are also listed.

<table>
<thead>
<tr>
<th>Step #</th>
<th>Displacement (cm)</th>
<th>Base Shear (kN)</th>
<th>A-B</th>
<th>B-IO</th>
<th>IO-LS</th>
<th>LS-CP</th>
<th>CP-C</th>
<th>C-D</th>
<th>D-E</th>
<th>&gt;E</th>
<th>Total</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-0.005</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1696</td>
</tr>
<tr>
<td>1</td>
<td>1.28</td>
<td>269.13</td>
<td>1695</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1695</td>
</tr>
<tr>
<td>2</td>
<td>3.33</td>
<td>669.91</td>
<td>1597</td>
<td>99</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1696</td>
</tr>
<tr>
<td>3</td>
<td>5.39</td>
<td>931.14</td>
<td>1449</td>
<td>247</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1696</td>
</tr>
<tr>
<td>4</td>
<td>7.39</td>
<td>1064.1</td>
<td>1304</td>
<td>392</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1696</td>
</tr>
</tbody>
</table>
The capacity curve is shown in Figure 9. Note that the performance point occurred at step number 7 at a displacement of 13.53 cm and a base shear of 1249 kN.

Referring to Table 4, the table shows that for each step of pushing the number of plastic hinges that occurred in members increases for each performance level till total collapse of structure. This can also be observed visually in the deformed shapes of Figure 10.

In addition, deformation limits can be checked at the performance point level as follows:

Total displacement at the performance point = 135.3 mm

Total height of structure = 30.4 m = 30400 mm

Ratio of performance point displacement / total height = 1353/30400 = 0.005

Referring to Table 5, drift limitations are met for the immediate occupancy performance level.

Table 5. Deformation Limits (ATC-96)

<table>
<thead>
<tr>
<th>Performance level</th>
<th>Immediate Occupancy</th>
<th>Damage Control</th>
<th>Life Safety</th>
<th>Structural Stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interstory Drift Limit</td>
<td>^h</td>
<td>0.01</td>
<td>0.01-0.02</td>
<td>0.02</td>
</tr>
<tr>
<td>Maximum Total drift $h_i$</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Where $V_i$ is the total calculated lateral shear force in story $i$ and $P_i$ is the total gravity load (i.e. dead plus likely live load) at story $i$. 

| Maximum inelastic drift | 0.005 | 0.005-0.015 | No limit | No limit |

Figure 10. Deformed shape at step # 16
No. of hinges =158 B, 222 LS, 0 CP, 1 D, 1 E
Displacements (266.9 mm)
Discussion of Results

The nonlinear static procedure is intended to provide a simplified approach for directly determining the nonlinear response behavior of a structure at different levels of lateral displacements, ranging from initial elastic response through development of a failure mechanism and initiation of collapse. Response behavior is gauged by a measurement of the strength of the structure at various increments of lateral displacement.

Generally, if a structure is subjected to lateral loads larger than those that represented by the elastic strength, a number of elements will yield, eventually forming a mechanism. Standard methods of plastic or limit analysis can be used to determine the strength corresponding to such mechanism. If after the structure develops a mechanism, it deforms an additional substantial amount, elements within the structure may fail and thus cease to contribute strength to the structural system. In such cases, the strength of the structure will diminish with increasing deformation. Figure 11, which is a plot of the lateral structural strength vs. deformation (or pushover curve) for a hypothetical structure, illustrates these concepts.

Figure 11. Strength–deformation relation for a frame structure(www.bsscoline.org)

As shown in the figure, many structures exhibit a range of behavior between the development of first yielding and development of a mechanism. When the structure deforms while elements are yielding (shown as progressive yielding), the relation between external forces and deformations cannot be determined by simple limit analysis. For such a case, other methods of analysis are required. The purpose of nonlinear static procedure
is to provide a simplified method of determining the structural response behavior at deformation levels between those that cannot be conveniently analyzed using limit state methods.

Figure 12. Fundamental Period, in seconds

Figure 13. Base shear in x-direction
10. CONCLUSIONS

(1). Nonlinear static pushover analysis has served well as an efficient and easy-to-use alternative to dynamic time-history analysis, since, despite its simplicity, it is capable of providing important structural response information. Indeed, pushover can be employed to identify critical regions, where inelastic deformations are expected to be high, and strength irregularities in plan or elevation that might cause important changes in the inelastic dynamic response.

(2). This type of analysis is also capable of predicting the sequence of yielding and/or failure of structural components and the progress of the overall capacity curve of the structure, thus verifying the adequacy of the seismic load.

(3). When a structure deforms while elements are yielding (known as progressive yielding), the relation between external forces and deformations cannot be determined by a simple limit analysis. For such a case, other methods of analysis are required. The purpose of nonlinear static procedure is to provide a simplified method of determining the structural response behavior at deformation levels between those that cannot be conveniently analyzed using limit state methods.

Nonlinear static procedure can be used efficiently to evaluate the performance level of reinforced concrete buildings subjected to seismic loading.

12. Recommendations

1. Nonlinear static pushover analysis is recommended as an efficient and easy-to-use alternative to dynamic time-history analysis due to its simplicity and capability of predicting the sequence of yielding and/or failure of structural components and evaluating the performance of reinforced concrete buildings.

Figure 14. Maximum Displacement in x-direction
2. Nonlinear static procedures are especially recommended for analysis of buildings with irregularities.
3. Pushover method should not be used for structures in which higher mode effects are significant unless a LDP evaluation is also performed to capture the effect of higher modes.

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Sincere thanks and appreciation to Dr. Hamed Allayed.

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