Assessment of Nonlinear Static (Pushover) Procedures Using Time-History Direct Integration Analysis

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Abstract: The seismic performance of structures subjected to earthquake always becomes critical issues. The principal objective of this study is the assessment of nonlinear static procedures using nonlinear time-history analysis. Pushover procedures in this research consist of Coefficient Method (CM), Capacity Spectrum Method (CSM) and Modal Pushover Analysis (MPA), as the most commonly used methods. In this context 5, 8 and 12-story frames were selected to represent the low, medium and high rise regular reinforcement concrete structures. The results revealed that the MPA method and CM can be used for analysis of structure, while the results obtained by CSM were far from realistic behaviour of structure under earthquake.

Key words: Seismic Performance, Coefficient Method, Capacity Spectrum Method, Modal Pushover Analysis.

INTRODUCTION

Earthquake is one of the most serious natural disasters known by human beings. Although analyses of ruins due to earthquake have showed that new structural analysis methods used in buildings to preserve and minimize human life, economic losses due to property damage and disruption on business and companies might be very large. Hence, the seismic performance of structures subjected to earthquake always becomes critical issues.

Behaviour of structure in earthquake has shown that many buildings cannot tolerate the earthquake forces, even some of the buildings that was designed based on proposed liner static analysis by codes. Since, the dynamic methods were very complicated and time-consuming; thus, nonlinear static (pushover) method has made great progress among other methods. Consequently, recently an issue regarding the using of pushover analysis method had arisen against nonlinear dynamic method. This is due to better accuracy and simplicity when compared with linear static and dynamic procedures, respectively (Gencturk and Elnashai 2008).

Foundation of nonlinear static analysis (Pushover Analysis) is often attributed to the work of Takeda et al. (1970), Freeman et al. (1975) and later Saiidi and Sozen (1981) proposed an approach wherein the response of a Multi-Degree-of-Freedom (MDOF) system was determined from dynamic response analysis of an equivalent Single-Degree-of-Freedom (SDOF) system. A SDOF system is defined as that in which only one type of motion is possible, or in other words the location of the system at any instant of time can be defined in terms of single coordinate (Sen 2009). Therefore, the proposed simplified nonlinear analysis procedures and structural models are usually based on the reduction of MDOF model of structures to an equivalent SDOF system (Chopra and Goel 2002). Many analysis methods for nonlinear static category were developed over recent century described in the next section.

History of Pushover Analysis:

A list of studies on pushover analysis is presented below:

1. Fajfar and Fischinger (1987) proposed the N2 method as a simple nonlinear procedure applicable to the logical design of reasonable regular structure oscillating mainly in a single mode. Its basic variant has been implemented in Eurocode 8 (CEN 2005). An extension of the N2 method proposed important higher mode effects in plan and along the elevation. New version based on the supposition that the structure remainders in the elastic range when vibrating in higher modes, (Kreslin and Fajfar 2011, 2012).

2. The capacity spectrum method (CSM), adopted in ATC-40 (ATC 1996), was first introduced in the 1970s by Freeman as a fast evaluation procedure for evaluating the seismic vulnerability of buildings (Freeman et al. 1975; Freeman 1978). This procedure compares the capacity of the structure in the form of a pushover curve with demands of earthquake ground motion on the structure in the form of an elastic response spectrum.

3. Mahaney et al. (1993) proposed the Acceleration-Displacement Response Spectrum (ADRS) format, that spectral accelerations are plotted against spectral displacements, with the periods T represented by radial lines. The crossing of the capacity spectrum and the demand spectrum provides an estimation of the displacement demand and inelastic acceleration (Strength) (Fajfar 1999). Capacity spectrum method was later updated in FEMA-440 (2006).

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4. One of the famed procedures in class of pushover analysis is Coefficient Method (CM) that was first introduced in Federal Emergency Management Agency (FEMA) of the U.S.A (FEMA-273 1997), and was further developed and published as a pre-standard for seismic rehabilitation of buildings in FEMA-356 (2000). The method was later updated in FEMA-440 (2006). The displacement demand of the method is determined from the elastic one by using a series of modification factors based on statistical analyses.

5. Paret et al. (1996) and Sasaki et al. (1998) proposed Multi-Modal Pushover (MMP) procedure to identify failure mechanisms due to higher modes for structures with significant higher-order modal response. The procedure is intuitive and in fact provided qualitative information on higher mode effects, which conventional single mode pushover analysis, fails to highlight. Nevertheless, that is not easily to quantify of the effects of these higher modes, since the method does not provide an assessment of the response (Antoniou and Pinho 2004). Later, Moghadam and Tso (2002) proposed Pushover Result Combination (PRC) method that was an improvement of the multi-modal pushover procedure. In this method estimation of the maximum seismic response was derived from combining the results of several pushover analyses, which are carried out by a predefined mode shape as a load pattern.


7. Hernandez-Montes et al. (2004) considered the energy-based (energy absorbed or the work done) formulation in the pushover analysis procedure. The proposed method to establish the capacity curve used a target-displacement consequent from the work done by the lateral loads instead of using the roof displacement. The work done by the lateral loads related with each mode is calculated using an incremental formulation in each step of the pushover procedure. More recently Manoukas et al. (2011) proposed a new energy-based pushover method in order to obtain an approximate estimate of structural performance under strong earthquakes.

8. Kalkan and Kunnath (2006) proposed Adaptive Modal Combination (AMC) procedure in which a set of adaptive mode-shape based inertia load patterns is exerted on structure. More recently Shakeri et al. (2010) proposed advanced adaptive pushover method, called (SSAP) that is based on the story shears which takes into account the setback of sign in the higher modes.

As results of previous study, it can be concluded that many researchers have proposed various methods in the form of pushover analysis, which have shown different results. Furthermore, comparison of the proposed methods shows that the main differences between the methods were in the selection of lateral load distribution and in defining the dynamic properties of sections (Cardone 2007; Ferracuti et al. 2009; Goel 2008; Goel 2003, 2004; Goel and Chadwell 2007; Kalkan and Kunnath 2007; Kreslin and Fajfar 2011). Hence, it is seems that, there is a gap of knowledge in assessment of proposed pushover procedures by researchers.

Assessment of Pushover Procedure:

Pushover analyses of the models were performed to analysis the lateral base shear and roof displacement relationship of the structure. The pushover curve for structures is including an initial linear branch in which the structural members deform in their elastic range and then followed by a yield region where the beam or column members start developing plastic hinges. As lateral deformation continues, extra plastic deformation happens, and the post-yield strength of the structure softens. This research is an assessment of tree pushover analysis procedure using nonlinear time-history analysis which is described in the following sections.

Capacity Spectrum Method (CSM):

CSM provides a specially treatment of the reduction of earthquake demand for maximum displacement of existing reinforcement structures. The capacity spectrum method is a very useful instrument in the assessment and retrofit design of nonlinear SDOF system which provides a graphical depiction of the global force-displacement capacity curve of the structure (i.e., pushover) compared to the response spectra depictions of the seismic demands. The target displacement (Performance Point) in the background of the CSM is obtained at the crossing of the elastic response spectrum and capacity curve.

To use the CSM it is required to convert the capacity curve and demand spectrum curve to ADRS format. In this method for converting the capacity curve to the ADRS format, we first calculate the modal mass coefficient \( \alpha_k \) and the modal participation factor \( \mu_k \) using equations 1 and 2. Then for each point on the capacity curve, \( \bar{V} \), \( \Delta_{\text{roof}} \) calculate the associated point \( S_{\Delta_k} \) on the capacity spectrum using equations 3 and 4:
Also to convert a demand spectrum from the standard format (Sa vs T) based on the building code to ADRS format, it is required to determine the value of for each point on the curve, \( S_a, T \). This can be done by equation 5:

\[
S_{\delta i} = \frac{T_i^2}{4\pi^2} S_{\alpha i} \delta
\]

**Coefficient Method (CM) of FEMA-356:**

In CM the maximum inelastic displacement of an MDOF system is obtained using modifying the elastic displacement of the equivalent SDOF with an effective period (Equation 6).

\[
T_e = T_i \sqrt{\frac{K}{K_e}}
\]

Where \( T_i \) is elastic fundamental period and \( K, K_e \) are the elastic lateral stiffness and effective lateral stiffness, respectively. In continuing calculated the target displacement (\( \delta_t \)) from equation 7.

\[
\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2}
\]

where;

- \( T_e \) = Effective fundamental period as calculated in equation 6.
- \( C_0 \) = Modification factor to relate spectral displacement.
- \( C_1 \) = Modification factor to relate anticipated maximum inelastic displacements to displacements obtained from linear elastic response.
- \( C_2 \) = Modification factor to represent the effect of hysteresis shape on the maximum displacement response.
- \( C_3 \) = Modification factor to represent increased displacements because of dynamic \( P - \Delta \) effects.

**Modal Pushover Analysis (MPA):**

MPA method has been established to account for the higher mode effects. This method is highly exact unless the structure is very deformed into the zone of stiffness and decline in strength. MPA is based on the supposition of the classical modal analysis concepts that extended to inelastic systems. Due to individual terms seismic responses are determined in the modal development of the effective earthquake forces by a pushover analysis. Calculation of the complete response of the building is obtained by combining these modal responses by SRSS modal combination method. Remembering the equation of a nonlinear system subjected to ground acceleration;

\[
M\ddot{u} + C\dot{u} + R(u) = -M\ddot{a}_g
\]

According to \( u = \phi, g \) and use of the orthogonally (vertically) of mode shapes with deference to M and C, the equation 9 obtained from motion in the \( n^{th} \) mode.
\[ \ddot{q} + 2 \zeta_n \omega_n \dot{q} + \frac{\omega_n^2 q}{M_n} = -\Gamma_n \ddot{a}_g \] (9)

In fact the phrase \( R(q) \) is a function of all the modal amplitudes \( \{q\} \) but in modal procedure it is supposed to be only a function of \( q_n \). Due to \( D_n = \frac{q_n}{\Gamma_n} \), equation 10 presented the final shape of the equation from motion for the \( n^{th} \)-mode SDOF.

\[ \ddot{D}_n + 2 \zeta_n \omega_n \dot{D}_n + F_m = -\ddot{a}_g \] (10)

Force-deformation relationship of the \( n^{th} \)-mode SDOF system presented by \( F_m \) and \( D_n \) is obtained from the pushover analysis under a lateral load corresponding to the mode shapes, namely, \( S_n^* = M \varphi_n \). Changing the plot of base shear \( (V_n) \) against roof displacement \( (U_n) \) provides the force-deformation relationship of the SDOF system (Equation 11 and 12).

\[ F_m = \frac{V_n}{2M\omega_n}, \] (11)

\[ D_n = \frac{U_n}{\omega_n}, \] (12)

where \( \varphi_{n, r} \) is introduced as the \( n^{th} \)-mode shape at the roof. The \( n^{th} \)-mode response of the MDOF system is got from the pushover analysis under \( S_n^* \) at a roof displacement. \( U_m = \Gamma_n \varphi_n D_n \), wherein \( D_n \) obtained from solution of equation 10.

Obtained a bilinear idealization of the \( F_m - D_n \) relationship from solving of equation 9, which is used and the term \( W_n^* \) for calculated at the yield point of the bilinear representation.

\[ W_n^* = \frac{\Gamma_{n, y}}{D_y} \] (13)

Following is a concise summary of this method.

1. Calculate the natural frequencies, \( \omega_n \) and modes, \( \varphi_n \), for linearly elastic vibration of the structure.
2. For each mode \( n \), expand the base shear \( (V_{n, \gamma}) \) roof displacement \( (U_m) \) and pushover curve, \( S_n^* = M \varphi_n \), where \( m \) is the mass matrix of the building. Also Gravity loads, including that current gravity on the interior frames, are applied before the MPA.
3. Obtain a bilinear curve of the capacity curve and adapt it to the force-displacement relation of the equivalent SDOF system using equation 11 and 12.
4. Calculate the peak deformation \( D_n \) of the \( n^{th} \)-mode inelastic (SDF) system.
5. Compute peak roof displacement \( U_{m, \gamma} \) related with the \( n^{th} \)-mode inelastic SDF system from \( U_m = \Gamma_n \varphi_n D_n \).
6. Calculate the dynamic response and contribution of gravity loads \( (r_{\gamma}) \) due to \( n^{th} \)-mode.

Obtain the demand (total response) by combining gravity response and the peak dynamic (modal) response by SRSS approach: \( r = \sqrt{\max[r_{\gamma}^2 + \sum (r_{\gamma}^n)^2]} \)

**Nonlinear Time-History Analysis:**

Nonlinear time-history analysis is a step by step analysis of the dynamical response of a building to an identified loading that might change with time. Nonlinear time-history analysis is used to determine the dynamic response of a structure to random loading. The dynamic stability equation is obtained via Equation 14.

\[ K \ddot{u}(t) + C \dot{u}(t) + M \ddot{u}(t) = r(t) \] (14)

where \( K \) is the stiffness matrix, \( C \) is the damping matrix, \( M \) is the diagonal mass matrix, \( r \) is the applied load and \( u, \dot{u}, \ddot{u} \) are the displacements, velocities, and accelerations of the structure, respectively.

The nonlinear response of buildings is very sensitive to the structural modelling and ground motion characteristics. Consequently, a set of representative ground motion records with different characteristics in distance, magnitude, frequency and duration should be used. The input motions used in this research is included the two horizontal components of 10 ground motion with different magnitude (ranging from 6.4 to 7.6) and different Pick Ground Acceleration (PGA) obtained from Pacific earthquake engineering research centre (PEER 2006). Furthermore, as show in Table 1 five of the ground motion has been selected as a far fault ground motion
recorded with more than fifteen km distance and five of the other selected ground motion recorded with less than fifteen km distance.

Table 1: Parameters of Ground Motion Used in Analysis.

<table>
<thead>
<tr>
<th>No</th>
<th>Type</th>
<th>Earthquake</th>
<th>Mw</th>
<th>Station Name</th>
<th>Distance (km)</th>
<th>Component</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>FF</td>
<td>Chi-Chi Taiwan 1999</td>
<td>7.6</td>
<td>CWB 99999</td>
<td>26</td>
<td>TCU045</td>
<td>0.47</td>
</tr>
<tr>
<td>2</td>
<td>FF</td>
<td>Northridge 1994</td>
<td>6.7</td>
<td>Castaic-Old Ridge Route</td>
<td>21</td>
<td>ORR360</td>
<td>0.48</td>
</tr>
<tr>
<td>3</td>
<td>FF</td>
<td>Imperial Valley 1979</td>
<td>6.5</td>
<td>Calipatria Fire Station</td>
<td>25</td>
<td>H-CAL225</td>
<td>0.10</td>
</tr>
<tr>
<td>4</td>
<td>FF</td>
<td>San Fernando 1971</td>
<td>6.6</td>
<td>Castaic-Old Ridge Route</td>
<td>22</td>
<td>ORR291</td>
<td>0.29</td>
</tr>
<tr>
<td>5</td>
<td>FF</td>
<td>Victoria Mexico 1980</td>
<td>6.3</td>
<td>SAHOF Casa Flores</td>
<td>39</td>
<td>CPI045</td>
<td>0.07</td>
</tr>
<tr>
<td>6</td>
<td>NF</td>
<td>Cape Mendocino 1992</td>
<td>7</td>
<td>Petrolia</td>
<td>4</td>
<td>PET090</td>
<td>0.62</td>
</tr>
<tr>
<td>7</td>
<td>NF</td>
<td>Erzican, Turkey 1992</td>
<td>6.7</td>
<td>Erzican</td>
<td>9</td>
<td>ERZ-EW</td>
<td>0.48</td>
</tr>
<tr>
<td>8</td>
<td>NF</td>
<td>Landers 1992</td>
<td>7.3</td>
<td>Joshua Tree</td>
<td>11</td>
<td>JOS000</td>
<td>0.24</td>
</tr>
<tr>
<td>9</td>
<td>NF</td>
<td>Kobe, Japan 1995</td>
<td>6.9</td>
<td>KJMA</td>
<td>0.96</td>
<td>KJMN000</td>
<td>0.7</td>
</tr>
<tr>
<td>10</td>
<td>NF</td>
<td>Tabas, Iran 1978</td>
<td>7.3</td>
<td>Tabas</td>
<td>2</td>
<td>TAB-TR</td>
<td>0.81</td>
</tr>
</tbody>
</table>

1. FF: Far-Fault; NF: Near-Fault.

Description of Case Study Modelling:
This study proposed three reinforced concrete frames with different heights (5, 8 and 12 stories) which used to cover a broad range of fundamental periods, as shown in Figure 1. The presented frames were designed by using ACI 318-05 (2005), then defined as a two dimensional frames using SAP2000 version 14 software (CSI 2010). The structural models were based on centreline dimensions that assigned columns and beams among the nodes. Rigid floor diaphragm was assigned at each story level also, in the mass centre of each story the seismic mass of the frames were lumped. Proposed gravity loads in pushover analysis and time-history analysis are including dead loads and 25 percent of live loads that were assigned in all frames. Furthermore, P-Δ effect for both pushover and time-history analysis is not taking to account. Free vibration analyses were carried out to obtain mode shapes and elastic periods of the frames. The summarized of dynamic properties are presented in Table 2.

Fig. 1: Case Study Reinforcement Concrete Frames.

Table 2: Dynamic Properties of Case Study Frames.

<table>
<thead>
<tr>
<th>Frame</th>
<th>Period (T)</th>
<th>Modal Mass Factor (α)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>T₁</td>
<td>T₂</td>
</tr>
<tr>
<td>5-Story RC</td>
<td>0.956</td>
<td>0.302</td>
</tr>
<tr>
<td>8-Story RC</td>
<td>1.195</td>
<td>0.424</td>
</tr>
<tr>
<td>12-Story RC</td>
<td>1.690</td>
<td>0.605</td>
</tr>
</tbody>
</table>

SAP2000 (CSI, 2010) can be inserted plastic hinges at every location along the clear length of every frame element object. Every hinge represents concentrated post-yield conduct in single or multi degrees of freedom. Hinges only have an effect on the behaviour of the structure in pushover analysis and nonlinear time history
(direct-integration) analyses. Uncoupled moment (M2 and M3), axial force (P), torsion (T) and shear hinges (V2 and V3) can be assigned in element ends as well as at any number of locations along the span of the frame element. Furthermore, can be assigned a coupled P-M2-M3 (PMM) hinge that yield based on the interaction of bending moments and axial force at the hinge location. In this study, PMM and M3 are assigned to the columns and beams, respectively.

Three types of hinge properties defined in SAP2000 consisting of user-defined hinge properties, default hinge properties based on FEMA-356 and generated hinge properties. It should be noted that only user-defined hinge properties and default hinge properties can be assigned to frame elements. In this research default hinge properties assigned to frame elements for both pushover and time-history analysis.

RESULTS AND DISCUSSION

For comparison methods, this section presents the results of CM, CSM, MPA and nonlinear time-history method. Then, compute target displacements of each selected method to idealized capacity curves.

Coefficient Method (CM):
FEMA-356 (2000) guidelines proposed the lateral loads based on the dynamic properties of each structure. According to Goel (Goel 2003, 2004), this research using Equivalent Lateral Force (ELF) as a lateral load for all frames. ELF distribution proposed by Eurocode 8 (CEN, 2005) is the force at any story which is dependent to the height of story and mass in that story:

\[
F_i = \frac{m_{bi} h_i^k}{\sum m_{bi} h_i^k}
\]

where \(h_i\) is the height of the i-th story above the base and \(k\) is equal to 1 for main period \(T_1 \leq 0.5 \, \text{sec}\), and \(k\) is equal to 2 for main period \(T_1 \geq 2.5 \, \text{sec}\).

Computation Target Displacement:
Based on the effective lateral stiffness (\(K_a\)) and elastic lateral stiffness (\(K_d\)) defined as the initial elastic gradient of the capacity curve (pushover curve) and initial elastic gradient of the bilinear idealization, the effective fundamental period (\(T_e\)) was calculated from Equation 6. Moreover, modification factors (\(C_p\), \(C_d\), \(C_f\) and \(C_t\)) were obtained using equations and tables indicated in FEMA-356.

Then insert the values obtained above into Equation 7 to obtain target displacement (\(\delta_t\)). Table 3 show results for proposed values and target displacement of case study frames.

<table>
<thead>
<tr>
<th>Story</th>
<th>(C_p)</th>
<th>(C_d)</th>
<th>(C_f)</th>
<th>(C_t)</th>
<th>(S_e)</th>
<th>(T_e)</th>
<th>(K_e)</th>
<th>(K_d)</th>
<th>(\delta_t)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-Story</td>
<td>1.296</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.6743</td>
<td>0.9491</td>
<td>0.9491</td>
<td>16011</td>
<td>16011</td>
</tr>
<tr>
<td>8-Story</td>
<td>1.4187</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.4365</td>
<td>1.4663</td>
<td>1.1696</td>
<td>35431</td>
<td>22542</td>
</tr>
<tr>
<td>12-Story</td>
<td>1.358</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0.3217</td>
<td>1.9894</td>
<td>1.6628</td>
<td>38580</td>
<td>26950</td>
</tr>
</tbody>
</table>

Capacity Spectrum Method (CSM):
CSM provides a specially treatment of the reduction of earthquake demand for maximum displacement of existing reinforcement structures. In CSM target displacement obtained from Performance Point in the ADRS format of capacity curve and demand spectrum curve.

Computation Performance Point:
Target displacement (Performance Point) in the background of the CSM is obtained at the crossing of the elastic response spectrum and capacity curve. Since this point is obtained in ADRS format, therefore, the target displacement is necessary to converted into a capacity curve format. The target displacement can be converted by equations:

\[
d_{\text{roof}} = \delta_t \cdot \Delta_{\text{roof}} \cdot \Gamma \\
V = \alpha_t \cdot M \cdot g \cdot \delta_a
\]

According to the ATC-40 using of the uniform and ELF lateral load distribution is suggested for pushover analysis. Intersection of the elastic response spectrum and capacity curve is the coordinates of the performance point in ADRS format. This coordinates using Equation 16 and 17 convert to capacity curve format for use in
calculation of displacement and drift. Table 4 show the summarized of performance points in both ADRS and capacity curve format for case study frames.

Table 4: Performance Point in ADRS and Capacity Curve Format.

<table>
<thead>
<tr>
<th>Story</th>
<th>$S_a$</th>
<th>$S_d$</th>
<th>V</th>
<th>d</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-Story</td>
<td>0.292</td>
<td>0.058</td>
<td>720.89</td>
<td>0.077</td>
</tr>
<tr>
<td>5-Story</td>
<td>0.15</td>
<td>0.096</td>
<td>776.16</td>
<td>0.126</td>
</tr>
<tr>
<td>8-Story</td>
<td>0.116</td>
<td>0.125</td>
<td>2052.68</td>
<td>0.176</td>
</tr>
<tr>
<td>12-Story</td>
<td>0.081</td>
<td>0.192</td>
<td>2954.65</td>
<td>0.262</td>
</tr>
</tbody>
</table>

Modal Pushover Analysis (MPA):

MPA method has been established to account for the higher mode effects. Calculation of the complete response of the building is obtained by combining all modal responses in SRSS combination format. Therefore for estimation target displacement first should be computed modal capacity curve of structures. Figure 2 show capacity curve of modal load distribution for proposed frames in this research.

Fig. 2: Modal Capacity Curve of 5, 8 and 12-Story Reinforced Concrete Frames.
In MPA estimation of peak roof displacement in each mode is obtained from $U_m = \sum_i^n q_n D_n$, wherein $D_n$ obtained from solving of equation 12. As shown in Table 5 roof displacement calculated for the first three modes and then combined and idealized using the MPA method.

Table 5: Target Displacement in MPA Method.

<table>
<thead>
<tr>
<th>Story</th>
<th>Mode 1</th>
<th>Mode 2</th>
<th>Mode 3</th>
<th>MPA</th>
</tr>
</thead>
<tbody>
<tr>
<td>5-Story</td>
<td>0.32128</td>
<td>0.13393</td>
<td>0.07655</td>
<td>0.191</td>
</tr>
<tr>
<td>8-Story</td>
<td>0.8005</td>
<td>0.29201</td>
<td>0.13687</td>
<td>0.335</td>
</tr>
<tr>
<td>12-Story</td>
<td>1.21894</td>
<td>0.38072</td>
<td>0.24212</td>
<td>0.431</td>
</tr>
</tbody>
</table>

Nonlinear Time-History Analysis:

As shown in Table 1 five near field and five far field ground motions were defined to show the real response of proposed structures. The results of these ground motions idealized based on proposed target displacement by FEMA-356. Each of presented ground motions are including a duration (sec) which are recorded by seismograph into equal small steps to represent the more accurately the behaviour of structure. Nonlinear time-history analysis for each ground motion provides a function of joint in each story, in which shows displacement of each story under ground motion. This function is consisting of maximum and minimum displacement in story, which is used to plot story displacement curve.

Comparison the Results of Selected Pushover Methods:

The correctness of selected methods to predict the real behaviour of the structure was evaluated in this study. Therefore, nonlinear dynamic analysis method is used to evaluate the proposed methods. This evaluation can be performed via comparing displacement, drift and behaviour of structures. A single coordinate system is required to compare the curves of each case studies frame. Therefore the capacity curves of CM, CSM and MPA methods are plotted in a single coordinate system for each case study frames.

Story Displacement:

Story displacements using selected methods are different for each selected frames. These displacements are obtained using target displacement of each method to show the displacement of each story. Furthermore, the nonlinear time-history analysis is idealized base on target displacement of FEMA-356 to comparison methods. The selected CM, CSM and MPA methods are evaluated by using selected ground motions. For this purpose, the floor displacements from CM method, CSM and MPA methods are compared with the obtained values from the nonlinear time-history analysis. The story displacement of 5, 8 and 12-story frame for set ground motion are shown in Figure 3.

The results comparison shows that displacements obtained from 5 and 8-story frames in CM and MPA methods are overestimated at all stories especially in the middle stories (Figure 3a and 3b). Furthermore, the displacement in CSM are slightly overestimated at lower stories and underestimated at upper stories.

The results of displacement in a 12-story frame show that story displacements of CSM in comparison selected ground motions displacement are underestimated at the all stories (Figure 3c). In addition, it can be seen that displacements of MPA and CM methods are almost similar at the all stories in comparison with selected ground motions.

Inter-Story Drift:

Drift is the comparative horizontal displacement of two adjoining story in structure. Inter-story drift is a percentage of the story height separating in the two adjoining floor. The selected CM, CSM and MPA methods are evaluated by using selected ground motions. For this purpose, the inter-story drift from the CM method, CSM and MPA methods are compared with the obtained values from the selected ground motions. The inter-story drifts of 5, 8 and 12-story frame for set of ground motion are shown in Figure 4.

The results presented for the inter-story drifts shows that all methods, excluding CSM lead to inter-story drift that are essentially similar to selected ground motions with slight difference.

In 5-story frame, the comparison of inter-story drifts from the all pushover methods and the selected ground motion (Figure 4a) shows that the all pushover methods, excluding CSM method, lead to gross overestimation of drifts in the lower stories. Also results show that all pushover method lead to slightly underestimation of drift in the upper stories.

In 8-story frame, the comparison of results shows that the all pushover methods, excluding CSM method, lead to gross overestimation of drift in the lower story (Figure 4b). Furthermore, results show although CSM method lead to gross underestimation of drift in the upper stories, MPA and CM method lead to almost similar estimation of drift in the upper stories. In addition, as shown in Figure 4b MPA method lead to almost similar estimation of drift in the middle stories.
Fig. 3: Story Displacement of 5, 8 and 12-Story Reinforced Concrete Frames.
In 12-story frame, the comparison of results shows that CSM method lead to underestimation of drift in the all stories. Also the results show that the MPA and CM methods, lead to almost similar estimation of drift in the all stories, which among these methods, MPA method results is more similar than the others. Table 6 shows the amounts of drift of different stories obtained using various methods, including MPA and CM, compared to that of Northridge ground motion for 12-story frame.
Table 6: Difference in Different Stories Drift from MPA, CM and Northridge Ground Motion for 12-story frame.

<table>
<thead>
<tr>
<th>Story</th>
<th>Drifts</th>
<th></th>
<th>Drifts</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>MPA</td>
<td>Northridge</td>
<td>Difference (%)</td>
<td>CM (ELF)</td>
</tr>
<tr>
<td>1</td>
<td>25</td>
<td>30</td>
<td>-16</td>
<td>1</td>
</tr>
<tr>
<td>3</td>
<td>78</td>
<td>75</td>
<td>+4</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>133</td>
<td>136</td>
<td>-2</td>
<td>6</td>
</tr>
<tr>
<td>10</td>
<td>85</td>
<td>90</td>
<td>-5</td>
<td>10</td>
</tr>
<tr>
<td>12</td>
<td>45</td>
<td>45</td>
<td>0</td>
<td>12</td>
</tr>
</tbody>
</table>

As can be seen in Tables 6, the amount of drifts obtained using MPA method are very close to that of Northridge grand motion rather than CM method in 12-story frame.

**Conclusion:**

Motivated by previous researches correspond to that issue, this research presents an assessment of current nonlinear static procedures on the different type of structures. This research used the 5, 8 and 12-story frames to represent the real low, medium and high rise regular reinforced concrete structure. Furthermore, this research using nonlinear time-history analysis for assessment of selected procedures. Selected procedures are including, CM method proposed by FEMA- 356 (2000), CSM method adopted by ATC-40 (1996) and MPA method proposed by Chopra and Goel (2002).

Based on results obtained by comparing the selected pushover methods and nonlinear time history the following conclusion can be made that fulfilled the objectives of this research:

The comparison of capacity curves from the selected methods showed that the behaviour of selected case study frames for CM, CSM and MPA methods are almost similar. Moreover, according to the story displacements of the selected methods in comparison with selected ground motions, MPA and CM methods are almost similar. Nevertheless, the story displacements at the roof level for CSM are different. In addition, the comparing of inter-story drift from selected methods reveal that MPA and CM lead to almost similar estimation of drift in the, especially in high rise frames, while CSM lead to inaccurate estimation of selected frame. Therefore, it can be concluded that the MPA method proposed by Chopra and Goel (2002) and CM method proposed by FEMA-356 can be used for analysis of structure. However, it seems CSM method adopted by ATC-40 (1996) cannot show the real behaviour of structure under earthquake in this research.

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