Seismic Assessment of a RC Building according to FEMA 356 and Eurocode 8

Ioannis P. GIANNOPOULOS

Key words: Pushover analysis, FEMA 356, Eurocode 8, seismic assessment, plastic rotation, limit states

ABSTRACT: Seismic engineers are increasingly turning to non-linear methods of analysis which predict directly the amount and location of plastic yielding within a structure. This is in many ways a more satisfactory procedure than relying on elastic analysis reduced by a ductility factor. Non-linear static (pushover) analysis is a commonly used technique, which is finding prominence in standards and guidance material like Eurocode 8 and FEMA 356. The purpose of this paper is to compare the methods given by these two documents.

In this paper a typical existing five-story RC frame which has been designed for moderate seismicity according to the past generation of Greek seismic codes is investigated. A non-linear static (pushover) analysis is carried out for the building using SAP 2000 and the ultimate capacity of the building is established. Few critical sections are selected and the rotational ductility supply at various limit states as predicted by FEMA 356 and Annexe A of EC8 Part 3 is calculated. The two predictions are compared with each other and with results from the SAP2000 analysis. The outcome of this paper is to provide useful information for further development of Eurocode 8.

INTRODUCTION

Among the engineering community involved with the development of seismic design procedures, there is a general belief that the conventional elastic design and analysis methods cannot capture many important aspects that control the seismic performance of structures in severe earthquakes. Moreover, another powerful tool, inelastic time-history analysis, is computationally expensive and not feasible for many cases.

1 Civil Eng., MSc, DIC, PhD, Dept of Engineering, University of Cambridge, UK
email: ig245@cam.ac.uk
Nowadays, engineers are seeking a technique, which would solve the drawbacks described above. The search for a more useful and rational design process is a big issue for the future. Design has always been a compromise between simplicity and reality. The latter term, reality, seems to be very complex due to big uncertainties in imposed demands and available capacities. The first term, simplicity, is a necessity driven by computational cost and at the same time the limited ability to implement complexity with available knowledge and tools.

Pushover is a static non-linear analysis. The term static implies that a static method is being applied to represent a dynamic phenomenon. The term analysis implies that a pushover is being carried out to an already existing building and evaluates the existing solution and modifies it as needed. Therefore, pushover is a powerful tool for assessment purposes.


Non-linear static analysis has been developed extensively over the last years, and the reason is because powerful computers are able to support new computer programs. Some of these programs that are nowadays available are IDARC, DRAIN-2DX, INDYAS, SAP, STRUDL, STATIK, ETABS and SOFISTIK. In the present paper SAP 2000 V8 is used to carry out the pushover analysis to a test building.

A simple example of a pushover analysis is illustrated in Figure 1. This procedure requires the execution of a non-linear static analysis of a structure, which allows monitoring progressive yielding of the structure and establishing the capacity curve. The structure is 'pushed' with a lateral loading shape that follows the fundamental mode shape of the pre-yielding building to specific target displacements levels, while vertical earthquake loading is ignored. The resulting plot of Base Shear - Roof Displacement (Figure 1) is called the 'capacity curve'. The internal forces and deformations computed at the target displacement levels are estimates of the strength and deformation demands, which need to be compared to available capacities.

Figure 1  Illustration of pushover analysis
The first step in the construction of capacity and demand spectra curves is the conversion of the pushover curve, e.g. Base Shear - Roof Displacement, to an equivalent capacity curve, e.g. Spectral Acceleration - Spectral Displacement. The formulas for conversion of a pushover curve to a capacity spectrum are described elsewhere (Giannopoulos, 2006). The second step is to plot demand spectrum according to ATC – 40 or EC 8 (Figure 3).

**Code Provisions for Plastic Hinge Rotation Capacities of RC Members**  
(a) FEMA Plastic Hinge Rotation Capacities

The nonlinear procedures of FEMA require definition of the nonlinear load-deformation relation. Such a curve is given in Figure 4.

Point A corresponds to the unloaded condition. Point B corresponds to the nominal steel yield strength. The slope of line BC is usually taken equal to between 0% and 10% of the initial slope (line AB). Point C has resistance equal to the nominal strength. Line CD corresponds to initial failure of the member. It may be associated with phenomena such as fracture of the bending reinforcement, spalling of concrete or shear failure following initial yield. Line DE represents the residual strength of the member. It may be non-zero in some cases, or practically zero in others. Point E corresponds to the deformation limit. However, usually initial failure at C defines the limiting deformation, and in that case point E is a point having deformation equal to that at C and zero resistance.
The five points (A, B, C, D and E) are used to define the hinge rotation behaviour of RC members according to FEMA. Three more points Immediate Occupancy (IO), Life Safety (LS) and (Collapse Prevention) CP, are used to define the acceptance criteria for the hinge.

It is worth mentioning that the newest US code (ASCE 41-06, 2007) is using the same limiting values for the member plastic hinge rotation demand for beams. However, the corresponding limiting values for columns have slightly changed.

(b) EC 8 Plastic Hinge Rotation Capacities

The deformation capacity of beams, columns and walls, is defined in terms of the chord rotation $\theta$, i.e. of the angle between the tangent to the axis at the yielding end and the chord connecting that end with the end of the shear span (the point of contraflexure). The chord rotation is also equal to the element drift ratio, i.e. the deflection at the end of the shear span with respect to the tangent to the axis at the yielding end, divided by the shear span.

The state of damage in a structure is defined in EN 1998-3, Eurocode 8, by three Limit States, namely Near Collapse (NC), Significant Damage (SD), and Damage Limitation (DL).

Limit State of Near Collapse (NC)

The structure is heavily damaged, with low residual lateral strength and stiffness, although vertical elements are still capable of sustaining vertical loads. The value of the total chord rotation capacity at ultimate of concrete members under cyclic loading may be calculated from the following expression

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016 (0.3^\gamma) \left[ \frac{\max(0.01; \omega)}{\max(0.01; \omega)} \right] 0.225 \left( \frac{L_V}{h} \right)^{0.35} 25 \left( \frac{f_y}{f_c} \right) \left( \frac{h}{f_c} \right) \left( 1,25^{100p_a} \right)$$

(1)

The confinement effectiveness factor according to Sheikh and Uzumeri, (1982) is

$$a = (1 - \frac{s_h}{2b_0})(1 - \frac{s_h}{2b_0})(1 - \frac{\sum b_i^2}{6h_0b_0})$$

(2)

The value of the plastic part of the chord rotation capacity of concrete members under cyclic loading may be calculated from the following expression

$$\theta_{um}^{pl} = \theta_{um} - \theta_y = \frac{1}{\gamma_{el}} 0.045 (0.25^\gamma) \left[ \frac{\max(0.01; \omega)}{\max(0.01; \omega)} \right] 0.3 \left( \frac{L_V}{h} \right)^{0.35} 25 \left( \frac{f_y}{f_c} \right) \left( \frac{h}{f_c} \right) \left( 1,25^{100p_a} \right)$$

(3)

In members without detailing for earthquake resistance the values given by Equations 1 & 3 are multiplied by 0.825.
In members with smooth (plain) longitudinal bars $\theta_{um}$ given by Equation 1 is multiplied by 0.575, while $\theta_{um}^{pl}$ given by Equation 3 is multiplied by 0.375 (where these factors include the reduction factor of 0.825 given above).

For the evaluation of the ultimate chord rotation capacity EC 8 proposes also an alternative expression to Equation 1

$$\theta_{um} = \frac{1}{\gamma_{el}} (\theta_y + (\phi_u - \phi_y) L_{pl} (1 - \frac{0.5L_{pl}}{L_y}))$$

(4)

The value of the length $L_{pl}$ of the plastic hinge depends on how the enhancement of strength and deformation capacity of concrete due to confinement is taken into account in the calculation of $\phi_u$. There are two procedures given for this in EC8.

**Limit State of Significant Damage (SD)**

The structure is significantly damaged, with some residual lateral strength and stiffness, and vertical elements are capable of sustaining vertical loads. The chord rotation capacity corresponding to significant damage $\theta_{SD}$ may be assumed to be 75% of the ultimate chord rotation $\theta_u$ given by Equation 1.

**Limit State of Damage Limitation (DL)**

The structure is only lightly damaged, with structural elements prevented from significant yielding and retaining their strength and stiffness properties.

The capacity for this limit state used in the verifications is the yielding bending moment under the design value of the axial load. In case the verification is carried out in terms of deformations the corresponding capacity is given by the chord rotation at yielding $\theta_y$, evaluated for beams and columns using the following Equation

$$\theta_y = \phi_y \frac{L_{pl} + a_y z}{3} + 0.00135(1+1,5 \frac{h}{L_y}) + \frac{\varepsilon_y}{d-d'} 6f_y \sqrt{f_c}$$

(5)

An alternative expression is

$$\theta_y = \phi_y \frac{L_{pl} + a_y z}{3} + 0.0013(1+1,5 \frac{h}{L_y}) + 0.13\phi_y \frac{d_y f_y}{\sqrt{f_c}}$$

(6)

The first term in the above expressions accounts for flexure, the second term for shear deformation and the third for anchorage slip of bars.
DESCRIPTION OF THE TEST BUILDING

The test building is a five storeys reinforced concrete building, with a storey height of 3.00 m and regular 3.50 m bay sizes in both directions and 12 cm thick slabs. The columns at the base are rectangular with dimensions 350x350 mm, gradually reduced to 250x250 mm from the third floor to the roof. Column longitudinal reinforcement ranges between 1.0% and 2.5%, while transverse 8 mm diameter ties are used, spaced from 250 mm at the lower storeys to 400 mm at the roof. Beams are kept to dimensions 200/500 mm in all storeys and are lightly reinforced (about 0.4% steel ratio). The cross-section dimensions of columns are relatively narrow, reflecting the tendency of early designs to be as much as possible economic in concrete usage, since it was in situ mixed and manually conveyed and placed and because of the relatively low level of seismic action. Therefore, in the test structure the columns are slender and not strong enough to carry a large magnitude of bending caused by lateral forces generated during an earthquake, and consequently are more flexible than beams. More details about formwork and reinforcement details can be found elsewhere (Giannopoulos, 2006).

The building has been designed according to RD59 (1959), that is the old Greek design code, following allowable design stress procedures and simplified structural analysis models. Nominal values of dead and live loads were specified in the Greek Loading Codes (LC45, 1945), which are still in effect today. Structural elements possess no special reinforcement for confinement in the critical region and no capacity design provisions were used in their design.

Longitudinal bars in beams, are bent upwards at their end. The purpose of that is to resist negative moments at beams due to gravity loads. However, strong earthquake vibrations can change the moments at the ends of the beams (from sagging to hogging moment). As a result the bottom steel at beams at support may not be adequate for earthquake resistance. Transverse reinforcement in beams and columns are designed to resist shear under gravity loads only. Moreover, widely spaced stirrups (250 to 400 mm) do not provided sufficient confinement. Hence, stirrups are unable to withstand large curvature demand due to earthquake loads.
ANALYTICAL MODELLING OF MEMBERS

A summary of the modelling assumptions is given in Table 1. A more detailed review of all modeling assumption can be found elsewhere (Giannopoulos, 2006).

<table>
<thead>
<tr>
<th>Modelling Assumptions</th>
<th>Assumptions</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Material</strong></td>
<td></td>
</tr>
</tbody>
</table>
| Concrete [ B 160 ]    | Young's Modulus [21000 MPa]  
Compressive strength [16 MPa]  
No confining effect |
| Steel [ St 1 ]        | Young’s Modulus [200000 MPa]  
Yield strength [310 MPa]  
Ultimate strength [430 MPa] |
| Stress - Strain relationship | Concrete [Unconfined and confined according to Mander et al (1988)]  
Steel [bilinear Elasto-plastic model] |
| **Loading**           |             |
| Self-weight of members | weight per unit volume [24 KN/m³] |
| Gravity loads and masses | (dead load) + 0.3 (live load) |
| seismic design situation | (dead load) + 0.3 (live load) + (seismic action) |
| Mass distribution | Distributed at beam-column joints |
| P-delta effect | Not considered |
| Viscous damping | No |
| **Structural modeling** | SAP 200 V8.2.3 |
| Element model | Linear elements for beams and columns  
Shell elements for slabs |
| Rigid offset at beam-column joints | Yes, at both beam and column ends |
| Effective flange width of T-beams | Considered  
[b_{eff}=8h_f+b_c for internal beam]  
[b_{eff}=4h_f+b_c for external beam] |
| Diaphragm action | Taken into account using shell elements for the slabs |

Table 1  Modelling Assumptions

RESULTS OF ANALYSIS

Mode Shapes and Periods

An eigenvalue analysis of the test building is carried out using SAP 2000. The resulting elastic periods and mode shapes have been determined for the X and Y direction and torsion θ. The fundamental mode shape has a period $T_1 = 0.80$ sec.
This fundamental period can be compared with the fundamental period from EC8 formula, which is given in EC8 Part 1 § 4.3.3.2.2(3):

\[ T_f = C_f \cdot H^{3/4} = 0.075 \cdot 15^{3/4} = 0.57 \text{ sec} \]

where \( C_f \) is 0.075 for moment resistance space concrete frames and \( H \) is the height of the building, in m.

The fundamental period calculated from EC8 formula is less than the one calculated by the analysis. This is due to the fact that in the analysis partitions and cladding are not considered in the model and as a result the model is less stiff comparing with the real structure.

Pushover Curves
Having created the model of the test building, a static nonlinear (pushover) analysis of the building was carried out using SAP 2000. A maximum roof displacement of 0.20 m was chosen to be applied. Pushover analysis was carried out separately in the X and Y directions. The resulting pushover curves, in terms of Base Shear – Roof Displacement (V-\( \Delta \)), are given in Figure 5 for X and Y directions respectively. The slope of the pushover curves is gradually reduced with increase of the lateral displacement of the building. This is due to the progressive formation of plastic hinges in beams and columns throughout the structure. The pushover curves reach a maximum point and afterwards there is a sudden drop of the curve. This maximum point corresponds to failure of the building, i.e. there are many plastic hinges formed with big plastic rotations and the building can no longer sustain them.

Comparison of the pushover curves in the X and Y directions shows that they are almost identical. The max point for X direction is \( V = 1650.7 \text{ kN} \) and \( \Delta = 0.159 \text{ m} \) and for Y direction is \( V = 1627.7 \text{ kN} \) and \( \Delta = 0.156 \text{ m} \), with the X pushover curve being slightly stiffer than the Y curve. This is explained by the fact that the building has 4 bays in the X directions and 3 bays in the Y direction.

![Figure 5 Pushover curves for X and Y direction](image)

Plastic Hinge Formation
The formation of plastic hinges based on FEMA 356 rules are introduced as input into the SAP 2000 program. At every deformation step of the pushover analysis the program can do the following. (a) Determine the position and plastic rotation...
of hinges in beams and columns. (b) Determine which hinges have reached one of the three FEMA limit states: IO, LS and CP using suitable colors for their identification (Figures 6 & 7). The steps at which the three limit states of plastic hinges are reached and the corresponding values on the pushover curve are given in Table 2. From Figures 6 & 7 it is observed at step 10 (CP limit state) that the 3rd floor is a weak storey with plastic hinges formed at the top and bottom of almost all columns having reached the LS limit state.

The FEMA 356 rules, with the IO, LS and CP limit states for hinge rotation, have been used in the analysis with the SAP 2000 program. All the above shown information is calculated by the program automatically. Applying the Annex A of EC8 Part 3 values for DL, SD and NC limit states cannot be done automatically using the program. Nevertheless some critical positions in columns and beams have been chosen and the Moment-Rotation values calculated by the program, at each deformation step, will be used to calculate the rotational ductility supply by Annex A of EC8 Part 3 and compare it with that by FEMA and with the rotational ductility demand.

<table>
<thead>
<tr>
<th>Direction</th>
<th>Limit State (performance level)</th>
<th>Colour</th>
<th>Step at which Limit State is reached</th>
<th>Base Shear [kN]</th>
<th>Roof Displacement [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>X</td>
<td>Immediate Occupancy (IO)</td>
<td></td>
<td>5</td>
<td>1301,0</td>
<td>0,064</td>
</tr>
<tr>
<td></td>
<td>Life Safety (LS)</td>
<td></td>
<td>8</td>
<td>1524,2</td>
<td>0,111</td>
</tr>
<tr>
<td></td>
<td>Collapse Prevention (CP)</td>
<td></td>
<td>10</td>
<td>1650,7</td>
<td>0,159</td>
</tr>
<tr>
<td>Y</td>
<td>Immediate Occupancy (IO)</td>
<td></td>
<td>5</td>
<td>1298,5</td>
<td>0,065</td>
</tr>
<tr>
<td></td>
<td>Life Safety (LS)</td>
<td></td>
<td>8</td>
<td>1512,1</td>
<td>0,111</td>
</tr>
<tr>
<td></td>
<td>Collapse Prevention (CP)</td>
<td></td>
<td>10</td>
<td>1627,7</td>
<td>0,156</td>
</tr>
</tbody>
</table>

**Figure 6** Step 5 (Immediate Occupancy)  **Figure 7** Step 10 (Collapse Prevention)

**Table 2** Steps at which limit states IO, LS and CP of plastic hinges are reached and corresponding values on pushover curve
Rotational Ductility Supply by FEMA 356 and Annex A of EC8 Part 3

The program calculates at every plastic hinge the moment – rotation values at every deformation step of the analysis. These values can be used by hand to calculate at a number of selected critical positions in columns and beams the rotational ductility demand curve and compare it with the rotational ductility supply curve by FEMA and EC8. This was done at the top and bottom end of a critical column and one end of a critical beam of the test building.

A column from the 3rd floor, which is a weak floor, is the most critical column of this floor as it can be seen from Figure 6, and was chosen for calculations of rotational ductility supply by FEMA and EC8 and comparison with rotational ductility demand.

The geometry, materials and reinforcement of the column are summarized elsewhere (Giannopoulos, 2006). The confinement effectiveness factor $\alpha$ is determined using Equation 2. The curvatures $\phi_y$ and $\phi_u$ are determined using XTRACT program. The plastic hinge rotation values $P$, $V$, $M$, $\theta_{pl}$ calculated from the pushover analysis at every deformation step are also considered. More details are given elsewhere (Giannopoulos, 2006).

All these data are used to calculate at every deformation step the hinge rotation supply prescribed by EC8 at the NC, SD and NC limit states. For this all the equations of EC8 given in Equations 1-6 are used. Finally all these values are used to plot curves of the hinge plastic rotation supply and demand and are given in Figure 8.

From the curves of hinge plastic rotation supply and demand given in Figure 8a for FEMA and EC8 it is observed that the EC8 limit states are approximately 1,2 times the corresponding values of FEMA. This column section is one of the most heavily stressed (critical section) and at the last deformation step, where the building cannot sustain any more lateral displacement, the FEMA Collapse Prevention (CP) limit state is reached, while the corresponding EC8 Near Collapse (NC) limit state is not reached.

Similarly, all data for the bottom of the critical column are determined and Figure 8b is showing the plastic rotation and FEMA and EC8 limit states at each deformation step. It can be concluded that bottom end column section is less critical than the top end section and does not reach the CP limit state at the last deformation step. Regarding the EC8 limits of this bottom end column section they are practically the same as the corresponding ones of the top end section.

A critical beam is chosen from the 1st floor of the building. All the data and results of analysis and calculations using the FEMA and EC8 limit states for plastic hinge rotations for this section are given Figure 9. It is observed that the EC8 limit states are increasing with roof displacement, while in columns they remain almost constant. Additionally the EC8 NC limit state values are less than the
corresponding FEMA CP values, while in columns it is the opposite. The same applies for the SD EC8 and LS FEMA limit states.

![Plastic rotation (magenta) and FEMA (blue) and EC8 (green) limit states for column K4-3](image)

**Figure 8** Plastic rotation (magenta) and FEMA (blue) and EC8 (green) limit states for column K4-3

![Plastic rotation and FEMA and EC8 limit states (Beam B1.3 right)](image)

**Figure 9** Plastic rotation and FEMA and EC8 limit states (Beam B1.3 right)

**CONCLUSIONS**

A typical five storey non-ductile RC frame building which has been designed following past seismic regulations in Greece has been analyzed using a nonlinear static (pushover) analysis. Few critical sections are selected and the rotational ductility supply at various limit states as predicted by FEMA 356 and Annex A of EC8 Part 3 is calculated. The two predictions are compared with each other and with results from the SAP2000 analysis. The comparison demonstrates that there are differences in the results produced by the two approaches.
(a) From the curves of hinge plastic rotation supply for FEMA and EC8 it is observed that for beams the EC8 limit states are increasing with roof displacement, while in columns they remain almost constant. Additionally the EC8 NC limit state values for beams are less than the corresponding FEMA CP values, while in columns it is the opposite. The same applies for the SD EC8 and LS FEMA limit states.

(b) The resulting $\theta_y$ values from Equation 5 of EC8 consist of three terms accounting for flexure, shear, and anchorage slip of bars respectively. It is observed that the first term is roughly equal to 60% of the sum of all terms. Comparison of values from Equation 5 & 6 shows almost exact values and therefore the two equations are correctly given by EC8 as alternative and equivalent. Another point that it is noticed is that $\theta_y$ remains practically constant in all deformation steps.

(c) For columns the resulting $\theta_u$ values from Equation 4 are slightly less (93%) than those from Equation 1 and therefore are correctly given by EC8 as alternative and equivalent. Also the resulting $\theta_pl$ values from Equation 3 and from $\theta_u$ (from 1) minus $\theta_y$ (from 5) are practically the same.

(d) On the other hand, for beams the resulting $\theta_u$ values from Equation 4 are approximately 40% of those from Equation 1 and therefore are not correctly given by EC8 as alternative and equivalent. The Equation 4 includes $\phi_u$ and $\phi_y$ parameters which are for beams 6 and 2 times respectively the corresponding values for columns, while Equation 1 does not contain $\phi_u$ and $\phi_y$ parameters. Therefore it seems that Equation 4 needs reassessment. Also the resulting $\theta_pl$ values from Equation 3 and from $\theta_u$ (from Eq. 1) minus $\theta_y$ (from Eq. 5) are different for the same reasons as previous and this needs reassessment.

REFERENCES


