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# COMPARISON OF THE METHODS APPROVED FOR THE SEISMIC PERFORMANCE ASSESSMENT OF EXISTING BUILDINGS

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## ABSTRACT

In this study, it is aimed to compare the seismic damage assessments of TEC-2007 and Eurocode 8/3 methods. For this purpose 40 low and mid-rise existing residential buildings were selected and 3D nonlinear models were constructed by using the design projects of buildings. Each building was analyzed in X and Y directions and therefore 80 non-linear analyses were performed in total. Building inventory includes both new and old buildings. Buildings constructed before Turkish Earthquake Code-1998 are denoted as old, constructed after 1998 are denoted as new buildings. By this method, sensitivity of the selected methods on the new and old buildings was assessed. Non-linear response of inventory buildings (capacity curves of buildings) was obtained by using nonlinear pushover analysis. Member damage limits and the building drift ratios corresponding to these limits were determined according to both Turkish Earthquake Code-2007 and EuroCode-8. Inelastic deformation demands of buildings were calculated by using code compatible demand spectrum. Results have shown that the Eurocode 8/3 method presented in gives more critical results with respect to Turkish Earthquake Code-2007 method. Drift capacities corresponding to Eurocode formulations were smaller than that of Turkish code. This observation becomes clearer especially in older buildings. This situation reveals that individual building performances obtained from different code methods can be different. All of these observations indicate that assessed code methods can produce different performance estimations and hence decision makings. Therefore this situation should be taken into consideration during the seismic performance assessment studies.

*Keywords: Existing Reinforced Concrete Buildings; Seismic Performance Assessment; Nonlinear Analysis; Seismic Damage*

## 1. INTRODUCTION

Estimation of seismic performance of existing buildings and the methods used for the determination of damage levels are the important issues of structural engineering field. Existing modern structural codes include the regulations about not only new buildings, but also existing ones. Turkish Earthquake Code-2007 (TEC-2007) and Eurocode 8/3 (EC8/3) are the examples of these codes. In these codes seismic performance of buildings are defined depending on the deformation based damage states of beams and/or columns. Although demand estimation methods are similar, seismic performance predictions produced by TEC-2007 and Eurocode 8/3 (EC8/3) can be quite different. Main reason behind this situation is related with the different code expressions that explain member damage limits. In TEC-2007 strain based damage limits are used. In EC8/3, however, empirical equations are given to define member damage levels. Eurocode method and the expressions approved in this method are mainly based on studies performed by Fardis and Panagiotakos (Panagiotakos and Fardis, 2001, Fardis, 2009).

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## 2. CAPACITY CURVES AND DAMAGE LEVELS OF BUILDINGS

All of the 40 RC buildings considered in this study are real buildings and still occupied for residential purposes. Numbers of stories of selected buildings are 3, 4, 5 and 6. In each story group there are 5 old and 5 new buildings. Buildings designed and constructed after Turkish Earthquake Code of 1998 (TEC-1998) are called as new buildings since substantial changes and modern regulations were enforced first time by this code. Previous buildings were notated as old buildings. Non-linear models of buildings were constructed by using Sap2000 software according to design projects of buildings. Concrete and steel grades are accepted according to design projects. Distribution of buildings considered in this study is presented in Table 1.

Table 1. Distribution of story numbers of old and new buildings

| Building Story Number | Old (1998-) | New (1998+) |
|---|---|---|
| 3 | 5 | 5 |
| 4 | 5 | 5 |
| 5 | 5 | 5 |
| 6 | 5 | 5 |
| Σ | 20 | 20 |

Methods approved for the calculation of plastic deformation capacities of members by the TEC-2007 and EC 8/3 are quite different from each other. In TEC-2007 member damage limits are determined by considering the compression strains of concrete and tensile strains of longitudinal strains. Confinement level of the members determines the compression strain capacity of the sections. Plastic rotation capacities and the rotation levels corresponding to strain based member damage limits were determined by performing moment-curvature analyses. In EC 8/3, on the other hand, empirical formulations based on studies performed by Fardis and Panagiotakos (Panagiotakos and Fardis, 2001) were provided. Different assumptions and different calculation methods enforced by these codes may yield significantly different deformation capacities and hence different performance predictions. The other important parameter that can change the non-linear response of building according to these codes is related with the calculation of effective stiffness. TEC-2007 and EC8/3 use different formulations for effective stiffness calculations. This situation can affect the shape of capacity curves. Period and the demand estimation calculations are also affected from effective stiffness. Formulations and assumptions followed according to TEC-2007 and EC8/3 are summarized below.

### *2.1 Member Damage Definitions in Turkish Earthquake Code-2007*

As being aforementioned in text, strength and ductility capacities of columns and beams were determined by moment-curvature analyses. During the moment-curvature analysis, confined concrete behavior was represented by Modified Kent-Park (Park et al. 1982) model and section damage levels were obtained by considering concrete and steel strain limits given in TEC-2007. Plastic rotation capacity of each damage level were calculated by using moment-area theorems and assigned to critical sections. Plastic hinge lengths were taken as half of the section height (TEC-2007). Effective stiffness of members was determined depending on the axial load levels according to TEC-2007 (Equation 1). Interpolation was performed for intermediate values.

$$\frac{N}{A_c f_c} < 10\% \rightarrow EI_e = 0.4$$

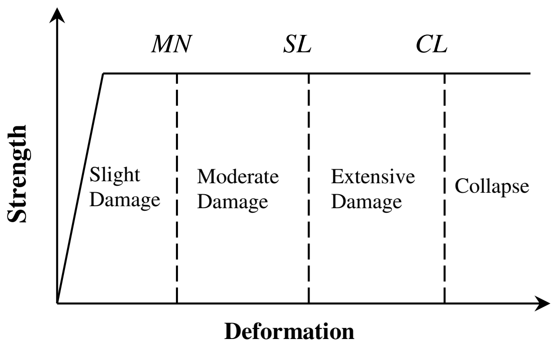
$$\frac{N}{A_c f_c} \geq 40\% \rightarrow EI_e = 0.8 \qquad (1)$$

Definition of three section damage limits Minimum Damage (MN), Safety Limit (SL), and Collapse Limit (CL) are presented in Table 2. Strain based formulations for both concrete and steel can be seen in this table. Notations $\rho_s$ and $\rho_{sm}$ defines the existing and required transverse reinforcement ratios according to TEC-2007. Details of the method can be found in the TEC-2007.

Table 2. Strain limits corresponding member damage limits in TEC-2007

| Section Damage Limits | Conctere | Steel |
|---|---|---|
| Minimum Damage (MN) | 0.0035 | 0.01 |
| Safety Limit (SL) | $0.0035 + 0.010\ (\rho_s / \rho_{sm}) \leq 0.0135$ | 0.04 |
| Collapse Limit (CL) | $0.0040 + 0.014\ (\rho_s / \rho_{sm}) \leq 0.0180$ | 0.06 |

Capacity curve of buildings were obtained by using nonlinear static pushover analysis. In order to compare strength and deformation capacities of buildings, capacity curves are represented in terms of lateral strength ratio ($V_t$/W) and drift ratio (Δ/H). Definition of member damage limits and the damage levels among these limits are shown on the left side of the Figure 1. Formation of these member damages depending on the increasing deformations are also shown on a sample curve in Figure 1. It can be clearly seen that MN levels of members are exceeded at the initial stages of plastic deformations of the building. CL levels which define the collapse limit appear at higher drift levels. Each point presented on the capacity curve represents the exceedance of predefined damage level at each member.



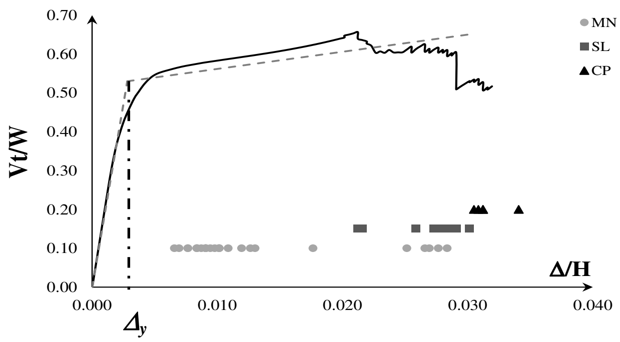




Figure 1. Member damage limits according to TEC-2007 and the representation of these limits on building capacity curve

### *2.2 Member Damage Definitions in Eurocode 8/3*

Eurocode 8/3 is also defines three damage limits named as Damage Limitation (DL), Significant Damage (SD), Near Collapse (NC). Damage Limitation and Near Collapse is defined by empirical equations. On the other hand, Significant Damage (SD) level is accepted as 3/4 of the ultimate chord rotation capacity. Alternative expressions can be found in EC8/3 for yield (DL) and ultimate (NC) chord rotation capacities. In this study, yield rotation (DL) was calculated by using Equation 2 and ultimate (NC) chord rotation capacity is calculated by using Equation 3. Plastic chord rotation capacity, used in the non-linear analysis stage, was obtained by extracting the yield rotation from the total capacity. DL limit in EC8/3 used to define yielding level. However, it can be said that damage limit of IO (in TEC-2007) corresponds to higher deformation level than that of DL.

$$\theta_y = \emptyset_y \frac{L_v + a_v z}{3} + 0.0013\left(1 + 1.5\frac{h}{L_v}\right) + 0.13\emptyset_y \frac{d_b f_y}{\sqrt{f_c}} \tag{2}$$

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016(0.3^{\nu}) \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225} \left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1.25^{100\rho_d}) \tag{3}$$

In these equations $g_{el}$ is coefficient and changes for primary and secondary elements. $L_v$ represents M/V ratio and $\nu$ defines the axial load ratio. *w, w'* are the mechanical reinforcement ratios for tension and compression longitudinal reinforcement. $\alpha$ represents the confinement effectiveness factor. $d_b$ defines mean diameter of the tension reinforcement. During the nonlinear pushover analysis, effective stiffness of the members was calculated by using Equation 4 (Eurocode 8).

$$EI_{eff} = \frac{M_y L_v}{3\theta_y} \tag{4}$$

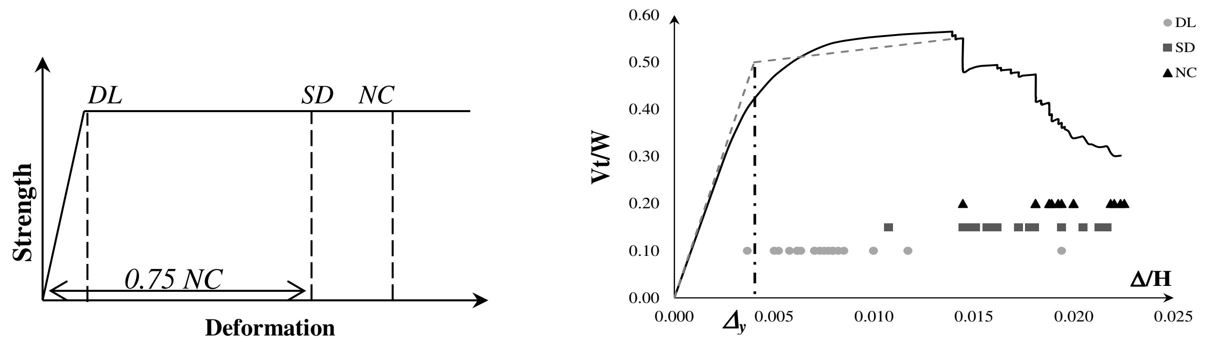



Figure 2. Member damage limits according to EC8/3 and the representation of these limits on building capacity curve

## 3. COMPARISON OF DAMAGE ESTIMATIONS

Capacity curves of buildings according to TEC-2007 and EC8/3 are presented in Figures 3 to 6. During the construction of non-linear building models it was observed that TEC-2007 method (Equation 1) gives relatively higher effective stiffness coefficients with respect to EC8/3 (Equation 4). Differences in the slope of elastic portion of the capacity curves (Figures 3 to 6) show the influence of the effective stiffness formulations. It can be said that TEC-2007 models are stiffer than that of EC8/3 ones. During the modelling stage it was also observed that TEC-2007 method gives higher rotation capacities at plastic hinges with respect to EC8/3. Higher ductile response of capacity curves of TEC-2007 models can be seen on the figures.

Damages obtained from TEC-2007 and EC8/3 methods were compared by investigating the distribution of the damage limits along the capacity curve. Dots presented on Figures 3 to 6 represent the formation of each damage limit at each element step by step. In order to investigate and compare the column and beam damages, they are plotted separately for each damage level.

In Figure 3, the points corresponding to minimum (MN and DL at left) and maximum (CL and NC at right) damage limits of columns, taken from one of the 3 story buildings, are shown. Both charts indicate that EC8/3 formulations give conservative estimations for minimum and maximum damage limits in columns. DL limits start to occur at earlier ductility levels with respect to MN. Similarly, NC limits also occur much before than CL limits. Validity of this situation for columns and beams can also be seen in other capacity curves presented from Figures 3 to 6. This situation indicates that expressions offered in Eurocode always give conservative estimations for beams and columns at both of the minimum and maximum damage states.

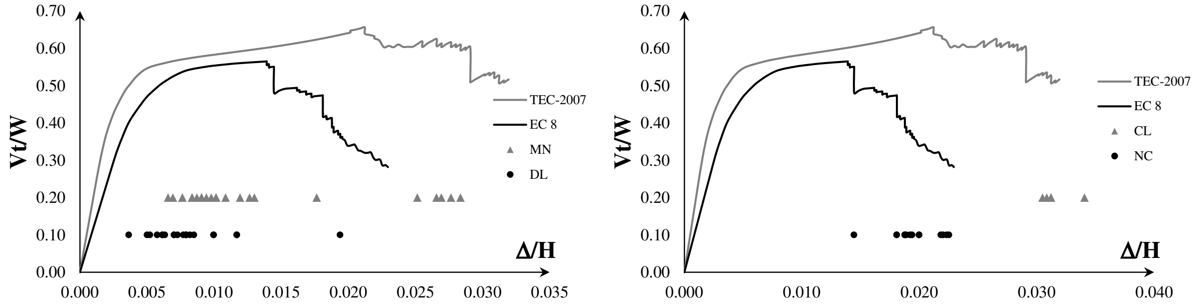



Figure 3. Capacity curve of 3 story new building and the formation of column damages

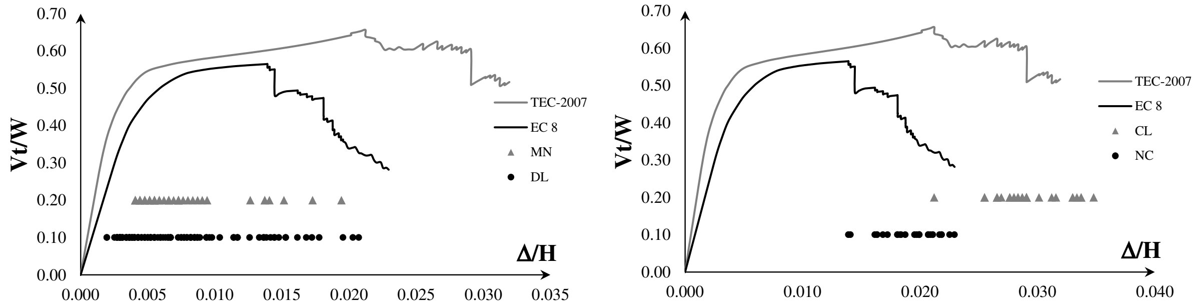



Figure 4. Capacity curve of 3 story new building and the formation of beam damages

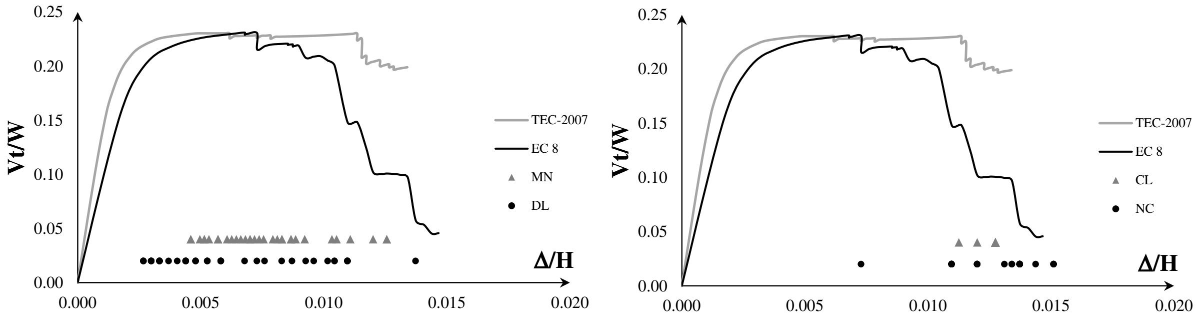



Figure 5. Capacity curve of 4 story old building and the formation of column damages

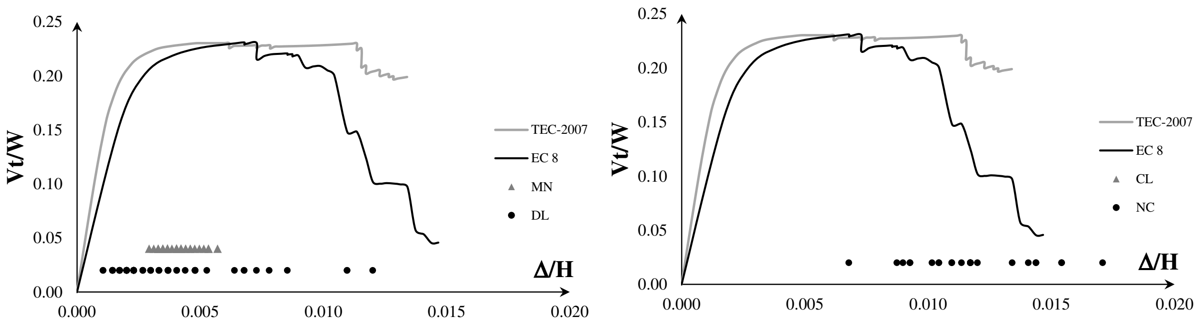



Figure 6. Capacity curve of 4 story old building and the formation of beam damages

In order to investigate the accumulation of damage limits, displacement ductility levels corresponding to formation of each limit for each member were calculated. Building yield displacement, which is required for the displacement ductility calculation, was obtained from bi-linearized capacity curve as shown in Figure 1 and 2. Roof displacement at the formation of each damage limit was divided into yield deformation of capacity curve ($\Delta_y$) and ductility level corresponding to each damage limit was obtained. This process was followed for both of the TEC-2007 and EC8/3 models. Distribution of ductility levels and the mean values corresponding different codes are presented from Figures 5 to 16. It can be said that smaller mean values of ductility limits corresponding to EC8/3 formulations substantiate the conservativeness of the Eurocode method.

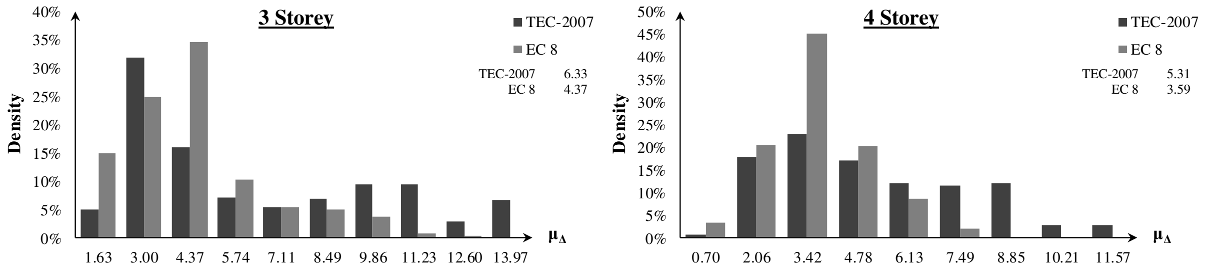



Figure 7. Distribution of building ductility corresponding MN and DL levels in columns (New building)

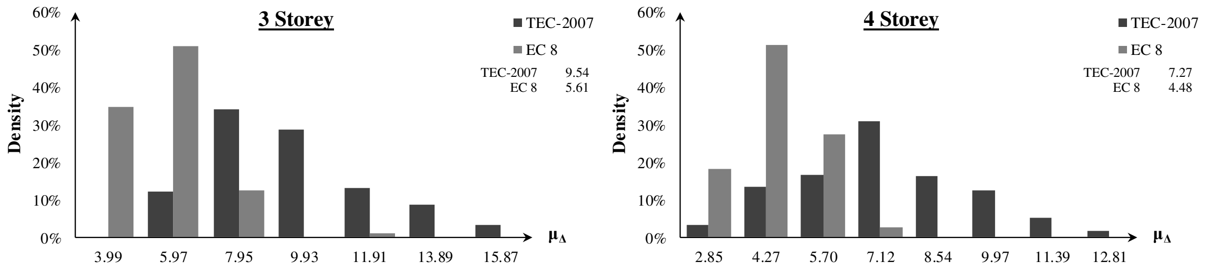



Figure 8. Distribution of building ductility corresponding SL and SD levels in columns (New building)

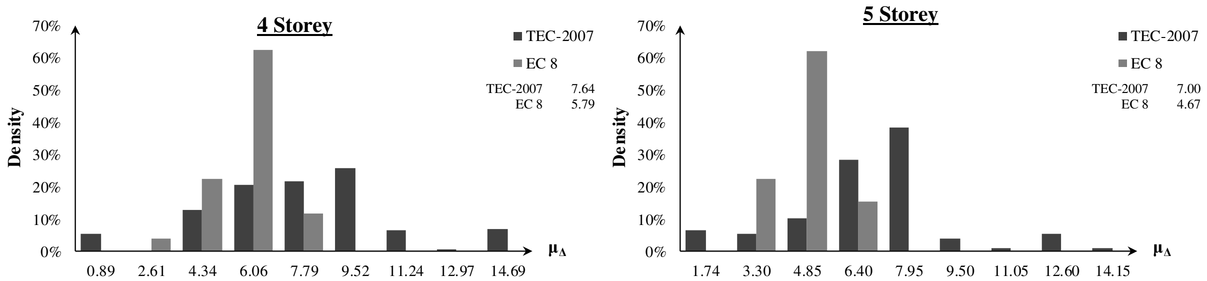



Figure 9. Distribution of building ductility corresponding CL and NC levels in columns (New building)

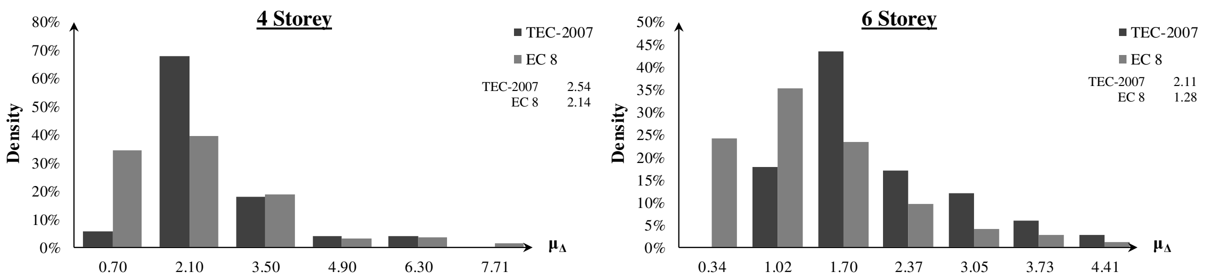



Figure 10. Distribution of building ductility corresponding MN and DL levels in beams (New building)

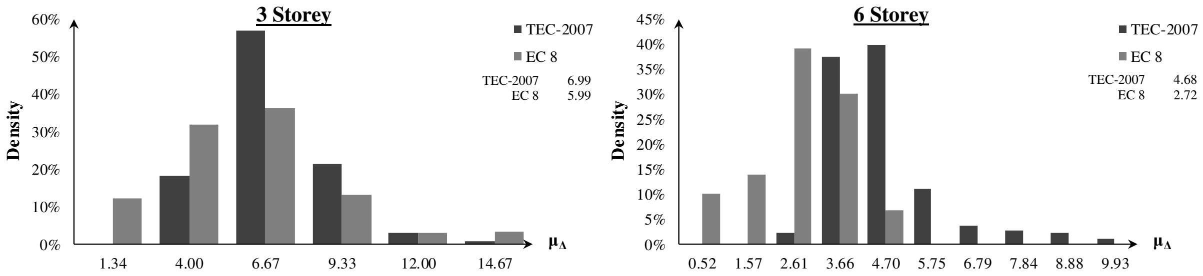



Figure 11. Distribution of building ductility corresponding SL and SD levels in beams (New building)

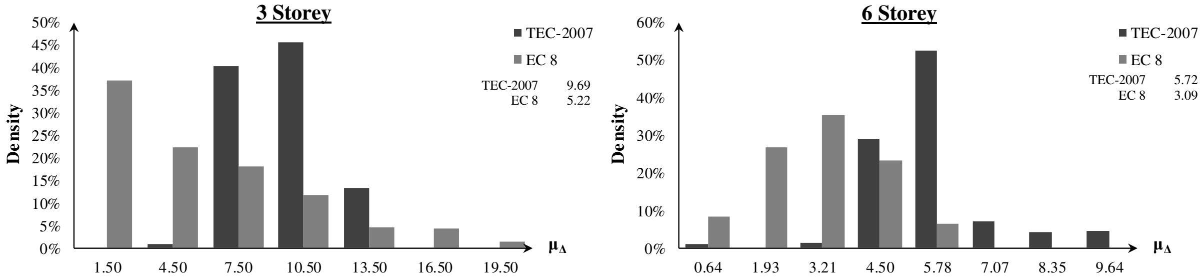



Figure 12. Distribution of building ductility corresponding CL and NC levels in beams (New building)

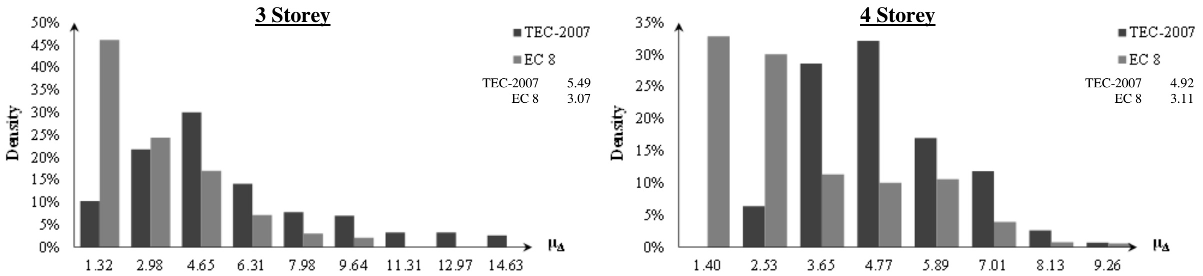



Figure 13. Distribution of building ductilities corresponding MN and DL levels in columns (Old building)

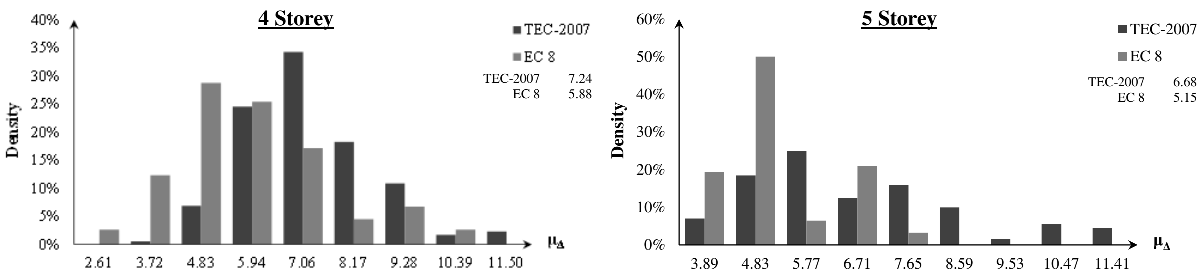



Figure 14. Distribution of building ductility corresponding SL and SD levels in columns (Old building)

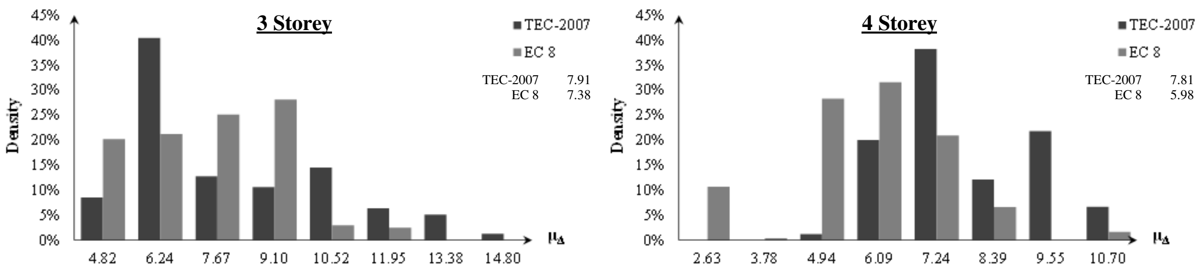



Figure 15. Distribution of building ductility corresponding CL and NC levels in columns (Old building)

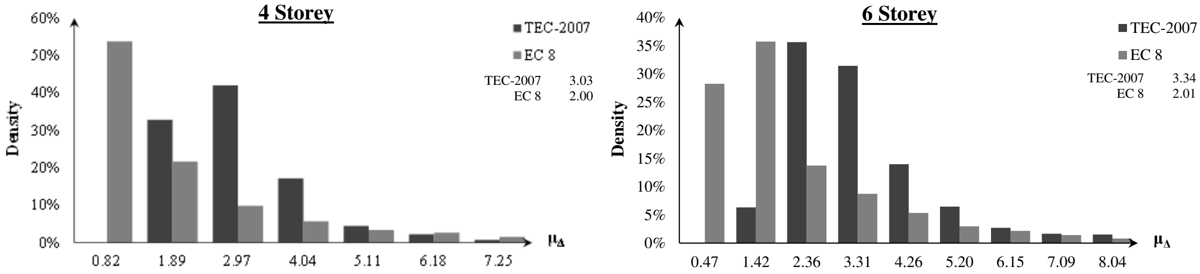



Figure 16. Distribution of building ductility corresponding MN and DL levels in beams (Old building)

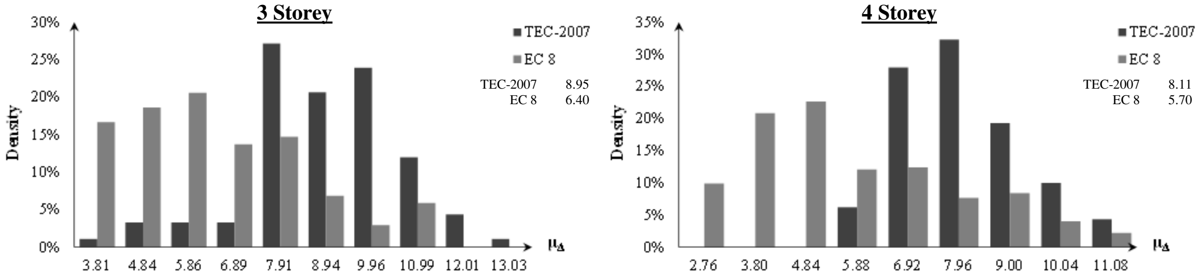



Figure 17. Distribution of building ductility corresponding SL and SD levels in beams (Old building)

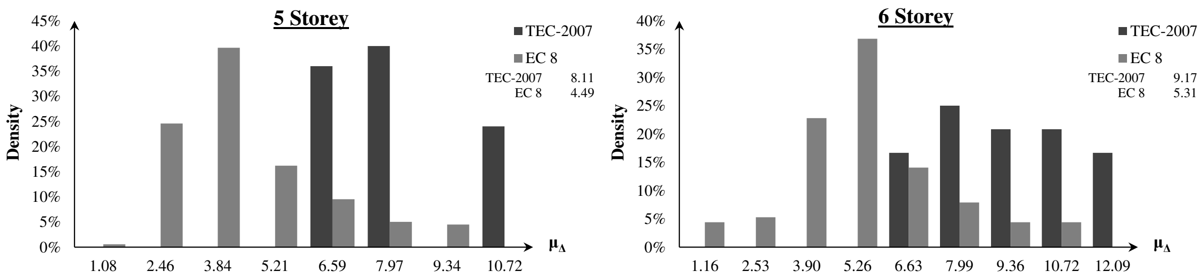



Figure 18. Distribution of building ductility corresponding CL and NC levels in beams (Old building)

Besides the effect of the code methods, effect of member type (beam or column) and the effect of building story number on the distribution of ductility levels were also discussed. Information, presented from Figure 5 to 16, is summarized in Tables 3 to 6. In these tables mean values of the distributions are presented.

Variation of results indicates that DL limit of Eurocode 8/3 corresponds to yield point and this situation explains significantly lower ductility levels at DL. This situation can be clearly seen both in graphs and tables. On the other hand, graphs and tables show that CL limits of Turkish code are also higher than NC limits of Eurocode. This situation shows that drift capacities estimated by strain based method of Turkish code are significantly higher than Eurocode method.

Member based assessments have shown that ductility levels calculated for beams and columns are also significantly different from each other. Both of the code methods showed that beams reach the MN limits at lower steps with respect to columns in both new and old buildings. In other words beams start to damage earlier. Low deformation capacities of short beams explain the higher damageability of beams. On the other hand, it was observed that beam CL limits (TEC-2007) are higher than that of columns especially in old buildings. In new buildings, differences between CL limits of columns and beams become smaller. However it should be stated that variations of distributions are very high.

Similar situation is also valid for EC8/3 models. DL limits of beams are attained at lower ductility levels with respect to columns in new and old buildings. NC limits of beams and columns are similar in old buildings. However, in new buildings, ductility capacities of columns are significantly higher than beams at NC. This situation is much more evident in low story buildings.

Distribution of results presented in Table 3 to 6 showed that ductility levels tend to decrease with increasing story numbers. Ductility values calculated for 3 story building are higher than that of 6 story ones. This situation is valid for both code methods and for all damage levels. However, high scatter shown in the graphs means that all of these comments are explains the general situation. Configuration of structural system, cross sectional dimensions and the length of columns and beams can significantly change the damage capacity of members. To make common evaluations it may necessary to increase the number of buildings in the inventory.

Table 3. Average building ductilities corresponding column damage levels (Old building)

| | | **TEC** | | | **EC8** | |
|---|---|---|---|---|---|---|
| | MN | SL | CL | DL | SD | NC |
| 3 | 5.49 | 7.24 | 7.91 | 3.07 | 6.28 | 7.38 |
| 4 | 4.92 | 7.24 | 7.81 | 3.11 | 5.88 | 5.98 |
| 5 | 4.50 | 6.68 | 6.24 | 2.71 | 5.15 | 5.57 |
| 6 | 4.88 | 6.49 | 6.63 | 2.58 | 5.96 | 5.66 |

Table 4. Average building ductilities corresponding column damage levels (New building)

| | | **TEC** | | | **EC8** | |
|---|---|---|---|---|---|---|
| | MN | SL | CL | DL | SD | NC |
| 3 | 6.33 | 9.54 | 10.18 | 4.37 | 5.61 | 10.69 |
| 4 | 5.31 | 7.27 | 7.64 | 3.59 | 4.48 | 5.79 |
| 5 | 5.47 | 7.56 | 7.00 | 3.11 | 4.17 | 4.67 |
| 6 | 4.20 | 5.06 | 5.31 | 2.30 | 4.84 | 3.22 |

Table 5. Average building ductilities corresponding beam damage levels (Old building)

| | | **TEC** | | | **EC8** | |
|---|---|---|---|---|---|---|
| | MN | SL | CL | DL | SD | NC |
| 3 | 3.40 | 8.95 | 11.49 | 2.64 | 6.40 | 8.22 |
| 4 | 3.03 | 8.11 | 10.66 | 2.00 | 5.70 | 6.21 |
| 5 | 2.86 | 7.59 | 8.11 | 1.51 | 3.98 | 4.49 |
| 6 | 3.34 | 8.36 | 9.17 | 2.01 | 5.10 | 5.31 |

Table 6. Average building ductilities corresponding beam damage levels (New building)

| | | **TEC** | | | **EC8** | |
|---|---|---|---|---|---|---|
| | MN | SL | CL | DL | SD | NC |
| 3 | 2.67 | 6.99 | 9.69 | 3.09 | 5.99 | 5.22 |
| 4 | 2.54 | 6.67 | 9.13 | 2.14 | 2.34 | 3.19 |
| 5 | 2.58 | 5.90 | 7.87 | 1.61 | 3.36 | 3.62 |
| 6 | 2.11 | 4.68 | 5.72 | 1.28 | 2.72 | 3.09 |

## 4. CONCLUSIONS

Estimation of non-linear deformation capacity of existing buildings is important and necessary for performance assessment of existing buildings. Different code methods approved by TEC-2007 and EC8/3 were compared by using 40 existing buildings occupied. Static pushover analysis was performed for all buildings in both X and Y directions.

Different elastic slopes of the capacity curves show the influence of different effective stiffness formulations of TEC-2007 and EC8/3. It was observed that TEC-2007 method gives higher effective stiffness coefficients with respect to EC8/3. This situation means that TEC-2007 based modeling approach can produce lower vibration period and hence lower drift demand.

Comparison of the capacity curves clearly showed that strain based formulations of TEC-2007 method gives higher plastic rotation capacities at plastic hinges with respect to EC8/3 method. This situation explains the higher drift limits of the capacity curves of TEC-2007 models. Investigation of results showed that DL limits of Eurocode correspond to significantly lower drift levels with respect to MN limits of TEC-2007.

Different ductility values corresponding to beam and column damages revealed the differences in the damageability of different members. In both code methods, it was investigated that beams start to damage at lower drift levels.

Results and assessments showed that investigated code methods can produce quite different damage estimations and hence decision makings. Therefore this situation should be taken into consideration during the seismic performance assessment studies.

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