



EVALUATION OF DEFORMATION LIMITS IN CODES FOR REINFORCED CONCRETE COLUMNS

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ABSTRACT

Predicting the deformation capacities of reinforced concrete columns is necessary for seismic evaluation of existing and new buildings. These deformation capacities have been set in terms of the rotation angle in EC-8 (2005) and ASCE/SEI 41 (2013) and strain in Turkish Seismic Code (2007-2016). In this study, finite element models of sixty-nine experimentally tested column specimens were modelled and analyzed by using nonlinear finite element method. As a result of the analyzes, data, which is difficult to measure in experiments such as the strain of outer fiber of confined concrete, were calculated and compared with code deformation limits. As a result of this study, EC-8 column ultimate deformation limits found to remain unsafe according to calculated deformation capacities herein. Taking into account TSC and ASCE/SEI 41, calculated capacities are in the safe region. However, for the axial load ratios over 0.45 and columns produced of high strength concrete, code limit state expressions are unable to predict the limit values correctly. In addition, TSC may significantly limit the deformation capacity of RC columns under their actual capacities. There is a need to improve these code limits using a more extensive column database which includes different design parameters.

Keywords: Deformation limits, RC Column, Finite element method

1. INTRODUCTION

Predicting the deformation capacity of reinforced concrete structural elements is important for seismic evaluation of existing or new buildings. The ability to remain without significant loss of load-carrying capacity of reinforced concrete (RC) structural members under seismic loads is vital for the assurance of life safety performance criteria. Understanding the behavior of columns, which are the primary components of RC structure, is important for the evaluation of whole structural frame. Although behavior of RC columns was investigated for years, the problem of accurate calculation of the behavior of RC columns under seismic loads is still needs improvements.

There are codes and performance based design expressions which have been developed for estimating ultimate deformation capacity of RC columns. Design codes specify deformation limits for RC columns in terms of the rotation angle (θ) [1-4] and strain (ϵ) [5-7]. During the seismic performance evaluation of reinforced member of buildings, designers use these proposed deformation limits. However, a limited number of column test results were used for verifying of these expressions and limit states specified at the codes, especially which are based on strain criteria [8]. Proposed expressions and acceptance criteria in seismic action for RC columns needs a through examination using more extensive column databases to improve limit states and their corresponding values. For deficient columns, Acun and Sucuoglu [9] conducted twelve full-scale column tests to evaluate performance limits and they found that available deformation limits in EC-8 [1] and ASCE/SEI 41[2] are very conservative. Same results were found in the drift capacity of RC columns tested by Bae and Bayrak [10]. Due to lack of time and testing difficulties, limited tests were conducted on large-scale RC columns with different design parameters, such as confinement, concrete strength, longitudinal reinforcement ratio, axial load ratio, and cross sectional dimensions. In addition, the results obtained

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even for similar columns differ from each other, though many column tests were done. Analytical studies can be conducted to minimize the margin of error by doing large number of analyses from a single hand using reliable models.

Evaluation of these code limits and proposed equations by analyzing large number of column database using a well-calibrated finite element model is easier than conducting experiments in nowadays technology. Kazaz et al. [11-13] evaluated deformation limits of reinforced concrete shear walls by using nonlinear finite element procedures (FEM). They proposed more reliable and improved prediction equations for the deformation capacity of shear walls. Using the similar analytical framework, this study utilizes a well calibrated modeling tool to investigate the deformation measures defined in terms of plastic rotation, local concrete and steel strains at the extreme fiber of rectangular RC column sections. In addition, crushing of concrete cover, bond failure, buckling of compression bars, strain profile, strain of outer fiber of confined concrete can be evaluated at ultimate state. For the assessment of deformation limits of rectangular RC columns, sixty-nine experimental studies were selected from PEER Structural Performance Database [14] and analyzed with finite element method. In this study, the adequacy of deformation limits specified by codes and researchers are investigated. The validity of proposed equations was examined using various types of RC column tests. It has been concluded that the performance limits must be refined in terms of member geometry and mechanical characteristics. Moreover, additional extensive analytical studies are needed.

2. CURRENT REGULATIONS AND PREVIOUS STUDIES

2.1. Turkish Seismic Code, 2007 (TSC-07)

For Collapse Limit (CL) concrete and steel strain limits at the fibers of cross section given below.

$$(\varepsilon_{cg})_{CL} = 0.004 + 0.014(\rho_s/\rho_{sm}) \leq 0.018 ; (\varepsilon_s)_{CL} = 0.060 \quad (1)$$

In Eq. (1), ε_{cg} is concrete strain at the outer fiber inside of the lateral reinforcement, ε_s is deformation of reinforcement steel unit, ρ_s is volumetric ratio of existing transverse reinforcement and ρ_{sm} is volumetric ratio of the transverse reinforcement necessary to be existed in the cross section required by the code.

2.2. Turkish Seismic Code, 2016 (Draft) (TSC-16 (Draft))

TSC-16 employs the same approach as the previous version about calculating deformation limits. In the new version, same strain limit requirements are used with multiplying by reduction factor (ϕ). The reduction factor depends on shear strength rate (ν) (Figure 1)

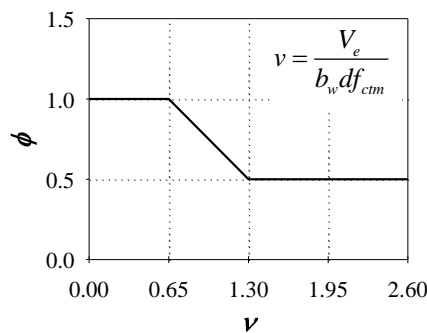


Figure 1. Reduction factor (ϕ)-shear strength rate (ν) relation

2.3. ASCE/SEI 41-13

ASCE/SEI 41 [2] is commonly used throughout the US and internationally for the seismic assessment and retrofit of existing concrete buildings. Modelling parameters for Collapse Prevention (CP) performance level is given in tables in the code book. To replace modeling parameters table in ASCE/SEI 41 [2], linear regression was performed to establish a relation between the chosen parameters and ultimate plastic rotation angle as linear regression can provide a good estimate for median values. Regression equations in place of discrete limits are seen in Table 1.

Table 1. Ultimate plastic rotation angle equations and failure modes

Failure Modes	Ultimate Plastic Rotation Angle (rad)
Condition i Flexure Failure	$0.0292 - 0.047 \left(\frac{P}{A_g f'_c} \right) + 1.625(\rho)$
Condition ii Flexure-Shear Failure where yielding is expected before shear failure	$0.016 + 2.8636\rho - (4.5455\rho + 0.00117) \left(\frac{P}{A_g f'_c} \right) + (0.3636\rho - 0.00202) \left(\frac{V}{b_w d \sqrt{f'_c}} \right)$
Condition iii Flexure-Shear Failure	$0.00678 - 0.013 \left(\frac{P}{A_g f'_c} \right) + 1.5454\rho$

2.4. Eurocode 8 Part-3 (EN 1998-3:2005, EC-8)

The limit states are given based on total chord rotation capacity of structural elements. The value of the total chord rotation capacity (elastic plus inelastic part) at ultimate, θ_u , of concrete members may be calculated from the following expression:

$$\theta_u = \frac{1}{\gamma_{el}} 0.016 (0.3^v) \left[\frac{\max(0.01; \omega')}{\max(0.01; \omega')} \right]^{0.225} \left(\min \left(9; \frac{L_v}{h} \right) \right)^{0.35} 25^{\left(\alpha_{psx} \frac{f_{yw}}{f_c} \right)} (1.25^{100\rho_d}) \quad (2)$$

In addition, EC-8 [1] states a deformation limit for concrete ultimate strain of the extreme fiber of the cross section. Expression is given in Eq. (3) for ultimate strain of the compression zone.

$$\varepsilon_{cm} = 0.004 + 0.5 \frac{\alpha_{ps} f_{yw}}{f_{cc}} \quad (3)$$

2.5. Limits by Researchers

Panagiotakos and Fardis [3] studied on a comprehensive set of experimental tests results to reveal deformation behavior of RC members. They reported an alternative expression Eq. (4) for ultimate chord rotation capacity (θ_u).

$$\theta_{um} = a_{st} a_{cyc} \left(1 + \frac{a_{sl}}{2.3} \right) a_{wall} (0.2^v) \left[\frac{\max(0.01; \omega_2)}{\max(0.01; \omega_1)} \right]^{0.275} \left(\frac{L_s}{h} \right)^{0.45} (1.1^{100\alpha_{wx}}) (1.3\rho_d) \quad (4)$$

Grammatikou et al. claim that the code limits are not safe and general [7]. They investigated the analytical relation between moment and curvature to estimate strains in the bars and the extreme concrete fibers of section. They derived strain limit formula for extreme fiber of concrete (Eq. (5)).

$$\varepsilon_{cu} = 0.035 + 0.04 \sqrt{\alpha \rho_w \frac{f_{yw}}{f'_c}} \quad (5)$$

Haselton et al. [4] used 255 columns test results to create empirical equation that predicts deformation capacity. In their study, statistically significant design parameters were determined and correlation between plastic rotation and design parameters were identified. Following equation is proposed by them (Eq. (6)).

$$\theta_{cap,pl} = 0.12(1 + 0.55a_{sl})(0.16)^v(0.02 + 40\rho_{sh})^{0.43}(0.54)^{0.01c_{units}f'_c}(0.66)^{0.1s_n}(2.27)^{10.0\rho} \quad (6)$$

3. METHODOLOGY AND PARAMETRIC STUDY

3.1. Experimental Database

The database used in this study were taken from PEER Structural Performance Database User's Manual [14] and researcher's published articles. Mechanical and geometric features of each experimentally evaluated rectangular RC column tests and their measured values such as Δ_y , V_{max} , Δ_u , and cyclic loading history and other relevant information were reported.

The selected column database satisfies the following criteria: column aspect ratio, $1.65 < L/h < 7.63$; concrete compression strength, $21 < f_c < 102$ MPa; longitudinal and transverse reinforcement nominal yield stress, f_y and f_{yw} , in the range of 300-580 MPa; longitudinal reinforcement ratio, $0.01 < \rho < 0.036$; transverse reinforcement ratio, $0.00082 < \rho_s < 0.032$ and axial load ratio, $0 < P/P_0 < 0.6$.

3.2. Finite Element Analysis (FEA) Method and Validation

Response of designed columns was calculated using nonlinear finite element analysis program ANSYS v14.0 [15]. Material models for concrete and steel can be found in Kazaz (11-13). Concrete confinement, reinforced bar buckling features and bond-slip model [16] are added in to FE models. Lateral and axial loads were taken as in the experimental set-ups. Each analyses of test specimen were performed under monotonic static loads.

Figure 2(a) and (c) display that there is a good agreement between calculated (FEA) and experimental (EXP) ultimate tip displacements (Δ_u) and base shear force (V_{max}) of the columns. However as shown in Figure 2(b) and (d), tip displacement at yield (Δ_y) and plastic rotation (θ) responses for EXP and FEM results have some differences.

After comparing deformation capacity of specimens, reliability of finite element model was evaluated. By this way, the finite element models' ability to represent the experimental study was verified. As the second step, rebar and concrete core ultimate displacements (Δ_t and Δ_c) were calculated over the plastic hinge length ($L_p=h/2$) then column clear height (h_0) used to calculate base rotation angle (θ) of column. In addition, curvature of section (ϕ), ultimate average strains at core and cover concrete ($\varepsilon_{cu,ave_cover}$, $\varepsilon_{cu,ave}$) and steel ($\varepsilon_{su,ave}$) of column were calculated.

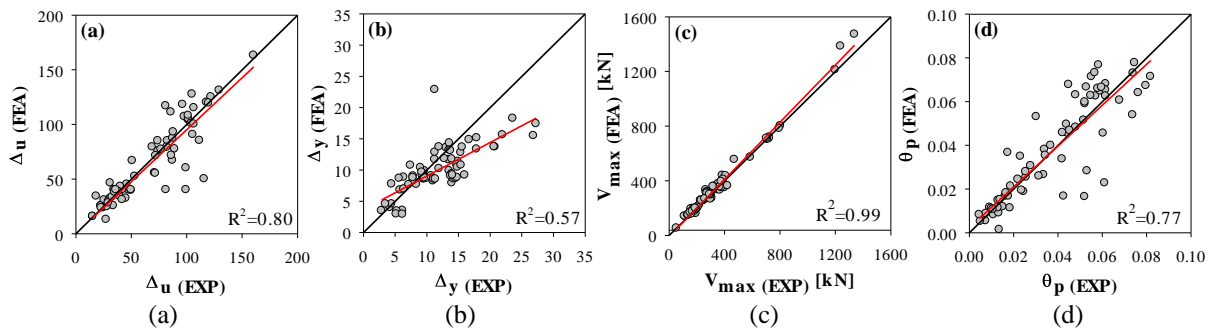


Figure 2. Comparison between experimental and FEM values of: (a) tip displacement; (b) tip displacement at yield; (c) base shear force; (d) plastic rotation

Next, local strains over column length were calculated from each element node displacement. Strain profiles of columns were obtained at the ultimate deformation limit. Also, maximum local concrete strains ($\epsilon_{cu,max}$) and curvature (ϕ_{max}) at core were taken from strain profiles.

4. RELATION WITH FEA AND SECTION ANALYZES

Due to its complexity nonlinear finite element method is not well suited for engineers in practice to calculate the response of reinforced concrete members. All studied columns behavior was investigated by using conventional section analysis (SA) technique to make correlation between FEM and SA results. However, SA does not consider bar buckling, bond-slip, high strength concrete and size effects while calculating the column behavior. CUMBIA, a reinforced concrete section analysis code written in Matlab [17], was used to calculate the moment-rotation and force-deformation response of rectangular RC columns. For each column ultimate curvature values, which are calculated from maximum strains, taken from FEM results and this curvature matched with SA calculation. Both concrete and steel strains at ultimate curvature of column specimens were obtained from SA results. SA strain values were compared with average strains over plastic hinge length $L_p=0.5h$ from FEM results. As seen Figure 3, there are strong agreements with FEM and SA strains which are calculated at the same curvature.

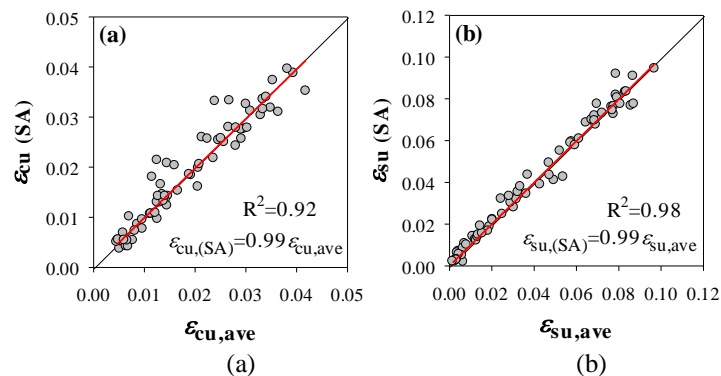


Figure 3. Comparison of ultimate strains from FEM and SA results at the same ultimate curvature (a) concrete ultimate strains and (b) steel ultimate strains

5. RESULTS AND DISCUSSION

5.1. Evaluation of Code and Previously Proposed Limits

ASCE/SEI 41 specify the plastic rotation limits for RC members. When calculated and compared to code plastic rotation limits, the comparison indicates that ultimate plastic rotation limits of ASCE/SEI corresponding to significant loss of lateral-force capacity are mostly conservative. However, for the

columns, which have axial load ratios more than 0.45 and made of high strength concrete (HSC) (70-102 MPa), ASCE/SEI 41 limits are on the unsafe zone (Figure 4(a)) especially in Bayrak and Sheikh [18] tests.

Haselton et al. [4] proposed another equation (Eq. (6)) to determine deformation capacity of columns based on plastic rotation angle. When Figure 4(b) is examined, it is seen that there is an improvement in the stated limit in ASCE/SEI 41 [2]. On the other hand, proposed equation is inadequate for columns made of HSC, aspect ratio is larger than 6, transverse reinforcement diameter larger than 15 mm and have axial load ratio more than 0.45 likewise ASCE/SEI 41 [2]. These nonconforming columns tests were conducted by Saatcioglu and Grira [19], Xiao and Yun [20] and Bayrak and Sheikh [21].

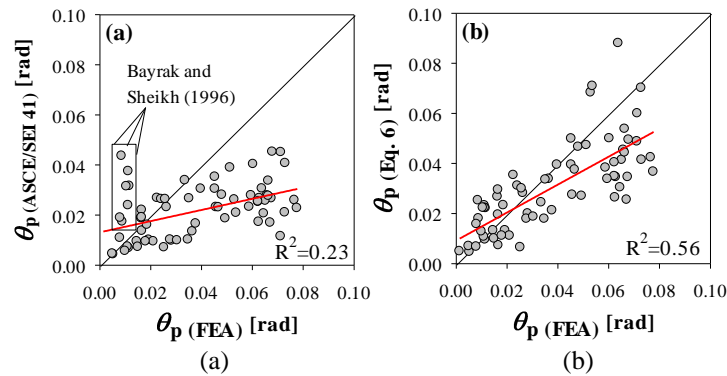


Figure 4. Comparison of calculated and proposed plastic rotation capacities: (a) ASCE/SEI limits; (b) Equation 6 proposed by Haselton et al. [4]

EC-8 [1] and Panagiotakos and Fardis [3] suggest similar deformation capacity limits of beams, columns and walls based on the total chord rotation. Panagiotakos and Fardis proposed an equation after examined 878 RC structural element tests. As seen Figure 5(a), 73% of RC column specimens is at unsafe zone as regard of chord rotation angle when consider EC-8 limit states. Proposed equation from Panagiotakos and Fardis has insignificant improvement over EC8 and just 36% of them are at the safe zone (Figure 5(b)).

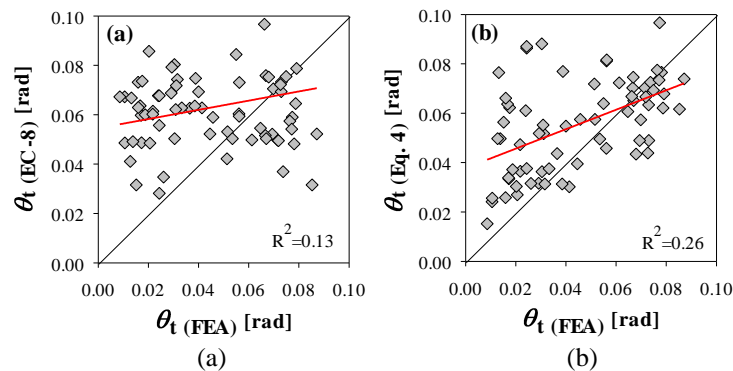


Figure 5. Total chord rotation limits from (a) EC-8 and (b) Panagiotakos and Fardis [3] (Eq. (4)) versus calculated values

Another approach for predicting deformation capacity of RC columns is controlling of concrete and steel strains. As mentioned before TSC [5], EC-8 [1] and Grammatikou et al. [7] set strain limits for structural RC members. Unlike the proposed strain limits, TSC [5] take in consideration of outer core fiber instead of extreme fiber of compression zone. Figure 6(a) compares the calculated core outer fiber strains with TSC code requirements. It has been concluded that 39% of specimens, which subjected to high axial load and aspect ratio, are at the unsafe zone when code limits compared with calculated strain values. Moreover, concrete strain capacities of columns are under estimated. As seen

Figure 6(b), EC-8 has no association between calculated values of concrete strains. Figure 6(c) shows that when proposed equation for extreme concrete fiber strain of compression zone by Grammatikou et al. [7] evaluated by using calculated strain values, majority of column specimens (83%) are in the safe zone. However, the problem for estimating deformation capacity of columns under high axial load level and aspect ratio still exists ($\nu > 0.45$ and $L/h > 6$).

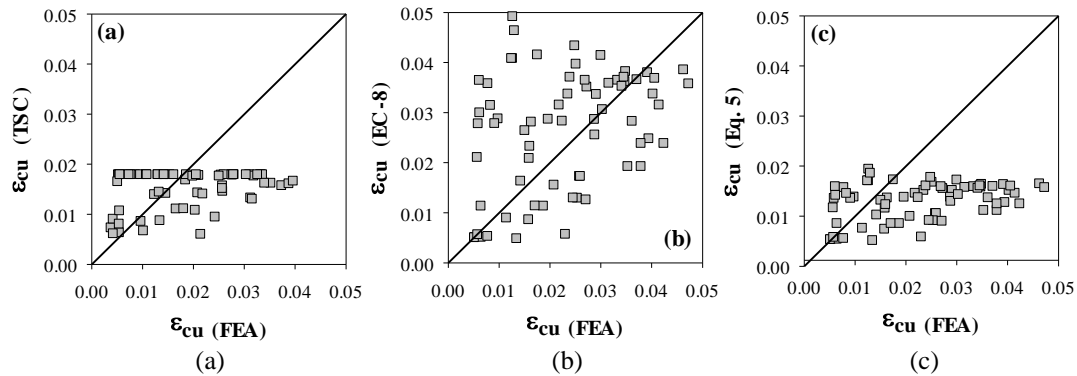


Figure 6. Comparison between concrete strains of FEM and proposed limits: (a) TSC; (b) EC-8; (c) Grammatikou et al. [7] (Eq. (5))

5.2. Relationship of Column Design Parameter with Deformation Criteria

Trends between deformations at ultimate damage states, corresponding to transverse reinforcement ratio, column aspect ratio, axial load ratio, and concrete compression strength were evaluated. These column design parameters are used to calculate column deformation capacities in codes and proposed equations. When transverse reinforcement ratio, column compression strength and axial load ratio are increased, the deformation capacities of RC columns are reduced. Majority of columns were used in this study have transverse reinforcement ratio (ρ_s) more than 0.007 and because of that reason ρ_s become insignificant design parameter. Levels of confinement take important role in determining the deformation capacity of columns. For transverse reinforcement ratio (ρ_s) less than 0.007, ρ_s takes role in concrete strain capacity. However, in the cases where ρ_s level more than 0.007, ρ_s become insignificant design parameter on determining deformation capacity of columns. Aspect ratio (L/h) and axial load ratio (ν) have negative correlations with deformation criteria. Axial load ratio, confinement reinforcement ratio and shear stress are determinant factors on deformation capacities of studied column database.

6. CONCLUSION

Evaluation of the ultimate deformation capacity for RC columns was mentioned in many standards and studies. These standards and studies define different ultimate deformation capacity for RC columns. Besides traditionally used plastic rotation limits, local deformation measures like strain and curvature began to gain acceptance as performance criteria due to developments in the computational field especially owing to nonlinear modelling with fiber-based elements. There are various studies investigating the predictability and reliability of these measures as damage limit mainly employing statistical evaluation of experimental data. However, the strains and curvatures are rarely available from experiments impeding thorough evaluation of strain-based criteria.

This study, using nonlinear finite element modelling strategy investigates deformation measures calculated on 69 experimental column specimens. It is clearly seen that the damage limit expressions in existing regulations and previous studies partially satisfy the need for being a reliable deformation measure as a damage limit.

The authors of this study think that the experimental studies are indispensable to the understanding of the behavior of reinforced concrete members. On the other hand, computational methods may produce considerable insight to the behavior of structural elements where it is difficult to measure local response and enables obtaining results that are more consistent among themselves by conducting large amount of analyses using several variables.

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