Performance based Displacement Limits for Reinforced Concrete Columns under Flexure

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SUMMARY:
Seismic performance of reinforced concrete frame buildings is dominated by columns which can be classified as primary members of these structures. In order to assess the performance of these columns under a given earthquake effect, performance based displacement limits are proposed by certain codes and/or guidelines. These limits are generally given for different damage and performance levels. Adequacy of the displacement limits given in the most common codes such as Eurocode 8, FEMA 356, ASCE41 as well as Turkish Earthquake Code (TDY) 2007 were evaluated by carrying out parametric studies for a set of flexure critical reinforced concrete columns. A total number of 144 flexure critical columns are generated in parametric studies to present the effects of various parameters such as column geometry, concrete strength, axial load ratio, transverse reinforcement ratio, and yielding strength of longitudinal reinforcement on performance based displacement limits. These columns are modeled in OPENSEES and their response is determined. Performance based displacement limits were obtained from analytical results for each performance level in terms of drift ratio. These limits were then compared with the code limits. Performance based displacement limits proposed by TEC (2007), FEMA 356 (2000), and Eurocode 8 (2003) are found very conservative compared to limits obtained from analytical behavior. Improvements on these limits are proposed in the form of expression representing the effect most important parameters.

Keywords: RC Columns, Displacement limits, Performance assessment

1. INTRODUCTION

Determination of expected seismic performance of existing buildings under likely earthquake effects is performed using assessment procedures. Guidelines and codes have recently been prepared to provide procedures and acceptance criteria for seismic assessment of especially Reinforced Concrete buildings. FEMA 356 (2000), Eurocode 8 (EC8 2003) and Turkish Earthquake Code (TEC 2007) are among the most recent ones containing sections on seismic assessment of existing RC buildings. In all of these documents, assessment criteria are given on the basis of components forming the structural system. Columns are the primary and the most important load carrying components for a reinforced concrete frame structure. Most of the failures in previous major earthquakes for reinforced concrete frame structures are directly related to seismic behavior of the columns existing in the frames. The component level evaluations are generally done based on the performance based displacement capacities specified for members. These deformation limits depend on the predominant behavior mode of the members. Behavior mode of RC columns is generally classified as shear, flexure or a combination of these. In FEMA 356 plastic rotation is adopted as the deformation quantity whereas EC8 uses chord rotation as the deformation measure. Unlike others, TEC 2007 gives deformation limits in terms of strains.

This study aims to evaluate the adequacy of the existing common displacement limits given for RC columns that respond primarily in Flexure. A total number of 144 flexure critical columns are generated in parametric studies to present the effects of various parameters such as column geometry, concrete strength, axial load ratio, transverse reinforcement ratio, and yielding strength of longitudinal reinforcement on performance based displacement limits. Deformation limits of columns were first determined from their load-displacement plots obtained from finite element analysis for different performance levels and later compared with the limits given in FEMA 356, EC8 and TEC2007.

2. PROPERTIES OF COLUMN SPECIMENS

The columns generated for the study have four different cross sectional types that are commonly used. Those sections are 400 mm x 400 mm and 500 mm x 500 mm square columns, 300 mm x 500 mm and 300 mm x 600 mm rectangular columns. To handle the deficiencies arisen from the properties of selected database, additional columns are analyzed by utilizing OpenSees (2005).

Concrete compressive strength is taken as 10 MPa, 14 MPa, and 20 MPa. Selected yielding strength of longitudinal reinforcement is 220 MPa and 420 MPa, and for the transverse reinforcement this value is 420 MPa. Transverse reinforcement ratio is selected as 0.0075 and 0.02. For all specimens, longitudinal reinforcement ratios are the same with the value of 0.01. Also, spacing between the transverse reinforcement is considered as 100 mm for all specimens. In addition, axial load ratios are chosen as 0.10, 0.25, and 0.40 to reflect accurate properties of the columns constructed in Turkey. After applying these parameters, 144 column specimens are generated. The range of the parameters used in this part of the study is summarized in Table 2.1.

Table 2.1. Range of Parameters Used in Parametric Study

Dimension (mm)	f_{ck} (MPa)	N/N_o	ρ	f_{yk} (MPa)	ρ_s	f_{ywk} (MPa)
400x400	10	0.10	0.01	220	0.0075	420
500x500	14	0.25		420	0.02	
300x500	20	0.40				
300x600						

Where:

f_{ck}: Concrete compressive strength

N/N_0: Axial load ratio

ρ: Longitudinal reinforcement ratio

f_{yk}: Yielding strength of reinforcement

ρ_s: Transverse reinforcement ratio

f_{ywk}: Yielding strength of transverse reinforcement

3. MODELING AND ANALYSIS

Seismic behavior of flexure critical reinforced concrete columns are influenced by several parameters such as axial load ratio, concrete compressive strength, yielding strength of reinforcement, longitudinal reinforcement ratio, and transverse reinforcement ratio. Thus, these parameters are modeled accurately using the computer program developed by PEER (Opensees 2005). Displacement controlled nonlinear static analyses were performed by applying incremental displacements at the tip of the column. While applying incremental displacements, at the critical section of the flexural member, element forces in addition to stress-strain values of concrete and steel are recorded for each step. Also, chord rotations are calculated by dividing recorded tip displacement values to column length (half-height). As a result, for each step material strains, chord rotations, and element forces are related to each other. This relationship provides an opportunity to estimate displacement limits for each performance level in terms of material strains or chord rotations.

3.1. Modeling of Columns

Reinforced concrete flexural components are modeled by using the concept of fiber analysis. Unidirectional concrete and steel fibers represent the flexural component in this type of analysis. A fiber section has a general geometric configuration formed by sub-regions of simpler, regular shapes (quadrilateral, circular or triangular regions) called patches. In addition, layers of reinforcement bars can be specified. Concrete and steel fiber behaviors can be modeled and defined in the direction of the member length, so fiber based analysis may be used for all type of flexural components regardless of cross sectional shape and the direction of the horizontal load.

Modified Kent and Park model (1982) is used to define concrete behavior accurately. Kent and Park model (1982) was modified with the idea inspired from the relationship of Roy and Sozen confined concrete model (1964). For confined and unconfined concrete regions, two different stress-strain relations are applied. For both unconfined and confined concrete models, first part of the stress-strain curves has a second order parabolic region as Hognestad model suggests. Second part of the curves that show the decreasing part of the stress values, are first order lines with negative slope. Negative slope of confined part is smaller than the negative slope of unconfined part. Unconfined concrete has a limited maximum strain ε_{cu}=0.004, on the other hand for confined concrete there is no such a limitation of compressive strain. In Opensees (2005), “Concrete01 Material” is used to construct a uniaxial Kent and Park concrete material object with degraded linear unloading/reloading stiffness according to the work of Karsan-Jirsa with zero tensile strength.

The stress versus strain curve of reinforcing steel has been taken identical in compression and tension. Idealized one dimensional stress-strain curve is convenient in order to provide the simplicity of calculations. It was assumed that reinforcing steel has a linear hardening region. In Opensees (2005), “Steel01 Material” is used to construct a uniaxial bilinear steel material object with kinematic hardening and optional isotropic hardening described by a nonlinear evolution equation.

3.2. Modeling of Slip

Experimental behavior of reinforced concrete members, which are subjected to both static and dynamic loads, is extremely related to the effects of bond slip. Strain penetration effects are caused by the slippage between fully fixed longitudinal reinforcing bars and the concrete section of the connecting member. In linear and nonlinear analyses of reinforced concrete members, ignoring the effects of strain penetration causes miscalculation of the deflections and member elongations. In addition member stiffness and hysteretic energy dissipation capacity are overestimated if one ignores the strain penetration effects. In order to estimate the member end rotation due to reinforcement slip, bond slip effects were accurately modeled by using a zero-length section element accessible in OpenSees (2005). Zero-length section element is used in a section analysis for which the strain values of concrete and steel fibers are calculated. Stress-strain relationships of concrete and steel fibers are necessary for the calculation of fiber forces which provide the section moment by integrating these forces all over the section.

Figure 3.1 illustrates the difference between considering bond-slip effects and ignoring them. Member stiffness and hysteretic energy dissipation capacity are overestimated when bond-slip effects are ignored. In addition deflections and member elongations are miscalculated while modeling without bond-slip effects. For the specimen shown in Figure 3.1, capacity curve obtained from analytical study fits well with the backbone curve obtained from experimental study. Additionally, analytical studies showed that 15-20% of the drift ratio at collapse prevention performance level (ultimate point) is caused by the effects of bond-slip.

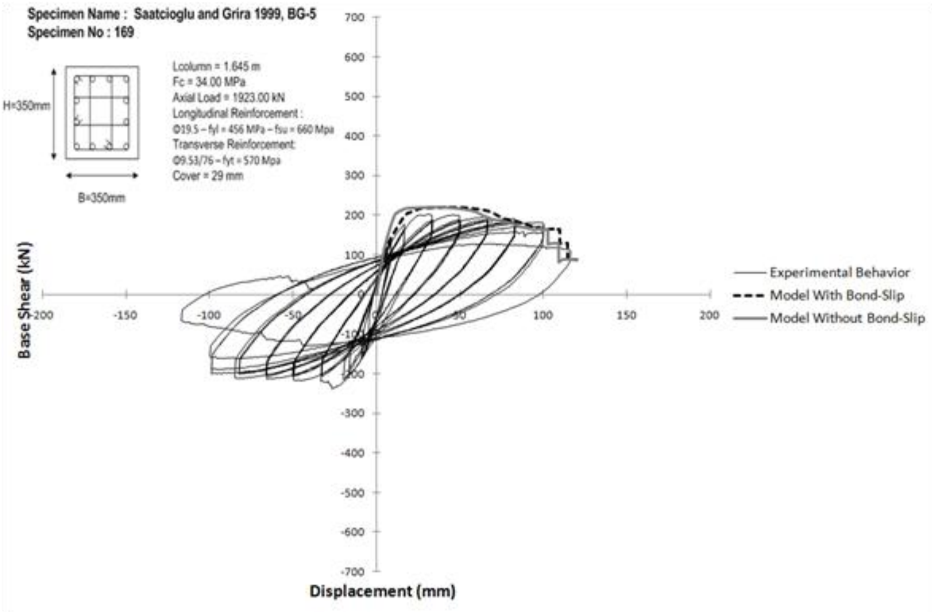

Figure 3.1. Effect of Modeling Bond-Slip on Envelope Response

4. DETERMINATION OF COLUMN DEFORMATION LIMITS

From nonlinear static analyses capacity curve of each column is obtained and approximated as bilinear curve. These curves have an initial elastic branch until the stiffness changes, and afterwards plastic behavior is observed. Priestley et. al. (2007) defined first yield point as the point where the outer part of tension reinforcement reaches yielding strain or the outer part of concrete fiber reaches the strain value of 0.002. The line which is obtained from the first yield point is extended up to the point where the outer part of tension reinforcement reaches the strain value of 0.015 or the outer part of concrete fiber reaches the strain value of 0.004. The elastic part obtained from this extrapolated line is combined with the plastic part up to the ultimate point to complete the bilinear curve.

In this study, estimated immediate occupancy is a performance limit where the elastic branch gives place to plastic branch. Thus, performance limit of immediate occupancy (IO) has a strain value of 0.015 for outer part of the tension reinforcement or strain value of 0.004 for the outer part of concrete fiber. In addition to this, estimated performance limit of collapse prevention (CP) is the ultimate point where the shear capacity drops 20 percent from the maximum value. Estimated performance limit of life safety (LS) is taken as 75 percent of the ultimate point (estimated collapse prevention). Figure 4.1 shows the performance limits on an idealized capacity curve.

The deformation limits estimated for each performance level are determined using the criteria explained above. These deformation limits are then compared to the limits given by various guidelines and codes as discussed in the next section.

4.1. Deformation Limits in Codes and Guidelines

In the scope of this study, three different seismic assessment guidelines were evaluated. These guidelines are respectively Turkish Earthquake Code (2007), American pre-standard, FEMA 356 (2000) and European seismic code, Eurocode 8 (2003).

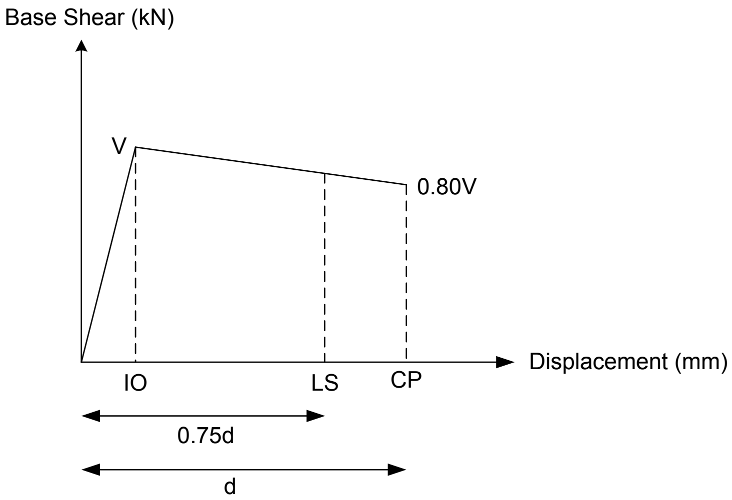

Figure 4.1. Estimated Performance Limits

4.1.1. Turkish Seismic Code (2007)

Turkish Earthquake Code (2007) is the one which was revised in 2007 as a result of the deficiencies based on the seismic code written in 1998. The new code consists of a part that includes provisions for seismic assessment and retrofitting methods of existing buildings. In order to assess and decide the performance level of existing reinforced concrete structures, performance based analysis procedure is required. This procedure is totally different from the one which consists of force based capacity design method. To estimate performance level of a structure, all components in the critical sections should be investigated for a code specified demand.

In the nonlinear static procedure of TEC, in order to predict the performance level, the strain limits of concrete and steel are used as the main parameters. In addition, Turkish Earthquake Code (2007) defines three damage levels based on the ductility capacity and predicted failure mode. Seismic performance of a structure can be determined by considering the distribution of structural damage along the building. For a reinforced concrete column, sectional damage state should be calculated by determining the strain values of concrete fibers and reinforcement. Strain limits are provided as:

For Immediate Occupancy (IO):

$$(\varepsilon_{cunconfined})_{IO} = 0.0035 \,;\, (\varepsilon_s)_{IO} = 0.01 \tag{4.1}$$

For Life Safety (LS):

$$(\varepsilon_{cconfined})_{LS} = 0.0035 + 0.01\left({\rho_s}/{\rho_{sm}}\right) \leq 0.0135 \,;\, (\varepsilon_s)_{LS} = 0.04 \tag{4.2}$$

For Collapse Prevention (CP):

$$(\varepsilon_{cconfined})_{CP} = 0.004 + 0.014\left({\rho_s}/{\rho_{sm}}\right) \leq 0.018 \,;\, (\varepsilon_s)_{CP} = 0.06 \tag{4.3}$$

Where;
$\varepsilon_{cunconfined}$: Cover concrete strain at the outer fiber of the unconfined region
$\varepsilon_{cconfined}$: Core concrete strain at the outer fiber of the confined region

ε_s : Steel strain at the critical section
ρ_s : Volumetric ratio of the confinement reinforcement present at the critical section
ρ_{sm}: Volumetric ratio of the confinement reinforcement required at the critical section

Using the performance limits specified by TEC, deformation limits for each performance level were calculated for each column. Deformation limits were converted to chord rotation, plastic rotation and drift using section analysis for the columns. In section analyses, length of plastic zone was taken as half of the depth of the member as suggested by the code.

4.1.2. FEMA (1997)

FEMA 356 (2000) is the American pre-standard and commentary for the seismic rehabilitation of buildings. The document presents deformation limits, which are in the form of plastic rotations, for different structural members and these limits should be compared with the deformation demands obtained from nonlinear structural analysis. This guideline has then been upgraded to ASCE 41 (2007).

In FEMA 356, deformation limits are specified in terms of plastic rotation for reinforced concrete columns. These limiting values are directly related with the dominated mode of behavior (shear or flexure), $N/(bdf_c)$ ratio (axial load ratio), the spacing of stirrups, and $(V/(b_w d\,(f_c)^{1/2})$ ratio. For flexure critical columns selected from PEER database (2005), deformation limits for each performance level were calculated using the plastic rotation limits given in FEMA 356. These plastic rotation limits were then converted to chord rotation and drift values to be able to make comparisons with the limits specified by other specifications.

4.1.3. EUROCODE 8 (2003)

Eurocode 8 (2003) includes a part for the assessment of reinforced concrete columns that recommends the calculation of chord rotations with the given equations in the code. These equations are functions of many variables such as axial load ratio, longitudinal reinforcement ratio, transverse reinforcement ratio, and yield strength of the transverse reinforcement. Calculated chord rotations should be compared with the demands obtained from nonlinear analysis.

In Eurocode 8, three limit states that correspond to the previously mentioned performance levels are employed; Damage Limitation (DL), Significant Damage (SD) and Near Collapse (NC). For each limit state a corresponding chord rotation value is given. The total chord rotation capacity for the limit state of NC (sum of elastic and plastic behavior) should be calculated from the following expression:

$$\theta_{um} = \frac{1}{\gamma_{el}} 0.016(0.3^{\nu}) \left[\frac{\max(0.01;\omega')}{\max(0.01;\omega)} f_c\right]^{0.225} \left(\frac{L_v}{h}\right)^{0.35} 25^{\left(\alpha\rho_{sx}\frac{f_{yw}}{f_c}\right)} (1.25^{100\,\rho_d}) \tag{4.4}$$

Where;
γ_{el}: Equal 1.5 for the primary seismic elements and 1.0 for secondary seismic elements
h: Depth of the cross-section
$L_v = M/V$; Ratio of moment versus shear at the end section
$\nu = N/bhf_c$ (b: Width of the compression zone, N: Axial force)
ω, ω': Mechanical reinforcement ratio of the tension and compression
f_c, f_{yw}: Concrete compressive strength and steel yield strength (MPa)

$\rho_{sx} = A_{sx}/b_w s_h$; Ratio of transverse steel to parallel to the direction x of loading (s_h is the stirrup spacing)
ρ_d: Steel ratio of diagonal reinforcement
α: Confinement effectiveness factor

$$\alpha = \left(1 - \frac{s_h}{2b_o}\right)\left(1 - \frac{s_h}{2h_o}\right)(1 - \frac{\sum b_i^2}{6h_o b_o})$$

b_o, h_o: Dimensions of confined core to the centerline of the hoop
b_i: Centerline spacing of longitudinal bars laterally restrained by a stirrup corner or a cross-tie along the perimeter of the cross-section

The total chord rotation capacity corresponding to limit state of significant damage is calculated from the following expression:

$$\theta_{SD} = \frac{3}{4}\theta_{um} \tag{4.5}$$

For the limit state of DL the chord rotation that corresponds to the yield rotation is expressed as:

$$\theta_y = \phi_y \frac{L_v + \alpha_v z}{3} + 0.00135\left(1 + 1.5\frac{h}{L_v}\right) + \frac{\varepsilon_y}{d - d'}\frac{d_b f_y}{6\sqrt{f_c}} \tag{4.6}$$

In Equation 4.6, ϕ_y: Yield curvature of the end section, $\alpha_v z$: Tension shift of the bending moment diagram, f_y: Steel yield stress, ε_y: Steel strain at yielding and d_b: Diameter of the tension reinforcement.

Using the expression of EC8, chord rotation limits were calculated for each column in the database that were later converted to drift limits for each limit state.

4.2. Comparisons of Performance Limits

Flexure critical columns selected for parametric study are analyzed and capacity curves are obtained. According to TEC (2007), FEMA 356 (2000), EC 8 (2003), and ASCE/SEI 41 Update (2009), performance limits for each performance level are determined.

Performance limits corresponding to each performance level obtained by different seismic guidelines were compared. Tables 4.1 and 4.2 illustrate statistical analysis results obtained according to different seismic guidelines. Table 4.1 shows real drift ratios; on the other hand Table 4.2 shows normalized drift ratios corresponding to each seismic guideline. For the performance level of immediate occupancy, average drift ratios for TEC (2007), Eurocode 8 (2003), FEMA 356 (2000), and ASCE/SEI 41 (2009) are 1.03, 0.66, 0.88, and 0.91%, respectively. For the performance level of life safety, average drift ratios for TEC (2007), Eurocode 8 (2003), FEMA 356 (2000), and ASCE/SEI 41 (2009) are 1.77, 2.17, 1.81, and 2.22%, respectively. For the performance level of collapse prevention, average drift ratios for TEC (2007), Eurocode 8 (2003), FEMA 356 (2000), and ASCE/SEI 41 (2009) are respectively 2.22, 2.89, 2.20, and 2.81%. According to evaluation of capacity curves obtained from analytical studies, estimated drift ratios for each performance level have average values of 0.51, 2.53, and 3.37 percent. When standard deviations are compared, it is seen that FEMA 356 provides closer and smaller standard deviation values for all performance levels compared to other seismic provisions.

Table 4.1. Statistical Analysis Results Obtained According to Different Seismic Guidelines

		Drift Ratio (%)		
Procedure	**Stat.**	**IO**	**LS**	**CP**
TEC 2007	μ	1.03	1.77	2.22
	σ	0.25	0.40	0.68
	cov	0.24	0.23	0.31
FEMA 356	μ	0.88	1.81	2.20
	σ	0.12	0.14	0.19
	cov	0.14	0.08	0.09
EC 8	μ	0.66	2.17	2.89
	σ	0.52	0.57	0.76
	cov	0.78	0.26	0.26
ASCE/SEI 41 Update	μ	0.91	2.22	2.81
	σ	0.12	0.42	0.59
	cov	0.14	0.19	0.21
ESTIMATED	μ	0.51	2.53	3.37
	σ	0.14	1.09	1.45
	cov	0.27	0.43	0.43

Table 4.2. Statistical Analysis Results Obtained for the Comparison of Different Seismic Guidelines

		Drift Ratio (%)		
Procedure	**Stat.**	**IO**	**LS**	**CP**
TEC 2007	μ	2.17	0.80	0.72
	σ	0.83	0.28	0.19
	cov	0.38	0.35	0.26
FEMA 356	μ	1.81	0.87	0.78
	σ	0.36	0.43	0.37
	cov	0.20	0.49	0.47
EC 8	μ	1.24	0.97	0.97
	σ	0.20	0.34	0.34
	cov	0.16	0.35	0.35
ASCE/SEI 41 Update	μ	1.86	1.01	0.95
	σ	0.36	0.37	0.34
	cov	0.19	0.37	0.36
ESTIMATED	μ	1.00	1.00	1.00
	σ	0.00	0.00	0.00
	cov	0.00	0.00	0.00

4.3. Proposed Equations for Performance Levels

According to detailed examination of seismic behavior of selected columns, main parameters affecting displacement capacities of assessed columns were determined as axial load ratio, concrete compressive strength, yielding strength of reinforcement, transverse reinforcement ratio, slenderness ratio and normalized shear stress (Solmaz, 2010). In order to generate appropriate equations for immediate occupancy, life safety, and collapse prevention performance levels, these parameters are considered.

Nonlinear regression analyses are performed and Equations 4.7, 4.8, and 4.9 are obtained with R^2 values of 0.738, 0.738, and 0.822, respectively.

Drift ratio (percent) at collapse prevention performance level can be calculated by Equation 4.7.

$$(DR)_{CP} = -1.42 + 70.212\rho_s + 5.324(0.01)^{N/N_0} + 0.0074\left(\frac{V}{b_w d\sqrt{f_c}}\right)^{-0.818} + 0.00142 f_{yk} + 0.822\left(L/_H\right) - 0.077 f_c \tag{4.7}$$

Drift ratio (percent) at life safety performance level can be calculated by Equation 4.8.

$$(DR)_{LS} = 0.75(DR)_{CP} \tag{4.8}$$

Drift ratio (percent) at immediate occupancy performance level can be calculated by Equation 4.9;

$$(DR)_{IO} = -0.30 + 0.001 f_{yk} + 0.16\left(L/_H\right) \tag{4.9}$$

Where:
ρ_s: Volumetric ratio of transverse reinforcement
$N/_{N_o}$: Axial load ratio
V: Shear force at the critical section
b_w : Width of the web reinforcement
d: Flexural depth of the section
f_c: Concrete compressive strength
f_{yk}: Yielding strength of longitudinal reinforcement
$L/_H$: Slenderness ratio

Estimated versus calculated drift ratios are plotted in Figure 4.2 for both PEER database (2005) and parametric study columns for immediate occupancy, life safety, and collapse prevention performance levels. Calculated values indicate the ones computed from the proposed equations.

5. CONCLUSIONS

In this study, seismic behavior of reinforced concrete columns, whose failure modes were flexural, was evaluated analytically with a finite element analysis program OpenSees (2005). Performance based displacement limits for each performance levels were compared with the limits obtained from different seismic provisions. New relationships that take into account the influential parameters are proposed to improve the code recommended performance based displacements limits.

For most of the flexure critical columns, TEC (2007), FEMA 356 (2000), Eurocode 8 (2003), and ASCE/SEI 41 Update (2009) provide underestimated seismic performance. When these columns reach performance level of collapse prevention according to these seismic assessment guidelines, all of them do not even reach performance level of life safety according to analytical results. Eurocode 8 (2003) and ASCE/SEI 41 Update (2009) provide closer performance limits compared to TEC (2007) and FEMA 356 (2000).

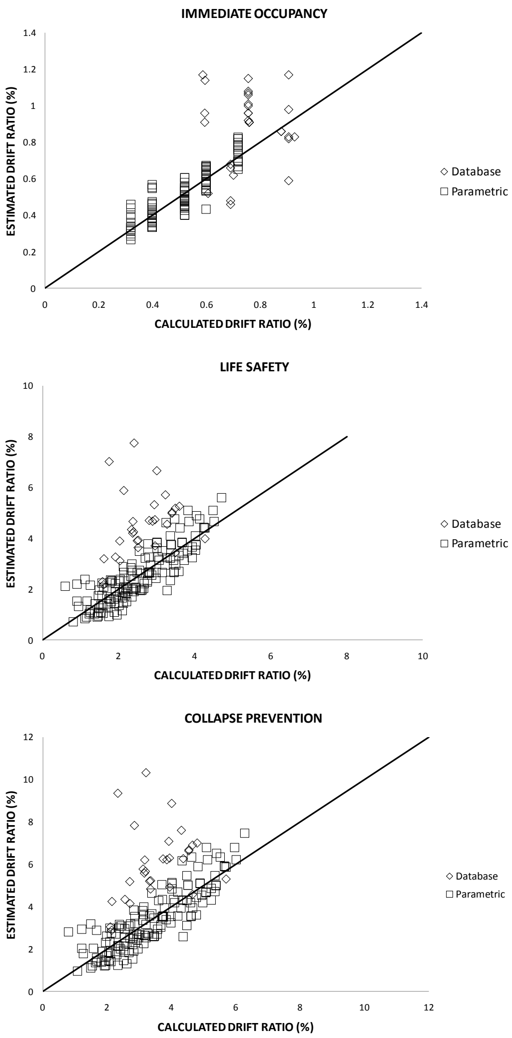

Figure 4-2 Comparison of Estimated and Calculated Drift Ratios

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REFERENCES

European Committee for Standardization (2003). Eurocode 8: Design of Structures for Earthquake Resistance, Part 3: Strengthening and Repair of Buildings, Eurocode 8, Brussels.

Federal Emergency Management Agency (FEMA) (2000). Prestandart and Commentary for the Seismic Rehabilitation of Buildings, FEMA-356, Washington, D.C.

Open System for Earthquake Engineering Simulation (OpenSees) (2005). Pacific Earthquake Engineering Research Center, University of California, Berkeley, http://opensees.berkeley.edu/, last visited date 25/08/2010.

PEER (2005). Pacific Earthquake Engineering Center: Structural Performance Database, University of California, Berkeley, http://nisee.berkeley.edu/spd/ and http://maximus.ce.washington.edu/~peera1/, last visited date 24/08/2010.

Priestley, M.J.N., Calvi, G.M., Kowalsky, M.K. (2007). Displacement-Based Seismic Design of Structures, IUSS Press, Pavia.

TEC, Ministry of Public Works and Settlement, Government of Republic of Turkey (TEC) (2007). Specifications for Structures to be Built in Disaster Areas, Ankara.

Solmaz, T., (2010). Evaluation of Performance Based Displacement Limits for Reinforced Concrete Columns Under Flexure. Master’s Thesis, Middle East Technical University, Ankara.