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SEISMIC PERFORMANCE ASSESSMENT OF REINFORCED CONCRETE STRUCTURES

(BETONARME YAPILARIN SİSMİK PERFORMANSININ DEĞERLENDİRİLMESİ)

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ABSTRACT/*ÖZET*

In this study, determination of the performance levels of structural systems by the Capacity Spectrum Method and the Displacement Coefficients Method, which are used to determine performance levels of structures by considering structural capacity obtained from pushover analysis is intended. For this purpose, five-storey reinforced concrete 3D frame system structures having the same floor plan are considered in the analysis. Also, to observe the differences in the performance levels, a sample structure having same story heights in the first phase and then first story height is increased in order to examine the weak story irregularity is taken into consideration. Structures are designed in accordance with Turkish Standards (TS 500) and the Turkish Earthquake Code. Life safety (LS) structural performance level is chosen as a target for the sample structures under a design earthquake that may be exceeded in a 50-year period with 10 percent probability. The capacity curves of the sample structures are obtained from pushover analysis and their performance levels are determined by the Capacity Spectrum Method and the Displacement Coefficients Method.

Bu çalışmada yapı sistemlerinin artımsal itme analizi ile belirlenen yapısal kapasitesini esas alan ve yapıların performans seviyesinin belirlenmesinde kullanılan Kapasite Spektrumu Yöntemi ve Deplasman Katsayıları Yöntemi ile performans seviyelerinin belirlenmesi amaçlanmıştır. Bu amaçla, kat planları aynı olan beş katlı betonarme çerçeveli sistemli yapılar dikkate alınmıştır. Ayrıca performans seviyelerindeki farklılıkları görmek üzere seçilen yapının önce tüm katları aynı yükseklikte tasarlanmış, daha sonra zemin kat yükseklikleri arttırılarak ortaya çıkan zayıf kat düzensizliğinin etkisi irdelenmeye çalışılmıştır. Yapılar, TS 500 ve DBYBHY 2007 kuralları çerçevesinde tasarlanmıştır.Kullanılan yapı sistemleri için 50 yıllık süreç içinde aşılma olasılığı %10 olan tasarım depremi etkisi altında can güvenliği (CG) performans seviyesi hedeflenmiştir. Artımsal itme analizi ile yapıların kapasite eğrileri elde edilmiş ve Kapasite Spektrumu Yöntemi ve Deplasman Katsayıları Yöntemi ile performans seviyeleri belirlenmiştir.

KEYWORDS/*ANAHTAR KELİMELER*

Pushover analysis, Capacity spectrum method, Displacement coefficients method, Performance level
Artımsal itme analizi, Kapasite spektrumu yöntemi, Deplasman katsayıları yöntemi, Performans seviyesi

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1. INTRODUCTION

Various analysis methods, both elastic (linear) and inelastic (nonlinear), are available for the analysis of reinforced concrete structures. The methods that take part in codes and used for analyses of structures under lateral loads, are generally based on linear-elastic behaviour of structures under earthquake effects. In linear-elastic analysis of structural systems, elastic earthquake forces are reduced according to defined reduction factor, which varies based on typical inelastic response of structural systems. Although an elastic analysis gives a good indication of the elastic capacity of structures and indicates where first yielding will occur, but it can not predict failure mechanism of structure and account for redistribution of forces during the progressive yieldings (ATC 40, 1996).

The damages and the economical losses during the last major earthquakes (Loma Prieta earthquake and Northridge earthquake), introduced a new approach in seismic design of structures called ‘Performance Based Design and Evaluation’. In performance based design and evaluation of structures under earthquake effects, it is necessary to determine the nonlinear behaviour of structures. Both nonlinear time history analysis and nonlinear static analysis procedures are used for this objective. The most basic inelastic analysis method, known as the Time History Analysis, is considered very complex and impractical for general use. For this reason, nonlinear static analysis methods have become popular.

The central focus of the nonlinear static analysis methods is the generation of the capacity curve or pushover curve. This curve represents lateral displacement as a function of the force applied to the structure. The use of nonlinear static analysis methods for design and evaluation helps engineers to understand better how structures will behave when subjected to major earthquakes, where it is assumed that the elastic capacity of the structure will be exceeded (ATC 40, 1996).

The most common used nonlinear static analysis procedures for the evaluation of the performance levels of structures are the Capacity Spectrum Method, which uses the intersection of the capacity curve with a reduced response spectrum to estimate the maximum displacement and the Displacement Coefficients Method, which provides a direct numerical process for calculating the displacement demand.

2. CONCEPTUAL DEVELOPMENT OF THE CAPACITY SPECTRUM METHOD

Application of the Capacity Spectrum Method requires both the demand response spectra and structural capacity curves to be plotted in the spectral acceleration vs. spectral displacement format which is known as Acceleration-Displacement Response Spectra (ADRS). In order to convert a spectrum from the traditional spectral acceleration, S_a vs period, T format found in the building code to ADRS format, it is necessary to determine the value of S_{di} for each point on the curve, S_{ai}, T_i (Figure 1). This can be done by Equation 1 (ATC 40, 1996).

$$S_{di} = S_{ai} \frac{T^2}{4\pi^2} \tag{1}$$

In order to use the Capacity Spectrum Method, it is also necessary to convert the capacity curve obtained from pushover analysis in terms of base shear, V_T and roof displacement, δ_{max} to the capacity spectrum (Figure 2). Capacity spectrum is the representation of the capacity curve in ADRS format. This transformation can be done by using Equation 2 and Equation 3.

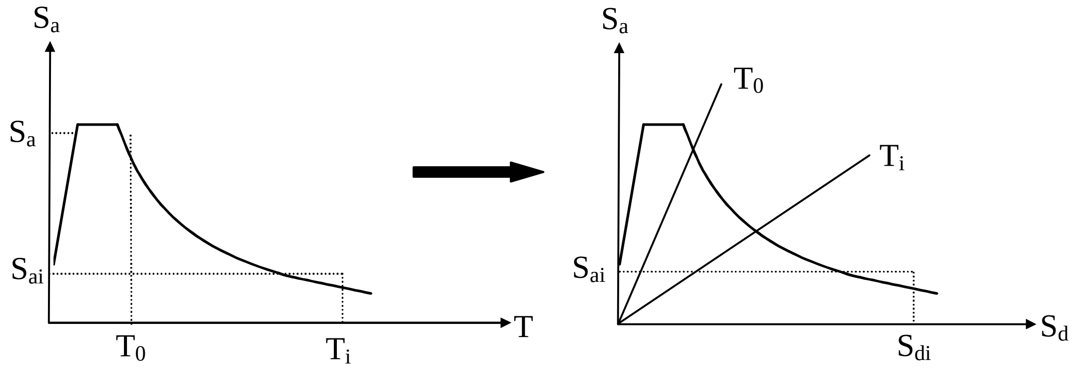

Figure1. Response spectra in traditional and ADRS format

$$S_a = \frac{V_T / W}{\alpha_1} \tag{2}$$

$$S_d = \frac{\delta_{max}}{PF_1 \phi_{roof,1}} \tag{3}$$

In Equation 2 and Equation 3, the coefficients α_1 and PF_1 are calculated as follows in Equation 4 and Equation 5.

$$\alpha_1 = \frac{\left[\sum_{i=1}^{N} (w_i \phi_{i,1} / g)\right]^2}{\left[\sum_{i=1}^{N} (w_i / g)\right]\left[\sum_{i=1}^{N} (w_i \phi_{i,1}^2 / g)\right]} \tag{4}$$

$$PF_1 = \left[\frac{\sum_{i=1}^{N} (w_i \phi_{i,1} / g)}{\sum_{i=1}^{N} (w_i \phi_{i,1}^2 / g)}\right] \tag{5}$$

In these equations, S_a is spectral acceleration, S_d is spectral displacement, PF_1 is modal participation of the first natural mode, α_1 is modal mass coefficient for the first natural mode, N is the uppermost level in the main portion of the structure, W is total building weight (dead weight of building plus likely live loads), W_i/g is mass assigned to level i, $\phi_{i,1}$ is amplitude of mode 1 at level i and $\phi_{roof,1}$ is normalized amplitude of mode 1 at roof level.

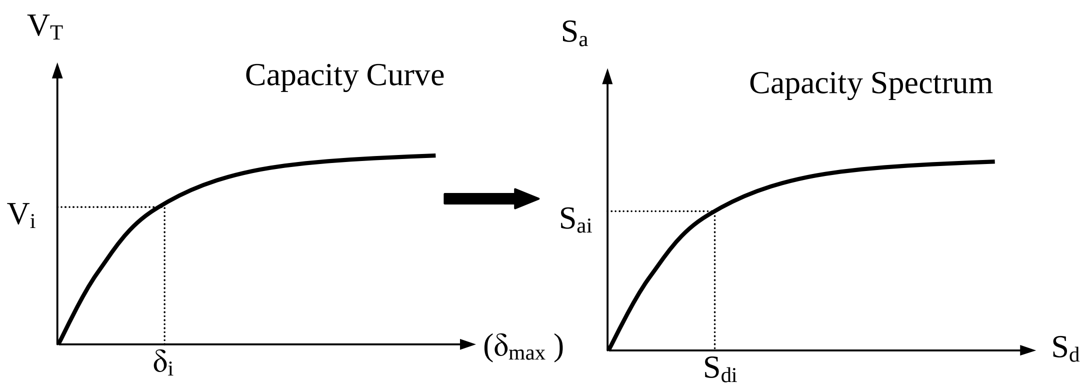

Figure 2. Conversion of capacity curve to capacity spectrum

After the representation of response spectra and capacity curves in ADRS format, an initial performance (maximum acceleration a_{pi} and displacement d_{pi}) point is selected. This may be based on equal-displacement approximation as shown in Figure 3 or any other point based on engineering judgment (FEMA 440, 2004).

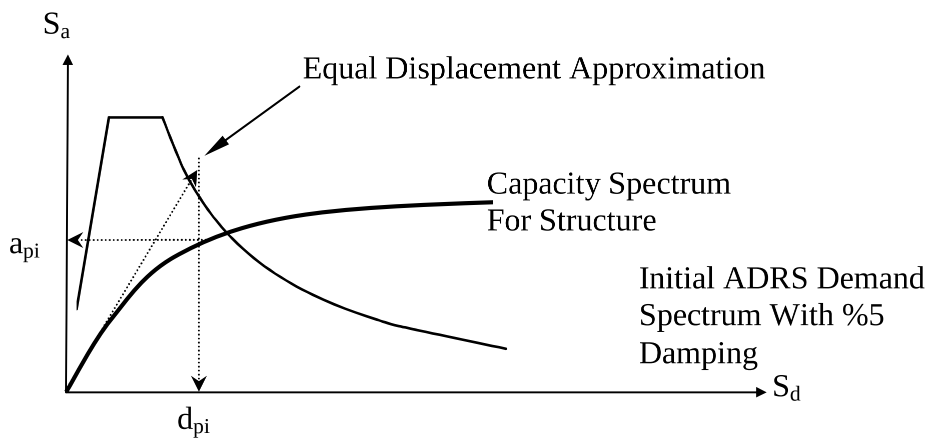

Figure 3. Selection of initial performance point

Another step in the Capacity Spectrum Method is the construction of bilinear representation of capacity spectrum. A bilinear representation of the capacity spectrum is needed to estimate the effective damping and appropriate reduction of spectral demand. This bilinearezation defines the initial period, T_0, yield displacement, $d_{y,}$ and yield acceleration, a_y (Figure 4).

By using the values obtained from bilinear representation of capacity spectrum, the values of post-elastic stiffness, α, and ductility, μ, can be calculated as follows in Equation 6 and Equation 7 (FEMA 440, 2004).

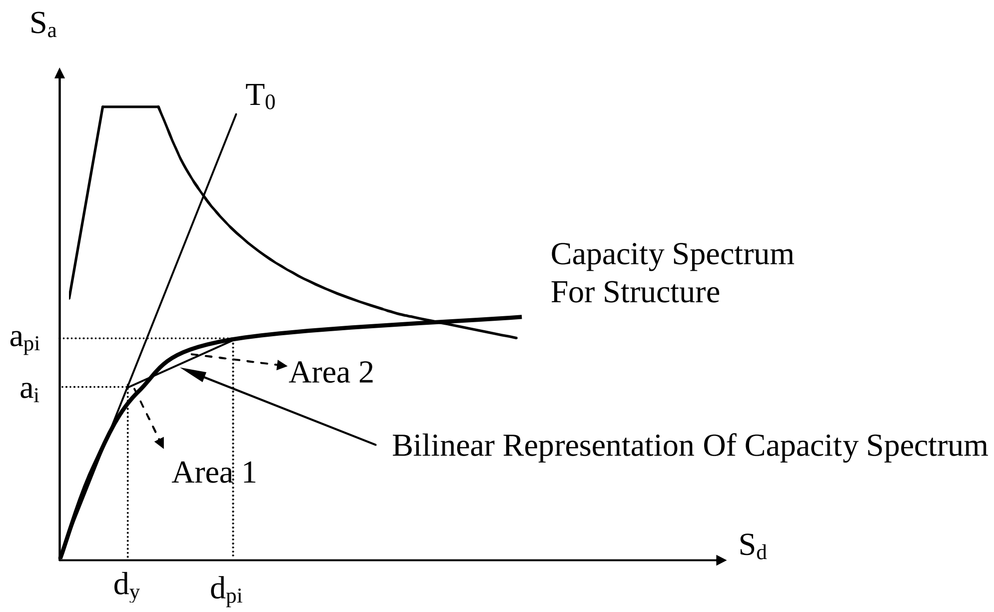

Figure 4. Bilinear representation of capacity spectrum

$$\alpha = \frac{\left(\dfrac{a_{pi} - a_y}{d_{pi} - d_y}\right)}{\left(\dfrac{a_y}{d_y}\right)} \tag{6}$$

$$\mu = \frac{d_{pi}}{d_y} \tag{7}$$

By using the calculated values of post-elastic stiffness and ductility, the corresponding effective damping, β_{eff}, and the corresponding effective period, T_{eff}, can be calculated. Effective viscous damping values for all hysteretic model types and alpha values have the following form, where β_0 is hysteretic damping (FEMA 440, 2004):

$$\beta_{eff} = A(\mu-1)^2 + B(\mu-1)^3 + \beta_0 \quad \text{for } \mu < 4.0 \tag{8a}$$

$$\beta_{eff} = C + D(\mu-1) + \beta_0 \quad \text{for } 4.0 \le \mu \le 6.5 \tag{8b}$$

$$\beta_{eff} = E\left[\frac{F(\mu-1)-1}{F(\mu-1)^2}\right]\left(\frac{T_{eff}}{T_0}\right)^2 + \beta_0 \quad \text{for } \mu > 6.5 \tag{8c}$$

Values of the coefficients in these equations for effective damping of the hysteretic model type are given in Table 1 (FEMA 440, 2004).

Table 1. Coefficients to be used in equations 8a, 8b, 8c for effective damping

Hysteretic Model Type	**α (%)**	**A**	**B**	**C**	**D**	**E**	**F**
Stiffness Degrading	0	5.1	-1.1	12	1.4	20	0.62
	2	5.3	-1.2	11	1.6	20	0.51
	5	5.6	-1.3	10	1.8	20	0.38
	10	5.3	-1.2	9.2	1.9	21	0.37
	20	4.6	-1.0	9.6	1.3	23	0.34

Effective period values for all hysteretic model types and alpha values have the following form (FEMA 440, 2004):

$$T_{eff} = \left[G(\mu-1)^2 + H(\mu-1)^3 + 1\right]T_0 \quad \text{for } \mu < 4.0 \tag{9a}$$

$$T_{eff} = \left[I + J(\mu-1) + 1\right]T_0 \quad \text{for } 4.0 \le \mu \le 6.5 \tag{9b}$$

$$T_{eff} = \left\{K\left[\sqrt{\frac{(\mu-1)}{1+L(\mu-2)}} - 1\right] + 1\right\}T_0 \quad \text{for } \mu > 6.5 \tag{9c}$$

Values of the coefficients in the equations for effective period of the hysteretic model type are shown in Table 2 (FEMA 440, 2004).

Using the effective damping, β_{eff}, $B(\beta_{eff})$ which is used to adjust spectral acceleration ordinates, is calculated (FEMA 440, 2004).

$$B = \frac{4}{5.6 - \ln\beta_{eff}\,(\%)} \tag{10}$$

Spectral acceleration ordinates are adjusted with the equation given below (FEMA 440, 2004).

$$(S_a)_\beta = \frac{(S_a)_{\%5}}{B(\beta_{eff})} \tag{11}$$

Table 2. Coefficients to be used in equations 9a, 9b, 9c for effective period

Hysteretic Model Type	α (%)	G	H	I	J	K	L
Stiffness Degrading	0	0.17	-0.032	0.10	0.19	0.85	0.00
	2	0.18	-0.034	0.22	0.16	0.88	0.02
	5	0.18	-0.037	0.15	0.16	0.92	0.05
	10	0.17	-0.034	0.26	0.12	0.97	0.10
	20	0.13	-0.027	0.11	0.11	1.00	0.20

The estimated maximum displacement, d_i, is determined by using the intersection of the radial effective period, T_{eff}, with the ADRS adjusted for β_{eff}. The estimated maximum acceleration, a_i, corresponds to d_i on the capacity curve (Figure 5).

The estimated maximum displacement, d_i, is compared with the initial assumption. If it is within acceptable tolerance ($0.95d_{pi} \leq d_i \leq 1.05d_{pi}$), the performance point corresponds to a_i and d_i. If it is not within the acceptable tolarance, the procedure mentioned above is repeated.

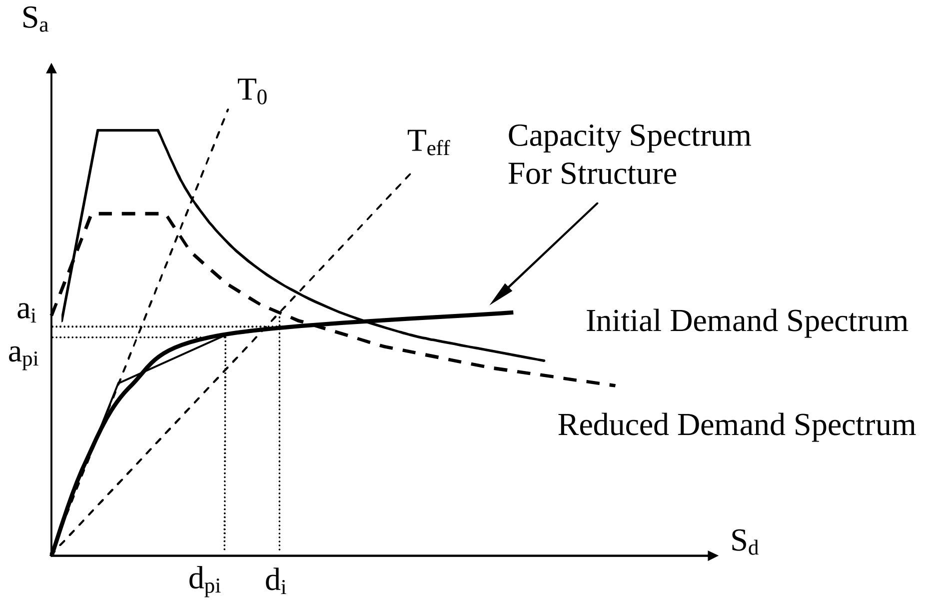

Figure 5. Determination of estimated maximum displacement using direct iteration

The obtained a_i and d_i values are converted to base shear and maximum roof displacement by using the following equations.

$$V_T = \alpha_1 S_a W \tag{12}$$

$$\delta_{max} = PF_1 \phi_{roof,1} S_d \tag{13}$$

To determine the performance level of a structure, the structure is statically pushed to the performance point. Comparing the maximum displacement value and internal force-deformation states with the limit values, the performance level of the structure can be determined (İrtem and Türker, 2002) (Figure 6).

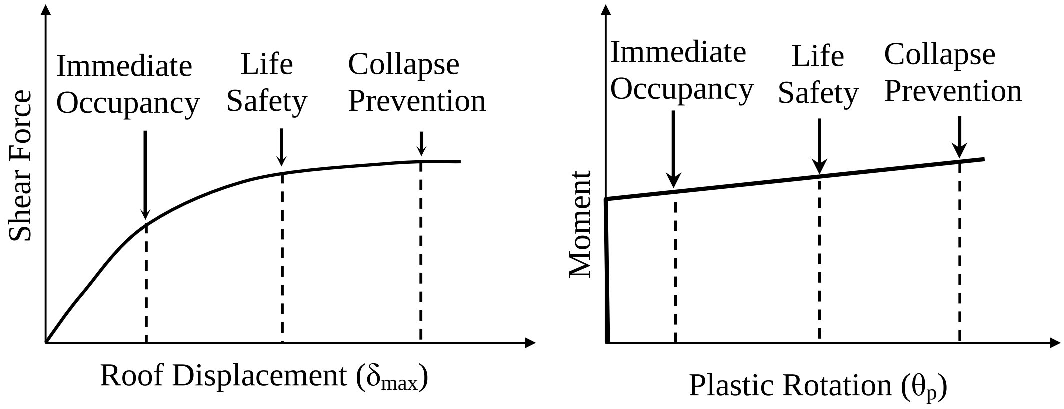

Figure 6. Determination of performance level

3. THE DISPLACEMENT COEFFICIENTS METHOD

The Displacement Coefficients Method provides a direct numerical process for calculating the displacement demand. It does not require converting the capacity curve into spectral coordinates.

The nonlinear force-displacement relationship between base shear and displacement shall be replaced with an idealized relationship to calculate the effective lateral stiffness, K_e, and effective yield strength, V_y, of the structure. This relationship shall be bilinear, with initial slope K_e and post yield slope K_s. Line segments on the idealized force-displacement curve shall be located using an iterative graphical procedure that approximately balances the area above and below the curve. The effective lateral stiffness, K_e, shall be taken as the secant stiffness calculated at the base shear force corresponding to 60% of the effective yield strength of the structure (ATC 40, 1996) (Figure 7).

The effective fundamantal period in the direction under consideration shall be based on the idealized force-displacement curve and can be calculated in accordance with the Equation 14.

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \tag{14}$$

where T_i is elastic fundamental period in the direction under consideration calculated by elastic dynamic analysis, T_e is effective period of the structure, K_i is elastic lateral stiffness of the structure in the direction under consideration and K_e is effective lateral stiffness of the structure in the direction under consideration.

The target displacement shall be calculated in accordance with Equation 15 (FEMA 440, 2004).

$$\delta_T = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \tag{15}$$

In this equation C_0 is modification factor to relate spectral displacement of an equivalent single degree of freedom system to the roof displacement of multi degree of freedom system. C_0 can be taken as the first modal participation factor ($PF_1 \phi_{roof,1}$) at the level of the control node, the modal participation factor at the level of the control node is calculated using a shape vector corresponding to the deflected shape of the structure at the target displacement or the appropriate value from Table 3.

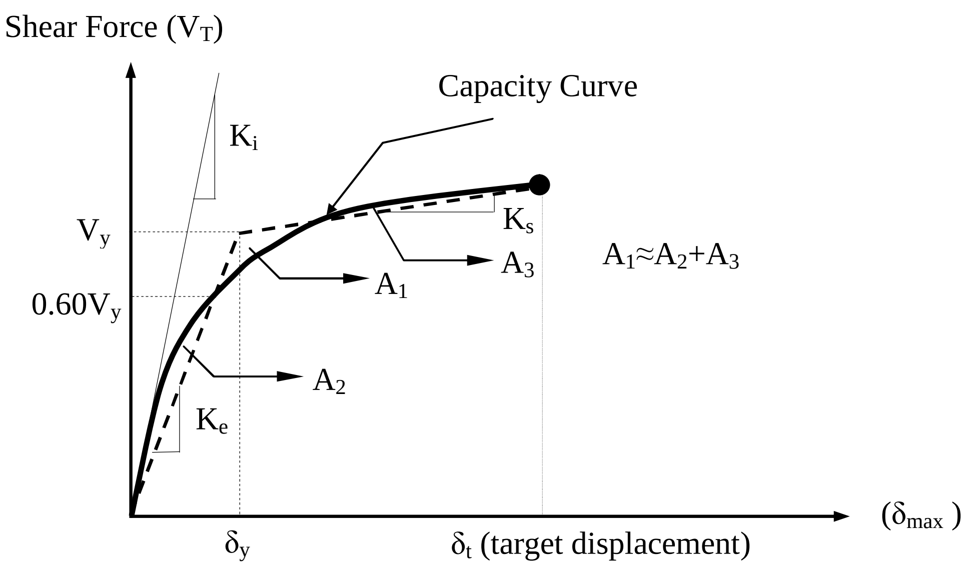

Figure 7. Idealization of capacity curve

Table 3. Values for modification factor C_0

Number of Stories	**Shear Buildings**		**Other Buildings**
	Triangular Load Pattern	**Uniform Load Pattern**	**Any Load Pattern**
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

C_1 is the modification factor which relates expected maximum inelastic displacements to displacements calculated for linear elastic response. C_1 can be determined from Equation 16.

$$C_1 = 1 + \frac{R-1}{aT_e^2} \qquad (16)$$

where T_e is the effective fundamental period of the single degree of freedom model of the structure in seconds and R is the strength ratio computed with Equation 17. The constant , a, is equal to 130, 90 and 60 for the site classes B, C and D, respectively.

$$R = \frac{S_a}{V_y / W} C_m \qquad (17)$$

C_m is the effective mass factor from Table 4. C_m shall be taken as 1.0 if the fundamental period, T, is greater than 1.0 second. For periods greater than 1.0 sec., C_1 may be assumed to be 1.0.

C_2 is the modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration on maximum displacement response. It is recommended that the C_2 coefficient to be as follows.

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 \qquad (18)$$

C_1 may be assumed to be equal to 1.0 for periods greater than 0.7 sec.

Table 4. Values for effective mass factor C_m

No. of Stories	Concrete Moment Frame	Concrete Shear Wall	Concrete Pier-Spandrel	Steel Moment Frame	Steel Concentric Braced Frame	Steel Eccentric Braced Frame	Other
1-2	1.0	1.0	1.0	1.0	1.0	1.0	1.0
≥3	0.9	0.8	0.8	0.9	0.9	0.9	1.0

4. CASE STUDIES

To determine the performance levels, five-storey reinforced concrete 3D frame system structures having the same floor plan are considered in the analyses. To observe the differences in the performance levels, the above mentioned five-storey reinforced concrete structure is taken into consideration with the same story heights (story height=3m) and then in the second phase, the first story height (first story heights=5m) is increased intentionally to examine the weak story irregularity.

Two five-storey reinforced concrete 3D frame system structures are designed geometrically and materially in accordance with TS 500 and the Turkish Earthquake Code (TS500, 2000; DBYBHY, 2007). Life safety structural performance level is chosen as a target for the sample structures under a design earthquake that may be exceeded in a 50-year period with 10 percent probability (DBYBHY, 2007).

In the study; the five-storey reinforced concrete 3D frame system structure with the same story heights is symbolized as RC 5.1., the five-storey reinforced concrete 3D frame system structure with the increased first story height is symbolized as RC 5.2..

The floor plan of the structures considered in the analyses are given in Figure 8.

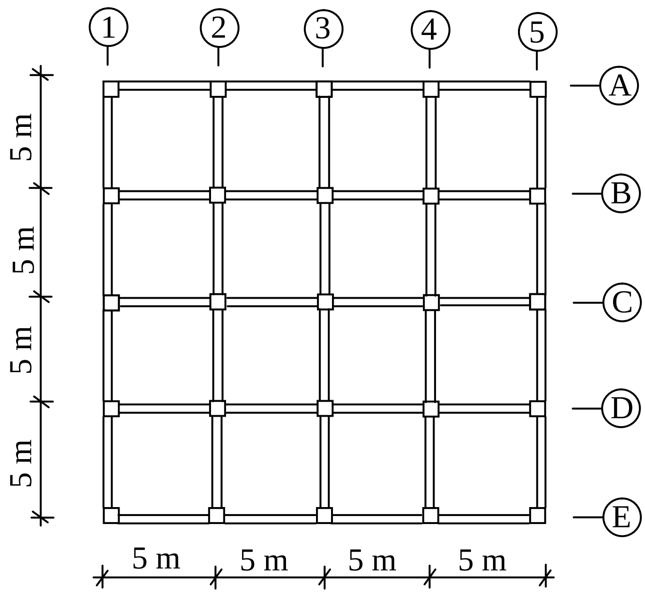

Figure 8. Floor plan for the RC 5.1. and the RC 5.2..

The parameters used in the design of the five-storey reinforced concrete 3D frame system structures and the dimensions of structural members are given in Table 5.

Table 5. The parameters used in the design of the five-storey reinforced concrete 3D frame system structures and the dimensions of the structural members

Five-storey Reinforced Concrete 3D Frame System Structures (RC 5.1. and RC 5.2.)	
Earthquake Zone	1
Effective Ground Acceleration Coefficient, A_0	0.40
Soil Type	Z2
Structure Importance Coefficient, I	1
Structural System Behaviour Coefficient, R	8
Concrete Type	C20
Steel Type	S420
Slab Thickness (cm)	12
Dimensions of Beams (cm) [In all stories]	25x50
Dimensions of Columns (cm) [In all stories]	50x50

4.1. Pushover Curves of The Structures

To obtain the pushover curves of the structures SAP 2000 Structural Analysis Programme is used (CSI SAP 2000 V-8.1.5, 2002). In pushover analyses of the structures, plastic hinge hypothesis is taken into consideration. In that hypothesis, it is assumed that plastic deformations are considered to gather in sections called plastic hinge and other parts of system behave linear elastic. For effective stiffness values, the values given in FEMA 356 are used. In the performance evaluations of the structures, limit values for different performance levels given in FEMA 356 in terms of plastic hinge rotation are taken into consideration (FEMA 356, 2000). Capacity curves obtained from the pushover analyses are presented in Figure 10 for the sample structures.

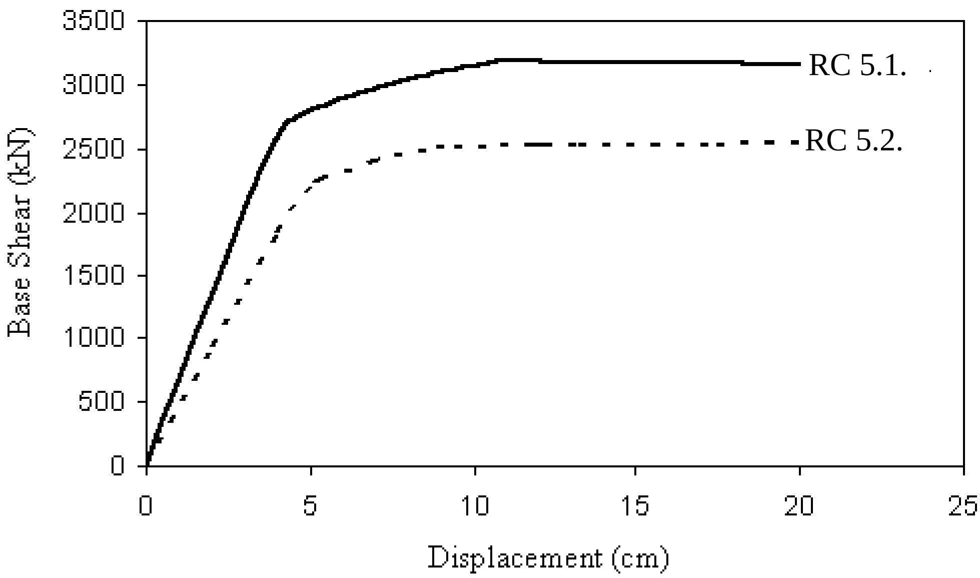

Figure 10. Capacity curves of the five-storey reinforced concrete 3D frame system structures

The results obtained from the Capacity Spectrum Method For the sample structures are presented below. In Table 7, δ_{max} is the displacement value and V_T is the shear force value at the calculated performance point.

Table 7. Performance point values of the structures obtained by the Capacity Spectrum Method

Structure Type	**$PF_1 f_{roof,1}$**	**α_1**	**T_{eff} (sec)**	**B_{eff} (%)**	**S_a (g)**	**S_d (cm)**	**δ_{max} (cm)**	**V_T (kN)**
RC 5.1.	1.275	0.828	1.192	15.800	0.225	10.40	13.260	3160.040
RC 5.2.	1.225	0.931	1.410	13.197	0.155	13.20	16.170	2525.928

By performing the calculation steps of the Displacement Coefficients Method, the performance point of the sample structures are determined and the corresponding values of that point are given in Table 8.

Table 8. Performance point values of the structures obtained by the Displacement Coefficients Method

Structure Type	**C_0**	**C_1**	**C_2**	**S_a (g)**	**T_e (sec)**	**δ_{max} (cm)**	**V_T (kN)**
RC 5.1.	1.275	1.026	1.009	0.576	0.797	12.000	3180.5181
RC 5.2.	1.225	1.017	1.000	0.468	1.034	15.478	2531.7773

4.2. Determination of Performance Levels of the Structures

The performance levels of the sample structures are determined by both using the Capacity Spectrum Method and the Displacement Coefficients Method. The numbers of plastic hinges occurred in structural members, the maximum plastic hinge rotations, the maximum story drifts and the performance levels which are determined with the two methods, are given below.

4.2.1. Determination of Performance Levels by Using the Capacity Spectrum Method

To evaluate the performance levels of the sample structures, the structures are statically pushed to the target displacement value, which was determined by the Capacity Spectrum Method. The numbers of plastic hinges in beams and columns, the maximum plastic hinge rotations in the performance point and the maximum total drifts of the structures statically pushed to the performance point are given in Table 10, Table 11 and Table 12 respectively.

Table 10. The numbers of plastic hinges according to performance levels

Structure Type	**Plastic Hinge Numbers According To Performance Levels**							
	Beam				Column			
	<IO	IO-LS	LS-CP	>CP	<IO	IO-LS	LS-CP	>CP
RC 5.1.	60	140	---	---	23	2	---	---
RC 5.2.	62	78	42	---	---	23	2	---

Table 11. The maximum plastic hinge rotations

Structure Type	**Maximum Plastic Hinge Rotation Values (rad)**	
	Beam	Column
RC 5.1.	0.009666	0.002635
RC 5.2.	0.011929	0.007892

Table 12. The maximum story drifts

Structure Type	Maximum Story Drift (%)	Performance Level
RC 5.1.	1.146	IO-LS
RC 5.2.	1.319	IO-LS

According to the values given in Table 10, Table 11 and Table 12, the performance levels of the sample structures are determined. The performance level of the five-storey reinforced concrete 3D frame system structure with the same story heights (RC 5.1.) is found to be between the Immediate Occupancy and the Life Safety. For the five-storey reinforced concrete 3D frame system structure with the increased first story height (RC 5.2.), the performance level between the Life Safety and the Collapse Prevention is obtained.

4.2.2. Determination of Performance Levels by Using the Displacement Coefficients Method

To evaluate the performance levels of the sample structures, the structures are statically pushed to the target displacement value, which was determined by the Displacement Coefficients Method. The numbers of plastic hinges in beams and columns, the maximum plastic hinge rotations in the performance point and the maximum total drifts of the structures statically pushed to the performance point are given in Table 13, Table 14 and Table 15 respectively.

Table 13. The numbers of plastic hinges according to performance levels

Structure Type	Plastic Hinge Numbers According To Performance Levels							
	Beam				Column			
	<IO	IO-LS	LS-CP	>CP	<IO	IO-LS	LS-CP	>CP
RC 5.1.	69	131	---	---	25	---	---	---
RC 5.2.	70	80	32	---	---	25	---	---

Table 14. The maximum plastic hinge rotations

Structure Type	Maximum Plastic Hinge Rotation Values (rad)	
	Beam	Column
RC 5.1.	0.008731	0.00180
RC 5.2.	0.011345	0.00733

Table 15. The maximum story drifts

Structure Type	Maximum Story Drift (%)	Performance Level
RC 5.1.	1.060	IO-LS
RC 5.2.	1.262	IO-LS

Using the values given in Table 13, Table 14 and Table 15, the performance levels of sample structures are determined. The performance level of the five-storey reinforced concrete 3D frame system structure with the same story heights (RC 5.1.) is found to be between the Immediate Occupancy and the Life Safety. For the five-storey reinforced concrete 3D frame system structure with the increased first story height (RC 5.2.), the performance level between the Life Safety and the Collapse Prevention is obtained.

5. CONCLUSIONS

In this study, it is intended to determine the performance levels of structural systems under earthquake effects by using the Capacity Spectrum Method and the Displacement Coefficients Method, which are both used to determine performance levels of structures by considering structural capacity obtained from pushover analysis. For this purpose, five-storey reinforced concrete 3D frame system structures having the same floor plan are considered in the analyses. To observe the differences in the performance levels, the above mentioned five-storey reinforced concrete structure is taken into consideration having the same story heights in the first phase and then the first story height is increased intentionally in order to examine the weak story irregularity.

Two five-storey reinforced concrete 3D frame system structures are designed geometrically and materially in accordance with TS 500 and the Turkish Earthquake Code. Life safety structural performance level is chosen as a target for the sample structures under a design earthquake that may be exceeded in a 50-year period with 10 percent probability.

Considering the performance levels for the sample structures obtained by the Capacity Spectrum Method and the Displacement Coefficients Method, it can be said that the mentioned two methods gives nearly the same performance levels.

According to the performance evaluation with the Capacity Spectrum Method and the Displacement Coefficients Method, the five-storey reinforced concrete 3D frame system structure with the same story heights (RC 5.1.) can easily get the Life Safety performance level under the design earthquake. For the five-storey reinforced concrete 3D frame system structure with the weak story irregularity (RC 5.2.), the obtained performance levels are worse than the performance levels of the five-storey reinforced concrete 3D frame system structure having the same story heights (RC 5.1.). In the sample structure with the weak story irregularity, the values of plastic hinge rotations in beams and columns (especially in the first story colums) and the story drifts are increased.

Considering the results obtained from this study, the earthquake behaviour of the structure with the weak story irregularity is in the negative aspect. In case with weak story irregularity, the total earthquake force that the structure can resist, which is called the seismic capacity of the structure, is decreased.

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