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Performance Analysis of A Reinforced Concrete Frame System According to TBEC-2018

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Abstract

Turkey is located on active earthquake zone so earthquake resistant building design becomes more important with the rapidly increasing population and urbanization. In the great earthquakes occurred in our country for centuries, many people lost their lives after the earthquake damaged the buildings and this also increase the importance of building an earthquake resistant structure. The performance based seismic design evaluates how the buildings are likely to implement under an earthquake motion and is comprised linear elastic and nonlinear elastic methods in recent seismic codes. In this study, the performance based design of a four-storey and three-span reinforced concrete frame system is performed according to the Turkish Building Earthquake Code (TBEC-2018). The nonlineer static pushover analysis of the reinforced concrete (RC) frame system carried out for DD-2 level earthquake and it has been determined whether it has the performance criteria targeted in the code.

Betonarme Bir Çerçeve Sistemin TBDY-2018 Yönetmeliğine Göre Performans Analizi

Anahtar Kelimeler;
Depreme Dayanıklı Yapı Tasarımı, Statik İtme Analizi, Performansa Dayalı Tasarım, TBDY-2018

Özet

Aktif deprem kuşağı üzerinde bulunan Türkiye'de hızla artan nüfus ve şehirleşmeyle beraber depreme dayanıklı yapı tasarımı daha da önemli hale gelmektedir. Ülkemizde yüzyıllardır meydana gelen büyük depremlerde, deprem sonrası yapıların büyük hasar görmesiyle birçok kişi hayatını kaybetmiş ve aynı zamanda bu, depreme dayanıklı yapı yapmanın önemini daha da arttırmıştır. Özellikle son yıllarda performansa dayalı tasarım kavramı, tasarım ve değerlendirme yöntemlerindeki gelişmelerle birlikte öne çıkmaktadır. Performansa göre tasarımda, tasarım depremi altında yapımızın nasıl bir davranış sergileyebileceği ve deprem sonrası yapımınız durumunun önceden belirlenmesi amaçlanmıştır. Performansa dayalı değerlendirme yöntemleri mevcut deprem yönetmeliklerinde doğrusal ve doğrusal olmayan yöntemler olarak iki ana başlık altında değerlendirilmektedir. Bu çalışmada Türkiye Bina Deprem Yönetmeliği (TBDY 2018)'ne göre 4 katlı ve 3 açıklıklı betonarme bir düzlem çerçeve sistemin performansa dayalı tasarımı gerçekleştirilmiştir. Düzlem çerçeve sistemin artımsal statik itme analizi DD-2 düzeyinde deprem için yapılmış ve yönetmelikte hedeflenen performans kriterlerini sağlayıp sağlamadığı belirlenmiştir.

1 INTRODUCTION

Structures subject to significant inelastic deformation under an earthquake motion and this causes changes in the dynamic characteristics of structures such as the natural frequencies and damping ratios with time. Therefore, determining the real behavior of structure under seismic loading requires inelastic analytical procedures to obtain dynamic characteristics of structures. The use of inelastic analysis methods instead of the traditional elastic analysis methods helps us about how a structure behaves under an earthquake. Inelastic analysis procedure on nonlineer analysis includes inelastic static and inelastic time history analyses. Inelastic time history analysis is the most accurate method to predict the force and deformation demands at various components of the structure. Inelastic time history analysis must be used for assessment post-elastic behavior cannot be implemented directly by an elastic analysis. However, the use of inelastic time history analysis is limited and is impractical because dynamic responses are very susceptible of ground motion characteristics and modeling of the system. Therefore, a simplified nonlinear analysis procedure is developed to evaluate inelastic seismic demands by the researchers. Inelastic static analysis, which is also known as pushover analysis, is the widely used simplified nonlinear static analysis procedure due to being uncomplicated and its simplicity.

In the pushover analysis, the structure undergoes vertical load and gradually increasing lateral load distributed along the building height. The equivalent static lateral loads approximately represent earthquake-induced forces. The structural loading is incrementally increased in compliance with an accurate predefined pattern. The total base shear forces versus top displacements in a structure are obtained by this analysis that may occur any failure or damage. The analysis is performed up to failure and collapse load and ductility capacity are determined. The capacity (pushover) curve, which identifies the behavior of a structure under increasing lateral loads, is obtained from the analysis for the building. The target displacement is determined based on the capacity curve. Many methods were presented to apply the nonlinear static pushover to structures. These inelastic static analysis procedures can be listed as Capacity Spectrum Method (ATC-40, 1996), Displacement Coefficient Method (FEMA-356, 2000) and the Secant Method (COLA, 1995), constant ductility procedure (modal pushover analysis) (Chopra and Goel, 2001). In the pushover analysis, plastic yielding effects will dominate in the inelastic performance of RC structures due to behave highly inelastic under seismic loads. Therefore, the accuracy of the pushover analysis depends on the ability of the analytical models, which accurately represent these effects. Generally, analytical models for the pushover analysis may be divided into two main types for frame structures: the first is distributed plasticity (plastic zone) and the second is concentrated plasticity (plastic hinge). In this study, incremental single mode pushover analysis, which become an acting analysis for performance based design and has been extensively applied in practice for seismic design, is performed according to the TBEC-2018 for modeling four-stories and three-bay RC simple plane frame with commercial finite element software package, SAP2000. In structural model, dimensions of beams and columns are chosen according to minimum design conditions of TBEC-2018. All beams $30x50cm^2$ and columns $45x45cm^2$ are selected. According to nonlineer pushover analysis results the upper limit values of the strains corresponding to the cross-sectional damage level of the sections are obtained and damage limits and damage states of the considered system have been determined.

Fajfar and Fischinger (1987) determined stiffness, strength and supplied ductility by the nonlinear static analysis of a MDOF system under a monotonically increasing lateral load. Bracci et. al (1997) proposed a procedure about the use of stiffness-dependent lateral force distributions in which story forces are proportional to story shear resistances at the previous step for evaluating the seismic performance and retrofit of existing low-to-mid rise RC buildings. It was obtained that the procedure can provide reliable estimates of story demands versus capacities for use in seismic performance and retrofit assessment of the structures. Krawinkler and Seneviratna (1998) dealt with the pros and cross of pushover analysis by taking into account different aspects of the method. Sasaki et al. (1998) developed the multi-mode pushover procedure to try to account for the effects of higher modal response and determine failure mechanisms due to higher modes in a pushover analysis. In the study, it was explained the steps to perform multi-mode pushover procedure and applied the method to several buildings. They used capacity spectrum method and structure's capacity (pushover curve) for each mode was compared with earthquake demand by using Capacity Spectrum Method (ATC-40, 1996). Gupta (1999) analyzed the recorded responses of eight real buildings that experienced ground accelerations to understand and to evaluate the behavior of the structures. Kim and D'Amore (1999) set out to assess pushover analysis in comparison with inelastic time history procedures. Mwafy and Elnashai (2001) performed a series of pushover analyses and incremental dynamic collapse analyses to investigate the validity and the applicability of pushover analysis. They considered twelve RC buildings according to different parameters, such as structural systems, design accelerations and design ductility levels. Moghadam (2002) proposed a procedure to quantify the effects of higher mode responses in tall buildings and performed a series of pushover analysis using elastic mode shapes as load pattern. Inel and Ozmen (2006) investigated the possible differences in the results of pushover analysis due to default and user-defined nonlinear component properties. Four- and seven-story buildings are considered to represent low and medium rise buildings located in a high-seismicity region of Turkey. It is obtained from the study the user-defined hinge model is better than the default-hinge model in reflecting nonlinear behavior compatible with the element properties. Chaudhari and Dhoot (2016) is used the non-linear static procedures to analyze the performance of a four-storey RC building under lateral loads. Atmaca et al. (2018) investigated relative floor displacements for linear time history analyses of a six-storied reinforced concrete building by using real and scaled earthquake records. Çavdar (2019) used performance-based design method to determine the level of expected performance of the structures under the earthquake effects.

2 METHOD

2.1 Pushover Analysis

Pushover analysis is a method, which consists of a series of sequential elastic analyses, to evaluate earthquake performance of the structures due to its computational simplicity steps. The aim of the analysis is to estimate its strength and deformation demands in design seismic motions by the help of static inelastic analysis and is to compare these demands available structure capacities at the specific performance levels. The assessments for the performance parameters include global drift and inter-story drift member deformations and etc.

The pushover analysis load cases can be implemented as force-controlled which pushes to a certain defined force level and as displacement controlled which pushes to a specified displacement. In the displacement-controlled pushover analysis proposed by Allahabadi (1987), specified drifts are sought where the magnitude of applied load is not known previously. The internal forces and deformations computed at the target displacement are used to estimates of inelastic strength and deformation demands that have to be compared with available capacities for a given performance level (Allahabadi (1987), Oguz (2005)). The expectation from pushover analysis is to estimate critical response parameters imposed on structural system. In the analysis, the model is firstly created and gravity loads are applied. Then, a predefined incremental lateral load distributed along the building height is applied to the model. The applied lateral forces are increased until some members of the system yield. The structural model is modified to account for the reduced stiffness of yielded members and lateral forces are again increased until additional members of the system yield. This process is continued until a control displacement at the top of building reaches a certain level of deformation or structure becomes unstable. The roof displacement is drawed with base shear to get the global capacity (pushover) curve like as in Figure 1 Oguz (2005). This capacity curve represents nonlineer behavior of the system.

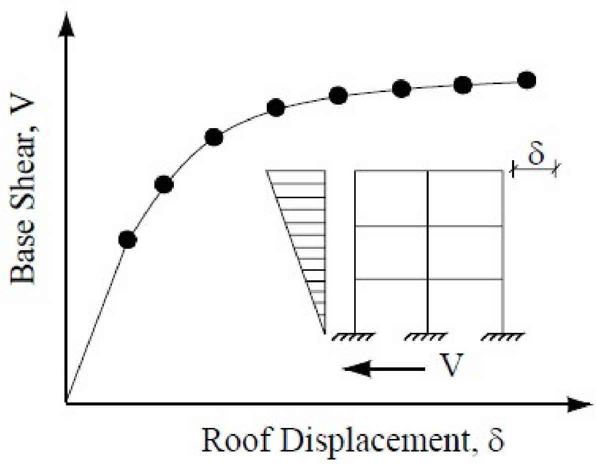
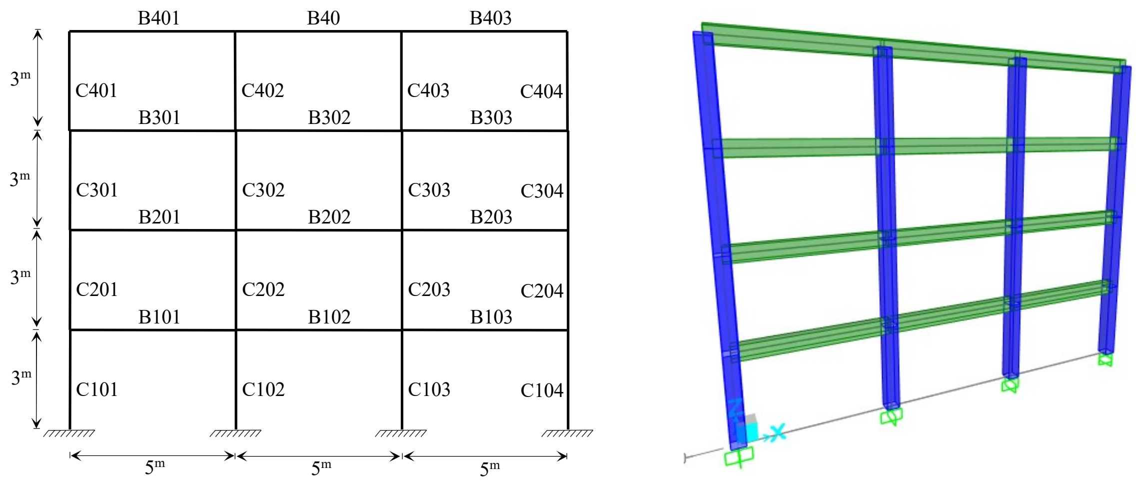


Figure 1. Pushover curve of a structure

3 NUMERICAL EXAMPLE

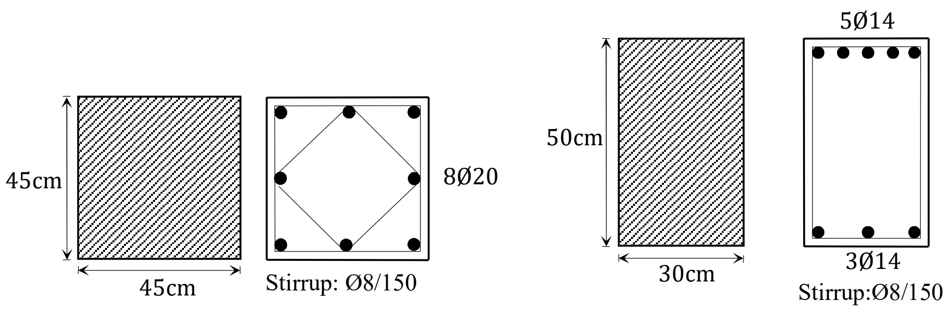
3.1 Details of sample system

In this study, a 4-storey and 3-bay simple RC frame system is chosen in order to better understand the Chapter 5 (Analysis Requirements for Displacement Based Design of Buildings under Earthquake Effect) of TBEC-2018 and is analyzed by the method of incremental single mode pushover analysis. The storey height is 3m and total storey height (H_N) is 12m. The considering frame plane system in this study is shown in Figure 2. While the dimensions and the reinforcement details of column and beam are shown in Figure 3, the material properties of the beam and column are given in Table 1. The concrete compressive strength are assumed to be 25MPa and the yield strength of the longitudinal and transverse reinforcement is 420MPa. The earthquake ground motion level is considered DD-2, which has a probability of exceeding 50 years in 10 years. The location selected for analysis is a region whose local soil class is as ZC with a high seismicity with a PGA value of 0.65g. The detailed numerical parameters considered the analysis are given in Table 2. According to the analysis information in Table 2, there is no drawback in applying the incremental single mode pushover analysis for the RC plane frame system.



a)RC plane frame system b) 3D model of the system

Figure 2. a) RC plane frame system, b) 3D model of the system



a) Column cross section details

b) Beam cross section details in the support and the middle span

Figure 3. The Column and beam cross-sections of the model

Table 1. The material properties of the beam and column

Member	Materials	Dimensions (cm)	Concrete Young' Modulus (MPa)	Reinforcement Young' Modulus (MPa)
Beam	C25-B420C	30*x*50	30000	200000
Column	C25-B420C	45*x*45	30000	200000

Table 2. Four-storey RC building information for the analysis

TBEC-2018	
Earthquake ground motion level	DD-2
Type of Structure	Ordinary Building
Load resistance system	Moment Frame system
Storey of height (m)	3
Local soil class	ZC
Latitude	38.883337
Longitude	40.494507
Short period map spectral acceleration coefficient (S_s)	1.602
Long period map spectral acceleration coefficient (S_l)	0.420
Short period design spectral acceleration coefficient (S_{ds})	1.922
Long period design spectral acceleration coefficient (S_{dl})	0.630
The peak ground acceleration (PGA) [g]	0.651
The peak ground velocity (PGV) [cm/sn]	42.761
Spectrum characteristic periods (TA and TB)	0.0437 and 0.2185
Building usage class (BKS)	3
Building Importance Factor (I)	1
Earthquake Design Class (DTS)	1
Building Height Class (BYS)	6
Analysis Type	Pushover analysis

In the analysis, vertical dead and live loads are shown in Figure 4.

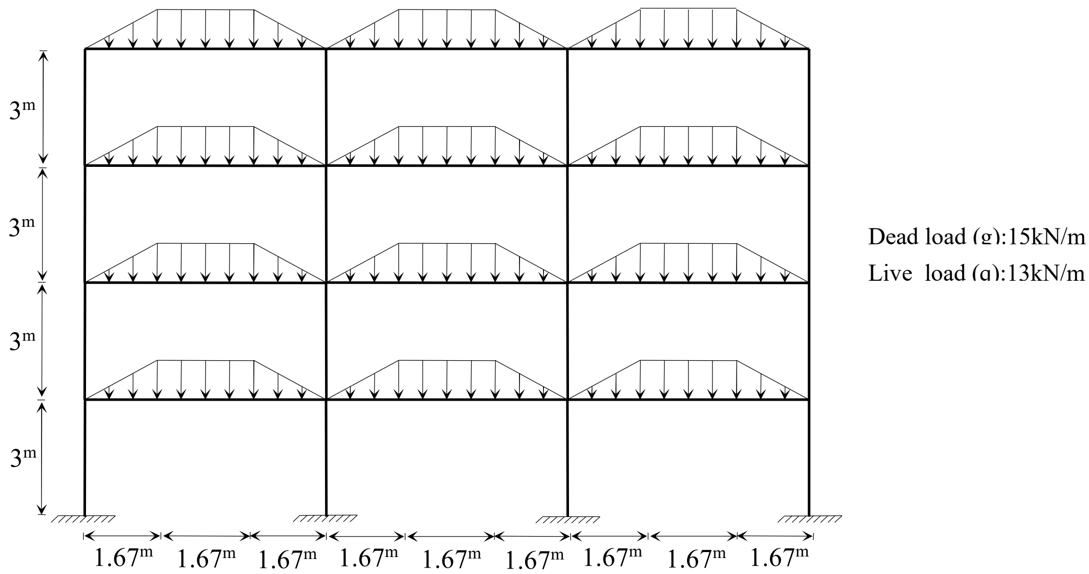


Figure 4. Vertical dead and quake loads in the analysis

3.2 The Obtaining Flexural Stiffness of Cracked Section in RC Structural Elements

While determining the structural performance of RC structures under earthquake effects, the stiffness of the members is determined by taking into account the flexural stiffness of the cracked section (TBEC-2018). The effective flexural stiffness of the cracked cross section is realistically obtained from the moment-curvature relationship.

Material nonlinearities in structural elements are modeled with two types of plastic hinge behavior, namely lumped and spread plastic hinge behavior assumptions. It is the assumption that beam and columns behave as linear elastic except in given points where plastic hinges can form and plastic deformations will occur at the end of the element in the lumped hinge approach. However, in the spread plastic hinge approach, plastic deformations occur in areas close to the end of element. In order to model the plastic hinge behavior, it is important to get the length of the plastic hinge. (Papadrakakis et al. 2008). Although the spread plastic hinge approach idealizes real behavior more realistically, in this study the lumped plastic hinge assumption is considered in terms of ease of calculation and is used in the modeling of the beam and column. According to TBEC-2018 the effective flexural stiffness's of RC columns and beams [$(EI)_e$] is calculated Eq. (1)

$$(EI)_e = \frac{M_y * L_s}{\theta_Y * 3} \tag{1}$$

where M_y and θ_Y are, respectively, the means of yield moment and yield rotation of plastic hinges at the ends of the beam and column. L_s is shear span, which is the ratio of bending moment to shear force. Besides, it can be taken as approximately half of the span in columns and beams (TBEC-2018). According to TBEC-2018, yield rotation of plastic hinges can be calculated as below

$$\theta_Y = \frac{f_y * L_s}{3} + 0.0015\eta\left(1 + 1.5\frac{h}{L_s}\right) + \frac{f_y * d_b f_{ye}}{8\sqrt{f_{ce}}} \tag{2}$$

where f_y is effective yield curvature in the plastic hinge cross section, η is 1.0 in beams and columns, h is the height of section, d_b is the average diameter of the reinforcement interlocking to the node. f_{ce} and f_{ye} are the average compressive strength of concrete and average yield strength of reinforcement, respectively.

In this study, the means of yield moment (M_y) and the means of yield rotation (θ_Y) of plastic hinges at the ends of the beam and column are obtained in SAP2000's section designer and the steps how to achieve these values are given in Figures 5-6. These values are also given in Table 4 for the beam and column. However, before these values are obtained, the material properties must be introduced to the SAP2000 program. To do this, expected (average) strength of the material given in Table 3 will be based on for concrete and reinforcement in

TBEC-2018. According to this values, concrete and reinforcement are introduced to the program.

Table 3. Expected strength of the material

Concrete	f_{ce}=1.3f_{ck}
Reinforcement	f_{ye}=1.2f_{yk}

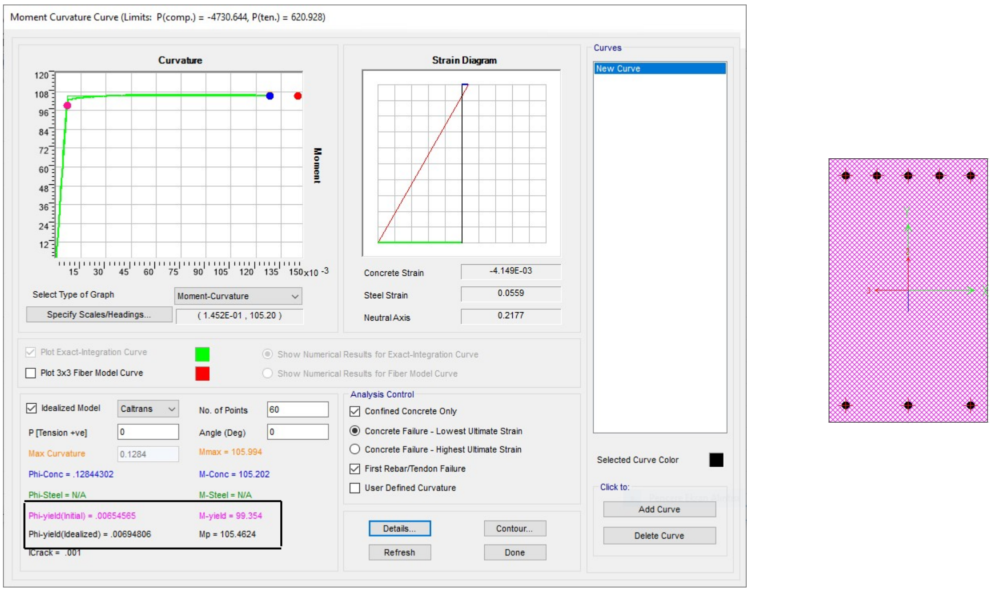


Figure 5. The obtaining of the yield moment and the yield rotation for the beam

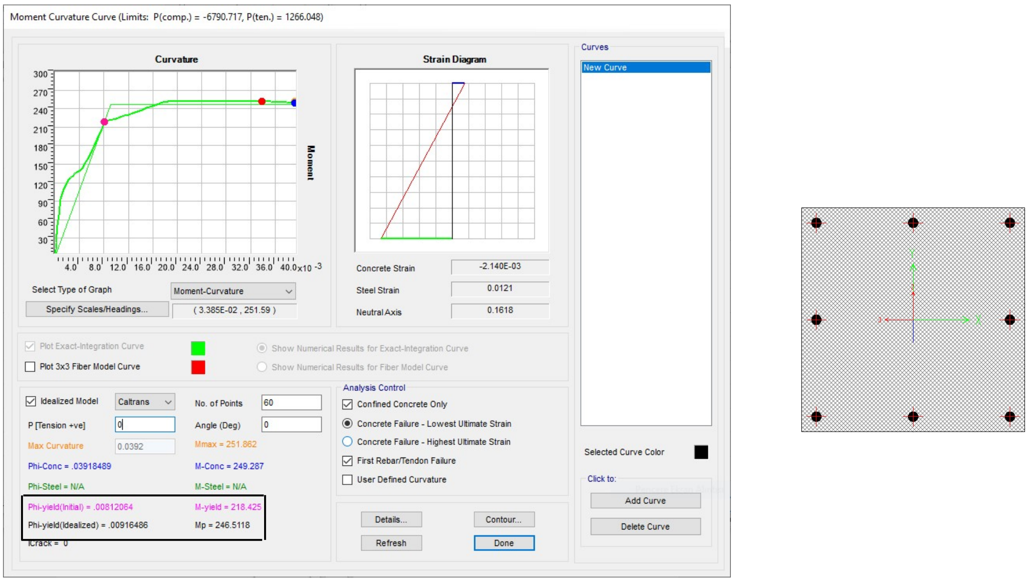


Figure 6. The obtaining of the yield moment and the yield rotation for the column

In the Table 4, M_u is the required bending moment strength determined based on load combinations.

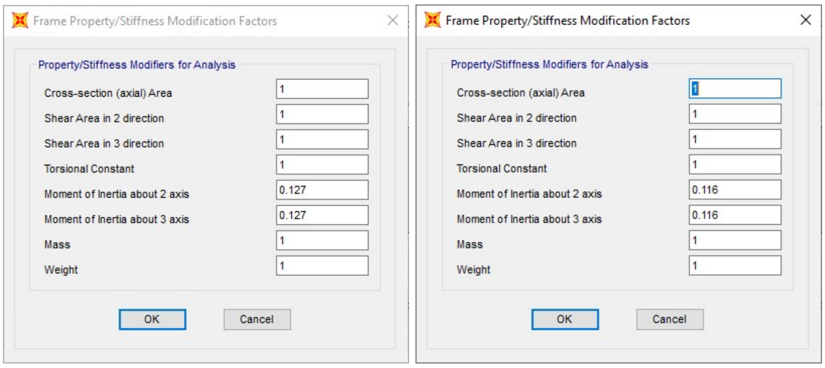
Table 4. θ_Y, M_y and M_u values for the beam and column

	Beam	*Column*
θ_Y *(rad)*	0.00694806	0.00916486
M_y *(kNm)*	99.354	218.425
M_u *(kNm)*	105.462	246.5118

The effective section stiffness's for the beam and column are given in Table 5 and how to enter these values into SAP2000 in section of set modifiers is shown in Figure 7. In Table 5, $(EI)_b$ and $(EI)_e$ are the uncracked and cracked sections flexural stiffness's, respectively.

Table 5. The effective section stiffness values for the beam and column

Beam		*Column*	
$(EI)_b$ (kNm^2)	$(EI)_e$ (kNm^2)	$(EI)_b$ (kNm^2)	$(EI)_e$ (kNm^2)
93750	11916.28	102515.6	11916.44
Section Stiffness Ratio		Section Stiffness Ratio	
$(EI)_e/(EI)_b$=0.127		$(EI)_e/(EI)_b$=0.116	



a) Beam b) Column

Figure 7. The effective section stiffness's for a) Beam and b) Column

3.3 The Determining Pushover Forces Proportional Mass

In the conventional pushover analysis, it was assumed that the response of the multi-degree-of freedom system could be represented by an equivalent single degree of freedom system (Krawinkler and Seneviratna, 1998). This implies that the response is controlled by a single mode, and that the shape of this mode remains constant throughout the time history response, regardless to the level of deformation. Accordingly, in the single mode pushover analysis, it is assumed that seismic response is mainly controlled by the fundamental mode. With this method, the structure is exposed to monotonically increasing predefined lateral forces until a predetermined target displacement is reached. However, this procedure is suitable for the structures that its dynamic behavior depends only on a single elastic vibration mode, as in general low-rise and medium-rise structures.

In order to reflect the effect of the lateral earthquake load, forces proportional to story masses and modal amplitudes must be applied at nodes of story levels. Modal amplitudes are obtained as a result of modal analysis as shown in Figure 8 and lateral earthquake forces are obtained by multiplying the masses of stories and modal amplitudes obtained. The obtained lateral earthquake forces are presented in Table 6.

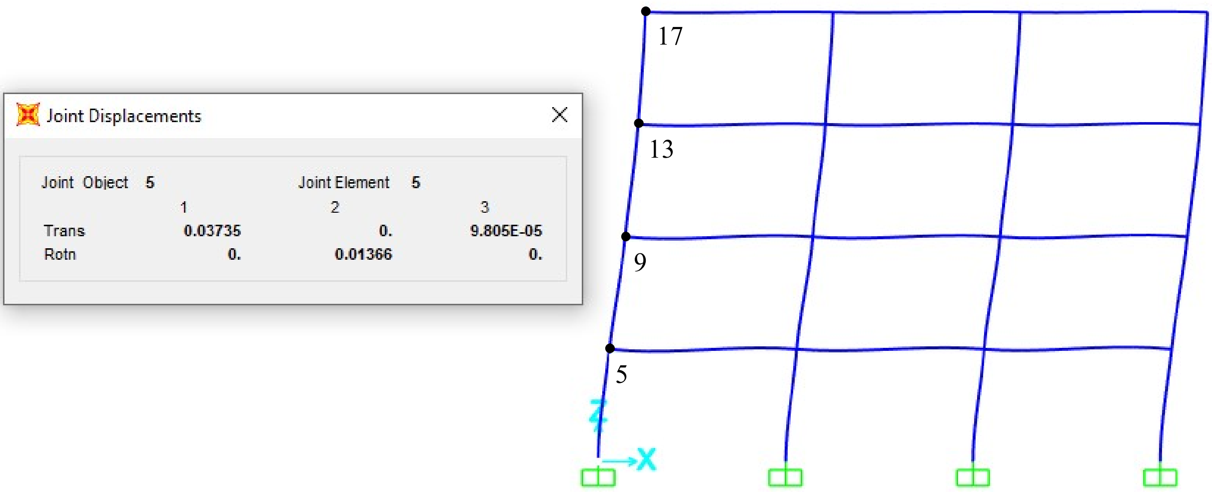


Figure 8. The obtaining of modal amplitudes

Table 6. Lateral loads applied in the nodes of the frame plane system

Node number	Node mass (kNs^2/m)	Node Amplitude (m)	Node Load (kN)
5	3.21	0.03735	0.120
9	3.21	0.09012	0.289
13	3.21	0.13259	0.426
17	3.21	0.15736	0.505

3.4 The Obtaining Capacity Curve

It is needed that earthquake ground motion level, local soil class and latitude and longitude values depending on location to obtain capacity curve. Therefore, horizontal elastic response spectrum is obtained from the page, Ministry of Interior Disaster and Emergency Management Presidency, which is known shortly AFAD in Turkey, as shown in Figure 9. According to related data, shown also in Figure 9, horizontal elastic response spectrum for DD-2 earthquake ground motion level and for 5% damping is given in Figure 10. It can be also seen from the Table 2 for the spectrum data depending on the location.

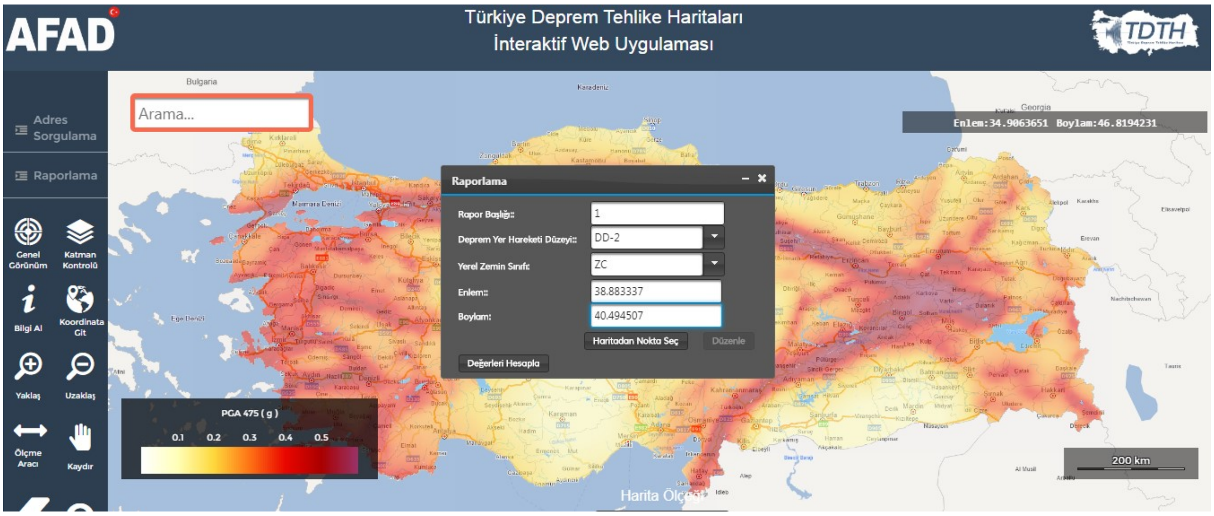


Figure 9. Turkey Earthquake Hazard Maps Interactive Web page of AFAD

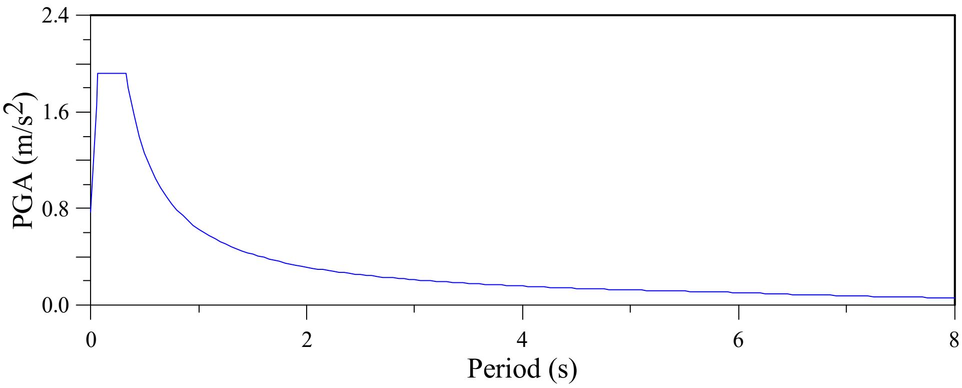


Figure 10. Horizontal response spectrum for DD-2

3.4.1 Identification of Plastic Hinge

In analysis requirements for displacement-based design of buildings, there are three options for defining plastic hinges in SAP2000 program. In the first of these, preliminary dimensioning of structural members is done under load combinations and then required reinforcement area is determined from the SAP2000 program. The default hinge features are automatically defined by the program based on the obtained reinforcement areas. In the second option, reinforcement arrangement and areas of structural members are indicated in the section definition. The default hinge features are automatically defined by the program based on the indicated reinforcement areas. In the last option, hinge properties defined by the user are assigned by obtaining moment-curvature relationships for both positive and negative bending and interaction diagrams based on reinforcement arrangement and areas of structural members. In this study, the default plastic hinge properties are assigned by using the second option.

The location of the hinge must be determined when assigning the plastic hinge. In the lumped plastic hinge approach, TBEC-2018 suggests that plastic hinge length (L_p) equals to half of the section depth in the direction of loading (h) is an acceptable value which generally gives conservative results, shown in Eq. (3). This suggestion is adapted to calculate plastic hinge length. In this study, the section depth in the direction of loading for the beam and column are 0.5m and 0.45m, respectively.

$$L_p \cong 0.5h \quad (3)$$

3.4.2 The Obtaining Pushover Curve and Performance Point

In the constant single mode pushover analysis, it is noted that the lateral load may be a set of displacements or forces, but it should have a constant ratio and a constant shape during the analysis. In this way, at the end of the iteration, the reaction force of the structure is assembled from the contribution of all finite elements. The process terminates when either a predefined limit state is reached, or structural collapse is identified. At the end of the pushover calculation, the roof displacement versus base shear is then interpreted as the capacity curves. Using this process, the structural behavior from elastic state to collapse state can be traced (Behnam, 2017). In this study, as a result of nonlinear performance analysis under PUSHX loading, modal acceleration-modal displacement curve is obtained, and this is overlaid with design spectrum curve. Thus, performance point is identified and is shown in Figure 11. It is obtained that displacement of performance point is 0.167m for DD-2 earthquake ground motion level from the figure. Therefore, displacement-controlled pushover analysis should be maintained at least until this displacement value is obtained.

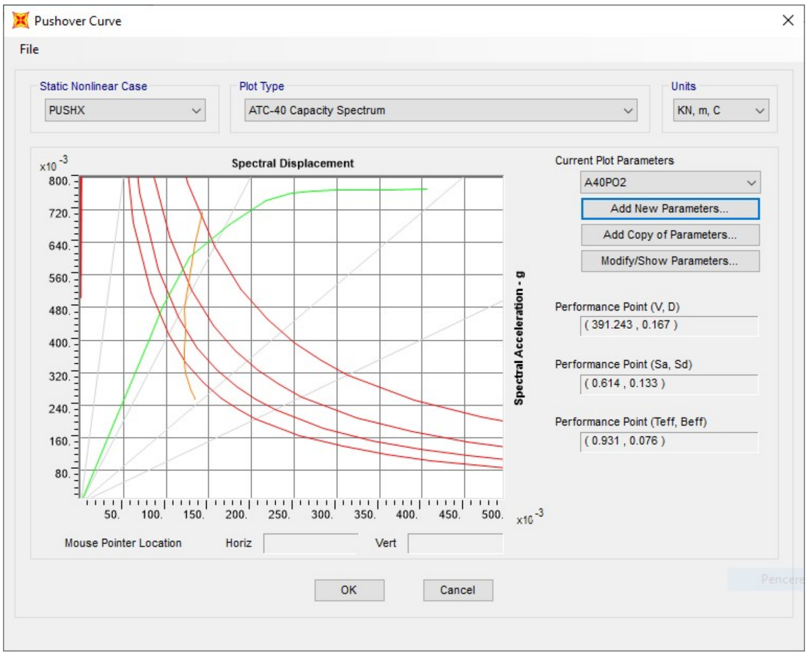


Figure 11. Performance point for DD-2 earthquake ground motion level

3.5 Determination of Damage Limits and Damage Zones According to TBEC-2018

Seismic performance levels of the structures are defined with respect to expected damages during the earthquake. In TBEC-2018, three damage cases and damage limits are defined for ductile members at cross section level according to performance-based design. These performance levels are "*Minimum damage performance level*"," Controlled *damage performance level*" and "*Excessive damage performance level*" and are shown in Figure 12. On the other hand, design earthquakes have been classified in three levels with probability of exceeding of 50%, 10% and 2% in 50 years, respectively. Minimum damage (MD) performance level is defined as a damage where no or a very limited damage occurs in structural members under an earthquake. Controlled damage (CD) performance level is defined as a damage level where damages occurring due to seismic motions are permitted provided that such damages are not very serious structurally and can be repaired. Excessive damage (ED) performance level is defined as a damage where extensive damage occurs in the structures under an earthquake.

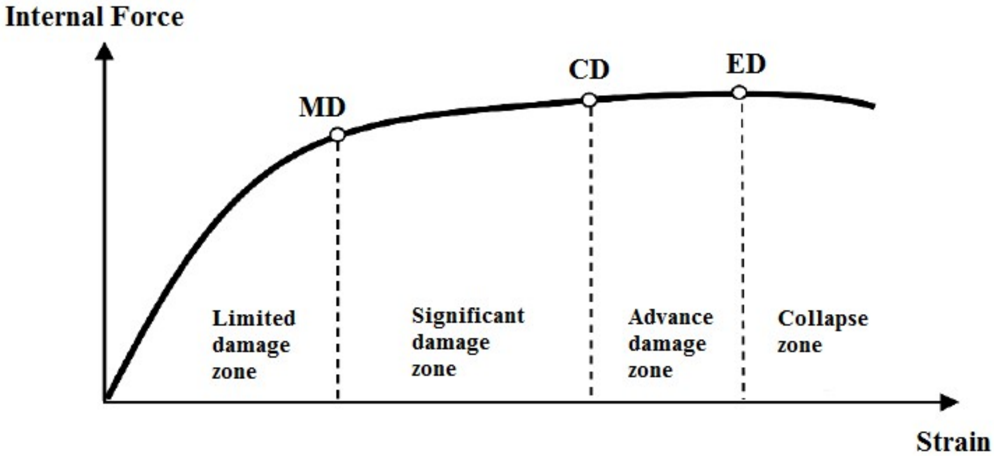


Figure 12. The damage limits and damage zones of the cross-section (TBEC-2018)

3.5.1 Comparison of the Obtained Strain with Evaluation Criteria

The status of plastic hinge of each element should be first examined to determine the status of the cross-sections or whether the strains in the cross-sections exceed the limit values given in TBEC-2018 regulation. The status of plastic hinges of structural members, namely beams and columns are given according to earthquake ground motion levels in Tables 7-8. The internal forces obtained static pushover analysis has been entered as data in Response2000 program and therefore, unit strain deformations formed in concrete and reinforcement are obtained and unit strain limits given in TBEC-2018 for various damage situations have been considered by omitting the confinement effect. In the relevant tables, strain values are obtained via Response2000 program. In the tables, evaluation criteria's, namely MD, CD and ED, represent respectively minimum damage performance level, controlled damage performance level and excessive damage performance level. C is structural members that do not provide damage status before collapse. ε_c is unit shortening at concrete and ε_s is unit elongation of the reinforcement.

Table 7. Damage cases for the beam

Beam							
Member number	Assigned plastic hinge	The obtained strain values		Evaluation criteria (MD, CD, ED)		Case	
		ε_c	ε_s	ε_c	ε_s		
B101	10H1	-0.00202	0.02036	MD		CD	C
	10H2	-0.00500	0.05320			C	
B102	11H1	-0.00199	0.01988			CD	C
	11H2	-0.00466	0.04980			C	
B103	12H1	-0.00199	0.01965			CD	C
	12H2	-0.00483	0.05147			C	
B201	13H1	-0.00252	0.02328	ε_c =0.0025	ε_s =0.0075	CD	C
	13H2	-0.00472	0.05038			C	
B202	14H1	-0.00211	0.02184	CD		CD	C
	14H2	-0.00475	0.05065			C	
B203	15H1	-0.00196	0.01998			CD	C
	15H2	-0.00468	0.05000			C	
B301	16H1	-0.00299	0.02740			ED	ED
	16H2	-0.00063	0.00216			MD	
B302	17H1	-0.00532	0.05249	ε_c =0.002625	ε_s =0.024	C	C
	17H2	-0.00058	0.00208			CD	
B303	18H1	-0.00444	0.04620	ED		C	C
	18H2	-0.00053	0.00225			MD	
B401	33H1	-0.00590	0.05354			C	C
	33H2	-0.00161	0.01089			CD	
B402	34H1	-0.00527	0.05051			C	C
	34H2	-0.00165	0.01284			CD	
B403	35H1	-0.00491	0.04996	ε_c =0.003500	ε_s =0.032	C	C
	35H2	-0.00134	0.01056			CD	

Table 8. Damage cases for the column

<table>
<tr><th colspan="8">Column</th></tr>
<tr><th rowspan="2">Member number</th><th rowspan="2">Assigned plastic hinge</th><th colspan="2">The obtained strain values</th><th colspan="2">Evaluation criteria (MD, CD, ED)</th><th rowspan="2" colspan="2">Case</th></tr>
<tr><th>ε_c</th><th>ε_s</th><th>ε_c</th><th>ε_s</th></tr>
<tr><td rowspan="2">C101</td><td>19H1</td><td>-0.00180</td><td>0.00948</td><td rowspan="10" colspan="2">MD</td><td>CD</td><td rowspan="2">CD</td></tr>
<tr><td>19H2</td><td>-0.00184</td><td>0.00977</td><td>CD</td></tr>
<tr><td rowspan="2">C102</td><td>22H1</td><td>-0.00192</td><td>0.00664</td><td>MD</td><td rowspan="2">MD</td></tr>
<tr><td>22H2</td><td>-0.00162</td><td>0.00573</td><td>MD</td></tr>
<tr><td rowspan="2">C103</td><td>25H1</td><td>-0.00151</td><td>0.00519</td><td>MD</td><td rowspan="2">MD</td></tr>
<tr><td>25H2</td><td>-0.00187</td><td>0.00655</td><td>MD</td></tr>
<tr><td rowspan="2">C104</td><td>28H1</td><td>-0.00319</td><td>0.01313</td><td>ED</td><td rowspan="2">ED</td></tr>
<tr><td>28H2</td><td>-0.00317</td><td>0.01303</td><td>ED</td></tr>
<tr><td rowspan="2">C201</td><td>20H1</td><td>-0.00261</td><td>0.01549</td><td>CD</td><td rowspan="2">CD</td></tr>
<tr><td>20H2</td><td>-0.00247</td><td>0.01440</td><td>CD</td></tr>
<tr><td rowspan="2">C202</td><td>23H1</td><td>-0.00150</td><td>0.00556</td><td rowspan="2">ε_c =0.0025</td><td rowspan="2">ε_s =0.0075</td><td>MD</td><td rowspan="2">MD</td></tr>
<tr><td>23H2</td><td>-0.00150</td><td>0.00561</td><td>MD</td></tr>
<tr><td rowspan="2">C203</td><td>26H1</td><td>-0.00141</td><td>0.00514</td><td rowspan="8" colspan="2">CD</td><td>MD</td><td rowspan="2">MD</td></tr>
<tr><td>26H2</td><td>-0.00142</td><td>0.00519</td><td>MD</td></tr>
<tr><td rowspan="2">C204</td><td>29H1</td><td>-0.00298</td><td>0.01338</td><td>ED</td><td rowspan="2">ED</td></tr>
<tr><td>29H2</td><td>-0.00298</td><td>0.01339</td><td>ED</td></tr>
<tr><td rowspan="2">C301</td><td>21H1</td><td>-0.00581</td><td>0.03590</td><td>C</td><td rowspan="2">C</td></tr>
<tr><td>21H2</td><td>-0.00600</td><td>0.03709</td><td>C</td></tr>
<tr><td rowspan="2">C302</td><td>24H1</td><td>-0.00198</td><td>0.00813</td><td>CD</td><td rowspan="2">CD</td></tr>
<tr><td>24H2</td><td>-0.00208</td><td>0.00880</td><td>CD</td></tr>
<tr><td rowspan="2">C303</td><td>27H1</td><td>-0.00200</td><td>0.00850</td><td rowspan="2">ε_c =0.002625</td><td rowspan="2">ε_s =0.024</td><td>CD</td><td rowspan="2">CD</td></tr>
<tr><td>27H2</td><td>-0.00195</td><td>0.00818</td><td>CD</td></tr>
<tr><td rowspan="2">C304</td><td>30H1</td><td>-0.00336</td><td>0.01741</td><td rowspan="8" colspan="2">ED</td><td>ED</td><td rowspan="2">ED</td></tr>
<tr><td>30H2</td><td>-0.00326</td><td>0.01678</td><td>ED</td></tr>
<tr><td rowspan="2">C401</td><td>8H1</td><td>-0.00133</td><td>0.00524</td><td>MD</td><td rowspan="2">MD</td></tr>
<tr><td>8H2</td><td>-0.00133</td><td>0.00524</td><td>MD</td></tr>
<tr><td rowspan="2">C402</td><td>9H1</td><td>-0.00363</td><td>0.00203</td><td>C</td><td rowspan="2">C</td></tr>
<tr><td>9H2</td><td>-0.00559</td><td>0.03185</td><td>C</td></tr>
<tr><td rowspan="2">C403</td><td>31H1</td><td>-0.00530</td><td>0.03072</td><td>C</td><td rowspan="2">C</td></tr>
<tr><td>31H2</td><td>-0.00545</td><td>0.03162</td><td>C</td></tr>
<tr><td rowspan="2">C404</td><td>32H1</td><td>-0.00349</td><td>0.01981</td><td rowspan="2">ε_c =0.003500</td><td rowspan="2">ε_s =0.032</td><td>ED</td><td rowspan="2">C</td></tr>
<tr><td>32H2</td><td>-0.00552</td><td>0.03247</td><td>C</td></tr>
</table>

4 CONLUSIONS

In this study, the performance analysis of a four-storey and three-span RC plane frame has been made for the DD-2 design earthquake. In addition, information about static pushover analysis terms and stages has been explained according to TBEC-2018. The internal forces obtained static pushover analysis has been entered as data in Response2000 program and therefore, unit strain deformations formed in concrete and reinforcement are obtained and unit strain limits given in TBEC-2018 for various damage situations have been considered by omitting the confinement effect. The location selected for analysis is a region with a high seismicity with a PGA value of 0.65g. In this framework, as a result of the analysis it has been seen that the beam elements do not provide the pre-collapse boundary condition and that all beam element except for B301 beam collapse. As to the columns, most of the elements in the lower stories have provided the status of MD or CD performance levels, whereas some of them in the upper stories have reached the damage status of ED performance level and others have collapsed. In this context, it can be stated that the selected model provide the strong-column/weak-beam rule and plastic hinges are primarily formed on the beams and so it is observed that the beam mechanism firstly formed in the model.

In the great earthquakes that occurred in countries for centuries, many people lost their lives after the earthquake. Thus, earthquake resistant building design becomes more important with the rapidly increasing population and urbanization with the developments in earthquake engineering and earthquakes occurring in our country in Turkey located on active earthquake zone. Accordingly, the importance of performance analysis is increasing in structural engineering day by day. With this study, information about how to perform performance analysis has been systematically given and its steps is explained. It has been hoped that this study will serve as an example especially for structural engineers working in the project offices.

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