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RESEARCH ARTICLE

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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.

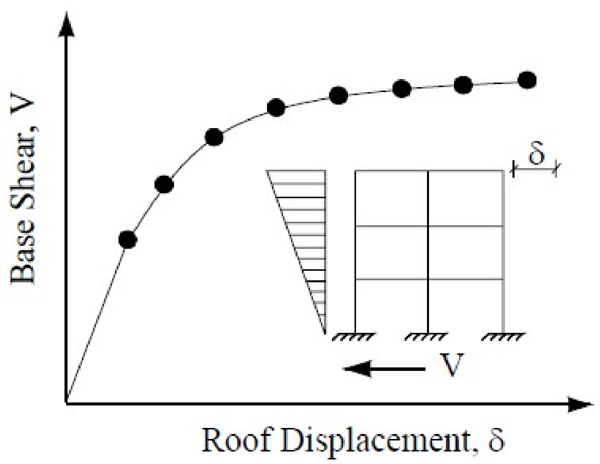
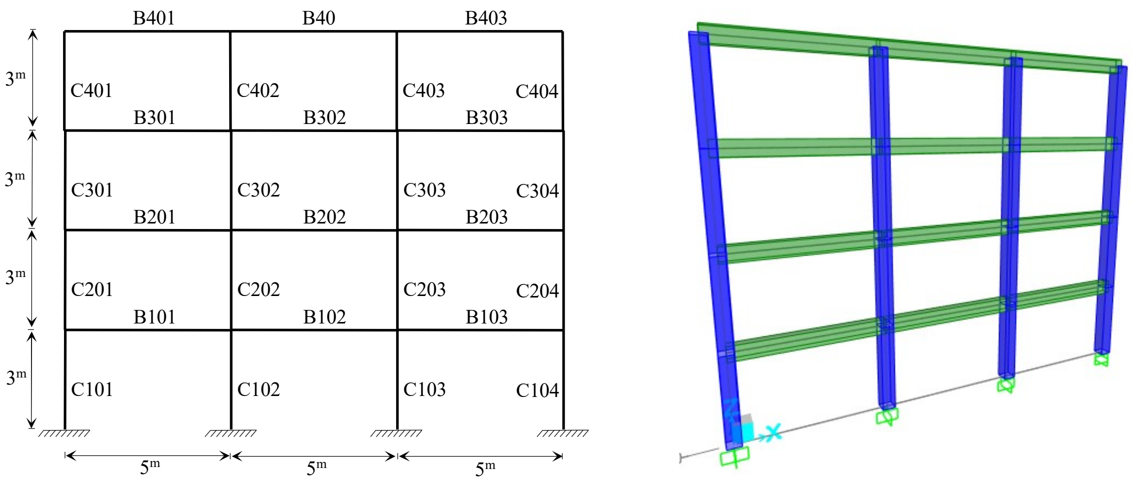


Figure 1. Pushover curve of a structure

3 NUMERICAL EXAMPLE

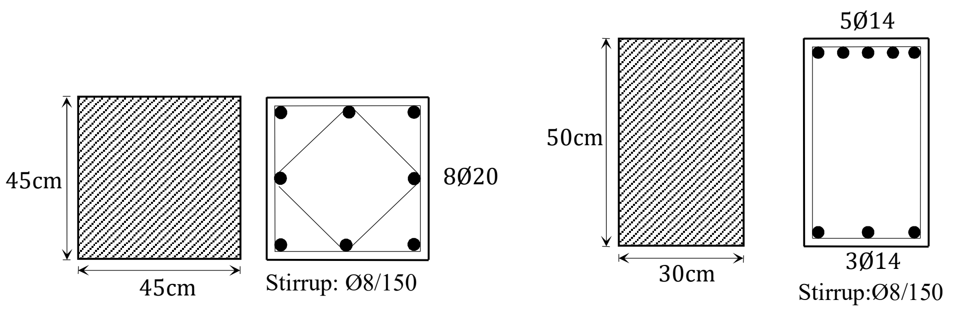
3.1 Details of sample system

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.

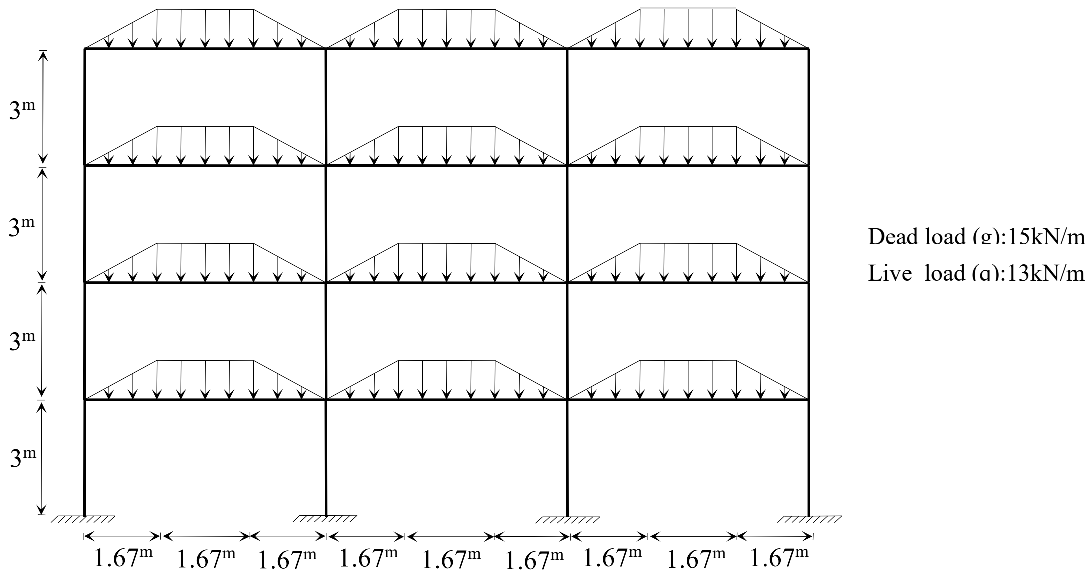


Figure 4. Vertical dead and quake loads in the analysis

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	$f_{ye}=1.2f_{yk}$

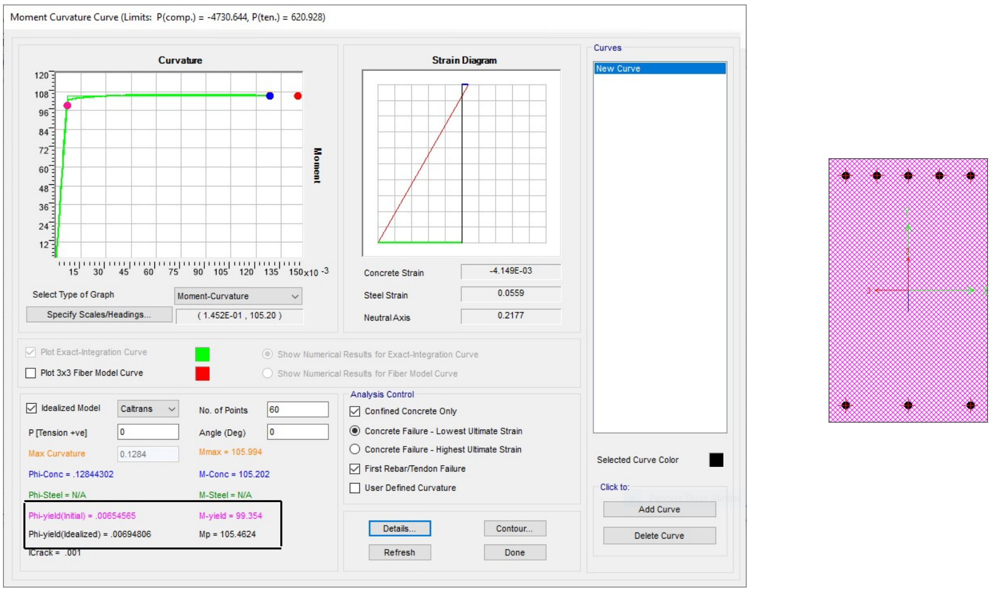


Figure 5. The obtaining of the yield moment and the yield rotation for the beam

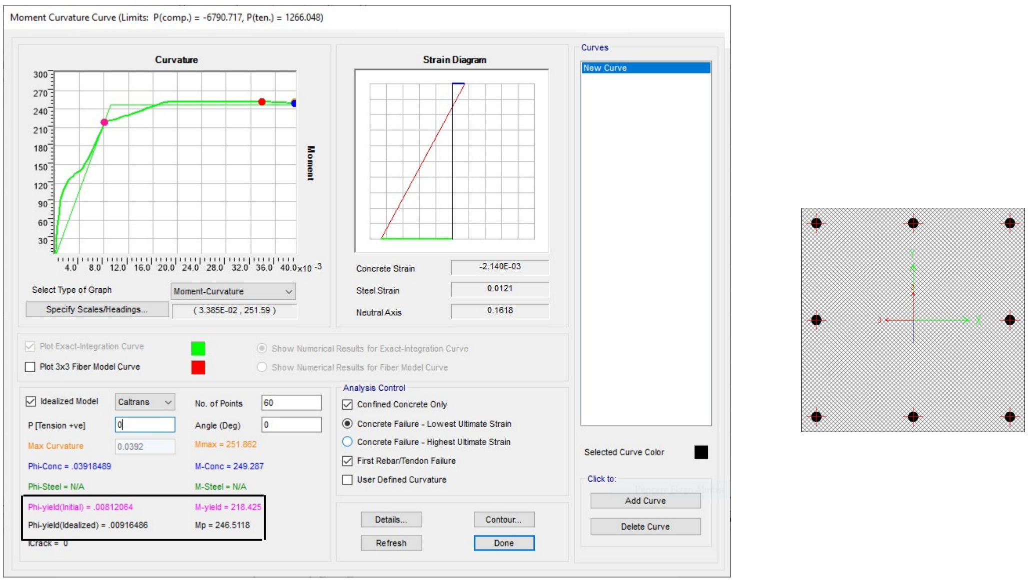


Figure 6. The obtaining of the yield moment and the yield rotation for the column

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

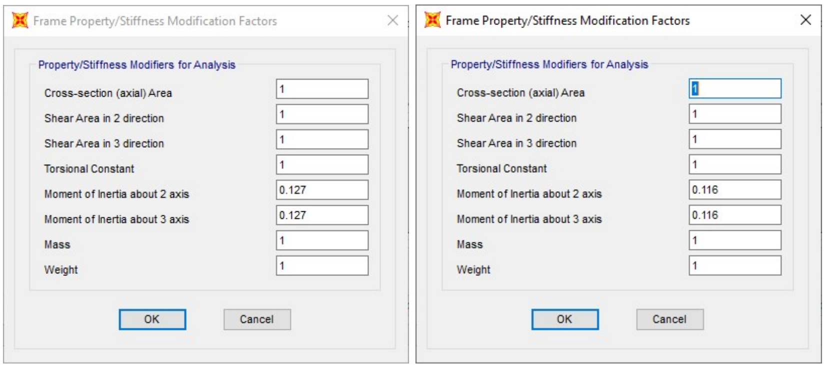
Table 4. θ_Y , M_y and M_u values for the beam and column

	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.

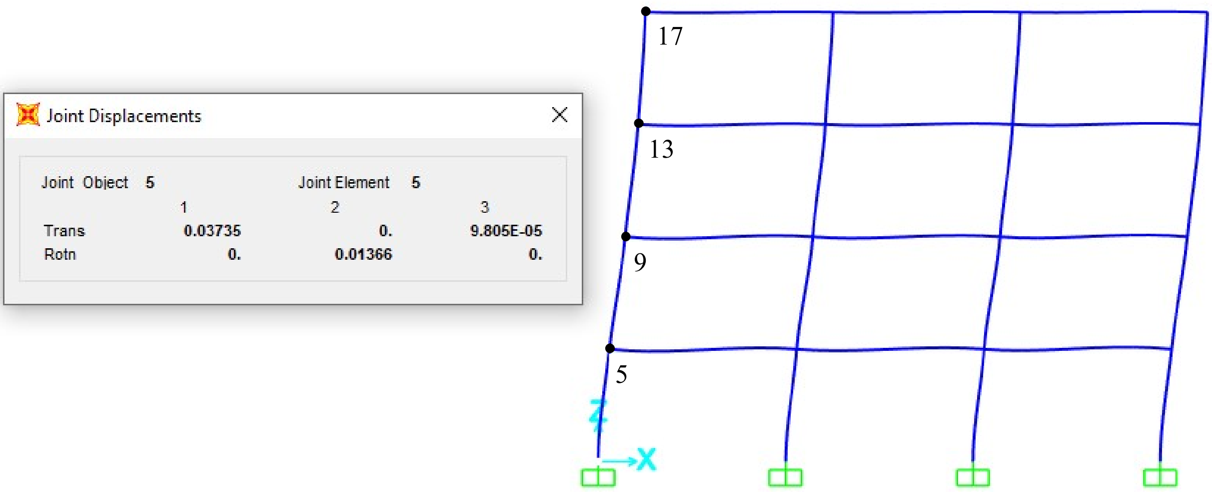


Figure 8. The obtaining of modal amplitudes

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.

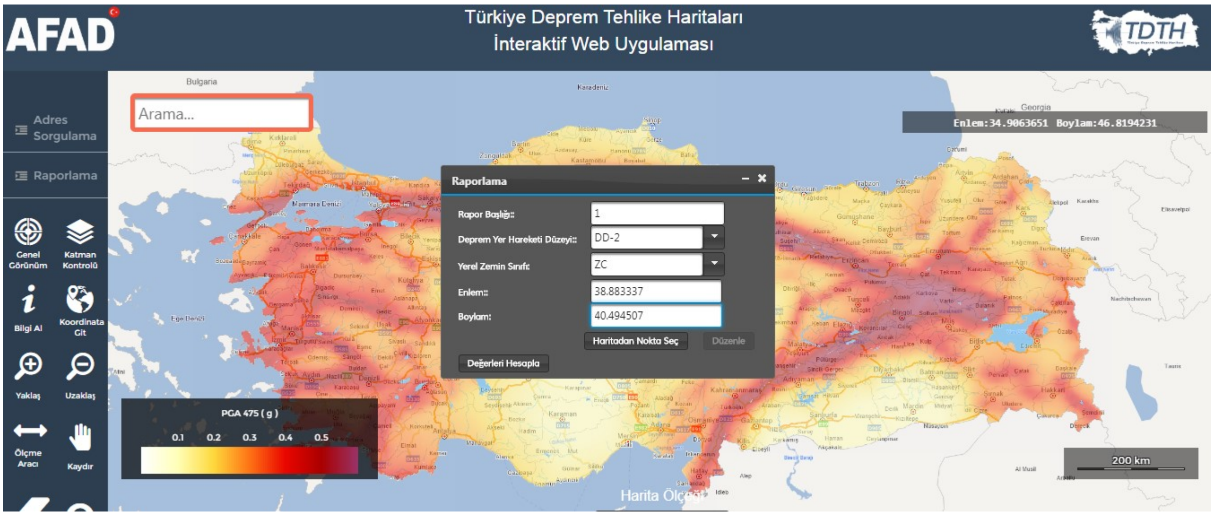


Figure 9. Turkey Earthquake Hazard Maps Interactive Web page of AFAD

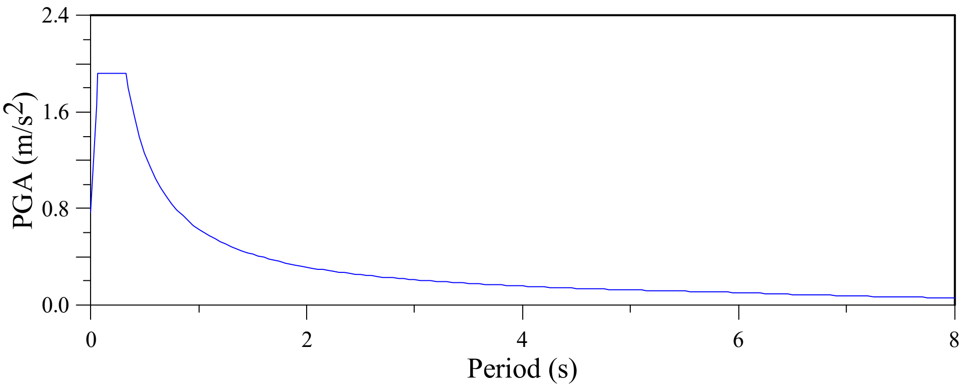


Figure 10. Horizontal response spectrum for DD-2

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.

$$L_p \cong 0.5h \tag{3}$$

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.

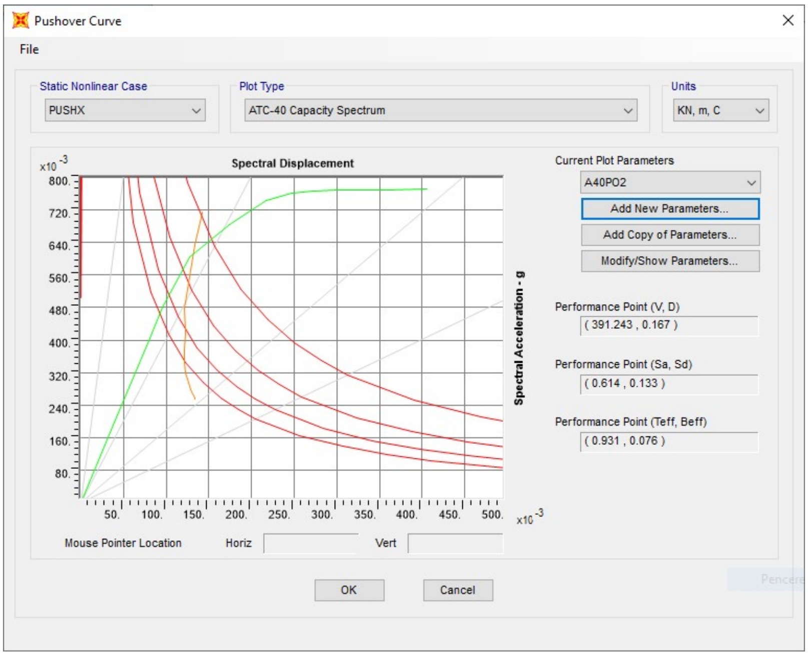


Figure 11. Performance point for DD-2 earthquake ground motion level

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.

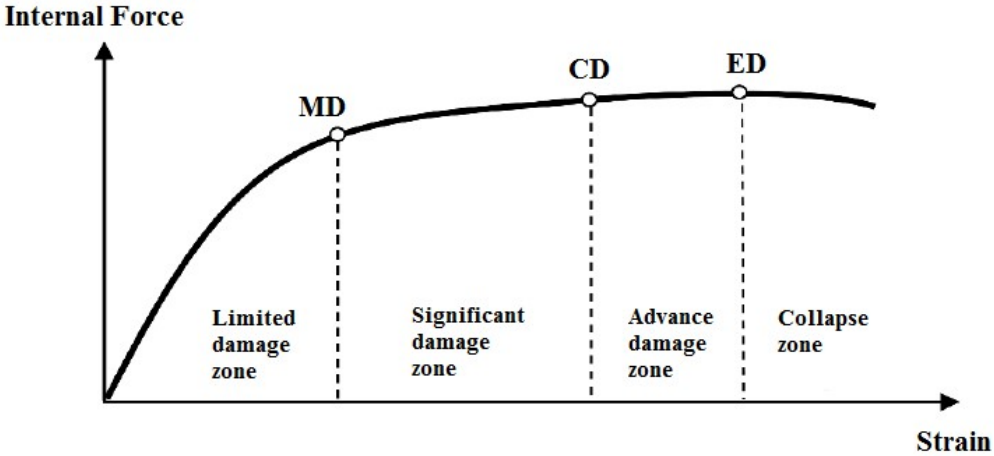


Figure 12. The damage limits and damage zones of the cross-section (TBEC-2018)

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

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

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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Review Article

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Controlled Demolition Techniques and Demolition Direction

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Abstract

Every structure has a service life that can be used safely. The demolition works of the buildings that complete the period of use are quite troublesome. Various demolition techniques have been developed by structural engineers over time. The main purpose of this study is to compare demolition techniques with their advantages and disadvantages. The most important problem with demolition works is safe operation. In this article, the necessary techniques have been investigated to perform the demolition work safely in the desired direction.

Kontrollü Yıkım Teknikleri ve Yıkım Yönü

Anahtar Kelimeler;
Blok Zinciri, İnşaat mühendisliğinde Blok zinciri uygulamaları.

Özet

Her yapının güven içinde kullanılabileceği bir servis ömrü vardır. Kullanım süresini tamamlayan yapıların yıkım işleri ise oldukça zahmetlidir. Yapı mühendisleri tarafından zaman içinde çeşitli yıkım teknikleri geliştirilmiştir. Bu çalışmanın başlıca amacı yıkım tekniklerini avantaj ve dezavantajlarıyla karşılaştırmaktır. Yıkım çalışmaları ile en önemli sorun güvenli çalışmadır. Bu makale kapsamında yıkım işinin istenilen yönde güvenli biçimde yapılabilmesi için gerekli teknikler araştırılmıştır.

1. INTRODUCTION

Turkey has a large existing building stock has completed its service life. These existing structures need to be renewed for four different reasons. These reasons are;

1. End of life of structures,
2. Non-compliance with structure specifications and regulations,
3. Structural and design weaknesses against earthquakes,
4. Natural disasters, wars etc.

The reasons for urban transformation include place requirement due to increasing population, revisions in the zoning plans and rapid urbanization factors. When the data of the Turkish Statistical Institute were examined, the urban transformation rate was 25% in the early 1950s, 40% in the 1980s, 65% in the early 2000s, 77% in 2012, 91.5% in 2014, 92.1% in 2015 and 92.5% in 2017. This increase in urban population day by day causes housing shortages. Due to the growing housing shortage, renovation or demolition of older dwellings has become important. Due to the need for housing due to urbanization and the presence of structures damaged by natural disasters, it is predicted that more than 6000000 houses will be demolished and rebuilt in our country in twenty years.

2. DEMOLITION TECHNIQUES

Within the scope of urban transformation in our country, many studies have been carried out and new techniques have been developed to ensure that the demolition or renovation of old and inadequate structures can continue more effectively. Techniques used for complete or partial destruction of structures can be classified as follows (Koca, 2006):

- Demolition by crushing or by breaking or by separating with mechanical tools,
- Demolition by hitting the structure with the help of a vince-attached steel sphere,
- Demolition with high-access machines and scissors machines,
- Demolition by tow rope,
- Controlled demolition by explosives,
- Demolition by floor reduction method,
- Demolition of the structure by breaking down with chemical materials,
- Demolition of the elements of the structure with diamond saws.

Although there are many techniques developed for the demolition work, the technique that provides the most suitable conditions for demolition work must be selected. When selecting the most suitable technique to use for a demolition job, the following elements are taken into account:

• Cost of demolition work and allocated time for demolition,

• Determination of whether demolition work is partial or complete,

• Determination of whether the field capacity for mechanical vehicles is sufficient,

• The quality of the material used in the structure to be demolished,

• Geometry of the structure or structural elements to be demolished,

• The size and location of the structure to be demolished (Koca, 2006),

• Environment and traffic situation of the structure to be demolished,

• The properties of the ground where the structure is built and the structural system of the structure,

• Equipment supply, demolition experience and occupational health and safety precaution,

• Safety of demolition work and presence of hazardous materials,

• Allowable level of disturbance (noise, dust and vibration),

• Reuse of rubble after demolition.

Some of demolition techniques are shown in figure 1.

(a) (b) (c)

(d) (e) (f)

Figure 1. (a) Destruction by mechanical tools, (b) Destruction by steel sphere, (c) Demolition with concrete shears, (d) Demolition with high access machine, (e) Demolition by tow rope, (f) Demolition by explosive.

The three most important of these elements; location, dimensions and cost of demolition.
The destruction of these structures by traditional methods by inexperienced and uninformed people leads to dangerous consequences. On November 23, 2017, that the chimney above the demolished block toppled over the excavator in the Triangle Bazaar in Denizli (Figure 2 (a)) ("Denizli'de yıkımı yapılan", 2017) and on January 8, 2017 that the overturning of the mechanical vehicle during the demolition of the 6-storey building in Battalgazi District of Malatya (Figure 2 (b)) ("Malatya'da yıkım yapan", 2017) are examples of these dangerous consequences.

(a) Demolition accident in Denizli

b) Demolition accident in Malatya

Figure 2. Example Demolition Accidents

In order to continue urban transformation activities quickly and effectively in our country, controlled demolition method by explosives should be preferred instead of traditional demolition methods.

3. CONTROLLED DEMOLITION BY EXPLOSIVES

Controlled demolition technique by explosives of the structures is based on the principle that the carrier elements in the lower floors of the structure are broken down by explosives and the other carrier elements lose their carrier properties under the influence of increasing forces.
The advantages and disadvantages of the demolition technique by explosives compared to traditional demolition techniques can be summarized as follows.

Advantages of demolition technique by explosive:

- Lower cost, especially when applied in higher structures,
- Faster application than other demolition techniques,
- Disturbances limited to a short time,
- Safer when carried out on or near traffic arteries,
- It can be used in situations where the use of construction machines is difficult,

Disadvantages of demolition technique by explosives:

- If there is no project of the building to be demolished and the material properties of the building are not known, data collection may lead to time loss,
- The necessity of a team of expert and experienced people in explosive, static and security issues for demolition work (Koca, 2006),
- To obtain the necessary permits for detonation is time consuming,
- The necessity of stopping traffic flow around detonation (Jimeno ve diğerleri, 1995),
- Potential danger risk of explosives,
- Possibility of damaging nearby structures,
- The possibility that the structure will not be completely demolished.

Two methods are commonly used for controlled destruction of structures by explosives . The first of these is based on the principle that the structure is toppled sideways as a result of changing the centre of gravity of the structure (Figure 3 (a)). The second is based on the principle that the structure collapses within the boundaries of the structure as a result of the loss of the carrier properties of some of the carrier elements of the structure (Figure 3 (b)).

It is possible to come across applications where these two methods are used together.

(a)

Figure 3. (a) Toppling of the structure and (b) Collapse of the structure within its boundaries

The firing sequence in toppling the structure to the side starts from the carrier elements which are in the direction of the toppling and continues in a delayed way towards the carrier elements in the inner parts of the structure. The firing sequence in the inward collapse of the structure starts from the carrier elements that are near the centre of gravity of the structure and continues towards the carrier elements that are located at the edges. Detonation of the carrier elements by using milliseconds-delayed capsules will increase the loads on the other structural elements, so the structure will start to deform itself. In non-delay explosions or explosions with very little delay time, the structure does not have the time to deform itself (Özyurt, 2013). The delay intervals determined according to the demolition direction of the structures are shown in Figure 4(a) and Figure 4(b).

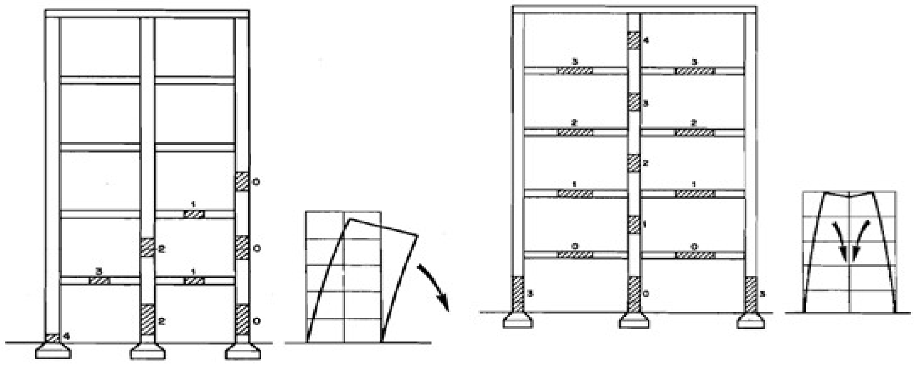

Figure 4. (a) Toppling the structure in a certain direction and (b) Collapse of the structure within its boundaries

In this demolition technique, the material properties of the structure, the selection of the elements, the design of the hole geometry in which explosives will be placed, the type and amount of explosives to be used in the demolition, the firing system to be used must be determined by the experts in the demolition area (Doğan et. al., 2009). Due to the diversity of features, such as properties of materials used in the structure, location of the structure and purpose of demolition, etc., the most suitable blasting and firing design of each structure is different (Olofsson, 1980; Özyurt, 2013). For example, if there is no space of sufficient size for demolition, the inward collapse method can be preferred. If there is, toppling method can be preferred.

Dangerous results can occur if the dynamic effects of the structure planned to be detonated are not calculated correctly or if the explosion sequence of the elements that are the most important stage of the controlled detonation technique is not properly determined. For example, on November 10, 2010, the tower of the Mad River Power Station in the USA State of Ohio was demolished on solid power lines, not in the direction it was planned after the explosion of the dynamite (Figure 5 (a)) ("Yanlış hesap kuleyi", 2010). The 10-storey structure, which was intended to be demolished by explosives in Sevastopol on 26 December 2014, was not demolished as desired as a result of detonation and the structure was tilted at an angle of 20° (Figure 5(b)) ("Demolition fail", 2014). On December 3, 2017, Pontiac Silverdome Stadium in Michigan, USA, was intended to be demolished by explosives, but the structure remained standing (Figure 5 (c)) ("Detroit Stadı", 2017). On April 4, 2018, a 53-meter-long silo was attempted to be demolished by explosives in Vordingborg, Denmark, but the giant silo was demolished towards the wrong side (Figure 5 (d)) ("Danimarka'da korkutan", 2018). For planning controlled demolition of buildings by explosives in the most accurate way, in other words, in order to avoid such situations or minimize this risk, a demolition simulation program is needed, which includes modern mechanical results and provides the demolition mechanism to be estimated as close to reality as possible.

Figure 5. (a) Toppling of the Power Plant in the wrong direction, (b) Demolition of the structure in the not intended direction, (c) Demolition of the structure in the not intended direction, (d) Toppling of the silo in the wrong direction.

4. CONCLUSION

In this study, demolition techniques have compared with their advantages and disadvantages. Among the techniques discussed, explosive demolition technique stands out with its time and cost advantages. However, the technique with the highest safety risk due to faulty demolitions and dentures is also an explosive demolition technique. There is always a risk of unplanned collapse in demolition work. However, this risk is at maximum level in explosive destruction. In cases where there is not enough space in the demolition site, this risk increases one more time. In this technique, it can be much more complex to correctly predict the direction of destruction. Although there are computer software designed for this purpose, these software increase the cost of the project and using these software requires a new skill. As a continuation of this article, the authors suggest to focus on the studies on demolition simulations. By creating simple but effective demolition models, the level of safety can be brought to upper levels in demolition works.

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RESEARCH ARTICLE

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Investigation of Earthquake Behaviour of Different Building Materials Used in Masonry Structures

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Keywords:
Masonry structure, Structural materials, Effect of different materials on masonry structure. Earthquake behaviour of masonry structure.

Abstract

The use of masonry structures dates back many years. It is important to determine the behaviour of masonry structures, which are widely used today. The most important factor determining the behaviour of the masonry structure is the structural material used in the structure. In this study, a masonry wall built with aerated concrete, pumice, brick and stone building materials used in masonry structures was modelled in 3D in ANSYS program and its behaviour against 3 different earthquakes was investigated. As a result of the investigation, the reliability of construction materials according to earthquake records was listed.

Yığma Yapılarda Kullanılan Farklı Yapı Malzemelerinin Deprem Davranışlarının İncelenmesi

Anahtar Kelimeler;

Yığma yapılar, Yapısal malzeme, Farklı malzemelerin yığma yapıya etkisi, Yığma yapının deprem davranışı.

Özet

Yığma yapıların kullanımı uzun yıllara dayanmaktadır. Günümüzde de kullanımı oldukça yaygın olan yığma yapıların davranışlarının belirlenmesi önemlidir. Yığma yapının davranışını belirleyen en önemli etken, yapıda kullanılan yapısal malzemedir. Bu çalışmada yığma yapılarda kullanılan gazbeton, ponza, tuğla ve taş yapı malzemeleri ile inşa edilen bir yığma duvar, ANSYS programında 3 boyutlu olarak modellenmiş ve 3 farklı deprem karşısındaki davranışı incelenmiştir. İnceleme sonucunda yapı malzemelerinin deprem kayıtlarına göre güvenilirlikleri sıralanmıştır.

1 INTRODUCTION

When the structures used in our country are examined, the use of masonry structures as well as reinforced concrete structures draws attention. (Korkmaz et al., 2014). Seismic behaviour of these structures will be examined and their safety will contribute to the prevention of possible damages (Korkmaz et al., 2014). Masonry structures are generally constructed from different materials such as stone, brick, adobe, briquette and have been used from past to present (Çırak, 2011). In a masonry structure, the walls serve as carriers (Çırak, 2011).

When determining the damage level of a masonry structure after an earthquake, the repair and strengthening status of the structure is examined (İnangu, A., Kırbaş, H.,1999). Earthquake behavior of each material used in masonry structure is different (Çırak, 2011). The best method for determining earthquake behavior is nonlinear time history analysis (Badry and Satyam, 2014). For this reason, Time history analysis method was preferred in this study.

The earthquake behaviour of the masonry structure can be shown as in figure1. Damages occurring or expected in the structure can be classified according to the severity of the earthquake. Damages at level A and B are the expected damage levels for earthquakes of magnitude 6-7, damages at level C and D are for earthquakes of magnitude 8-9, and earthquakes at level E are the expected damage levels for earthquakes greater than 9. (İnangu, A., Kırbaş, H.,1999).

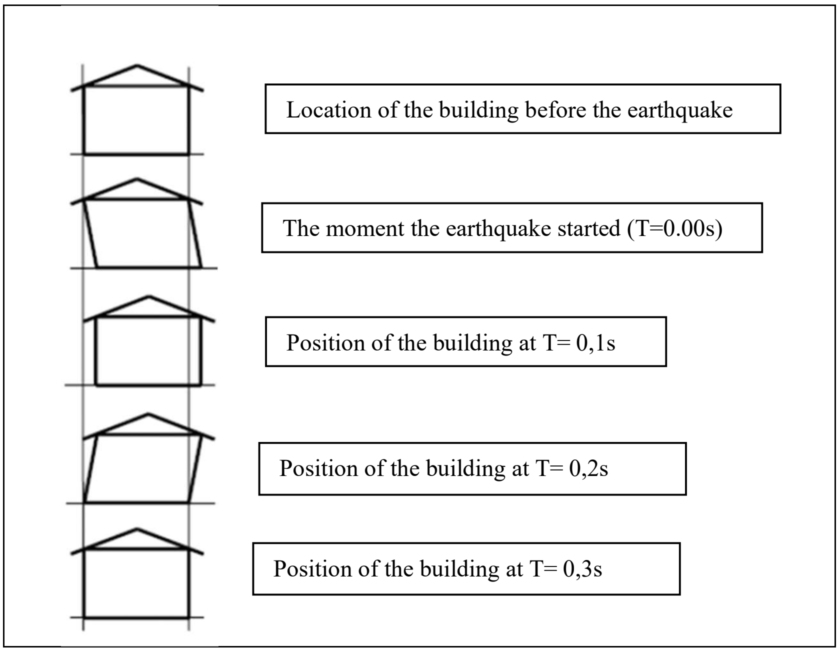

Figure 1. “Earthquake Behaviour of Structures” (İnangu, A., Kırbaş, H.,1999).

A. "Undamaged or slightly damaged structure: No cracks or plaster cracks in the structure." (İnangu, A., Kırbaş, H.,1999).

B. "Slightly damaged structure: 45 degrees of cutting cracks on structure." (İnangu, A., Kırbaş, H.,1999).
C. "Moderately damaged wall structures: The walls have 45 degree cut cracks. However, shear stress on the wall decreased. (%30-%40)" (İnangu, A., Kırbaş, H.,1999).

D. "Heavy damaged masonry structure: Crack gap exceeds 25 mm in structures and walls are separated at the corners, the effect of shear forces is weakened, and the fragmented walls become incapable of carrying vertical loads, causing swelling and collapse of the walls due to vertical loads." (İnangu, A., Kırbaş, H.,1999).

E. "Demolished masonry structure: A large part of the carrier wall is demolished, and the floors are stacked on top of each other, this damaged structure is no longer repairable." (İnangu, A., Kırbaş, H.,1999).

2 MATERIAL AND METHOD

Materials used in masonry structure may vary depending on the region built (Bayülke, 2011). Wooden structures are used in the Black Sea Region, mudbrick structures are used in Southeast Anatolia and stone walls are used in Eastern Anatolia (Bayülke, 2011).

Robustness, strength, economy, sound and heat insulation, workmanship is important in the choice of construction material (Bayülke, 1998; Köktürk, 1997). The lightness of the material used in the structure reduces the building load (Korkmaz et al., 2014). Porous and lightweight construction materials have sound and heat insulation (Bayülke, 1998; Köktürk, 1997). It is also cheaper and does not require much labour, so it is more preferred (Bayülke, 1998; Köktürk, 1997).

The brick is used both as a carrier and as a partition wall (Bayülke et al., 1989). Ductile behaviour of brick masonry structures is poor (Bayülke et al., 1989). That is why brick masonry structures exhibit brittle behaviour (Bayülke et al., 1989).

"Stones are natural, crystalline internal and inorganic building materials." (Köktürk, 1997). Used in the construction of the carrier wall since the past (Köktürk, 1997). But because it is heavy and the conditions of use are difficult, its use is limited (Köktürk, 1997). Granite, basalt, andesite, sandstone, tuff slate, limestone and sandstone are natural Stones (Köktürk, 1997).

Pumice of volcanic origin, glassy and porous structure is a lightweight construction element (Bayülke et al., 1989). It has low permeability and high heat and sound insulation due to interpore spaces (Bayülke et al., 1989). Due to these physical properties, pumice is used in the construction of concrete briquettes and blocks, and in heat and sound insulation in constructions (Benedetti et al., 1998).

Aerated concrete, porous, lightweight, has heat and sound insulation and is a fireproof material (Benedetti et al., 1998). It is economical because of its easy workability and low workmanship (Benedetti et al., 1998). It is a light material and reduces the load (Benedetti et al., 1998). "In the masonry constructions constructed with aerated concrete, it is observed that the rigidity and strength are maintained against horizontal forces in the earthquakes." (Benedetti et al., 1998). Nowadays it is more preferred than other materials (Benedetti et al., 1998).

3 CASE STUDY

In the study a masonry structure made of brick, stone, pumice and aerated concrete material was modelled in three dimensions in ANSYS program and its behaviour against earthquake was investigated. Three earthquake acceleration recordings were applied by using the analysis method for four different material states in the time domain and the displacement and stress values obtained were compared and the reliability of the structure under earthquake effect was determined. The 3D of 4m/4m/0,25m masonry wall is shown in figure 2.

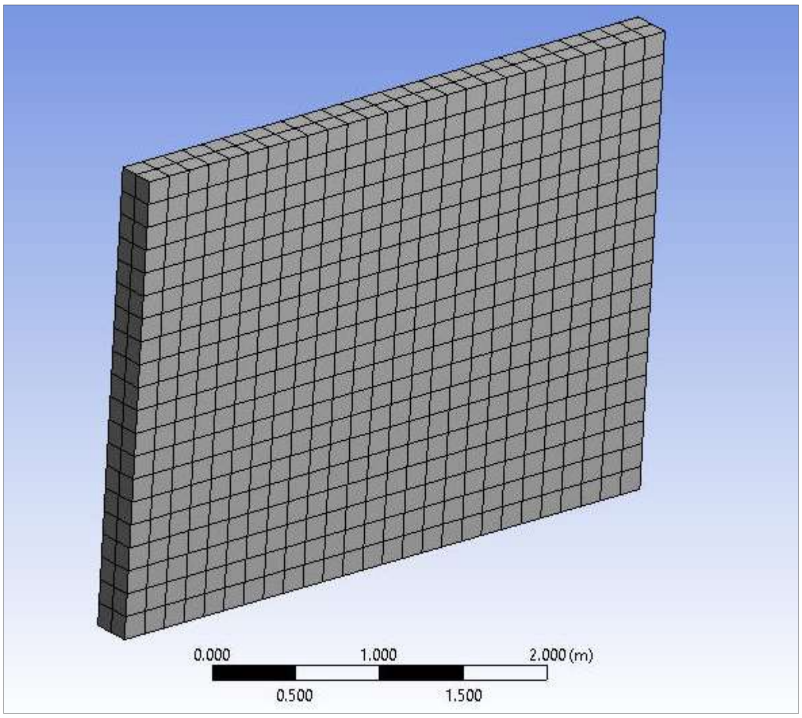

Figure 2. 3D Model Of Masonry Wall Example (4,00x4,00x0,25m).

Table 1. Material properties used in the analyses (Korkmaz et al. , 2014).

Material Type	Modulus of Elasticity (MPa)	Poisson Ratio	Unit Weight (kN/m3)
Brick	3000	0,2	20
Stone	26000	0,2	25
Pumice	22000	0,2	16
Aerated Concrete	25000	0,2	6

Table 2. Earthquake characteristics used in analysis (Korkmaz et al. , 2014)

No	Earthquake	Year	Moment Size	Scale Factor	Arias Intensity	Tp	Distance
			(Mg)		(m/s)	(s)	(km)
1	El Centro	1940	6,95	1.0	1,6	-	6,09
2	Shandon	1966	6,19	1.0	0,4		12,90
3	Gilroy	1979	5,74	1.0	0,8	1,232	3,11

Figure 3-11 show X, Y and Z component of ground motion records

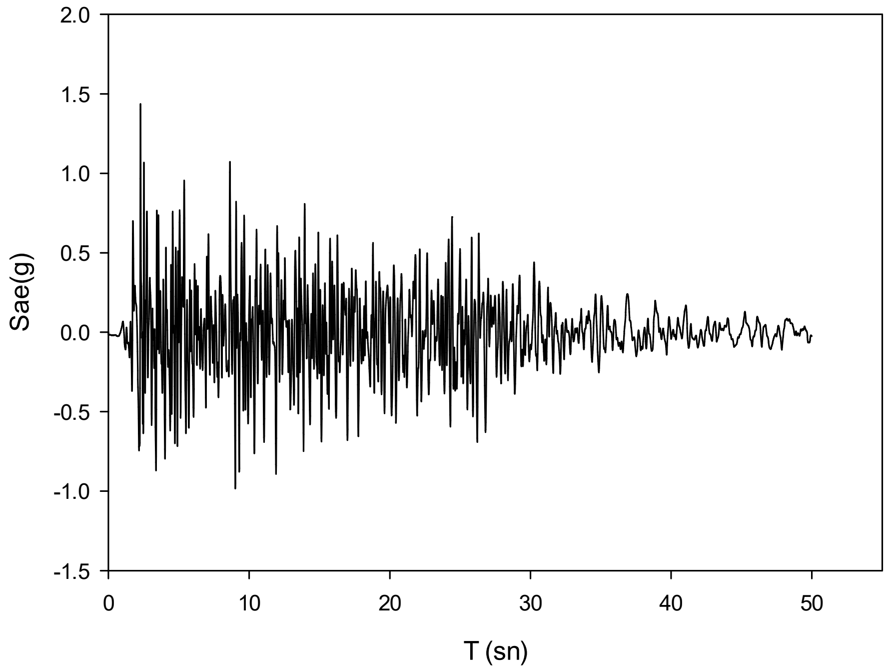

Figure 3. X Component of 1940 El Centro earthquake

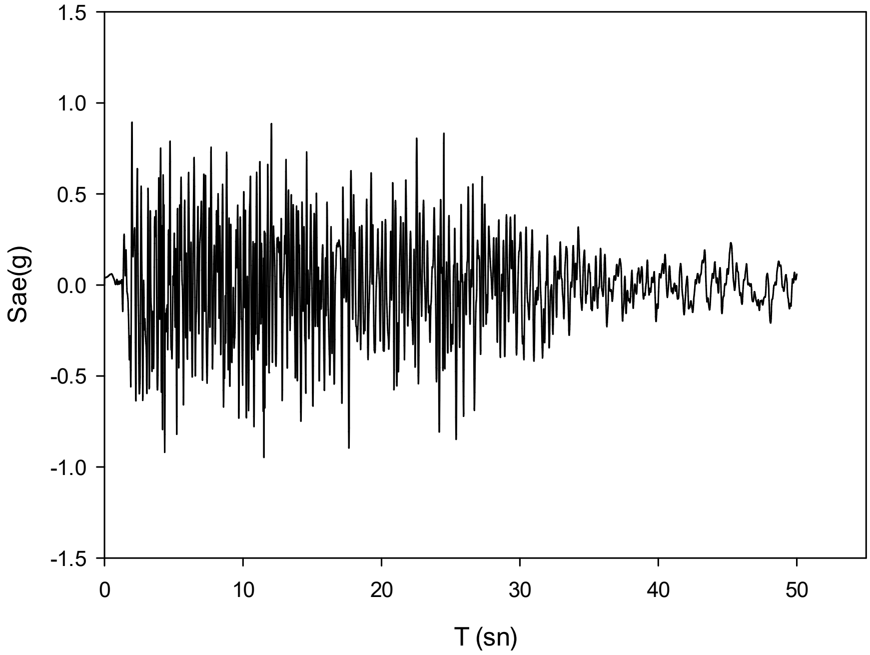

Figure 4. Y Component of 1940 El Centro earthquake

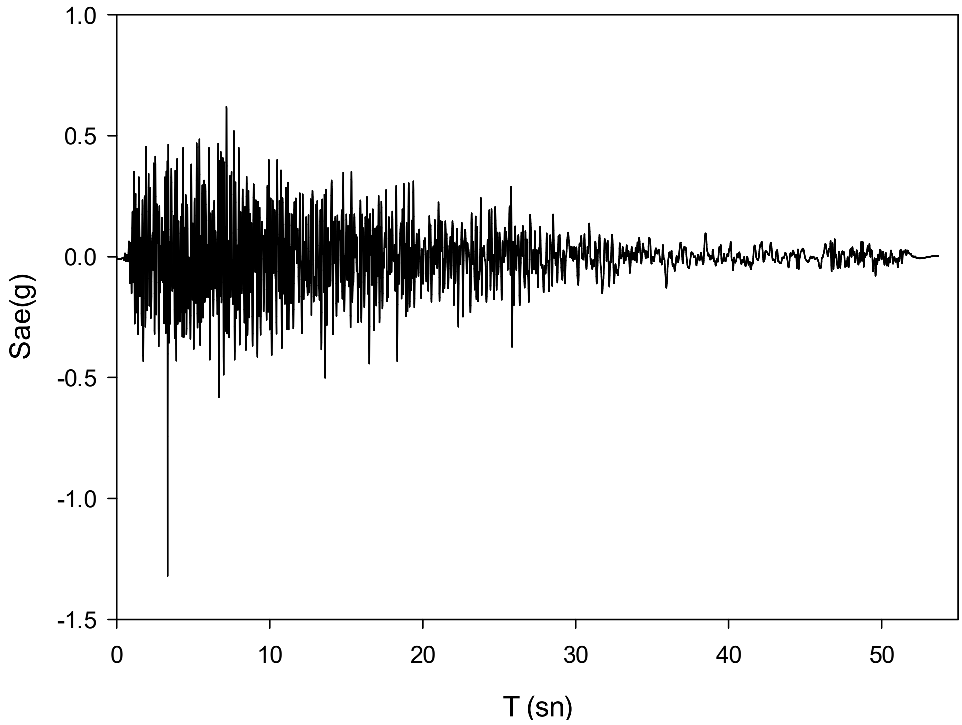

Figure 5. Z Component of 1940 El Centro earthquake

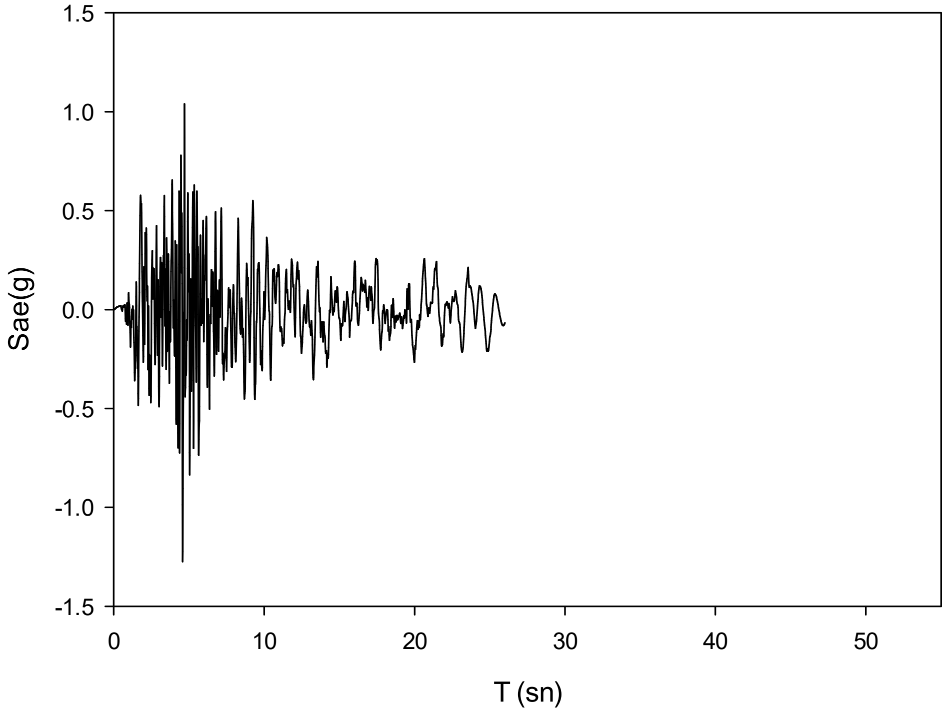

Figure 6. X Component of 1966 Shandon earthquake

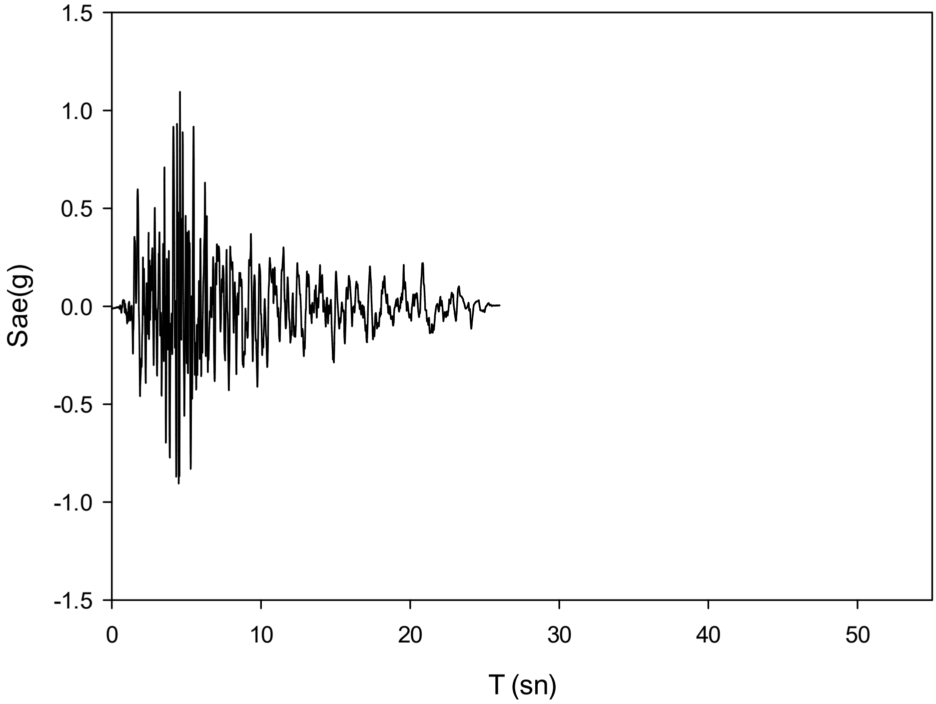

Figure 7. Y Component of 1966 Shandon earthquake

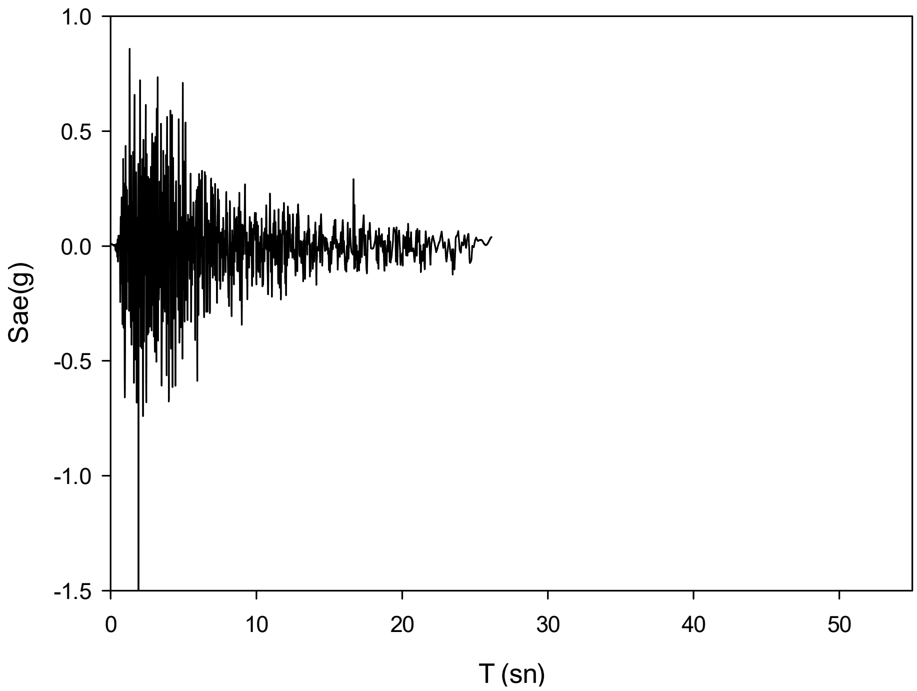

Figure 8. Z Component of 1966 Shandon earthquake

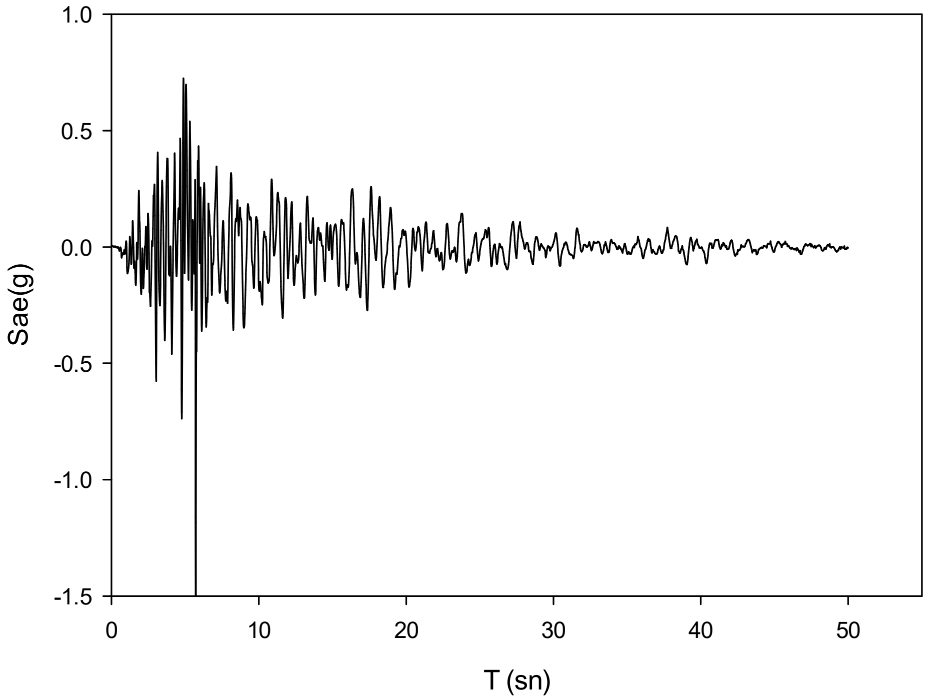

Figure 9. X Component of 1979 Gilroy earthquake

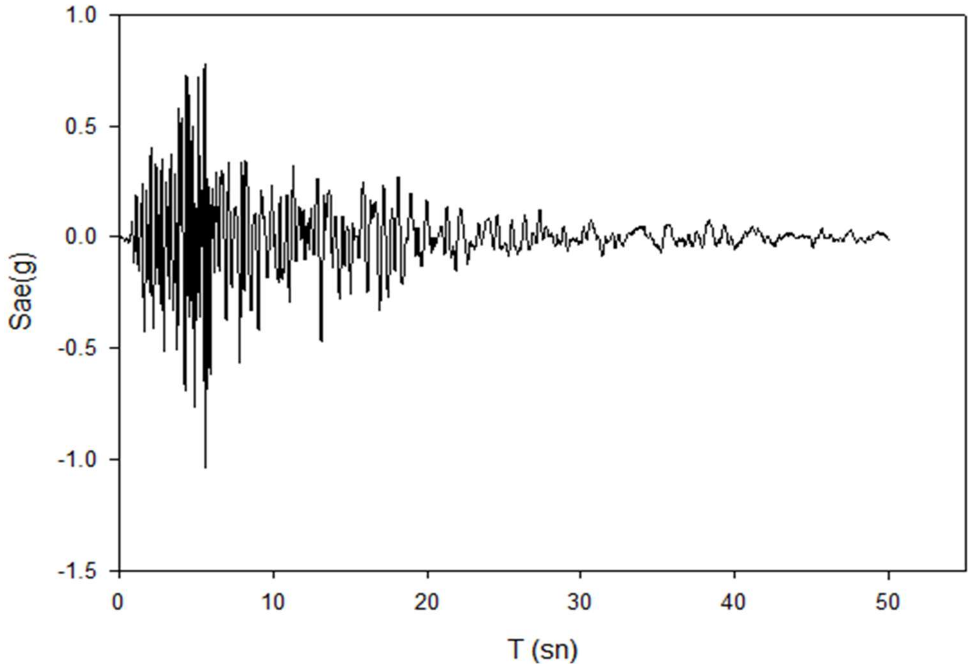

Figure 10. Y Component of 1979 Gilroy earthquake

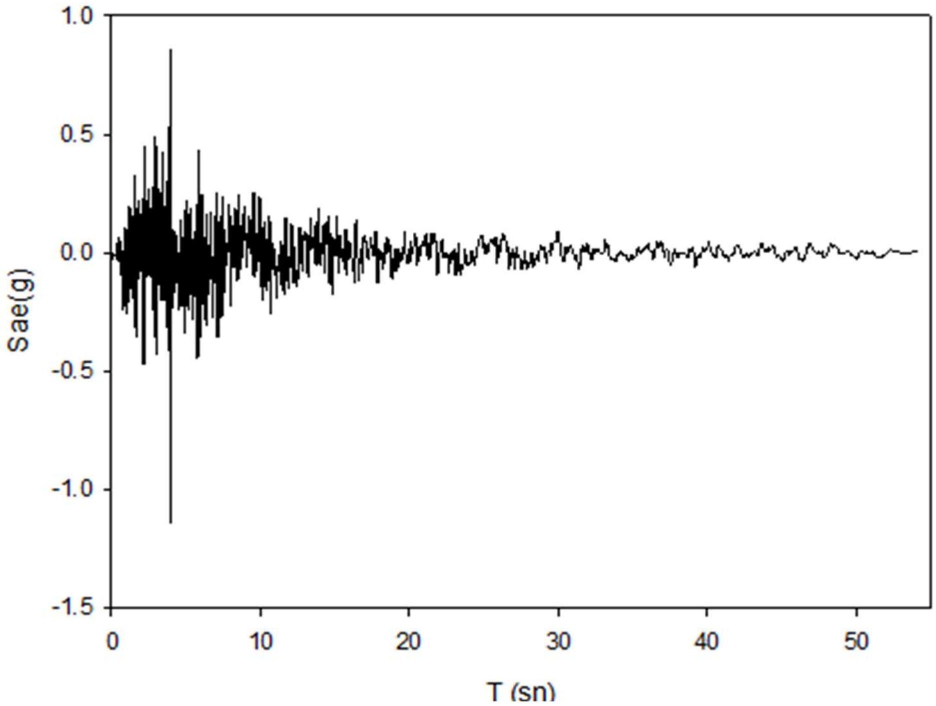

Figure 11. Z Component of 1979 Gilroy earthquake

Figure 12 shows that maximum values of lateral displacements for different materials according to earthquake records.

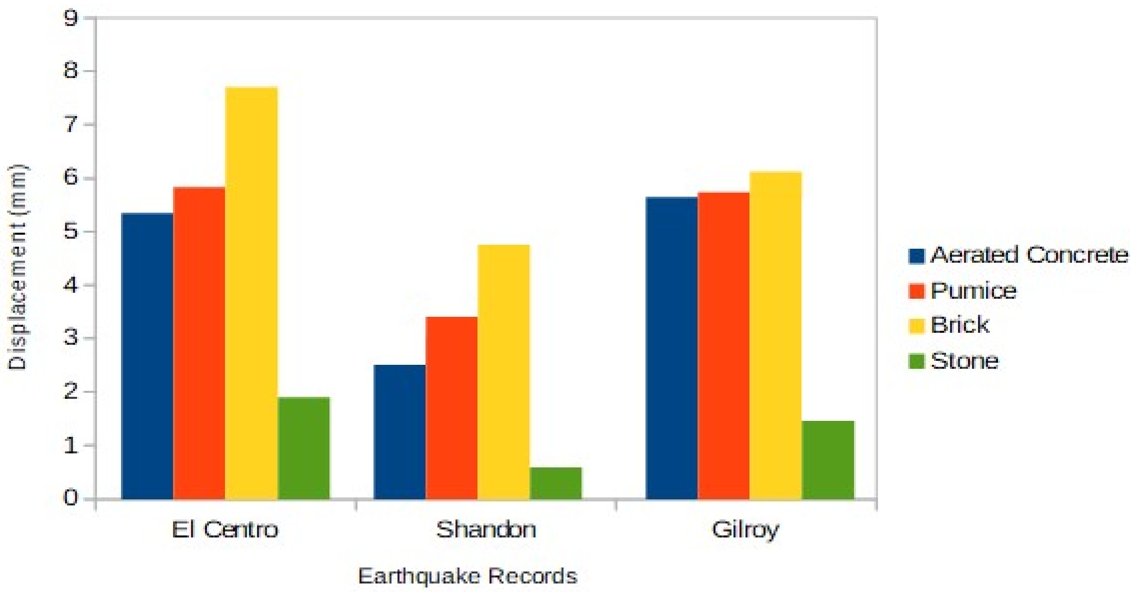

Figure 12. Lateral displacement values

3. CONCLUSION

In this article, the working principle of the masonry structures used from past to present, the materials used in the structure and how these materials affect the structure are examined. A masonry wall is modelled in ANSYS program. the dimensions of the wall are 4m x 4m x 0.25m. Aerated concrete, pumice, brick and stone materials were introduced separately as wall materials. Analyses were made for each earthquake applied to the wall. When the analysis results were examined, it was found that the brick with the lowest flexibility module had higher displacement and the stone with the highest flexibility module had the lowest displacement. When we sort the displacement order from big to small, it becomes brick, pumice, aerated concrete and stone. The reason why earthquakes with different dimensions have the same displacement is seen when focal depths are examined. The amount of displacement is related to the depth of focus of the earthquake. When we look at the El Centro and Gilroy earthquakes, the earthquake dimensions are 6.95 Mg for El Centro and 5.74 Mg for Gilroy. However, the displacement amount is the same in both earthquakes. The reason for this is that the depth of focus is 6.09 km in the El Centro earthquake and 3.11 km for the Gilroy earthquake. As a result, in the earthquake of the same size, the amount of displacement in the earthquake region with a close focus depth will increase and will decrease reliability.

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SCIENNOVATION
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RESEARCH ARTICLE

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Analysis Of An Educational Building According To TEC2007 And TEC2018

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Keywords:
Analysis, Building importance coefficient
Training building

Abstract

It was mentioned in Turkey Building Earthquake Regulation, which was published in 2018 and entered into force on 1 January 2019, that many changes have been made in many articles of (TDY 2007) for the buildings to be constructed in earthquake zones. One of these articles is the change of building importance coefficient according to purpose of the building use. Building importance coefficient is, a coefficient which is determined at designing stage according to the use of building aftern an earthquake load. According to the new regulation, School and education buildings and facilities, dormitories and dining halls, military barracks, prisons and classes of museums and so on, they have been changed as compulsory buildings after the earthquake. Within the scope of the study, ideal education building was analyzed in the program according to tdy 2007 and tbdy 2018 and the resulting data were compared.

Bir Eğitim Binasının TDY 2007 ve TBDY 2018'e Göre Analizi

Anahtar Kelimeler;
Analiz,
Bina önem katsayısı,
Eğitim binası

Özet

2018 yılında yayımlanan ve 1 Ocak 2019 tarihinde yürürlüğe giren Türkiye Bina Deprem Yönetmeliğinde (TBDY 2018) Deprem Bölgelerinde Yapılacak Binalar Hakkında Yönetmelik (TDY 2007)'e göre birçok alanda değişiklikler yapılmıştır. Bu alanlardan bir tanesi de binanın kullanım amacına göre belirlenen bina önem katsayısının değişmesidir. Bina önem katsayısı; tasarım aşamasındaki bir yapıda oluşan deprem yükünün depremden sonra kullanılma durumuna göre belirlenen katsayıdır. Okul ve eğitim bina ve tesisleri, yurt ve yatakhaneler, askeri kışlalar, cezaevleri vb. ve müzelerin kullanım sınıfları yeni yönetmeliğe göre depremden sonra kullanılması zorunlu binalar olarak değiştirilmiştir. Yapılan çalışma kapsamında; programda bulunan örnek eğitim binasının TDY2007 ve TBDY2018'e göre analizi yapılmış ve çıkan veriler karşılaştırılmıştır.

1 INTRODUCTION

Approximately 92% of our country's territory, 95% of the population, almost all of the industrial centers are located in the active earthquake zone (Taşan, 2012). Therefore, all structures must be constructed against earthquake (Öztürk 2005); (Nemrutlu and Sarı, 2018); (Tunc ve Tanfener, 2016). In 2018, a new earthquake regulation (TEC 2018) was published and the old earthquake regulation (TEC 2007) was repealed. With the new earthquake regulation that came into force in 2019, studies where two regulations were compared with each other started to be published (Haj Ahmet, 2018); (Ulutaş, 2018). One of the changes in the new regulation is the building importance factor, which is determined according to the purpose of use of the building. Educational buildings, dormitories, military barracks, prisons etc. usage classes have been changed to buildings that must be used after the earthquake according to the new regulation. The building importance coefficient of these structures has increased from "I" = 1.4 to "I" = 1.5.

In this study; A sample education building was analysed with the protastructure program. The building importance coefficient, which changed according to the new regulation, was analysed according to TEC 2007 and TEC 2018 and the data released were compared.

2 BUILDING INFORMATION

Number of Floors = 7

Rigid basement = 1

Concrete Class = C30/37(Foundation)/ C5/45 (other stories)

Rebar Class = B420C

Table 1. Building Parameters

Floor	Height (cm)	Elevation (cm)	Coefficient of Live Load
6	350.00	2450.00	0.30
5	350.00	2100.00	0.30
4	350.00	1750.00	0.30
3	350.00	1400.00	0.30
2	350.00	1050.00	0.30
1	350.00	700.00	0.30
Basement	350.00	350.00	0.30

Figure 1. 3D model of example building

The sample building shown in Figure 1 measures 56 * 24 * 24.5 m and consists of a basement and six floors.

2. SEISMIC PARAMETERS

Analyze Type	= Static Analyze
Degrees of Freedom	= X, Y and Rotation
Rigid zones in the junction	= Will be reduced by 25%
Earthquake Code	= **TEC 2007 / TEC2018**
Earthquake Zone	= 1. Zone
Effective ground acceleration(Ao)	= 0.40 (TDTH, 2020).
Structural system type	= 1.4/1.5
Structural system concrete frames beam	= Buildings where seismic effects are met by reinforced with high ductility level transmitting momentum and bond (hollow) reinforced concrete curtains with high ductility level

S.S. behavior coefficient, (R)	= 6.66
Ductility Level	= High
Building Purpose	= School
Eccentricity, (%)	= 5.0
Soil Class	=Z2/ZB

3. ANALYSIS CHECKS

Table 2. Comparison of analysis results according to two codes

TEC2007	TEC2018
No (B2) Soft Story Irregularity	**No Soft Story Irregularity** √
(A1) Torsional irregularity control ηC : Δmax / Δort A1 irregularity in 1 direction (0.00 degrees with X-Axis) It was detected in the 2 direction (90.00 degrees with X-Axis) irregularity in A1. Max. Torsional Irregularity Coefficient = 1.640 ≤ 2.0 The building was re-analyzed by applying Additional Eccentricities. √	**(A1) BURULMA DÜZENSİZLİĞİ KONTROLU**: ηC : Δmax / Δort No A1 torsional irregularity in 1 direction (0.00 degrees with X-Axis) In the direction of 2 (90.00 degrees with X-axis), there is a torsional irregularity in the structure. Max. Torsional Irregularity Coefficient = 1.668 ≤ 2.0 Earthquake Design Class: DTS = 1a Building Height Class: BYS=5 ≥ BYS=4 (Hn = 21.00 m) (TBDY 2018 - Article 4.6.2.2) The building was re-analyzed by applying Additional Eccentricities.
--	BUILDING BASE AND BUILDING HEIGHT CONTROL: Tp,all / Tp,up = 1.0000 ≤ 1.1√
STRUCTURAL SYSTEM CONTROL: 1 direction (0.00 degrees with X-Axis) αS = Vp / Vt = (E +) = 0.67 ≤ 0.75 / (E-) = 0.67 ≤ 0.75 Structural System can be accepted as Shearwall + Frame. √ 2 direction (90.00 degree with X-Axis) αS = Vp / Vt = (E +) = 0.83> 0.75 / (E-) = 0.83> 0.75 Structural System Behavior Coefficient: R = 10 - 4 αS = 6.66 was used. Relative floor shifts control: Relative Floor Shifts provide Limit Values in 1 and 2 directions. OK.√	**STRUCTURAL SYSTEM CONTROL:** 1 direction (0.00 degrees with X-Axis) αM = MDev / Mo = 0.53 <0.75 Structural System: It can be accepted as A15. √ R = 7.00 and D = 2.50 will be used in the calculation of the design results. 2 direction (90.00 degree with X-Axis) αM = MDev / Mo = 0.71 <0.75 Building Carrier System: It can be accepted as A15. √ In the calculation of the design results: R = 7.00 and D =2.50 will be used Relative floor shifts control: Relative Floor Shifts provide Limit Values in 1 and 2 directions. OK.√
533/5000 EARTHQUAKE STATUS BUILDING TILTING CONTROL: Earthquake effects F1 and F2 were calculated using R = 6.664. ACTIVE EFFECTS: Total Ma1 (kN.m): 293135.95 Ma2 (kN.m): 341575.12 EFFECTS AGAINST TIPPING (Negative Earthquake Direction): Mp1 (kN.m): 3.295E + 06 Mp2 (kN.m): 1.412E + 06 EFFECTS AGAINST TIPPING (Positive Earthquake Direction): Mp1 (kN.m): 3.857E + 06 Mp2 (kN.m): 1.653E + 06 Roll Over Control: Direction 1 ... Mp1 / Ma1 = 3.295E + 06 / 293135.95 = 11.2404 ≥ 2.0 OK. √ Tip Over Control: Direction 2 ... Mp2 / Ma2 = 1.412E + 06 / 341575.12 = 4.1342 ≥ 2.0 OK. √	EARTHQUAKE STATUS BUILDING TILTING CONTROL: Earthquake effects F1 and F2 were calculated using R = 7.00. ACTIVE EFFECTS: Total Ma1 (kN.m): 113493.53 Ma2 (kN.m): 148 347.26 EFFECTS AGAINST TIPPING (Negative Earthquake Direction): Mp1 (kN.m): 3.295E + 06 Mp2 (kN.m): 1.412E + 06 EFFECTS AGAINST TIPPING (Positive Earthquake Direction): Mp1 (kN.m): 3.857E + 06 Mp2 (kN.m): 1.653E + 06 Roll Over Control: Direction 1 ... Mp1 / Ma1 = 3.295E + 06 / 113493.53 = 29.0323 ≥ 2.0 OK. √ Tip Over Control: Direction 2 ... Mp2 / Ma2 = 1.412E + 06 / 148347.26 = 9.5191 ≥ 2.0 OK. √

The protastructure outputs of the analysis results are given in figure 2 and 3.

O Z D E G E R   S I S T E M   M O D A L   C A S E S : MODALALL

DENKLEM SAYISI	:	34005
KUTLE SAYISI	:	21
OZDEGER SAYISI	:	6

O Z D E G E R L E R   V E   F R E K A N S L A R

MODE SAYISI	PERIYOD (SN)	FREKANS (ÇEVRİM/SN)	ACISALFREKAN (RAD/SN)	OZDEGER (RAD/SN)**2
1	0.595772	1.678495	10.546297	111.224386
2	0.561554	1.780771	11.188916	125.191847
3	0.463156	2.159101	13.566032	184.037216
4	0.181818	5.500009	34.557576	1194.226036
5	0.158449	6.311173	39.654268	1572.460979
6	0.111633	8.957893	56.284103	3167.900277

K A T K I   C A R P A N I

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.595772	0.915148E-04	-.180238E-03	0.000000	0.000000	0.000000	1606.287592
2	0.561554	99.191185	0.111732E-04	0.000000	0.000000	0.000000	-.141012E-02
3	0.463156	-.891801E-05	98.202012	0.000000	0.000000	0.000000	0.350918E-03
4	0.181818	-.185084E-04	0.606666E-03	0.000000	0.000000	0.000000	-591.888440
5	0.158449	39.985172	0.749131E-05	0.000000	0.000000	0.000000	-.284979E-03
6	0.111633	-.344255E-05	48.361780	0.000000	0.000000	0.000000	-.550152E-02

E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.595772	0.596358E-10	0.231323E-09	0.000000	0.000000	0.000000	65.574966
2	0.561554	70.060107	0.888947E-12	0.000000	0.000000	0.000000	0.505362E-10
3	0.463156	0.566318E-12	68.669742	0.000000	0.000000	0.000000	0.312971E-11
4	0.181818	0.243930E-11	0.262074E-08	0.000000	0.000000	0.000000	8.903714
5	0.158449	11.384726	0.399613E-12	0.000000	0.000000	0.000000	0.206403E-11
6	0.111633	0.843889E-13	16.654408	0.000000	0.000000	0.000000	0.769231E-09

T O P L A M   E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-YON	Y-YON	Z-YON
1	0.595772	0.596358E-10	0.231323E-09	0.000000	0.000000	0.000000	65.574966
2	0.561554	70.060107	0.232212E-09	0.000000	0.000000	0.000000	65.574966
3	0.463156	70.060107	68.669742	0.000000	0.000000	0.000000	65.574966
4	0.181818	70.060107	68.669742	0.000000	0.000000	0.000000	74.478680
5	0.158449	81.444833	68.669742	0.000000	0.000000	0.000000	74.478680
6	0.111633	81.444833	85.324150	0.000000	0.000000	0.000000	74.478680

O Z D E G E R   S I S T E M   M O D A L   C A S E S : MODALSUPER

DENKLEM SAYISI	:	34005
KUTLE SAYISI	:	18

O Z D E G E R L E R   V E   F R E K A N S L A R

MODE SAYISI	PERIYOD (SN)	FREKANS (ÇEVRİM/SN)	ACISALFREKAN (RAD/SN)	OZDEGER (RAD/SN)**2
1	0.595769	1.678503	10.546348	111.225453
2	0.561545	1.780801	11.189100	125.195952
3	0.463107	2.159327	13.567453	184.075781
4	0.181806	5.500362	34.559792	1194.379233
5	0.158402	6.313065	39.666156	1573.403940
6	0.111211	8.991907	56.497819	3192.003497

K A T K I   C A R P A N I

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.595769	0.910600E-04	-.139413E-04	0.000000	0.000000	0.000000	1603.437888
2	0.561545	98.932816	0.108754E-04	0.000000	0.000000	0.000000	-.140919E-02
3	0.463107	-.896862E-05	97.551779	0.000000	0.000000	0.000000	0.208549E-03
4	0.181806	-.172941E-04	-.397468E-05	0.000000	0.000000	0.000000	-581.428020
5	0.158402	38.880664	0.615046E-05	0.000000	0.000000	0.000000	-.286993E-03
6	0.111211	-.390090E-05	44.503453	0.000000	0.000000	0.000000	-.595475E-04

E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.595769	0.691243E-10	0.162024E-11	0.000000	0.000000	0.000000	83.313908
2	0.561545	81.593557	0.985979E-12	0.000000	0.000000	0.000000	0.643505E-10
3	0.463107	0.670542E-12	79.331472	0.000000	0.000000	0.000000	0.140938E-11
4	0.181806	0.249327E-11	0.131698E-12	0.000000	0.000000	0.000000	10.954817
5	0.158402	12.602087	0.315348E-12	0.000000	0.000000	0.000000	0.266904E-11
6	0.111211	0.126854E-12	16.510588	0.000000	0.000000	0.000000	0.114905E-12

T O P L A M   E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-YON	Y-YON	Z-YON
1	0.595769	0.691243E-10	0.162024E-11	0.000000	0.000000	0.000000	83.313908
2	0.561545	81.593557	0.260622E-11	0.000000	0.000000	0.000000	83.313908
3	0.463107	81.593557	79.331472	0.000000	0.000000	0.000000	83.313908
4	0.181806	81.593557	79.331472	0.000000	0.000000	0.000000	94.268726
5	0.158402	94.195644	79.331472	0.000000	0.000000	0.000000	94.268726
6	0.111211	94.195644	95.842061	0.000000	0.000000	0.000000	94.268726

Figure 2; Analysis results according to TEC2007 (ProtaStructure, 2020).

O Z D E G E R   S I S T E M   M O D A L   C A S E S : MODALALL

DENKLEM SAYISI : 34005
KUTLE SAYISI : 21
OZDEGER SAYISI : 6

O Z D E G E R L E R   V E   F R E K A N S L A R

MODE SAYISI	PERIYOD (SN)	FREKANS (ÇEVRİM/SN)	ACISALFREKAN (RAD/SN)	OZDEGER (RAD/SN)**2
1	0.891391	1.121842	7.048743	49.684785
2	0.869012	1.150732	7.230261	52.276678
3	0.668903	1.494986	9.393274	88.233589
4	0.263695	3.792255	23.827441	567.746964
5	0.238491	4.193034	26.345607	694.090989
6	0.158845	6.295458	39.555529	1564.639905

K A T K I   C A R P A N I

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.891391	0.001283	-.144556E-03	0.000000	0.000000	0.000000	1594.403766
2	0.869012	98.677253	0.658245E-05	0.000000	0.000000	0.000000	-0.020446
3	0.668903	-.442246E-05	97.999572	0.000000	0.000000	0.000000	0.382424E-04
4	0.263695	-.876685E-04	0.596530E-03	0.000000	0.000000	0.000000	-606.169157
5	0.238491	40.406931	0.679631E-05	0.000000	0.000000	0.000000	-.160414E-02
6	0.158845	-.191085E-05	48.549361	0.000000	0.000000	0.000000	-.522057E-02

E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.891391	0.117285E-07	0.148799E-09	0.000000	0.000000	0.000000	64.608266
2	0.869012	69.335993	0.308532E-12	0.000000	0.000000	0.000000	0.106245E-07
3	0.668903	0.139268E-12	68.386913	0.000000	0.000000	0.000000	0.371690E-13
4	0.263695	0.547283E-10	0.253390E-08	0.000000	0.000000	0.000000	9.338543
5	0.238491	11.626162	0.328905E-12	0.000000	0.000000	0.000000	0.653998E-10
6	0.158845	0.260002E-13	16.783853	0.000000	0.000000	0.000000	0.692670E-09

T O P L A M   E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-YON	Y-YON	Z-YON
1	0.891391	0.117285E-07	0.148799E-09	0.000000	0.000000	0.000000	64.608266
2	0.869012	69.335993	0.149107E-09	0.000000	0.000000	0.000000	64.608266
3	0.668903	69.335993	68.386913	0.000000	0.000000	0.000000	64.608266
4	0.263695	69.335993	68.386913	0.000000	0.000000	0.000000	73.946810
5	0.238491	80.962155	68.386913	0.000000	0.000000	0.000000	73.946810
6	0.158845	80.962155	85.170766	0.000000	0.000000	0.000000	73.946810

OZDEGER SAYISI : 6

O Z D E G E R L E R   V E   F R E K A N S L A R

MODE SAYISI	PERIYOD (SN)	FREKANS (ÇEVRİM/SN)	ACISALFREKAN (RAD/SN)	OZDEGER (RAD/SN)**2
1	0.891387	1.121847	7.048772	49.685183
2	0.869001	1.150746	7.230352	52.277994
3	0.668838	1.495130	9.394178	88.250576
4	0.263679	3.792494	23.828945	567.818622
5	0.238430	4.194096	26.352280	694.442687
6	0.158266	6.318496	39.700280	1576.112213

K A T K I   C A R P A N I

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.891387	0.001279	0.568192E-05	0.000000	0.000000	0.000000	1591.797315
2	0.869001	98.450801	0.633206E-05	0.000000	0.000000	0.000000	-0.020407
3	0.668838	-.451432E-05	97.376119	0.000000	0.000000	0.000000	-.636227E-04
4	0.263679	0.839277E-04	0.547029E-05	0.000000	0.000000	0.000000	595.791677
5	0.238430	39.391164	0.547233E-05	0.000000	0.000000	0.000000	-.158862E-02
6	0.158266	-.259854E-05	44.757805	0.000000	0.000000	0.000000	-.389060E-04

E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-DON	Y-DON	Z-DON
1	0.891387	0.136435E-07	0.269132E-12	0.000000	0.000000	0.000000	82.108622
2	0.869001	80.800423	0.334245E-12	0.000000	0.000000	0.000000	0.134953E-07
3	0.668838	0.169887E-12	79.046027	0.000000	0.000000	0.000000	0.131171E-12
4	0.263679	0.587199E-10	0.249457E-12	0.000000	0.000000	0.000000	11.502761
5	0.238430	12.935189	0.249643E-12	0.000000	0.000000	0.000000	0.817811E-10
6	0.158266	0.562902E-13	16.699855	0.000000	0.000000	0.000000	0.490508E-13

T O P L A M   E T K I N   K U T L E   O R A N I - (%)

MOD	PERIYOD	X-YON	Y-YON	Z-YON	X-YON	Y-YON	Z-YON
1	0.891387	0.136435E-07	0.269132E-12	0.000000	0.000000	0.000000	82.108622
2	0.869001	80.800423	0.603378E-12	0.000000	0.000000	0.000000	82.108622
3	0.668838	80.800423	79.046027	0.000000	0.000000	0.000000	82.108622
4	0.263679	80.800423	79.046027	0.000000	0.000000	0.000000	93.611382
5	0.238430	93.735612	79.046027	0.000000	0.000000	0.000000	93.611382
6	0.158266	93.735612	95.745882	0.000000	0.000000	0.000000	93.611382

Figure 3; Analysis results according to TEC2018 (ProtaStructure, 2020)

4. CONCLUSION

In this study, a sample education building was analyzed under the same conditions according to TEC2007 and TEC2018 and the results were compared. The results of the analysis showed that the numerical data have changed somewhat. No significant difference was observed in the general situation of the building. Some period difference in the earthquake report results draw attention. The reason for the period difference is that in TEC2007 only the horizontal spectrum is created for one earthquake. In the new regulation, both horizontal and vertical spectra are created for repetition periods of 2475 years, 475 years, 72 years and 43 years. In the new regulation, the fixed displacement plateau and the TL (transition period to the fixed displacement zone) determining this plateau are included in the spectrum. In TEC2018, this period is considered as 6s. Thus, displacement request does not increase uncontrolled. In addition, in the new earthquake regulation, the results are more sensitive since the earthquake parameters that affect the account are taken on the coordinate, that is, a more realistic calculation approach.

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RESEARCH ARTICLE

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A Study on the Effects of Highway Reinforcement on Load Distribution

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Keywords:
Base-subbase reinforcement,
Geogrid,
Base-Subbase Layers

Abstract

Highways, one of the most fundamental elements of civil engineering, are constantly exposed to dynamic loads. Undesirable damage due to these loads can be occurred. This study discusses the geogrids used to strengthen highway layers. Using the probability density functions obtained from the experimental results, damage probability tables were created for highway layers reinforced with geogrids. Vertical pressure distribution according to traffic loads, tire configuration and layer thickness were examined by field and laboratory tests. Within the scope of the study, five types of geogrids were used for experimental and analytical studies. “As a result, damage for any road reinforced with different geogrid types can be estimated using the damage probability table obtained in this study.

Otoyol Güçlendirmenin Yük Dağılımına Etkileri Üzerine bir Çalışma

Anahtar Kelimeler;
Temel-alttemel güçlendirmesi,
Geogrid,
Temel alttemel tabakaları

Özet

İnşaat mühendisliğinin en temel unsurlarından biri olan otoyollar sürekli olarak dinamik yüklemelere maruz kalmaktadır. Bu yüklerin sebep olduğu istenmeyen hasarlar ortaya çıkabilmektedir. Bu çalışma otoyol tabakalarının güçlendirilmesinde kullanılan geogridleri ele almaktadır. Deneysel sonuçlardan elde edilen olasılık yoğunluk fonksiyonları kullanılarak geogridler ile güçlendirilmiş otoyol tabakaları için hasar olasılık cetvelleri oluşturulmuştur. Trafik yüklerine, lastik konfigürasyonuna ve tabaka kalınlığına göre dikey basınç dağılımı saha ve laboratuvar testleri ile incelenmiştir. Deneysel ve analitik çalışmalar için beş tür geogrid kullanılmıştır. Bu çalışma sonucunda elde edilen hasar olasılık cetveli ile; farklı geogrid tipleriyle güçlendirilmiş herhangi bir yol için hasar tahmini yapılabilir.

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1. INTRODUCTION

According to the developing infrastructure, in addition to the rapid increase in urbanization and more carrying capacity of transportation trucks, the importance of road and highway transportation technology has also grown worldwide. Also, high traffic characteristics bring out higher traffic volumes and tire pressures; consequently, development in vehicle axles, tyres and technologies causes road damages (Cebon D., 2000; Sebaaly P. E., 1992). Excessive axle load increases road damaging potential, exponentially. Therefore, transport planning for future and improving economic life standards are very crucial. Recent research indicates truck weights over 113.4 ton and tire pressures of 150 psi (Emery J., 2007). Due to the increased traffic capacity and heavier vehicle loads, the roads are exposed to more stress and strain factors, permanent deformations, rutting and fatigue damages than ever before (Mulungye et al., 1987). If these stresses measured on the road surface (definite point on the pavement) or interface of sub-layers have too high values, they may induce a permanent road surface or sub-layer damage. These permanent deformations reduce the road economic life, increase the maintenance costs and may give rise to failure of the asphalt pavements, rutting and fatigue failure, particularly or completely.

A flexible pavement consists of asphalt surfacing layer and combined unbound aggregate road base, on a subgrade of natural soil. The stress distribution can be seen in Figure 1. (Mulungye et al., 1987). Truck loads bring out the top and bottom of the pavement to shift rapidly from compression to tension, and fatigue cracks result from the repeated tensile strains. According to the elastic layer theory, the maximum strain is located at the bottom of the asphalt surfacing layer (Ullidtz P., 1987). In order to predict the formation of fatigue cracking, a lot of pavement design models are based on straining at the bottom of the asphalt layer (European Commission, 1997). The vertical strain on the road layers and the subgrade causes deformation as a result of rutting.

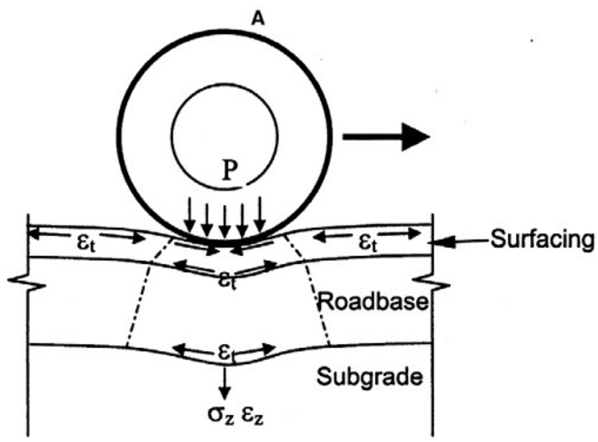

Figure 1. Strain distribution in road layers (Mulungye et al., 1987).

Since the first pneumatic tire was made in 1888 by J.B. Dunlop, many researches have fulfilled the related truck loads effecting on road layers (surface layer, base, subbase and subgrade) and the interaction between vehicle tires and pavement (Behiry, 2013). Some studies related to the tire science. A tire is the connection between truck and road surface and it is made through the tire contact area, which occurs due to all forces included in the vehicle and its movement.

The tire contact area is related to the rough surface, aggregate and cements characteristics. For the hot-mix asphalt concrete, these aggregates are of varies sizes, ranging from very fine particle size to 20 mm dimensions and road surfacing seals have approximately 13 mm dimensions (Beer and Fisher, 2013).

2. RESEARCH APPROACH

2.1. Loading Unit and Experiments

The tests were performed using a loading unit and a test box of pull-out device illustrated in Figure 2. The pull-out test device was constituted of a rigid pull-out box which had steel profiles, a loading and clamping system, measurement sensors (pressure gages, strain gages, LVDT) and a data acquisition system.

Figure 2. The loading unit and test.

In the test unit, the subgrade material was put into the half of the box then geogrid was laid and, subbase material was spread above the geogrid. Optimum water contents of the subgrade material and the subbase material were 18% and 4.7%, respectively. The loose granular subgrade fill material was placed in the 100 mm lifts. The total fill thickness of 800 mm was maintained prior to pull-out testing.

2.2. Soil Properties

The aggregates used in this study were chosen as mainly existed in the Black Sea Region in Turkey. Subbase materials were provided by the Highway Regional Officials. After subbase material was subject to the drying process in oven, sieve analysis was performed with ASTM sieves in KTU Department of Civil Engineering Structures and Materials Laboratory. Subbase and subgrade materials contained 20 % filler, 60 % fine aggregate, 20 % coarse aggregate and 3 % filler, 45 % fine aggregate, and 52 % coarse aggregate, respectively. Gradation curves are shown in Figure 3.

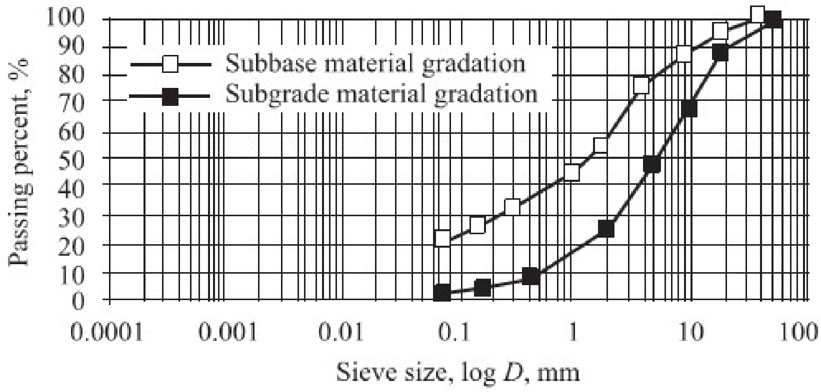

Figure 3. Gradation distribution of soils.

2.3. Geogrid Materials

The test specimen was cut from the sample rolls of the geogrid material. Five types of geogrid samples were used in this loading unit. These geogrids which were 50×50 mm, 40×40 mm, 30×30 mm square aperture size, hexagonal and crosswise aperture shape geogrids are shown in Figure 4.

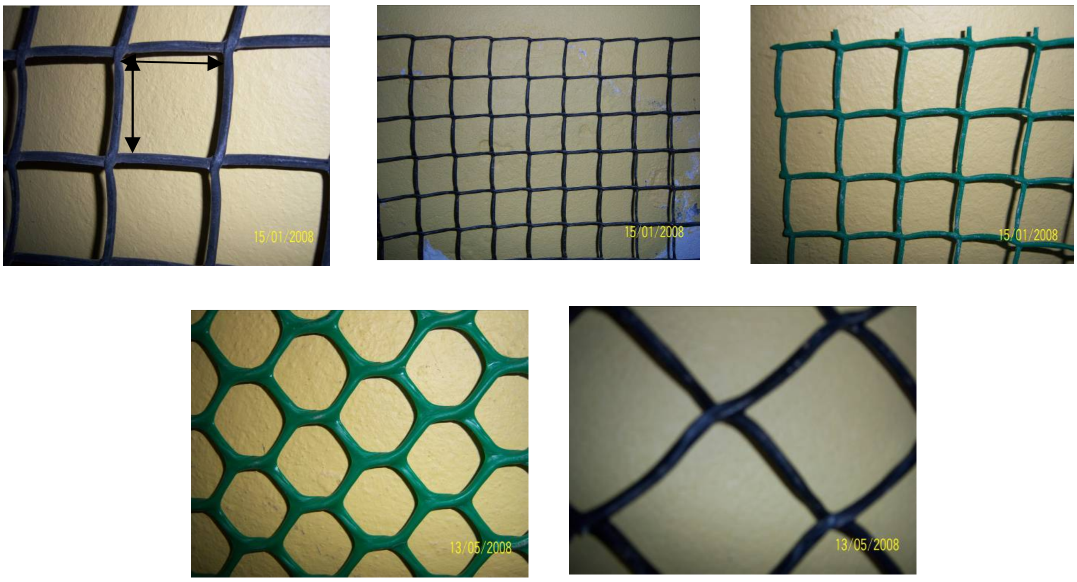

Figure 4. Geogrid Examples.

2.4. Field Analyses

In literature, an 80 kN single axle load (Mulungye et al., 1987), (Sert and Akpinar, 2012), (Fange et al., 2007) and 700 kPa pressure are considered (Wu, 2007). The most commonly used equivalent load in the U.S. is the 18,000 lb (80 kN) equivalent single axle load (generally designated as ESAL) (2002 Guide for the Design of New and Rehabilitated Pavement Structures). Park (2008) determined that tire pressures on the pavements ranged from 550 kPa to 890 kPa. (Priest and Tim, 2005) showed the vertical pressure on the subbase layer as 35 kPa. In this study, vertical pressure and tire load were measured on the field by using the 200 mm diameter pressure gauges installed on the top of the asphalt pavement layer and subbase layer. The pressure values ranging from 550 kPa to 790 kPa on the top of the pavement surface and from 31 to 33 kPa on the top of the subbase layer were obtained. In each test, in accordance to the field, 35 kPa vertical pressure and ESAL's of 80 kN were applied by the vertical piston.

4. RESULTS AND DISCUSSIONS

The data obtained from pressure cells settled in sublayers are seen in Table 1. Two different levels existed. One of them was upper level and the other one was lower level of geogrid. For each level, four pressure sensors were used and every test was repeated three times.

Table. 1. Pressure Distributions According to Geogrid Samples.

Geogrid Aperture Shapes	Test No	Pressures from lower level of geogrid (kPa)				Pressures from upper level of geogrid (kPa)			
		Sensor No				Sensor No			
		1	2	3	4	5	6	7	8
50×50 mm	1	91	14	27	64	113	18	13	39
	2	54	39	17	56	63	73	4	29
	3	14	5	17	34	22	31	14	75
40×40 mm	1	5	17	21	40	1	2	3	53
	2	31	13	16	18	25	16	17	9
	3	26	19	32	13	40	41	16	66
30×30 mm	1	5	6	19	_	5	7	11	_
	2	17	3	65	26	2	11	109	101
	3	7	16	36	17	155	11	20	31
Hexagonal	1	29	24	34	19	51	23	21	6
	2	15	58	50	28	27	66	43	72
	3	18	2	19	1	24	25	14	15
Crosswise	1	31	38	45	34	19	51	37	_
	2	44	37	163	82	56	34	95	186
	3	29	44	65	70	25	40	46	102

After obtaining vertical pressure measurements of sublayers; lognormal mean vertical pressure value and lognormal standard deviation of values for each material type were utilized. The probability of reaching or exceeding a "damage state" as demand parameters of vertical pressure was calculated. The obtained distribution graphic of sublayers reinforced with geogrids are shown in Figure 5.

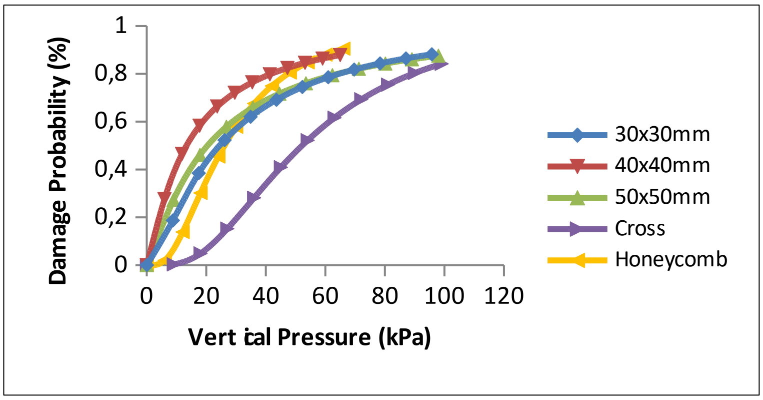

Figure 5. Cumulative distribution functions on the top of the geogrids.

Load distributions for the different types of geogrid at upper level are shown on the same graph. The horizontal axis of the graph shows the vertical pressure values acting on the soil layer, while the vertical axis of the graph shows the potential for these pressures.

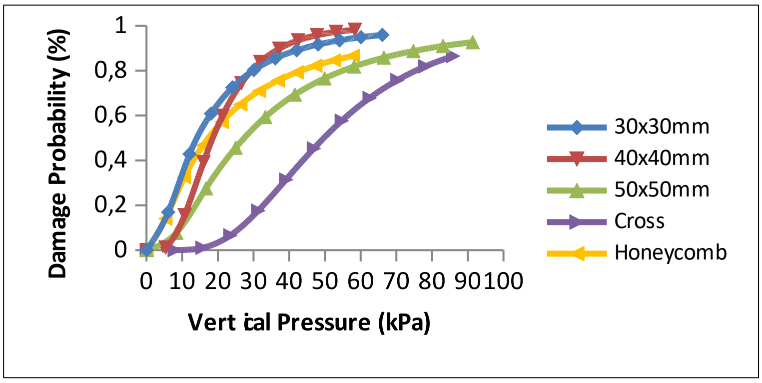

Figure 6. Cumulative distribution functions at the bottom of the geogrids.

Figure 6 shows the load distributions in the lower layers. When Figure 5 and Figure 6 are analysed together, vertical pressure distributions obtained from different shaped would significantly change between top and bottom layers.

Pressure gauges installed on upper and lower levels of the geogrid indicated that the geogrids reduce the vertical stress significantly by distributing the vertical load to a wide range over the subgrade soil. The vertical pressures were obtained from tests. The graphics indicated that the 40×40 mm aperture size geogrid and the hexagonal aperture size geogrid showed close performances. The 30×30 mm aperture size geogrid showed some more resistance to vertical pressure. But when it started to lose its strength, it showed less resistance to vertical pressure distribution than the 40×40 mm aperture size geogrid. The reduction in the vertical stress on upper level of the geogrid was 12%, 14%, 52%, 10% and 25% for 50×50 mm, 40×40 mm, 30×30 mm square aperture size, crosswise and hexagonal aperture shape geogrids, respectively. This crucial result shows that smaller aperture size geogrids can improve the subgrade bearing capacity in terms of vertical stresses.

When it is thought that tensile stress behaviour has a different tendency than vertical stress distribution, the 50×50 mm aperture size geogrid gives a better performance with the respect to tensile stress behavior than the other geogrid types. As long as high strength junctions and square aperture shape advantage are provided with the effective mechanical interlock of aggregate particles into the aperture, the permanent deformation of subbase layer is expected to result in a high resistance. Because of its larger aperture size, 50x50mm aperture size geogrid effects more than the other square size aperture geogrids, and so it can penetrate more aggregate inside aperture. According to this feature, 50x50mm aperture size geogrid indicates a perfect interlock effect. However, in terms of vertical stress distribution in sub-soils of highway, an opposite behaviour is observed for the tensile stress distribution. In this case, smaller aperture size geogrids can improve the sub-soil bearing capacity

5. CONCLUSIONS

In this study, vertical pressure distributions in the loading unit were analysed and the cumulative distribution of damage was formed for five different type geogrids. According to experimental and analytical results; the basic conclusions obtained from the study are as follows:

1. The geogrids reduce the vertical stress significantly by distributing the vertical load to a wide range over the subgrade soil. It can be said that using geogrid for road embankments is so efficient.
2. The square aperture size geogrids are more efficient than other type geogrids (hexagonal and crosswise) in terms of load transfer.
3. The reduction in the vertical stress on upper level of the geogrid was 12%, 14%, 52%, 10% and 25% for 50×50 mm, 40×40 mm, 30×30 mm square aperture size, crosswise and hexagonal aperture shape geogrids, respectively.
4. The 30×30 mm aperture size geogrid has shown some more resistance to vertical pressure.
5. Smaller aperture size geogrids can improve the subgrade bearing capacity in terms of vertical stresses.

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