

ASSESSMENT OF EXISTING BUILDINGS BY USING THE LINEAR
ELASTIC PROCEDURE ACCORDING TO THE TURKISH BUILDING
EARTHQUAKE CODE (2018)

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EARTHQUAKE CODE (2018)**

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ABSTRACT

ASSESSMENT OF EXISTING BUILDINGS BY USING THE LINEAR ELASTIC PROCEDURE ACCORDING TO THE TURKISH BUILDING EARTHQUAKE CODE (2018)

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Evaluation of existing structures under earthquake loads has an important place in the field of performance-based earthquake engineering. In this respect, linear and non-linear analysis methods can be used in the evaluation of existing structures. Linear elastic analysis and evaluation techniques are notable for their simplicity in terms of modeling and analysis. Nevertheless, the precision of estimating seismic demands and the reliability of outcomes may differ depending on the seismic hazard level and the characteristics of the structural system. Conversely, nonlinear analysis computation methods employ nonlinear material models and geometric relationships that account for the damage behavior of structural materials and systems. These approaches can be more intricate than linear elastic methods and necessitate greater computational power due to the utilization of material models and solution algorithms. Analysis procedures and condition assessment methods for existing structures are thoroughly explained in the Turkish Building Earthquake Code 2018 (TBEC-2018). Plastic rotation demands can be calculated using the linear elastic force-deformation relationships and the seismic performance of the structure can be evaluated on the basis of these computed demands. In this study, three existing buildings with different using purposes and different structural features such as

school, dormitory building, which have an important place in the structure stock of Turkey are investigated as case studies. In the scope of this study, the parameters which are used in the application of the linear elastic assessment method are studied. Seismic performance evaluation is carried out using the TBEC (2018) linear elastic assessment procedure. Obtained results are then compared with those compiled from the nonlinear dynamic analyses. The comparative investigation reveals that the TBEC (2018) linear elastic assessment procedure is generally successful in estimating the global seismic response as well as individual member plastic rotation demands. In addition, this study is carried out within the scope of the UDAP project named as Ç-21-30 “Condition Assessment of Existing Buildings By Using The TEC 2018 Linear Elastic Procedure” and supported by AFAD.

Keywords: Condition Assessment, Seismic Design, Linear Analysis Procedures

ÖZ

MEVCUT BİNALARIN TÜRKİYE BİNA DEPREM YÖNETMELİĞİ 2018'E GÖRE DOĞRUSAL ELASTİK HESAP YÖNTEMİ İLE DEĞERLENDİRİLMESİ

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Mevcut yapıların deprem yükleri altında değerlendirilmesi, performans esaslı deprem mühendisliği alanında önemli bir yere sahiptir. Bu doğrultuda mevcut yapıların değerlendirilmesinde doğrusal ve doğrusal olmayan analiz yöntemleri kullanılabilmektedir. Doğrusal elastik analiz ve değerlendirme teknikleri, modelleme ve analiz açısından basitlikleri ile dikkat çekmektedir. Bununla birlikte, sismik talepleri tahmin etmenin kesinliği ve sonuçların güvenilirliği, sismik tehlike düzeyine ve yapısal sistemin özelliklerine bağlı olarak farklılık gösterebilir. Doğrusal olmayan analiz hesaplama yöntemleri ise, yapısal malzeme ve sistemlerin hasar davranışını açıklayan doğrusal olmayan malzeme modellerini ve geometrik ilişkileri kullanır. Bu yaklaşımlar, doğrusal elastik yöntemlerden daha karmaşık olabilmektedir ve malzeme modellerinin ve çözüm algoritmalarının kullanılması nedeniyle daha fazla hesaplama gücü gerektirmektedir. Mevcut yapılar için analiz prosedürleri ve durum değerlendirme yöntemleri, Türkiye Bina Deprem Yönetmeliği'nde (TBDY-2018) ayrıntılı olarak açıklanmıştır. Plastik dönme talepleri doğrusal elastik kuvvet-deformasyon ilişkileri kullanılarak hesaplanabilir ve yapının sismik performansı bu hesaplanan talepler temelinde değerlendirilebilmektedir.

Bu çalışmada, Türkiye yapı stoğunda önemli bir yere sahip olan okul, yurt binası gibi farklı kullanım amaçlarına ve farklı yapısal özelliklere sahip mevcut üç bina örnek olay olarak incelenmiştir. Bu çalışma kapsamında doğrusal elastik değerlendirme yönteminin uygulanmasında kullanılan parametreler incelenmiştir. Elde edilen sonuçlar daha sonra doğrusal olmayan dinamik analizlerden derlenenlerle karşılaştırılır. Karşılaştırmalı araştırma, TBDY (2018) doğrusal elastik değerlendirme prosedürünün, genel olarak sismik performansın yanı sıra yapısal elemanların plastik dönüş taleplerini tahmin etmede başarılı olduğunu ortaya koymaktadır. Ayrıca bu çalışma, Ç-21-30 “Mevcut Binaların TBDY 2018’e Göre Doğrusal Elastik Hesap Yöntemi İle Değerlendirilmesi” adlı UDAP projesi kapsamında yürütülmekte olup AFAD tarafından desteklenmektedir.

Anahtar Kelimeler: Durum Değerlendirmesi, Sismik Tasarım, Doğrusal Analiz



To my family,

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CHAPTER 1

INTRODUCTION

1.1 General

Seismic evaluation of existing structures is a popular research area in the context of performance-based earthquake engineering. In this respect, linear and non-linear calculation methods can be used in the evaluation of existing structures. Analysis procedures and condition assessment methods for existing structures are discussed in detail in seismic regulations ASCE 41-17 (2017) and Eurocode 8-3 (EN 1998-3, 2005), which are widely used in the world. These procedures are thoroughly explained in the Turkish Building Earthquake Code 2018 (TBEC-2018), as well.

Linear elastic analysis and evaluation methods come to the fore with their aspects such as simplicity in modeling and analysis. However, the accuracy of estimating the seismic demands and the validity of the results can be a critical issue depending on the level of seismic hazard and the characteristics of the structural system. Although linear elastic calculation methods are based on the basic principles of structural behavior and they are not difficult to implement, they can show limited success in accurate prediction of structural performance due to the simplifications and various assumptions brought by the simplified application-based approaches. This situation arises especially in systems where significant inelastic deformation is expected in cases with highly asymmetric structural systems and/or high levels of seismic hazard. On the other hand, nonlinear analysis calculation methods use nonlinear material models and complex geometric relationships that reflect the damage characteristics of structural materials and systems in a more reliable manner. Numerical models created with these enhanced features are analyzed with static and dynamic analysis methods such as push over analysis or time history analysis. As the analysis output, nonlinear deformations and internal forces in the structural system are obtained.

These methods can be more complex than linear elastic methods and they require more computational effort due to the complex material models and advanced solution algorithms used. For this reason, more expertise and qualified information are required to employ these methods compared to linear elastic methods. Nonlinear calculation methods can give more accurate predictions on the demands of the structures due to earthquake forces despite its difficulty and complexity.

1.2 Literature Review

There exist studies which have been focused on the efficiency of linear elastic analysis and nonlinear analysis approaches available in the literature. Various studies on the performance evaluation of existing buildings have been examined and discussed in this section.

Linear elastic methods form the basis of structural analysis used in the force-based seismic design of new buildings or in the seismic evaluation of existing buildings. Linear elastic methods are theoretically consistent and computationally stable. Compared to nonlinear calculation methods, they are simpler in terms of practical applicability. However, it has limited capacity to simulate nonlinear behavior. In the study conducted by Sucuoğlu and Özçamur (2015), the limits in the application of linear elastic methods were discussed. These limits were obtained by comparative analysis of the displacement-based structural response under spectrum analysis and nonlinear time history analysis. Both methods were applied comparatively for the analysis of structures with different number of stories under twenty different ground motion records. The plastic rotation demands of the structural members and the relative drift ratios between the stories were calculated. As a result of this study, it was determined that the ratio of average column demand to capacity and the ratio of average beam demand to capacity at the critical story are the most effective parameters in determining the validity range of linear elastic calculation methods.

One of the important issues in seismic evaluation of existing structures is to estimate inelastic maximum displacement demands under seismic action. Although this can be obtained with methods such as nonlinear time history analysis, it is mostly carried out with simpler procedures. The well-known practical applications can be stated as the displacement coefficient method and the equivalent linear elastic method. Equivalent linear elastic methods are basically based on the principle of estimating nonlinear demands using linear elastic numerical models with an equivalent period and damping ratio. Lin and Miranda (2009) evaluated the accuracy of estimating the nonlinear displacement demands by calculating the maximum values through four different equivalent linear methods. They used the equivalent linear method defined by the force reduction factor (R) in their study, unlike the linear equivalent methods defined by the ductility ratio (μ) employed by some other researchers. The reason is that the parameter μ is not known for existing structures and this value is to be determined by the expert opinion of structural engineers. This situation causes the inelastic maximum displacement demands of existing structures to be obtained after an iterative process. In order to prevent this iterative process and increase the accuracy, Lin and Miranda presented an equivalent linear method defined by the R factor. In their study, they defined R with a formula that correlates with the mass, spectral acceleration, and yield strength of the nonlinear system. Equivalent linear method and linear time history methods were applied under 72 ground motions and inelastic maximum displacements were examined. For all the R values used, the method presented by Lin and Miranda predicted inelastic displacements with a fairly good accuracy. As a result of their study, they stated that using the R factor instead of μ provides an advantage in terms of accuracy while formulating the equivalent linear elastic system parameters.

An advanced linear elastic assessment methodology for existing structures was proposed by Günay and Sucuoğlu (2010). In this method, structural members that are expected to show non-linear behavior under seismic demand are determined by comparing their roles within the structure, capacities, and the stiffness of these members is reduced in a single iteration for the entire structure. Predicted response

obtained from the proposed method were evaluated comparatively with the results of nonlinear time history analysis and multimodal pushover analysis. It was observed that the proposed method is successful in estimating the flexural demands of the deformation-controlled members and shear force demands in force-controlled members.

In the study conducted by Fragiakadis et al. (2014), the feasibility of nonlinear static methods in calculating the seismic demands of typical, regular reinforced concrete structures was discussed. It was stated that nonlinear static methods are quite common in the evaluation of structures exposed to seismic loads. It was also considered to be used in seismic design of new buildings. Nonlinear methods developed for the evaluation of existing buildings require a detailed and complex numerical modeling. Although nonlinear response history analysis is generally accepted as the most precise analysis method available, the high computational cost and difficulty in choosing suitable ground motions shift the focus on nonlinear static methods. In the study, nonlinear static methods were compared with nonlinear dynamic methods in order to better determine the applicability of single-mode and multi-mode pushover analysis. The accuracy of nonlinear static methods was presented as a result of the study, which is directly related to the characteristic features of the modeled structure and the nonlinear demands occurring in the structure. It has been shown that the displacement and relative story drift demands obtained by the mode superposition method, which is a linear elastic analysis method, can predict the average results obtained from the nonlinear dynamic analysis with an acceptable accuracy for multi-story structures.

In the study conducted by Memari et al. (2000), a seismic evaluation was performed on an existing 32-story building constructed with a reinforced concrete frame. The assessment conducted inelastic dynamic time-history analysis to determine the structural response in terms of force and deformation under the influence of three different ground motion records. Within the scope of the study, modeling process, which was based on the DRAIN-2D program, plastic rotation, rotation and curvature ductility requirements, and the assessment of member capacities was investigated.

The study also examines recent recommendations pertaining to plastic hinge rotation and adjustments in plastic hinge length. Furthermore, it assesses the outcomes of implementing some of these recommendations. This study provides insights into the shear lag effect and highlights certain challenges associated with applying the direct results obtained from inelastic dynamic time-history analysis to practical seismic assessments.

Browning et al. (2008) investigated the displacement responses of multi-degree-of-freedom (MDoF) nonlinear reinforced concrete building models and the displacement responses of equivalent single-degree-of-freedom (SDoF) and MDoF linear models comparatively under seismic loading. Displacement responses were characterized by roof displacement, maximum story deformation, and location of failure. A relationship was derived to determine the maximum nonlinear roof drift ratios. For determining nonlinear roof displacement, the relationship between linear response spectrum and effective period was determined as a result of a parametric analysis using 105 reinforced concrete frame models and 10 selected ground motions. In line with the results of the study, it was observed that the maximum story drift calculated by using nonlinear analysis was 1.5 times higher than the values calculated using linear modal analysis.

In the study conducted by Kalkan and Kunnath (2007), the efficiency of various nonlinear static procedures (NSPs) on seismic assessment of reinforced concrete structures were investigated. This evaluation is carried out by contrasting their predictions with benchmark responses derived from nonlinear time-history (NTH) analyses. A crucial element in the development of performance-based design methods is the precise estimation of seismic demand parameters. In engineering practice, nonlinear static procedures (NSPs) are now commonly employed to predict seismic demands in building structures. While NSPs can directly calculate seismic demands using a site-specific hazard spectrum, nonlinear time-history (NTH) analyses necessitate multiple ground motion records and a probabilistic assessment to consider the inherent variability in earthquake recordings. Despite this advantage, simplified versions of NSPs based on unchanging load patterns, such as those

recommended in ATC-40 and FEMA-356, have well-documented limitations. They struggle to account for higher mode effects and the modal variations resulting from inelastic behavior. Consequently, several improved pushover procedures have been proposed. The outcomes of the analytical investigation reveal that, among the various NSPs examined in the study, the Adaptive Modal Combination approach consistently provided more accurate predictions of peak response measures, including inter-story drift and component plastic rotations.

The estimation of inelastic deformation demands has a significant role in the earthquake resistant design of new structures or in the seismic evaluation of existing structures. In the study conducted by Sucuoğlu and Alici (2020), estimation of inelastic deformations in structural building frames through the implementation of response spectrum analysis. The difficulty of estimating inelastic seismic deformations by analyzing the linear elastic system dynamically has been a challenging task in earthquake engineering because of the practical reasons widely acknowledged. If it becomes feasible to represent the maximum displacement of a yielding system in relation to the maximum displacement of its corresponding linear elastic system, this presents a considerable advantage in terms of seismic design and the subsequent evaluation of structural system performance. In this study, first, the damping spectra are formulated for single-degree-of-freedom systems, which are defined by their initial period or initial stiffness. The aim is to achieve equivalent maximum displacements as the corresponding inelastic systems. Mean spectral relationships, along with their standard deviations, are computed for a selected ground motion set. Subsequently, optimal damping scaling coefficients are determined to convert the standard 5% damped linear elastic spectra into inelastic spectra. 5-story and 12-story reinforced concrete buildings which are code-based structure have been investigated as a case studies. Then, employing mode superposition analysis and the optimal spectral factors, they estimate the maximum deformations of multi-degree-of-freedom building frames. These results are then compared with outcomes obtained from nonlinear response history analysis conducted under the ground motion set. The proposed methodology yields

reasonably accurate predictions of mean inelastic deformations. Consequently, it is recommended that damping spectra providing optimal damping ratios be used as a practical tool for evaluating the seismic performance of newly designed and code-based structures.

In the study conducted by Panagiotakos and Fardis (1999), a method has been proposed for multi-story frames without infill walls. This new method is applied for simple but relatively accurate estimation of inelastic deformations for the entire structure. The deformation measures of interest in this study are the chord rotations at the member ends and the interstory drift ratios. Calculated chord rotations are used to determine the deformation capacity of a member or the detailed member design that must be done to accommodate lateral deformation due to seismic action. The inelastic interstory drift ratios and the chord rotations at the structural member ends, which were calculated in that study, were presented as the ratios to the corresponding deformation measurements as a result of the elastic analysis. Two types of elastic analysis were considered in the study. These are equivalent static lateral force analysis and multi-modal response spectrum analysis. Nonlinear story drifts and chord rotations are obtained by applying elastic analysis and scaling the obtained results by a conversion factor. Although this method was developed for the nonlinear analysis of completely regular and symmetrical frame structures, it was expected that the proposed method can also be applied to multi-modal response spectrum analysis of structures that are highly irregular in plan.

Previous studies on the verification of simplified methods given in seismic codes and standards for the performance evaluation of existing buildings are also examined. The results are summarized below.

Arujo and Castro (2017) examined the reliability of safe building performance controls defined in Eurocode 8-3 (EN 1998-3, 2005), which are based on considering plastic deformations. Their work showed that linear analysis fails to predict inelastic deformation demands. The accuracy of using linear analysis to predict nonlinear response in existing structures was discussed. Although the linear analysis

applicability criteria were met, it was determined that the local deformation demands could not be accurately estimated. For this reason, a simplified method was proposed to estimate the inelastic deformation demands using elastic analysis methods. A comparative study was presented to measure inelastic deformation demands by using linear analysis. The aim of this study was to propose a simplified method that allows the estimation of local inelastic chord rotation demands obtained by using linear elastic analysis. Alternative methodology has been proposed to overcome the limitations of linear elastic methods in predicting nonlinear deformation demands in steel moment frame systems. As a result of the study, it was stated that chord rotations were obtained consistently with this methodology.

There are also previous studies that focused on the comparison of linear and non-linear calculation methods applied for the performance assessment of existing structures in the 2007 Turkish Earthquake Code (DBYBHY, 2007). Şengöz and Sucuoğlu (2007) compared the methods which are presented in the 2007 Turkish Earthquake Code through an analytical study. In that study, two pairs of existing and strengthened residential buildings were evaluated by using the methods in the 2007 Turkish Earthquake Code to examine the differences between the seismic evaluation of existing structure and strengthening methods in the code. Then, the obtained results were compared with the actual damage observations in the 1999 Düzce Earthquake, in which a 5-story sample building was employed. As a result, it was revealed that both methods have some advantages and disadvantages, and the regulation does not impose any limitation on the selection of the procedures. The difference between these methods and the philosophy underlying these methods should be well understood by the structural engineer in order to get accurate and reliable results. It was stated that linear elastic and non-linear methods could not estimate the column damage states that occur during real earthquakes accurately. In addition, the relative story drift limits were unrealistic, but the linear elastic method seemed to be more conservative than the non-linear method.

Toprak et al. (2008) compared the damage observed in an existing reinforced concrete structure during the 1998 Adana-Ceyhan earthquake with the results of the

condition assessment using linear elastic methods. It was found that the linear elastic methods of Eurocode 8-3 (2005) and DBYBHY (2007) give similar demands in terms of estimating the failure performance of the critical story. It was stated that the calculated curvature demands are similar to each other by using the approach which has been given in both regulations.

In the study conducted by Karakas et al. (2021), eight different old structures (constructed before 1998) and a new (built after 1998) structure were investigated according to the calculation methods in DBYBHY(2007), TBEC(2018), Eurocode 8.3 (EN 1998-3, 2005) and ASCE 41-17 (2017) regulations and the results were compared. Building capacity curves of the structures were obtained, strength and displacement capacities were examined. By using more than 300 acceleration records, fragility graphs of structures were obtained through nonlinear time history analyses. It was stated that the strength, stiffness, and deformation capacity information of new structures are significantly higher for all the regulations taken as basis. It was concluded that this situation also affects the fragility responses of structures with the information that the vulnerability of old buildings is much higher than the new ones. The investigations have revealed that the displacement ductility calculated according to Eurocode 8-3 (EN 1998-3, 2005) is quite low compared to other standards. Lower effective stiffness estimations cause high yield displacements and low ductility capacities. On the other hand, considering the displacement capacities, it was concluded that the method proposed in TBEC (2018) is the most conservative method and gives the lowest plastic drift capacities compared to other regulations. The variation of vulnerabilities showed that the number of stories for mid-rise buildings is not as effective as other parameters. Instead, it was stated that the most effective parameters were the date of construction and the differences in code-based evaluation methodologies. The study showed that the damage probabilities are significantly different when the methods specified in different regulations have been used.

As can be seen from the detailed investigation of the previous studies in the literature, the calculation methods presented for the performance assessment of existing

structures show significant differences. Among the studies based on linear elastic procedures, the ones that examine the applicability limits of this method are very common. Studies on nonlinear calculation methods focus on structural performance evaluation by using incremental pushover analysis. Such studies generally present the methods developed as an alternative for time history dynamic analysis. Both approaches have their advantages and disadvantages. Although the linear elastic analysis and evaluation methods come to the fore with aspects such as simplified modeling and ease of analysis, the accuracy of estimating the demands under earthquake forces and the validity of the results obtained may vary according to the level of seismic hazard level and the characteristics of the structural system. In the previous studies, it was determined that the average column demand to capacity ratio (DCR) values calculated for the application limits of linear elastic methods and the average beam DCR values calculated at the critical story of the moment-frame system are the most effective parameters.

The preparation of the numerical structural model can be complex when the nonlinear calculation method is used. However, despite its complexity, the demands of the structures due to seismic forces can be estimated more accurately. Therefore, modeling approaches and the analysis methods can affect the obtained results.

1.3 Objective and Scope

The aim of this study is to evaluate how accurately the linear elastic calculation methods can predict the earthquake performance demands of the existing structures. Also, the results obtained serve as a validation for the linear elastic evaluation method in TBEC (2018) and contribute significantly to its applicability.

In scope of the study, the linear elastic calculation method in TBEC(2018) has been examined and evaluation methods in other similar standards have been compared. Afterwards, the parameters which are used in the application of the linear elastic calculation method within the scope of TBEC (2018) are studied. Following this stage, three existing buildings with different occupancy classes and structural

systems such as school, dormitory, which are commonly encountered in the Turkish building stock, have been inspected as case study examples. Analytical models of these structures are prepared, and the seismic demand estimation performance of the linear elastic method is investigated. Seismic responses estimated by the linear elastic analysis are compared with nonlinear dynamic analysis results. This study is carried out within the scope of the UDAP project named as Ç-21-30 “Condition Assessment of Existing Buildings By Using The TEC 2018 Linear Elastic Procedure” and supported by AFAD. The findings as a result of the study are also presented in project progress reports.

The outline of the thesis can be described as follows:

- Chapter 2 focuses on comparison of available performance assessment procedures for existing structures.
- In Chapter 3, the linear elastic method in TBEC (2018) is examined.
- Chapter 4 is dedicated to nonlinear dynamic analysis for selected case studies.
- Chapter 5 presents comparisons of TBEC (2018) linear elastic analysis and nonlinear dynamic analysis results.
- Chapter 6 focuses on summary and conclusions.

CHAPTER 2

COMPARISON OF AVAILABLE PERFORMANCE ASSESSMENT PROCEDURES FOR EXISTING BUILDINGS

2.1 General

Seismic assessment of existing buildings under seismic loads has an important place in globally accepted standards and regulations. Within the scope of this study, existing performance evaluation methods in ASCE 41-17(2017), TBEC (2018) and Eurocode 8-3(EN 1998-3, 2005) regulations are introduced and compared in this chapter.

2.2 The ASCE 41-17 Standard (2017)

The analysis methods which have been employed for the evaluation and seismic rehabilitation of existing buildings and the acceptance criteria for these methods are given in detail in Chapter 7 of ASCE41-17 (2017). Requirements for mathematical modeling of structures, basic assumptions, P- Δ effects, soil-structure interaction (SSI), multi-directional effects are also discussed. Four analysis methods are proposed in the regulation and the criteria to choose one of these four analysis methods are explained by taking into account the limitations of each method in application. Section 7.3 of the Standard specifies the criteria for the selection of analysis methods. A brief explanation can be provided as follows.

Analysis methods that can be used to evaluate the seismic performance of existing structures are specified as linear static method (LSP), linear dynamic method (LDP), nonlinear static method (NSP) and nonlinear dynamic method (NDP). The analysis of the structures is carried out to obtain the internal forces in the structural members and the deformations that occur in the building under the seismic action of the related

seismic hazard level. Linear elastic analysis methods are subjected to the limitations set out in Section 7.3.1. LSP and LDP are applied according to Sections 7.4.1 and 7.4.2 of the Standard. Nonlinear analysis methods are subjected to the limitations set out in section 7.3.2. NSP and NDP are applied according to Sections 7.4.3 and 7.4.4 of the Standard.

It is stated that one of the basic conditions for the application of linear elastic methods is that the buildings which are examined should meet the given irregularity constraints. These irregularity limitations are set out in Section 7.3.1.1 of the ASCE 41-17 Standard. These irregularities are grouped under three main headings, which are in-plane discontinuous irregularities, out-of-plane discontinuous irregularities, and weak storey irregularities. These situations are illustrated in Figures 2.1 and 2.2.

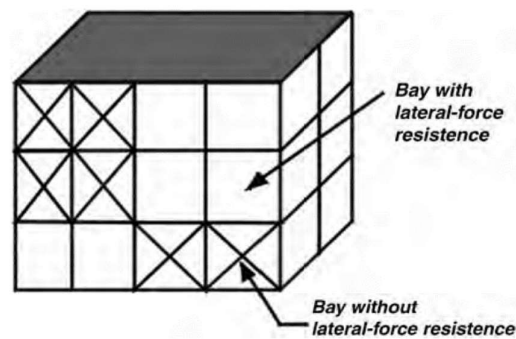


Figure 2.1 In-plane discontinuous irregularities (ASCE 41-17)

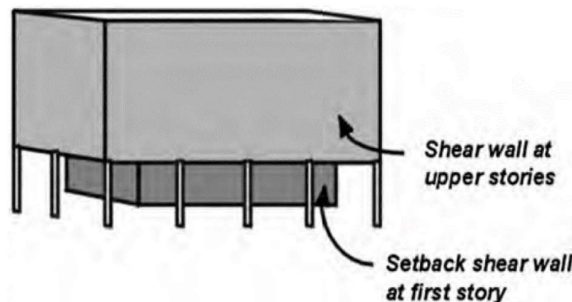


Figure 2.2 Out-of-plane discontinuous irregularities (ASCE 41-17)

In addition, the structural members should meet the defined Demand Capacity Ratios (DCRs) in order to apply linear elastic methods. In structures with one or more of the specified irregularities, linear elastic methods are not allowed if the defined DCRs of the structural members could not be met. Therefore, it is necessary to determine the irregularities in the structure and to determine the DCR values of the structural members in order to decide the applicability of linear elastic methods.

First, a numerical model of the existing building should be created and subjected to linear elastic analysis under the applied earthquake forces in order to determine DCR values. The highest value is determined as the member DCR value. According to the ASCE 41-17 Standard, the DCR value of any structural member is found by the ratio of the force acting on the member (Q_{UD}) to the calculated capacity of the member as a result of the analysis of the structure (Q_{CE}), as shown in Equation 2.1.

$$DCR = \frac{Q_{UD}}{Q_{CE}} \quad (2.1)$$

The conditions required for the application of linear elastic methods are listed below. Linear elastic methods are not allowed to be applied in cases where any of the mentioned conditions could not be met. In cases where the irregularities in the structure are below the defined limits, but the DCR values do not meet the defined limits, the use of linear elastic methods is not allowed.

- Soft storey and torsional stiffness irregularities are below the limits defined in Section 7.3.1 of ASCE 41-17.
- In cases where the soft storey or torsional stiffness irregularities in the structure are met, the calculated DCR values of the structural members are less than 3 and the limiting value of the member demand modification factor (m-factor) specified in ASCE 41-17 Sections 8 to 12.

- Absence of in-plane and out-of-plane discontinuity irregularities in the structure as explained in Section 7.3.1.1 of ASCE 41-17.

The linear elastic condition assessment methods considered in the Standard are also divided into sub sections. Linear elastic analysis are classified under two main headings as static method and dynamic method. For the linear elastic static method, the fundamental period of the structure has to be determined first. Then, equivalent static horizontal loads are calculated according to the determined period and these loads are applied to the structure. Another calculation method which is discussed under linear elastic analysis is linear elastic dynamic method. Mode superposition or dynamic analysis methods are employed as dynamic methods.

Moreover, some limitations are defined for the applicability of linear elastic static and dynamic analysis methods. According to these limitations, the application of linear elastic static analysis methods is limited in structures with long periods and some irregularities. Long period structures are defined according to Equation 2.2 below. In this equation, parameter T_s is the characteristic period of the response spectrum and it is the period value in which the spectrum shifts from the constant acceleration region to the constant velocity region.

$$T \geq 3.5 * T_s \quad (2.2)$$

Performance targets of existing structures are determined by selecting parameters such as seismic hazard level and structural performance level. Performance targets for existing structures vary according to risk categories and they are shown in Table 2.1 below as indicated in Section 2.2 in ASCE 41-17.

Table 2.1 Performance targets for existing structure (ASCE 41-17)

Table 2-1. Basic Performance Objective for Existing Buildings (BPOE)

Risk Category	BSE-1E	BSE-2E
I and II	Life Safety Structural Performance	Collapse Prevention Structural Performance
	Life Safety Nonstructural Performance (3-C)	Hazards Reduced Nonstructural Performance ^a (5-D)
III	Damage Control Structural Performance	Limited Safety Structural Performance
	Position Retention Nonstructural Performance (2-B)	Hazards Reduced Nonstructural Performance ^a (4-D)
IV	Immediate Occupancy Structural Performance	Life Safety Structural Performance
	Position Retention Nonstructural Performance (1-B)	Hazards Reduced Nonstructural Performance ^a (3-D)

^a Compliance with ASCE 7 provisions for new construction is deemed to comply.

The seismic performance of existing structures is related to the expected structural damage of the building under the effect of the applied earthquake loads. It is defined on the basis of three different damage states. Performance levels for these damage states are given in Section 2.3 as “Immediate Occupancy Performance Level (S-1)”, “Damage Control Performance Level (S-2)”, “Life Safety Performance Level (S-3)”, “Limited Safety Performance Level (S-4)” and “Collapse Prevention Performance Level (S-5)”. S-2 and S-4 performance levels are specified as intermediate performance levels. “Immediate Occupancy Performance Level” (S-1) is defined for buildings that should remain safely after an earthquake and maintain their pre-earthquake condition. “Life Safety Performance Level” (S-3) is defined for structures in which structural members are damaged after an earthquake but there is no partial or complete collapse. “Collapse Prevention Performance Level” (S-5) is defined for structures in which structural members are severely damaged after an

earthquake and seismic resistance against the possibility of collapse is lost. Non-structural performance targets are also grouped under five main headings. These are defined as “Operational” (N-A), “Position Retention” (N-B), “Life Safety” (N-C), “Hazard Reduced” (N-D), and “Not Considered” (N-E). The damage control and structural performance levels defined in the ASCE 41-17 (2017) Standard are given in detail in Table 2.2.

Table 2.2 Damage control and performance levels (ASCE 41-17)

Table C2-3. Damage Control and Building Performance Levels

Target Building Performance Levels				
	Collapse Prevention Level (5-D)	Life Safety Level (3-C)	Immediate Occupancy Level (1-B)	Operational Level (1-A)
Overall damage	Severe	Moderate	Light	Very light
Structural components	Little residual stiffness and strength to resist lateral loads, but gravity load-bearing columns and walls function. Large permanent drifts. Some exits blocked. Building is near collapse in aftershocks and should not continue to be occupied.	Some residual strength and stiffness left in all stories. Gravity-load-bearing elements function. No out-of-plane failure of walls. Some permanent drift. Damage to partitions. Continued occupancy might not be likely before repair. Building might not be economical to repair.	No permanent drift. Structure substantially retains original strength and stiffness. Continued occupancy likely.	No permanent drift. Structure substantially retains original strength and stiffness. Minor cracking of facades, partitions, and ceilings as well as structural elements. All systems important to normal operation are functional. Continued occupancy and use highly likely.
Nonstructural components	Extensive damage. Infills and unbraced parapets failed or at incipient failure.	Falling hazards, such as parapets, mitigated, but many architectural, mechanical, and electrical systems are damaged.	Equipment and contents are generally secure but might not operate due to mechanical failure or lack of utilities. Some cracking of facades, partitions, and ceilings as well as structural elements. Elevators can be restarted. Fire protection operable.	Negligible damage occurs. GPower and other utilities are available, possibly from standby sources.
Comparison with performance intended for typical buildings designed to codes or standards for new buildings, for the design earthquake	Significantly more damage and greater life-safety risk.	Somewhat more damage and slightly higher life-safety risk.	Less damage and low life-safety risk.	Much less damage and very low life-safety risk.

The scope of knowledge which is collected from structures for performance assesment is specified in Section 6.2.4 of the ASCE 41-17 Standard. Knowledge factors which are used to determine the capacities of structural members of existing structures and to evaluate their earthquake resistance are given in Table 2.3. The knowledge factor (κ) varies depending on factors such as the selected performance target and the analysis method. The knowledge factors specified in Table 2.3 are used in the capacity calculations of the members.

Table 2.3 Knowledge levels (ASCE 41-17)

Data	Level of Knowledge					
	Minimum		Usual		Comprehensive	
Performance Level	Life Safety (S-3) or lower		Damage Control (S-2) or lower		Immediate Occupancy (S-1) or lower	
Analysis Procedures	LSP, LDP		All		All	
Testing	No tests ^a		Usual testing		Comprehensive testing	
Drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings	Design drawings	Field survey drawings prepared in absence of design drawings
Condition Assessment ^b	Visual	Comprehensive	Visual	Comprehensive	Visual	Comprehensive
Material Properties	From design drawings (or documents) ^c	From default values	From design drawings (or documents) and tests	From usual tests	From design drawings (or documents) and tests	From comprehensive tests
Knowledge Factor (κ) ^d	0.9 ^{e,f}	0.75	1.00	1.00	1.00	1.00

Note: LSP, linear static procedure; LDP, linear dynamic procedure.

^a Except where no default material properties are provided in this standard.

^b In the absence of sufficient component detailing information from the design drawings, missing information shall be supplemented by a comprehensive condition assessment.

^c Where material properties are missing from the design drawings (or documents), default values may be used in conjunction with $\kappa = 0.75$.

^d $\kappa = 0.75$.

^e Refer to Chapters 9 through 12 for additional material-specific requirements and limitations.

^f If the building meets the benchmark requirements of Table 3-2, then $\kappa = 1.0$.

^g If inspection or testing records are available to substantiate the design drawings, then $\kappa = 1.0$.

Performance acceptance criteria for linear elastic methods are given in detail in Section 7.5.2.2 of ASCE 41-17. These acceptance criteria are examined considering the predefined performance levels. Members effects to be checked for acceptance criteria are classified according to two limit states. These are defined as force controlled effects and deformation controlled effects. Force controlled effects are associated with limited ductile behavior, and deformation-controlled effects are associated with ductile behavior. For example, for a beam member, the bending moment represents the deformation controlled effect, while the shear force represents the force controlled effect. Details on this classification are given in the ASCE 41-17 Standard. The demands are obtained under load combinations defined by the type of effect on the structural members (Q_{UD} for strain-controlled effects, Q_{UF} for force-controlled effects). For strain-controlled effects, the existing strength is compared with expected strength which is multiplied with the knowledge factor (κ) and the m -factor. The m -factor is a capacity modification factor used to account for the expected ductile behavior. This factor is determined by the selected performance level. The m -factor is presented in tabular form between Chapters 8 and

12 of the ASCE 41-17 standard for each performance level. As an example, the m -factors for steel members for different earthquake levels are shown in Table 2.4.

Table 2.4 m -factors for structural steel members

Component/Action	<i>m</i> -Factors for Linear Procedures ^a				
	IO	Primary		Secondary	
		LS	CP	LS	CP
Beams—Flexure					
a. $\frac{b_f}{2t_f} \leq 0.30 \sqrt{\frac{E}{F_{ye}}}$ and $\frac{h}{t_w} \leq 2.45 \sqrt{\frac{E}{F_{ye}}}$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \geq 0.38 \sqrt{\frac{E}{F_{ye}}}$ or $\frac{h}{t_w} \geq 3.76 \sqrt{\frac{E}{F_{ye}}}$	1.25	2	3	3	4
c. Other	Linear interpolation between the values on lines <i>a</i> and <i>b</i> for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.				
Columns in Compression or Tension—Flexure^{b,c}					
For $ P_{UF} /P_{ye} < 0.2$					
a. $\frac{b_f}{2t_f} \leq 0.30 \sqrt{\frac{E}{F_{ye}}}$ and $\frac{h}{t_w} \leq 2.45 \sqrt{\frac{E}{F_{ye}}} \left(1 - 0.71 \frac{ P_{UF} }{P_{ye}}\right)$	2	6	8	10	12
b. $\frac{b_f}{2t_f} \geq 0.38 \sqrt{\frac{E}{F_{ye}}}$ or $\frac{h}{t_w} \geq 3.76 \sqrt{\frac{E}{F_{ye}}} \left(1 - 1.83 \frac{ P_{UF} }{P_{ye}}\right)$	1.25	1.25	2	2	3
c. Other	Linear interpolation between the values on lines <i>a</i> and <i>b</i> for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.				
For $ P_{UF} /P_{ye} \geq 0.2$					
a. $\frac{b_f}{2t_f} \leq 0.30 \sqrt{\frac{E}{F_{ye}}}$ and $\frac{h}{t_w} \leq 0.77 \sqrt{\frac{E}{F_{ye}}} \left(2.93 - \frac{ P_{UF} }{P_{ye}}\right) \leq 1.49 \sqrt{\frac{E}{F_{ye}}}$	$1.5 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$7.5 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$10.5 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$13.5 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$16.5 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$
b. $\frac{b_f}{2t_f} \geq 0.38 \sqrt{\frac{E}{F_{ye}}}$ or $\frac{h}{t_w} \geq 1.12 \sqrt{\frac{E}{F_{ye}}} \left(2.33 - \frac{ P_{UF} }{P_{ye}}\right) \geq 1.49 \sqrt{\frac{E}{F_{ye}}}$	$0.375 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$0.375 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$1.5 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$1.5 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$	$3 \left(1 - \frac{5}{3} \frac{ P_{UF} }{P_{ye}}\right) + 1 > 1$
c. Other	Linear interpolation between the values on lines <i>a</i> and <i>b</i> for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.				

Deformation controlled effects (Q_{UD}) are calculated according to Equation 2.3 as shown below. In this equation, parameter Q_G shows the effect resulting from gravitational forces while Q_E indicates the effect resulting from earthquake forces.

$$Q_{UD} = Q_G + Q_E \quad (2.3)$$

In order to ensure that the performance target has been met, the available strength should be more than the calculated demand for strain-controlled effects. This comparison is shown in Equation 2.4 below as indicated in the regulation.

$$m * k * Q_{CE} > Q_{UD} \quad (2.4)$$

Force controlled effects (Q_{UF}) are calculated according to Equation 2.5 as shown below. The X coefficient in the equation is determined according to the target performance level. The J value is taken as the smallest DCR value. Alternatively, the J value can be taken as 2 for high seismic level, 1.5 for medium seismic level, and 1 for low seismic level. $C1$ and $C2$ values also vary depending on the ground conditions, the effective period of the structure and the ratio of elastic strength demand to yield strength coefficient ($\mu_{strength}$).

$$Q_{UF} = Q_G \pm \frac{X * Q_E}{C1C2J} \quad (2.5)$$

For acceptance condition, the existing strength has to be more than the calculated demand for force controlled effects. This comparison is shown in Equation 2.6 below as indicated in the regulation.

$$k * Q_{CL} > Q_{UF} \quad (2.6)$$

The members of the structure are checked in accordance with the above mentioned inequalities using the m -factors given for each performance level in tables between Chapters 8 and 12 of ASCE 41-17. In Chapter 10, the m -factors to be used in the linear elastic analysis method for the elements of RC structures are given in tables. By using these m -factors, the existing strengths are controlled. If these conditions are met, it is concluded that performance assessment of the existing structures analyzed with linear elastic analysis methods satisfies the performance target under the given seismic hazard level.

2.3 Turkish Building Earthquake Code (TBEC-2018)

The linear elastic method which is used for the evaluation of existing buildings, included in the Turkish Building Earthquake Code (TBEC-2018), is a unique procedure that has been specifically developed for TBEC. This method is based on chord rotations and section curvatures, which are internal deformation demand parameters of the structural members in the building. First, a numerical model is created, and then this model is subjected to lateral force analysis for the application of the method. The validity of the method is checked by the DCRs of the members and several limitations in the structure as well. By using the results obtained from this analysis, plastic rotation demands can be calculated and the performance of the structure can be evaluated on the basis of deformation.

In Chapter 15 of TBEC (2018), the principles regarding the evaluation of existing buildings are given in detail. The calculation rules to be applied in the evaluation of the performance of existing structures, strengthened buildings and building-type structures under the effect of earthquake, the principles to be taken as basis in the retrofitting decisions and the principles of the retrofitting design of the buildings that are decided to be strengthened are discussed in Chapter 15. Information to be collected, knowledge level coefficients, performance targets and earthquake hazard levels are also defined in this section.

The level of knowledge to be collected from the structures is specified in Section 15.2.1 of TBEC (2018). Accordingly, the details and dimensions of the members to be used in the determination of the capacities of the structural system elements of the existing buildings and in the evaluation of their earthquake resistance, as well as the information on the structural system geometry and material properties can be obtained from the projects, reports of the buildings, the observations and measurements to be made in the building. The knowledge levels for each building type and accordingly the knowledge level coefficients are specified in Section 15.2.12. These coefficients are shown in Table 2.5. Knowledge levels are classified

as limited and comprehensive, respectively. Obtained knowledge levels are used to calculate the member capacities.

Table 2.5 Knowledge level coefficients for structures (TBEC)

Knowledge Level	Knowledge Coefficient
Limited	0.75
Comprehensive	1

The information regarding the earthquake ground motions to be considered in the design of buildings under earthquake effects given in Section 2.1 of TBEC (2018). Within the scope of this Code, four different seismic intensity levels are defined from DD-1 to DD-4. DD-1 level describes a very rare earthquake ground motion with a 2% exceedance probability in 50 years and a corresponding recurrence period of 2475 years. This earthquake ground motion is also called the largest earthquake ground motion considered. DD-2 level characterizes the design-basis earthquake ground motion in which the probability of exceedance in 50 years is 10% and the corresponding recurrence period is 475 years. DD-3 level represents frequent earthquake ground motions for which the probability of exceedance in 50 years is 50% and the corresponding recurrence period is 72 years. Finally, DD-4 level describes very frequent earthquake ground motions where the probability of exceedance in 50 years is 68% (50% in 30 years) and the corresponding recurrence period is 43 years.

Building Occupancy Classes (BKS) are defined in Table 2.6 depending on the intended use of the buildings, as a basis for determining the Earthquake Design Classes defined in Section 3.2 of TBEC (2018). Building importance factors (I) are defined according to the building occupancy classes.

Table 2.6 Building occupancy classes and building importance coefficients
(TBEC–2018)

Building Use Class	Building Usage Purpose	Building Importance Factor
BKS = 1	Buildings that need to be used after an earthquake, buildings where people stay for a long time and intensely, buildings where valuable goods are stored and buildings containing dangerous substances a) Buildings that must be used immediately after an earthquake (Hospitals, dispensaries, health centers, fire brigade buildings and facilities, PTT and other communication facilities, transportation stations and terminals, energy generation and distribution facilities, province, district and municipality administration buildings, first aid and disaster planning stations)) b) Schools, other educational buildings and facilities, dormitories and dormitories, military barracks, prisons, etc. c) museums d) Toxic, explosive, flammable, etc. with features buildings where substances are located or stored	1.5
BKS = 2	Shopping malls, sports facilities, cinema, theatre, concert halls, places of worship, etc.	1.2
BKS = 3	Other buildings that are not included in the definitions given for BKS=1 and BKS=2 (Houses, workplaces, hotels, building type industrial structures, etc.)	1.0

Earthquake Design Classes (DTS), which are taken as basis in design under seismic action are determined according to Table 2.7 depending on the Building Use Classes and the short period design spectral acceleration coefficient (S_{DS}) defined in Section 2.3.2.2 for the DD-2 earthquake ground motion level.

Table 2.7 Earthquake design classes (DTS) (TBEC – 2018)

Acceleration Design Spectrum Factor (S_{DS}) for DD-2 Earthquake level and short period	Building Usage Classification	
	BKS = 1	BKS = 2, 3
$S_{DS} < 0.33$	DTS 4a	DTS = 4
$0.33 \leq S_{DS} < 0.5$	DTS = 3a	DTS = 3
$0.5 \leq S_{DS} < 0.75$	DTS = 2a	DTS = 2
$0.75 \leq S_{DS}$	DTS = 1a	DTS = 1

In TBEC (2018), buildings are classified in 8 Building Height Classes (BYS) in terms of total building height. The Building Height Classes are given in Table 2.8.

Table 2.8 Building height classes

Building Height Class	Building Allowable Height According to Earthquake Design Classes and Building Height Category (m)		
	DTS = 1, 1a, 2, 2a	DTS = 3, 3a	DTS = 4, 4a
BYS = 1	$H_N > 70$	$H_N > 91$	$H_N > 105$
BYS = 2	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
BYS = 3	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
BYS = 4	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
BYS = 5	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
BYS = 6	$10.5 < H_N \leq 17.5$	$17.5 < H_N \leq 28$	
BYS = 7	$7 < H_N \leq 10.5$	$10.5 < H_N \leq 17.5$	
BYS = 8	$H_N \leq 7$	$H_N \leq 10.5$	

According to TBEC (2018), seismic performance of existing buildings is related to the expected damage seismic action and it is defined on the basis of four different damage states. These performance levels are given as the “Immediate Occupancy Performance Level”, “Limited Damage Performance Level”, “Controlled Damage Performance Level”, and “Collapse Prevention Performance Level”. Immediate Occupancy (QC) Performance Level corresponds to the situation in which structural damage does not occur in the structural elements of the building or the damage remains negligible. The Limited Damage (LD) Performance Level corresponds to the damage level at which limited damage occurs in the structural elements of the building, in other words, the non-linear behavior is limited. Controlled Damage (CD) Performance Level corresponds to the damage level that is not too severe and extensive and mostly repairable in the building structural system elements to ensure life safety. The Prevention of Collapse (CP) Performance Level corresponds to the pre-failure situation where severe damage to the structural elements of the building occurs. Partial or complete collapse of the building should be prevented. For the given seismic intensity levels, nominal performance targets defined for earthquake

design classes DTS =1, 2, 3, 3a, 4, 4a and advanced performance targets defined for earthquake design classes DTS =1a, 2a are given in Table 2.9.

Table 2.9 Performance targets and evaluation/design approaches for new or existing buildings according to earthquake design classes (TBEC – 2018)

(a) New Reinforced Concrete, Prefabricated Reinforced Concrete and Steel Buildings (Except High- Buildings $BYS \geq 2$)

Earthquake Level	(1) DTS = 1.1a , 2 2a , 3, 3a, 4, 4a		(2) (2) DTS = 1a , 2a	
	Normal Performance Target	Evaluation/Design Approach	Advanced Performance Target	Evaluation/Design Approach
DD-3	—	—	Limited Damage	Deformation-Based Design
DD-2	Controlled Damage	Force-Based Design	Controlled Damage	Force-Based Design
DD-1	—	—	Controlled Damage	Deformation-Based Design

(b) New Constructed or Existing High-Rise Buildings ($BYS = 1$)

Earthquake Level	DTS = 1, 2, 3, 3a, 4, 4a		DTS = 1a, 2a	
	Normal Performance Target	Evaluation/Design Approach	Advanced Performance Target	Evaluation/Design Approach
DD-4	Immediate Occupancy	Force-Based Design	—	—
DD-3	—	—	Limited Damage	Deformation-Based Design
DD-2	Controlled Damage	Force-Based Design	Controlled Damage	Force-Based Design
DD-1	Prevention of Collapse	Deformation-Based Design	Controlled Damage	Deformation-Based Design

(c) Existing Reinforced Concrete, Prefabricated Reinforced Concrete and Steel Buildings (Except Tall Buildings – $BYS \geq 2$)

Earthquake Level	DTS = 1, 2, 3, 3a, 4, 4a		DTS = 1a, 2a	
	Normal Performance Target	Evaluation/Design Approach	Advanced Performance Target	Evaluation/Design Approach
DD-3	—	—	Limited Damage	Deformation-Based Design
DD-2	Controlled Damage	Deformation-Based Design	—	—
DD-1	—	—	Controlled Damage	Deformation-Based Design

Analysis methods are specified following the performance levels. Analysis methods are divided into linear elastic methods and nonlinear methods. The application limits for linear elastic methods are defined in Section 15.5.3.1 of TBEC (2018). These limitations restrain the linear elastic calculation method to be used in regular and mid-rise buildings. DCR values, which has significant role in the American

Standards (i.e. ASCE 41-17) as shown before, are considered as an important parameter also in TBEC. According to Chapter 15.5.3 in TBEC, application limits of linear calculation methods are given as follows:

- The building height class should be greater than 5 ($BYS > 5$).
- B3 irregularity defined in Section 3.6.2.4 should not be present in the structure. In the code, B3 irregularity is defined as the situations where the vertical elements (columns or shear walls) of the load-bearing system are removed on some floors and placed on the top or end of beams. In addition, placing the shear walls on the upper floor over the columns on the lower floors is also defined as B3 irregularity.
- For each floor of RC buildings, excluding the top floor, the average of the DCR values scaled by the shear force of the vertical ductile elements (columns, shear walls and reinforced partition walls) should be less than the DCR value of the beams for the each earthquake direction.
- The average of the DCR values scaled by the shear force of the vertical ductile elements on any floor of the reinforced concrete buildings, excluding the top floor, should be less than 3 for each earthquake direction.
- The average DCR value of ductile beams should be less than 5 for any floor of RC buildings, excluding the top floor in each earthquake direction.

According to TBEC (2018), linear calculation methods can not be applied in case of any of the above mentioned situations are not satisfied. These constraints are generally controlled to ensure that the columns are stronger than the beams (i.e. strong column weak beam principle) in the structural system that is examined, and no collapse conditions such as soft storeys occur. These are to ensure the capacity design principles in general. The DCR limits given for beams and vertical members are the upper limit values in terms of estimation performance of nonlinear dynamic analysis results of seismic behavior obtained by linear elastic methods. In a system where DCR values are calculated below the relevant limits, the evaluations made with linear elastic calculation methods can accurately represent the nonlinear

behavior. Therefore, the above criteria check is carried out before the calculation method is applied.

After checking the applicability limits, plastic rotation demands are obtained by using the total chord rotation (θ_k) obtained from the linear elastic analysis. The deformed shape of a typical member under double curvature bending is shown in Figure 2.3. In this figure, l shows the total member length, l_c shows the net span, Δ shows the inter-floor translation, θ_i and θ_j indicates the rotations of i and j nodes, respectively. θ_{ki} and θ_{kj} shows the chord rotations at the i and j ends of member.

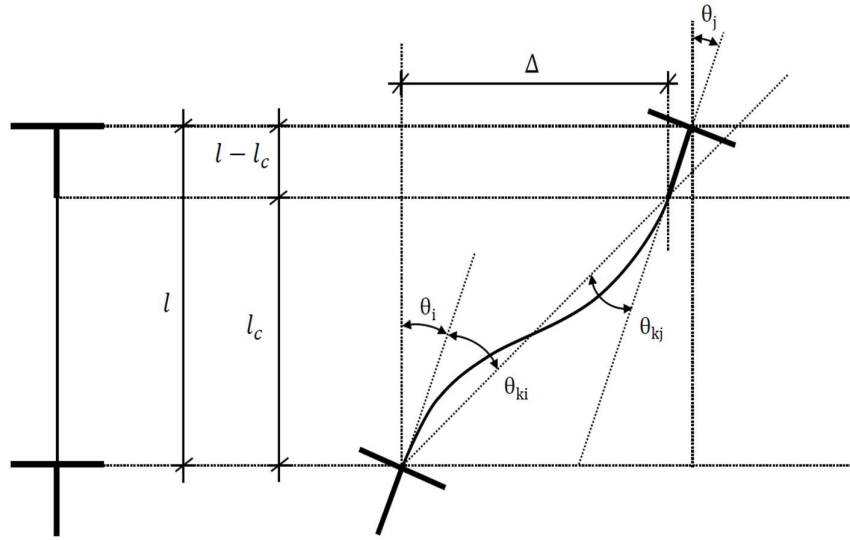


Figure 2.3 Chord rotations for a column member

The chord rotation (θ_k) is calculated as shown in Equation 2.7 with the parameters defined in Figure 2.3. In beam members, the interstory displacement value is generally zero ($\Delta = 0$).

$$\theta_{ki} = \frac{\Delta}{l_c} - \theta_i \quad (2.7)$$

Using the θ_k value calculated in Equation 2.7, the plastic rotation (θ_p) at the member end can be calculated by using Equation 2.8 shown below.

$$\theta_{ki} = \theta_{yi} + \theta_{pi} \quad (2.8)$$

The θ_y value in Equation 2.8 represents the yield rotation. The calculation of yield rotation is also explained in Section 15A.3 of TBEC (2018). The relationships between the end rotations at yield and end moments at the i and j ends of a bending member that has yielded at both ends are defined in Equations 2.9 and 2.10. In these equations, EI is the bending stiffness of the uncracked section. M_{yi} and M_{yj} represent the effective yield moments at the ends i and j , respectively. The directions of the yielding moments are positive in the counterclockwise direction and negative in the clockwise direction. Equations 2.9 and 2.10 define the yield rotations (θ_{yi} and θ_{yj}) at both ends of a bending member under the effect of double curvature when the yield moment strength is reached at both ends.

$$\theta_{yi} = \frac{M_{yi} * l_c}{3EI} * \left(1 - \frac{M_{yj}}{2M_{yi}}\right) \quad (2.9)$$

$$\theta_{yj} = \frac{M_{yj} * l_c}{3EI} * \left(1 - \frac{M_{yi}}{2M_{yj}}\right) \quad (2.10)$$

Similarly, for wall type members, the associated yield moment is calculated by Equation 2.11 since it behaves in a single curvature bending condition along the section. In Equation 2.11, l_s is the shear opening (the ratio of moment/shear force in

the section). It can be taken as approximately half the distance from the floor to the top of the shear wall.

$$\theta_y = \frac{My * l_s}{3EI} \quad (2.11)$$

After obtaining the plastic rotations (θ_p), which is an important response parameter, the structural performance is checked for the allowable limits in the design guidelines according to Section 5.8.1 of TBEC (2018). At this stage, the plastic rotation limit of each member is calculated in accordance with the performance target of the structure by using the curvature values obtained from the section analysis. Then, these plastic rotation limits are compared with the plastic rotation demands of the members which are obtained by using the linear elastic methods.

In order to be used in the performance evaluation for the Collapse Prevention Performance Level (*CP*), the allowable limit for plastic rotation is defined as shown in Equation 2.12 according to the plastic behavior model in RC building elements, the axial force acting on the section, and the curvature analysis. In this equation, φ_u and φ_y represents the ultimate and yield curvature. L_p and L_s are the plastic hinge length and shear opening respectively. The term d_b represents the diameter of the longitudinal steel reinforcement.

$$\theta_p^{(CP)} = \frac{2}{3} * [(\varphi_u - \varphi_y) * L_p * \left(1 - 0.5 * \frac{L_p}{L_s}\right) + 4.5 * \varphi_u * d_b] \quad (2.12)$$

The allowable plastic rotation (θ_p) limits for performance evaluation of the Controlled Damage Performance Level (*CD*) are defined in Equation 2.13 below, depending on the values for of Collapse Prevention performance level.

$$\theta_p^{(CD)} = 0.75 * \theta_p^{(CP)} \quad (2.13)$$

In order to be used in the performance evaluation for the Limited Damage Performance Level (*LD*), the formation of plastic hinges in the structural system in RC members is not allowed as shown in Equation 2.14.

$$\theta_p^{(LD)} = 0 \quad (2.14)$$

The structural members should be classified according to their ductile and brittle behavior in order to evaluate the seismic performance of the examined structure. Afterwards, the performance of the building is determined by comparing the internal deformation limits which are associated with the determined performance level with the calculated values. According to Figure 2.4, elements whose critical sections do not reach limited limit are in the Limited Damage State, elements whose critical sections are between limited damage and controlled damage limits are in the Significant Damage State elements whose critical sections are between controlled damage and collapse prevention limits are in the Advanced Damage State and, elements whose critical sections exceeds the collapse prevention limit of the sections are located in the Collapse State.

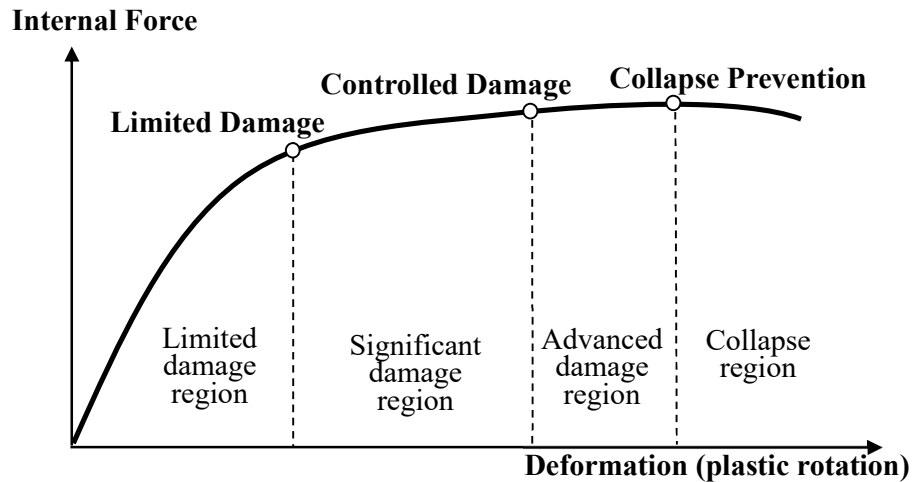


Figure 2.4 Damage states for sections (TBEC – 2018)

For Limited Damage performance level, at any floor of RC buildings, at most 20% of the beams can pass into the significant damage state as a result of the calculation made for each earthquake direction applied. However, all other members should be in the Limited Damage state. Buildings in this condition are considered to be in the *Limited Damage Performance Level*, provided that brittle damaged elements, if any, are strengthened.

For a Controlled Damage performance level, RC buildings must meet the specified conditions. As a result of the calculations made for each earthquake direction of RC buildings, at any floor, excluding the secondary beams (not included in the horizontal load bearing system), maximum 35% of the beams can pass into the advanced damage state. The total contribution of the vertical elements in the advanced damage state to the shear force carried by the vertical elements at each floor should be less than 20%. The ratio of the sum of the shear forces of the vertical elements in the advanced damage state at the top floor to the sum of the shear forces of all the vertical elements in that floor can be at most 40%. All other elements are in the limited damage or significant damage state. However, the ratio of the shear forces carried by the vertical elements in which the significant damage limit has been exceeded in both the lower and upper sections of members at any floor to the shear force carried by all the vertical elements on that floor should not exceed 30%. Buildings that meet the above mentioned conditions are considered to be at the *Controlled Damage Performance Level*, provided that the brittle damaged elements, if any, are strengthened.

RC buildings have to satisfy the specified conditions for the Collapse Prevention performance level. In any floor of RC buildings with the exception of the secondary (not included in the horizontal load bearing system) beams, a maximum of 20% of the beams can pass into the collapse state. All other elements should be in limited damage, significant damage, or advanced damage states. However, the ratio of the shear forces carried by the vertical elements in which the significant damage limit has been exceeded in both the lower and upper sections of members at any floor to the shear force carried by all the vertical elements on that floor should not exceed

30%. The use of the building in its current state is inconvenient in terms of life safety. Buildings that satisfy the above conditions are considered to be at the *Collapse Prevention Performance Level*, taking into account that all brittle damaged elements are in the collapse zone.

The building is in a Collapse state if it fails to meet the collapse prevention performance level conditions. In this case, it is stated that the use of the building is inconvenient in terms of life safety. According to TBEC (2018), the seismic performance of existing buildings is determined according to the four performance criteria mentioned above by considering the expected damage to the building under the effect of seismic forces.

2.4 Eurocode 8.3 (EN 1998-3, 2005)

Evaluation procedures of existing buildings, strengthening and repair of buildings against seismic effects are examined in Chapter 3 of the Eurocode-8 (EN 1998-3, 2005). The performance limit states are defined and the earthquake hazard levels related to the performance targets are specified. Performance limit states are examined and discussed in detail under three main headings as “Damage Limitation”, “Significant Damage” and “Near Collapse”. The Damage Limitation performance state is defined for structures that have only been slightly damaged but where the structural members do not lose their stiffness and strength properties. It has been stated that cracks can be seen in non-structural elements, but this type of damage can be easily repaired. “Significant Damage” performance state is defined for structures with severe damage, permanent displacements, and damage to non-structural elements. The “Near Collapse” performance situation is defined for structures that are heavily damaged, are at the onset of collapse, and most of the non-structural elements have collapsed.

The seismic hazard is defined by very rare seismic activity, with a 2% probability of exceeding the spectral magnitudes in 50 years for the “Near Collapse” performance

level and the corresponding recurrence period of 2475 years. For the "Significant Damage" performance level, the seismic hazard is defined by the probability of exceeding the spectral magnitudes in 50 years is 10% and the corresponding recurrence period is 475 years. For the "Damage Limitation" performance level, the seismic hazard is defined by the probability of exceeding the spectral magnitudes in 50 years is 20% and the corresponding recurrence period is 225 years.

Buildings are divided into four importance classes depending on the consequences of the collapse for human life, their importance in terms of public safety and protection in the immediate post-earthquake period, and the social and economic consequences of the collapse. These importance classes are defined by different importance factors (γ_I). If the severity factor (γ_I) is equal to one, it is associated with a seismic event with a reference return period of 475 years. Table 2.10 shows the importance classes and importance factors for buildings.

Table 2.10 Importance classes for buildings in Eurocode 8-3 (EN 1998-3, 2005)

Importance Class	Buildings
I	Buildings of minor importance for public safety, e.g. Agricultural buildings etc.
II	Ordinary buildings, not belonging in the other categories
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. Schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. Hospitals, power plants etc.

Building importance coefficient is taken as 0.8, 1.0, 1.2 and 1.4 for buildings belonging to I, II, III, IV building importance classes, respectively. The building importance coefficient is determined depending on the occupancy class of the

building. It is necessary to consider a more severe earthquake with a long recurrence period for structures that must be used immediately after the earthquake. For this purpose, earthquake load is increased with the building importance factor and earthquakes with a larger recurrence period and more severe earthquakes are taken into account. For the seismic analysis of typical RC structures, earthquake ground motion is used for the "Significant Damage" performance level, for which the probability of exceedance in 50 years is 10% and the corresponding recurrence period is 475 years. According to Table 2.10, this type of RC structures are in the "II" building importance class and the building importance coefficient is chosen as 1.0.

The eligibility criteria of the investigated structures are discussed in Section 2.2 of Eurocode 8-3. As an important point, structural members should be classified as ductile and brittle because their behavior during an earthquake have a important effect on the performance of the structure. Ductile members have the ability to undergo significant deformation before they fail. This means that they can absorb and dissipate energy during an earthquake providing the structure to remain functional and allow occupants with time to evacuate safely. Ductile members are typically designed to undergo large deformations without catastrophic failure. On the other hand, brittle members do not exhibit significant deformation before they fail. They tend to fail suddenly without warning, which can lead to the rapid collapse of a structure during an earthquake. Therefore, the classification of structural members as ductile or brittle is crucial for seismic assessment because it directly impacts the safety and performance of buildings during earthquakes. First of all, it should be verified that the deformation demands calculated according to the performance targets of the structural members do not exceed their capacities. Then, taking into account the strength of the structural elements, their demands and capacities should be compared and verified. Eligibility criteria are determined according to each performance target and are examined in detail. First, input data and knowledge levels are discussed. The knowledge levels that can be used in the evaluation of existing buildings are given in Section 7.3 of Eurocode 8-3. These

knowledge levels are divided into three groups as KL1, KL2 and KL3. Knowledge levels are examined in terms of geometry, detailing and materials. The geometry includes the general geometry of the structure and the member section dimensions. Detailing includes reinforcement quantity and arrangement. The material includes the material properties. In Table 2.11, knowledge levels are discussed comparatively.

Table 2.11 Levels of knowledge defined for existing structures in Eurocode 8-3

Knowledge Level	Geometry	Details	Materials
KL1	From original outline construction drawings with sample visual survey or from full survey	Simulated design in accordance with relevant practice and from limited <i>in-situ</i> inspection	Default values in accordance with standards of the time of construction and from limited <i>in-situ</i> testing
KL2		From incomplete original detailed construction drawings with limited <i>in-situ</i> inspection or from extended <i>in-situ</i> inspection	From original design specifications with limited <i>in-situ</i> testing or from extended <i>in-situ</i> testing
KL3		From original detailed construction drawings with limited <i>in-situ</i> inspection or from comprehensive <i>in-situ</i> inspection	From original test reports with limited <i>in-situ</i> testing or from comprehensive <i>in-situ</i> testing

The corresponding geometry, detailing, and material uncertainties for the various levels of knowledge need to be properly considered to make a structural assessment. In the Eurocode 8-3 (EN 1998-3, 2005), a Confidence Factor (CF) has been created in order to collect all kinds of uncertainty in a single factor and apply it only to the mechanical properties of materials. The recommended CF values are 1.35, 1.20 and 1.0 for KL1, KL2 and KL3, respectively. The relationship between the knowledge levels specified in the regulation and the applicable analysis methods and the confidence factors related to them are shown in Table 2.12.

Table 2.12 Knowledge levels and corresponding methods of analysis (LF: Lateral Force Method, MRS: Modal Response Spectrum analysis) and values for Confidence Factor (CF)

Knowledge Level	Geometry	Details	Materials	Analysis	CF
KL1	From original outline construction drawings with sample visual survey or from full survey	Simulated design in accordance with relevant practice and from limited in-situ inspection	Default values in accordance with standards of the time of construction and from limited in-situ testing	LF-MRS	CF _{KL1}
KL2		From incomplete original detailed construction drawings with limited in-situ inspection or from extended in-situ inspection	From original design specifications with limited in-situ testing or from extended in-situ testing	All	CF _{KL2}
KL3		From original detailed construction drawings with limited in-situ inspection or from comprehensive in-situ inspection	From original test reports with limited in-situ testing or from comprehensive in-situ testing	All	CF _{KL3}
NOTE: The values ascribed to the confidence factors to be used in a country may be found in its National Annex. The recommended values are CF _{KL1} =1.35, CF _{KL2} =1.20 and CF _{KL3} =1.00					

CF values defined for each knowledge level are used in capacity calculations of structural members. The capacities of the members are calculated by dividing the values of the material properties which are obtained from the available information and on-site tests by these safety factors. It is also used as a multiplier of the mechanical properties of ductile members to determine actions affecting brittle elements or mechanisms. The verification approaches for ductile and brittle mechanisms and the use of CFs are shown in Table 2.13.

Table 2.13 Validation approaches for ductile and brittle mechanisms (Eurocode 8-3)

		Linear Model (LM)		Non-linear Model	
		Demand	Capacity	Demand	Capacity
Type of element or mechanism (e/m)	Ductile	Acceptability of Linear Model (for checking of $\rho_i = D_i/C_i$ values)		From analysis. Use mean values of properties in model.	In terms of deformation. Use mean values of properties divided by CF.
		From analysis. Use mean values of properties in model.	In terms of strength. Use mean values of properties		
		Verifications (if LM accepted)			
		From analysis.	In terms of deformation. Use mean values of properties divided by CF.		
	Brittle	Verifications (if LM accepted)		From analysis. Use mean values of properties in model.	In terms of strength. Use mean values of properties divided by CF and by partial factor.
		If $\rho_i \leq 1$: from analysis.	In terms of strength. Use mean values of properties divided by CF and by partial factor.		
		If $\rho_i > 1$: from equilibrium with strength of ductile e/m. Use mean values of properties multiplied by CF.			

After the input data and knowledge levels are defined, the allowed analysis methods are presented in Eurocode 8-3. These methods are given under two main headings as linear analysis method and nonlinear analysis method. In the context of linear analysis methods, lateral force analysis and modal analysis methods are presented. Static pushover analysis and dynamic analysis methods are given in the context of nonlinear methods.

Eurocode 8-3 defines limitations for the lateral force analysis and modal analysis methods. The limitations related to the applicability of these methods are associated with the DCR values in terms of bending moments of the structural members. In order to apply linear elastic methods, the ratio between the highest and lowest DCR values calculated for the overall structure should be between 2 and 3 in addition to the limitations given in EN 1998-1:2004 Section 4.3.3.2.2. A recommendation is given for the DCR cutoff value and a value of 2.5 is generally recommended. One of the limitations in the application of linear analysis methods is the structural irregularity. According to Eurocode 8-3, linear elastic methods are allowed to be

applied where the structure is geometrically regular and the period of the structure is less than or equal to 2 seconds. The geometrical irregularities of the structure are explained in Section 4.2.3 of Eurocode 8-3. In order to classify a building as regular in plan, the necessary conditions should be satisfied. The design should ensure an approximately symmetrical layout in the plan, considering both lateral stiffness and the distribution of mass along the two perpendicular axes.

The planar stiffness of the stories must be sufficiently large compared to the lateral stiffness of the vertical structural members. Thus, the deformation of the slabs has little effect on the distribution of forces on the vertical structural members. In plan, the slenderness of the building is calculated by the ratio of the $L_{\max}$ value to the $L_{\min}$ value and it should not be greater than 4, where $L_{\max}$ and $L_{\min}$ represent the largest and smallest plan dimensions of the building measured in the vertical directions, respectively. In order to classify a structure as regular in cross-section, the required conditions should be satisfied. All lateral load bearing systems should have no irregularities from the foundation to the top story of the building. Both the lateral stiffness and mass of the individual stories should remain constant from the foundation to the top of the building or decrease gradually without sudden changes. In frame type buildings, the ratio of the actual strength of the floor to the required strength according to the analysis should not vary disproportionately between adjacent stories.

In the verification method, a distinction is made between "ductile" and "brittle" structural members. It is defined in two ways for primary and secondary seismic elements. Ductile elements are defined as columns, beams and walls under bending without axial force, brittle elements are defined as beams, columns, walls and joints under shear force. Ductile elements are checked for deformation, brittle elements are checked for forces. For ductile elements, the demand to be controlled is the chord rotation at the member ends, obtained from linear or nonlinear analysis. For brittle mechanisms, the demand to be controlled is the force acting on the mechanism. For linear analysis methods, this force is the value transmitted by the ductile elements through equilibrium conditions. The calculation principles for RC structures related

to the evaluation and strengthening of existing structures are specified in Eurocode 8.3 Information Annex A.

Limit values for structural members are specified in Section A.3.2. Accordingly, the rotational limit defined for the Near-Collapse Limit State (NC) of beams, columns and walls (ductile members) operating only under bending without shear is shown in Equation 2.15. In this equation, θ_u represents the maximum curvature of the section, θ_y represents the curvature of the section corresponding to the yield capacity, L_{pl} represents the length of the plastic hinge, L_v represents the ratio of moment to shear. The parameter γ_{el} takes different values in primary seismic elements and secondary seismic elements. Some of the existing structural elements may be referred to as "secondary seismic elements" according to the definitions in EN 1998-1: 2004, 4.2.2 (1)P, (2) and (3). "Secondary seismic elements" and "primary seismic elements" are verified by the same eligibility criteria. However, less conservative estimates of their capacity are used. Therefore, the value of the parameter γ_{el} is taken as 1.5 for primary seismic elements and as 1 for secondary seismic elements.

$$\theta_{um=NC} = \frac{1}{\gamma_{el}} * (\theta_y + (\varphi_u - \varphi_y)) * L_{pl} * (1 - 0.5 * \frac{L_{pl}}{L_v}) \quad (2.15)$$

The rotation limit defined for the Significant Damage Performance Level is shown in Equation 2.16.

$$\theta_{SD} = 0.75 * \theta_{um} \quad (2.16)$$

The chord rotation calculation corresponding to the yield limit is specified in Equation 2.17 for beams and columns. Also, for rectangular walls, T-joints or heavy building sections, the rotational strain capacity calculation is shown in Equation 2.18.

$$\theta_y = \varphi_y * \left(\frac{L_v + a_v * Z}{3} \right) + 0.0014 * \left(1 + 1.5 * \frac{h}{L_v} \right) + \varphi_y * \left(\frac{d_b f_y}{8 \sqrt{f_c}} \right) \quad (2.17)$$

$$\theta_y = \varphi_y * \left(\frac{L_v + a_v * Z}{3} \right) + 0.0013 + \varphi_y * \left(\frac{d_b f_y}{8 \sqrt{f_c}} \right) \quad (2.18)$$

In the seismic performance assessment, the internal forces and deformations that are obtained under the specified seismic hazard level are compared with the predefined limit values. Regarding the comparison of the results, the seismic performance of the building is determined.

2.5 Comparison of The Procedures

Within the scope of the study, linear elastic calculation methods that can be applied to determine the performance level of existing buildings under earthquake forces are examined. ASCE 41-17 (2017), Eurocode 8-3 (EN 1998-3, 2005) and TBEC (2018) regulations, which are widely used in the world are investigated. In all three regulations, the calculation rules to be applied in the evaluation of the performance of all existing buildings, strengthened buildings and building-type structures under the effect of earthquake loads are discussed. The principles to be taken as basis in the retrofitting decisions and the principles of the retrofitting design of the buildings that are decided to be strengthened are included. Information to be collected, knowledge level coefficients, performance targets and seismic hazard levels are defined. In accordance with the performance-based earthquake engineering approach, the seismic hazard levels at which the existing structures will be evaluated are determined and explained in detail in all of the documents examined. Performance targets of existing buildings are determined by selecting parameters such as seismic hazard level and structural performance level.

Limits are defined according to the expected seismic performance of existing structures. Linear elastic method which is given in the TBEC (2018) is based on chord rotations and section curvatures of structural members. By using the relations obtained from the horizontal force analysis with a linear elastic numerical model, the plastic rotational demands can be calculated and the performance of the structure can be evaluated on the basis of deformation. In Eurocode 8-3, limit states are determined according to the deformation capacities of the load-bearing elements of the structure, taking into account the cross-sectional characteristics. In ASCE 41-17, the seismic performance of existing buildings is related to the expected damage to the building under seismic action and it is defined on the basis of three different damage states. Acceptance criteria are examined considering the determined performance levels. In ASCE 41-17, the strength and deformation capacities of elements are obtained using modeling parameters. These parameters are calculated according to the formulas given in the regulation. The capacity modification factor (m -factor) which is used to account for ductile behavior is determined by the performance level chosen and this factor is tabulated in Chapters 8 through 12 of ASCE 41-17 for each performance level. By using the m -factors and knowledge level coefficients (κ) given for linear elastic analysis methods, the existing strengths are checked and the existing structure condition assessment is evaluated accordingly.

When the three regulations studied within the scope of the work are compared with each other, it is observed that there are similarities in the performance targets. In ASCE 41-17, three performance targets are specified as Immediate Occupancy Performance Level, Life Safety Performance Level, and Collapse Prevention Performance Level. While "Immediate Occupancy Performance Level" is specified to buildings that must remain safe after the earthquake, "Life Safety Performance Level" is designated for structures where structural components may sustain damage during an earthquake, but there should be no partial or complete collapse. Lastly, "Collapse Prevention Performance Level" is used for structures where the structural elements are severely affected by an earthquake, resulting in a loss of seismic resistance against the risk of collapse. Similarly, in TBEC (2018), three performance

targets are specified as Limited Damage Performance Level, Controlled Damaged Performance Level, and Collapse Prevention Performance Level. The Limited Damage Performance Level addresses the level of damage in building structural system elements where limited damage occurs, in other words, where non-linear behavior is limited. The Controlled Damage Performance Level is designed to withstand a level of damage in building structural system elements that is not very severe and is mostly repairable, in order to ensure the safety. The Prevention of Collapse Performance Level is designed to withstand the advanced level of severe damage occurring in building structural system elements, preventing the building from partially or completely collapsing. Similar to these two regulations in Eurocode 8.3, the performance limit states are classified in three primary categories which are "Damage Limitation," "Significant Damage," and "Near Collapse." The "Damage Limitation" performance condition applies to structures that have sustained minimal damage, with structural components retaining their stiffness and strength properties. The "Significant Damage" performance state pertains to structures experiencing severe damage, permanent displacements, and harm to non-structural elements. The "Near Collapse" performance scenario is applicable to structures that have suffered extensive damage often resulting in the collapse of most non-structural elements.

In addition to the similarity in performance targets in three regulations, the criterias determined the application of linear elastic methods are also similar. The structural irregularities must be within certain limits which are indicated in detail in all three regulations. In addition to that, one prominent feature in all three standards is the limitation of applying linear elastic methods based on DCR values. High DCR values and irregularities in the structure make the structural behavior more complex. As a result, linear elastic calculation methods can not accurately predict real earthquake behavior. The limiting effects of DCR values and structural irregularities on the application of linear elastic methods are discussed in detail in all three documents. In the ASCE 41-17, the application of linear elastic methods is not allowed if the calculated DCR values of the structural members are greater than 3. According to the Eurocode 8-3, the ratio between the highest and the lowest DCR values calculated

throughout the building should be between 2 and 3 in order to apply linear elastic methods. A recommendation is given for the DCR cutoff value and a value of 2.5 is generally recommended.

Furthermore, in all three standards, structural elements are classified according to the expected failure mode, which is either brittle or ductile in nature. ASCE 41-13 provides control for brittle elements using different load combinations, while Eurocode 8-3 focuses on strength checks for brittle elements. In TBDY 2018, brittle elements are considered as elements that do not directly achieve the desired performance level.

CHAPTER 3

LINEAR ELASTIC METHOD IN TBEC (2018)

3.1 Development of Analysis Algorithm for Linear Elastic Method

The flow chart prepared for the linear elastic method used for the evaluation of existing buildings in the Turkish Building Earthquake Code (TBEC-2018) is presented in Figure 3.1. After checking the validity of the linear elastic method by the DCRs of the members and by several limitations regarding the height class and irregularities in the structure, structural members should be classified as ductile and brittle. Then, the damage observed in the member is determined after the demands, capacities and allowable limits are compared with each other. After the damage levels of all structural members are determined, the seismic performance of the existing structure is evaluated on the basis of four different damage states expected to occur according to TBEC (2018) Section 15.8.2 provided that all brittle elements are strengthened. Structural strengthening procedures are assessed in order to prevent potential failure modes and improve the seismic behavior of existing members.

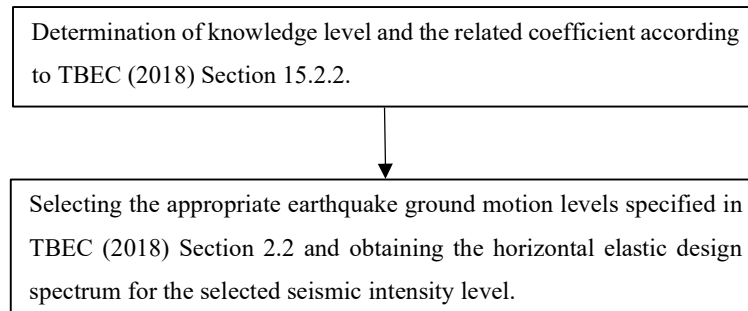


Figure 3.1 Flow-chart that represents the application of linear elastic methods in TBEC (2018)

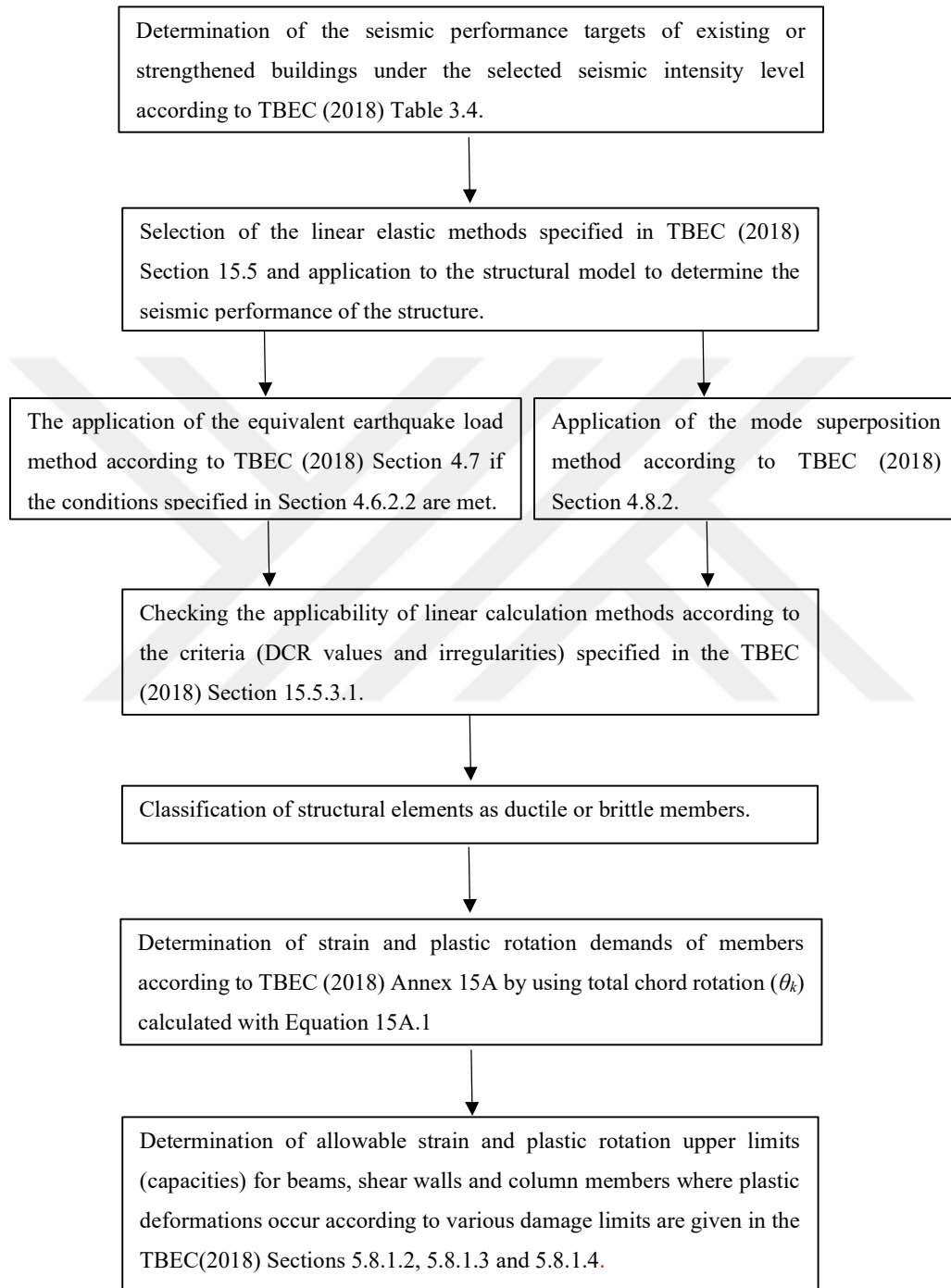


Figure 3.1 (Continued)

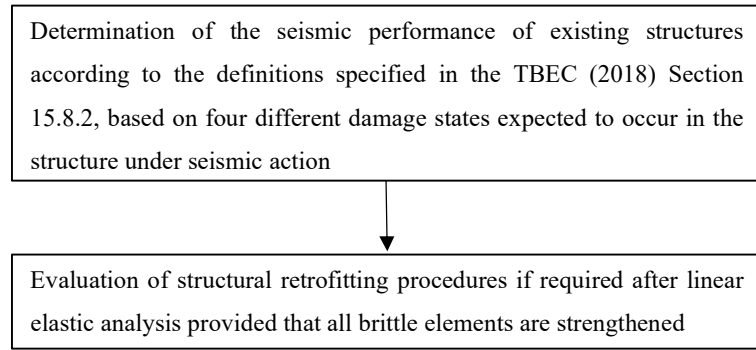


Figure 3.1 (Continued)

3.1.1 Details of the Analysis Algorithm

A linear elastic method algorithm is developed, and all the calculation steps are methodically performed by using the flowchart in Figure 3.1. Sample calculations are given for a column and a beam belonging to a 4-story existing school building in order to demonstrate the important calculation stages of the analysis algorithm. This structure is examined in detail in Section 3.2. The selected column and beam members for which the calculations are shown are marked in Figure 3.2.

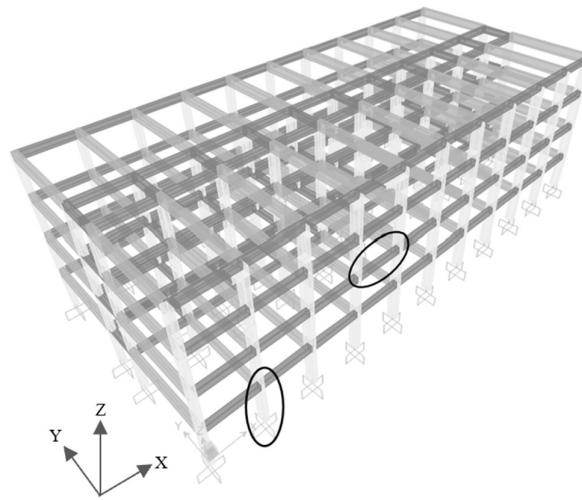


Figure 3.2 Numerical model of the existing school building (selected members are marked for calculations)

Total chord rotations, plastic rotations, moment-curvature relationships, DCR values, allowable plastic deformation limits that are specified according to damage status of the members are calculated for selected members in the X direction of applied earthquake load according to flow chart that is presented in Figure 3.1.

First, the chord rotation (θ_k) is calculated for both i and j ends of the members as shown in Equations 3.1 and 3.2. In these equations, l represents the total length of member, l_c represents the net span, Δ is the inter-story drift, θ_i and θ_j the rotations which are obtained from the structural analysis at the i and j nodes, respectively. Parameters θ_{ki} and θ_{kj} are the chord rotations at the member ends.

$$\theta_{ki} = \frac{\Delta}{l_c} - \theta_i \quad (3.1)$$

$$\theta_{kj} = \frac{\Delta}{l_c} - \theta_j \quad (3.2)$$

The values of θ_i and θ_j are obtained as a result of combinations from lateral force analyses, and they are shown in Table 3.1 for the selected column and beam members. Since the column is located on the ground floor and it is modeled with fixed support conditions, the deformation at the bottom end (i) is obtained as zero.

Table 3.1 Linear elastic analysis results under the worst load combination for selected members

Node	U_x (m)	U_y (m)	U_z (m)	θ_x (rad)	θ_y (rad)	θ_z (rad)
Column I End	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Column J End	0.0846	0.0148	-0.0003	-0.0063	0.0159	0.0000
Beam I End	0.1889	0.0376	-0.0003	-0.0063	0.0153	0.0000
Beam J End	0.1889	0.0376	-0.0003	-0.0063	0.0153	0.0000

According to the analysis results presented in Table 3.1, the total chord rotations for both ends of the column and beam members have been calculated using Equation 3.1 and Equation 3.2. As can be seen in Table 3.1, the translational deformation at the beam ends is zero. Therefore, the chord rotation is mainly related to the magnitude of the rotational deformation of the nodal point to which the element is attached for beam member. On the other hand, for the column member, axial displacement due to translation under horizontal forces is significantly observed and behavior of the column is compatible with the deformed shape presented in Figure 2.3.

The chord rotations for the selected column member can be calculated by Equation 3.3 and Equation 3.4.

$$\theta_{ki} = \frac{\Delta}{l_c} - \theta_i = \frac{0,0846-0}{3,1-0,5} - 0 = 0,0325 \text{ rad} \quad (3.3)$$

$$\theta_{kj} = \frac{\Delta}{l_c} - \theta_j = \frac{0,084}{3,1-0,5} - 0,0159 = 0,0166 \text{ rad} \quad (3.4)$$

The chord rotations for the selected beam member can be calculated by Equation 3.5 and Equation 3.6.

$$\theta_{ki} = \frac{\Delta}{l_c} - \theta_i = \frac{0,0003-0,0003}{3,6-0,3} - 0,0153 = -0,0153 \text{ rad} \quad (3.5)$$

$$\theta_{kj} = \frac{\Delta}{l_c} - \theta_j = \frac{0,0003-0,0003}{3,6-0,3} - 0,0153 = -0,0153 \text{ rad} \quad (3.6)$$

The plastic rotations (θ_p) at the member ends are calculated by Equations 3.7 and Equation 3.8 by using the θ_k values which are calculated from Equations 3.5 and Equation 3.6.

$$\theta_{ki} = \theta_{yi} + \theta_{pi} \quad (3.7)$$

$$\theta_{kj} = \theta_{yj} + \theta_{pj} \quad (3.8)$$

The values of yield rotation (θ_y) occurring at the member ends in Equations 3.7 and Equation 3.8 are calculated to determine the plastic rotations at both ends of the members. The calculation of θ_y is also explained in TBEC (2018) Section 15A.3. The relationships between yield rotations and moments at the i and j ends of a member are defined in Equations 3.9 and Equation 3.10.

$$\theta_{yi} = \frac{Myi*lc}{3EI} * (1 - \frac{Myj}{2M}) \quad (3.9)$$

$$\theta_{yj} = \frac{Myj*lc}{3EI} * (1 - \frac{Myi}{2Myj}) \quad (3.10)$$

The yield moments at the ends of the members are calculated by using the moment-curvature analysis. Moment-curvature is a graphical representation of the relationship between the applied moment (bending moment) and the curvature (strain) in a reinforced concrete section. While yield curvature is the curvature at which the reinforcement steel within the section reaches its yielding point and starts to undergo plastic deformation, ultimate curvature is the maximum curvature that the concrete section can achieve before it fails. Ultimate curvature corresponds to the point on the curve where the curvature begins to increase rapidly as the applied moment approaches the ultimate capacity of the section. Beyond this point, the section is no longer able to carry additional load, and it typically experiences significant cracking and deformation. Yield moment and ultimate moment are the moments at these curvature points. As an example, the cross-section of the selected column and the moment-curvature diagram obtained under the axial load carried by the column are shown in Figure 3.3. The moment curvature relationship is obtained by defining the

section properties with the 540 kN axial load acting on the selected column in the Response 2000 software.

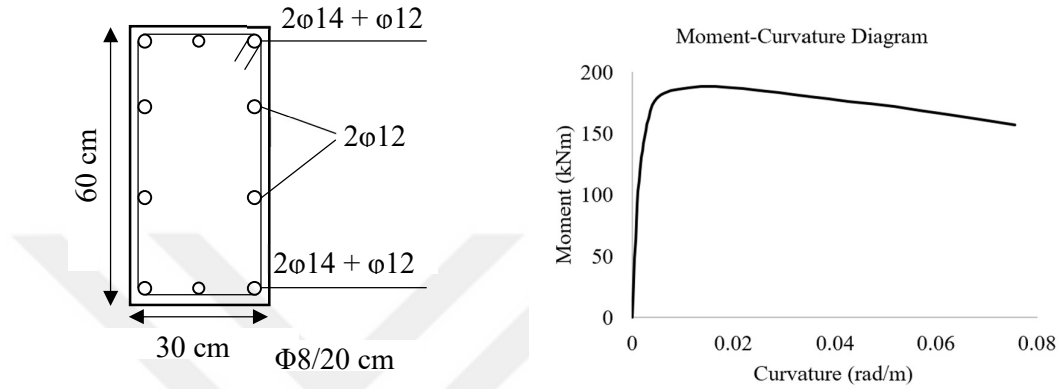


Figure 3.3 Cross-section and moment-curvature diagram of the column

The cross section of the selected beam and the related moment-curvature diagram obtained from the section analysis are shown in Figure 3.4. For a beam with double curvature behavior, the moment-curvature response at the ends differs. Figure 3.4 shows the results of the moment-curvature analysis for the *i*-end and *j*-end of the beam that is consistent with the lateral force analysis.

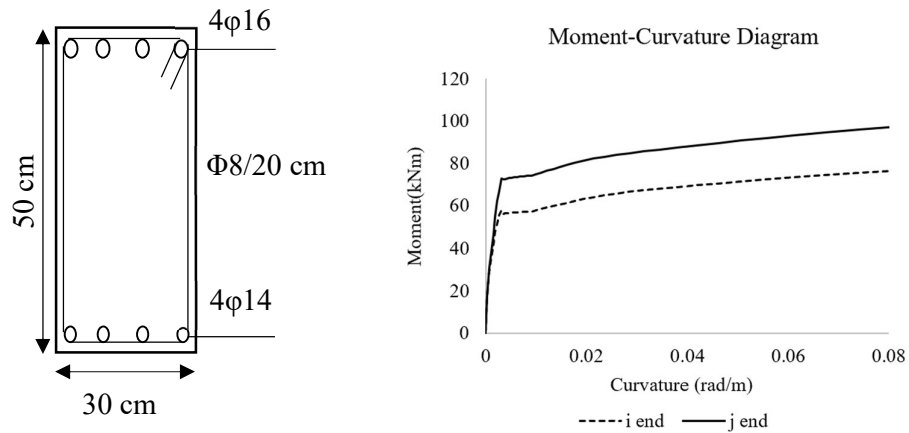


Figure 3.4 Cross-section and moment-curvature diagram of the beam

Yield and ultimate moment values and also curvatures are obtained from the moment curvature graph by using the bilinearization method. The bilinearization process involves identifying two linear segments on the moment-curvature graph. The first segment usually represents the initial elastic behavior of the material, and the second represents the post-yield behavior. Before the material reaches its yield point, the relationship between moment and curvature is linear, following Hooke's Law. This segment is characterized by the material's elastic modulus (E). After the material yields, the relationship between moment and curvature may still be linear, but with a different slope. This segment may be characterized by the effective stiffness of the material after yielding. According to the cross-section analysis, the yield moment M_y is calculated as 176 kN.m under the axial load carried for the selected column member. The yield moments at i and j ends of the beam, i.e., M_{yi} and M_{yj} , are determined as 57 kNm and 76 kNm, respectively. The yield moments at the ends of the members as a result of the section analyses are presented in Table 3.2.

Table 3.2 Linear elastic analysis results for selected members

Member	Moment- i end (kN.m)	Moment- j end (kN.m)	M_{yi} (kN.m)	M_{yj} (kN.m)
Beam	681	709	57	76
Column	998	752	176	176

According to the moment values specified in Table 3.2, the DCR values for column members can be calculated by Equation 3.11 and Equation 3.12.

$$DCR_i = \frac{M_i}{M_{yi}} = \frac{998,06}{176} = 5,67 \quad (3.11)$$

$$DCR_j = \frac{M_j}{M_{yj}} = \frac{752,15}{176} = 4,27 \quad (3.12)$$

The same calculations can also be carried out for beam members as shown in Equation 3.13 and Equation 3.14.

$$DCR_i = \frac{M_i}{M_{yi}} = \frac{681,63}{57} = 11,95 \quad (3.13)$$

$$DCR_j = \frac{M_j}{M_{yj}} = \frac{709,17}{76} = 9.33 \quad (3.14)$$

The calculated DCR values are above the allowable limits given in Section 15.5.3.1 in TBEC (2018). The selected case study structure has poor seismic performance due to the RC element sections and existing low material strengths. At the same time, the school building is located in an area of high seismic hazard. These situations cause high DCR values to be obtained in structural members. Although the DCR limits are exceeded, the rest of calculations for the linear elastic method are carried out in order to demonstrate the workflow and the corresponding steps.

According to the results of the section analysis in Figures 3.3 and 3.4, yield rotations that occurred at the ends of the column and beam members have been calculated using Equations 3.9 and 3.10. Parameter EI represents the bending stiffness of the uncracked section of the member. The EI value is multiplied by 0.35 for beams and 0.7 for columns in order to use the crack section properties of the member. Yield rotations at i and j ends for the selected column members are calculated by Equation 3.15 and Equation 3.16.

$$\theta_{yi} = \frac{176*3.1}{3*0.7*(\sqrt{12*3250+14000})*1000*\frac{1}{12}*0.3*0.6^3} * \left(1 - \frac{176}{2*176}\right) = 0.00095 \text{ rad} \quad (3.15)$$

$$\theta_{yj} = \frac{176*3.1}{3*0.7*(\sqrt{12*3250+1400})*1000*\frac{1}{12}*0.3*0.6^3} * \left(1 - \frac{176}{2*176}\right) = 0.00095 \text{ rad} \quad (3.16)$$

Similarly, yield rotations at i and j ends for the selected beam members are calculated by Equation 3.17 and Equation 3.18.

$$\theta_{yi} = \frac{57 \cdot 3.6}{3 \cdot 0.35 \cdot (\sqrt{12} \cdot 3250 + 1400) \cdot 1000 \cdot \frac{1}{12} \cdot 0.3 \cdot 0.5^3} \cdot \left(1 - \frac{76}{2 \cdot 57}\right) = 0.0008 \text{ rad} \quad (3.17)$$

$$\theta_{yj} = \frac{76 \cdot (3.6)}{3 \cdot 0.35 \cdot (\sqrt{12} \cdot 3250 + 1400) \cdot 1000 \cdot \frac{1}{12} \cdot 0.3 \cdot 0.5^3} \cdot \left(1 - \frac{57}{2 \cdot 76}\right) = 0.0021 \text{ rad} \quad (3.18)$$

After calculating the yield rotations at the ends of the column and beam members, the plastic rotations for both ends of the members are determined by Equations 3.19, Equations 3.20, Equations 3.21, and Equations 3.22 as shown below. The first two calculations represent plastic rotation values at column ends whereas the next two calculations stand for plastic rotations at beam ends.

$$\theta_{ki} = \theta_{yi} + \theta_{pi} \quad \theta_{pi} = 0.0325 - 0.00095 = 0.0316 \text{ rad} \quad (3.19)$$

$$\theta_{kj} = \theta_{yj} + \theta_{pj} \quad \theta_{pj} = 0.0166 - 0.00095 = 0.0156 \text{ rad} \quad (3.20)$$

$$\theta_{ki} = \theta_{yi} + \theta_{pi} \quad \theta_{pi} = 0.0153 - 0.0008 = 0.0145 \text{ rad} \quad (3.21)$$

$$\theta_{kj} = \theta_{yj} + \theta_{pj} \quad \theta_{pj} = 0.0153 - 0.0021 = 0.0132 \text{ rad} \quad (3.22)$$

The damage states observed at the member ends are determined by comparing the calculated plastic rotations with the plastic rotation allowable limit values specified in TBEC (2018). The allowable limit for the calculated plastic rotations to be used in the performance evaluation for the Collapse Prevention Performance Level (CP)

is calculated by using the Equation 2.12. Similarly, plastic rotation limit for Controlled Damage Performance Level (*CD*) is calculated by using the Equation 2.13. Plastic rotation limit for Limited Damage Performance Level (*LD*) is calculated by using the Equation 2.14. According to Equation 2.14, the value of allowed plastic rotation for Limited Damage Performance Level (*LD*) in RC building under concern is zero. Hence plastic rotation limits in CP and CD Performance Levels for column ends can be given by Equation 3.23, Equation 3.24, Equation 3.25, and Equation 3.26.

$$\theta_{pi}^{(CP)} = \frac{2}{3} * [(0.075 - 0.004) * 0.3 * (1 - 0.5 * \frac{0.3}{1.55}) + 4.5 * 0.075 * 0.014] = 0.016 \text{ rad} \quad (3.23)$$

$$\theta_{pj}^{(CP)} = \frac{2}{3} * [(0.075 - 0.004) * 0.3 * (1 - 0.5 * \frac{0.3}{1.55}) + 4.5 * 0.075 * 0.014] = 0.016 \text{ rad} \quad (3.24)$$

$$\theta_{pi}^{(CD)} = 0.75 * 0.016 = 0.012 \text{ rad} \quad (3.25)$$

$$\theta_{pj}^{(CD)} = 0.75 * 0.016 = 0.012 \text{ rad} \quad (3.26)$$

Similarly, plastic rotation limits in CP and CD Performance Levels for beam ends are calculated by Equation 3.27, Equation 3.28, Equation 3.29, and Equation 3.30.

$$\theta_{pi}^{(CP)} = \frac{2}{3} * [(0.12 - 0.003) * 0.25 * (1 - 0.5 * \frac{0.25}{1.8}) + 4.5 * 0.119 * 0.014] = 0.023 \text{ rad} \quad (3.27)$$

$$\theta_{pj}^{(CP)} = \frac{2}{3} * [(0.12 - 0.011) * 0.25 * (1 - 0.5 * \frac{0.25}{1.8}) + 4.5 * 0.119 * 0.016] = 0.022 \text{ rad} \quad (3.28)$$

$$\theta_{pi}^{(CD)} = 0.75 * 0.023 = 0.0172 \text{ rad} \quad (3.29)$$

$$\theta_{pj}^{(CD)} = 0.75 * 0.0225 = 0.0168 \text{ rad} \quad (3.30)$$

The damage states obtained from the comparison of the plastic rotation demands calculated at the ends of the column member with the plastic rotation limits are shown in Table 3.3. Since the plastic rotation that occurred at the i end of the member is above the plastic rotation limit for Collapse Prevention Performance Level (CP), the i end exhibits collapse (C) performance. Since the plastic rotation at the other end is between the CP and CD plastic rotation limits, the j end shows the advanced damage (AD) performance.

Table 3.3 Performance evaluation of column member

Member	θ_{pi} (rad)	θ_{pj} (rad)	$\theta_{pi}^{(CP)}$ (rad)	$\theta_{pj}^{(CP)}$ (rad)	$\theta_{pi}^{(CD)}$ (rad)	$\theta_{pj}^{(CD)}$ (rad)	Check – i end	Check – j end
Column	0.032	0.015	0.016	0.016	0.012	0.012	C	AD

The damage states obtained at the ends of the beam member are shown in Table 3.4. Since the plastic rotation demand at both ends of the beam is below the controlled damage plastic rotation limit, the ends of the member exhibit significant damage (SD) performance.

Table 3.4 Performance evaluation of beam member

Member	θ_{pi} (rad)	θ_{pj} (rad)	$\theta_{pi}^{(CP)}$ (rad)	$\theta_{pj}^{(CP)}$ (rad)	$\theta_{pi}^{(CD)}$ (rad)	$\theta_{pj}^{(CD)}$ (rad)	Check – i end	Check – j end
Beam	0.014	0.013	0.023	0.022	0.017	0.016	SD	SD

To summarize, the calculations for selected beam and column members in the existing 4 story school building are presented as a demonstration of the flowchart in Figure 3.1 for the Linear Elastic Method of TBEC (2018). In the continuation of the calculation stages presented above, the structural performance target will be determined on story basis. Thus, the calculations presented for the sample members are performed for all the members of a structure with the routines written in the MATLAB environment. DCR checks and linear elastic performance evaluations are made throughout the structure by using the developed code.

3.2 Linear Elastic Method Application 1: Existing 4-Story RC School Building

In this section, as the first case study, the 4-Story existing school building is systematically examined with the linear elastic analysis method in TBEC (2018). First of all, the seismic performance of the existing building is evaluated. Then, a strengthening project is proposed for this existing school building. The performance analysis of the building strengthened with shear walls are carried out again with the linear elastic analysis method and the results are discussed.

3.2.1 Seismic Performance Evaluation under Existing Condition

The photo of the 4-story existing school building in Istanbul Bakırköy is shown in Figure 3.5. The building, which has a symmetrical plan, consists of four typical floors. It is 36 meters in width and 16 meters in length. The total height of the building is 12.4 meters. The plan geometry and dimensions of the building are given in Figure 3.6.



Figure 3.5 The photo of the 4-story existing school building

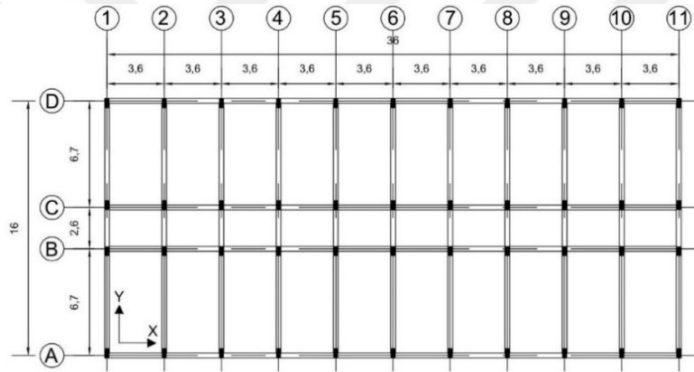


Figure 3.6 General geometry of the structure and principal axes of analysis

The structure has one type of column and two types of beam sizes. Cross sections of the typical column and the beams are shown in Figure 3.7 and 3.8, respectively. According to Figure 3.7, the existing columns have cross-section dimensions of $0.3 \text{ m} \times 0.6 \text{ m}$. The net column height (l_n) in the building is 2.6 meters. According to Figure 3.8, the type 1 beams have cross-section dimensions of $0.3 \text{ m} \times 0.5 \text{ m}$. Net length of the Type 1 beam (l_n) in the building is 3.3 meters. Type 2 beams have cross-section dimensions of $0.3 \text{ m} \times 0.7 \text{ m}$, and the net beam length (l_n) is 6.1 meters. C12 class of concrete and S220 class undeformed steel reinforcement bars had been used in the existing school building. Therefore, in the analysis, the concrete strength is taken as 12MPa, and the steel yield strength is taken as 220 MPa.

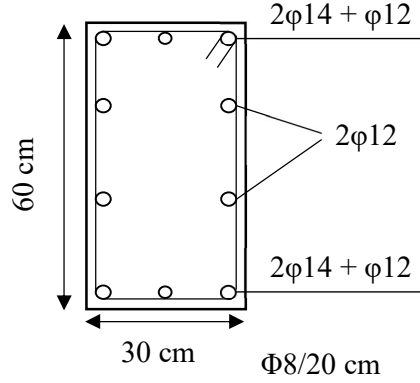


Figure 3.7 Cross-section of a typical column member

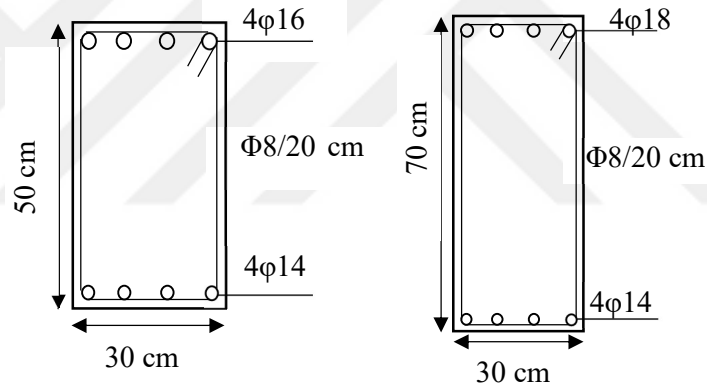


Figure 3.8 Cross-section of type 1 and type 2 beam members

The ground floor plan of the structure is shown in Figure 3.9. The labels of the column and beam members used in the modeling of the structure are also indicated on the floor plan. For each floor, the third digit of the member label is employed to represent the floor number. With this labelling approach, the column, and the beam in the building that the information obtained from the analysis results belongs to can be tracked easily.

301401	101401	301402	101402	301403	101403	301404	101404	301405	101405	301406	101406	301407	101407	301408	101408	301409	101409	301410	101410	301411	
201301		301301		201201		301201		201101		301101		201201		301201		201101		301101		201101	
201302		301302		201202		301202		201102		301102		201202		301202		201102		301102		201102	
201303		301303		201203		301203		201103		301103		201203		301203		201103		301103		201103	
201304		301304		201204		301204		201104		301104		201204		301204		201104		301104		201104	
201305		301305		201205		301205		201105		301105		201205		301205		201105		301105		201105	
201306		301306		201206		301206		201106		301106		201206		301206		201106		301106		201106	
201307		301307		201207		301207		201107		301107		201207		301207		201107		301107		201107	
201308		301308		201208		301208		201108		301108		201208		301208		201108		301108		201108	
201309		301309		201209		301209		201109		301109		201209		301209		201109		301109		201109	
201310		301310		201210		301210		201110		301110		201210		301210		201110		301110		201110	
201311		301311		201211		301211		201111		301111		201211		301211		201111		301111		201111	

Figure 3.9 Ground floor plan and labelling of structural members

Within the scope of the analysis of the 4-story school building with the linear elastic method, first the existing structure is modeled by using the SAP2000 (2022-v22) finite element analysis program. The SAP2000 analysis model of the 4-story school building is shown in Figure 3.10. Fixed support conditions are defined for the bottom ends of the columns on the ground floor. Beams and columns are modeled as 1-D frame elements. The effective flexural stiffness coefficients have been applied to all structural elements according to TBEC (2018) Section 4.5.8 in order to consider the cracked section properties of the structural members.

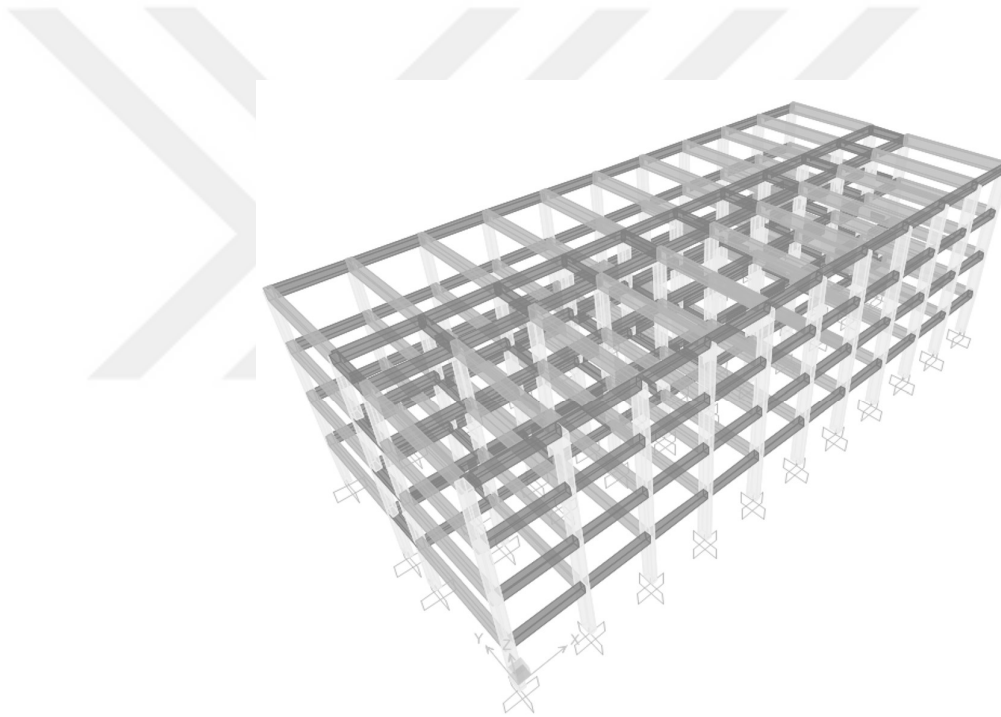


Figure 3.10 SAP2000 analysis model of 4-Story school building

As a result of the modal analysis, the fundamental periods of the structure are obtained. Accordingly, the period of the structure in the X direction (T_X) is determined as 0.91 seconds whereas the period in the Y direction (T_Y) is determined as 0.64 seconds. The mode shapes with the determined periods are shown in Figures 3.11 and 3.12, respectively.

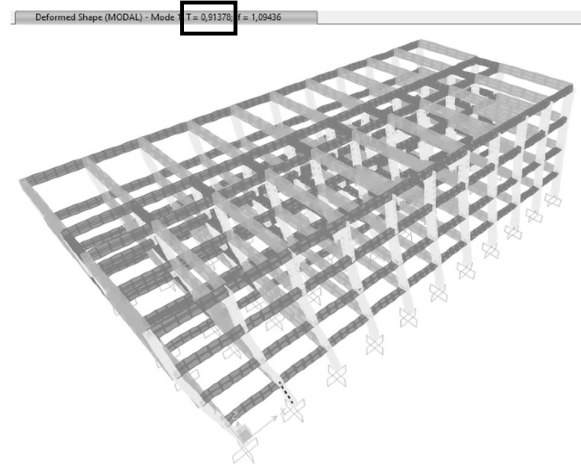


Figure 3.11 Mode-shape and the period of the structure in X direction (T_X)

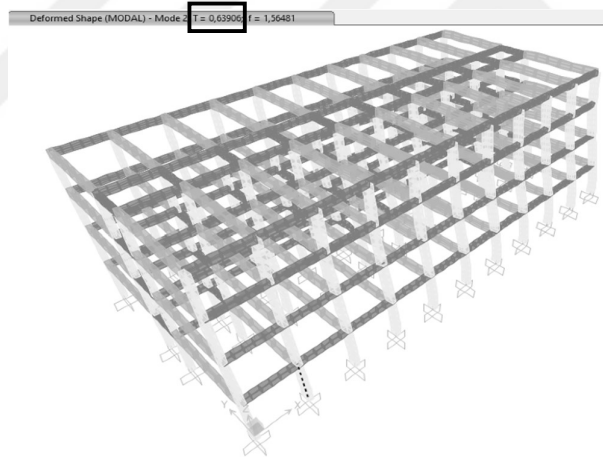


Figure 3.12 Mode shape and the period of the structure in Y direction (T_Y)

The building occupancy class (BKS) is 1 as indicated in Table 2.6 for the examined structure. Earthquake Design Classes (DTS), which will be taken as a basis in the design under the effect of earthquake, are given in Table 2.7. The short period design spectral acceleration coefficient (S_{DS}) under the DD-2 seismic intensity level for this structure is greater than 0.75. In this case, according to Table 2.7, the Earthquake

Design Class (*DTs*) for the existing school building is selected as “1a”. Since the earthquake design class is “1a” and the building height is 12.4 m, the Building Height Class (*BYS*) value is taken as 6 according to Table 2.8. In relation with the coefficients mentioned above, the target performance level for the school building is determined as the “Controlled Damage Performance Level” under DD-1 seismic intensity level. For this case, the existing structure is analyzed under the DD-1 seismic intensity level, which represents very rare earthquake ground motion with the probability of 2% exceedance in 50 years. and the corresponding recurrence period is 2475 years. The site conditions of the current location of the school building are assumed to be ZC (compact sand, hard clay, or weak fragmented rock). The horizontal linear elastic design spectrum representing the seismic hazard on building site is presented in Figure 3.13. The first two period values of the structure in the *X* direction and in the *Y* direction and the corresponding spectral acceleration demands are plotted on the spectrum graph shown in Figure 3.13. The acceleration values obtained from the graph are used in the calculations of the equivalent static horizontal loads to be applied to the structure in the next step.

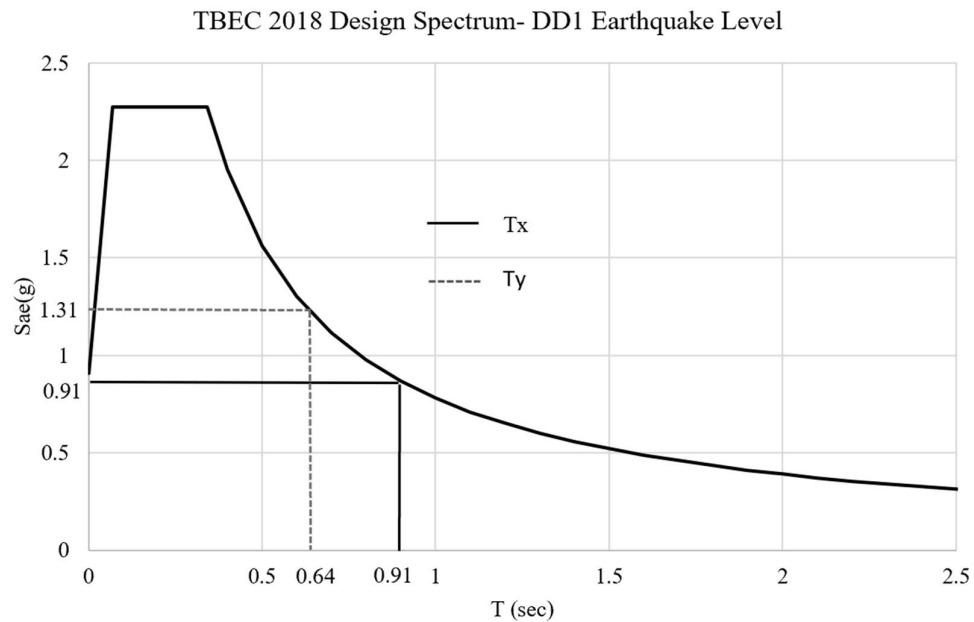


Figure 3.13 Response spectrum for DD-1 seismic intensity level

The applicability limits of the Equivalent Lateral Earthquake Load Method, which is one of the linear elastic calculation methods, are specified in TBEC (2018) Section 4.6.2. Accordingly, permitted BYS is specified for the method in order to be applied according to the building type and earthquake design classes. When these applicability limits are evaluated, it is concluded that the Equivalent Lateral Earthquake Load Method could be used for the existing 4-story school building where the BYS is determined as 6 according to Table 2.8. The periods of the structure are used in the calculation of earthquake forces with the Equivalent Lateral Earthquake Load Method. The forces are determined by applying this method which is described in Section 4.7 in TBEC (2018) in orthogonal X and Y earthquake directions for investigating the behavior of the building under the effect of earthquake forces. The total equivalent earthquake load (base shear force) acting on the whole building is calculated according to TBEC (2018) Section 4.7. Equations 3.31 and 3.32 determine the total equivalent lateral earthquake load acting on the entire building in the direction of the earthquake under consideration.

$$V_{tE}^{(X)} = m_t * S_{aR}(T_X) \quad (3.31)$$

$$V_{tE}^{(Y)} = m_t * S_{aR}(T_Y) \quad (3.32)$$

$S_{aR}(T_x)$ and $S_{aR}(T_y)$ in Equations 3.31 and 3.32 indicate the reduced design spectral acceleration value calculated along the X and Y earthquake directions. These values are calculated according to Equation 3.33.

$$S_{aR}(T) = \frac{S_{aE}(T)}{R_a(T)} \quad (3.33)$$

According to Section 15.5.1.1 in TBEC (2018), R_a value is taken as 1 in the calculation of the total equivalent earthquake load. Table 3.5 shows the period values used in the equivalent lateral static load calculation and the spectral acceleration values corresponding to these period values. The total shear force acting on the structure is calculated for the X and Y directions as shown in Table 3.5.

Table 3.5 Equivalent seismic loads calculated for two principal axes of the structure

$T_x=$	0.91 s	$T_A=$	0.06 s
$T_y=$	0.64 s	$T_B=$	0.34 s
$S_{DS}=$	2.28 g	$S_{ae}(T_x)=$	0.91 g
$S_{D1}=$	0.78 g	$S_{ae}(T_y)=$	1.31 g
$DTS=$	1 a	$R_a(T)=$	1
$BYS=$	6	$S_{aR}(T_x)=$	8.95 m/s ²
$BKS=$	1	$S_{aR}(T_y)=$	12.80 m/s ²
$M_{total}=$	2708 kg	$V_x=$	24249.51 kN
$W=$	26569 kN	$V_y=$	34673.92 kN

Equivalent lateral earthquake loads acting on the floors are calculated according to TBEC (2018) Section 4.7.2. The total equivalent earthquake load in the X direction is defined by Equation 3.34 as the sum of the equivalent earthquake loads acting on the building floors in this direction.

$$V_{tE}^{(X)} = \Delta F_{NE}^{(X)} + \sum_{i=1}^N F_{iE}^{(X)} \quad (3.34)$$

The value of the additional equivalent earthquake load $\Delta F_{NE}^{(X)}$ acting on the N^{th} floor (top floor) of the building is determined by Equation 3.35.

$$\Delta F_{NE}^{(X)} = 0.0075 * N * V_{tE}^{(X)} \quad (3.35)$$

The rest of the total equivalent earthquake load, except $\Delta F_{NE}^{(X)}$, is distributed to the building floors including the N^{th} floor by Equation 3.36. The indices i and j in the equation indicate the floor number, m indicates the floor mass, and H indicates the height of the floor from the ground level.

$$F_{iE}^{(X)} = \left(V_{tE}^{(X)} - \Delta F_{NE}^{(X)} \right) * \frac{m_i H_i}{\sum_{j=1}^N m_j * H_j} \quad (3.36)$$

The additional equivalent earthquake load ΔF_{NE} acting on the 4th floor of the existing building is calculated by Equation 3.35 as 727 kN and 1040 kN for the X and Y directions, respectively. The total equivalent earthquake load is distributed to the building floors using Equation 3.36. Table 3.6 shows the equivalent lateral seismic loads acting on the floors in the X and Y directions.

Table 3.6 Equivalent earthquake loads acting on the each floor

Floor Number	W_i	H_i	$W_i.H_i$	F_X (kN)	F_Y (kN)
Floor 1	6642.3	3.1	20591.1	2352.2	3363.3
Floor 2	6642.3	6.2	41182.2	4704.4	6726.7
Floor 3	6642.3	9.3	61773.3	7056.6	10090.1
Floor 4	6642.3	12.4	82364.4	9408.8	13453.4

Equivalent lateral earthquake loads acting on the floors are applied to the analysis model separately for both directions. The calculated earthquake forces are applied to the model as two separate analysis cases. The results obtained for both principal directions of the structure have been compiled from the dead load and live load ($G+Q$) cases and the earthquake load cases applied in linear combinations. For both principal axis, earthquake force acting in the other direction is included in the combination with a coefficient of 0.3. The load combinations which are considered in the analysis are shown in Equation 3.37 and Equation 3.38.

$$G + Q \pm E^{(X)} \pm 0.3E^{(Y)} \quad (3.37)$$

$$G + Q \pm E^{(Y)} \pm 0.3E^{(X)} \quad (3.38)$$

After linear elastic modeling of the structure and analysis under equivalent lateral static lateral load, DCR values are calculated in order to check the applicability limits of the linear elastic method. The required M_y value in the calculation of the DCRs of the structural members is obtained as a result of the moment-curvature analysis for both ends of the beam members showing double curvature behavior. For the column members, M_y values are calculated using the moment-curvature diagram obtained under the axial load.

DCR calculations are made for both ends of all members in the existing structure. Tables 3.7 and 3.8 show the average DCR values of beam and column members on a floor basis under the earthquake forces applied in the X and Y directions. The average DCR values given on the floor basis for the column members are scaled with the shear force and they are calculated with the following Equation 3.39 in TBEC (2018) Section 15.5.3.2. In Equation 3.39, index i shows the column number in the relevant floor, V_i shows the shear force calculated in column i , and $(DCR)_i$ shows the DCR value calculated in this column.

$$DCR = \frac{\sum_i V_i * (DCR)_i}{\sum_i V_i} \quad (3.39)$$

Table 3.7 Average DCR values of beam members for each floor

Floor Number	Under the Earthquake force in <i>X</i> direction	Under the Earthquake force in <i>Y</i> direction
Floor 1	13.4	14.4
Floor 2	12.2	14.6
Floor 3	8.4	10.4
Floor 4	3.4	5.0

Table 3.8 Shear force scaled average DCR values of column members for each floor

Floor Number	Under the Earthquake force in <i>X</i> direction	Under the Earthquake force in <i>Y</i> direction
Floor 1	5.5	9.4
Floor 2	5.1	6.2
Floor 3	4.7	7.0
Floor 4	3.5	6.4

The calculated DCR values are above the limits given in TBEC (2018) Section 15.5.3.1. The structure is very weak due to the poor material strength of the RC element sections used in the existing school building. At the same time, it is under the influence of high intensity seismic action since it is evaluated under the DD-1 seismic intensity level. This consideration eventually yields high DCR values in structural members. Although the DCR limits are exceeded, further calculations are

carried on and the evaluation results are obtained due to the advantage of the simple structural geometry of the case study building and the intention to examine the performance of the linear elastic method in detail for structures with high DCR values.

After the DCR values are checked, the performances of all column and beam members in the existing structure are evaluated. In Table 3.9, damage state distribution in the beams are classified for each floor. In Table 3.10, the share of shear force in vertical elements for each damage state (in %) is presented for each floor. In Table 3.11, the shear force distribution of the vertical elements exceeding the significant damage zone in the lower and upper sections is presented on a story basis.

Table 3.9 Damage state distribution for beam members (%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	0	100	0	0	0	100	0	0
3	0	100	0	0	0	100	0	0
2	0	80	20	0	0	33	67	0
1	0	80	10	10	0	67	33	0

Table 3.10 Damage state distribution proportional to shear force for vertical elements(%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	0	100	0	0	0	100	0	0
3	0	12	88	0	0	100	0	0
2	0	0	12	88	0	14	53	32
1	0	0	0	100	0	0	0	100

Table 3.11 Damage state distribution of vertical members exceeding the significant damage zone in lower and upper sections by shear force ratio (%)

Floor	X Direction		Y Direction	
	LD+SD	AD+C	LD+SD	AD+C
4	100	0	100	0
3	12	0	100	0
2	0	88	14	82
1	0	85	0	0

According to the results presented in Tables 3.9-3.11, the criterion if the existing 4-story school building satisfies the CD performance level under DD-1 seismic intensity level or not is determined as a result of some checks as stated below.

- Beam Damage Ratio:
Advanced Damage (AD) = %67 > %35 **NOT OK!**
Collapse (C) = %10 > 0 **NOT OK!**
- Vertical Member Shear Force (V_c) Ratio :
Advanced Damage (AD) = %88 > %20 **NOT OK!**
Collapse (C) = %100 > 0 **NOT OK!**
- Top Floor Vertical Member Shear Force (V_c) Ratio:
Advanced Damage (AD) = %0 ≤ %40 **OK!**
Collapse (C) = %0 **OK!**
- Vertical Element Shear Force (V_c) Ratio Exhibiting Plastic Behavior:
Advanced Damage +Collapse (AD+C) = %88 > %30 **NOT OK!**

As a result of the analysis, the “CD Performance Level” can not be achieved for the existing 4-story school building. The performance of the structure, which is analyzed by the linear elastic analysis method, is determined as the "Collapse Performance Level".

3.2.2 Seismic Performance Evaluation under Strengthened Condition

Due to the failure to meet the performance target, it is aimed to enhance the seismic behavior of the existing members and to achieve the target performance level. For this purpose, the RC framing system of the existing 4-story school building, which has insufficient lateral rigidity and strength, is strengthened with cast-in-situ RC shear walls. Within the scope of this examination, the linear elastic analysis is repeated by adding RC shear walls to the existing 4-story school building model. Then, the seismic performance of the structure is re-evaluated with the linear elastic analysis method. Shear walls which are added to the building are of two types and their locations are given in the floor plan presented in Figure 3.14.

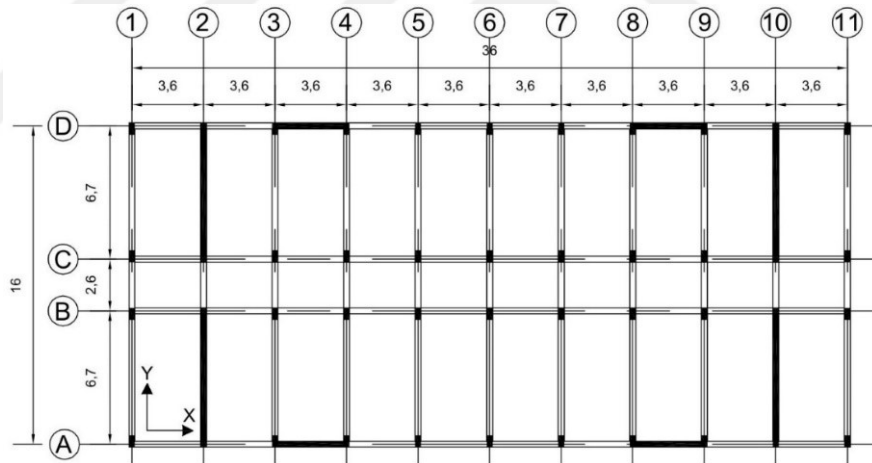


Figure 3.14 Structural layout of strengthened building

The cross-sectional properties of the two additional shear walls are shown in Figures 3.15 and 3.16. The first type of shear wall has cross-sectional dimensions of $3.9 \text{ m} \times 0.3 \text{ m}$ according to Figure 3.15. Clear height of shear wall is 2.6 meters. The steel reinforcement details in the cross-section are given in Figure 3.15. The cross-sectional features of the second type shear wall are shown in Figure 3.16. According

to Figure 3.16, the member has cross-sectional dimensions of 7 m $\times$ 0.3 m. Clear height of this shear wall type is 2.6 meters. The steel reinforcement details in the cross section are given in Figure 3.16.

Concrete class C30 and reinforcement steel class S420 (deformed bar) are used in the shear walls. Therefore, the concrete strength is taken as 30 MPa and the steel yield strength is taken as 420 MPa in the analysis. The effective stiffness coefficients defined for the shear walls according to TBEC (2018) Section 4.5.8 is applied in order to use the cracked cross-section properties of the structural elements.

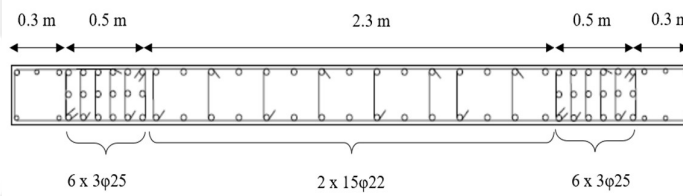


Figure 3.15 Cross-sectional properties of type 1 RC shear wall

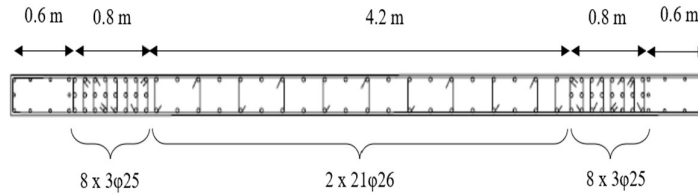


Figure 3.16 Cross-sectional properties of type 2 RC shear wall

The yield moments at the ends of the shear walls are calculated as a result of the moment-curvature analysis. The moment curvature diagram under the axial load that belongs to the Type 1 shear wall member on the first floor is shown in Figure 3.17.

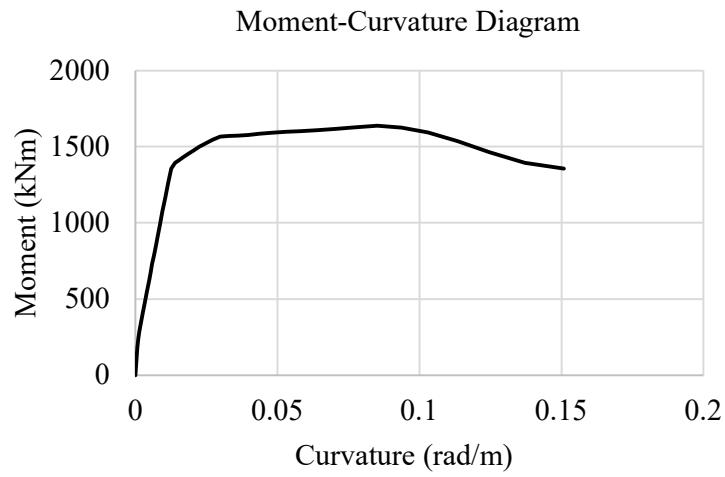


Figure 3.17 Moment- curvature diagram for type 1 shear wall

The moment curvature diagram under the axial load that belongs to the Type 2 shear wall member on the first floor is shown in Figure 3.18.

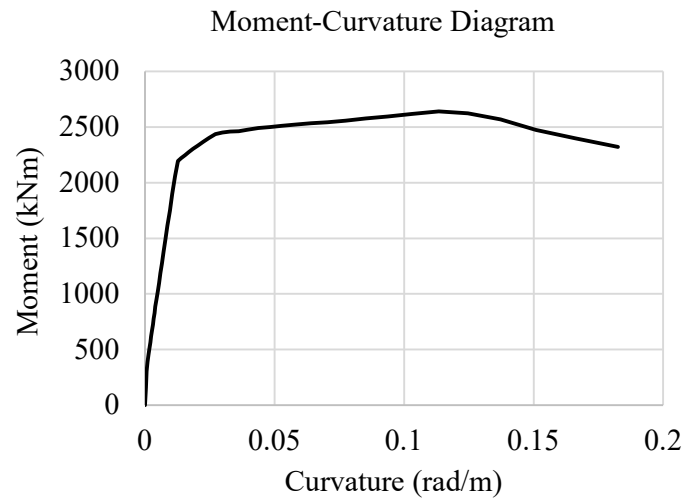


Figure 3.18 Moment- curvature diagram for type 2 shear wall

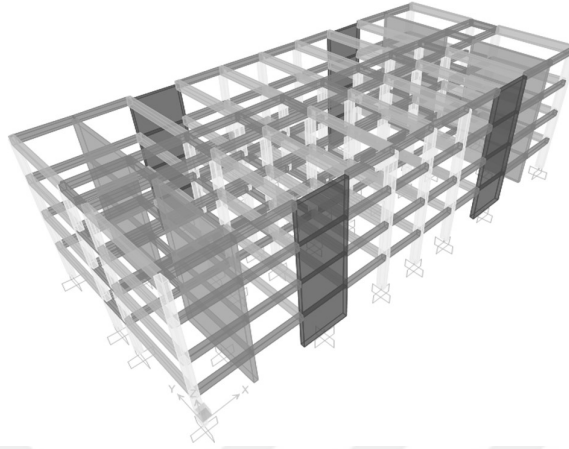


Figure 3.19 Analysis model of strengthened 4-Story school building

The 3D analysis model of the strengthened school building is shown in Figure 3.19. As a result of the modal analysis of the strengthened structure, the fundamental period of the structure in the X direction (T_X) is determined as 0.418 seconds, and the fundamental period in the Y direction (T_Y) is determined as 0.21 seconds. The mode shapes are presented in Figure 3.20 and Figure 3.21, respectively. Moreover, equivalent lateral static loads are recalculated for the strengthened structure. The results are given in Table 3.12.

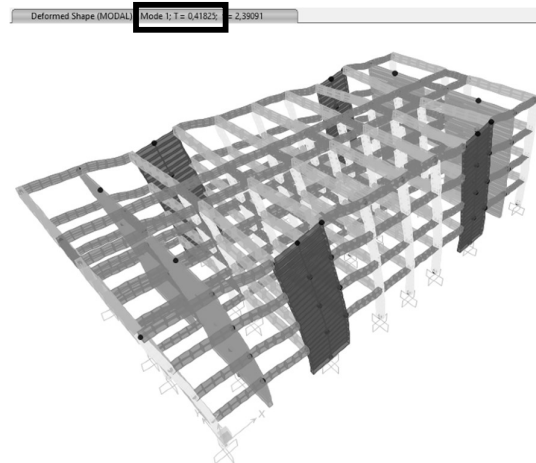


Figure 3.20 Mode shape and period of the strengthened structure in X direction (T_X)

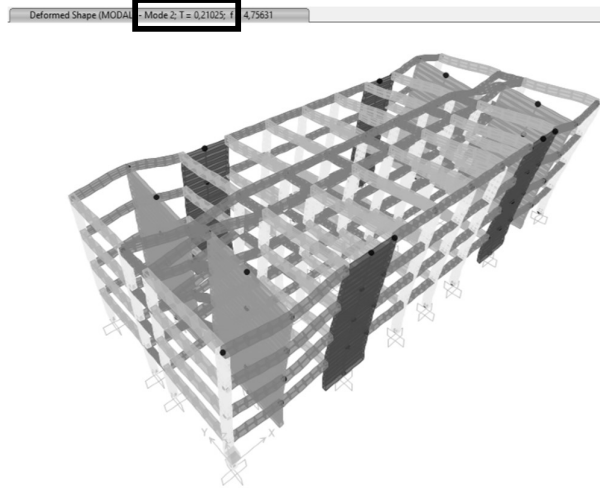


Figure 3.21 Mode shape and period of the strengthened structure in Y direction (T_y)

Table 3.12 Equivalent lateral seismic loads calculated for two principal axis of the strengthened structure

$T_x =$	0.42 s	$T_A =$	0.06 s
$T_y =$	0.21 s	$T_B =$	0.34 s
$S_{DS} =$	2.28 g	$S_{ae}(T_x) =$	1.99 g
$S_{D1} =$	0.78 g	$S_{ae}(T_y) =$	2.53 g
$DTS =$	1a	$R_a(T) =$	1
$BYS =$	6	$S_{aR}(T_x) =$	19.56 m/s ²
$BKS =$	1	$S_{aR}(T_y) =$	24.86 m/s ²
$M_{total} =$	2656 kg	$V_x =$	51956.10 kN
$W =$	26055 kN	$V_y =$	66025.71 kN

The total equivalent base shear force is distributed to the floors using Equation 3.36. Equivalent lateral earthquake loads acting at each floor in X and Y directions are shown in Table 3.13.

Table 3.13 Equivalent earthquake loads acting on the each floor for strengthened structure

Floor Number	W_i	H_i	$W_i.H_i$	F_x (kN)	F_y (kN)
Floor 1	6513.9	3.1	20193.3	5039.7	6404.4
Floor 2	6513.9	6.2	40386.6	10079.4	12808.9
Floor 3	6513.9	9.3	60580.1	15119.2	19213.4
Floor 4	6513.9	12.4	80773.3	20158.9	25617.9

After modeling of the structure in the linear elastic range and analysis under equivalent static lateral load, DCR values are calculated in order to check the applicability limits of the linear elastic method in TBEC (2018). Tables 3.14- 3.16 show the DCR values of the average beam, column, and shear wall members on a story basis under the seismic forces applied in the X and Y directions. The average DCR values given on floor basis for vertical elements are scaled with shear force and these values are calculated with Equation 3.39 as stated in the previous section.

Table 3.14 Average DCR values for beam members at each floor in strengthened structure

Floor Number	Under the Earthquake force in X direction	Under the Earthquake force in Y direction
Floor 1	5.6	2.8
Floor 2	8.0	3.6
Floor 3	8.9	3.9
Floor 4	6.5	3.1

Table 3.15 Shear force scaled average DCR values of column members at each floor in strengthened structure

Floor Number	Under the Earthquake force in X direction	Under the Earthquake force in Y direction
Floor 1	1.5	2.0
Floor 2	2.3	1.6
Floor 3	3.0	2.0
Floor 4	4.4	3.2

Table 3.16 Shear force scaled average DCR values of shear wall members at each floor in strengthened structure

Floor Number	Under the Earthquake force in X direction	Under the Earthquake force in Y direction
Floor 1	16.5	22.6
Floor 2	9.9	14.7
Floor 3	4.2	7.7
Floor 4	2.1	7.1

When the DCR values calculated for the strengthened structure are compared with the DCR values obtained for the existing school building, it is observed that the DCR values have decreased significantly. However, the DCR values which are obtained are still above the limits given in TBEC (2018) Section 15.5.3.1. However, the calculations are carried on for the reasons discussed in the previous section and in order to examine the effect of the strengthening method applied to the structure on the performance by the linear elastic method.

After the DCR values are checked, the performances of all vertical and beam members in the structure are evaluated. In Table 3.17, the damage state distribution in the beams are classified for each floor. In Table 3.18, the share of shear forces in vertical elements for each damage state (in %) is presented for each floor. In Table 3.19, the shear force distribution of the vertical elements exceeding the significant damage zone in the lower and upper sections is presented on a story basis.

Table 3.17 Damage state distribution of beam members in strengthened structure (%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	0	100	0	0	0	100	0	0
3	0	100	0	0	0	100	0	0
2	0	100	0	0	0	100	0	0
1	0	100	0	0	0	100	0	0

Table 3.18 Damage state distribution proportional to shear force of vertical elements in strengthened structure (%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	53	37	10	0	82	18	0	0
3	80	20	0	0	91	9	0	0
2	87	13	0	0	94	6	0	0
1	0	100	0	0	94	6	0	0

Table 3.19 Damage state distribution of vertical members exceeding the significant damage zone in lower and upper sections by shear force ratio in strengthened structure (%)

Floor	X Direction		Y Direction	
	LD+SD	AD+C	LD+SD	AD+C
4	90	0	100	0
3	100	0	100	0
2	100	0	100	0
1	100	0	100	0

According to the results presented in Tables 3.17- 3.19, the criterion if the 4-story school building strengthened with shear walls satisfies the “CD Performance Level” under DD-1 seismic intensity level or not is determined as a result of the checks as stated below.

- Beam Damage Ratio:
Advanced Damage (AD) = %0 ≤ %35 **OK!**
Collapse (C) = %0 **OK!**
- Vertical Member Shear Force (V_c) Ratio :
Advanced Damage (AD) = %10 ≤ %20 **OK!**
Collapse (C) = %0 **OK!**
- Top Floor Vertical Member Shear Force (V_c) Ratio:
Advanced Damage (AD) = %10 ≤ %40 **OK!**
Collapse (C) = %0 **OK!**
- Vertical Element Shear Force (V_c) Ratio Exhibiting Plastic Behavior:
Advanced Damage +Collapse (AD+C) = %0 ≤ %30 **OK!**

As a result of the seismic performance analysis, the “CD Performance Level” is achieved for the strengthened 4-story school building. In other words, it has been

revealed by the linear elastic method that the existing school building achieved the target performance level after being strengthened with shear walls.

3.3 Linear Elastic Method Application 2: Existing 6-Story Dormitory Building

A 6-story dormitory building is chosen as the second existing structure for which the seismic performance is evaluated by using the TBEC (2018) linear elastic method. In the selection of this structure, the condition that the DCRs of the structural elements in the building to be examined should be slightly above the limits specified by the TBEC (2018) linear elastic method is taken as a criterion. Considering the structural features and material properties of the dormitory building, DCRs are expected to meet this condition. At the same time, the fact that the building has a non-symmetrical floor plan provides an opportunity to test the estimation accuracy of the linear elastic method in structures with this feature. These are the reasons of selecting this dormitory building as a case study structure.

3.3.1 Seismic Performance Evaluation of 6-Story Dormitory Building

The dormitory building was built before 2007 and it is located in the Gebze/Kocaeli region. The extruded 3D view and typical floor plan of the building are given in Figures 3.22 and 3.23, respectively. The approximate plan dimensions are 30 meters by 15 meters. The total height is 20.7 meters. A single type of column and a single type of beam section are used in the structure. The columns have cross-sectional dimensions of 0.4 m $\times$ 0.4 m. The clear column height (l_n) in the structure is 2.95 meters. The beams have cross-sectional dimensions of 0.3 m $\times$ 0.5 m, and their clear span (l_n) is 3.8 m. There are two 20 cm thick shear walls in both principal directions. The cross-sections of columns, beams and shear walls are given in Figures 3.24-3.26, respectively. Concrete class is determined as C20, and S220 class undeformed reinforcement steel is used in the building. Consistent with the material classes,

concrete strength is taken as 20 MPa and the yield strength of the reinforcement steel is taken as 220 MPa in the structural modeling.

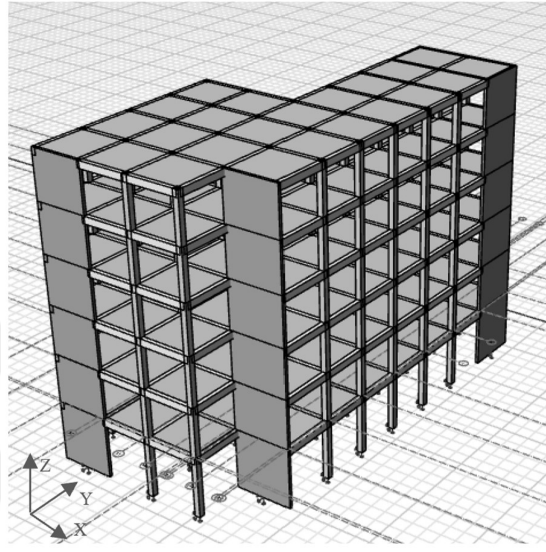


Figure 3.22 Extruded 3D view of the numerical model of the 6-story dormitory building

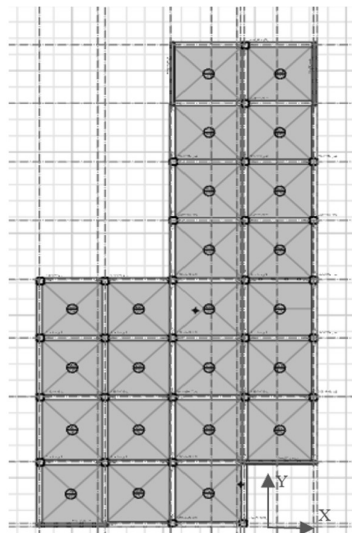


Figure 3.23 Typical floor plan of 6-story dormitory building

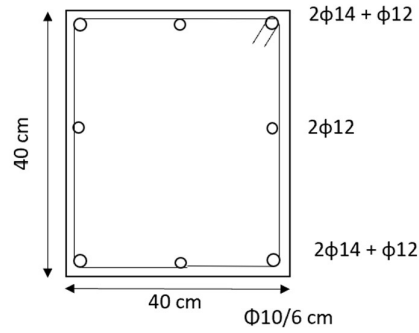


Figure 3.24 Cross-sectional properties of a typical column

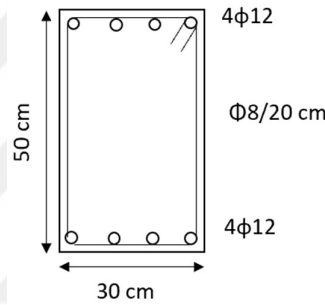


Figure 3.25 Cross-sectional properties of a typical beam

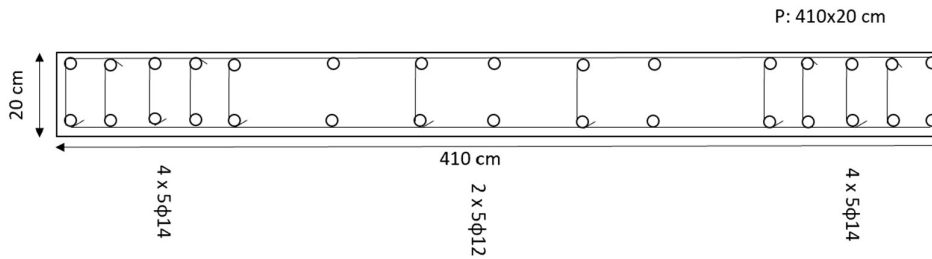


Figure 3.26 Cross-sectional properties of a typical shear wall

The building occupancy class (BKS) is taken as 1 as indicated in Table 2.6 for the case study structure. Earthquake Design Classes (DTS), which will be taken as a basis in the design under the effect of earthquake, are given in Table 2.7. The short period design spectral acceleration coefficient (S_{Ds}) under the DD-2 seismic intensity level for this structure is greater than 0.75. In this case, according to Table

2.7, the Earthquake Design Class (*DTS*) for the existing school building becomes 1a. Since the earthquake design class is 1a and the building height is 20.7 m, the Building Height Class (*BYS*) value is taken as 5 according to Table 2.8.

According to the factors mentioned above, the target performance level for the school building is determined as the “CD Performance Level” under DD-1 seismic intensity level. The site conditions of the current location of the dormitory building are assumed to be ZC (compact sand, hard clay, or weak fragmented rock). The horizontal linear elastic design spectrum representing the seismic hazard on building site is presented in Figure 3.27. The values of the first two vibration period of the structure in x and y directions and the corresponding spectral acceleration demands are plotted on the spectrum graph shown in Figure 3.27.

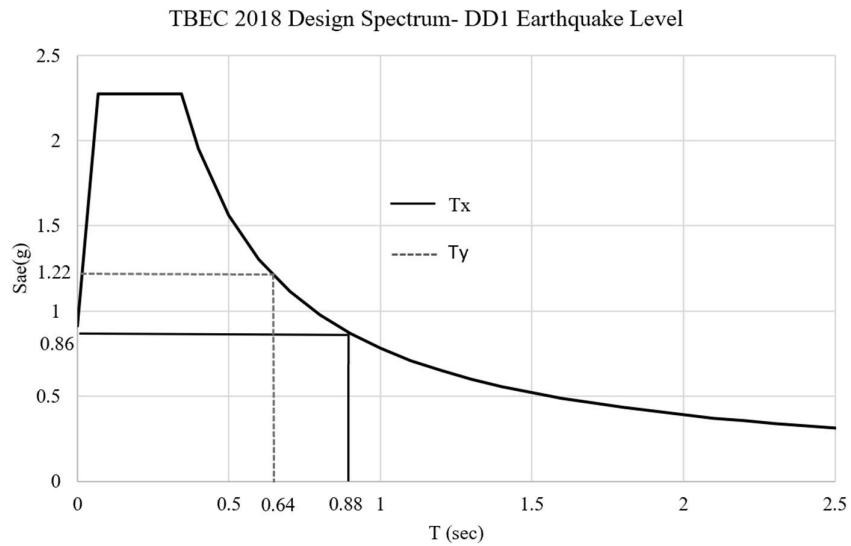


Figure 3.27 Response spectrum for DD-1 earthquake ground motion level

In order to apply the linear elastic method in TBEC (2018), numerical models of the structure with linear elastic properties are created on the OpenSees (2021) platform. The modeling approaches adopted for the previously examined building are also

used in the analysis model of the dormitory building. Cracked section properties are defined in linear elastic members. Shear walls are modeled using the wide column analogy.

The modal analysis is applied, and the basic dynamic properties of the structure are determined. The dominant period of the structure in the X direction (T_X) is calculated as 0.88 seconds whereas the dominant period in the Y direction (T_Y) is determined as 0.64 seconds. The corresponding mode shapes are presented in Figures 3.28 and 3.29, respectively. The structure of the dormitory building exhibits a dominant torsional behavior due to the non-symmetrical floor plan, as can be seen from the mode shapes.

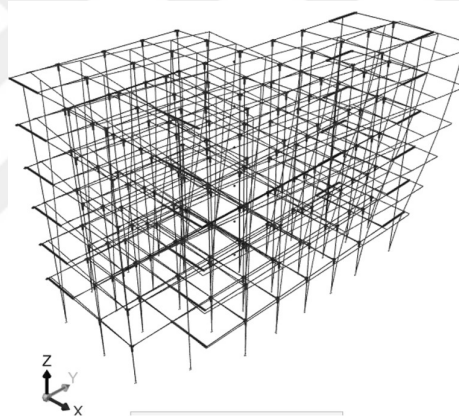


Figure 3.28 Mode shape for $T_X = 0.88$ sec

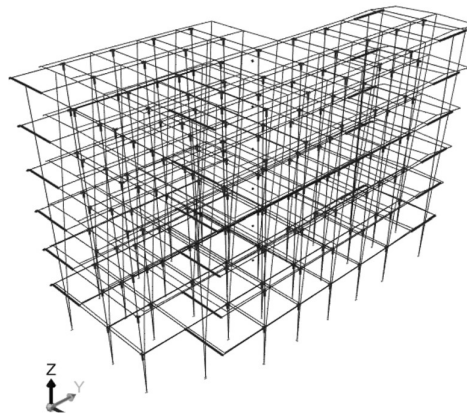


Figure 3.29 Mode shape for $T_Y = 0.64$ sec

The linear elastic method in TBEC (2018) is applied to the numerical model of the dormitory building through the routines presented in the previous section. Firstly, DCRs for the structural elements on each floor except the top floor of the building are obtained and the average values per floor are determined. The DCR values are presented in Table 3.20 for both principal directions.

Table 3.20 Calculated DCR values for structural members

Floor	Member	DCR _X	DCR _Y
1	Column / Wall	3.28	1.42
	Beam	7.51	5.23
2	Column / Wall	3.36	2.33
	Beam	8.75	7.27
3	Column / Wall	3.63	3.28
	Beam	8.47	7.80
4	Column / Wall	3.57	3.81
	Beam	7.44	7.37
5	Column / Wall	3.32	4.08
	Beam	6.10	6.67

According to the results presented in Table 3.20, the DCR values calculated for the X direction are slightly above the DCR limits that are specified as 3 for ductile columns and 5 for beams in TBEC (2018) Section 15.5.3. A similar condition is examined for the beams in principal Y direction and upper floor columns. It is thought that the mentioned DCR values are caused by the low material strength of the building and the torsional effect due to the non-symmetrical floor plan ($\eta_{bi} = 1.705$ according to TBEC (2018) Table 3.6). For this structure, the estimation accuracy of the linear elastic method, which is not valid according to TBEC (2018)

Section 15.5.3, can be examined for the exceeded DCR values. As a result of the application of the linear elastic method, the summary results compiled for the two principal directions of the structure on a story basis are given in Tables 3.21- 3.23.

Table 3.21 Damage state distribution of beam members in dormitory building (%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
6	0	100	0	0	0	100	0	0
5	0	100	0	0	0	100	0	0
4	0	100	0	0	0	100	0	0
3	0	100	0	0	0	100	0	0
2	0	100	0	0	0	100	0	0
1	0	100	0	0	0	100	0	0

Table 3.22 Damage state distribution proportional to shear force of vertical elements in dormitory building (%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
6	0	100	0	0	0	100	0	0
5	0	100	0	0	0	100	0	0
4	0	100	0	0	0	64.55	35.45	0
3	0	100	0	0	0	100	0	0
2	0	100	0	0	0	100	0	0
1	0	100	0	0	0	100	0	0

Table 3.23 Damage state distribution of vertical members exceeding the significant damage level in lower and upper sections by shear force ratio in dormitory building (%)

Floor	X Direction		Y Direction	
	LD+SD	AD+C	LD+SD	AD+C
6	100	0	100	0
5	100	0	100	0
4	100	0	87	13
3	100	0	100	0
2	100	0	100	0
1	100	0	100	0

According to the results presented in Tables 3.21-3.23, the criterion if the 6-story dormitory building satisfies the “CD Performance Level” under DD-1 seismic intensity level or not is determined as a result of the checks as stated below.

- Beam Damage Ratio:
Advanced Damage (AD) = %0 $\leq$ %35 **OK!**
Collapse (C) = %0 **OK!**
- Vertical Member Shear Force (V_c) Ratio :
Advanced Damage (AD) = %35.45 $>$ %20 **NOT OK!**
Collapse (C) = %0 **OK!**
- Top Floor Vertical Member Shear Force (V_c) Ratio:
Advanced Damage (AD) = %0 $\leq$ %40 **OK!**
Collapse (C) = %0 **OK!**
- Vertical Element Shear Force (V_c) Ratio Exhibiting Plastic Behavior:
Advanced Damage +Collapse (AD+C) = %13 $\leq$ %30 **OK!**

Since 2 columns on the 4th floor of the building that pass into the "AD State" contribute more than 20% (35.45%) to the total shear force carried on this floor,

"Collapse Prevention" performance level is achieved on the floor. It is evaluated that the structure generally provides the "CD" performance level, since the criterion fails due to 2 structural elements only in the whole structure.



CHAPTER 4

NONLINEAR DYNAMIC ANALYSIS APPROACH IN TBEC (2018)

4.1 Introduction

In Chapter 4, the analysis results which are obtained by using the the nonlinear dynamic analysis are presented for the examined existing structures. Plastic rotations occurring at the ends of the members, interstory drift ratios, damage states and structural performance levels have been shown.

4.2 Case Study 1: 4-Story RC School Building

In this section, 4-story existing school building which is presented in previous chapter is systematically examined with the nonlinear dynamic analysis method in TBEC (2018). First, a numerical analysis model with nonlinear material properties for the school building is developed. Then, a set of strong ground motion records representing the seismic hazard specific to the structure is obtained by using scaling methods. Nonlinear dynamic analyses of the numerical model for the case study building under the effect of biaxial earthquake are conducted and response parameters such as element plastic rotations are obtained. Post-processing routines are developed to compile the results of the nonlinear dynamic analyses and averages plastic rotation values are obtained for all structural members in the structure.

4.2.1 Numerical Modeling of the Case Study Building

A numerical model of the building has been developed by using the inelastic material properties to perform nonlinear dynamic analysis of the structure. The model has been prepared by employing the OpenSees (2021) platform. The OpenSees platform

has been chosen to be used in nonlinear modeling and analysis because it is academically based, verified by studies in the literature, and is a reliable and open-source software. In the three-dimensional numerical model, nonlinear modeling properties are defined for all structural members. It is possible to perform modeling with various options ranging from lumped plasticity elements to fiber section distributed plasticity elements in the OpenSees platform. Lumped plasticity modeling is an approach which is commonly used in structural engineering and finite element analysis to simulate the nonlinear behavior of materials. The material behavior is simplified into discrete plastic hinges at certain locations within the structure. These zones are often placed at critical points where plasticity is expected to occur. Each plastic hinge is typically assumed to exhibit elastic behavior up to a certain yield point, beyond which it becomes perfectly plastic, which means that it can deform without any increase in stress. Unlike more detailed plasticity models, lumped plasticity models do not consider strain hardening, where the material's stiffness changes as it deforms. Stress redistribution between different plastic hinges is not typically considered in lumped plasticity models. Once a hinge yields, it remains at a constant yield stress. Lumped plasticity models are beneficial because they significantly reduce the computational complexity of analyzing structures, making it easier to perform nonlinear analyses. On the other hand, distributed plasticity modeling is an advanced approach used in structural engineering and finite element analysis to simulate the nonlinear behavior of materials and structures. It is a more detailed and realistic method compared to lumped plasticity modeling. Distributed plasticity modeling considers the continuous distribution of plasticity and deformation within the structural elements rather than simplifying the material behavior into discrete plastic hinges. This approach accounts for the gradual transition from elastic to plastic behavior across the entire element. Distributed plasticity models capture the spatial distribution of plastic strains and stresses within structural elements. This means that plastic deformation occurs progressively along the length and across the cross-section of the element. Contrary to the lumped plasticity modeling, distributed plasticity models can account for strain hardening

effects. Strain hardening refers to the phenomenon where the material's stiffness increases as it deforms in the plastic range. Nonlinear behavior models which are used in establishing nonlinear analysis models of the structures are given in Section 5.3 in TBEC (2018). Sections 5.3.1 and 5.3.2 includes detail information about lumped plasticity behavior and distributed plasticity behavior, respectively. Nonlinear models that have been created with the preferred modeling approach give variable results in terms of analysis convergence performance. It is aimed that the non-linear dynamic analyses in this study are intended to be at an optimal level in terms of both result accuracy and convergence. For this purpose, lumped plasticity models have been used in all structural elements. Lumped plasticity elements can be defined as elements with nonlinear cross-section relationship along a certain joint length in the end regions where the highest deformation demands are expected in columns and beams. In this modeling approach, linear elastic section definitions are employed in the remaining region between the end regions. These element models are located under the element type "forceBeamColumn" in the OpenSees library. According to the formulation of the element, the cross-section response is calculated with the two-point Gauss-Radau quadrature in the member end regions determined in proportion to the plastic hinge lengths. In the remaining inner region, two-point Gaussian quadrature is used. With this formulation, the section response can be calculated at a total of 6 integration points within the element. Effective stiffness of structural members modeling with the lumped plasticity approach, are determined according to Section 5.4.5 in TBEC (2018). The nonlinear behavior of the limited plasticity elements has been obtained by defining the moment-curvature relationships along the sections at the plastic hinge regions. Section analyses have been performed for both positive and negative bending directions and moment-curvature relationships of the end regions have been obtained. Moment-curvature relationships in positive and negative directions are represented by bilinear curves, thus, the moment-curvature behavior of the beam end regions is obtained in both directions. These moment curvature relationships are obtained at the end regions of all beam and column members. Differently, the moment-curvature relationships of

the columns have been obtained under the joint effect of the vertical loading states and the earthquake loads calculated by taking $R_a = 4 (G + Q \pm E/R_a)$ in both directions in accordance with TBEC (2018) Chapter 15. These relations are obtained for both principal directions of the columns that are defined in the numerical model. Axial force-bending moment interactions and moment-curvature interactions in two principal directions are neglected during the dynamic action in the columns. This approach has been adopted with the intention of obtaining accurate analysis results with a modeling approach as simple as possible and with high convergence performance.

4.2.2 Selected Strong Ground Motion Records

A set of strong ground motion records has been selected by considering the ground characteristics of the location for the case study building, and the seismic hazard. The parameters of the elastic design spectrum in TBEC (2018) obtained according to the location of the building are shown in Table 4.1. Strong ground motion records have been compiled from the NGA (2011) database. The scaled horizontal components of strong ground motion records are employed in the relevant calculations of the nonlinear dynamic analysis method in TBEC (2018). The ground motion record set which is used in the analyses has been prepared in three stages.

Table 4.1 TBEC (2018) design spectrum parameters for the 4-story school building

S_{DS}	2.28
S_{D1}	0.781
T_A (s)	0.07
T_B (s)	0.34
Soil Type	ZC

Ground motion records from earthquakes which have moment magnitudes (M_w) between 6.5 and 7.0, with a maximum ground acceleration value of (PGA) at least 0.05g, with closest distances to the fault (R_{clst}) between 20 km and 60 km, with a soil

type *C*, have been selected and compiled in the first stage. Ground motion records that are very close to the fault are not included so that the effects of near-field earthquakes do not affect the results of the dynamic analysis. There are 84 strong ground motion pairs from different earthquakes in the pre-set. The scatter plot of the $M_w - R_{clst}$ values of the pre-cluster is presented in Figure 4.1.

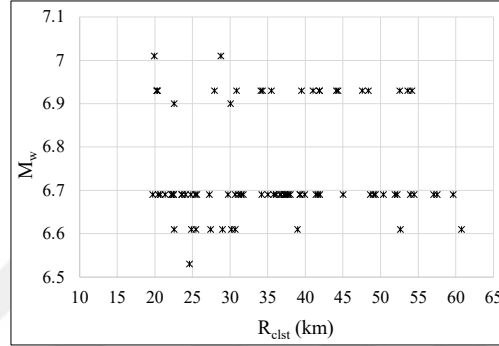


Figure 4.1 Distribution of $M_w - R_{clst}$ values of strong ground motions

In the second stage, an elimination process has been performed on the ground motion cluster based on the employment of the scaling factors obtained according to the TBEC (2018) elastic design spectrum. In the scaling method, the scaling factors have been obtained by taking the ratio of the spectral values between the periods of $0.75T_p$ and $1.25T_p$ (where T_p is the natural vibration period of the building) in the elastic design spectrum and the spectral values between the same periods of the SRSS acceleration response spectra. Considering the dynamic properties of the existing and strengthened states of the 4-story school building, the scaling factors obtained from the ground motions in the pre-set cluster are presented in Figure 4.2, ordered from smallest to largest. The SRSS acceleration response spectra of 84 strong ground motions scaled with the factors calculated according to the dynamic properties of the existing and strengthened structure, along with the target spectra, are given in Figure 4.3. The factors obtained as a result of the scaling processes have been evaluated and strong ground motion pairs for which the scaling factors are not between the limits between 0.70 and 4.5 have been removed from the pre-set. This scaling and filtering process have been applied separately for two different T_p values for the existing state

of the 4-story school building ($T_p = 0.92$ s.) and its strengthened state ($T_p = 0.42$ s.). Then, the filtered clusters have been created for two different dominant vibration periods. Relevant limit values are also marked in Figure 4.2.

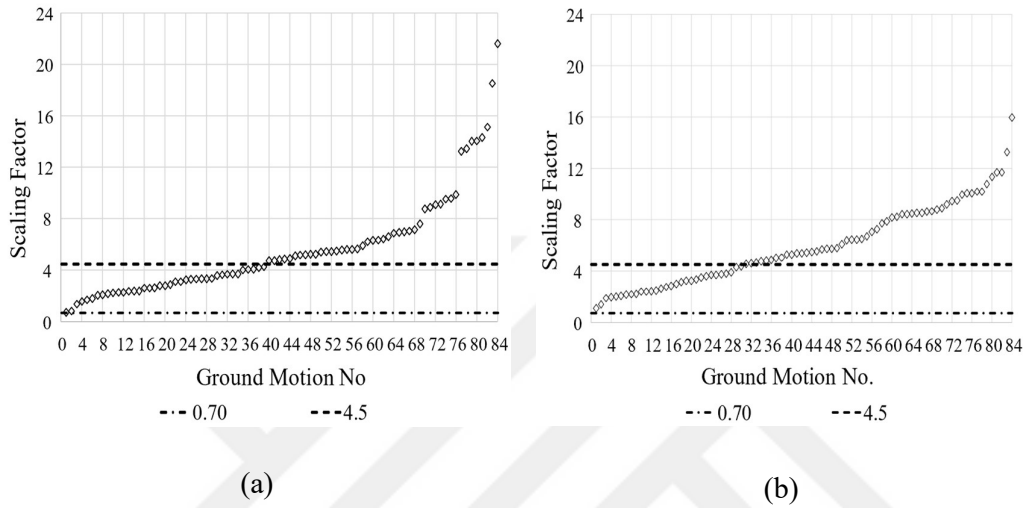


Figure 4.2 The scaling factors obtained according to the TBEC (2018) design spectrum for the 4-story school building in its (a) existing state and (b) strengthened state

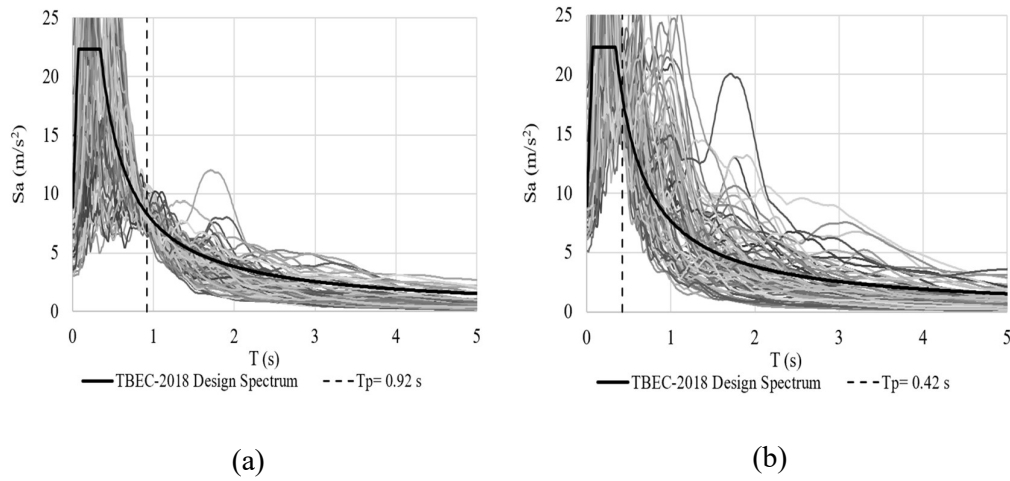


Figure 4.3 SRSS acceleration response spectra of 84 strong ground motion couples scaled according to the (a) existing state (b) strengthened state of the 4-story school building

In the third and final stages, the specific records in the ground motion clusters, which have been determined for both the existing state and strengthened state of the 4-story school building are taken into consideration. Accordingly, a total of 21 strong ground motion pairs have been compiled as clusters that could be used in the analysis. By establishing a common strong ground motion cluster, dynamic analyses of both structures can be performed by changing only the scale factors, and the analysis results under the same earthquake action can be evaluated comparatively. The characteristics of the common records and the scaling factors of both structures are given in Table 4.2.

Table 4.2 Strong ground motion properties and the corresponding scaling factors that can be used in dynamic analysis of the existing and strengthened school building

Earthquake	Year	Station	M_w	Fault Mechanism	Depth (km)	R_{dist} (km)	Soil Type	Scaling Factors for Current Condition	Scaling Factors for Strengthened Condition
Northridge-01	1994	Castaic - Old Ridge Route	6.69	Reverse Fault	17.50	20.72	C	0.72	1.09
Northridge-01	1994	LA - Univ. Hospital	6.69	Reverse Fault	17.50	34.20	C	3.38	1.39
Loma Prieta	1989	Hollister - South & Pine	6.93	Reverse Fault	17.48	27.93	C	0.84	1.88
Northridge-01	1994	Pacific Palisades - Sunset	6.69	Reverse Fault	17.50	24.08	C	2.82	1.96
Loma Prieta	1989	Coyote Lake Dam (SW Abut)	6.93	Reverse Fault	17.48	20.34	C	1.58	2.03
Northridge-01	1994	LA - City Terrace	6.69	Reverse Fault	17.50	36.62	C	3.69	2.17
Loma Prieta	1989	Palo Alto - SLAC Lab	6.93	Reverse Fault	17.48	30.86	C	1.37	2.18
Northridge-01	1994	LA - Chalon Rd	6.69	Reverse Fault	17.50	20.45	C	1.70	2.22
Loma Prieta	1989	Anderson Dam (Downstream)	6.93	Reverse Fault	17.48	20.26	C	2.38	2.38
Northridge-01	1994	Moorpark - Fire Sta	6.69	Reverse Fault	17.50	24.76	C	2.30	2.38
Northridge-01	1994	LA - Fletcher Dr	6.69	Reverse Fault	17.50	27.26	C	2.37	2.41
Northridge-01	1994	LA - Wadsworth VA Hospital South	6.69	Reverse Fault	17.50	23.60	C	1.81	2.61
Northridge-01	1994	LA - Brentwood VA Hospital	6.69	Reverse Fault	17.50	22.50	C	2.07	2.75
Northridge-01	1994	LA - UCLA Grounds	6.69	Reverse Fault	17.50	22.49	C	2.25	2.81
Northridge-01	1994	LA - Wadsworth VA Hospital North	6.69	Reverse Fault	17.50	23.60	C	2.11	3.21
Northridge-01	1994	LA - Cypress Ave	6.69	Reverse Fault	17.50	30.70	C	3.70	3.22
Northridge-01	1994	Manhattan Beach - Manhattan	6.69	Reverse Fault	17.50	39.29	C	3.34	3.71
Northridge-01	1994	LA - Temple & Hope	6.69	Reverse Fault	17.50	31.48	C	2.63	3.78
Loma Prieta	1989	APEEL 7 - Pulgas	6.93	Reverse Fault	17.48	41.86	C	2.65	4.28
Northridge-01	1994	LA - W 15th St	6.69	Reverse Fault	17.50	29.74	C	3.27	4.53
San Fernando	1971	Pasadena - CIT Athenaeum	6.61	Reverse Fault	13.00	25.47	C	3.66	4.61

The SRSS acceleration response spectra of the selected ground motions which are scaled according to the existing structure and strengthened structure dynamic features are presented in Figure 4.4, respectively.

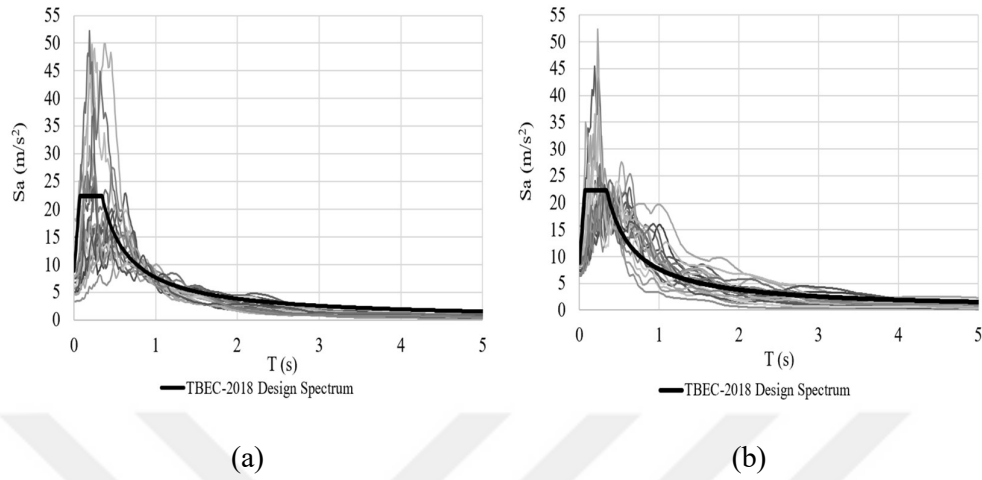


Figure 4.4 SRSS spectra of 21 strong ground motion pairs scaled to be used in analyses according to (a) existing dynamic properties ($T_p = 0.92$ s) and (b) strengthened design properties ($T_p = 0.42$ s) of a four-story school building.

4.2.3 Seismic Performance Evaluation of the School Building at Existing Condition by Using Nonlinear Dynamic Analysis

Nonlinear dynamic analyses are conducted by employing the strong ground motion records to the numerical model of the existing structure with nonlinear properties in both horizontal directions. It should be stated that in some of the analyses under specific strong ground motion pairs in the cluster caused divergence problems have been encountered. Hence these specific strong ground motions are removed from the analysis.

First, the ground motion set mean plastic rotations occurring at the ends of the structural members as a result of dynamic analyses are presented on a frame-by-frame basis. According to the typical floor plan presented in Figure 3.6, considering the X direction of the existing school building, the bottom and top end plastic rotations for the columns on frame A and frame B, which are the outer frame and the inner frame respectively, are presented in Figure 4.5.

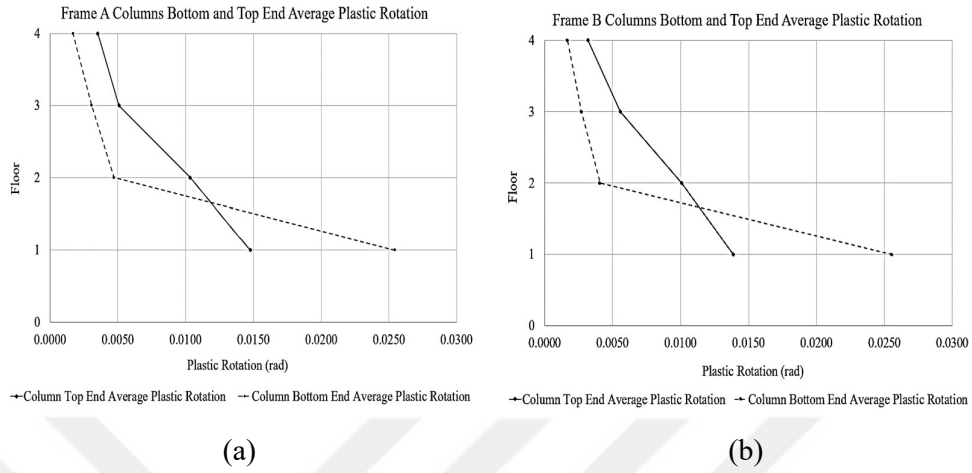


Figure 4.5 Mean plastic rotations for the bottom and top ends of (a) Frame A columns and (b) Frame B columns under the given ground motion set.

The end plastic rotations of Frame A and Frame B beams for the X direction are given in Figure 4.6.

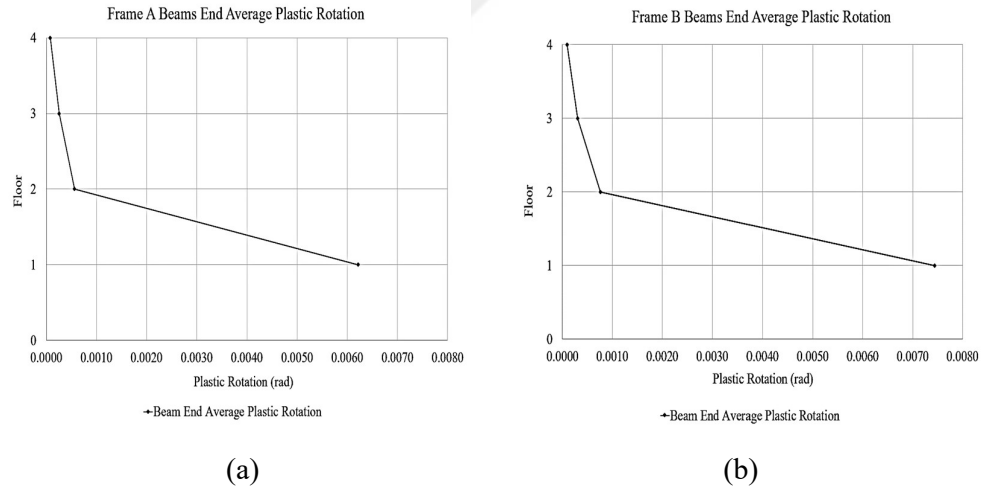


Figure 4.6 Mean plastic rotations for the ends of (a) Frame A beams and (b) Frame B beams under the given ground motion set.

Considering the Y direction of the existing structure, the bottom and top end plastic rotations for the columns of Frame 1 representing the outer frame and Frame 6 representing the inner frame are presented in Figure 4.7.

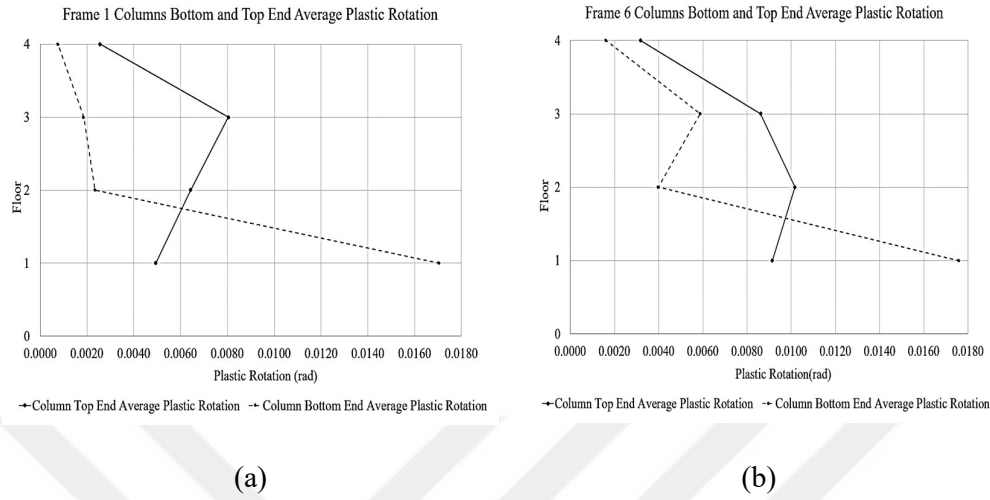


Figure 4.7 Mean plastic rotations for the bottom and top ends of (a) Frame 1 columns and (b) Frame 6 columns under the given ground motion set.

The end plastic rotations of Frame 1 and Frame 6 beams for the Y direction are shown in Figure 4.8.

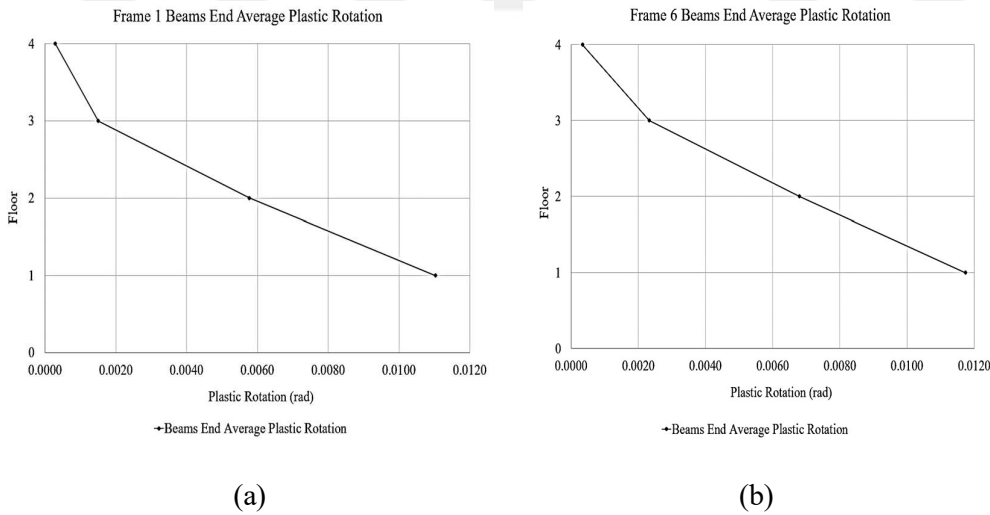


Figure 4.8 Mean plastic rotations for ends of (a) Frame 1 beams and (b) Frame 6 beams under the given ground motion set.

The interstory drift ratios for the X and Y principal directions of the building according to existing state, are also presented in Figure 4.9.

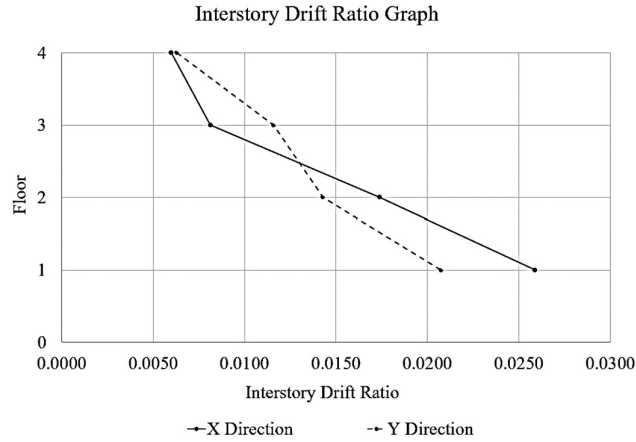


Figure 4.9 Interstory drift ratios for *X* and *Y* directions

As a result of the comparison of the obtained plastic rotation demands with the limit values in TBEC (2018) Section 5.8, the damage states of each structural member are determined. Performance evaluation of all column and beam members in the building is carried out in the same manner. The obtained results are tabulated in accordance with TBEC (2018) Section 15.8. In Table 4.3, the damage percentages in beams are classified according to their damage states and presented on a floor basis. Similarly, in Table 4.4, the shear force distribution of vertical members (%) has been classified according to the damage states and presented on floor basis. In Table 4.5, the shear force distribution of the vertical elements exceeding the SD limit in the lower and upper sections is presented on a story basis.

Table 4.3 Damage state distribution of beam members (%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	0	100	0	0	0	100	0	0
3	0	100	0	0	0	100	0	0
2	0	100	0	0	0	100	0	0
1	0	100	0	0	0	100	0	0

Table 4.4 Damage distribution proportional to shear force of vertical elements (%)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	0	100	0	0	0	100	0	0
3	0	100	0	0	0	100	0	0
2	0	100	0	0	0	100	0	0
1	0	0	0	100	0	0	0	100

Table 4.5 Damage distribution of vertical members exceeding the SD limit in lower and upper sections by shear force ratio (%)

Floor	X Direction		Y Direction	
	LD+SD	AD+C	LD+SD	AD+C
4	100	0	100	0
3	100	0	100	0
2	100	0	100	0
1	0	82	0	85

According to the results of nonlinear dynamic analysis, the existing 4-story school building can not satisfy the “Controlled Damage Performance” target according to TBEC (2018) Section 15.8. The performance of the 4-story existing school building under the nonlinear dynamic analysis is determined to be in “Collapse Performance Level” according to Tables 4.3, 4.4 and 4.5.

4.2.4 Seismic Performance Evaluation of the School Building at Strengthened Condition by Using Nonlinear Dynamic Analysis

Nonlinear dynamic analyses have been conducted by applying the scaled strong ground motion records presented in the previous section to the numerical model of

the strengthened school building in both horizontal directions. Results are compiled from dynamic analyses for strong motion pairs that did not encounter divergence problem during the analyses. The ground motion set mean plastic rotations for the structural elements are given on a frame basis. According to the typical floor plan presented in Figure 3.6 and considering the X direction for the existing structure, the bottom and top end mean plastic rotations for the frame A representing the outer frame and frame B columns are presented in Figure 4.10.

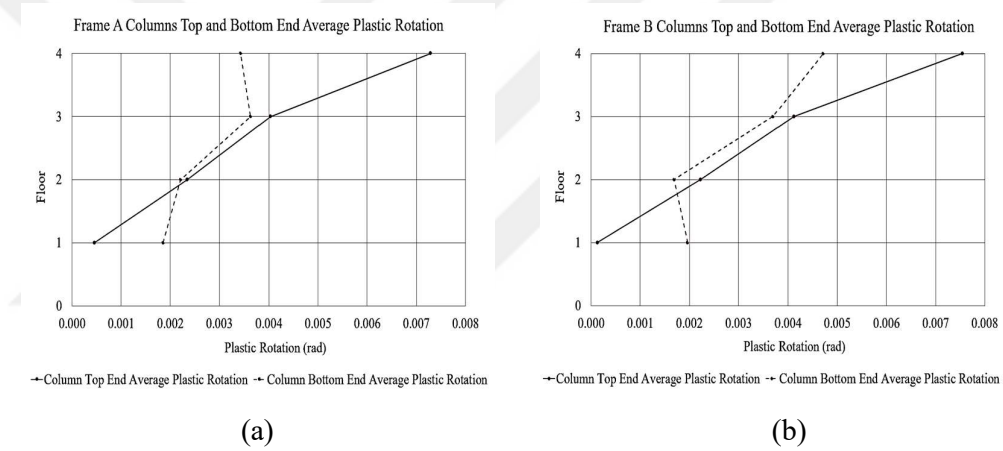


Figure 4.10 Mean plastic rotations for the bottom and top ends of (a) Frame A columns, (b) Frame B columns under the mean values for the ground motion set.

The end plastic rotations of Frame A and Frame B beams for the X direction are given in Figure 4.11.

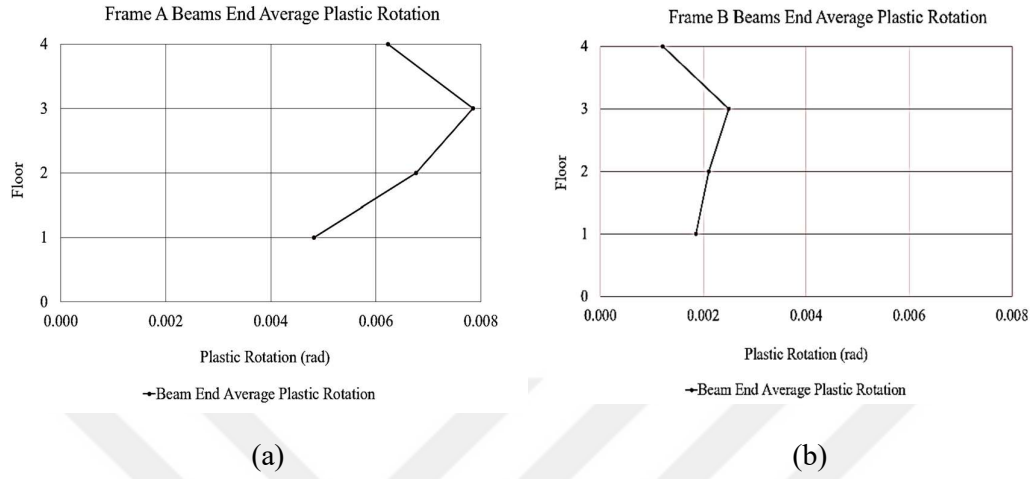


Figure 4.11 Mean plastic rotations for ends of (a) Frame A beams and (b) Frame B beams under the mean values of ground motion set.

Considering the Y direction of the strengthened structure, the bottom and top end plastic rotations for the columns of outer Frame 1 and inner Frame 6 are presented in Figure 4.12.

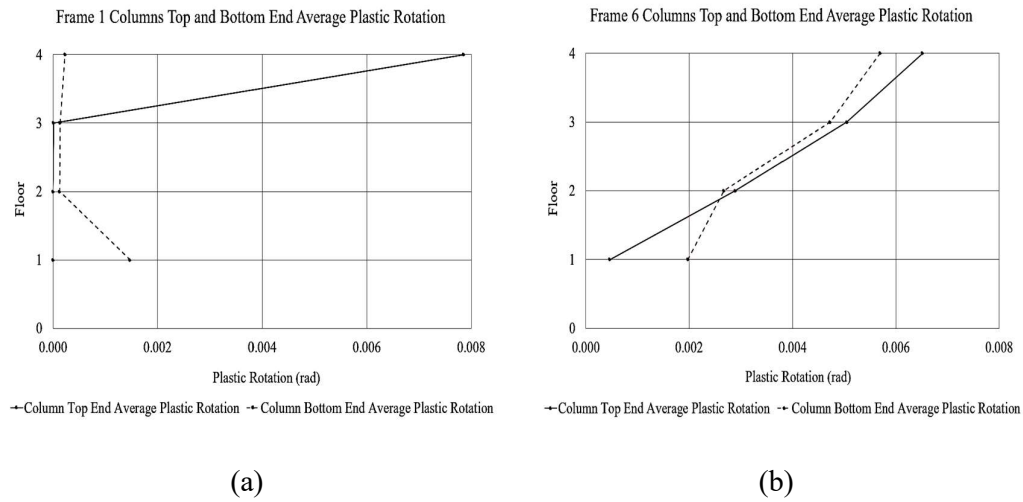


Figure 4.12 Mean plastic rotations for the bottom and top ends of (a) Frame 1 columns and (b) Frame 6 columns under the mean values for the ground motion set.

The end plastic rotations of Frame 1 and Frame 6 beams for the Y direction are given in Figure 4.13.

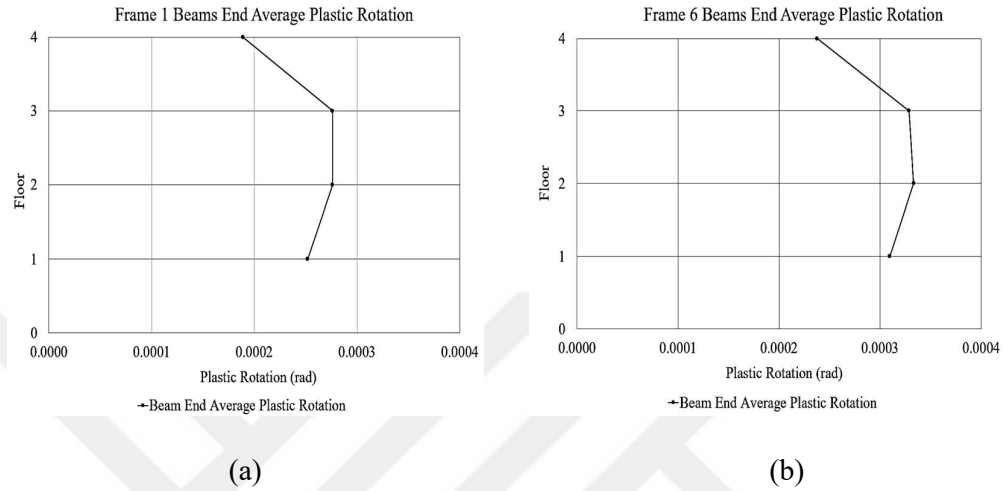


Figure 4.13 Mean Plastic rotations for ends of (a) Frame 1 beams and (b) Frame 6 beams under the mean values of the ground motion set.

The ground motion set mean plastic rotations calculated in the shear walls as a result of dynamic analyses are given in Figure 4.14. Considering the *X* direction of the strengthened structure, the plastic rotations for the outer Frame A shear walls are presented in Figure 4.14. Plastic rotation demands on shear walls in the *Y* direction are not calculated.

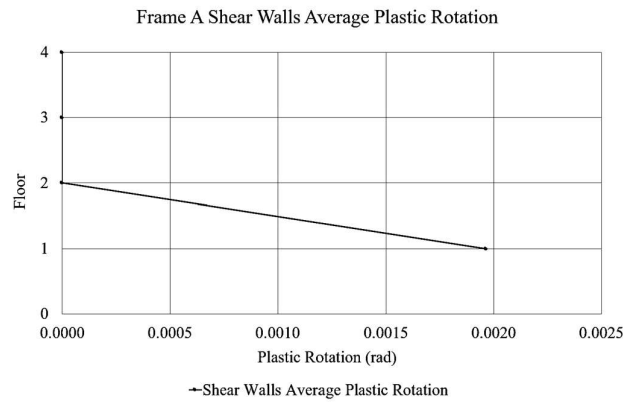


Figure 4.14 Mean plastic rotations for Frame A shear walls under the mean values of the ground motion set.

The interstory drift ratios for the X and Y directions of the building according to strengthened state, are also presented in Figure 4.15.

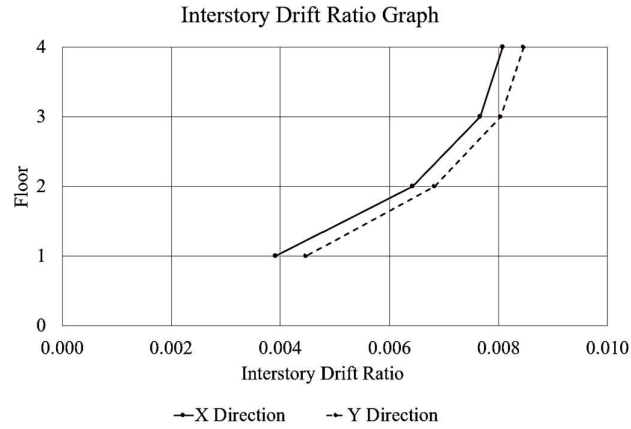


Figure 4.15 Interstory drift ratios for X and Y directions of the building

As a result of the comparison of the plastic rotation demands obtained from the nonlinear dynamic analysis of the strengthened structure with the limit values in TBEC (2018) Section 5.8, the damage states of each structural member are determined. In Table 4.6, the damage percentages in beams are classified according to their damage and presented on a floor basis. Similarly, in Table 4.7, the shear force distribution of vertical elements (%) is presented on a floor basis by classifying them according to their damage levels.

Table 4.6 Damage distribution of beam members in strengthened structure (in %)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	0	100	0	0	0	100	0	0
3	0	100	0	0	0	100	0	0
2	0	100	0	0	0	100	0	0
1	0	100	0	0	0	100	0	0

Table 4.7 Damage distribution proportional to shear force of vertical elements in strengthened structure (in %)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
4	69	31	0	0	66	34	0	0
3	84	16	0	0	81	19	0	0
2	86	14	0	0	84	16	0	0
1	0	100	0	0	91	9	0	0

Table 4.8 Damage distribution of vertical members exceeding the significant damage limit in lower and upper sections by shear force ratio in strengthened structure (in %)

Floor	X Direction		Y Direction	
	LD+SD	AD+C	LD+SD	AD+C
4	100	0	100	0
3	100	0	100	0
2	100	0	100	0
1	100	0	100	0

According to the results, very low plastic demands occur in the strengthened 4-story existing school building and the added shear walls provide a significant performance improvement. In the light of nonlinear dynamic analyses with strong ground motions compatible with the DD-1 seismic intensity level, the “CD Performance” target determined for the strengthened 4-story existing school building is achieved in accordance with TBEC (2018) Section 15.8. The performance of the structure is determined as “Controlled Damage” in accordance with Tables 4.6, 4.7 and 4.8.

4.3 Case Study 2: 6-Story Dormitory Building

Nonlinear dynamic analyses are carried out by applying horizontal components of strong ground motions to the numerical model of the existing 6-story dormitory building. The strong ground motion set presented in the previous section is used in dynamic analysis of Case Study 2 building, as well. It is aimed that the strong ground motion records are compatible with the spectral values of earthquake intensity. For this purpose, the basic period of the structure ($T_X = T_p = 0.88$ s) is considered to perform the scaling process presented in the previous section. The main period of the building, $T_p = 0.88$ s, is very close to the main period of the existing school building ($T_{p_school} = 0.91$ s) examined as the first case study. For this reason, the scale values obtained as a result of the scaling process for the school building and the associated strong ground motions are also used in the nonlinear dynamic analyses of the 6-story dormitory building. The mentioned strong ground motion list and scale factors are presented in Table 4.2 in the previous sections. In the light of the facts discussed above, the "Scaling Factors for the school Building" given in Table 4.2 are used for the nonlinear dynamic analysis of the existing dormitory building.

4.3.1 Seismic Performance Evaluation of the 6-Story Dormitory Building by Using Nonlinear Dynamic Analysis

No divergence problem is encountered in the dynamic analyses made with the application of strong ground motion pairs in two principal directions. The ground motion set mean plastic rotations of the end plastic rotations of the structural members by using the ground motion set are calculated. The summary results which are compiled on the basis of floors for the two principal directions of the dormitory building are given in Tables 4.9- 4.11. According to these results, the dormitory building satisfies the "CD" performance target. The plastic rotation demands calculated separately for each structural member are presented in the next section in comparison with the results of the linear elastic method.

Table 4.9 Damage distribution of beam members in dormitory building (in %)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
6	52	48	0	0	62.07	37.93	0	0
5	8	92	0	0	0	100	0	0
4	8	92	0	0	0	100	0	0
3	8	92	0	0	0	100	0	0
2	4	96	0	0	0	100	0	0
1	0	100	0	0	3.45	96.55	0	0

Table 4.10 Damage distribution proportional to shear force of vertical elements in dormitory building (in %)

Floor	X Direction				Y Direction			
	LD	SD	AD	C	LD	SD	AD	C
6	9.55	90.45	0	0	18.53	81.47	0	0
5	37.84	62.16	0	0	36.19	63.81	0	0
4	29.2	54.97	0	0	8.44	57.53	0	0
3	47.73	52.27	0	0	48.28	51.72	0	0
2	58.9	41.1	0	0	80.58	19.42	0	0
1	0	100	0	0	0	100	0	0

Table 4.11 Damage distribution of vertical members exceeding the significant damage limit in lower and upper sections by shear force ratio in strengthened structure (in %)

Floor	X Direction		Y Direction	
	LD+SD	AD+C	LD+SD	AD+C
6	100	0	100	0
5	100	0	100	0
4	100	0	100	0
3	100	0	100	0
2	100	0	100	0
1	100	0	100	0

CHAPTER 5

COMPARISON OF TBEC (2018) LINEAR ELASTIC METHOD AND NON-LINEAR DYNAMIC ANALYSIS

5.1 Introduction

In Chapter 5, the analysis results which are obtained by using the linear elastic analysis method in TBEC (2018) and the nonlinear dynamic analysis are compared for the examined existing structures. In the comparisons, plastic rotations occurring at the ends of the members, interstory drift ratios, damage states and structural performance levels have been presented.

5.2 Comparison of Analysis Results for the Existing Condition of School Building

First of all, the plastic rotation demands obtained by the two analysis methods are compared on a frame-by-frame basis. The bottom and top end plastic rotations for the Frame A columns are compared in the graphs presented in Figure 5.1 for the TBEC (2018) linear elastic analysis method and the nonlinear dynamic analysis method. Likewise, the bottom and top end plastic rotation values of the Frame B columns are presented in the graphs given in Figure 5.2.

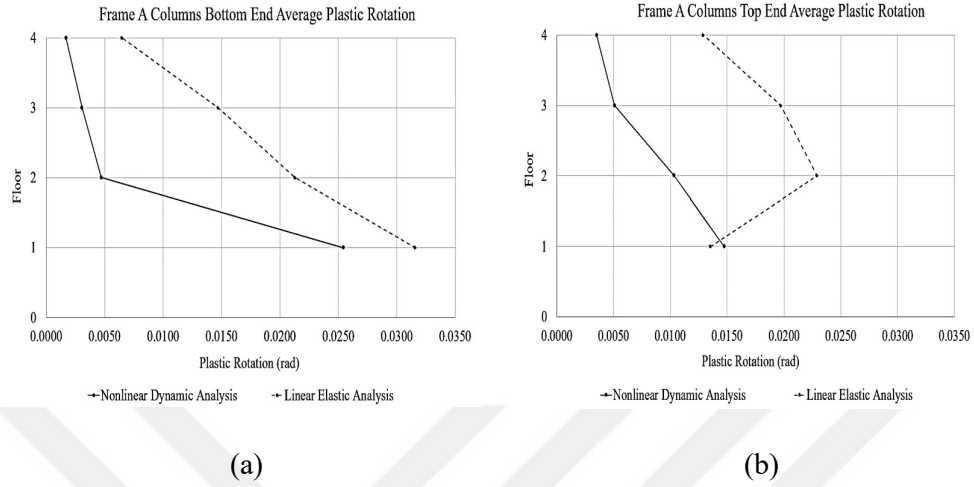


Figure 5.1 Comparison of (a) bottom end and (b) top end plastic rotations for frame A columns.

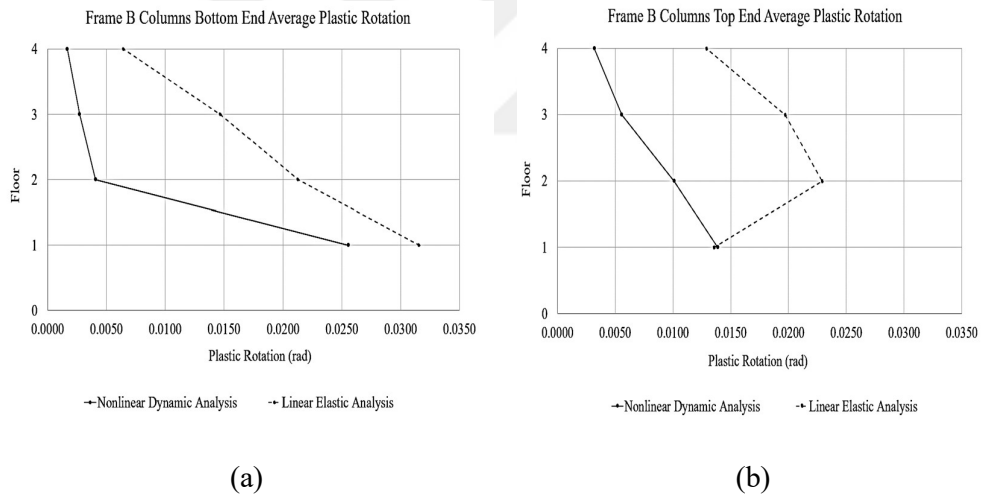
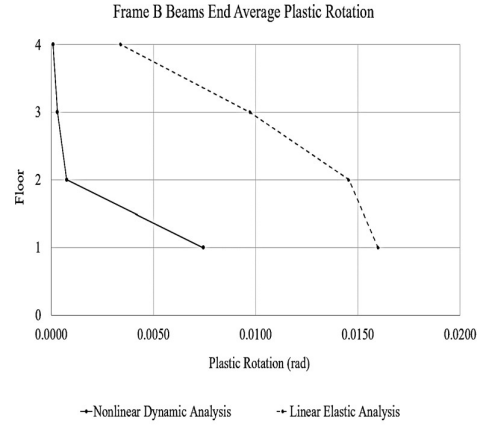
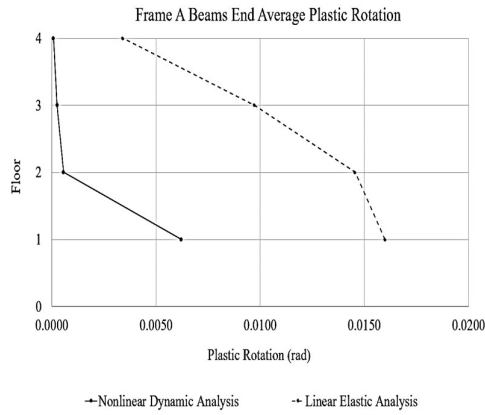


Figure 5.2 Comparison of (a) bottom end and (b) top end plastic rotations for frame B columns.

End plastic rotations of Frame A and Frame B beams are given comparatively with the graph presented in Figure 5.3 for TBEC (2018) linear elastic analysis method and nonlinear dynamic analysis method.

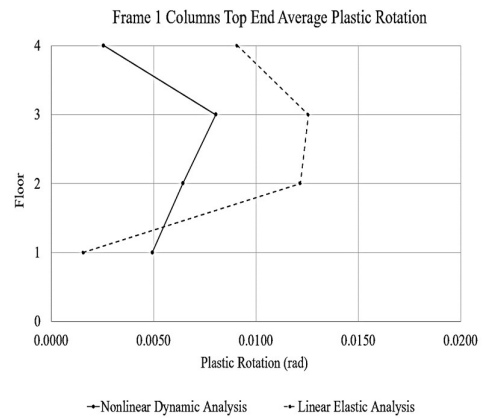
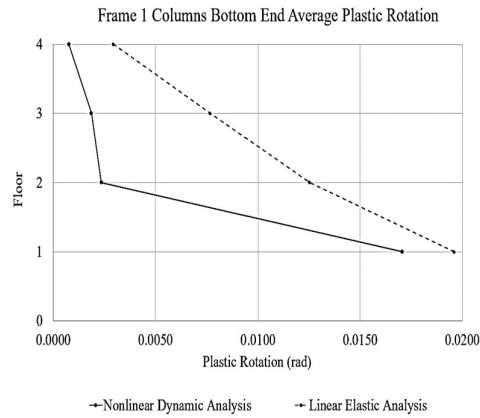


(a)

(b)

Figure 5.3 Comparison of end plastic rotations for (a) Frame A and (b) Frame B beams.

The bottom and top end plastic rotations for the columns of Frames 1 and 6 are compared in Figures 5.4 and 5.5 for the TBEC (2018) linear elastic analysis method and the nonlinear dynamic analysis method.



(a)

(b)

Figure 5.4 Comparison of (a) bottom end and (b) top end plastic rotations for columns of Frame 1.

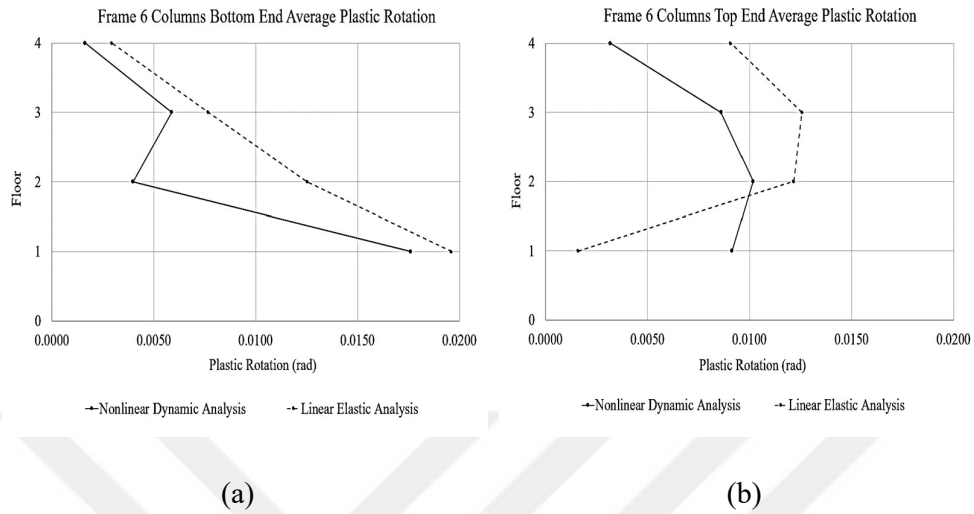


Figure 5.5 Comparison of (a) bottom end and (b) top end plastic rotations for columns of Frame 6.

End plastic rotations for beams of Frames 1 and 6 are given comparatively with the graph presented in Figure 5.6 for TBEC (2018) linear elastic analysis method and nonlinear dynamic analysis method.

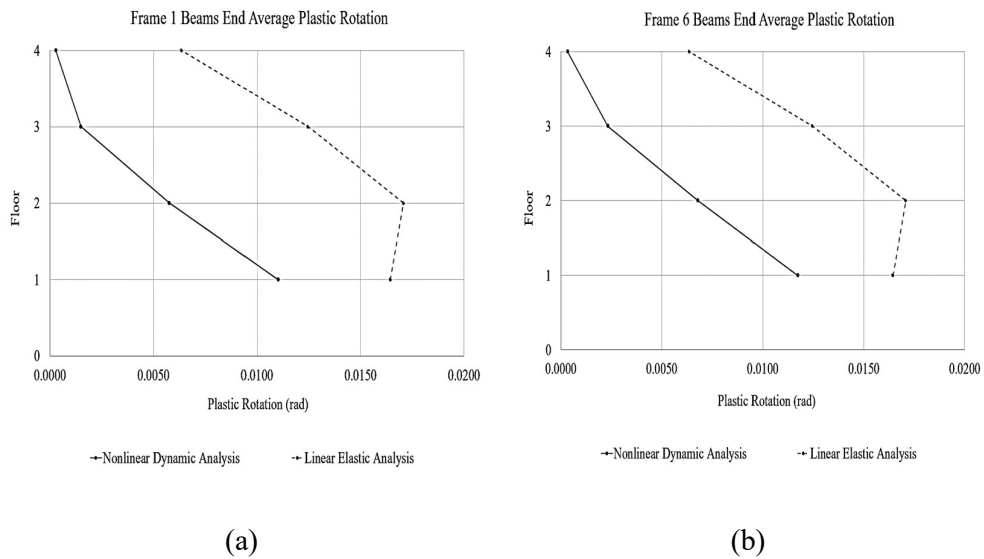


Figure 5.6 Comparison of end plastic rotations for (a) Frame 1 and (b) Frame 6 beams.

Interstory drift ratio plots obtained by using TBEC (2018) linear elastic analysis method and nonlinear dynamic analysis method are presented in Figure 5.7 by comparing them separately for X and Y directions.

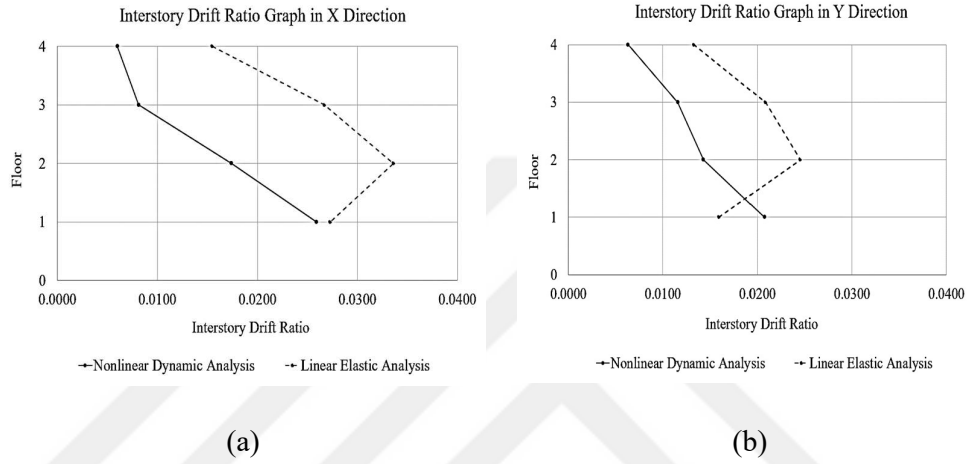


Figure 5.7 Comparison of interstory drift ratios in (a) X direction and (b) Y direction for the existing structure.

Correlation graphs of mean plastic rotations obtained from TBEC (2018) linear elastic analysis method and nonlinear dynamic analyses are also obtained for each structural member. The plastic rotation demands of the 4-story existing school building, which occur at the bottom and top ends of the columns under the X -direction earthquake motion, are shown for each floor by using both analysis methods and they are presented in Figures 5.8 and 5.9.

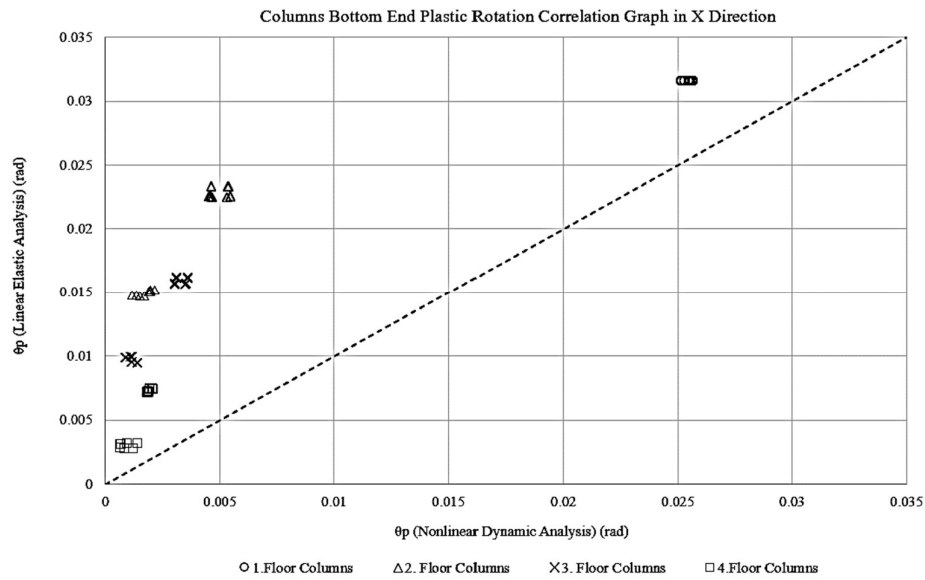


Figure 5.8 Correlation of bottom end plastic rotations for columns in the X -direction for both analysis methods

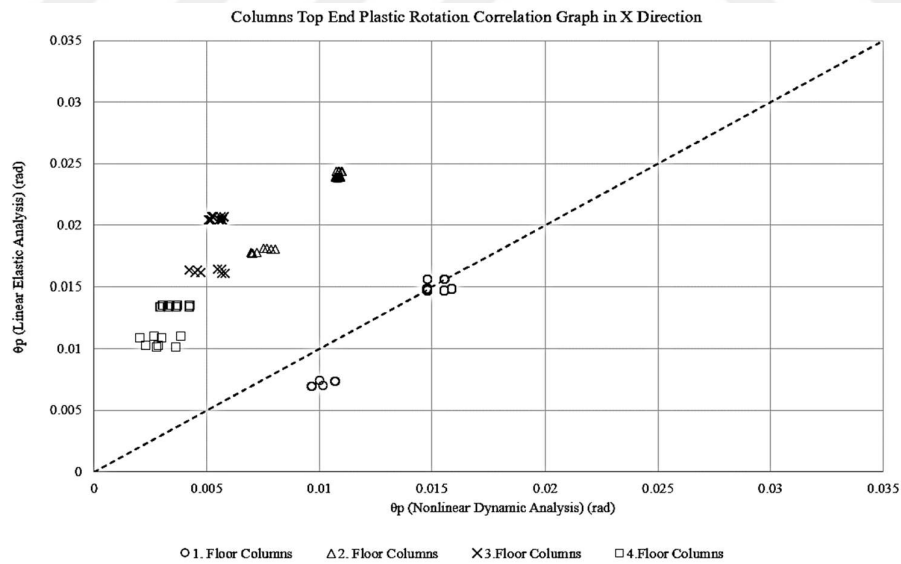


Figure 5.9 Correlation of top end plastic rotations for columns in the X -direction for both analysis methods

For X -direction beams, the correlation of end plastic rotations obtained by using both analysis methods are presented in Figure 5.10.

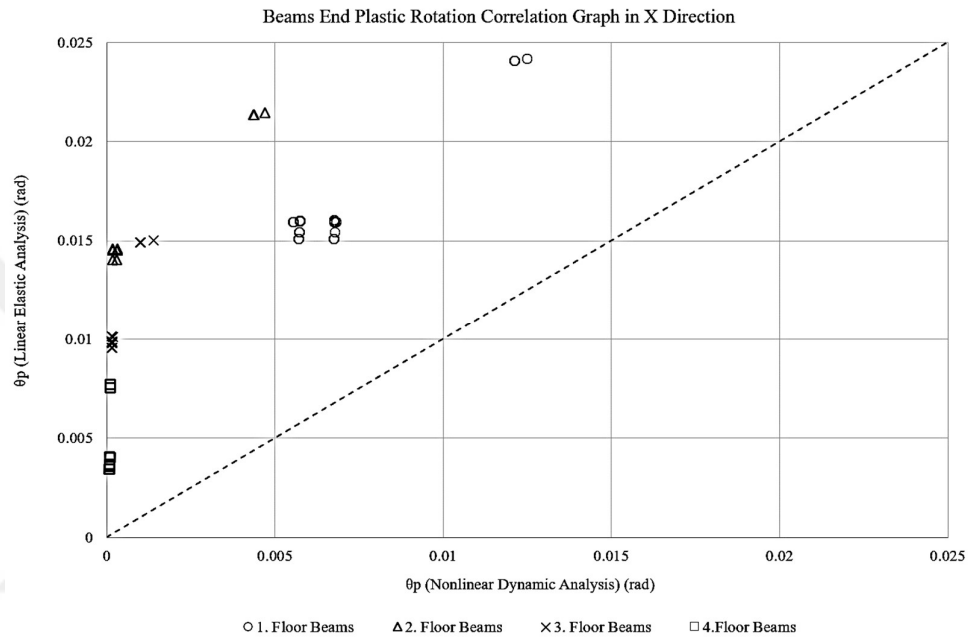


Figure 5.10 Correlation of plastic rotations at the ends of the beams in the X direction.

The plastic rotation demands of the existing structure, which occur at the bottom and top ends of the columns under the influence of ground motion in the Y -direction, are shown for each floor by using both analysis methods, and they are presented in Figures 5.11 and 5.12.

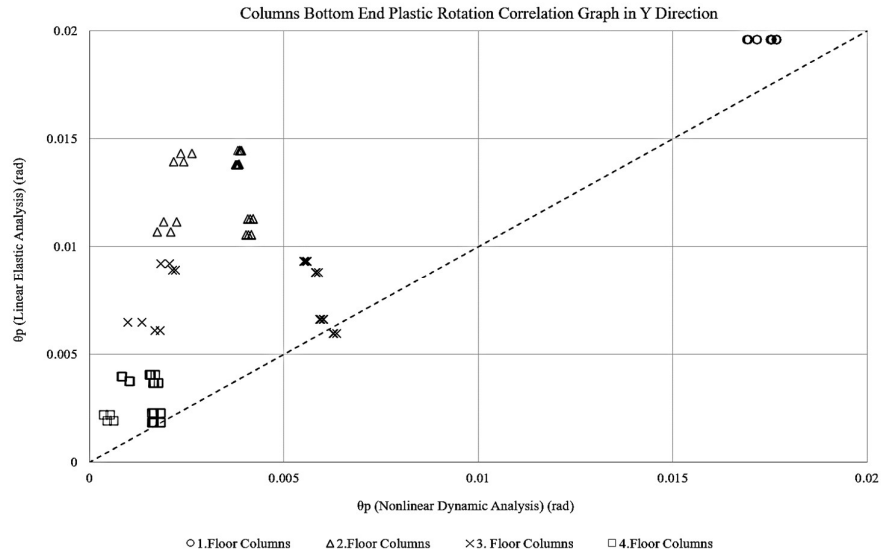


Figure 5.11 Correlation of bottom end plastic rotations for columns in the Y-direction for both analysis methods

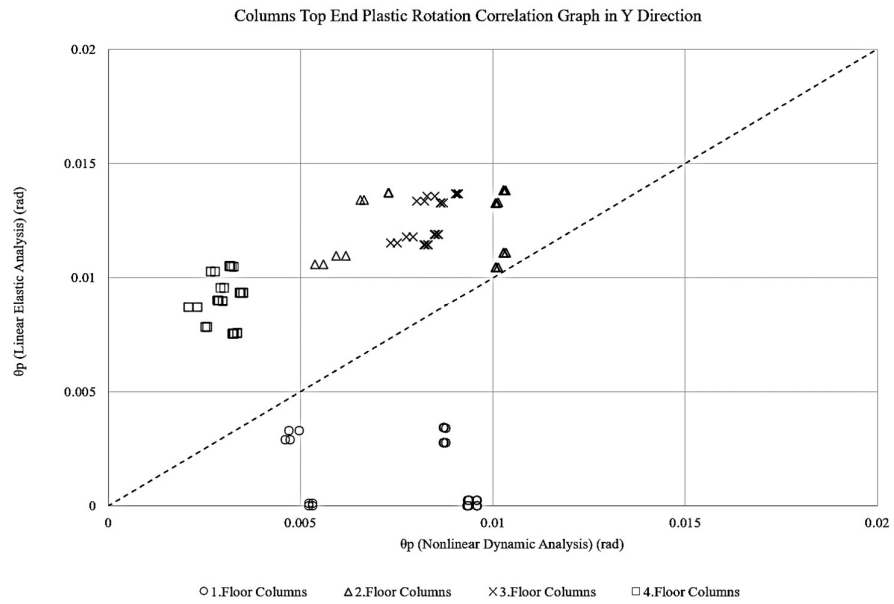


Figure 5.12 Correlation of top end plastic rotations for columns in the Y-direction for both analysis methods

For Y-direction beams, the correlation of end plastic rotations obtained by using both analysis methods have been presented in Figure 5.13.

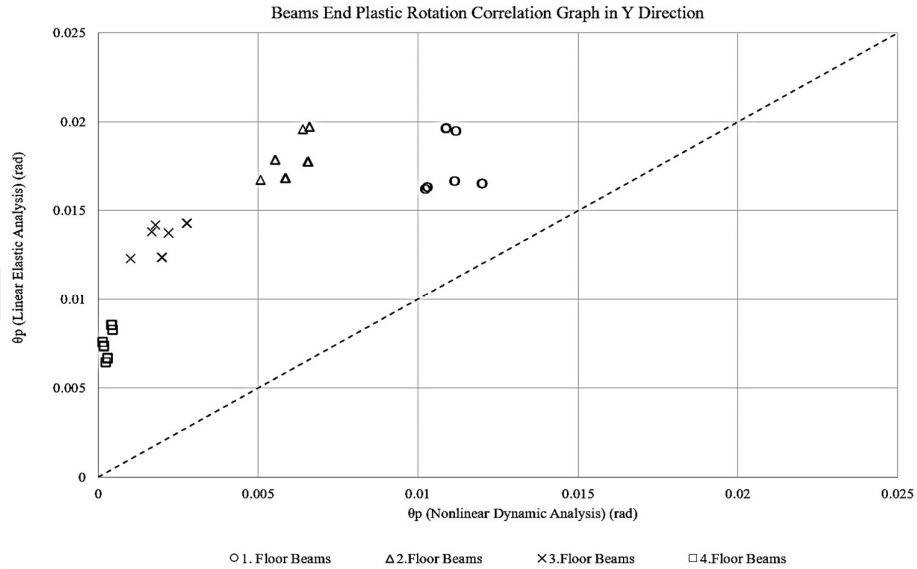


Figure 5.13 Correlation of plastic rotations at the ends of the beams in the Y direction

5.3 Comparison of Analysis Results for the Strengthened Condition of the School Building

In this section, the performance of the strengthened school building obtained by using the linear elastic analysis method of TBEC (2018) is compared with the nonlinear dynamic analysis results. The plastic rotation demands obtained by the two analysis methods are compared on a frame-by-frame basis. The bottom end plastic rotations of columns in Frames A and B are given in Figure 5.14 for TBEC (2018) linear elastic analysis method and nonlinear dynamic analysis method.

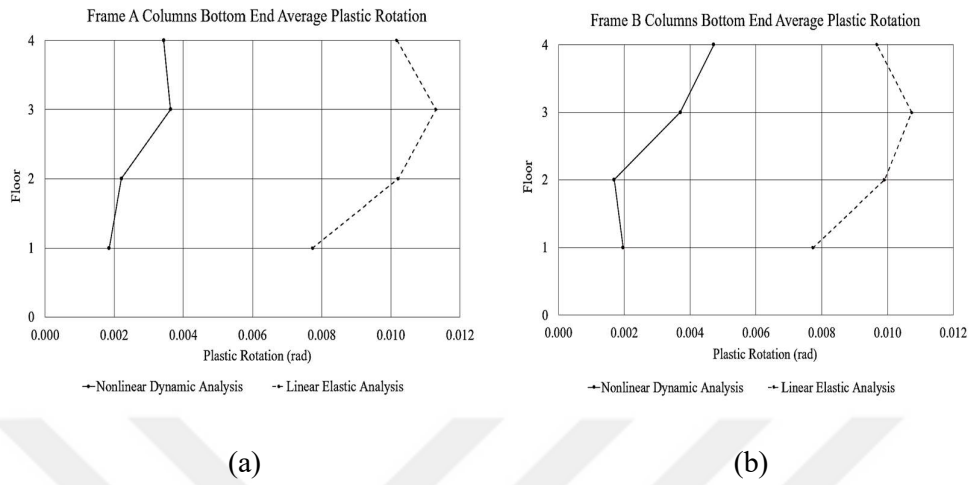


Figure 5.14. Comparison of bottom end plastic rotations for columns in (a) Frame A and (b) Frame B

End plastic rotations of beams in Frames A and B are given comparatively with the graph presented in Figure 5.15.

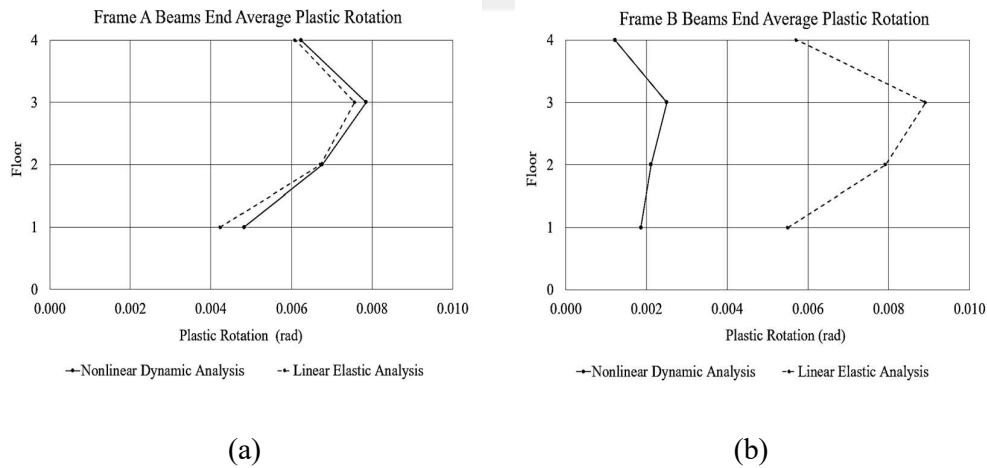
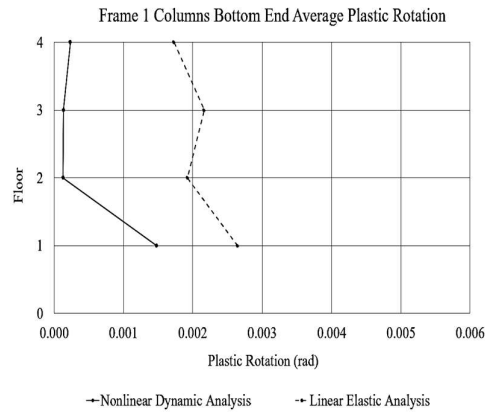
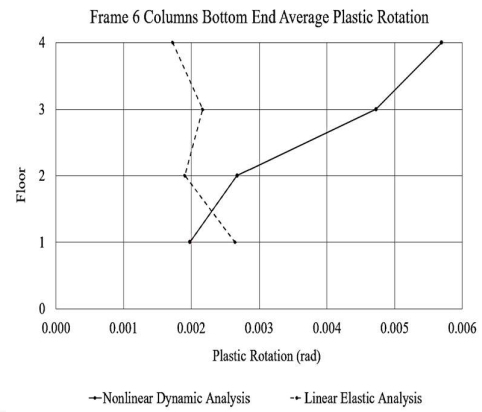


Figure 5.15 Comparison of end plastic rotations for beams in (a) Frame A and (b) Frame B

The bottom end plastic rotations for columns in Frames 1 and 6 are compared in Figure 5.16 for the TBEC (2018) linear elastic analysis method and the nonlinear dynamic analysis method.



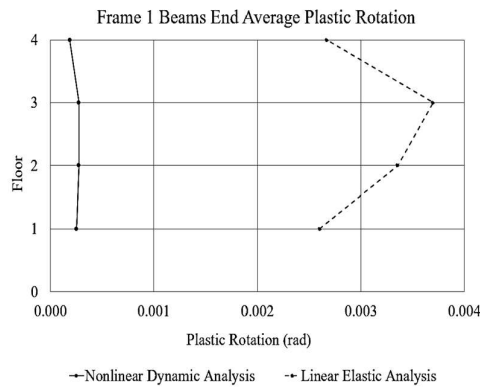
(a)



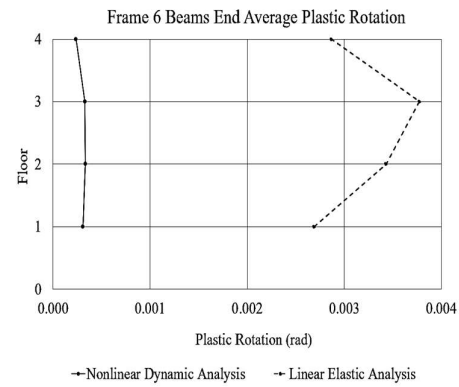
(b)

Figure 5.16 Comparison of bottom end plastic rotations for columns in (a) Frame 1 and (b) Frame 6

The end plastic rotations for beams in Frames 1 and 6 are compared in Figure 5.17 for the TBEC (2018) linear elastic analysis method and the nonlinear dynamic analysis method.



(a)



(b)

Figure 5.17 Comparison of end plastic rotations for beams in (a) Frame 1 and (b) Frame 6

The plastic rotations calculated by TBEC (2018) linear elastic analysis method and nonlinear dynamic analysis methods of the shear walls which are located in Frame

A are given together in Figure 5.18. This comparison is not made for Frame 2 shear walls, since plastic rotation demands are not observed in the evaluations made with both methods in the analyses on the *Y* direction of the building.

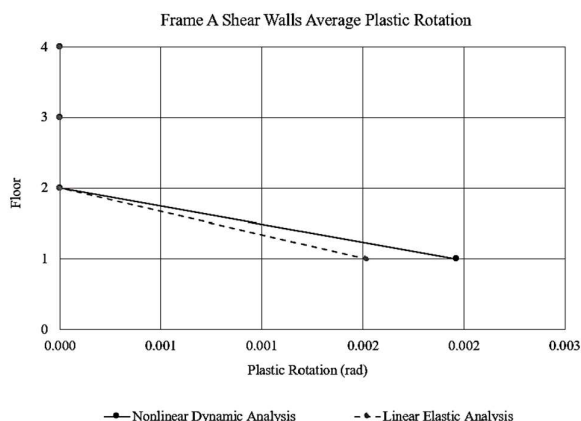


Figure 5.18 Comparison of plastic rotations for Frame A shear walls

Detailed examinations of the 4-story existing school building are completed with these results. When the results are considered in general, it is observed that the plastic rotation demands are calculated slightly higher by the linear elastic method. In the light of these findings, the performance of the linear elastic method regarding dynamic behavior prediction is discussed in detail in Chapter 6.

5.4 Comparison of Analysis Results for the Existing 6-Story Dormitory Building

In this section, the plastic rotation values calculated by both analysis methods in the structural members of the existing 6-story dormitory building are compared with each other and also with the limit values of plastic rotation. The end plastic rotation demands obtained from the nonlinear dynamic analysis are compiled as the average values obtained from the ground motion set. Due to the more complex plan geometry of the dormitory building compared to the school building, a different way of showing the results is followed. Instead of the average results on the frame basis, the

calculated plastic rotations of the columns, shear walls and beams in each floor are presented in a graphical way. According to the floor plan of the existing dormitory building presented in Figure 3.23, the plastic rotation demands calculated by the TBEC (2018) linear elastic method and non-linear dynamic analysis at the bottom ends of the columns in the X direction are presented in Figure 5.19 for each floor. Plastic rotation limits calculated according to TBEC (2018) for these members are also given in Figure 5.19.

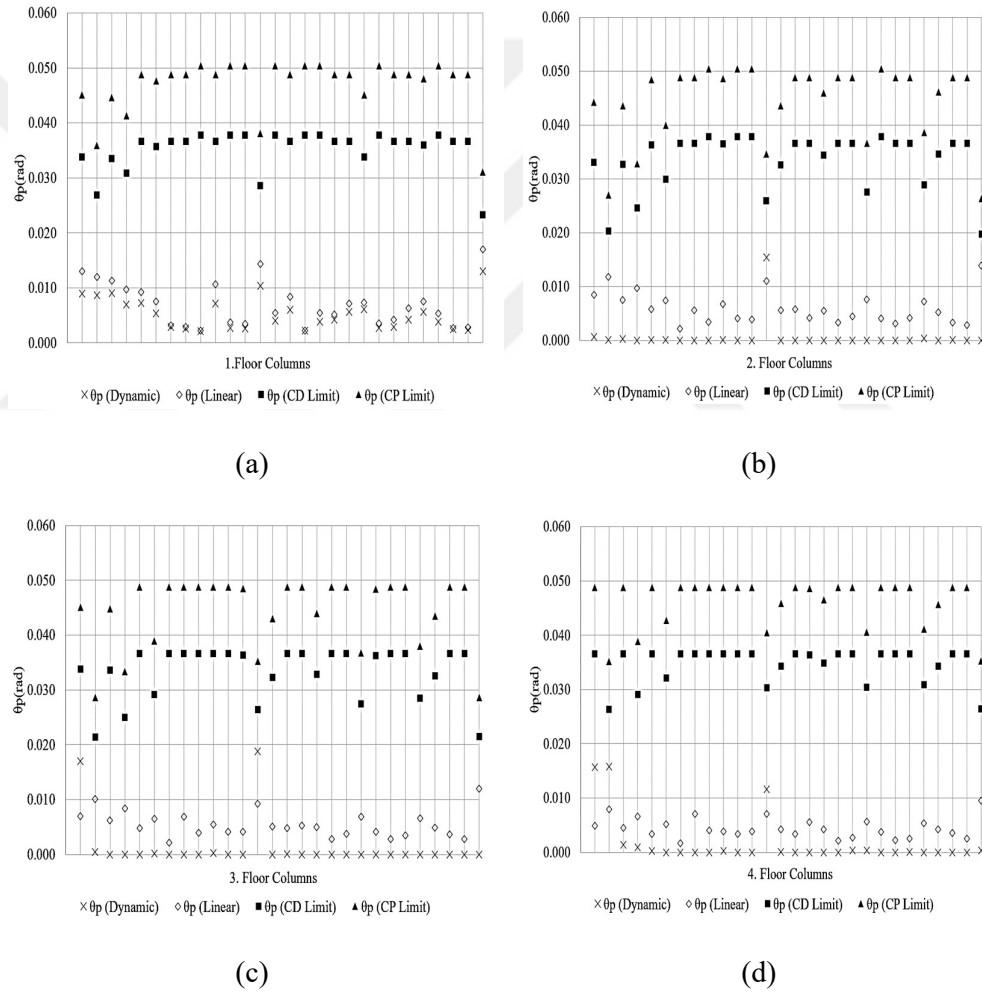
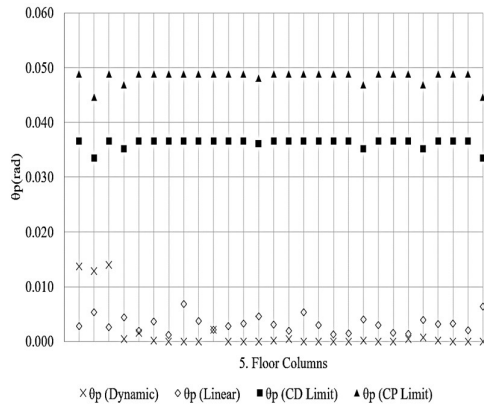
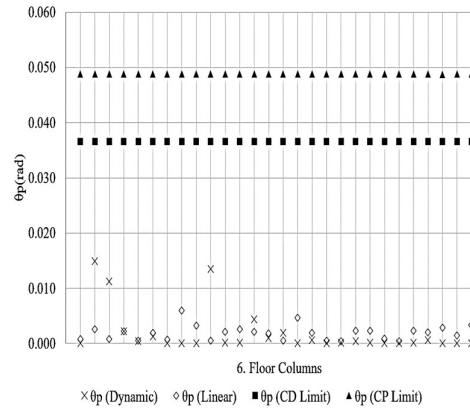


Figure 5.19 Comparison of the bottom end plastic rotations of the (a) 1st floor, (b) 2nd floor, (c) 3rd floor, (d) 4th floor, (e) 5th floor and (f) 6th floor columns calculated in the X direction



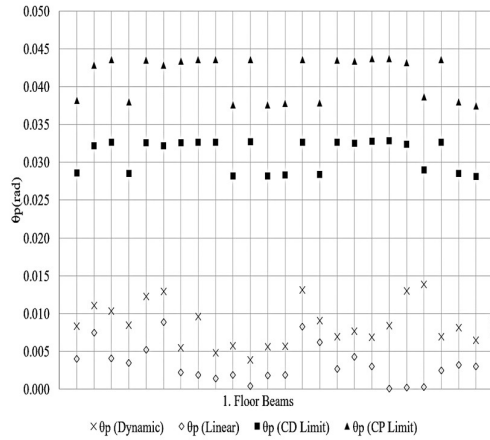
(e)



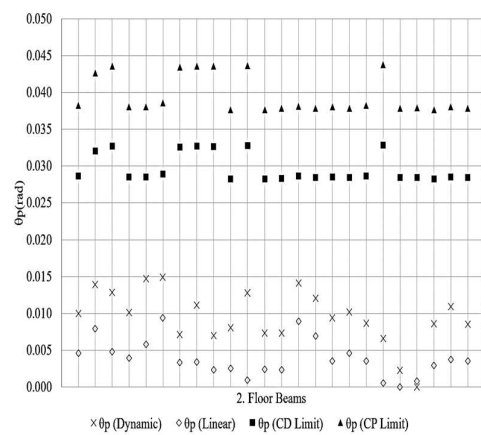
(f)

Figure 5.19 (Continued)

The plastic rotation demands calculated by TBEC (2018) linear elastic method and nonlinear dynamic analysis at the beam ends in the X principal direction of the dormitory building are also presented together with the plastic rotation limits in Figure 5.20.

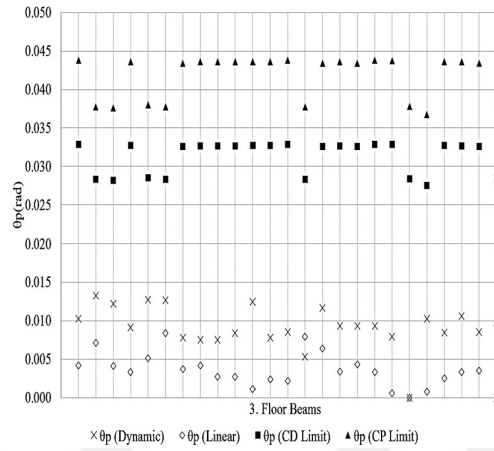


(a)

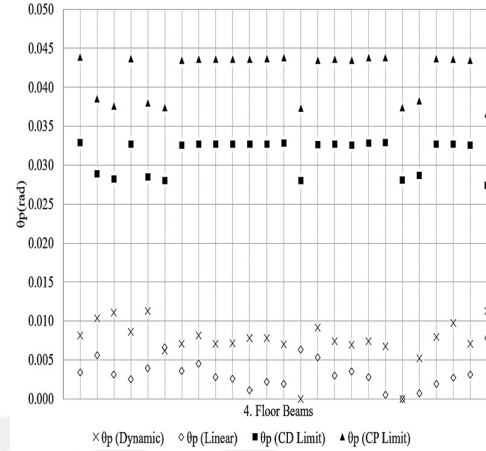


(b)

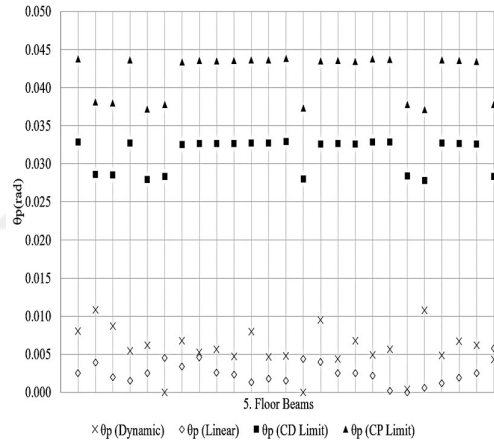
Figure 5.20 Comparison of the end plastic rotations of the (a) 1st floor, (b) 2nd floor, (c) 3rd floor, (d) 4th floor, (e) 5th floor and (f) 6th floor beams calculated in the X direction



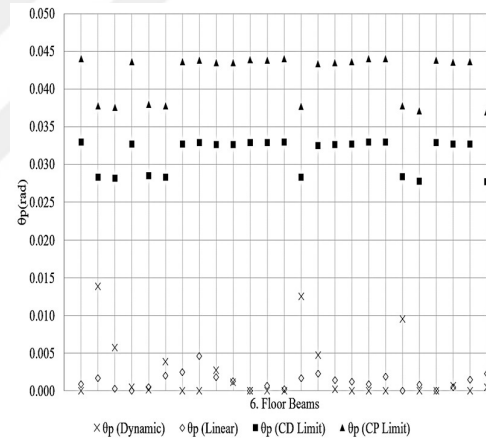
(c)



(d)



(e)



(f)

Figure 5.20 (Continued)

The plastic rotational demands calculated by TBEC (2018) linear elastic method and nonlinear dynamic analysis at the bottom end of the columns and beams for the Y direction of the building are given in Figures 5.21 and 5.22, respectively, with the plastic rotation limits for all floors.

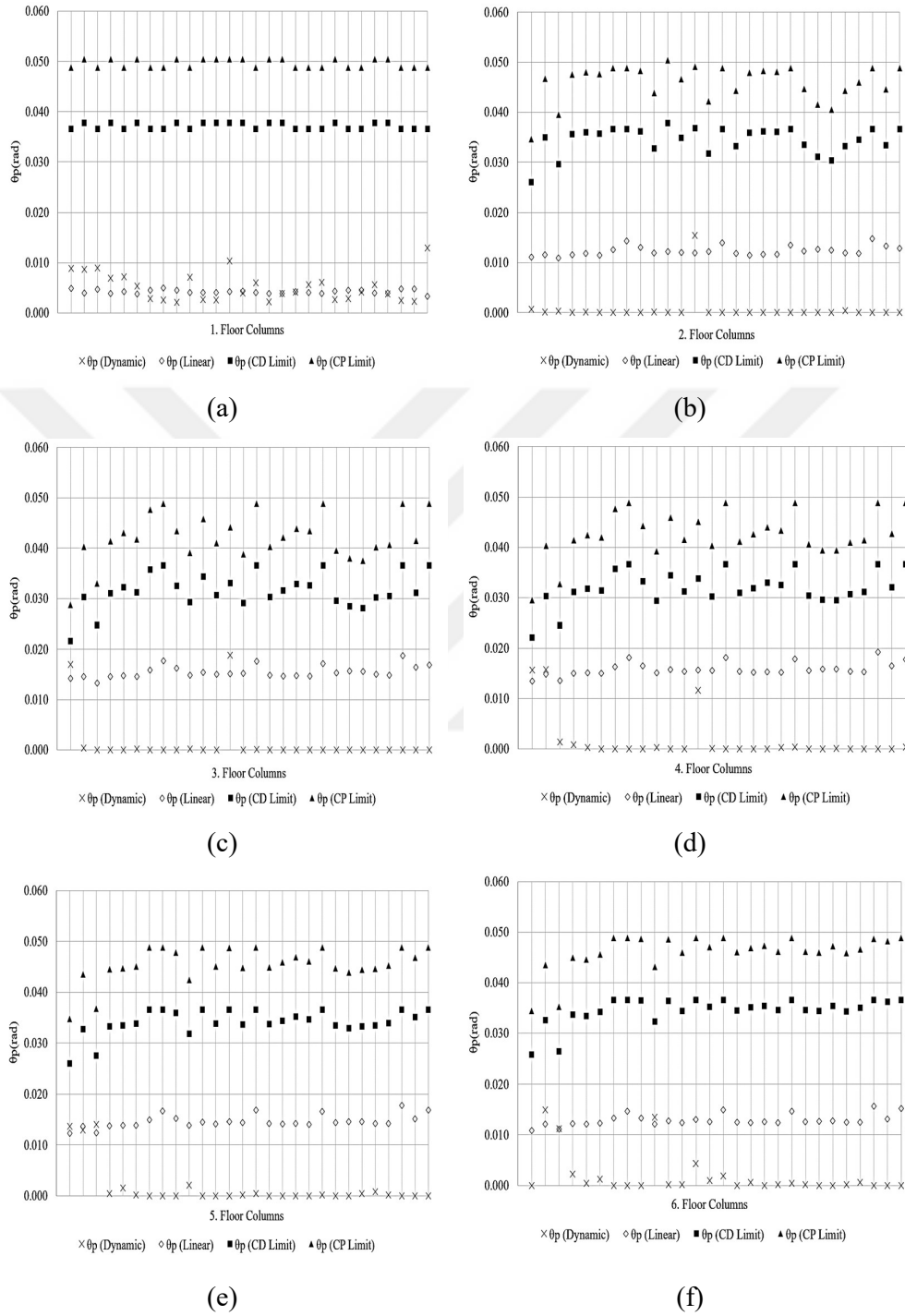
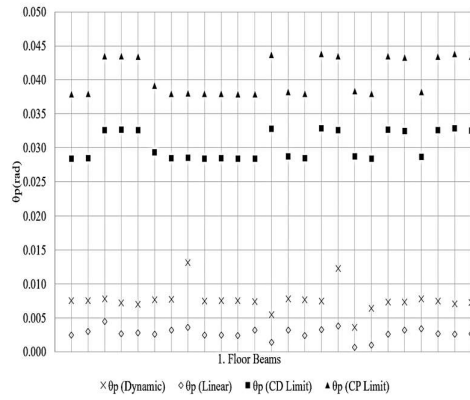
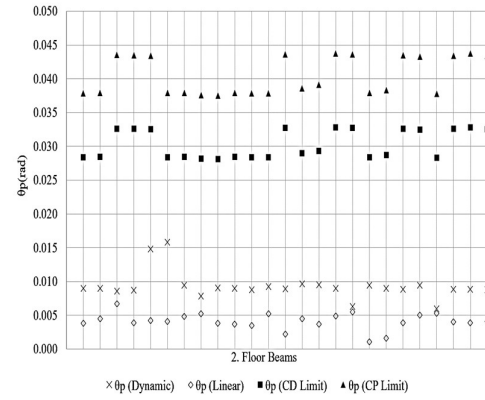


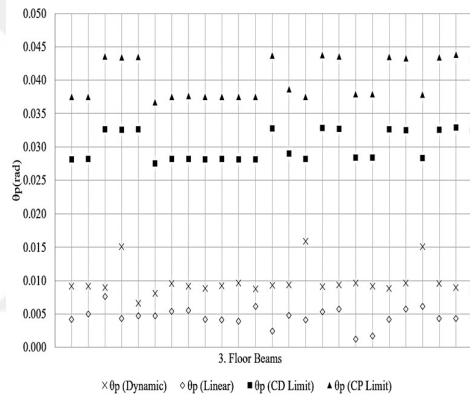
Figure 5.21 Comparison of the bottom end plastic rotations of the (a) 1st floor, (b) 2nd floor, (c) 3rd floor, (d) 4th floor, (e) 5th floor and (f) 6th floor columns calculated in the Y direction



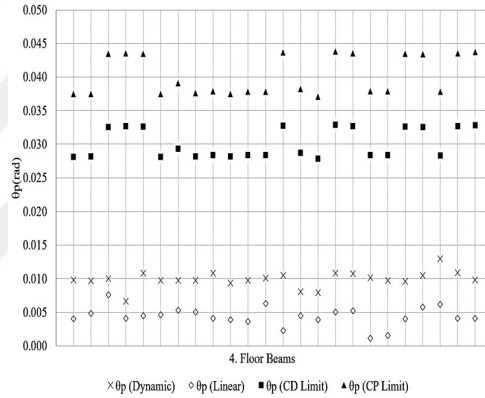
(a)



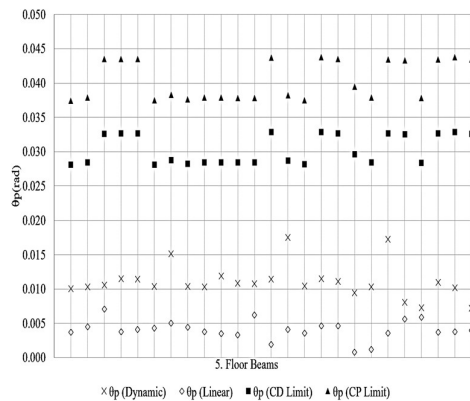
(b)



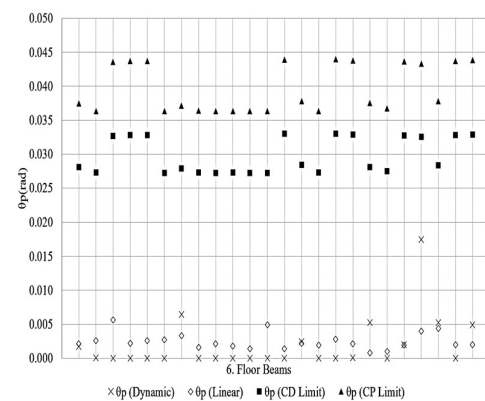
(c)



(d)



(e)



(f)

Figure 5.22 Comparison of the end plastic rotations of the (a) 1st floor, (b) 2nd floor, (c) 3rd floor, (d) 4th floor, (e) 5th floor and (f) 6th floor beams calculated in the Y direction

The plastic rotation values on the shear walls located in the X and Y directions of the building for both analysis methods are presented comparatively in Tables 5.1 and 5.2.

Table 5.1 Plastic rotation comparisons of shear walls in X direction

Floor	Shear Wall Number	X Direction	
		Linear Analysis θ_p	Dynamic Analysis θ_p
1. Floor	P101	0.0011	0.0010
	P102	0.0046	0.0007
2. Floor	P201	0.0046	0.0000
	P202	0.0124	0.0000
3. Floor	P301	0.0056	0.0000
	P302	0.0126	0.0000
4. Floor	P401	0.0058	0.0000
	P402	0.0114	0.0000
5. Floor	P501	0.0055	0.0007
	P502	0.0094	0.0000
6. Floor	P601	0.0048	0.0019
	P602	0.0069	0.0000

Table 5.2 Plastic rotation comparisons of shear walls in Y direction

Floor	Shear Wall Number	Y Direction	
		Linear Analysis θ_p	Dynamic Analysis θ_p
1. Floor	P104	0.0037	0.0037
	P103	0.0041	0.0038
2. Floor	P203	0.0142	0.0000
	P204	0.0137	0.0000

Table 5.2 (Continued)

3. Floor	P303	0.0187	0.0000
	P304	0.0182	0.0000
4. Floor	P403	0.0195	0.0000
	P404	0.0192	0.0000
5. Floor	P503	0.0183	0.0000
	P504	0.0182	0.0000
6. Floor	P603	0.0163	0.0001
	P604	0.0163	0.0001

The differences in ratios of the average plastic rotation demands compiled as a result of non-linear dynamic analyses and member plastic rotation demands calculated with the TBEC (2018) linear elastic method are determined as percentages. The ratios obtained for the bottom end of the first-floor columns are presented in Table 5.3. According to the data presented in Table 5.3, the linear elastic method predicts 22% higher demand than the dynamic analysis in the X direction, while it gives 25% lower plastic rotation demands than the dynamic analysis in the Y direction of the structure.

Table 5.3 Difference in Mean Plastic Rotation for Column Bottom End (in %)

X Direction		Y Direction	
Floor	Mean % Difference	Floor	Mean % Difference
Floor 1	22.31	Floor 1	-25.77

Calculations and comparisons regarding the 6-story dormitory building, which is examined as a second case study, are also completed. When the results are evaluated in general, it has been determined by dynamic analyses that the plastic rotation demands are realized at the bottom ends of columns in the first floor and beams in

accordance with the capacity design. TBEC (2018) linear elastic method can also predict plastic rotation demand with varying rates in these regions. The highest values calculated for the 6-story dormitory building are around 0.01 rad at the bottom ends of the columns. Due to the observed values are quite small, the deviation values stated in the Table 5.3 were obtained. When the plastic rotation values obtained as a result of linear analysis and dynamic analysis are evaluated, it is observed that the plastic rotation values are quite close to each other. These results are discussed in detail in Chapter 6.



CHAPTER 6

SUMMARY AND CONCLUSIONS

6.1 Summary

This study has been focused on the condition assessment of existing structures under earthquake forces according to the TBEC (2018). Linear elastic or nonlinear procedures could be utilized for this purpose. The first part of the study is devoted to the introduction of the related procedure in TBEC (2018) as well as similar procedures in other international codes. In the second part, three existing buildings with different structural features and occupancy classes are investigated as case studies.

First, the seismic performance is evaluated for a low-rise school building by using the linear elastic method. This case study building, for which the prepared calculation routines have been checked and used, is chosen because it provides ease of control due to its simple geometry and represents the class of buildings with critical importance in the building stock. Since the school building is an old structure, it has prominent features such as low material strength and RC member sections with non-ductile details. In addition, it is assumed that the building is located in a region with high seismic intensity in order to provide more unfavorable conditions for the case study. Eventually, the calculated DCR values for the building are expected to be high.

The building is examined first by using the linear elastic method in TBEC (2018). Regardless of the DCR limits, the performance of the structure is determined as "Collapse Performance Level" after the application of the linear elastic method. Considering the structural deficiencies of the building together with the high seismic

demand, and the earthquake hazard, it can be considered that the determined performance level reflects the expected earthquake performance for this building.

When the non-linear dynamic behavior of the 4-story school building in the existing condition is examined in the light of the results obtained from the analyses carried out, it is observed that a story mechanism has occurred in the first story columns. The building experiences plastic hinging at the top and bottom ends of the first story columns, resulting in less non-linear demands on the beams and column member in the upper floors. This is an expected result since the examined school building has not been designed according to the principles of current earthquake codes, hence it does not satisfy the capacity design requirements. Considering the high demand capacity ratios (DCR) presented, it is concluded that the existing school building shows a significant amount of nonlinear behavior under the earthquake hazard level. Also, the performance of the structure is determined as the "Collapse Performance Level".

Comparing linear elastic and nonlinear dynamic analysis methods in TBEC (2018) for the first case study building, it is observed that linear elastic analysis method can accurately predict the first story mechanism (i.e. hinging for first story columns). There are similar values in terms of the interstory drift ratio of the first story columns. The linear elastic method generally remains on the conservative side and estimates higher demands for rotations and interstory drifts in other story columns. The only exception to this situation is observed in the plastic rotations of the top ends of the columns in some floors. Slightly higher plastic rotation demands are calculated at the top ends of these members when the nonlinear dynamic analysis is applied. In general, higher demands are estimated by the linear elastic method for beams.

In addition, a strengthened project is also considered for the examined school building and the same evaluation is made for the strengthened structure. When the results compiled from the non-linear dynamic analyses of the strengthened school building are examined, it is observed that the plastic rotation demands are very limited. The additional shear walls provide significant lateral rigidity to the structure

and greatly reduce non-linear demands in both principal directions. In this context, the school structure has become able to satisfy the performance target according to TBEC (2018).

The plastic rotation demands obtained in the performance evaluation of the strengthened school structure with the linear elastic method are compared with the plastic rotation demands obtained from the non-linear dynamic analysis. It is determined that the linear elastic method in TBEC (2018) predicts the plastic rotational demands on the conservative side. When the frame-based results are examined, it is determined that while the linear elastic method estimates approximately similar plastic deformation demands in the inner and outer frames, the frame mean plastic rotations differ as a result of nonlinear dynamic analyses. Structural features such as the presence of shear walls in the frame cause variation in beam plastic deformation demands in nonlinear dynamic analyses. The linear elastic method cannot accurately predict this variability in demand. The reason is the linear elastic analysis method assumes that structural materials behave elastically which means that the members deform linearly in proportion to the applied load and return to their original shape when the load is removed. This assumption is not valid when the members experience plastic deformation. In reality, under high levels of loading, concrete exhibit nonlinear behavior beyond their elastic limits, leading to plastic deformations, including rotations. Beams in a building which consist of shear walls are subject to significant lateral forces during seismic events. These lateral forces induce bending and shear in the beams. As the shear walls themselves may undergo significant inelastic deformation and rotations, they can transfer these demands to the beams. In the linear elastic method, plastic rotations are not explicitly accounted for, and the distribution of inelastic deformations and rotations is assumed to be idealized, which may not reflect the actual behavior of the structure. Also, the column beam connections can experience plastic deformations and rotations under ground motion. Linear elastic analysis does not capture the interaction between beams and columns under inelastic conditions.

The strengthened school structure does not satisfy the DCR limits specified in TBEC (2018). For this reason, the correlation between the responses calculated by the linear elastic method and the nonlinear dynamic analysis can be expected to be weak. In addition, the plastic rotation demands calculated in both analyses remain very low and the structure basically behaves elastically. The plastic rotations increase the margins of deviation between the results obtained by the two methods. To give an example, the average plastic rotational demand at the bottom end of the first story columns in Frame 1 is 0.0015 rad. for the dynamic analysis and 0.0025 rad for the linear-elastic method (see Figure 5.16). Although the difference between the results seems large, these values indicate very small plastic deformations. Considering the general behavior of the building under seismic action, both methods consistently revealed that the building exhibited a highly elastic response.

The 6-story dormitory building, which is the second case study building, provides a more realistic testing opportunity for the linear elastic method in TBEC (2018). Although this structure was built before 2007 and has low material strength according to current standards, it is a building where the capacity design principle is mostly valid because it has a frame system with small and similar spans including ductile RC elements according to TEC (1998). The fact that the DCR values for columns determined for linear elastic method calculations are lower than the beam DCR values throughout the structure indicates this situation. In addition, low material strength and structural geometry are thought to cause slightly higher values than the DCR limits specified in TBEC (2018). The structure can be considered as a different case in the examination of the linear elastic method due to its properties.

The results compiled from nonlinear dynamic analyses indicate that plastic hinges occur at the bottom ends of the first story columns and shear walls, and also in the beams. This situation is considered to be compatible with the expected performance from the capacity design. On the other hand, low plastic rotational demands are calculated throughout the structure. The calculated maximum values are 0.01 rad at the bottom end of the column and beam ends. It can be stated that the structure which is analyzed under the seismic intensity compatible with DD-1 earthquake level gives

a relatively low nonlinear response due to its design features. The fact that there is no plastic rotation demand at the bottom ends of the shear walls also verifies this situation.

When the plastic rotation demands obtained by the linear elastic method in TBEC (2018) are compared with the dynamic analysis results, it can be concluded that the linear elastic method can detect the plastic hinging at the bottom ends of columns and also at beam ends. It is noteworthy that the differences between two methods, especially at the bottom ends of the columns, are not significant. Although the linear elastic method predicts the behavior at the beam ends where the DCR values are calculated above the allowable limits with a lower accuracy, it generally yields lower plastic rotation demand than the dynamic analysis. While the linear elastic method estimates the plastic rotation demand in all columns in the other stories, the columns and shear walls on the upper stories remain in the elastic range according to the results of nonlinear dynamic analysis. This is attributed to the fact that plastic hinges occur with a certain hierarchy at the bottom ends of columns and at the beam ends due to the capacity design principle. With the changing stiffness properties of the structure, non-linear deformations are concentrated in these regions, and non-damaging (linear elastic) members resist forces with the principle of redistribution. As the linear elastic method in TBEC (2018) predicts plastic rotations over displaced axis rotations as per the formulation presented, it cannot detect the articulation mechanism and redistribution behavior. As a result, it can be concluded that the linear elastic method can predict plastic deformations at the ground story column ends and at the beam ends with an acceptable margin whereas it provides a poor estimate for the ends of the column members in the upper stories of the structure, which is designed according to the capacity design principles. It is thought that the accuracy of the estimations can be improved if the DCR limit values determined for applicability of the linear elastic method are satisfied.

6.2 Conclusions

The following conclusions can be stated based on the observations and results obtained from the numerical analyses. All comments stated below belong to the case study buildings selected with specific structural characteristics under imposed seismic loading conditions.

- In the light of the observations from the first case study building, it can be stated that the linear elastic method in TBEC (2018) can accurately predict the nonlinear behavior of the structure at the first floor because of fixity at bottom of the first-floor column. For members which are in other floors, the prediction performance remains highly conservative. As discussed, this is related to the story mechanism that is encountered on the first floor of the inspected buildings during the seismic action.
- For the strengthened school building, it can be concluded that the linear elastic method in TBEC (2018) predicts a behavior that is compatible with nonlinear dynamic analyses on the basis of the whole structure, if not on member basis. It is also concluded that the ease of application provided by the linear elastic method compared to nonlinear dynamic analysis in cases where low plastic deformation demands are expected to occur in buildings, creates an advantageous situation despite the conservative results obtained.
- According to obtained results, the existing school building has higher DCR values than the limit values given for the linear elastic method in TBEC (2018). On the other hand, when the demands from linear elastic method are compared with the nonlinear dynamic analysis results, it is observed that a good estimation accuracy in the column members are obtained where the story mechanism has occurred. This gave rise to the idea of further testing the applicability of the method for the identification of critical members, independent of the DCR limits.

- The 6-story dormitory building, which is the second structure examined within the scope of the study, has provided a different testing opportunity for the TBDY-2018 linear elastic method. Although this building was constructed before 2007 and has lower material strengths compared to today's standards, it stands out as a building where capacity design is largely valid due to its regular frame system with small spans and its concrete elements designed with ductility according to the 1998 seismic code. The fact that the column DCR values determined for linear elastic method calculations are lower than the beam DCR values throughout the structure also points to this situation. Although it presents a conservative performance prediction for the column members on upper floors, it can be concluded that the linear elastic method can reasonably predict the plastic deformations at the lower ends of the first-floor columns and beam ends of a structure designed according to the principles of capacity design. It is believed that the accuracy of predictions can be improved if the method meets the specified DCR limit values. Considering the 6-story dormitory building, it can be stated that the results obtained from the linear elastic method in TBEC (2018) are promising in terms of evaluating the existing structures which are well designed building according to earthquake regulations.
- For the 6-story dormitory building, the results obtained from nonlinear dynamic analyses indicate that plastic hinging occurs at the lower ends of the first-floor column and shear walls, as well as in the beams. This situation is believed to be in line with the expected performance from capacity design. When the plastic rotation demands obtained from the TBEC (2018) linear elastic method are compared with the dynamic analysis results, it can be concluded that the method can detect plasticity at the lower ends of columns and beam ends. It is noteworthy that the error rates, especially at the lower ends of columns, are low. However, the method's prediction performance is weaker at beam ends where DCR values are calculated above the allowed limits, but it generally results in less plastic rotation demand compared to

dynamic analysis. While the linear elastic method predicts plastic rotation demand in all columns located on other floors, according to the results of nonlinear dynamic analysis, columns and shear walls on upper floors remain elastic. This situation is attributed to the behavior of capacity design, which leads to a certain sequence of plastic hinging occurring at the lower ends of columns and beam ends. During dynamic analysis, the increase in nonlinear behavior and the change in structural stiffness results in a redistribution in internal force demands. The failure of linear elastic analysis methods to capture this behavior in general is a well-known phenomenon in performance-based earthquake engineering applications and this situation is observed also in this study.

- In general, this research focuses on the analysis of the buildings which have higher DCR (Demand Capacity Ratio) values than the applicable limits outlined in the TBEC(2018). Therefore, this study extensively explores the effectiveness and precision of the linear analysis approach when these DCR limits are exceeded. According to obtained results, the linear elastic method predicts the nonlinear behavior of the structure in an acceptable margin especially for the floor that story mechanism occurs although the DCR limits are exceeded. When it is thought that DCR limits will be exceeded in most existing old structures, linear elastic method has a very limited application window considering DCR limits according to TBEC (2018).
- As a general conclusion, the most important observation in this study is that the performance of the members where non-linear demands are expected under the influence of earthquake forces can be predicted with a reasonable accuracy by the application of the linear elastic method in TBEC (2018), which provides an ease of application for seismic assessment of existing structures.

6.3 Future Recommendations

The following recommendations can be made for future research:

- The case studies examined in this study are buildings in Turkey where the DCR limits specified in the TBEC (2018) are exceeded. In the case of exceeding DCR limits, the accuracy of the linear analysis method has been thoroughly investigated. Therefore, in future studies, existing structures that satisfy the DCR limits values presented in Section 15.5.3 of TBEC (2018) can be subjected to performance evaluation using both linear elastic and nonlinear dynamic analysis methods. Thus, it will be possible to examine the accuracy of the TBEC (2018) linear elastic analysis method across a wide spectrum of variable DCR limits.
- In the future research, the performance evaluations of building types with different usage purposes and structural characteristics, can be conducted through linear elastic method and nonlinear dynamic analysis. Therefore, a relationship can be established and investigated between the change of structural properties and the accuracy of the linear elastic analysis method on the prediction of plastic rotation demands.

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