

**T.C.  
İSTANBUL KÜLTÜR UNIVERSITY  
INSTITUTE OF GRADUATE STUDIES**

**SEISMIC DESIGN OF A REINFORCED CONCRETE SCHOOL BUILDING**

**Master of Science Thesis**

**Mutasem AL OSMAN**

**1800001171**

**Department: Civil Engineering**

**Programme: Structural Engineering**

**Supervisor: Assist. Prof Dr. Gökhan YAZICI**

**AUGUST 2020**

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**University** : **İstanbul Kültür University**  
**Institute** : **Institute of Graduate Studies**  
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**Programme** : **Structural Engineering**  
**Supervisor** : **Assist. Prof. Dr. Gökhan YAZICI**  
**Degree Awarded and Date** : **MSc Thesis – AUGUST 2020**

## **ABSTRACT**

### **SEISMIC DESIGN OF A REINFORCED CONCRETE SCHOOL BUILDING**

**Mutasem AL OSMAN**

A large number of school buildings were damaged during the last century due to earthquakes all over the world. This led to the constant revision of seismic codes worldwide to improve safety for school buildings against earthquakes. In this study, seismic analysis and design carried out for a fictitious five-storey reinforced concrete school building located in a high seismic zone in Istanbul, Turkey. The study consists of eight sections. In the first section, provides a brief description and objective of the study. Second and third sections, give a theoretical review on methods of seismic analysis, design and performance evaluation. In addition, the description of the model school building is presented. At section 4, a comparison is performed between four seismic codes on the calculation of base shear for equivalent lateral force analysis (EFL). In the fifth chapter, seismic design is performed for a school building using CSI ETABS according to TEC2007, TS498 and TS500. At sixth and seventh sections, seismic performance of the designed school building is evaluated by using static nonlinear pushover analysis. Afterwards, retrofitting strategies are proposed to enhance the actual school performance. Finally, the performance of retrofitted cases is evaluated according to TBDY2018. In end of the study, conclusions and recommendations made according to obtained results.

**Keywords:** Seismic design, Equivalent Lateral Force Method, Reinforced Concrete, pushover analysis, School buildings.

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## **KISA ÖZET**

### **BETONARME BİR OKUL BİNASININ SİSMİK TASARIMI**

**Mutasem AL OSMAN**

Geçen yüzyılda tüm dünyada depremler nedeniyle çok sayıda okul binası hasar görmüştür. Bu, okul binalarının depreme karşı güvenliğini artırmak için dünya çapında sismik kodların sürekli revize edilmesine yol açmıştır. Bu çalışmada, İstanbul, Türkiye'de yüksek deprem bölgesinde yer alan farazi beş katlı betonarme bir okul binasının sismik analizi ve tasarımı yapılmıştır. Çalışma, sekiz bölümden oluşmaktadır. İlk bölümde, çalışmanın kısa bir açıklaması ve amacı verilmektedir. İkinci ve üçüncü bölümler, sismik analiz, tasarım ve performans değerlendirme yöntemleri üzerine teorik bir inceleme sunmaktadır. Buna ek olarak, model okul binasının özellikleri verilmiştir. Bölüm 4'te, eşdeğer deprem yükü yöntemi (EFL) için taban kesme kuvveti hesaplamasına ilişkin dört sismik kod arasında bir karşılaştırma yapılmıştır. Beşinci bölümde, TEC2007, TS498 ve TS500'e göre CSI ETABS kullanılarak model okul binası için sismik tasarım yapılmıştır. Altıncı ve yedinci bölümlerde, tasarlanan okul binasının sismik performansı, statik doğrusal olmayan itme analizi kullanılarak değerlendirilmiştir. Daha sonra, model okul yapısının performansını iyileştirmek için güçlendirme önerileri verilmiştir. Son olarak, güçlendirme önerilerinin performansı TBDY2018'e göre değerlendirilmiştir. Çalışma sonucunda elde edilen sonuçlara göre sonuç ve öneriler yapılmıştır.

**Anahtar Kelimeler:** Sismik tasarım, Eşdeğer Yanal Kuvvet Yöntemi, Betonarme, itme analizi, Okul binaları.

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## ABBREVIATIONS

TEC2007	: Turkish earthquake code 2007
TBDY2018	: Türkiye Bina Deprem Yönetmeliği 2018
ASCE	: American Society of Civil Engineers
ASCE 7-16	: Minimum Design Loads and Associated Criteria for Buildings and Other Structures 7-16 version
ASCE 41-13	: Seismic Evaluation and Retrofit of Existing Buildings
EC8	: Eurocode 8 for Seismic Design of Buildings
FEMA	: Federal Emergency Management Agency
FEMA 273	: NEHRP Guidelines for The Seismic Rehabilitation of Buildings
FEMA 356	: Prestandard And Commentary for The Seismic Rehabilitation of Buildings
FEMA 440	: Improvement of Nonlinear Static Seismic Analysis Procedures
NSP	: Nonlinear static procedure
ATC	: Applied Technology Council
ATC-40	: Seismic Evaluation and Retrofit of Concrete Buildings
ATC-50	: Evaluation and Improvement of Inelastic Seismic Analysis Procedures
CSI	: Computers and Structures, Inc.
ETABS	: Ultimate integrated software package for the structural analysis and design of buildings
ELF	: Equivalent Lateral Force
DTS	: Deprem Tasarım Sınıfına
BYS	: Bina Yükseklik Sınıfı
BKS	: Bina Kullanım Sınıfı
PSHA	: Probabilistic Seismic Hazard Analysis
PE	: Probability of exceeding
PGA	: Peak ground acceleration
PGV	: Peak ground velocity
RSA	: Response spectral acceleration
PA	: Pushover analysis

IO	: Immediate occupancy
LS	: Life safety
CP	: Collapse prevention
ADRS	: Acceleration-Displacement response spectra
NTC 2008	: Italian technical standards for construction 2008
SSI	: Soil-Structure interaction
DCM	: Displacement Coefficient Method
TS500	: Turkish code requirements for design and construction of. reinforced concrete
TS498	: Turkish design load for buildings
EC1	: Eurocode 1 for Actions on structures
DCH	: High ductility class
CSM	: Capacity spectrum method

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## LIST OF SYMBOLS

$K$	: Stiffness
$E$	: Young's modulus
$I_m, I_g$	: Moment of inertia
$V, V_{tE}^{(X)}, V_t, F_b$	: Base shear
ft	: Feet
ft <sup>2</sup>	: Feet square
m	: Meter
m <sup>2</sup>	: Meter square
$T, T_{pA}, T_1, T_a$	: Fundamental period
$T_1$	: Fundamental period in Eurocode
$T_c$	: Upper limit period in Eurocode
s	: Seconds
$\eta_{bi}$	: Torsional irregularity coefficient
$H_N, H, h_n$	: Building height
$H_i$	: Height of i'th story
$H_j$	: Height of j'th story
$S_s$	: 0.2 sec spectral acceleration
$S_1$	: 1 sec spectral acceleration
$g$	: Ground acceleration
PI	: Plasticity index
R	: Response modification factor
q	: Behavior factor
$g$	: Ground acceleration
$W_j^{(s)}$	: Singular weight acting on the typical finite element joint j
$W_{G,j}^{(s)}$	: Single permanent weight acting on the typical finite element joint j
$W_{Q,j}^{(s)}$	: Single permanent weight acting on the typical finite element joint j
n	: Live load participation factor

$w_i$	: Weight of i'th story
$w_j$	: Weight of j'th story
$M, m_t$	: Mass
$W$	: Weight
$g_i$	: Total dead load at i'th story of building
$q_i$	: Total live load at i'th story of building
$G_{k,j}$	: Permanent weight
$Q_{k,l}$	: Quasi-permanent weight
$\Psi_{(E,i)}$	: the combination coefficient for variable action i
kpa	: Kilopascal
f	: Frequency
$\omega$	: Radial velocity
$C_t, \alpha$	: Factor depends on type of seismic resisting structural system
N	: Number of floors
$S_{ae}(T), S_e(T), S(T), S_a$	: Elastic response spectrum
$T_0, T_A, T_B, T_C, T_D, T_s$	: Elastic response spectrum limits
$T_L$	: long-period transition period
$C_t$	: Building period coefficient
$C_u$	: coefficient for upper limit on calculated period
$\eta$	: damping correction factor
S	: Soil factor
$a_g, a_{gR}$	: Peak ground acceleration
$\gamma_I, I, I_e$	: Importance factor
$F_a, F_s$	: Short-period site coefficient (at 0.2-s period)
$F_v, F_1$	: Long-period site coefficient (at 1.0-s period)
$S_{DS}$	: Short period design spectral acceleration coefficient
$S_{D1}$	: 1.0 second period design spectral acceleration coefficient
$S_d(T), S_{aR}(T)$	: Design response spectrum
$C_s$	: Seismic response coefficient
$A(T_1)$	: Spectral acceleration coefficient

$A_0$	: Effective ground acceleration coefficient
$R_a(T_1)$	: Seismic load reduction factor
$D$	: Over strength factor
$\beta_i$	: Initial damping
$\beta_0$	: damping factor
$a_{pi}$	: Maximum acceleration
$d_{pi}$	: Maximum displacement
$a_y$	: Yield acceleration
$d_y$	: Yield displacement
$\mu$	: Ductility
$\alpha$	: Post-Elastic stiffness
$\beta_{eff}$	: Effective damping
$T_{eff}, T_e$	: Effective period
$T_{sec}$	: The radial secant period
$\delta_t$	: Target displacement
$C_0$	: Modification factor to relate spectral displacement to building roof displacement
$C_1$	: Modification factor to relate spectral displacement to building roof displacement
$\mu_{strength}$	: Ratio of elastic strength demand to calculated yield strength coefficient.
$a$	: Site class factor
$T_s$	: Period associated with transition from constant acceleration segment of the spectrum to the constant velocity segment of the spectrum
$T_i$	: Elastic fundamental period.
$K_i$	: Elastic lateral stiffness of the building.
$V_y$	: Effective yield strength calculated using the capacity curve.
$W$	: Effective seismic weight.
$C_m$	: Effective mass factor
$C_2$	: Modification factor to represent the effect of pinched hysteresis shape, cyclic
$m/s$	: Meter per second

$m/s^2$	: Meter per second square
ft/s	: Feet per second
$\gamma_c$	: Concrete unit weight
kN/m	: Kilo newton per meter
$kN/m^2$	: Kilo newton per meter square
$kN/m^3$	: Kilo newton per meter cubic
$f_{ck}$	: Characteristic compressive strength of concrete
$f_{ctk}$	: Concrete tensile strength
$f_{ctd}$	: Design concrete tensile strength
$f_{cd}$	: Design compressive strength of concrete
$E_c$	: Concrete modulus of elasticity
$f_{yk}$	: Yield strength
$f_{su}$	: Rupture strength
$E_s$	: Steel modulus of elasticity
$\gamma_s$	: Strength reduction factor for steel
$f_{yk}$	: Yield strength
$F_{yd}$	: Design yield strength of longitudinal reinforcement
$F_{ud}$	: Design ultimate strength of longitudinal reinforcement
$t_s$	: Slab thickness
$\gamma_b$	: Unit weight of block
$b_b$	: Beam width
$h_b$	: Beam depth
$b_c$	: Column width
$h_c$	: Column depth
$\gamma_t$	: Unit weight of tile
$\gamma_s$	: Unit weight of sand
$(EI)_o$	: Bending stiffness for non-cracked section
$(EI)_e$	: Bending stiffness for cracked section
$A_c$	: Area of gross cross section
$f_{cm}$	: Strength of concrete section member
$N_d$	: Factored axial force

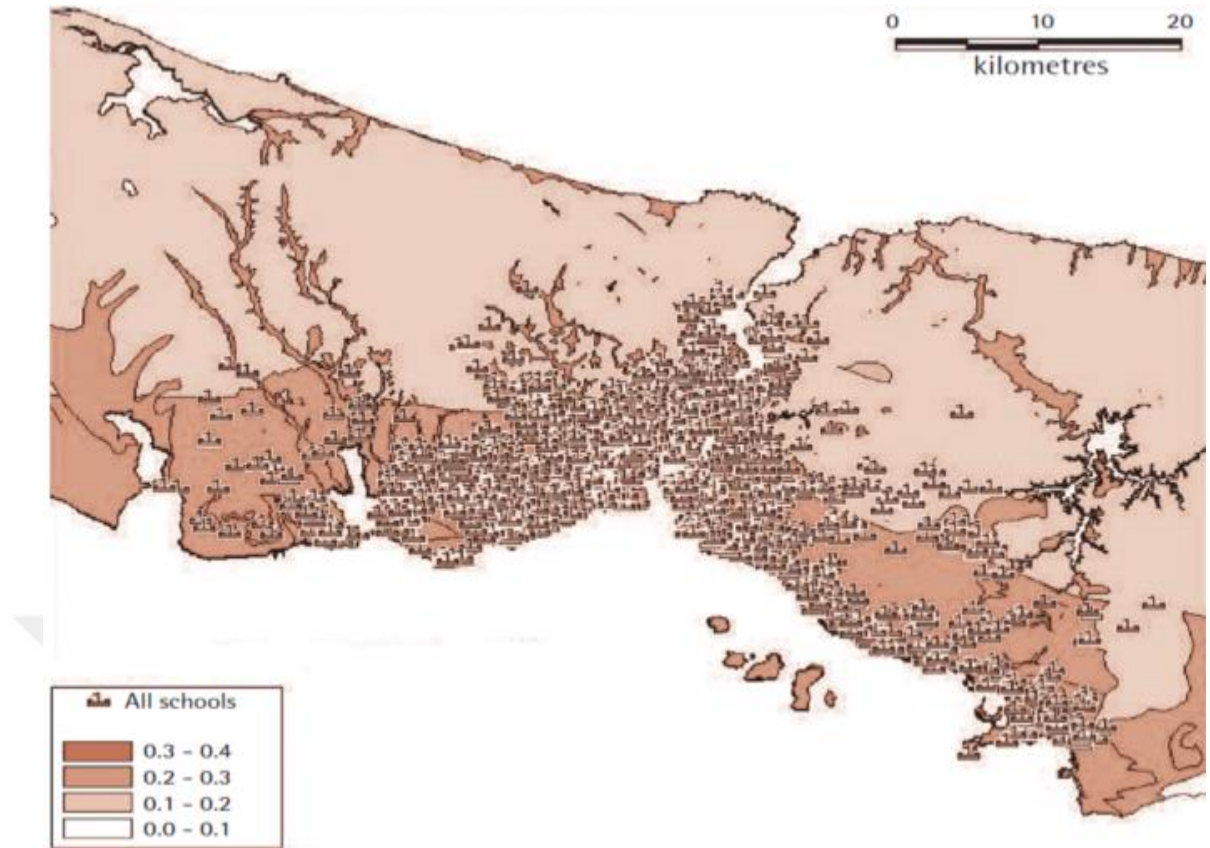
$q_0$	: Basic value of the behavior factor
$k_w$	: Factor reflecting the prevailing failure mode in structural systems with walls
$\lambda$	: Correction factor
$S_d$	: Monitored displacement
$S_a$	: Spectral acceleration
$Q$	: Generalized force
$Q_y$	: Yield strength



## 1. INTRODUCTION

The 1999 Kocaeli earthquake showed how destructive earthquakes can be. Kocaeli earthquake caused the deaths of 17,000 people and left more than 250,000 people homeless [1]. Kocaeli earthquake also damaged a significant number of schools around the earthquake region. 43 schools were beyond repair and 381 schools had minor to medium damage [2]. Even in Istanbul, 820 schools were reported to be affected [2]. This reminds humanity the importance of design buildings especially schools against earthquakes. In addition, Figure 1.1 shows how schools in Istanbul, located near a fault line that can cause catastrophic earthquakes over magnitude 7.0 according to Richter scale, are exposed to seismic risk due to their location in a high seismic zone [2].

This thesis deals with the performance-based analysis for a 5-story model reinforced concrete school building that was designed according to seismic Turkish code (TEC2007). Performance based design method will also be used for this thesis in the analysis of this school building.



**Figure 1.1 : Schools in Istanbul with Seismic Zones [2]**

### **1.1. Purpose of the Thesis**

The main objective of my thesis is to make accurate performance-based analyzing with use lateral and gravity load according to new Turkish code (TBDY2018) global acceptance criteria for school building that designed according to old seismic Turkish code (TEC2007). This give us information about response and how much they can sustain of present exciting school building during earthquake.

Code provisions for equivalent static lateral force method procedures for school buildings according to TBDY2018, TEC2007, ASCE 7-16 and EC8 codes are also discussed within the scope of this study.

## 1.2. Background

Seismic evaluation and retrofit of concrete buildings – ATC 40 [3] by the Applied Technology Council, ASCE41-13 [4] by AMERICAN NATIONAL STANDARDS INSTITUTE, NEHRP GUIDELINES FOR THE SEISMIC REHABILITATION OF BUILDINGS FEMA 273 [5] and FEMA 356 [6] by FEDERAL EMERGENCY MANAGEMENT AGENCY have been developed to evaluate the existing buildings performance and improve the concept of performance analysis of a building to know the level of building safety. Then, ATC 55 [7] and FEMA 440 [8] have been developed for non-linear seismic analysis. The previous Turkish seismic code (TEC2007) [9] and new seismic Turkish code (TBDY2018) [10], was developed and revised for seismic assessment for structures. Also, European code (EC8) [11] and American society of civil engineering code (ASCE 7-16) [12] was developed and revised for seismic assessment for structures.

## 1.3. Previous Studies

This includes literature survey of researches about area of seismic design of school buildings. Seismic design includes seismic analysis and performance evaluation. Listed below are studies conducted on this field of research.

- H. Bilgin (2015), presents “Seismic performance evaluation of an existing school building in Turkey”. In this research, Seismic performance evaluation by using PA performed for a typical school building according to TEC2007. Then, the study shows performance of school building not satisfy the acceptance criteria, so the school building strengthened in order to satisfy the required performance [13].
- M. Dogru and G. Arslan (2017). Present “SEISMIC PERFORMANCE OF A RC SCHOOL BUILDING CONSIDERING DIFFERENT SOIL CLASSES”. Performance evaluation carried out for five story school RC buildings with considering different soil classes in which this building designed according Turkish Seismic Code (TEC) 1975. Then, seismic performance for this existing building is estimated according to TEC2007 by using PA and different soil classes. The building pushed by use SAP2000 software with 200 mm maximum displacement limit. The research concludes, the damage level of structural sections is getting to has higher values with movement to soften soil class because of increasing demand displacement values. Also, the ready for usage performance level is not provided for any soil site class but life safety level just provided for site class A. Finally, this

research shows the necessity of performance evaluation for important structures built on weak grounds [14].

- F. Hsiao, Y. Oktavianus, Y. Ou, C. Luu and S. Hwang (2015), present “A Pushover Seismic Analysis and Retrofitting Method Applied to Low-Rise RC School Buildings”. In this research, a modified CSM called Taiwan Earthquake Assessment for Structures by Pushover Analysis carried out by use results from an experimental of two pushover field tests in low rise RC school building (The Guanmiao school building) and ASCE 41 CSM. Also, some problems are discussed in this research and their solution such as optimal selection of the RC jacketing’s position. Finally, the research show Taiwan Earthquake Assessment for Structures by Pushover Analysis is capable of providing accurate results for assessing a low-rise RC building’s capacity and is more appropriate to pushover field tests [15].
- E. El-Arab (2011) [13], present “Seismic Analysis of Existing School Buildings Using Different Egyptian Seismic Provesions”. In this research, a comparative study for a seismic analysis of a certain class of existing school buildings according to importance and wide spread buildings in Egypt considerations. This study, focuses on a comparison between all versions of the Egyptian Code of Practice for loading (ECP-201), versions published in 1993, 2003, and the draft of final version October, 2008] and the Regulations of the Egyptian Society of Earthquake Engineering (ESEE, 1988). For each building case, base shear and moment value are calculated by using response spectrum, time history and ELF analysis according to each code provisions that mentioned above. Finally, the study show normalized base shear and base moment obtained using the ECP editions were less matched with real earthquake time history analysis results with error percentage reaching to 90% [16].

## 2. METHODOLOGY

### 2.1. Overview

The following methods will be used: modal analysis will be used to determine the mode shapes and periods for the structure, static linear analysis will be used to check and design the frames of structure, pushover (nonlinear) analysis used to obtain capacity curve for structure and performance-based design used to calculate performance point.

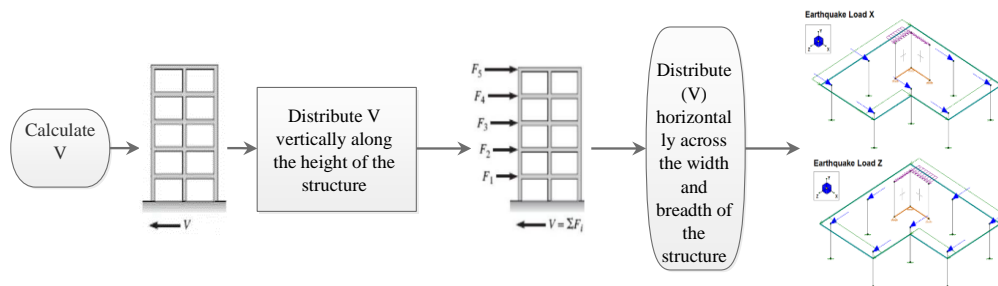
Actually, for seismic analysis for our building we will use Equivalent Lateral Force procedure (ELF). Also, two types of procedures will be used to calculate performance point of structure.

### 2.2. ELF Procedures in ASCE 7-16, EC8, TBDY2018 and TEC2007

Equivalent Lateral Force procedure is a static linear analysis method used for seismic analysis, which uses the following steps:

1. Calculate the seismic base shear ( $V$ ) for structure.
2. Distribute base shear ( $V$ ) vertically along the height of the structure.
3. Distribute base shear ( $V$ ) horizontally across the width and breadth of the structure.

Each step presented above depends on a set of simplifying assumptions.



**Figure 2.1 : ELF Procedure**

### 2.2.1. ELF Procedures in ASCE 7-16, EC8, TBDY2018 and TEC2007

The ELF method can be used only for buildings that not have significant elevation and plan irregularity in mass and stiffness and up to mid-height buildings. The conditions of use ELF method for different codes are shown in Table 2.1 and Table 2.2.

**Table 2.1** : ELF Conditions for ASCE7-16 & EC8 Codes [12, 11]

ASCE 7-16 CODE	EC8 CODE
<ul style="list-style-type: none"> <li>• Can be used for all structures that have seismic design category B,C               <ul style="list-style-type: none"> <li>• For D,E,F seismic design category there are the following constraints :                   <ol style="list-style-type: none"> <li>1. Risk Category I or II buildings not exceeding two stories above the base.</li> <li>2. ELF can be used for structures of light-frame construction.</li> <li>3. ELF can be used for structures with no structural irregularities and not exceeding 160 ft (48.8 m) in structural height.</li> <li>4. ELF can be used for structures exceeding 160 ft (48.8 m) in structural height with no structural irregularities and with <math>T &lt; 3.5T_s</math>.</li> <li>5. ELF can be used for structures not exceeding 160 ft (48.8 m) in structural height and having only horizontal irregularities of Type 2, 3, 4, or 5 in Table 12.3-1 or vertical irregularities of Type 4, 5a, or 5b in Table 12.3-2.</li> </ol> </li> </ul> </li> </ul>	<ul style="list-style-type: none"> <li>• The building in must meet these two conditions:               <ol style="list-style-type: none"> <li>1. They have fundamental periods of vibration <math>T_1</math> in the two main directions which are smaller than the following values                   <math display="block">T_1 \leq \begin{cases} 4. T_c \\ 2,0 s \end{cases}</math>                   , where <math>T_c</math> is the upper limit of the period of the constant spectral acceleration branch.                 </li> <li>2. They meet the criteria for regularity in elevation given in Section 4.2.3.3.</li> </ol> </li> </ul>

**Table 2.2 : ELF Conditions for TBDY2018 & TEC2007 Codes [10, 9]**

TBDY2018			TEC2007 CODE		
<ul style="list-style-type: none"> <li>After determining the DTS from Table 3.2 in code and BYS from Table 3.3 in code values for structure we can check if the ELF method can be used or not as following:</li> </ul>			<ul style="list-style-type: none"> <li>After determining the seismic zone from seismic hazard map for structure we can check if the ELF method can be used or not as following:</li> </ul>		
Building Type	Allowed Building Height Class		Seismic zone	Building Type	Allowed Building Height (H <sub>N</sub> )
	DTS = 1, 1a, 2, 2a	DTS = 3, 3a, 4, 4a			
Buildings where the torsional irregularity coefficient on each floor satisfies the condition $\eta_{bi} \leq 2.0$ and also does not have B2 type irregularity	BYS $\geq 4$	BYS $\geq 5$	1,2	Buildings where the torsional irregularity coefficient on each floor satisfies the condition $\eta_{bi} \leq 2.0$ .	25m
All other buildings	BYS $\geq 5$	BYS $\geq 6$	1,2	Buildings where the torsional irregularity coefficient on each floor satisfies the condition $\eta_{bi} \leq 2.0$ and also does not have B2 type irregularity.	40m
			3,4	All other buildings	40m

### 2.2.2. Base Shear Calculation

During earthquakes, the mass of structure accelerates laterally, these masses produce inertial forces that accumulate over the height of structure that produce the base shear. Base shear as shown in Table 2.3 depends on many factors in which the most important one is the weight of structure as its increase the seismic force and base shear increase. Also, the other factors are type of structural system used in resist seismic force, usage type of building and location of building.

**Table 2.3 :** The Factors That Base Shear Depends on and Their Affect

<b>Factors</b>	<b>The effect on base shear</b>
Weight of structure	Increase base shear
Ductility of structure system	Decrease base shear
Height of structure	Decrease base shear by increase natural period
Importance of building	Increase base shear
Location of building	As the building approach to high seismicity zone the base shear will increase

In general, ASCE 7-16, EC8, TBDY2018 and TEC2007 Codes are similar in calculation of base shear for ELF method in the side of taken the base shear values as percentage of weight of structure. Actually, this percentage depends on factors that are presented above.

### 2.3. Seismic Coefficients

#### 2.3.1. Determination of Seismic and Site Coefficients

Seismic coefficients depend on earthquake itself and the occurrence probability of earthquake and site coefficients depend on type of soil in desired site. Seismic coefficients include 0.2 sec spectral acceleration ( $S_s$ ), 1 sec spectral acceleration ( $S_1$ ) and ground acceleration in terms of  $g$ . Usually, each country give values of Seismic coefficients called seismic Hazard maps according to seismic statistical studies

Soil, site or ground class tables in four codes classify soil according to shear wave velocity for the 30m depth in soil. TBDY2018 and ASCE7-16 [10, 12], the site classes are divided into 6 groups (Table 2.4) and almost same for both codes which they are rock site class for first and second class, soft rock with very dense soil for third class, stiff soil for fourth class and fifth and sixth classes for weak, have high plasticity index (PI), high moisture content soil and soil require special analysis.

**Table 2.4 : Site Class for TBDY2018 and ASCE7-16 Codes [10, 12]**

site class	Description of soil layer	shear wave velocity (m/s)
A or ZA	Hard rock	>1500
B or ZB	Rock	760 to 1500
C or ZC	soft rock with very dense soil	360 to 760
D or ZD	stiff soil	180 to 360
E or ZE	Soft clay soil and have high PI & high moisture content	<180
F or ZF	Require special analysis	-

EC8 code [11], site classes are described more elaborately and divided into 7 groups (Table 2.5) which are rock for Group A, very dense sand gravel, or very stiff clay for Group B, dense or medium dense sand gravel, or stiff clay for Group C, loose to medium cohesionless soil for Group D, alluvium soil has shear wave velocity from 100 m/s to 360 m/s located above stiff material for Group E, weak, have high plasticity, high moisture content soil for Group S1 and group S2 for liquefiable soil that require special analysis.

**Table 2.5 : Ground Types for Ec8 [11]**

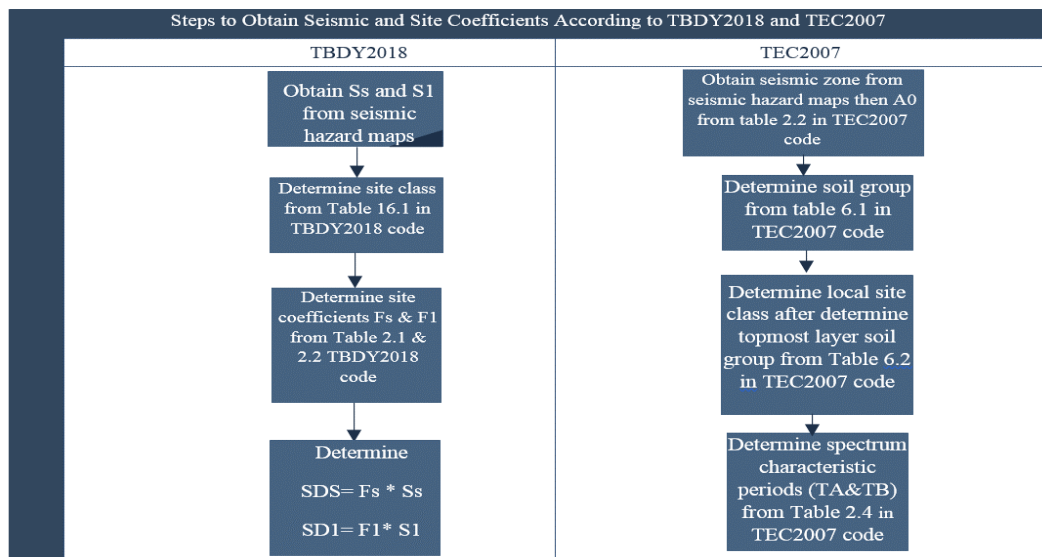
Ground Type	Description of Soil Layer	Shear Wave Velocity (m/s)
A	Rock	>800
B	very dense sand gravel, or very stiff clay	360 to 800
C	dense or medium dense sand gravel, or stiff clay	180 to 360
D	loose to medium cohesionless soil	<180
E	alluvium soil has shear wave velocity from 100 m/s to 360 m/s located above stiff material	
S1	have high plasticity, high moisture content soil	<100
S2	liquefiable soil that require special analysis	

TEC2007 [9] code site classes (Table 2.6) are illustrated in another way by use two tables (6.1 and 6.2) in code. The first table divided soil to four group from A to D. Also, it describes soil group, type of soil such as rock or clay and give the shear velocity for each soil group. The second table, divided soil group into four parts called local site class and noted Z1 up to Z4 which depends on soil group and top layer thickness. In local site class table of TEC2007 code, Z1 and Z2 represent hard rock or clay and very dense soil to soft or stiff clay and medium dense soil. Also, Z3 and Z4 represents soft or stiff clay and medium dense soil to loosen, very weak soil or high moisture content soil.

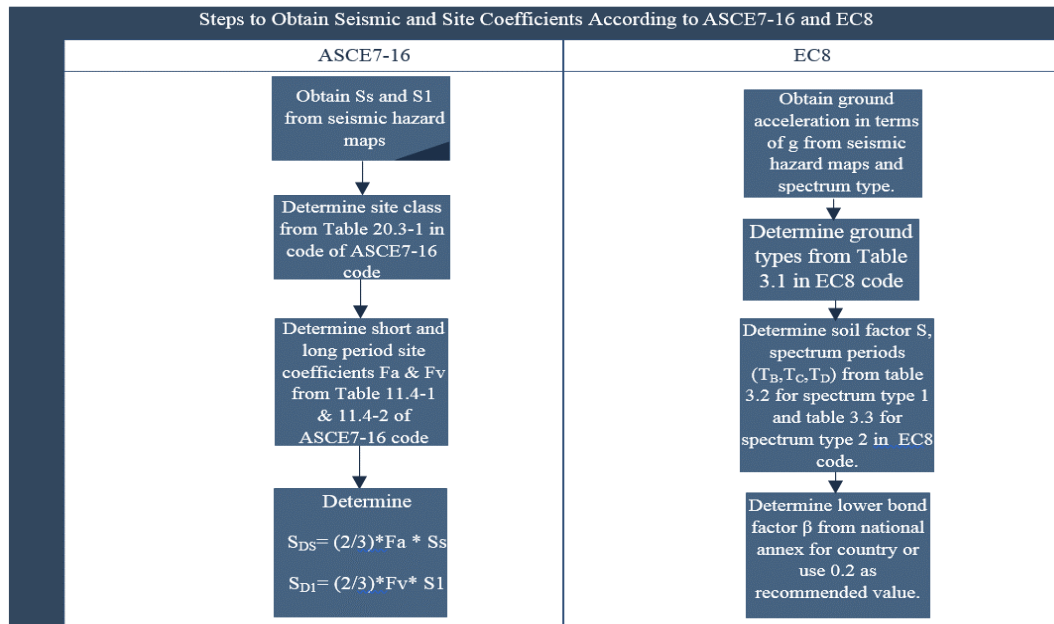
**Table 2.6 : Site Classes for TEC2007 Code [9]**

Soil group	Description of soil Group	Shear wave velocity (m/s)	Local site class
A	Hard rock Very dense soil or Hard clay	>1000 >700	Z1
B	Soft Rock Dense soil High stiff clay	700 to 1000	Z1 if Topmost soil layer of this group $\leq 15$ m
		400 to 700 300 to 700	Z2 if Topmost soil layer of this group $> 15$ m
C	Highly weathered soft metamorphic rocks. Medium dense soil. Stiff clay	400 to 700	Z2 if Topmost soil layer of this group $\leq 15$ m
		200 to 400	Z3 if Topmost soil layer of this group 15 m to 50 m
		200 to 300	Z4 if Topmost soil layer of this group $> 50$ m
D	Soft, deep alluvial layers with high moisture content, loose soil or Soft clay.	<200	Z3 if Topmost soil layer of this group $\leq 10$ m
			Z4 if Topmost soil layer of this group $> 10$ m

Also, the Figure 2.2 and Figure 2.3 will summarize the steps of obtain Seismic and Site Coefficients for TBDY2018, TEC2007, ASCE7-16 & EC8:



**Figure 2.2 : Steps to Obtain Seismic and Site coefficients According to TBDY2018 and TEC2007 [10, 9]**



**Figure 2.3 :** Steps to Obtain Seismic and Site coefficients According to ASCE7-16 and EC8 [12, 11]

### 2.3.2. Seismic Load Resisting Structural System, Ductility & Importance Factors

There are many seismic systems used to resist earthquake forces but each system has their advantages and disadvantages. For example, the use of shear walls system has limit in building height according to ASCE7-16 [12] and TBDY2018 [10]. In addition, some of these systems may have undesirable effects on the architectural aspects of the building. On the other hand, some of seismic systems are favorable because they enhance the ductility of building. Essentially, the selection of the seismic load resisting system depends on:

1. Financial criteria.
2. Architectural factors.
3. Code design limit factors

TBDY2018 [10] & ASCE7-16 [12] give limits to use some of seismic systems which they illustrated in Table 4.1 in TBDY2018 code after determining BYS for building and Table 12.2-1 in ASCE 7-16 code after determining seismic design category for building. Also, TEC2007 [9] gives some height condition limit to the building at section 2.5.1 of TEC2007 code.

During earthquakes, buildings cannot remain at elastic behavior and it can damage or go through plastic state or fracture state or damage, where deformations will dramatically increase even for small load. Engineers must ensure that the building is able to sustain seismic loads without having excessive deformations or collapse. For this purpose, we should increase the ductility of our building.

Actually, codes accounts for building ductility by using Structural Behavior Factor from Table 4.1 in TBDY2018 [10], structural system behavior factors (R) from Table 2.5 in TEC2007 [9] code, behavior factor (q) from section 5.2.2.2 for RC buildings and section 6.3.2 for steel buildings in EC8 [11] code and response modification factor (R) from Table 12.2-1 in ASCE 7-16 [12] code.

In addition, during earthquake some buildings must be able to sustain more loads than other buildings because they are having important usage after earthquake such as hospitals or buildings with high occupancy such as schools or when the contents of the building may cause catastrophe such as chemical plants. In order to account for these, TBDY2018 [10], TEC2007 [9], ASCE7-16 [9] & EC8 [11] codes use importance factor to increase strength of these building.

### **2.3.3. Seismic Weight**

During of structure acceleration due to earthquake, just the mass that tied with structure can be considered as effective seismic weight. In fact, TBDY2018 [10], TEC2007 [9], EC8 [11] and ASCE 7-16 [12] codes suppose the weight of each story lumped at story level at center of mass in story plan for simplicity where each story lumps the masses on it and the half weight of column and shear walls above and under of desired story. Also, roof story lumps the above weight and half of below story columns and shear walls.

TBDY2018 [10] (in section 4.5.9) , TEC2007 [9] (in section 2.7.1), EC8 [11] (in section 3.2.4) and ASCE 7-16 [12] (in section 12.7.2) divide the seismic weight into two parts , the first one is permanent (such as electrical generator, electrical conditions and walls) and the second is due moving loads or quasi-permanent loads (such as people, vehicles and snow weights) as shown in Table 2.7.

**Table 2.7 :** Equations of seismic weight according to TBDY2018, TEC2007, EC8 and ASCE 7-16 [10, 9, 11, 12]

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**TDBY2018** 
$$w_j^{(s)} = w_{G,j}^{(s)} + nw_{Q,j}^{(s)} \quad (2.1)$$

Where  $W_j^{(s)}$  = Singular weight acting on the typical finite element joint j [kN]

$W_{G,j}^{(s)}$  = Single permanent weight acting on the typical finite element joint j [kN]

$W_{Q,j}^{(s)}$  = Single additional (moving) weight acting on the typical finite element joint j [kN]

n= moving weight participation factor

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**TEC2007** 
$$w_i = g_i + nq_i \quad (2.2)$$

Where  $w_i$  = Story weight [kN]

$g_i$  = Total dead load at i'th story of building [kN]

$q_i$  = Total live load at i'th storey of building [kN]

n= live load participation factor

---

**EC8** 
$$\Sigma G_{k,j} + \Sigma \psi_{E,i} \cdot Q_{k,i} \quad (2.3)$$

Where  $G_{k,j}$  = Permanent weight [kN]

$Q_{k,i}$  = Quasi-permanent weight [kN]

$\psi_{E,i}$  = is the combination coefficient for variable action i

---

**ASCE 7-16** dead load+ %of storage live load +%of snow load + live load that tied with structure.

---

Live load participation factor has a variable value and depends on type of structure used and type of load. Table 2.8 summarize these values according to TBDY2018, TEC2007, EC8 and ASCE 7-16.

**Table 2.8 :** Moving weight participation factor values according to TBDY2018, TEC2007, EC8 and ASCE 7-16 [10, 9, 11, 12]

Type of load / usage of area	TBDY2018&TEC2007) (for n values)	EC8 (for $\psi_{E,i}$ values)	ASCE 7- 16 (%of moving load)
For snow load	0.3	0.2	0.2
Storage building, Warehouse, etc....	0.8	0.8	0.25
Shopping, store	0.6	0.6	0
School, dormitory, sports facility, cinema, theater, auditorium, temple, restaurant, masjid, etc.	0.6	For roof = 0.6	0
		For storeys with correlated occupancies = 0.48	
		For independently occupied storeys = 0.3	
Residential, office, hotel, hospital, vehicle parking, etc.	0.3 (in TEC2007, vehicle parking we take n=0.6)	For roof = 0.3	0
		For storeys with correlated occupancies = 0.24	
		For independently occupied storeys = 0.15	
Fixed equipment weight in industrial buildings	1	1	1
Lifting loads by crane	0	-	0
Partitions weight	Use same value of area usage	Use same value of area usage	1 and minimum 0.5 kpa

#### 2.4. Modal Analysis

Modal analysis is technique used to determine dynamic characteristics for structure such as mode shapes and modal vibration periods. The mass (M) and stiffness (K) of

structure are the main factors that the modal analysis depends on [17]. The following show the relationships between K, M, T, f for one degree of freedom structure [17]:

$$[\omega] = \sqrt{\frac{[K]}{[M]}} = 2. \pi. f \quad (2.4)$$

$$T = \frac{1}{f} \quad (2.5)$$

#### **2.4.1. Fundamental Time Period Calculation**

Fundamental vibration period for structure is used to calculate seismic base shear in ELF. The fundamental period of buildings depends on factors such as the height of building, type of seismic resisting structural system, mass and lateral stiffness of structure.

#### **2.4.2. Approximate Fundamental Period**

For calculating fundamental period, we can use modal analysis for structure to give results that are more accurate. Actually, calculating the period by modal analysis for structure is difficult and can be done by computer So TBDY2018 [10], TEC2007 [9], EC8 [11] and ASCE 7-16 [12] codes give approximate method to calculate the fundamental period under conditions as shown in Table 2.9.

**Table 2.9 :** Approximate Fundamental Time Period According to TBDY2018, TEC2007, EC8 and ASCE 7-16 [10, 9, 11, 12]

Code	Formula	Conditions to use
TBDY2018	$T_{pA} = C_t H_N^{\frac{3}{4}} (2.6)$ $T_{pA} = \text{fundamental period in sec.}$ $C_t$ $= \text{factor depends on type of seismic resisting structural system}$ $C_t$ $= 0.07 \text{ for reinforced concrete building , } 0.08 \text{ for steel building}$ $H_N = \text{height of building from the base}$	-
TEC2007	$T = 0.1N (2.7)$ $T = \text{fundamental period in sec}$ $N = \text{number of floor without basement}$	-
EC8	$T_1 = C_t \cdot H^{\frac{3}{4}} (2.8)$ $T_1 = \text{fundamental period in sec}$ $C_t$ $= \text{factor depends on type of seismic resisting structural system}$ $C_t = \text{is } 0,085 \text{ for moment resistant space steel frames,}$ $0,075 \text{ for moment resistant space concrete frames and}$ $\text{for eccentrically braced steel frames and } 0,050 \text{ for all}$ $\text{other structures;}$ $H = \text{is the height of the building, in m, from the foundation or from the}$ $\text{top of a rigid basement.}$	For buildings with heights of up to 40 m
ASCE 7-16	$T_a = C_t h_n^x (2.9)$ $T_a = \text{fundamental period in sec}$ $C_t \text{ and } x$ $= \text{factor depends on type of seismic resisting structural system}$ $C_t \text{ and } x = \text{can be determined from Table 12.8 – 2 in ASCE 7 – 16}$ $h_n = \text{is the height of the building from the foundation or from the top of}$ $\text{a rigid basement}$	-

## 2.5. Nonlinear Analysis

A nonlinear analysis for structure is an analysis with non-linear relationship between load and displacement or with stress and strain. Nonlinearity in structure can be from material nonlinearity (such as elastic-plastic material), geometric nonlinearity (due to large deformations) and contact (boundary) nonlinearity [17]. The effect of nonlinearity in structure is reflected in a stiffness which will change during apply the load or displacement on structure. Because of that, the nonlinear analysis took long time to solve by computer solver.

Actually, the nonlinear analysis can represent the behavior of structure when applying equal increment of load or displacement and calculating the stiffness and reactions for each frame element in structure at each step of analysis [17]. The final solution can be conducted by summation of different solution of each step [17].

In fact, the nonlinear analysis gives more accurate and realistic solution to structure comparing with linear analysis [17]. There are essentially two approaches in nonlinear analysis: displacement controlled and load controlled analysis.

### **2.5.1. Displacement Controlled Approach**

In this approach the displacement controls the analysis, where the displacement during analysis is increasing by use equal value step by step that require increasing unequal value of load. By this way, we can represent the nonlinear behavior for structure [17].

Usually, this approach is preferred than load-controlled approach because the designer can carry out the analysis according to desired value of displacement.

### **2.5.2. Load Controlled Approach**

In this approach the load controls the analysis, where the load during analysis is increasing by use equal value that require increasing unequal value of displacement. At the point where the load value exceeds the strength value, there are no solution. By this way, we can represent the nonlinear behavior for structure [17].

## **2.6. Performance Based Design**

Performance based design is a method used for determining and understanding structure which allow engineers to make assessment for structures and able redesign the structures according to safety and economic criteria [17].

One goal of engineers is to enhance the performance of structures against earthquakes and let them withstand a given level of earthquake shaking with desired performance level [17]. However, what level of ground shaking should they use in analysis? In addition, there are uncertainties about the location, size and intensity of future ground shaking. In order to quantify these uncertainties, Engineers develop Probabilistic Seismic Hazard Analysis (PSHA). Also, PSHA is constructed by use annual probability of exceeding (PE) specific level of ground shaking at specific site location with a range of intensity level [17].

The PSHA can take many forms, which are [17]:

1. Peak ground acceleration (PGA).
2. Peak ground velocity (PGV).

### 3. Response spectral acceleration (RSA).

Typically, with some key structural periods of 0.3 and 0.1 seconds.

In fact, PSHA can be used in two ways:

They can be as hazard map plotting (Figure 2.4) such as the map used in seismic codes or can be used in the construction of the elastic response spectrum. Elastic response spectrum is a plot that represent peak response for linear elastic system single degree of freedom oscillators at a range of natural frequencies or periods [17]. Engineers use PSHA with two ways as stated above in order to visualize seismic hazard in codes. This PSHA based approach, define seismic hazards in the function of geographical location and site conditions. Also, this approach includes earthquakes maps with counters lines and codified Earthquake zoning maps provided from earthquake building codes according to given annual PE at specific level of ground shaking [17].

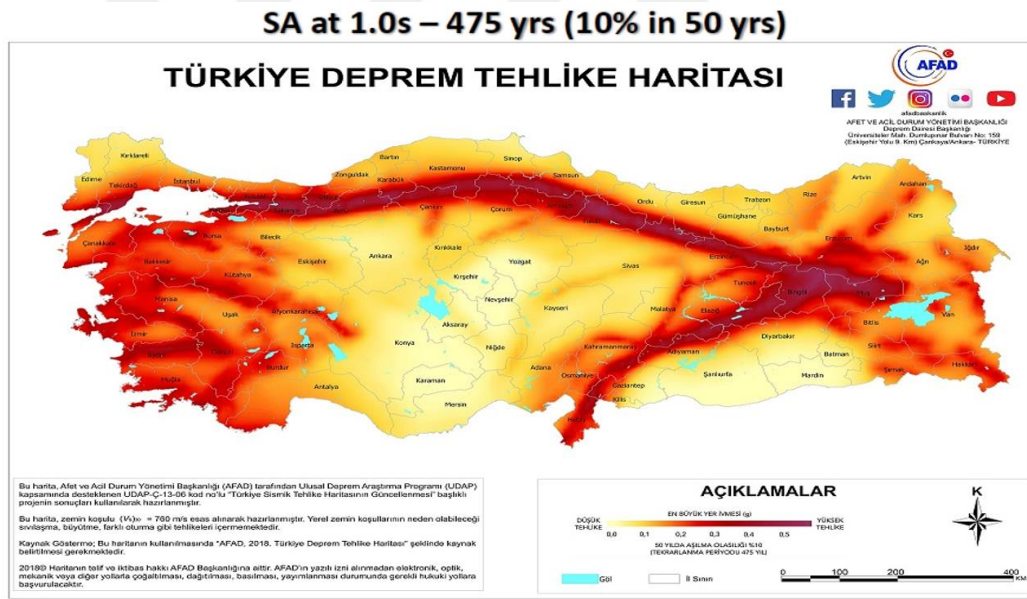


Figure 2.4 : Seismic Hazard Map Plotting [18]

#### 2.6.1. Seismic hazard Map of Turkey

The school building in this study is located in Turkey, so we will use Seismic hazard map for Turkey country that is available at <https://tdth.afad.gov.tr/> website. The seismic hazard maps in turkey have four different probability levels and with different ground motion parameters (PGA, PGV, RSA with damping 5% at 0.2 and 1.0 sec) for return periods of 43, 72, 475 & 2475 years [10] as shown in Table 2.10.

**Table 2.10 : Types of Seismic Hazard Maps in Turkey Description [10]**

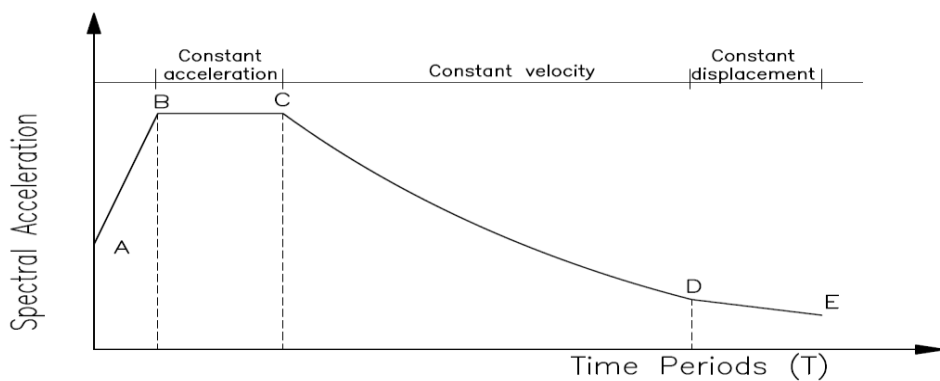
Map type	Probability of exceeding spectral magnitudes in 50 years	Return periods (years)
Earthquake Ground Motion Level-1 (DD-1)	2%	2475
Earthquake Ground Motion Level-2 (DD-2)	10%	475
Earthquake Ground Motion Level-3 (DD-3)	50%	72
Earthquake Ground Motion Level-4 (DD-4)	68%	43

### 2.6.2. Acceleration Response Spectrum

In order to represent seismic actions, all codes used form of spectrum for absolute acceleration against natural time periods as we stated above. ALL codes have the same typical shape of acceleration response spectrum that shown in figure below.

The typical shape of spectrum has four zones, which they are [12]:

- Linear relationship with time periods from A to B.
- Constant acceleration for the range of time periods from B to C.
- Constant velocity for the range of time periods from C to D.
- Constant acceleration for the range of time periods from B to C.
- Constant displacement for the range of time periods from D to E.



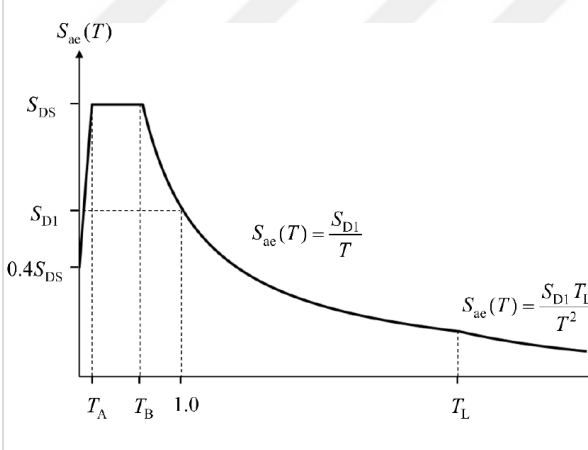
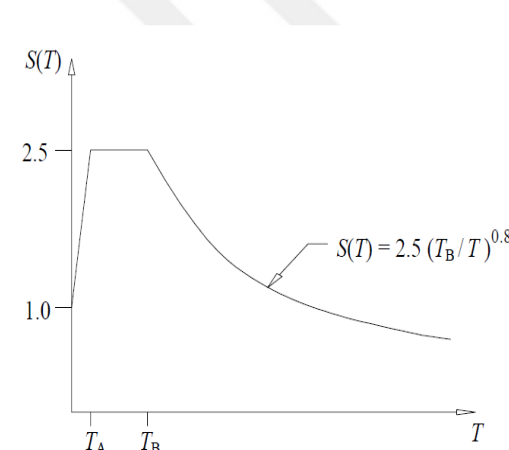
**Figure 2.5 : Typical Shape of Response Spectrum**

EC8 [11] code has two type of acceleration response spectrum, Type 1 & 2. Type 1 can be used for high seismic zone and type 2 can be used as EC8 [11] code state in section 3.2.2.2. “If the earthquakes that contribute most to the seismic hazard defined for the site for the purpose of probabilistic hazard assessment have a surface-wave magnitude,  $M_s$ , not greater than 5.5, it is recommended that the Type 2 spectrum is adopted”.

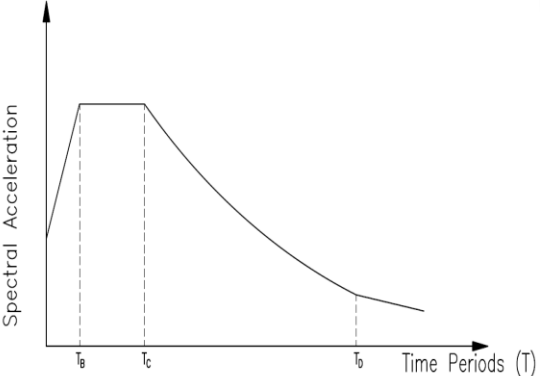
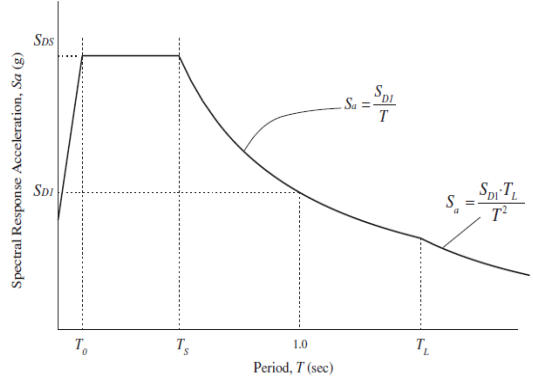
The equations for elastic response spectrum & drawings according to TBDY2018, TEC2007, EC8 and ASCE 7-16 codes are shown in Table 2.11, Table 2.12, Figure 2.6, Figure 2.7, Figure 2.8, Figure 2.9.



**Table 2.11** : Elastic Response Spectrum Equations According to TBDY2018 and TEC2007 [10, 9]

TBDY2018	TEC2007
<p>for <math>(0 \leq T \leq T_A)</math>:</p> $S_{ae}(T) = (0.4 + 0.6 \frac{T}{T_A}) S_{DS} \quad (2.10)$ <p>for <math>(T_A \leq T \leq T_B)</math>:</p> $S_{ae}(T) = S_{DS} \quad (2.11)$ <p>for <math>(T_B \leq T \leq T_L)</math>:</p> $S_{ae}(T) = \frac{S_{D1}}{T} \quad (2.12)$ <p>for <math>(T_L \leq T)</math>:</p> $S_{ae}(T) = \frac{S_{D1} T_L}{T^2} \quad (2.13)$ <p>Where <math>T_A = 0.2 \frac{S_{D1}}{S_{DS}}</math> ; <math>T_B = \frac{S_{D1}}{S_{DS}}</math> ; <math>T_L = 6S</math></p>	<p>for <math>(0 \leq T \leq T_A)</math>:</p> $S(T) = 1 + 1.5 \frac{T}{T_A} \quad (2.14)$ <p>for <math>(T_A \leq T \leq T_B)</math>:</p> $S(T) = 2.5 \quad (2.15)$ <p>for <math>(T_B \leq T)</math>:</p> $S(T) = 2.5 (\frac{T_B}{T})^{0.8} \quad (2.16)$ <p>Where <math>T_A</math> and <math>T_B</math> found in table 2.4 of TEC2007 code</p>
TBDY2018	TEC2007
	
<p><b>Figure 2.6</b> : Shape of Elastic Response Spectrum for TBDY2018 [9]</p>	<p><b>Figure 2.7</b> : Shape of Elastic Response Spectrum for TEC2007 [9]</p>

**Table 2.12** : Elastic Response Spectrum Equations According to EC8 and ASCE 7-16 [11, 12]

EC8	ASCE 7-16
<p>for <math>(0 \leq T \leq T_B)</math>:  <math display="block">\mathbf{S}_e(\mathbf{T}) = \mathbf{a}_g \cdot \mathbf{S} \cdot \left[ 1 + \frac{T}{T_B} \cdot (\eta \cdot 2, 5 - 1) \right] \quad (2.17)</math> </p> <p>for <math>(T_B \leq T \leq T_C)</math>:  <math display="block">\mathbf{S}_e(\mathbf{T}) = \mathbf{a}_g \cdot \mathbf{S} \cdot \eta \cdot 2, 5 \quad (2.18)</math> </p> <p>for <math>(T_C \leq T \leq T_D)</math>:  <math display="block">\mathbf{S}_e(\mathbf{T}) = \mathbf{a}_g \cdot \mathbf{S} \cdot \eta \cdot 2, 5 \left[ \frac{T_C}{T} \right] \quad (2.19)</math> </p> <p>for <math>(T_D \leq T)</math>:  <math display="block">\mathbf{S}_e(\mathbf{T}) = \mathbf{a}_g \cdot \mathbf{S} \cdot \eta \cdot 2, 5 \left[ \frac{T_C T_D}{T^2} \right] \quad (2.20)</math> </p> <p>Where <math>\mathbf{a}_g = \gamma_I \cdot \mathbf{a}_{gR}</math> ,  <math>\mathbf{a}_{gR}</math> = peak ground acceleration on type A ground,  <math>\gamma_I</math> = importance factor  <math>\mathbf{S}</math> = soil factor  <math>\eta</math> = damping correction factor where <math>\eta = 1</math> for 5% viscous damping</p>	<p>for <math>(0 \leq T \leq T_0)</math>:  <math display="block">S_a = S_{DS} (0.4 + 0.6 \frac{T}{T_0}) \quad (2.21)</math> </p> <p>for <math>(T_0 \leq T \leq T_s)</math>:  <math display="block">S_a = S_{DS} \quad (2.22)</math> </p> <p>for <math>(T_s \leq T \leq T_L)</math>:  <math display="block">S_a = \frac{S_{D1}}{T} \quad (2.23)</math> </p> <p>for <math>(T_L \leq T)</math>:  <math display="block">S_a = \frac{S_{D1} T_L}{T^2} \quad (2.24)</math> </p> <p>Where <math>T_0 = 0.2 \frac{S_{D1}}{S_{DS}}</math> ; <math>T_s = \frac{S_{D1}}{S_{DS}}</math> ;  <math>T_L</math> = long-period transition period(s)</p>
EC8	ASCE 7-16
 <p><b>Figure 2.8</b> : Shape of Elastic Response Spectrum for EC8 [11]</p>	 <p><b>Figure 2.9</b> : Shape of Elastic Response Spectrum for ASCE 7-16 [12]</p>

The equations for design response spectrum according to TBDY2018, TEC2007, EC8 and ASCE 7-16 codes are shown below in Table 2.13:

**Table 2.13** : Design Response Spectrum Equations According to TBDY2018, TEC2007, EC8 and ASCE 7-16 [10, 9, 11, 12]

TBDY2018	TEC2007
<p>for (<math>0 \leq T \leq T_A</math>):</p> $S_{aR}(T) = \left(0.4 + 0.6 \frac{T}{T_A}\right) S_{DS} * \frac{T_B}{T * [D + (\frac{R}{I} - D)]} \quad (2.25)$ <p>for (<math>T_A \leq T \leq T_B</math>):</p> $S_{aR}(T) = S_{DS} * \frac{T_B}{T * [D + (\frac{R}{I} - D)]} \quad (2.26)$ <p>for (<math>T_B \leq T \leq T_L</math>):</p> $S_{ae}(T) = \frac{S_{D1}}{T} * \frac{1}{R} \quad (2.27)$ <p>for (<math>T_L \leq T</math>):</p> $S_{ae}(T) = \frac{S_{D1} T_L}{T^2} * \frac{1}{R} \quad (2.28)$ <p>Where <math>T_A = 0.2 \frac{S_{D1}}{S_{DS}}</math> ; <math>T_B = \frac{S_{D1}}{S_{DS}}</math> ; <math>T_L = 6S</math>  R= response modification factor  D=over strength factor  I= importance factor</p>	<p>for (<math>0 \leq T \leq T_A</math>):</p> $S(T) = \frac{\left(1 + 1.5 \frac{T}{T_A}\right) A_0 I}{1.5 + (R - 1.5) \left(\frac{T}{T_A}\right)} \quad (2.29)$ <p>for (<math>T_A \leq T \leq T_B</math>):</p> $S(T) = \frac{2.5 * A_0 I}{R} \quad (2.30)$ <p>for (<math>T_B \leq T</math>):</p> $S(T) = \frac{2.5 \left(\frac{T_B}{T}\right)^{0.8} * A_0 I}{R} \quad (2.31)$ <p>Where <math>T_A</math> and <math>T_B</math> found in table 2.4 in TEC2007 code  R= response modification factor  <math>A_0</math>=Effective Ground Acceleration  Coefficient from table 2.2 of TEC2007 code  I= importance factor</p>
EC8	ASCE 7-16
<p>for (<math>0 \leq T \leq T_B</math>):</p> $S_d(T) = a_g \cdot S \cdot \left[ \frac{2}{3} + \frac{T}{T_B} \cdot \left( \frac{2.5}{q} - \frac{2}{3} \right) \right] \quad (2.32)$ <p>for (<math>T_B \leq T \leq T_C</math>):</p> $S_d(T) = a_g \cdot S \cdot \frac{2.5}{q} \quad (2.33)$ <p>for (<math>T_C \leq T \leq T_D</math>):</p> $S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \left[ \frac{T_C}{T} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (2.34)$ <p>for (<math>T_D \leq T</math>):</p> $S_d(T) \begin{cases} = a_g \cdot S \cdot \frac{2.5}{q} \left[ \frac{T_C T_D}{T^2} \right] \\ \geq \beta \cdot a_g \end{cases} \quad (2.35)$ <p>Where <math>a_g = \gamma_I \cdot a_{gR}</math> ,  <math>a_{gR}</math> = <b>peak ground acceleration on type A ground</b>,  <math>\gamma_I</math> = <b>importance factor</b></p>	<p>for (<math>0 \leq T \leq T_0</math>):</p> $S_d = S_{DS} \left(0.4 + 0.6 \frac{T}{T_0}\right) * \frac{I_e}{R} \quad (2.36)$ <p>for (<math>T_0 \leq T \leq T_S</math>):</p> $S_d = S_{DS} * \frac{I_e}{R} \quad (2.37)$ <p>for (<math>T_S \leq T \leq T_L</math>):</p> $S_d = \frac{S_{D1}}{T} * \frac{I_e}{R} \quad (2.38)$ <p>for (<math>T_L \leq T</math>):</p> $S_d = \frac{S_{D1} T_L}{T^2} * \frac{I_e}{R} \quad (2.39)$ <p>Where <math>T_0 = 0.2 \frac{S_{D1}}{S_{DS}}</math> ; <math>T_S = \frac{S_{D1}}{S_{DS}}</math> ;  <math>T_L</math>= long-period transition period(s)  R= response modification factor  <math>I_e</math>= importance factor</p>

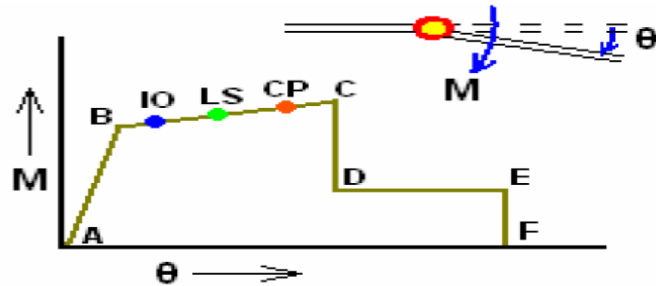
### 2.6.3. Push-over Analysis

One of the techniques used in performance-based design is push-over analysis (PA). PA is a simplified non-linear technique used to obtain the strength capacity of a structure beyond its elastic limit [19].

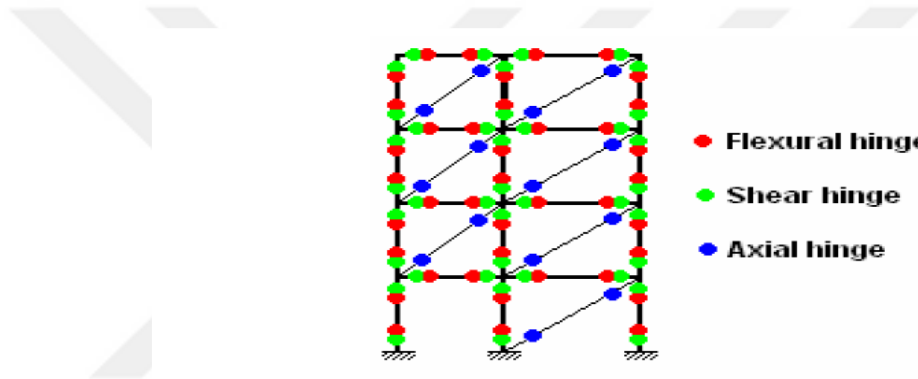
The need for engineers to know the non-linear behavior of a structure due to seismic loads encouraged the development of the PA technique. Actually, PA helps engineers demonstrate how failure occurs in a structure in reality and determine the final failure mode. In addition, the PA process helps in determining the weak point and area in a structure by tracking the damage sequences of each structure element and frame by use of hinges [19].

Hinges are zones or points in structure elements where it is expected to be cracked and yielded in a relatively higher degree than other zones and show high flexural or shear displacements when it is close to its ultimate strength under cyclic loading [19]. Hinges have more than one type which they are flexural, shear and axial hinges. The flexural and shear hinges are located at the ends of beams and columns. The axial hinges are located at the ends of diagonal struts [19].

A hinge represents the local relationship between force and displacement for a structural element from its elastic range to inelastic that under seismic loads [19]. For instance, a flexural hinge represents the local relationship between moment and rotation for a beam as shown in Figure 2.10. A to B represents the linear elastic range from the unloaded state A to its effective yield B. Then, B to C represents the inelastic linear response of ductile stiffness and C to D represents a sudden drop in load resistance. After that, D to E represents a reduced resistance [19]. Finally, E to F represents a total loss of resistance [19]. Typically, hinges are placed in the structural elements for a framed structure as shown in Figure 2.11. These hinges have nonlinear states identified as 'Immediate Occupancy' (IO), 'Life Safety' (LS) and 'Collapse Prevention' (CP) at B to C (ductile) range [19]. This can be done by dividing the B to C range into four parts and denoting IO, LS and CP, which are the states of each individual hinge [19]. Also, the structure as a whole has these states which are called drift limits. The BC range is divided according to different criteria. For example, according to (Inel & Ozmen, 2006) it is at 10%, 60%, and 90% of the range BC for IO, LS and CP respectively [19].



**Figure 2.10** : A Typical Flexural Hinge Property, Showing IO, LS, CP Performance Level [19]



**Figure 2.11** : Locations of Hinges in Framed structure [19]

#### 2.6.4. Push-over Procedures

Many codes describe the PA procedure such as ATC 40, FEMA 273, FEMA 356, 440. Two approaches are introduced in FEMA 440 and ASCE 41-13. In addition, they describe acceptable limits and modeling different components. In FEMA 440 [8], these are the capacity spectrum approach and the displacement coefficient approach. One of limitations of single mode pushover procedure is just considering the first mode shape of the equivalent single degree of freedom system but it is still high effective performance-based analysis procedure because it gives excellent prediction of nonlinear behavior of the structure [20]. The most important thing in any performance-based analysis is the ability to obtain seismic demand and capacity for structure with acceptable degree of certainty [20].

The structure capacity at all depends on the capacity of strength and deformation of each individual component in the structure. For determination the capacities for

structure components beyond the elastic limit, some form of nonlinear analysis is applied such as pushover. Pushover uses a series of sequential elastic analysis, superimposed to obtain approximate force with displacement capacity diagram for the structure. Then, a distribution of lateral force is applied again until more components yield. Actually, the pushover analysis is continued till reaches to instable in structure or till reaches to a previously determined limit.

During an earthquake, structure exposed by complex horizontal lateral displacement due to ground movement. Actually, it is difficult and not practical to trace this horizontal lateral displacement at each time step to obtain design parameter for structure. In order to solve this problem, codes used traditional design method such as ELF method to represent this horizontal lateral displacement. For nonlinear analysis methods, it is better, easier and more direct to represent a complex horizontal displacement Patterns by use a set of lateral displacements [20]. For any given structure and ground movement, the displacement demand is an estimate of the maximum expected response of the building during the ground movement [20]. When a capacity curve and demand displacement are obtained, a performance evaluation can be performed.

#### **2.6.4.1 Capacity Curve**

Capacity curve represents the whole capacity for structure. Usually, it represented by relationship between force applied and deformation for the structure. Capacity curve procedure is obtained by using elastic analysis, superimposed and lateral load pattern applied for the structure [20]. At the start of the procedure, the structure behaves in elastic domain and the force-deformation relationship will be linear. When the structure reaches the elastic limit with increasing the load and starts to behave in inelastic domain and the force-deformation relationship will be non-linear [20]. Finally, the procedure ends upon reaching to pre-specified load, deformation or structure collapse.

#### **2.6.4.2 Demand Curve**

Demand curve represents the maximum ground motion exposed to structure. Also, demand curve is represented by acceleration of ground versus displacement.

### 2.6.4.3 Performance Point and Target Displacement

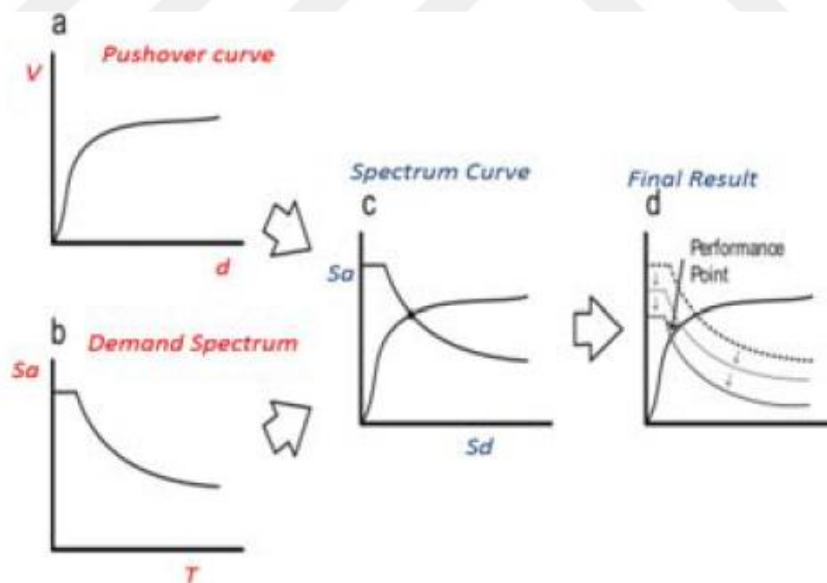
Performance point or target displacement is the aim of push-over analysis method. In order to obtain performance point, Capacity and demand curve are presented at same coordinate and same format. The format of two curves called Acceleration-Displacement Response Spectra (ADRS). The intersection of these two curves is called performance point (Figure 2.12) [21].

In order to obtain target displacement, codes used direct numerical mathematical method and equations.

Finally, there are many methods to obtain performance point but CSI ETABS V18 use four methods which they are:

1. FEMA 440 EQUIVALENT LINEARIZATION [8].
2. NTC 2008 TARGET DISPLACEMENT [22].
3. EC8 2004 TARGET DISPLACEMENT [11].
4. ASCE 41-13 NSP [4].

In this thesis, we will consider FEMA 440 EQUIVALENT LINEARIZATION & ASCE 41-13 NSP methods.



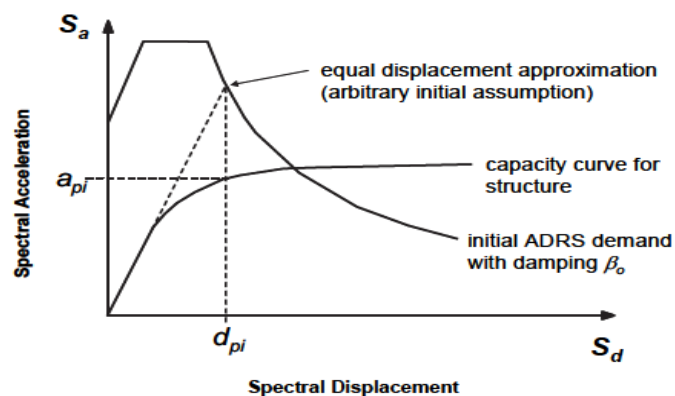
**Figure 2.12** : Performance Point Determination [21]

### 2.6.4.4 FEMA 440 EQUIVALENT LINEARIZATION Procedure

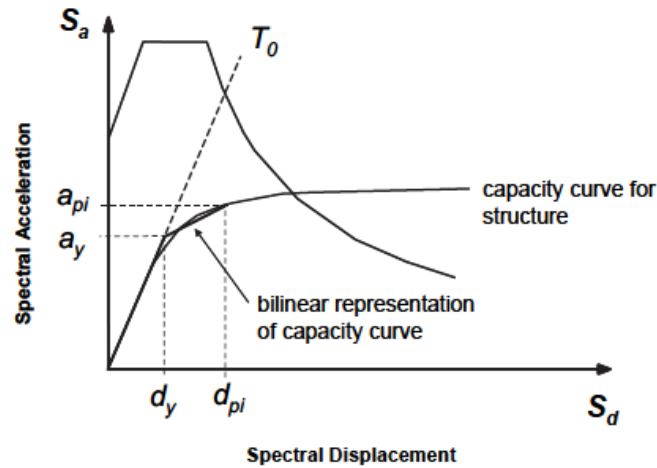
The procedure used in CSI ETABS are called procedure C or EQUIVALENT LINEARIZATION in FEMA 440 [8] code. This approach uses the modified acceleration response spectrum for multiple assumed solutions ( $a_{pi}$ ,  $d_{pi}$ ) and the

corresponding ductilities to generate a locus of possible performance points [8]. The actual performance point is located at the intersection of this locus and the capacity spectrum [8].

1. Select a spectral representation of the ground motion of interest with an initial damping,  $\beta_i$  (normally 5%). This may be a design spectrum from ATC-40 [3] or FEMA 356 [6], a site-specific deterministic spectrum, or an equal hazard probabilistic spectrum [8].
2. Modify the selected spectrum, as appropriate, for soil-structure interaction (SSI) in accordance with the procedures in Chapter 9 in FEMA 440 code. This involves both potential reduction in spectral ordinates for kinematic interaction and a modification in the system damping from the initial value,  $\beta_i$  to  $\beta_0$ , to account for foundation damping. If foundation damping is ignored,  $\beta_0$  is equal to  $\beta_i$  [8].
3. Convert the selected spectrum, modified for SSI when appropriate, to an acceleration-displacement response spectrum format in accordance with the guidance in ATC-40 [3]. This spectrum is the initial ADRS demand (see Figure 2.13) [8].
4. Obtain capacity curve for structure as described in ATC 40 [3] and FEMA 356 [6] code (see Figure 2.13). Also, Capacity curve must be in ADRS format.
5. Select an initial performance point (maximum acceleration,  $a_{pi}$ , and displacement,  $d_{pi}$ ). This may be based on an equal-displacement approximation as shown in Figure 2.13 or any other point based on engineering judgment [8].



**Figure 2.13 : Initial ADRS Demand and Capacity [8]**



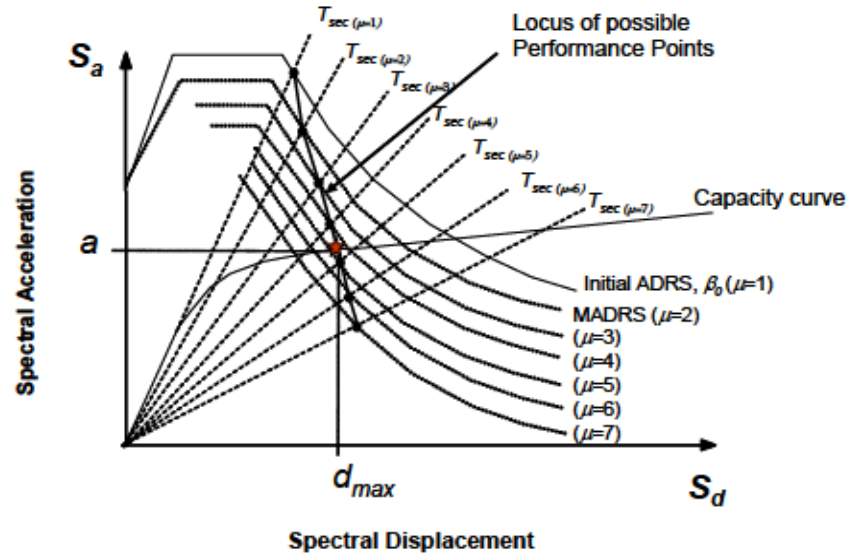
**Figure 2.14** : Bilinear Representation of Capacity Spectrum [7]

6. Develop a bilinear representation of the capacity spectrum in accordance with the procedures in ATC-40 [3]. This defines the initial period,  $T_0$ , yield displacement,  $d_y$ , and yield acceleration,  $a_y$ . (see Figure 2.14). Note that these parameters may vary for differing assumptions  $a_{pi}$  and  $d_{pi}$  [8].
7. For the bilinear representation developed in Step 6, calculate the values of post-elastic stiffness,  $\alpha$ , and ductility,  $\mu$ , as follows [8]:

$$\alpha = \frac{\left(\frac{a_{pi} - a_y}{d_{pi} - d_y}\right)}{\left(\frac{a_y}{d_y}\right)} \quad (2.40)$$

$$\mu = \frac{d_{pi}}{d_y} \quad (2.41)$$

8. Using the calculated values for post-elastic stiffness,  $\alpha$ , and ductility,  $\mu$ , from Step 7, calculate the corresponding effective damping,  $\beta_{eff}$ , (see Section 6.2.1 in FEMA 440 code). Similarly calculate the corresponding effective period,  $T_{eff}$ , (see Section 6.2.2 in FEMA 440 code) [8].
9. Using the effective damping determined from Step 8, adjust the initial ADRS to  $\beta_{eff}$  (see Section 6.3 in FEMA 440 code) [8].
10. Multiply the acceleration ordinates of the ADRS for  $\beta_{eff}$  by the modification factor,  $M$ , determined using the calculated effective period,  $T_{eff}$ , in accordance with Section 6.2.3 in FEMA 440 code to generate the modified acceleration-displacement response spectrum (MADRS) [8].



**Figure 2.15 :** Locus of Possible Performance Points Using MADRS Spectrum [7]

11. A possible performance point is generated by the intersection of the radial secant period,  $T_{sec}$ , with the MADRS (see Figure 2.15) [8].
12. Increase or decrease the assumed performance point and repeat the process to generate a series of possible performance points [8].
13. The actual performance point is defined by the intersection of the locus of points from Step 12 and the capacity spectrum [8].

#### 2.6.4.5 ASCE 41-13 NSP Procedure

ASCE 41-13 NSP [4] procedure called displacement coefficient method (DCM) also. The main object of DCM is to find the target displacement ( $\delta_t$ ). Target displacement is the maximum displacement can roof of structure reach during seismic load design. DCM is direct numerical method used number of coefficients so it can be used in programming unlike FEMA 440 EQUIVALENT LINEARIZATION that used graphical solution which it is difficult in programming.

Also, DCM method explained in Section 3.3.3.3.2 of FEMA 356: 2000 [6]code.

The target displacement can be found by apply equation 2.42 as following [4]:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \quad (2.42)$$

where

$S_a$  = Response spectrum acceleration at the effective fundamental period and damping ratio of the building in the direction under consideration (taken from demand spectrum curve), as calculated in Sections 2.4.1 or 2.4.2 of ASCE 41-13.

$g$  = gravity acceleration.

$C_0$  = Modification factor to relate spectral displacement to building roof displacement.

$C_1$  = Modification factor to relate spectral displacement to building roof displacement. For periods less than 0.2 s,  $C_1$  need not be taken greater than the value at  $T = 0.2$  s. For periods greater than 1.0 s,  $C_1 = 1.0$  [4].

$$= 1 + \frac{\mu_{strength}^{-1}}{aT_e^2} \quad (2.43)$$

where  $a$  = Site class factor:

= 130 Site Class A or B;

= 90 Site Class C;

= 60 Site Class D, E, or F;

$T_s$  = Characteristic period of the response spectrum, defined as period associated with transition from constant acceleration segment of the spectrum to the constant velocity segment of the spectrum (to be calculated from demand spectrum curve).

$T_e$  = Effective fundamental time period [4].

$$= T_i \sqrt{\frac{K_i}{K_e}} \quad (2.44)$$

$K_i$  = Elastic lateral stiffness of the building.

$\mu_{strength}$  = Ratio of elastic strength demand to calculated yield strength coefficient [4].

$$= \frac{S_a}{V_y/W} \cdot C_m \quad (2.45)$$

$V_y$  = Effective yield strength calculated using the capacity curve.

$W$  = Effective seismic weight.

$C_m$  = Effective mass factor as determined by Table 7-4 of ASCE 41-13 [4] code.

$C_2$  = Modification factor to represent the effect of pinched hysteresis shape, cyclic stiffness degradation, and strength deterioration on the maximum displacement response. For periods greater than 0.7 s,  $C_2 = 1.0$  [4];

$$= 1 + \frac{1}{800} \left( \frac{\mu_{strength}^{-1}}{T_e} \right)^2 \quad (2.46)$$



### 3. DESCRIPTION OF THE BUILDING MODEL

#### 3.1. Objective

The purpose of this chapter is to present the description of the building model, including information about loading, materials, location, cross sections and type of building.

#### 3.2. General Information on the Structure

The fictitious structure used in this study, is a reinforced concrete frame structure which is used as a school building and located in Bakırköy, Istanbul with Cartesian coordinates (40.97616419057289°, 28.873216939473792°). It consists of five stories and has a square plan, which has length of 20 meter with equal story heights. The total height of building is 15 meters and each story has a 3-meter height. The structure is symmetric in both plan and elevation. Each story plan has an area equal to 20X20=400-meter square and has 5 bays in both X and Y directions.

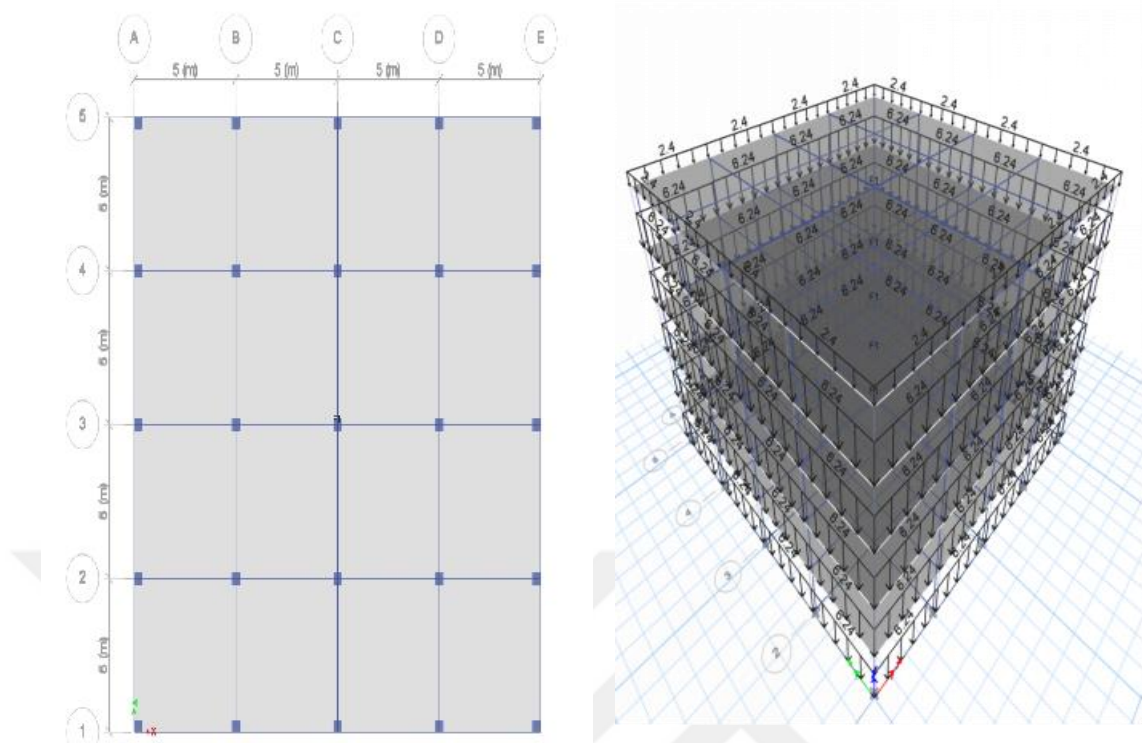
The slab in each story has a thickness of 0.2 meter. The beams in each story have a cross section of 0.3 m X 0.6 m. All columns of structure have 0.4-meter depth and 0.4-meter width.

The structure is located in soil has a shear velocity equal to 1500 feet per second or 457.2 meter per second.

The data of the structure is summarized in Table 3.1.

**Table 3.1 :** Data of Structure

No of stories	5
Dimension of structure b x w (m)	20 x 20
Height of each story (m) (for all stories)	3
Slab thickness (m) (for all stories)	0.2
Beam cross section b x h (m) (for all stories)	0.3 x 0.6
Columns section for all stories b x h (m)	0.4 x 0.4
Location of building	40.97616419057289°, 28.873216939473792°
Shear velocity of soil	1500 ft/s (457.2 m/s)
Type of seismic resisting system	Reinforced concrete moment resisting frames



**Figure 3.1 : 3D and Plan for Structure**

### 3.3. Properties of Structure Materials

This section presents the mechanical characteristics of structure materials (concrete and reinforcement steel) used in our school building according to Turkish Standard TS500 [23] as following:

➤ For concrete:

Unit weight of concrete =  $\gamma_c = 25 \text{ kN/m}^3$

Concrete Grade = C25 for beams and slabs and C30 for columns

Characteristic of concrete strength =  $f_{ck}$  for C25= 25 MPa and  $f_{ck}$  for C30 = 30 MPa

Working compression strength [23] :

$$0.85 f_{ck} \quad (3.1)$$

$$\text{C25: } 0.85 * 25 = 21.25 \text{ MPa}$$

$$\text{C30: } 0.85 * 30 = 25.5 \text{ MPa}$$

Tensile strength [23] :

$$f_{ctk} = 0.35 \sqrt{f_{ck}} \quad (3.2)$$

$$C25: 0.35 \sqrt{25} = 1.75 \text{ MPa}$$

$$C30: 0.35 \sqrt{30} = 1.92 \text{ MPa}$$

Design compressive and tensile strength of concrete [23]:

$$f_{cd} = \frac{0.85 f_{ck}}{\gamma_c} \quad (3.3)$$

$$f_{ctd} = \frac{0.85 f_{ctk}}{\gamma_c} \quad (3.4)$$

Where strength reduction factor for concrete  $\gamma_c = 1.5$

C25:

$$f_{cd} = \frac{21.25}{1.5} = 14.17 \text{ MPa}$$

$$f_{ctd} = \frac{1.75}{1.5} = 1.167 \text{ MPa}$$

C30:

$$f_{cd} = \frac{21.25}{1.5} = 17 \text{ MPa}$$

$$f_{ctd} = \frac{1.92}{1.5} = 1.28 \text{ MPa}$$

Concrete Modulus of elasticity ( $E_c$ ) [23]:

$$E_c = 3250 * \sqrt{f_{ck}} + 14000 \quad (\text{MPa}) \quad (3.5)$$

$$C25: E_c = 3250 * \sqrt{25} + 14000 = 30250 \text{ MPa}$$

$$C30: E_c = 3250 * \sqrt{30} + 14000 = 31800 \text{ MPa}$$

➤ Reinforcement steel:

Reinforcement steel Grade = S420a

Yield strength =  $f_{yk} = 420 \text{ MPa}$

Rupture strength =  $f_{su} = 500 \text{ MPa}$

Steel Modulus of elasticity ( $E_s$ ) =  $2 \times 10^5 \text{ MPa}$

Strength reduction factor for steel  $\gamma_s = 1.15$  so the reduced strength for reinforcement steel can be calculated as following [23]:

$$F_{yd} = \frac{f_{yk}}{\gamma_s} \quad (3.6)$$

$$F_{yd} = \frac{420}{1.15} = 365.22 \text{ MPa}$$

$$F_{ud} = \frac{f_{su}}{\gamma_s} \quad (3.7)$$

$$F_{ud} = \frac{500}{1.15} = 434.78 \text{ MPa}$$

### 3.4. Loading

For Dead load:

The dead load of chosen school building consists of the self-weight of structure, Perimeter infill walls for roof that have height of 1 m and for rest of building that have height of 2.6 and finishing load for floor consist of 10 cm height and 2 cm height of tiles as following:

### 3.5. Dead Load on Slab

- Slab thickness =  $t_s = 0.2 \text{ m}$  (for all slab)
- Unit weight of concrete =  $\gamma_c = 25 \text{ kN/m}^3$
- Unit weight of sand =  $\gamma_s = 18 \text{ kN/m}^3$
- Unit weight of Tile =  $\gamma_t = 20 \text{ kN/m}^3$
- Self-weight of slab =  $t_s * \gamma_c = 0.2 * 25 = 5 \text{ kN/m}^2$
- Dead loads on slabs that include 10 cm sand height =  $0.1 * \gamma_s = 0.1 * 18 = 1.8 \text{ kN/m}^2$
- Dead loads on slabs that 2 cm tile thickness =  $0.02 * \gamma_t = 0.02 * 20 = 0.4 \text{ kN/m}^2$
- So the total dead load on slab =  $5 + 1.8 + 0.4 = 7.2 \text{ kN/m}^2$

➤ Beams and columns dead load:

This include self-weight of beam and columns, perimeter infill walls on beams as following:

- Unit weight of concrete =  $\gamma_c = 25 \text{ kN/m}^3$
- Unit weight of block =  $\gamma_b = 12 \text{ kN/m}^3$
- Beam cross section  $b_b \times h_b$  for all stories =  $0.3 \text{ m} \times 0.6 \text{ m}$

- Columns section for all stories  $b_c \times h_c$  (m) = 0.4 m x 0.4 m
- Self-weight of drop part of beams =  $\gamma_c \times b_b \times (h_b - t_s) = 25 \times 0.3 \times (0.6 - 0.2) = 3$  kN/m
- Self-weight of columns =  $\gamma_c \times b_c \times h_c = 25 \times 0.4 \times 0.4 = 4$  kN/m
- weight of infill walls on beams =  $\gamma_b \times$  width of infill walls x height of infill walls.
- weight of roof infill walls on roof perimeter beams =  $12 \times 0.2 \times 1 = 2.4$  kN/m
- weight of infill walls on the rest of perimeter beams =  $12 \times 0.2 \times 2.6 = 6.24$  kN/m

Live load:

The live load on slab for school building is obtained according to Turkish Standard TS498 [24], Eurocode (EC1) [25], ASCE 7-16 [12] separately that shown in Table 3.2 but the partition weight chosen uniform value that equal about 0.75 kN/m<sup>2</sup> for all cases.

**Table 3.2 :** Live Load Value for School According to Turkish Standard TS498, Eurocode (EC1), ASCE 7-16 [24, 25, 12]

Type of live load	Value of live load		
	TS498	EC1	ASCE 7-16
Roof live load (kN/m <sup>2</sup> )	2 (ÇİZELGE 7)	1 (CATEGORY H IN TABLE 6.1)	1 (for ordinary flat roof and used for maintenance work maybe TABLE 4.3-1)
Live load for school usage floor(kN/m <sup>2</sup> )	5 (ÇİZELGE 7)	2 (FROM TABLE 6.1&6.2 FOR SCHOOL USAGE AREA)	2 (if we consider all floors area are classrooms TABLE 4.3-1)
Partitions live load for all stories except roof story (kN/m <sup>2</sup> )	0.75	0.75	0.75

### 3.6. Cracked Section Stiffness Modification Factors

The structural members during earthquake exposed to cracks and this make the stiffness of structural members to be lower. In order to take account for cracking, codes introduce load, safety and modification factors.

In designing against only gravity load, codes introduce load factor to take account for this stiffness modification but if lateral loads are present, the codes give recommendation of stiffens reduction for each structural member to take account for this cracking rather than load factor as following:

- For TBDY2018 [10] & ASCE 7-16 [12] code:

A) For beam member:

$$\text{Moment of inertia} = 0.35 I_g \quad (3.8)$$

B) For Column member:

$$\text{Moment of inertia} = 0.70 I_g \quad (3.9)$$

Where  $I_g$  = moment of inertia for non-cracked section.

- For TEC2007 code [9]:

A) For beam member:

$$(EI)_e = 0.40 (EI)_o \quad (3.10)$$

B) For columns and frames members:

$$(EI)_e = 0.40 (EI)_o \quad \text{if } ND / (Ac f_{cm}) \leq 0.10 \quad (3.11)$$

$$(EI)_e = 0.80 (EI)_o \quad \text{if } ND / (Ac f_{cm}) \geq 0.40 \quad (3.12)$$

For  $ND / (Ac f_{cm})$  values between 0.1 to 0.4, we can use linear interpolation.

Where  $(EI)_o$  = bending stiffness for non-cracked section.

$(EI)_e$  = bending stiffness for cracked section.

$A_c$  = area of gross cross section.

$f_{cm}$  = strength of concrete section member.

$N_d$  = factored axial force.

$N_d$  &  $N_d / (Ac f_{cm})$  values for our school building can be found in appendix A.2. for each column by take maximum value of load combination that is taken from ETABS.

For simplification, we try to uniform the value of  $(EI)_e$  for columns at each story as following Table 3.3:

**Table 3.3** : Values of Stiffness Modification Factors for Beams & Columns According to TEC2007 code

Story	Columns modification factor $(EI)_e$	beams modification factor $(EI)_e$	Notes
Story 5 (roof)	0.4	0.4	For each story columns, we take average of values.
Story 4	0.46	0.4	
Story 3	0.56	0.4	
Story 2	0.65	0.4	
Story 1	0.71	0.4	

- For EC8 code [11]:

- For beam and column members:

$$\text{Moment of inertia} = 0.5 I_g \quad (3.13)$$

Where  $I_g$  = moment of inertia for non-cracked section.

## 4. COMPARATIVE STUDY OF SEISMIC ANALYSIS FOR SCHOOL BUILDING ACCORDING TO TBDY2018, TEC2007, ASCE7-16 AND EC8 CODES CASE STUDY

### 4.1. Objective

The objective of this chapter is to make comparison between the base shear obtained from four seismic codes.

### 4.2. Calculations of Seismic and Site Coefficients for our School Building

For 0.2 sec spectral acceleration ( $S_s$ ), 1 sec spectral acceleration ( $S_1$ ) and ground acceleration in terms of g (PGA) and for ground acceleration ( $A_0$ ) from old Turkey seismic map at Earthquake Ground Motion Level-2 (DD-2) to ensure life safety are presented in Table 4.1 as following:

**Table 4.1 :** Value of  $S_s$ ,  $S_1$ , PGA,  $A_0$

$S_s$	$S_1$	PGA(g)	$A_0$
1.208	0.328	0.495	0.4

For spectrum type in EC8 code, spectrum type 1 chosen for high seismic zone because building located in Turkey-Istanbul.

For site class and ground types are presented in Table 4.2 as following:

**Table 4.2 :** Value of Site Class and Ground Types

TBDY2018	TEC2007	EC8	ASCE 7-16
ZC	Z2	B	C

For site coefficients, short and long period site coefficients, soil factor and spectrum periods for spectrum type 1 are presented in Table 4.3 as following:

**Table 4.3 :** Spectrum Periods

TBDY2018		TEC2007		EC8				ASCE 7-16	
$F_s$	F1	$T_A$	$T_B$	S	$T_B$	$T_C$	$T_D$	Fa	Fv
1.2	1.5	0.15	0.4	1.2	0.15	0.5	2.0	1.2	1.5

For  $S_{DS}$ ,  $S_{D1}$  and lower bond factor  $\beta$  are presented in Table 4.4:

**Table 4.4 :** Value of  $S_{DS}$ ,  $S_{D1}$  and Lower Bond Factor  $\beta$

TBDY2018		EC8	ASCE 7-16	
$S_{DS}$	$S_{D1}$	$\beta$	$S_{DS}$	$S_{D1}$
1.4472	0.492	0.2	0.9648	0.328

Table 4.5 show the section where importance factor can be determined for buildings and for our school building:

**Table 4.5 :** TBDY2018, TEC2007, EC8 & ASCE 7-16 Sections to Determine Importance Factor for School Building [10, 9, 11, 12]

TBDY2018 (Table 3.1)	TEC2007 (Table 2.3)	EC8 (Table 4.3)	ASCE 7-16 (Table 12.2-1, C,5)
BKS =1		III	III (Risk category in Table 1.5-1)
Importance factor (I) = 1.5	Importance factor (I) = 1.4	Importance factor ( $\gamma_I$ ) = 1.2	Importance factor ( $I_e$ )=1.25

The Table 4.6 show the Ductility factors for our school:

**Table 4.6 :** Value of Ductility Factors for the School Building

TBDY2018 (Table 4.1, A11.) [10]		TEC2007 (Table 2.5) [9]
Response modification (R)	Overstrength factor (D)	Response modification (R)
8	3	8
EC8 [11]		ASCE 7-16 (Table 12.2-1,C,5) [12]
q = $q_0 \times k_w$ ( $q_0$ from table 5.1) & $k_w$ for structural system with walls, by choose DCH (high ductility) structure		Response modification (R)
4.5x1.3= 5.85		8

### 4.3. Seismic Weight Calculation

#### 4.3.1. Dead Weight Calculation

The dead weight calculations are done by using loads values from section 3.4 where the calculation is being summarized in Table 4.7 as following:

**Table 4.7 :** Dead Load Calculation for School

Type of loading	Height or thickness(m)	Area Load (kN/m <sup>2</sup> ) or linear load (kN/m)	Total length (m), area (m <sup>2</sup> ) or number	Weight (kN)
<b>Roof Story Dead load (kN)</b>				
Perimeter infill walls for roof floor	-	2.4 kN/m	80	192
Dead loads on slab	-	7.2 kN/m <sup>2</sup>	Area of slab =20 x 20 = 400 m <sup>2</sup>	2880
Self-weight of drop part of beams	-	3 kN/m	Total length = 200 m	600
Self-weight of columns	-	4 kN/m	Total length = 37.5 m	150
<b>4,3,2,1 Story Dead load (kN)</b>				
Perimeter infill walls	-	6.24 kN/m	80	499.2 (for one story)
Dead loads on slab	-	7.2 kN/m <sup>2</sup>	Area of slab =20 x 20 = 400 m <sup>2</sup>	2880 (for one story)
Self-weight of drop part of beams	-	3 kN/m	Total length = 200 m	600 (for one story)
Self-weight of columns	-	4 kN/m	Total length = 75 m	300 (for one story)
Total (kN)		192+2880+600+150+4x (499.2+2880+600+300) =20938.8		

### 4.3.2. Live Load Calculation

The live load is taken also from section 3.4 where both old and new seismic Turkish code use same loading code (TS498 [24]). The seismic live load is calculated by multiplying each load value with live load participation factor according to each code as following in Table 4.8.

**Table 4.8 :** Live Load Calculation with Participation Factor for School According to TBDY2018, TEC2007, EC8, ASCE 7-16 [10, 9, 11, 12]

No. of story	Value of live load		
	TBDY2018 & TEC2007	EC8	ASCE 7-16
Roof Story live load weight (kN)	$2 \times 400 \times 0.6 = 480 \text{ kN}$	$1 \times 400 \times 0.6 = 240 \text{ kN}$	-
4 <sup>th</sup> story live load weight (kN)	$5 \times 400 \times 0.6 + 0.75 \times 400 \times 0.6 = 1380 \text{ kN}$	$2 \times 400 \times 0.48 + 0.75 \times 400 \times 0.48 = 528 \text{ kN}$	$0.75 \times 400 = 300 \text{ kN}$
3 <sup>rd</sup> story live load weight (kN)	$5 \times 400 \times 0.6 + 0.75 \times 400 \times 0.6 = 1380 \text{ kN}$	$2 \times 400 \times 0.48 + 0.75 \times 400 \times 0.48 = 528 \text{ kN}$	$0.75 \times 400 = 300 \text{ kN}$
2 <sup>nd</sup> story live load weight (kN)	$5 \times 400 \times 0.6 + 0.75 \times 400 \times 0.6 = 1380 \text{ kN}$	$2 \times 400 \times 0.48 + 0.75 \times 400 \times 0.48 = 528 \text{ kN}$	$0.75 \times 400 = 300 \text{ kN}$
1 <sup>st</sup> story live load weight (kN)	$5 \times 400 \times 0.6 + 0.75 \times 400 \times 0.6 = 1380 \text{ kN}$	$2 \times 400 \times 0.48 + 0.75 \times 400 \times 0.48 = 528 \text{ kN}$	$0.75 \times 400 = 300 \text{ kN}$
Total (kN)	6000 kN	2352 kN	1500 kN

### 4.3.3. Total Seismic Weight Calculation

The total seismic weight for each case is calculated according to TBDY2018, TEC2007, EC8, ASCE 7-16 code separately as shown in Table 4.9.

**Table 4.9 :** Total Seismic Weight Calculation for School According to TBDY2018, TEC2007, EC8, ASCE 7-16 [10, 9, 11, 12]

No. of story	Value of load		
	TBDY2018 & TEC2007	EC8	ASCE 7-16
Total (kN)	$6000 + 20938.8 = 26938.8 \text{ kN}$	$2352 + 20938.8 = 23290.8 \text{ kN}$	$1200 + 20938.8 = 22138.8 \text{ kN}$

#### 4.4. Time Period Calculation

For our school building, the fundamental time period is calculated according to approximate equations and by computer for cracked and un-cracked sections, where the results are shown in Table 4.10 below:

**Table 4.10** : Fundamental Time Period Calculation Results for School Building According to TBDY2018, TEC2007, EC8, ASCE 7-16 [10, 9, 11, 12]

CODE	Data	Approx. Time period value	Time period calculated by ETABS	
			Un-Cracked section	Cracked section
TBDY2018	$C_t = 0.07$ for reinforced concrete building $H_N = 15$ m	$T_{pA} = 0.533$ sec.	$T_{pA}(\text{sec.})$	
			0.7462	0.7462
TEC2007	N = 5 story	T = 0.5 sec.	$T(\text{sec.})$	
			0.789	1.1
EC8	$C_t = 0,075$ for moment resistant space concrete frames $H = 15$ m	$T_1 = 0.572$ sec.	$T_1(\text{sec.})$	
			0.739	1.03
ASCE 7-16	$T_a = C_t h_n^x (12.8 - 7)$ $C_t$ and $x = (0.0466, 0.9)$ for Concrete moment – resisting frames $h_n = 15$ m	$T_a = 0.533$ sec.	$T_a(\text{sec.})$	
			0.72	0.7462

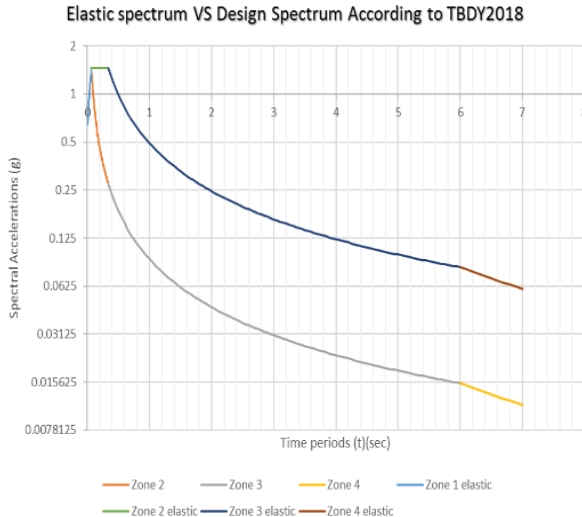
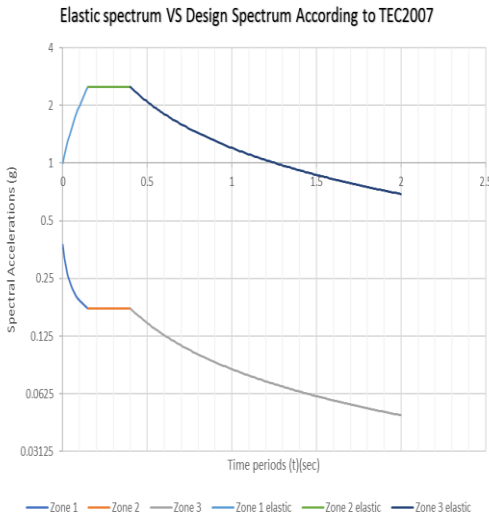
For TBDY2018, Actual calculated ETABS time period is 0.789 sec for un-cracked section and 1.087 sec for cracked section but according to Section 4.7.3.2 of TBDY2018 code  $T_{pA}$  must not be more than  $1.4 \cdot \text{Approx. period}$  [10].

Also for ASCE7-16, the Actual calculated time period is 0.993 sec for Cracked section but according to Section 12.8.2 of ASCE 7-16 code  $T_a$  must not more than Coefficient  $C_u$  (in Table 12.8-1 of ASCE7-16 code )  $\cdot \text{Approx. period}$ , where  $C_u = 1.4$  in our case. ( $1.4 \cdot 0.533 = 0.7462$  sec) [12].

#### 4.5. Spectrum Acceleration Calculation

The calculations for design spectrum acceleration are shown in Table 4.11 and Table 4.12, then plots of elastic spectrum versus design spectrum are perform (Figure 4.1, Figure 4.2, Figure 4.3 & Figure 4.4) for the school building according to TBDY2018, TEC2007, EC8, ASCE 7-16 codes are shown below (where equations parameters are found in Tables 4.3.1, 4.3.2, 4.3.3, 4.3.4 & Tables 5.4.1, 5.2.1 for  $q$  and  $\gamma_I$  of EC8 Code [11]):

**Table 4.11** : Design Response Spectrum Calculations According to TBDY2018 and TEC2007 [10, 9]

<b>TBDY2018</b>	<b>TEC2007</b>
<p>for (<math>0 \leq T \leq T_A</math>):</p> $S_{aR}(T) = \left(0.4 + 0.6 \frac{T}{0.068}\right) * 1.4472 * \frac{0.34}{T * 5.333}$ <p>for (<math>T_A \leq T \leq T_B</math>):</p> $S_{aR}(T) = 1.4472 * \frac{0.34}{T * 5.333}$ <p>for (<math>T_B \leq T \leq T_L</math>):</p> $S_{aR}(T) = \frac{0.492}{T} * \frac{1.5}{8}$ <p>for (<math>T_L \leq T</math>):</p> $S_{aR}(T) = \frac{2.95}{T^2} * \frac{1.5}{8}$ <p>Where <math>T_A = 0.2 * \frac{0.492}{1.4472} = 0.068 \text{ sec}</math>;  <math>T_B = \frac{0.492}{1.4472} = 0.34 \text{ sec}</math>  <math>T_L = 6 \text{ sec}</math>  <math>R = 8</math>  <math>D = 3</math>  <math>I = 1.5</math></p>	<p>for (<math>0 \leq T \leq T_A</math>):</p> $S(T) = \frac{\left(1 + 1.5 \frac{T}{0.15}\right) 0.4 * 1.4}{1.5 + (8 - 1.5) \left(\frac{T}{0.15}\right)}$ <p>for (<math>T_A \leq T \leq T_B</math>):</p> $S(T) = \frac{2.5 * 0.4 * 1.4}{8}$ <p>for (<math>T_B \leq T</math>):</p> $S(T) = \frac{2.5 \left(\frac{0.4}{T}\right)^{0.8} * 0.4 * 1.4}{8}$ <p>Where <math>T_A = 0.15 \text{ sec}</math> and <math>T_B = 0.4 \text{ sec}</math>  <math>R = 8</math>  <math>A_0 = 0.4</math>  <math>I = 1.4</math></p>
<b>TBDY2018</b>	<b>TEC2007</b>
<p>Elastic spectrum VS Design Spectrum According to TBDY2018</p> 	<p>Elastic spectrum VS Design Spectrum According to TEC2007</p> 
<p><b>Figure 4.1</b> : Elastic Spectrum vs Design Spectrum According to TBDY2018 [10]</p>	<p><b>Figure 4.2</b> : Elastic Spectrum vs Design Spectrum According to TEC2007 [9]</p>

**Table 4.12 :** Design Response Spectrum Calculations According to EC8 and ASCE 7-16 [11, 12]

EC8	ASCE 7-16
<p>for (<math>0 \leq T \leq T_B</math>):</p> $S_d(T) = 0.7128 * \left[ \frac{2}{3} + \frac{T}{0.15} * \left( \frac{2.5}{5.85} - \frac{2}{3} \right) \right]$ <p>for (<math>T_B \leq T \leq T_C</math>):</p> $S_d(T) = 0.7128 * \frac{2.5}{5.85}$ <p>for (<math>T_C \leq T \leq T_D</math>):</p> $S_d(T) \left\{ \begin{aligned} &= 0.7128 * \frac{2.5}{5.85} * \left[ \frac{0.5}{T} \right] \\ &\geq 0.1188 \end{aligned} \right.$ <p>for (<math>T_D \leq T</math>):</p> $S_d(T) \left\{ \begin{aligned} &= 0.7128 * \frac{2.5}{5.85} * \left[ \frac{0.5 * 2}{T^2} \right] \\ &\geq 0.1188 \end{aligned} \right.$ <p>Where  <math>a_g = \gamma_I \cdot a_{gR} = 1.2 * 0.495 = 0.594g</math>,  <math>a_{gR} = 0.495</math>  <math>\gamma_I = 1.2</math>; <math>\beta = 0.2</math>; <math>S = 1.2</math>  <math>T_B = 0.15</math> s; <math>T_C = 0.5</math> s; <math>T_D = 2</math> s</p>	<p>for (<math>0 \leq T \leq T_0</math>):</p> $S_d = 0.9648 * \left( 0.4 + 0.6 \frac{T}{0.068} \right) * \frac{1.25}{8}$ <p>for (<math>T_0 \leq T \leq T_s</math>):</p> $S_d = 0.9648 * \frac{1.25}{8}$ <p>for (<math>T_s \leq T \leq T_L</math>):</p> $S_d = \frac{0.328}{T} * \frac{1.25}{8}$ <p>for (<math>T_L \leq T</math>):</p> $S_d = \frac{1.968}{T^2} * \frac{1.25}{8}$ <p>Where <math>T_0 = 0.2 * \frac{0.328}{0.9648} = 0.068</math> sec;  <math>T_s = \frac{0.328}{0.9648} = 0.34</math> sec  <math>T_L = 6</math> sec  <math>R = 8</math>  <math>I_e = 1.25</math></p>
EC8	ASCE 7-16
<p><b>Figure 4.3 :</b> Elastic Spectrum vs Design Spectrum According to EC8 [11]</p>	<p><b>Figure 4.4 :</b> Elastic Spectrum vs Design Spectrum According to ASCE 7-16 [12]</p>

#### 4.6. Base Shear Calculation

In this section, base shear calculation of the school building is carried out according to approximate time period formula (Table 4.13) and calculated fundamental time period by ETABS for un-cracked (Table 4.14) and cracked (Table 4.15) section as following:

**Table 4.13 :** Value of Base Shear for the School Building by the use of Approx. Period

TBDY2018	TEC2007
For approx. period $T_{pA} = 0.533 \text{ sec}$	For approx. period $T = 0.5 \text{ sec}$
$V_{tE}^{(X)} = m_t S_{aR} \left( T_p^{(X)} \right) \geq 0.04 m_t I S_{DS} g$ $V_{tE}^{(X)} = 26938 * \frac{0.492}{0.533} * \frac{1.5}{8} = 4662 \text{ kN}$ $\geq 0.04 * 26938 * 1.5 * 1.4472$ $= 2339 \text{ kN}$	$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10 A_0 I W$ $= \frac{26938 * 0.4 * 1.4 * 2.09}{8} = 3941 \text{ kN} \geq 0.10 * 0.4 * 1.4 * 26938 = 1509 \text{ kN}$
EC8	ASCE 7-16
For approx. period $T_1 = 0.572 \text{ sec}$	For approx. period $T_a = 0.533 \text{ sec}$
$F_b = S_d(T_1) \cdot m \cdot \lambda$ $= 0.7128 * \frac{2.5}{5.85} * \left[ \frac{0.5}{0.572} \right] * 23290.8 * 0.85 = 5271 \text{ kN}$	$V = C_s W \geq 0.044 S_{DS} I_e W$ $V = \frac{0.328}{0.533} * \frac{1.25}{8} * 22138 = 2129 \text{ kN}$ $\geq 0.044 * 0.9648 * 1.25 * 22138 = 1175 \text{ kN}$

Note:  $\lambda$  is the correction factor, the value of which is equal to:  $\lambda = 0,85$  if  $T_1 < 2 T_c$  and the building has more than two stories, or  $\lambda = 1,0$  otherwise [11].

**Table 4.14 :** Value of Base Shear for the School Building by the use of ETABS Calculated Time Period for Un-Cracked Section

TBDY2018	TEC2007
For period $T_{pA} = 0.7462 \text{ sec}$	For period $T_1 = 0.789 \text{ sec}$
$V_{tE}^{(X)} = m_t S_{aR} \left( T_p^{(X)} \right) \geq 0.04 m_t I S_{DS} g$ $V_{tE}^{(X)} = 26938 * \frac{0.492}{0.7462} * \frac{1.5}{8} = 3330 \text{ kN}$ $\geq 0.04 * 26938 * 1.5 * 1.4472 = 2339 \text{ kN}$	$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10 A_0 I W$ $= \frac{26938 * 0.4 * 1.4 * 1.45}{8} = 2734 \text{ kN} \geq 0.10 * 0.4 * 1.4 * 26938 = 1509 \text{ kN}$
EC8	ASCE 7-16
For period $T_1 = 0.739 \text{ sec}$	For period $T_a = 0.72 \text{ sec}$
$F_b = S_d(T_1) \cdot m \cdot \lambda$ $= 0.7128 * \frac{2.5}{5.85} * \left[ \frac{0.5}{0.739} \right] * 23290.8 * 0.85 = 4080 \text{ kN}$	$V = C_s W \geq 0.044 S_{DS} I_e W$ $V = \frac{0.328}{0.72} * \frac{1.25}{8} * 22138 = 1576 \text{ kN}$ $\geq 0.044 * 0.9648 * 1.25 * 22138 = 1175 \text{ kN}$

Note:  $\lambda$  is the correction factor, the value of which is equal to:  $\lambda = 0,85$  if  $T_1 < 2 T_c$  and the building has more than two stories, or  $\lambda = 1,0$  otherwise [11].

**Table 4.15** : Value of Base Shear for Our School Building by Use ETABS Calculated Time Period for Cracked Section

TBDY2018	TEC2007
For period $T_{pA} = 0.7462$ sec	For period $T_1 = 1.1$ sec
$V_{tE}^{(X)} = m_t S_{aR}(T_p^{(X)}) \geq 0.04 m_t I S_{DS} g$ $V_{tE}^{(X)} = 26938 * \frac{0.492}{0.7462} * \frac{1.5}{8}$ $= 3330 \text{ kN}$ $\geq 0.04 * 26938 * 1.5 * 1.4472$ $= 2339 \text{ kN}$	$V_t = \frac{WA(T_1)}{R_a(T_1)} \geq 0.10 A_0 I W$ $= \frac{26938 * 0.4 * 1.4 * 1.11}{8} = 2193 \text{ kN} \geq 0.10 * 0.4 * 1.4 * 26938 = 1509 \text{ kN}$
EC8	ASCE 7-16
For period $T_1 = 0.739$ sec	For period $T_a = 0.7462$ sec
$F_b = S_d(T_1) \cdot m \cdot \lambda$ $= 0.7128 * \frac{2.5}{5.85} * \left[ \frac{0.5}{1.03} \right] * 23290.8 * 1.0 = 3444 \text{ kN}$	$V = C_s W \geq 0.044 S_{DS} I_e W$ $V = \frac{0.328}{0.7462} * \frac{1.25}{8} * 22138$ $= 1520 \text{ kN}$ $\geq 0.044 * 0.9648 * 1.25 * 22138$ $= 1175 \text{ kN}$

Note:  $\lambda$  is the correction factor, the value of which is equal to:  $\lambda = 0,85$  if  $T_1 < 2 T_c$  and the building has more than two stories, or  $\lambda = 1,0$  otherwise [11].

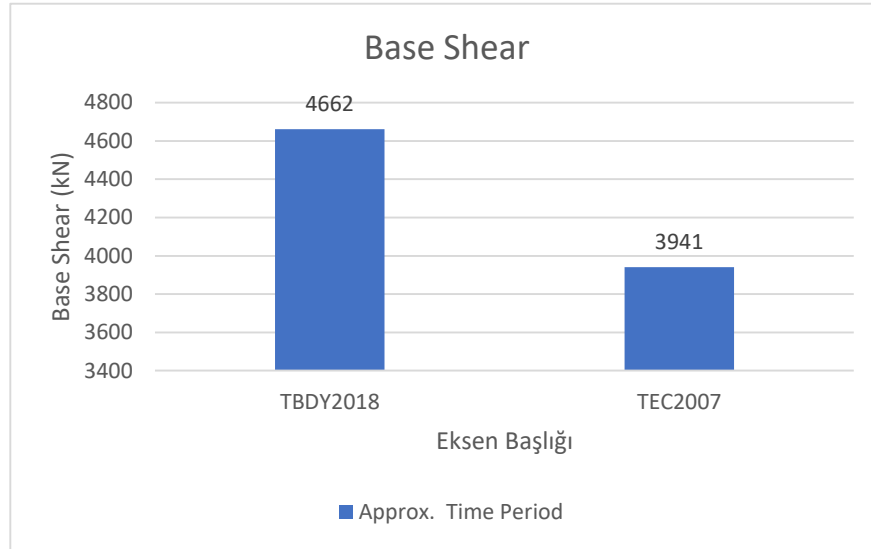
## 4.7. Case Study Conclusion

### 4.7.1. TBDY2018 VS TEC2007

As shown in Figure 4.5 the base shear of TEC2007 code for approx. period is less than TBDY2018 code by 15% although both codes are Turkish code and the seismic weight is similar. The difference in values of base shear is due to following reasons:

1. The importance factor for school usage building is increased in TBDY2018 by 6.7% compared with old Turkish code (TEC2007), so this cause to the base shear to increase by same percentage.
2. The spectral acceleration value is less than in old Turkish code.

According to above reasons, TBDY2018 code increase the safety factors for school buildings which make it safer to use but less economic.



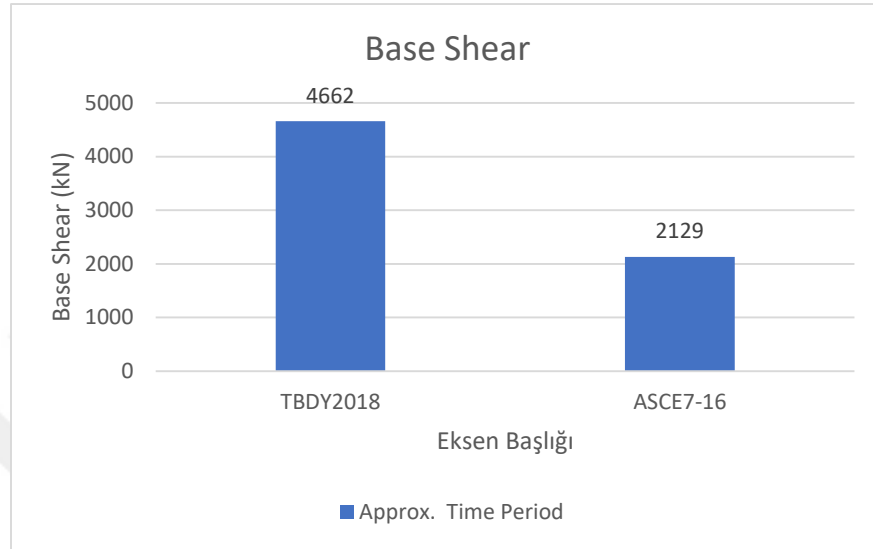
**Figure 4.5 :** Value of Base Shear for School by Use Approx. Time Period According to TBDY2018 and TEC2007

#### 4.7.2. TBDY2018 VS ASCE 7-16

If we observe results of base shear for approx. period according to TBDY2018 and ASCE 7-16 (Table 4.6), ASCE7-16 code gives less value by 54% compared to TBDY2018 code although there is similarity in steps and equations used in both codes. Actually, ASCE7-16 code gives this result because of the followings:

1. The effective seismic weight in ASCE 7-16 code for our school building is smaller than other code (17.8% less than TBDY2018) due to that ASCE7-16 code does not consider most of live load as shown in Table 4.8.
2. The importance factor for school building in ASCE7-16 is less than TBDY2018 which they are 1.25 (16.67% less than TBDY2018).

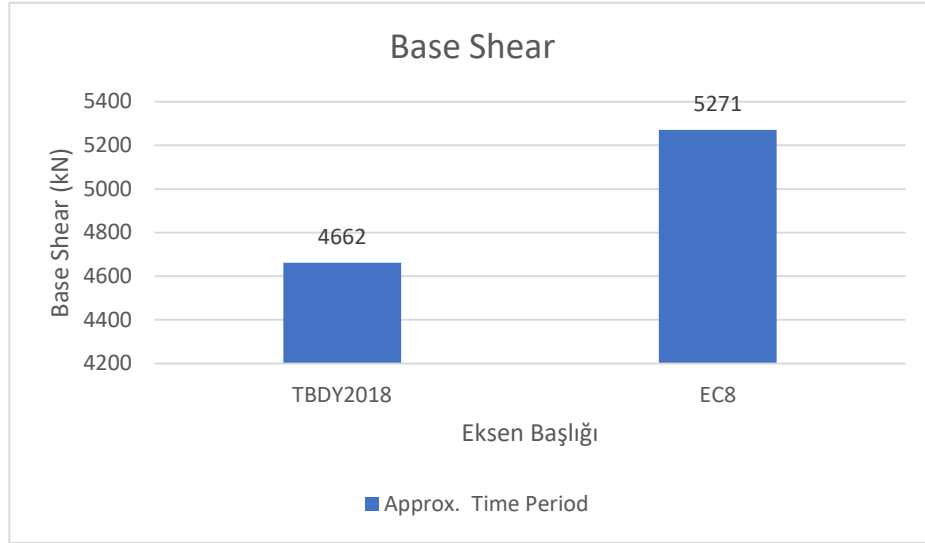
- The ASCE7-16 code just considers 2/3 percentage of design spectral response acceleration parameter at a period of 1.0 s (SD1) and design spectral response acceleration parameter in the short period (SDS) so this will make ASCE7-16 value less than TBDY2018 value for about 33.3% for our school building.



**Figure 4.6 :** Value of Base Shear for School by Use Approx. Time Period According to TBDY2018 and ASCE 7-16

#### 4.7.3. TBDY2018 VS EC8

For approx. time period according to TBDY2018 and EC8 (Figure 4.7), we found the difference in base shear between TBDY2018 and EC8 is 13% more in EC8 although the effective seismic weight in TBDY2018 is more by 15.7% and importance factor is less by 20%, where it is 1.2 in EC8 for school building and 1.5 in TBDY2018. The major reason that make EC8 base shear value is bigger in our school building is the behavior factor in EC8 ( $q=5.85$ ) is less than TBDY2018 ( $R=8$ ) by 26.9% for RC moment resistant frames which make base shear larger.



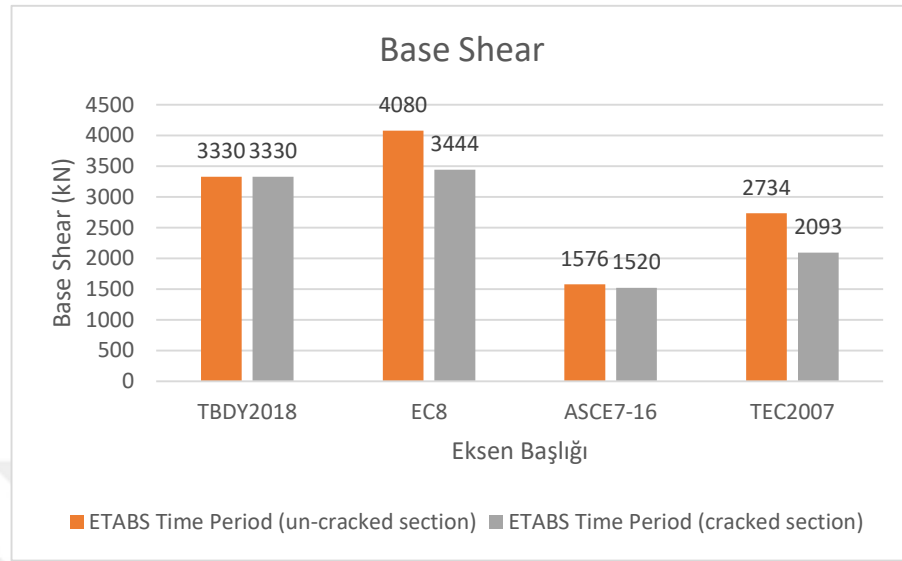
**Figure 4.7 :** Value of Base Shear for School by Use Approx. Time Period According to TBDY2018 and EC8

#### 4.7.4. ETABS Calculated Time Period for cracked & and un-cracked section:

If we observe base shear values calculated by ETABS time period (Figure 4.8) we conduct that:

1. The base shear of ASCE 7-16 code for our school building is still smallest value because the reasons that stated in 4.7.2 section.
2. The values of base in EC8 is still the biggest one.
3. The values of base shear in TEC2007 is lower than TBDY2018 because reasons stated in section 4.7.1. Also, TEC2007 base shear lower than EC8 because the behavior (ductility) factor ( $q=5.85$ ) in EC8 is smaller than ductility factor ( $R=8$ ) in TEC2007 by 26.9%. In other hand, the value of TEC2007 is bigger than ASCE7-16 because Two major factors which they are the effective seismic weight is smaller in ASCE7-16, the importance factor ( $I=1.25$ ) is less by 10.7% in ASCE7-16 compared with one in TEC2007 ( $I=1.4$ ).
4. The values of base shear in TBDY2018 is same for both uncracked and cracked section case because the values of actual time periods for both cases are capped by maximum value of time period for TBDY2018 code.
5. The values of base shear in ASCE 7-16 code is almost same for both uncracked and cracked section case because the value of time period for cracked

section case is capped by maximum value of time period ( $T_a = 0.7462$  sec) of ASCE 7-16 code that make it so close to cracked one ( $T_a = 0.72$  sec)

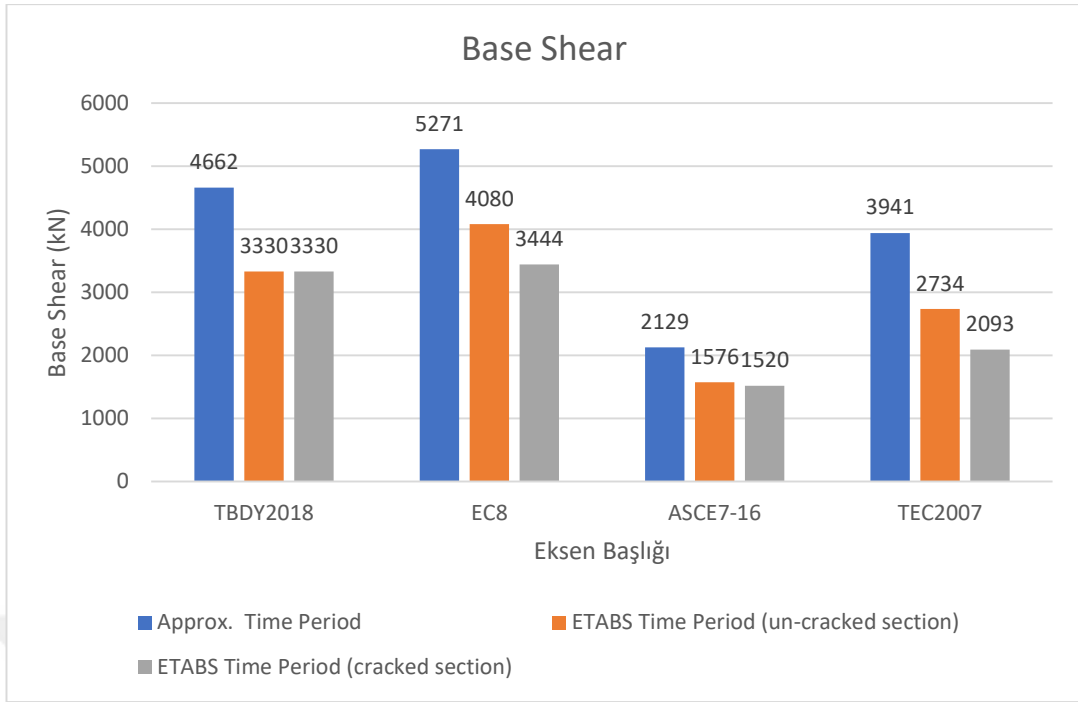


**Figure 4.8 :** Value of Base Shear for School by ETABS Time period according to TBDY2018, TEC2007, EC8 & ASCE7-16

#### 4.7.5. Approximate Time Period VS ETABS Calculated Time Period

As shown in Figure 4.9, the value of base shear is dropped, when it is calculated by ETABS calculated time period due to increase of time period value that make base shear be less for our case of school building in constant velocity zone at spectrum.

The time period calculated by ETABS is approach to reality rather than the approx. value because we take into account the effect of mass & stiffness by using ETABS program. Also, the drop in base shear value is 28.6% in TBDY2018 code and the drop for ASCE7-16 code for both uncracked and cracked section cases is 27.3%. In addition, drop in base shear for TEC2007 code is 30.6%&46.9% for both uncracked and cracked section cases, respectively. Finally, EC8 code base shear dropped by 22.6%&34.7 loss for both uncracked and cracked section cases, respectively.



**Figure 4.9 :** Value of Base Shear for School by Use Approx. and ETABS Time Period According to TBDY2018, TEC2007, EC8 & ASCE7-16

## 5. DESIGN OF SCHOOL STRUCTURE ACCORDING TO THE 2007 TURKISH SEISMIC DESIGN CODE

### 5.1. Objective

The purpose of this chapter to design columns and beams in order to find out missing data such as reinforcement of our school building for make results of pushover analysis more accurate and realistic according to TEC2007, TS500 and TS498 codes.

### 5.2. Mathematical Model by Using CSI ETABS

The structure is modeled, analyzed and designed by using CSI ETABS V18.1.1 [26] structural analysis software. CSI ETABS [26] are structural analysis package programmed by Computer and Structure. CSI ETABS is finite element-based software. Also, it can be used for PA and modal analysis by use two methods, which they are eigen value and Ritz vector. In Addition, it supports almost all of universal code such ASCE7-16 and TBDY2018.

The steps of modelling structure are summarized as following:

1. Choose of design codes and units.
2. Define material properties and cross sections according to codes, section 3.3 & **Hata! Başvuru kaynağı bulunamadı.** in this thesis, where the slabs are defined as rigid diaphragm.
3. Define loads, load cases, modal cases, loads combination and story mass source.
4. Draw model by use draw menu.
5. Assign loads to frames and slabs. Then, assign base supports.

Finally, the model is ready to run static analysis but for modal and nonlinear analysis further steps will needed.

### 5.3. Loads

Loads are defined and assigned in modelling according to section 3.4 for TEC2007 [9], TS500 [23] and TS498 [24] codes.

Loads combination according to TS500 Turkish code as following [23]:

- a) If we consider just gravity load, we use the following load combination:

$$1.4 \text{ Dead load} + 1.6 \text{ Live load} \quad (5.1)$$

b) If we consider gravity load and seismic load, we use the following loads combinations:

$$1.0 \text{ Dead load} + 1.0 \text{ Live load} \pm 1.0 \text{ Seismic load} \quad (5.2)$$

$$0.9 \text{ Dead load} \pm 1.0 \text{ Seismic load} \quad (5.3)$$

### **5.3.1. Statically Seismic Load According to TEC2007**

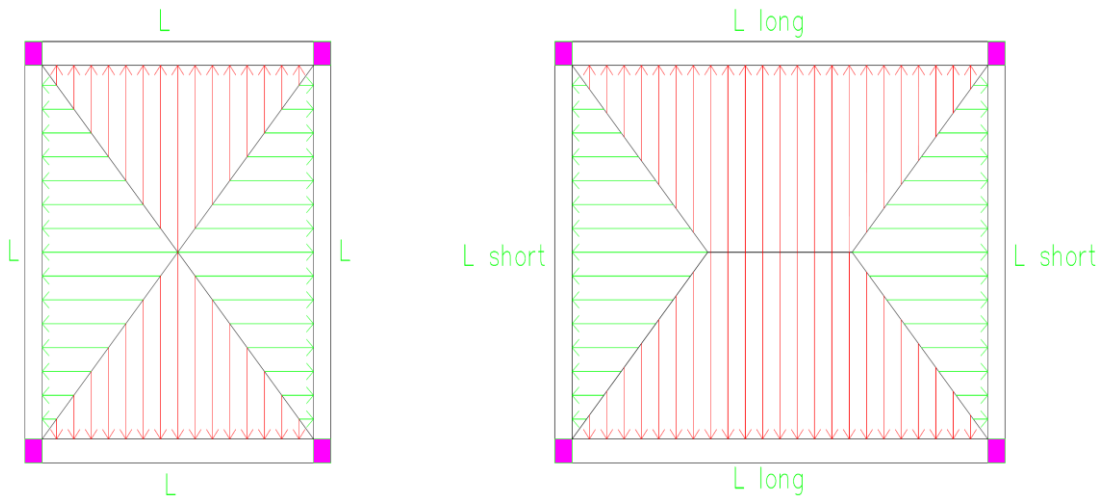
For seismic static analysis, statically lateral load must be applied according to desired seismic code in which each code has its parameters and coefficient as presented in chapter 4.

The required seismic coefficients and parameters for our school building are presented in chapter 4 for TEC2007 code.

The seismic forces are applied to center of mass for each floor in both direction (x and y) and this force resisted by lateral structural system rigidity or what we can say it resisted by center of rigidity of this system. This seismic force causes lateral displacement to building and cause rotation in building if the center of rigidity and center of mass not have same coordinate for each floor or there is eccentricity between them. Actually, seismic codes take account for this eccentricity in calculation and give minimum eccentricity by  $\pm 0.05 * \text{length of story plan}$  that perpendicular to force direction for both axes (x and y) for regular building.

### **5.3.2. Slab Load Distribution to Beam**

The slabs in our school structure modeled as membrane type, so the tributary areas will have triangle shapes for short beams and trapezoidal shapes for long beams but for equal lengths of surrounded beams (square slab) the tributary areas will have just triangle shape as illustrate in Figure 5.1.



**Figure 5.1 : Slab to Beams Load Distribution**

#### **5.4. Story Mass Source:**

In order to perform seismic, modal and nonlinear analysis, the masses that participated in those analysis must be determined. In CSI ETABS, the user defines the mass sources. Then, CSI ETABS lumps each story mass at center of mass of each slab level as discussed in section 2.3.3 for seismic weight in which it is same for seismic weight or mass.

The calculations of dead and load story mass (seismic weight) for each story are presented in section 4.3 according to TEC2007. Also, total mass for each story are presented in Figure 5.2.



**Figure 5.2 :** Story Masses According to TEC2007

### 5.5. Structural Analysis & Design:

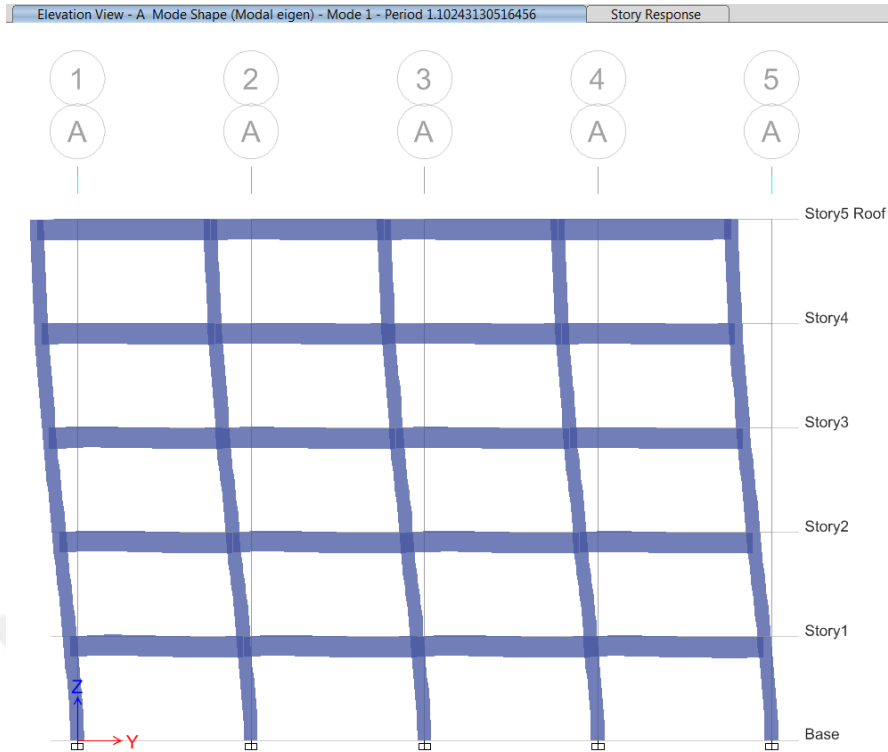
After model of structure in ETABS and check modeling, we can run analysis and make design according to analysis results as following steps.

#### 5.5.1. Modal (eigen) Analysis

After determine of story masses, the modal analysis is ready to run in order to determine time period or frequency for our structure.

The result of fundamental natural time Period (T) equal to 1.1 sec for our school building by use ETABS modal eigen vector analysis as shown in Figure 5.3.

Time period value is same one that calculated in section 4.4 for cracked section according to TEC2007 code, in which it calculated by introduce stiffness modification factor. Stiffness modification factor decrease Cross section stiffness that cause to lesser period time than un-cracked section.



**Figure 5.3 : Fundamental Modal Shape with Time Period (CSI ETABS)**

### 5.5.2 Static Linear Analysis

This analysis is in order to obtain seismic forces by use ELF Method, then obtain members forces, stress, strains and displacements that be used for design of structural members elements.

#### 5.5.2.1. Static Seismic Forces of Stories

CSI ETABS in analysis, calculate base shear as presented in chapter 4 for TEC2007 code. Then, the base shear is distributed to each story in structure. The seismic force distribution depends on Story seismic weight or mass and height of story as following:

- The base shear calculation according to  $T = 1.1$  sec (Cracked section):

$$V_t = 2093 \text{ kN}$$

- $i$ 'th story seismic force [9]:

$$F_i = (V_t - \Delta F_N) \frac{w_i H_i}{\sum_{j=1}^N w_j H_j} \quad (5.4)$$

Where

$$\Delta F_N = 0.0075NV_t \quad (5.5)$$

$w_i$ = weight of i'th story

$w_j$ = weight of j'th story

$H_i$ = height of i'th story

$H_j$ = height of j'th story

$N$ = no of story

- for  $N=5$  and  $H=3$  for all stories and  $w$  from section 5.4, we can calculate seismic story forces as following in Table 5.1 :

**Table 5.1** : Stories Seismic Forces Calculation for School Building According to TEC2007

No. of story	Weight of story( $w_i$ ) (kN)	Height of story ( $H_i$ ) (m)	$w_i * H_i$	$(V_t - \Delta F_N)$	$F_i$ (kN)
Story 5 (roof)	4302	15	64530	2014.51	554.81 +0.0075NV <sub>t</sub> = 633.30
Story 4	5659.2	12	67910.4		583.88
Story 3	5659.2	9	50932.8		437.91
Story 2	5659.2	6	33955.2		291.94
Story 1	5659.2	3	16977.6		145.97
$\sum_{j=1}^N w_j H_j =$			234306	$\sum_{i=1}^N F_i =$	2093

### 5.6. Longitudinal Reinforcement Determination

The longitudinal reinforcement of columns and beam will determinate for our school building in order to use this reinforcement date in PA analysis. The reinforcement determination done by use CSI ETABS according to TS500-2000 code preferences as shown in Figure D.1 in appendix. CSI ETABS design structural against each given load combination. Then, the results for maximum load combination are selected where the governor load combination is:

$$1.0 \text{ Dead load} + 1.0 \text{ Live load} \pm 1.0 \text{ Seismic load}$$

### 5.6.1. Columns Reinforcements

In order to determine adequate reinforcement for columns, the longitudinal reinforcement ratios are selected. Then, number of bars are inputs to CSI ETABS. After that, CSI ETABS check reinforcement and give capacity ratio where it must be lower than 1, if not reinforcement must be increased and make process again.

Table 5.2 show chosen reinforcement for columns.

**Table 5.2 :** Columns Reinforcements According to TS500-2000

no of Story	For longitudinal reinforcement (column cover =40 mm)				
	Dimensions for columns b x h	cross section area cm <sup>2</sup>	Chosen reinforcement ratio %	Chosen Diameter of bars (mm)	Number of bars
Story5 (roof)	40 X 40	1600	1	16	8
Story4	40 X 40	1600	1	16	8
Story3	40 X 40	1600	1	16	8
Story2	40 X 40	1600	1.50	16	12
Story1	40 X 40	1600	1.90	18	12

### 5.6.2. Beams Reinforcements

The beams are designed as rectangular section against bending moment by use CSI ETABS according to Turkish code. CSI ETABS calculates the reinforcement area for beams. Then, designer calculates number of bars as following in Table 5.3.

**Table 5.3 : Beams Reinforcement According to TS500-2000**

no of beam	For negative moment			For positive moment		
	Req reinf. Area cm <sup>2</sup>	Bars Diameter	No of bars	Req reinf. Area cm <sup>2</sup>	Bars Diameter	No of bars
Edge beams Story5	4.2	12	4	4.2	12	4
Internal beams Story5	4.2	12	4	4.2	12	4
Edge beams Story4	6.3	14	5	4.2	12	4
Internal beams Story4	7.3	18	3	5.9	14	4
Edge beams Story3	8.9	16	5	5.4	16	3
Internal beams Story3	9.8	18	4	5.9	14	4
Edge beams Story2	10.7	20	4	7	16	4
Internal beams Story2	11.6	20	4	6.2	16	4
Edge beams Story1	10.7	20	4	7.3	16	4
Internal beam Story1	11.5	20	4	6.5	16	4



## 6. PERFORMANCE BASED ANALYSIS BY PUSHOVER FOR THE SCHOOL BUILDING

### 6.1. Introduction

Performance based analysis are used to determine the behavior of building during earthquake. PA is procedure used in performance-based analysis. The purpose of this chapter is to check performance of our existing school building against seismic load by use pushover analysis. This give answer if our design gives good or poor performance during earthquake. Also, CSI ETABS V18.1.1 [26] is the software will used in pushover.

The steps of PA in CSI ETABS are as following:

1. Create of analyzing model that include defined frame members and shells and materials and applied load.
2. Assign longitudinal reinforcements to frame members as actual.
3. Assign hinges properties to frames.
4. Assign load cases to structure as following.
  - a. Gravity load that applied on structure before lateral load.
  - b. Seismic or lateral load that used in pushing the structure.
5. Assign nonlinear pushover cases that include two cases as following:
  - a. Gravity load nonlinear case that start from initial zero condition by using load control approach.
  - b. Lateral load nonlinear case that start from previous gravity load nonlinear case by using displacement control approach.
6. Run the pushover cases analysis.
7. Finally, CSI ETABS plot capacity curve, give hinge states and forces and reaction results. Also, the performance point or target displacement can be found by assign required demand curve or response spectrum.

Gravity load used in pushover analysis is according to TBDY2018 [10] as following:

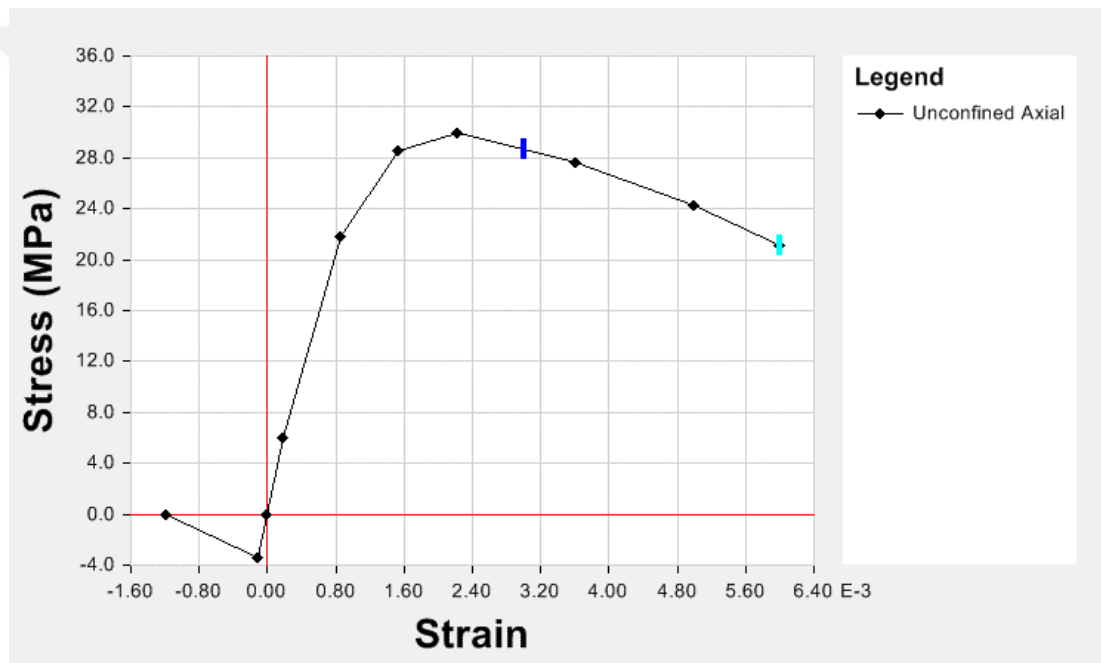
$$\text{Gravity load for pushover analysis} = \text{DEAD LOAD} + n * \text{LIVE LOAD} \quad (66.1)$$

Where  $n$  = moving weight participation factor

$n=0.6$  for school buildings.

In displacement-controlled approach, the displacements are predefined in CSI ETABS and the program monitors this displacement by monitoring predefined joint displacement at roof story. In our case, this joint has label 10 at story 5 roof as shown in Figure D.12.

In addition, ETABS develops for each structural material stress-strain curve that used to determine non-linear properties for structural members as shown in Figure 6.1 for C30 concrete. Also, ETABS may develop more than stress-strain curve for one material for example confined and non-confined reinforced concrete.



**Figure 6.1** : Stress – Strain Curve for C30 Unconfined Concrete [26]

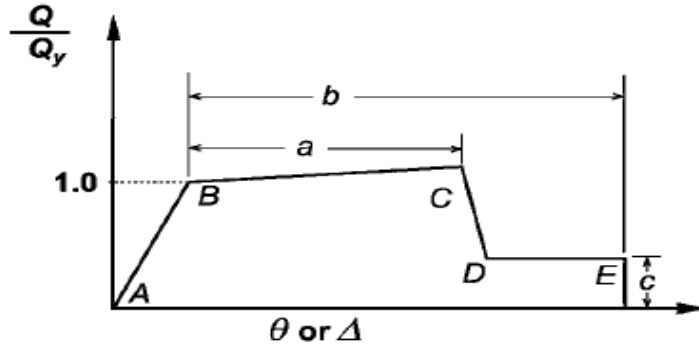
## 6.2. Cracked Section Stiffness Modification Factors for PA analysis

In pushover analysis, the structural sections (beams and columns) are considered to be cracked. The stiffness modification factors are taken according to TBDY2018.

## 6.3. Plastic Hinges Definitions

The plastic hinges are defined at the ends of beams and columns at relative distances that equal 0.05 and 0.95 multiply with member clear length.

The performance acceptance criteria for plastic hinges will be according to ASCE 41-13 code. Also, the plastic hinge generalized moment-rotation curve are shown in Figure 6.2.



**Figure 6.2 :** Generalized Moment-Rotation Curve of Plastic Hinge [4]

The value of  $a$ ,  $b$  and  $c$  can be found in table 6.1 for beam and 6.2 for column. Also,  $Q$  represents the generalized force of the member,  $Q_y$  represent yield strength of member [4].

CSI ETABS V18.1.1 [26] has auto hinge assignment features for frames according to ASCE 41-13 code.

### 6.3.1. Beams Plastic Hinges Definitions

Flexure (M3) hinges are assigned to concrete beams ends according to condition “i” in TABLE 10-7 of ASCE 41-13 [4] code that shown in Table 6.1. M3 hinges means the moment is around strong axis in moment rotation relationship.

The transverse shear reinforcing is considered to be not conformed to ASCE 41-13 provision.

**Table 6.1 : Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures for RC Beams [4]**

**Table 10-7. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Beams**

Conditions			Modeling Parameters <sup>a</sup>			Acceptance Criteria <sup>a</sup>		
			Plastic Rotations Angle (radians)		Residual Strength Ratio	Plastic Rotations Angle (radians)		
			a	b		Performance Level		
					IO	LS	CP	
Condition i. Beams controlled by flexure <sup>b</sup>								
$\rho - \rho'$	Transverse reinforcement <sup>c</sup>	$\frac{V}{b_w d \sqrt{f'_c}}$ <sup>d</sup>						
$\leq 0.0$	C	$\leq 3$ (0.25)	0.025	0.05	0.2	0.010	0.025	0.05
$\leq 0.0$	C	$\geq 6$ (0.5)	0.02	0.04	0.2	0.005	0.02	0.04
$\geq 0.5$	C	$\leq 3$ (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
$\geq 0.5$	C	$\geq 6$ (0.5)	0.015	0.02	0.2	0.005	0.015	0.02
$\leq 0.0$	NC	$\leq 3$ (0.25)	0.02	0.03	0.2	0.005	0.02	0.03
$\leq 0.0$	NC	$\geq 6$ (0.5)	0.01	0.015	0.2	0.0015	0.01	0.015
$\geq 0.5$	NC	$\leq 3$ (0.25)	0.01	0.015	0.2	0.005	0.01	0.015
$\geq 0.5$	NC	$\geq 6$ (0.5)	0.005	0.01	0.2	0.0015	0.005	0.01
Condition ii. Beams controlled by shear <sup>b</sup>								
Stirrup spacing $\leq d/2$			0.0030	0.02	0.2	0.0015	0.01	0.02
Stirrup spacing $> d/2$			0.0030	0.01	0.2	0.0015	0.005	0.01
Condition iii. Beams controlled by inadequate development or splicing along the span <sup>b</sup>								
Stirrup spacing $\leq d/2$			0.0030	0.02	0.0	0.0015	0.01	0.02
Stirrup spacing $> d/2$			0.0030	0.01	0.0	0.0015	0.005	0.01
Condition iv. Beams controlled by inadequate embedment into beam-column joint <sup>b</sup>								
			0.015	0.03	0.2	0.01	0.02	0.03

NOTE:  $f'_c$  is in lb/in.<sup>2</sup> (MPa) units.  
<sup>a</sup>Values between those listed in the table should be determined by linear interpolation.  
<sup>b</sup>Where more than one of conditions i, ii, iii, and iv occur for a given component, use the minimum appropriate numerical value from the table.  
<sup>c</sup>“C” and “NC” are abbreviations for conforming and nonconforming transverse reinforcement, respectively. Transverse reinforcement is conforming if, within the flexural plastic hinge region, hoops are spaced at  $\leq d/3$ , and if, for components of moderate and high ductility demand, the strength provided by the hoops ( $V_s$ ) is at least 3/4 of the design shear. Otherwise, the transverse reinforcement is considered nonconforming.  
<sup>d</sup> $V$  is the design shear force from NSP or NDP.

### 6.3.2. Columns Plastic Hinges Definitions

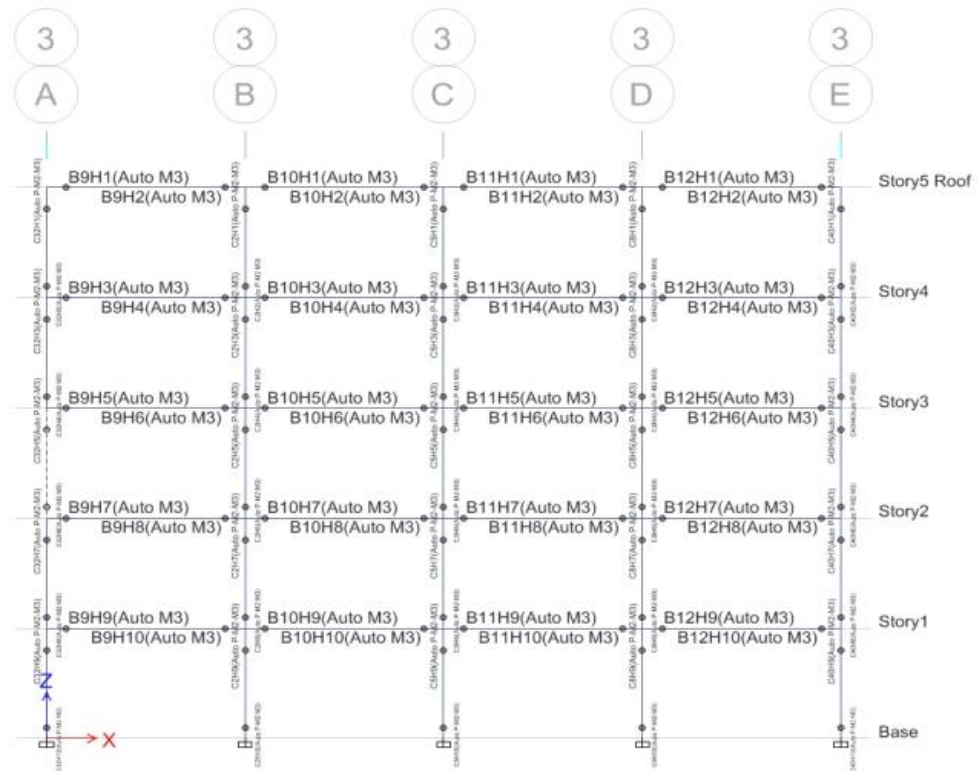
Axial force – Moment (P-M2-M3) hinges are assigned to concrete columns ends (Figure 6.3) according to condition “ii” for flexure or shear failure in TABLE 10-8 of ASCE 41-13 code that shown in Table 6.2. P-M2-M3 hinges means the Axial force and moments around both strong and weak axes relationship.

**Table 6.2 : Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures for RC Columns [4]**

**Table 10-8. Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures—Reinforced Concrete Columns**

Conditions			Modeling Parameters <sup>a</sup>			Acceptance Criteria <sup>a</sup>		
			Plastic Rotations Angle (radians)		Residual Strength Ratio	Plastic Rotations Angle (radians)		
			a	b		Performance Level		
					IO	LS	CP	
Condition ii. <sup>b</sup>								
$\frac{P}{A_g f'_c}$	$\rho = \frac{A_s}{b_w s}$	$\frac{V}{b_w d \sqrt{f'_c}}$ <sup>d</sup>						
$\leq 0.1$	$\geq 0.006$	$\leq 3$ (0.25)	0.032	0.060	0.2	0.005	0.045	0.060
$\leq 0.1$	$\geq 0.006$	$\geq 6$ (0.5)	0.025	0.060	0.2	0.005	0.045	0.060
$\geq 0.6$	$\geq 0.006$	$\leq 3$ (0.25)	0.010	0.010	0.0	0.003	0.009	0.010
$\geq 0.6$	$\geq 0.006$	$\geq 6$ (0.5)	0.008	0.008	0.0	0.003	0.007	0.008
$\leq 0.1$	$\leq 0.0005$	$\leq 3$ (0.25)	0.012	0.012	0.2	0.005	0.010	0.012
$\leq 0.1$	$\leq 0.0005$	$\geq 6$ (0.5)	0.006	0.006	0.2	0.004	0.005	0.006
$\geq 0.6$	$\leq 0.0005$	$\leq 3$ (0.25)	0.004	0.004	0.0	0.002	0.003	0.004
$\geq 0.6$	$\leq 0.0005$	$\geq 6$ (0.5)	0.0	0.0	0.0	0.0	0.0	0.0

NOTE:  $f'_c$  is in lb/in.<sup>2</sup> (MPa) units.  
<sup>a</sup>Values between those listed in the table should be determined by linear interpolation.  
<sup>b</sup>Refer to Section 10.4.2.2.2 for definition of conditions i, ii, and iii. Columns are considered to be controlled by inadequate development or splices where the calculated steel stress at the splice exceeds the steel stress specified by Eq. (10-2). Where more than one of conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.  
<sup>c</sup>Where  $P > 0.7A_g f'_c$ , the plastic rotation angles should be taken as zero for all performance levels unless the column has transverse reinforcement consisting of hoops with 135-degree hooks spaced at  $\leq d/3$  and the strength provided by the hoops ( $V_s$ ) is at least 3/4 of the design shear. Axial load  $P$  should be based on the maximum expected axial loads caused by gravity and earthquake loads.  
<sup>d</sup> $V$  is the design shear force from NSP or NDP.

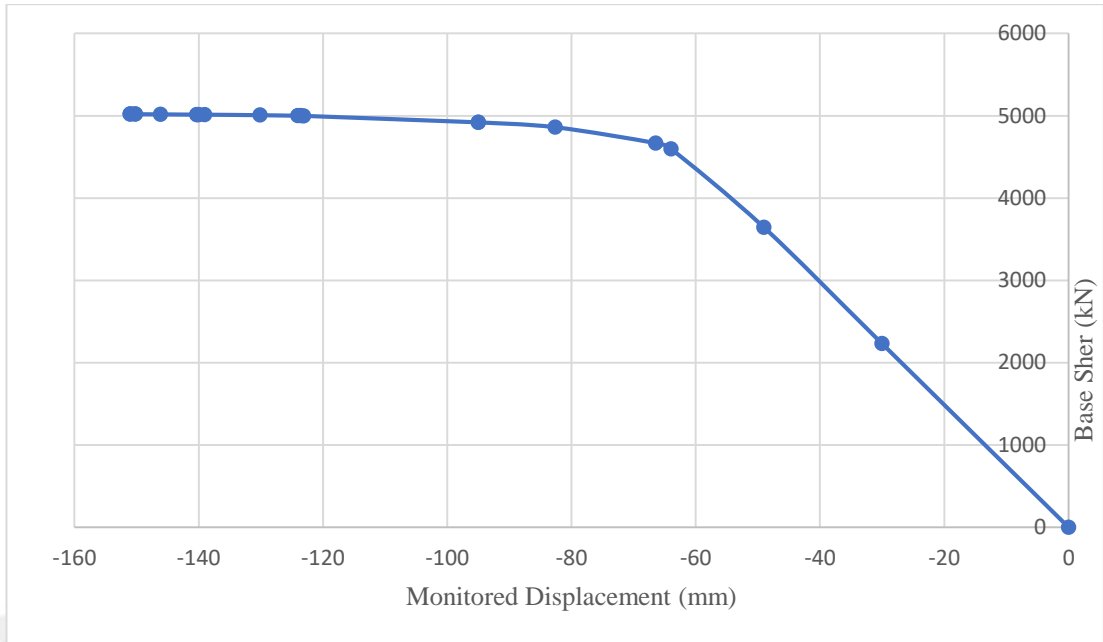


**Figure 6.3 : Frames Hinges Assignment**

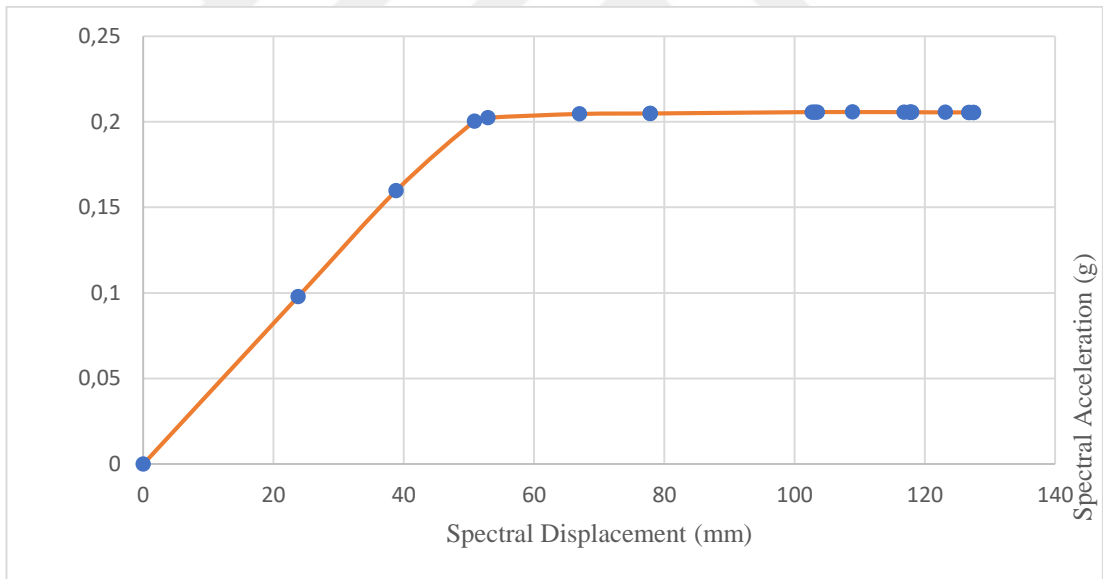
#### **6.4. Pushover Results**

The pushover analysis starts by nonlinear gravity analysis for given dead and live load. Then, CSI ETABS push the structure in chosen direction by use defined lateral load until the predefined displacement value (in our case it is 300mm) is reach to control displacement or the structure became unstable.

During pushover analysis, base shear values versus top displacement for control joint are recorded and plotted to obtain capacity curve as shown in Figure 6.4. Then, the capacity curve is converted to ADRS format (capacity spectrum) ( $S_a$  vs  $S_d$ ) shown in Figure 6.5 in order to find performance point. Actually, one direction (x-axis) is chosen to be pushed because the structure is symmetric in both directions.



**Figure 6.4 :** V vs Monitored Displacement Capacity Curve for Actual School Building

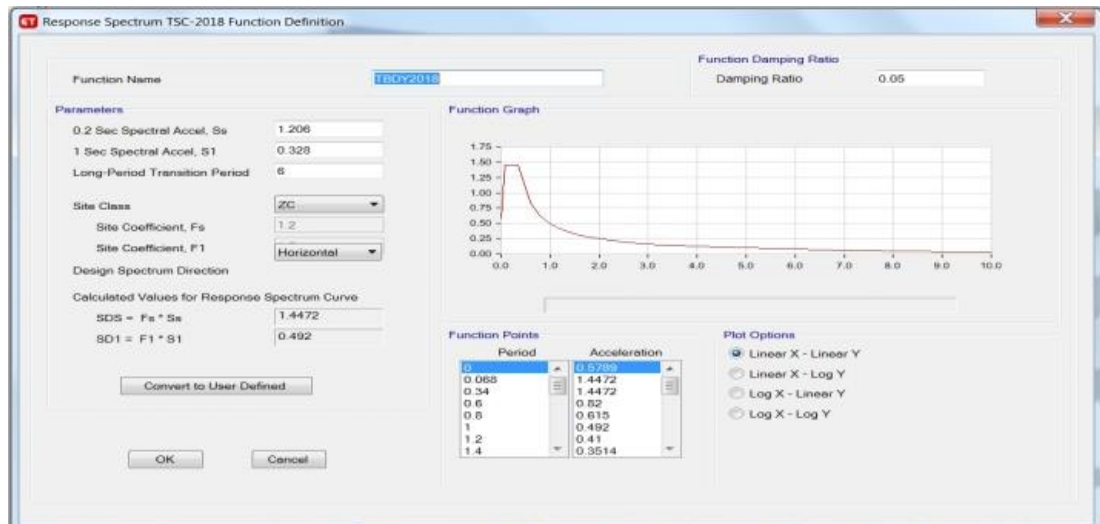


**Figure 6.5 :** Capacity Spectrum (Sa VS Sd) for Actual School Building

### 6.5. Demand Curve

Demand curve for our school building is calculated from elastic response spectrum according to TBDY2018 code (DD-2) &(DD-1) &(DD-3).

CSI ETABS can generate demand curve according to TBDY2018 automatically by inputting the desired response spectrum coefficient and factors as shown in Figure 6.6.



**Figure 6.6 :** Demand Response Spectrum Definition in CSI ETABS for Our School Building

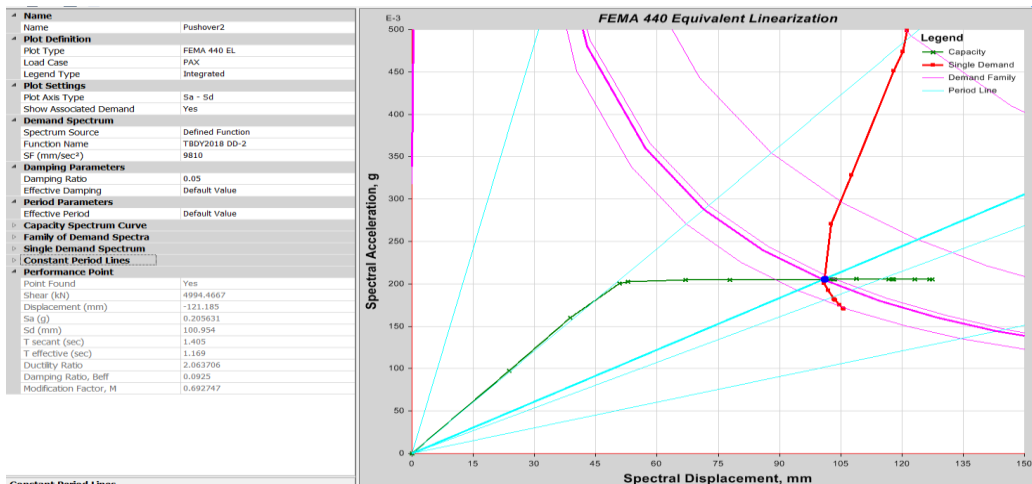
### 6.6. Performance of the School Building

The performance evaluation of structure against seismic loads include the determination of the maximum deformation of the structure and checking if the structure can accommodate this deformation globally and locally. This maximum deformation can be found by determining the performance point or target displacement.

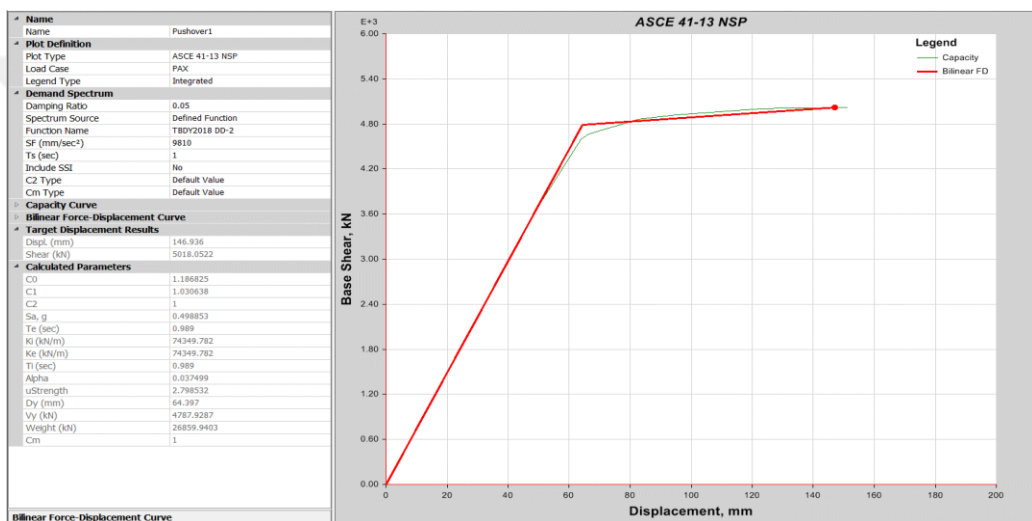
The performance point will be determined by capacity spectrum and demand spectrum intersection by using FEMA 440 EQUIVALENT LINEARIZATION [8] method and target displacement will be determined by using direct calculation method according to ASCE 41-13 NSP [4].

If the performance point is located near the elastic domain of capacity spectrum, so the structure will have good performance during earthquakes. In other words, as the performance point location far from elastic domain as the structure behave poorly during earthquakes.

Figure 6.7 and Figure 6.8 show the calculation of the performance point and target displacement for the school building in CSI ETABS for DD-2 Demand curve and for DD-3&DD-1 can be found in APPENDIX F:.



**Figure 6.7 : Performance Point Calculation for Actual School Building**



**Figure 6.8 : Target Displacement Calculation for Actual School Building**

## 6.7. Results discussion

The maximum top displacement that the school building before collapse is about 151 mm.

During pushover analysis, the hinges formations start firstly at some beams in story1 (Figure 6.9). Then, they extended to all beams and columns of same story and extend to some columns in story 2. Next, hinge continue formation of story 2 at beams and columns. Finally, plastic hinges extended to story 3 (Figure 6.9) that make the structure to became unstable.

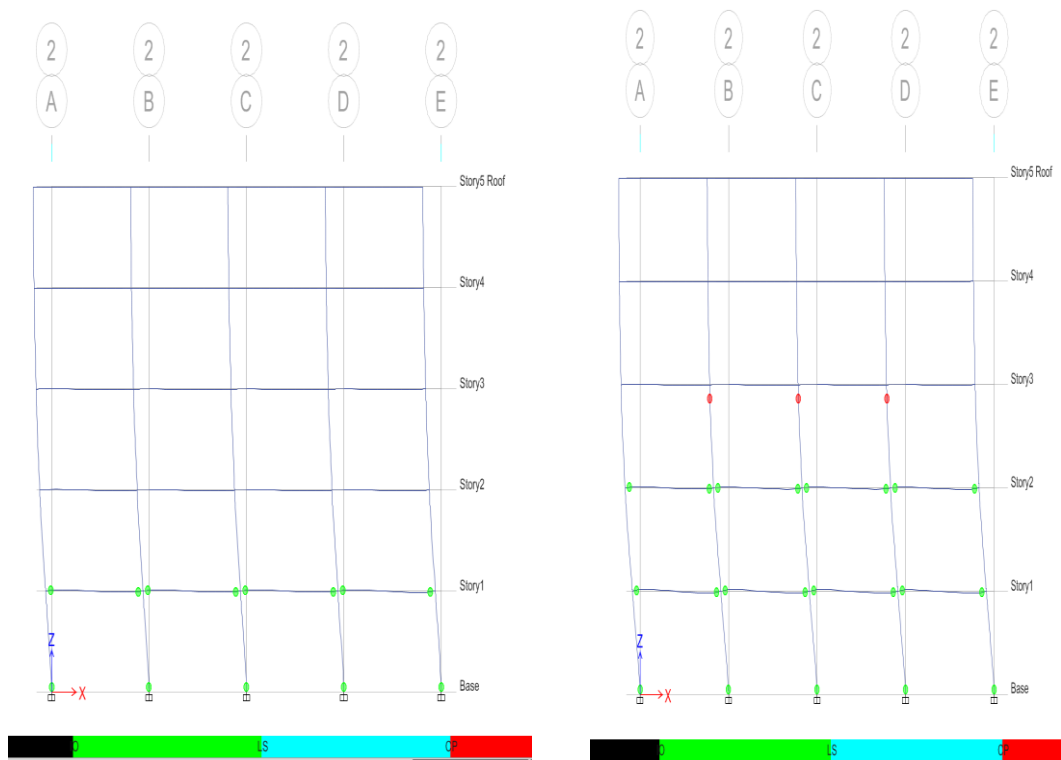
Performance Point and Target Displacement of Actual School Building presented in **Table 6.3** for DD-1&DD-2&DD-3 demand curves.

**Table 6.3 :** Performance Point and Target Displacement of Actual School Building

Demand Curve	Performance Point		Target Displacement		% Difference	
	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)	Displacement	Base shear
DD-1	Not found	Not found	Far from capacity curve		-	-
DD-2	121	4994	147	5018	21.5%	0.48%
DD-3	55	4040	55	4051	0%	0.27%

According to Table 6.3 results, the building has poor performance and poor behavior during earthquake especially at story 1&3. This may cause major damage or collapse, so this will make the life safety of children at risk. Actually, the building performance is good for DD-3 demand capacity but this not enough for existing school buildings where for DD-1 the performance point and target displacement are not found.

Finally, the actual school building will be retrofitted in order to reach to acceptable and good performance during earthquake.



**Figure 6.9 :** Beginning and End of Plastic Hinges Formation During PA Analysis of Actual School Building



## 7. PERFORMANCE BASED ANALYSIS BY PUSHOVER FOR RETROFITTED SCHOOL BUILDING

### 7.1. Introduction

The PA analysis of the building shows poor performance against seismic loads. Actually, the critical story was the ground and third story where almost all hinges form at these stories. So, the actual building will be retrofitted by use more than one cases. Then, each retrofitted case performance will be checked by PA to reach to adequate retrofitted case that give adequate performance level.

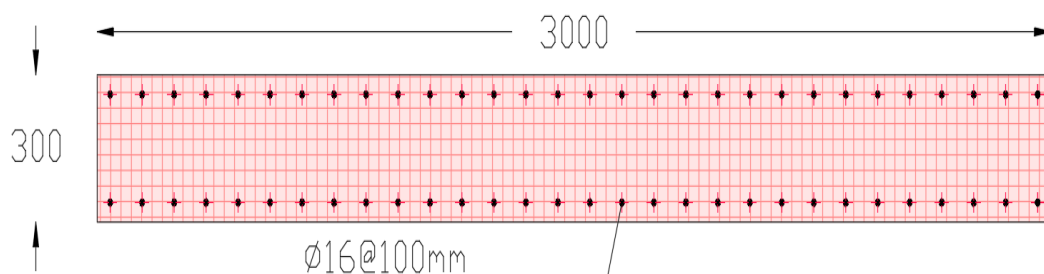
### 7.2. Retrofitting Cases

The retrofitting for building is done by technique of columns Jacketing and shear wall addition by use of high compressive strength (C50) reinforced concrete and S420a Reinforcement steel Grade. Also, five retrofitting cases used as following:

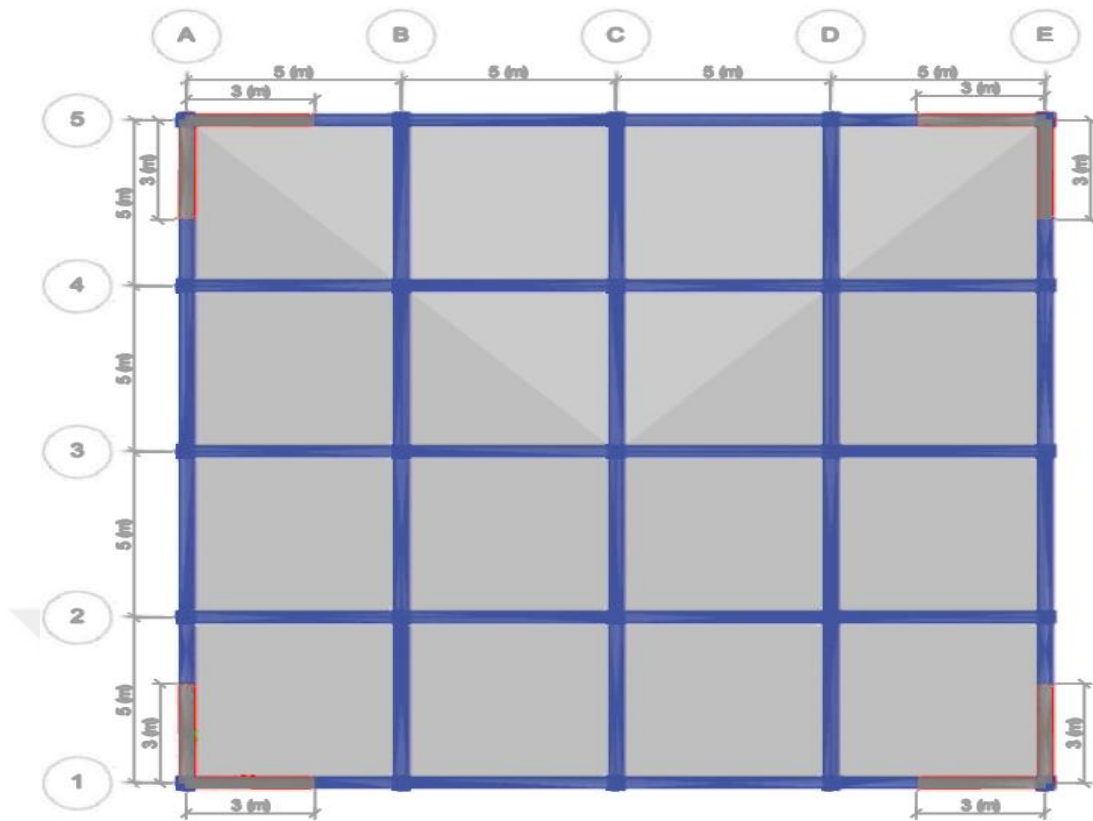
- a) Columns jacketing for story 1.
- b) Columns jacketing for story 1&2.
- c) Columns jacketing for all stories.
- d) Shear walls addition at structure corners.
- e) Shear walls addition at structure corners and middle of edges.

The stories columns jacketing data are as followings Table 7.1

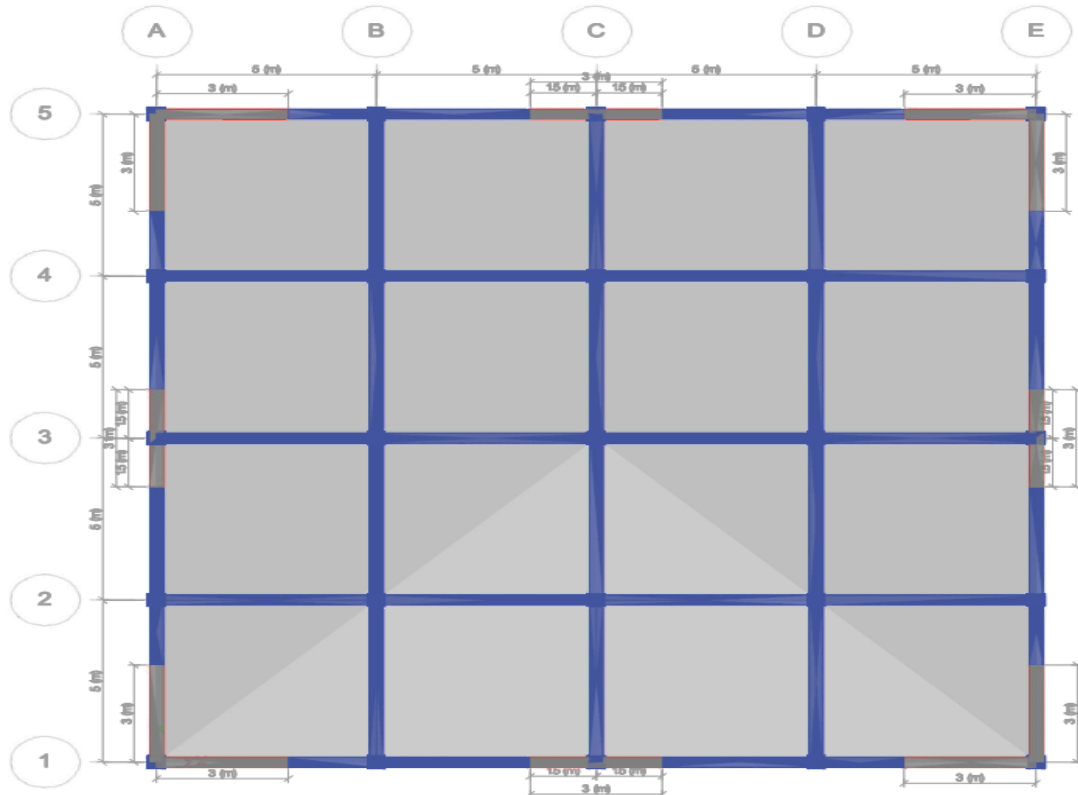
Shear walls will have rectangular cross sections with uniform steel reinforcement (Figure 7.1) by use 2  $\phi 16@100$  mm  $\approx 1.3\%$  and have 30mm clear cover of concrete. Cross-section dimensions of shear walls are 30 cm width and 300 cm length for each one. Locations of shear walls for each case are shown in Figure 7.2 and Figure 7.3.



**Figure 7.1** : Shear Walls Cross Section

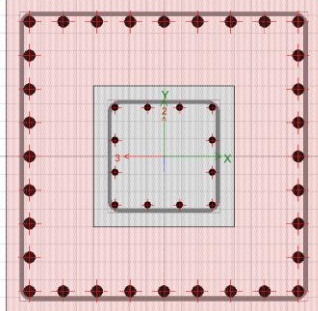
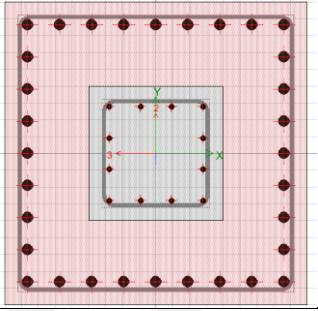
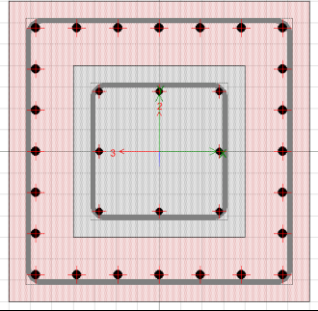
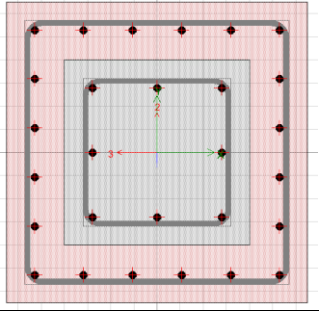


**Figure 7.2 :** Shear Wall Addition at Structure Corners Locations



**Figure 7.3 :** Shear Wall Addition at Structure Corners and Middle of Edges Locations

**Table 7.1 : Stories Column Jacketing**

Story columns	Jacketing longitudinal reinforcements	New column dimension (cm × cm)	Cross section description
Story1	32ø32 ≈4%	90X90	<p>C40 X 40 + 25 cm Jacketing with clear cover 40 mm</p> 
Story2	32ø32 ≈4%	90X90	<p>C40 X 40 + 25 cm Jacketing with clear cover 40 mm</p> 
Story3	24ø20 ≈2.3%	70X70	<p>C40 X 40 + 15 cm Jacketing with clear cover 40 mm</p> 
Story4&5	20ø16 ≈1.5%	65X65	<p>C40 X 40 + 12.5 cm Jacketing with clear cover 40 mm</p> 

The columns jacketing is drawn by using section designer tool in CSI ETABS V18.1.1, in order to obtain moment-curvature relationship. This have benefits in yield strength calculations for each section.

Also, the C50 concrete class has mechanical properties according to TS00 as following [23]:

Unit weight of concrete =  $\gamma_c = 25 \text{ kN/m}^3$

Characteristic of concrete strength =  $f_{ck} = 50 \text{ MPa}$

Working compression strength:

C40:  $0.85 f_{ck} = 0.85 * 50 = 42.5 \text{ MPa}$

Strength reduction factor for concrete  $\gamma_c = 1.5$

$$F_{cd} = \frac{42.5}{1.5} = 28.33 \text{ MPa}$$

Concrete Modulus of elasticity ( $E_c$ ):

$$E_c = 3250 * \sqrt{50} + 14000 = 36980 \text{ MPa}$$

### **7.3. Cracked Section Stiffness Modification Factors for Pushover Analysis**

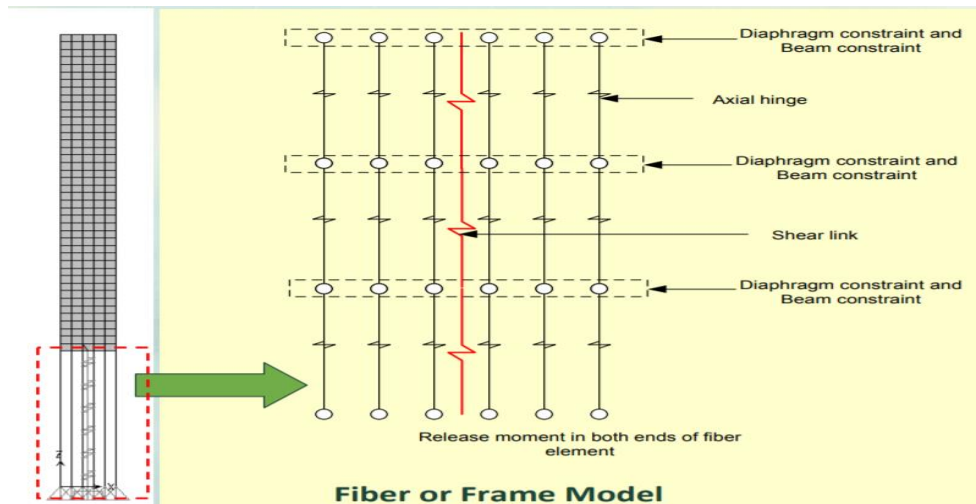
In pushover analysis, the structural sections such as beams, jacketed columns and shear walls are considered to be cracked same as in actual building modeling, where the stiffness modification factors are taken according to TBDY2018.

### **7.4. Plastic Hinges Definitions**

The plastic hinges are defined same as the school building model.

### **7.5. Shear walls modeling**

Shear walls modeling for nonlinear behavior are by use fiber element. Fiber element consist of discrete shear wall cross section into smaller columns that have close space. Each smaller column represents part of shear wall and called fiber. Each fiber includes concrete fiber and steel fiber. In order to represent shear stiffness these fibers are connected with shear elements as shown in Figure 7.4 [27].

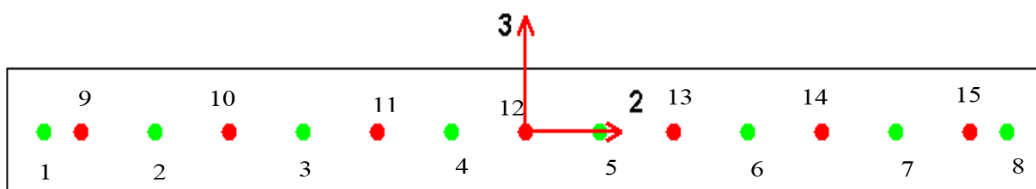


**Figure 7.4 : Shear Wall Fiber Modeling Concept [27]**

The shear walls fiber modelling done by use auto fiber assignment for shear walls feature in ETABS V18. Table 7.2 and Figure 7.5 present the chosen fiber area and locations.

**Table 7.2 : Shear Walls Fibers Modelling Data**

Fiber Number	Fiber cm <sup>2</sup>	Area	Fiber mm	Coordinate	2	Fiber Material
1	634.3		-1392.9			C50 concrete
2	1268.6		-1071.4			C50 concrete
3	1268.6		-642.9			C50 concrete
4	1268.6		-214.3			C50 concrete
5	1268.6		214.3			C50 concrete
6	1268.6		642.9			C50 concrete
7	1268.6		1071.4			C50 concrete
8	634.3		1392.9			C50 concrete
9	17.1		-1285.7			S420a steel
10	17.1		-857.1			S420a steel
11	17.1		-428.6			S420a steel
12	17.1		0			S420a steel
13	17.1		428.6			S420a steel
14	17.1		857.1			S420a steel
15	17.1		1285.7			S420a steel

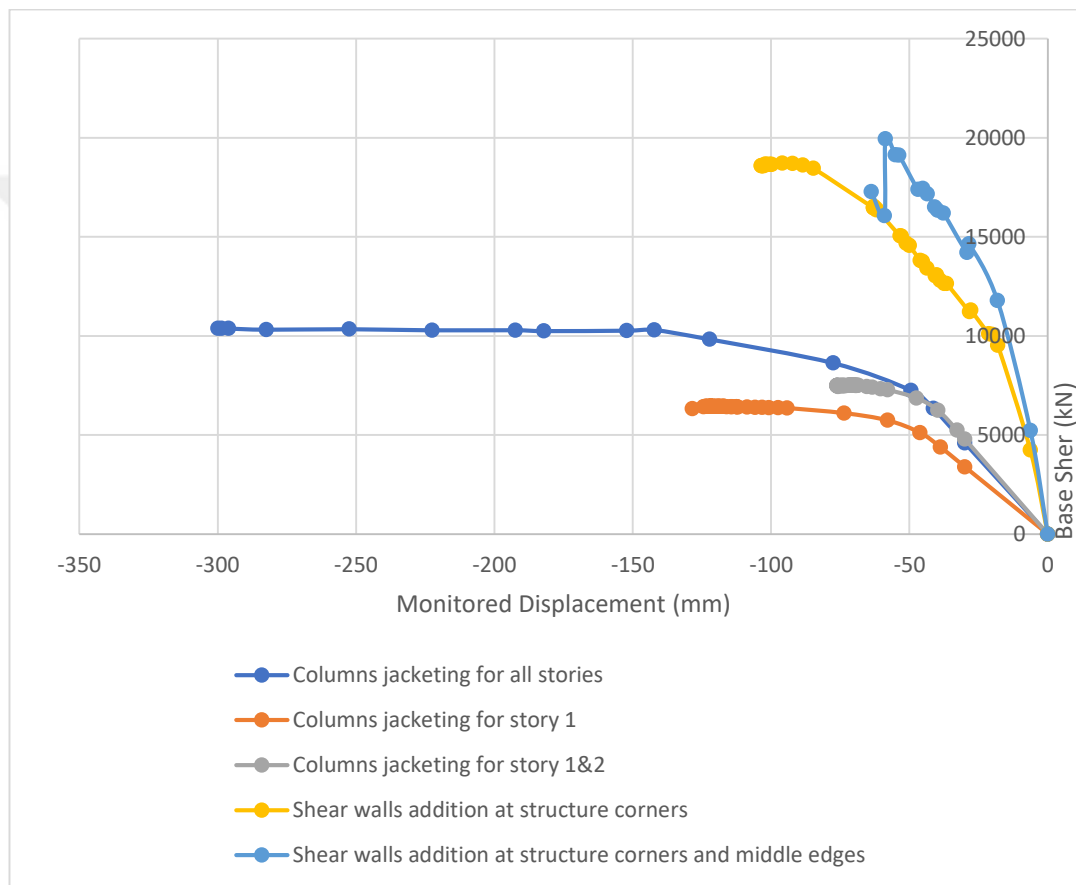


**Figure 7.5 : Shear Walls Fibers Modelling Locations**

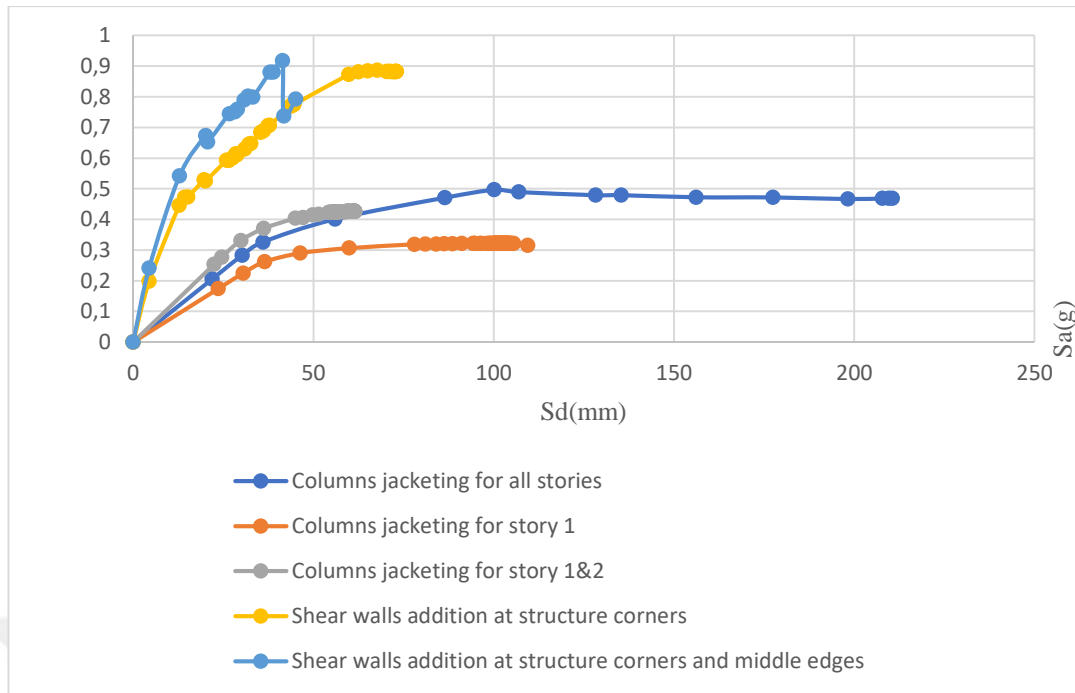
## 7.6. Pushover Results

The pushover analysis for retrofitted buildings cases are use same actual building mathematical model except the column sections are different and shear walls added for shear walls addition cases according to each retrofitted case.

Also, same steps followed to obtain capacity curve for Retrofitted School Building (Figure 7.6) table and data. Then, capacity spectrum obtained for each of retrofitted case (Figure 7.7).



**Figure 7.6 :** V vs Monitored Displacement Capacity Curve for Retrofitted School Building



**Figure 7.7 :** Capacity Spectrum (Sa VS Sd) for Retrofitted School Building

### 7.7. Demand Curve

Demand curve for our retrofitted school building calculated from elastic response spectrum according to TBDY2018 code as for actual one for DD-1, DD-2 and DD-3. CSI ETABS inputs for demand curves as shown in Figure 6.6.

### 7.8. Performance of Retrofitted School Building

The performance of retrofitted school building is determined by calculation of performance point and target displacement for each case. FEMA 440 EQUIVALENT LINEARIZATION [8] method is used to determine performance point and ASCE 41-13 NSP [4] method is used to determine target displacement as done for actual building. APPENDIX F.2: present calculations of performance point and target displacement for Retrofitted school building in CSI ETABS and Table 7.3 summarize these results.

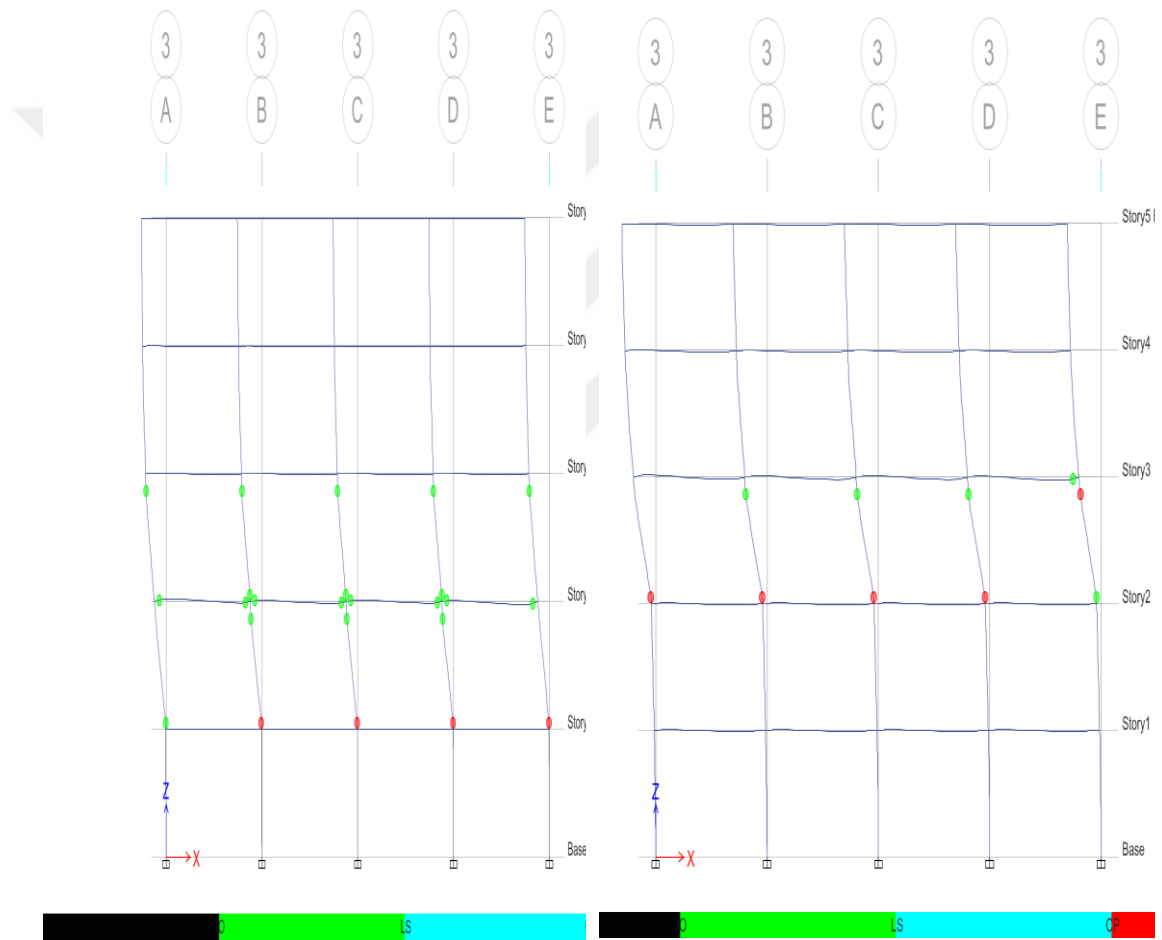
**Table 7.3** : Performance Point and Target Displacement of Retrofitting School Building

Demand Curve	Performance Point		Target Displacement		% Difference	
	Displacement (mm)	Base shear (kN)	Displacement (mm)	Base shear (kN)	Displacement	Base shear
Columns jacketing for story 1						
DD-1	Not found		Far from capacity curve		-	-
DD-2	96	6368	115	6432	19.8%	1%
DD-3	42	4721	43	4759	2.4%	0.8%
Columns jacketing for story 1&2						
DD-1	Not found		Far from capacity curve		-	-
DD-2	Not found		Far from capacity curve		-	-
DD-3	35	5624	36	5723	2.9%	1.73%
Columns jacketing for all stories						
DD-1	168	19955	194	20350	15.5%	1.98%
DD-2	103	15793	110	16468	6.8%	4.27%
DD-3	35	7956	36	8024	2.9%	0.85%
Shear walls addition at structure corners						
DD-1	109	18468	Far from capacity curve		-	-
DD-2	74	17474	84	18185	13.5%	4.1%
DD-3	22	10305	25	10692	13.6%	3.8%
Shear walls addition at structure corners and middle edges						
DD-1	Not found		Far from capacity curve		-	-
DD-2	Not found		Far from capacity curve		-	-
DD-3	23	13014	21	12652	8.7%	2.8%

## 7.9. Results Discussion and Acceptance Criteria

Among these retrofitted cases, just column jacketing for all story's performance point can be found for all demand curves and the target displacement located at pushover curve.

Figure 7.8 and Figure 7.9 show collapse mechanism of retrofitted cases by column jacketing for story 1 and story 1&2 which show the collapse mechanism happen in not retrofitted stories, so this give the reason of these retrofitted technique failure and poor performance in which collapse mechanism transferred to not retrofitted stories.



**Figure 7.8 :** Collapse Mechanism of Retrofitted Cases by Column Jacketing for Story 1

**Figure 7.9 :** Collapse Mechanism of Retrofitted Cases by Column Jacketing for Story 1&2

Table 7.3 show the displacement difference between FEMA 440 EQUIVALENT LINEARIZATION & ASCE 41-13 are acceptable, when the structure performance close to elastic domain and the difference become unacceptable when the building performance at plastic domain. On the other hand, the base shear value of both methods is almost same.

Table 7.4 show global Performance levels and Acceptance criteria for buildings according to TBDY2018 code and Table 7.5 show performance level of retrofitted cases according to different demand curves.

**Table 7.4** : Performance levels and Acceptance criteria according to TBDY2018 [10]

Performance level	Performance acceptance criteria
<p>Immediate Occupancy (IO)</p> <p>(post-earthquake damage state that remains safe to occupy, essentially retains the pre-earthquake design strength and stiffness of the structure)</p>	<ol style="list-style-type: none"> <li>1. There shall not be any beams beyond LS.</li> <li>2. There shall not be any column or shear walls beyond IO level.</li> <li>3. The ratio of beams in IO-LS region shall not exceed 20% in any story.</li> </ol>
<p>Life Safety (LS)</p> <p>(the post-earthquake damage state that includes damage to structural components but retains a margin against onset of partial or total collapse)</p>	<ol style="list-style-type: none"> <li>1. The ratio of beams and columns in LS-CP region shall not exceed 35% in any story.</li> <li>2. In any story, the shear carried by columns or shear walls in LS-CP region shall not exceed 20% of story shear. This ratio can be taken as 40% for roof story.</li> <li>3. In any story, the shear carried by columns or shear walls yielded at both ends shall not exceed 30% of story shear.</li> <li>4. There shall not be any columns or shear walls beyond CP.</li> </ol>
<p>Collapse Prevention (CP)</p> <p>(post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse. Also, The use of the building in its current state is unfavorable for life safety.)</p>	<ol style="list-style-type: none"> <li>1. The ratio of beams beyond CP region shall not exceed 20% in any story.</li> <li>2. In any story, the shear carried by columns or shear walls beyond CP region shall not exceed 30% of story shear.</li> <li>3. In any story, the shear carried by columns or shear walls yielded at both ends shall not exceed 30% of story shear.</li> </ol>

**Table 7.5 : Performance Level of Retrofitted Cases**

Demand Curve	Performance level at Performance Point	Performance level at Target Displacement
Columns jacketing for story 1		
DD-2	LS (Figure F.24)	Not applicable
DD-3	IO (Figure F.25)	IO (Figure F.25)
Columns jacketing for story 1&2		
DD-3	IO (Figure F.26)	IO (Figure F.26)
Columns jacketing for all stories		
DD-1	LS (Figure F.27)	LS (Figure F.28)
DD-2	LS (Figure F.29)	LS (Figure F.30)
DD-3	IO (Figure F.31)	IO (Figure F.31)
Shear walls addition at structure corners		
DD-1	CP (Figure F.32)	-
DD-2	LS (Figure F.33)	LS (Figure F.34)
DD-3	IO (Figure F.35)	IO (Figure F.35)
Shear walls addition at structure corners and middle edges		
DD-3	IO (Figure F.36)	IO (Figure F.36)

According to section 3.5.1 of TBDY2018, we can determine the required performance for building by obtain DTS from Table C.7 , BKS from Table C.5 and BYS from Table C.8. As a result, the required performance for school building is Immediate Occupancy (SH) performance level at DD-3 demand curve and Life Safety (KH) performance level at DD-1 demand curve according to Table 7.6, in which  $S_{DS} = 1.4472$  (Table 4.4),  $BYS = 6$  and  $BKS = 1$  (Table 4.5) in section 4.2 for school, so  $DTS = 1a$  for our school building [10].

**Table 7.6 : Performance Goal of Existing RC building (BYS $\geq$ 2) [10]**

(c) Mevcut Yerinde Dökme Betonarme, Önüretimli Betonarme ve Çelik Binalar  
(Yüksek Binalar dışında – BYS  $\geq$  2)

Deprem Yer H. Düzeyi	DTS = 1, 2, 3, 3a, 4, 4a		DTS = 1a, 2a	
	Normal Performans Hedefi	Değerlendirme/Tasarım Yaklaşımı	İleri Performans Hedefi	Değerlendirme/Tasarım Yaklaşımı
DD-3	—	—	SH	ŞGDT
DD-2	KH	ŞGDT	—	—
DD-1	—	—	KH	ŞGDT

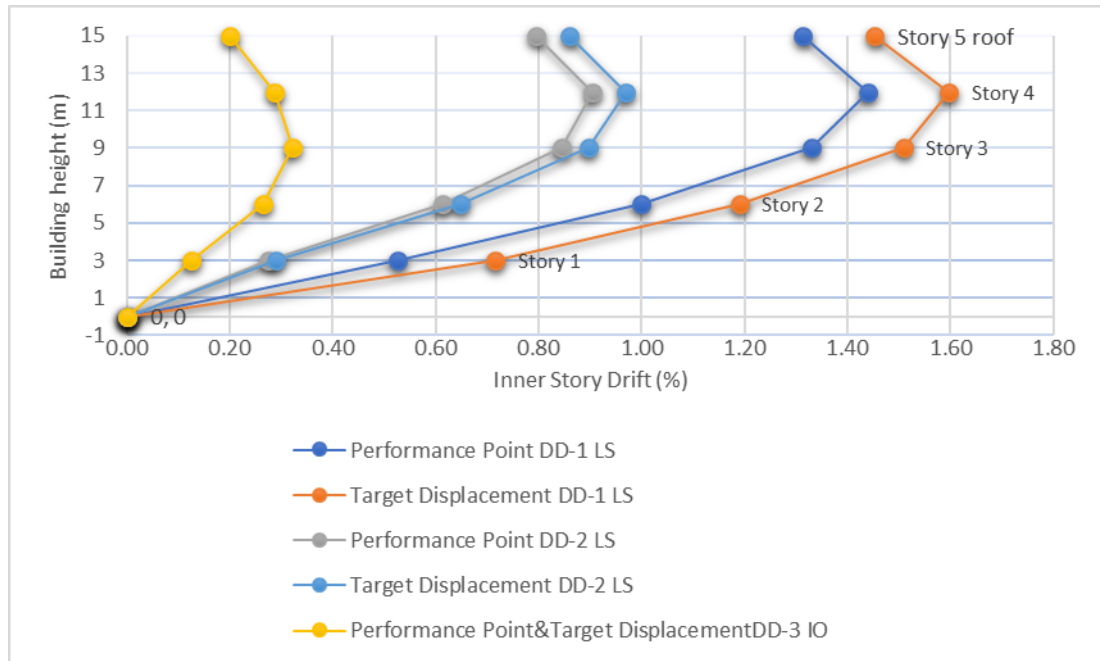
- (<sup>1</sup>) BYS > 3 olan binalarda uygulanacaktır.  
(<sup>2</sup>) BYS = 2,3 olan binalarda uygulanacaktır.  
(<sup>3</sup>) Ön tasarım olarak yapılacaktır.  
(<sup>4</sup>)  $I = 1,5$  alınarak uygulanacaktır.  
(<sup>5</sup>) Bkz. 3.5.2.2.

According to Table 7.5, just columns jacketing for all stories retrofitted case met the requirement of performance level for performance calculated by FEMA 440 Equivalent Linearization and ASCE 41-13 NSP.

In addition, TBDY2018 code specified limits for inner story drift as section 14.14.5.1 of code. The inner story drift limits are  $0.01h_i$  (1%) for IO performance level and  $0.015h_i$  (1.5%) for LS performance level [10].

Figure 7.10 and Table F.13 show the calculated inner story drift for retrofitted school building.

The results show drift calculated by use DD-1 ground motion and at target displacement that calculated by ASCE 41-13 NSP slightly exceeded drift limit.



**Figure 7.10 : Inner Story Drift Ratio of Column Jacketing for all Stories Retrofitted Case**

Finally, shear walls addition technique, relies on increasing the stiffness of structure by shear walls until demand capacity is satisfied. However, in the case of school building that located in high seismic zone (zone 1) large shear walls are required to reach to adequate performance goal. Addition of shear walls at structure corners technique gave the best result after full column jacketing technique.



## 8. CONCLUSIONS AND RECOMMENDATIONS

### 8.1. Conclusions

In this thesis, a comparative seismic analysis study was conducted according to four design codes (TBDY2018, TEC2007, ASCE 7-16 and EC8) for a five-story reinforced concrete school building located in Istanbul. Afterwards, the frames in structure were designed according to TEC2007 code. Then, performed-based analysis by PA was carried out to the school building to check the performance during earthquake. After that, school structure model was retrofitted in order to enhance performance with the use of columns jacketing and shear walls addition techniques. Finally, performance-based design was carried out for retrofitted cases by using PA analysis in order to choose the adequate retrofitted case. Based on this, the following conclusions are found.

1. Seismic analysis comparative study shows both EC8 and TBDY2018 codes are given the biggest base shear values for our school structure at all case of fundamental time period calculations that make them the safest.
2. The comparison between base shear values which were calculated by using different fundamental time periods cases (Approx. and ETABS time period for cracked and uncracked sections) show ETABS calculated fundamental time period for cracked section give the smallest values of base shear.
3. Performance-Based Analysis for the school building show the importance of seismic performance check for new design structures or the existing ones. Especially, when structure is located at a high seismic zone or for school usage to ensure life safety performance during earthquakes.
4. The retrofitting by use columns jacketing for partial stories is not a good choice because the collapse mechanism will transfer to unretrofitted stories.
5. At high seismic zone school structure, the retrofitting techniques that in increase ductility of structure are more effective than one that increase the stiffness.
6. Retrofitting of actual school building by jacketing all stories columns is the best case because this will increase the ductile behavior of structure.
7. Both FEMA 440 EQUIVALENT LINEARIZATION & ASCE 41-13 NSP methods give almost same base shear value at performance point or target displacement but give high variation at displacement values especially when performance in plastic domain.



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**APPENDIX**



## APPENDIX A: Location of School Building



Figure A.1 : Location of School Building

## APPENDIX B: $N_d$ & $N_d / (A_c f_{cm})$ values

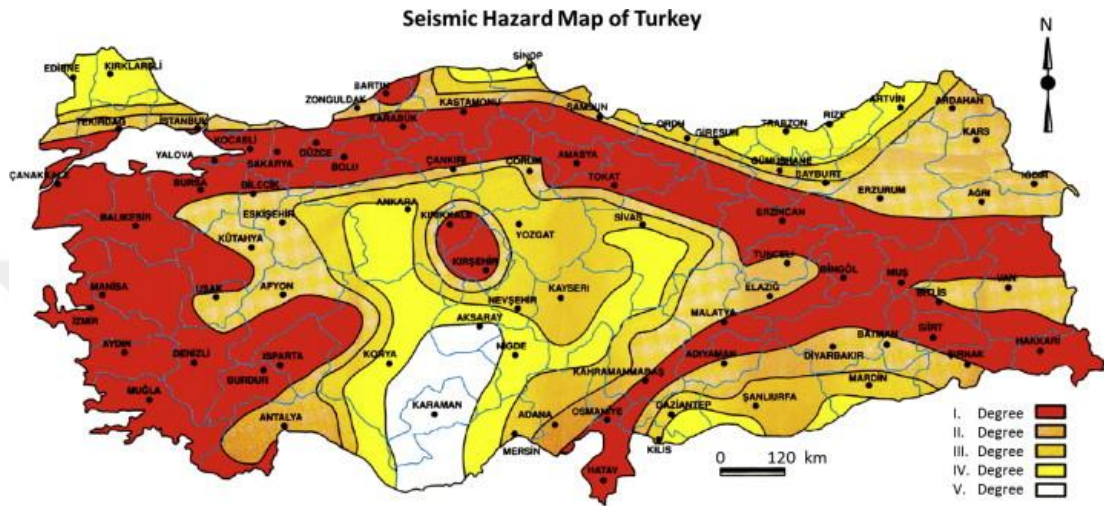
**Table B.1** :  $N_d$  &  $N_d / (A_c * f_{cm})$  values Calculations

Story	Column	Combo	P or $N_d$ (kN)	$A_c$ ( $m^2$ )	$f_{cm}$ (kN/ $m^2$ )	$N_d /$ ( $A_c f_{cm}$ )	Stiffness reduction factor	Average	
Story5 Roof	C1	1.4 Dead + 1.6 Live	-405.069	0.16	30000	0.08	0.40	0.40	
Story5 Roof	C2	1.4 Dead + 1.6 Live	-398.05	0.16	30000	0.08	0.40		
Story5 Roof	C3	1.4 Dead + 1.6 Live	-405.069	0.16	30000	0.08	0.40		
Story5 Roof	C4	1.4 Dead + 1.6 Live	-398.05	0.16	30000	0.08	0.40		
Story5 Roof	C5	1.4 Dead + 1.6 Live	-389.877	0.16	30000	0.08	0.40		
Story5 Roof	C6	1.4 Dead + 1.6 Live	-398.05	0.16	30000	0.08	0.40		
Story5 Roof	C7	1.4 Dead + 1.6 Live	-405.069	0.16	30000	0.08	0.40		
Story5 Roof	C8	1.4 Dead + 1.6 Live	-398.05	0.16	30000	0.08	0.40		
Story5 Roof	C9	1.4 Dead + 1.6 Live	-405.069	0.16	30000	0.08	0.40		
Story5 Roof	C10	1.4 Dead + 1.6 Live	-127.534	0.16	30000	0.03	0.40		
Story5 Roof	C16	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story5 Roof	C17	1.4 Dead + 1.6 Live	-223.315	0.16	30000	0.05	0.40		
Story5 Roof	C18	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story5 Roof	C19	1.4 Dead + 1.6 Live	-127.534	0.16	30000	0.03	0.40		
Story5 Roof	C31	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story5 Roof	C32	1.4 Dead + 1.6 Live	-223.315	0.16	30000	0.05	0.40		
Story5 Roof	C33	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story5 Roof	C34	1.4 Dead + 1.6 Live	-127.534	0.16	30000	0.03	0.40		
Story5 Roof	C35	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story5 Roof	C36	1.4 Dead + 1.6 Live	-223.315	0.16	30000	0.05	0.40		
Story5 Roof	C37	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story5 Roof	C38	1.4 Dead + 1.6 Live	-127.534	0.16	30000	0.03	0.40		
Story5 Roof	C39	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story5 Roof	C40	1.4 Dead + 1.6 Live	-223.315	0.16	30000	0.05	0.40		
Story5 Roof	C41	1.4 Dead + 1.6 Live	-229.382	0.16	30000	0.05	0.40		
Story4	C1	1.4 Dead + 1.6 Live	-952.59	0.16	30000	0.20	0.53		0.46
Story4	C2	1.4 Dead + 1.6 Live	-943.226	0.16	30000	0.20	0.53		
Story4	C3	1.4 Dead + 1.6 Live	-952.59	0.16	30000	0.20	0.53		
Story4	C4	1.4 Dead + 1.6 Live	-943.226	0.16	30000	0.20	0.53		
Story4	C5	1.4 Dead + 1.6 Live	-932.446	0.16	30000	0.19	0.53		
Story4	C6	1.4 Dead + 1.6 Live	-943.226	0.16	30000	0.20	0.53		
Story4	C7	1.4 Dead + 1.6 Live	-952.59	0.16	30000	0.20	0.53		
Story4	C8	1.4 Dead + 1.6 Live	-943.226	0.16	30000	0.20	0.53		
Story4	C9	1.4 Dead + 1.6 Live	-952.59	0.16	30000	0.20	0.53		
Story4	C10	1.4 Dead + 1.6 Live	-323.001	0.16	30000	0.07	0.40		
Story4	C16	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story4	C17	1.4 Dead + 1.6 Live	-552.763	0.16	30000	0.12	0.42		
Story4	C18	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story4	C19	1.4 Dead + 1.6 Live	-323.001	0.16	30000	0.07	0.40		
Story4	C31	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story4	C32	1.4 Dead + 1.6 Live	-552.763	0.16	30000	0.12	0.42		
Story4	C33	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story4	C34	1.4 Dead + 1.6 Live	-323.001	0.16	30000	0.07	0.40		
Story4	C35	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story4	C36	1.4 Dead + 1.6 Live	-552.763	0.16	30000	0.12	0.42		
Story4	C37	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story4	C38	1.4 Dead + 1.6 Live	-323.001	0.16	30000	0.07	0.40		
Story4	C39	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story4	C40	1.4 Dead + 1.6 Live	-552.763	0.16	30000	0.12	0.42		
Story4	C41	1.4 Dead + 1.6 Live	-561.614	0.16	30000	0.12	0.42		
Story3	C1	1.4 Dead + 1.6 Live	-1499.46	0.16	30000	0.31	0.68	0.56	
Story3	C2	1.4 Dead + 1.6 Live	-1487.77	0.16	30000	0.31	0.68		
Story3	C3	1.4 Dead + 1.6 Live	-1499.46	0.16	30000	0.31	0.68		
Story3	C4	1.4 Dead + 1.6 Live	-1487.77	0.16	30000	0.31	0.68		
Story3	C5	1.4 Dead + 1.6 Live	-1474.38	0.16	30000	0.31	0.68		
Story3	C6	1.4 Dead + 1.6 Live	-1487.77	0.16	30000	0.31	0.68		
Story3	C7	1.4 Dead + 1.6 Live	-1499.46	0.16	30000	0.31	0.68		
Story3	C8	1.4 Dead + 1.6 Live	-1487.77	0.16	30000	0.31	0.68		
Story3	C9	1.4 Dead + 1.6 Live	-1499.46	0.16	30000	0.31	0.68		
Story3	C10	1.4 Dead + 1.6 Live	-519.434	0.16	30000	0.11	0.41		
Story3	C16	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story3	C17	1.4 Dead + 1.6 Live	-882.556	0.16	30000	0.18	0.51		

Story3	C18	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story3	C19	1.4 Dead + 1.6 Live	-519.434	0.16	30000	0.11	0.41		
Story3	C31	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story3	C32	1.4 Dead + 1.6 Live	-882.556	0.16	30000	0.18	0.51		
Story3	C33	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story3	C34	1.4 Dead + 1.6 Live	-519.434	0.16	30000	0.11	0.41		
Story3	C35	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story3	C36	1.4 Dead + 1.6 Live	-882.556	0.16	30000	0.18	0.51		
Story3	C37	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story3	C38	1.4 Dead + 1.6 Live	-519.434	0.16	30000	0.11	0.41		
Story3	C39	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story3	C40	1.4 Dead + 1.6 Live	-882.556	0.16	30000	0.18	0.51		
Story3	C41	1.4 Dead + 1.6 Live	-893.909	0.16	30000	0.19	0.51		
Story2	C1	1.4 Dead + 1.6 Live	-2046.39	0.16	30000	0.43	0.80		0.65
Story2	C2	1.4 Dead + 1.6 Live	-2032.16	0.16	30000	0.42	0.80		
Story2	C3	1.4 Dead + 1.6 Live	-2046.39	0.16	30000	0.43	0.80		
Story2	C4	1.4 Dead + 1.6 Live	-2032.16	0.16	30000	0.42	0.80		
Story2	C5	1.4 Dead + 1.6 Live	-2016.04	0.16	30000	0.42	0.80		
Story2	C6	1.4 Dead + 1.6 Live	-2032.16	0.16	30000	0.42	0.80		
Story2	C7	1.4 Dead + 1.6 Live	-2046.39	0.16	30000	0.43	0.80		
Story2	C8	1.4 Dead + 1.6 Live	-2032.16	0.16	30000	0.42	0.80		
Story2	C9	1.4 Dead + 1.6 Live	-2046.39	0.16	30000	0.43	0.80		
Story2	C10	1.4 Dead + 1.6 Live	-716.223	0.16	30000	0.15	0.47		
Story2	C16	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story2	C17	1.4 Dead + 1.6 Live	-1212.39	0.16	30000	0.25	0.60		
Story2	C18	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story2	C19	1.4 Dead + 1.6 Live	-716.223	0.16	30000	0.15	0.47		
Story2	C31	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story2	C32	1.4 Dead + 1.6 Live	-1212.39	0.16	30000	0.25	0.60		
Story2	C33	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story2	C34	1.4 Dead + 1.6 Live	-716.223	0.16	30000	0.15	0.47		
Story2	C35	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story2	C36	1.4 Dead + 1.6 Live	-1212.39	0.16	30000	0.25	0.60		
Story2	C37	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story2	C38	1.4 Dead + 1.6 Live	-716.223	0.16	30000	0.15	0.47		
Story2	C39	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story2	C40	1.4 Dead + 1.6 Live	-1212.39	0.16	30000	0.25	0.60		
Story2	C41	1.4 Dead + 1.6 Live	-1226.1	0.16	30000	0.26	0.61		
Story1	C1	1.4 Dead + 1.6 Live	-2601.19	0.16	30000	0.54	0.80	0.71	
Story1	C2	1.4 Dead + 1.6 Live	-2579.06	0.16	30000	0.54	0.80		
Story1	C3	1.4 Dead + 1.6 Live	-2601.19	0.16	30000	0.54	0.80		
Story1	C4	1.4 Dead + 1.6 Live	-2579.06	0.16	30000	0.54	0.80		
Story1	C5	1.4 Dead + 1.6 Live	-2554.72	0.16	30000	0.53	0.80		
Story1	C6	1.4 Dead + 1.6 Live	-2579.06	0.16	30000	0.54	0.80		
Story1	C7	1.4 Dead + 1.6 Live	-2601.19	0.16	30000	0.54	0.80		
Story1	C8	1.4 Dead + 1.6 Live	-2579.06	0.16	30000	0.54	0.80		
Story1	C9	1.4 Dead + 1.6 Live	-2601.19	0.16	30000	0.54	0.80		
Story1	C10	1.4 Dead + 1.6 Live	-908.831	0.16	30000	0.19	0.52		
Story1	C16	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		
Story1	C17	1.4 Dead + 1.6 Live	-1537.98	0.16	30000	0.32	0.69		
Story1	C18	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		
Story1	C19	1.4 Dead + 1.6 Live	-908.831	0.16	30000	0.19	0.52		
Story1	C31	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		
Story1	C32	1.4 Dead + 1.6 Live	-1537.98	0.16	30000	0.32	0.69		
Story1	C33	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		
Story1	C34	1.4 Dead + 1.6 Live	-908.831	0.16	30000	0.19	0.52		
Story1	C35	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		
Story1	C36	1.4 Dead + 1.6 Live	-1537.98	0.16	30000	0.32	0.69		
Story1	C37	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		
Story1	C38	1.4 Dead + 1.6 Live	-908.831	0.16	30000	0.19	0.52		
Story1	C39	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		
Story1	C40	1.4 Dead + 1.6 Live	-1537.98	0.16	30000	0.32	0.69		
Story1	C41	1.4 Dead + 1.6 Live	-1557.67	0.16	30000	0.32	0.70		

**APPENDIX C: SEISMIC AND SITE COEFFICIENTS FOR OUR SCHOOL BUILDING ACCORDING TO TBDY2018, TEC2007, ASCE7-16 AND EC8**

**APPENDIX C.1: Ground Motion Parameters**



**Figure C.1 : Old Seismic Hazard Map of Turkey [28]**

**Table C.1 : Ground Motion Parameters for our School Building**

Ground motion level	$S_s$	$S_1$	PGA	PGV
DD-1	2.099	0.588	0.824	52.635
DD-2	1.206	0.328	0.495	30.458
DD-3	0.454	0.119	0.590	0.178

**Table C.2 : Effective ground acceleration coefficient ( $A_0$ ) According to TEC2007 [9]**

<i>Seismic Zone</i>	$A_0$
1	0.40
2	0.30
3	0.20
4	0.10

## APPENDIX C.2: TBDY 2018 Seismic and Site Coefficients

**Table C.3** : Long-Period Site Coefficient (at 1.0-s period) ( $F_1$ ) [10]

Yerel Zemin Sınıfı	1.0 saniye periyot için Yerel Zemin Etki Katsayısı $F_1$					
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 = 0.50$	$S_1 \geq 0.60$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.8	0.8	0.8	0.8	0.8	0.8
ZC	1.5	1.5	1.5	1.5	1.5	1.4
ZD	2.4	2.2	2.0	1.9	1.8	1.7
ZE	4.2	3.3	2.8	2.4	2.2	2.0

**Table C.4** : Short-Period Site Coefficient (at 0.2-s period) ( $F_s$ ) [10]

Yerel Zemin Sınıfı	Kısa periyot bölgesi için Yerel Zemin Etki Katsayısı $F_s$					
	$S_s \leq 0.25$	$S_s = 0.50$	$S_s = 0.75$	$S_s = 1.00$	$S_s = 1.25$	$S_s \geq 1.50$
ZA	0.8	0.8	0.8	0.8	0.8	0.8
ZB	0.9	0.9	0.9	0.9	0.9	0.9
ZC	1.3	1.3	1.2	1.2	1.2	1.2
ZD	1.6	1.4	1.2	1.1	1.0	1.0
ZE	2.4	1.7	1.3	1.1	0.9	0.8

**Table C.5 : Importance factor [10]**

Bina Kullanım Sınıfı	Binanın Kullanım Amacı	Bina Önem Katsayısı (I)
BKS = 1	<b>Deprem sonrası kullanımı gereken binalar, insanların uzun süreli ve yoğun olarak bulunduğu binalar, değerli eşyanın saklandığı binalar ve tehlikeli madde içeren binalar</b> <b>a)</b> Deprem sonrasında hemen kullanılması gerekli binalar (Hastaneler, dispanserler, sağlık ocakları, itfaiye bina ve tesisleri, PTT ve diğer haberleşme tesisleri, ulaşım istasyonları ve terminalleri, enerji üretim ve dağıtım tesisleri, vilayet, kaymakamlık ve belediye yönetim binaları, ilk yardım ve afet planlama istasyonları) <b>b)</b> Okullar, diğer eğitim bina ve tesisleri, yurt ve yatakhaneler, askeri kışlalar, cezaevleri, vb. <b>c)</b> Müzeler <b>d)</b> Toksik, patlayıcı, parlayıcı, vb. özellikleri olan maddelerin bulunduğu veya depolandığı binalar	1.5
BKS = 2	<b>İnsanların kısa süreli ve yoğun olarak bulunduğu binalar</b> Alışveriş merkezleri, spor tesisleri, sinema, tiyatro, konser salonları, ibadethaneler, vb.	1.2
BKS = 3	<b>Diğer binalar</b> BKS=1 ve BKS=2 için verilen tanımlara girmeyen diğer binalar (Konutlar, işyerleri, oteller, bina türü endüstri yapıları, vb.)	1.0

**Table C.6 : Response Modification and Over Strength Factor (R&D) [10]**

Bina Taşıyıcı Sistemi	Taşıyıcı Sistem Davranış Katsayısı $R$	Dayanım Fazlalığı Katsayısı $D$	İzin Verilen Bina Yükseklik Sınıfları $BYS$
<b>A. YERİNDE DÖKME BETONARME BİNA TAŞIYICI SİSTEMLERİ</b>			
<b>A1. Süneklik Düzeyi Yüksek Taşıyıcı Sistemler</b>			
<b>A11.</b> Deprem etkilerinin tamamının moment aktaran <i>süneklik düzeyi yüksek</i> betonarme çerçevelerle karşılandığı binalar	8	3	$BYS \geq 3$
<b>A12.</b> Deprem etkilerinin tamamının <i>süneklik düzeyi yüksek</i> bağ kirişli (boşluklu) betonarme perdelerle karşılandığı binalar	7	2.5	$BYS \geq 2$
<b>A13.</b> Deprem etkilerinin tamamının <i>süneklik düzeyi yüksek</i> boşluksuz betonarme perdelerle karşılandığı binalar	6	2.5	$BYS \geq 2$
<b>A14.</b> Deprem etkilerinin moment aktaran <i>süneklik düzeyi yüksek</i> betonarme çerçeveler ile <i>süneklik düzeyi yüksek</i> bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.4.5)	8	2.5	$BYS \geq 2$
<b>A15.</b> Deprem etkilerinin moment aktaran <i>süneklik düzeyi yüksek</i> betonarme çerçeveler ile <i>süneklik düzeyi yüksek</i> boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.4.5)	7	2.5	$BYS \geq 2$
<b>A16.</b> Deprem etkilerinin tamamının çatı düzeyindeki bağlantıları mafsallı olan ve yüksekliği 12 m'yi geçmeyen <i>süneklik düzeyi yüksek</i> betonarme kolonlar tarafından karşılandığı tek katlı binalar	3	2	–
<b>A2. Süneklik Düzeyi Karma Taşıyıcı Sistemler (Bkz. 4.3.4.1, 4.3.4.6)</b>			
<b>A21.</b> Deprem etkilerinin moment aktaran <i>süneklik düzeyi sınırlı</i> betonarme çerçeveler ile <i>süneklik düzeyi yüksek</i> bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.1.2)	6	2.5	$BYS \geq 4$
<b>A22.</b> Deprem etkilerinin moment aktaran <i>süneklik düzeyi sınırlı</i> betonarme çerçeveler ile <i>süneklik düzeyi yüksek</i> boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar (Bkz.4.3.1.2)	5	2.5	$BYS \geq 4$
<b>A23.</b> Deprem etkilerinin moment aktaran <i>süneklik düzeyi sınırlı dolgulu (asmolen) veya dolgusuz tek doğrultulu dişi döşemeli</i> betonarme çerçeveler ile <i>süneklik düzeyi yüksek</i> bağ kirişli (boşluklu) betonarme perdeler tarafından birlikte karşılandığı binalar	6	2.5	$BYS \geq 6$
<b>A24.</b> Deprem etkilerinin moment aktaran <i>süneklik düzeyi sınırlı dolgulu (asmolen) veya dolgusuz tek doğrultulu dişi döşemeli</i> betonarme çerçeveler ile <i>süneklik düzeyi yüksek</i> boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar	5	2.5	$BYS \geq 6$
<b>A3. Süneklik Düzeyi Sınırlı Taşıyıcı Sistemler (Bkz. 4.3.4.1, 4.3.4.3, 4.3.4.7)</b>			
<b>A31.</b> Deprem etkilerinin tamamının moment aktaran <i>süneklik düzeyi sınırlı</i> betonarme çerçevelerle karşılandığı binalar	4	2.5	$BYS \geq 7$
<b>A32.</b> Deprem etkilerinin tamamının <i>süneklik düzeyi sınırlı</i> boşluksuz betonarme perdelerle karşılandığı binalar	4	2	$BYS \geq 6$
<b>A33.</b> Deprem etkilerinin moment aktaran <i>süneklik düzeyi sınırlı</i> betonarme çerçeveler ile <i>süneklik düzeyi sınırlı</i> boşluksuz betonarme perdeler tarafından birlikte karşılandığı binalar	4	2	$BYS \geq 6$

**Table C.7 : Seismic Design Classes (DTS) [10]**

DD-2 Deprem Yer Hareketi Düzeyinde Kısa Periyot Tasarım Spektral İvme Katsayısı ( $S_{DS}$ )	Bina Kullanım Sınıfı	
	BKS = 1	BKS = 2, 3
$S_{DS} < 0.33$	DTS = 4a	DTS = 4
$0.33 \leq S_{DS} < 0.50$	DTS = 3a	DTS = 3
$0.50 \leq S_{DS} < 0.75$	DTS = 2a	DTS = 2
$0.75 \leq S_{DS}$	DTS = 1a	DTS = 1

**Table C.8 : Building Height Classes (BYS) [10]**

Bina Yükseklik Sınıfı	Bina Yükseklik Sınıfları ve Deprem Tasarım Sınıflarına Göre Tanımlanan Bina Yükseklik Aralıkları [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	$H_N \leq 7$	$H_N \leq 10.5$	

### APPENDIX C.3: TEC2007 Seismic and Site Coefficients

**Table C.9 :** Spectrum Periods  $T_A$  &  $T_B$  [9]

<i>Local Site Class according to Table 6.2</i>	$T_A$ (second)	$T_B$ (second)
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	0.20	0.90

**Table C.10 :** Importance Factor [9]

<i>Purpose of Occupancy or Type of Building</i>	<i>Importance Factor (I)</i>
<p><b><u>1. Buildings required to be utilized after the earthquake and buildings containing hazardous materials</u></b>            a) Buildings required to be utilized immediately after the earthquake (Hospitals, dispensaries, health wards, fire fighting buildings and facilities, PTT and other telecommunication facilities, transportation stations and terminals, power generation and distribution facilities; governorate, county and municipality administration buildings, first aid and emergency planning stations)            b) Buildings containing or storing toxic, explosive and flammable materials, etc.</p>	1.5
<p><b><u>2. Intensively and long-term occupied buildings and buildings preserving valuable goods</u></b>            a) Schools, other educational buildings and facilities, dormitories and hostels, military barracks, prisons, etc.            b) Museums</p>	1.4
<p><b><u>3. Intensively but short-term occupied buildings</u></b>            Sport facilities, cinema, theatre and concert halls, etc.</p>	1.2
<p><b><u>4. Other buildings</u></b>            Buildings other than above defined buildings. (Residential and office buildings, hotels, building-like industrial structures, etc.)</p>	1.0

**Table C.11 : Response Modification Factor (R) [9]**

<b>TABLE 2.5 - STRUCTURAL SYSTEM BEHAVIOUR FACTORS (R)</b>		
<b>BUILDING STRUCTURAL SYSTEM</b>	<b>Systems of Nominal Ductility Level</b>	<b>Systems of High Ductility Level</b>
<b><u>(1) CAST-IN-SITE REINFORCED CONCRETE BUILDINGS</u></b>		
(1.1) Buildings in which seismic loads are fully resisted by frames.....	4	8
(1.2) Buildings in which seismic loads are fully resisted by coupled structural walls.....	4	7
(1.3) Buildings in which seismic loads are fully resisted by solid structural walls.....	4	6
(1.4) Buildings in which seismic loads are jointly resisted by frames and solid and / or coupled structural walls.....	4	7
<b><u>(2) PREFABRICATED REINFORCED CONCRETE BUILDINGS</u></b>		
(2.1) Buildings in which seismic loads are fully resisted by frames with connections capable of cyclic moment transfer	3	7
(2.2) Single-storey buildings in which seismic loads are fully resisted by columns with hinged upper connections	—	3
(2.3) Prefabricated buildings with hinged frame connections in which seismic loads are fully resisted by prefabricated or cast – in – situ solid structural walls and / or coupled structural walls.	—	5
(2.4) Buildings in which seismic loads are jointly resisted by frames with connections capable of cyclic moment transfer and cast-in-situ solid and / or coupled structural walls	3	6
<b><u>(3) STRUCTURAL STEEL BUILDINGS</u></b>		
(3.1) Buildings in which seismic loads are fully resisted by frames.....	5	8
(3.2) Single – storey buildings in which seismic loads are fully resisted by columns with connections hinged at the top.....	—	4
(3.3) Buildings in which seismic loads are fully resisted by braced frames or cast-in-situ reinforced concrete structural walls		
(a) Centrally braced frames .....	4	5
(b) Eccentrically braced frames .....	—	7
(c) Reinforced concrete structural walls.....	4	6
(3.4) Buildings in which seismic loads are jointly resisted by structural steel braced frames or cast-in-situ reinforced concrete structural walls		
(a) Centrally braced frames..... (b)	5	6
Eccentrically braced frames..... (c)	—	8
Reinforced concrete structural walls.....	4	7

## APPENDIX C.4: EC8 Seismic and Site Coefficients

**Table C.12 :** Type1 Response Spectra Parameters  $S$ ,  $T_B$ ,  $T_C$  and  $T_D$  [11]

Ground type	$S$	$T_B$ (s)	$T_C$ (s)	$T_D$ (s)
A	1,0	0,15	0,4	2,0
B	1,2	0,15	0,5	2,0
C	1,15	0,20	0,6	2,0
D	1,35	0,20	0,8	2,0
E	1,4	0,15	0,5	2,0

**Table C.13 :** Importance Factor [11]

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, fire stations, power plants, etc.

**Table C.14 :** Behavior factor (q) Upper Limit [11]

STRUCTURAL TYPE	Ductility Class	
	DCM	DCH
a) Moment resisting frames	4	$5\alpha_w/\alpha_1$
b) Frame with concentric bracings		
Diagonal bracings	4	4
V-bracings	2	2,5
c) Frame with eccentric bracings	4	$5\alpha_w/\alpha_1$
d) Inverted pendulum	2	$2\alpha_w/\alpha_1$
e) Structures with concrete cores or concrete walls	See section 5	
f) Moment resisting frame with concentric bracing	4	$4\alpha_w/\alpha_1$
g) Moment resisting frames with infills		
Unconnected concrete or masonry infills, in contact with the frame	2	2
Connected reinforced concrete infills	See section 7	
Infills isolated from moment frame (see moment frames)	4	$5\alpha_w/\alpha_1$

## APPENDIX C.5: ASCE 7-16 Seismic and Site Coefficients

**Table C.15** : Long-Period Site Coefficient (at 1.0-s period) ( $F_V$ ) [12]

Mapped Risk-Targeted Maximum Considered Earthquake ( $MCE_R$ ) Spectral Response Acceleration Parameter at 1-s Period						
Site Class	$S_1 \leq 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 = 0.5$	$S_1 \geq 0.6$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2 <sup>a</sup>	2.0 <sup>a</sup>	1.9 <sup>a</sup>	1.8 <sup>a</sup>	1.7 <sup>a</sup>
E	4.2	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of  $S_1$ .

**Table C.16** : Short-Period Site Coefficient (at 0.2-s period) ( $F_a$ ) [12]

Mapped Risk-Targeted Maximum Considered Earthquake ( $MCE_R$ ) Spectral Response Acceleration Parameter at Short Period						
Site Class	$S_s \leq 0.25$	$S_s = 0.5$	$S_s = 0.75$	$S_s = 1.0$	$S_s = 1.25$	$S_s \geq 1.5$
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.9	0.9	0.9	0.9	0.9	0.9
C	1.3	1.3	1.2	1.2	1.2	1.2
D	1.6	1.4	1.2	1.1	1.0	1.0
E	2.4	1.7	1.3	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: Use straight-line interpolation for intermediate values of  $S_s$ .

**Table C.17 : Risk Category [12]**

Use or Occupancy of Buildings and Structures	Risk Category
Buildings and other structures that represent low risk to human life in the event of failure	I
All buildings and other structures except those listed in Risk Categories I, III, and IV	II
Buildings and other structures, the failure of which could pose a substantial risk to human life	III
Buildings and other structures, not included in Risk Category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure	
Buildings and other structures not included in Risk Category IV (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing toxic or explosive substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released <sup>a</sup>	
Buildings and other structures designated as essential facilities	IV
Buildings and other structures, the failure of which could pose a substantial hazard to the community	
Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, or hazardous waste) containing sufficient quantities of highly toxic substances where the quantity of the material exceeds a threshold quantity established by the Authority Having Jurisdiction and is sufficient to pose a threat to the public if released <sup>a</sup>	
Buildings and other structures required to maintain the functionality of other Risk Category IV structures	

<sup>a</sup>Buildings and other structures containing toxic, highly toxic, or explosive substances shall be eligible for classification to a lower Risk Category if it can be demonstrated to the satisfaction of the Authority Having Jurisdiction by a hazard assessment as described in Section 1.5.3 that a release of the substances is commensurate with the risk associated with that Risk Category.

**Table C.18 : Importance Factor [12]**

Risk Category from Table 1.5-1	Snow Importance Factor, $I_s$	Ice Importance Factor—Thickness, $I_t$	Ice Importance Factor—Wind, $I_w$	Seismic Importance Factor, $I_e$
I	0.80	0.80	1.00	1.00
II	1.00	1.00	1.00	1.00
III	1.10	1.15	1.00	1.25
IV	1.20	1.25	1.00	1.50

Note: The component importance factor,  $I_p$ , applicable to earthquake loads, is not included in this table because it depends on the importance of the individual component rather than that of the building as a whole, or its occupancy. Refer to Section 13.1.3.

**Table C.19 : Response Modification and Over Strength Factor (R&D) [11]**

Seismic Force-Resisting System	ASCE 7 Section Where Detailing Requirements Are Specified	Response Modification Coefficient, $R^a$	Overstrength Factor, $\Omega_o^b$	Deflection Amplification Factor, $C_d^c$	Structural System Limitations Including Structural Height, $h_s$ (ft) Limits <sup>d</sup>				
					Seismic Design Category				
					B	C	D <sup>e</sup>	E <sup>e</sup>	F <sup>e</sup>
18. Ordinary reinforced masonry shear walls	14.4	2	2½	2	NL	160	NP	NP	NP
19. Detailed plain masonry shear walls	14.4	2	2½	2	NL	NP	NP	NP	NP
20. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP	NP	NP
21. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP	NP	NP
22. Light-frame (wood) walls sheathed with wood structural panels rated for shear resistance	14.5	7	2½	4½	NL	NL	65	65	65
23. Light-frame (cold-formed steel) walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1	7	2½	4½	NL	NL	65	65	65
24. Light-frame walls with shear panels of all other materials	14.1 and 14.5	2½	2½	2½	NL	NL	35	NP	NP
25. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	160	160	100
26. Steel special plate shear walls	14.1	7	2	6	NL	NL	160	160	100
<b>C. MOMENT-RESISTING FRAME SYSTEMS</b>									
1. Steel special moment frames	14.1 and 12.2.5.5	8	3	5½	NL	NL	NL	NL	NL
2. Steel special truss moment frames	14.1	7	3	5½	NL	NL	160	100	NP
3. Steel intermediate moment frames	12.2.5.7 and 14.1	4½	3	4	NL	NL	35 <sup>k</sup>	NP <sup>k</sup>	NP <sup>k</sup>
4. Steel ordinary moment frames	12.2.5.6 and 14.1	3½	3	3	NL	NL	NP <sup>j</sup>	NP <sup>j</sup>	NP <sup>j</sup>
5. Special reinforced concrete moment frames <sup>m</sup>	12.2.5.5 and 14.2	8	3	5½	NL	NL	NL	NL	NL
6. Intermediate reinforced concrete moment frames	14.2	5	3	4½	NL	NL	NP	NP	NP
7. Ordinary reinforced concrete moment frames	14.2	3	3	2½	NL	NP	NP	NP	NP
8. Steel and concrete composite special moment frames	12.2.5.5 and 14.3	8	3	5½	NL	NL	NL	NL	NL
9. Steel and concrete composite intermediate moment frames	14.3	5	3	4½	NL	NL	NP	NP	NP
10. Steel and concrete composite partially restrained moment frames	14.3	6	3	5½	160	160	100	NP	NP
11. Steel and concrete composite ordinary moment frames	14.3	3	3	2½	NL	NP	NP	NP	NP
12. Cold-formed steel—special bolted moment frame <sup>n</sup>	14.1	3½	3 <sup>o</sup>	3½	35	35	35	35	35
<b>D. DUAL SYSTEMS WITH SPECIAL MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES</b>									
12.2.5.1									
1. Steel eccentrically braced frames	14.1	8	2½	4	NL	NL	NL	NL	NL
2. Steel special concentrically braced frames	14.1	7	2½	5½	NL	NL	NL	NL	NL
3. Special reinforced concrete shear walls <sup>e,h</sup>	14.2	7	2½	5½	NL	NL	NL	NL	NL
4. Ordinary reinforced concrete shear walls <sup>f</sup>	14.2	6	2½	5	NL	NL	NP	NP	NP
5. Steel and concrete composite eccentrically braced frames	14.3	8	2½	4	NL	NL	NL	NL	NL
6. Steel and concrete composite special concentrically braced frames	14.3	6	2½	5	NL	NL	NL	NL	NL
7. Steel and concrete composite plate shear walls	14.3	7½	2½	6	NL	NL	NL	NL	NL
8. Steel and concrete composite special shear walls	14.3	7	2½	6	NL	NL	NL	NL	NL
9. Steel and concrete composite ordinary shear walls	14.3	6	2½	5	NL	NL	NP	NP	NP
10. Special reinforced masonry shear walls	14.4	5½	3	5	NL	NL	NL	NL	NL
11. Intermediate reinforced masonry shear walls	14.4	4	3	3½	NL	NL	NP	NP	NP
12. Steel buckling-restrained braced frames	14.1	8	2½	5	NL	NL	NL	NL	NL
13. Steel special plate shear walls	14.1	8	2½	6½	NL	NL	NL	NL	NL
<b>E. DUAL SYSTEMS WITH INTERMEDIATE MOMENT FRAMES CAPABLE OF RESISTING AT LEAST 25% OF PRESCRIBED SEISMIC FORCES</b>									
12.2.5.1									
1. Steel special concentrically braced frames <sup>p</sup>	14.1	6	2½	5	NL	NL	35	NP	NP
2. Special reinforced concrete shear walls <sup>e,h</sup>	14.2	6½	2½	5	NL	NL	160	100	100
3. Ordinary reinforced masonry shear walls	14.4	3	3	2½	NL	160	NP	NP	NP
4. Intermediate reinforced masonry shear walls	14.4	3½	3	3	NL	NL	NP	NP	NP

## APPENDIX D: ETABS MODELING

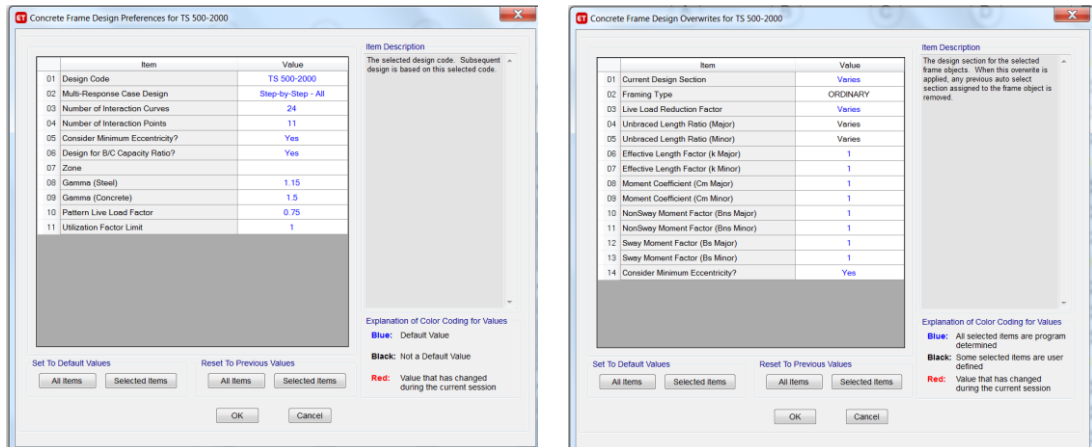


Figure D.1 : Concrete Frame Design Preferences

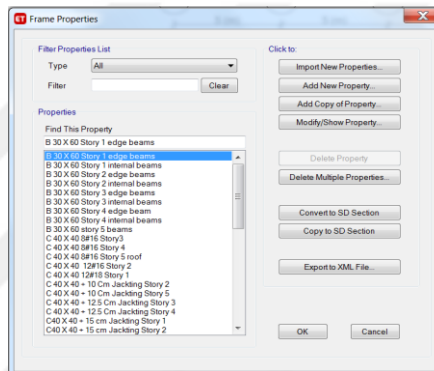


Figure D.2 : Used Frames Sections

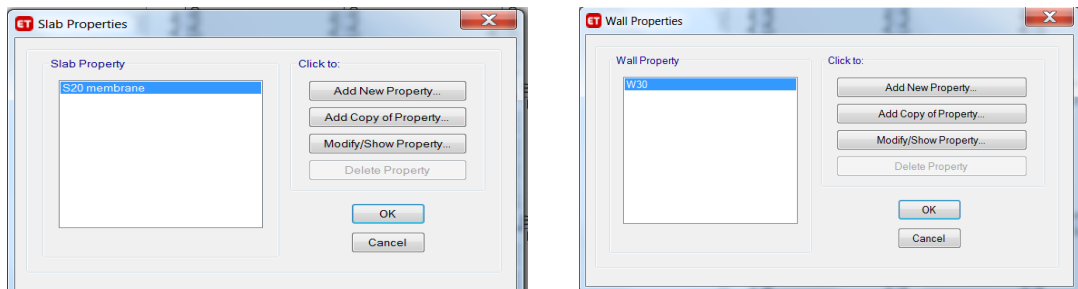


Figure D.3 : Used Slab and shear wall sections

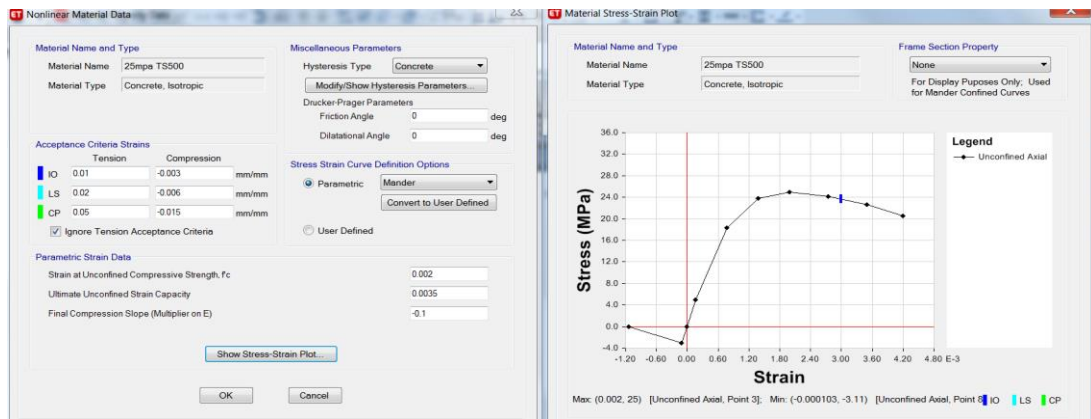


Figure D.4 : Nonlinear Material Data and Stress-Strain Plot of C25 Concrete

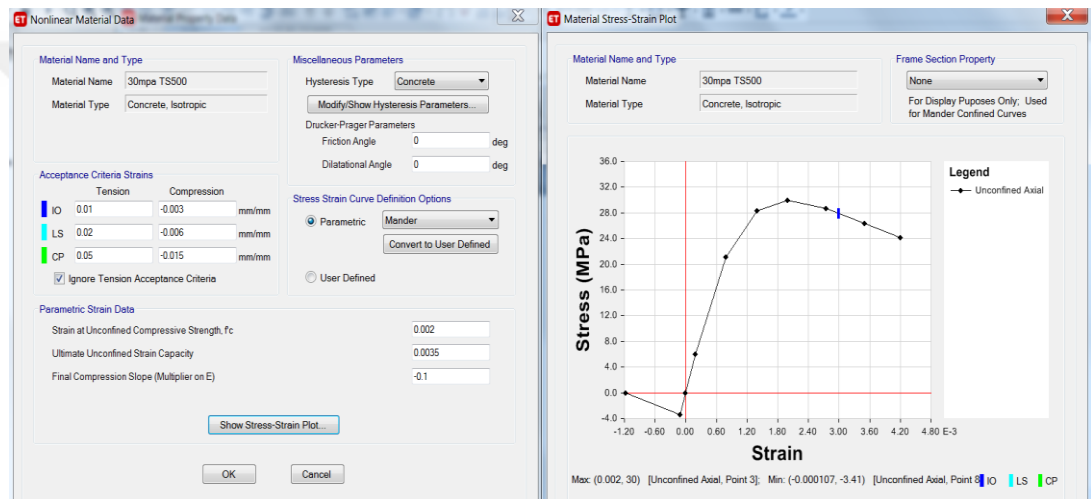


Figure D.5 : Nonlinear Material Data and Stress-Strain Plot of C30 Concrete

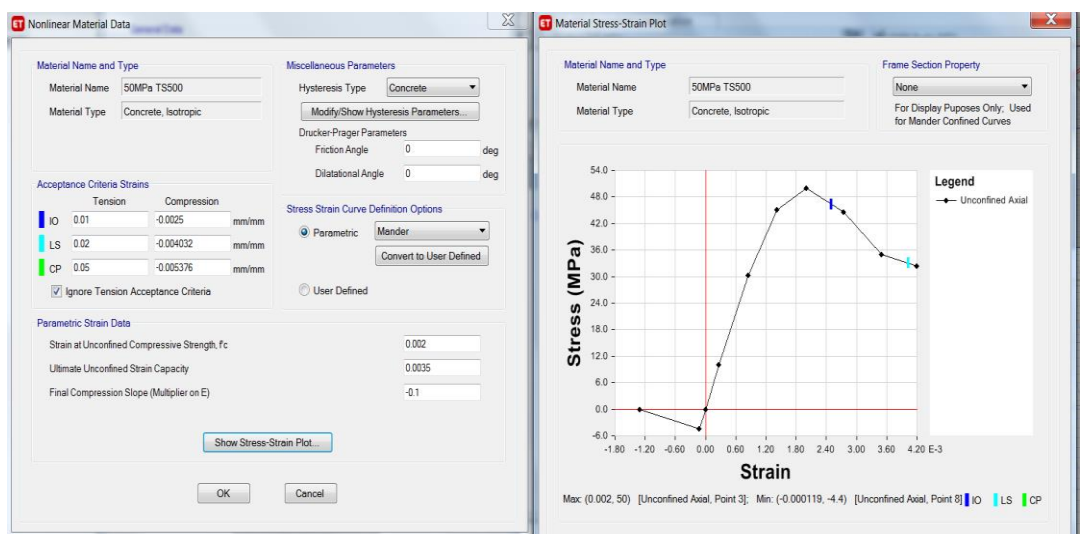
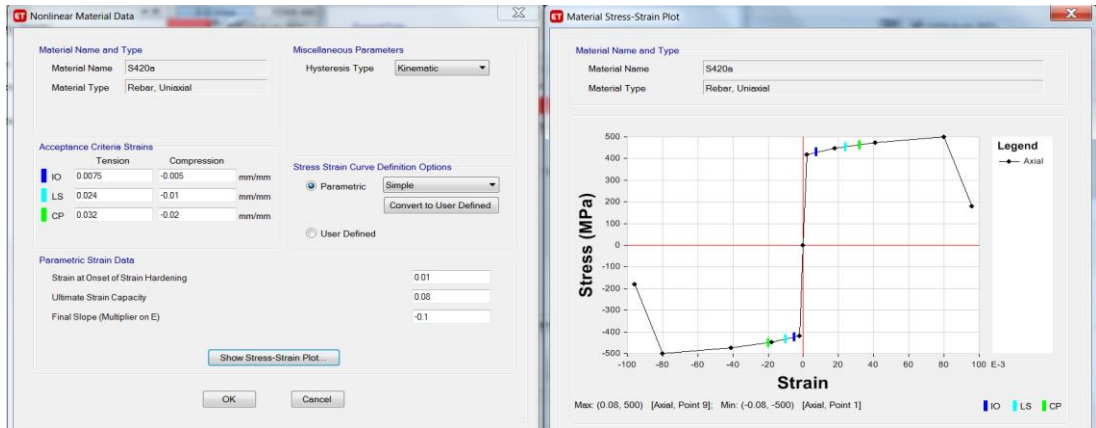
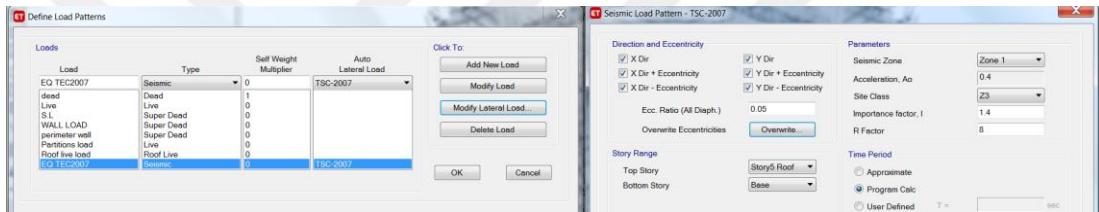


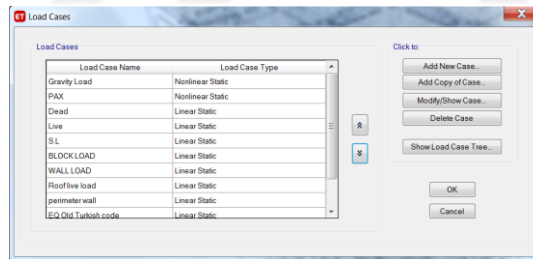
Figure D.6 : Nonlinear Material Data and Stress-Strain Plot of C50 Concrete



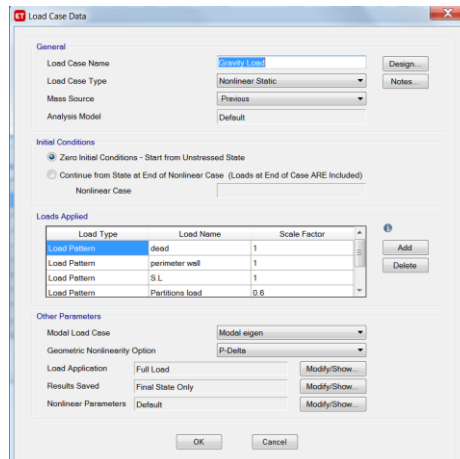
**Figure D.7 :** Nonlinear Material Data and Stress-Strain Plot of S420a Concrete



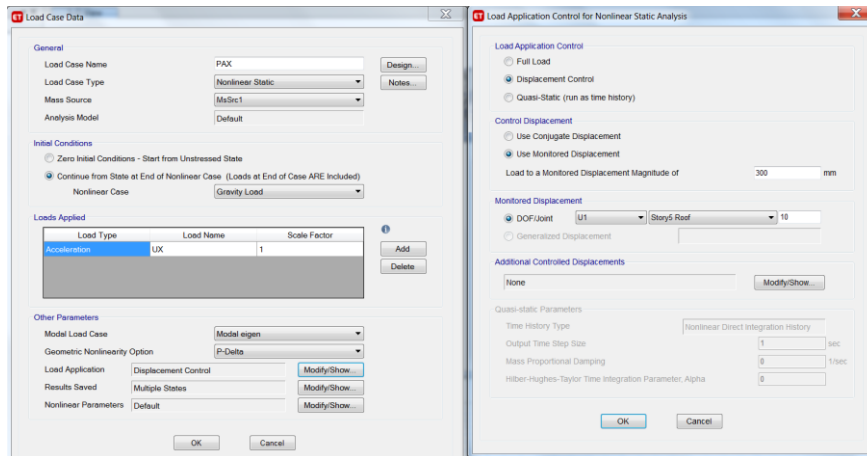
**Figure D.8 :** Load patterns & TEC2007 Seismic Load Configurations



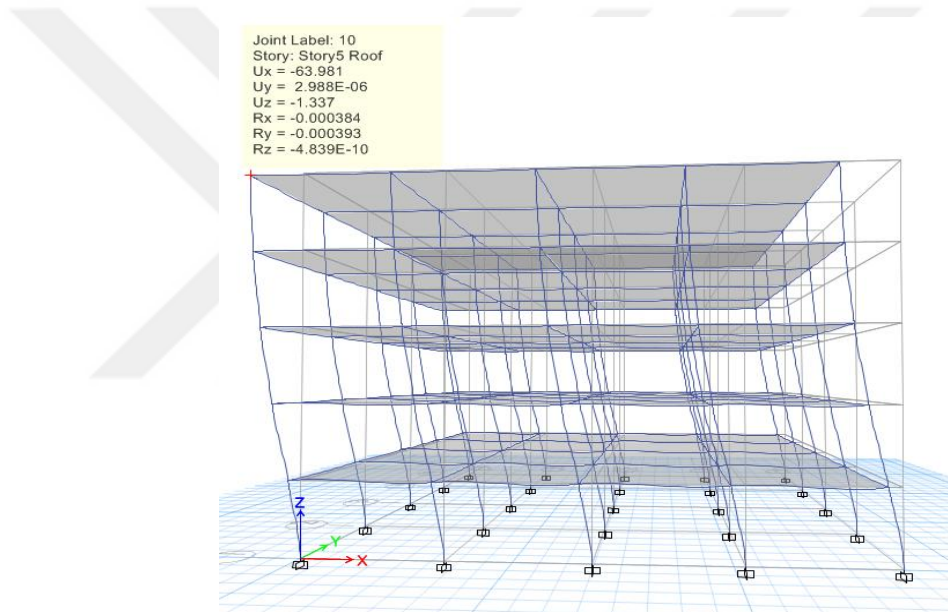
**Figure D.9 :** Used Load Cases



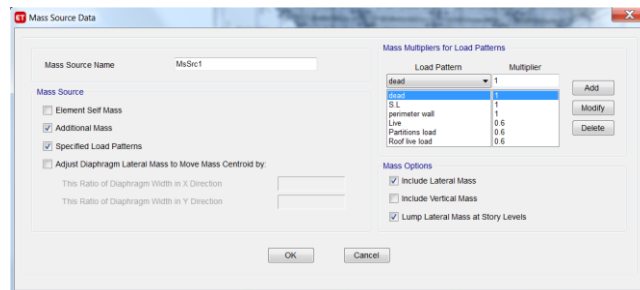
**Figure D.10 :** Gravity Load Case Data for PA (1.0 Dead loads +0.6 live loads)



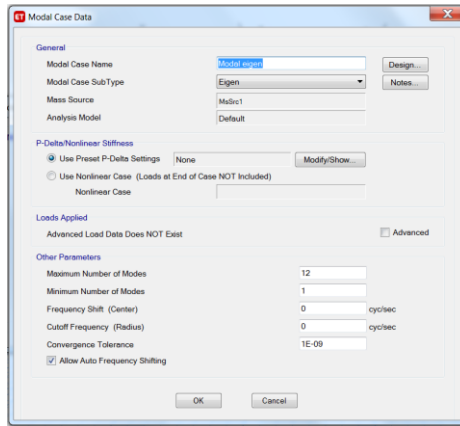
**Figure D.11 : Load Case Data for X Direction-Push**



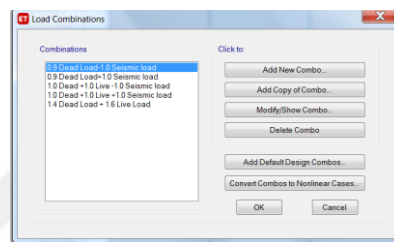
**Figure D.12 : Monitored Displacement Joint**



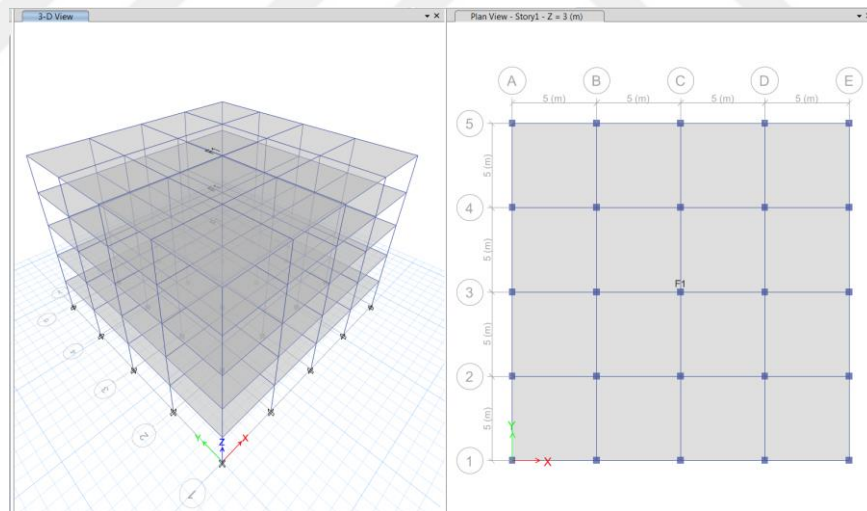
**Figure D.13 : Mass Source Definition for Modal & Static Analysis**



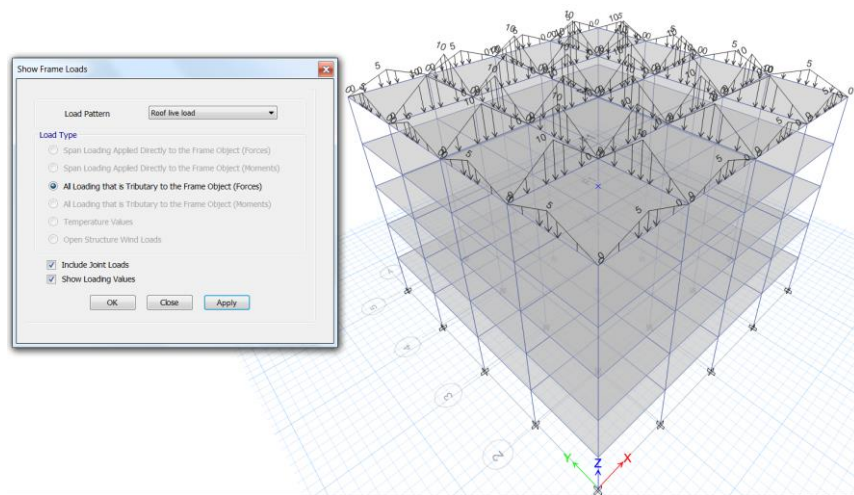
**Figure D.14 : Modal Analysis Case Data**



**Figure D.15 : Used Load Combinations**



**Figure D.16 : 3D and 2D View for Model Drawing**



**Figure D.17 : Example of Slab Load Distribution to Beam**



**Figure D.18 : Auto Seismic Load (TEC2007) Assigned to Stories by ETABS**

**ET Auto Hinge Assignment Data**

Auto Hinge Type  
From Tables In ASCE 41-13

Select a Hinge Table  
Table 10-7 (Concrete Beams - Flexure) Item i

Degree of Freedom  
 M2  
 M3

V Value From  
 Case/Combo PAX  
 User Value V2  kN

Transverse Reinforcing  
 Transverse Reinforcing is Conforming

Reinforcing Ratio (p - p') / pbalanced  
 From Current Design  
 User Value (for positive bending)

Deformation Controlled Hinge Load Carrying Capacity  
 Drops Load After Point E  
 Is Extrapolated After Point E

OK Cancel

**ET Auto Hinge Assignment Data**

Auto Hinge Type  
From Tables In ASCE 41-13

Select a Hinge Table  
Table 10-8 (Concrete Columns)

Degree of Freedom  
 M2     P-M2     Parametric P-M2-M3  
 M3     P-M3  
 M2-M3     P-M2-M3

P and V Values From  
 Case/Combo PAX  
 User Value  
V2  kN    V3  kN

Concrete Column Failure Condition  
 Condition i - Flexure     Condition iii - Shear  
 Condition ii - Flexure/Shear     Condition iv - Development

Shear Reinforcing Ratio  $p = A_v / (b_w * s)$   
 From Current Design  
 User Value 0.014

Deformation Controlled Hinge Load Carrying Capacity  
 Drops Load After Point E  
 Is Extrapolated After Point E

OK Cancel

**Figure D.19 :** Auto Hinge Assignment date for Beams and Columns

# APPENDIX E: FRAMES LONGITUDINAL REINFORCEMENT ETABS RESULTS

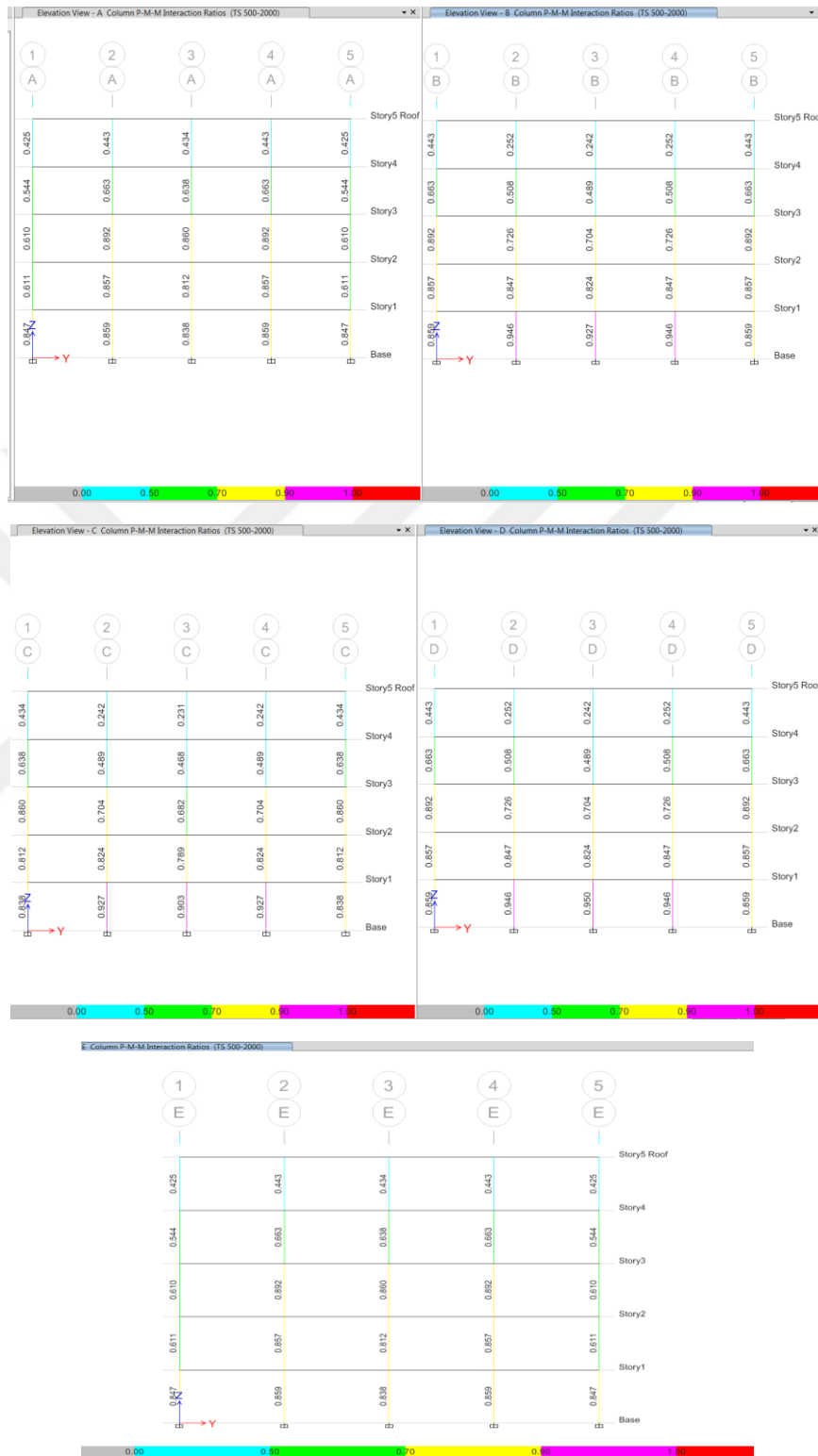


Figure E.1 : Column Capacity Ratio for Chosen reinforcement

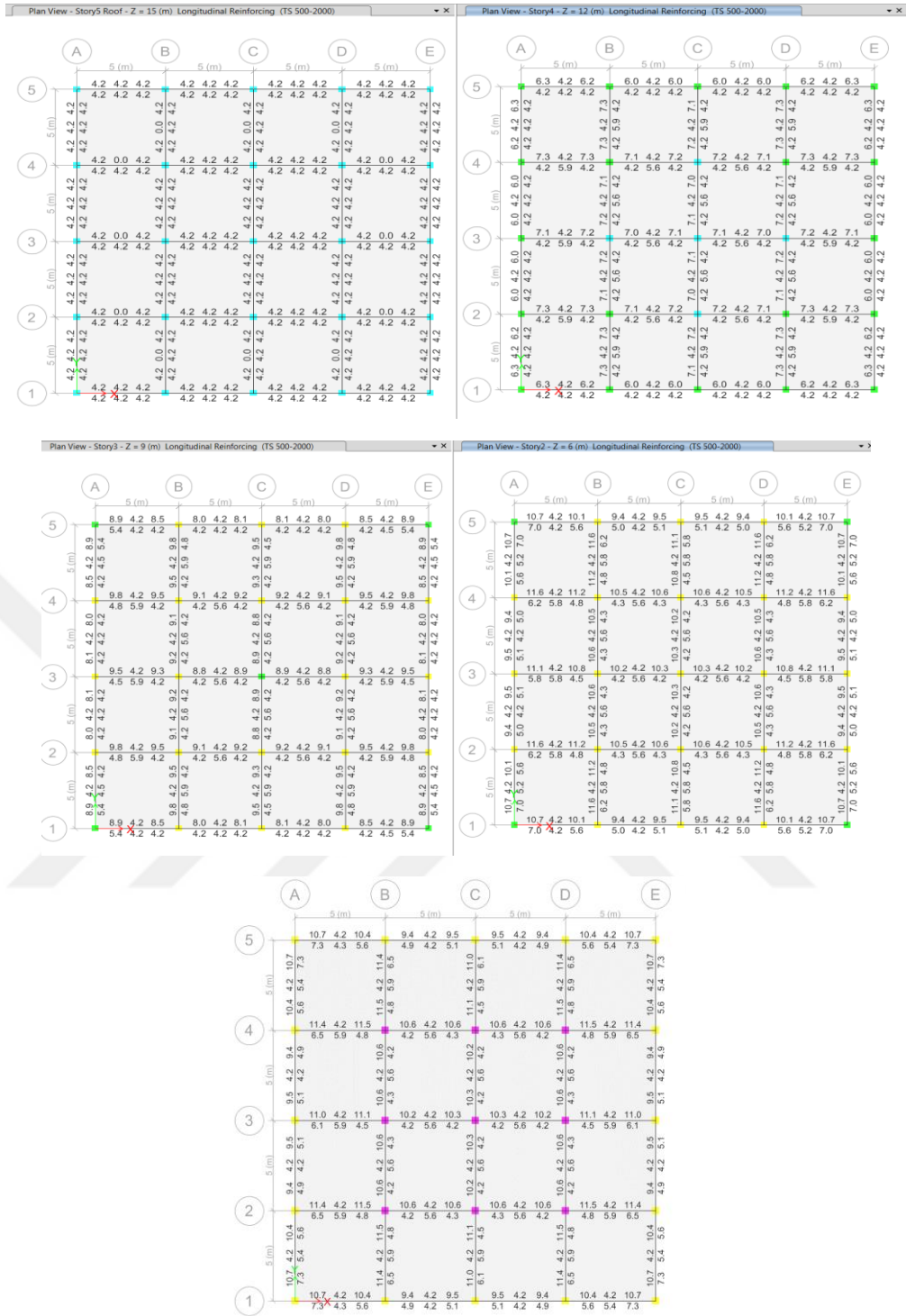


Figure E.2 : Beams longitudinal Reinforcement Area (cm<sup>2</sup>)

## APPENDIX F: PUSHOVER RESULTS

### APPENDIX F.1: Capacity Curve & Capacity Spectrum Data

**Table F.1 : Base Shear vs Monitored Displacement Capacity Curve Data for Actual school building**

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	650	0	0	0	0	650	0	0	0	650
1	-30	2230.4935	650	0	0	0	0	650	0	0	0	650
2	-49.013	3644.1143	648	2	0	0	0	650	0	0	0	650
3	-63.981	4598.2628	595	55	0	0	0	650	0	0	0	650
4	-66.449	4668.2918	580	70	0	0	0	650	0	0	0	650
5	-82.65	4861.7832	537	113	0	0	0	645	5	0	0	650
6	-94.997	4920.8069	516	134	0	0	0	591	59	0	0	650
7	-95	4921.0297	516	134	0	0	0	591	59	0	0	650
8	-123.125	4999.9086	491	159	0	0	0	577	73	0	0	650
9	-123.594	5000.5747	491	159	0	0	0	577	73	0	0	650
10	-124.062	5000.8305	491	159	0	0	0	577	73	0	0	650
11	-130.156	5007.8752	487	163	0	0	0	575	75	0	0	650
12	-139.062	5014.2058	479	171	0	0	0	555	89	0	6	650
13	-140	5014.4639	479	171	0	0	0	551	93	0	6	650
14	-140.234	5014.7699	479	171	0	0	0	551	93	0	6	650
15	-140.351	5014.7824	479	171	0	0	0	551	93	0	6	650
16	-146.152	5017.5767	476	174	0	0	0	539	105	0	6	650
17	-150.195	5020.0281	473	177	0	0	0	539	105	0	6	650
18	-150.312	5019.9136	473	177	0	0	0	539	105	0	6	650
19	-151.03	5020.5135	471	179	0	0	0	539	105	0	6	650
20	-151.037	5020.3178	471	179	0	0	0	539	105	0	6	650

**Table F.2 : Capacity Spectrum for Actual School Building Data**

Sd	Sa	Period
mm	g	Sec
0	0	0
23.765	0.097786	0.989
38.827	0.159759	0.989
50.855	0.200341	1.011
52.928	0.202376	1.026
66.953	0.204581	1.148
77.826	0.204847	1.237
77.829	0.204856	1.237
102.668	0.205688	1.418
103.082	0.205697	1.42
103.497	0.205684	1.423
108.886	0.205725	1.46
116.799	0.205632	1.512
117.633	0.205608	1.518
117.841	0.205613	1.519
117.945	0.205609	1.52
123.111	0.205518	1.553
126.702	0.205502	1.575
126.807	0.205492	1.576
127.443	0.2055	1.58
127.451	0.205491	1.58

**Table F.3 : Base Shear vs Monitored Displacement Capacity Curve for Retrofitted School Building by Columns Jacketing Story 1**

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	650	0	0	0	0	650	0	0	0	650
1	-30	3397.5224	650	0	0	0	0	650	0	0	0	650
2	-38.775	4391.248	648	2	0	0	0	650	0	0	0	650
3	-46.233	5122.5425	612	38	0	0	0	650	0	0	0	650
4	-57.942	5741.5051	567	83	0	0	0	650	0	0	0	650
5	-73.624	6105.3674	516	134	0	0	0	620	30	0	0	650
6	-94.219	6361.6816	484	166	0	0	0	597	53	0	0	650
7	-97.522	6377.7722	484	166	0	0	0	587	63	0	0	650
8	-100.825	6385.0726	484	166	0	0	0	581	69	0	0	650
9	-103.302	6399.8031	484	166	0	0	0	579	69	0	2	650
10	-105.82	6399.7673	484	166	0	0	0	570	78	0	2	650
11	-108.71	6416.4005	484	166	0	0	0	568	78	0	4	650
12	-112.013	6419.3156	484	166	0	0	0	563	83	0	4	650
13	-112.508	6421.173	484	166	0	0	0	561	85	0	4	650
14	-112.511	6420.8341	484	166	0	0	0	561	85	0	4	650
15	-114.167	6427.8718	484	166	0	0	0	557	89	0	4	650
16	-114.655	6430.9854	484	166	0	0	0	557	89	0	4	650
17	-116.137	6433.901	484	166	0	0	0	552	94	0	4	650
18	-117.065	6437.1967	484	166	0	0	0	551	94	0	5	650
19	-117.068	6436.3595	484	166	0	0	0	551	94	0	5	650
20	-117.356	6438.0881	484	166	0	0	0	551	94	0	5	650
21	-117.884	6439.2669	484	166	0	0	0	549	96	0	5	650
22	-118.468	6440.0836	484	166	0	0	0	549	96	0	5	650
23	-118.85	6442.1023	484	166	0	0	0	549	96	0	5	650
24	-119.041	6441.8564	484	166	0	0	0	549	96	0	5	650
25	-119.232	6442.859	484	166	0	0	0	548	97	0	5	650
26	-119.235	6438.819	484	166	0	0	0	548	97	0	5	650
27	-119.998	6441.0693	484	166	0	0	0	547	98	0	5	650
28	-120.38	6442.9732	484	166	0	0	0	547	98	0	5	650
29	-120.761	6444.157	484	166	0	0	0	547	98	0	5	650
30	-120.857	6444.6648	484	166	0	0	0	547	98	0	5	650
31	-121.444	6445.4631	484	166	0	0	0	547	98	0	5	650
32	-121.725	6446.9754	484	164	2	0	0	547	98	0	5	650
33	-121.728	6447.3151	483	165	2	0	0	547	98	0	5	650
34	-121.731	6447.5187	483	165	2	0	0	547	98	0	5	650
35	-121.734	6447.7712	483	165	2	0	0	547	98	0	5	650
36	-121.737	6448.0097	483	165	2	0	0	547	98	0	5	650
37	-121.74	6448.234	483	165	2	0	0	547	98	0	5	650
38	-121.743	6448.4918	483	165	2	0	0	547	98	0	5	650
39	-121.746	6448.6749	483	165	2	0	0	547	98	0	5	650
40	-121.749	6448.9105	483	165	2	0	0	547	98	0	5	650
41	-121.752	6449.1339	483	165	2	0	0	547	98	0	5	650
42	-121.755	6449.363	482	166	2	0	0	547	98	0	5	650
43	-121.758	6449.562	482	166	2	0	0	547	98	0	5	650
44	-121.761	6449.6945	482	166	2	0	0	547	98	0	5	650
45	-121.764	6449.8633	482	166	2	0	0	547	98	0	5	650
46	-121.767	6446.9937	482	166	2	0	0	547	96	0	7	650
47	-121.99	6447.0231	482	166	2	0	0	547	96	0	7	650
48	-122.101	6447.0306	482	166	2	0	0	547	96	0	7	650
49	-122.212	6445.4541	482	166	2	0	0	547	96	0	7	650
50	-122.881	6443.5972	482	165	3	0	0	547	96	0	7	650
51	-123.327	6441.5567	482	165	3	0	0	547	96	0	7	650
52	-123.549	6441.0164	482	165	3	0	0	547	96	0	7	650
53	-123.772	6439.7778	482	163	5	0	0	547	96	0	7	650
54	-124.441	6430.3428	482	160	8	0	0	544	95	0	11	650
55	-128.451	6322.8404	482	159	9	0	0	544	91	0	15	650

**Table F.4 : Capacity Spectrum for Retrofitted School Building by Columns**

## Jacketing Story 1

<b>Sd</b>	<b>Sa</b>	<b>Period</b>
mm	g	Sec
0	0	0
23.59	0.174163	0.738
30.49	0.225103	0.738
36.457	0.261913	0.749
46.326	0.290312	0.801
59.919	0.30686	0.887
78.049	0.319206	0.992
81.034	0.319906	1.01
84.04	0.320067	1.028
86.269	0.320784	1.04
88.575	0.320642	1.055
91.185	0.321431	1.069
94.207	0.321439	1.086
94.656	0.321526	1.089
94.659	0.321509	1.089
96.159	0.321828	1.097
96.599	0.321981	1.099
97.951	0.322102	1.106
98.797	0.322244	1.111
98.8	0.322202	1.111
99.057	0.322286	1.112
99.539	0.322332	1.115
100.08	0.322374	1.118
100.424	0.322469	1.12
100.594	0.322442	1.121
100.766	0.322492	1.122
100.774	0.322288	1.122
101.473	0.322398	1.126
101.816	0.322478	1.127
102.159	0.322517	1.129
102.247	0.322546	1.13
102.786	0.322581	1.133
103.04	0.322649	1.134
103.042	0.322666	1.134
103.045	0.322676	1.134
103.047	0.322689	1.134
103.05	0.322701	1.134
103.052	0.322712	1.134
103.055	0.322725	1.134
103.057	0.322735	1.134
103.06	0.322746	1.134
103.062	0.322758	1.134
103.065	0.322769	1.134
103.067	0.322779	1.134
103.07	0.322786	1.134
103.072	0.322794	1.134
103.078	0.322648	1.134
103.282	0.322641	1.135
103.383	0.322631	1.136
103.487	0.322548	1.136
104.104	0.322425	1.14
104.52	0.322297	1.143
104.723	0.322253	1.144
104.933	0.322179	1.145
105.557	0.321631	1.149
109.396	0.3156	1.181

**Table F.5 : Base Shear vs Monitored Displacement Capacity Curve for Retrofitted School Building by Columns Jacketing Story 1&2**

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	650	0	0	0	0	650	0	0	0	650
1	-30	4800.5222	650	0	0	0	0	650	0	0	0	650
2	-32.8	5248.4998	648	2	0	0	0	650	0	0	0	650
3	-39.745	6249.9641	620	30	0	0	0	650	0	0	0	650
4	-47.503	6868.9102	589	61	0	0	0	650	0	0	0	650
5	-57.851	7277.9423	550	100	0	0	0	637	6	0	7	650
6	-60.42	7328.649	542	108	0	0	0	627	16	0	7	650
7	-63.425	7417.6898	538	112	0	0	0	624	19	0	7	650
8	-65.378	7440.0039	538	112	0	0	0	624	19	0	7	650
9	-68.522	7505.2911	523	127	0	0	0	621	22	0	7	650
10	-68.955	7509.5571	523	127	0	0	0	619	23	0	8	650
11	-69.388	7511.6977	523	127	0	0	0	618	24	0	8	650
12	-69.391	7500.5171	523	127	0	0	0	618	24	0	8	650
13	-69.625	7504.049	523	127	0	0	0	618	24	0	8	650
14	-69.977	7507.1828	523	127	0	0	0	618	24	0	8	650
15	-70.094	7507.2292	523	127	0	0	0	618	24	0	8	650
16	-70.211	7508.4761	523	127	0	0	0	618	24	0	8	650
17	-70.328	7507.492	523	127	0	0	0	617	25	0	8	650
18	-70.445	7508.7444	523	127	0	0	0	615	27	0	8	650
19	-70.562	7507.9066	523	127	0	0	0	614	28	0	8	650
20	-70.68	7508.9746	523	127	0	0	0	612	30	0	8	650
21	-70.738	7507.6268	523	127	0	0	0	612	30	0	8	650
22	-70.797	7507.2132	523	127	0	0	0	612	30	0	8	650
23	-70.914	7508.0499	523	127	0	0	0	612	30	0	8	650
24	-71.031	7507.3371	523	127	0	0	0	612	30	0	8	650
25	-71.148	7508.3654	523	127	0	0	0	612	30	0	8	650
26	-71.266	7507.2223	523	127	0	0	0	612	30	0	8	650
27	-71.383	7508.1351	523	127	0	0	0	612	30	0	8	650
28	-71.5	7507.1762	523	127	0	0	0	612	30	0	8	650
29	-71.617	7508.4466	523	127	0	0	0	612	30	0	8	650
30	-71.734	7504.2525	523	127	0	0	0	612	30	0	8	650
31	-71.852	7507.11	523	127	0	0	0	612	30	0	8	650
32	-71.969	7504.6003	523	127	0	0	0	612	30	0	8	650
33	-72.906	7493.4848	523	127	0	0	0	611	30	0	9	650
34	-73.141	7496.5316	523	127	0	0	0	611	30	0	9	650
35	-73.375	7496.6371	523	127	0	0	0	611	30	0	9	650
36	-73.609	7499.0567	523	127	0	0	0	611	30	0	9	650
37	-73.856	7499.479	523	127	0	0	0	611	30	0	9	650
38	-73.867	7499.182	523	127	0	0	0	611	30	0	9	650
39	-73.985	7499.9091	523	127	0	0	0	610	31	0	9	650
40	-74.102	7499.7547	523	127	0	0	0	610	31	0	9	650
41	-74.219	7499.0176	523	127	0	0	0	608	33	0	9	650
42	-74.336	7500.1706	523	127	0	0	0	603	38	0	9	650
43	-74.395	7497.6714	523	127	0	0	0	603	38	0	9	650
44	-74.746	7494.0284	523	127	0	0	0	603	38	0	9	650
45	-75.039	7496.8569	523	127	0	0	0	603	38	0	9	650
46	-75.068	7494.498	523	127	0	0	0	603	38	0	9	650
47	-75.127	7494.9919	523	127	0	0	0	603	38	0	9	650
48	-75.186	7493.4603	523	127	0	0	0	603	38	0	9	650
49	-75.654	7487.3092	523	127	0	0	0	603	38	0	9	650
50	-75.889	7489.3472	523	127	0	0	0	603	38	0	9	650
51	-76.123	7489.9354	523	127	0	0	0	603	38	0	9	650
52	-76.126	7490.6181	523	127	0	0	0	603	38	0	9	650
53	-76.129	7491.0525	523	127	0	0	0	603	38	0	9	650
54	-76.132	7491.5432	523	127	0	0	0	603	38	0	9	650
55	-76.135	7492.1033	523	127	0	0	0	603	38	0	9	650
56	-76.138	7492.4191	523	127	0	0	0	603	38	0	9	650
57	-76.141	7492.8694	523	127	0	0	0	603	38	0	9	650
58	-76.144	7493.3013	523	127	0	0	0	603	38	0	9	650
59	-76.147	7493.7134	523	127	0	0	0	603	38	0	9	650
60	-76.15	7494.112	523	127	0	0	0	603	38	0	9	650
61	-76.153	7494.4877	523	127	0	0	0	603	38	0	9	650
62	-76.156	7494.8491	523	127	0	0	0	603	38	0	9	650
63	-75.879	7450.4155	523	127	0	0	0	603	38	0	9	650

**Table F.6 : Capacity Spectrum for Retrofitted School Building by Columns Jacketing Story 1&2**

<b>Sd</b>	<b>Sa</b>	<b>Period</b>
mm	g	Sec
0	0	0
22.473	0.253628	0.597
24.57	0.277297	0.597
29.861	0.33138	0.602
36.213	0.370413	0.627
44.999	0.403803	0.67
47.077	0.406741	0.683
49.739	0.415053	0.695
51.442	0.417125	0.705
54.219	0.422971	0.718
54.609	0.423442	0.721
54.996	0.423666	0.723
55.01	0.423133	0.723
55.219	0.423458	0.725
55.539	0.423838	0.726
55.643	0.423895	0.727
55.754	0.424051	0.728
55.86	0.424076	0.728
55.972	0.424205	0.729
56.082	0.424248	0.729
56.19	0.424373	0.73
56.25	0.424352	0.73
56.305	0.424367	0.731
56.415	0.424534	0.731
56.527	0.424525	0.732
56.637	0.424691	0.733
56.747	0.42464	0.733
56.854	0.424794	0.734
56.965	0.424774	0.735
57.071	0.42491	0.735
57.188	0.424758	0.736
57.294	0.424972	0.737
57.408	0.424917	0.737
58.319	0.424936	0.743
58.531	0.425157	0.744
58.761	0.42537	0.746
58.971	0.425578	0.747
59.209	0.425775	0.748
59.218	0.425769	0.748
59.323	0.425831	0.749
59.432	0.42591	0.749
59.543	0.425937	0.75
59.65	0.426051	0.751
59.708	0.425963	0.751
60.054	0.425989	0.753
60.327	0.426306	0.755
60.356	0.4262	0.755
60.41	0.426239	0.755
60.469	0.426202	0.756
60.927	0.426086	0.759
61.149	0.426344	0.76
61.371	0.426531	0.761
61.373	0.426568	0.761
61.375	0.42659	0.761
61.377	0.426617	0.761
61.379	0.426646	0.761
61.382	0.426665	0.761
61.384	0.426689	0.761
61.386	0.426713	0.761
61.389	0.426735	0.761
61.391	0.426757	0.761
61.393	0.426778	0.761
61.396	0.426798	0.761

**Table F.7 : Base Shear vs Monitored Displacement Capacity Curve for Retrofitted School Building by Columns Jacketing for All Stories.**

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	650	0	0	0	0	650	0	0	0	650
1	-30	6970.9544	650	0	0	0	0	650	0	0	0	650
2	-31.418	7300.3694	647	3	0	0	0	650	0	0	0	650
3	-53.197	10991.0148	519	131	0	0	0	650	0	0	0	650
4	-102.869	15803.0125	458	192	0	0	0	542	108	0	0	650
5	-137.919	18975.7353	448	202	0	0	0	501	149	0	0	650
6	-147.058	19747.6	420	230	0	0	0	496	154	0	0	650
7	-147.702	19780.0244	417	233	0	0	0	492	158	0	0	650
8	-151.438	19883.6905	417	233	0	0	0	490	160	0	0	650
9	-152.057	19894.6921	417	233	0	0	0	490	160	0	0	650
10	-152.363	19905.6871	417	233	0	0	0	490	160	0	0	650
11	-152.366	19903.8436	417	233	0	0	0	490	160	0	0	650
12	-153.267	19926.3561	417	233	0	0	0	490	160	0	0	650
13	-153.996	19922.5339	417	233	0	0	0	490	160	0	0	650
14	-155.452	19961.6193	417	233	0	0	0	490	160	0	0	650
15	-155.455	19958.9926	417	233	0	0	0	490	160	0	0	650
16	-157.881	19935.4766	417	233	0	0	0	490	160	0	0	650
17	-161.063	20056.3252	417	233	0	0	0	490	160	0	0	650
18	-161.071	20057.7325	417	233	0	0	0	490	160	0	0	650
19	-161.074	20057.4956	417	233	0	0	0	490	160	0	0	650
20	-170.622	19909.7367	417	233	0	0	0	470	180	0	0	650
21	-181.97	20353.6347	417	233	0	0	0	470	180	0	0	650
22	-196.936	20349.3007	417	233	0	0	0	450	200	0	0	650
23	-226.946	20939.4141	417	233	0	0	0	425	225	0	0	650
24	-256.915	21365.2024	417	233	0	0	0	425	225	0	0	650
25	-276.857	22008.7567	417	193	40	0	0	425	200	25	0	650
26	-279.395	22056.2504	417	182	51	0	0	425	185	40	0	650
27	-281.592	22072.2312	417	173	60	0	0	425	174	51	0	650
28	-287.941	22060.4703	417	158	75	0	0	425	165	60	0	650
29	-291.177	22061.4931	417	145	88	0	0	425	150	75	0	650
30	-293.165	22026.042	408	148	94	0	0	425	137	88	0	650
31	-294.426	21981.3317	408	142	100	0	0	425	131	94	0	650
32	-298.966	21619.1302	400	138	112	0	0	425	113	112	0	650
33	-300	21482.0823	400	136	114	0	0	425	113	112	0	650

**Table F.8 : Capacity Spectrum for Retrofitted School Building Columns Jacketing for All Stories.**

<b>Sd</b>	<b>Sa</b>	<b>Period</b>
mm	g	Sec
0	0	0
21.797	0.316063	0.527
22.827	0.330999	0.527
38.094	0.509059	0.549
71.868	0.76344	0.616
95.849	0.9262	0.645
102.119	0.964633	0.653
102.58	0.966111	0.654
105.156	0.966067	0.662
105.555	0.96552	0.663
105.781	0.96579	0.664
105.782	0.965689	0.664
106.394	0.965357	0.666
106.93	0.963784	0.668
107.904	0.963722	0.671
107.905	0.963584	0.671
109.654	0.957574	0.679
111.817	0.960213	0.685
111.822	0.960269	0.685
111.824	0.960255	0.685
118.248	0.934183	0.714
126.284	0.948562	0.732
137.336	0.932096	0.77
157.791	0.939371	0.822
179.997	0.944841	0.876
193.658	0.970181	0.896
195.341	0.971434	0.9
196.824	0.972117	0.903
201.081	0.971687	0.913
203.236	0.972251	0.917
204.485	0.971555	0.92
205.239	0.970657	0.923
207.81	0.959257	0.934
208.396	0.954453	0.938

**Table F.9 : Base Shear vs Monitored Displacement Capacity Curve for Retrofitted School Building by Shear Wall Addition at Structure Corners**

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	690	0	0	0	0	690	0	0	0	690
1	-6.296	4246.9623	688	2	0	0	0	690	0	0	0	690
2	-18.049	9529.2807	654	36	0	0	0	690	0	0	0	690
3	-20.221	10082.1087	652	38	0	0	0	690	0	0	0	690
4	-21.361	10116.14	652	38	0	0	0	690	0	0	0	690
5	-27.86	11304.0627	640	50	0	0	0	690	0	0	0	690
6	-28.293	11221.9037	638	52	0	0	0	690	0	0	0	690
7	-36.601	12642.2916	630	60	0	0	0	680	10	0	0	690
8	-37.468	12641.3516	628	62	0	0	0	680	10	0	0	690
9	-38.917	12814.9646	625	65	0	0	0	680	10	0	0	690
10	-40.247	13088.8387	622	68	0	0	0	680	10	0	0	690
11	-40.69	13038.7732	622	68	0	0	0	680	10	0	0	690
12	-43.689	13417.9609	611	79	0	0	0	680	10	0	0	690
13	-45.314	13761.4538	603	87	0	0	0	676	14	0	0	690
14	-46.074	13802.1764	599	91	0	0	0	670	20	0	0	690
15	-50.051	14568.3635	586	104	0	0	0	668	22	0	0	690
16	-51.189	14675.136	580	110	0	0	0	666	24	0	0	690
17	-52.918	15035.7376	569	121	0	0	0	664	26	0	0	690
18	-53.447	15047.7335	569	121	0	0	0	664	26	0	0	690
19	-62.001	16405.4234	540	150	0	0	0	656	34	0	0	690
20	-62.004	16356.8485	540	150	0	0	0	656	34	0	0	690
21	-62.934	16535.2225	535	155	0	0	0	654	36	0	0	690
22	-62.937	16477.9491	535	155	0	0	0	654	36	0	0	690
23	-62.94	16465.5552	535	155	0	0	0	654	36	0	0	690
24	-63.044	16488.9413	535	155	0	0	0	654	36	0	0	690
25	-63.047	16468.8798	535	155	0	0	0	654	36	0	0	690
26	-84.769	18464.9735	453	235	2	0	0	631	59	0	0	690
27	-88.519	18631.1157	443	241	6	0	0	623	61	2	4	690
28	-92.269	18697.9632	432	252	6	0	0	620	61	4	5	690
29	-96.019	18723.9142	422	258	10	0	0	615	64	6	5	690
30	-99.769	18651.1701	419	261	10	0	0	611	64	8	7	690
31	-100.238	18650.2819	419	260	11	0	0	608	64	8	10	690
32	-100.706	18651.8942	419	260	11	0	0	607	63	9	11	690
33	-101.644	18659.4611	415	263	12	0	0	604	64	9	13	690
34	-102.113	18670.9565	415	263	12	0	0	602	63	10	15	690
35	-102.581	18587.4166	414	264	11	1	0	601	63	10	16	690
36	-103.05	18584.3949	413	265	11	1	0	601	63	10	16	690
37	-103.167	18586.7161	413	265	11	1	0	601	63	10	16	690
38	-103.285	18586.4704	413	265	11	1	0	601	63	10	16	690
39	-103.519	18587.0236	413	265	11	1	0	601	63	10	16	690
40	-103.587	18586.9307	413	265	11	1	0	601	63	10	16	690

**Table F.10** : Capacity spectrum for retrofitted school building by Shear wall addition at structure corners

<b>Sd</b>	<b>Sa</b>	<b>Period</b>
mm	g	Sec
0	0	0
4.455	0.198873	0.3
12.775	0.445874	0.34
14.31	0.472028	0.349
15.122	0.473427	0.359
19.721	0.529617	0.387
20.032	0.525575	0.392
25.908	0.592819	0.419
26.524	0.592665	0.424
27.55	0.600859	0.43
28.489	0.613888	0.432
28.805	0.611432	0.435
30.927	0.629353	0.445
32.072	0.645722	0.447
32.611	0.647573	0.45
35.415	0.684035	0.457
36.219	0.689039	0.46
37.436	0.706291	0.462
37.811	0.706808	0.464
43.834	0.771699	0.478
43.836	0.769405	0.479
44.49	0.777952	0.48
44.493	0.775205	0.481
44.495	0.774643	0.481
44.568	0.775755	0.481
44.57	0.7748	0.481
59.827	0.87318	0.525
62.445	0.881467	0.534
65.075	0.884808	0.544
67.686	0.886289	0.554
70.27	0.882953	0.566
70.58	0.882881	0.567
70.831	0.882597	0.568
71.341	0.882243	0.571
71.625	0.882546	0.572
72.246	0.881055	0.575
72.671	0.882186	0.576
72.76	0.88247	0.576
72.823	0.882356	0.576
72.957	0.882259	0.577
72.993	0.882182	0.577



**Table F.11 : Base Shear vs Monitored Displacement Capacity Curve for Retrofitted School Building by Shear Wall Addition at Structure Corners and Middle Edges**

Step	Monitored Displ	Base Force	A-B	B-C	C-D	D-E	>E	A-IO	IO-LS	LS-CP	>CP	Total
0	0	0	670	0	0	0	0	670	0	0	0	670
1	-6.274	5233.4446	668	2	0	0	0	670	0	0	0	670
2	-18.2	11791.5498	636	34	0	0	0	670	0	0	0	670
3	-28.486	14642.9159	611	59	0	0	0	670	0	0	0	670
4	-29.235	14213.9759	607	63	0	0	0	670	0	0	0	670
5	-37.772	16204.494	588	82	0	0	0	658	10	0	2	670
6	-39.905	16341.8827	584	84	2	0	0	658	10	0	2	670
7	-40.941	16511.8059	582	86	2	0	0	658	10	0	2	670
8	-43.543	17162.3021	574	94	2	0	0	656	12	0	2	670
9	-45.147	17437.1504	573	95	2	0	0	656	12	0	2	670
10	-46.942	17388.6371	567	101	2	0	0	654	14	0	2	670
11	-53.82	19125.5981	537	131	2	0	0	634	34	0	2	670
12	-55.112	19141.7242	531	135	4	0	0	632	36	0	2	670
13	-58.7	19947.3443	517	149	4	0	0	626	42	0	2	670
14	-59.098	16063.7223	487	175	6	0	2	604	64	0	2	670
15	-63.803	17282.2771	469	193	6	0	2	600	66	0	4	670
16	-61.782	16070.9632	469	193	6	0	2	600	66	0	4	670

**Table F.12 : Capacity Spectrum for Retrofitted School Building by Shear Wall Addition at Structure Corners and Middle Edges**

Sd	Sa	Period
mm	g	Sec
0	0	0
4.425	0.241332	0.272
12.847	0.54175	0.309
20.104	0.673216	0.347
20.643	0.652743	0.357
26.663	0.744865	0.38
28.173	0.750995	0.389
28.905	0.758831	0.392
30.736	0.789159	0.396
31.868	0.801891	0.4
33.143	0.799104	0.409
37.979	0.880216	0.417
38.894	0.880796	0.422
41.417	0.918437	0.426
41.756	0.737133	0.478
45.089	0.793078	0.478

## APPENDIX F.2: Performance Point & Target Displacement Calculations

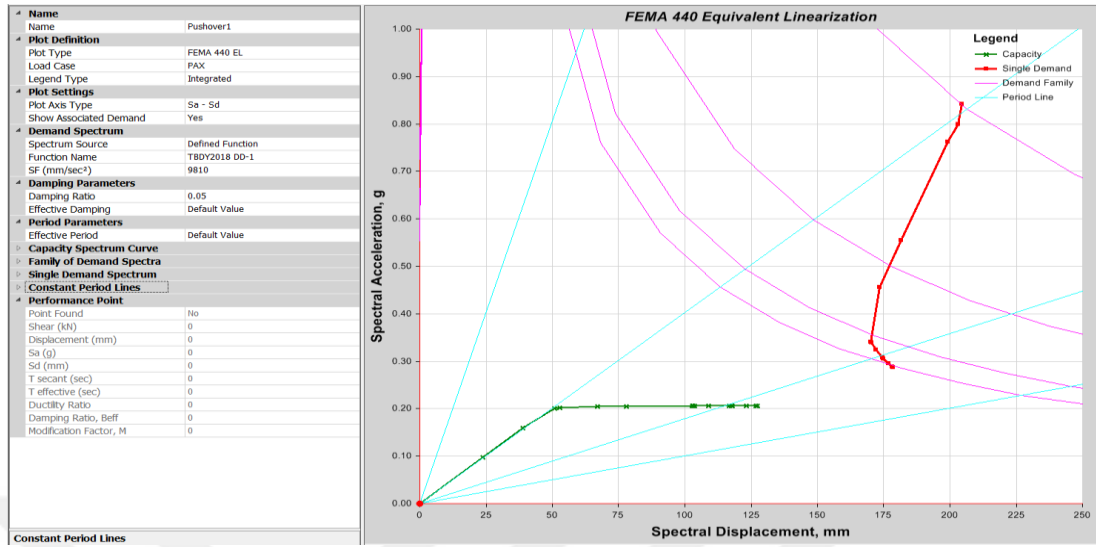


Figure F.1 : Performance Point Calculation for Actual School Building (DD-1)

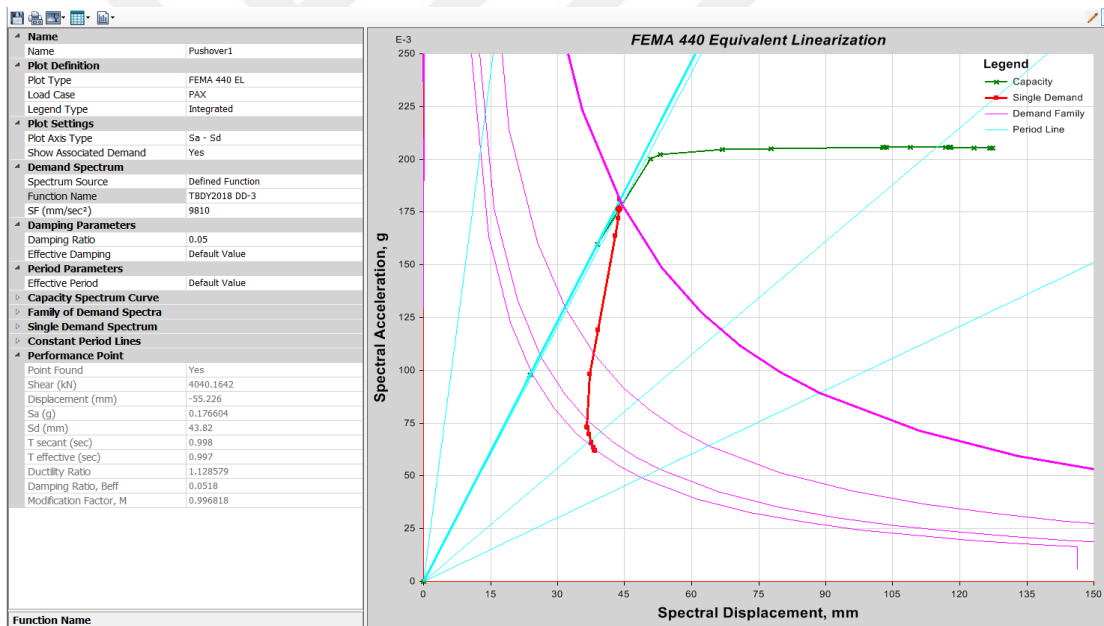


Figure F.2 : Performance Point Calculation for Actual School Building (DD-3)

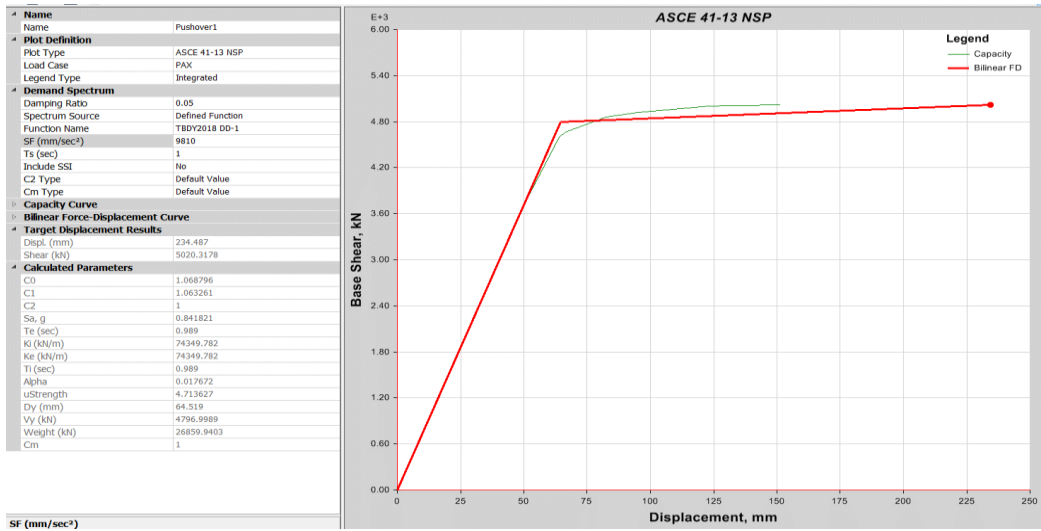


Figure F.3 : Target Displacement Calculation for Actual School Building (DD-1)

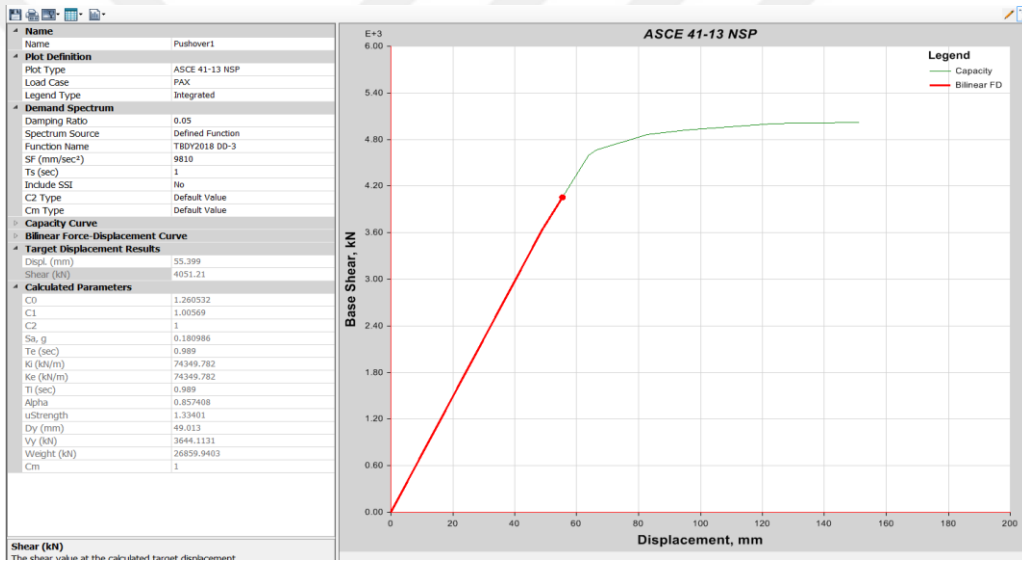


Figure F.4 : Target Displacement Calculation for Actual School Building (DD-3)

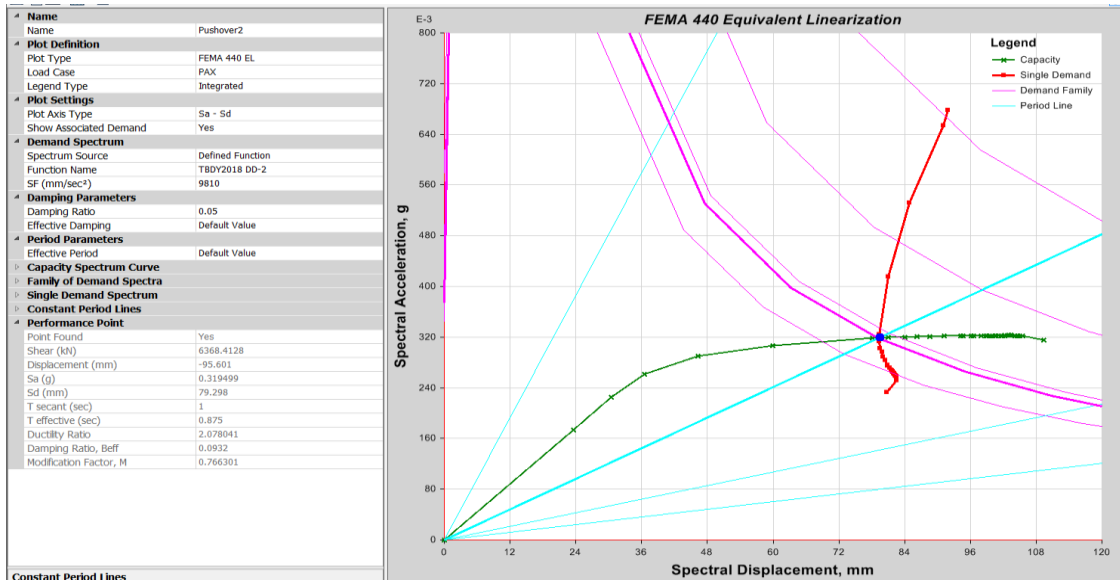


Figure F.5 : Performance Point Calculation for Columns Jacketing for Story1(DD-2)

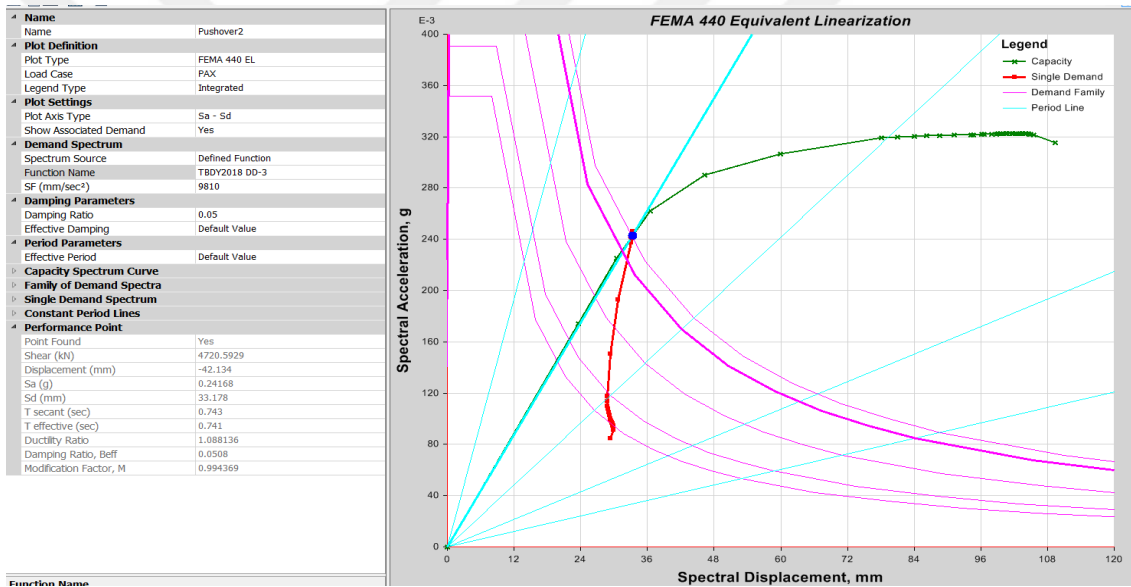
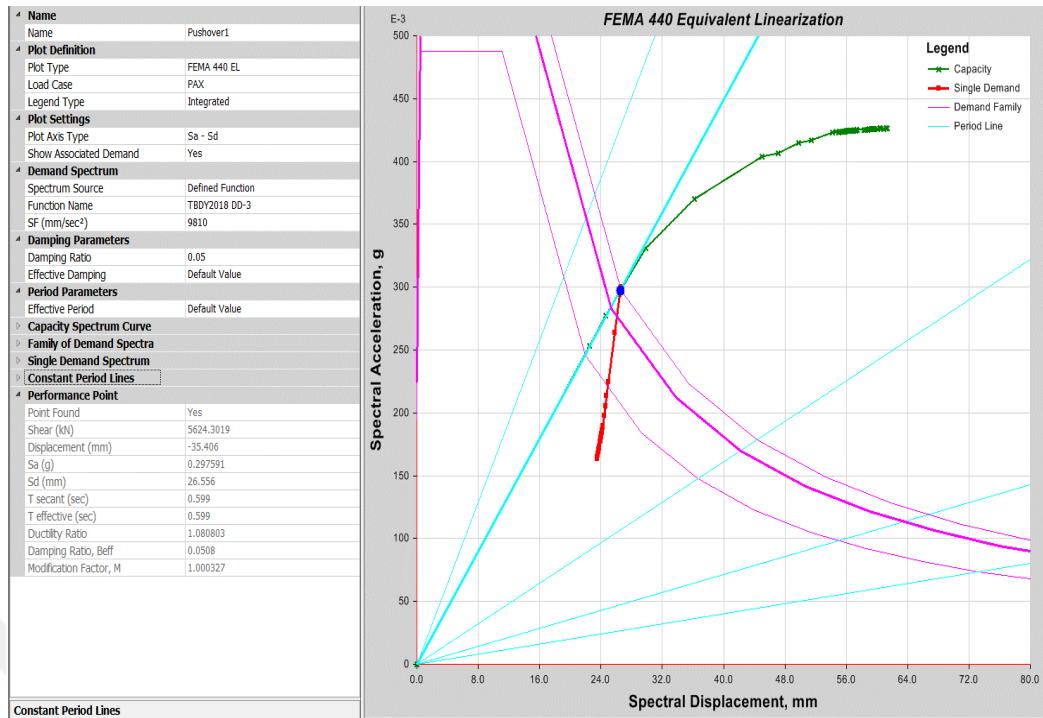
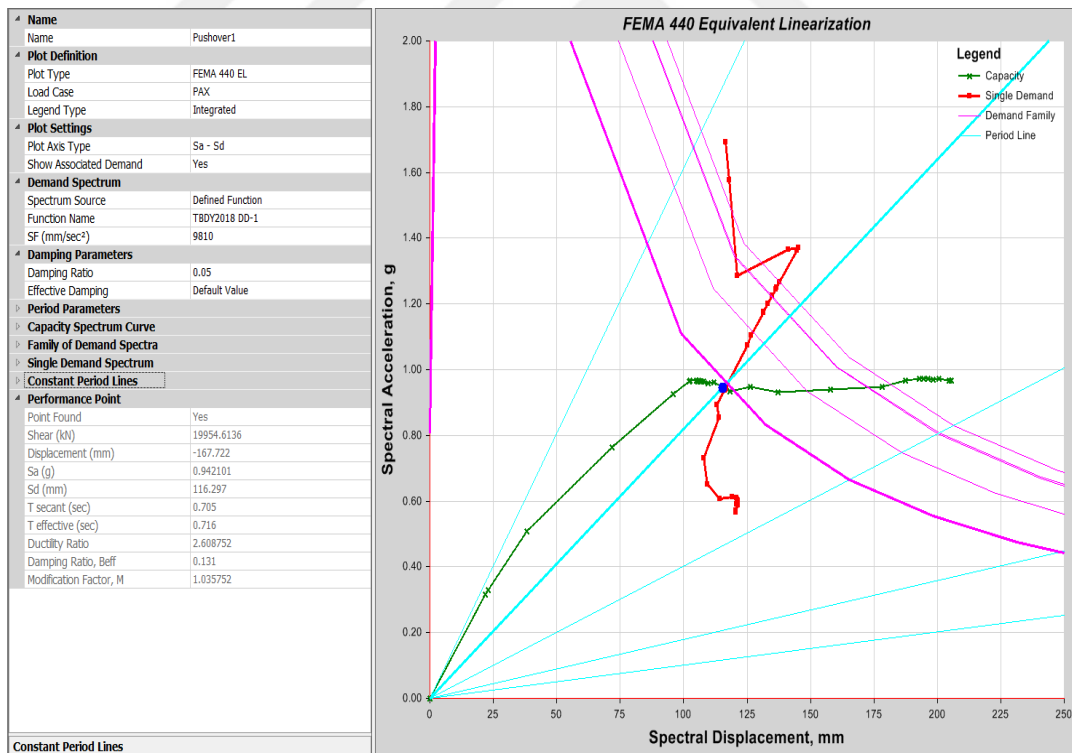


Figure F.6 : Performance Point Calculation for Columns Jacketing for Story1(DD-3)



**Figure F.7 : Performance Point Calculation for Columns Jacketing for Story 1&2 (DD-3)**



**Figure F.8 : Performance Point Calculation for Columns Jacketing for All Stories (DD-1).**

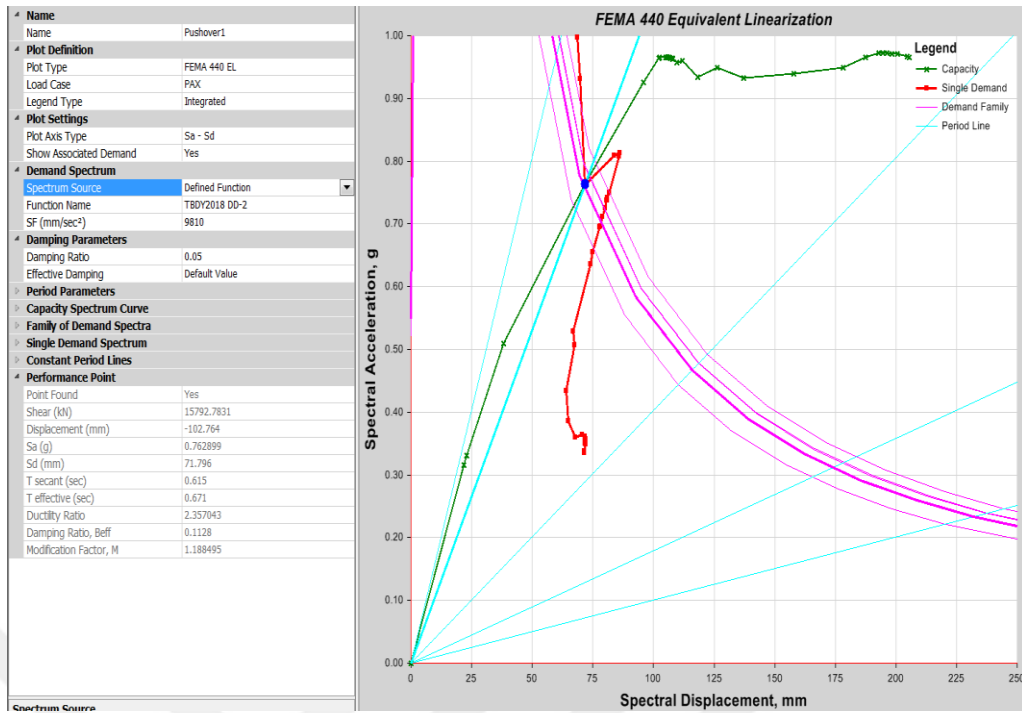


Figure F.9 : Performance Point Calculation for Columns Jacketing for All Stories (DD-2).

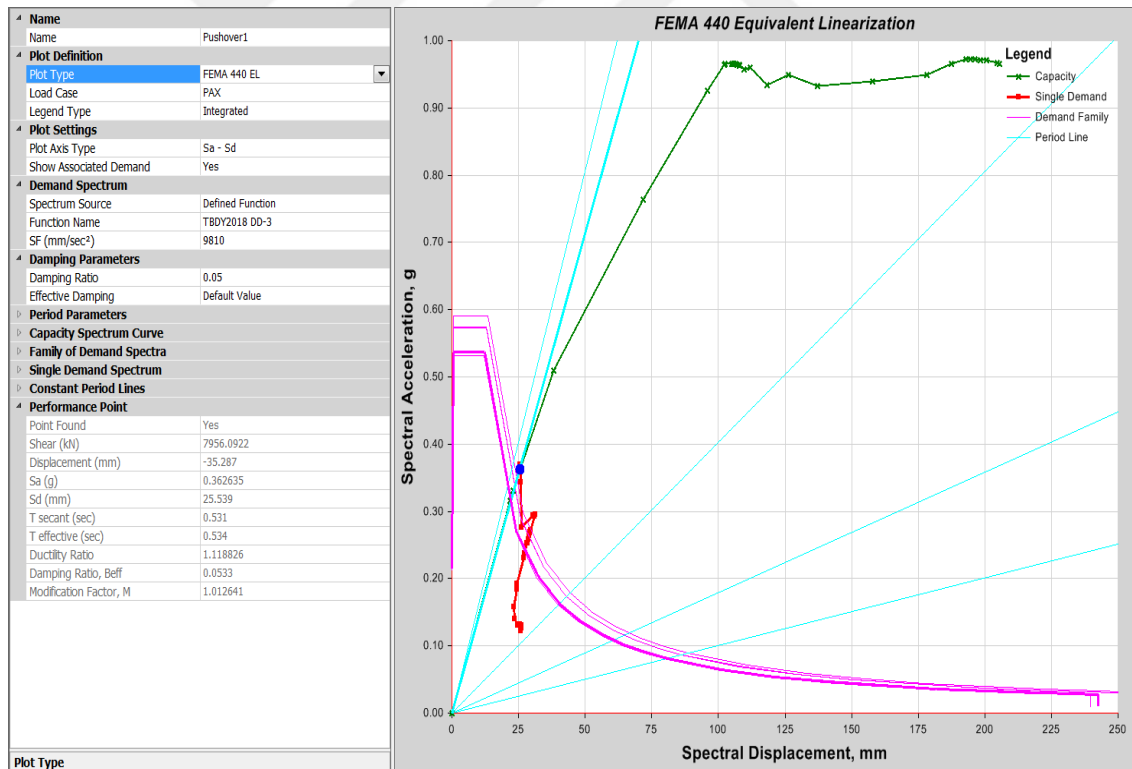
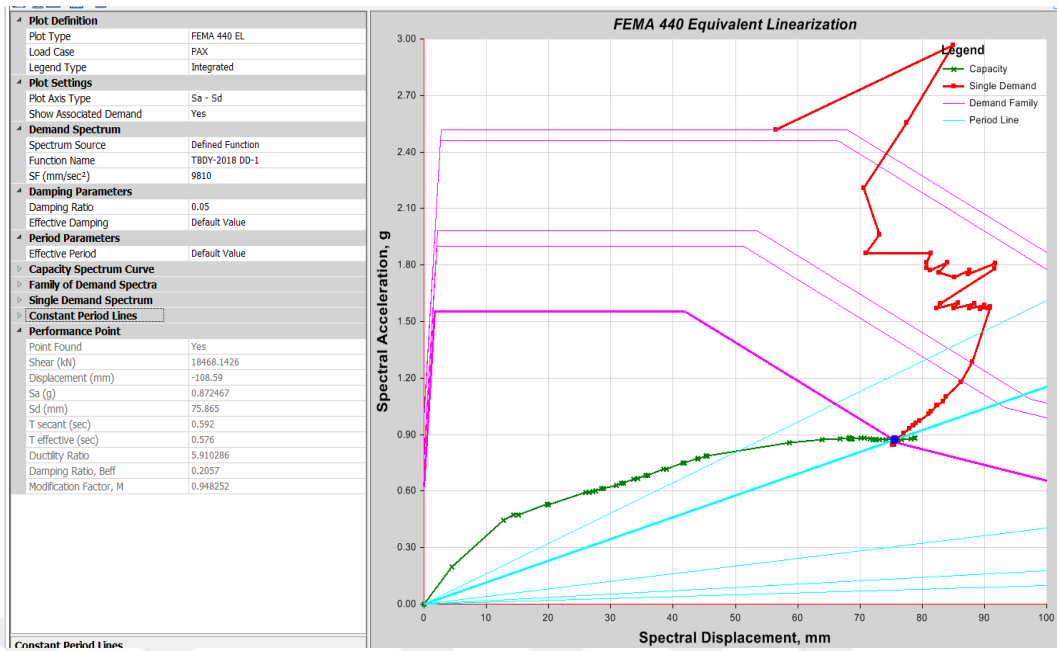
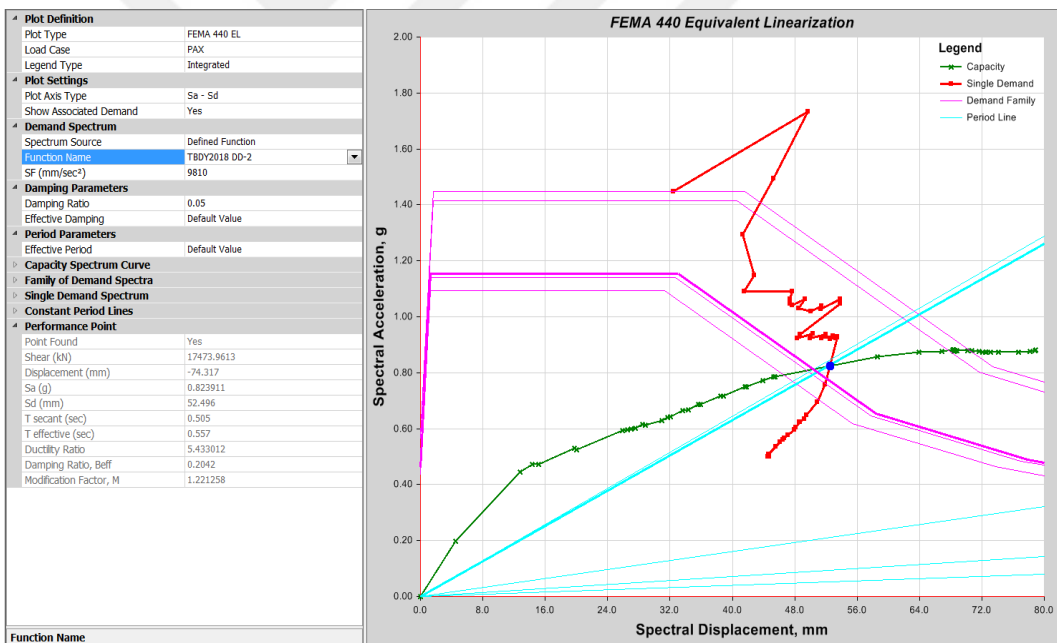


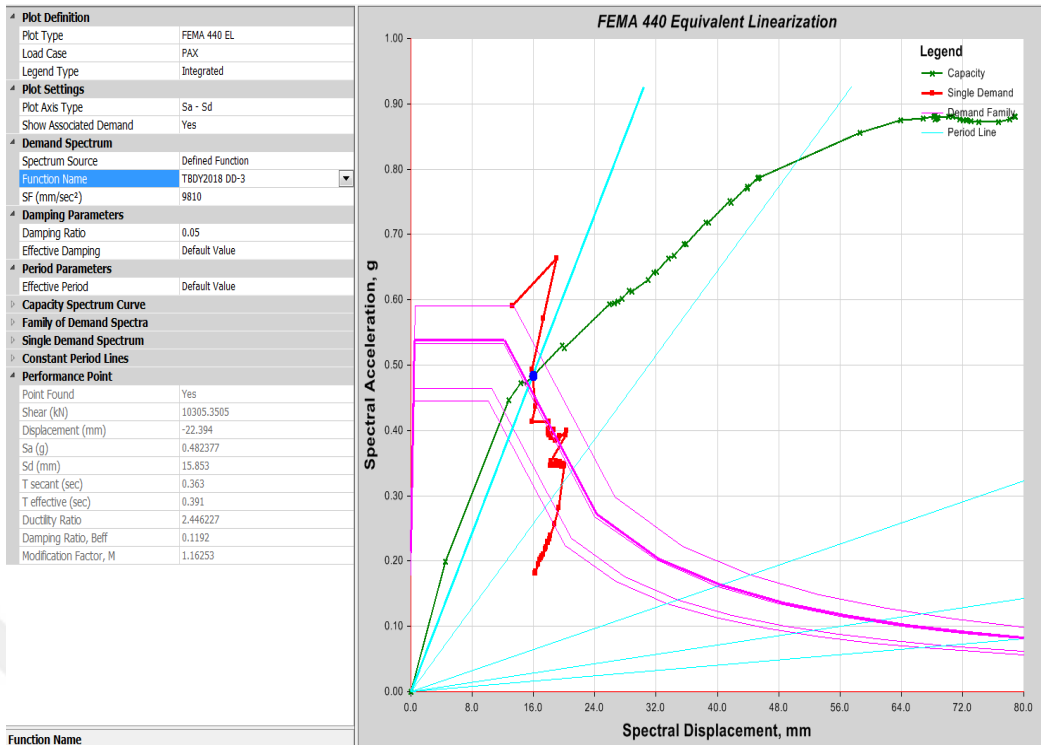
Figure F.10 : Performance Point Calculation for Columns Jacketing for All Stories (DD-3).



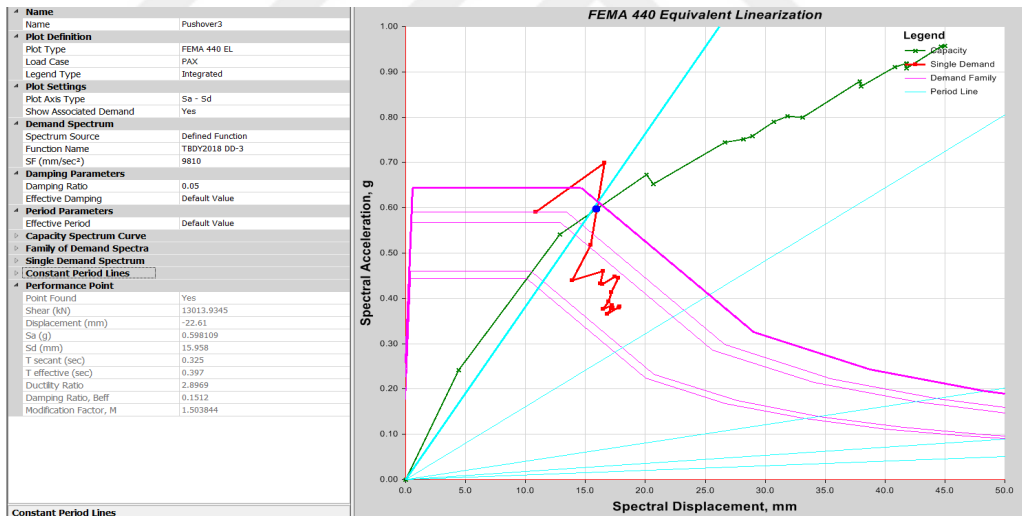
**Figure F.11** : Performance Point Calculation for Shear walls addition at structure corners (DD-1)



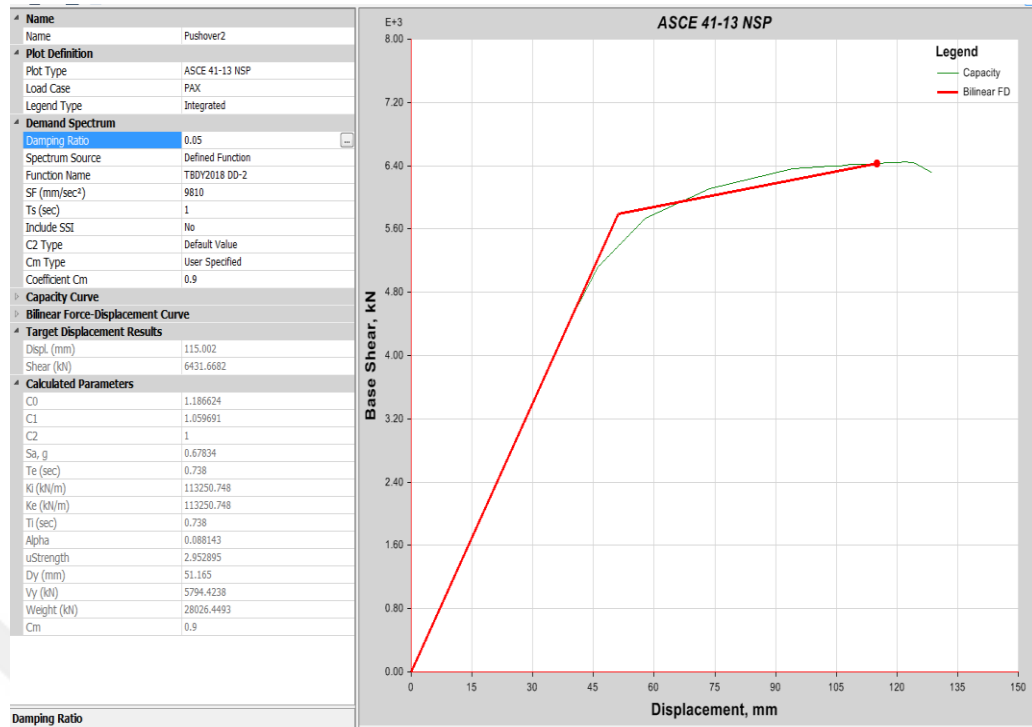
**Figure F.12** : Performance Point Calculation for Shear walls addition at structure corners (DD-2)



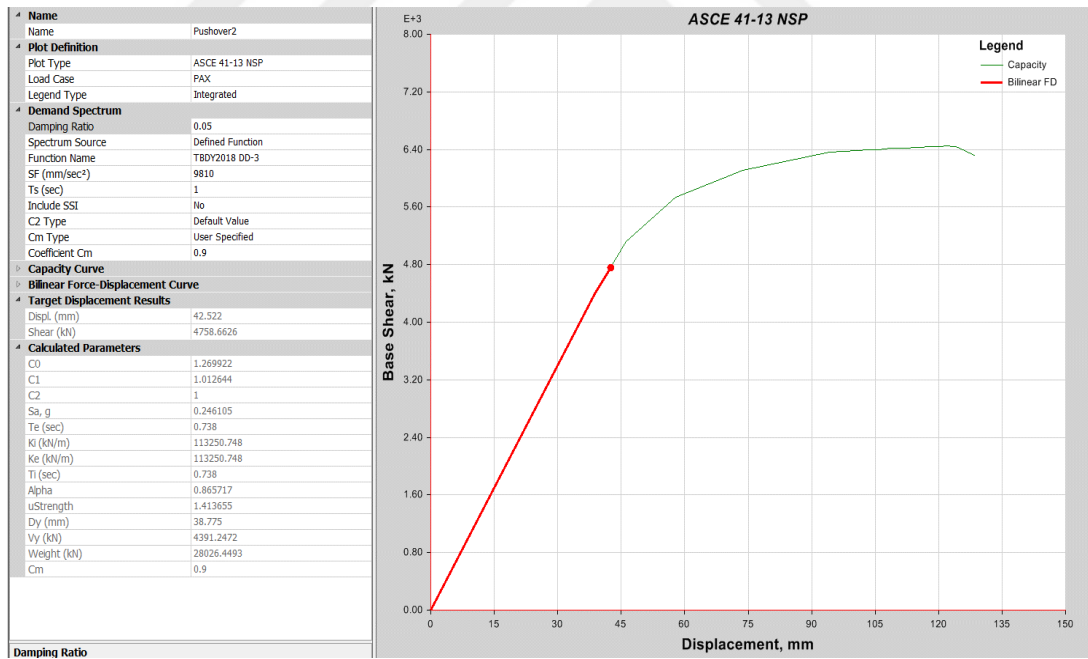
**Figure F.13 ::** Performance Point Calculation for Shear walls addition at structure corners (DD-3)



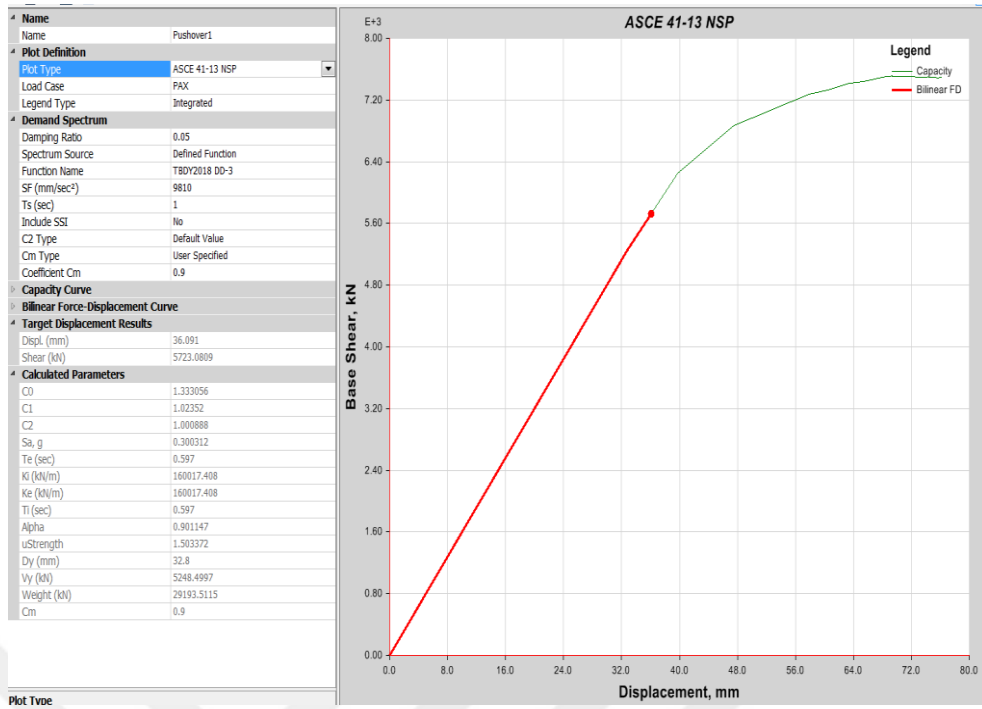
**Figure F.14 :** Performance Point Calculation for Shear walls addition at structure corners and middle edges of Retrofitted School Building (DD-3)



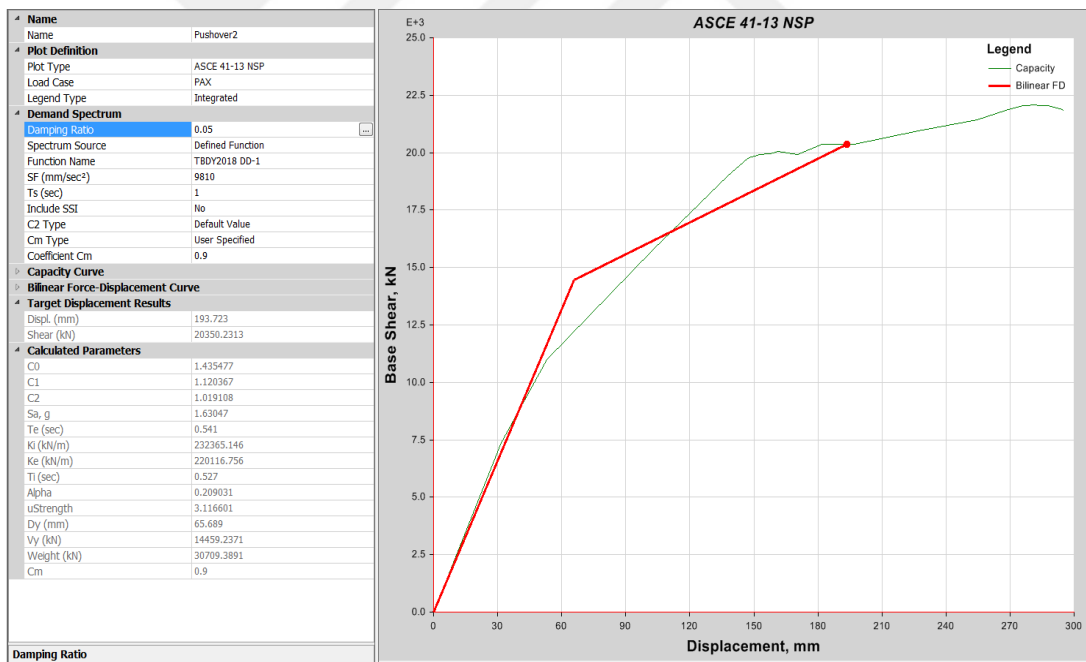
**Figure F.15 : Target Displacement Calculation for Columns Jacketing for Story 1 (DD-2)**



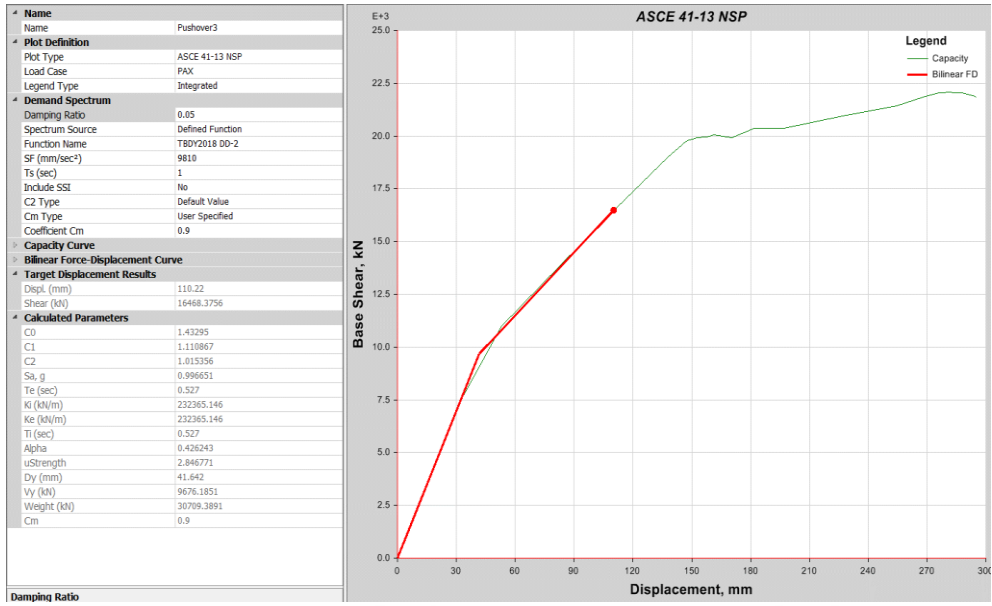
**Figure F.16 : Target Displacement Calculation for Columns Jacketing for Story 1 (DD-3)**



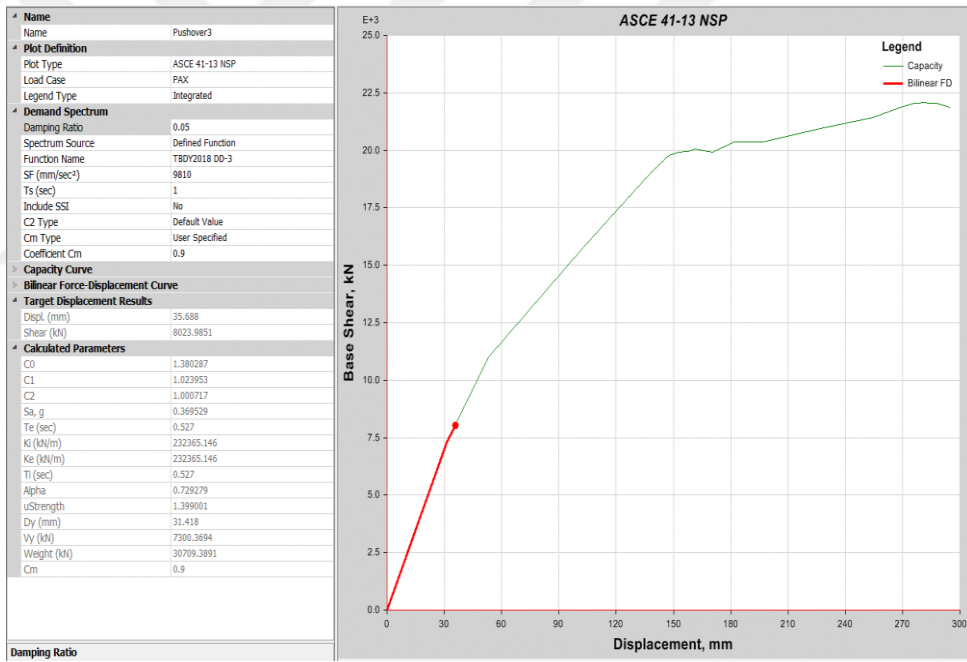
**Figure F.17 : Target Displacement Calculation for Columns Jacketing for Story 1&2 (DD-3)**



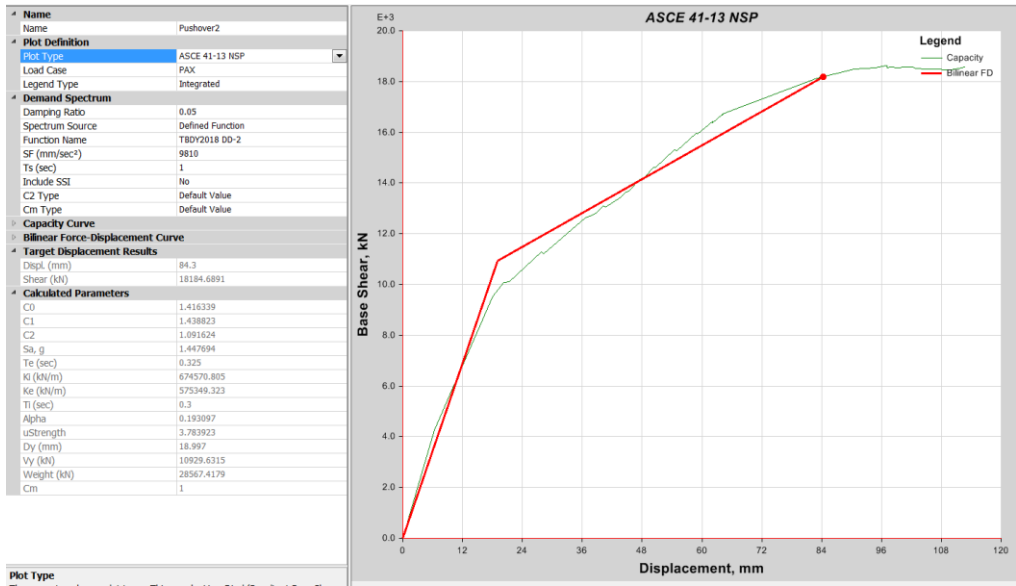
**Figure F.18 : Target Displacement Calculation of Columns Jacketing for All Stories (DD-1)**



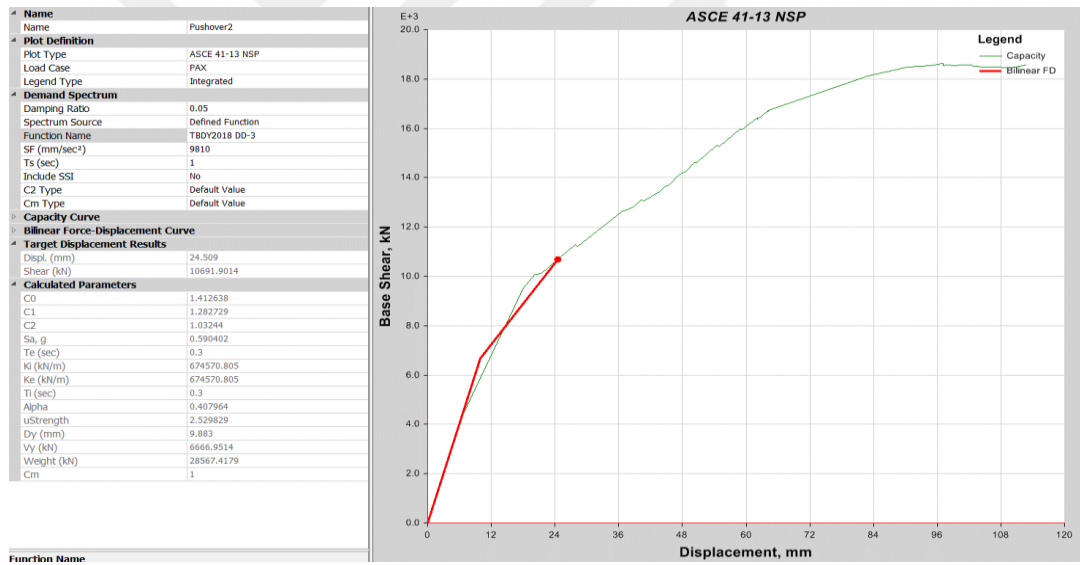
**Figure F.19 : Target Displacement Calculation of Columns Jacketing for All Stories (DD-2)**



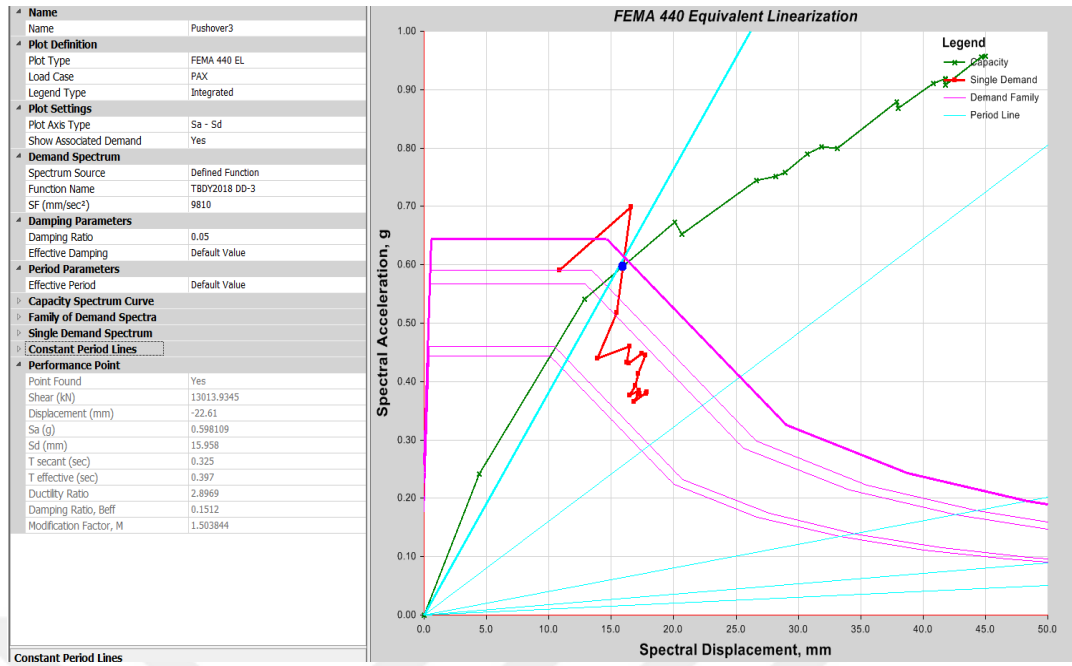
**Figure F.20 : Target Displacement Calculation of Columns Jacketing for All Stories (DD-3)**



**Figure F.21** : Target Displacement Calculation for Shear walls addition at structure corners of Retrofitted School Building (DD-2)

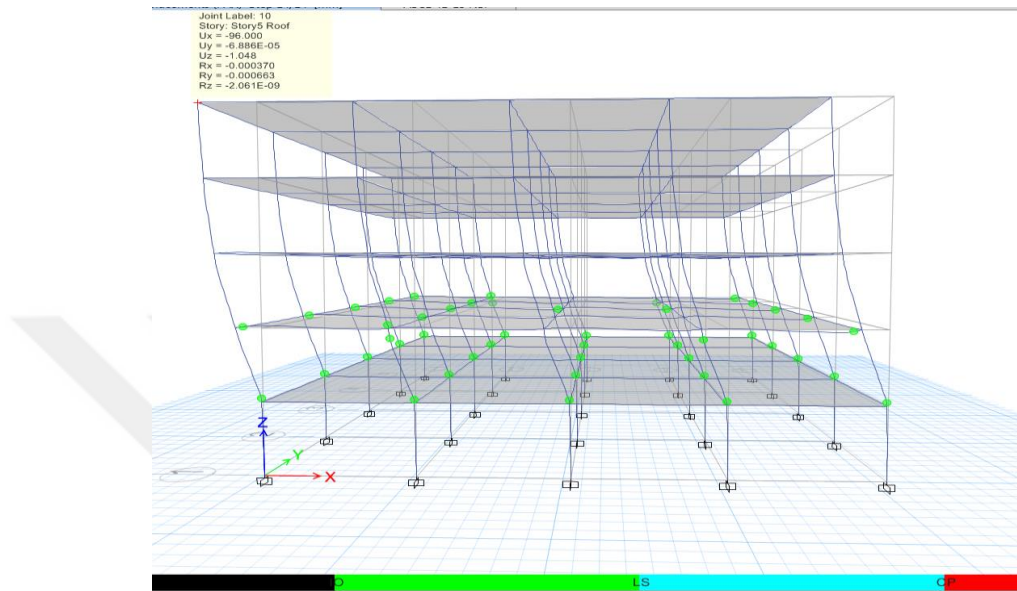


**Figure F.22** : Target Displacement Calculation for Shear walls addition at structure corners of Retrofitted School Building (DD-3)

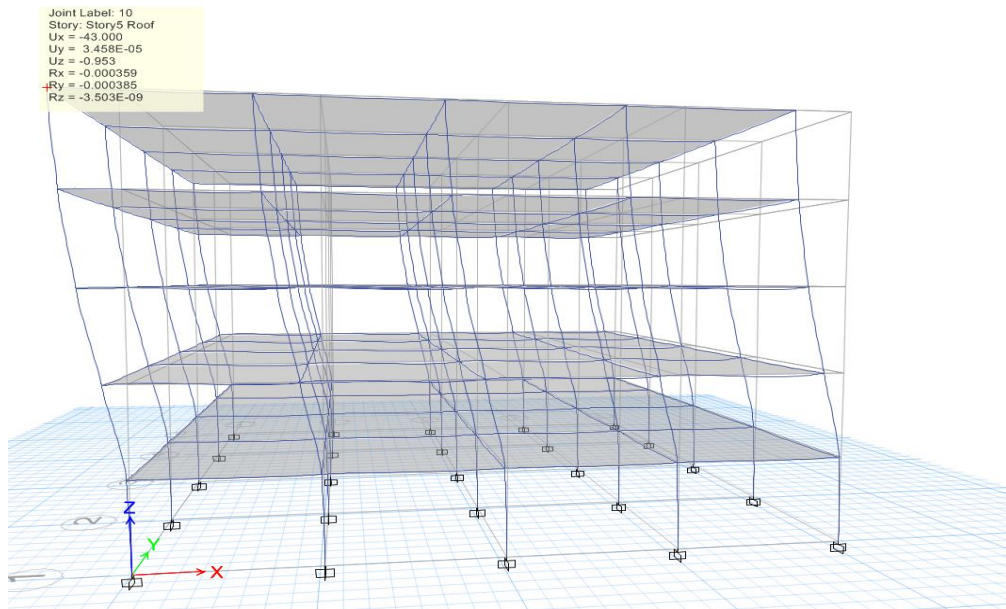


**Figure F.23** : Target Displacement Calculation for Shear walls addition at structure corners and middle edges of Retrofitted School Building (DD-3)

### APPENDIX F.3: Structure State at Performance Point & Target Displacement

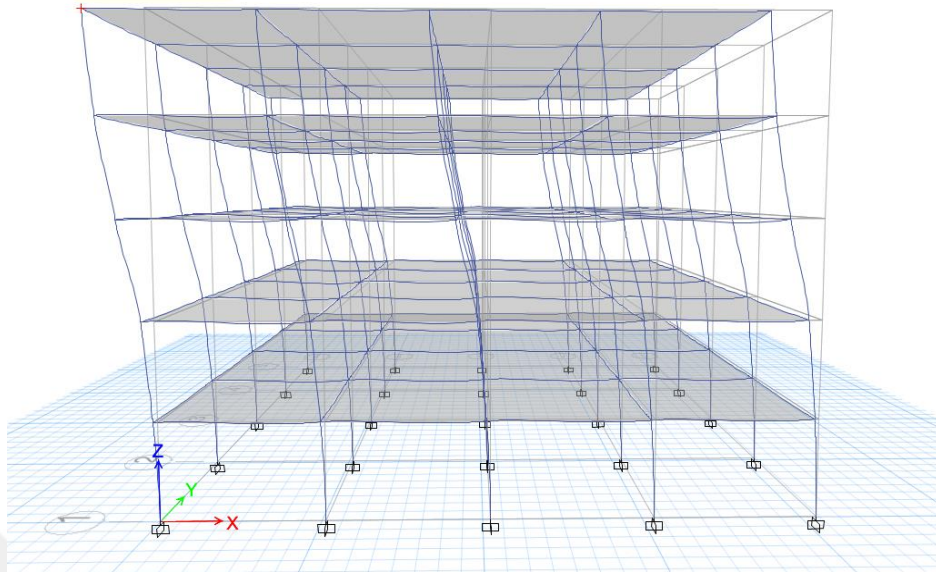


**Figure F.24 :** Structure state at performance point (96 mm) of column jacketing for story 1 retrofitted case (All Hinges at IO-LS Region)



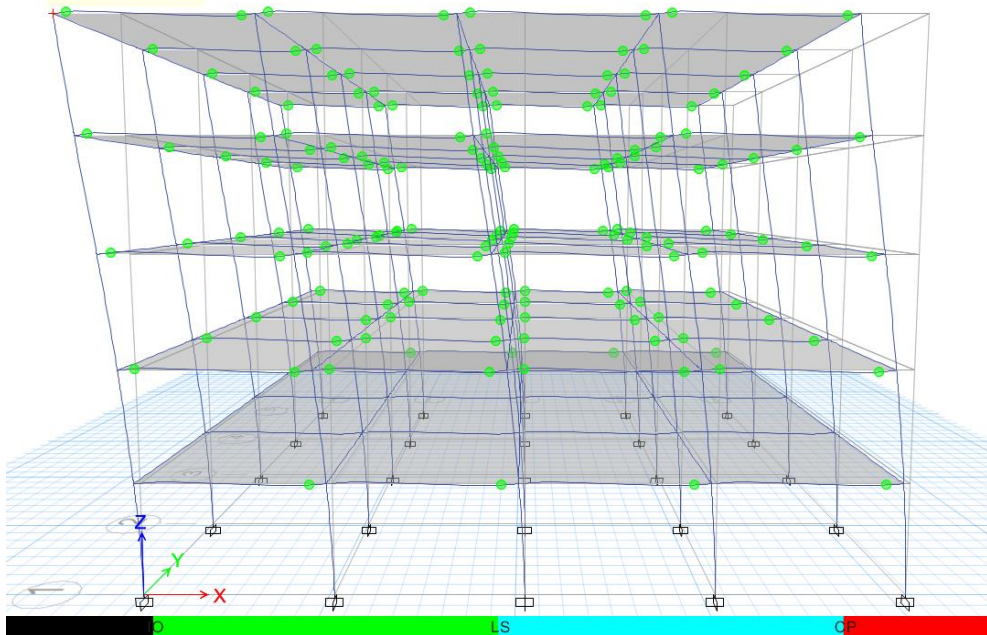
**Figure F.25 :** Structure state at performance point & Target displacement (43mm) of column jacketing for story 1 retrofitted case (No Hinge Formation)

Joint Label: 10  
Story: Story5 Roof  
Ux = -36.000  
Uy = 2.861E-05  
Uz = -0.643  
Rx = -0.000349  
Ry = -0.000479  
Rz = -2.887E-09

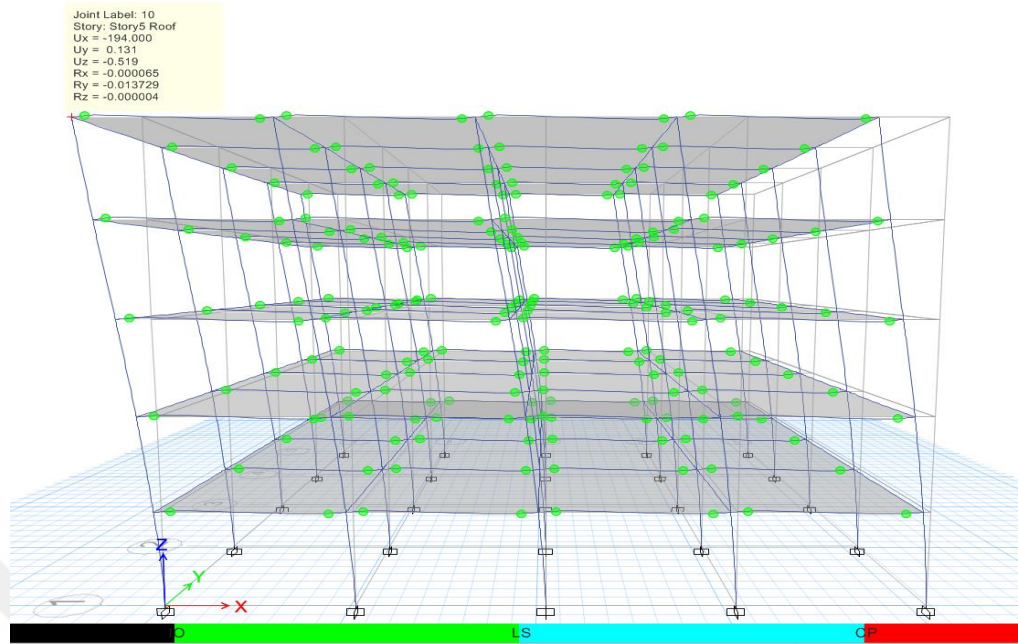


**Figure F.26 :** Structure state at performance point & Target displacement (36mm) of column jacking for story 1 & 2 retrofitted case (No Hinge Formation)

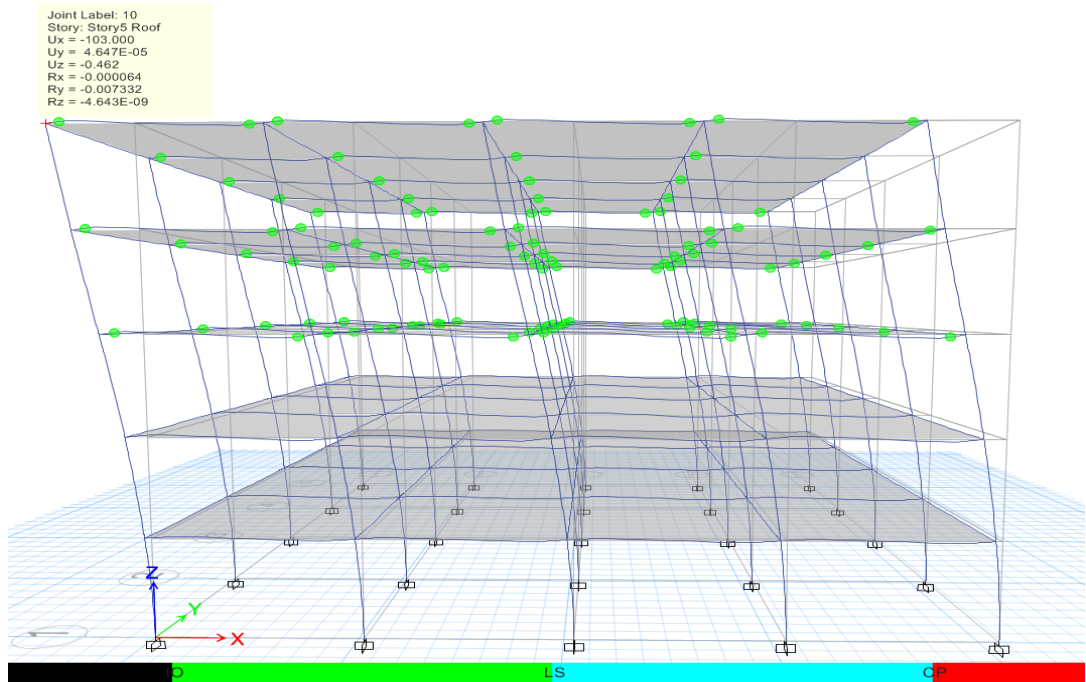
Joint Label: 10  
Story: Story5 Roof  
Ux = -168.000  
Uy = -0.156  
Uz = -0.506  
Rx = -0.000061  
Ry = -0.012373  
Rz = 0.000013



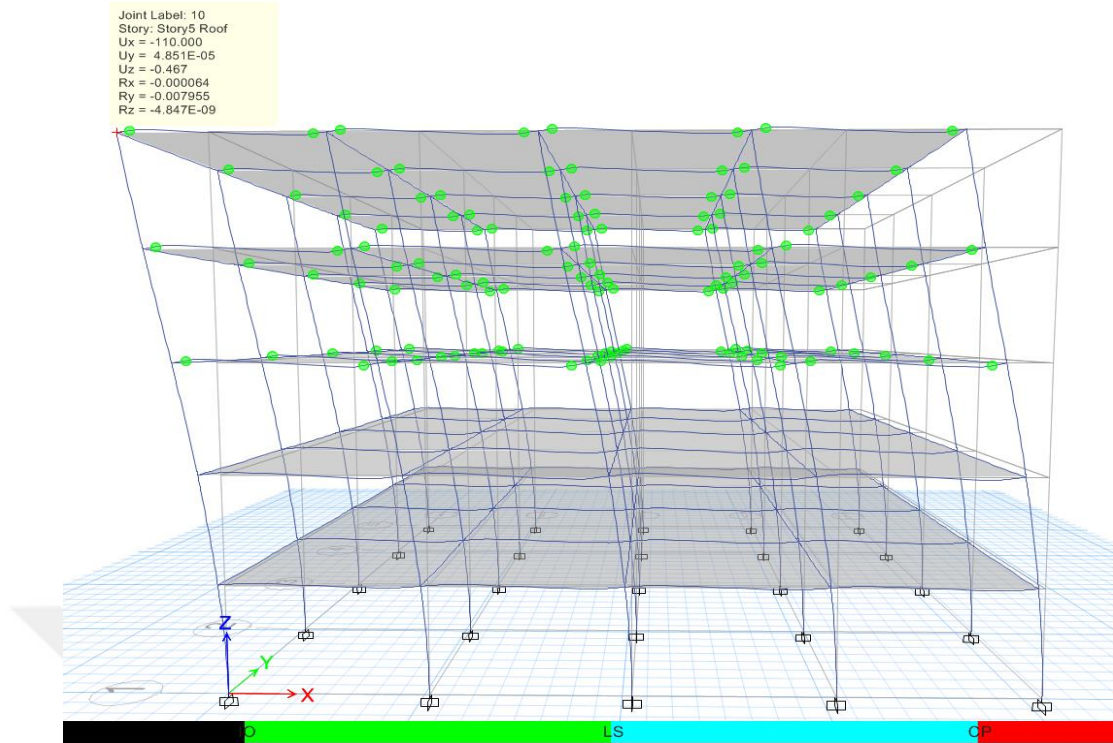
**Figure F.27 :** Structure state at Performance Point (168 mm) of Columns Jacketing for All Stories Retrofitted Case (All Hinges at IO-LS Region)



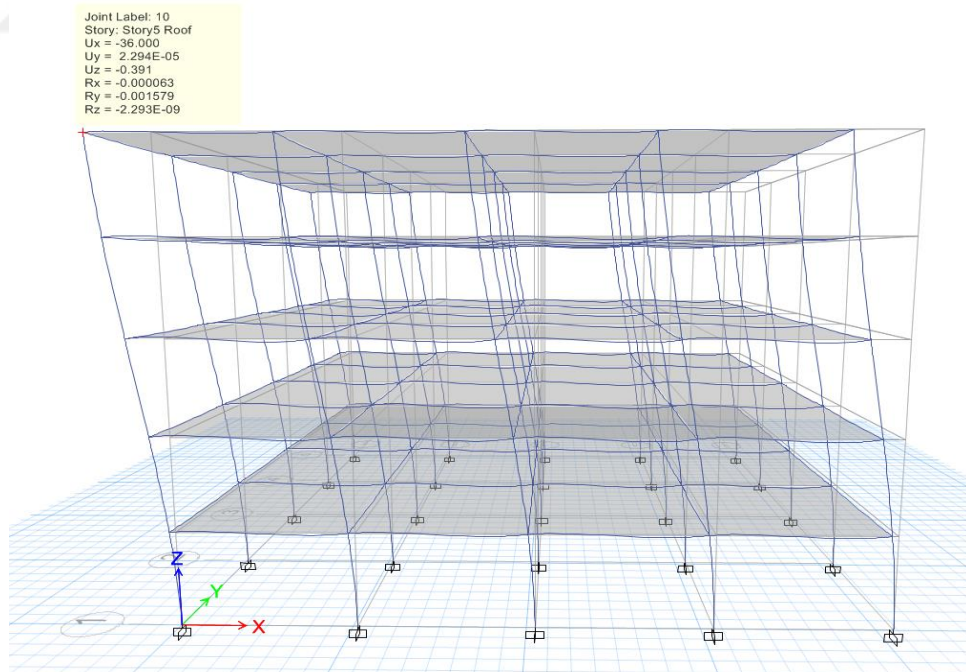
**Figure F.28 :** Structure State at Target Displacement (194mm) of Columns Jacketing for All Stories Retrofitted Case (All Hinges at IO-LS Region)



**Figure F.29 :** Structure State at Performance point (103mm) of Columns Jacketing for All Stories Retrofitted Case (All Hinges at IO-LS Region)

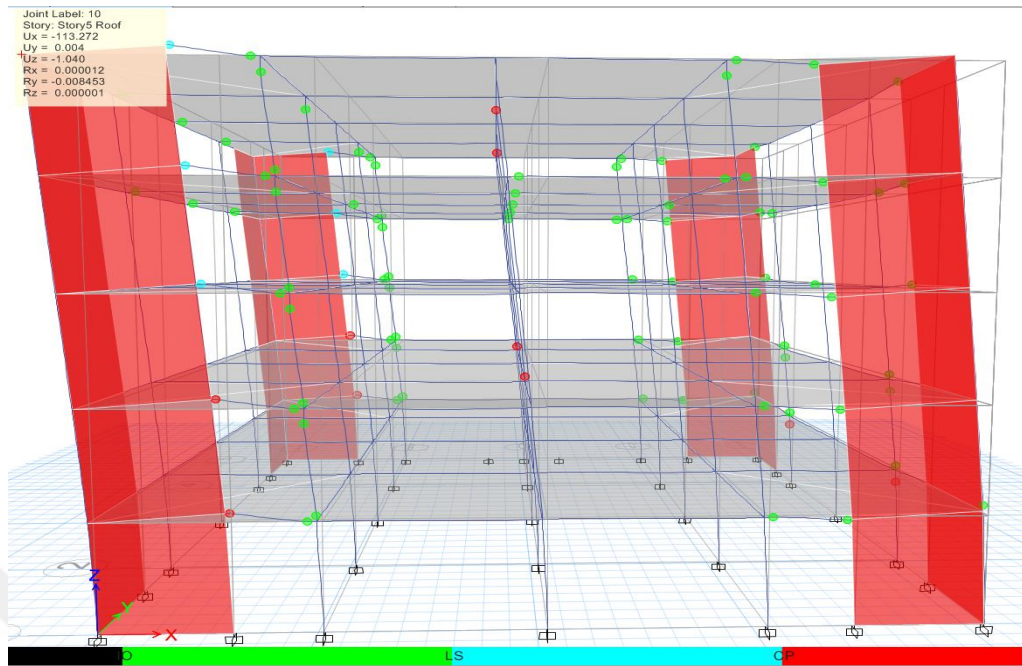


**Figure F.30 :** Structure State at Target Displacement (110 mm) of Columns Jacketing for All Stories Retrofitted Case (All Hinges at IO-LS Region)

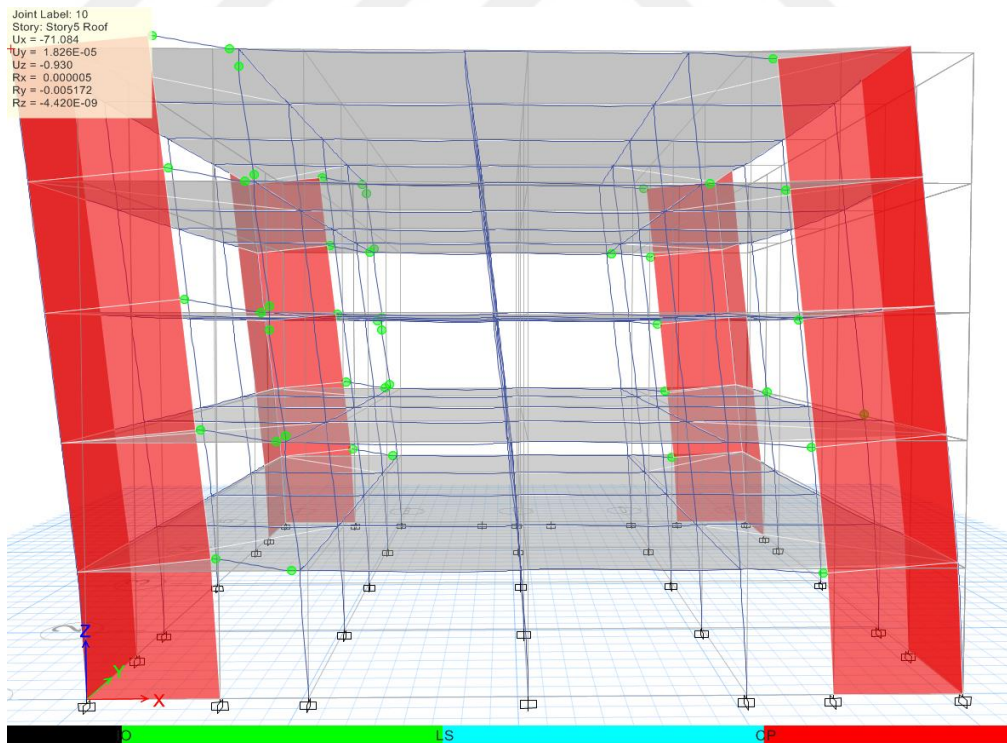


**Figure F.31 :** Structure state at Performance point & Target Displacement (36mm) of Columns Jacketing for All Stories Retrofitted Case (No hinges formation)

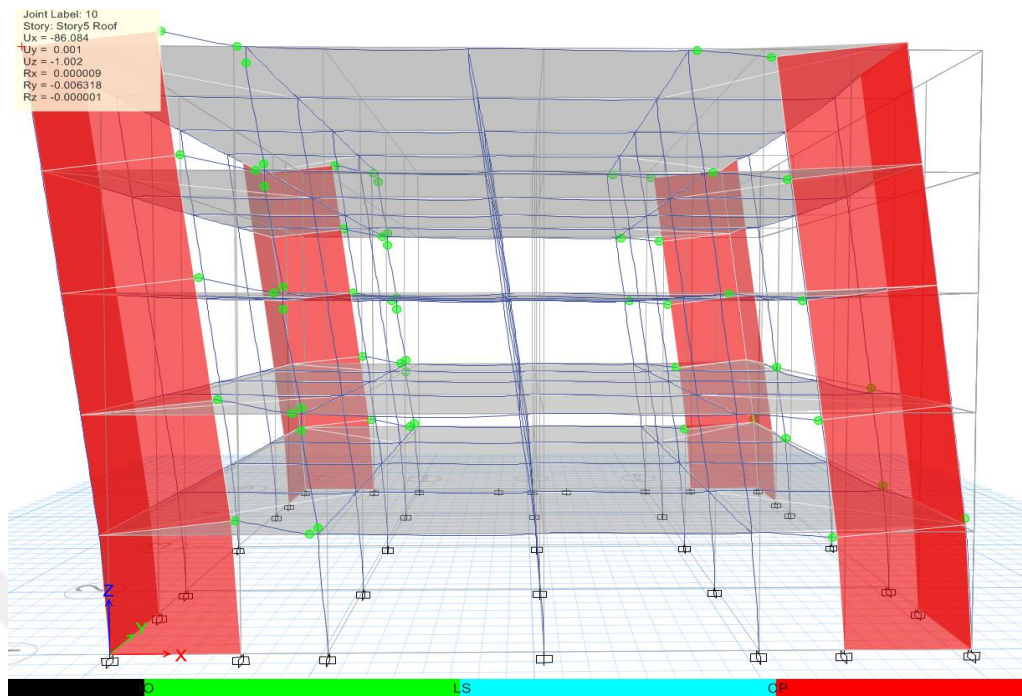
Figure F.32 show 82 hinges at IO-LS region, 6 hinges at LS-CP region and 10 beyond CP limit



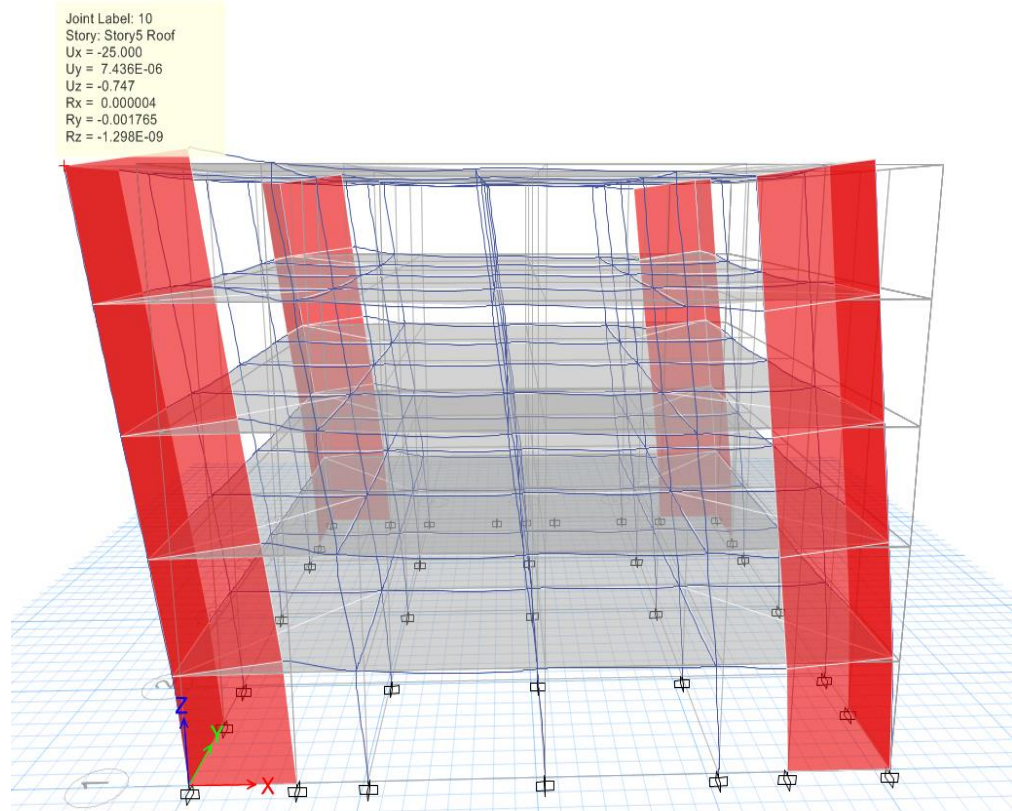
**Figure F.32:** Structure State at Performance point ( $\approx 109\text{mm}$ ) for Shear walls addition at structure corners Retrofitted Case



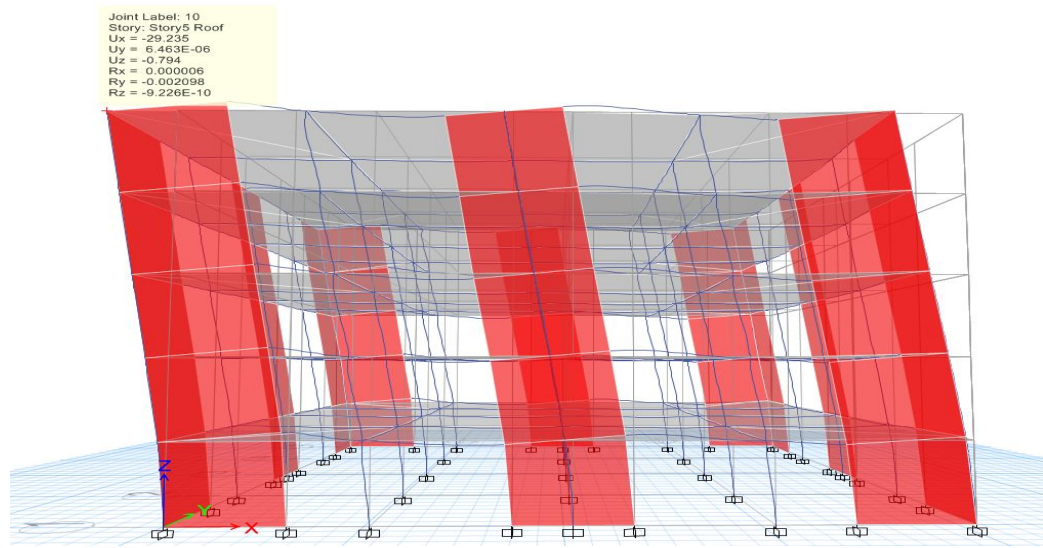
**Figure F.33:** Structure State at Performance point (74mm) for Shear walls addition at structure corners Retrofitted Case (All Hinges at IO-LS Region)



**Figure F.34:** Structure State at Target Displacement ( $\approx 84\text{mm}$ ) for Shear walls addition at structure corners Retrofitted Case (All Hinges at IO-LS Region)



**Figure F.35:** Structure State at Performance point & Target Displacement (25mm) for Shear walls addition at structure corners Retrofitted Case



**Figure F.36:** Structure State at Performance point & Target Displacement (23mm) for Shear walls addition at structure corners and middle edges Retrofitted Case (No hinges formation)

**APPENDIX F.4: Inner story Drift Ratio of Column Jacketing for All Stories  
Retrofitted Case**

**Table F.13 :** Inner story Drift Ratio of Column Jacketing for All Stories Retrofitted Case

Story No.	Calculated Drift Case				
	DD-1 demand curve		DD-2 demand curve		DD-3 demand curve
	At Performance Point	At Target Displacement	At Performance Point	At Target Displacement	At Performance point & Target Displacement
Story 5 roof	0.013129	0.014533	0.007951	0.008588	0.002008
Story 4	0.014395	0.015975	0.009054	0.009694	0.002871
Story 3	0.013308	0.015094	0.008447	0.008984	0.00322
Story 2	0.009992	0.011905	0.006126	0.006491	0.002641
Story 1	0.005266	0.007159	0.002755	0.00291	0.00126