

T.R.
GEBZE TECHNICAL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

SEISMIC DESIGN OPTIMIZATION AND
SEISMIC PERFORMANCE OF TALL BUILDINGS AND
SEISMIC PERFORMANCE

ABDULHAMİT EROĞLU
A THESIS SUBMITTED FOR THE DEGREE OF
MASTER OF SCIENCE
DEPARTMENT OF CIVIL ENGINEERING
EARTHQUAKE AND STRUCTURAL ENGINEERING PROGRAM

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GEBZE

2017

T.C.
GEBZE TEKNİK ÜNİVERSİTESİ
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OPTİMİZASYONU VE DEPREM
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ABDULHAMİT EROĞLU
YÜKSEK LİSANS TEZİ
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GEBZE

2017



YÜKSEK LİSANS JÜRİ ONAY FORMU

GTÜ Fen Bilimleri Enstitüsü Yönetim Kurulu'nun 29/06/2017 tarih ve 2017/35 sayılı kararıyla oluşturulan jüri tarafından 28/07/2017 tarihinde tez savunma sınavı yapılan Abdulhamit Eroğlu'nun tez çalışması İnşaat Mühendisliği Anabilim Dalı Deprem ve Yapı Mühendisliği Programında YÜKSEK LİSANS tezi olarak kabul edilmiştir.

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SUMMARY

Due to increasing urbanization, high-rise buildings stock is expanding in Turkey. Traditionally, these buildings are designed with core shear walls and frames as structural system in our country. Any changes in design will affect the seismic performance while defining the structural system of high-rise buildings. In this respect, the characteristics of the structural system are important.

In this study, it was examined how to design and evaluate the structural system in order to improve the seismic performance of the high-rise buildings or to get the best. Eventhough currently, there is no official Code or regulation document on the design of high-rise buildings in the cities with high seismic risk, the draft of "Turkish Building Seismic Code" in June 2016 includes rules for the design of high-rise building. The first system was a typical core-shear wall with frames while the second system was typical system enhanced with Buckling-Retrained Braced Frame.

In this study, two reinforced concrete high-rise buildings with different structural systems were designed based on Turkish Building Seismic Code Draft 2016 to evaluate seismic performance.

In addition to that, nonlinear time history analysis was performed to evaluate and seismic performance was investigated.

Key Words: High-Rise Building Design, Code-Based Design, Nonlinear Time History Analysis, Performance Based Design.

ÖZET

Günümüzde artan ihtiyaçlar doğrultusunda Türkiye’de yüksek yapı stoku giderek artmaktadır. Geleneksel olarak ülkemizdeki tüm yüksek binaların taşıyıcı sistemi betonarme çekirdek perde ve çerçevelerden oluşturulmaktadır. Yüksek binaların taşıyıcı sistemini oluştururken tasarımdaki herhangi bir değişiklik deprem performansını etkileyecektir. Bu açıdan taşıyıcı sistemin özellikleri önemi arz etmektedir.

Bu çalışma kapsamında yüksek yapıların deprem davranışı ve performanslarının iyileştirilmesi veya en iyi sonuçları alabilmek için taşıyıcı sistemi en doğru şekilde tasarlanması ve değerlendirilmesi irdelenmiştir. Ayrıca, deprem riskinin yüksek olduğu büyük şehirlerde yüksek binaların tasarımı için resmi bir yönetmelik bulunmamaktadır ancak Haziran 2016 yılında taslak olarak yayınlanan «Türkiye Bina Deprem Yönetmeliği» yüksek yapı tasarımı için kuralları içermektedir.

Bu tez çalışmasında, taşıyıcı sistemleri farklı iki betonarme yüksek bir bina, TBDY (2016)’da önerilen tasarım ilkelerine göre analiz edilerek taşıyıcı sistem belirlenecek, deprem performansı değerlendirilmiştir. Taşıyıcı sistemleri farklı olan iki betonarme yüksek binadan birincisi çekirdek-perdeli çerçeveli, ikincisi ise Burkulması Önlenmiş Çaprazlı sistemdir.

Bu çalışmada, zaman tanım alanında doğrusal olmayan analiz yöntemi kullanılarak yüksek yapıların deprem performanslarının değerlendirilmesi konusunda çeşitli tasarım parametreleri incelenerek oldukça faydalı bilgiler sağlanmıştır.

Anahtar Kelimeler: Yüksek Yapı Tasarımı, Doğrusal Tasarım, Zaman Tanım Alanında Doğrusal Olmayan Analiz, Performansa Dayalı Tasarım.

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LIST of ABBREVIATIONS and ACRONYMS

<u>Abbreviations and Acronyms</u>	<u>Explanations</u>
$E_d^{(H)}$	: Horizontal Design Spectrum
G	: Dead Load
g	: Acceleration of gravity [m/s^2]
H_N	: Total height above basement floors
I	: Building importance factor
m_t	: Total mass above basement floors
n	: Live load participation factor
R	: Coefficient of structural behaviour
Q	: Live load
Q_e	: Effective Live load
S_{DS}	: Spectral acceleration coefficient in short period domain
$V_{tE}^{(X)}$	: Base shear force for X component [kN]
α	: Amprical coefficient for space of seismic joint
η_{bi}	: Torsional irregularity coeeficient for story i
γ_E	: Coefficient of amplification for base shear force
CBH	: Class of Building Height
CSD	: Class of Seismic Design
D	: Overstrength Factor
	Earthquake Level with 2% probability of being exceeded in 50
DD-1	: years (2475-year return period)
	Earthquake Level with 10% probability of being exceeded in 50
DD-2	: years (475-year return period)
	Earthquake Level with 50% probability of being exceeded in 50
DD-3	: years (72-year return period)
	Earthquake Level with 68% probability of being exceeded in 50
DD-4	: years (43-year return period)

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1. INTRODUCTION

The high-rise building of 'Park Row Building' is considered to be the first example which was built in New York in 1899 at height of 119 meters. The main purpose of the construction was to make people be taken advantages of sunlights to increase its usage area. This building was also constructed with using steel braces-frame structural sytem as the most high-rise buildings that were built in the late 19th century and early 20th century. 'Empire State Building' was built in 1931 with height of 381 meters. The building was constructed with very poorly known, about behavior of building materials and lack of analysis methods and equipments; but in this building like other buildings built in same periods was evidenced by the evolving technology of using excess materials. At that time, the living population and the cities increased and started to grow gradually. However, land prices were inevitable to rise so that economic growth and resulting demand for office space is a good indication of demand for high-rise buildings.

On the other hand, depending on the use of developing of technology and high-strenght materials, it was started to use innovator structural system and using analysis methods other than older and weak methods. In Figure 1.1, geographical distribution of high-rise buildings are shown for countries.

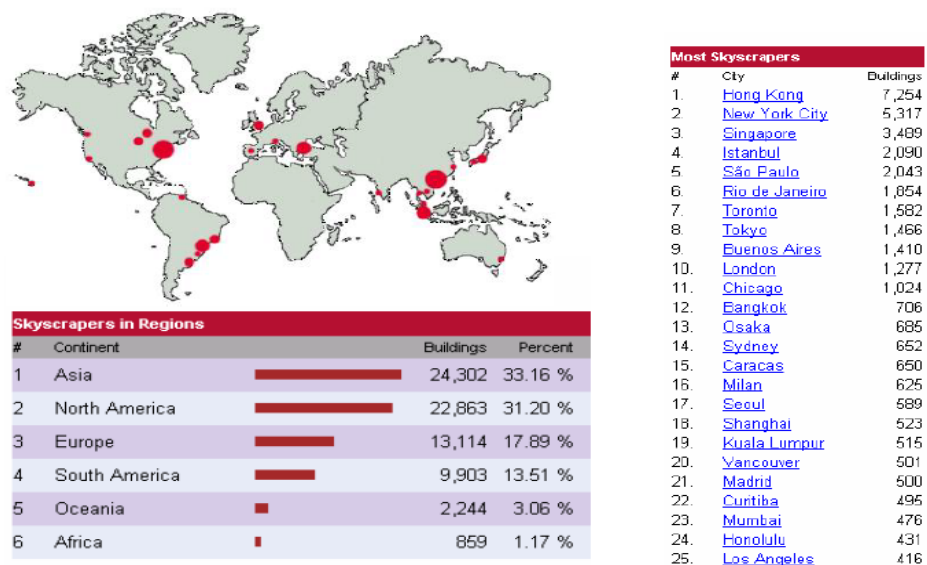


Figure 1.1: Geographical distribution of high-rise buildings.

However in tall buildings, using with the new developing technology and high-strength materials, new methods and elements are started to be used in tall buildings to increase resistant horizontal loads such as Buckling-Restrained Braces. Buckling-Restrained Braced Frames (BRBFs) are a relatively new type of concentrically braced system characterized by the use of braces that yield inelastically both in tension and compression at their adjusted strengths (Clark et al., 1999). The most basic properties of the BRBs are that they have a symmetrical hysteresis in tension and compression due to the restraining of buckling, and also have high energy capacity with exhibiting ductile behavior. The provision of ductile behavior in the buildings is considered to be an important criterion in the seismic design of the structure.

The buckling of the braces under horizontal loads, such as earthquake loads, has led engineers to develop structural systems that is not buckled under pressure and have smooth hysteretic behavior.

The first application of BRB in Japan was made in 1987. In the United States, it began to be used from the beginning of the 2000s, and the interest in BRB since then has increased rapidly. Many experimental and analytical studies on this subject have been carried out.

In Turkey, Istanbul Technical University Technopark buildings, BRBs has been used. On the other hand, Çelik and Bruneau (2009) worked analytically on the BRBs of the bridges. Karataş and Çelik (2009) also studied the design and rules of BRBs.

After all, in this study the design and seismic performance of tall buildings will be investigated depending on new Turkish Building Seismic Code [TBSC, Draft 2016].

When all of these are considered together, modern high-rise structures have begun to take place in our lives where were designed using innovator structural systems by engineers and architects.

If we have to define “what’s a tall building?”, the first answer can be related to the height so it is defined by [The Council of Tall Buildings and Urban Habitat] that ‘*A building whose height creates different conditions in the design, construction, and use than those that exist in common buildings of a certain region and period.*’. But it is obviously that high-rise buildings are defined depending on codes. For example,

according to LATBSDS (2015), AB-083, PEER-TBI (2010), high-rise building must be higher than 50 meters. Istanbul Seismic Design Code for Tall Buildings [ISDCTB, 2008] defines high-rise building which must be higher than 60 meters and according to Turkish Building Seismic Code [TBSC, Draft 2016] it is defined to be buildings that are higher than 70 meters.

1.1.Motivation

In this study, two models of tall buildings were designed and with different structural systems for a building site in Istanbul at Latitude 41.0724, Longitude 29.0112 on a local site class ZC. The chosen tall building was designed with height of 175 meters according to specified by the code on Turkish Building Seismic Code [TBSC, Draft 2016].

In this study, the most appropriate and advantageous design of the structural system was examined in order to improve the seismic performance to be within the performance levels specified by the code, therefore two building structural systems types were investigated,

- i) Reinforced concrete tall building with core shear wall and frame structural system.
- ii) Reinforced concrete tall building with buckling-restrained braced frame structural system.

The first structural system is one of the most common building types all over the world. Second structural system is less common building types and still developing.

The design steps of the two models of tall buildings are given below:

- i) Modeling of high-rise building
- ii) Defining nonlinear materials properties for structures
- iii) Choosing scaled ground motion datas according to site seismic hazard
- iv) Obtaining seismic performance by analysing nonlinear time history with the selected and scaled ground motions
- v) Comparison between structural systems

As it is stated above, the first step is prepared by ETABS (2000) software. The second step is prepared by SAP 2000 by [CSI, 2000] that exported to PERFORM 3D [CSI, 2008] for nonlinear dynamic analysis.

This study will provide useful information on the seismic performance of high-rise structures using nonlinear time history analysis. Also, it will provide design guidance for engineers who lives in Turkey or would like to use new Turkish Seismic Code.

1.2.Scope

This thesis includes the design and procedure various tasks of analysis of the tall buildings which includes rules and specifications, design and analysis of the buildings based on new Turkish Building Seismic Code which are typical high-rise building and using BRB (buckling-restrained brace) in high-rise building, results and discussion. The report is prepared as follows:

Chapter 2 presents the a new Turkish Building Seismic Code specifications for the design, together with the other International Codes.

Chapter 3 and 4 present design and performance information for selected structural system of tall building that includes design of structural system, preparing analytical modeling and summary of response results.

Chapter 5 presents results discussions and comparison for the structural systems of two different tall buildings to understand which structural system is more advantageous for seismic behaviour.

Chapter 6 presents assesments and discussion for the performance levels of two high-rise buildings.

2.BASIC RULES AND SPECIFICATIONS FOR SEISMIC DESIGN OF TALL BUILDINGS

Reinforced concrete(RC) Tall Buildings are usually designed with frame systems and core shear wall in the center. Unlike traditional structural systems, in tall buildings, core shear walls must be designed to be resistant horizontal loads as earthquake. Therefore, loads can cause more damage to the structural systems in tall buildings than typical building. For this reason, tall buildings have a low safety factor against collapse.

The use of the Nonlinear Time History Analysis (NTHA) to design tall buildings with more realistic engineering approaches is necessary for different reasons. In order to implement NTHA, the structural members must be preliminary designed and the necessary equipment must be specified. In tall buildings, the minimum base shear which is defined in codes, influences the design due to their long natural periods. Alternative specifications during the preliminary design phase can be followed with some differences. There are two different approaches: two-stage process and three-stage process. The main purpose is to ensure safety and ductility behaviour under target spectrum while construction is designed. The existing seismic codes provide the ductility criterion indirectly by linear methods with reduced forces and capacity design principles. Other than, specifications of codes for seismic design can be very restrictive and deficient in the design of tall buildings. The two-stage process method is based on preliminary design and two checks of performance assessments. The preliminary design is completely left to the knowledge and experience of the design engineer, in other words, seismic codes based on existing elastic methods are totally ignored. It is expected that the specified performance level which is defined immediate occupancy performance level under small effects as collapse prevention level under strong effects are revised if there are unreachable performance level and two-stage analysis is done accordingly. In three-stage process, the preliminary design is occurred by seismic codes as in typical buildings. Minimum base shear must be provided according to seismic codes. After preliminary design, necessary controls are performed. According to performance level in codes as in two-stage process. When all required checks are provided, the design is completed. After the, preliminary design completed satisfactorily, NLTHA are utilized for performance assessment.

In this chapter, the rules and specifications for tall buildings and Buckling-Restrained Braces (BRBFs) are explained according to Los Angeles Tall Buildings Structural Design Council [LATBSDC, 2017], Istanbul Sesmic Design Code for Tall Buildings [ISDCTB, 2008] and Draft Turkish Building Seismic Code [TBSC, 2016], respectively.

2.1.Los Angeles Tall Buildings Structural Design Council (LATBSDC, 2017)

LATBSDC is defined an alternative procedure for seismic analysis and design of tall buildings located in the Los Angeles region as two-stage process as it is explained earlier.

This document is developed for seismic design of tall buildings different building types can be designed by this code. The intent of the document is to provide performance-based design for seismic design of tall buildings with safe performance under earthquake actions. The tall buildings are defined to exceed higher than 50 meters. Measured dimensions for the height of tall buildings are defined above average adjacent ground surface excluding mechanical penthouses and other projections above the roof. The reasons is that, small areas above roof are occurred small mass compared to the mass of the roof.

In this document, analysis is based on capacity design principles procedure by evaluation of performance based design. In LATBSDC, linear analysis are not based on strength based design so linear analysis for tall buildings may be utilized for given performance level to the required strength.

Acceptance criteria for tall buildings of this document [LATBSDC, 2017] is given in Table 2.1,

Table 2.1: Acceptance criteria of LATBSDC.

Level of Earthquake	Earthquake Performance
Frequent/Service	Serviceability
Maximum Considered Earthquake(MCE)	Collapse Prevention

According to Table 2.2, the service level design earthquake are taken to have 50% probability of being exceeded in 30 years (43-year return period.) The purpose of this evaluation is that structural members retain their functionality against events.

However, MCE level of earthquake is defined as Risk Targeted Maximum Considered Earthquake ground motions. The purpose of this evaluation is subjected to demonstrate a low probability of collapse.

Table 2.2: Summary of basic requirements in LATBSDC.

Design / Evaluation Step	Ground Motion Intensity ¹	Type of Analysis	Type of Mathematical Model	Accidental Torsion Considered?	Material Reduction Factors (ϕ)	Material Strength
1	Nonlinear Behavior Defined / Capacity Design					
2	50/30	LDP ² or NDP ³	3D ⁴	Evaluated	1.0	Expected properties are used throughout
3	MCE _R ⁵	NDP	3D ⁴	Yes, if flagged during Step 2. No, otherwise.	1.0 unless otherwise specified in this document	

¹ probability of exceedance in percent / number of years

² linear dynamic procedure

³ nonlinear dynamic procedure

⁴ three-dimensional

⁵ per ASCE 7-16 with modifications and exceptions as noted in this document

2.2.Istanbul Sesmic Design Code for Tall Buildings (ISDCTB, 2008)

ISDCTB Istanbul Seismic Design Code for Tall Buildings is defined as three-stage process as it is explained earlier which is based on PEER Performance Based Design approach. The tall buildings are defined must exceed higher than 60 meters.

The intent of the document is to provide performance-based design for seismic design of tall buildings with safe performance under earthquake actions. In ISDCTB code, 3 earthquakes level are defined which are D1, D2 and D3, respectively to obtain performances level of all buildings. Performance levels are defined in Figure 2.2,

- Minimum Damage/ Immediate Occupancy
- Controlled Damage/ Life Safety

- Extensive Damage/ No-Collapse Safety

In Table 2.3, performance targets of building class are shown,

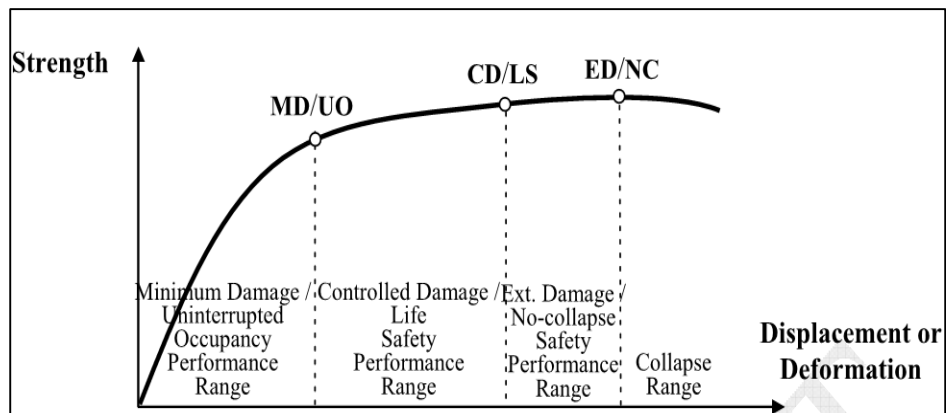


Figure 2.1: Performance levels and regions defined in ISDCTB, 2008.

Table 2.3: Building class minimum performance objectives relationship.

Building Occupancy Class	(E1) Earthquake Level	(E2) Earthquake Level	(E3) Earthquake Level
Normal occupancy class (residence, hotel, office building, etc.)	MD/UO	CD/LS	ED/NC
Special occupancy class (health, education, public admin. Building, etc)	-	MD/OU	CD/

In ISDCTB-2008 code three stage process is defined in Table 2.4 which is I-A, I-B, II and III, respectively. Stages are defined as follows,

- Design Stage(I-A): Preliminary Design with Linear Analysis for Life Safety under D2 Level Earthquake
- Design Stage(I-B): Design with Nonlinear Analysis for Life Safety under D2 Level Earthquake
- Design Stage(II): Design with Linear Analysis for Minimum Damage under D1 Level Earthquake
- Design Stage(III): Design with Nonlinear Analysis for No-Collapse under D3 Level Earthquake

Table 2.4: Analysis and design procedure defined in ISDCTB 2008.

<i>Design Stage</i>	<i>Design Stage I-A</i>	<i>Design Stage I-B</i>	<i>Design Stage II</i>	<i>Design Stage III</i>
Design Type	Preliminary Design	Design	Verification	Verification
Earthquake Level	Normal class buildings (D2- earthquake)	Normal class buildings (D2- earthquake)	Normal class buildings (D2- earthquake)	Normal class buildings (D3- earthquake)
	Special class buildings (D3- earthquake)	Special class buildings (D3- earthquake)	Special class buildings (D2- earthquake)	
Performance Objective	Life Safety	Life Safety	Immediate Occupancy	Collapse Prevention
Analysis Type	3D linear response spectrum	3D nonlinear time history	3D linear response spectrum	3D nonlinear time history
Structural System Behavior Coef.	$R \leq 7$	-	$R=1.5$	-
Story drift ratio limit	%2	%2.5	%1	%3.5
Section stiffness in RC frame members	Effective stiffness (from TSC2007)	Effective stiffness (moment curvature analysis)	Effective stiffness (moment curvature analysis)	Effective stiffness (moment curvature analysis)
Material Strength	Characteristic strength	Expected Strength	Expected Strength	Expected Strength
Acceptance Criteria	Strength & story drift ratio	Strain & story drift ratio	Strength & story drift ratio	Strain & story drift ratio

2.3.Draft Turkish Building Seismic Code (TBSC, 2016)

TBSC is defined new Turkish Building Seismic Code as three-stage process as it is explained earlier which is based on PEER Performance Based Design approach. The tall buildings are defined Class 1 of Buildings that have heights presented in Table 2.5.

Table 2.5: Class of building height in TBSC 2016.

Class of Building Height	According to Class of Building Height and Seismic Design		
	CSD= 1, 1a, 2, 2a	CSD=3, 3a	CSD= 4, 4a
CBH=1	$H_N > 70$	$H_N > 91$	$H_N > 105$
CBH=2	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
CBH=3	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
CBH=4	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
CBH=5	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
CBH=6	$10.5 < H_N \leq 17.5$	$17.5 < H_N \leq 28$	
CBH=7	$7 < H_N \leq 10.5$	$10.5 < H_N \leq 17.5$	
CBH=8	$H_N \leq 7$	$H_N \leq 10.5$	

The four seismic input levels are defined as DD-1, DD-2, DD-3, DD-4, respectively to obtain performance level of all buildings. Performance levels are defined as the same rules of (ISDCTB, 2008) and it is explained previous issues in Table 2.6 and Figure 2.3,

Table 2.6: Target performance level and designing methods.

Earthquakes	CSD= 1, 2, 3, 3a, 4, 4a			CSD= 1a, 2a	
	PERFORMANCE LEVEL	DESIGN		PERFORMANCE LEVEL	DESIGN
DD-4	MD	SBD	PBD	-	
DD-3	-	-		UO	PBD
DD-2	LS	SBD		LS	SBD
DD-1	NC	PBD		LS	PBD

- Minimum Damage/ Uninterrupted Occupancy
- Controlled Damage/ Life Safety
- Extensive Damage/ No-Collapse Safety

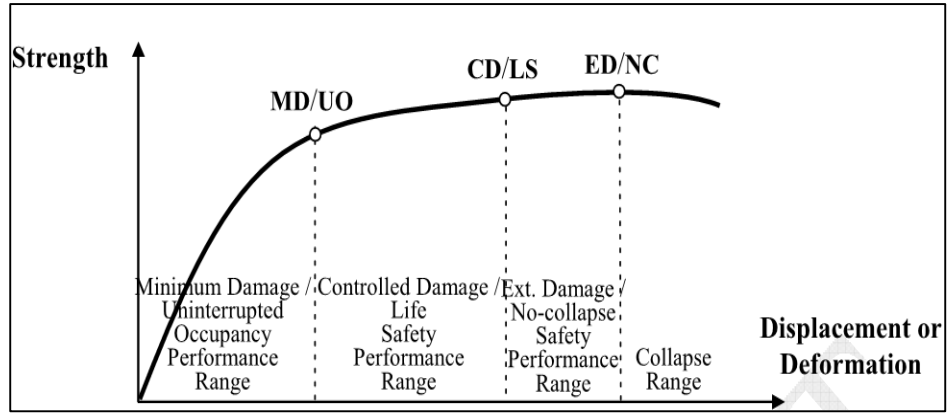


Figure 2.2: Performance levels and regions defined in TSC 2016.

In TBSC 2016 code three stage process is defined which is I, II and III, respectively. Stages are defined as follows,

- Design Stage(I): Preliminary Design with Linear Analysis for Life Safety under design spectrum DD-2 Level Earthquake
- Design Stage(II): Design with Linear or Nonlinear Analysis for Uninterrupted Occupancy under DD-4 Level Earthquake and Design with Nonlinear Analysis for Immediate Occupancy under DD-3 Level Earthquake
- Design Stage(III): Design with Nonlinear Analysis for No-Collapse under DD-1 (MCE) Level Earthquake

In this study, tall buildings are according to depending on Turkish Building Seismic Code Draft 2016 explained in Chapter 13 which is called rules for tall building design. Design procedure for tall building according to Building Seismic Code Draft 2016 will be explained step by step in the next chapters.

3. STRENGTH BASED DESIGN

In this chapter, the design of tall building, which is given and explained previous chapters, will be presented according to 1st Design Step of Chapter 13 in.

In this study, TBSC 2016. First Design step which is a preliminary design was realized according to the new Turkish Building Seismic Code 2016 Chapter 13, 5, 4 and regulations from Chapter 7 the procedure is explained step by step as follows:

- i) Preparing model of tall building in softwares
- ii) Calculating creep effect of tall building for gravity loads and wind loads without earthquake.
- iii) Defining design response spectrum which is called DD-2, and combinations for design based on TS 500 and TBSC 2016
- iv) Design of sections and reinforcement details of elements of tall building.

In the view of such information, high-rise building is modeled using ETABS 2016 for this chapter. For further analysis of model, it is also modeled utilizing SAP2000 then exporting to PERFORM 3D for performance based design of tall building.

However, properties of tall building are explained above for preliminary design;

Tall building is a residential building located in Istanbul. The building consists of a centrally located core shear wall with beams surrounded by concrete columns. Table 3.1 shows general information about reinforced concrete RC tall building and properties of materials. 3D view of the prototype model is shown at Figure 3.1 and typical formwork plan is shown at Figure 3.2.

Table 3.1: General information of tall building.

RC TALL BUILDING	
Number of Storey	50
Story Height	3.5 m
Total Story Height	175 m
Structural System	Core Shear Wall and Frames (Tower Type)
MATERIAL	
Concrete	C50
Reinforcing Steel	S420
Concrete Modulus of Elasticity(E_c)	37000 Mpa
Rebar Modulus of Elasticity(E_s)	200000 Mpa

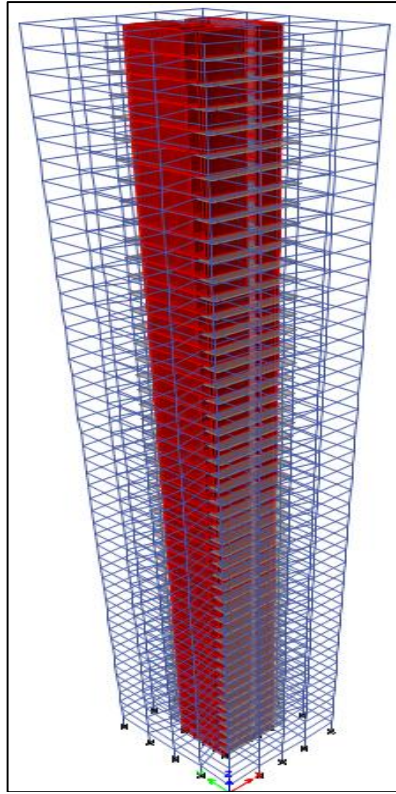


Figure 3.1: Isotropic view of the prototype building.

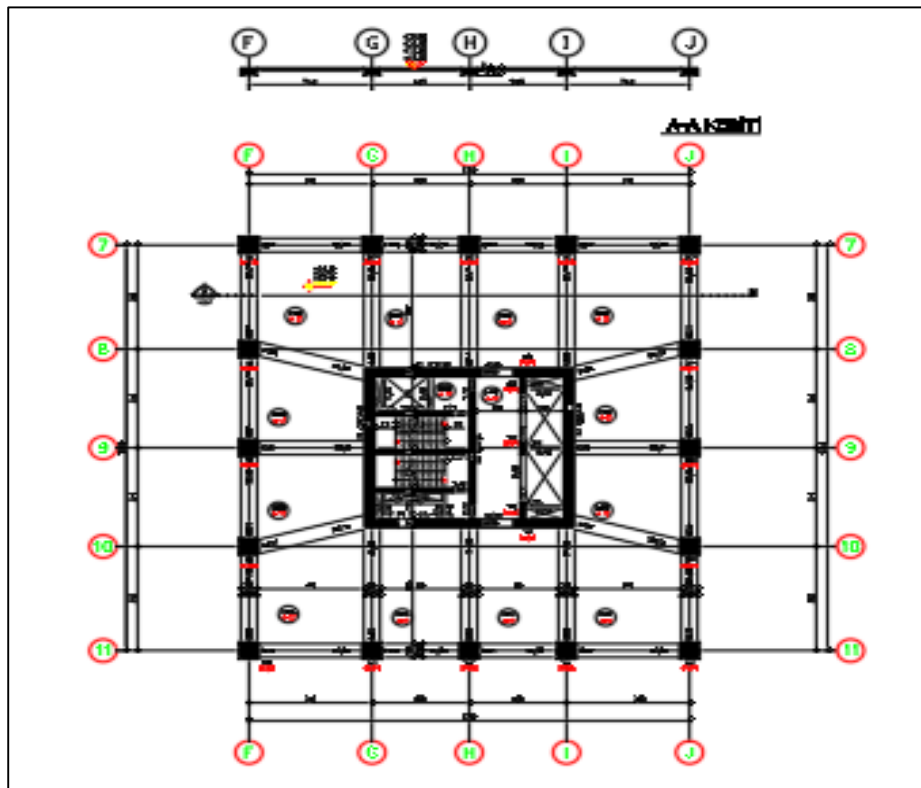


Figure 3.2: Typical formwork of the prototype building.

The building geometrical model can be defined as follows:

- According to formwork plan, as it is shown at Figure 3.2, the dimensions are defined as 27.2 meters and 29.6 meters for direction of X and Y, respectively.
- Core walls' dimensions are defined as 12.90 meters for X way and 11.60 meters for Y way.
- Exterior shear walls where core walls are occurred by, thickness is 60 centimeters and interior shear walls are defined thickness of 30 centimeters.
- Exterior columns are defined in the corner by dimensions of 120^{cm}x120^{cm} and defined at the middle by dimensions of 150^{cm}x150^{cm}, interior columns are defined by dimensions of 30^{cm}x100^{cm} at the core.
- Beams are defined by dimensions of 100^{cm}x50^{cm} and beams at the core, defined dimensions of 25^{cm}x50^{cm} and 30^{cm}x50^{cm}.

3.1.Loads

The loads are specified according to turkish codes. Considering proper loads for model is the most significant issue for analysis therefore the loads type and values are listed and screenshots for different aspects for analysis procedure of the tall building are provided.

3.1.1.Gravity Loading Data

Dead loads and live loads are considered from the code which is “TS 498, Design Loads for Buildings” as presented in Table 3.2-3.4. The weight of structure is calculated by ETABS 2016.

Table 3.2: Dead loads for slabs of rooms.

SLABS FOR ROOMS			
Materials	Density(kg/m ³)	Thickness(m)	Dead Load(kN/m ²)
2 cm insulation	3	0.02	0.06
Finishes	20	0.06	1.2
Ceramic	20	0.01	0.2
Plaster+Dye	2.2	0.02	0.44
Mechanic			0.1
TOTAL			2

Table 3.3: Dead loads for slabs at roof.

SLABS FOR ROOF			
Materials	Density(kN/m³)	Thickness(m)	Dead Load(kN/m²)
Ceramic	20	0.01	0.2
400 dosage of finishes	20	0.05	1
Insulation			0.2
Finishes	20	0.05	1
Mechanic			0.2
Insulation			0.2
TOTAL			2.80

Table 3.4: Gravity loading data.

STORY		Dead Load	Live Load	Unit
ROOF		2.8	5	kN/m ²
TYPICAL	Rooms	2	2.5	kN/m ²
	Halls and Stairs		3.5	kN/m ²

3.1.2.Wind Loading

Wind loads are defined by ETABS due to wind code which is called IYBRY (Wind Codes for Tall Buildings in Istanbul,2009), as input parameters and loads are given in Table 3.5 and Figure 3.3-3.4,

Table 3.5: Wind load parameters.

h=z	175 m	PLAN	L_x	27.2 m
V_b	25 m/s		L_y	29.6 m
z₀	1 m	h/d	h/L_x	6.4
z_{min}	10 m		h/L_y	6.0
k_r	0.23	C_{px}	Front	0.8
Δ	119 m		Behind	-0.7
C_t	1.12	C_{py}	Front	0.8
ρ	1.25 N/m ³		Behind	-0.7

The wind load are calculated by $W=q_p(z) \times C_p \times A$ and applied to structure. In ETABS, the wind are defined by the user and are shown in Figure 3.3 and 3.4.

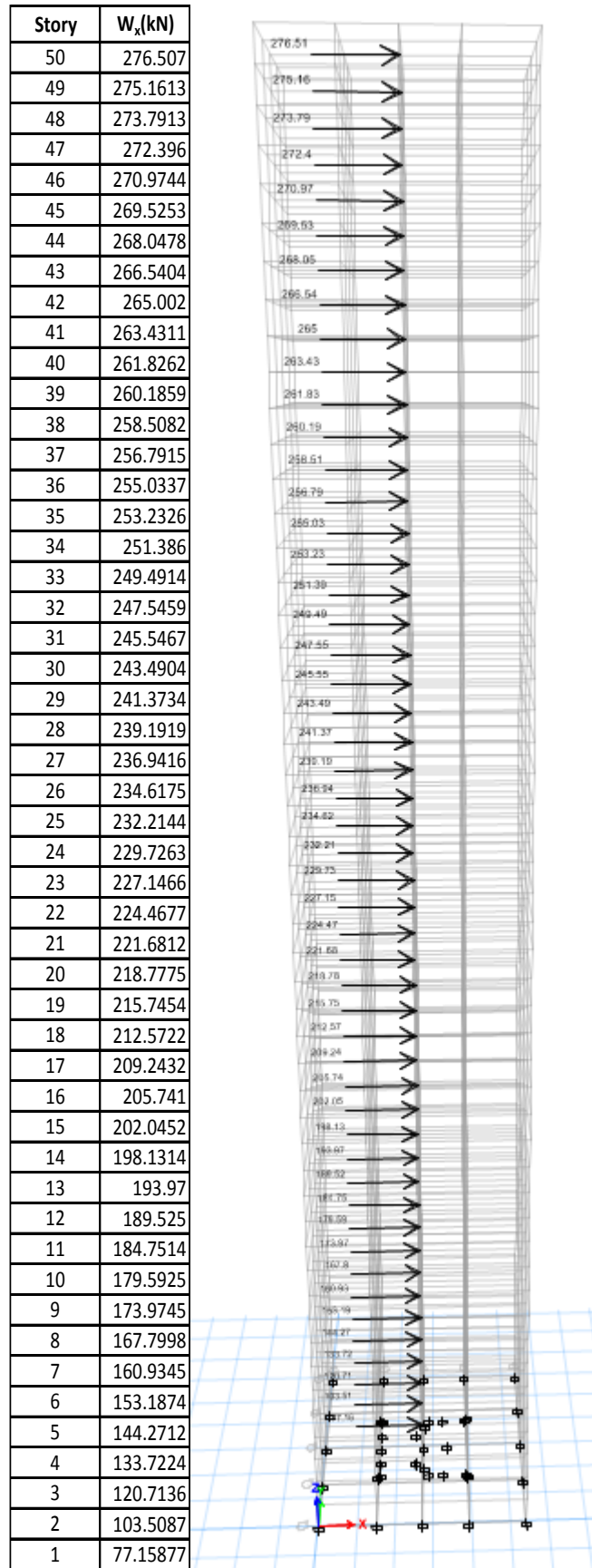


Figure 3.3: Wind loading for X direction.

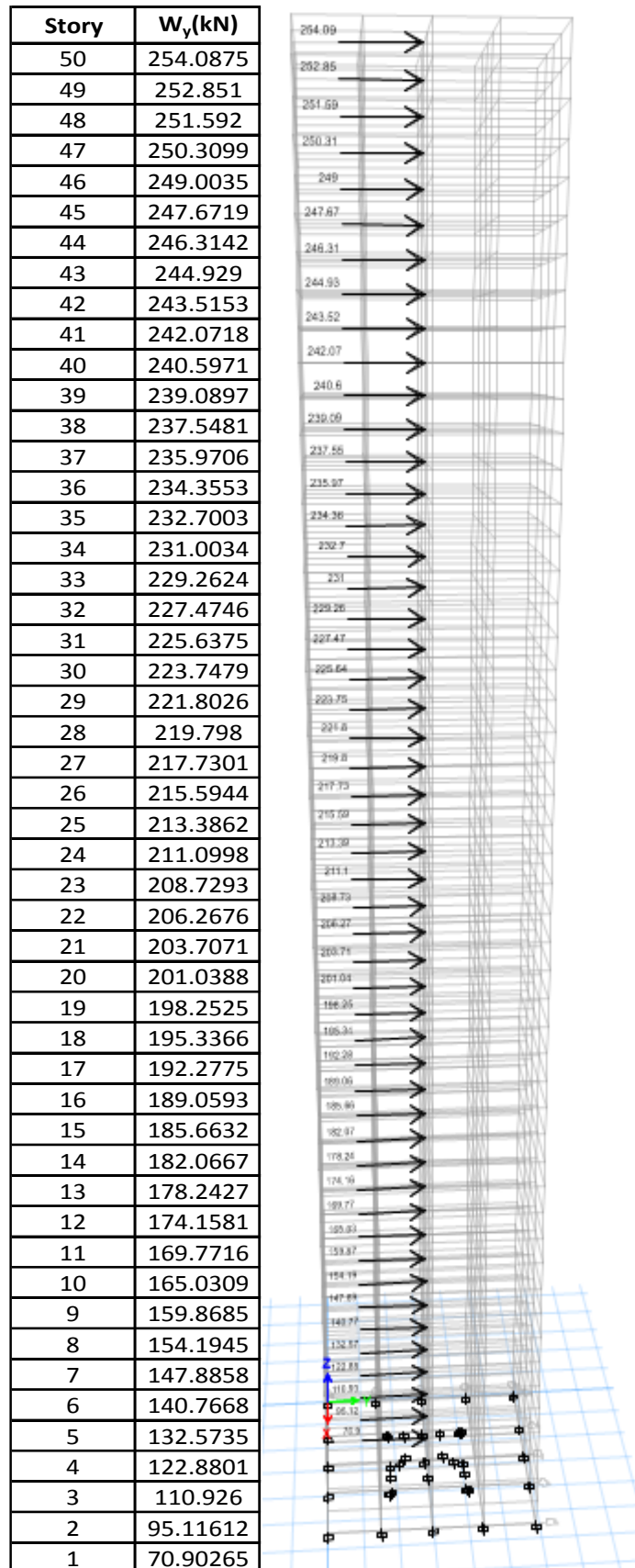


Figure 3.4: Wind loading for Y direction.

3.1.3. Seismic Loading

The seismic parameters are considered according to Turkish Building Seismic Code 2016 which specifications and rules were explained in Chapter 2 the parameters used in the analysis are provided in Table 3.6.

In first step, which is about strenght based design for high-rise buildings, DD-2 design spectrum needs to be considered for preliminary design which is called standard design spectra. DD-2 spectra for acceleration and displacements are shown at Figure 3.5-3.6, respectively.

Table 3.6: Seismic parameters.

Soil Site	ZC
Acceleration Factor (S_{DS} , S_{D1})	0.931, 0.331
Corner Periods (T_A , T_B)	0.071 sec, 0.356 sec

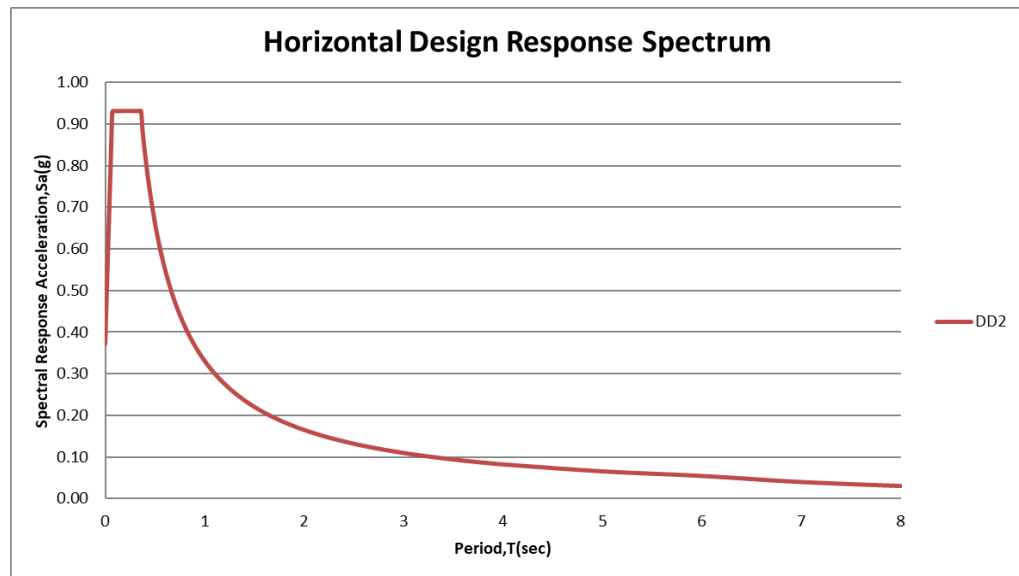


Figure 3.5: DD-2 Horizontal design acceleration response spectrum.

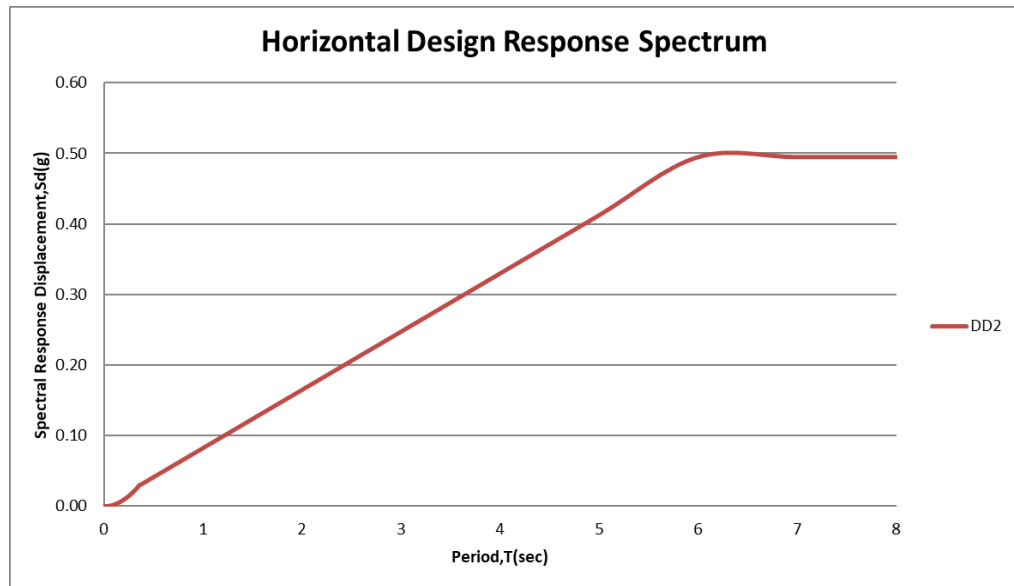


Figure 3.6: DD-2 Horizontal design displacement response spectrum.

In the preliminary design stage, structural behaviour coefficient which is called R , is taken as $4/5$ times of the listed value as defined in Chapter 13 of TBSC 2016. The design acceleration spectra is given in Figure 3.7. For a given tall building, Coefficient of Structural System Behaviour " R ", is considered to be taken 7 because sections of the frame elements and shear walls of tall building was already decided therefore it is not necessary to multiply it by coefficient of 0.8.

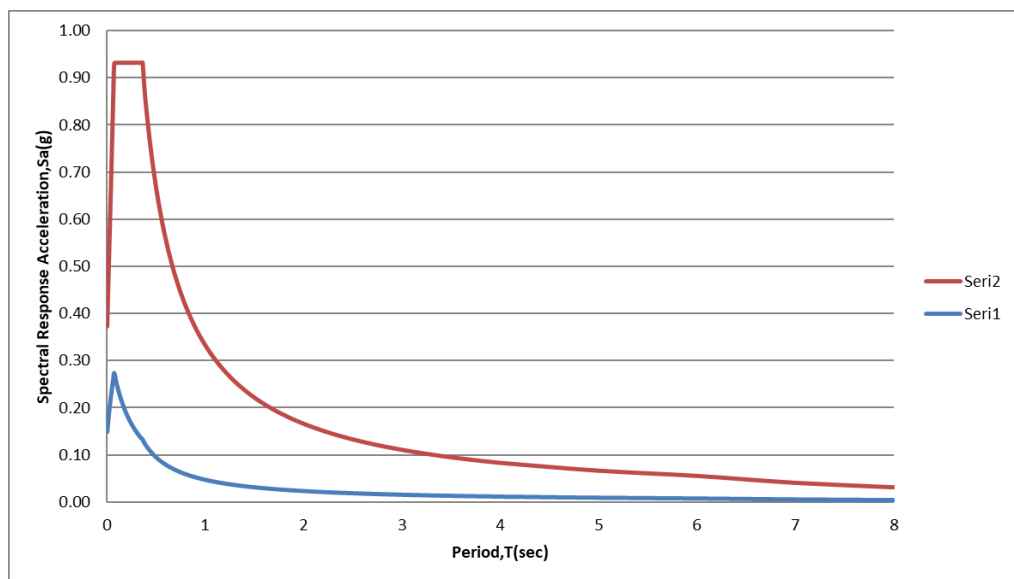


Figure 3.7: Specific response spectrum of tall building.

According to TBSC 2016, 5 % eccentricity of seismic loads are considered for both X and Y direction for designing which are defined in ETABS and given below:

- SPECXPE: Positive 5 % eccentricity of seismic loads for X direction.
- SPECXNE: Negative 5 % eccentricity of seismic loads for X direction.
- SPECYPE: Positive 5 % eccentricity of seismic loads for Y direction.
- SPECYNE: Negative 5 % eccentricity of seismic loads for Y direction.

However, the vertical design spectrum as given in TBSC 2016 in Section 4.4.3 the building complies with Eq (4.6) was neglected for this tall building.

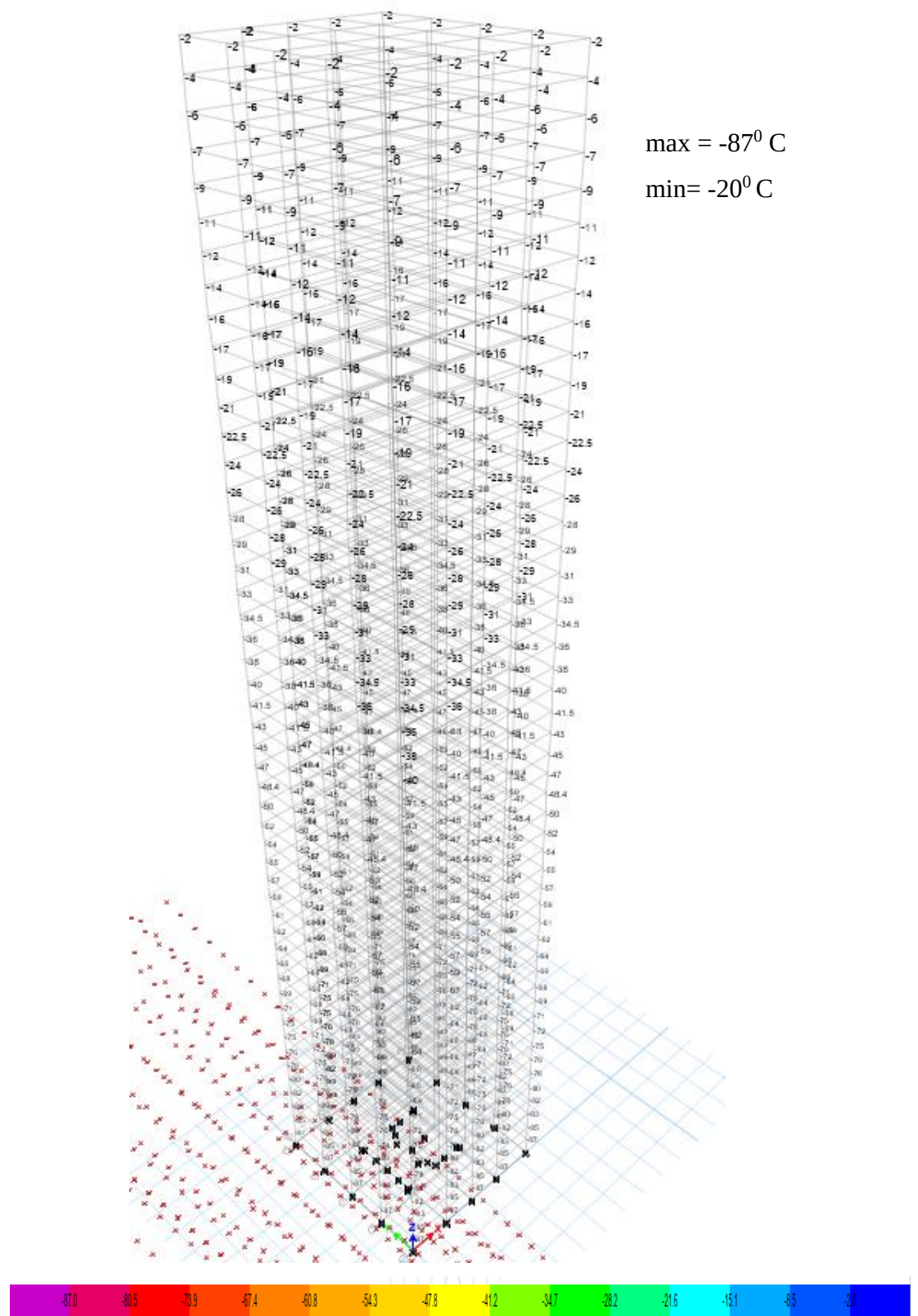
3.1.4. Thermal Loading due to Creep & Shrinkage Effects

Temperature loading is the most significant issue for tall buildings. The reason is that, the creep and shrinkage are occurred in concrete while construction staged so temperature loads are applied due to creep and shrinkage strain. Temperature loads are defined in ETABS to columns which are the most effected by displacements about vertical component as it is shown Figure 3.8. Temperature loads can be applied for beams and shear wall but the values are very smaller which can be neglected for defining.

Temperature loads are defined by calculation of creep and shrinkage effects with formula given in Eq. (3.1);

$$\Delta T = \frac{\epsilon_{sh}}{\alpha(10^{-5})} \quad (3.1)$$

The vertical displacements are calculated for columns to define temperature loads.



3.1.5.Load Combinations

The combinations are defined in ETABS according to codes which are TS 500-2000 and TBSC 2016 as follows:

- **GRAVITY** : $1.4G + 1.6Q$
- **G+nQ** : $G + 0.3Q$
- **G+Q** : $G + Q$
- **EQXPE** : $G + Q + \text{SPECXPE} + 0.3\text{SPECYPE}$
- **EQXNE** : $G + Q + \text{SPECXNE} + 0.3\text{SPECYNE}$
- **EQYPE** : $G + Q + \text{SPECYPE} + 0.3\text{SPECXPE}$
- **EQYNE** : $G + Q + \text{SPECYNE} + 0.3\text{SPECXNE}$
- **EQXNEN** : $G + Q - \text{SPECXNE} - 0.3\text{SPECYNE}$
- **EQXPEN** : $G + Q - \text{SPECXPE} - 0.3\text{SPECYPE}$
- **EQYNEN** : $G + Q - \text{SPECYNE} - 0.3\text{SPECXNE}$
- **EQYPEN** : $G + Q - \text{SPECYPE} - 0.3\text{SPECXPE}$
- **09EQXPE** : $0.9G + \text{SPECXPE} + 0.3\text{SPECYPE}$
- **09EQXNE** : $0.9G + \text{SPECXNE} + 0.3\text{SPECYNE}$
- **09EQYPE** : $0.9G + \text{SPECYPE} + 0.3\text{SPECXPE}$
- **09EQYNE** : $0.9G + \text{SPECYNE} + 0.3\text{SPECXNE}$
- **GQWXP** : $G + 1.3Q + WX$
- **GQWXN** : $G + 1.3Q - WX$
- **GQWYP** : $G + 1.3Q + WY$
- **GQWYN** : $G + 1.3Q - WY$
- **09GWXP** : $0.9G + WX$
- **09GWXN** : $0.9G - WX$
- **09GWYP** : $0.9G + WY$
- **09GWYN** : $0.9G - WY$
- **TEMP** : $G + 1.2Q + 1.2T$

All the data and design parameters necessary for the design of the building are ready for analysis. Tall building model is already for analysis and for design sections of slabs, beams, columns and shear walls.

3.2.Strength Based Design Results

In this part, designing of elements of tall buildings will be considered as a continues topics. Preliminary design will be explained and some calculations and examples and tables will be provided step by step which is the main topic of this chapter as it is explained head of chapter.

The model of tall building was analyzed by ETABS to obtaine the most incapale loads of given combinations for elements of structure that will be chosen for flexural and shear design.

3.2.1.Creep&Shrinkage Effects

Behavior of reinforced concrete structures under temperature effect and a variety of methods have been proposed in the literature to determine internal forces and deformations. Behavior like concrete is highly affected by temperature and for a material that becomes complicated. It should be kept in mind that these proposed methods include approximate and specific assumptions.

On the other hand, the values obtained as a result of analytically developed methods. It should be noted that validity is directly related to the attention and sensitivity in practice.

It should be summarized that the deformations coming from the concrete column as a result of the temperature effect ad strain values which is occured by effecton of Creep and Shrinkage.The design of the concrete slab, the application of the construction slab should be planned accordingly. For this reason, depending on the design, a detailed the construction staged procedure should be established.

However, creep is a time dependent increase of strain. Shrinkage is a time dependent decrease in concrete volumes which occurs temperature loads on the elements of structure. The effecton type of model is CEB-FIP 90 used in given tall building which is calculated by ETABS as it is shown at Figure 3.9.

Figure 3.9: Time dependent properties for concrete material.

In this part, only creep effect will be calculated with construction staged. Shrinkage and creep effect will be calculated and shown deformations will be considered in the calculation of slabs.

In calculation of creep and shrinkage effects, the basic formula Eq. (3.2) is considered;

$$\sigma = \epsilon * E \rightarrow \frac{P}{A} = \frac{\Delta l}{l} * E \rightarrow \Delta l = \frac{P * l}{A * E} \quad (3.2)$$

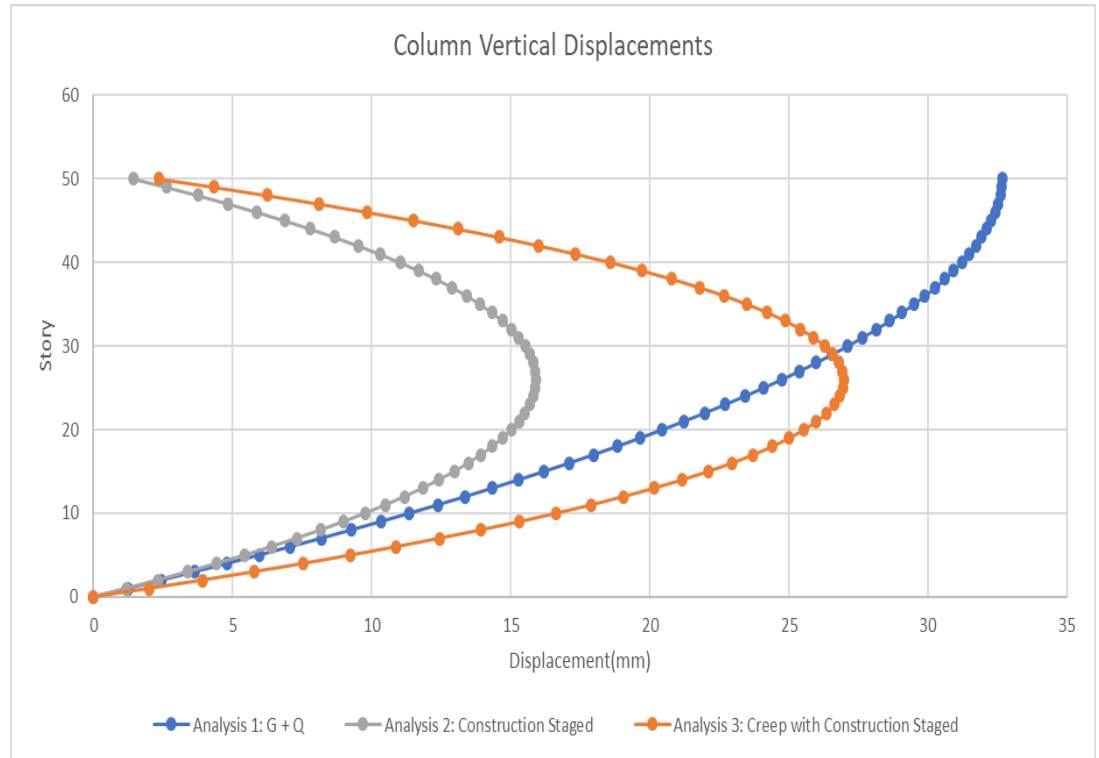


Figure 3.10: Comparison of vertical displacements.

According to Figure 3.10, analyses are defined as 1,2 and 3 which presents only gravity loads, gravity loads with construction staged and gravity loads with construction staged and creep effects, respectively. That is clearly that effect of creep is the most significant issue for tall buildings. If it is not considered carefully in staging construction, absolutely, the fatality error will be appeared.

Vertical displacements, occurred by construction staged and creep effects with gravity and wind loads, are the most effective on vertical frames that are columns and shear walls. Axial load of beams can be calculated but it is not necessary since small loads applied on beams.

The calculation of creep effects are used in, slabs, and were controlled for rules of displacements according to Chapter 13 and Table 13.3 in TS500 by shortening between columns and shear walls where slabs are connected by beams. The formula of deflection rules based on TS500 is shown Eq. (3.3).

$$\Delta u \leq l_n / 240 \quad (3.3)$$

Δu is deflection for flexural elements which are defined as slabs, l_n is dimension of slabs' space which are modeled like one element for this tall building as is defined maximum space about 29.6 meters. The calculation of creep effects were calculated by two methods. First method is calculated by hand that vertical displacements of columns and shear walls were obtained. Second method of calculation are obtained for displacements for slabs by ETABS so results can be obtained easy as it is shown at Figure 3.11;

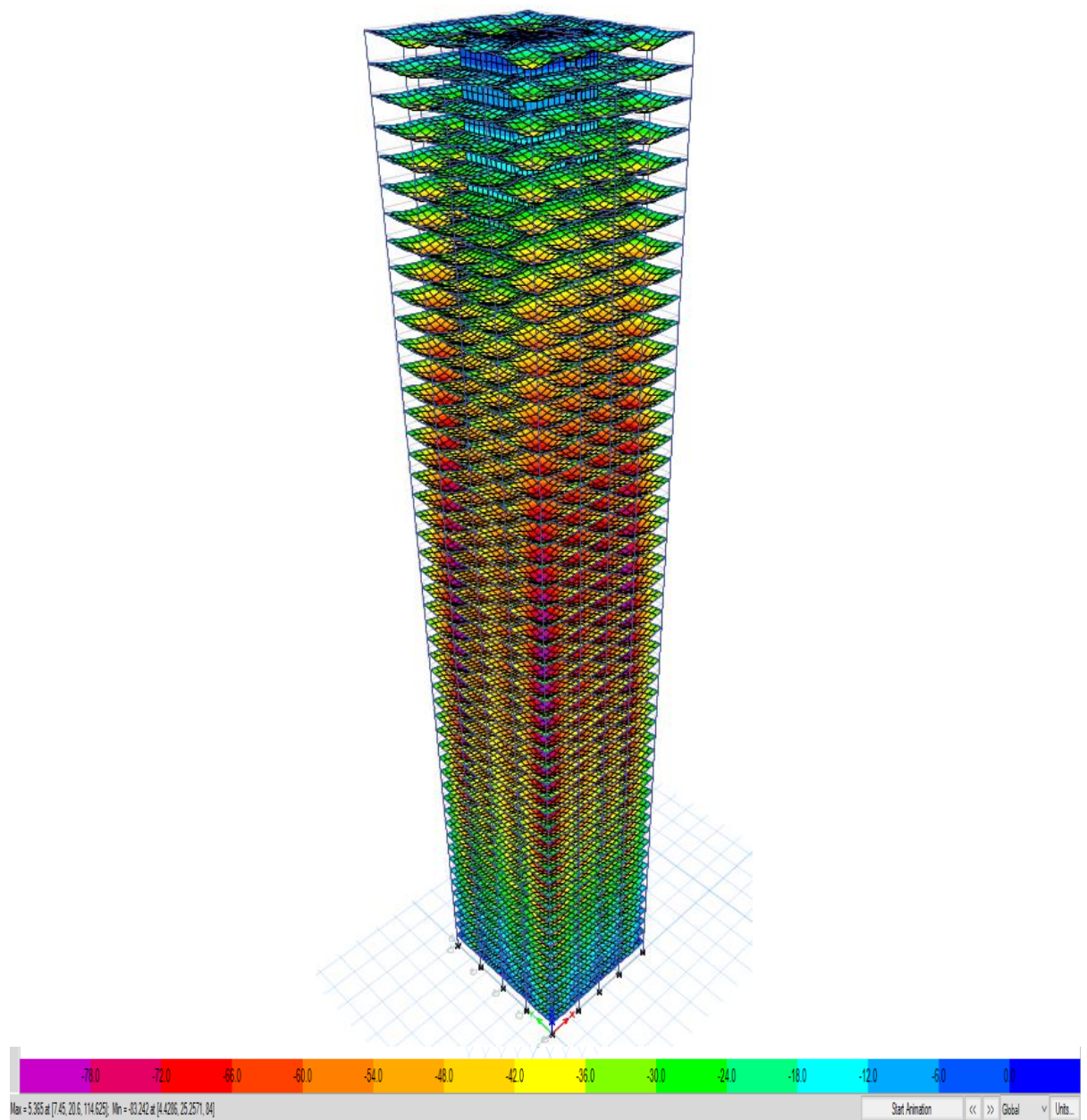


Figure 3.11: Displacements for slabs.

According to Figure 3.11, max displacements of slabs for construction stage and creep are shown.

However, displacements are controlled and provided by range of displacements as following Eq. (3.4).

$$\max \Delta u = 114.625 \text{ mm} \leq \frac{l_n = 29600}{240} = 123.33 \text{ mm} \quad (3.4)$$

3.2.2. Seismic Loading Calculations

Prior to analysis at the model, some rules must be checked according to TBSC 2016. In this part, irregularities of given tall building will be presented and controlled by ranges based on TBSC 2016. Controlling of irregularities are defined by seismic load DD-2.

3.2.3. Checking of Irregularities

- A1-Torsional Irregularity and B2-Interstory Strength Irregularity

The case of A1 where *Torsional Irregularity Coefficient* η_{bi} , which is defined for any of the two orthogonal earthquake directions as the ratio of the maximum relative story drift at any story to the average relative story drift at the same story in the same direction, is greater than 1.2.

The irregularity B2 is defined that case in each of the orthogonal earthquake directions, *Stiffness Irregularity Coefficient* η_{ki} which is defined as the ratio of the average relative story drift at any of the storey to the average relative story drift at the story immediately above or below, is greater than 2.0.

For each story of each component is shown at Table 3.7-3.8 for controlling the A1 and B2 irregularities.

Table 3.7: Irregularities of A1 and B2 for X direction.

Story	$h_i(m)$	$\Delta_{ix,max}$	$\Delta_{ix,min}$	$\Delta_{ix,ort}$	$\Delta_{ix,ort}/h_i$	A1		B2	
						$\eta_{bi(x)}$	CHECK	$\eta_{ki(x)}$	CHECK
50	3.5	0.006	0.006	0.006	0.002	1.01	OK	-	-
49	3.5	0.006	0.006	0.006	0.002	1.01	OK	1.02	OK
48	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.02	OK
47	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.02	OK
46	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.02	OK
45	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.02	OK
44	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.02	OK
43	3.5	0.007	0.006	0.007	0.002	1.02	OK	1.03	OK
42	3.5	0.007	0.007	0.007	0.002	1.02	OK	1.03	OK
41	3.5	0.007	0.007	0.007	0.002	1.02	OK	1.02	OK
40	3.5	0.007	0.007	0.007	0.002	1.02	OK	1.02	OK
39	3.5	0.007	0.007	0.007	0.002	1.02	OK	1.02	OK
38	3.5	0.008	0.007	0.007	0.002	1.02	OK	1.02	OK
37	3.5	0.008	0.007	0.008	0.002	1.02	OK	1.02	OK
36	3.5	0.008	0.008	0.008	0.002	1.02	OK	1.02	OK
35	3.5	0.008	0.008	0.008	0.002	1.02	OK	1.02	OK
34	3.5	0.008	0.008	0.008	0.002	1.02	OK	1.02	OK
33	3.5	0.008	0.008	0.008	0.002	1.02	OK	1.02	OK
32	3.5	0.008	0.008	0.008	0.002	1.02	OK	1.02	OK
31	3.5	0.009	0.008	0.008	0.002	1.02	OK	1.02	OK
30	3.5	0.009	0.008	0.009	0.002	1.02	OK	1.01	OK
29	3.5	0.009	0.008	0.009	0.002	1.02	OK	1.01	OK
28	3.5	0.009	0.009	0.009	0.002	1.02	OK	1.01	OK
27	3.5	0.009	0.009	0.009	0.003	1.02	OK	1.01	OK
26	3.5	0.009	0.009	0.009	0.003	1.02	OK	1.01	OK
25	3.5	0.009	0.009	0.009	0.003	1.02	OK	1.01	OK
24	3.5	0.009	0.009	0.009	0.003	1.02	OK	1.01	OK
23	3.5	0.009	0.009	0.009	0.003	1.02	OK	1.00	OK
22	3.5	0.009	0.009	0.009	0.003	1.02	OK	1.00	OK
21	3.5	0.009	0.009	0.009	0.003	1.03	OK	1.00	OK
20	3.5	0.009	0.009	0.009	0.003	1.03	OK	1.00	OK
19	3.5	0.009	0.009	0.009	0.003	1.03	OK	1.00	OK
18	3.5	0.009	0.009	0.009	0.003	1.03	OK	0.99	OK
17	3.5	0.009	0.009	0.009	0.003	1.03	OK	0.99	OK
16	3.5	0.009	0.009	0.009	0.002	1.03	OK	0.99	OK
15	3.5	0.009	0.008	0.009	0.002	1.03	OK	0.98	OK
14	3.5	0.009	0.008	0.008	0.002	1.03	OK	0.98	OK
13	3.5	0.009	0.008	0.008	0.002	1.03	OK	0.98	OK
12	3.5	0.008	0.008	0.008	0.002	1.03	OK	0.97	OK
11	3.5	0.008	0.008	0.008	0.002	1.03	OK	0.97	OK
10	3.5	0.008	0.007	0.008	0.002	1.03	OK	0.96	OK
9	3.5	0.007	0.007	0.007	0.002	1.03	OK	0.96	OK
8	3.5	0.007	0.007	0.007	0.002	1.03	OK	0.94	OK
7	3.5	0.007	0.006	0.006	0.002	1.03	OK	0.93	OK
6	3.5	0.006	0.006	0.006	0.002	1.03	OK	0.92	OK
5	3.5	0.005	0.005	0.005	0.001	1.04	OK	0.89	OK
4	3.5	0.005	0.004	0.004	0.001	1.04	OK	0.86	OK
3	3.5	0.004	0.003	0.004	0.001	1.04	OK	0.80	OK
2	3.5	0.003	0.002	0.002	0.001	1.05	OK	0.70	OK
1	3.5	0.001	0.001	0.001	0.0003	1.05	OK	0.42	OK

Table 3.8: Irregularities of A1 and B2 for Y direction.

Story	$h_i(m)$	$\Delta_{iy,max}$	$\Delta_{iy,min}$	$\Delta_{iy,ort}$	$\Delta_{iy,ort}/h_i$	A1		B2	
						$\eta_{bi(Y)}$	CHECK	$\eta_{ki(Y)}$	CHECK
50	3.5	0.006	0.006	0.006	0.002	1.02	OK	-	-
49	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.01	OK
48	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.01	OK
47	3.5	0.006	0.006	0.006	0.002	1.02	OK	1.01	OK
46	3.5	0.007	0.006	0.006	0.002	1.02	OK	1.01	OK
45	3.5	0.007	0.006	0.006	0.002	1.02	OK	1.01	OK
44	3.5	0.007	0.006	0.007	0.002	1.03	OK	1.01	OK
43	3.5	0.007	0.006	0.007	0.002	1.03	OK	1.01	OK
42	3.5	0.007	0.006	0.007	0.002	1.03	OK	1.01	OK
41	3.5	0.007	0.006	0.007	0.002	1.03	OK	1.01	OK
40	3.5	0.007	0.007	0.007	0.002	1.04	OK	1.01	OK
39	3.5	0.007	0.007	0.007	0.002	1.04	OK	1.01	OK
38	3.5	0.007	0.007	0.007	0.002	1.04	OK	1.01	OK
37	3.5	0.007	0.007	0.007	0.002	1.04	OK	1.01	OK
36	3.5	0.007	0.007	0.007	0.002	1.04	OK	1.01	OK
35	3.5	0.007	0.007	0.007	0.002	1.04	OK	1.01	OK
34	3.5	0.007	0.007	0.007	0.002	1.05	OK	1.01	OK
33	3.5	0.007	0.007	0.007	0.002	1.05	OK	1.00	OK
32	3.5	0.007	0.007	0.007	0.002	1.05	OK	1.00	OK
31	3.5	0.007	0.007	0.007	0.002	1.05	OK	1.00	OK
30	3.5	0.008	0.007	0.007	0.002	1.05	OK	1.00	OK
29	3.5	0.008	0.007	0.007	0.002	1.05	OK	1.00	OK
28	3.5	0.008	0.007	0.007	0.002	1.06	OK	1.00	OK
27	3.5	0.008	0.007	0.007	0.002	1.06	OK	1.00	OK
26	3.5	0.008	0.007	0.007	0.002	1.06	OK	1.00	OK
25	3.5	0.008	0.007	0.007	0.002	1.06	OK	1.00	OK
24	3.5	0.007	0.007	0.007	0.002	1.07	OK	0.99	OK
23	3.5	0.007	0.006	0.007	0.002	1.07	OK	0.99	OK
22	3.5	0.007	0.006	0.007	0.002	1.07	OK	0.99	OK
21	3.5	0.007	0.006	0.007	0.002	1.08	OK	0.99	OK
20	3.5	0.007	0.006	0.007	0.002	1.08	OK	0.99	OK
19	3.5	0.007	0.006	0.007	0.002	1.08	OK	0.99	OK
18	3.5	0.007	0.006	0.007	0.002	1.09	OK	0.98	OK
17	3.5	0.007	0.006	0.006	0.002	1.09	OK	0.98	OK
16	3.5	0.007	0.006	0.006	0.002	1.10	OK	0.98	OK
15	3.5	0.007	0.005	0.006	0.002	1.10	OK	0.97	OK
14	3.5	0.007	0.005	0.006	0.002	1.11	OK	0.97	OK
13	3.5	0.006	0.005	0.006	0.002	1.11	OK	0.97	OK
12	3.5	0.006	0.005	0.005	0.002	1.12	OK	0.96	OK
11	3.5	0.006	0.005	0.005	0.002	1.12	OK	0.96	OK
10	3.5	0.006	0.004	0.005	0.001	1.12	OK	0.95	OK
9	3.5	0.005	0.004	0.005	0.001	1.13	OK	0.94	OK
8	3.5	0.005	0.004	0.004	0.001	1.12	OK	0.94	OK
7	3.5	0.005	0.004	0.004	0.001	1.12	OK	0.92	OK
6	3.5	0.004	0.003	0.004	0.001	1.12	OK	0.91	OK
5	3.5	0.004	0.003	0.003	0.001	1.12	OK	0.88	OK
4	3.5	0.003	0.002	0.003	0.001	1.13	OK	0.84	OK
3	3.5	0.002	0.002	0.002	0.001	1.13	OK	0.79	OK
2	3.5	0.002	0.001	0.002	0.0004	1.13	OK	0.70	OK
1	3.5	0.001	0.001	0.001	0.0002	1.12	OK	0.44	OK

- A2-Floor Discontinuities

In any story;

- i) The case where total area of the openings including those of stairs and elevator shafts exceeds 1/3 of the gross floors area as it is checked and presented at Table 3.9.

Table 3.9: Irregularities of A2 for building.

STORY	STORY AREA	OPENINGS AREA	RATIO	CHECK
	m ²	m ²		
50	874.72	12.3	0.014	OK
49	874.72	12.3	0.014	OK
48	874.72	12.3	0.014	OK
47	874.72	12.3	0.014	OK
46	874.72	12.3	0.014	OK
45	874.72	12.3	0.014	OK
44	874.72	12.3	0.014	OK
43	874.72	12.3	0.014	OK
42	874.72	12.3	0.014	OK
41	874.72	12.3	0.014	OK
40	874.72	12.3	0.014	OK
39	874.72	12.3	0.014	OK
38	874.72	12.3	0.014	OK
37	874.72	12.3	0.014	OK
36	874.72	12.3	0.014	OK
35	874.72	12.3	0.014	OK
34	874.72	12.3	0.014	OK
33	874.72	12.3	0.014	OK
32	874.72	12.3	0.014	OK
31	874.72	12.3	0.014	OK
30	874.72	12.3	0.014	OK
29	874.72	12.3	0.014	OK
28	874.72	12.3	0.014	OK
27	874.72	12.3	0.014	OK
26	874.72	12.3	0.014	OK
25	874.72	12.3	0.014	OK
24	874.72	12.3	0.014	OK
23	874.72	12.3	0.014	OK
22	874.72	12.3	0.014	OK
21	874.72	12.3	0.014	OK
20	874.72	12.3	0.014	OK
19	874.72	12.3	0.014	OK
18	874.72	12.3	0.014	OK
17	874.72	12.3	0.014	OK
16	874.72	12.3	0.014	OK
15	874.72	12.3	0.014	OK
14	874.72	12.3	0.014	OK
13	874.72	12.3	0.014	OK
12	874.72	12.3	0.014	OK
11	874.72	12.3	0.014	OK
10	874.72	12.3	0.014	OK
9	874.72	12.3	0.014	OK
8	874.72	12.3	0.014	OK
7	874.72	12.3	0.014	OK
6	874.72	12.3	0.014	OK
5	874.72	12.3	0.014	OK
4	874.72	12.3	0.014	OK
3	874.72	12.3	0.014	OK
2	874.72	12.3	0.014	OK
1	874.72	12.3	0.014	OK

- ii) The case is not applied for the building model where local floor openings which make the safe transfer of seismic loads difficult to vertical structural elements
- iii) The case is not applied for the building model of abrupt reductions in the in-plane stiffness and strength of floors.

- A3-Projections in Plan

The case is not existed in the building model where dimensions of projections in both directions in plan exceed the total plan dimensions of that story of the building in the respective directions by more than 20%.

- B1-Interstory Strength Irregularity and B3-Discontinuity of Vertical Structural Elements

Due to given building, irregularities of B1 and B3 are not existed in the building model. The reason is that in this building every story and elements of the structure are the same so calculation of irregularities for B1 and B3 are not necessary

According to checking irregularities, in building which used for this study, any irregularities are not existed which means calculation can be continued with selected structural system.

3.2.4. Seismic Mass Calculation

Masses are assembled in ETABS through various ways. The easiest model is defining everything as loads. ETABS transforms loads to masses with defined participation ratios. Since this is a residential building the seismic mass is $G + 0.3 Q$ as it is shown below by formulas as Eq. (3.8) -(3.11)

$$W = \sum_{i=1}^N w_i \quad (3.8)$$

$$w_i = g_i + nq_i \quad (3.9)$$

$$m_i = w_i / g \quad (3.10)$$

$$g = 9.81 \text{ m/s}^2 \quad (3.11)$$

According to all informations which are given above, the calculation of mass is presented at Table 3.10.

Table 3.10: Definition of mass.

STORY	Mass X	Mass Y	Mass o
	t	t	t-m ²
1-3	1101.145	1101.145	73879.49
4	1100.476	1100.476	73834.59
5	1099.36	1099.36	73759.76
6 - 49	1098.914	1098.914	73729.83
50	1063.586	1063.586	71359.54
TOTAL	54883.76	54883.76	3682334

3.2.5.Modal Analysis Results

The number of modes of vibration required to participate in the account in each of the perpendicular X and Y horizontal earthquake directions, which is specified in Chapter 4 in TBSC 2016 as the sum of the modal masses calculated for each mode should never be less than 90% of the total mass of the building. For earthquake actions, cracked section properties of structural members are taken into account. Multipliers for each structural member are given in the code, Table 4.2. in TBSC 2016. Mode shapes and participating mass ratios are given in the Table 3.11.

Table 3.11: Modal parcipating factor.

Mode	Period sec	UX	UY	Sum UX	Sum UY
1	7.496	0.7046	0.0001	70.46	0.01
2	7.04	0.0001	0.6631	70.47	66.32
3	4.904	0	0.0028	70.47	66.61
4	2.101	0.1378	0	84.25	66.61
5	1.709	0	0.1465	84.25	81.26
6	1.534	0	0.0183	84.25	83.09
7	1.012	0.0508	0	89.33	83.09
8	0.866	0	0.004	89.33	83.49
9	0.729	0	0.0567	89.33	89.16
10	0.616	0.0278	0	92.12	89.16
11	0.566	0	0.001	92.12	89.26
12	0.43	3.97E-06	0.0287	92.12	92.13

3.2.6. Minimum Base Shear Calculation

The minimum base shear for tall buildings is defined in Chapter 13 of TBSC 2016 as given in Eq. (3.12) -(3.15).

$$V_{\min} = 0.04 \alpha_H m_t S_{DS} g \quad (3.12)$$

$$\alpha_H: H_N > 155 \text{ m so } = 0.5 \quad (3.13)$$

$$S_{DS}: 0.932 g \quad (3.14)$$

$$V_{\min} = 10055.5 \text{ kN} \quad (3.15)$$

The calculated base shears are presented at Eq. (3.16) –(3.17)

$$V_x = 5638 \text{ kN} \quad (3.16)$$

$$V_y = 6478.5 \text{ kN} \quad (3.17)$$

TBSC 2016 Eq. (4.25) defines the check of base shear calculated with modal analysis against equivalent lateral force method. $\gamma_E = 1.0$ according to Chapter 13.

Modal base shear is smaller than the minimum for each direction so that earthquake which is design spectrum DD-2, will be multiplied by β which coefficient of amplification for base shear force as considered in TBSC 2016 Eq. (4.25) as following Eq. (3.18).

$$\beta_{tE}^X = 1.78 \quad \beta_{tE}^Y = 1.55 \quad (3.18)$$

3.2.7. Mode Superposition Analysis

A three-dimensional model of the structural system has been established to determine the internal forces and displacements that will occur in the structural system elements under the common influence of the dynamic analysis results using the Modal Superposition Method with the building loads. In the model, the shear walls are idealized using shell elements. Each floor is considered to be a rigid diaphragm in its own plane. The three-dimensional analysis model of the building

and the deformed shapes for both earthquake directions are presented at Figures 3.12-3.14.

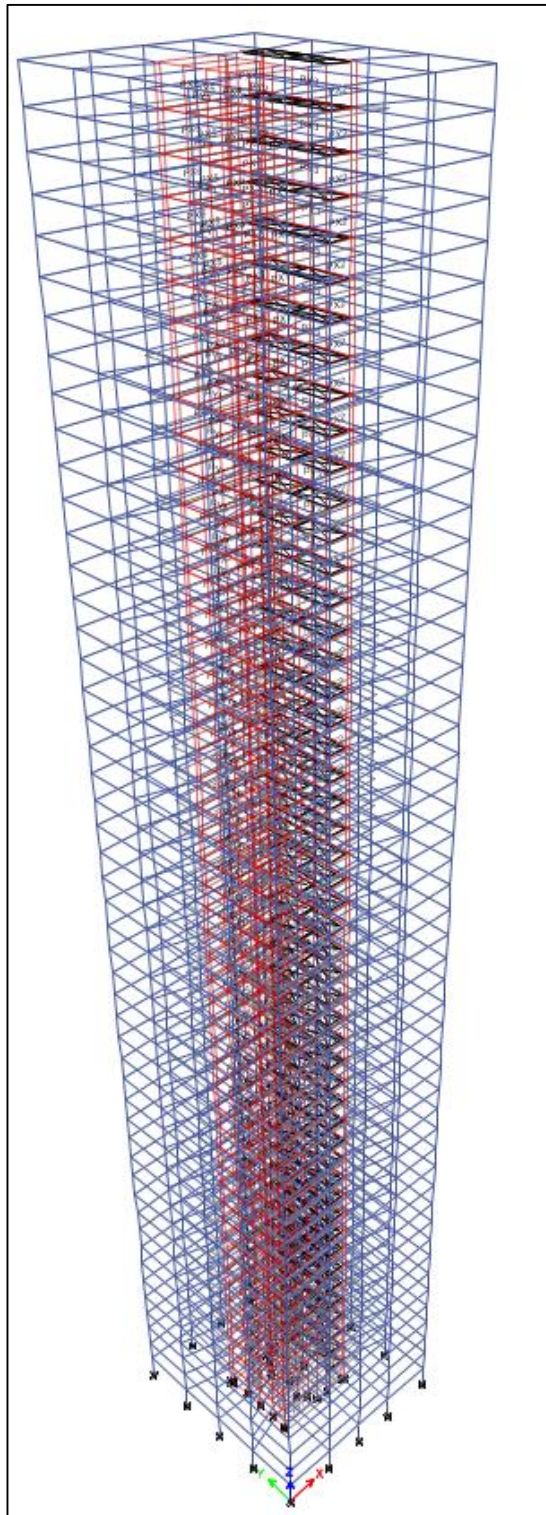


Figure 3.12: Mathematical 3-D model of building.

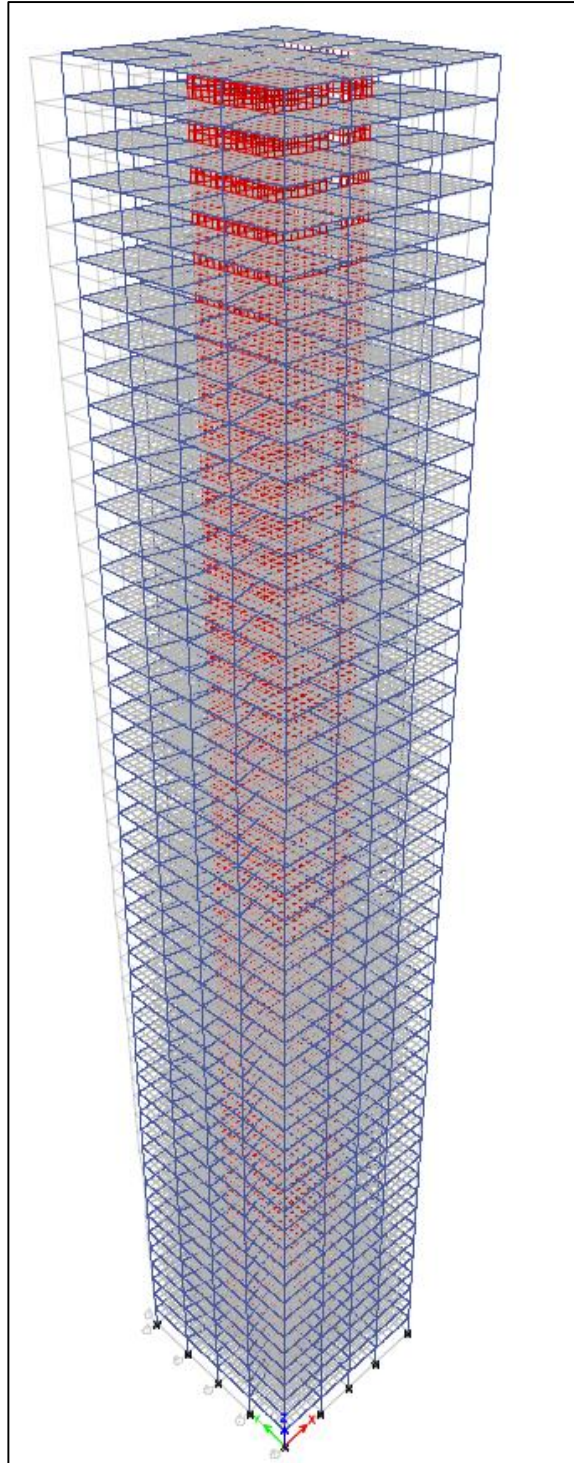


Figure 3.13: Deformed shape of building for + X loading.

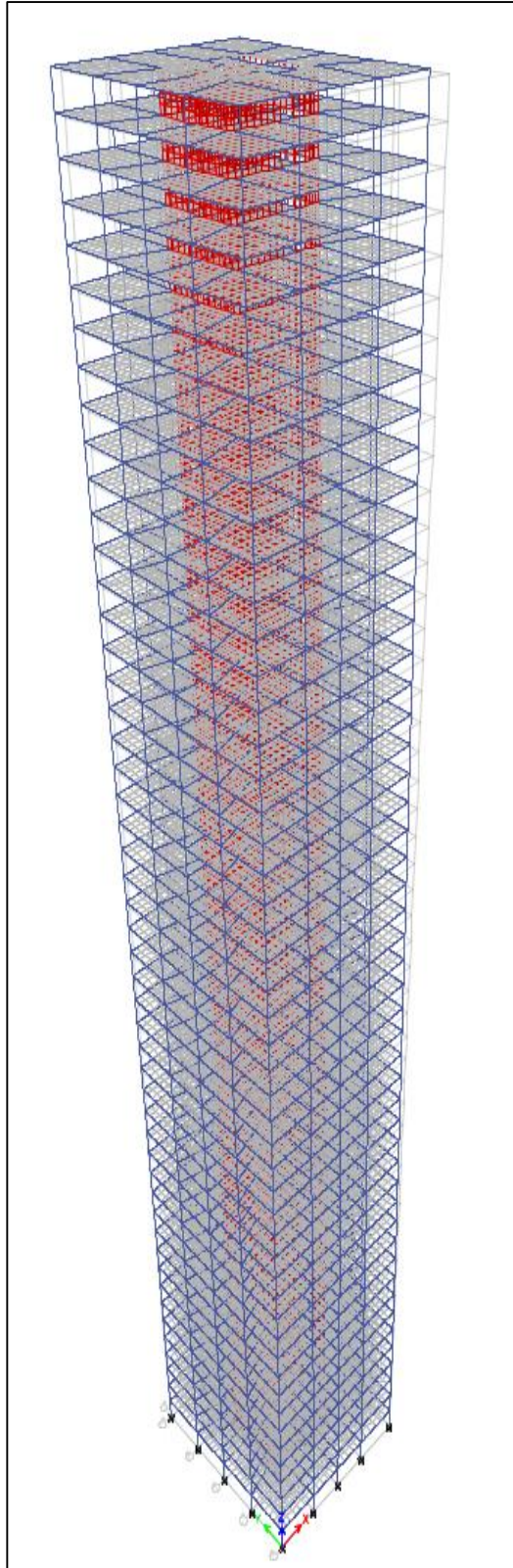


Figure 3.14: Deformed shape of building for +Y loading.

3.2.8.Drift Checks

All story drifts are checked with the limit of $\lambda \times \delta_{i, \max}^{(X)} / h_i \leq 0.016$. DD-2 level is taken into account and λ is calculated for component of X and Y with taken 0.353 and 0.39, respectively. All drifts are checked and found to be within the limitations of the code as it is presented at Table 3.12-3.13.

Table 3.12: Story drift for X direction.

Story	$h_i(m)$	$d_{ix}(m)$	$\Delta_{ix}(m)$	$\delta_{ix}(m)$	δ_{ix}/h_i	CHECK
50	3.5	0.3730	0.0057	0.0399	0.0040	OK
49	3.5	0.3674	0.0058	0.0406	0.0041	OK
48	3.5	0.3616	0.0059	0.0413	0.0042	OK
47	3.5	0.3557	0.0060	0.0422	0.0043	OK
46	3.5	0.3496	0.0062	0.0432	0.0044	OK
45	3.5	0.3435	0.0063	0.0442	0.0045	OK
44	3.5	0.3372	0.0065	0.0454	0.0046	OK
43	3.5	0.3307	0.0066	0.0465	0.0047	OK
42	3.5	0.3240	0.0068	0.0477	0.0048	OK
41	3.5	0.3172	0.0070	0.0489	0.0049	OK
40	3.5	0.3102	0.0072	0.0501	0.0051	OK
39	3.5	0.3031	0.0073	0.0514	0.0052	OK
38	3.5	0.2957	0.0075	0.0526	0.0053	OK
37	3.5	0.2882	0.0077	0.0538	0.0054	OK
36	3.5	0.2805	0.0079	0.0550	0.0055	OK
35	3.5	0.2727	0.0080	0.0561	0.0057	OK
34	3.5	0.2647	0.0082	0.0573	0.0058	OK
33	3.5	0.2565	0.0083	0.0583	0.0059	OK
32	3.5	0.2481	0.0085	0.0593	0.0060	OK
31	3.5	0.2397	0.0086	0.0603	0.0061	OK
30	3.5	0.2311	0.0087	0.0612	0.0062	OK
29	3.5	0.2223	0.0088	0.0619	0.0062	OK
28	3.5	0.2135	0.0090	0.0627	0.0063	OK
27	3.5	0.2045	0.0090	0.0633	0.0064	OK
26	3.5	0.1955	0.0091	0.0639	0.0064	OK
25	3.5	0.1864	0.0092	0.0643	0.0065	OK
24	3.5	0.1772	0.0092	0.0647	0.0065	OK
23	3.5	0.1679	0.0093	0.0649	0.0065	OK
22	3.5	0.1587	0.0093	0.0650	0.0066	OK
21	3.5	0.1494	0.0093	0.0651	0.0066	OK
20	3.5	0.1401	0.0093	0.0649	0.0065	OK
19	3.5	0.1308	0.0092	0.0646	0.0065	OK
18	3.5	0.1216	0.0092	0.0642	0.0065	OK
17	3.5	0.1124	0.0091	0.0636	0.0064	OK
16	3.5	0.1033	0.0090	0.0629	0.0063	OK
15	3.5	0.0943	0.0089	0.0620	0.0062	OK
14	3.5	0.0855	0.0087	0.0609	0.0061	OK
13	3.5	0.0768	0.0085	0.0595	0.0060	OK
12	3.5	0.0683	0.0083	0.0580	0.0059	OK
11	3.5	0.0600	0.0080	0.0562	0.0057	OK
10	3.5	0.0519	0.0077	0.0542	0.0055	OK
9	3.5	0.0442	0.0074	0.0518	0.0052	OK
8	3.5	0.0368	0.0070	0.0489	0.0049	OK
7	3.5	0.0298	0.0065	0.0457	0.0046	OK
6	3.5	0.0233	0.0060	0.0419	0.0042	OK
5	3.5	0.0173	0.0054	0.0375	0.0038	OK
4	3.5	0.0120	0.0046	0.0322	0.0032	OK
3	3.5	0.0074	0.0037	0.0258	0.0026	OK
2	3.5	0.0037	0.0026	0.0180	0.0018	OK
1	3.5	0.0011	0.0011	0.0076	0.0008	OK

Table 3.13: Story drifts for Y direction.

Story	$h_i(m)$	$d_{iy}(m)$	$\Delta_{iy}(m)$	$\delta_{iy}(m)$	δ_{iy}/h_i	CHECK
50	3.5	0.3169	0.0063	0.0442	0.0049	OK
49	3.5	0.3106	0.0064	0.0446	0.0050	OK
48	3.5	0.3043	0.0064	0.0450	0.0050	OK
47	3.5	0.2978	0.0065	0.0455	0.0051	OK
46	3.5	0.2913	0.0066	0.0460	0.0051	OK
45	3.5	0.2848	0.0066	0.0465	0.0052	OK
44	3.5	0.2781	0.0067	0.0470	0.0052	OK
43	3.5	0.2714	0.0068	0.0475	0.0053	OK
42	3.5	0.2646	0.0069	0.0481	0.0054	OK
41	3.5	0.2578	0.0069	0.0486	0.0054	OK
40	3.5	0.2508	0.0070	0.0491	0.0055	OK
39	3.5	0.2438	0.0071	0.0496	0.0055	OK
38	3.5	0.2367	0.0071	0.0500	0.0056	OK
37	3.5	0.2296	0.0072	0.0505	0.0056	OK
36	3.5	0.2224	0.0073	0.0509	0.0057	OK
35	3.5	0.2151	0.0073	0.0513	0.0057	OK
34	3.5	0.2078	0.0074	0.0516	0.0057	OK
33	3.5	0.2004	0.0074	0.0519	0.0058	OK
32	3.5	0.1930	0.0074	0.0521	0.0058	OK
31	3.5	0.1856	0.0075	0.0524	0.0058	OK
30	3.5	0.1781	0.0075	0.0525	0.0059	OK
29	3.5	0.1706	0.0075	0.0526	0.0059	OK
28	3.5	0.1631	0.0075	0.0527	0.0059	OK
27	3.5	0.1555	0.0075	0.0527	0.0059	OK
26	3.5	0.1480	0.0075	0.0526	0.0059	OK
25	3.5	0.1405	0.0075	0.0525	0.0059	OK
24	3.5	0.1330	0.0075	0.0523	0.0058	OK
23	3.5	0.1255	0.0074	0.0521	0.0058	OK
22	3.5	0.1181	0.0074	0.0518	0.0058	OK
21	3.5	0.1107	0.0073	0.0514	0.0057	OK
20	3.5	0.1033	0.0073	0.0509	0.0057	OK
19	3.5	0.0961	0.0072	0.0503	0.0056	OK
18	3.5	0.0889	0.0071	0.0496	0.0055	OK
17	3.5	0.0818	0.0070	0.0488	0.0054	OK
16	3.5	0.0748	0.0068	0.0479	0.0053	OK
15	3.5	0.0680	0.0067	0.0469	0.0052	OK
14	3.5	0.0613	0.0065	0.0457	0.0051	OK
13	3.5	0.0548	0.0063	0.0444	0.0049	OK
12	3.5	0.0484	0.0061	0.0429	0.0048	OK
11	3.5	0.0423	0.0059	0.0412	0.0046	OK
10	3.5	0.0364	0.0056	0.0393	0.0044	OK
9	3.5	0.0308	0.0053	0.0372	0.0041	OK
8	3.5	0.0255	0.0050	0.0349	0.0039	OK
7	3.5	0.0205	0.0046	0.0322	0.0036	OK
6	3.5	0.0159	0.0042	0.0291	0.0032	OK
5	3.5	0.0117	0.0037	0.0257	0.0029	OK
4	3.5	0.0081	0.0031	0.0218	0.0024	OK
3	3.5	0.0050	0.0025	0.0173	0.0019	OK
2	3.5	0.0025	0.0017	0.0121	0.0013	OK
1	3.5	0.0008	0.0008	0.0053	0.0006	OK

Second order effects of structure are calculated for both components according to TSC 2016 Chapter Eq. (4.29) and checked with Eq. (4.30) in $\chi = 0.5$ is considered for RC buildings. The results are presented at Table 3.14-3.15.

Table 3.14: Second order effect checks for both directions.

Story	$\sum_{j=1}^n w_j$	w_i	$\Delta_{ix,ort}$	$\Delta_{iy,ort}$	V_{ix}	V_{iy}	h_i	θ_{ix}	θ_{iy}
50	1063.6	1063586.2	0.0056	0.0062	815.0	956.9	3.5	0.0021	0.0020
49	1100.0	1098914.3	0.0057	0.0063	1565.2	1821.9	3.5	0.0011	0.0011
48	1100.0	1098914.3	0.0058	0.0063	2208.1	2544.5	3.5	0.0008	0.0008
47	1100.0	1098914.3	0.0059	0.0064	2744.5	3126.2	3.5	0.0007	0.0006
46	1100.0	1098914.3	0.0061	0.0064	3178.7	3572.9	3.5	0.0006	0.0006
45	1100.0	1098914.3	0.0062	0.0065	3519.2	3896.2	3.5	0.0006	0.0005
44	1100.0	1098914.3	0.0064	0.0065	3779.6	4113.8	3.5	0.0005	0.0005
43	1100.0	1098914.3	0.0065	0.0066	3977.8	4249.2	3.5	0.0005	0.0005
42	1100.0	1098914.3	0.0067	0.0067	4135.1	4330.5	3.5	0.0005	0.0005
41	1100.0	1098914.3	0.0069	0.0067	4274.4	4388.5	3.5	0.0005	0.0005
40	1100.0	1098914.3	0.0070	0.0068	4416.8	4452.3	3.5	0.0005	0.0005
39	1100.0	1098914.3	0.0072	0.0068	4578.4	4544.5	3.5	0.0005	0.0005
38	1100.0	1098914.3	0.0074	0.0069	4768.0	4677.2	3.5	0.0005	0.0005
37	1100.0	1098914.3	0.0075	0.0069	4986.2	4850.6	3.5	0.0005	0.0004
36	1100.0	1098914.3	0.0077	0.0070	5226.3	5054.0	3.5	0.0005	0.0004
35	1100.0	1098914.3	0.0079	0.0070	5477.3	5270.9	3.5	0.0005	0.0004
34	1100.0	1098914.3	0.0080	0.0070	5727.1	5483.4	3.5	0.0004	0.0004
33	1100.0	1098914.3	0.0082	0.0071	5964.6	5676.3	3.5	0.0004	0.0004
32	10781.4	10780349.7	0.0083	0.0071	6181.8	5839.3	3.5	0.0041	0.0037
31	1109.7	1098914.3	0.0084	0.0071	6375.2	5968.6	3.5	0.0004	0.0004
30	1100.0	1098914.3	0.0085	0.0071	6545.1	6066.6	3.5	0.0004	0.0004
29	1100.0	1098914.3	0.0086	0.0071	6696.0	6141.3	3.5	0.0004	0.0004
28	1100.0	1098914.3	0.0087	0.0071	6834.5	6204.5	3.5	0.0004	0.0004
27	1100.0	1098914.3	0.0088	0.0071	6968.3	6269.4	3.5	0.0004	0.0004
26	1100.0	1098914.3	0.0089	0.0071	7104.5	6348.0	3.5	0.0004	0.0004
25	1100.0	1098914.3	0.0090	0.0071	7247.9	6448.4	3.5	0.0004	0.0003
24	1100.0	1098914.3	0.0090	0.0070	7399.8	6572.9	3.5	0.0004	0.0003
23	1100.0	1098914.3	0.0090	0.0070	7558.2	6718.0	3.5	0.0004	0.0003
22	1100.0	1098914.3	0.0091	0.0069	7717.9	6875.4	3.5	0.0004	0.0003
21	1100.0	1098914.3	0.0091	0.0068	7872.2	7034.1	3.5	0.0004	0.0003
20	1100.0	1098914.3	0.0090	0.0067	8014.1	7183.3	3.5	0.0004	0.0003
19	1100.0	1098914.3	0.0090	0.0066	8138.2	7314.6	3.5	0.0003	0.0003
18	1100.0	1098914.3	0.0089	0.0065	8241.7	7424.1	3.5	0.0003	0.0003
17	1100.0	1098914.3	0.0089	0.0064	8325.6	7513.1	3.5	0.0003	0.0003
16	1100.0	1098914.3	0.0087	0.0062	8394.3	7588.4	3.5	0.0003	0.0003
15	1100.0	1098914.3	0.0086	0.0061	8455.9	7661.6	3.5	0.0003	0.0002
14	1100.0	1098914.3	0.0085	0.0059	8520.0	7746.7	3.5	0.0003	0.0002
13	1100.0	1098914.3	0.0083	0.0057	8596.7	7857.7	3.5	0.0003	0.0002
12	1100.0	1098914.3	0.0081	0.0055	8694.3	8005.2	3.5	0.0003	0.0002
11	1100.0	1098914.3	0.0078	0.0053	8817.6	8194.3	3.5	0.0003	0.0002
10	1100.0	1098914.3	0.0075	0.0050	8966.5	8422.4	3.5	0.0003	0.0002
9	1100.0	1098914.3	0.0072	0.0047	9135.9	8680.2	3.5	0.0002	0.0002
8	1100.0	1098914.3	0.0068	0.0044	9316.4	8952.5	3.5	0.0002	0.0002
7	1100.0	1098914.3	0.0063	0.0041	9496.1	9221.7	3.5	0.0002	0.0001
6	1100.0	1098914.3	0.0058	0.0037	9662.5	9469.8	3.5	0.0002	0.0001
5	1100.5	1099360.5	0.0052	0.0033	9804.19	9681.652	3.5	0.0002	0.0001
4	1102.2	1101145.0	0.0044	0.0028	9913.314	9846.2928	3.5	0.0001	0.0001
3	1102.2	1101145.0	0.0035	0.0022	9986.238	9958.8018	3.5	0.0001	0.0001
2	1102.2	1101145.0	0.0025	0.0015	10024.72	10020.993	3.5	0.0001	0.0000
1	1101.6	1100475.8	0.0010	0.0007	10036.46	10041.85	3.5	0.0000	0.0000

Table 3.15: Results of second order effects.

MAX COEFFICIENT		
θ_{ix}	θ_{iy}	CHECK
0.0041	0.0037	OK

3.2.9. Flexural Design of Structural Members

Reinforced concrete design engine of ETABS is used. ACI 318 – 99 code is used because of similarity with TS 500. To take into account the 2nd order effects the P-delta option of ETABS is used.

3.2.9.1. Slab Design

Slabs are designed with thickness about 16 cm and due to the most effective internal forces in end joints and middle of the slab. Clear cover concrete of slab is defined as $h_f=1.5$ cm. Slabs are modeled as rigid diagram as two-way loading so it is not necessary to consider earthquake loads. Design rules of the slab are defined in TS500 as following Eq. (3.19) –(3.22).

- Rebar Ratio for S420

$$A_{smin}=0.0035 \times 100 \times 16 = 5.6 \text{ cm}^2 \quad (3.19)$$

- Space of Rebar Rules

$$t \leq 1.5 \times h_f = 24 \text{ cm} \quad (3.20)$$

$$t \leq 20 \text{ cm for short way} \quad (3.21)$$

$$t \leq 25 \text{ cm for long way} \quad (3.22)$$

- Chosen Rebar

$\Phi 10/20(3.93)$ is chosen for long and short way.

However total minimum rebar is selected area of 7.86 cm^2 .

- Calculation Example for Slab

Table 3.16: Flexural design of slabs.

Story	kN-m/m	MIDDLE	END	MIDDLE	END	MIDDLE	END	MIDDLE	END	REQUIRED			
		MAX	MIN	K	K	ω	ω	ρ		MIDDLE	END	MIDDLE	END
50	M11	28.42	58.51	14.5	32.14286	40.5	87	0.00405	0.0087	As	As	MIN REBAR	1218
	M22	25.1	63	14.852071	32.14286	42	93	0.0042	0.0093	546	1302	MIN REBAR	1302

As it is given at Table 3.16, slabs calculation of reinforcement details are presented. Several methods of calculations can be considered for the design. However, min rebar is used for all slabs where unit is shown at Table 3.16.

- Displacement Rules

Slabs have largest displacement in the structure as it is exposed to bending moments so there are rules for displacements according to TS 500 as it is explained for part of the calculation of creep effects as following.

$$\max \Delta u \leq \frac{\max l_n = 760}{240} = 32 \text{ mm} \quad (3.23)$$

Due to explained information above, displacements of slabs under gravity loads (G+Q) are presented at Figure 3.15-3.16 to check displacements.

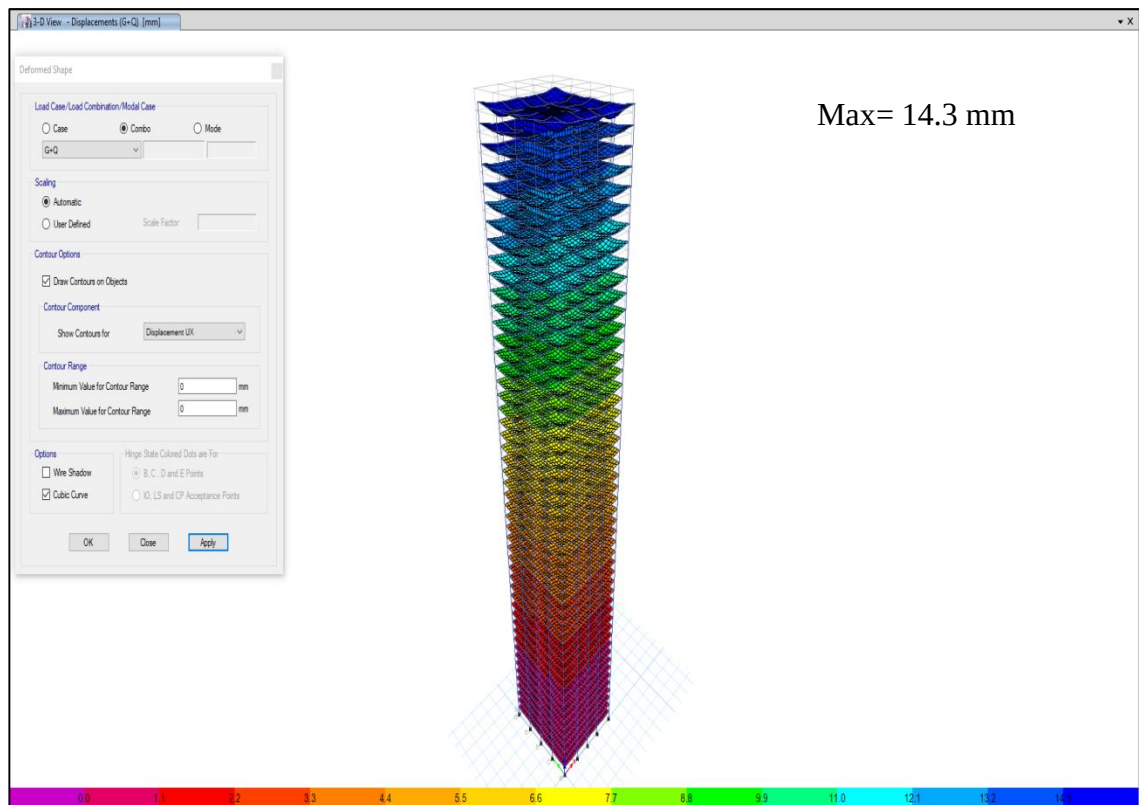


Figure 3.15: Displacements of slabs for X direction.

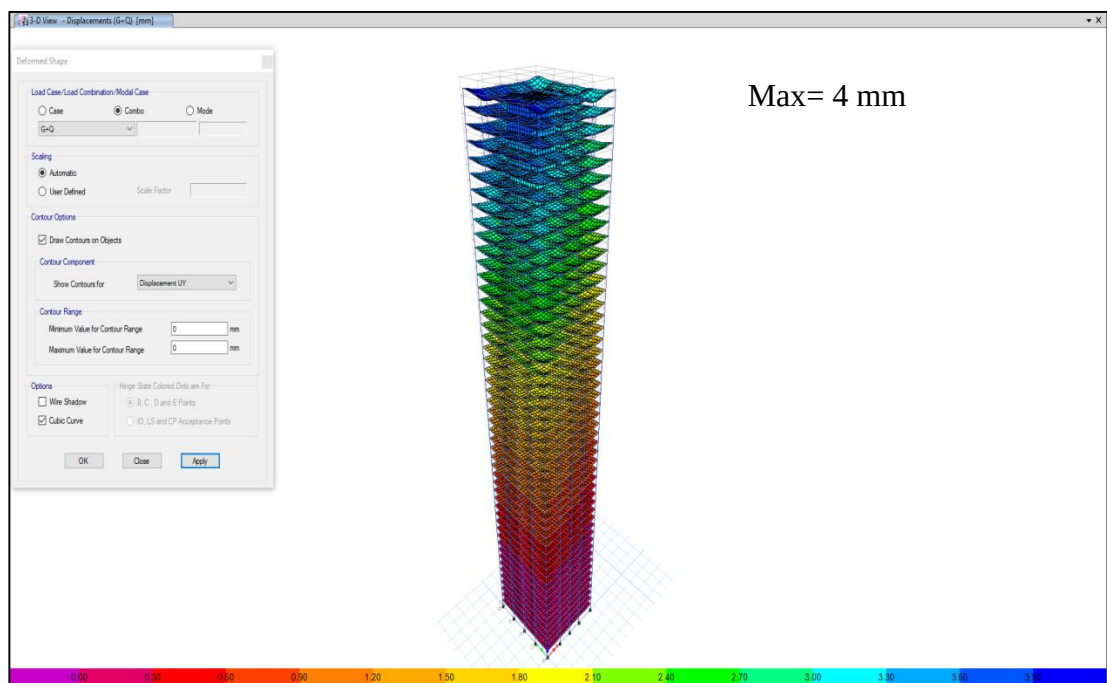


Figure 3.16: Displacements of slabs for Y direction.

It is clear that displacements are not exceeded the range defined by TS500.

3.2.9.2.Beam Design

Rules and properties for designing of beams are presented in Table 3.17-3.19.

- Cross Section Rules

Table 3.17: Rules for beam cross sections in codes.

TS 500	$b_w \geq 20 \text{ cm}$	$b_w \leq a+h_{kr}$	$h_{kr} \geq 30 \text{ cm}$	$h_{kr} \geq 3h_f$	$h_{kr} \geq L/y$	
TBSC 2016	$b_w \geq 25 \text{ cm}$	$b_w \leq a+h_{kr}$	$h_{kr} \geq 3h_f$	$h_{kr} \geq 30 \text{ cm}$	$h_{kr} \leq 3.5b_w$	$h_{kr} \leq l_n/4$

Perimeter beams are designed and enveloping rebar areas are used. Since both orthogonal axis' are similar in length, same rebar configuration is used.

Table 3.18: Configuration of beams.

f_{ctd}	1.67	Mpa
f_{ctk}	2.5	Mpa
f_{yd}	365	Mpa
f_{yk}	420	Mpa
f_{ck}	50	Mpa
f_{cd}	35	Mpa
ρ_{min}	0.00366	TS 500
ρ_{max}	0.02	
ρ_{min}	0.003571	TDY 2016
c_c	25	mm
c	40	Mm

- Calculation Example for Flexural Design Beams

Table 3.19: Flexural design of beams.

Label	Story	Section	b_w	h_w	d	Location	(-) Moment	(-) Combo	As Top	(+) Moment	(+) Combo	As Bot	ρ Top	ρ As Bot	TOP CHECK	BOT CHECK
			mm	mm	mm		kN-m		mm ²	kN-m		mm ²	mm ²	mm ²		
B1	Story50	B100/50	1000	500	460	End-I	-334.753	EQXNEN	2052	48.14	09EQYPEN	1662	0.004461	0.003613	OK.	OK.
B1	Story50	B100/50	1000	500	460	Middle	-40.7443	09EQYPEN	1662	65.0015	09EQYPEN	1662	0.003613	0.003613	OK.	OK.
B1	Story50	B100/50	1000	500	460	End-J	-372.009	EQXNEN	2288	81.4886	09EQYPEN	1662	0.004974	0.003613	OK.	OK.

Beam design procedure is considered to be the same as slabs which is explained at previous part. Rebar percentage are compared with minimum range

based on code requirements. According to computed areas of beams, longitudinal rebar are decided.

- Shear Strength for Beam

Shear for beams are checked by comparing the maximum shear probable on any section as defined in the code by using the over strength factor as,

$$V_e = V_{dy} + D \cdot V_e \quad (3.24)$$

The formulas and ranges about shear strength is shown due to codes as Table 3.20 as,

$$V_r \geq V_e = V_d \quad (3.25)$$

$$V_r = V_c + V_w \quad (3.26)$$

$$V_c = 0.8 V_{cr} \quad (3.27)$$

$$V_w = (A_{sw}/s) f_{ywd} d \quad (3.28)$$

$$V_{cr} = 0.65 f_{ctd} b_w d \quad (3.29)$$

$$V_{max} = 0.65 f_{cd} b_w d \text{ in TS500} \quad (3.30)$$

$$V_r = V_{max} = 0.85 (f_{cd}^{1/2}) b_w d \text{ in TSC 2016} \quad (3.31)$$

Table 3.20: Rules for transverse rebar details.

$V_d \leq V_{cr}$	Minimum rebar is used due to codes
$V_{cr} < V_d \leq V_{max}$	Calculation of transverse rebar is done.
$V_d > V_{max}$	Cross sections must be growth.

According to all information transverse rebar are selected and shown for one example at Table 3.21 and reinforcement details of beams are shown at Figure 3.17.

Table 3.21: Calculation of shear reinforcement details for beam.

Label	Story	Section	bw	hkr	d	Location	At	Vcr	TBSC 2016	TS500	Vc	Vw	Vr	Vd	CALCULATION	REBAR
									Vmax	Vmax						
									mm ² /m	kN					Vcr, Vmax	Asw
B1	Story50	B100/50	1000	500	460	End-I	1355.29	499.33	2764.788	3542	399.464	227.5532	627.0172	71.4422	DO NOT.	137.2603
B1	Story50	B100/50	1000	500	460	Middle	1355.29	499.33	2764.788	3542	399.464	227.5532	627.0172	93.3748	DO NOT.	137.2603
B1	Story50	B100/50	1000	500	460	End-J	1355.29	499.33	2764.788	3542	399.464	227.5532	627.0172	135.5544	DO NOT.	137.2603

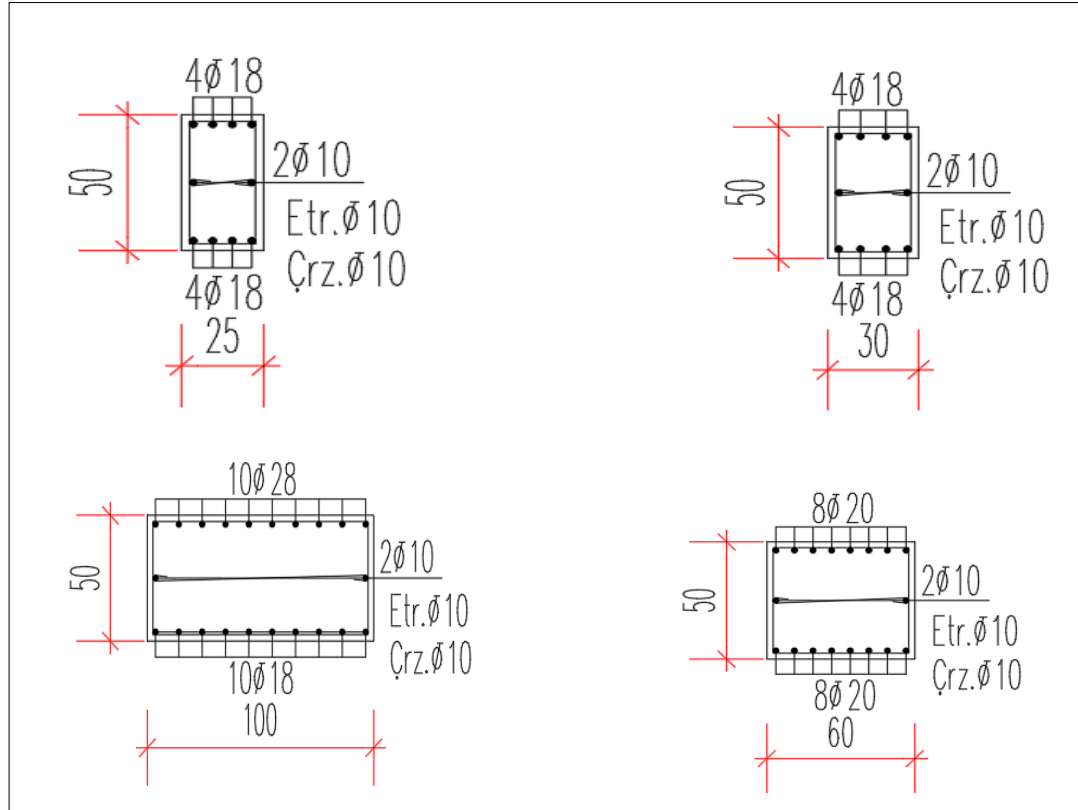


Figure 3.17: Reinforcement details of beams.

3.2.9.3. Column Design

Rules and properties for designing of columns are shown as Table 3.22-3.24.

- Cross Section Rules

Table 3.22: Rules for column cross sections in codes.

TS 500	$b, h \geq 25 \text{ cm}$	$A_c \geq N_d / 0.9 f_{cd}$
TBSC 2016	$b, h \geq 30 \text{ cm}$	$A_c \geq N_d / 0.4 f_{ck}$

The rules of columns are provided for all member used in building depending on Turkish Codes which is shown at Table 3.22.

- Reinforcement Detail Rules

Reinforcement details of longitudinal rebars are selected according to Turkish code TS500 and TBSC 2016 as it is presented Table 3.23.

Table 3.23: Rules for column reinforcement details.

Provision	TS 500		TBSC 2016	
Number	6 Φ 14 (9.24)		$\geq \Phi$ 14	
Ratio	$\rho_{min} = 0.01$	$\rho_{max}=0.04$	$\rho_{min} = 0.01$	$\rho_{max}=0.04$
Shape	rebar space ≤ 30 cm		rebar space $\leq 25 \Phi_{hoop}$	

- Calculation Example for Columns

According to all informations which are given at Table 3.23, reinforcement details are given for two column example at Table 3.24.

Table 3.24: Reinforcement details of longitudinal rebar of column.

Label	Story	Section	b	h	P	M Major	M Minor	PMM Ratio or Rebar %	As	Rebar	As
			mm	mm	kN	kN-m	kN-m		cm2	Φ	cm2
C1	Story50	120*120	1200	1200	411.4482	-180.814	-180.914	0.01	144	24 Φ 28	147.84
C2	Story50	150*150	1500	1500	717.416	-372.548	-36.7136	0.01	225	32 Φ 30	226.24

- Column Shear Design

For each typical column, Section 7.3.7 [TBSC, 2017] shear demands will be evaluated. The maximum shear is assumed to be the minimum of either $V_e = D*V_{ex, y} + V_d$ or $V_e=(M_{pi}+M_{pj}) / l_n$ where M_{pi} and M_{pj} are column moment capacities.

For each column, cross section, the minimum shear reinforcement is chosen according to maximum axial load. All columns are defined as compression members as controlled by Eq. (3.31) –(3.32), $N_d > 0.20A_c f_{ck}$. According to given formula, areas of shear reinforcement details are shown at Table 3.25.

$$A_{sh} \geq 0.30 s_{bk} [(A_c / A_{ck}) - 1] (f_{ck} / f_{ywk}) \quad (3.31)$$

$$A_{sh} \geq 0.075 s_{bk} (f_{ck} / f_{ywk}) \quad (3.32)$$

Given Table 3.25 presented example for the shear reinforcing chosen according to the formulations defined by the code.

Table 3.25: Shear reinforcement details of columns.

Label	Story	Section	b	h	b _k	h _k	p	TENSION	MIN As	MIN As	Rebar
			mm	mm	mm	mm	kN		mm ²	mm ²	Φ
C1	Story50	120*120	1200	1200	1150	1150	411.4482	NOT OK	364.9068	1026.786	Φ 14/100
C2	Story50	150*150	1500	1500	1450	1450	717.416	NOT OK	363.3005	1294.643	Φ 14/100

- Shear Strenght Checks

This part is the same as beams which is explained in previous part. Some examples of column are presented at table below,

Table 3.26: Shear strenght checks of column.

Label	Story	Section	V Major	At Major	V Minor	CHECK	V _{cr}	TDY	TS500	V _c	V _w	V _r	V _d	CHECK	REBAR CALCULATION
			kN	mm ² /m	kN			V _{max}	V _{max}					V _r ,V _e	V _{cr} ,V _{max}
C1	Story50	120*120	266.3624	1626.35	231.8985	OK	1497.99	6939.562	10626	1198.392	4309.933	5508.325	266.3624	OK	MIN REBAR
C2	Story50	150*150	500.7732	2032.93	584.8141	OK	2360.963	10937.35	16747.5	1888.77	6851.897	8740.667	500.7732	OK	MIN REBAR

- Strong Column Weak Beam Checks

Every joint in frames are investigated for every component of earthquake by formula based on TBSC 2016 Chapter 7. The axial load which is minimum on the cross section is giving the minimum column capacity. Axial loads are derived from ETABS load combinations. Two calculations of columns and beams are given for an example at Table 3.27.

Table 3.27: Strong column, weak beam.

Story	Design Section	B/C Major Combo	B/C Major Ratio	B/C Minor Combo	B/C Minor Ratio	CHECK	
Story50	120*120	SPECX+	0.115	SPECX+	0.115	OK	OK
Story50	150*150	SPECX+	0.056	SPECX+	0.132	OK	OK

According to Table 3.27, rules for story column and weak beam is proposed for TSC 2016 whic is defined as Eq. (3.33).

$$M_{ra} + M_{r\bar{u}} \geq 1.2 M_{ri} + M_{rj} \quad (3.33)$$

- Shear Checks of Beam-Column Joints

Shear force in beam-column joints along the earthquake direction considered should be calculated by formula as Eq. (3.34).

$$V_e = 1.25 f_{yk} (A_{s1} + A_{s2}) - V_{kol} \quad (3.34)$$

In confined joints are given Eq. (3.35)

$$V_e \leq 1.7 b_j h f_{ck}^{1/2} \quad (3.35)$$

In unconfined joints are given Eq. (3.36).

$$V_e \leq b_j h f_{cd}^{1/2} \quad (3.36)$$

According to all information about joints of Beam-Column, checking of shear strength is calculated and presented for every story at Table 3.28 and reinforcement details of columns are shown at Figure 3.18. However, all joints are defined as unconfined.

Table 3.28: Shear checks in joints.

[illegible]

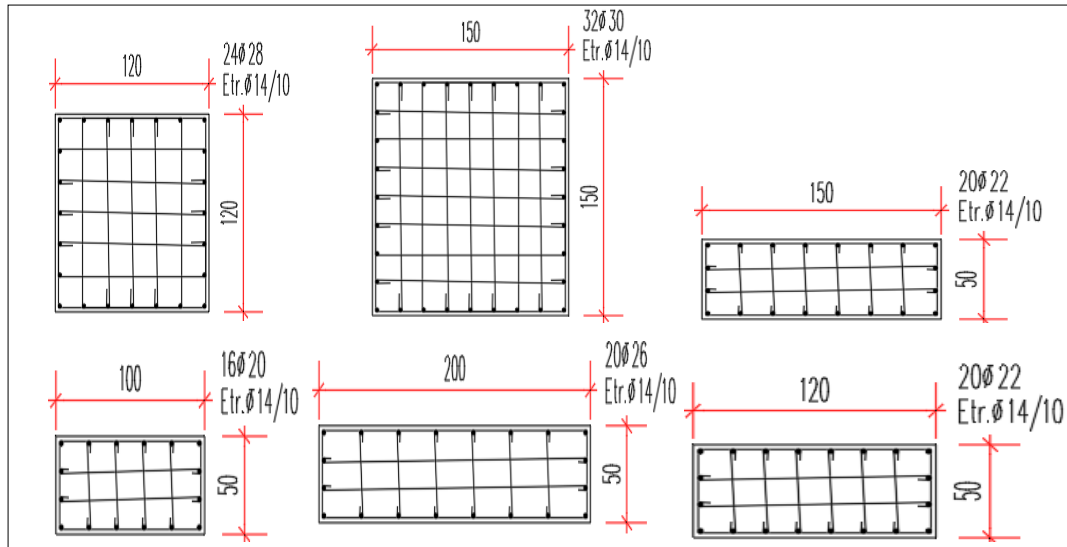


Figure 3.18: Reinforcement details of columns.

3.2.9.4.Wall Contribution

RC shear wall is modeled as shell element in ETABS. Piers must be defined in shells to obtain axial loads and moments in shear wall. For this study shear wall designed were given at Figure 3.19.

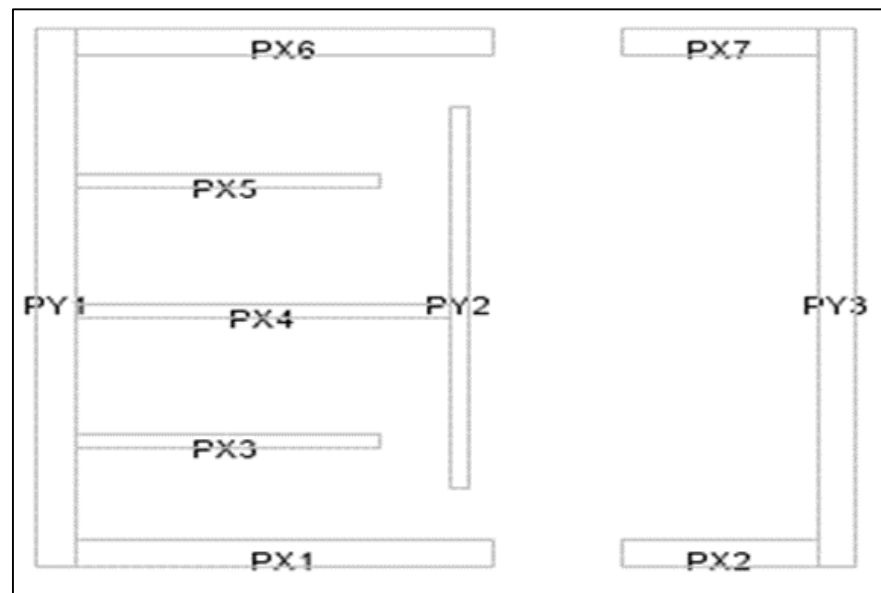


Figure 3.19-Model of shear wall.

- Cross Section Rules

Provision of shear walls are defined in TBSC 2016 by the formula,

$$A_c \geq N_{dm} / (0.25 f_{ck}) \quad (3.37)$$

Cross section of shear walls are checked by given formula. All section are provided. Two of pier sections are given at Table 3.29 as an example,

Table 3.29: Cross sections of shear walls.

Story	Pier Label	H	B	P	M2	M3	CHECK
		mm	mm	kN	kN-m	kN-m	SECTION
Story50	PX1	6602.6	600	-907.1958	220.4627	-23.9398	OK
Story49	PX1	6602.6	600	-1682.229	135.1628	-121.462	OK

Minimum reinforcement is found at each story level. Selection of reinforcing pattern is done according to code regulations.

At each pier shear forces are obtained and 2.5 times the shear forces from linear analysis is used. Calculations are given in Table 3.30-3.32.

- Flexural Design of Shear Wall

According to TBSC 2016 in 7.6.2.1, two webs are considered in shear wall due to $H_w/l_w < 2$

Critical wall height is taken $2 l_w \geq H_{cr} = 35 m \geq \max [l_w; H_w/6]$. Dimension of end zones of shear wall are defined in along critical height and out of critical height is calculated and reuirements for reinforcement details is explained as defined in TBSC 2016,

Along critical height is defined by Eq. (3.38) - (3.39).

$$l_u \geq 2b_w \quad (3.38)$$

$$l_u \geq 0.2l_w \quad (3.39)$$

Out of Critical Height is defined by Eq. (3.40) - (3.41).

$$l_u \geq b_w \quad (3.40)$$

$$l_u \geq 0.1 l_w \quad (3.41)$$

Web reinforcement requirements are defined by Eq. (3.42).

$$A_{sweb} \geq 0.0025(l_w - (2l_{end})) b_w \quad (3.42)$$

Reinforcement requirements at wall end zones are defined by Eq. (3.43) – (3.44).

$$A_s \geq 0.002 A_{end} \text{ along critical height} \quad (3.43)$$

$$A_s \geq 0.001 A_{end} \text{ out of critical height} \quad (3.44)$$

According to all informations flexural design of shear walls are completed. Shear wall flexural design calculations are shown at Table 3.30-3.31 according to requirements for an example.

Table 3.30: Properties of end zones.

Story	Pier Label	H	B	CHECK	HW/IW	CHECK	END ZONE of SW				
							CRITICAL WALL HEIGHT			OUT of CRITICAL WALL HEIGHT	
							0.20xIW	2xbW	SONUÇ(mm)	0.10xIW	RESULTS(mm)
Story50	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000
Story49	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000
Story48	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000
Story47	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000
Story46	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000
Story45	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000
Story44	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000
Story43	PX1	6602.6	600	OK	26.50471	>2.0	1320.52	1200	-	660.26	1000

Table 3.31: Reinforcement detail of shear walls.

Story	Pier Label	P	M2	M3	WEB REBAR	END ZONE REBAR		CHOSEN REBAR			
						CRITICAL	OUT of CRITICAL	END ZONE		WEB	
						Au,min(cm2)	Au,min(mm2)	φ	AS(mm2)	REQUIRED AS(mm2)	CHOSEN AS(mm2)
Story50	PX1	-907.196	220.4627	-23.9398	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840
Story49	PX1	-1682.23	135.1628	-121.462	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840
Story48	PX1	-2454.71	155.1873	-148.405	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840
Story47	PX1	-3226.55	150.3671	-162.652	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840
Story46	PX1	-3998.38	151.7	-167.092	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840
Story45	PX1	-4770.48	151.635	-167.214	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840
Story44	PX1	-5542.9	152.0043	-164.865	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840
Story43	PX1	-6315.63	152.3558	-161.132	6903.9	-	3961.56	12φ22	4561	6531.368	18φ22 6840

- Shear Design in Shear Walls

In shear design of shear wall, shear forces of earthquakes multiply by 1.25D to obtaine and decide shear reinforments of shear walls according to TBSC 2016. Requirements of shear provision are explained as it is given Eq. (3.45) – (3.48).

$$V_r = A_{ch} (0.65 f_{ctd} + \rho_{sh} f_{ywd}) \quad (3.45)$$

$$V_e \leq V_r \quad (3.46)$$

$$V_e \leq 0.85 A_{ch} f_{ck}^{1/2} \quad (3.47)$$

$$V_e \leq \min [0.2 f_{ck} A_c, (3.3 + 0.08 f_{ck}) A_c, 10 A_c] \quad (3.48)$$

According to provisions which are defined in TBSC 2016, some example of table is shown as Table 3.32 and reinforcement details of shear wall are given at Figure 3.20.

Table 3.32: Shear reinforcement detail and checks of shear walls.

Vcr	Vmax	Vc	Vw	Vr	Vd	REBAR CALCULATION	CALCULATED				WEB CHECK	END ZONE CHECK
							WEB	END	WEB	END		
							Asw(min)	Asw(min)	p	Asw		
kN	kN	kN	kN	kN	kN	Vcr,Vmax	mm2	mm2		mm2		
4300.273	30504.012	3440.218704	13327.02	16767.24	4300.273	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK
4300.273	30504.012	3440.218704	13327.02	16767.24	16767.24	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK
4300.273	30504.012	3440.218704	13327.02	16767.24	16767.24	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK
4300.273	30504.012	3440.218704	13327.02	16767.24	16767.24	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK
4300.273	30504.012	3440.218704	13327.02	16767.24	16767.24	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK
4300.273	30504.012	3440.218704	13327.02	16767.24	16767.24	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK
4300.273	30504.012	3440.218704	13327.02	16767.24	16767.24	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK
4300.273	30504.012	3440.218704	13327.02	16767.24	16767.24	NEEDED	150	309.5238	0.002617994	392.6990817	OK	OK

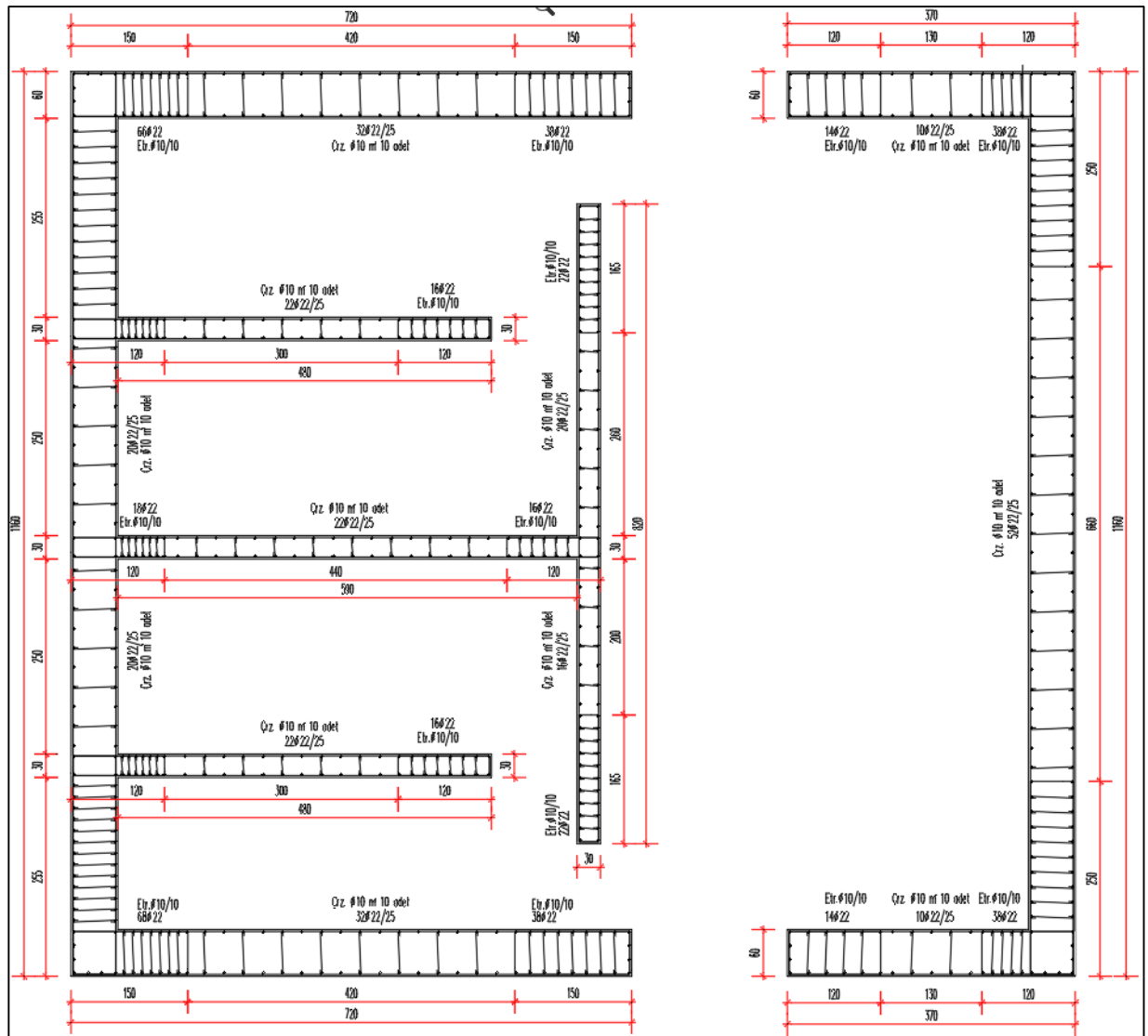


Figure 3.20: Reinforcement details for shear wall.

4.PERFORMANCE BASED DESIGN

The general aspects of displacement based design for the given building is presented. Chapter 13 of the TBSC 2016 is followed. A third stage collapse prevention analysis will be made using DD1 level earthquake. This level has a probability of %2 exceedance in 50 years. A non-linear dynamic analysis will be carried out by using Perform 3D software. All structural members will be investigated for their damage levels as defined in the TBSC 2016.

However, the model will be prepared by nonlinear materials to obtain behaviour of seismic performance and performance level due to new Turkish Building Seismic Code. This part presents the nonlinear time history analysis using selected scaled ground motions.

The seismic mass is calculated as “DL + SDL + 0.3LL” where DL and SDL are self-weight and superimposed dead loads while LL is the live load. At the calculated first stage, the gravity loading will be calculated according to this seismic weight component.

Nonlinear Time History Analysis (NTHA) is a 3-D dynamic analysis procedure performed in the time domain under appropriately selected and scaled ground motion records, where nonlinear properties of structural elements are appropriately taken into account in accordance with the modeling techniques described above. The procedure is accepted to be the most advanced and precise procedure to obtain the inelastic seismic demand quantities. Nonlinear Time History Analysis (NTHA) is based on direct, step-by-step integration of coupled equations of motion of the multi degree- of-freedom (MDOF) structural system in time domain.

In this study two model of buildings are designed and compared with each other the models can be defined as typical high-rise building and high-rise building with BRBFs. Buckling Restrained Braced Frames are defined in typical high-rise building without any changing of cross sections and reinforcement details as it was calculated and shown at Strength Based Design in Chapter 2. Design of BRBFs is explained in this chapter for the selected the comparison between typical tall building and tall building with BRBFs.

NTHA provides seismic demand quantities in terms of inelastic deformation components of ductile elements and internal force components corresponding brittle

behavior modes. These demand quantities are then checked against the corresponding performance criteria, that are specified in TBSC 2016.

In this study, the selected and scaled ground motion records for DD1 level are already prepared by linear scaling methods which are represented by Fahjan (2007).

4.1.Non-Linear Modeling

Estimation of seismic performance and risk involves the estimation of real seismic behavior of structures during earthquakes, which is achieved through appropriate nonlinear modeling of structural systems. Concentrated (lumped) plasticity and distributed plasticity are the two main approaches used for nonlinear modeling. In the present work, concentrated (lumped) plasticity approach is used in frame elements, i.e., in beams and columns, while distributed plasticity approach is used in walls in the form of fiber modeling. Both modeling options are available in the computer software used in the present work.

In accordance with the common engineering practice, rigid-diaphragm assumption is made for the in-plane stiffness of slabs. Stiffness contributions of infill walls and other nonstructural elements are neglected for being on the safe side.

In nonlinear performance assessment, “expected strength” values are used instead of “characteristic strength” values used in the regular design process. Accordingly, expected strengths of used materials are given at Table 4.1-4.2.

Table 4.1: Expected material strengths.

Material	Expected Str.
Reinforcing Steel	$f_{ye}=1.2 f_{yk}$
Concrete	$f_{ce}=1.3 f_{ck}$

Table 4.2: Properties of materials.

Concrete	C50	
Steel	S420	
f_{sy}	504	Mpa
f_{c0}	65	Mpa
E_{ce}	40202.34	Mpa
ϵ_{su}	0.08	
ϵ_{sy}	0.0021	
ϵ_{sh}	0.008	
f_{su}	660	Mpa
E_s	200000	Mpa

4.1.1.Lumped Plasticity Approach

The concentrated (lumped) plasticity approach is represented by the simplest model based on plastic hinges, which are zero-length elements through which the nonlinear behavior is assumed to be concentrated or lumped at predetermined sections. A typical plastic hinge is ideally located at the center of a plasticized zone called plastic hinge length, which is generally defined at each end of a clear length of a beam or column. A one-component plastic hinge model with or without strain hardening can be appropriately used to characterize a bi-linear moment-curvature relationship. Plastic hinge concept is ideally suited to the piecewise linear representation of concentrated nonlinear response. Linear behavior is assumed in between the predetermined plastic hinge sections as well as temporally in between the formation of two consecutive plastic hinges. As part of a piecewise linearization process, the yield surfaces of plastic hinge sections may be appropriately linearized, i.e., they may be represented by finite number of yield lines and yield planes in two- and three-dimensional hinge models, i.e., in the so-called PM hinges and PMM hinges, respectively.

In Practice, Oriented Nonlinear Analysis procedures based on pushover analysis, modeling of backbone curves of typical moment-rotation relationships of plastic sections are sufficient for nonlinear modeling which is given at Figure 4.1-4.2. In the Nonlinear Time History Analysis (NTHA), however, cyclic hysteretic behavior of plastic hinge is additionally required to be defined. Standard bi-linear model with parallel loading and unloading branches, peak-oriented model with or

without pinching and Takeda type models are the most well-known hinge hysteretic models. In this study, the reinforced concrete beams with standard flexural reinforcement will be modelled with a bilinear plastic hysteresis model with cyclic degradation consisted with the test performed by Popov et-al [PEER/ATC, 2010]. The reinforced concrete columns are modelled with PMM hinges.

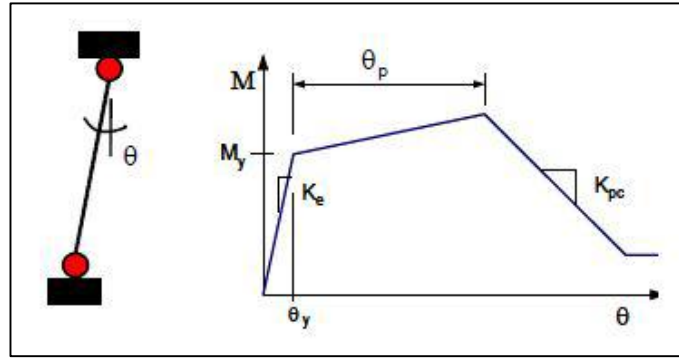


Figure 4.1: Plastic hinge model and moment-rotation relationship.

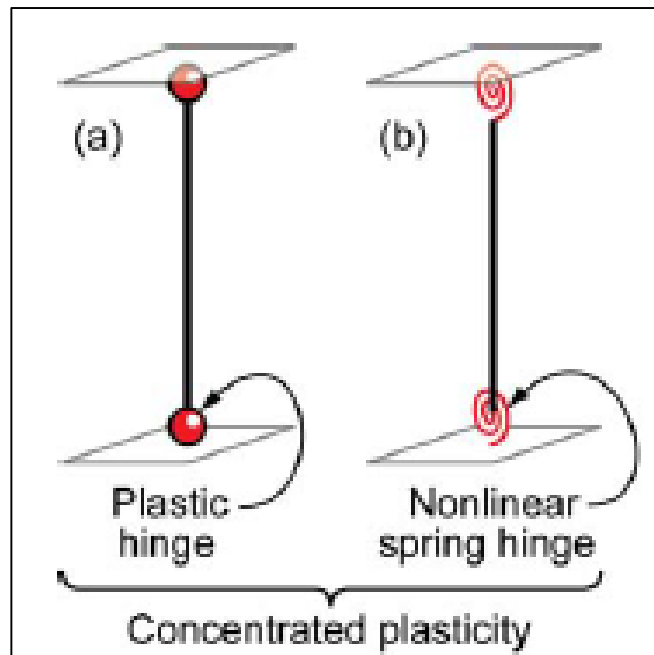


Figure 4.2: Concentrated plasticity approach idealization.

In nonlinear analysis, initial bending stiffness of reinforced concrete frame elements are considered on the basis of effective section properties. The used effective section stiffness properties are given in Table 4.3.

Table 4.3: Effective section stiffness modifiers in TBSC 2016.

Betonarme Taşıyıcı Sistem Elemanı	Etkin Kesit Rijitliği Çarpanı	
<i>Perde – Döşeme (Düzlem İçi)</i>	<i>Eksenel</i>	<i>Kayma</i>
Perde	0.50	0.50
Bodrum perdesi	0.80	0.50
Döşeme	0.25	0.25
<i>Perde – Döşeme (Düzlem Dışı)</i>	<i>Eğilme</i>	<i>Kesme</i>
Perde	0.25	1.00
Bodrum perdesi	0.50	1.00
Döşeme	0.25	1.00
<i>Çubuk eleman</i>	<i>Eğilme</i>	<i>Kesme</i>
Bağ kirişi	0.15	1.00
Çerçeve kirişi	0.35	1.00
Çerçeve kolonu	0.70	1.00
Perde (eşdeğer çubuk)	0.50	1.00

4.1.2.Distributed Plasticity Approach

The fiber model is the most popular distributed plasticity model (Figure 4.3) being used for the nonlinear modeling, where cross section of the structural element is subdivided into concrete fibers and steel fibers. Since the response is obtained in terms of uniaxial deformation of the fibers, the load versus deformation response of a fiber model is defined in terms of uniaxial stress-strain relations specified for concrete and reinforcement given in Figure 4.4. Various material models are available in the literature.

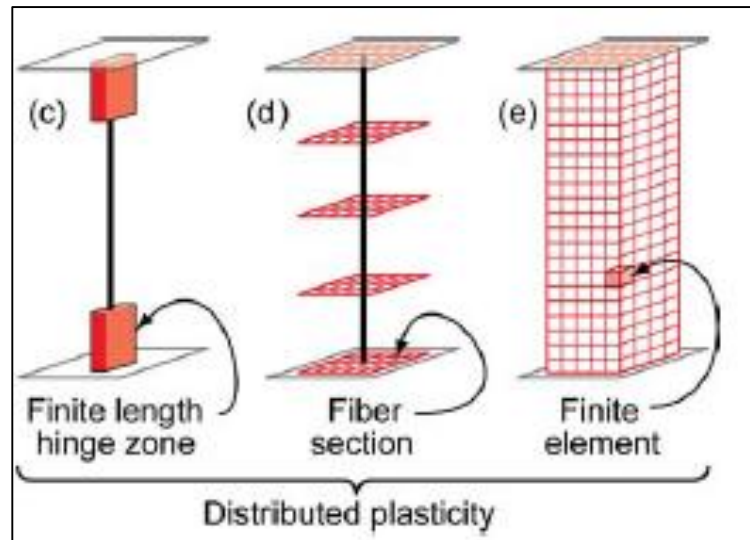


Figure 4.3: Distributed plasticity idealization.

Although fiber model is more advanced compared to plastic hinge model, its use in beams and columns is not warranted from practical viewpoint. In such elements, abundance of test data regarding stiffness modeling and rotation capacities leads instead to a wider use of the plastic hinge modeling. Fiber model, however, is more appropriate in flexural walls of rectangular and U or L shapes in plan. Acceptance criteria may be specified in terms of concrete and steel strain capacities or compatible plastic hinge rotations [PEER/TBI,2010]

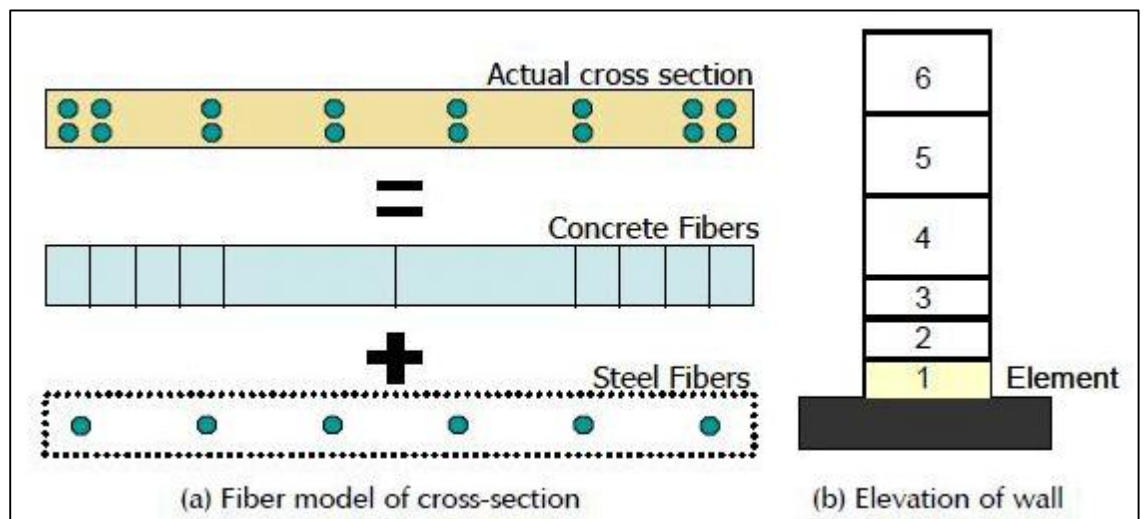


Figure 4.4: Fiber modeling of reinforced concrete shear walls.

4.1.3.Designing and Modeling of Critical Elements

In this part BRBFs are explained for designing and modeling which are called critical elements in buildings. BRBFs are designed in tall building without changing the design of tall building frames which were designed due to Chapter 3.

However, the reason for not changing is that the structural behaviour of the two models can be compared easily against earthquakes.

4.1.3.1.Designing of Buckling Restrained Braces

In this part, designing and modeling of BRBF of are explained. Buckling Restrained Braces are designed by MCE levels DD-1 earthquake. In extreme events (at the MCE level) the building should withstand shaking without collapse so drifts of typical tall building without BRB is checked by DD-1 Earthquake to decide the usage of BRBFs as shown in Figure 4.7-4.8.

Buckling-Restrained Braced Frames (BRBFs) are a relatively new type of concentrically braced system characterized (Figure 4.5) by the use of braces that yield inelastically both in tension and compression at their adjusted strengths [Clark et al., 1999]. The most basic properties of the BRBs are that they have a symmetrical hysteresis in tension and compression due to the restrained of buckling, and have high energy capacity with exhibiting ductile behavior which is given at Figure 4.6. The provision of ductile behavior in the buildings is considered as an important criterion in the seismic design of the structure.

The buckling of the braces under horizontal loads, such as earthquake loads, has led engineers to develop structural systems that is not buckled under pressure and have smooth hysteretic behavior.

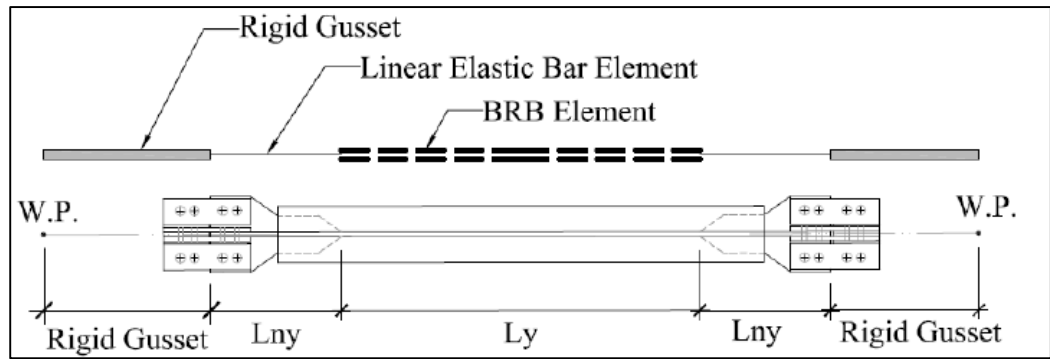


Figure 4.5: Diagram of Buckling Restrained Brace.

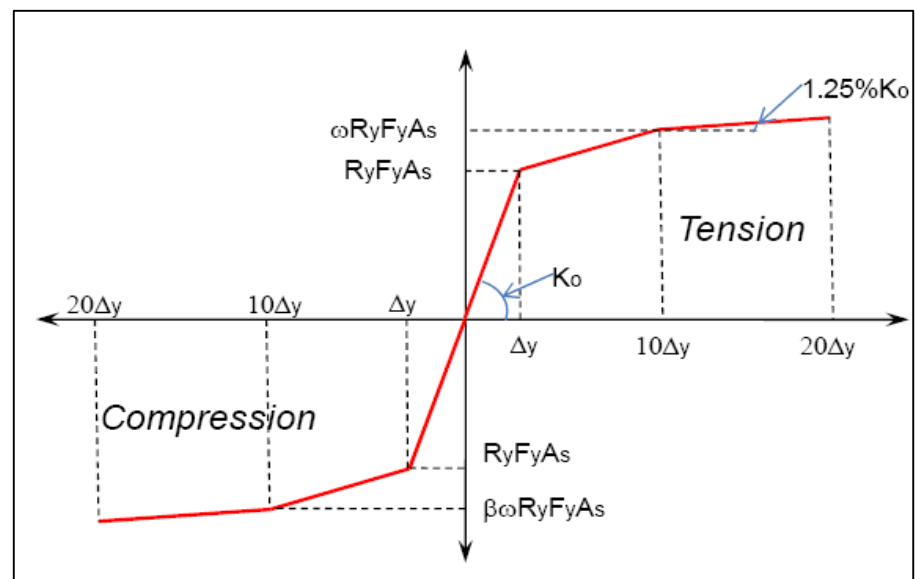


Figure 4.6: Properties of BRBs.

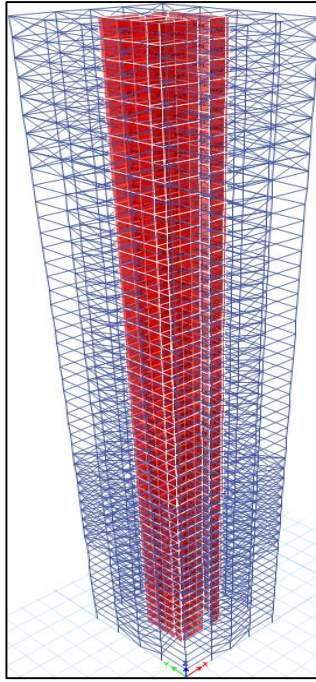


Figure 4.7: Tall Building with BRBFs.

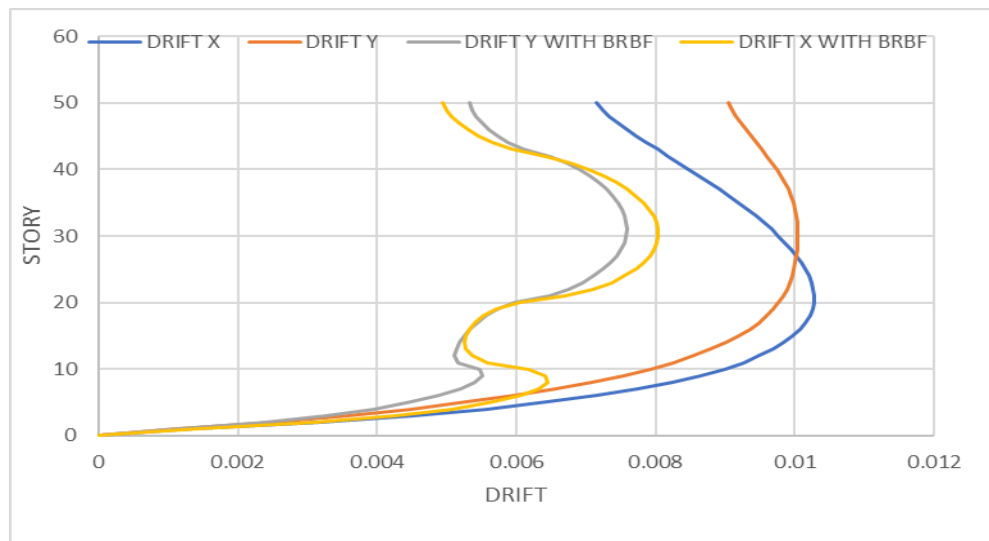


Figure 4.8: Drifts in DD-1 earthquake level in X and Y direction.

4.1.3.2. Modelling of Buckling Restrained Braces

BRB is modeled using Perform 3D BRB inelastic element in series with an elastic spring that models the end attachment of the braces. The backbone curve of the three types of BRBs were modeled as it is shown at Table 4.4-4.6.

Table 4.4: Properties of BRBs.

F_y	262	Mpa
R_y	1.3	
E	200000	Mpa
ω	1.25	
β	1.1	

Table 4.5: Used BRBFs in building.

TYPE	COMPONENT		L(mm)	0.4*BRB LENGHT	0.3*BRB LENGHT	Asy(mm ²)
	X(mm)	Z(mm)				
1	7600	3500	8367.198	5857.038501	2510.159357	25000
2	7200	3500	8005.623	5603.936117	2401.686907	25000
3	6000	3500	6946.222	4862.355396	2083.866598	25000

Table 4.6: Initial stiffnesses of BRBFs.

TYPE	K ₀	1.25K ₀	PERFORM	
			0.95K ₀	
1	853673.7	1067092	810990.1	kN/m
2	892230	1115288	847618.5	kN/m
3	1028308	1285385	976892.8	kN/m

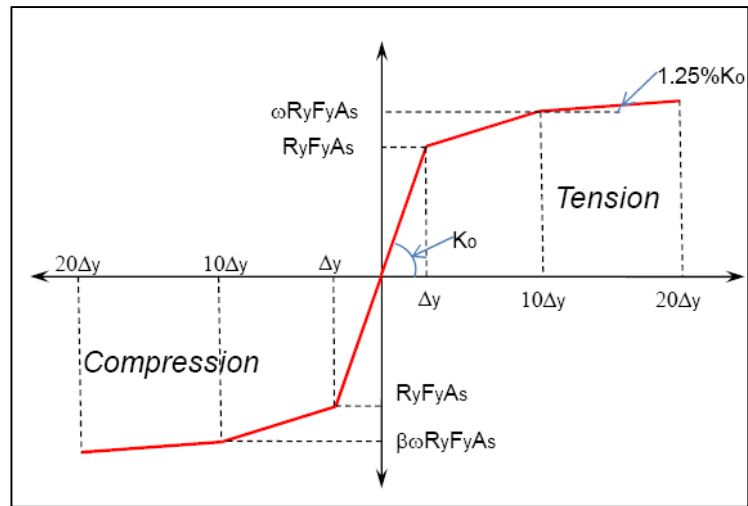


Figure 4.9: Presumed backbone curve for BRB in code.

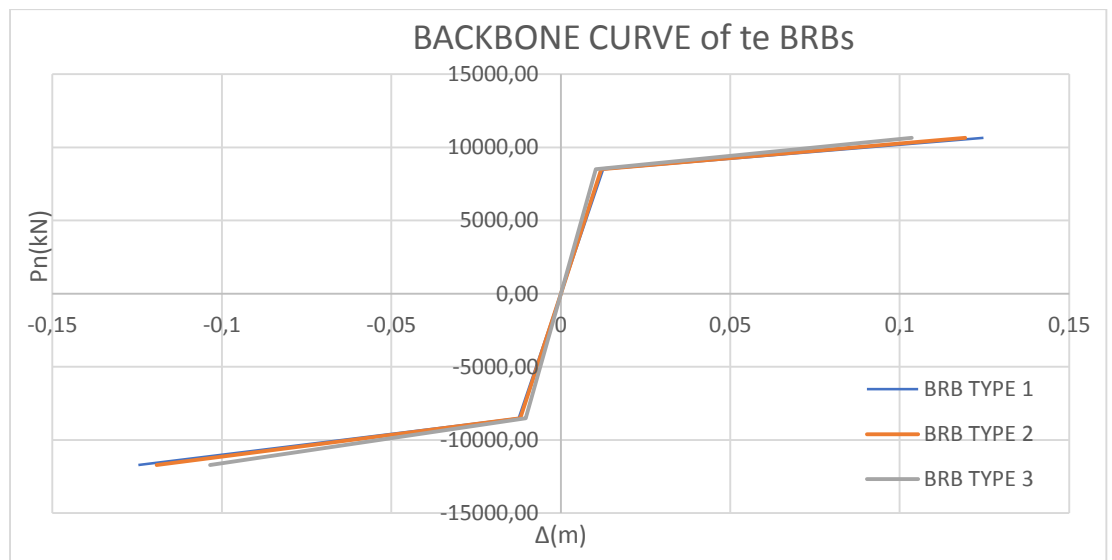


Figure 4.10: Backbone curve of BRB in used building.

According to Table 4.4-4.6 and Figure 4.9-4.10, Buckling Restrained Braces are designed and defined in PERFORM 3D. At the results section, assessment will be compared between two types of buildings.

4.1.4. Performance Acceptance Criteria

Acceptance criteria for plastic hinge response is generally defined in terms of plastic rotation capacities, which are specified in TBSC 2016. On the other hand,

concrete compressive strain and rebar steel strain capacities have also been specified in TBSC 2016 as acceptance criteria, which are to be compared with strain demands obtained from fiber analysis or from plastic hinge rotation demands.

In this study, the plastic rotations for beam and column hinges are compared with the limits given in TBSC 2016 while for fiber sections used in shear walls strains are checked.

The plastic rotation limits and strain limits will be presented later in this report.

4.1.5. Material Properties

The material properties used in the analysis are given in detail.

4.1.5.1. Concrete

The cylindrical 28 days concrete strength is chosen as 50 MPa. In the displacement based design expected values are used.

- | | |
|---------------------------------------|-----------------------------|
| • Characteristic compression strength | $f_{ck} = 65 \text{ MPa}$ |
| • Characteristic tension strength | $f_{ctk} = 4.5 \text{ MPa}$ |
| • Elasticity Modulus | $E = 40202 \text{ MPa}$ |
| • Shear Modulus | $G = 18.230 \text{ MPa}$ |
| • Weight density | $w = 25 \text{ kN/m}^3$ |

• Confined and Unconfined Concrete Definitions

Concrete compression strength increases with the reinforcing steel confinement ratio in a cross section. To consider real behavior of each cross section, confined and unconfined concrete models are used. As given in TBSC 2016, Mander equations are used. To calculate the concrete strengths Section Designer wizard of Sap2000 and Excel is used. Every section is modeled and confined concrete models are idealized to Perform 3D.

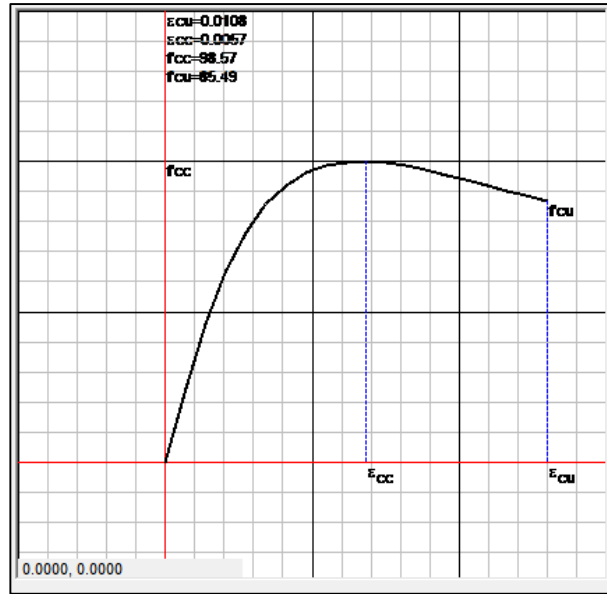


Figure 4.11: Mander confined concrete model for column in section designer.

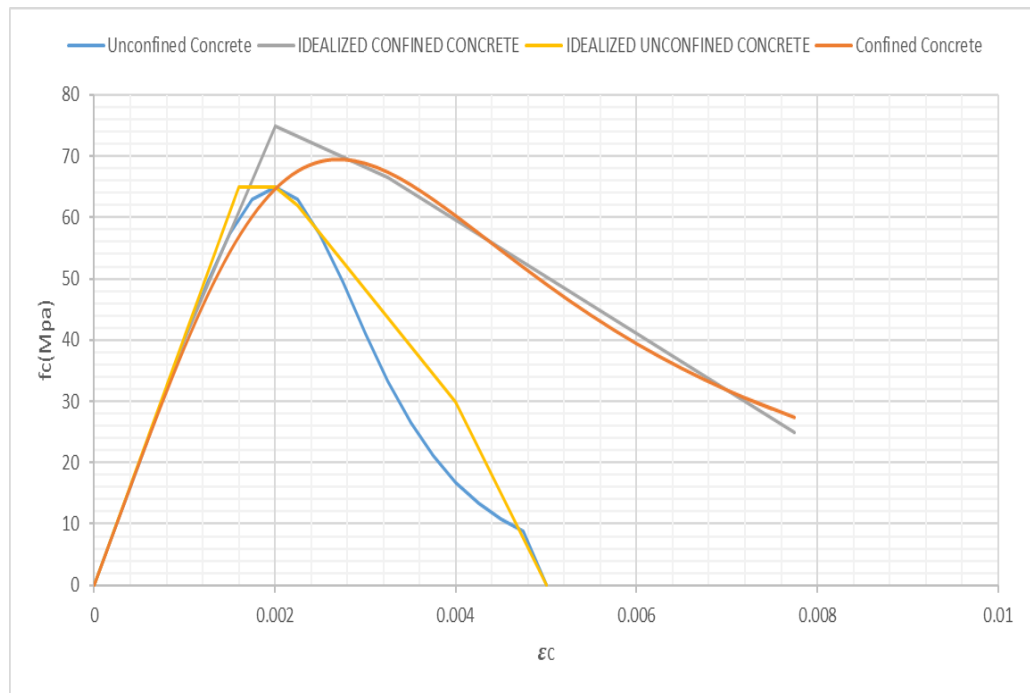


Figure 4.12: Concrete model for shear wall.

To define concrete model in PERFORM 3D, idealization of model must be defined as it is shown at Figure 4.11-4.14.

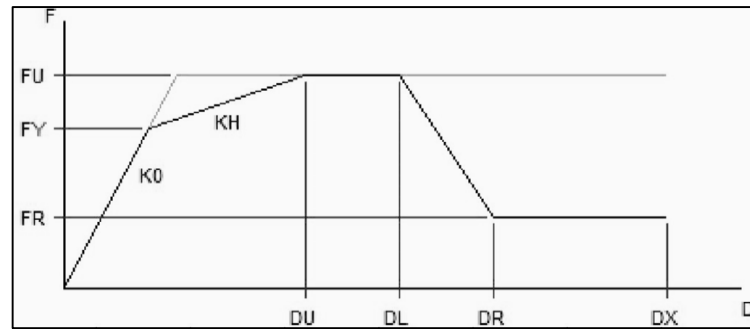


Figure 4.13: Idealization in Perform 3D.

To be able to implement the concrete material properties to Perform 3D, all confined section properties of columns and shear walls are calculated and taken from Section Designer in SAP2000 and Excel, respectively, processed and inputs are given to Perform 3D.

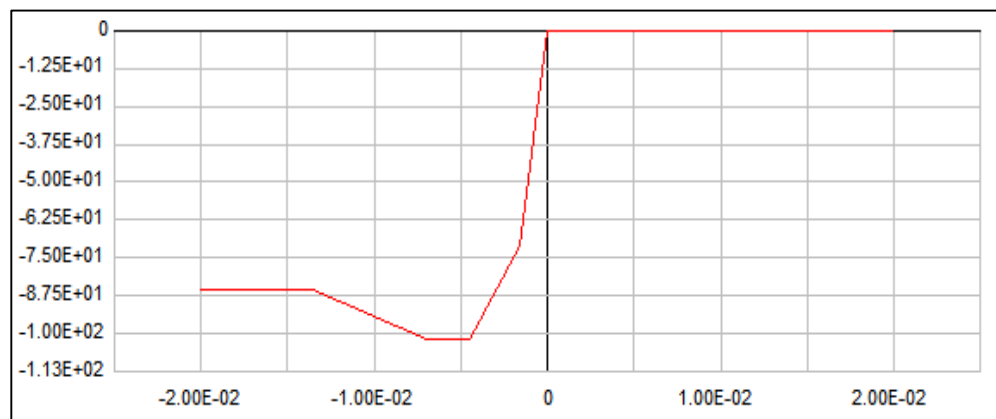


Figure 4.14: Implementation of concrete stress-strain relationship in Perform 3D.

4.1.5.2.Reinforcing Steel

The reinforcement steel will be characterized unless stated otherwise with a minimum yield strength of 420 MPa and a minimum fracture strength of 550 MPa. The minimum fracture strain should be higher than %10 which should be equivalent to BS420C.

- Characteristic yield strength $f_{yk}=420$ MPa
- Characteristic fracture strength $f_{su}=550$ MPa
- Elasticity modulus $E=200000$ MPa
- Expected yield strength $f_{ye}=1.2 \times 420 = 504$ MPa
- Expected fracture strength $f_{se}=1.2 \times 550 = 660$ MPa

Typical yield and fracture limit strains are $\varepsilon_y = 0.008$ and $\varepsilon_u = 0.08$ respectively.

The idealized stress strain relationship is defined in TBSC 2016 as in Figure 4.15.

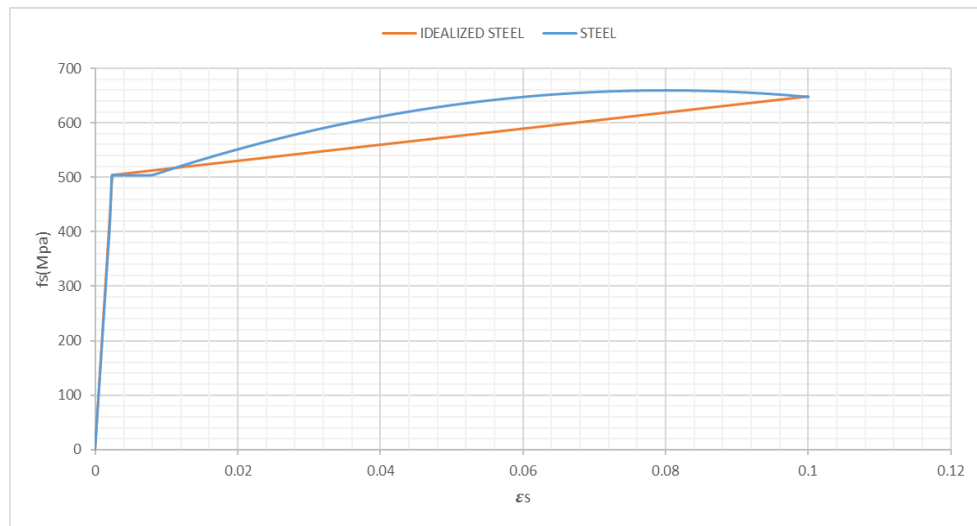


Figure 4.15: Idealized stress-strain curve for reinforcing steel in TBSC 2016.

- $\varepsilon_{sy} = 0.0021$
- $\varepsilon_{sh} = 0.008$
- $\varepsilon_{su} = 0.08$

However, for simplification an elasto perfect plastic model is used for reinforcing steel in Perform 3D. Strain capacities due to performance limit checks are marked on the graph in Figure 4.16.

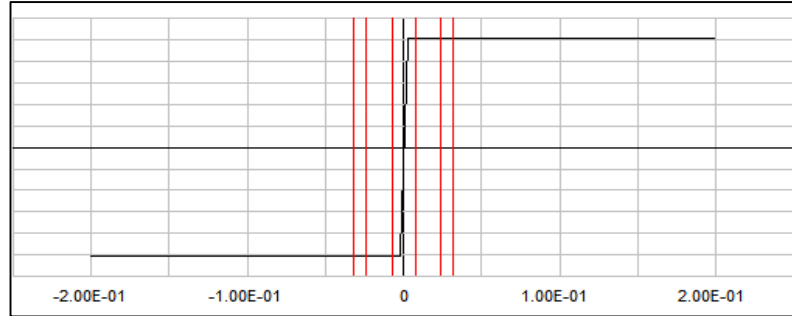


Figure 4.16: Stress strain relationship of BS420C reinforcing steel in Perform 3D.

4.2.Seismic Hazard

Ground motion records were used in assessment phases of this research for the purpose of nonlinear dynamic analysis of the model buildings which are typical and BRBFs of tall building. In this research, various implementations of the spectral matching and amplitude scaling methods for developing the ground motions were used which are explained in the following issues [Fahjan, 2008].

The critical damping ratio is taken as %2.5 as defined in TBSC 2016 Section 13.6.4.

According to TBSC 2016, while assessment of this building 11 pairs of ground motions must be selected but for this study 7 pairs of ground motions are selected whose spectra matched to requires in target spectrum in Figure 4.17 which are DD-3 and DD-1 (MCE) earthquakes for the location of tall building as it is explained in the previous chapters. In this part ground motions are given for DD-1 target spectrum at Table 4.7&Figure 4.18-4.26.

Table 4.7: Selected and matched ground motions for target spectrums.

EARTHQUAKE	YEAR	STATION	Mw
Kocaeli, Turkey	1999	Goynuk	7.51
Hector Mine	1999	Hector	7.13
Landers	1992	Joshua Tree	7.28
Hector Mine	1999	Joshua Tree	7.13
Duzce, Turkey	1999	Lamont 1061	7.14
Kocaeli, Turkey	1999	Arcelik	7.51
Landers	1992	Barstow	7.28

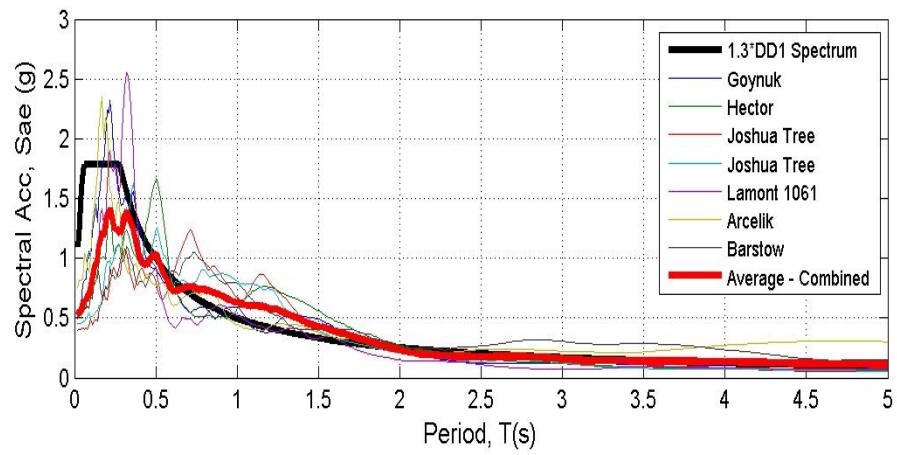


Figure 4.17: Response matching of used ground motion records.

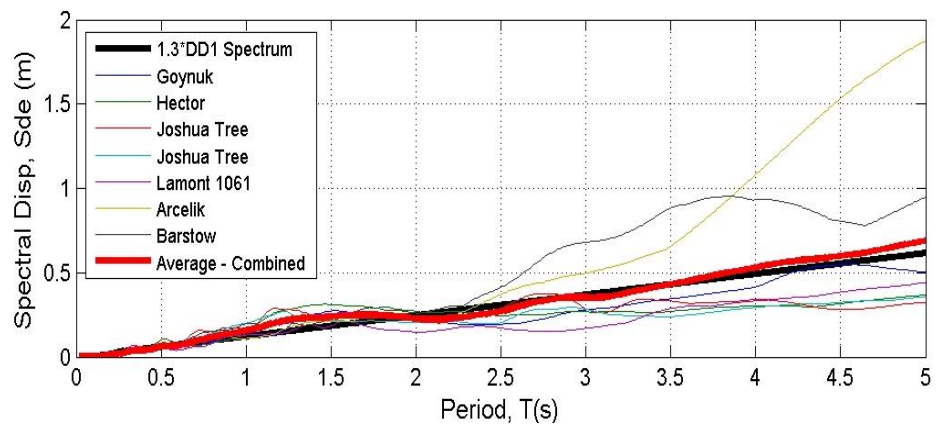


Figure 4.18: Spectral displacements showing scaling mean values.

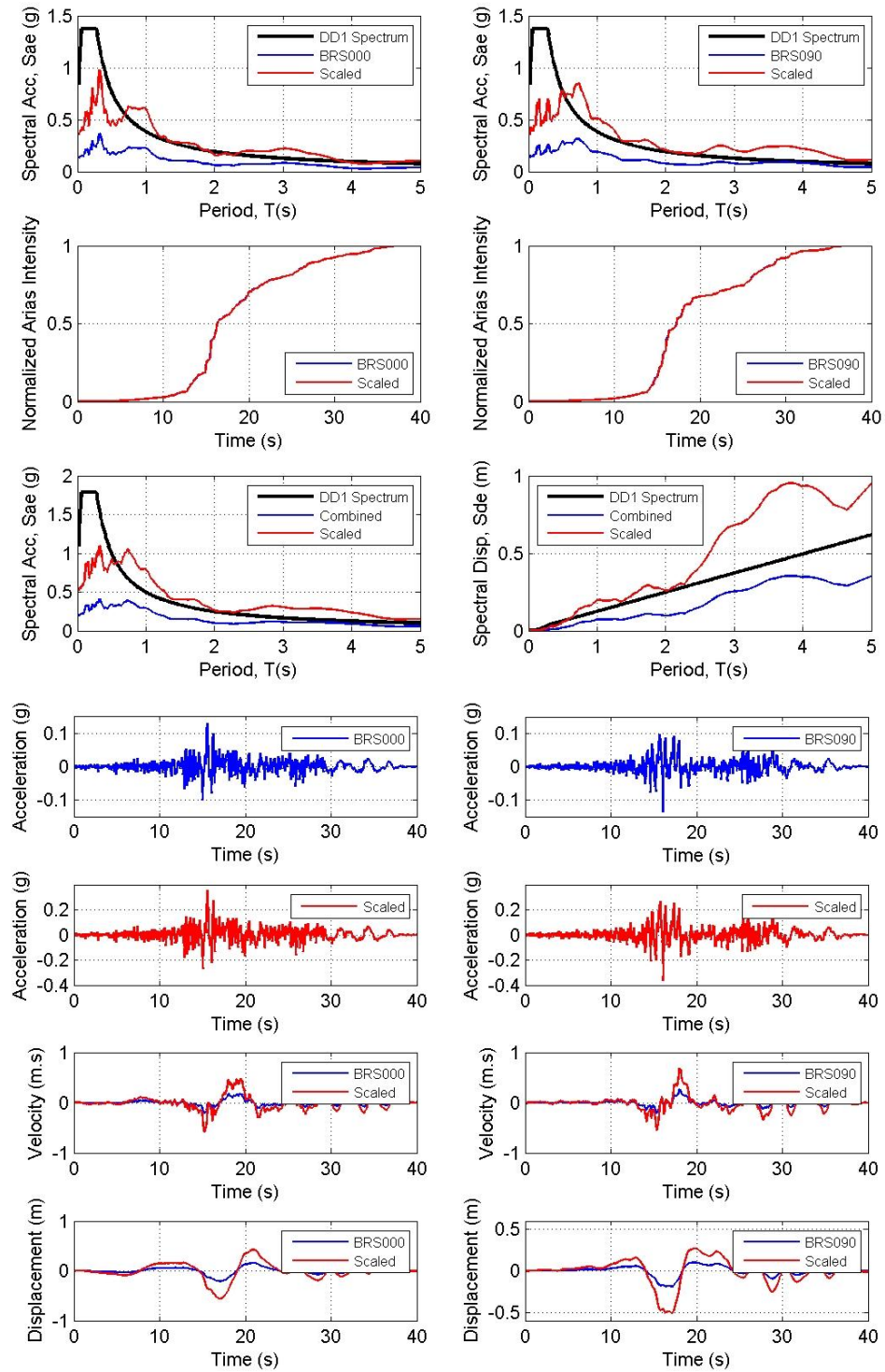


Figure 4.19: Landers RSN838 Ground Motion.

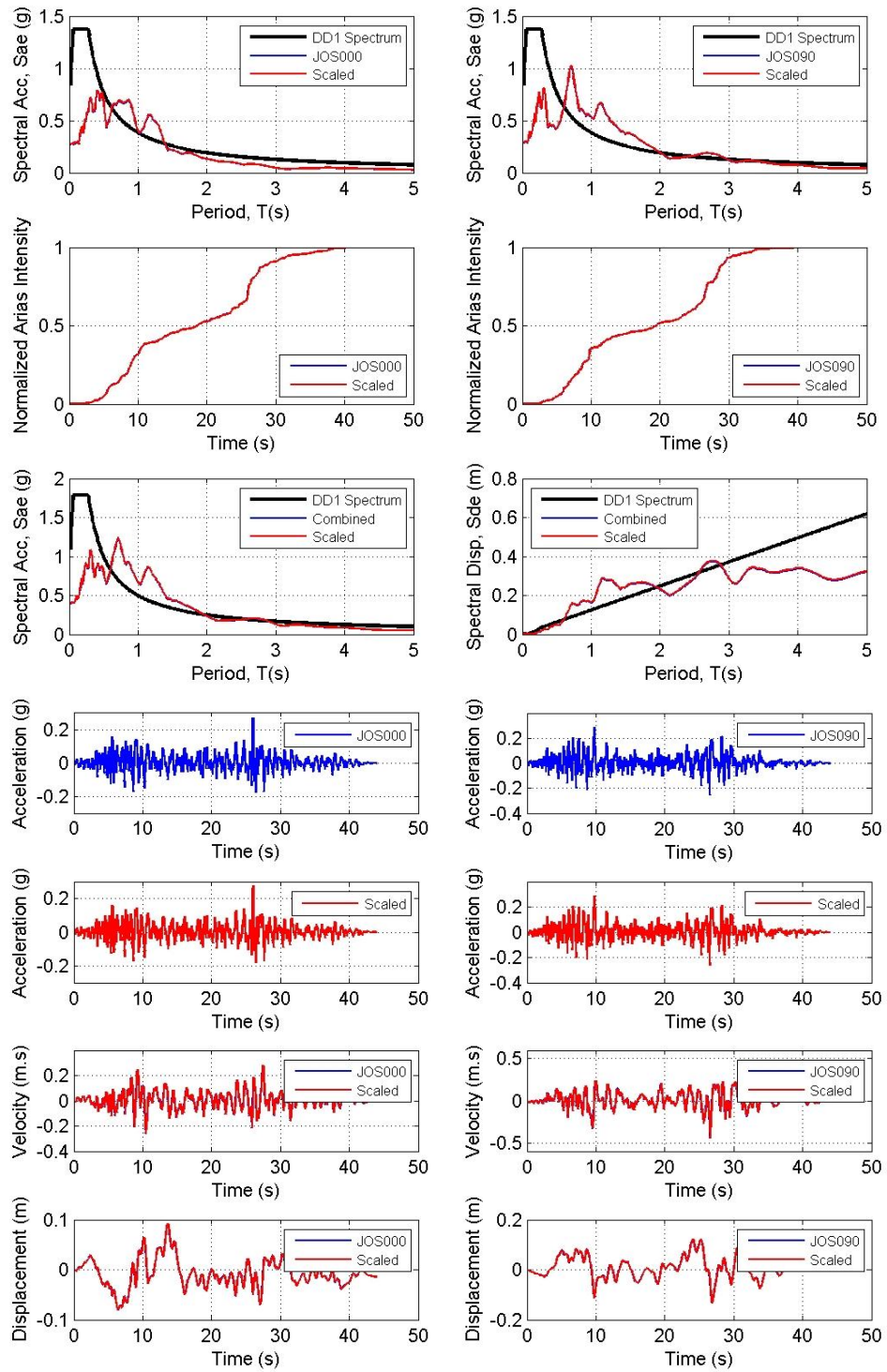


Figure 4.20: Landers RSN864 Ground Motion.

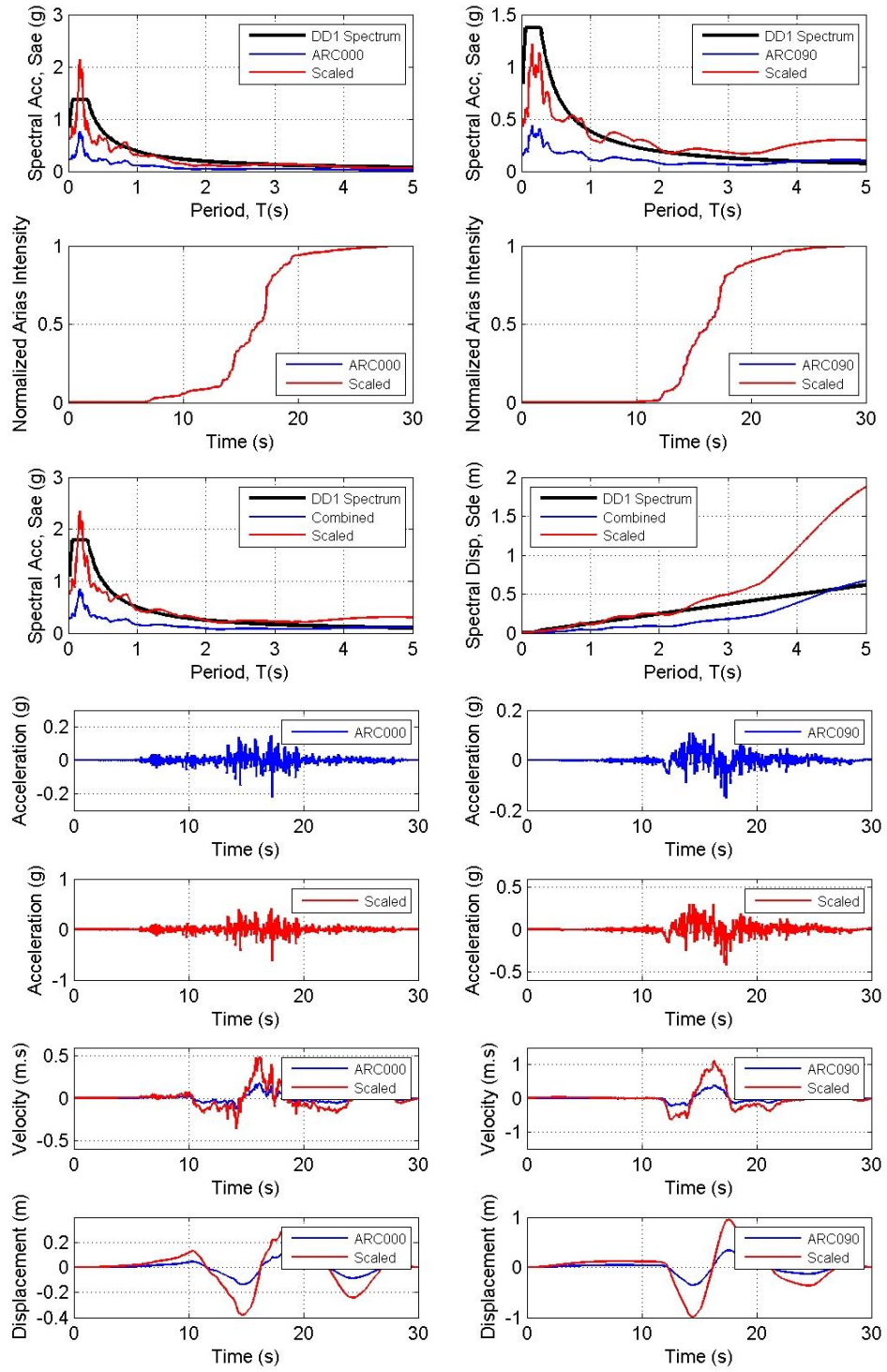


Figure 4.21: Kocaeli RSN1148 Ground Motion.

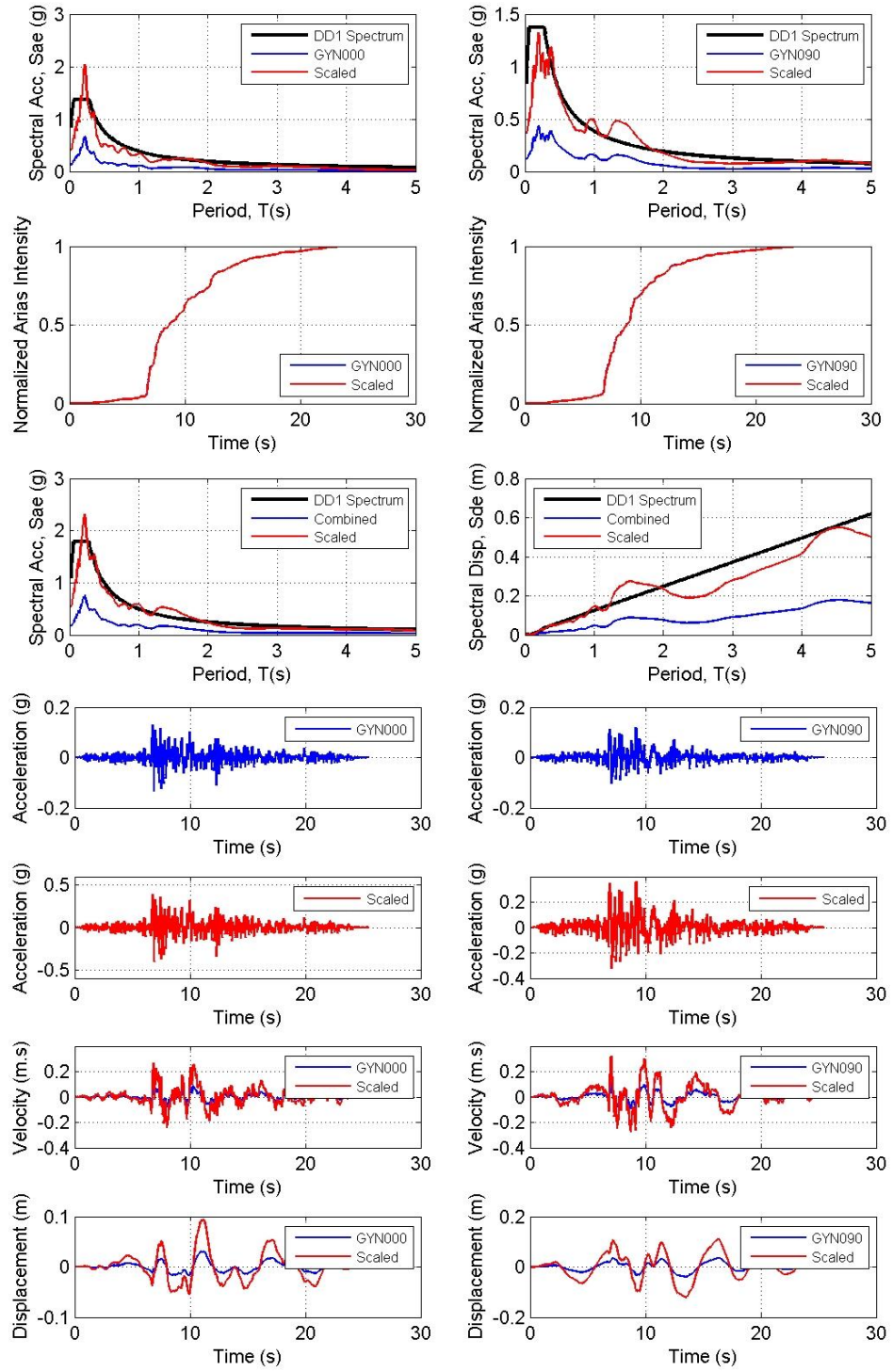


Figure 4.22: Kocaeli RSN1162 Ground Motion.

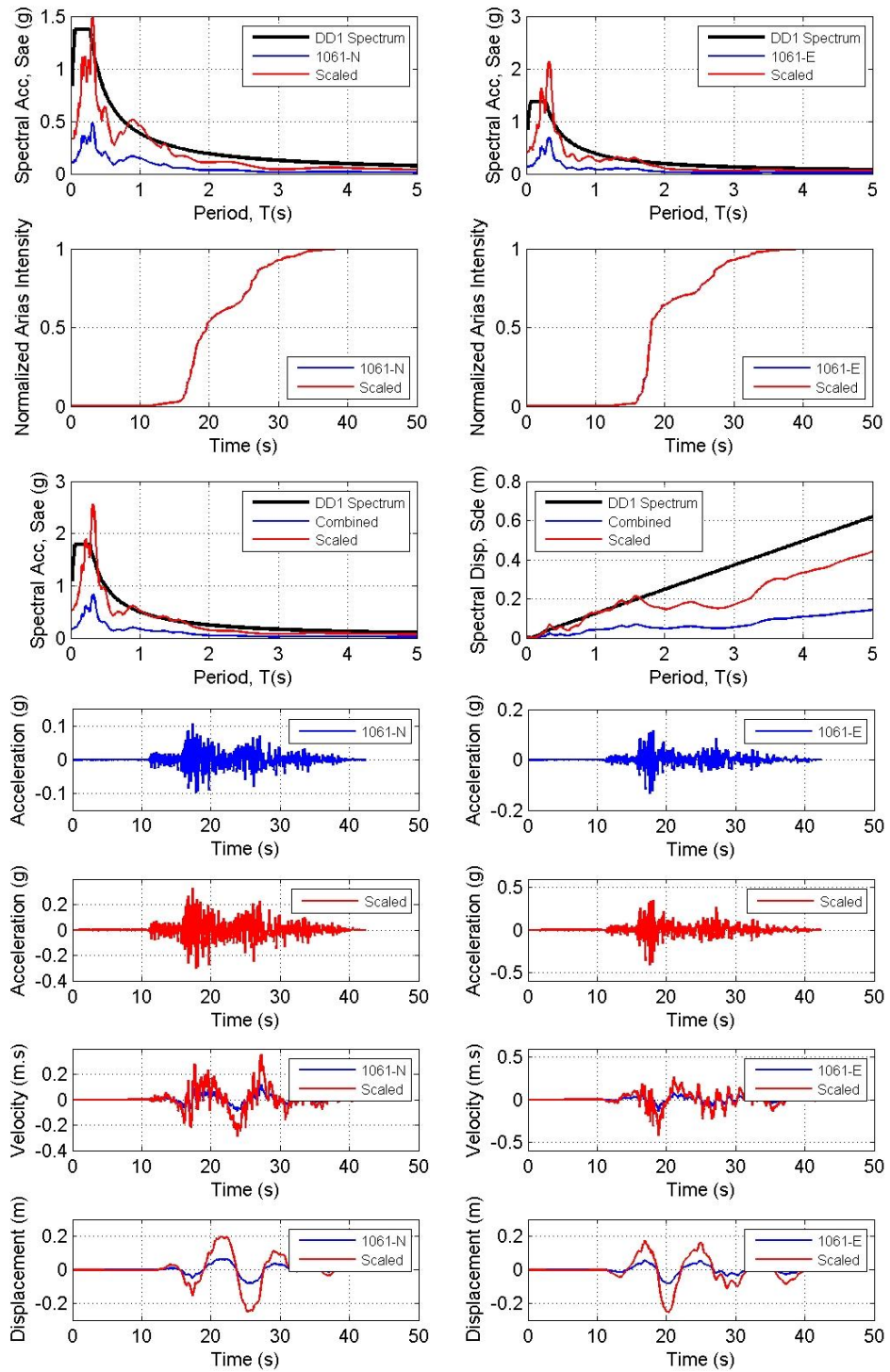


Figure 4.23: Duzce RSN1614 Ground Motion.

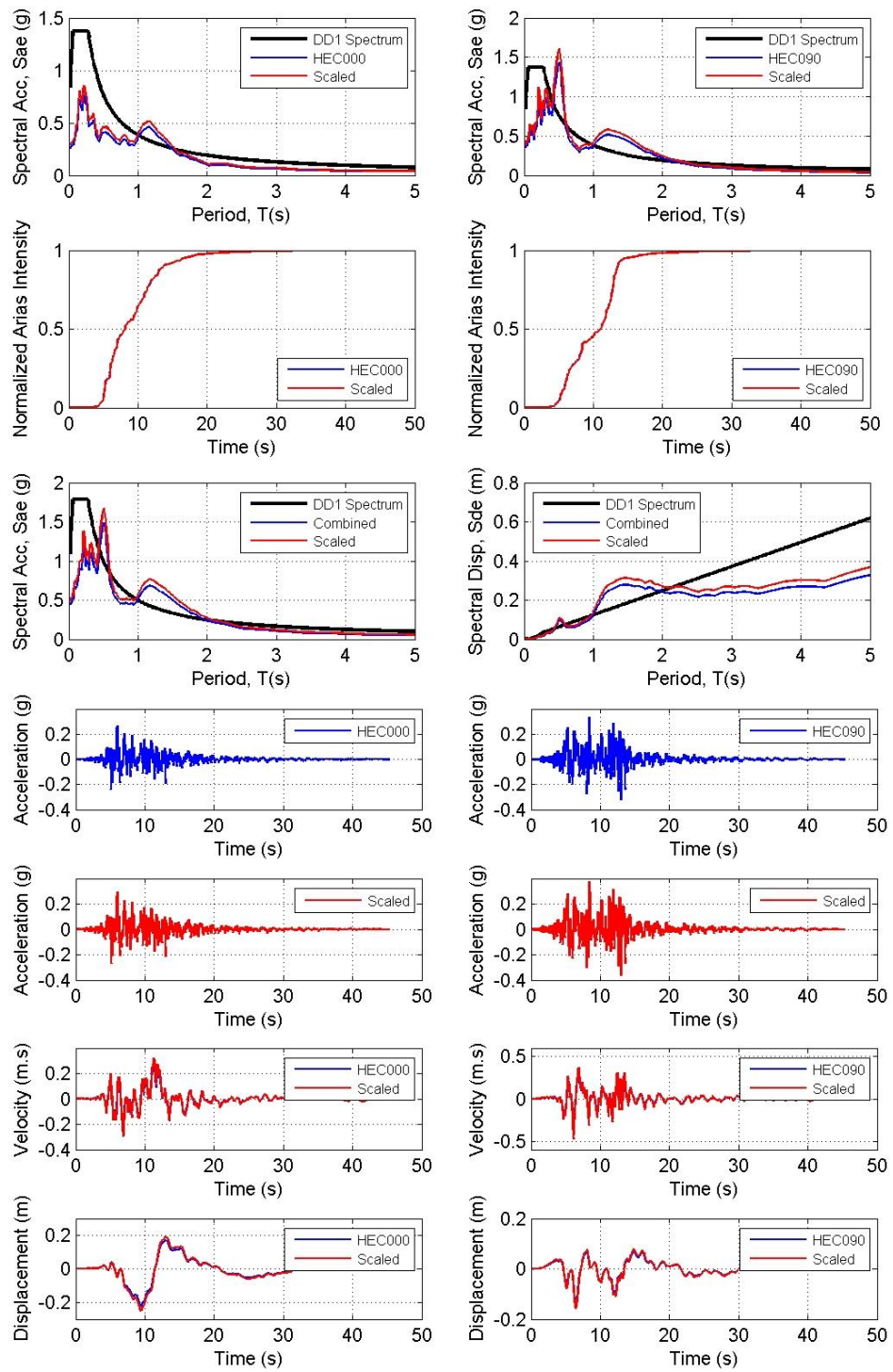


Figure 4.24: Hector Mine RSN1787 Ground Motion.

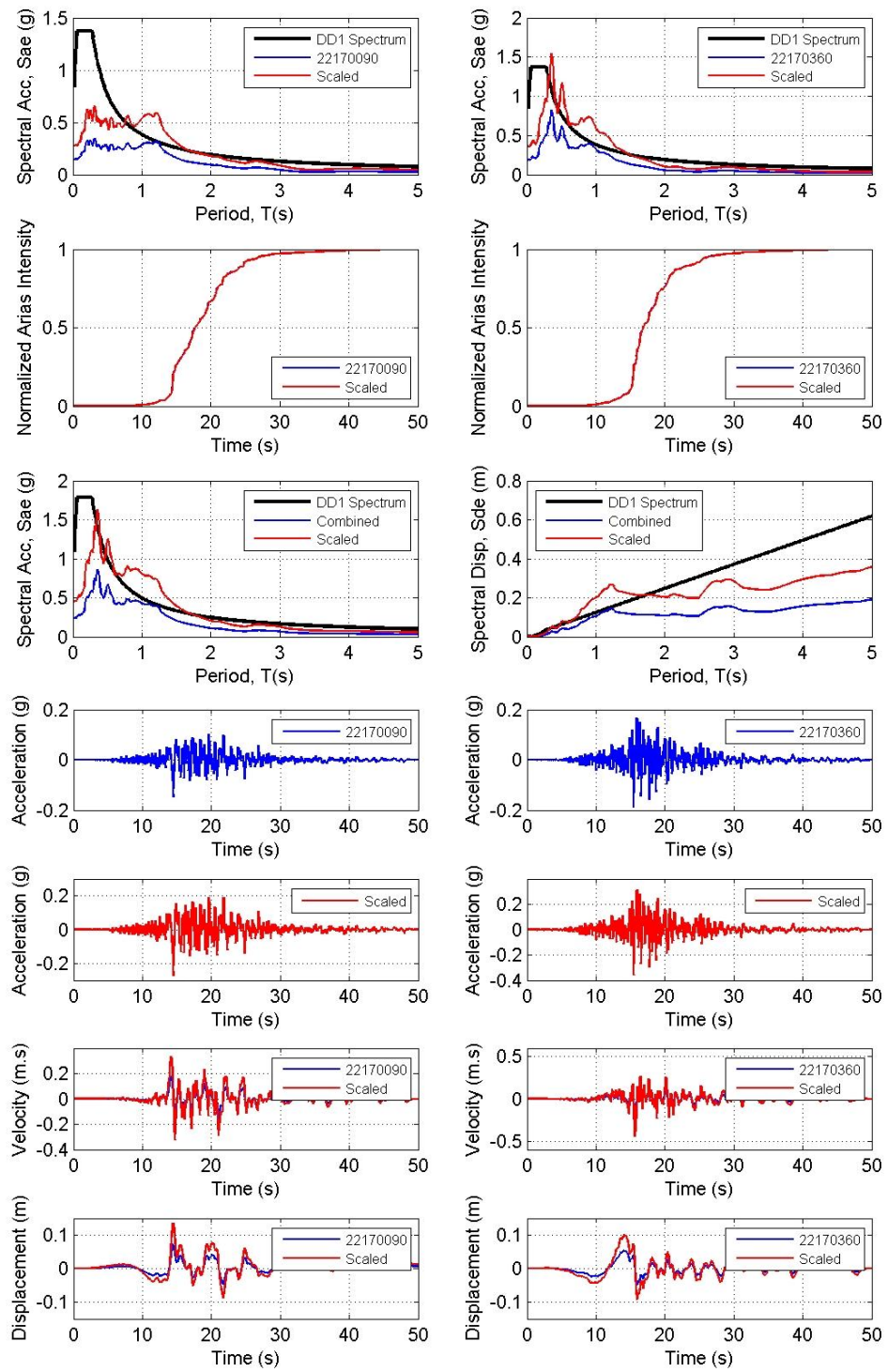


Figure 4.25: Hector Mine RSN1794 Ground Motion.

5. NONLINEAR ANALYSIS RESULTS

In this chapter, assessments of elements of typical and used BRBFs tall buildings will be shown at continues topics which are shown at Figure 5.1. Performance based design will be explained and shown by some calculations of examples and tables step by step which is the main topic of this chapter as it is explained in of Chapter 4.

However, the model of tall buildings was analyzed by PERFORM 3D to obtaine performance level of given two types of tall buildings.

This chapter is organised as follows;

- Results for Typical (with core SW and frames) Structural System
- Results for Buckling-Retrained Braced Frame Structural System
- Comparison of Results between Given 2 types of Tall Buildings

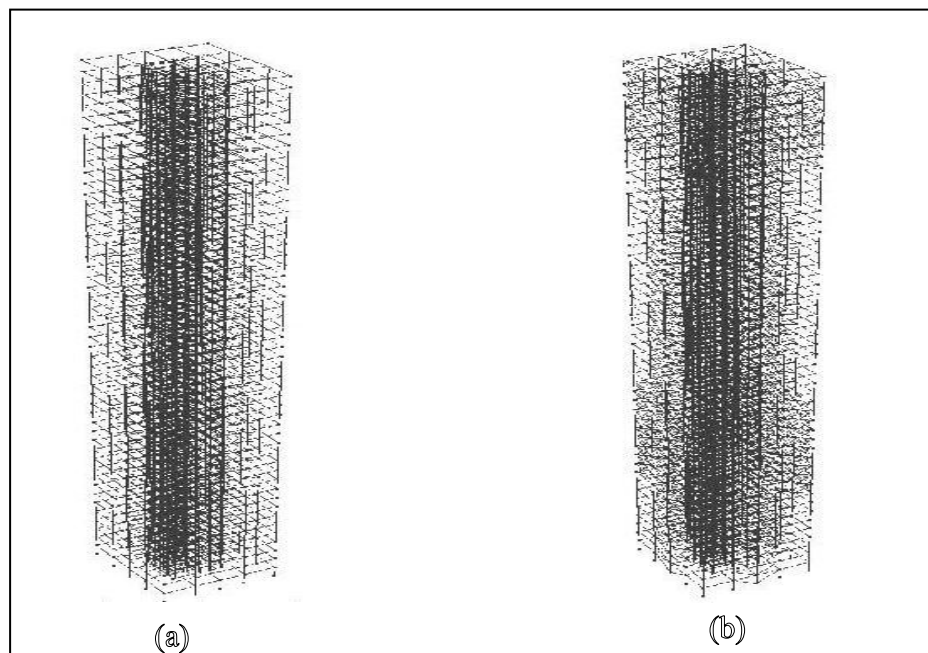


Figure 5.1: Three-dimensional views of the structures used in the study:
a) Typical tall building; b) Tall building with BRBFs.

5.1.Results for Typical Structural System

In this part, performance levels of members of typical tall building are shown subjected to DD-1 Level Earthquake (MCE) with ground motions having a 2% probability of being exceeded in 50 years (2475-year return period).

However, results for typical tall building are investigated and presented for performance levels of members as follows;

- Modal analysis results
- Energy diagrams
- Drift checks
- Strains
- Plastic hinge results
- Shear checks

5.1.1.Modal Analysis Results

The first 6 mode shapes of the structural model are shown in Figure 5.2-5.3. The effective mass ratio of the first 12 modes is given in Table 5.1. The mode shapes found in perform 3D model are in line with mode shapes found in ETABS. Only the structural periods are shorter which is to be expected since the ETABS model the wall are defined with effective crack section stiffness where the cracking is explicitly modelled in Perform 3D with fiber elements which uses the uncracked stiffness of the wall elements in the calculation of free vibration modes and mode-shapes.

Table 5.1: Modal properties of typical tall building.

Mode	Period	UX	UY	Sum UX	Sum UY
1	7.496	0.7046	0.0001	70.46	0.01
2	7.04	0.0001	0.6631	70.47	66.32
3	4.904	1.28E-06	0.0028	70.47	66.61
4	2.101	0.1378	1.80E-06	84.25	66.61
5	1.709	1.25E-06	0.1465	84.25	81.26
6	1.534	0	0.0183	84.25	83.09
7	1.012	0.0508	0	89.33	83.09
8	0.866	0	0.004	89.33	83.49
9	0.729	0	0.0567	89.33	89.16
10	0.616	0.0278	5.05E-07	92.12	89.16
11	0.566	2.27E-06	0.001	92.12	89.26
12	0.43	3.97E-06	0.0287	92.12	92.13

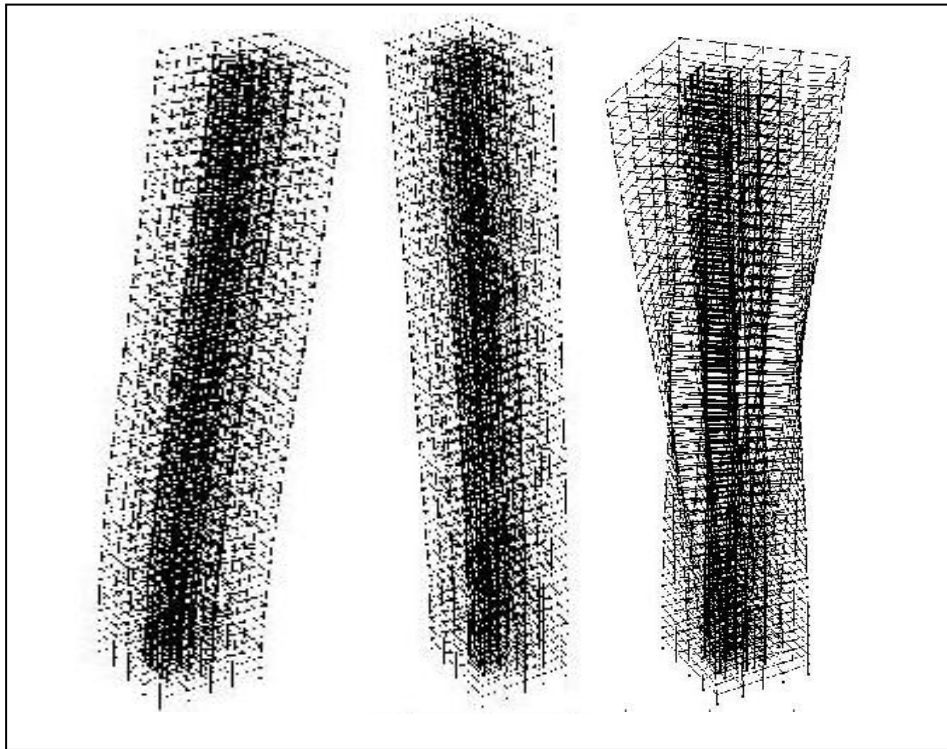


Figure 5.2: First 3 mode shapes at each direction of typical tall building.

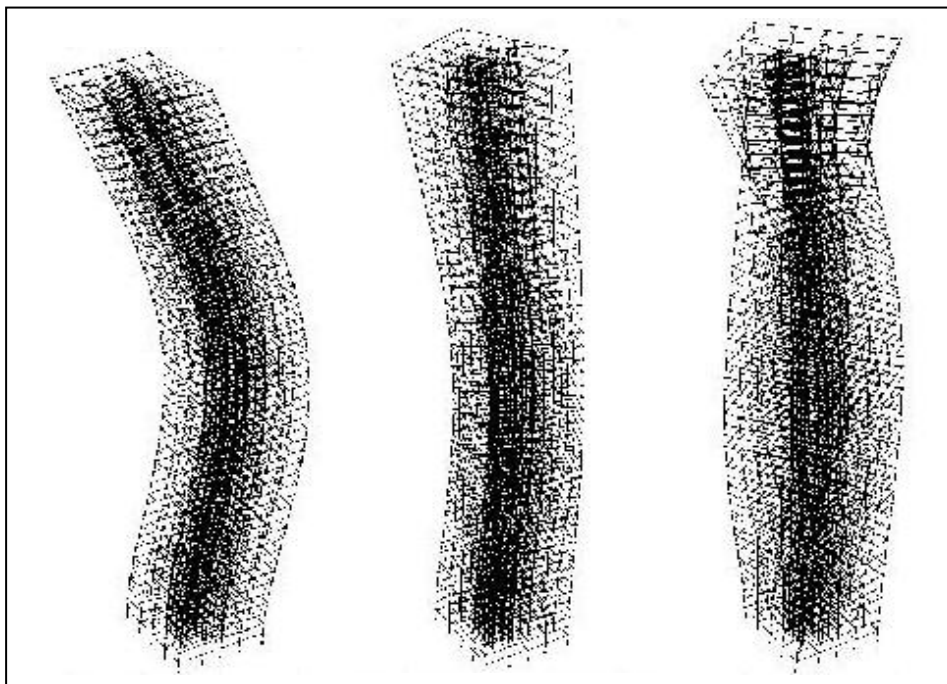


Figure 5.3: 4,5 and 6th mode shapes of typical tall building.

5.1.2. Energy Diagrams

Energy diagrams show the inelastic energy distributions due to total energy, kinetic energy, viscous damping energy and structural behavior which is occurred by earthquake ground motions.

In the figure as it is shown, red, green, yellow, orange, cyan and blue colours represent inelastic energy, Beta-K viscous energy, Alpha-M viscous energy, modal damping energy, strain energy and kinetic energy, respectively so the energy diagrams provide to obtain energy balance of structure under ground motions as it is shown at figure as Hector Mine ground motion.

However, performance level can be obtained by energy diagrams it is shown for Hector Mine ground motion, inelastic energy which is depending on plastic hinges results of structural members, is ranged between 30% and 40% of total energy. It's clearly that performance level is provided against DD-1 Level Earthquake for collapse prevention as it is demonstrated in previous topics.

All explanations about energy diagrams are shown at Figures 5.4-5.7.

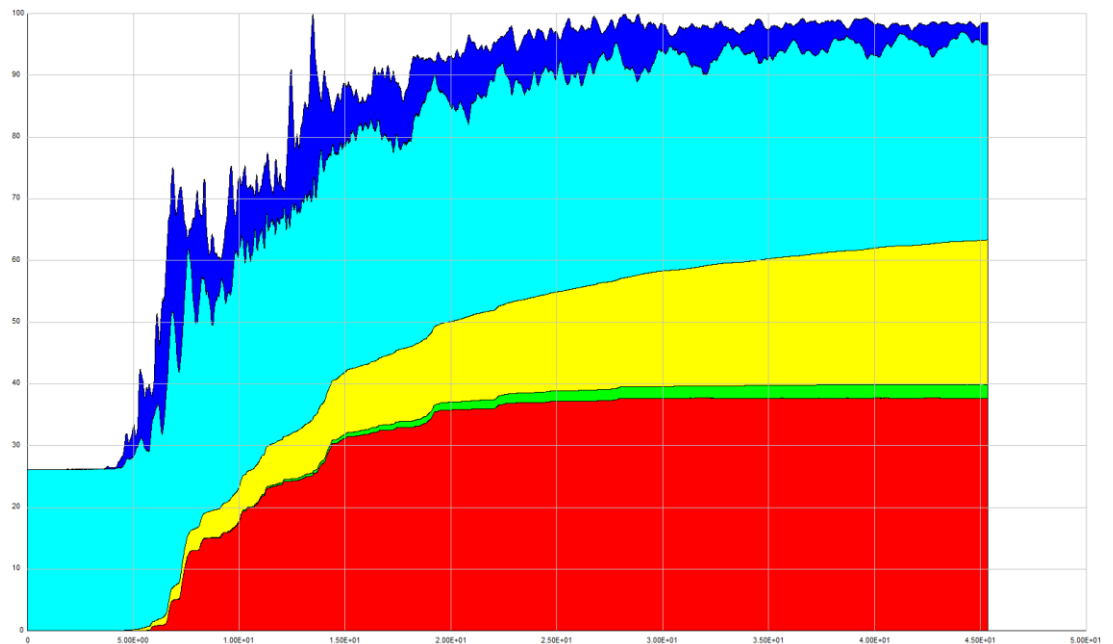


Figure 5.4: Composed total energy of typical tall building for Hector Mine Ground Motion.

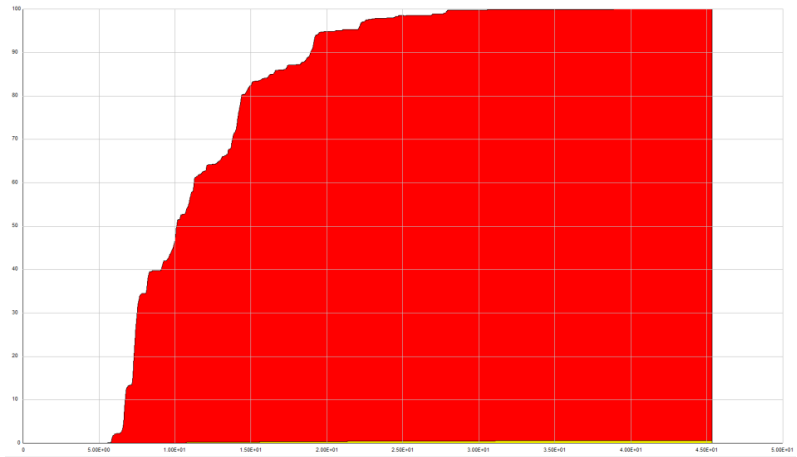


Figure 5.5: Inelastic energy of shear walls about 2%.

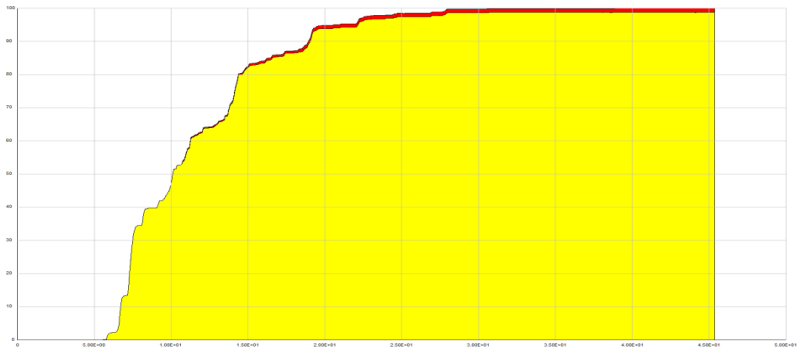


Figure 5.6: Inelastic energy of beams about 96%.

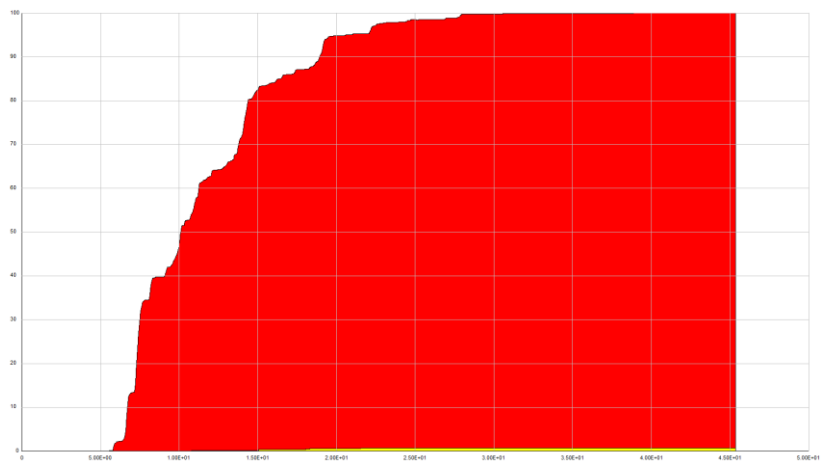


Figure 5.7: Inelastic energy of columns about 2%.

5.1.3. Drift Checks

The story drift ratios based on the average of nonlinear time-history analysis for the MCE plotted for the given corner location at each storey. Drift for typical tall building is shown that all the given story drift ratios that are shown in the following Figure 5.8-5.11 are less than 3% of average of drifts or less than 4.5% story drift ratio for each story, which is provided in TBSC 2016.

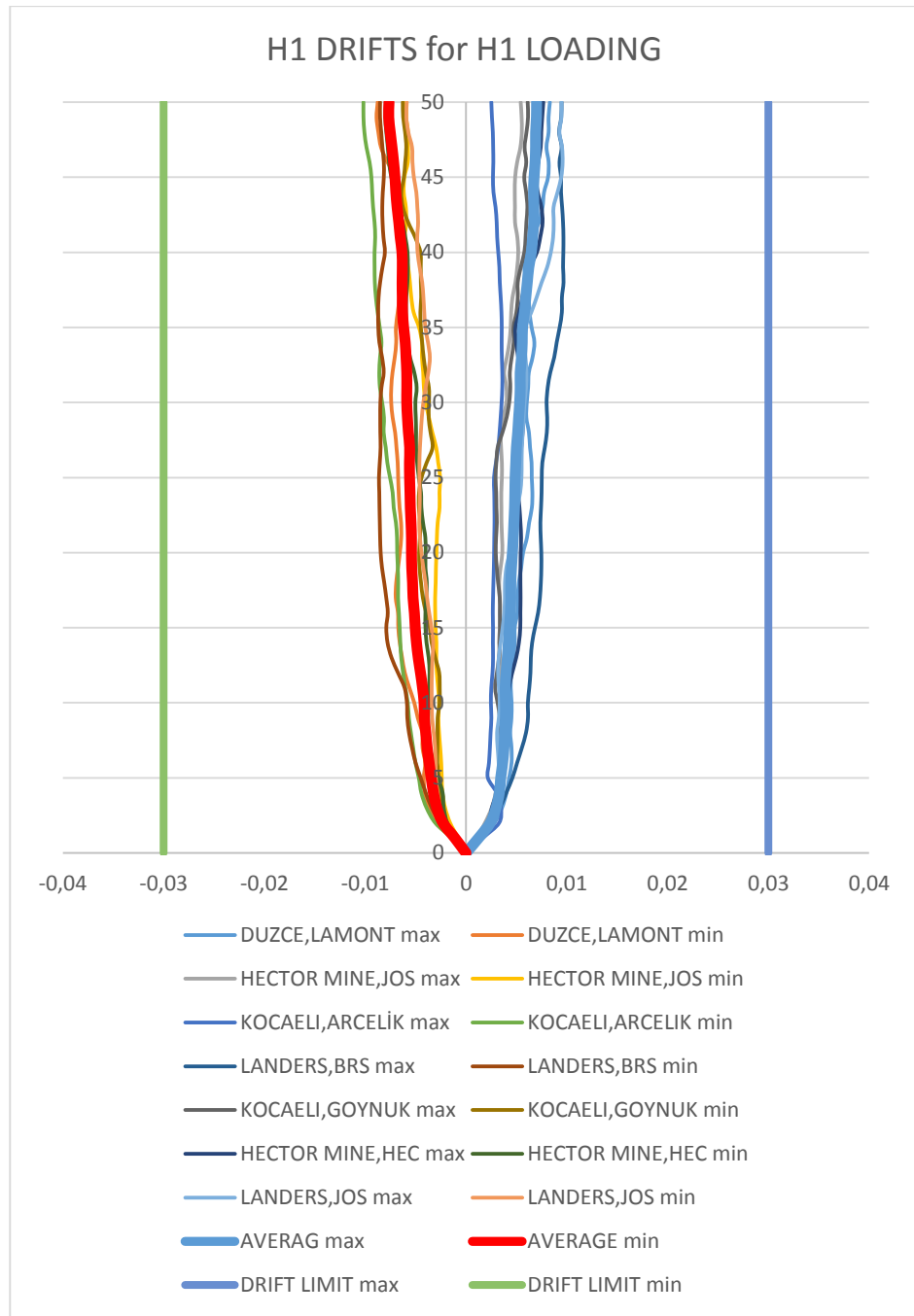


Figure 5.8: Drift H1 for H1 loading of typical tall building.

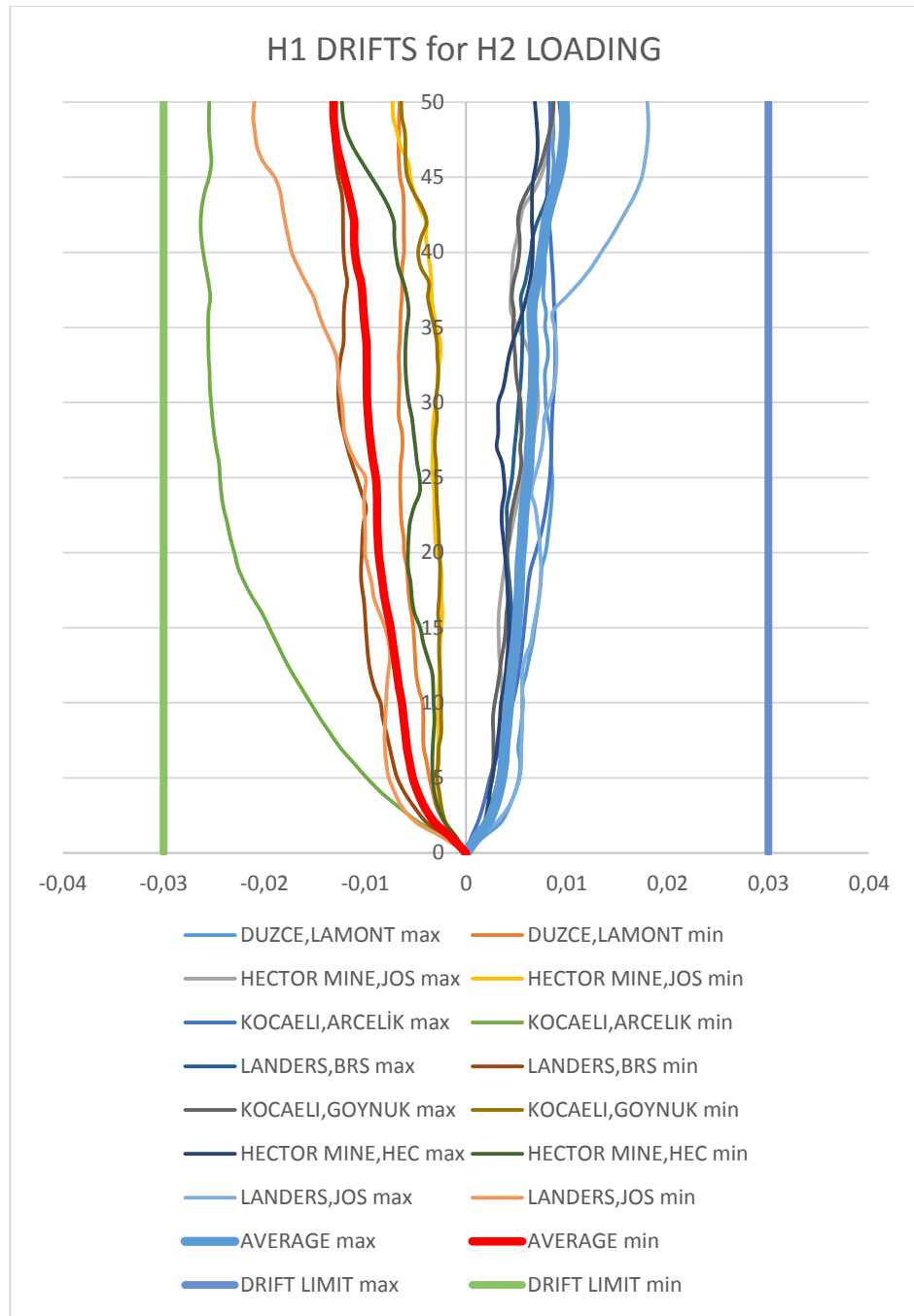


Figure 5.9: Drift H1 for H2 loading of typical tall building.

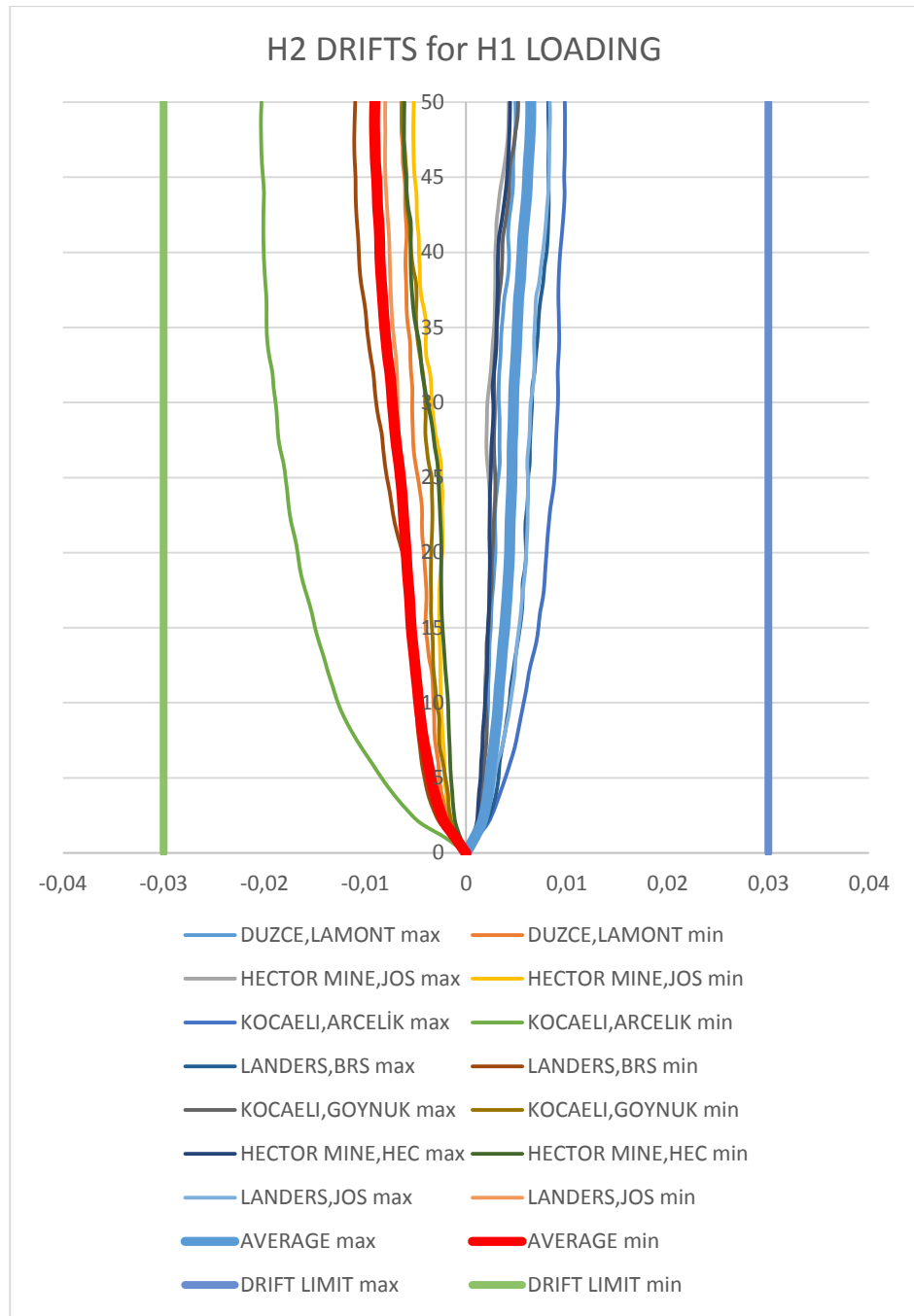


Figure 5.10: Drift H2 for H1 loading of typical tall building.

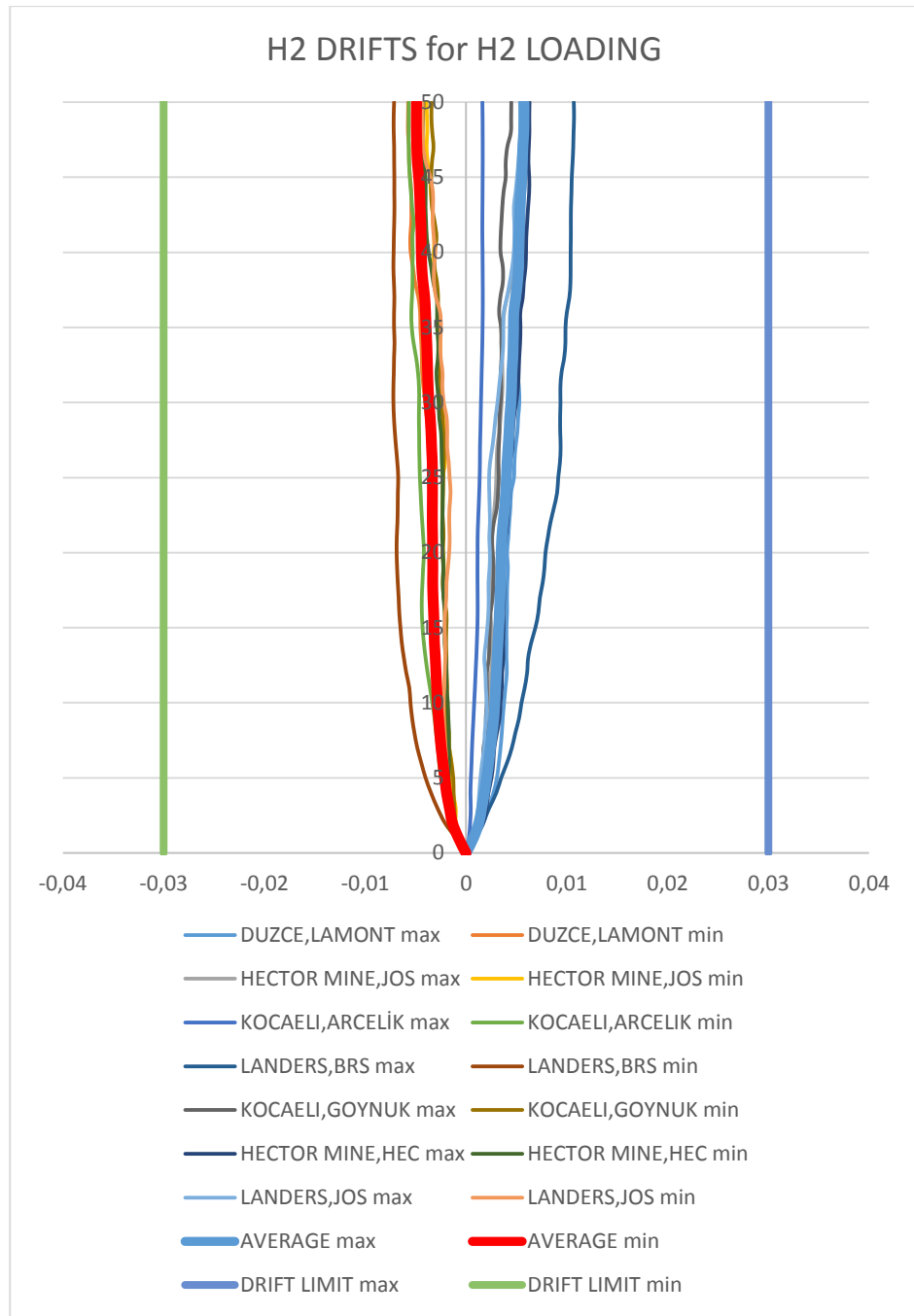


Figure 5.11: Drift H2 for H2 loading of typical tall building.

5.1.4. Performance Checks for Shear Walls

Shear walls are modeled by using fibers. When distributed plasticity, approach is accepted, limiting strains are checked through fibers. TBSC 2016 briefly defines limiting strains in Chapter 5 as Table 5.2. Since the performance level for this building is “Collapse Prevention” in DD1 earthquake;

- Limit strain for reinforcing steel = $\varepsilon_s^{(GP)} = 0.03$
- Limiting strain for concrete = $\varepsilon_c^{(GP)} = 0.015$

The strains are checked by using strain-gage elements in Perform 3D as given in Figure 5.13-5.16. The location of gages are shown at Figure 5.12.

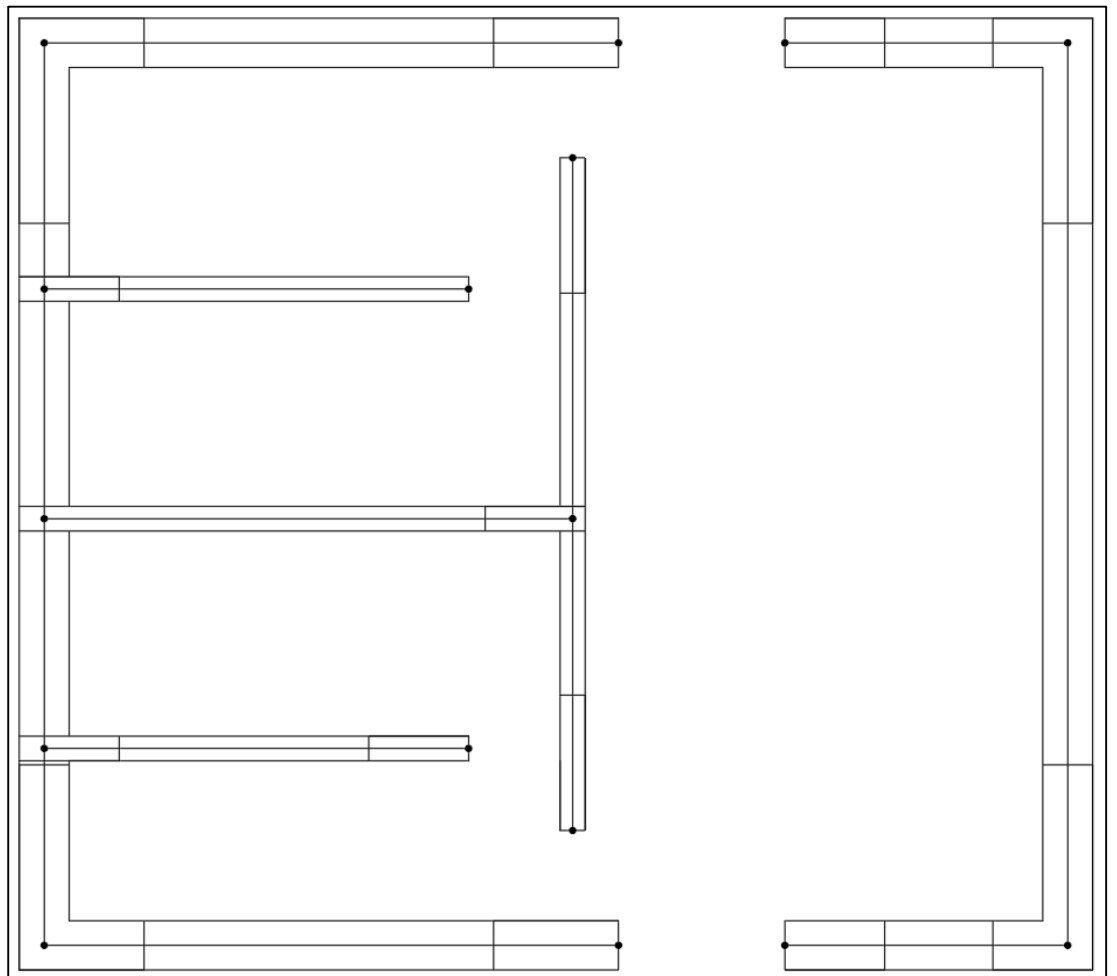


Figure 5.12: Strain-gage locations on a single pier drawn in black.

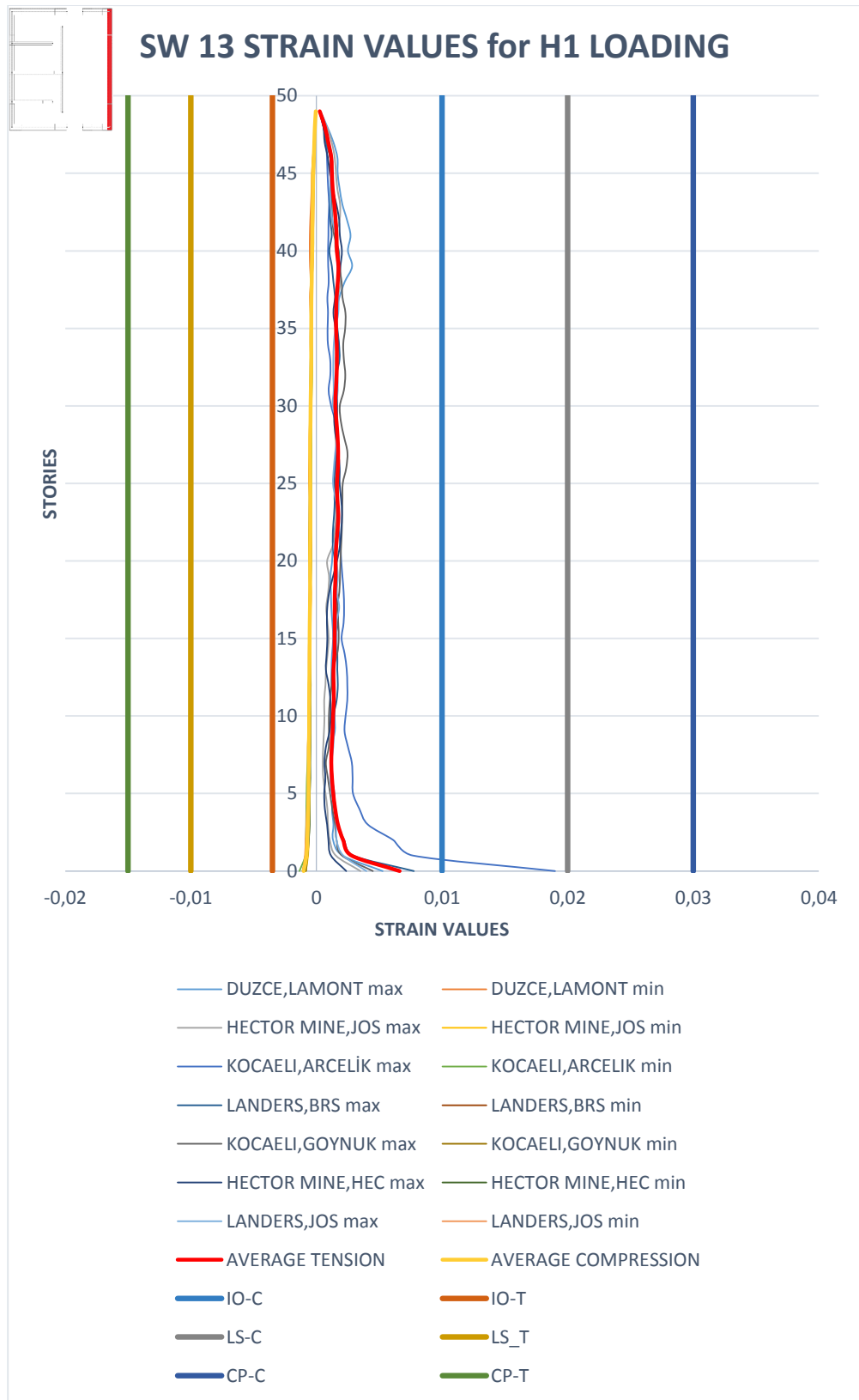


Figure 5.13: Strains for top strain-gage of sw 13 leg at H1 direction loading.

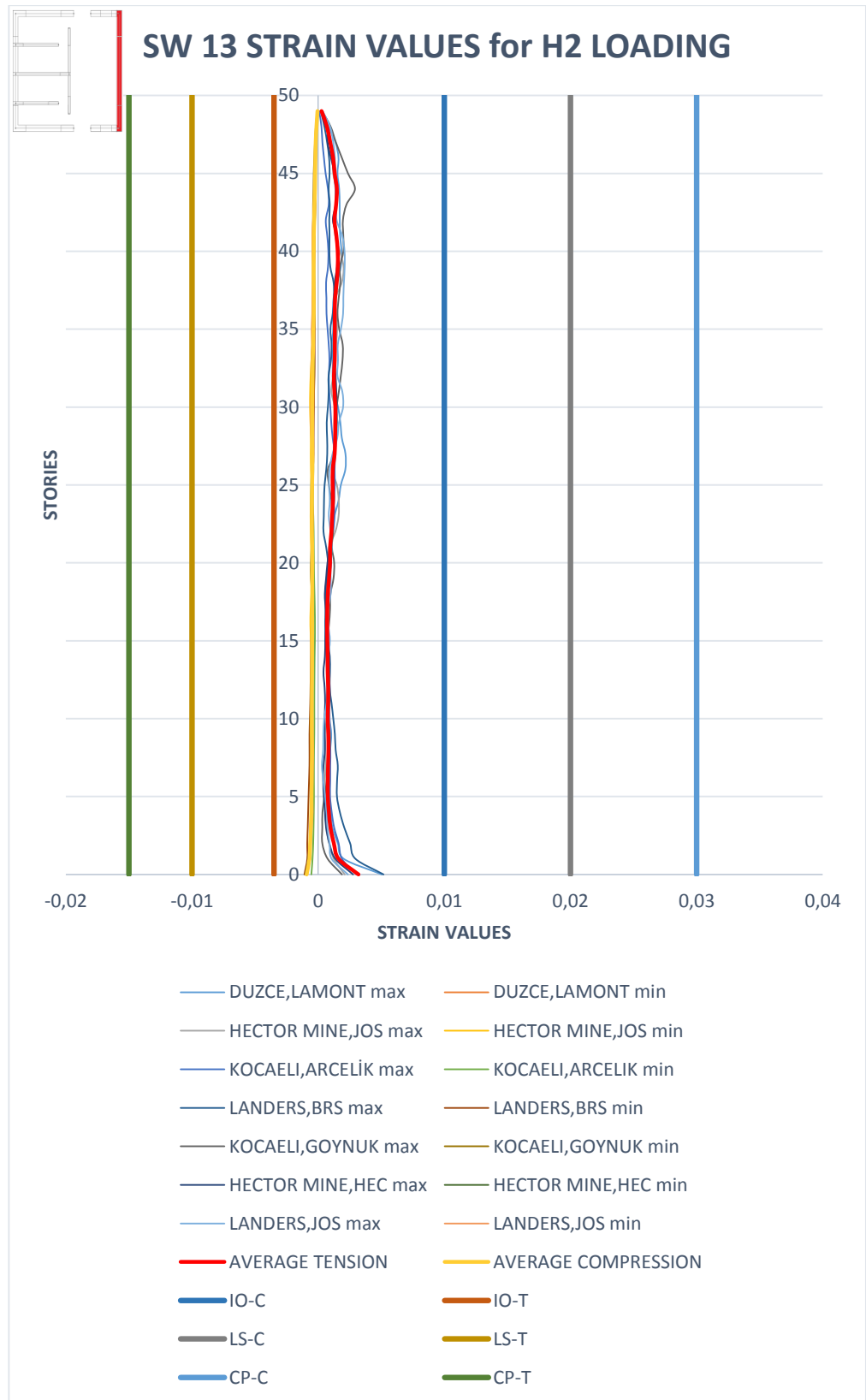


Figure 5.14: Strains for top strain-gage of sw 13 leg at H2 direction loading.

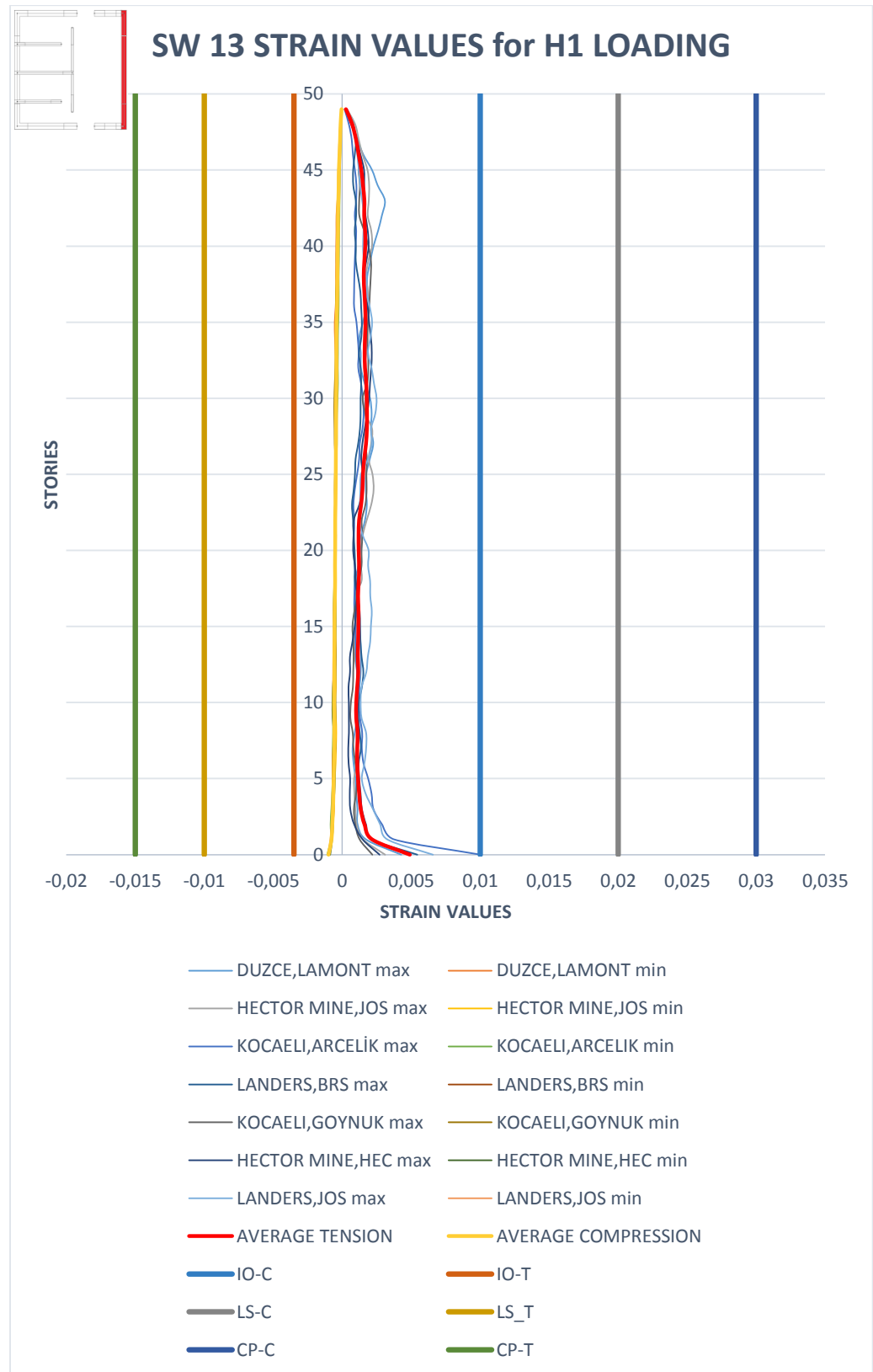


Figure 5.15: Strains for bottom strain-gage of sw 13 leg at H1 direction loading.

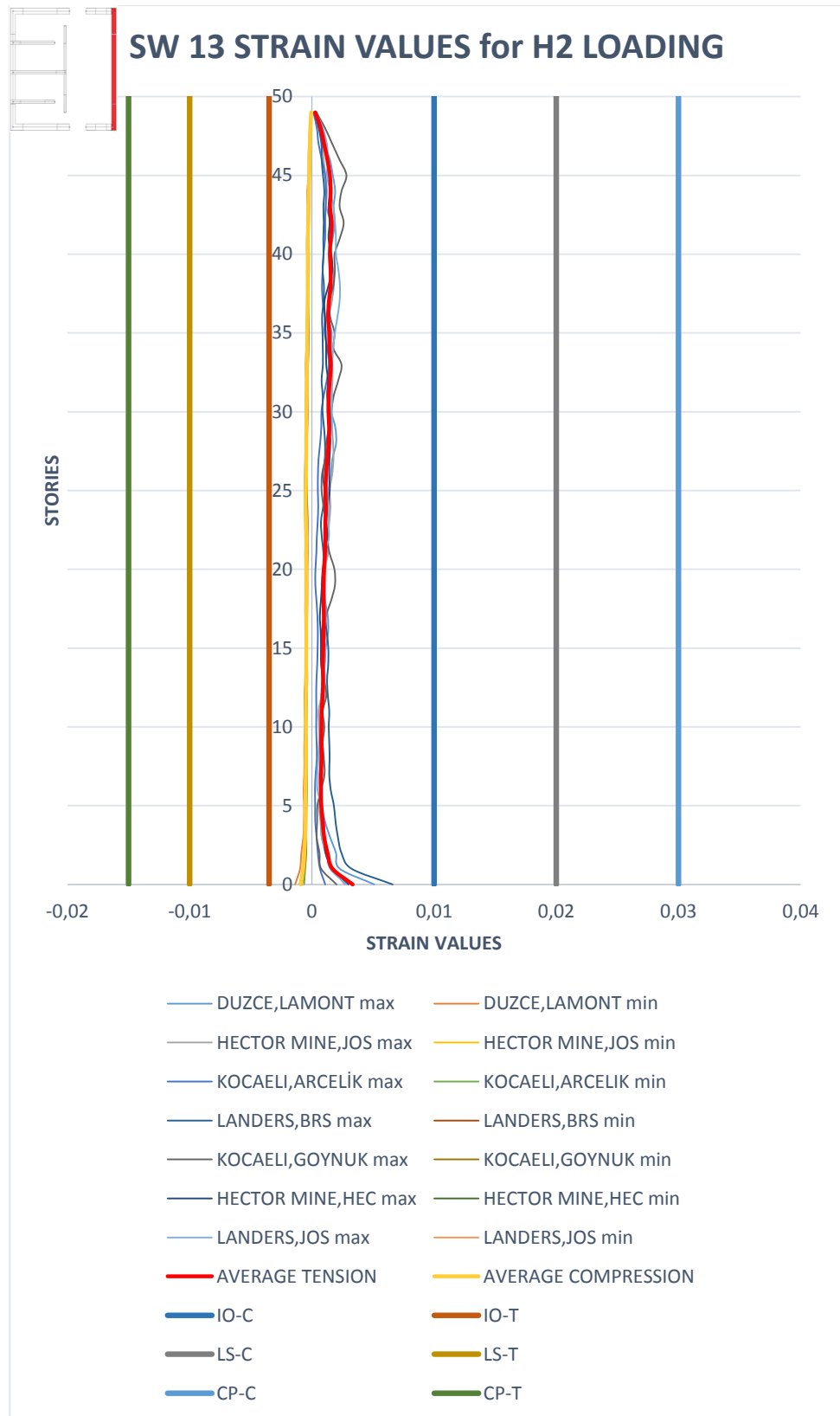


Figure 5.16: Strains for bottom strain-gage of sw 13 leg at H2 direction loading.

5.1.5. Performance Checks for Beams

Only one beam for each storey in plan is investigated which is shown in plan at Figure 5.17.

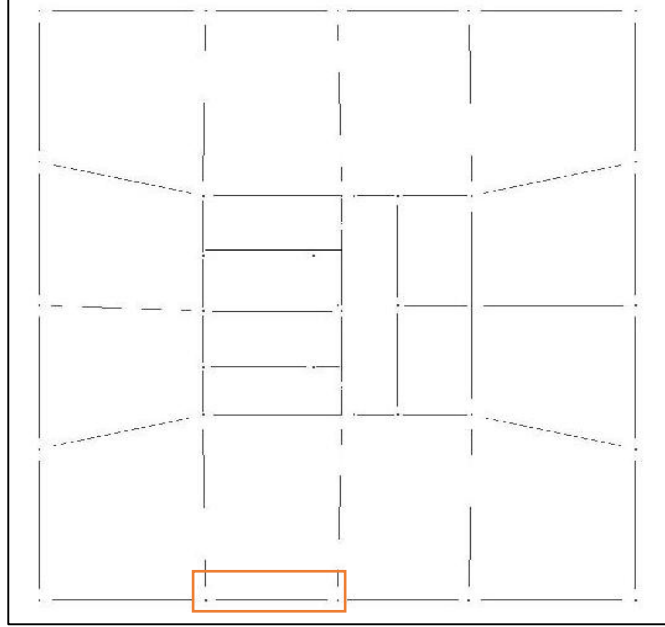


Figure 5.17: Selected beam for performance checks.

Performance demand limit as plastic rotation is calculated and shown in Figure 5.18-5.21 according to TSC 2016 as Life Safety and Collapse Prevention as Eq. (5.1) – (5.2).

$$\theta_p^{(CP)} = \frac{2}{3} \left[(\theta_u - \theta_y) L_p \left(1 - 0.5 \frac{L_p}{L_s} \right) + 4.5 \theta_u d_b \right] \quad (5.1)$$

$$\theta_p^{(LS)} = 0.75 \theta_p^{(CP)} \quad (5.2)$$

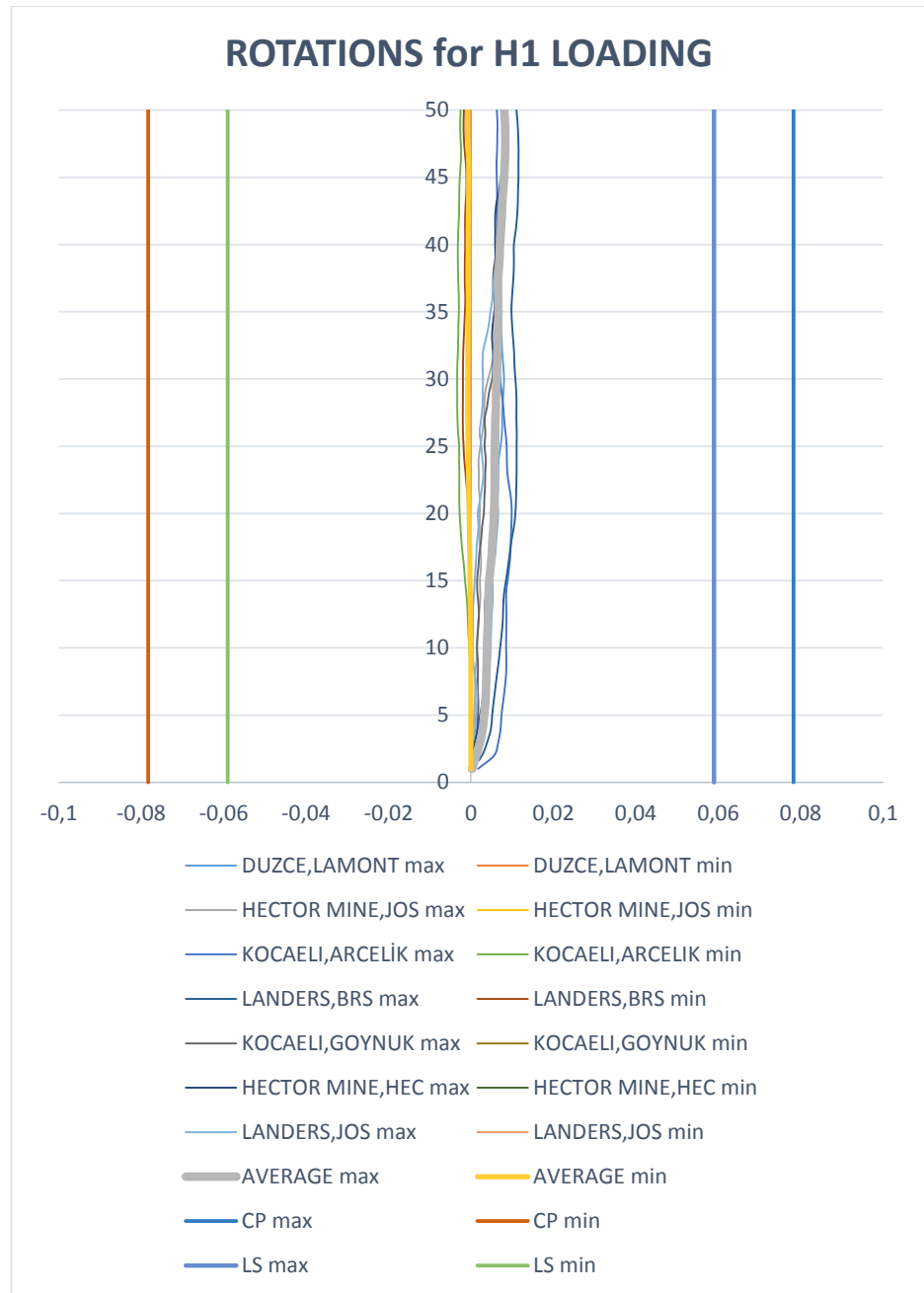


Figure 5.18: Beam plastic hinges rotations at I end for H1 loading.

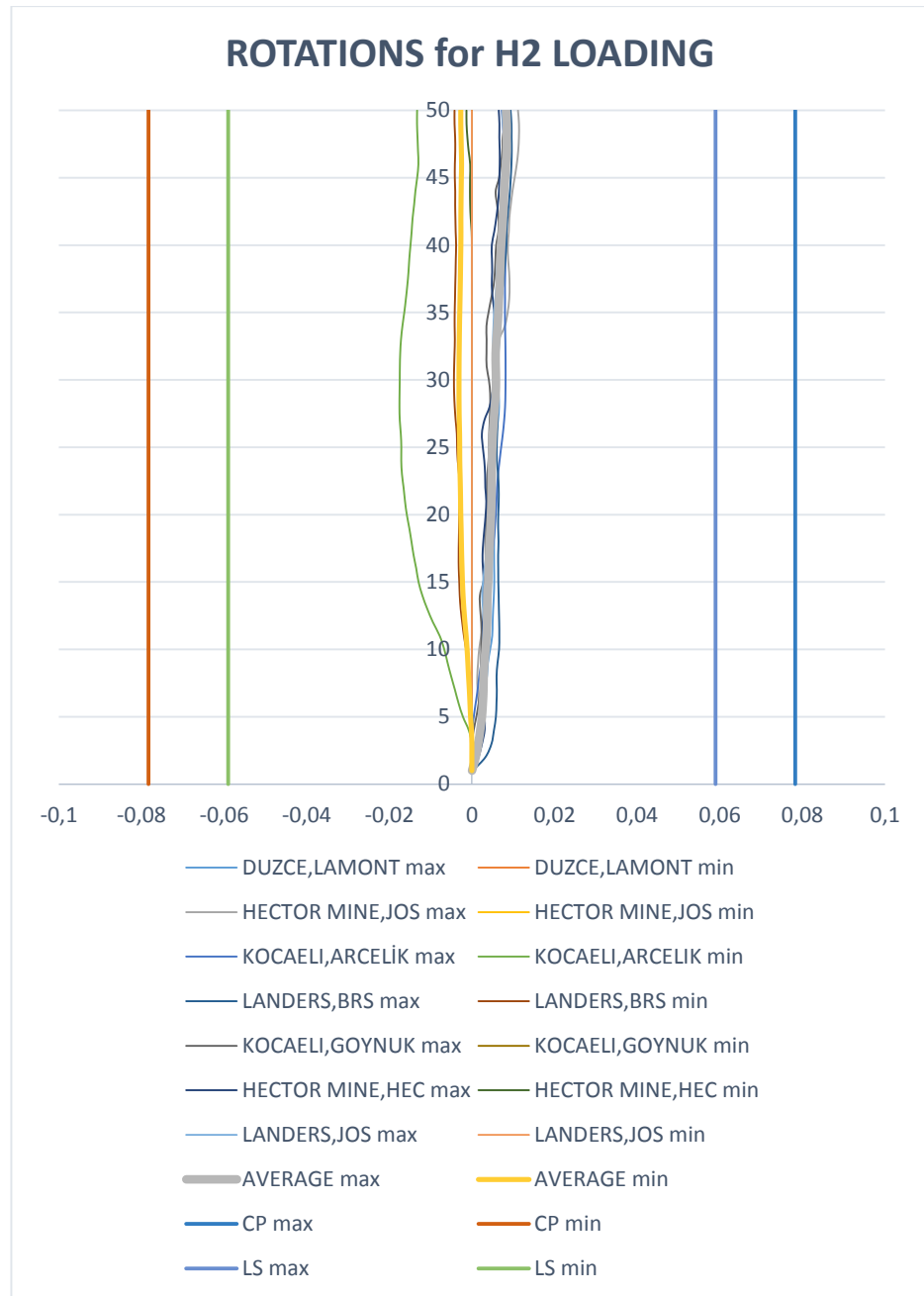


Figure 5.19: Beam plastic hinges rotations at I end for H2 loading.

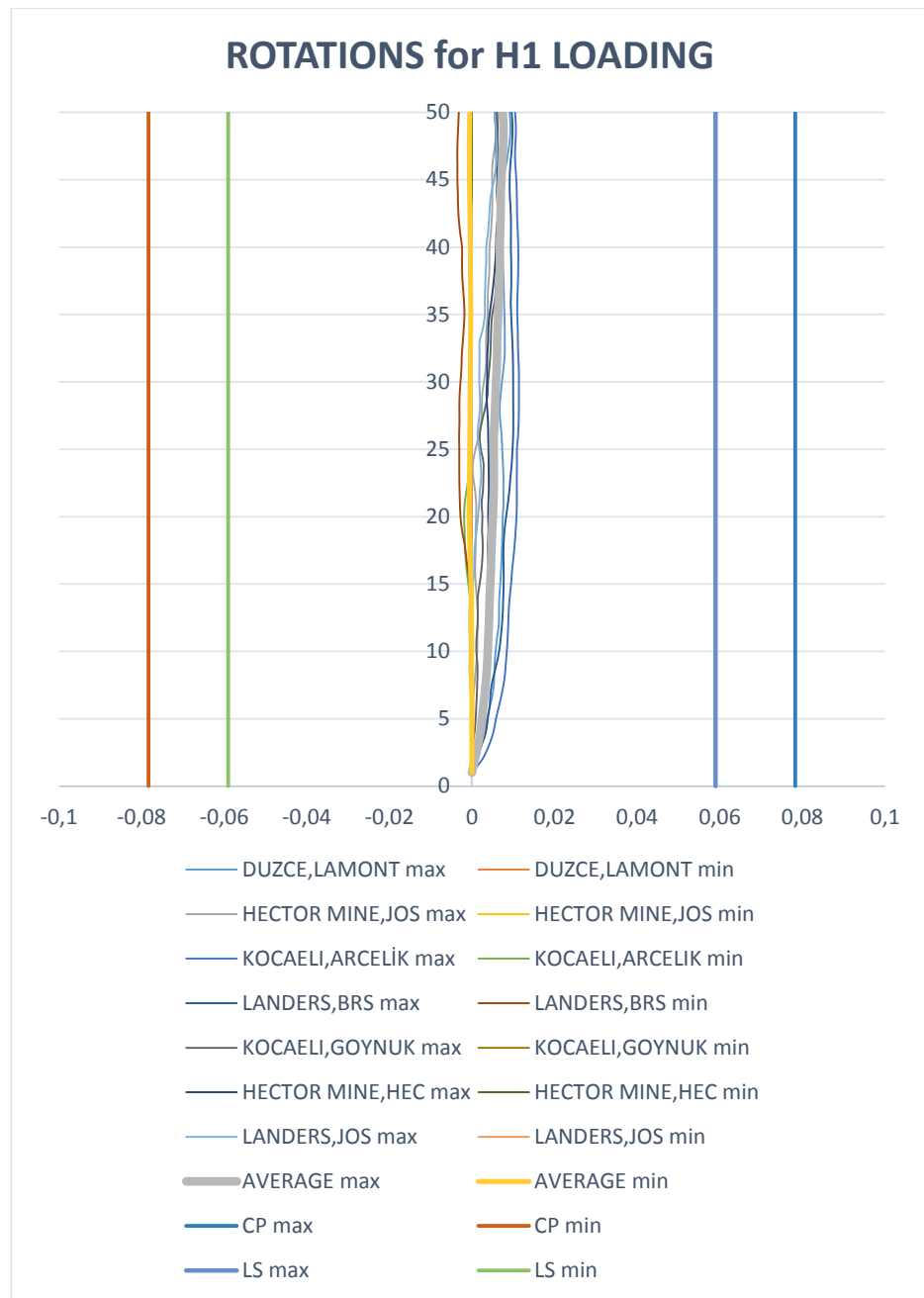


Figure 5.20: Beam plastic hinges rotations at J end for H1 loading.

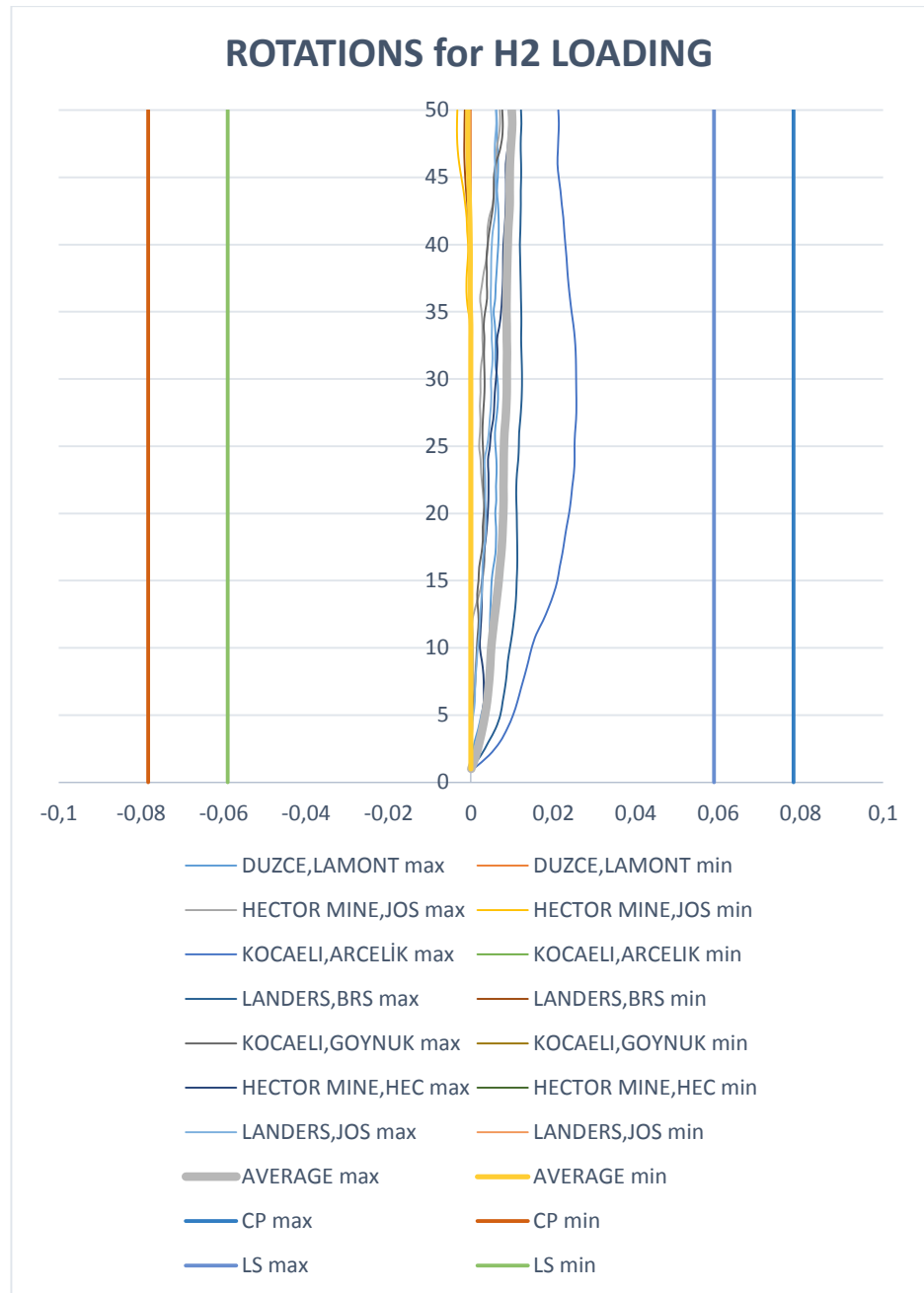


Figure 5.21: Beam plastic hinges rotations at J end for H2 loading.

5.1.6. Performance Checks for Columns

In the analysis results, we can compute demand of plastic rotation and demand of plastic curvature as it is shown at Eq. (5.3) as it is mentioned in Section 5.9.1.1 of TBSC 2016,

$$\Phi_p = \frac{\theta_p}{L_p} \quad (5.3)$$

In this study two kinds of sections of all columns are given which names' are 120x120 and 150x150. Total curvature is computed as it is shown at Eq. (5.4) where is shown in Section 5.9.1.2 of TBSC 2016,

$$\Phi_t = \Phi_y + \Phi_p \quad (5.4)$$

However, all the graphs are generated according to those informations as PMM interactions are shown at Figure 5.22-5.24.

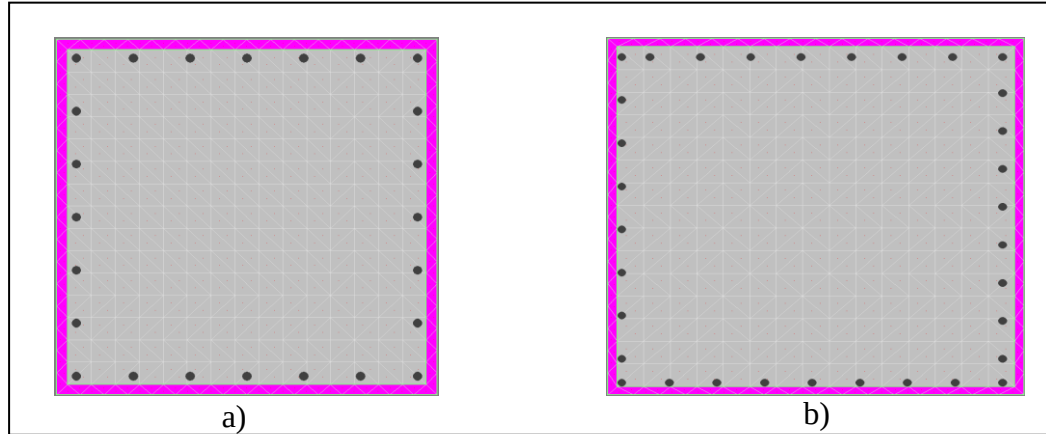


Figure 5.22: Sections of columns;
a) is 120x120, b) is 150x150.

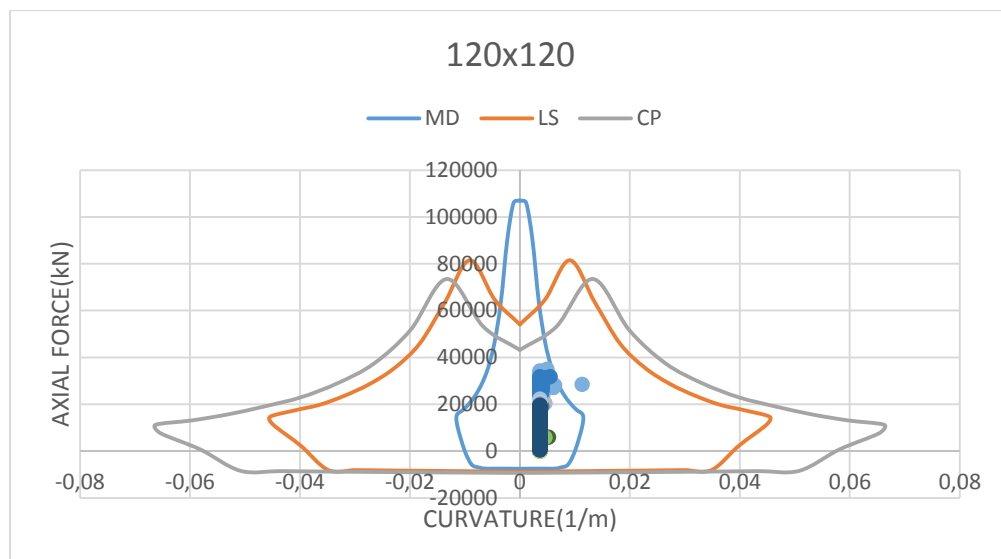


Figure 5.23: Axial force-curvature relationships for 120x120 of columns.

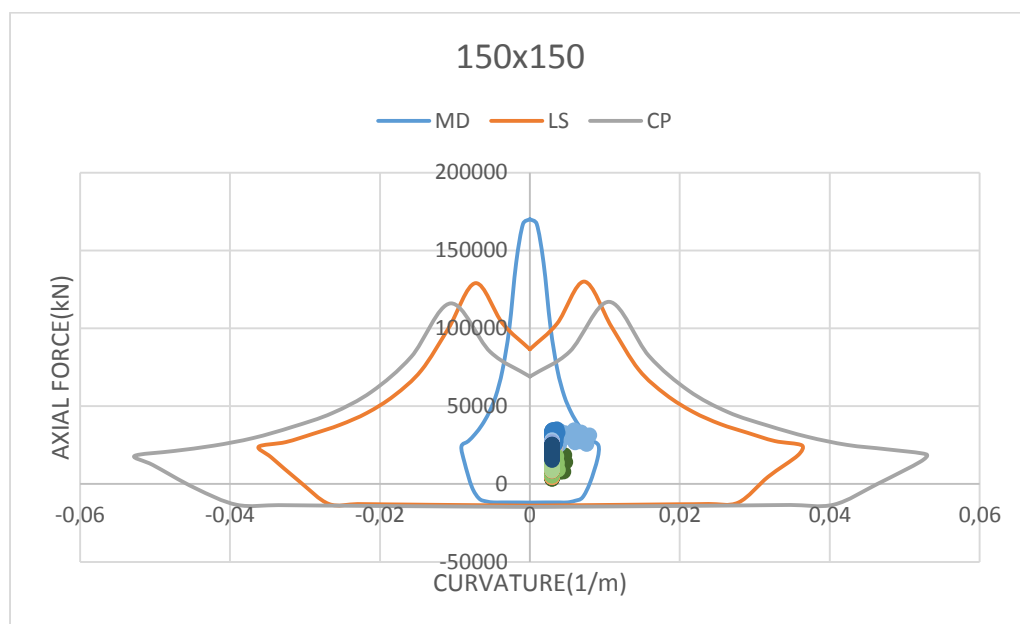


Figure 5.24: Axial force-curvature relationships for 150x150 of columns.

5.1.7. Shear Checks for Shear Walls

Shear forces are obtained to provide shear strength which is defined as Area Section Cut by Perform 3D. Shear forces are computed for all shear walls to obtain and provide performance demand limit for shear strength as limits are shown by Eq. (5.5).

$$V_e = 0.85 \times A_c h \times \sqrt{f_{ck}} \quad (5.5)$$

The maximum shear force that a section can bear by transverse reinforcement is defined in Eq. (5.6).

$$V_r = A_c h (0.65 f_{ctd} + \rho_{sh} f_{ywd}) \quad (5.6)$$

Shear limits are shown at Figure 5.25-5.28 which are the smallest value, with the maximum shear forces acting on piers. 1.3 times the average shear forces are also plotted.

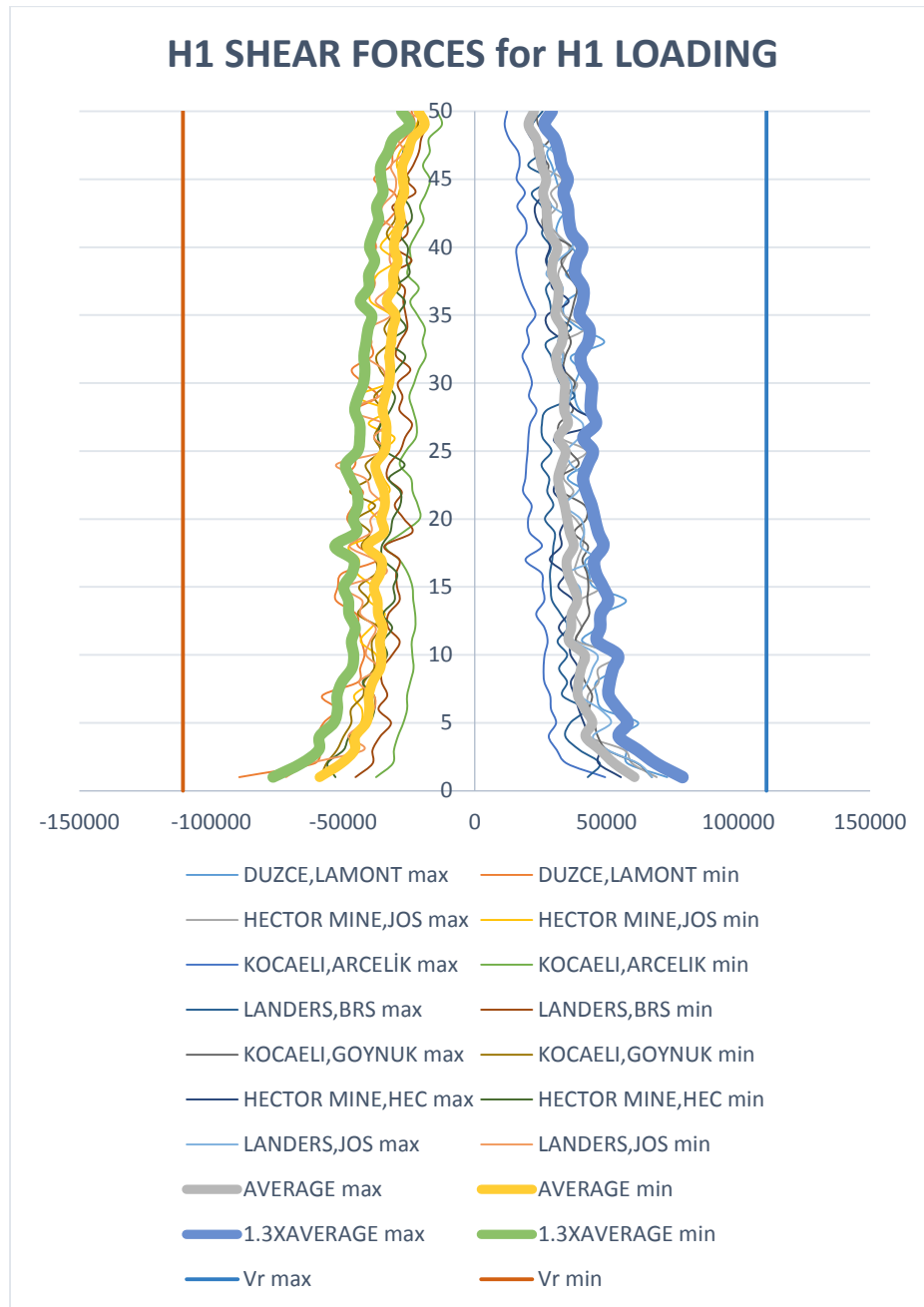


Figure 5.25: H1 shear checks of shear walls at H1 loading.

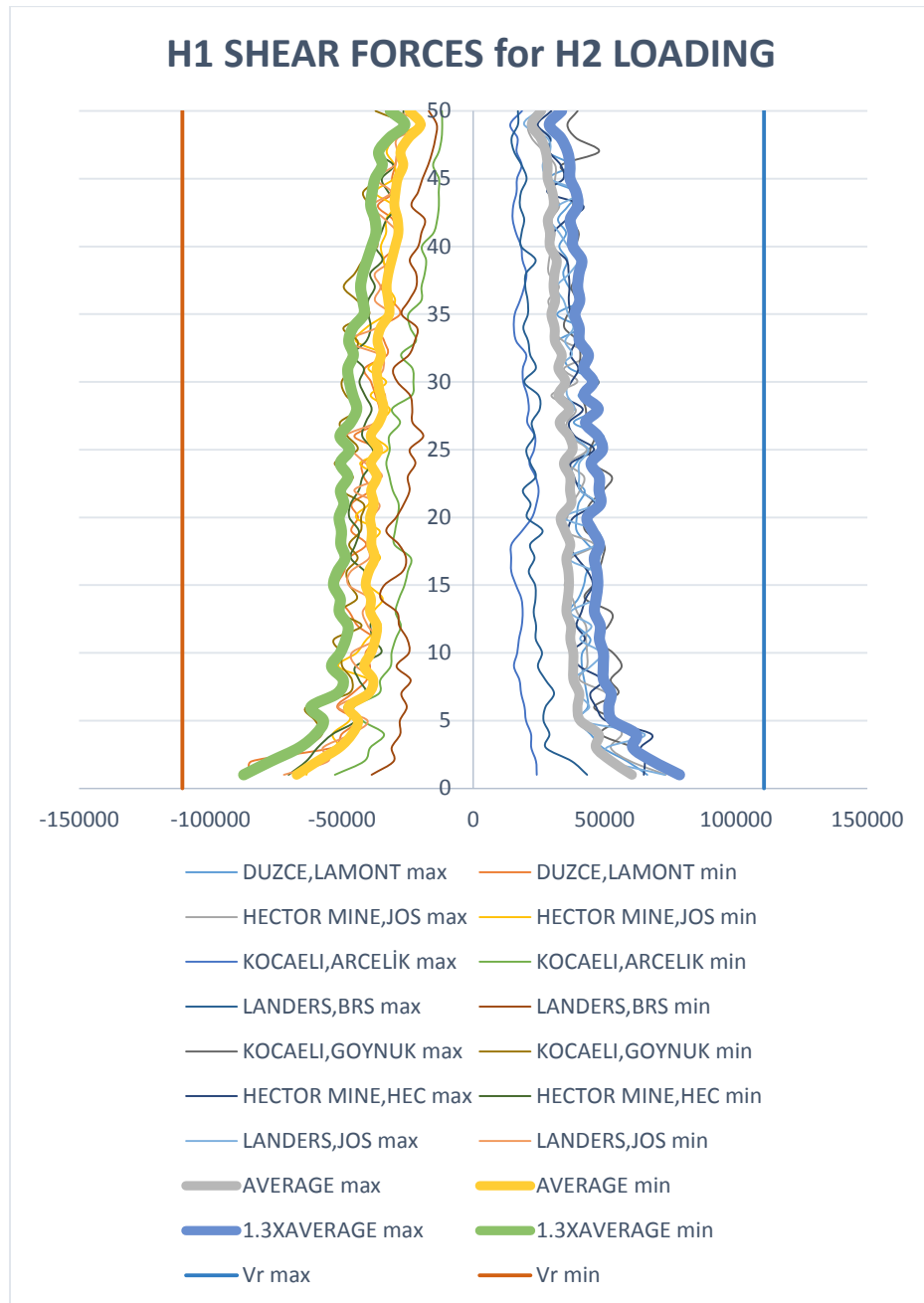


Figure 5.26: H1 shear checks of shear walls at H2 loading.

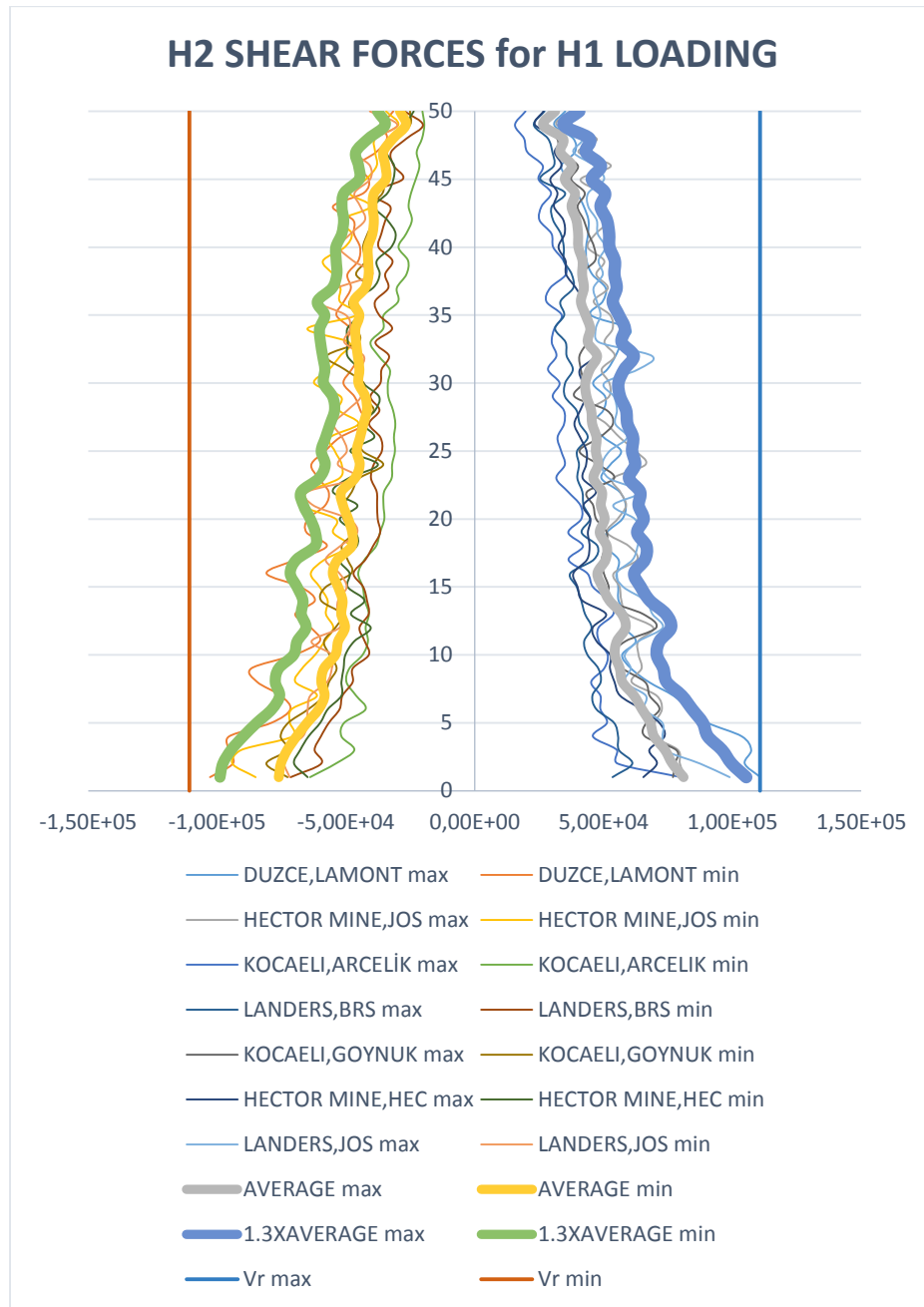


Figure 5.27: H2 shear checks of shear walls at H1 loading.

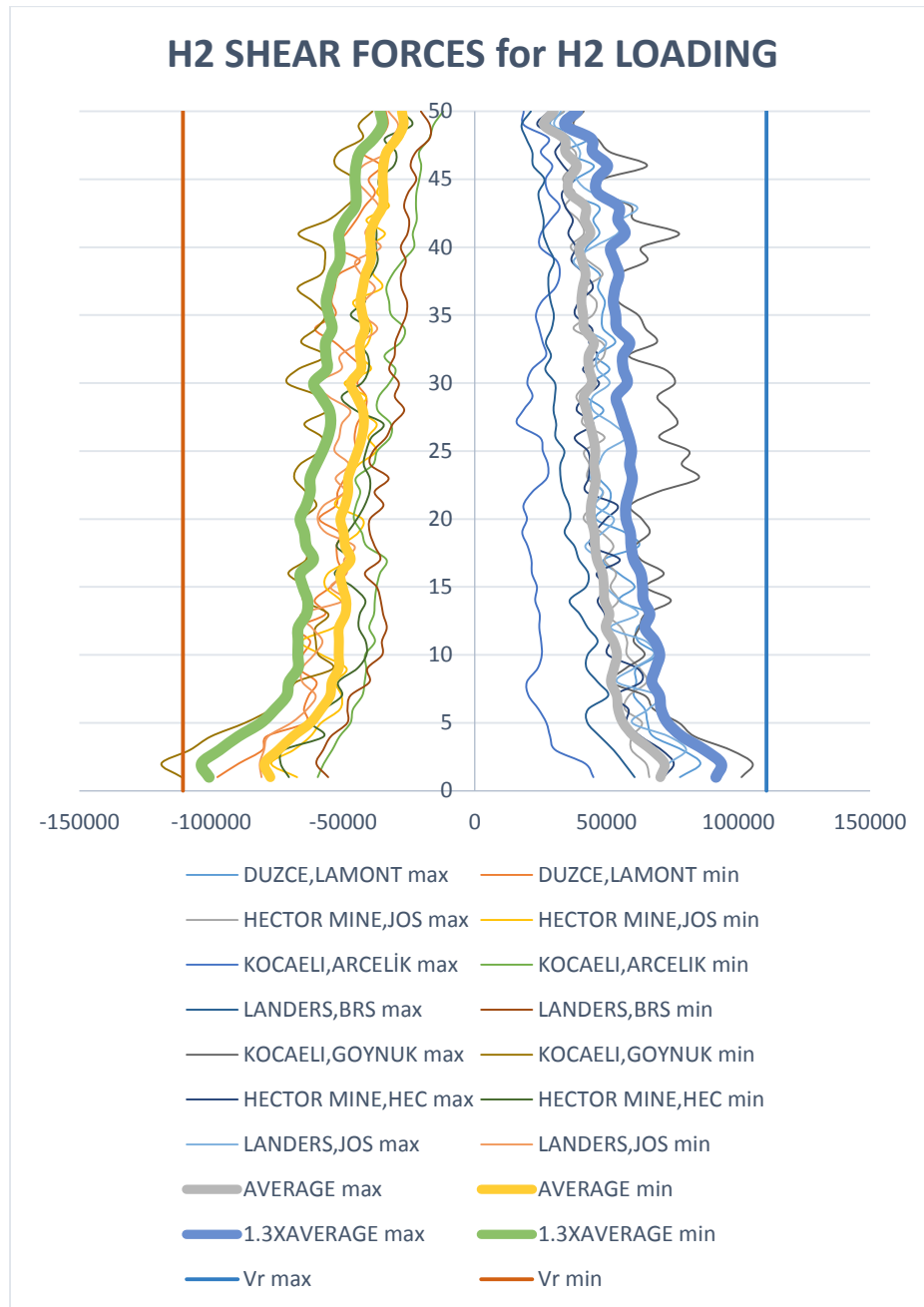


Figure 5.28: H2 shear checks of shear walls at H2 loading.

5.1.8. Shear Checks for Beams

Shear forces are obtained to provide shear strength. Shear forces are computed for selected beams which is explained for performance checks for beams and provide performance demand limit for shear strength as limits are shown by formula as follows,

$$V_e = 0.85 \times bw \times d \times \sqrt{f_{ck}} \quad (5.7)$$

The maximum shear force that a section can bear by transverse reinforcement is defined in Eq. (5.8) – (5.11).

$$V_r = V_c + V_w \quad (5.8)$$

$$V_c = 0.8V_{cr} \quad (5.9)$$

$$V_{cr} = 0.65 \times f_{ctd} \times bw \times d \quad (5.10)$$

$$V_w = \frac{A_{sw}}{s} f_{ywd} \times d \quad (5.11)$$

Shear limits are shown at Figure 5.29-5.32 which are the smallest value, with the maximum shear forces acting on piers. 1.3 times the average shear forces are also plotted.

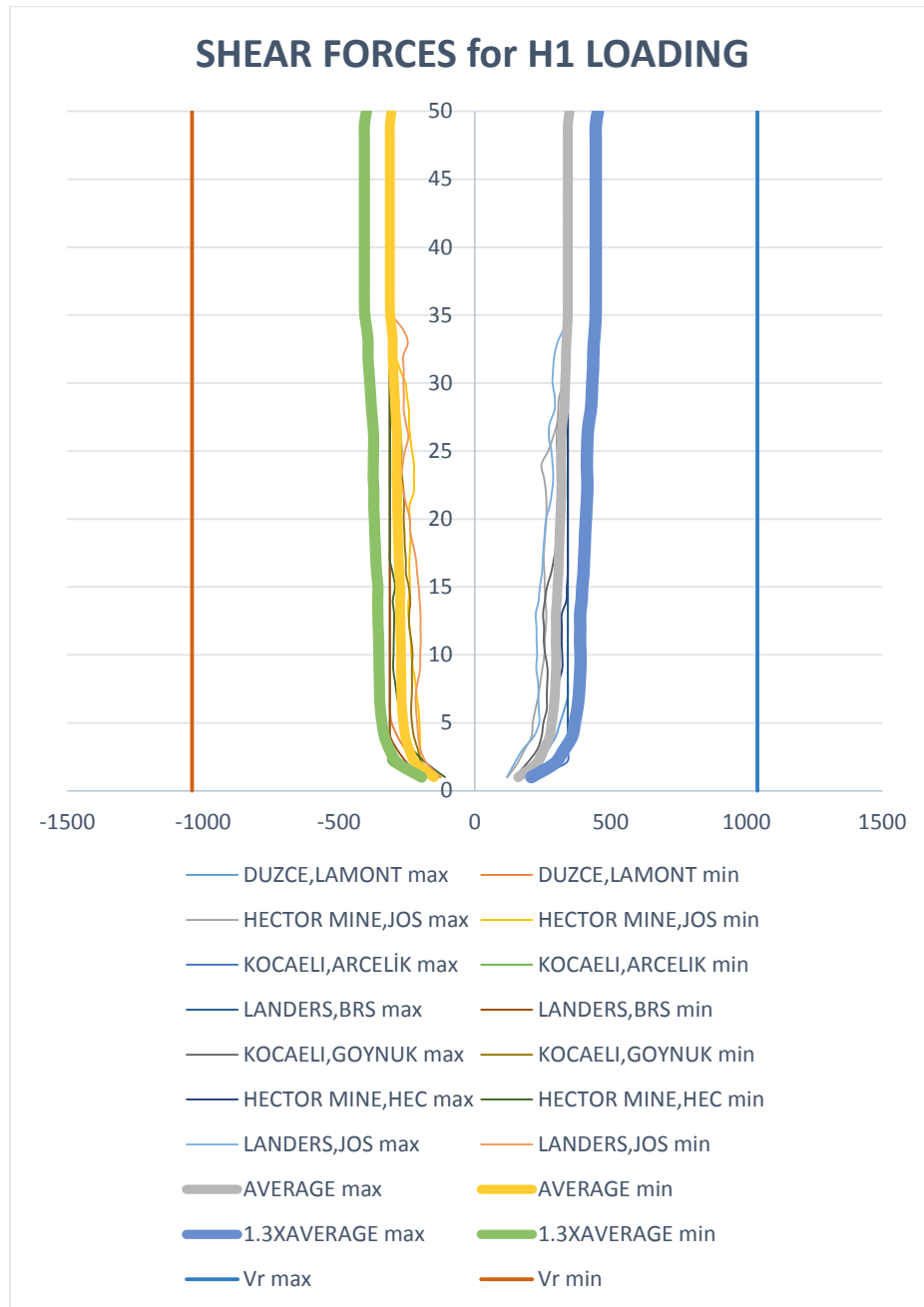


Figure 5.29: Shear checks of beams at I end for H1 loading.

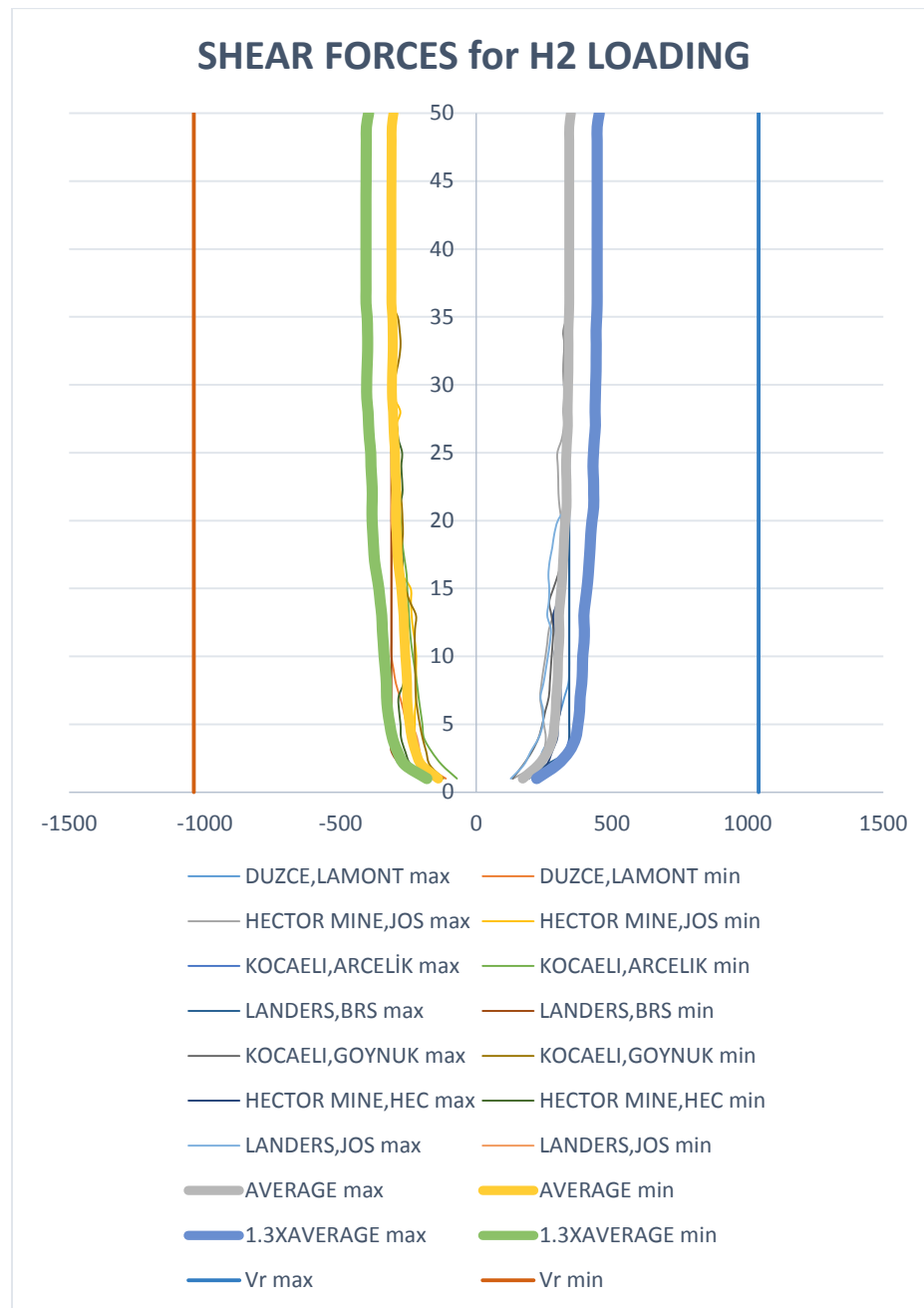


Figure 5.30: Shear checks of beams at I end for H2 loading.

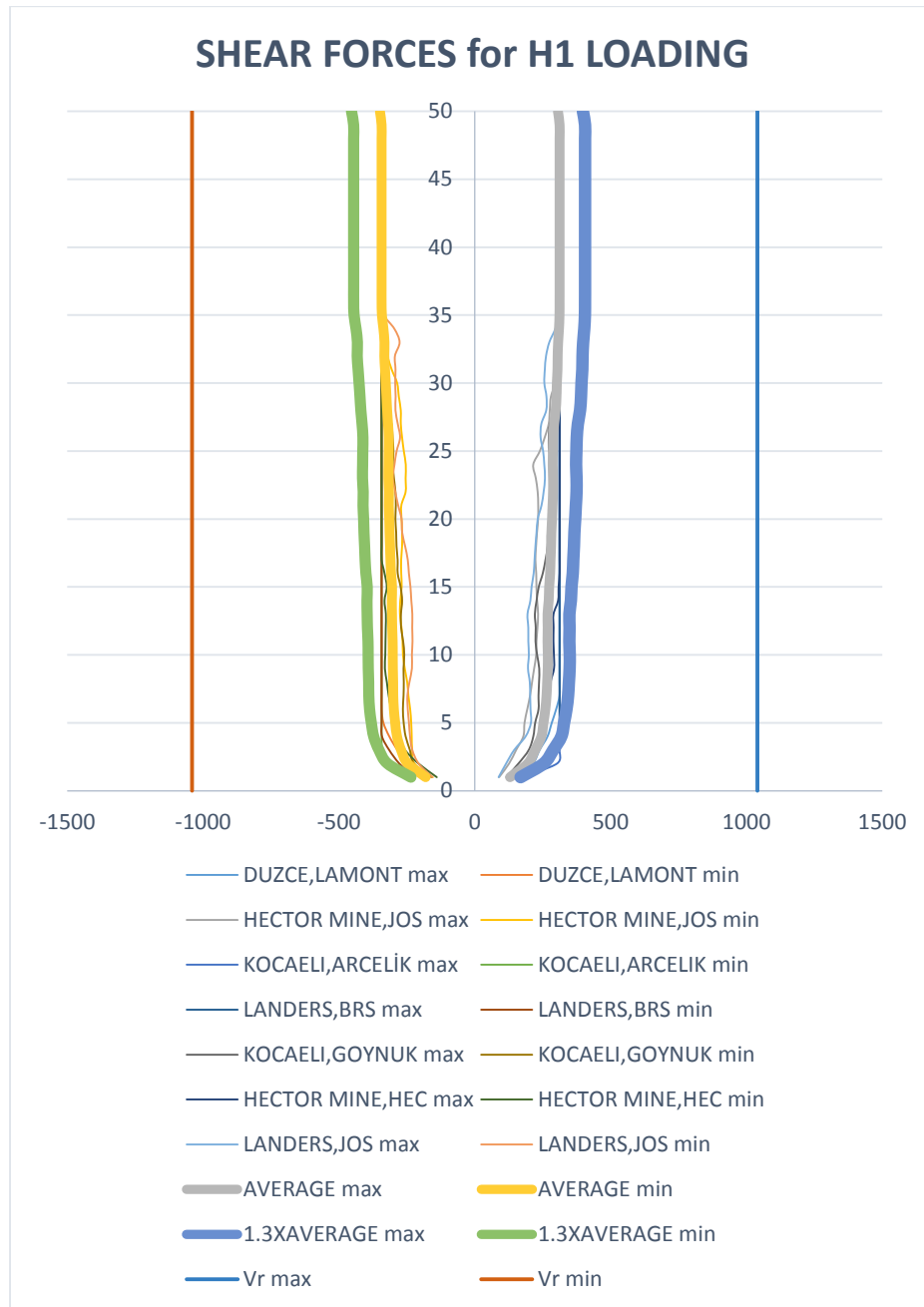


Figure 5.31: Shear checks of beams at J end for H1 loading.

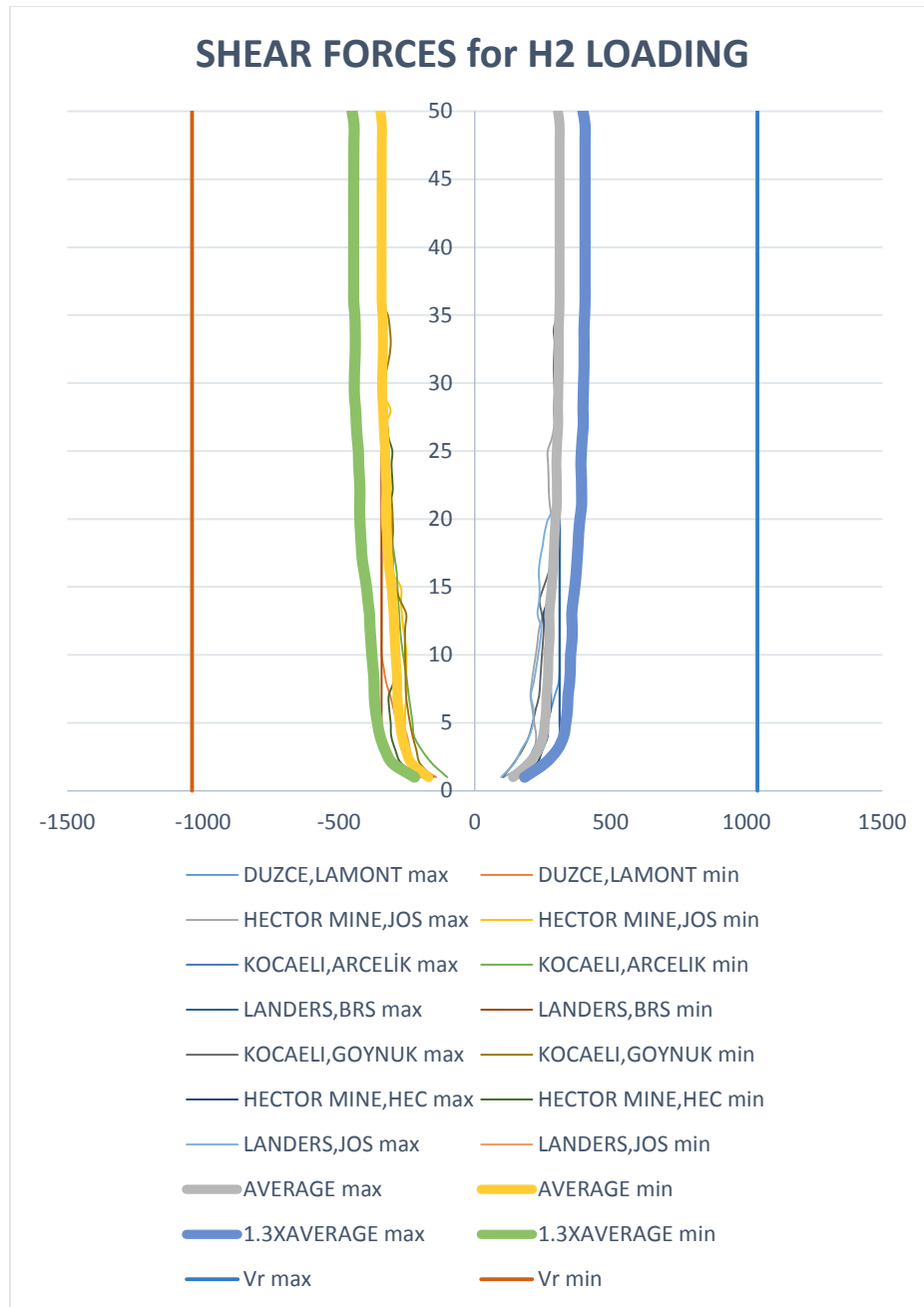


Figure 5.32: Shear checks of beams at J end for H2 loading.

5.1.9. Shear Checks for Columns

Shear forces are obtained to provide shear strength. Shear forces are computed for all column which is explained for performance checks for columns to obtain and provide performance demand limit for shear strength as limits are shown by Eq. (5.12).

$$V_e = 0.85 \times b_w \times d \times \sqrt{f_{ck}} \quad (5.12)$$

The maximum shear force that a section can bear by transverse reinforcement is defined in Eq. (5.13) – (5.16).

$$V_r = V_c + V_w \quad (5.13)$$

$$V_c = 0.8V_{cr} \quad (5.14)$$

$$V_{cr} = 0.65 \times f_{ctd} \times b_w \times d \quad (5.15)$$

$$V_w = \frac{A_{sw}}{s} f_{yw} d \times d \quad (5.16)$$

Shear limits are shown at Figure 5.33-5.36 which are the smallest value, with the maximum shear forces acting on piers. 1.3 times the average shear forces are also plotted.

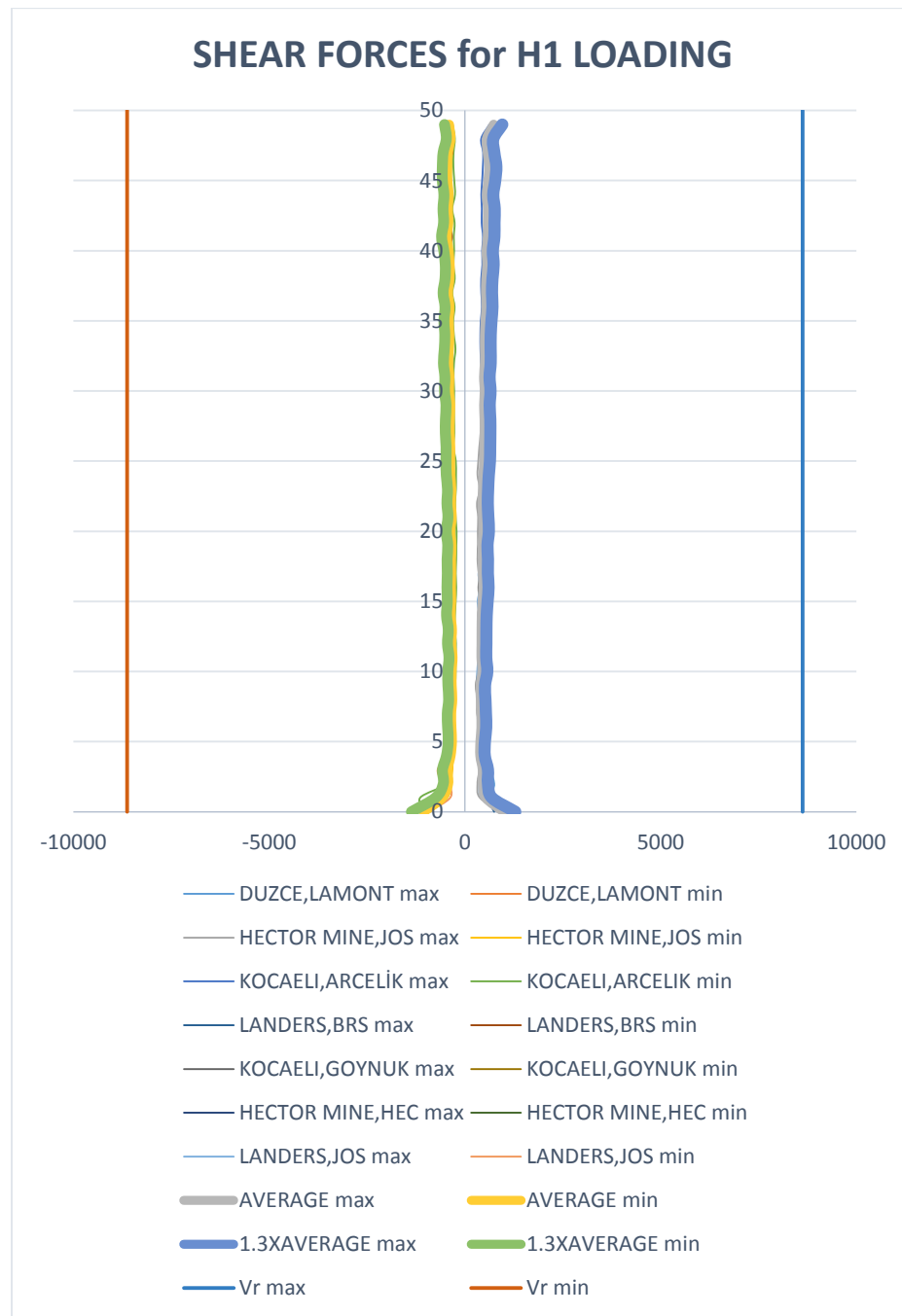


Figure 5.33: Shear checks of columns of 120X120 sections at H1 loading.

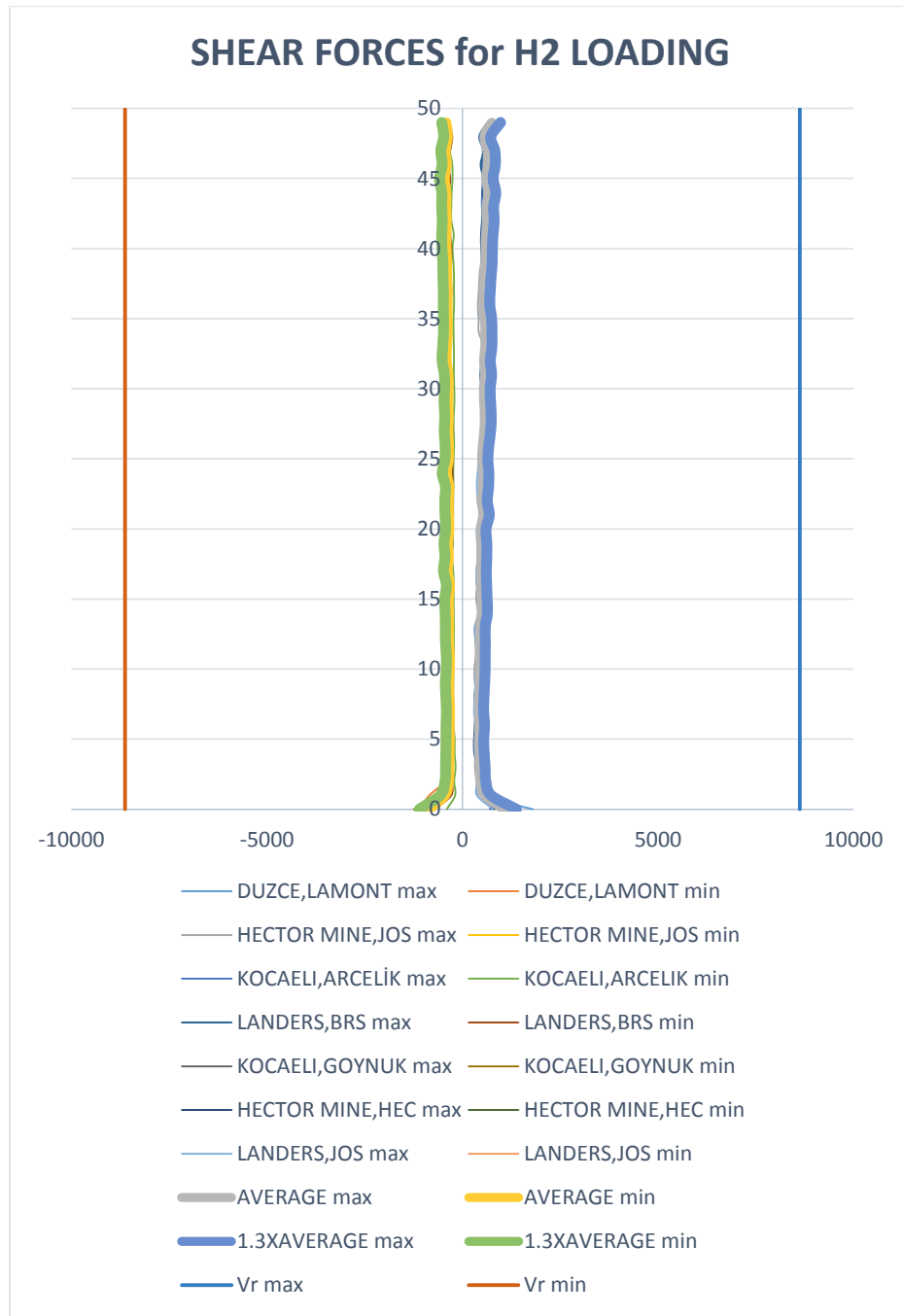


Figure 5.34: Shear checks of columns of 120X120 sections at H2 loading.



Figure 5.35: Shear checks of columns of 150X150 sections at H1 loading.



Figure 5.36: Shear checks of columns of 150X150 sections at H2 loading.

5.2.Results for Buckling-Restrained Braced Frame Structural System

In this part, performance levels of members of tall building with BRBFs are presented under DD-1 Level Earthquake (MCE) which ground motions having a 2% probability of being exceeded in 50 years (2475-year return period).

The results for tall building with BRBFs are investigated and occurred by performance levels of members as follows;

- Modal analysis results
- Energy diagrams
- Drift checks
- Hyteresis loops

5.2.1.Modal Analysis Results

The first 6 mode shapes of the structural model are given in Figure 5.37-5.38. The effective mass ratio of the first 12 modes is given in Table 5.2. The mode shapes found in perform 3D model are in line with mode shapes found in ETABS. Only the structural periods are shorter which is to be expected since the ETABS model the wall are defined with effective crack section stiffness where the cracking is explicitly modelled in Perform 3D with fiber elements which uses the uncracked stiffness of the wall elements in the calculation of free vibration modes and mode-shapes.

Table 5.2: Modal properties of tall building used BRBs.

Mode	Period	UX	UY	Sum UX	Sum UY
1	4.33	0.69	0.00	68.88	0.00
2	4.01	0.00	0.67	68.88	67.04
3	2.38	0.00	0.00	68.88	67.06
4	1.23	0.19	0.00	87.58	67.06
5	1.04	0.00	0.19	87.58	85.58
6	0.84	0.00	0.00	87.58	85.74
7	0.60	0.03	0.00	90.90	85.74
8	0.49	0.00	0.04	90.90	90.03
9	0.44	0.00	0.00	90.90	90.20
10	0.38	0.02	0.00	92.94	90.20
11	0.29	0.00	0.02	92.94	92.57
12	0.27	0.00	0.00	92.94	92.71

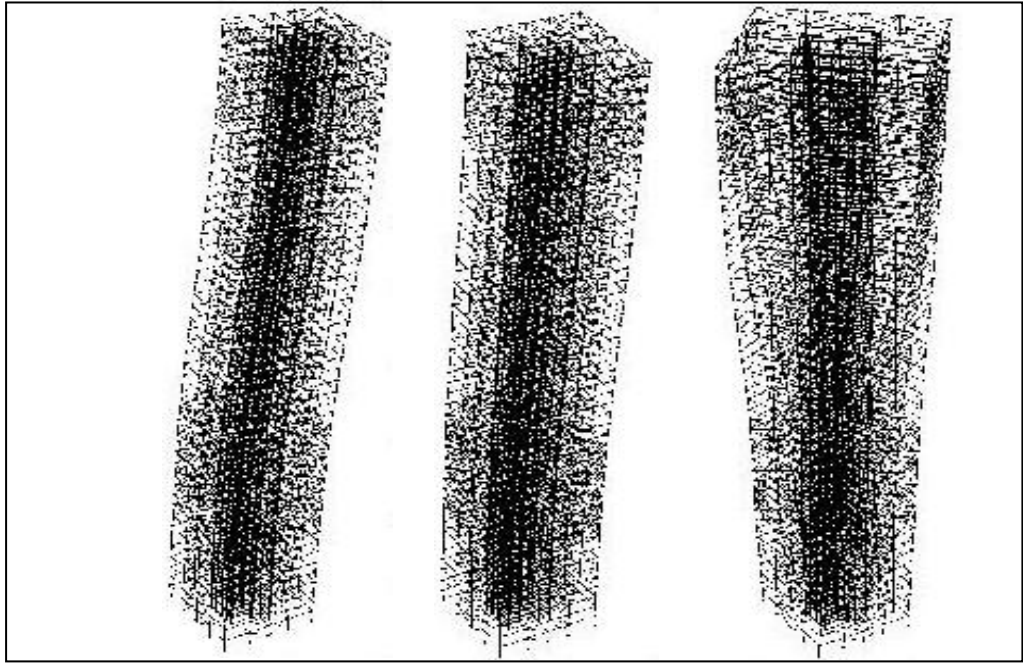


Figure 5.37: First 3 mode shapes at each direction of tall building with BRBs.

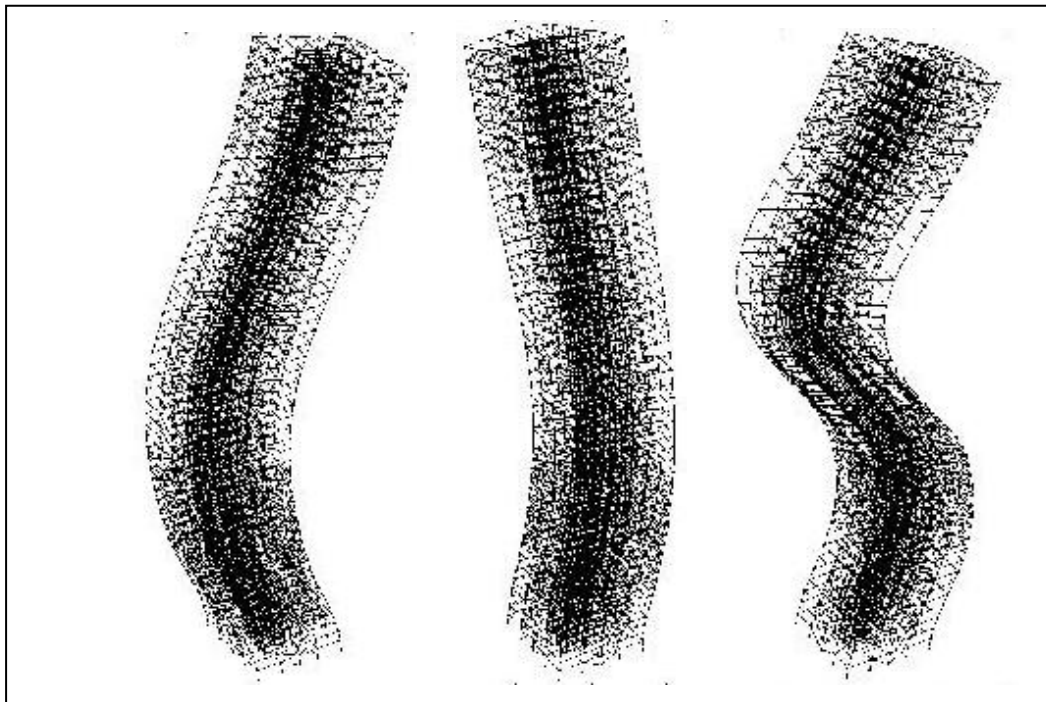


Figure 5.38: 4,5 and 6th mode shapes at each direction of tall building with BRBs.

5.2.2. Energy Diagrams

Energy diagrams show the inelastic energy distributions due to total energy, kinetic energy, viscous damping energy and structural behavior which is occurred by earthquake ground motions.

In the figure as it is shown, red, green, yellow, orange, cyan and blue colours represent inelastic energy, Beta-K viscous energy, Alpha-M viscous energy, modal damping energy, strain energy and kinetic energy, respectively so the energy diagrams provide to obtain energy balance of structure under ground motions as it is shown at figure as Hector Mine ground motion.

However, performance level can be obtained by energy diagrams as it is shown for El Centro ground motion, is inelastic energy which is depending on plastic hinges results of structural members, is stayed between 10% and 15% of total energy. It's clearly that performance level is provided against DD-1 Level Earthquake for collapse prevention as it is demonstrated previous topics.

All explanations about energy diagrams are shown as figures at Figure 5.39-5.43.

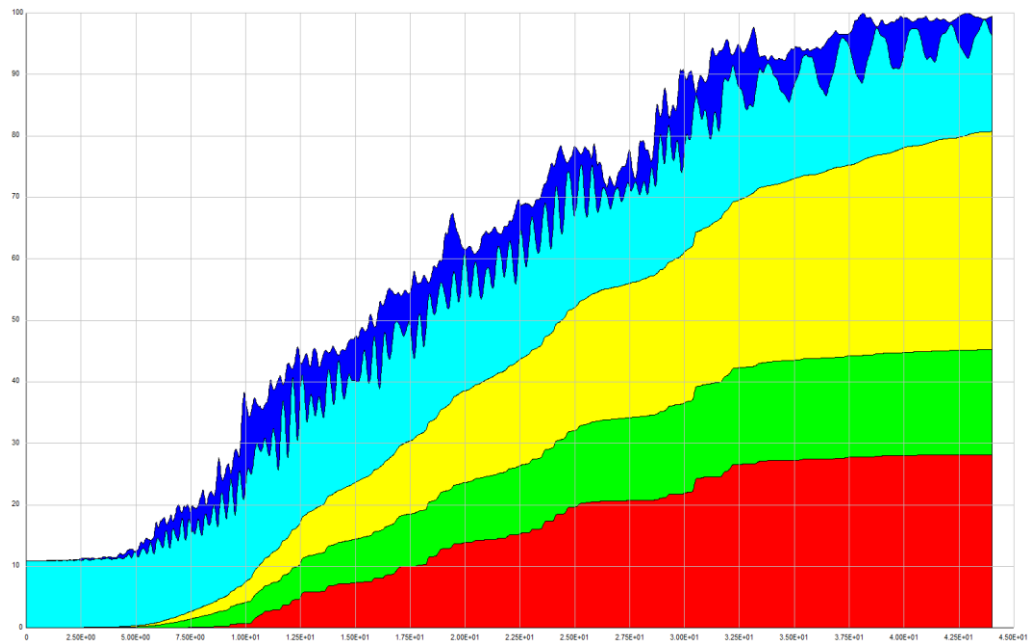


Figure 5.39: Composed total energy of typical tall building with BRBs for El Centro Ground Motion.

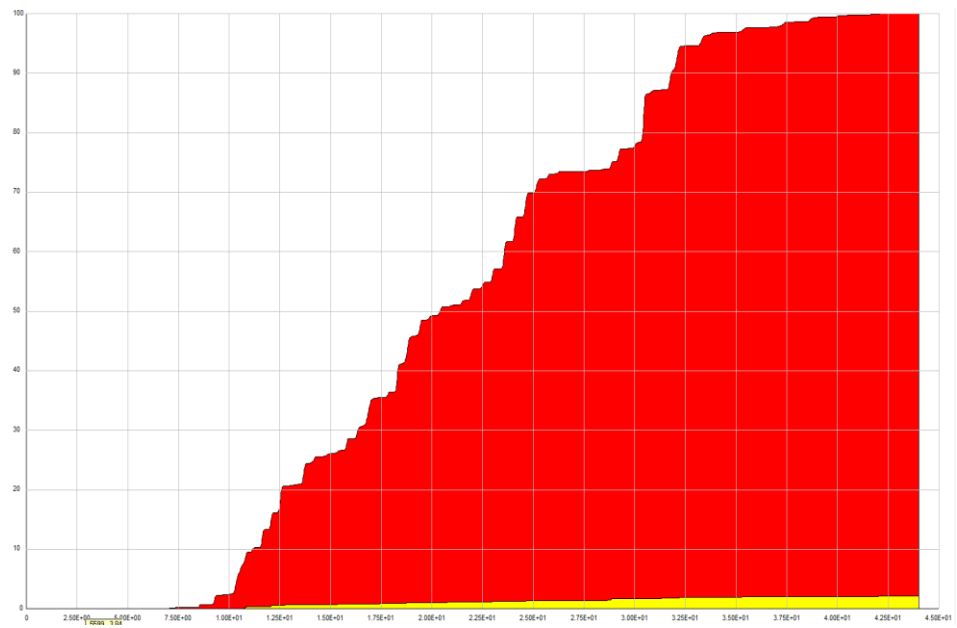


Figure 5.40: Inelastic energy of shear walls about 2%.

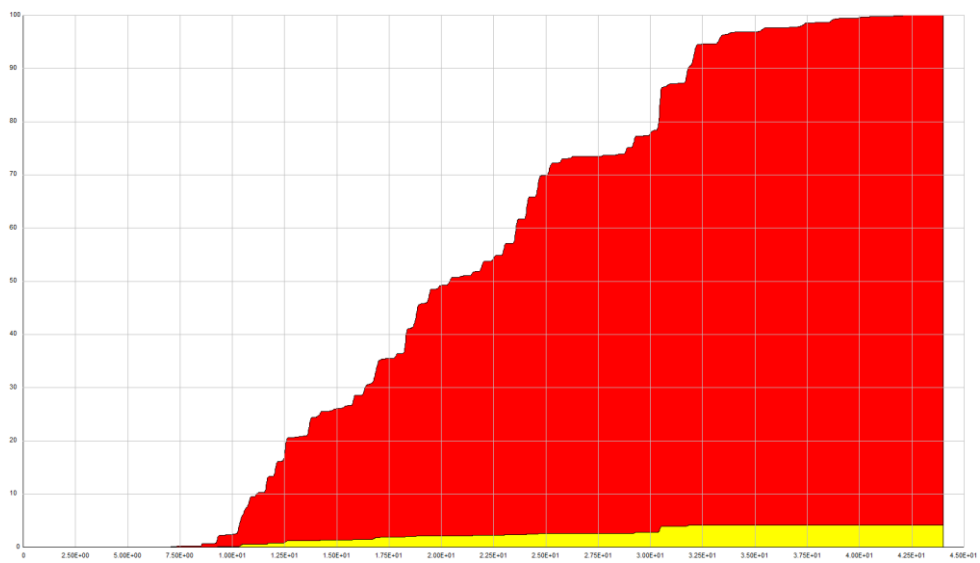


Figure 5.41: Inelastic energy of beams about 5%.

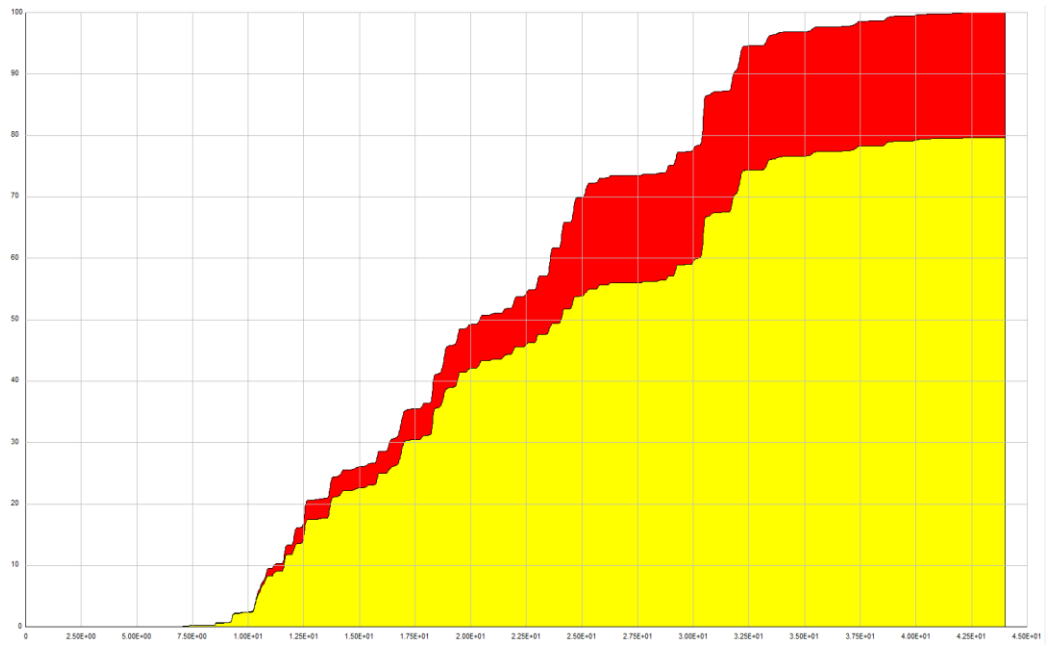


Figure 5.42: Inelastic energy of columns about 78%.

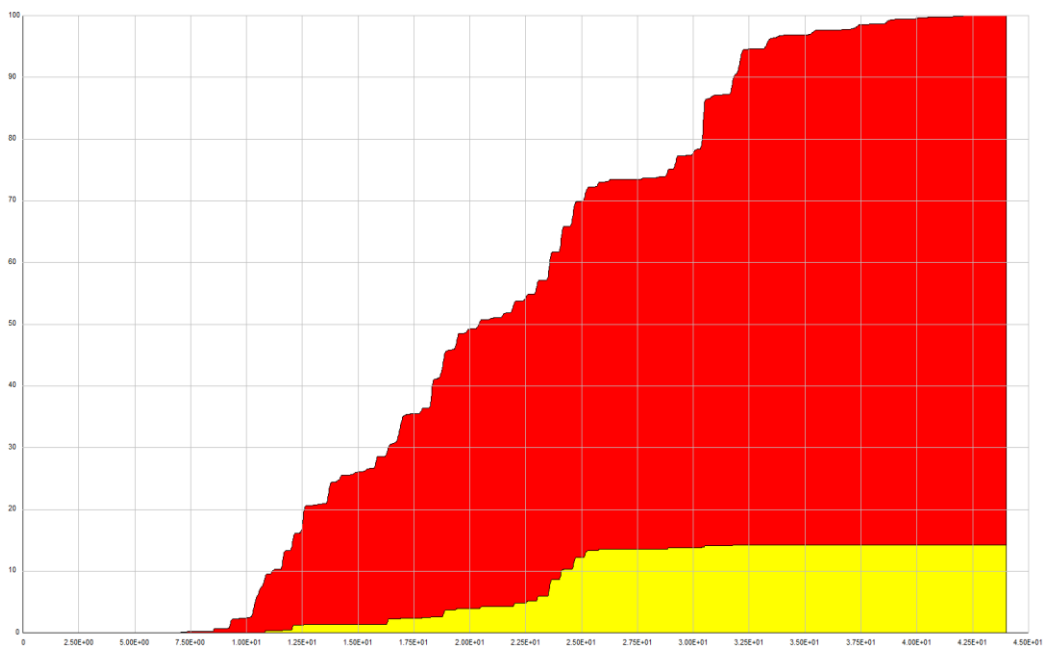


Figure 5.43: Inelastic energy of BRBF about 15%.

5.2.3. Drift Checks

The story drift ratios based on the average of nonlinear time-history analysis for the MCE plotted for the given corner location at storey. Drift for tall building used BRBFs are shown that all the given story drift ratios that are shown in the following Figure 5.44-5.47 are less than 3% of average of drifts or less than 4.5% story drift ratio for each story, which is provided in TBSC 2016.

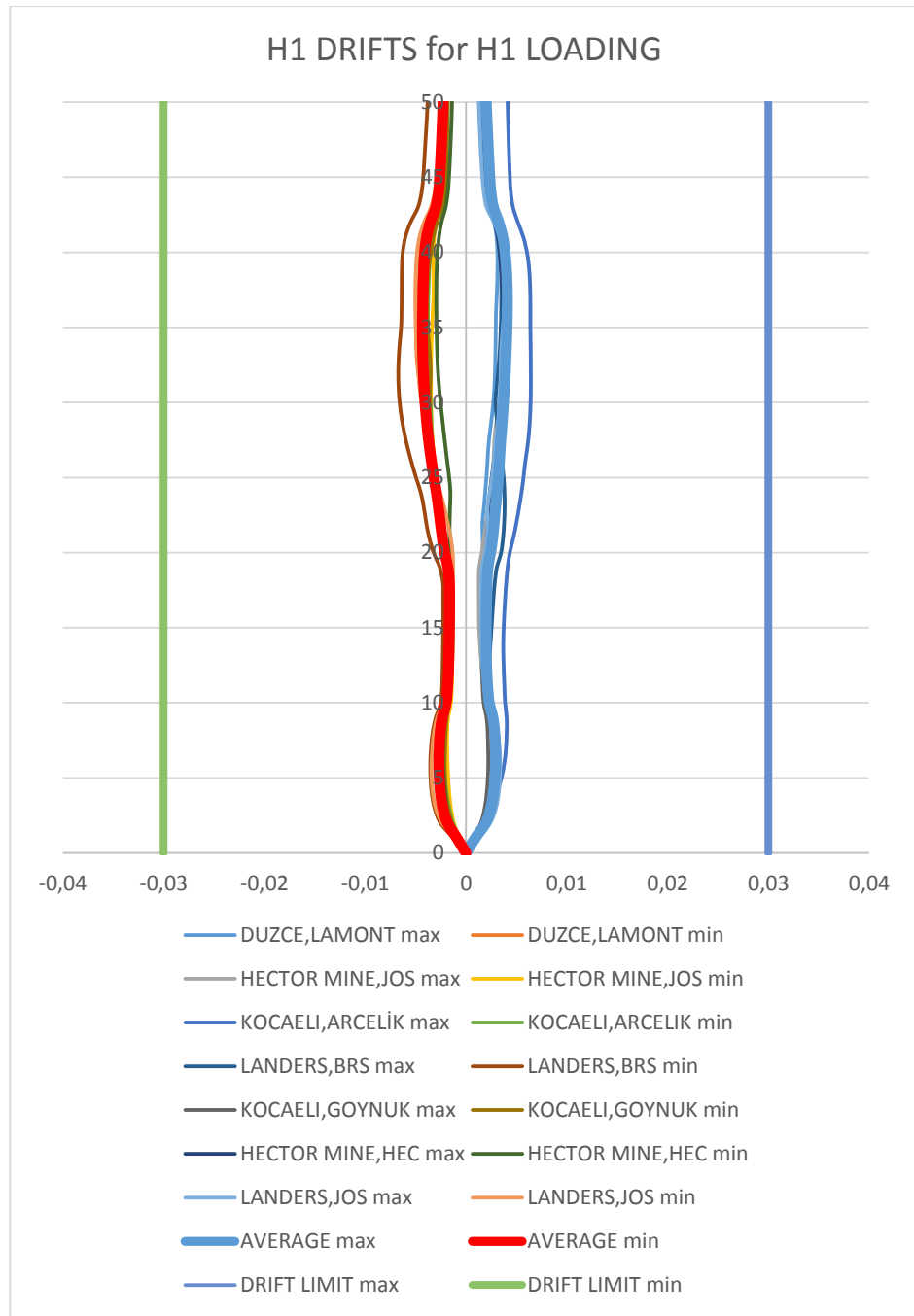


Figure 5.44: Drift H1 for H1 loading of tall building used BRBFs.

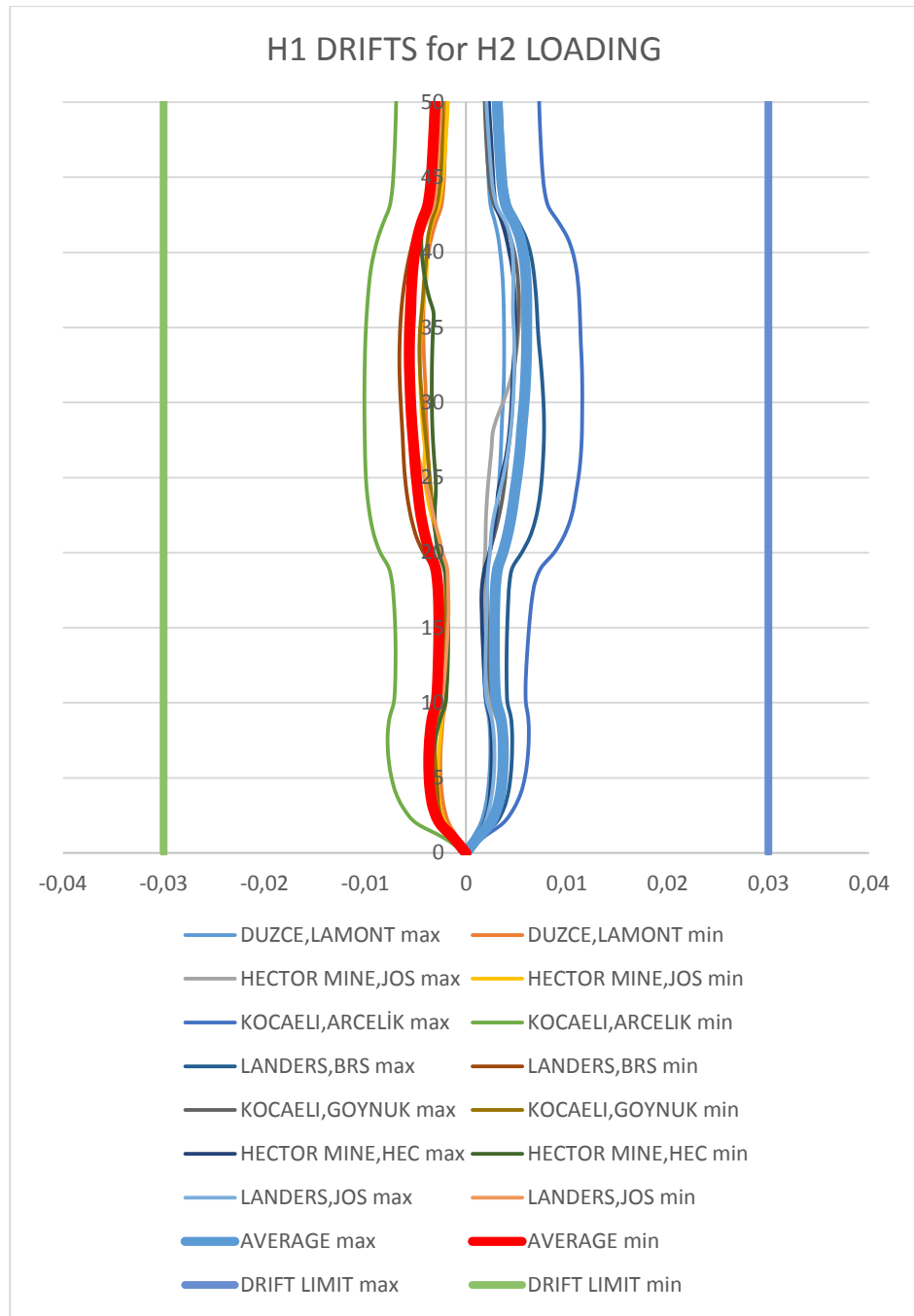


Figure 5.45: Drift H1 for H2 loading of tall building used BRBFs.

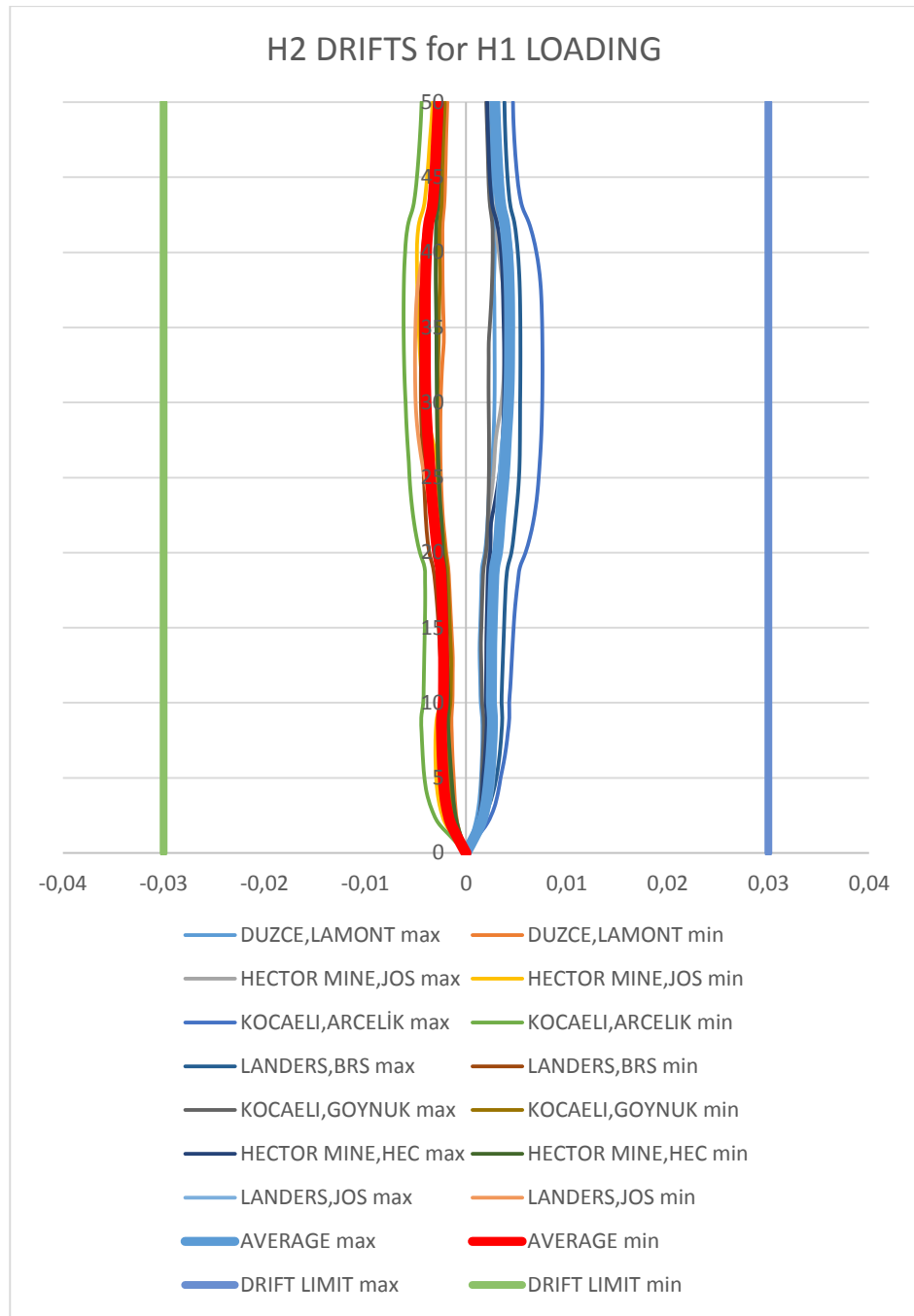


Figure 5.46: Drift H2 for H1 loading of tall building used BRBFs.

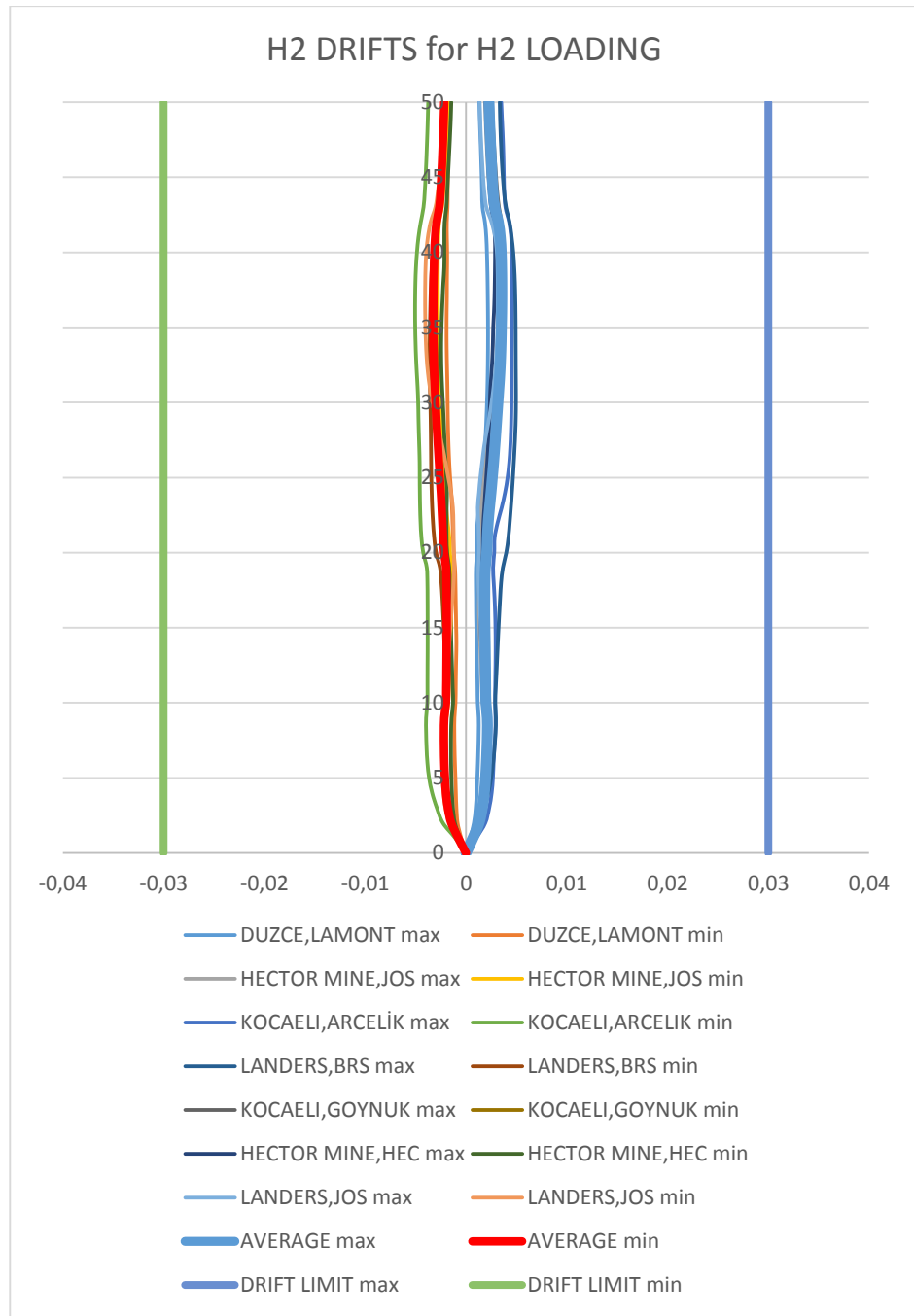


Figure 5.47: Drift H2 for H2 loading of tall building used BRBFs.

5.2.4.Hysteresis Loops

In this part, one Buckling-Restrained Brace Frame is chosen for hysteresis loops against ground motions as it is shown at Figure 5.48.

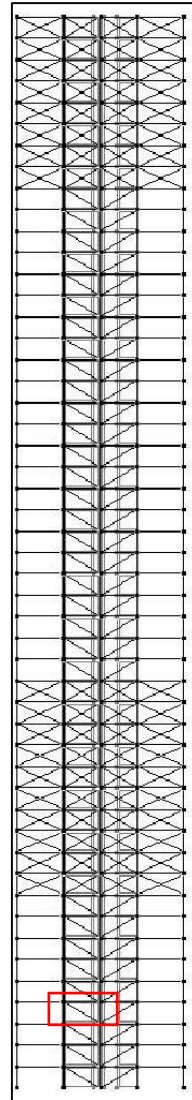


Figure 5.48: Selected BRBF for hysteresis loop.

The designing and modeling of BRBF is explained in previous topics and presented at Table 5.3 that is defined as Type 3.

Table 5.3: Hinge Properties of Type 3.

TYPE 3 AXIAL HINGE PROPERTIES									
TENSION					COMPRESSION				
$\omega R_y F_y A_s$	$R_y F_y A_s$	$\Delta(\text{mm})$	$10\Delta(\text{mm})$	$20\Delta(\text{mm})$	$R_y F_y A_s$	$\beta \omega R_y F_y A_s$	$\Delta(\text{mm})$	$10\Delta(\text{mm})$	$20\Delta(\text{mm})$
10643.75	8515.00	5.91	59.15	118.29	8515.00	11708.13	4.73	47.32	94.64

Hysteresis loops should be defined and checked to obtain nonlinear structural behavior which is called force-displacement relationships. The properties of first yield, strain hardening, ultimate strength, ductile limit, strength loss, residual strength and initial stiffness of structural members can be obtained by hysteresis loops. Axial force-axial displacements are shown at Figures 5.49-5.51 for BRBF of Type 3.

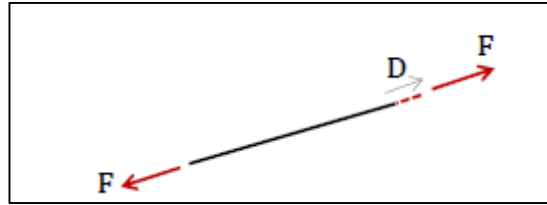


Figure 5.49: F-D relationship for a component.

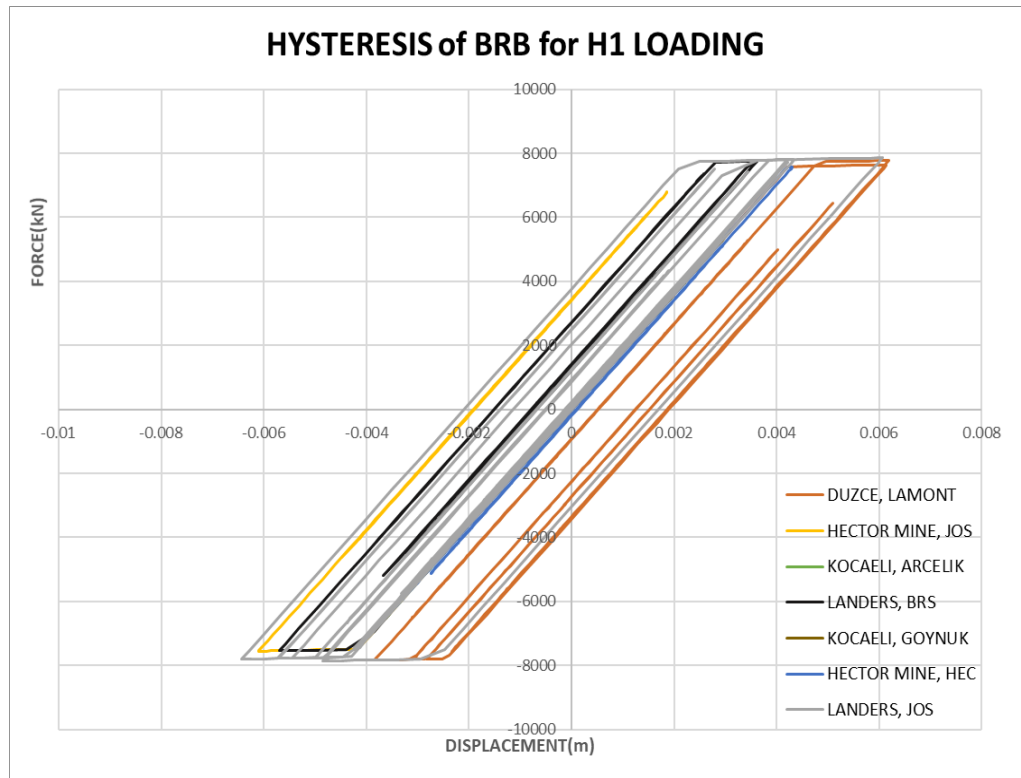


Figure 5.50: Hysteresis loop for H1 loading.

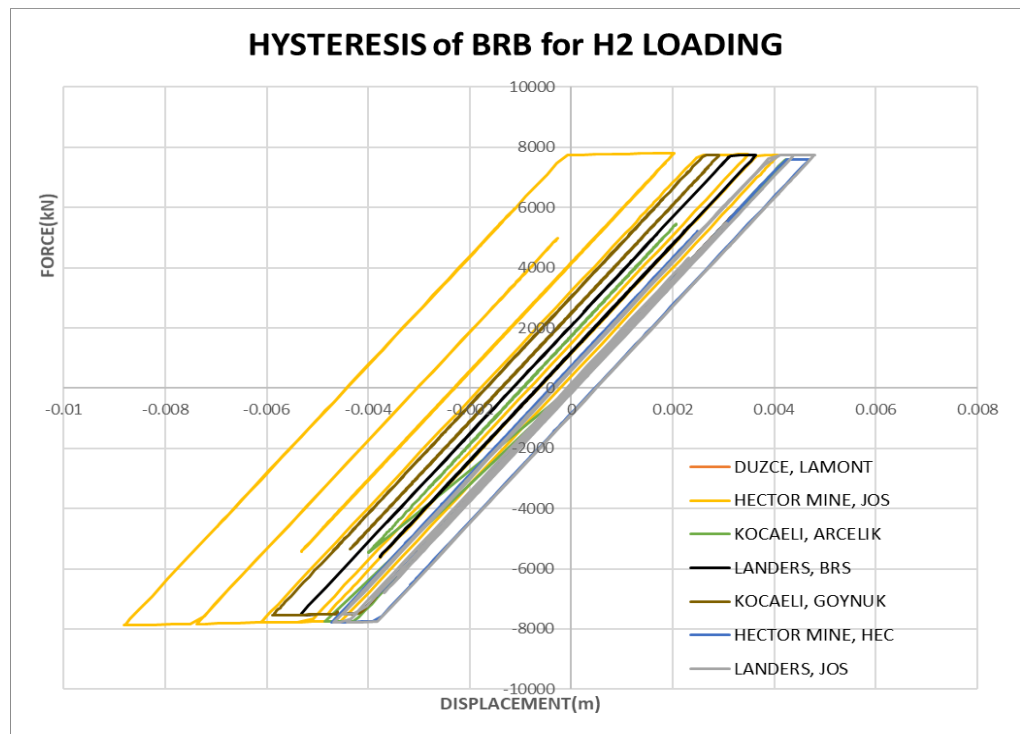


Figure 5.51: Hysteresis loop for H2 loading.

5.3.Comparison between Two Types of Tall Buildings

In this part, all conclusion of analysis for two types of tall building are presented. The comparison of structural behaviour of 2 types of tall buildings are explained under DD-1 Level Earthquake (MCE) with ground motions having a 2% probability of being exceeded in 50 years (2475-year return period).

Comparison of the results for two types of tall buildings are considered as follows;

- Modes
- Energy Diagrams
- Drifts

5.3.1.Modes

The first 6 mode shapes of the two types of structural models are compared and the effective mass ratio of the first 12 modes are given in Table 5.4.

Table 5.4: Modal Properties of 2 Types of Tall Buildings.

Mode	TALL BUILDINGS					
	TYPICAL			USED BRBF		
	Period sec	Sum UX	Sum UY	Period sec	Sum UX	Sum UY
1	7.50	70.46	0.01	4.325	68.88	0.001665
2	7.04	70.47	66.32	4.008	68.88	67.04
3	4.90	70.47	66.61	2.384	68.88	67.06
4	2.10	84.25	66.61	1.229	87.58	67.06
5	1.71	84.25	81.26	1.035	87.58	85.58
6	1.53	84.25	83.09	0.844	87.58	85.74
7	1.01	89.33	83.09	0.6	90.9	85.74
8	0.87	89.33	83.49	0.485	90.9	90.03
9	0.73	89.33	89.16	0.435	90.9	90.2
10	0.62	92.12	89.16	0.377	92.94	90.2
11	0.57	92.12	89.26	0.294	92.94	92.57
12	0.43	92.12	92.13	0.273	92.94	92.71

5.3.2. Energy Diagrams

Energy diagrams show the inelastic energy distributions due to total energy, kinetic energy, viscous damping energy and structural behavior which is occurred by earthquake ground motions.

In the Table 5.5, energy diagrams for two buildings are given to compare between typical and used BRBF of tall buildings. It is clearly that the effectiveness and behavior of BRBF are understood by energy diagrams under several ground motions.

Table 5.5: Energy diagrams of 2 types tall buildings.

	EARTHQUAKE	TYPICAL		USED BRBF	
		H1 LOADING (kNm)	H2 LOADING (kNm)	H1 LOADING (kNm)	H2 LOADING (kNm)
1	DUZCE, LAMONT	43140	41700	32120	26620
2	HECTOR MINE, JOS	24560	27460	43780	47250
3	KOCAELİ, ARCELİK	110100	65360	71840	108300
4	LANDERS, BRS	62430	66380	89030	108300
5	KOCAELİ, GOYNUK	29370	21390	24680	40340
6	HECTOR MINE, HEC	28740	29220	27580	32020
7	LANDERS, JOS	31440	30600	69140	53280

5.3.3. Drifts

The story drift ratios based on the average of nonlinear time-history analysis for the MCE plotted for the given corner location at each storey. Drift for typical tall building and tall building used BRBFs are presented at Table 5.6-5.7.

Maximum values of drifts are taken under each ground motions and the averages to compare between typical and used BRBF of tall buildings as it is shown at Table 5.6-5.7.

Table 5.6: Drifts for typical building.

	TYPICAL			
	H1 DRIFT		H2 DRIFT	
	H1 LOADING (m)	H2 LOADING (m)	H1 LOADING (m)	H2 LOADING (m)
EARTHQUAKE				
DUZCE, LAMONT	0.0089	0.0087	0.0064	0.006
HECTOR MINE, JOS	0.0062	0.0085	0.0052	0.0059
KOCAELİ, ARCELİK	0.01	0.026	0.02	0.006
LANDERS, BRS	0.0097	0.013	0.011	0.011
KOCAELİ, GOYNUK	0.0064	0.0087	0.0064	0.0045
HECTOR MINE, HEC	0.008	0.012	0.0062	0.0063
LANDERS, JOS	0.0096	0.021	0.0084	0.0054
AVERAGE	0.00766	0.013	0.009	0.006

Table 5.7: Drifts for tall building used BRBFs.

	USED BRBF			
	H1 DRIFT		H2 DRIFT	
	H1 LOADING (m)	H2 LOADING (m)	H1 LOADING (m)	H2 LOADING (m)
EARTHQUAKE				
DUZCE, LAMONT	0.005	0.0042	0.003	0.0022
HECTOR MINE, JOS	0.0042	0.006	0.005	0.004
KOCAELİ, ARCELİK	0.0064	0.012	0.0076	0.005
LANDERS, BRS	0.007	0.008	0.0054	0.005
KOCAELİ, GOYNUK	0.0042	0.0052	0.003	0.004
HECTOR MINE, HEC	0.004	0.005	0.004	0.005
LANDERS, JOS	0.005	0.006	0.005	0.004
AVERAGE	0.0043	0.006	0.0043	0.0036

6.CONCLUSION

Nonlinear time history analysis results with 7 ground motion records for each orthogonal direction were computed and compared with the limits of strain that in TBSC 2016 for columns and beams. For the shear walls, strains were calculated in joints as it is explained at previous chapters, where code limits and calculated of strains were shown. In beams, the values of strains were calculated in terms of rotations depending on moment-curvature relationships. Also, in columns, the values of strains were calculated by equation of transformation from rotations to curvatures. All these limits were presented in detail for two types of tall buildings that are typical and with BRBFs, respectively.

In the view of the results, the nonlinear strain limits of the shear strength, drifts and ductile members of structures were investigated in the structure expected to provide the target performance level of “Collapse Prevention” under DD-1 (MCE) level earthquake with a probability of exceeding 2% in the expected 50 years. The required performance level of the “Collapse Prevention” for drifts against chosen ground motions was provided for all structures. In the typical structure, which was designed by core shear walls with frames structural system, it was shown that the members of the structure have ductile strength against shear forces brittle to have ductile behavior under bending forces of the shear walls and frames. “Shear Strength” is provided which all members of structure behave linear in these brittle fracture modes.

It can easily be concluded that the performance level in all members in the whole structure is well below the limit values in the “Collapse Prevention”, and all members perform effectively and adequately under DD-1 (MCE) level earthquake with a probability of exceeding 2% in the expected 50 years.

Consequently, the two structures provides “Immediate Occupancy” target performance level.

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Abdulhamit Eroğlu was born in Istanbul in 1992. He succeeded undergraduate in 2015 from Sakarya University Department of Civil Engineering. He registered to Gebze Technical University in 2015 for M.Sc and still senior at Department of Civil Engineering Earthquake and Structure Engineering Program. He started to work in civil engineering office after graduation where is Yapi Engineering Co. Ltd. and is already working there as civil engineer.