REPUBLIC OF TÜRKİYE
MUĞLA SITKI KOÇMAN UNIVERSITY

GRADUATE SCHOOL OF
NATURAL AND APPLIED SCIENCES

DEPARTMENT OF CIVIL ENGINEERING

ANALYZING THE IMPACT OF STRONG-COLUMN WEAK-BEAM RATIO ON STRUCTURAL BEHAVIOR IN MID-RISE REINFORCED CONCRETE BUILDINGS

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MASTER OF SCIENCE

JANUARY 2025
MUĞLA

MUĞLA SITKI KOÇMAN UIVERSITY

Graduate School of Natural and Applied Sciences

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Date of Defense: 27/01/2025

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27/01/2025

ACKNOWLEDGEMENT

I would like to extend my deepest gratitude to my esteemed supervisor Assist. Prof. Dr. Ersan GURAY for their invaluable mentor and guidance throughout the entire process. This study would no have been possible without his support and guidance. And thire constant encouragement have been instrumental in shaping the process and quality of this research study. I am extremly grateful for thire patient , understanding , and dedication in helping me overcome challenges and navigate complex academic concepts.

I express my gratitude to my beloved family and friend for their unwavering support , understanding , and constant encouragement throghout my academic process. Their love, motivation, and belief in my abilities have been a constant source of inspiration, and I am truly grateful for their presence in my life.

Lastely, I acknowledge all those whose contributions, direct or indirect, have played a role in the successful completion of this thesis.

ABSTRACT

ANALYZING THE IMPACT OF STRONG-COLUMN WEAK-BEAM RATIO ON STRUCTURAL BEHAVIOR IN MID-RISE REINFORCED CONCRETE BUILDINGS

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27/01/2025 , 72 pages

Typical building according to Turkish Building Earthquake Code (TBEC, 2018) should follow the strong-column weak-beam (SCWB) approach to sustain stability and functionality during earthquake hit. The recommended ratio for SCWB is a minimum of 1.2 for every joint. However, empirical evidence shows that this ratio might not suffice the optimal seismic resilience when the height of the structures is getting higher.

This study aims to show the effect of SCWB ratio on mid-rise reinforced concrete (RC) structures. Different SCWB ratio is selected for 20-story RC frame as follows: 1) SCWB ratio ranging from 1.2-2.0; 2) SCWB ratio ranging from 2.0-3.0; 3) SCWB ratio ranging from 3.0-5.0. Nonlinear dynamic analysis was conducted to monitor the structure behavior.

Keywords: Strong-Column, Weak-Beam, Earthquake, Reinforced-Concrete

ÖZET

ORTA YÜKSEKLİKTE BETONARME BİNALARDA GÜÇLÜ KOLON ZAYIF KİRİŞ ORANININ YAPISAL DAVRANIŞ ÜZERİNE ETKİSİNİN ANALİZİ

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Ocak 2025

Türk Bina Deprem Yönetmeliği'ne (TBEC, 2018) göre tipik bir bina, deprem sırasında stabilite ve işlevselliği sürdürmek için güçlü kolon zayıf kiriş (SCWB) yaklaşımını izlemelidir. SCWB için önerilen oran her bir birleşim için en az 1,2'dir. Ancak, deneysel kanıtlar, bu oranın yapıların yüksekliği arttıkça optimum sismik dayanıklılığı sağlamak için yeterli olmayabileceğini göstermektedir.

Bu çalışma, SCWB oranının orta yükseklikteki betonarme (RC) yapılar üzerindeki etkisini göstermeyi amaçlamaktadır. 20 katlı RC çerçeve için farklı SCWB oranları aşağıdaki gibi seçilmiştir: 1) 1,2-2,0 aralığında SCWB oranı; 2) 2,0-3,0 aralığında SCWB oranı; 3) 3,0-5,0 aralığında SCWB oranı. Yapı davranışını izlemek için doğrusal olmayan dinamik analiz yapılmıştır.

Anahtar Kelimeler: Güçlü Kolon, Zayıf Kiriş, Deprem, Betonarm

TABLE OF CONTENTS

TABLE OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS AND ABBREVIATION

SCWB	Strong-Column Weak-Beam
RC	Reinforced Concrete
TBEC	Turkish Building Earthquake Code
ASC	American Society of Civil Engineers
GM	Ground Motion

1

1. INTRODUCTION

1.1. General

Over the past fifty years, structural engineers have embraced a widely used technique for earthquake-proof building design, involving systems that are deliberately designed to fail in flexural mode. This is accomplished by using moment frames. This approach is based on the observation that the displacement demands for a structure are comparable whether it shows linear or nonlinear behavior if the initial stiffness is unchanged. Professionals aimed for nonlinear response because it is more expensive to develop a system in the linear range.

The method is set up with systems that grow dispersed and sustainable displacements in the nonlinear zone. By ensuring ductile flexibility failures at the element level, sustainability is attained. It is also necessary to spread the displacement over the frames because the local concentration of the displacement requirements could result in early capacity overlimit. The goal of proportioning to have a failure mode in a beam mechanism is an attempt to ensure this criterion.

An improperly sized and detailed frame can cause a variety of problems with the structure's functionality, leading to story mechanism. In structural behavior, the distribution and transfer of lateral forces within a multi-story building is referred to as the "story mechanism". To sustain the stability and safety of a multi-story building, loads coming from earthquakes must be resisted. Structures with soft stories—one or more stories that are noticeably more flexible than the others—are more likely to collapse. For instance, these stories may sustain significant drift and damage during an earthquake, which could result in the building collapsing whole or partially as shown in figure 1.

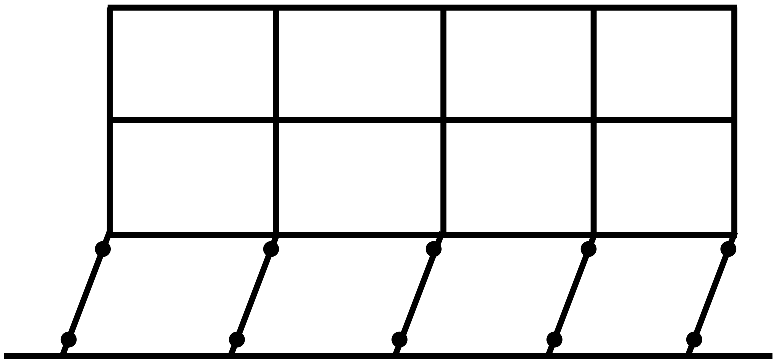

Figure 1.1 First story mechanism failure

The Turkish Building Earthquake Code (TBEC, 2018), American Society of Civil Engineers (ASCE-7, 2016), and other seismic regulations tried to regulate the total column moment capacity to total moment capacity at a beam-column joint to impose the beam mechanism construction. It is generally agreed upon that the limit ratio should be 1.2 (TBEC, 2018). Figure 2 shows the accepted moment pattern that facilitates ratio calculations. It is established that the specified condition alone is insufficient to guarantee the beam mechanism. Other Research indicates that additional factors may also impact the mechanism of failure (Discussed in Literature Review section).

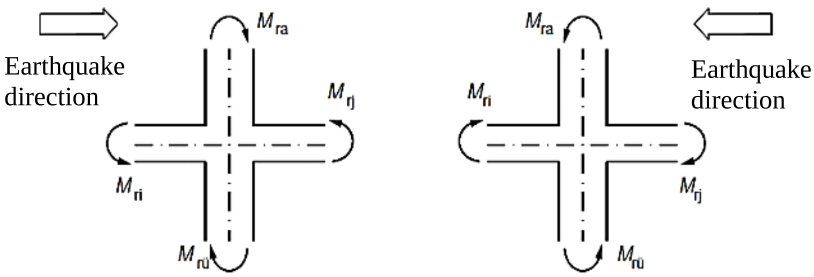

Figure 1.2 Moments transfer according to TBEC

1.2. Literature Review

Zareian, F. (2006) investigated the impact of column to beam strength ratio on moment-resisting frames. Two models for case studies are examined. Structures' collapse fragility curve characteristics—namely, how changes in the structural parameters affected the median collapse capacity—were thoroughly discussed. It is determined that P-Delta and the ratio of columns-beam strength (CBS) are the primary significant factors influencing the collapse potential. It was demonstrated that increasing the CBS from 1.2 to 2.4 causes the median collapse capacity to increase by 90%.

A study on SCWB design ratio was conducted on RC frames by Akhtari, R. (2023) using software implementation. Each case study contains three different SCWB design ratio ranging from 1.2 to 1.5, 1.5 to 2.0, and 2.0 to 3.0. The structural behavior of these case studies is monitored and evaluated using nonlinear static and dynamic analysis utilizing OpenSeesPy framework. It was shown that the performance and safety of the building enhances, and more beams involve dissipating energy coming from the earthquake.

Haselton and Deierlein (2007) studied comprehensively the base shear, drift limit, and SCWB design ratio. Different story heights were selected to represent the typical building, ranging from 4 to 12 stories. For each case study, a nonlinear model was created. The strong-column weak-beam (SCWB) design ratio is the most important parameter among the others, according to the study's findings. The result demonstrates how the structures' functionality and safety improved, resulting in a complete collapse mechanism.

In 2014, Zaghi et al. examined the impact of the column-to-beam strength ratio for steel moment frames subjected to earthquake response. An examination of multiple characteristics was given, including member ductility, inter-story drift, and floor acceleration. To achieve the required column beam strength ratios, the diameters of the column sections and the material's yield strength were adjusted for each frame. Monitoring the impacts of higher modes on the yield strength of columns revealed that these effects could lead to columns yielding even at strength ratios greater than 2.

The effect of column-to-beam strength on failure modes for a reinforced concrete frame was investigated by Hao Zhang et al. in 2019. To determine the system's behavior, a pushover analysis was carried out with strength ratios ranging from 1.2 to 2.0. It is suggested in this study to increase the column-to-beam ratio. A three-story, three-bay frame that was constructed in accordance with China's seismic design regulation was the subject of analysis. Furthermore, it was demonstrated that, considering the effects of slabs, the strength ratio should increase up to 1.8 to obtain the strong column and weak beam failure mode.

A method for calculating the lateral stiffness and inter-story drift ratio of frames while taking their relationship into consideration was proposed by Caterino et al. (2013). Based on the Italian Seismic Code, an approximate technique that was tested on 9 "ideal" and 2 "real" frames is evaluated. The outcomes were then contrasted with findings from previous research that provided a more accurate approximation of parameters for the structures that employed "capacity design." The first vibration mode shape served as the basis for the analysis, which estimated the stiffness and inter-story drift. Using the recommended formula, the authors evaluated the inter-story drift after first calculating the stiffness. In this investigation, shear-type framing yielded better approximations than flexure type.

According to a study by Sudarsana (2014), ductile reinforced concrete frames behave differently under static nonlinear pushover analysis depending on the column-to-beam strength ratio. The main study parameter is the column-to-beam design ratio, that utilizes the nominal strengths of the columns and beams to range from 1.2 to 2.0. It was discovered that ductility can be enhanced by a strength ratio of up to 1.4. Nevertheless, the increase in ductility is not noticeable for five- and ten-story models above this ratio. A beam sway collapse mechanism was found for five-story frame models for strength ratios of 1.4 up to 2.0; for ten-story frame models, this value varies from 1.6 to 2.0.

Mistri et al. (2016) conducted another study that shows the moment capacity ratio (MCR) on buildings with reinforced concrete (RC) frames. The study examines the reliability and fragility of four-story reinforced concrete frames that were constructed with MCR values ranging from 1.0 to 3.2. According to IS 1893(2002), RC frames are made for all seismic zones. Risk curves necessary for different seismic zones in India (such as zones II, III, IV, and V) have been chosen by the National Disaster Management Authority, which is part of the Indian government. Every designed building undergoes a seismic risk assessment, and a minimal MCR value is recommended based on the obtained Reliability Index and the Target Reliability Index. It is discovered that the MCR values significantly impact the seismic performance of frames. The MCR values have an important effect on how plastic hinges form in a particular order.

A study on the impact of column to beam moment capacity ratio was carried out by Sujan et al. in 2017. Three sets of structures have been designed with variable columns to beam moment capacity ratios (CBMCR) in each set, resulting in families of five structures each. For each model, pushover analysis is performed to assess how the CBMCR affects the lateral strength and displacement capacity of the structure. It has been shown that a rise in CBMCR enhances a structure's lateral strength and displacement capacity.

The impact of various column-to-beam strength ratio on the drift distribution of the reinforced concrete moment frames was examined in a study by Egemen et al. (2024). Different frames with varying numbers of floors and bays were taken into consideration for this aim. The numerical modeling was performed using the OpenSeesPy framework. Both nonlinear static and nonlinear time history analysis were employed to assess the seismic response of the model frames. The findings demonstrated that the distribution of inter-story drift ratios is strongly influenced by the column-to-beam strength ratio distribution. At the conclusion of the investigation, a distribution of the column-to-beam strength ratio was suggested.

A study by Sondilek et al. (2023) investigates how different column-to-beam strength ratios affect the reinforced concrete moment frames’ drift. For this reason, parametric tests using frames with varying story numbers were carried out. Using the OpenSeesPy framework, the frames were numerically modeled. The model frames' seismic response was evaluated using nonlinear dynamic analysis. Additionally, an analysis was conducted to determine how the various column-to-beam strength ratios affected the cost and material consumption.

The method for computing the column-to-beam strength ratios as it is illustrated in equation (1.1) and figure 2 is defined by TBEC (2018). The minimum ratio in TBEC is suggested to be 1.2. This ratio is generally utilized by designers to guarantee the SCWB approach.

$$(M_{ra} + M_{r\ddot{u}}) \geq 1.2 \left(M_{ri} + M_{rj}\right) \tag{1.1}$$

1.3. Scope Of This Study

This study's primary objective is to observe the RC moment frame failure mechanisms that meet the requirements of the TBEC (2018). The 20-story RC frames are chosen as a nod to the mid-rise RC buildings that are currently in Turkey. The two-dimensional three-bay frames that make up the selected structures are made to meet the standard span lengths and mass distributions for the industry. Along the height, stiffness is distributed. To observe how the frames responded, several column-to-beam flexural strength ratios were used. The TS500 and TBEC design specifications are met. Column-beam strength ratios between 1.2 and 5 are used for each frame's analysis.

To observe the frame failure mechanism, nonlinear time history analysis is performed. The SAP2000 software is utilized in this study. It was believed that the deformations of the members only occurred during flexural motions. It is believed that members will be able to withstand flexural failure with enough shear and bond capacity. This assumption is confirmed later by checking shear capacity under the developed demands. The Earthquake Hazard Map (https://www.afad.gov.tr) provided the parameters required for earthquake design purposes after the TBEC (2018). Every ground motion that was chosen is a distant field motion. In this study, near-fault consequences are not considered.

2. Analyze And Design Procedure In This Study

2.1. Introduction

Rather than constructing completely earthquake-resistant buildings, engineers prefer to design in a way to resist earthquakes despite of some local failure. Complete earthquake-resistant buildings will increase costs and take out an extensive element section. To get rid of over designing, one should know allowable principles and regulations for applying to the structure. It is desired to limit the failure of structural elements even for large loads coming from ground motion.

Despite of the gravity loads (Dead, live…), buildings should overcome the lateral loads imposed to them. These lateral loads typically can be earthquake or wind loads. However, for heavy structures, generally earthquake load is fatal and disastrous. To calculate and apply these loads, one should refer to some analysis and design procedures. Every country contains written regulation to deal with design and analysis. There are two types of design procedures: performance and prescriptive-based designs. In general, engineers prefer prescriptive-based design for their purposes due to its easiness and common usages.

Turkish Building Earthquake Code (TBEC, 2018) and Turkish Standards (TS, 2003) were developed along with those of other countries. Turkish Seismic Code allows buildings to face some local damage when earthquake happens. Under a low-intensity earthquake, it requires that no structural or nonstructural elements will be damaged. In a medium intensity earthquake, structural and nonstructural elements should be damaged but are repairable, while in a high-level earthquake, structural elements should be damaged in a way that they won't collapse for the safety of the people. A high-level earthquake is defined as an earthquake with a probability of exceeding 10% in 50 years. For available building analyses and strengthening, design codes provide some minimum performance limits.

2.2. Earthquake levels

In TBEC (2018), the earthquake levels are divided as follow:

- DD-1: Maximum probable earthquake
- DD-2: Standard Earthquake
- DD-3: Frequent Earthquake
- DD-4: Service Level Earthquake

DD-1 is the earthquake which has 2% probability of exceedance in 50 years. It has a high probability of occurrence but contains low intensity level. DD-2 earthquake level has high intensity but low occurrence probability. Its probability of exceedance is 10% in 475 years. DD-3 has a low probability of occurrence but high level of intensity. it refers to 50% probability of exceedance in 72 years. Finally, DD-4 contains 68% of exceedance probability in 43 years. For design purposes, standard earthquake level (DD-2) is applied for ordinary structures.

2.3. Standard Earthquake Spectra

The peak response of all potential linear single-degree-of-freedom (SDOF) systems to a specific ground motion component is represented by the elastic response spectrum. In other words, it is a description of a specific ground motion. Like this, the peak displacement relative to the ground of numerous SDOF systems with various period is commonly represented by a displacement response spectrum. As a result, constructing a response spectrum necessitates the examination of numerous SDOF systems.

The design spectrum is different from the response spectrum. Design spectrum is the envelope of several elastic design spectra. In new Turkish seismic code (section 2.3.1.), elastic design spectrum is divided into five sections (Figure 2.1). Each section refers to some parameters to be defined. These parameters can be obtained for specific location from the map using AFAD website (Turkey Earthquake Hazard map). By using these parameters and of course knowing the target location, one can construct the horizontal elastic design spectrum.

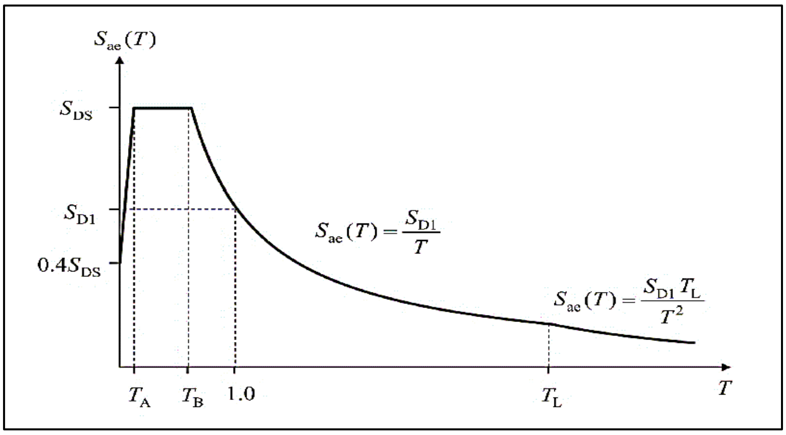

Figure 2.1. Horizontal design spectrum from TBEC (2018)

S_{D1}: Spectral Acceleration of 1[sec] Period (equation 2.2)

S_{Ds}: Spectral Acceleration for short period (equation 2.1)

T: Natural seismic period

T_A & T_B: Corner Periods (equation 2.3 & 2.4)

T_L: Long Period

$$S_{D1} = S_1 x F_1 \tag{2.1}$$

F_s: Short period site coefficient obtained from AFAD website

S_s: For short period spectral acceleration coefficient obtained from AFAD website.

$$S_{Ds} = S_s x F_s \tag{2.2}$$

S_1: For 1 second period spectral acceleration coefficient obtained from AFAD website.

F_1: Long period site coefficient from AFAD website

$$T_A = 0.2\frac{S_{D1}}{S_{DS}} \tag{2.3}$$

$$T_B = \frac{S_{D1}}{S_{DS}} \tag{2.4}$$

T_L = 6 second (Transition period according to TBEC 2018 for constant displacement region)

Horizontal elastic design spectral acceleration of the system can be found from following equations:

$$S_{ae}(T) = (0.4 + 0.6\frac{T}{T_A})S_{DS} \qquad 0 \leq T \leq T_A \tag{2.5}$$

$$S_{ae}(T) = S_{DS} \qquad T_A \leq T \leq T_B \tag{2.6}$$

$$S_{ae}(T) = \frac{S_{DS}}{T} \qquad T_B \leq T \leq T_L \tag{2.7}$$

$$S_{ae}(T) = \frac{S_{DS}T_L}{T^2} \qquad T_L \leq T \tag{2.8}$$

T: Fundamental period of the building

2.4. Local Soil Condition

In section (16.4) of TBEC (2018) defines the soil conditions from weaker to strong ones. These conditions permit the designer to judge about the behavior of soil under seismic action. Through this judgement, one can design a safe and serviceable structure. Figure 2.2 demonstrates the variation of soil types.

Local Soil Conditions	ZA	ZB	ZC	ZD	ZE	ZF
	Integral Rock	Fractured Rock	Densened Stong Clay	Medium Sand Clay	Loose Sand Weak Clay	Zone Specific Investigati on

Figure 2.2. Soil condition according to TEBC (2018)

Building Occupation Class (BKS)	Biulding Importance Factor	Building Usage Purpose
BKS =1	1.5	High impotance Class ,Immediate Usage, Crowded for Long duration , Toxic and Expolsive material storage
BKS=2	1.2	Short duration heavy usage,Malls ,Movie Theather, Sport Area and etc
BKS=3	1	Others, all other buildins (residentail, commercial, Hotels and etc)

Table 2.1. Importance factor according to TBEC (2018)

2.5. Building Importance Factor and Occupation Class

To find the earthquake design class (DTS), one needs to obtain building occupation class and its importance. Building importance factor defines how vital is the building to be safe after earthquake hit. Based on a building's importance class or occupancy category, occupancy importance factor (IF) multiplies the base shear design. TBEC (2018) provides a table (see table 5.1) to obtain the importance factor considering building usage purpose and building usage classification (BKS).

2.6. Earthquake Design Class (DTS)

To find the building height range, it is necessary to obtain the corresponding factor. For every different range of acceleration design spectrum factor (S_{DS}), TBEC (2018) provides a table (see table 2.2) to obtain the desired value for earthquake design class (DTS).

Acceleration Design Spectrum Factor (SDS) For DD-2 earthquake Level	Building Occupation Class	
	BKS=1	BKS=2, 3
SDS < 0.33	DTS =4a	DTS =4a
0.33 ≤ SDS < 0.5	DTS =3a	DTS =3a
0.5 ≤ SDS <0.75	DTS =2a	DTS =2a
0.75 ≤ SDS	DTS =1a	DTS =1a

Table 2.2. Earthquake Design Classes (TBEC, 2018)

2.7. Building Height Classes (BYS)

To find building height class for a specific building, first the earthquake design class (DTS) should be obtained from table 2.2 and then the height of the building should be known. Afterwards, one can utilize table 2.3 to find the building height class.

Building Height Class	Height Interval Based on Earthquake Design Class		
	DTS=1,1a,2,2a	DTS= 3,3a	DTS=4,4a
BYS=1	Hn>70	Hn>91	Hn>105
BYS=2	56<Hn≤70	70<Hn≤91	91<Hn≤105
BYS=3	42<Hn≤56	56<Hn≤70	56<Hn≤91
BYS=4	28<Hn≤42	42<Hn≤56	
BYS=5	17.5<Hn≤28	28<Hn≤42	
BYS=6	10.5<Hn≤17.5	4<Hn≤28	
BYS=7	7<Hn≤10.5	42<Hn≤56	
BYS=8	Hn≤7	Hn≤10.5	

Table 2.3. Building Height Classes (TBEC, 2018)

2.8. Reduced Acceleration Design Spectrum

Designing a building to stay in elastic zone is not economical and feasible. In general, designers mostly try to use the concept of ductility and inelastic energy dissipation. To apply this rule for the forced based design, one should find the reduction factor (Ra) of the structure. The reduction factor can be found using the following equations:

$$Ra(T) = \frac{R}{I} \qquad T > T_B \qquad (2.9)$$

$$Ra(T) = D + (\frac{R}{I} - D)\frac{T}{T_B} \qquad T < T_B \qquad (2.10)$$

R: Reduction Factor

D: Ductility Factor

T: Period of structure

R and D parameters can be found using Table 4.1 in TBEC (2018).

After calculating the elastic design spectrum (S_{ae}) and earthquake reduction factor (R_a), the reduced acceleration design spectrum can be found from equation 5.11:

$$S_{aR}(T) = \frac{S_{ae}(T)}{R_a(T)} \qquad (2.11)$$

The total Equivalent Earthquake Load is calculated using equation 5.11 and the total mass of the system by utilizing the following equation:

$$Vt = m_t \times S_{aR}(T) \geq 0.04 \times m_t \times I \times S_{DS} \times g \qquad (2.12)$$

The forces at the story levels can be calculated using equation 4.12 and 4.13 that were already discussed in the previous chapter.

2.9. Load Combination

There are several load types acting on the structure at the same time, resulting in a load combination. These loads can be dead, live, seismic, wind or snow loads in most cases. As a result of a variety of load combinations and load factors specified by building codes and regulations, the structure should be safe under a variety of maximum expected loads.

In this study, three types of load combinations have been used. These loads contain seismic, dead, and live effects to the model as shown below with their corresponding factors.

$$F_d = 1.4G + 1.6Q$$
$$F_d = 1.0G + 1.0Q + 1.0E \qquad 2.13$$
$$F_d = 0.9G + 1.0E$$

Where:

G is the permanent or dead load effects.

Q is the live load effects.

E is the earthquake load effects.

2.10. Effective Rigidity Factor And Live Load Effect

The live load participation factor must be used when designing and analyzing the building when considering the live load. Table 2.4 demonstrates the participation factor at the same time taking building usage purpose into account.

For strength-based building design, elements should be applied based on some factors. As a result of these factors, element sections will become less rigid. A different factor is given for each type of system, and it is selected for each type based on its characteristics. Table 2.5 below shows the rigidity factor for different elements according to TBEC (2018).

Building Usage Purpose	n
Store, Depo, Warhouse	0.8
School, Dormitories, Sport Complex, Cinema, Theatre, Concert Places, Religious Places, Shops and etc	0.6
Residence, Office , Hotel, Hospital, Parking etc	0.3

Table 2.3. Live load participation factor according to TBEC (2018)

Concrete Structural system elements	Effective Section rigidity factor	
Shear Wall- Slab(in Plane)	**Axial**	**Shear**
Shear Wall	0.5	0.5
Basement Shear Wall	0.8	0.5
Slab	0.25	0.25
Shear Wall- Slab(Out of Plane)	**Flexural**	**Shear**
Shear Wall	0.25	1.00
Shear Wall	0.5	1.00
Slab	0.25	1.00
Frame Element	**Flexural**	**Shear**
Spanderal Beam	0.15	1.00
Frame Beam	0.35	1.00
Frame Column	0.7	1.00
Shear Wall	0.5	0.5

Table 2.4. Rigidity Factor of different elements (TBEC, 2018)

2.11. Strength-Based Design

This method explains how to analyze a structure's design while taking its final strength into account. A key factor in safe construction and ensuring the occupants' maximum comfort is designing the building based on strength and serviceability limits. The structure's design analysis complies with the following criteria. Additional strength is considered in the design analysis by the strength design approach. Strain hardening causes the material to generate this extra strength. When compared to design procedure using the allowable stress method, the designers can provide more steel with the same area of concrete section and more concrete with less steel reinforcing.

2.12. Column Design Procedure

In TBEC (2018) the minimum permitted dimension for rectangular column sections is 300 mm, and for circular section columns, it is 350 mm. Equation 2.14 shows that the ratio of the axial compression load generated from G+Q+E over section gross area and concrete strength should be less than or equal to 0.4 after multiplying the live load with the reduced live load factor and combining them with other loads.

$$Ac \geq \frac{N_{dm}}{0.4 f_{ck}} \quad (2.14)$$

Where:

N_{dm} is axial load to the column.

f_{ck} is the characteristic strength of RC member in 28 days.

The high ductile column's longitudinal reinforcement ratio should be between 1% and 4% in value. Rebar less than Ø14 is not permitted for rectangular sections, and less than 6 rebar is not permitted for circular sections. The percentage of rebar in spliced sections shouldn't be higher than 6%.

For support sections of columns, the distance between ties shall be as shown in equation 2.15.

$$s \geq 50mm$$
$$s \leq 150mm \quad (2.15)$$
$$s \leq b_{min}/3$$

The minimum area for transverse reinforcement of a rectangular specific column's section can be calculated using equation 2.16 for Nd > $0.2A_cf_{ck}$. Ties with diameters less than 8 mm are not permitted to be used.

$$A_{sh} \geq 0.30\, s\, b_k[(A_c / A_{ck}) - 1](f_{ck} / f_{ywk})$$
$$A_{sh} \geq 0.075\, s\, b_k(f_{ck} / f_{ywk}) \quad (2.16)$$

Where:

s is spacing between shear reinforcement.

b_k is the core one side length.

A_c is the area of section.

A_{ck} is the area of core.

f_{ck} is the characteristic strength of RC member in 28 days.

f_{ywk} is the characteristic yield strength of shear rebar.

For $N_d \leq 0.2A_cf_{ck}$, at least 2/3 of the shear reinforcement calculated from equation 2.16 should be used.

2.13. Column Shear Safety

Equation 2.17 below can be used to compute the column shear force V_e that is employed in the column's shear design.

$$V_e = \frac{M_a + M_u}{l_n} \quad (2.17)$$

M_a and M_u are the moment capacity of the column at the top and bottom and Ln is its clear length. Equation 2.18 is then used to determine the total joint moment capacity of the column.

$$\sum M_p = M_{pi} + M_{pj} \quad (5.18)$$

If there is no specific calculation performed, then one can use the $M_{pi} = 1.4M_{ri}$ *and* $M_{pj} = 1.4M_{rj}$ to find parameters for equation 2.18. Detailed information is provided in TBEC (2018) section 7.3.7.

In addition, the following equation should be also satisfied for shear design of a specific column.

$$V_e \leq V_r \quad (2.19)$$

$$V_e \leq 0.85 A_w \sqrt{f_{ck}} \quad (2.20)$$

Where:

$V_r = V_w + V_c$

V_w is the sheer capacity of transverse reinforcement.

V_c is the shear capacity of the Concrete section.

For how to get these values, one can refer TBEC and TS500.

2.14. Strong-Column Weak-Beam Approach

As shown in equation 2.18, the total moment capacity of columns at the connection point of a structural system with beams, columns, and shear walls must be at least 20% more than the total moment capacity of the beam according to TBEC (2018). If this approach is applied to the model, the first plastic hinges will occur in beams rather than columns to prevent the building from story failure mechanism.

$$(M_{\text{ra}} + M_{\text{rü}}) \geq 1.2(M_{\text{ri}} + M_{\text{rj}}) \tag{2.18}$$

2.15. Beam Design Procedure

Full detail about how the beam and column should be connected and make the structural frame is provided in TBEC (2018). The beam's width must be at least 250 mm. The section's beams must have a height that is greater than three times the slab's 300 mm thickness. Furthermore, the beam's height cannot be greater than 3.5 times its width.

Shear reinforcement must have a minimum diameter of 12 mm and a minimum spacing of 300 mm. Additionally, supplementary ties should be used throughout the beam with a separation of no more than 600 mm in height and 400 mm in the axis. Detailed information about beams design procedure is described in TBEC (Section 7.4.3).

For structural building types with DTS = 1, 1a, 2, 2a (Earthquake Design Class), 50% of the negative moment section rebar should be placed at the bottom. This is because during an earthquake, the moment direction in the segment supporting the beams also changes periodically. Additionally, flexural reinforcement should be less than 2% of the gross section area.

There is a specific region for stirrups or transverse reinforcement that is designated as being twice the height of the beam. The first stirrup must be placed 50 mm from the face of the support and this portion of the beam must be carefully enclosed. In addition, stirrups smaller than 8 mm in diameter are to be avoided. The distance between stirrups must be less than 1/4 of the section height, which equals 8 or 150 mm according to TBEC (2018).

Furthermore, the following equation should be also satisfied for shear design of a specific beam.

$$V_e \leq V_r \tag{2.19}$$

$$V_e \leq 0.85 A_w \sqrt{f_{ck}} \tag{2.20}$$

Where:

$V_r = V_w + V_c$

V_w is shear capacity of transverse reinforcement.

V_c is the shear capacity of Concrete section.

For how to get these values, one can refer TBEC (2018) and TS500.

2.16. Nonlinear Dynamic Analysis

The computer technique of nonlinear dynamic analysis, also referred to as time-history analysis, is used in engineering and structural analysis to predict how structures will respond to dynamic loads, particularly when the response contains nonlinearities in material behavior or geometric deformations. Understanding how structures respond to occurrences like earthquakes, explosions, windstorms, and other dynamic forces requires this kind of analysis.

It is assumed in linear analysis that the relationship between loads and structural reaction is linear, i.e., that the response would double as the load doubled. However, many materials and structural systems display nonlinear behavior in real-world situations. Plastic deformations, significant displacements, material yielding, and changes in stiffness are a few examples of causes that might cause nonlinearities.

The following steps are commonly taken when performing a nonlinear dynamic analysis:

- Modelling: A finite element or other suitable numerical model is used to represent the structure. The geometry, composition, and boundary conditions of the structure are all included in this model.
- The applied loads: Time history—which may include seismic ground motions, wind gusts, or other dynamic forces—are defined.
- Time integration: Using numerical integration techniques, the equations of motion are solved across time. These equations in nonlinear analysis take changing stiffness, damping, and other nonlinear phenomena into consideration.
- Stiffness: The stiffness matrix can change when the structure is deformed. Based on the structure's deformed configuration, nonlinear dynamic analysis entails updating the stiffness matrix at each time step.
- Nonlinear analysis is iterative by its very nature. Until the convergence condition is satisfied, the solution is typically improved through iterations. To accurately depict the structure's nonlinear behavior, this is required.
- Results Interpretation: The analysis provides information about the structure's behavior in relation to dynamic loads throughout time. This includes data on internal forces, velocity, accelerations, and displacements.
- Nonlinear dynamic analysis is used to predict how important infrastructure such as bridges, buildings, and dams will respond to catastrophic events. The outcomes of these assessments are used by engineers to create structures that can safely disperse energy from dynamic forces. This is particularly crucial for buildings situated in seismically active regions since nonlinear dynamic analysis aids in the prediction of likely failure modes and the creation of efficient mitigation measures.

In general, nonlinear dynamic analysis is an effective tool for helping engineers comprehend and create structures that can survive the intricate and frequent nonlinear interactions between structures and dynamic forces. Ground motions are chosen and acquired for this investigation via the PEER website. Using SAP2000 software, the location's acceleration spectrum is then matched with the ground motions. This software is used to carry out the time-history analysis of all frames.

The ground motion recordings used in time history analysis should accurately represent the characteristics of the location and the level of seismic risk. Therefore, according to ASCE7-10, such assessments must use a minimum of three earthquake records. Table 2.7 presents the information on earthquake accelerograms utilized in this study.

Table 2.6. Selected ground motion for NDA

Earthquake	Year	Magnitude	Mechanism	NPTS	Dt (sec)
Chalfant Valley-02	1986	6.8	Strike slip	7999	0.005
Super. Hills-02	1987	6.2	Strike slip	11999	0.005
Landers	1992	6.0	Strike slip	7180	0.0039

3. DESIGN OF THE REPRESENTATIVE FRAMES

3.1. Introduction

This chapter presents the design considerations and the methodology of the 2D frames utilized in this research. This study investigates the structural behavior of planar frames with three-bay twenty-story frame utilizing three different SCWB design ratios. The design incorporates the Turkish Building Earthquake Code (TBEC, 2018) and the Turkish Reinforced Concrete Standard (TS 500, 2003). Time-history analyses were conducted to observe the behavior of each frame, specifically focusing on the period, inter-story drift, input energy, and base shear to monitor the efficiency of various range of SCWB as targeted. Different design ratios were chosen for the strong-column weak-beam, ranging from 1.2 to 2.0, 2.0 to 3.0, and 3.0 to 5.0. This chapter also presents the results and discussion of each frame's behavior.

Although the chosen frames for all case studies vary in SCWB, comparable designs are similar. Figure 3.1 illustrates that frames are integral components of buildings with standard spans. The 2D frames utilized in simulations are standard internal frames commonly found in existing prototype structures.

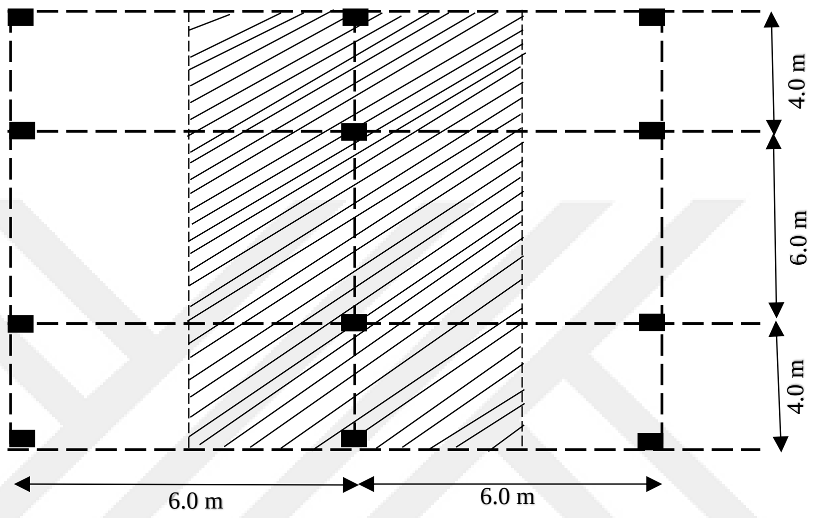

Figure 3.1 First story mechanism failure

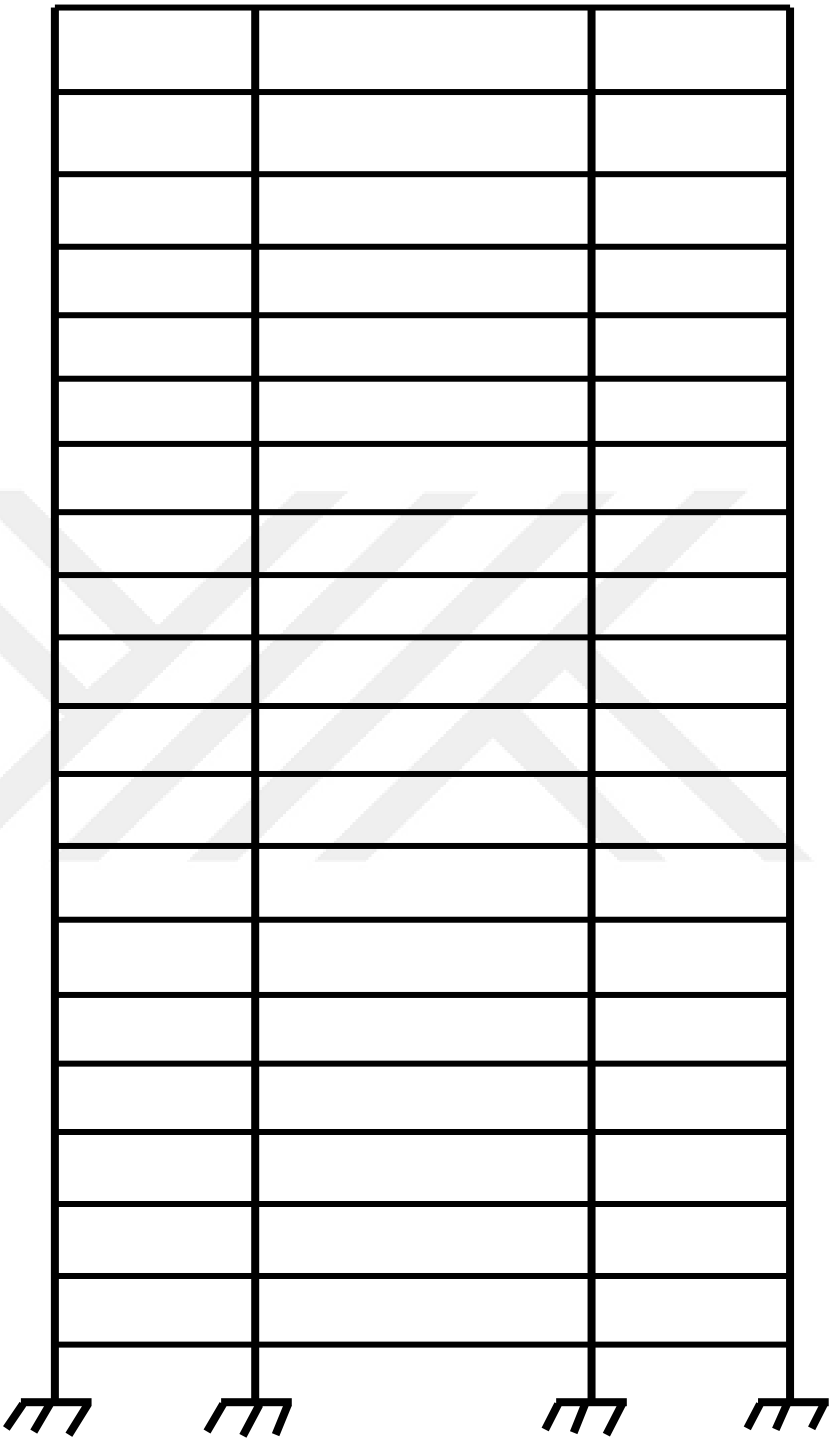

Figure 3.2 The configuration of selected 20-story RC frame

Figure 3.2 shows the configuration of a 20-story selected reinforced concrete frame. The selected concrete frame is typically 3.0-meter in height, corner bays are 4.0 meter, and middle bay is 6.0 meter in length. Moment frame system has been utilized for all three cases and with residential usage. The beams are intended as double reinforced concrete beams for flexural design. The design moments that were employed in the software are the moments at the column faces, and the constraints for reinforcement ratios, as indicated in TS500 and TBEC (2018) are considered.

The goal of shear design for structural components is to prevent failures caused by shear and ensure failures in flexure (i.e., if failures develop, flexure failure should be the reason). For this to happen, one must meet all the requirements in the Turkish Building Earthquake Code (2018). A spreadsheet was scripted to verify all these recommendations and requirements.

There could be numerous loads impacting on the building at the same time, so load combination ought to be considered. These forces could come from dead, live, seismic, wind, or snow loads in most circumstances. As a result of a variety of load combinations and load factors prescribed by building codes and regulations, the structure should be safe under a variety of maximum predicted loads. In this investigation, three kinds of load combinations have been employed. These loads contain seismic, dead and live impacts that are added to the model as illustrated below with their respective factors.

$$F_d = 1.4\ G + 1.6\ Q \tag{3.1}$$

$$F_d = 1.0\ G + 1.0\ Q + 1.0\ E \tag{3.2}$$

$$F_d = 0.9\ G + 1.0\ E \tag{3.3}$$

Where:

G is the Dead load

E is the seismic load

Q is the live load

In compliance with TBEC (2018), the fundamental design of frames was completed utilizing equivalent lateral load approach together with capacity design concepts. It is believed that the frames belong in a high-seismicity region with a ZD kind of soil (Figure 6). The linear elastic acceleration design spectrum has been achieved from the Turkish Earthquake Risk Map (AFAD) that the likelihood of exceeding is 10% in 50 years. Figure 7 shows the linear elastic design spectrum of the representative site.

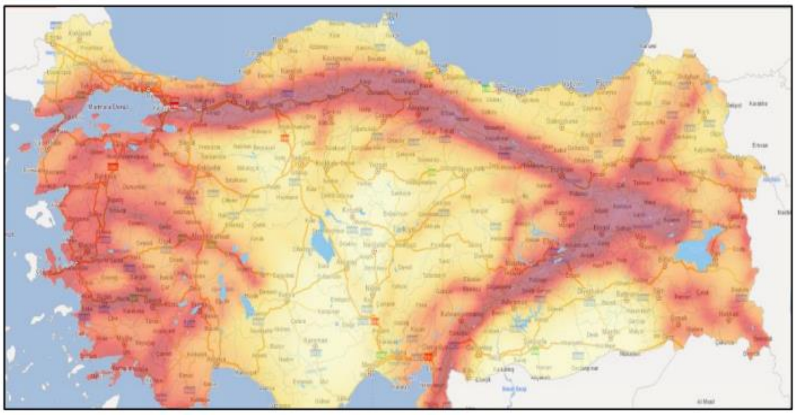

Figure 4. Turkey Earthquake Hazard Map (Source: AFAD website)

Figure 5.3 Selected location for the design process (Source: AFAD website)

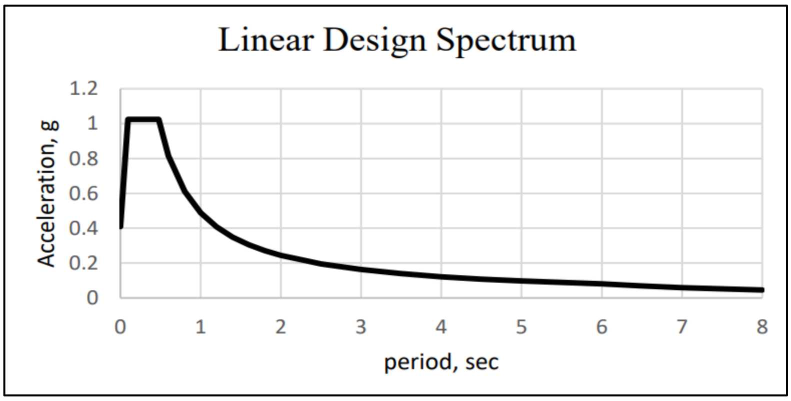

Figure 6. Linear Design spectrum for the representative location

The participation factor for live loads has been implemented. According to Turkish Building Earthquake Code (TBEC, 2018), residential structures should have a live load factor of 0.3. The frame members' effective stiffnesses are specified in TBEC (2018). For the columns and beams, reduction factors of 0.7 and 0.35 were used, respectively.

According to most seismic design codes, structures must offer the life-safety level during design-level earthquakes. Following an earthquake of design level, damage is likely to occur in the structures. The TBEC (2018) designates the design-level earthquake as DD2-level, with a 475-year return period and a 10% probability of surpassing it in 50 years. Ground motions at the design level were used to assess how the frames responded to such an incident.

3.2. Case Study I: SCWB ratio between 1.2-2.0

The design of the twenty-story three-bay frame was based on a selected SCWB ratio (1.2-2.0). Initial frame design was carried out according to TS500 (Turkish Standards, 2003), and subsequently the design of the conventional frame was primarily regulated by the Turkish Building Earthquake Code (TBEC, 2018), particularly in terms of reinforcement detailing. The specifics of the design process have been presented in the preceding chapters. Following the completion of the design process, the time-history analysis was conducted to the frame using SAP2000 software. This chapter presents the obtained analysis results and the subsequent discussion.

A recommended initial design ratio for the strong-column weak-beam is between 1.2 and 2.0. The component's cross-sections and reinforcing were found to satisfy the desired design ratio. The purpose is to track the inter-story drifts, input energy and base shear. The frames were 2-D moment frames. The external bays width is 4.0 meters, and the internal bay width is 6.0 meters. For all floors, the height of the story is 3.0 meters. Figure 32 illustrates the overall of the frame. The load applied to each floor is 4.0 KN/m^2 and 2.0 KN/m^2 for dead and live loads accordingly. The mass of each level is determined considering these loads. Reinforcement and concrete with typical strengths of 420 MPa and 30 MPa were assumed for the structures.

The component cross-sections and reinforcing details for the columns and beams are given in Tables 7 and 8 respectively. The initial period of the frame is shown in Figure 8.

Table 3.2.1. Cross sections for internal and external columns with reinforcement details. All dimensions are in mm.

Story	Longitudinal Reinforcement		Transverse Reinforcement				Cross Section Size	
			Inner		Outer			
	Inner	Outer	Confined Region	Unconfined Region	Confined Region	Unconfined Region	Inner	Outer
1-7	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	700x700	700x700
8-15	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	600x600	600x600
16-20	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	500x500	500x500

Table 3.2.2. Reinforcement and Cross-sections detail of beams, dimensions are in mm.

Story	Location	Inner Bay Support	Outer Bay Support	Cross Sectional Size	
	Longitudinal Reinforcement			Inner	Outer
1-20	Top	8Φ22	8Φ22		
	Bottom	4Φ22	4Φ22		
	Transverse Reinforcement			400x700	
1-20	Confined	3Φ10/100	3Φ10/100		
	Unconfined	3Φ10/130	3Φ10/130		

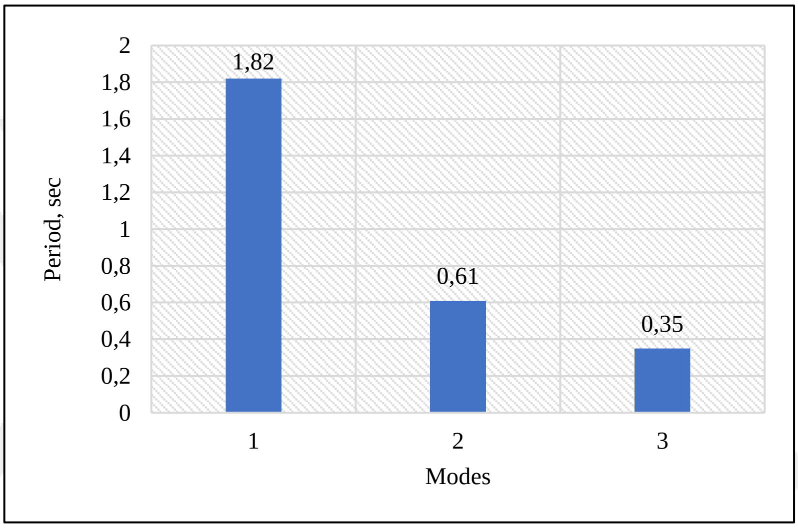

Figure 7. Selected frame different modes' period

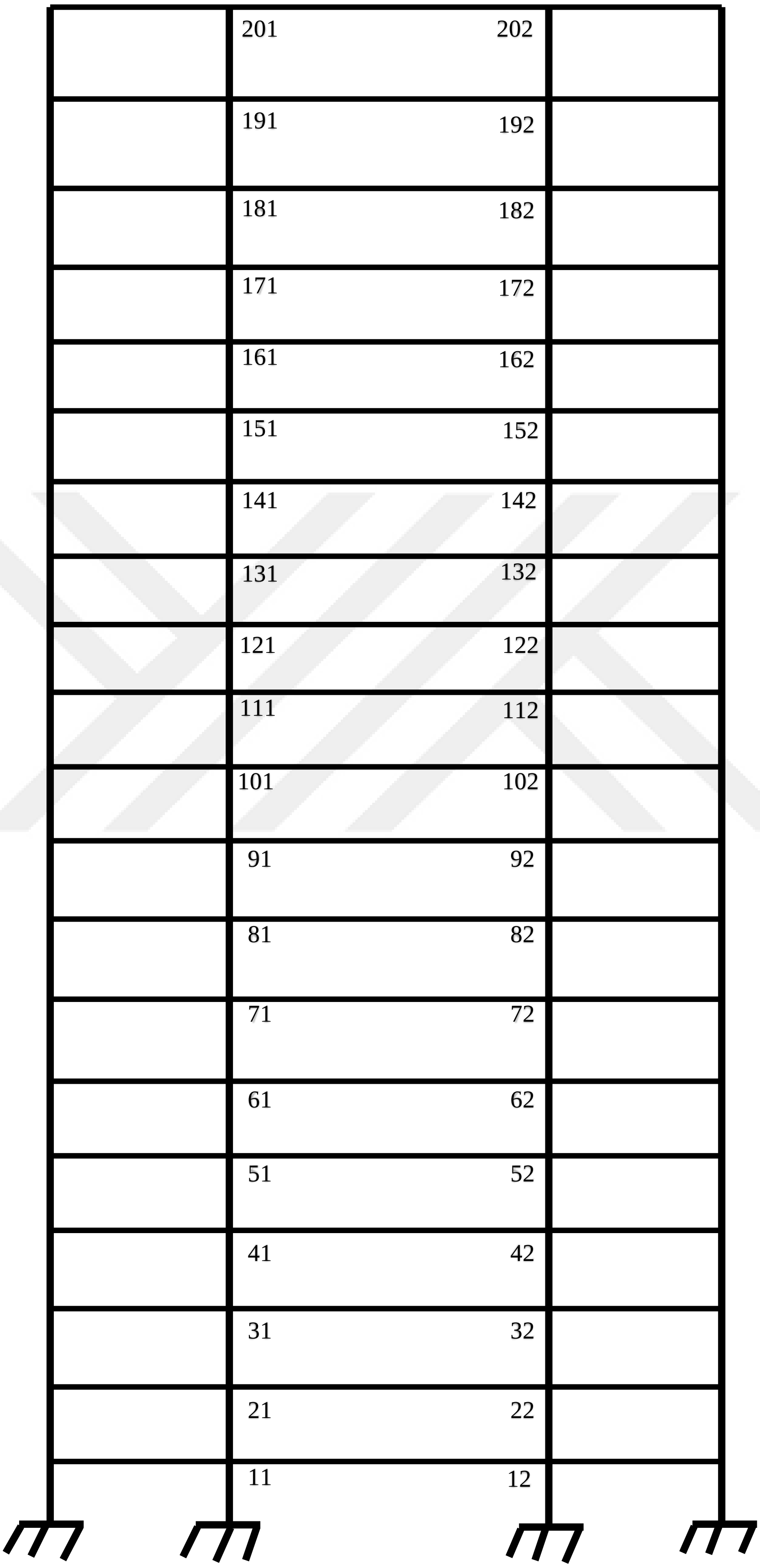

Figure 8. Joints' label of representative frame

Table 3.2.3. Targeted SCWB ratio: 1.2-2.0

Joints	SCWB ratio
11	2.0
12	2.0
21	2.0
22	2.0
31	2.0
32	2.0
41	1.97
42	1.98
51	1.92
52	1.95
61	1.88
62	1.92
71	1.69
72	1.73
81	1.51
82	1.55
91	1.47
92	1.52
101	1.43
102	1.48
111	1.39
112	1.45
121	1.36
122	1.44
131	1.33
132	1.4
141	1.21
142	1.27
151	1.35
152	1.40
161	1.57
162	1.61
171	1.53
172	1.58
181	1.51
182	1.55
191	1.49
192	1.51
201	0.74
202	0.75

3.2.1. Nonlinear analysis results for case study I

The influence of earthquake on the frame has been examined using three types of ground motions (Chalfant vally-02, Super. Hills-02, and Landers). Inter-story drift ratio, base shear, and total input energy coming from these earthquakes are presented. Figure 10 shows the inter-story drift ratio of the frame after the analysis. The ratio is typically between the 0.2 up to 0.8 for all floors.

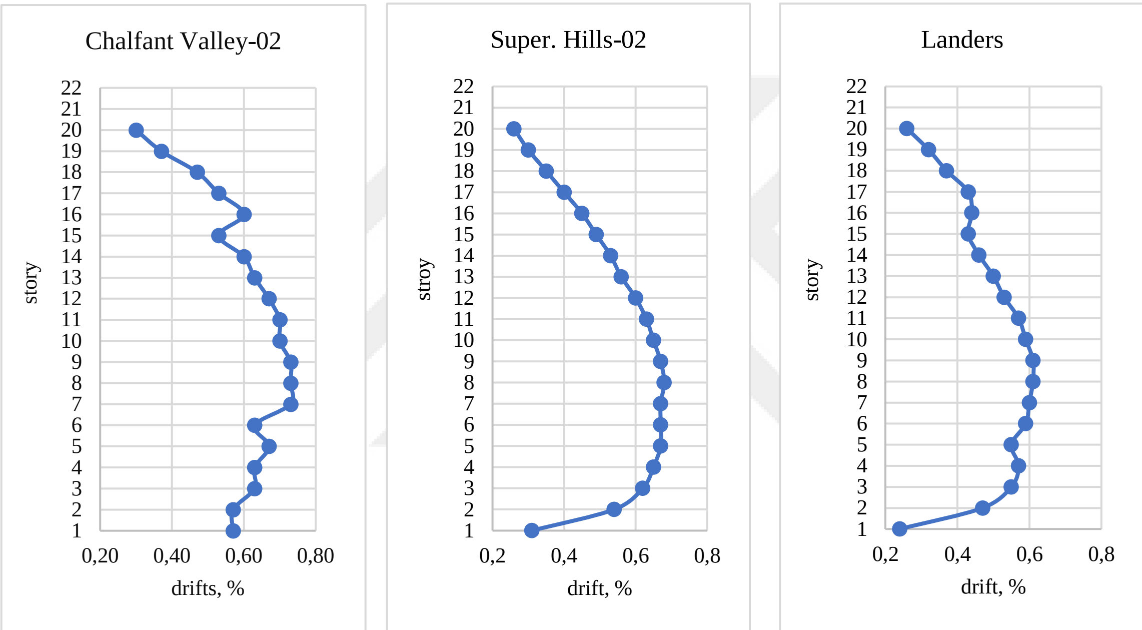

Figure 9. Inter-story drift after nonlinear time-history analysis

Unlike the static method, the structural dynamic response can be calculated using actual ground motions. This process is performed using time-history analysis. The base shear results for this case are presented for each ground motion. Figures 11-13 show the results for the selected ground motions. For Chalfant ground motion, the maximum base shear is 2700 KN which occurred at 10 seconds. The highest base shear is 3200 KN for Super. Hills ground motion. It also occurred at 10 seconds. However, for the Lander ground motion, the maximum base shear is almost 4000 KN which happened at 8 seconds.

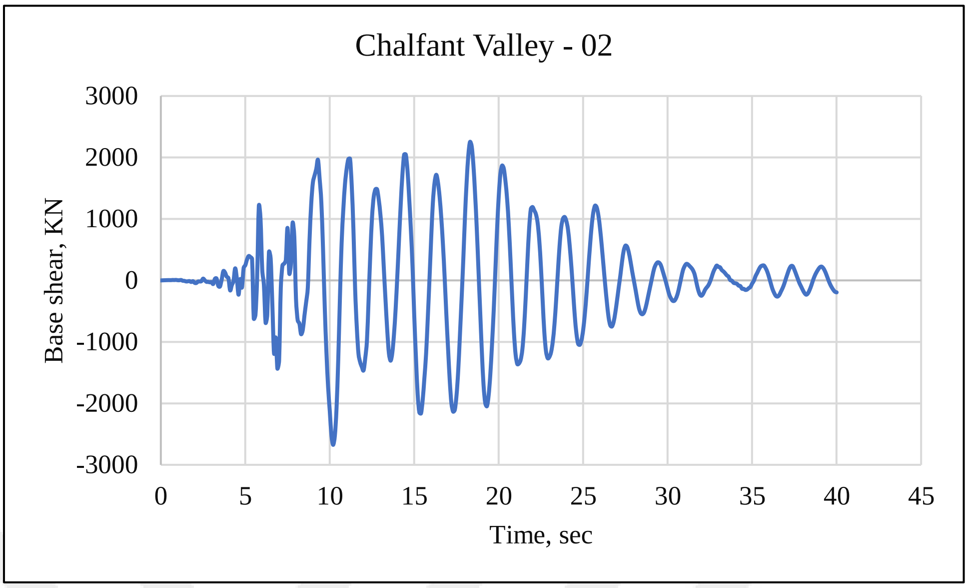

Figure 10. Base shear of the representative structure for Chalfant Valley-02 ground motion

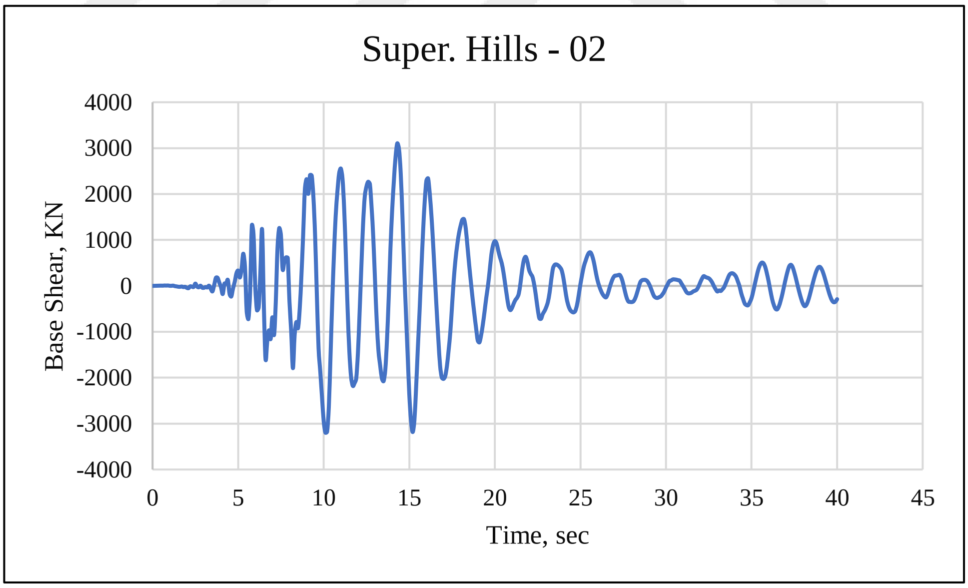

Figure 11. Base shear of the representative structure for Super. Hills-02

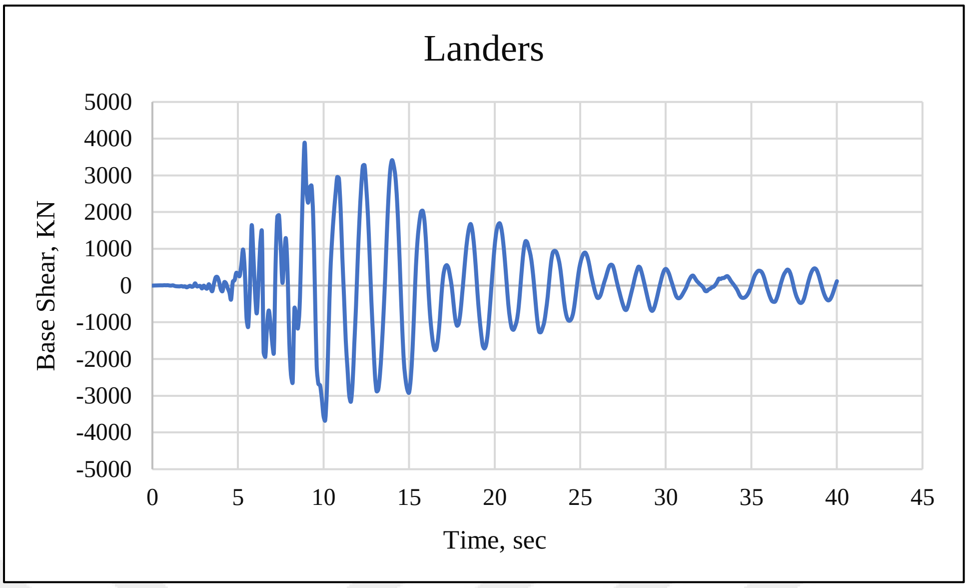

Figure 12. Base shear of the representative structure for Landers ground motion

An earthquake's ground motion intensity is reflected in input energy (Khakshaee et.al, 2003). The input energy of a structure must be estimated and distributed among its structural components in an energy-based seismic design. This result helps when the design of the structure is based on energy. Figres 14-16 show the total input energy released from the selected ground motions. The maximum input energy ranges between 1050 m^2/s^2 to 1300 m^2/s^2 for all ground motions. The time of occurrence is different that might be due to different peak ground acceleration for the selected ground motions.

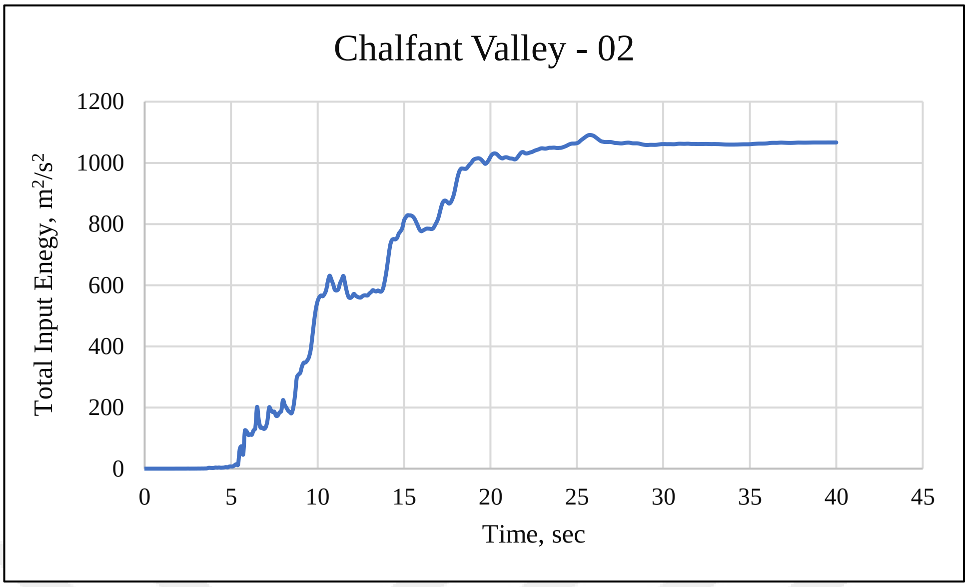

Figure 13. Total input energy coming from Chalfant Valley-02 ground motion

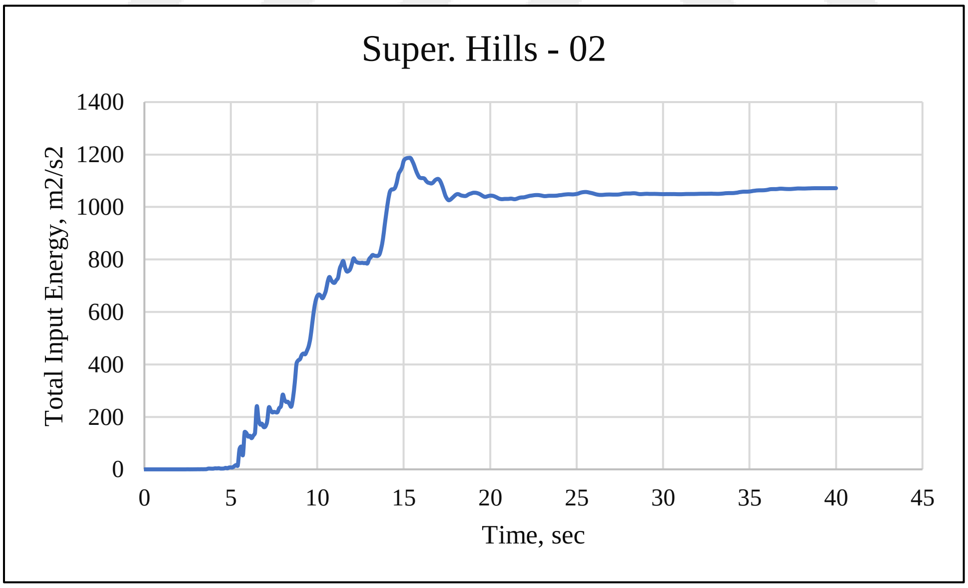

Figure 14. Total input energy coming from Super. Hills - 02 ground motion

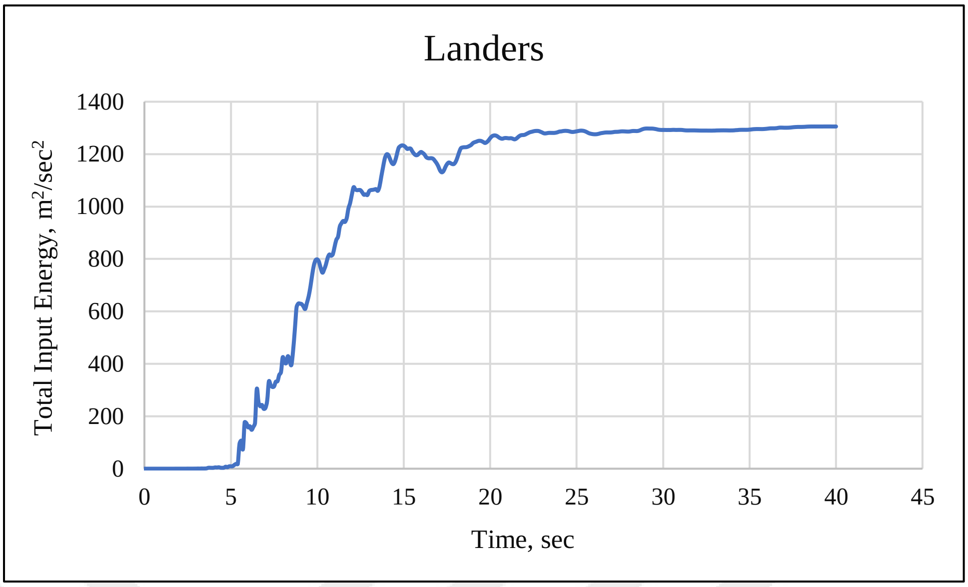

Figure 15. Total input energy coming from Chalfant Valley-02 ground motion

3.3 Case Study II: SCWB ratio between 2.0-3.0

This case study is a twenty-story three-bay frame based on a selected SCWB ratio (2.0-3.0). As the previous case study, the initial frame design was carried out according to TS500 (Turkish Standards, 2003), and consequently, the Turkish Building Earthquake Code (TBEC, 2018) mainly guided the design of the selected frame, primarily the reinforcement. Following the completion of the design process, the time-history analysis was performed with the aid of SAP2000 software.

For this case, a presumed initial design ratio for the strong-column weak-beam is between 2.0 and 3.0. The component's cross-sections and reinforcing were found to satisfy the desired design ratio. Like the previous case study, the aim is to monitor the inter-story drifts, input energy and base shear. The frame general properties and design method are like the previous case study. In contrast to the prior one, the SCWB and design details are different.

The component cross-sections and reinforcing details for the columns and beams are given in Tables 9 and 10 respectively. The initial period of the frame is shown in Figure 17.

Table 3.3.1. Cross sections for internal and external columns with reinforcement details. All dimensions are in mm.

Story	Longitudinal Reinforcement		Transverse Reinforcement				Cross Section Size	
			Inner		Outer			
	Inner	Outer	Confined Region	Unconfined Region	Confined Region	Unconfined Region	Inner	Outer
1-7	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	800x800	700x750
8-15	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	750x750	600x600
16-20	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	700x750	500x500

Table 3.3.2. Reinforcement and Cross-sections detail of beams, dimensions are in mm.

Story	Location	Inner Bay Support	Outer Bay Support	Cross Sectional Size	
	Longitudinal Reinforcement			Inner	Outer
1-20	Top	8Φ22	8Φ22		
	Bottom	4Φ22	4Φ22		
	Transverse Reinforcement			400x700	
1-20	Confined	3Φ10/100	3Φ10/100		
	Unconfined	3Φ10/130	3Φ10/130		

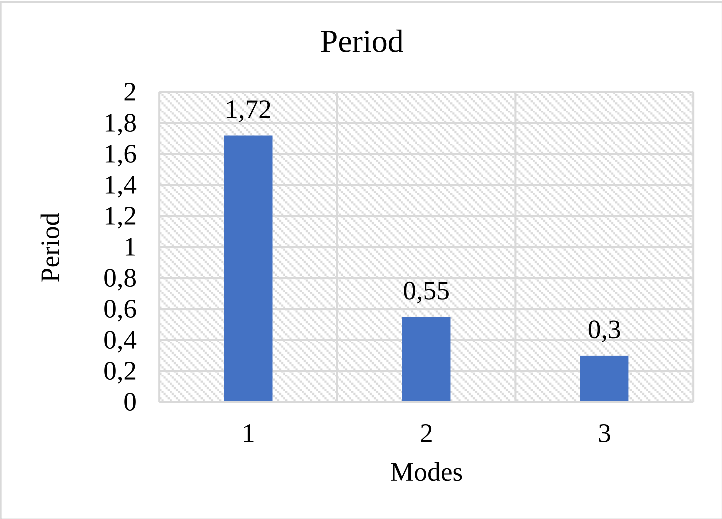

Figure 16. Period information for the case II

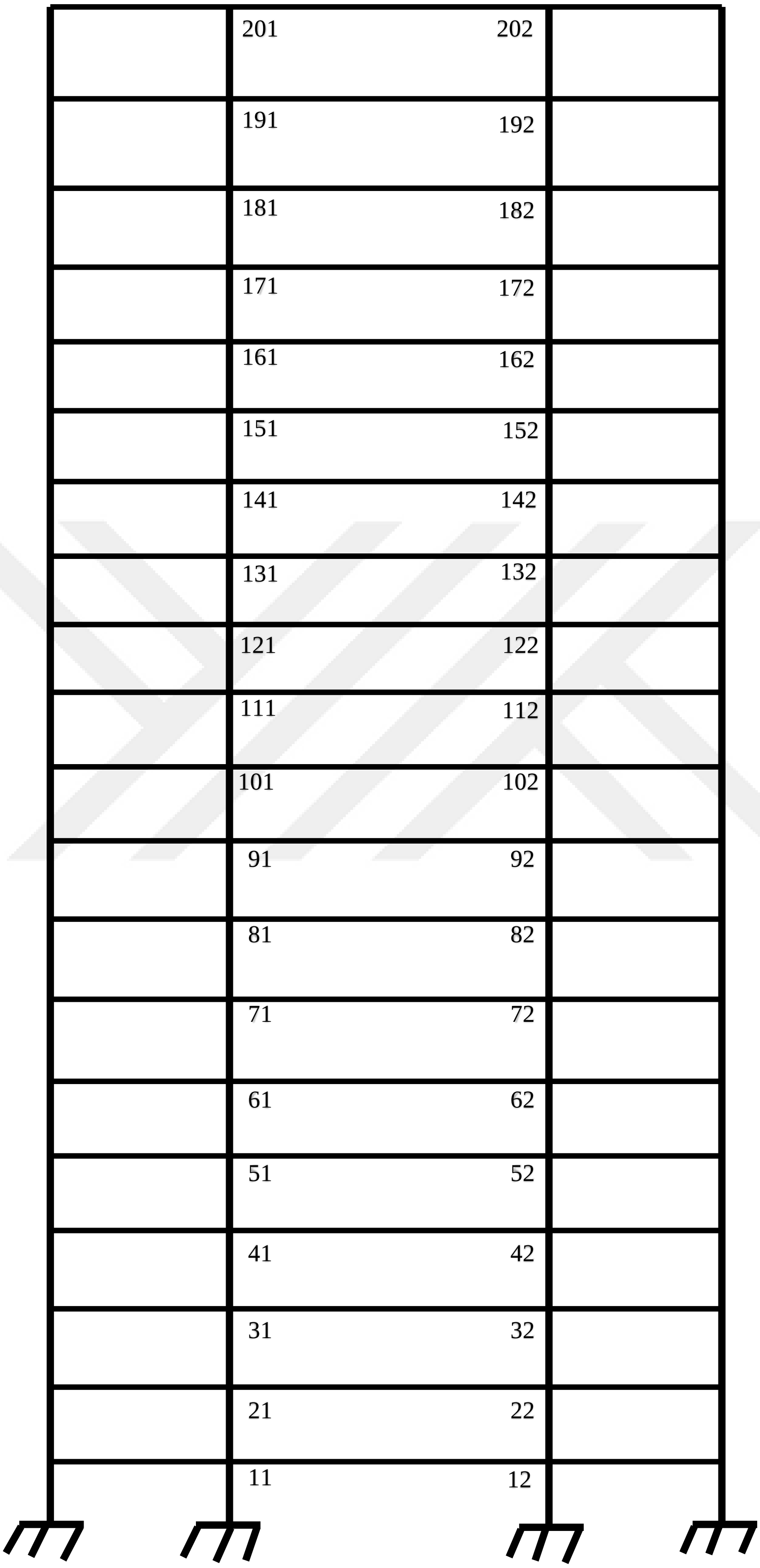

Figure 17. The joints' label for the selected frame - case study II

Table 3.3.3. SCWB ratio for case study II

Joints	SCWB ratio
11	3.05
12	3.03
21	3.05
22	2.99
31	3.00
32	2.95
41	2.93
42	2.92
51	2.88
52	2.88
61	2.83
62	2.84
71	2.64
72	2.68
81	2.47
82	2.51
91	2.42
92	2.47
101	2.37
102	2.43
111	2.33
112	2.40
121	2.29
122	2.36
131	2.24
132	2.32
141	2.21
142	2.28
151	2.15
152	2.23
161	2.10
162	2.18
171	2.04
172	2.12
181	2.00
182	2.05
191	2.0
192	2.0
201	0.96
202	0.97

3.3.1 Nonlinear analysis results for case study II

Plots showing the relative displacement between floors, or inter-story drift ratio (IDR), have been created for various ground motions. To evaluate damage and guarantee safety during seismic activity, IDR is a crucial metric. On the lower floors, the Lander ground motion had the least IDR. On the other hand, the Super Hills-02 ground motion at the fourth story produced the largest IDR. Larger IDRs are typically the outcome of ground motion with high peak ground acceleration (PGA). Figure 19 shows the graphical results of IDR for different ground motions and stories after the time-history analysis.

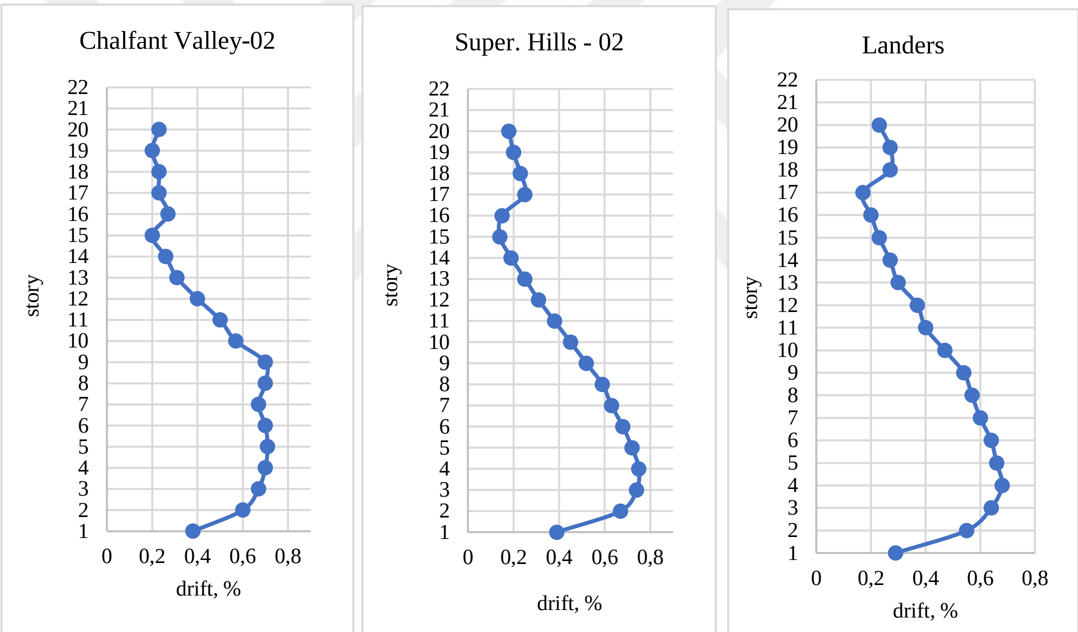

Figure 18. IDR result for different ground motions after time history analysis

Base shear, which indicates the total load on the structure, is one of the key parameters in seismic design. It is the entire lateral force carried on by the earthquake's ground motion. For the nonlinear dynamic analysis, three distinct ground motions of varying intensities were selected and used. The outcome is displayed in Figures 20–22. The greatest base shear, 4600 KN, is demonstrated to be the result of Lander ground motion at about 17 seconds.

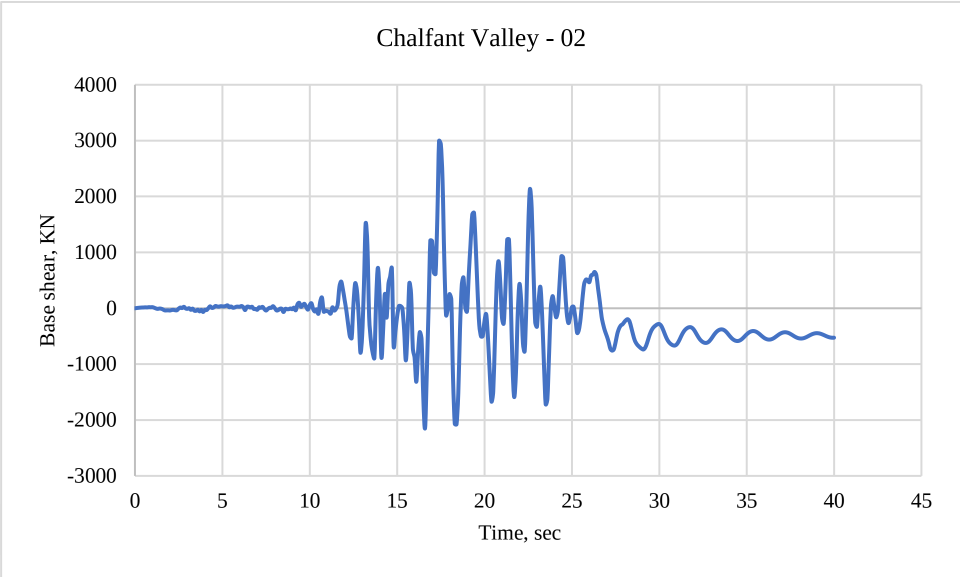

Figure 19. Base shear caused by Chalfant Valley -02 for case study II

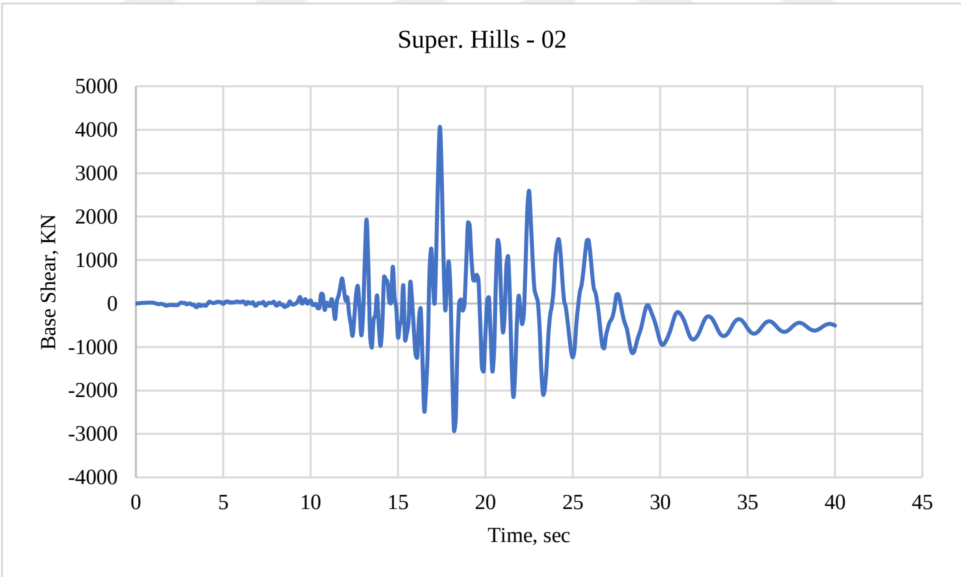

Figure 20. Base shear caused by Super. Hills -02 for case study II

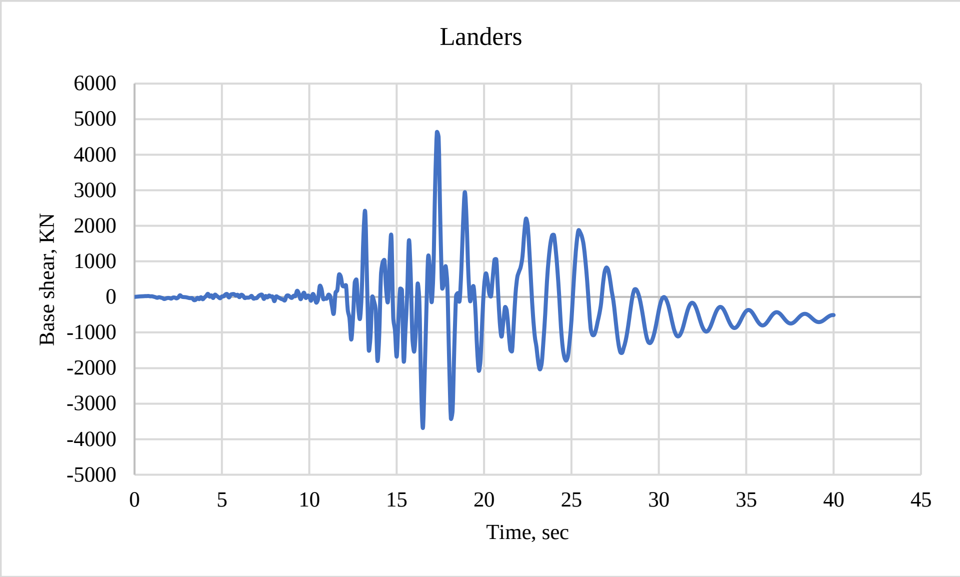

Figure 21. Base shear caused by Landers for case study II

Building design can be greatly influenced by total input energy, which is defined as the total energy transmitted to the structure. Figures 23-25 show the results of total input energy to the structure caused by different ground motion (Chalfant Valley-02, Super. Hills-02, and Landers) with different intensities and PGA. The greatest total input energy was transmitted to the structure from Landers ground motion which is around 960 m^2/s^2 at 26 seconds. The lowest was for Chalfant Valley-02 which is 650 m^2/s^2.

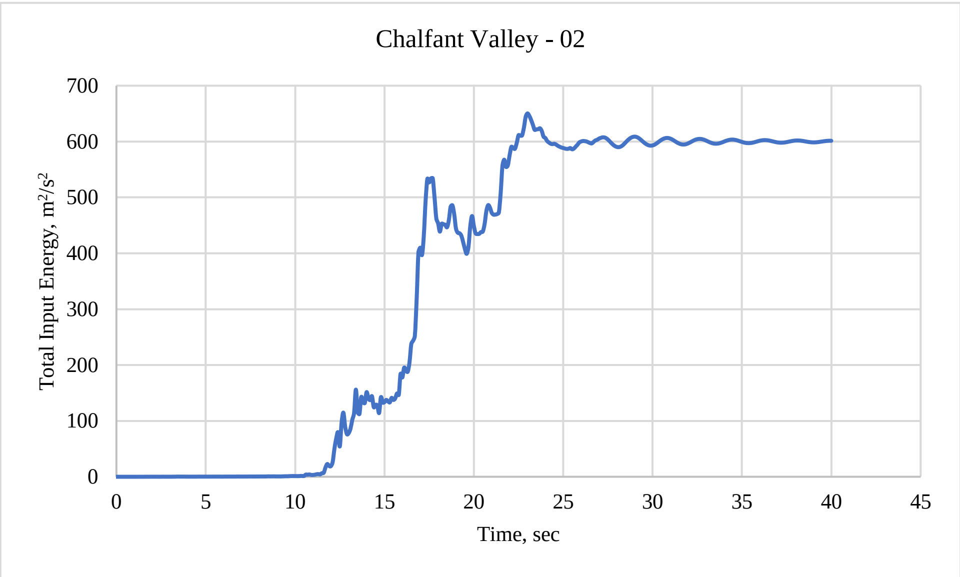

Figure 22. Total input energy - Chalfant Valley-02 for Case study II

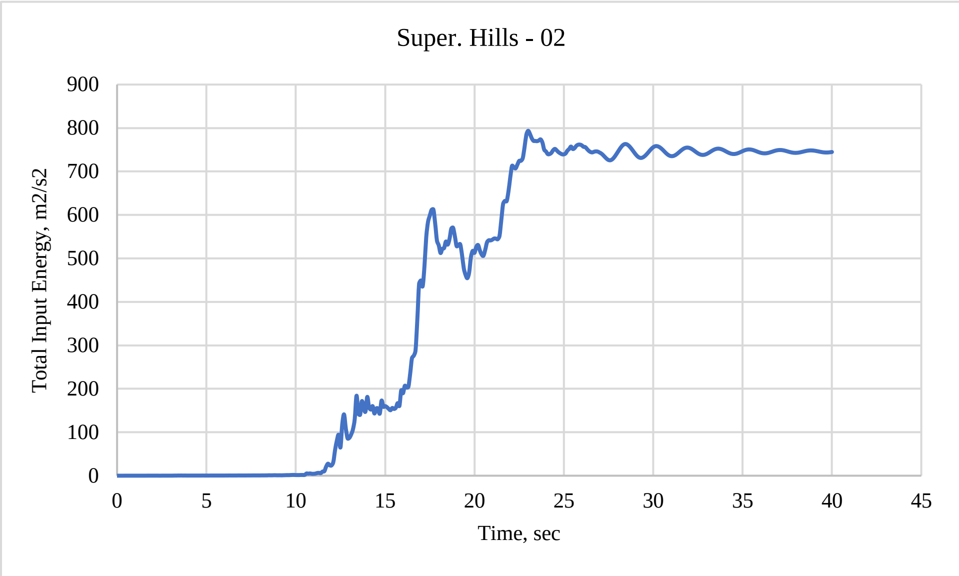

Figure 23. Total input energy – Super. Hills-02 for Case study II

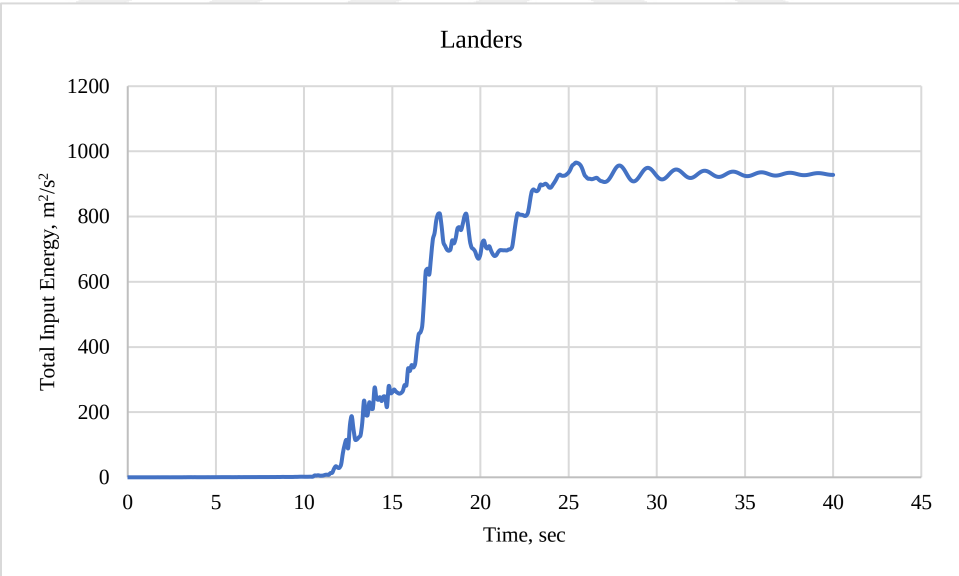

Figure 24. Total input energy - Landers for Case study II

3.4 Case Study III: SCWB ratio between 3.0-5.0

This case study is similar as the previous ones in term of configurations and details. However, the only difference is the range of SCWB ratio. The interval for the SCWB ratio is between 3.0 and 5.0 for the final trial. In fact, to achieve the targeted ratio, the frame particular properties such as cross section and reinforcement detail have been altered. As the previous case study, the initial frame design was carried out according to TS500 (Turkish Standards, 2003), and the Turkish Building Earthquake Code (TBEC, 2018). SAP2000 software has been utilized to perform the nonlinear dynamic analysis of the frame.

For this case, full detail about reinforcement and cross section detail can be found in Tables 9-10. The component's cross-sections and reinforcing were found to satisfy the desired design ratio. Like the previous ones, the purpose is to focus on the inter-story drifts, input energy and base shear results. As mentioned in contrast to the prior ones, the SCWB ratio and design details are different. The initial period of the frame is shown in Figure 26.

Table 3.4.1. Cross sections for internal and external columns with reinforcement details for case study III. All dimensions are in mm.

Story	Longitudinal Reinforcement		Transverse Reinforcement				Cross Section Size	
			Inner		Outer			
	Inner	Outer	Confined Region	Unconfined Region	Confined Region	Unconfined Region	Inner	Outer
1-7	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	1000x1000	700x750
8-15	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	900x900	600x600
16-20	12Φ28	12Φ28	Φ10/100	Φ10/130	Φ10/100	Φ10/150	800x850	500x500

Table 3.4.1. Reinforcement and Cross-sections detail of beams for case study III, dimensions are in mm.

Story	Location	Inner Bay Support	Outer Bay Support	Cross Sectional Size	
	Longitudinal Reinforcement			Inner	Outer
1-20	Top	8Φ22	8Φ22	400x700	
	Bottom	4Φ22	4Φ22		
	Transverse Reinforcement				
1-20	Confined	3Φ10/100	3Φ10/100		
	Unconfined	3Φ10/130	3Φ10/130		

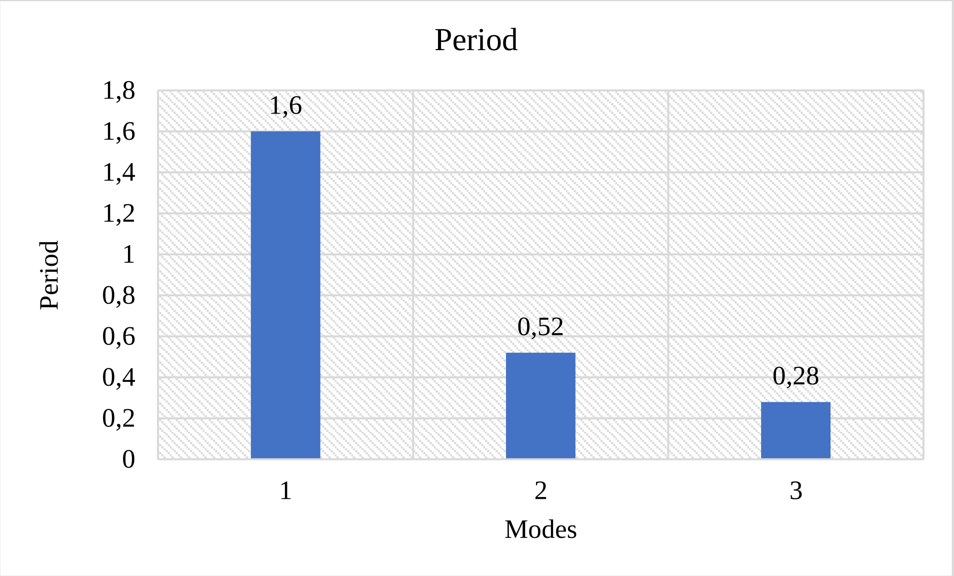

Figure 25. Period information for the case III

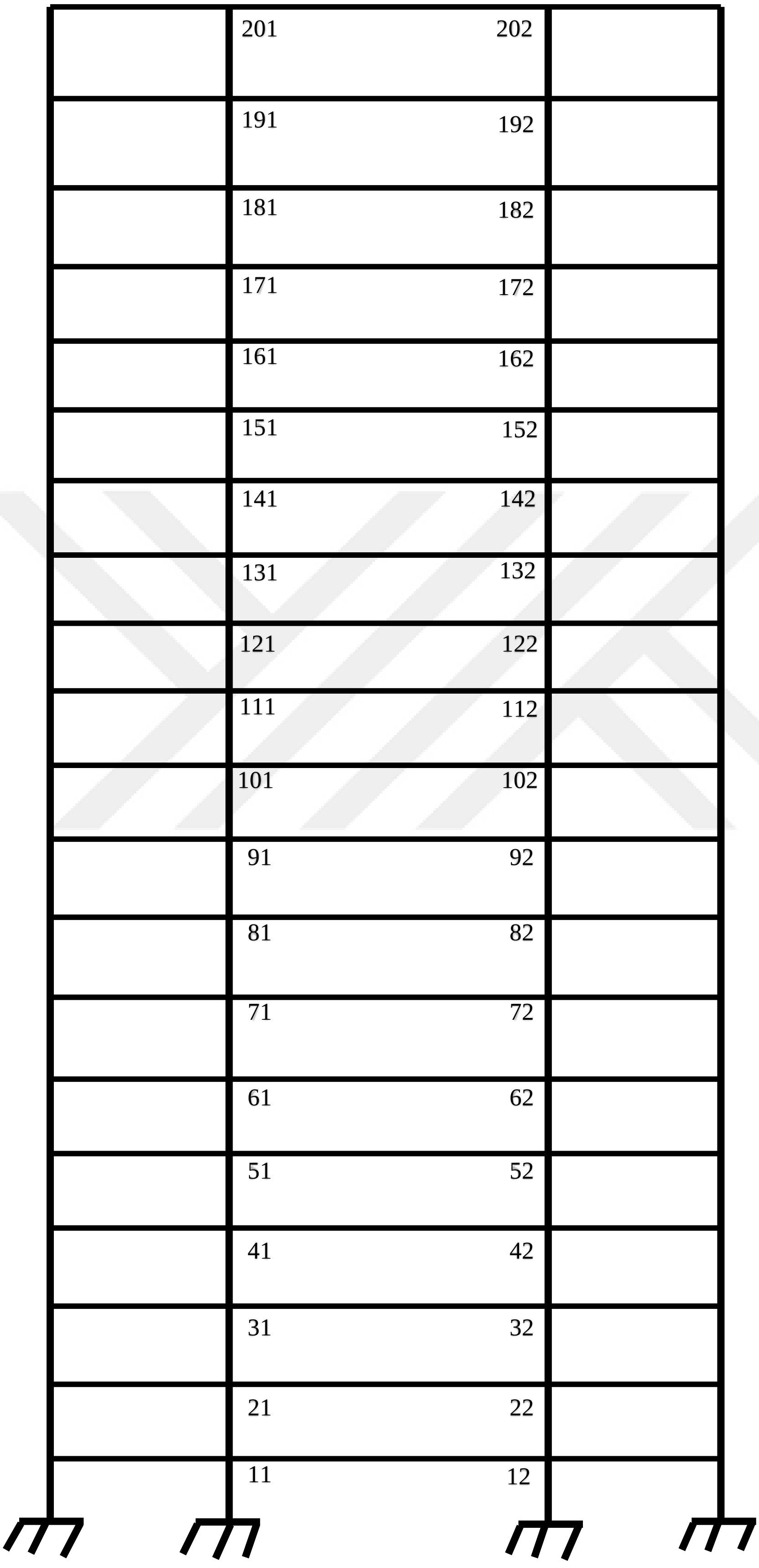

Figure 26. Joints' label for case study III

Table 3.4.3. SCWB ratio for case study III

Joints	SCWB ratio
11	4.83
12	4.46
21	4.71
22	4.40
31	4.58
32	4.33
41	4.46
42	4.27
51	4.34
52	4.22
61	4.24
62	4.16
71	3.95
72	3.91
81	3.67
82	3.67
91	3.59
92	3.61
101	3.48
102	3.56
111	3.37
112	3.48
121	3.26
122	3.39
131	3.16
132	3.31
141	3.12
142	3.20
151	3.10
152	3.15
161	3.05
162	3.11
171	3.01
172	3.07
181	3.00
182	3.03
191	3.00
192	3.00
201	0.96
202	1.06

3.4.1. Nonlinear analysis results for case study III

For a variety of ground motions, plots illustrating the relative displacement between floors, or inter-story drift ratio (IDR), have been produced. IDR is an essential statistic to assess damage and ensure safety during earthquake activity. The Lander ground motion has the lowest IDR on the first floor. However, the largest IDR was produced by the Chalfant Valley-02 ground motion at the seventeenth story. Larger IDRs usually result from increased peak ground acceleration (PGA) ground motion. Following the time-history analysis, Figure 28 displays the graphical findings of the IDR for various ground motions and tales.

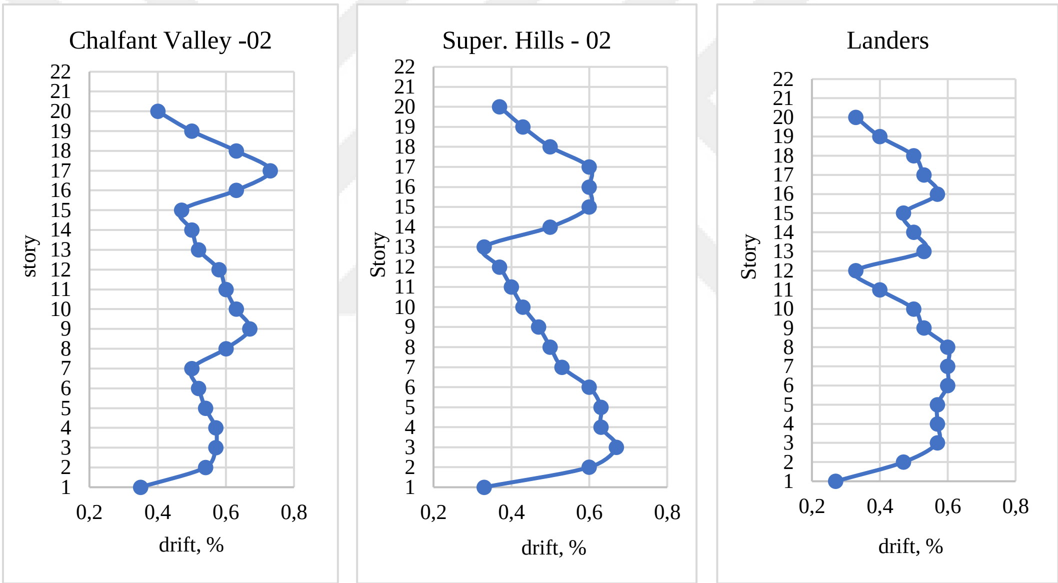

Figure 27. IDR results for selected ground motions

Base shear is one of the most important factors in seismic design since it shows the overall stress on the structure. It is the total lateral force generated by the ground motion of the earthquake. Three independent ground motions of varying intensities were chosen and employed for the nonlinear dynamic analysis for this case study as well. The results are shown in Figures 29–31. Lander ground motion is shown to contain the largest base shear (4200 KN) at roughly 15 seconds.

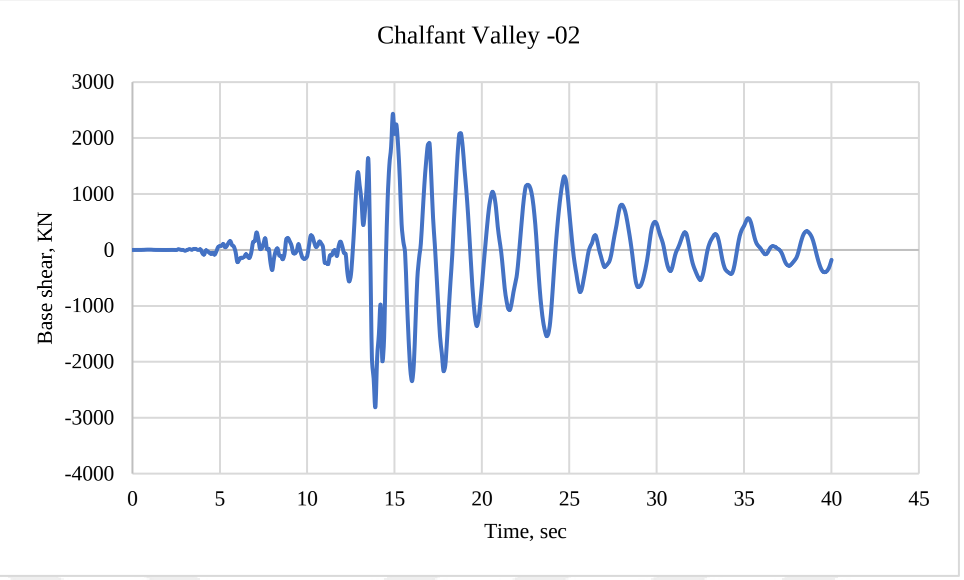

Figure 28. Base shear caused by Chalfant Valley -02 for case study III

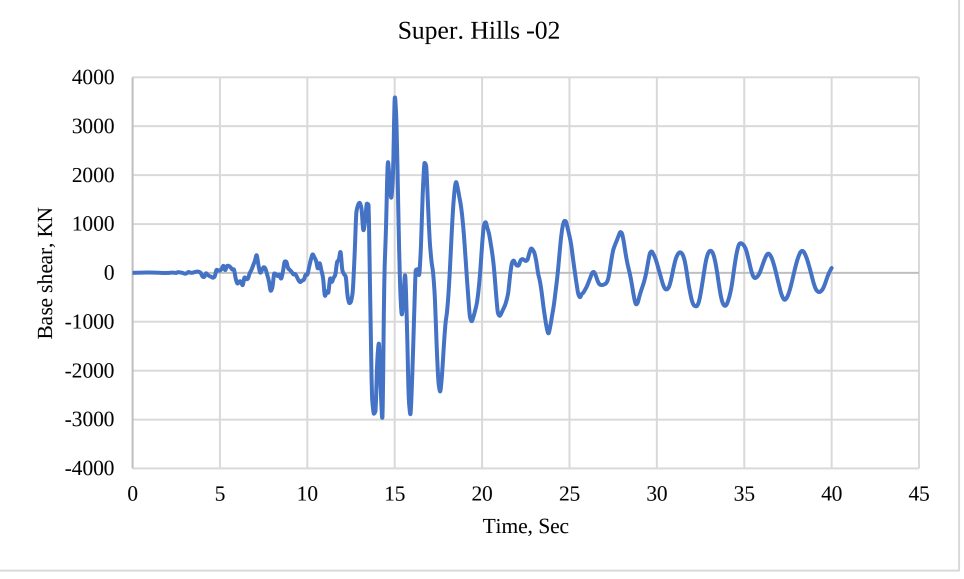

Figure 29. Base shear caused by Super. Hills -02 for case study III

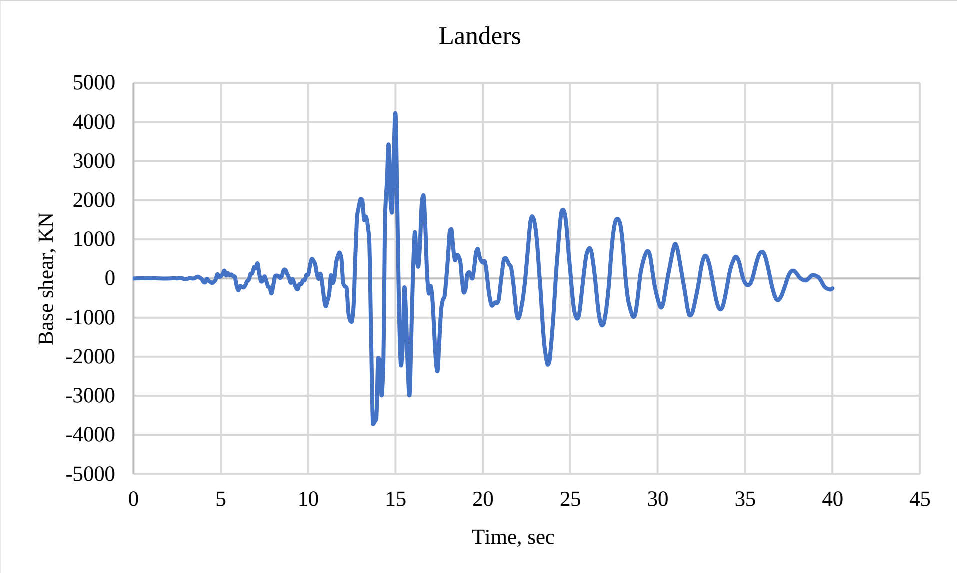

Figure 30. Base shear caused by Landers for case study III

The entire energy transmitted to the structure is known as total input energy, and it has a significant impact on building design. The total input energy to the structure resulting from various ground motions (Landers, Super Hills-02, and Halfant Valley-02) with varying intensities and PGA is displayed in Figures 32–34. At 28 seconds, the structure received the largest total input energy from Landers ground motion, which is approximately 800 m^2/s^2. Chalfant Valley-02 ground motion had the lowest energy experience (680 m^2/s^2).

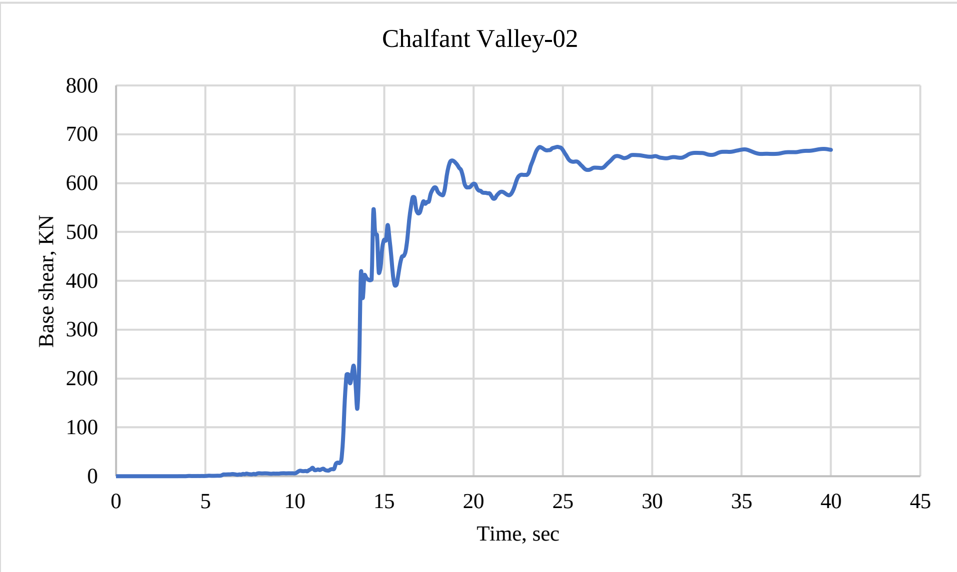

Figure 31. Total input energy - Chalfant Valley-02 for Case study III

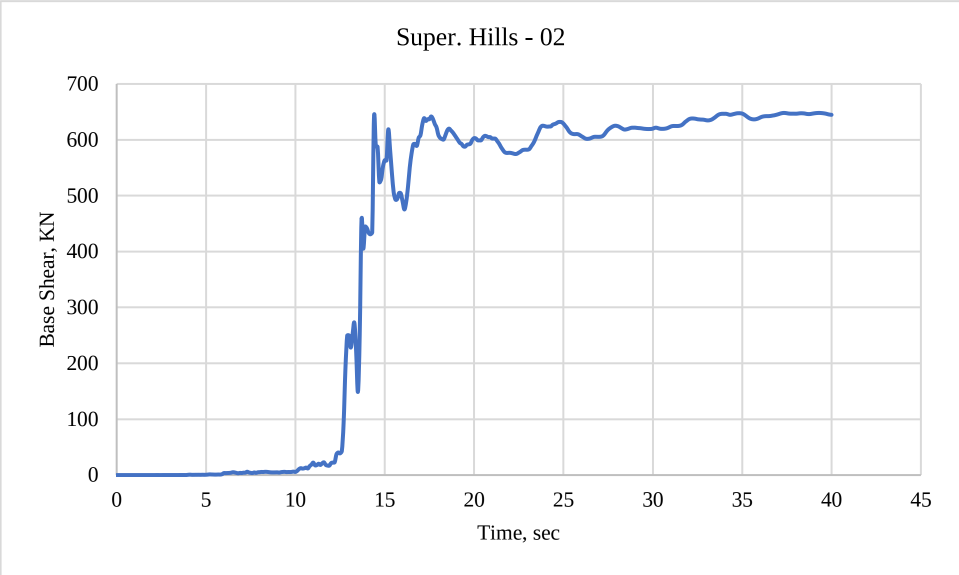

Figure 32. Total input energy Super. Hills-02 for Case study III

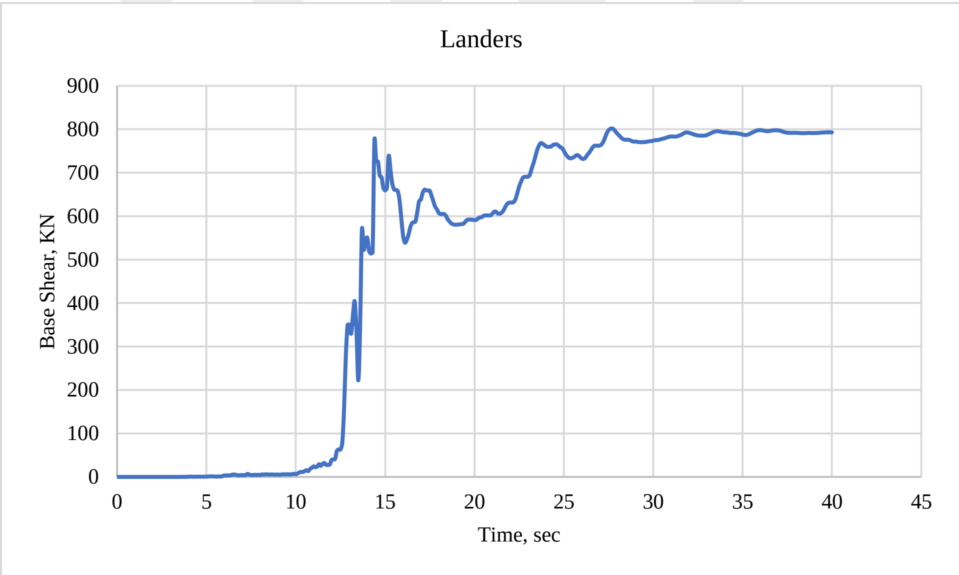

Figure 33. Total input energy- Landers- for Case study III

3.5. Result discussion and comparison

Figure 35 shows the comparison of periods for all modes and case studies. For the first mode, the highest period belongs to the case study I with SCWB of 1.2-2.0 for all modes. The lowest period belongs to case study III with SCWB of 3.0-5.0 for all modes.

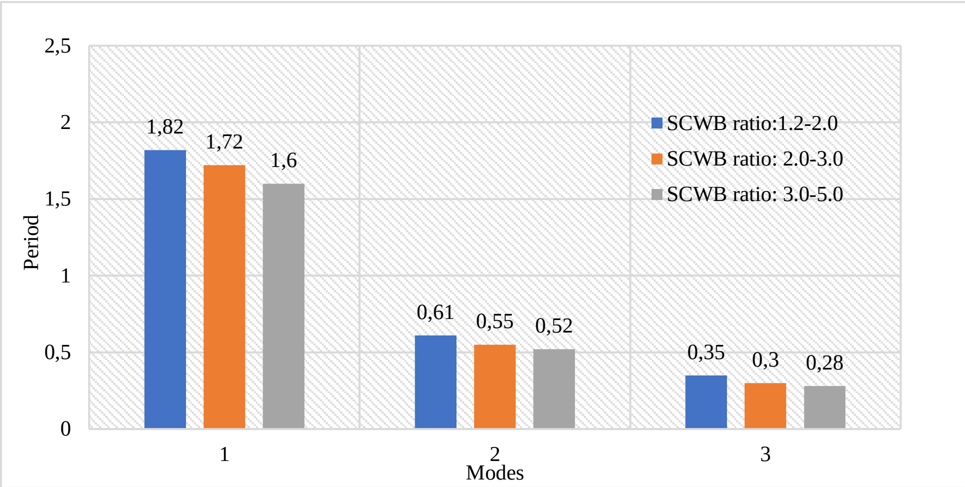

Figure 34. Period comparison for all cases

For Chalfant Valley-02 ground motion, the lowest IDR at the higher stories belongs to case study II where the SCWB ratio ranges between 2.0 and 3.0 as shown in Figure 36. However, for the lower stories, case study III shows lower IDR comparing with the others.

Figure 37 shows that for the Super. Hills-02 ground motion, the lowest IDR is for case study II (SCWB ratio of 2.0-3.0) at the higher stories. On the other hand, case study (I) experienced less IDR compared to the other case studies at the lower stories.

For Lander ground motion, case study I and III, both have the lowest IDR at the lower stories. The lowest IDR at the higher stories belongs to case study II which is shown in Figure 38.

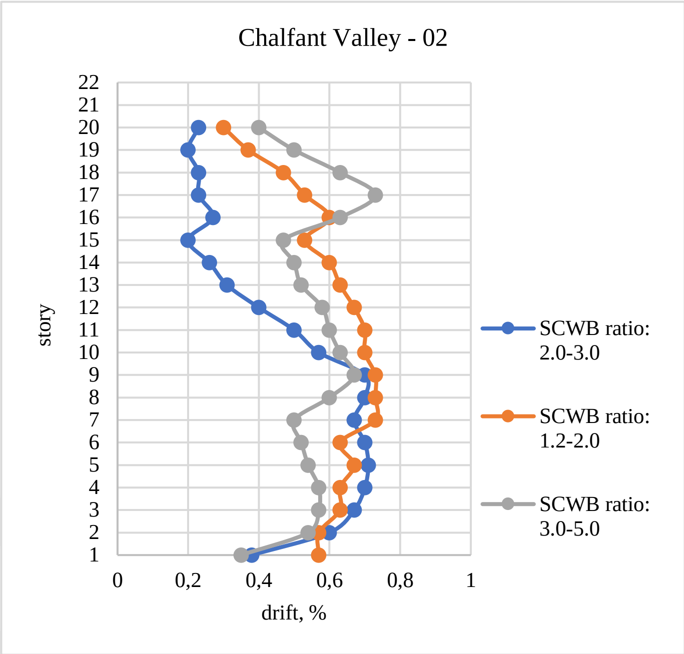

Figure 35. Comparison of IDR for Chalfant Valleys-02 ground motion for different SCWB ratios

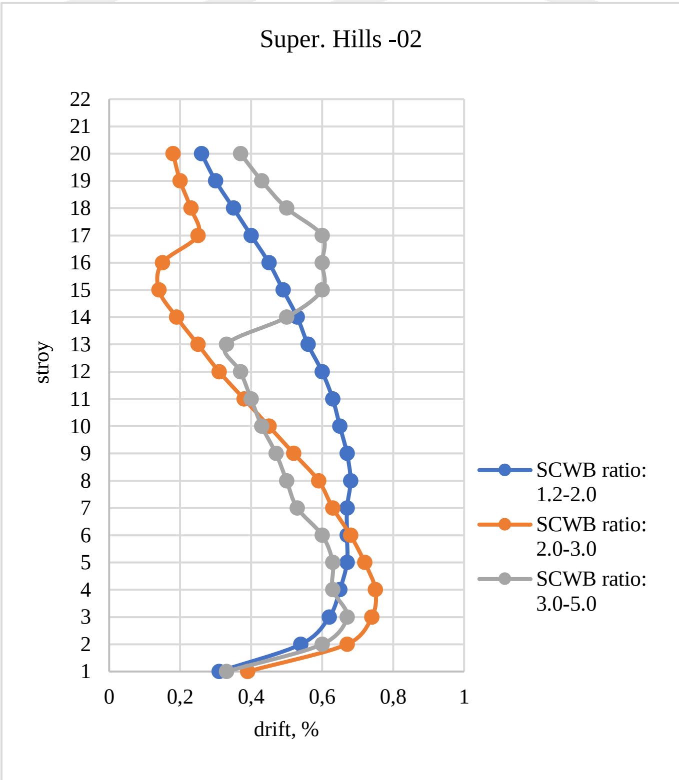

Figure 36. Comparison of IDR for Super. Hills-02 ground motion for different SCWB ratios

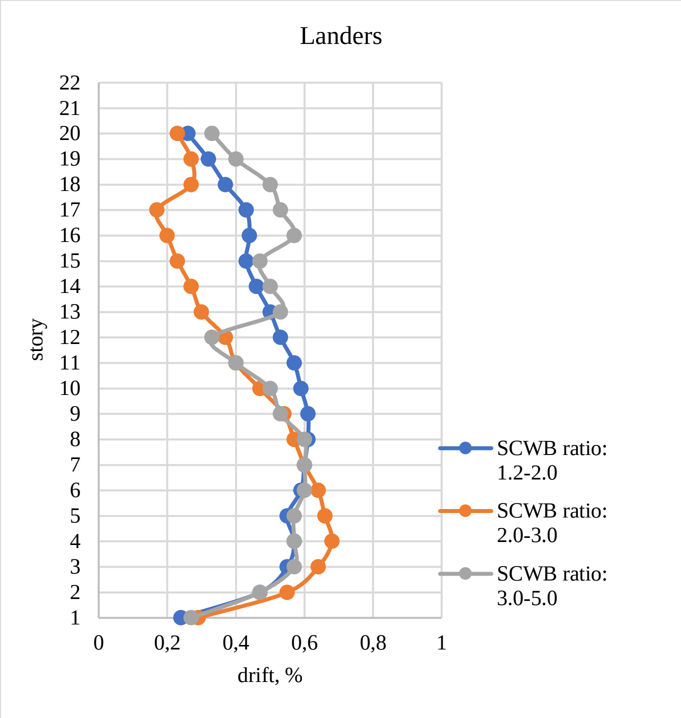

Figure 37. Comparison of IDR for Lander ground motion for different SCWB ratios

Figure 39 shows the base shear results for all case studies for Chalfant Valley-02 ground motion. It is shown that the case study with SCWB ratio of 2.0-3.0 experiences the highest base shear at around 17 seconds. It is also demonstrated that case study (I) has the highest base shear at early stages. The base shear reaches around 2700 KN in almost 10 seconds.

For Super. Hills-02 ground motion, the pattern is the same as the previous one as shown in Figure 40. At the early stage, case study (I) reaches the maximum base shear (3200 KN) and then the case study II experiences the highest value for the base shear (around 4000 KN) at around 16 seconds.

Lander ground motion like previous ones also experience the same pattern. At the early stages, case study (I) has the highest value of base shear and then the case study II governs as shown in Figure 41. The maximum base shear is around 4600 KN belongs to case study II that occurs at 18 second after the analysis starts.

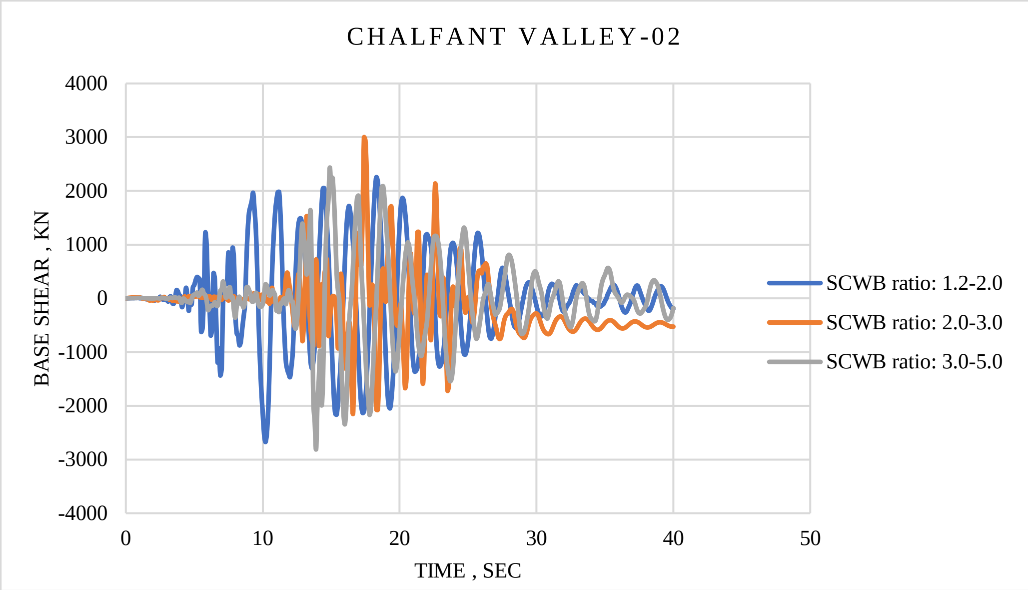

Figure 38. Comparison of base shear for different SCWB ratio - Chalfant Valley-02

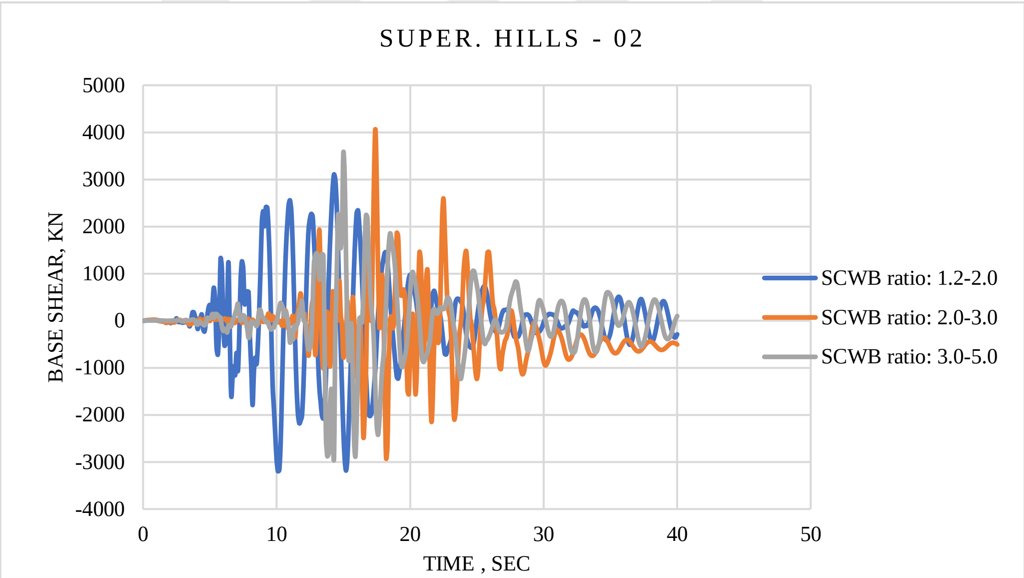

Figure 39. Comparison of base shear for different SCWB ratio - Super Hills. -02

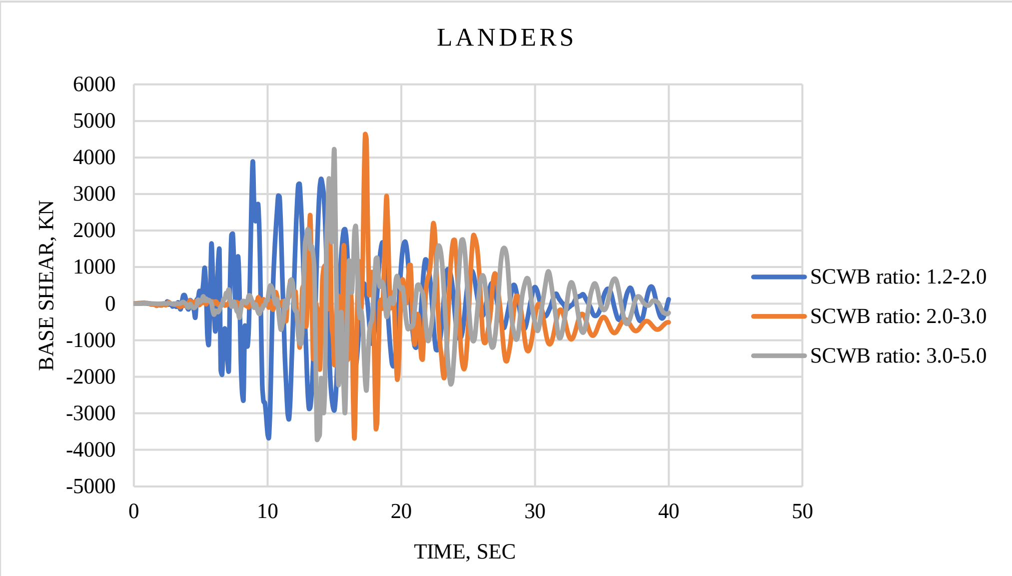

Figure 40. Comparison of base shear for different SCWB ratio - Landers

The highest total input energy belongs to case study (I) that is around 1050 m2/s2 at 26 seconds for Chalfant Valley-02 ground motion as shown in Figure 42. The other two case studies (II, III) have close results at some points.

For the Super. Hills-02 ground motion, the maximum total input energy also belongs to the case study (I) which is 1200 m2/s2. It occurs 15 seconds after the analysis starts. At the early stages, case studies II and III have almost the same total input energy and after 13 seconds, variation begins. However, this difference is not too much compared to case study (I).

For Lander ground motion, the path is the same as previous ones. Case study (I) governs and has the highest value of total input energy. For case studies II and III, at the early stages of analysis, the results are almost similar but after 13 seconds, the values begin to deviate from each other.

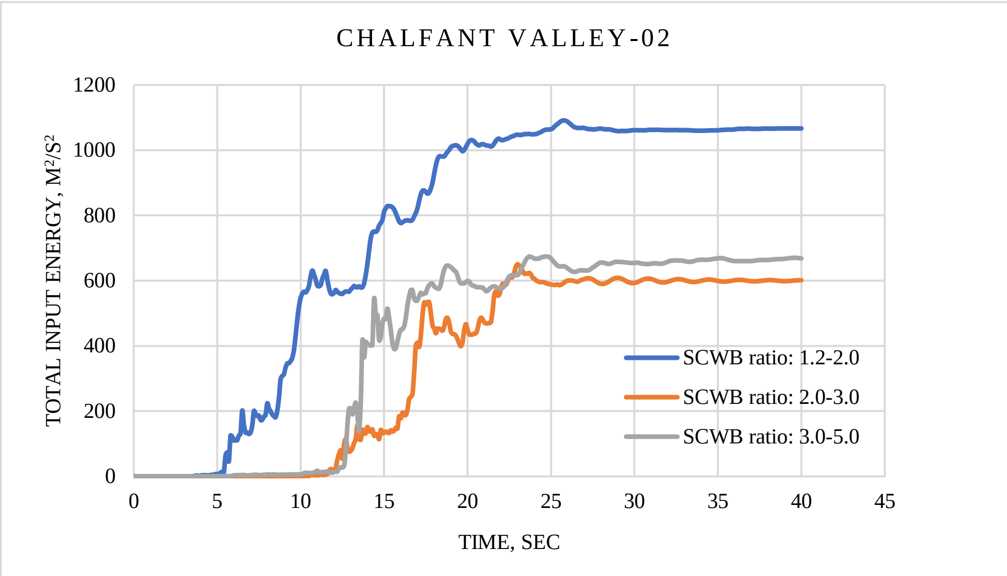

Figure 41. Total Input Energy comparison for all case studies- Chalfant Valley-02

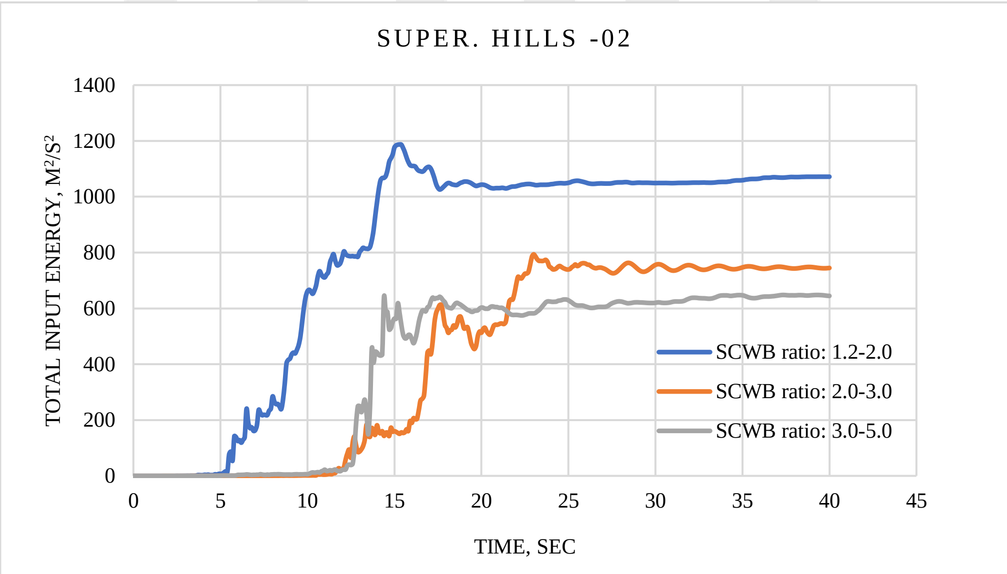

Figure 42. Total Input Energy comparison for all case studies- Super. Hills-02

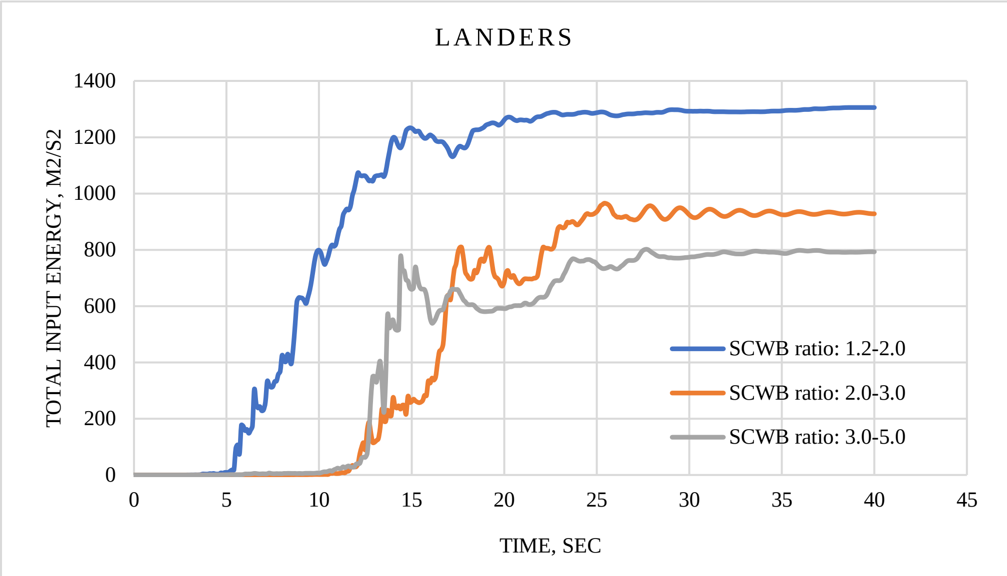

Figure 43. Total Input Energy comparison for all case studies- Landers

4. CONCLUSION

This study used nonlinear time-history analysis to examine the effect of strong-column weak-beam (SCWB) design ratio variation on a mid- rise reinforced concrete frame. Three discrete ground motions were used in the investigation of the frame, which has a 20-story moment frame system. The assessment was conducted utilizing the Turkish Building Earthquake Code (TBEC) with the aid of SAP2000 software.

With the use of SAP2000 software, the assessment was carried out using the Turkish Building Earthquake Code (TBEC). Although TBEC suggests a minimum SCWB design ratio of 1.2, several researchers have demonstrated that this figure may not be sufficient as building height increases. A time-history analysis for SCWB ratios between 1.2 and 5.0 was carried out. Three distinct case studies, each with a unique SCWB ratio range, were chosen, and each case study's design was completed in accordance with these ratios.

This study findings are listed below:

- The results indicate that, especially for mid-rise buildings, the suggested ratio for the SCWB design ratio might not be sufficient to provide the best possible structural performance. For two ground motions, the inter-story drift ratio (IDR) exhibits lower values, particularly for higher stories (Super. Hills-02, Landers).
- When the SCWB ratio is between 1.2 and 2.0 in the early stages, the base shear force achieves its highest value. However, when the SCWB ratio is more than 2.0, the base shear maximum value postpones.
- When compared to other cases, case study (I)'s total input energy has the highest value. This demonstrates that compared to other case studies, case study (I) absorbs greater energy from earthquakes.

In conclusion, this study's findings confirm the necessity of re-evaluating the present SCWB ratio recommendations in TBEC, especially for taller mid-rise RC buildings. Seismic resistance can be greatly increased with a higher SCWB ratio, but practical feasibility and economic factors must also be balanced in the design. Future studies should investigate other factors to further optimize SCWB ratios for various building types and seismic circumstances, such as structural imperfections and fluctuating ground motions.

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RESUME

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- Analyzed and designed a 5-story steel structure building.

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Auto CAT

Sap2000

Etabs

Computer programing (C++)

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