EARTHQUAKE RESISTANT DESIGN OF A MULTISTOREY RC BUILDING

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APPROVAL SHEET

This is to certify that we have read this thesis entitled **"Earthquake resistant design of a multistorey RC building"** and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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DECLARATION

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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ABSTRACT

EARTHQUAKE RESISTANT DESIGN OF A MULTISTOREY RC BUILDING

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By far, the highest natural hazards faced in the zones that have high seismicity are earthquakes. Albania is part of the Adriatic-Ionic and is considered as a moderate seismic region. During recent years Albania has faced a high number of small earthquakes, medium-sized Earthquakes (with a Magnitude M 5.4-5.9) and a smallest amount of large earthquake (considered earthquakes with a Magnitude M>6.5). Due to these earthquakes a lot of aspects of the communities where the earthquake happens are affected. Starting with life injuries, loss and damages on structures. Structures affected by earthquakes in recent years usually are left as they are, no structural repairs are made.

The main focus and purpose of this study is to analyze and design a multistorey building based on the seismic response of the building using the response spectrum and time history analysis. The finite element method (ETABS) is used to evaluate the behavior of structure, modelling and designing the multistorey building based on the ETABS analysis.

In this study, a 10-storey building, with 2 story's underground is taken into consideration. Based on the response spectrum analysis and time history analysis a comparison of results has been made and then the detailing and designing of the concrete frame elements. For designing and detailing the concrete frame elements CSI Detailing software is used. Results have been interpreted and an example of detailed design of concrete frame elements such as: slab, beam, wall and column are

presented in this study. To design the concrete frame elements the seismic method is used since one of this study objectives is doing the Earthquake Resistant design of a multistorey builing.

Keywords: *Structure, multistorey building, ETABS, earthquake, seismicity.*

ABSTRAKT

PROJEKTIMI ANTISIZMIK I NJË NDERTESE SHUMËKATËSHE BETONARME

Shaba, Denisa

Master Shkencor, Departamenti i Inxhinierisë së Ndërtimit

Udhëheqësi: Dr. Enea Mustafaraj

Deri më tani, rreziqet më të larta natyrore me të cilat përballen zonat që kanë sizmizëm të lartë janë tërmetet. Shqipëria është pjesë e territorit Adriatiko-Jon dhe konsiderohet si një rajon i moderuar sizmik. Gjatë viteve të fundit Shqipëria është përballur me një numër të lartë të tërmeteve të vogla, tërmete të mesme (me një madhësi M 5.4-5.9) dhe një sasi më të vogël të tërmetit të madh (konsiderohen tërmete me një madhësi M> 6.5). Për shkak të këtyre tërmeteve preken shumë aspekte të bashkësive ku ndodh tërmeti. Duke filluar me dëmtime në jetë, humbje dhe dëmtime në struktura. Strukturat e prekura nga tërmetet në vitet e fundit zakonisht lihen ashtu siç janë, nuk bëhen riparime strukturore.

Fokusi kryesor dhe qëllimi i këtij studimi është të analizojë dhe projektojë një ndërtesë shumëkatëshe bazuar në reagimin sizmik të ndërtesës duke përdorur spektrin e përgjigjes dhe analizën e historisë së kohës. Metoda e elementeve të fundme (ETABS) përdoret për të vlerësuar sjelljen e strukturës, modelimin dhe modelimin e ndërtesës shumëkatëshe bazuar në analizën ETABS.

Në këtë studim, një ndërtesë 10-katëshe, me 2 kate nën tokë është marrë në konsideratë. Bazuar në analizën e spektrit dhe analizës “Time History” është bërë një krahasim i rezultateve dhe më pas detajimi dhe modelimi i elementeve të betonit. Për dizenjimin dhe detajimin e elementëve betonarme të strukturës është përdorur program CSI Detailing. Rezultatet jane interpretuar dhe është marë një shembull dizenjimi dhe detajimi për secalin element betonarme si psh. : soleta, trarë, mure dhe

kolona dhe janë paraqitur në këtë tezë. Për projektimin e elementeve të struktures është përdorur metoda sizmike dhe projektimit të tyre duke qenë se një nga objektivat e këtij studimi është projektimi antisizmik I elementeve të një ndërtese shumëkatëshe.

Fjalët kyçe: *Strukturë, ndërtesë shumëkatëshe, ETABS, tërmet, sizmicitet*

ACKNOWLEDGEMENTS

In terms of gratifying my accomplishment of project, I would like to express that the project would not have been accomplished without the assistance and support of those who guided me in the course of this project.

I would like to express my appreciation to my honorable supervisor Prof. Dr. Enea Mustafaraj whose support, crtitics and help during this project has been key to materializing the project. Your insightful feedback pushed me to bring my work to a higher level and finalize the project in what I think is a good product.

In addition I would like to thank my family and friends. Words cannot express how grateful I am for all of the sacrifices you have made on my behalf.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

Problem Statement

Designing a multistorey building according to Eurocode and constructive rules can be a challenging process. Using the finite element method ETABS can reduce a lot of time in the designing procedure. It makes it easier to model the structure and gain different results that help in designing the structure elements and detailing. The structure being design should complete all Eurocode requirements and deisgning rules have to be brought in attention. In Albania construction industry is still on its early phases of growth and no seismic provisions have been incorporated into the existing buildings.

The aim of structural design is to accomplish an acceptable probability that the structure being designed will perform the function for which it is created and will safely withstand the influence that will act on it throughout its useful life.

An important part in the designing process is the Earthquake Resistant Design. The highest natural hazards faced in the zones that have high seismicity are earthquakes. Albania is part of the Adriatic-Ionic and is considered as a moderate seismic region. During recent years Albania has faced a high number of small earthquakes, medium-sized Earthquakes (with a Magnitude M 5.4-5.9) and a smallest amount of large earthquake (considered earthquakes with a Magnitude M>6.5). Due to these earthquakes a lot of aspects of the communities where the earthquake happens are affected. Starting with life injuries, loss and damages on structures.

The main focus and purpose of this study is to analyze and design a multistorey building based on the seismic response of the building using the response spectrum and time history analysis. The finite element method (ETABS) is used to evaluate the behavior of structure, modeling and designing the multistorey building based on the ETABS analysis.

In this study, a 10-storey building, with 2 storeys underground is taken into consideration. Based on the response spectrum analysis and time history analysis a comparison of results has been made and then the detailing and designing of the concrete frame elements. An evaluation and checking of results with Eurocode is also made in this study.

1.2 Objective

The objectives of this study are as follow:

1. To remodel a multistorey building structure with ETABS and designing and detailing its concrete frame elements based on ETABS rebar percentage results.
2. To make a comparison between the seismic response of the structure using the response spectrum and time history analysis.
3. To assess the structure design and model results with Eurocode standards
4. To asses the building performances in terms of structure displacement, deformed shape mode period and the comparison with Eurocode standards.
5. To detail and design concrete frame elements using CSI Detailing software.

1.3 Scope of study

1. Multistorey building ETABS modeling
2. Response spectrum and time history analysis
3. Seismic assesment response of multistorey building structure using response spectrum and time history analysis
4. Earthquake Resistant concrete frame elements design using CSI Detailing

1.4 Organization of the thesis

This thesis is divided in 5 chapters. The organization is done as follows:

In Chapter 1, the problem statement, thesis objective and scope of study is presented. Chapter 2, includes the literature review where general information for structurl systems of tall buildings and the seismicity of Albania is presented. Chapter 3, consists on the earthquake resistant design and the methodology of this study is described in details and a description of the case study is made. In Chapter 5, the experimental results and analysis are presented. In this chapter are interpreted the results obtained from ETABS oftware that are later on used to design the concrete frame elements. For the detailment of elements CSI Detailing software is used. In Chapter 6, conclusions and recommendations for further research are stated.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Tall building design is a complex process itself that takes various long studies and analytical investigations. While designing tall building Eurocode standards have to be brought in attention in order to design a structure that can safely carry lateral gravity and lateral loads. The design criteria are strength, serviceability, stability and human comfort. The structural engineer's aim is to arrive with suitable structural schemes to design a structure that satisfies all Eurocode standards and is a well design structure.

2.2.1. Basic rules of structural conception of buildings

- Structural simplicity

Structural systems must provide the easiest and most direct route of transfer or transmission of inertial forces arising from seismic action towards their footings. Structural simplicity is also characterized by the modeling, analysis, detailing and construction of simple structures, thu reducing the possibilities of various inaccuracies and thus predicting as accurately as possible the seismic response of the structure.

- Uniformity, symmetry and reserve

Structural systems must ensure uniformity of mass distribution and stiffness both in plan and in height. Systems must provide acceptable reserve, in the sense that if one of the structural elements can be destroyed in extreme cases, then there must be a range of other structural elements that can distribute the load that the destroyed element was carrying. So, the "redundancy" property is the ability of a structure to redistribute seismic forces, for a given design earthquake, in all its constructive elements, when partial damage has occurred and not complete collapse of the most charged element.

- The importance of intercostal behavior as a diaphragm

Simple and possibly symmetrical plans of buildings maintain the most efficient and predictable seismic response of each component. An important and relatively rigid interconnection of vertical components at appropriate levels is important. This is usually achieved with the use of interstices, which after ensuring the retention of vertical forces also realize the transmission of inertial forces caused by seismic oscillations in the structural elements of coping with horizontal forces. Vertical elements in this way will contribute to the total resistance against lateral forces, in proportion to their rigidity. Therefore, vertical elements will exist at every level.

- Tensile strength and stiffness

Such a phenomenon occurs in those buildings where the center of mass of the object does not coincide with the center of rigidity of the plane. Under seismic action appear seismic forces. These seismic forces are inertial forces and the point of their application is the center of mass of the floor taken in the study of the building. Meanwhile the resistive force of each floor will act on the center of rigidity. The center of rigidity is located depending on the planimetry of the building, the cross section of the vertical structural elements (column, wall) as well as the regularity of the building in height and plan. In this way we can express that the distance between the center of rigidity and the center of mass (d) causes the torque to arise. Moment is found as follows:

$M = F * \mathrm{d}$

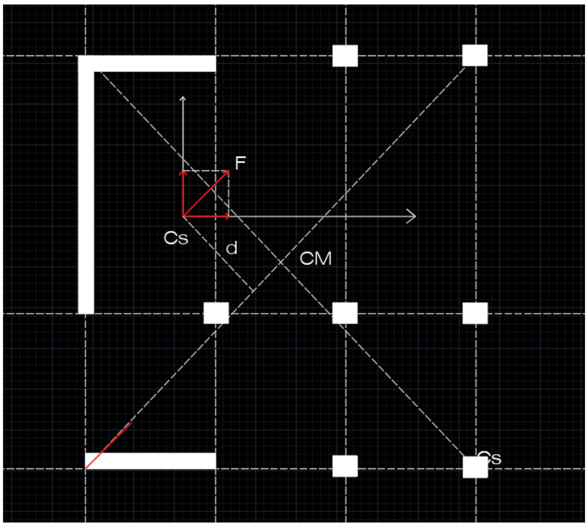


Figure 1 Geometric features

The planimetric shape of the building is a function of the investment demand and the investor's investment. But the task of the designer is to secure the object from the design earthquake. The configuration of the building is decisive in the way loads are distributed in the object. Regular shapes are very adequate against seismic actions because the torque that arises has relatively low values, while irregular shapes cause large values of torque at each floor level of the building. The regularity of the building has to do with the manner of distribution in plan and in height of mass, stiffness and resistance.

- Regularity in height

The most compact shapes in the height of buildings are cones, pyramids, orthogonal prisms and others. Irregular configurations are characterized by variations in the height of the object, giving narrowing’s (set-beck) and extensions (off-set) in the height of the object. It is these types of objects that have mass distributions and stiffnesses at the height of the non-uniform structure. This means that the transfer of seismic forces is non-uniform and consequently the concentration of absorbed energy occurs at specific points of the object. If the object consists of parts that change height, the use of ant seismic joints serves as a solution to the problem.

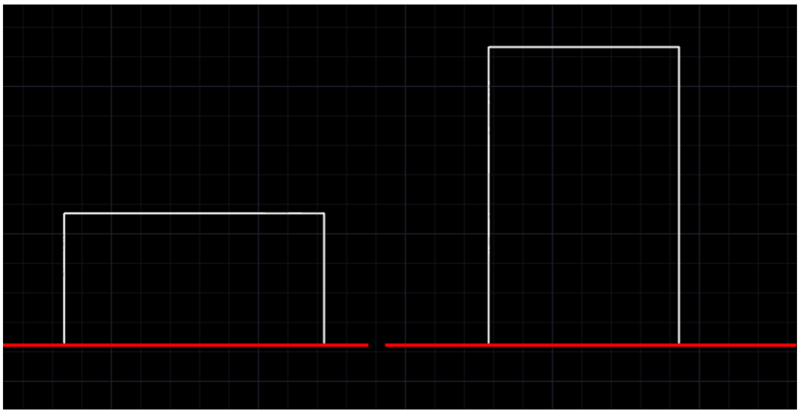


Figure 2 Buildings that are regular in height

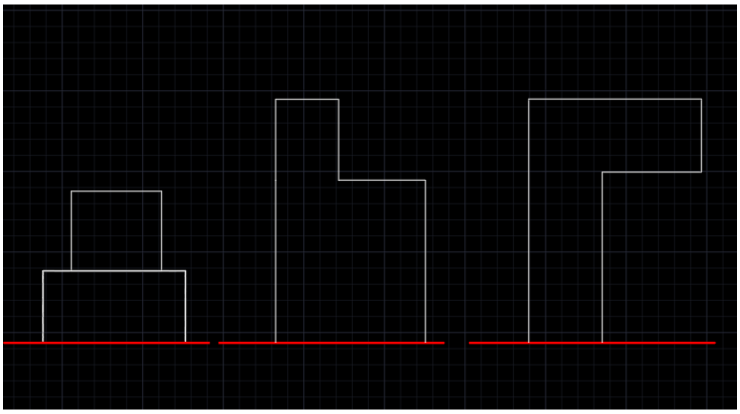


Figure 3 Non Regular in height buildings"

- Regularity in the plan

Preferred shapes in the plan are symmetrical ones like circle or square. This is because the seismic response of such objects is quite predictable. For irregular shapes in the plan, it is best to use antiseismic joints in order to divide the complex plan into smaller regular port plans.

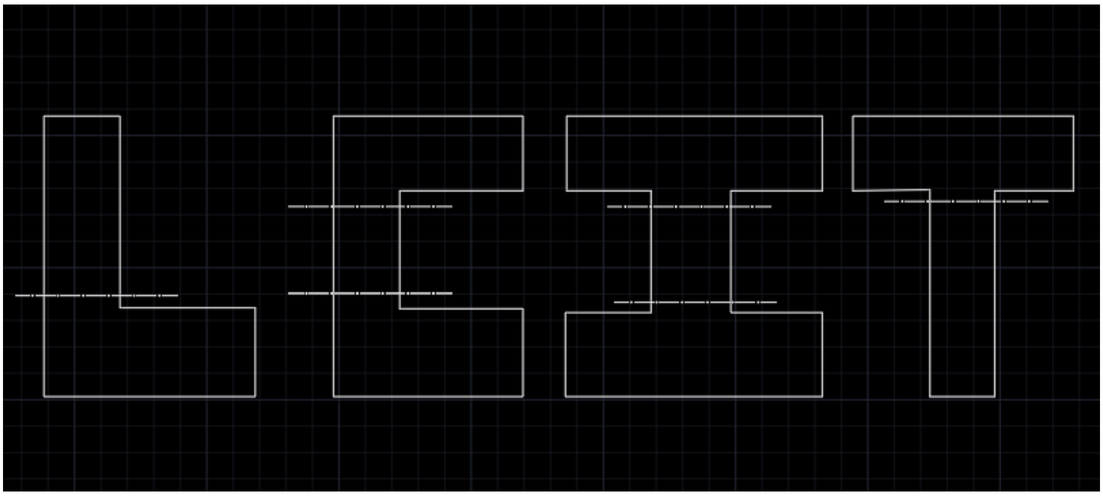


Figure 4 Non regular buildings that are divided with antiseisic deviders

2.2.2. Resistant systems of structures

The primary purpose of all structures used in buildings is to withstand the gravity load. However buildings can also be subject to loads or lateral forces due to wind or earthquakes. In a tall building the most important are the side loads and their effects. The three most commonly used structural systems are:

Frame systems

Frame system is the system in which vertical and lateral loads are borne mainly by spatial frames. Frames can be planar or spatial.

The concept of frame system is a simplification we make and is used in cases where we have complete symmetry both in plan and in height. Only in these cases can we perform the calculation of the structure in the plan. The resistance of horizontal forces (including vertical ones) is provided by means of resistance bending moments in the beam and column elements.

Capacity design is based on the concept of "strong column - soft beam". The formation of earthquake energy quenching points or as they are otherwise known: plastic cracks must be formed first in the beams of the frame. Only in this way will we have a maximum mobilization of the structural elements to withstand the forces caused by seismic oscillations. If plastic hinges were to be created initially in the columns, this would lead to the creation of a mechanism and consequently the loss of durability. Exactly such design conditions according to the capacities are provided through the analysis of the capacities of the joints.

$$\sum M_{RC} \geq 1.3 * \sum M_{Rb}$$

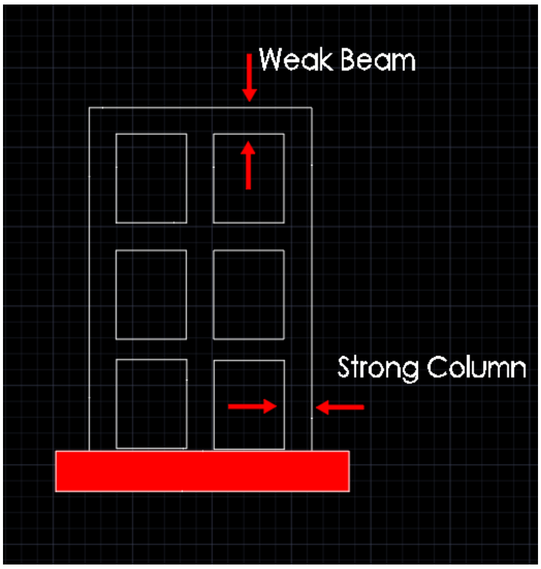


Figure 5 The concept of "Capacity Design" in frame systems

Wall Systems

This is a special type of systems where vertical and lateral loads are borne by vertical walls called "structural diaphragms". In these systems, the walls can work separately, but can be also connected by ductile beams that realize their cooperation, reducing the operating effects on them.

The advantages of wall systems are:

- Walls are elements with great rigidity as a result:
- They are unaffected by the local or global effects of the filler.
- They protect or limit damage in frequent or accidental earthquakes.
- The walls offer a perfect protection against collapse, as a result of the impossibility of double curvature they do not allow the creation of soft floors.

- Being less ductile wall element in contrast to the frame requires a less skilled workshop and supervision.
- Geometric effects and other phenomena on large walls are favorable for seismic performance.
- The wall system is more effective than the frame system in terms of seismic resistance.

The disadvantages of the wall system are sides of walled systems are:

- Walls are essentially less ductile elements compared to beams and columns and more sensitive to the action of shear forces and more difficult to detail for ductility.
- Walls offer a limited redundancy and few alternatives to the force transmission path.
- The walls do not allow flexibility to the architect, especially in the facade, and occupy more space in the plan compared to the frame system.
- It is not effective to use only walls to withstand gravitational forces, some beams and columns will be needed for this reason.
- To avoid large eccentricities or stiffness in the lower torsion of the floor, walls with a large contribution to the stiffness and stiffness in the side (stair and elevator cores as well as perimeter and large walls) require walls, stiff and rough hardness, to reduce these effects.
- It is hard to realize an appropriate footing for the walls especially for separate foundations. It is difficult to accurately assess the way the wall is fastened to the footing.

Here are some reservations concerning the seismic response of the walls:

- The performance for cyclic loads and seismic performance of walls is less well known than that of frames, because experimental research is more difficult to perform and their analytical model needs to be more advanced and sophisticated.

- Rotation against the neutral axis of the wall cannot be reliably assessed in design practice.
- Walls are more complex to model, analyze, dimension and reinforce in design practice especially for those who have a complex section (L-shaped, T-shaped etc.

Dual or Mixed Systems

We call dual systems those systems where the resistance of vertical loads is usually realized by the spatial frame, while the resistance against the lateral loads is provided partly by the frame and partly by the structural walls. Depending on the contributions that ying give to each of the component systems, above 50% and below 50%, dual systems are considered as frame-equivalent and wall-equivalent respectively. Dual structures or systems are effective antiseismic conceptions, especially for tall and very tall buildings.

Dual systems

In seismic areas the structure must be able to withstand not only gravitational forces or loads, but also lateral ones such as wind and mainly earthquakes. Seismic load in the structure causes deformations and stresses in different directions, which must be borne by the structure itself and its elements. Not all elements are critical regarding the seismic response of the structure. For example columns have a greater capacity to withstand seismic forces compared to beams. This is because the destruction of a column leads to the destruction of the beams and slabs that it holds, and consequently this leads to the destruction of other columns up to collapse of the structure. In the other way around when beams suffer local damage, this does not lead to collapse of the structure.

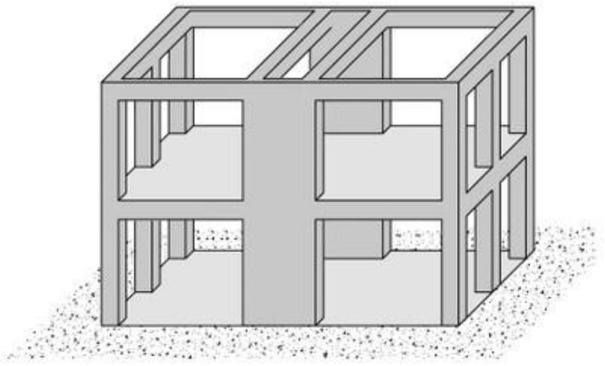


Figure 6 Dual Systems (https://taxonomy.openquake.org/)

To make the structures even more rigid, a combination of the two systems mentioned above is realized, thus creating mixed systems with frames and walls, otherwise called "Dual Systems".

2.2 SEISMICITY IN ALBANIA

2.2.1 Earthquake characteristics

An earthquake is a natural hazard, resut of rock shifting underneath the earth's crust. Earthquakes can cause injuries, life loss and structural damages. Most injuries related to earthquakes are from wall collapse, flying glass and object failing due to ground motion. Earthquake damages vary from one place to another due to the characteristics of the region. Earthquakes can also cause buildings and bridges to collapse.

2.2.2 Seismicity in Albania

Albania is a Balkan country located in the Alpine fold belt at the central part of the Dinarides-Hellenides arc with high rate of seismicity. Earthquake risk reduction has been an important concern for Albania. It is geologically and seismotectonically a complicated region. During recent years Albania has faced a large number of earthquakes that are categorized as small earthquakes with M less than 5, Earthquakes categorized as medium-sized have a a Magnitude of 5.4-5.9 and a smallest amount of large earthquakes that are considered the ones with a Magnitude $M>6.5$.

Antiseismic regulations have been taken in consideration in Albania since 1952. The first regulations for the seismic calculation of buildings was the static method that was later on replaced with the dynamic method in 1963. Albania is currently using the Asesismic Regulation KTP-N-2-89 which is in force in the moment of speaking.

In the last decades, a large amount of mass movements caused by rainfall and man-made works caused by earthquakes have been studied in Albania (Perrey 1853; Mihailoviç 1927; Mihailoviç 1951; Boué 1951; Shehu and Dhima 1983; Koçiaj 2000; Ambraseys 2009; Shehu and Dhima 1983; Dibra 1983; Sulstarova 1999; Aliaj et al. 2010, Muceku and Korini 2014; Muceku et al. 2016).

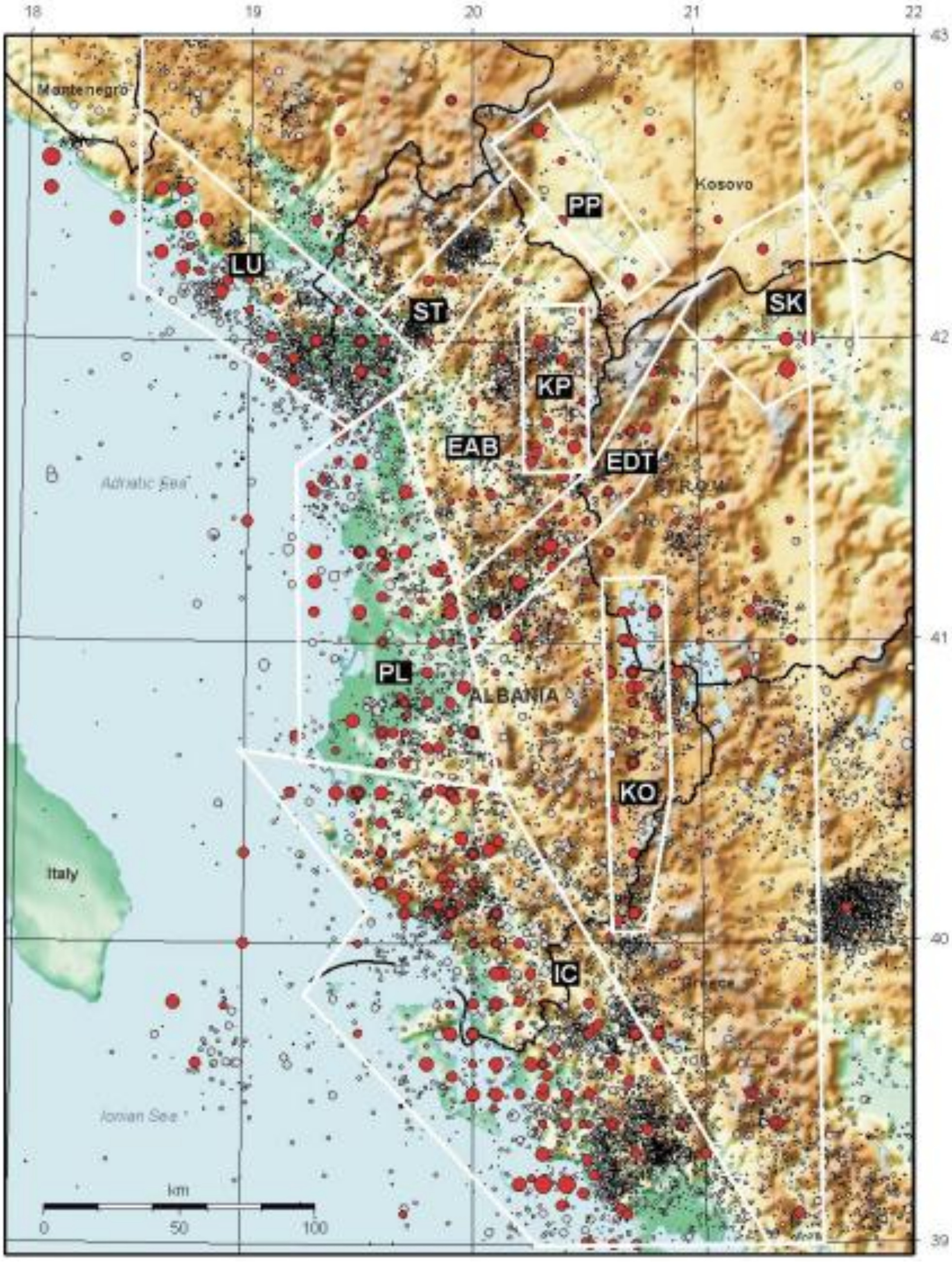
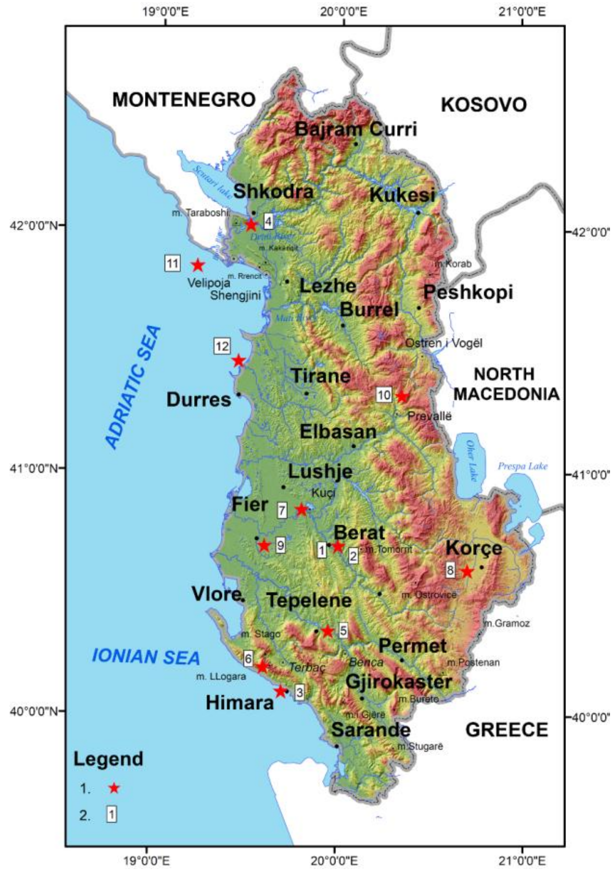


Figure 7 Albania's seismicity map

Albania has been exposed to mass movements that are strongly related to Earthquake's magnitude. Over the yers Albania has faced a lot of strong Erthquakes that are mostly historical evidence not supported by instrumental data records.

Nr	Date	Time (Local)	Lat (N^o)	Lon (E^o)	Hypocentral depth (km)	M	Location	Seismogenic zone
1	17.10.1851	01:30	40.70	20.00	33	6.6	Berat	Periadriatic Lowland
2	29.12.1851	10:30	40.70	20.00	13	6.0	Berat	Periadriatic Lowland
3	14.06.1893	-	40.10	19.70	6.6	6.6	Himarë	Ionian Coast
4	01.06.1905	04:42	42.02	19.50	15.6	6.6	Shkodër	Lezhë-Ulqin
5	26.11.1920	08:51	40.35	19.95	6.1	6.4	Tepelenë	Ionian Coast
6	21.11.1930	02:00	40.20	19.60	5.2	6.3	Vlorë	Ionian Coast
7	01.09.1959	11:37	40.85	19.80	16.9	6.4	Lushnje	Periadriatic Lowland
8	26.05.1960	05:10	40.60	20.70	7.6	6.5	Korçë	Ohrid-Korçë
9	18.03.1962	15:30	40.70	19.60	14.4	6.0	Fier	Periadriatic Lowland
10	30.11.1967	07:23	41.32	20.34	7.9	6.6	Dibër	Kukës-Peshkopi
11	15.04.1979	06:19	41.85	19.21	16.8	6.9	Montenegro	Lezhë-Ulqin
12	26.11.2019	03:54:12	41.46	19.44	22	6.4	Durrës	Periadriatic Lowland

Table 1 Strong earthquakes that have triggered the mass movements (Papazachos et al. 2001; Sulstarova and Aliaj 2001; Sulstarova and Koçiaj 1975; Koçi et al. 2019, Papadopoulos et al. 2020)



1. Earthquake location; 2. earthquake number.

Fig. 7. Epicenters of strong earthquakes, which have triggered mass movements. (June 2021, Earthquake-triggered mass movements in Albania article, Y.Muceku)

2.2.3 Earthquake zones

Albania is categorized into 10 seismic sources that are determined from present-day tectonic regime of the region and the full catalogues for smaller earthquakes from 1964-2000.

10 seismic zones are as listed below:

1. **Lezha-Ulqini (LU)**
2. **Periadriatic Lowland (PL)**
3. **Ionian Coast (IC)**
4. **Peja-Prizreni (PP)**
5. **Kukesi-Peshkopia (KP)**
6. **Ohrid-Korca (KO)**
7. **Shkodra-Tropoja (ST)**
8. **Elbasani-Dibra-Tetova (EDT)**
9. **Skopje (SK)**
10. **Eastern Albanian Background (EAB)**

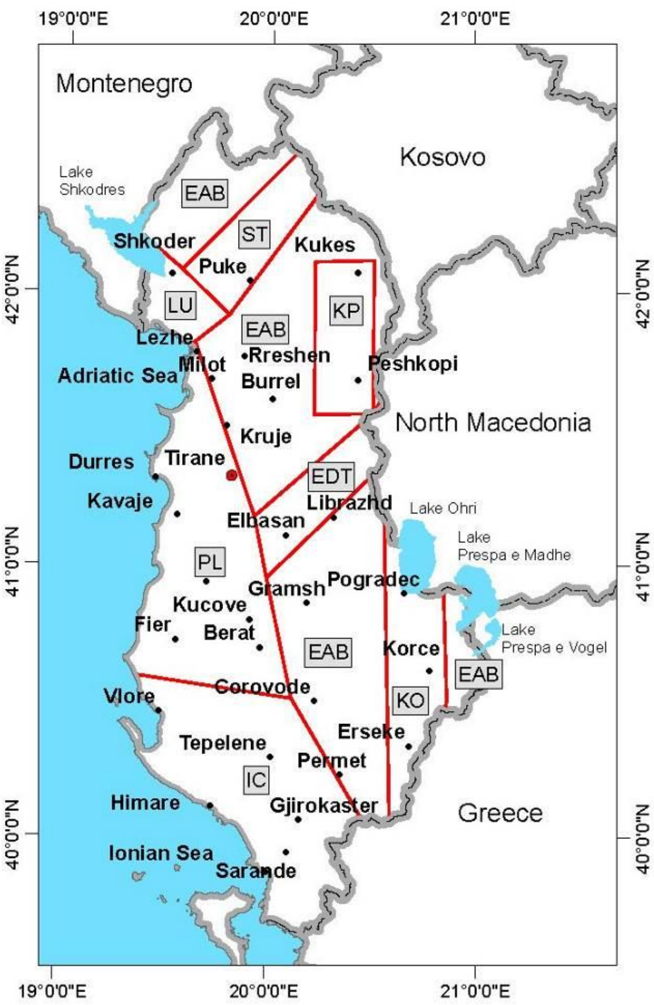


Figure 8 Map of 10 seismic zones in Albania (Aliaj 2004)

2.2.4 Maximum Magnitude

Albania as any other region with a high rate of seismicity has faced a lot of earthquakes while the higher amount of them is small-sized and medium sized. As for the large earthquakes with a Magnitude greater than 6.5 the number of them that Albania has faced is smaller.

2.2.5 Historical seismicity of Albania

Previous researches and studies about historical seismicity of Albania are made by Moreli, 1942; Mihajlovic, 1951; Shebalin et al. (Eds.), 1974; Sulstarova and Koçiaj, 1975; Makropoulos and Burton, 1981; Papazachos and Papazachou, 1989. For a period more than 2000 years Albania has faced 55 strong earthquakes. 36 of the 55 earthquakes date back to 19 century which makes us to believe that this number might be underestimated and that the history hides a lot more earthquakes that mights have occurred in Albanian territory. The most stricken by earthquakes towns are Durres and Butrint. In Durres strong earthquakes have caused a lot of damage on human life as well as on the economic aspect. Durres is the one city that not only has been nearly destroyed on the year 117 B.C, 334 or 345 A.C., 506, 1273, 1279, 1869 and 1870 but has also been stricken by strong earthquakes in the present time. As for Butrint, it was destroyed by a strong earthquake and the remains of the old town are found in the present time. Strongest earthquakes that date back in history of Albania over the years are as follow:

- **Earthquake of October 12, 1851 in Vlora**
- **Earthquake of October 17, 1851 in Berat**
- **Earthquake of June 14, 1851 in Himara, Kudhes village**

For the Earthquakes mentioned before no instrumental data records are available. The instrumental evidence of past earthquakes was available only on the end of 19th and beginning of 20th century. The instrumental evidence of earthquakes that hit Albania was made possible by seismological stations in Europe. Instrumental earthquake evidence in Albania date as follow:

- **1905 Earthquake** that hit Shkodra which aftershocks continued up to July 1906;
- **February 18, 191.** It’s aftershocks continued during all 1911 and 1912 year hit the region around Ohrid lake;
- **November 26, 1920** followed by a great number of aftershocks as well. This earthquake destroyed the town of Tepelena and surrounding villages
- **November 21, 1930** that hit in Llogara Pass of Çika mountain (Vlore) a very strong earthquake occurred, which destroyed totally the villages: Dukat, Terbaç, Palase and Dhermi.
- **September 1, 1959** (M=6.2) hit villages close to Kuçi bridge (Lushnje district) and towns of Lushnja, Fieri, Rogozhine, Peqin, Kuçove and Berat.

- **May 26, 1960** the region of Korca (M=6.4)
- **March 18**, **1962** with epicenter in Fier with a magnitude of 6.0
- **November 30, 1967** with a 6.6 magnitude hit Librazhdi and Dibra in Albania and in Western Macedonia
- **April 15, 1979**. This earthquake is also called the Montenegro earthquake and had a magnitude of 6.6 to 7.2.
- **The earthquake of Tirana, January 9, 1988**

2.2.6 Depth

While erathquakes take place in different depths most common depth for Albanian earthquakes is 10 km.

2.3 Response of multistorey building

In this study a multistorey building (10 storey and 2 underground storeys) is taken into consideration. The response of the building structure depends on strength, stiffness and ductility. Stiffness is the ability of a component or components for load resistance at a given response station. Ductility is the ability of a components or components to deform beyond the elastic limit.

Horizontal drift measures the lateral deformability of structural system. In buildings storey drifts Δ are the absolute displacement of any floor relative to the base, while inner storey drift δ defines the relative lateral displacement between two respective floors. The overall displacement is affected by the structural system that is used.

CHAPTER 3

METHODOLOGY

3.1 EARTHQUAKE RESISTANT DESIGN

Design philosophy is a great term we use for the basics of design. The importance of design philosophy becomes high when seismic consideration dominates design. The process of designing a building is a single one, but in the meantime, it must simultaneously satisfy different requirements and criteria, which correspond to the intensity and corresponding frequency of possible earthquakes. In order to carry out a construction project, presupposing for it a reasonable seismic safety, design criteria are applied such that, for any expected situation, the building is guaranteed acceptable seismic responses. Depending on the intensity of the earthquake considered these reactions are differentiated between them. In accordance with the intensity of earthquakes, the so-called "basic requirements" are determined, as well as the corresponding design criteria and the respective boundary conditions.

- Earthquake resistant design requirements
- Injury limitation claim

The design criteria that responds to this requirement is to withstand moderate earthquakes, is not strong and relatively frequent, in such a way that only some very limited deformations and damages are allowed, which do not compromise the specific requirements of the function. of the building. The design that refers to this requirement or the above criterion is known as "Design according to the limit state of use or functionality". The seismic action to be considered for this requirement has a probability of overcoming P_{DLR} = 10% in 10 years and a recurring period T_{DLR} = 95 years.

- No-collapse requirement

The structure shall be designed and constructed to withstand the design seismic action without local or global collapse, thus retaining its structural integrity and a residual load bearing capacity after the seismic event. (CEN 1998-1)

As a design criterion for this requirement is: to be able to withstand a strong and relatively rare earthquake, which may occur during the life of the object, in such a way that there is no such structural damage that causes collapse, local or global destruction (collapse) of the building, which would be dangerous to the safety of people. Antiseismic design should ensure that after the earthquake, the structure still retains a structural integrity and considerable bearing capacity. The corresponding earthquake of this requirement is considered as "design earthquake", while the design that refers to the above criteria is known as "Design according to the latest boundary condition". The seismic action to be considered for this requirement has a probability of overcoming P_{NCR} = 10% at 50 years and a repetitive period T_{NCR} = 475 years.

- The main structural features in antiseismic design

The specific structural data to be considered in relation to the antiseismic design of a structure under consideration are:

- Stiffness

A certain value of rigidity of a structural system (which must withstand the horizontal forces caused by seismic oscillations in order to provide displacements in the elastic stage of the primary elements for moderate and design earthquakes) must provide such deformations limited which do not bring about local or global collapse or collapse of the structure. These relationships are known and established by the principles of structural mechanics, using the geometric data of the component parts and the corresponding elasticity modulus. A typical nonlinear relationship force - displacement is as follows:

This relationship describes the reaction of a reinforced concrete component in relation to displacements with a constant increase. It presents two types of destructions: ductile and amorphous (brittle), as well as the corresponding idealized reactions. Through one of the ideal or bilinearized reactions stiffness of the structure is determined as follows:

$$K = S_y / \Delta_y$$

- Resistance

The structural system of coping with horizontal forces caused by seismic oscillations, must have a certain value of resistance, in order to ensure that no structural element under the action of a moderate earthquake is damaged. As in the case of rigidity, the strength is required to be sufficiently uniformly distributed in the structure under consideration, in order to avoid the differences in capacities required in the structural elements.

- Ductility

Most buildings possess a moderate solidity, reduced compared to what would be observed during a reaction assumed to be completely elastic to eventual strong earthquakes. To withstand the internal forces that would arise in each element in the elastic stage, heavy dimensions and constructions would be needed, surpassing the possibility of an economic design. Minimizing damage and ensuring the resistance of each structural element from strong earthquakes, is achieved by making such a design that these elements possess resistance even in their inelastic stage. This quality is known as ductility. Ductility means the property to allow relatively large deformations without having partial reductions in the rigidity, strength or bearing capacity of the structure.

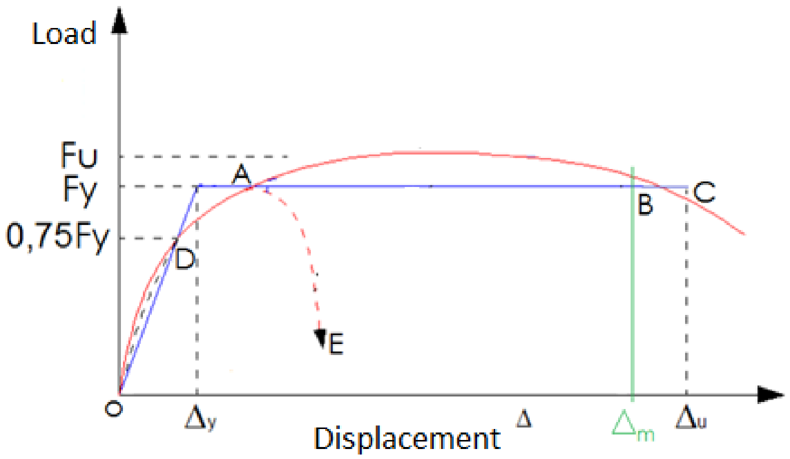


Figure 9 Typical load-displacement relationship for a reinforced concrete element (Pojani N.Seismic Engineering 2003)

A ductile destruction is represented by the OABC line (Figure 11), while an amorphous destruction is represented by the ODE line (Figure 11). Amorphous destruction means the almost complete loss of strength or solidity thus giving no possibility of warning of destruction or collapse of the structure. So amorphous destruction is the main cause for the collapse of buildings by earthquakes and the loss of human lives, and therefore must necessarily be avoided. Ductility is determined by the ratio of total displacements Δ_u to fluctuations of the onset of flow Δ_y, as follows:

-Ductility close to destruction

$$\mu_u = \frac{\Delta_u}{\Delta_y}$$

- Ductility for the determination of seismic resistance

$$\mu_m = \frac{\Delta_m}{\Delta_y}$$

Factors affecting the magnitude of ductility are as follow:

- Quality of construction materials and their physical-mechanical properties
- The shape of the cross sections, is the geometry of the structural components and of the structures as a whole .
- The connections between the structural elements and the way of their realization
- Amount of transverse and longitudinal reinforcement
- Nature of predominant joints (bending or pressing)
- How to apply loads during the loading-unloading process, static or dynamic

Ductility can also be expressed in forms other than the one expressed above, which is characterized by the magnitude of the displacements:

- Curvature ductility

$$\mu_\emptyset = \frac{\emptyset_u}{\emptyset_y}$$

- Rotation ductility

$$\mu_{\Theta} = \frac{\Theta_u}{\Theta_y}$$

- Ductility of the material

$$\mu_{\varepsilon} = \frac{\varepsilon_u}{\varepsilon_y}$$

For the above reasons ductility is the most important factor required by designers of buildings located in regions with high seismicity. Analytical and experimental studies that have been carried out for years have made possible the best possible knowledge of the ductile behavior of the elements and factors that influence this parameter so important for the design of structures that must withstand seismic loads.

3.2 Reaction of structures against seismic effects

During an earthquake, the movement of the ground caused by the arrival of seismic waves is transmitted to the building structures through their foundations. The inertia of the masses of buildings opposes this non-uniform motion, thus displaying inertial forces. Seismic actions and the corresponding reactions of structures are treated by the so-called "seismic analysis" or "seismicity calculations". It is noticed that in reinforced concrete construction structures the character of the vertical composition of the seismic action differs from the horizontal one. Ensuring or guaranteeing the necessary resistance of these structures, in order for them to withstand or resist the action of these forces, requires special calculations, as their effect is different from that of vertical static loads. Seismic action has an arbitrary direction in the plane. But, in practice, calculations in the seismicity of structures are made according to two perpendicular directions "x" and "y", which correspond to the axes of greater and lesser rigidity of the object being calculated.

The conception of inertial forces and the requirement to take into account the deformability (or stiffness) characteristics of structures highlight the necessity of including these calculations. Within the framework of Structural Dynamics and evaluating their response as systems with one and many degrees of freedom.

3.2.1 Elastic spectrum and design spectrum

Seismic motion at a given point on the surface is given by the elastic reaction spectrum of the ground acceleration, which in the following we will simply call the "elastic reaction spectrum". The shape of the elastic response spectrum is given the same for both levels of seismic action which refers to non-collapse and damage limitation requirements. Elastic response spectrum and design spectrum for the horizontal component of seismic action:

For the horizontal components of seismic motion the spectrum of the elastic reaction S_e (T) is defined by the following expressions:

$$0 \leq T \leq T_B \quad S_e(T) = a_g * S * \left[1 + \frac{T}{T_B} * (2.5 * \eta - 1)\right]$$

$$T_B \leq T \leq T_C \quad S_e(T) = a_g * S * 2.5 * \eta$$

$$T_c \leq T \leq T_D \quad S_e(T) = a_g * S * 2.5 * \eta * \left[\frac{T_C}{T}\right]$$

$$T_D \leq T \quad S_e(T) = a_g * S * 2.5 * \eta * \left[\frac{T_C * T_D}{T^2}\right]$$

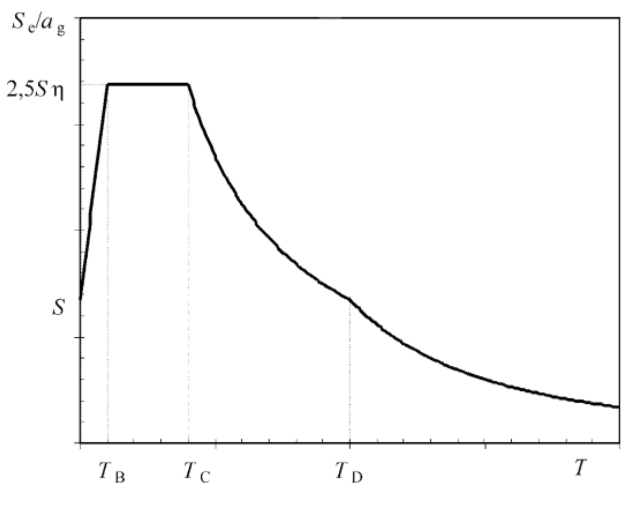


Figure 10 Elastic response spectrum (Pojani N.Seismic Engineering 2003)

3.2.2 Seismic forces in the structure

An earthquake causes the ground to move or shake for a relatively short time (15-30 seconds), although the tremors may continue for minutes. Earthquakes therefore cause earthquakes and consequently the structures located on the ground will move.

Because the ground and the highest level of the building are connected through vertical elements (such as columns or walls), then the effect of the motion reaches up to the roof and causes the generation of a force in it which we call inertial forces in the direction of opposite to ground acceleration.

To summarize, seismic assessments are intended to determine the forces exerted on the structure due to the design earthquake action. The earthquake in the structure is transmitted through the base of the building, ie through the oscillations that are caused on the ground. As a result, inertial forces arise in the structure.

$$F\ (Inertial\ Force) = M\ (mass\ ) * S_{ag}\ (\ acceleration)$$

As it appears from the above formula increasing the mass of the object increases the seismic force for a given earthquake. Although elegant structures or buildings have an advantage in antiseismic design. Also if in the structure will be generated large horizontal forces from seismic actions then we will have an increase in horizontal deformations. The seismic force has its point of application in the center of mass of the floor in the study of the building. The vibration modes of different systems are also called modal forms, where the number of modes is equal to the number of degrees of freedom of the system. The vibration forms have different periods and frequencies.

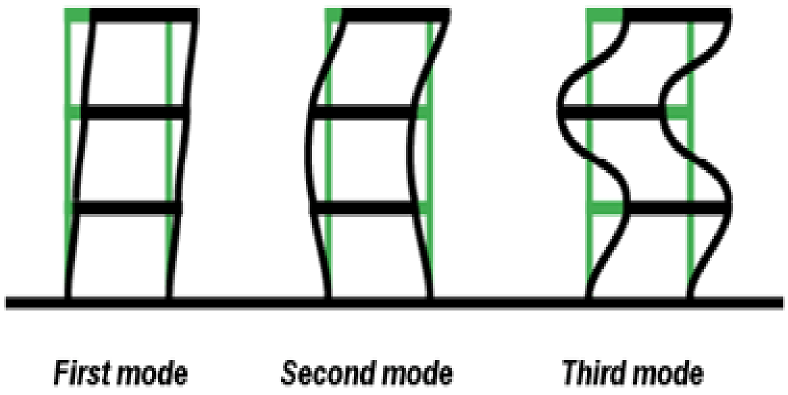


Figure 11 Modal shapes of the structure (Pojani N.Seismic Engineering 2003)

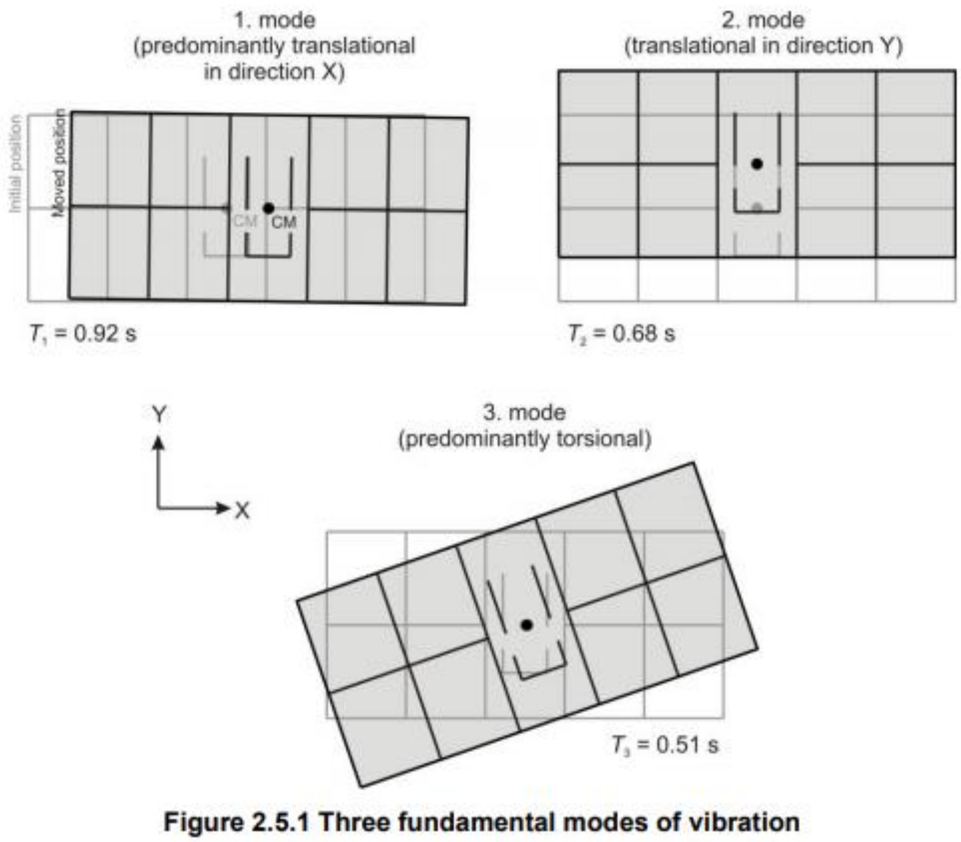


Figure 12 Three foundamental modes of vibration (EUR 25204 EN – 2012 JRC European Comission)

The three foundamental moodes of vibration are translational in the X direction in the first mode, in the second one translational in the Y direction and the third one is torsional. In accordance with EN 1998-1/4.3.3.3(3) the first six modes are sufficient to satisfy dhe requirements.

3.2.3 Design basics

- The basic principles of designing a building.
- Structural simplicity.
- Uniformity, symmetry and static indeterminacy and redistribution ability.
- Same resistance and stiffness in both directions. Structures as symmetrical as possible are preferred.
- Resistance and stiffness to torsion. Align the center of mass with that of stiffness.
- Diaphragmatic behavior of the interstices. The soles should be taken as absolutely tight in its plan.
- Selection of suitable foundations depending on the structure and type of land.

For irregular buildings in height, the behavior factor $q = 0.8q_0$

Wall or wall-equivalent wall system: q multiplied by $(1 + \alpha_0) / 3 \leq 1$

(α_0 dominant ratio for walls = $\Sigma H_i / \Sigma l_{wi}$)

Walls

	DCH	DCM	DCL
Wall length $b_{wo} \geq$	max (150 mm,hs/20)		

Critical area length,h_{cr}	≥max(lw,Hw/6) ≥min (2lw,h_{story}) if wall ≤6 storeys ≥min (2lw,2h_{story}) if wall >6 storeys	

Table Detailing and dimensioning of ductile walls (CEN, 2004)

Where: **Lw** – wall length

Hw - wall height

h_{story} - storey height

	DCH	DCM	DCL
Web thickness, b_{wo}≥	max(150mm, h_{storey}/20)		-
critical region length, h_{cr}≥	≥max(l_w, H_w/6) (1) ≤min(2l_w, h_{storey}) if wall ≤6 storeys ≤min(2l_w, 2h_{storey}) if wall > 6 storeys		-
Boundary elements:			
a) in critical region:			
- length l_c from edge ≥	0.15l_w, 1.5b_w, length over which ε_c> 0.0035		-
- thickness b_w over l_c ≥	200mm; h_{st}/15 if l_c≤max(2b_w, l_w/5), h_{st}/10 if l_c>max(2b_w, l_w/5)		-
- vertical reinforcement:			
ρ_{min} over A_c=$l_c b_w$	0.5%		0.2% (0)
ρ_{max} over A_c	4% (0)		
- confining hoops (w) (2):			
d_{bw}≥	8mm	in the part of the section where ρ_L>2%: as over the rest of the wall (case c, below)	
spacing s_w≤(3)	min(25d_{bh}, 250mm)		
ω_{wd}≥(2)	0.12	0.08	-
$\alpha\omega_{wd}$≥(3),(4)	$30\mu_\phi(\nu_d+\omega_\nu)\varepsilon_{sy,d}b_w/b_o-0.035$		-
b) storey above critical region	As in critical region, but $\alpha\omega_{wd}$ & ω_{wd}: 50% of those required in critical region	as over the rest of the wall (case c, below)	
c) over the rest of the wall height:	In parts of the section where ρ_L>2%: - distance of unrestrained bar in compression zone from nearest restrained bar ≤150mm; - hoops with d_{bw}≥ max(6mm, d_{bL}/4) & spacing s_w≤ min(12d_{bL}, 0.6b_{wo}, 240mm)(0) up to a distance of 4b_w above or below floor beams or slabs, or s_w≤ min(20d_{bL}, b_{wo}, 400mm)(0) beyond that distance		

Table EC8 rules for the detailing and dimensioning of ductile wall (CEN, 2004)

3.3 Response spectrum analysis

3.3.1 Definition of a response spectrum

Typical modal Equation for 3-dimensional seismic motion is as follows:

$$\ddot{y}(t)_n + 2\xi_n\ \omega_n \dot{y}(t)_n + \omega^2\, y(t)_n = \qquad \text{(Equation 1)}$$

$$p_{nx}\ddot{u}(t)_{gx} + p_{nx}\ddot{u}(t)_{gy} + p_{nz}\ddot{u}(t)_{gz}$$

where the three *Mode Participation Factors* are defined by $pni = -\emptyset nT\, \mathbf{M}i$ in which i is equal to x, y or z. In order to get a response spectrum solution 2 aspects have to be taken in consideration :

1. Evaluation of maximum peak forces and displacements for each direction of ground motion.
2. Evaluation of maximum response of earthquake motion acting at the same time. The modal combination will be rewritten as one motion component only.

3.3.2 Response spectrum analysis

Based on Eurocode 8 for a specified ground motion $u(t)_g$, damping value and assuming $p_{ni} = -1.0$ it we are able to solve Equation 1 at various values of ω and plot a curve of the maximum peak response $y(\omega)_{MAX}$. For this acceleration input, the curve is the displacement response spectrum for the earthquake motion. A different curve corresponds to each different value of damping.

A plot of $y(\omega)_{MAX}$ is defined as the pseudo-velocity spectrum and a plot of $\omega^2\ y(\omega)_{MAX}$ is defined as the pseudo-acceleration spectrum. These three curves are normally plotted as one curve on special log paper. These pseudovalues have minimum physical significance and are not an essential part of a response spectrum analysis. Equation 1 is used to calculate the true values for maximum velocity and acceleration.

There is a mathematical correspondance between the pseudo-acceleration spectrum and the total acceleration spectrum. The total acceleration of the unit mass, SDOF systems, governed by Equation 1, is given as follows:

$\ddot{u}(t)_T = \ddot{y}(t) + \ddot{u}(t)_g$ (Equation 2)

Equation can be solved for $\ddot{y}(t)$ and substituted into Equation 2 which yields

$\ddot{u}(t)_T = - \omega^2 y(t) - 2\xi\, \omega\dot{y}(t)$

For the special case of zero damping, the total acceleration of the system is equal to $\omega^2\, y(t)$. For this reason, the displacement response spectrum curve is normally not plotted as modal displacement $y(\omega)_{MAX}$ vs ω. It is standard to present the curve in terms of $S(\omega)$ vs. a period T in seconds where

$S(\omega)_a = \omega^2\, y(\omega)_{max}$ and $T=\frac{2\pi}{\omega}$

The pseudo-acceleration spectrum, $S(\omega)_a$, curve has the units of acceleration vs. period which has some physical significance for zero damping only. It is apparent that all response spectrum curves represent the properties of the earthquake at a specific site and are not a function of the properties of the structural system. After an estimation is made of the linear viscous damping properties of the structure, a specific response spectrum curve is selected.

3.3.3 Storey Drift Calculations

Response spectrum produces displacements that are always positive numbers. Damage to nonstructural items is estimated using inter-story displacements, which cannot be determined directly from likely peak displacement values. Placing a very thin panel element with a shear modulus of unity in the area where the deformation is to be evaluated is a straightforward way to acquire a likely peak value of shear strain. The shear stress peak value will be a good estimate.

3.3.4 Limitations of the Response spectrum method

It is clear that using the response spectrum method has drawbacks, some of which can be overcome with more research. However, it will never be accurate for nonlinear analysis of structures with multiple degrees of freedom. The author predicts that more time historical dynamic response analyses will be performed in the future, and that the various approximations related with the usage of the response spectral will be eliminated.

3.4 Time-History representation

3.4.1 General

- The seismic motion can also be represented with the time history that takes into consideration the velocity and corresponding displacement as input data.

- Artificial accelerograms, recorded or simulated accelerograms, and artificial accelerograms may be used to describe seismic motion depending on the nature of the application and the information really available.

3.4.2 Artificial accelerograms

- Artificial accelerograms should match the elastic response spectrum with a 5% viscous damping (ξ = 5%).
- The duration of the accelerograms must be consistent with the magnitude and other key characteristics of the seismic event that caused ag to form.

- When site-specific data is unavailable, the stationary section of the accelerograms should have a minimum duration Ts of 10 seconds.
-The suite of artificial accelerograms sadisfy the following directions:

1. the minimum accelerograms that should be used is three;

2. The value of ag shall not be less than the mean of the zero period spectral response acceleration values (derived from the individual time histories).
3. In the range of periods between 0,2T1 and 2T1, where T1 is the fundamental period of the structure in the direction where the accelerogram will be applied, no value of the mean 5% damping elastic spectrum calculated from all time histories should be less than 90% of the corresponding value of the 5% damping elastic response spectrum in the range of periods between 0,2T1 and 2T1.3.4.3. Recorded or simulated accelerograms
4. As long as the samples are adequately qualified in terms of the seismogenetic features of the sources and the soil conditions appropriate to the site, and their values are scaled to the value of a g.S for the zone under consideration, recorded accelerograms or accelerograms generated through a physical simulation of source and travel path mechanisms can be used.
5. Soil amplification analyses and dynamic slope stability verifications are specified in EN 1998-5:2004, 2.2.

3.4.3 Modal response spectrum analysis

- This sort of analysis should be used on structures that do not meet the requirements for using the lateral force technique of analysis;

- It is necessary to consider the reaction of all modes of vibration that contribute significantly to the overall response;

- the overall mass of the structure is at least 90% of the sum of the effective modal masses for the modes taken into account;
- Every mode having an effective modal mass greater than 5% of the total mass is considered;
- The preceding conditions should be validated for each relevant direction when utilizing a spatial model.

3.4.4 Combination of modal responses

(1) The response in two vibration modes i and j (including both translational and torsional modes) may be taken as independent of each other, if their periods Ti and T_j satisfy (with $T_j \leq T_i$) the following condition:

$Tj \leq 0{,}9 * T_i$

- When all relevant modal responses are considered independent of one another, the maximum EE of a seismic action effect can be calculated as follows:

$EE = \sqrt{\Sigma\, E\text{Ei}}2$

where

E is the seismic action effect under consideration (force, displacement, etc.)

EEi is the value of this seismic action effect due to the vibration mode i.

3.4.5 Displacement calculation

If a linear analysis is undertaken, the displacements caused by the design seismic action must be estimated using the following simplified statement based on the structural system's elastic deformations:

$ds = q_d * d_e$

where

-ds is the displacement of a point of the structural system induced by the design seismic action;

-qd is the displacement behaviour factor, assumed equal to q unless otherwise specified;

-de is the displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum

When determining the displacements de, the torsional effects of the seismic action shall be taken into account.

3.5 METHODOLOGY

3.5.1 PLANING OF STUDY

The purpose of this study is to design a multistorey building based on the results taken by ETABS modeling using the response spectrum analysis and time history analysis. The Durres earthquake (26/11/2019) records are taken into consideration when completing the time history analysis. A case study approach is chosen for modeling and analyzing.

This study’s stages are as listed below in order to take e fuller view of this study:

Phase 1: Project objectives and scopes of study evaluation

Phase 2: Data collecting and structure modeling

Phase 3: Structure modeling and structure response spectrum vs time history analysis.

Phase 4: Discussion and Conclusion

3.5.2 GATHERING INFORMATION AND DATA

For this study I have collected data for a building that will be soon constructed in Tirana. The data collected are as follow:

i) Case study’s building background information and data
ii) Location where the building will be constructed
iii) Configuration of the building
iv) Materials used for building construction and their properties

3.5.3 BUILDING INFORMATION

The building that I have considered for modeling and designing is a 10-storey building with 2 levels underground in Tirana. The two levels underground are used as a parking garage and other spaces are residential ones. The height of the building is 34.17 m.

This case's study building will be constructed in "Stacioni I Trenit" Tirane.

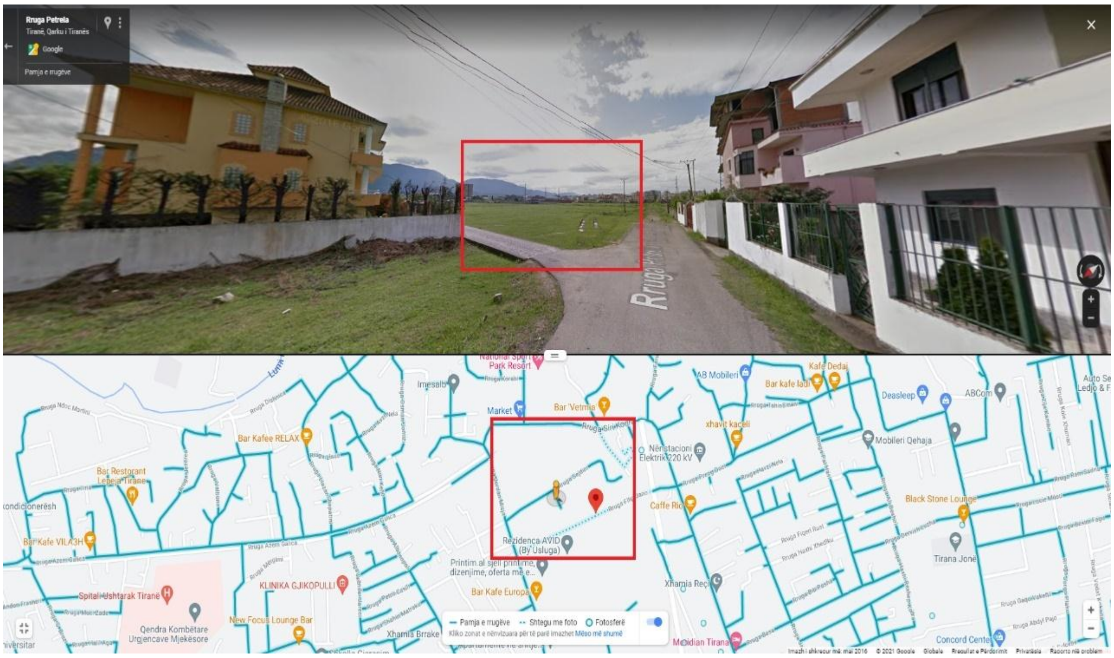


Figure 13 Loaction of the case study building

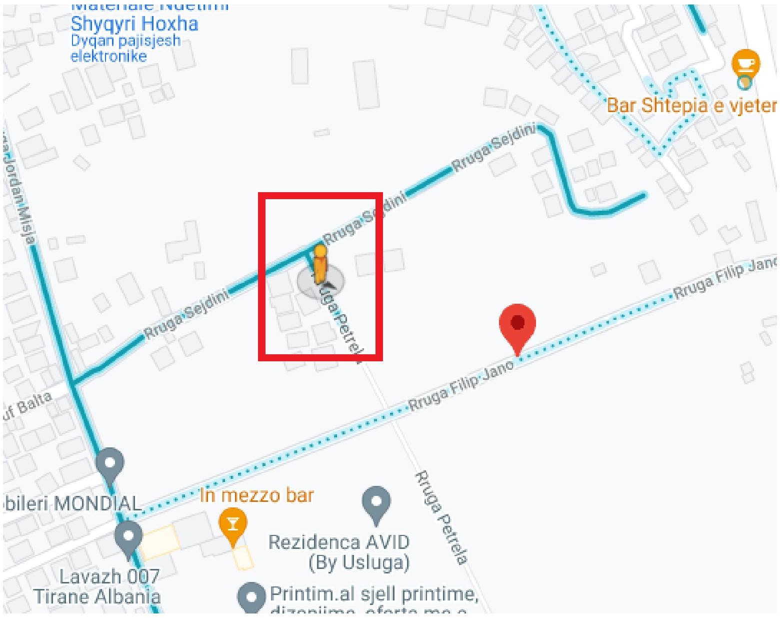


Figure 14 Loaction of the case study building

Figure 15 Figure 16 Loaction of the case study building

Based on the territory where the building is going to be constructed I have taken the data of that zone to correspond in the Response spectrum input data for the ETABS modelling and analysis.

3.5.4 ETABS MODELING

Etabs is a three-dimensional analysis and design program that has been used by structural engineers for nearly thirty years. It was developed by CSI (Computers and structures, Inc). Modeling tools and templates, code-based load prescriptions, analysis methods and solution techniques, all coordinate with the grid-like geometry unique to this class of structure.

Etabs software is used for multiple purposes such as:

1. Analysis of Skyscrapers steel & concrete structures
2. 3D modeling
3. Realistic modeling and distribution of loads based onvarious codes (ex. Eurocode, CSA, ACI etc.)

Benefits of Etabs software:

1. ETABS gives the user the chance of easy modification of the structure in a easy nd quick way;
2. 3D modeling with plan views and elevations;
3. Similar storey command that allows you to easy work with multistorey buildings;
4. Easy editing, modifying and navigation of the structure;
5. Different element modeling such as beams, columns, slabs and walls;
6. Create your model and editing has been easy through 3D view with different type of zoom option as well as panning command for moving the whole model easily without any rotation.
7. Automatic consideration of self-weight of material
8. Automatically creates Earthquake and Wind load also load combinations are easily automatically defined by building code
9. Interaction with various programs (ex. CSI Detailing, SAP, SAFE, CSI Revit etc) that makes the building design process easier and less time consuming.

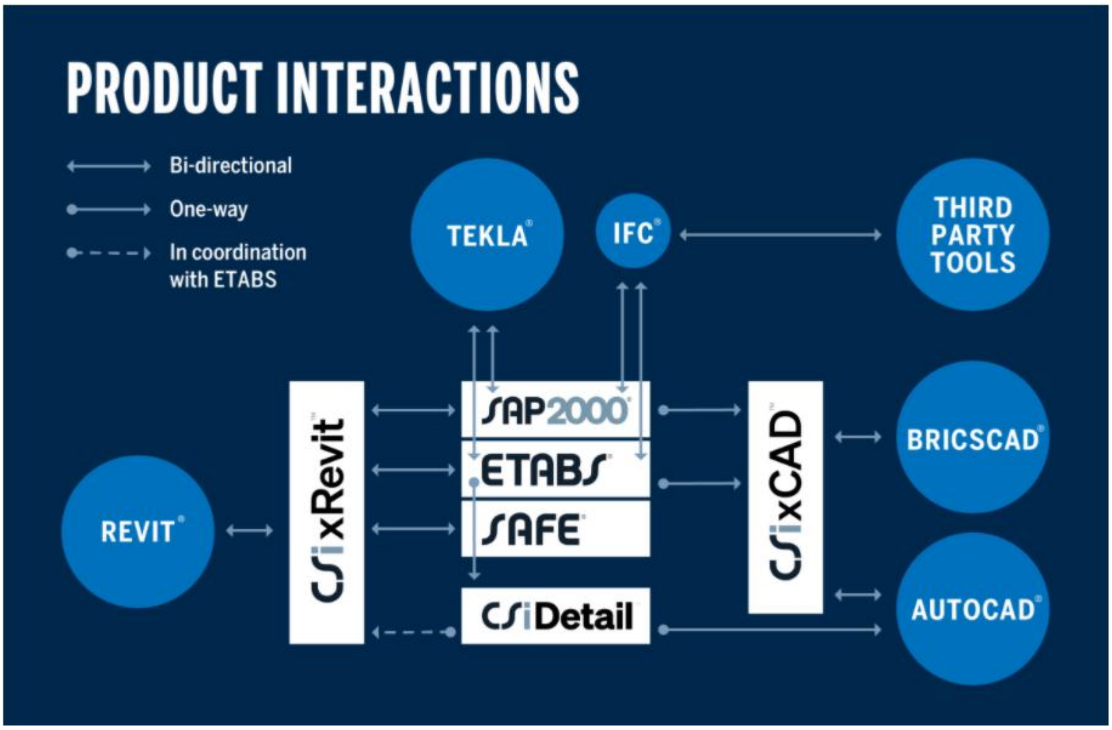


Figure 17 ETABS Iinteraction wth other programs (https://www.csiamerica.com/)

Modeling of this multistorey building is done using ETABS. The building's center of rigidity is assumed to match at the same point as the center of gravity.

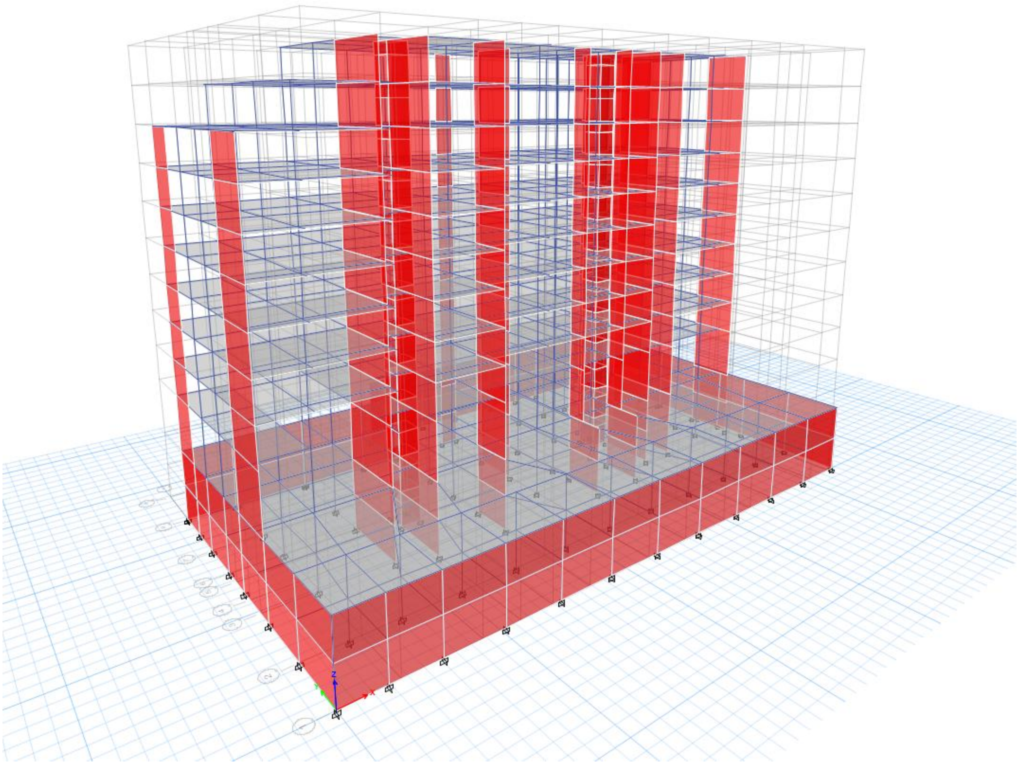


Figure 18 Multistorey building case study ETABS modeling

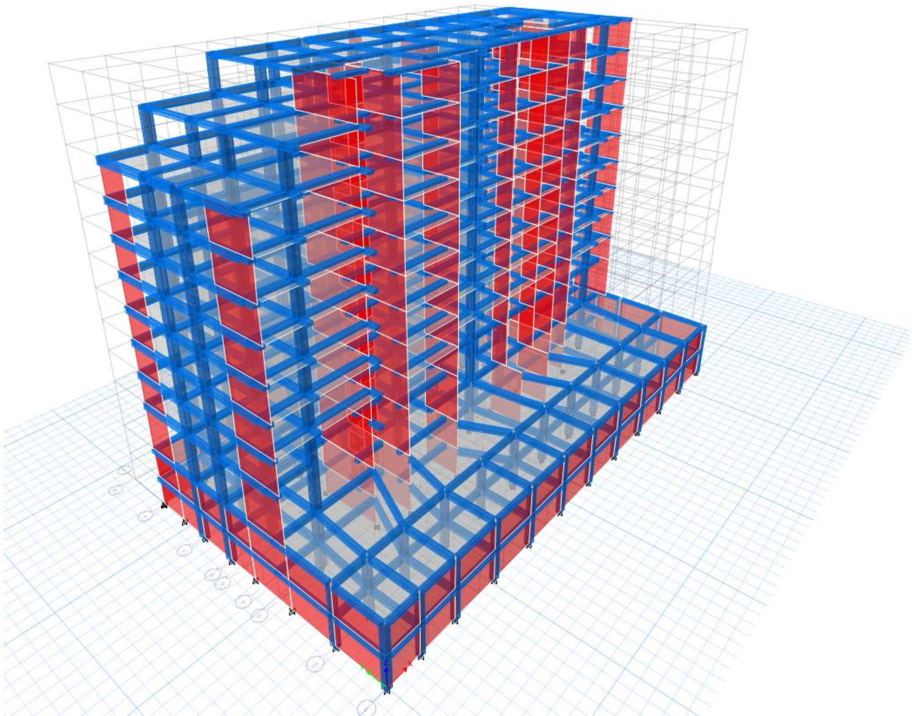


Figure 19 Multistorey building case study ETABS modeling

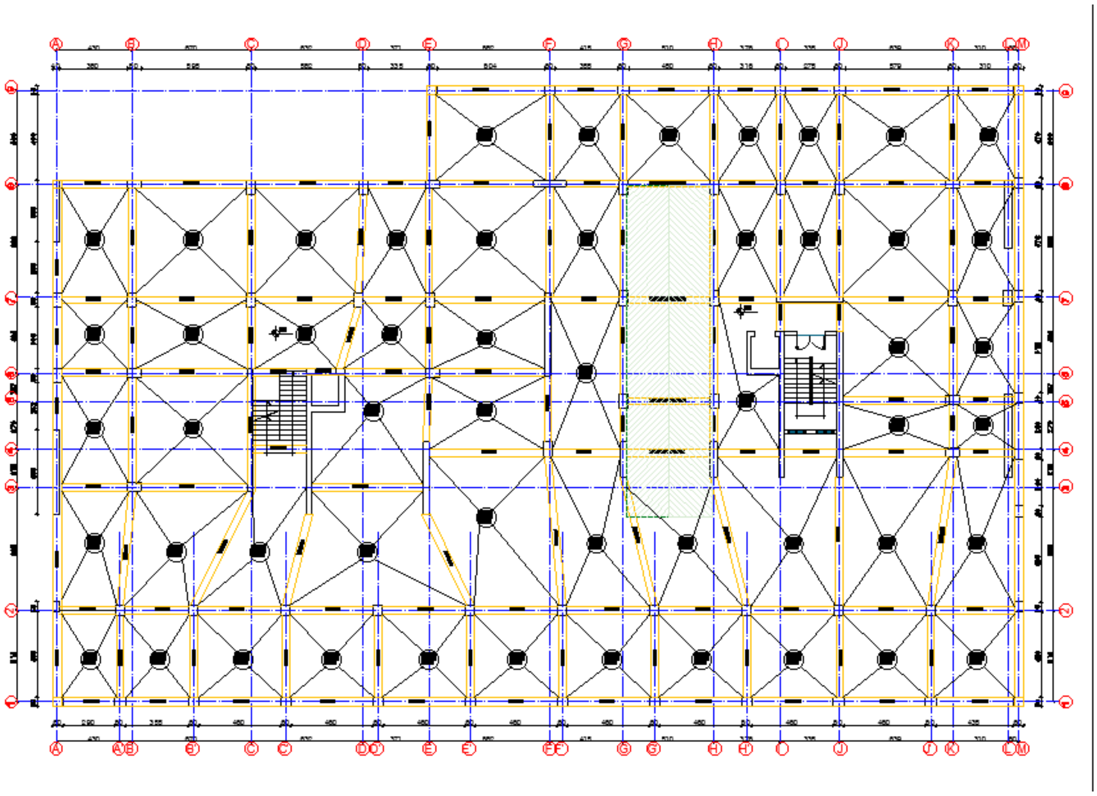


Figure 20 Multistorey building case study structural plan (Base)

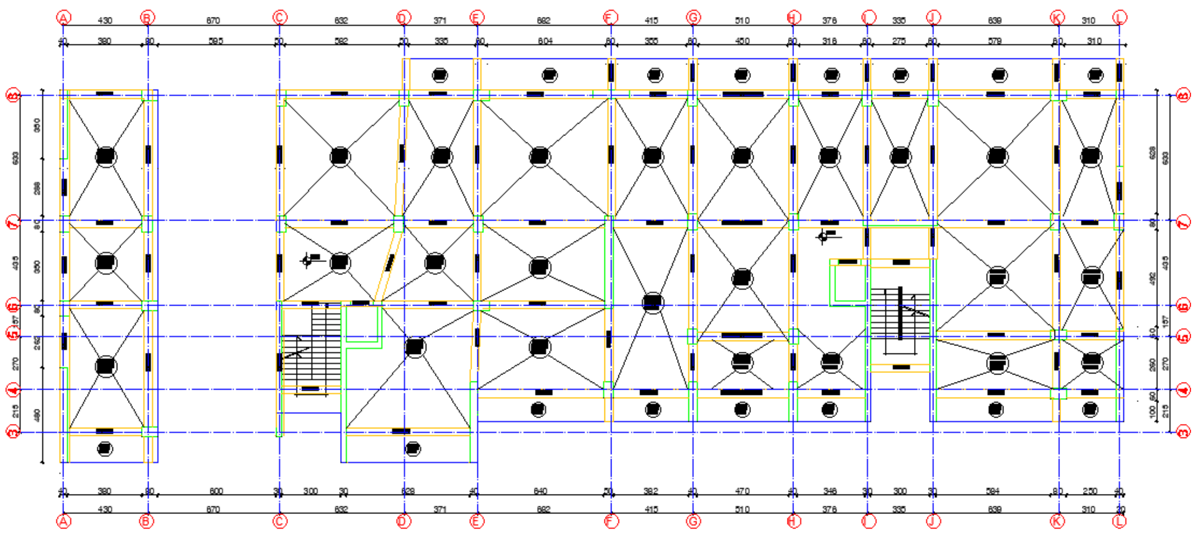


Figure 21 Multistorey building case study structural plan (Typical floor)

3.5.5 SUMMARY OF METHODOLOGY

In order to design the multistorey building based on the seismic response of the structure ETABS modeling is done. The response spectrum and time history analysis is done and a comparison of these results as well to understand the seismic performance of the case study. For the Time-history analysis 3 earthquake records are taken into consideration: Durres earthquake, El Centro and Santa Monica Earthquake. For the concrete frame element design the CSI Detailing software is used.

CHAPTER 4

RESULTS AND ANALYSIS

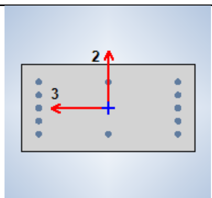
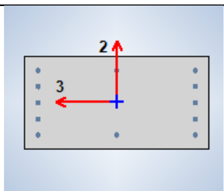
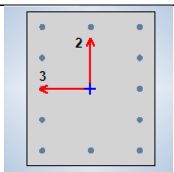
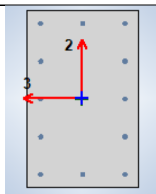
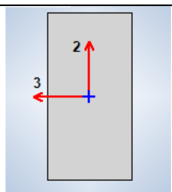
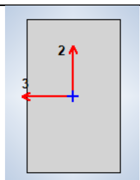
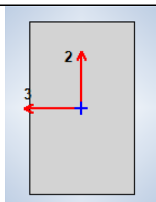
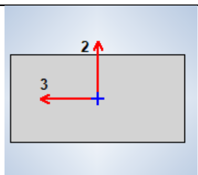
4.1 INTRODUCTION

This chapter is based in seismic behaviour of the multistorey building aspect using response spectrum analysis and time history analysis. A 10-storey building with 2 underground levels has been selected as a case study to design and analyze.

4.2 SECTION PROPERTIES

The following tables show the section properties of all structure's members.

No.	Frame Element	Section	Material	Area of section

1.	**Column 30x60** Depth=0.3m Width=0.6m		Concrete C0/37	0.18 m^2
2.	**Column 40x80** Depth=0.4m Width=0.8m		Concrete C0/37	0.32 m^2
3.	**Column 50x60** Depth=0.6m Width=0.5m		Concrete C0/37	0.3 m^2
4.	**Column 50x80** Depth=0.8m Width=0.5m		Concrete C0/37	0.4 m^2
5.	**Beam 30x60** Depth= Width=		Concrete C0/37	0.18 m^2
6.	**Beam 30x50** Depth=0.5m Width=0.3m		Concrete C0/37	0.15 m^2
7.	**Beam 40x60** Depth=0.6m Width=0.4m		Concrete C0/37	0.24 m^2
8.	**Beam 60x30** Depth=0.3m Width=0.6m		Concrete C0/37	0.18 m^2

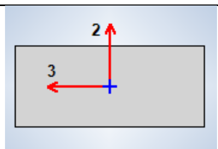
9.	**Beam 70x30** Depth=0.3m Width=0.7m		Concrete C0/37	0.21 m^2

Table Section properties of all structure's members of the case study building

No.	Element	Thicknes
1.	Slab	0.2m
2.	Ribbed Slab	0.5m

Table Slab properties

No.	Element	Thicknes	Color
1.	Wall	30 cm	
2.	Wall	30 cm	
3.	Wall	40 cm	
4.	Wall	50 cm	
5.	Wall	50 cm	

Table Wall properties different color representation

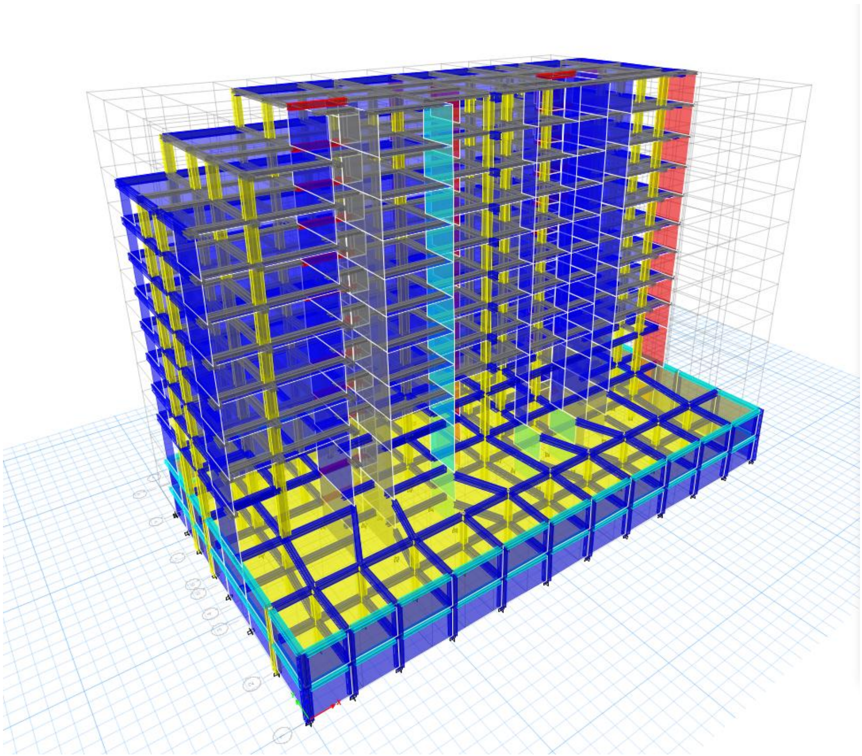


Figure 22 Wall properties different color representation ETABS 2D View

4.3 LOAD COMBINATIONS

Load combinations are put in the ETABS modeling as follow :

1. 1.35D
2. 1.35D+1.5L
3. 1D+0.3L+1E_X
4. 1D+0.3L+1E_Y
5. 1D+0.3L-1E_X
6. 1D+0.3L-1E_Y

According to EN 1990/6.4.3.4 the load combination of gravity and seismic loads have to be taken into consideration.

$1.0*G + \psi_{2i}*Q \pm E_{XY}(\pm M_a)$

where **G-** is the permanent gravity load such as self weight and additional dead loads **Q** is live load that can be variable or imposed load, which is reduced with factor ψ_{2i} =

0.3 (EN 1990/Table A.1.1, office building), and E_{XY} is the combined seismic action for both directions obtained by modal response spectrum analysis with included torsional effects (± Ma)

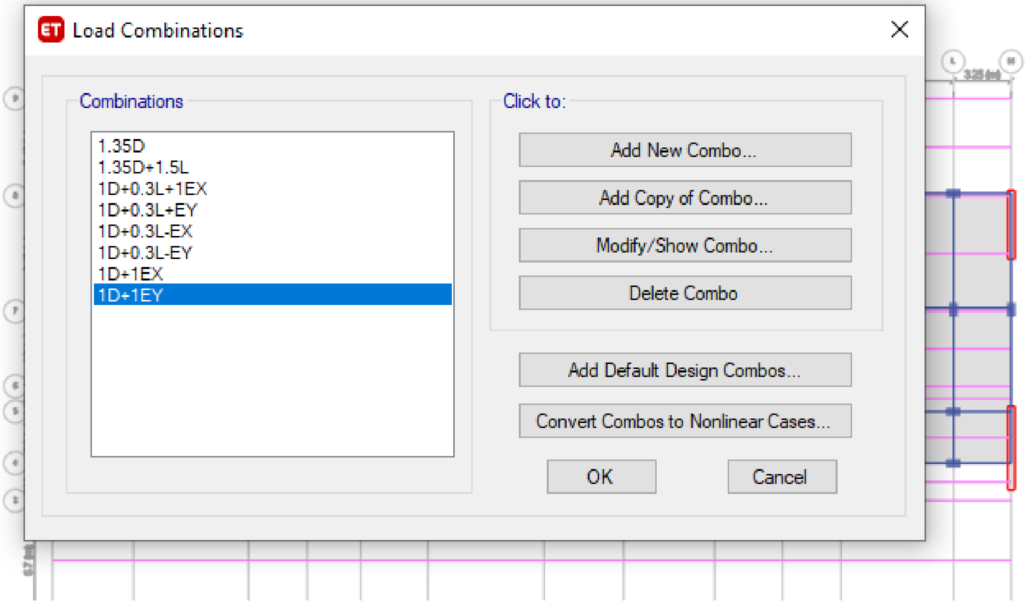


Figure 23 Load combinations used for the case study

4.4 RESPONSE SPECTRUM ANALYSIS

Response spectrum analysis is made to gain a view of the structural behaviour for dynamc impact. The natural periods and mode are the main factors to determine the dynamic characteristics of the structure.

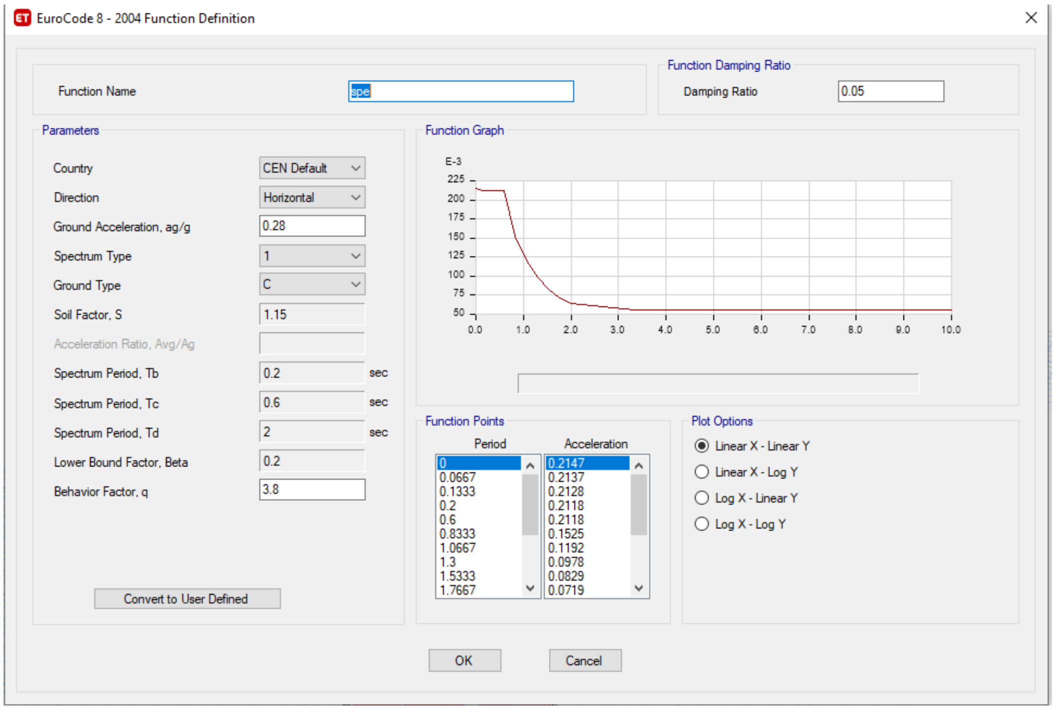


Figure 24 Response spectrum function ETABS input with case study data input

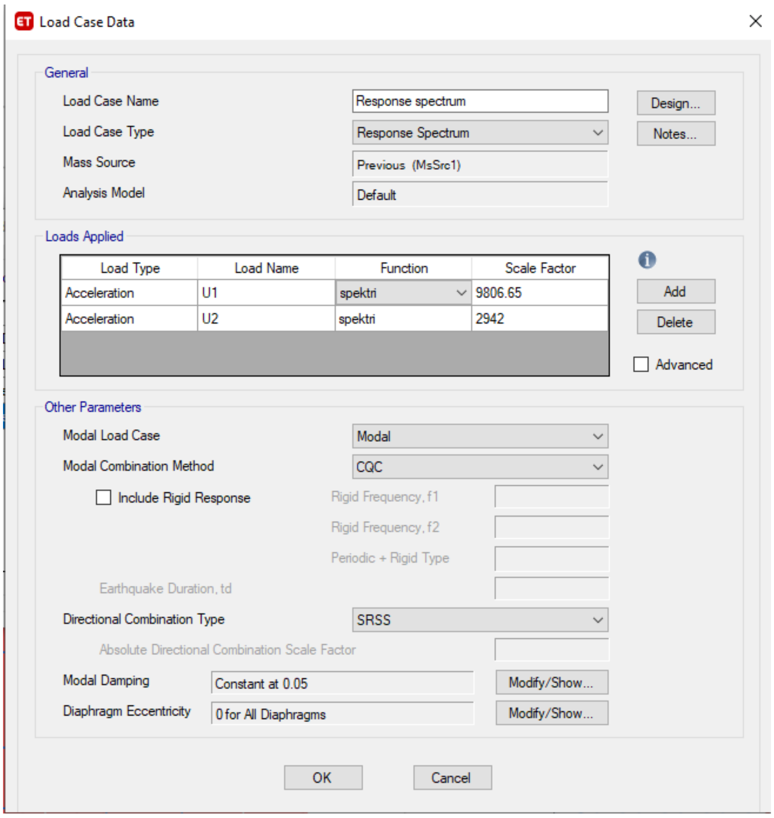


Figure 25 Response spectrum defined as a load case

Mode	Period	Structure movement
1	0.9251	X displacement
2	0.5714	Y displacement
3	0.4966	Torsion
4	0.2703	
5	0.1360	
6	0.1348	

Table Period values for each mode output from response spectrum analysis on ETABS

4.4.1 Mode deformed shapes and periods

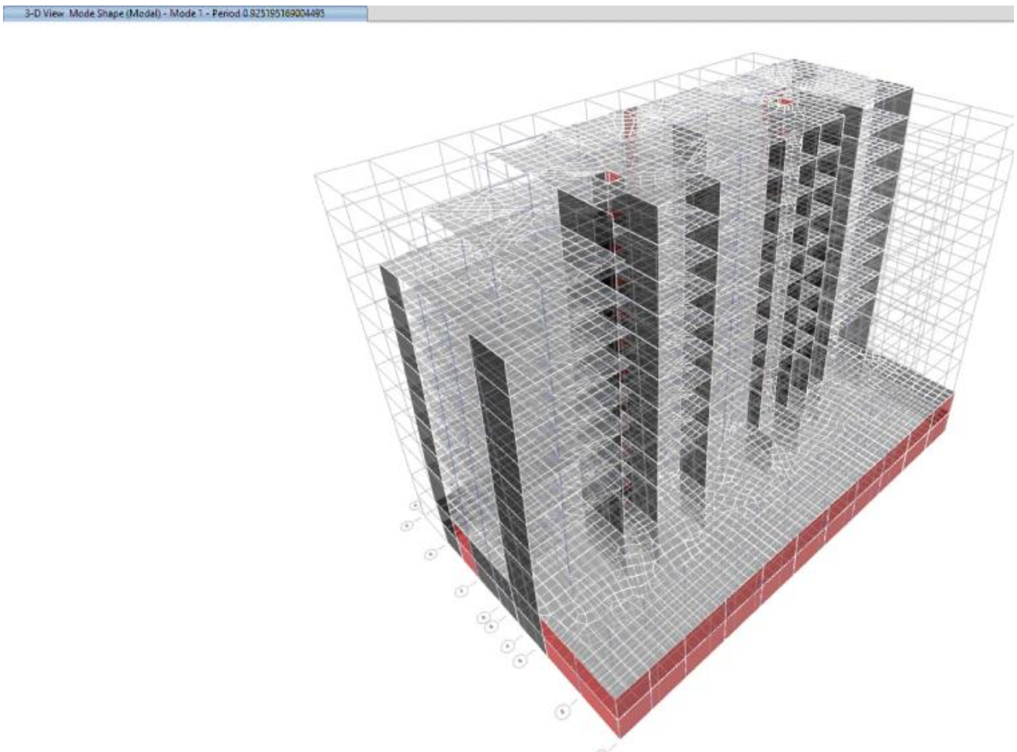


Figure 26 1st mode deformed shape and period obtained from RSA

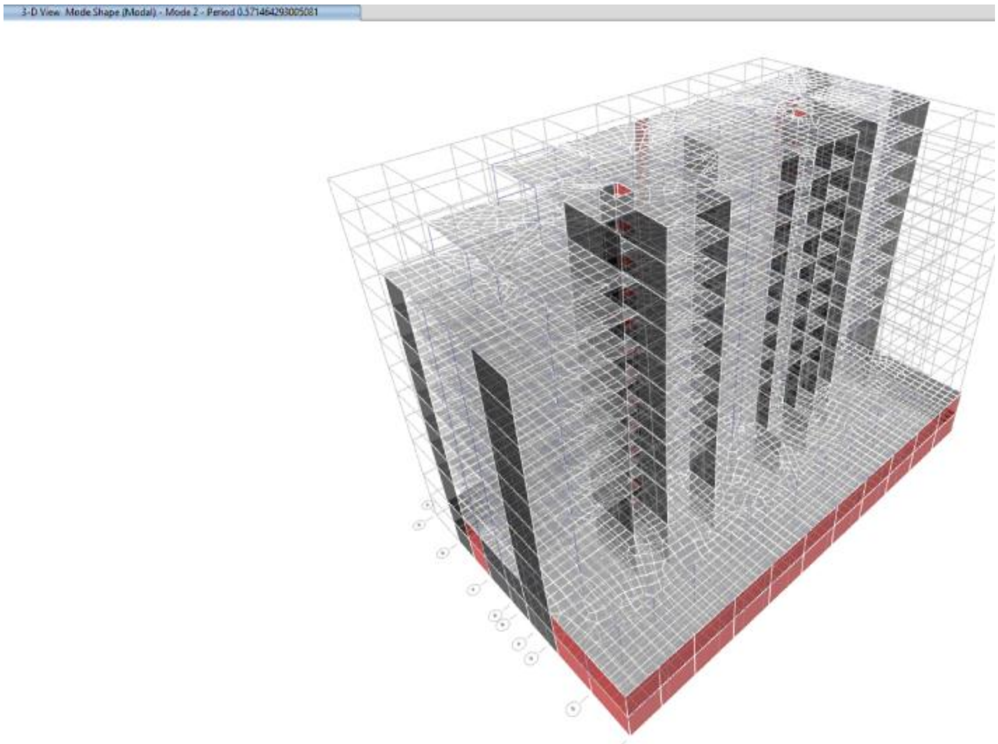


Figure 27 2nd mode deformed shape and period obtained from RSA

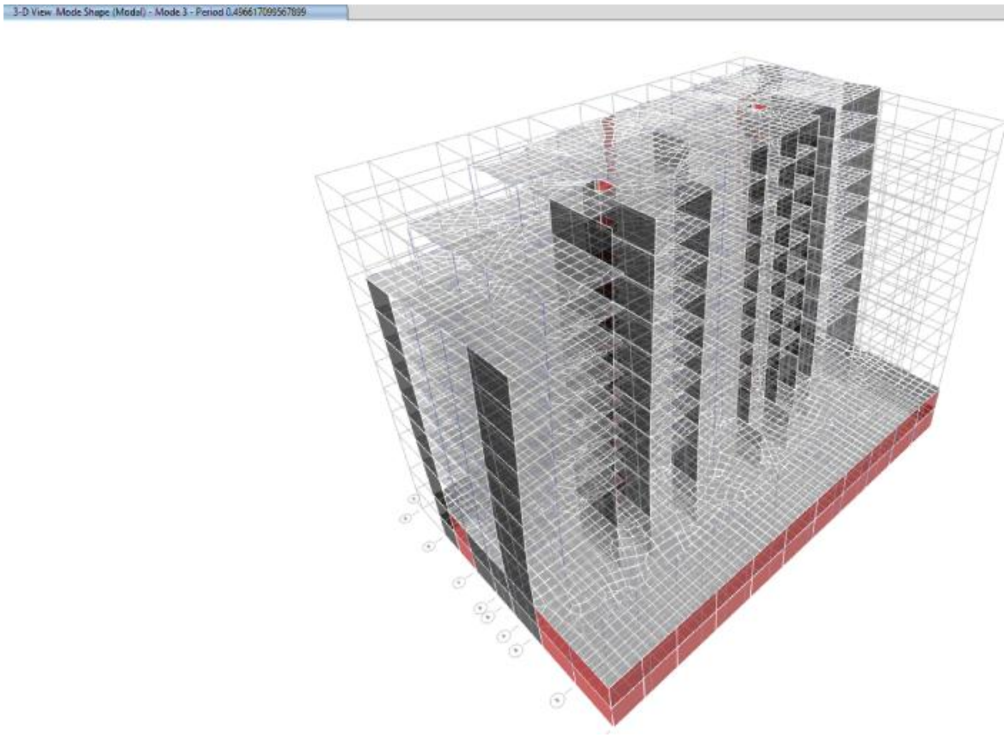


Figure 28 3rd mode deformed shape and period obtained from RSA

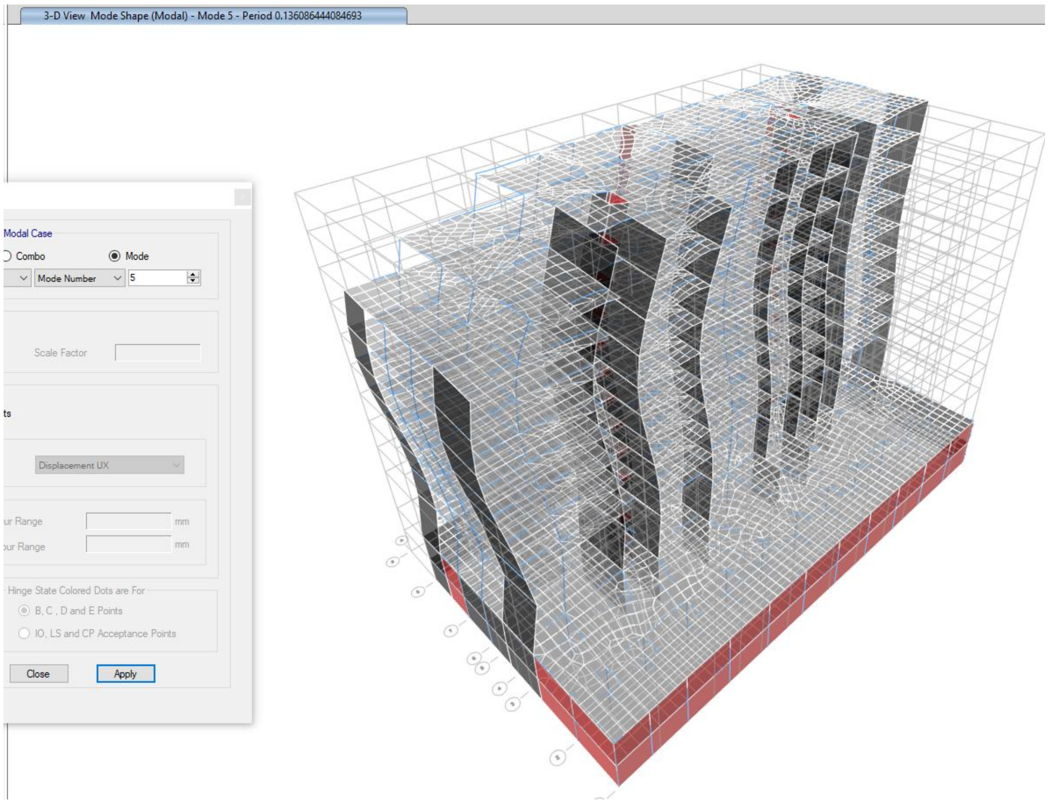


Figure 29 4th mode deformed shape and period obtained from RSA

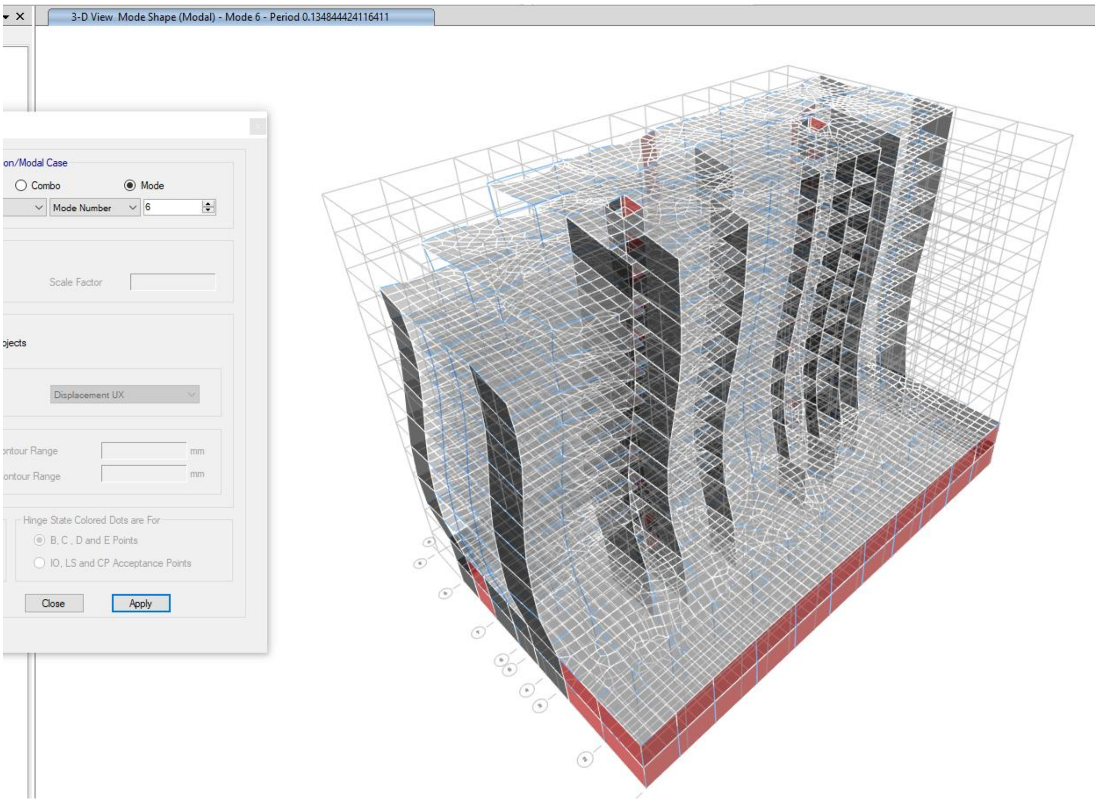


Figure 30 6th mode deformed shape and period obtained from RSA

According to EC8 for determination of the fundamental period the following expression may be used:

$T_1 = C_t\, H^{3/4}$

where

Ct is 0,085 for moment resistant space steel frames, 0,075 for moment resistant space concrete frames and for eccentrically braced steel frames and 0,050 for all other structures;

H is the height of the building, in m, from the foundation or from the top of a rigid basement.

For this case study : Where H=34.17, Ct=0.050

$T1 = 0.075 * (34.17)^{3/4} = 1.05$

4.4.2 Maximum story displacement

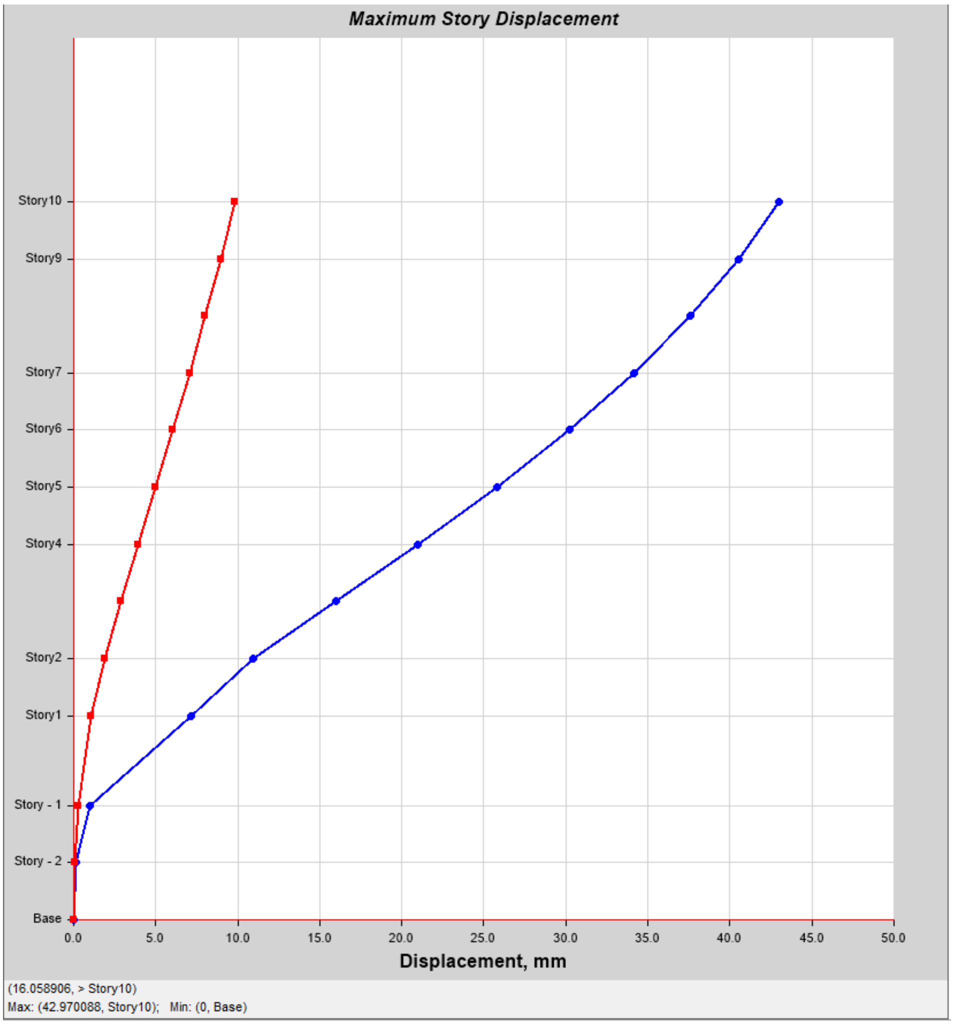


Figure 31 Maximum story displacement obtained from RSA

4.4.3 Story shears

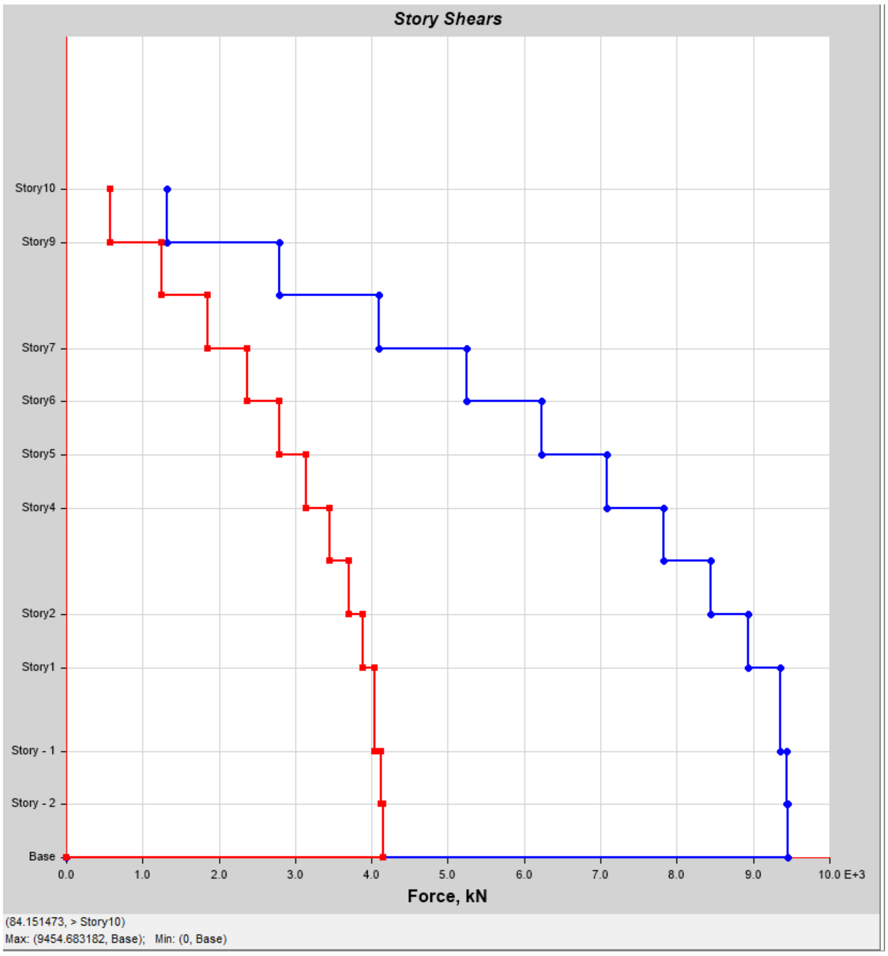


Figure 32 Story shears obtained from RSA

4.4.4 Story overturning moments

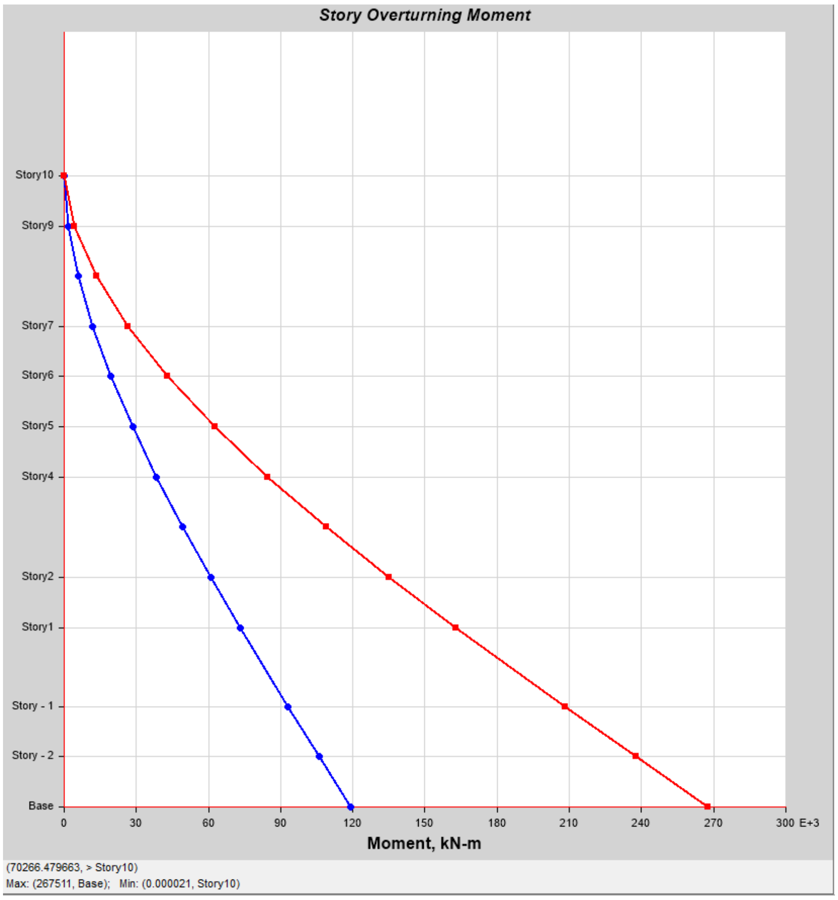


Figure 33 Story overturning moment obtained from RSA

4.5 TIME HISTORY ANALYSIS

In order to make a full time-history analysis I have considered not only Durres Earthquake data but also 2 other earthquakes, El Centro Earthquake that occurred in 1940 and also Santa Monica.

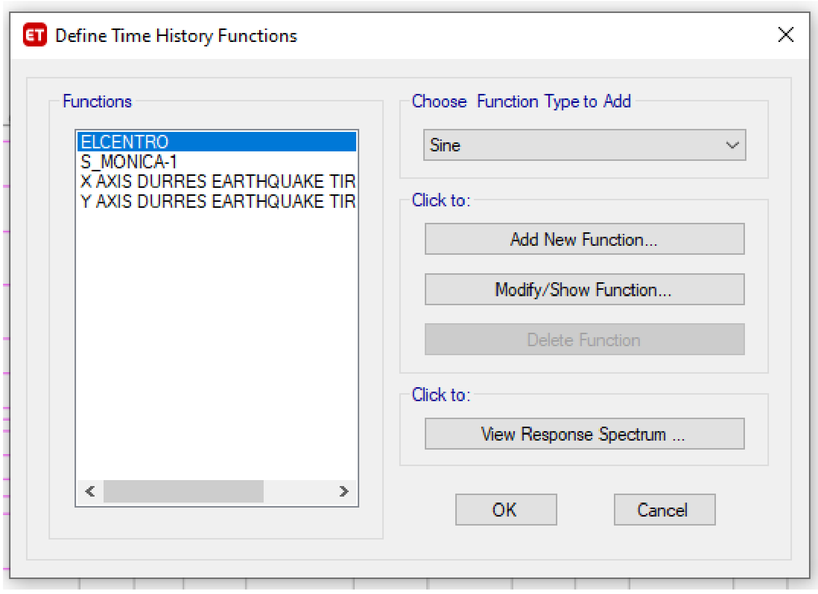
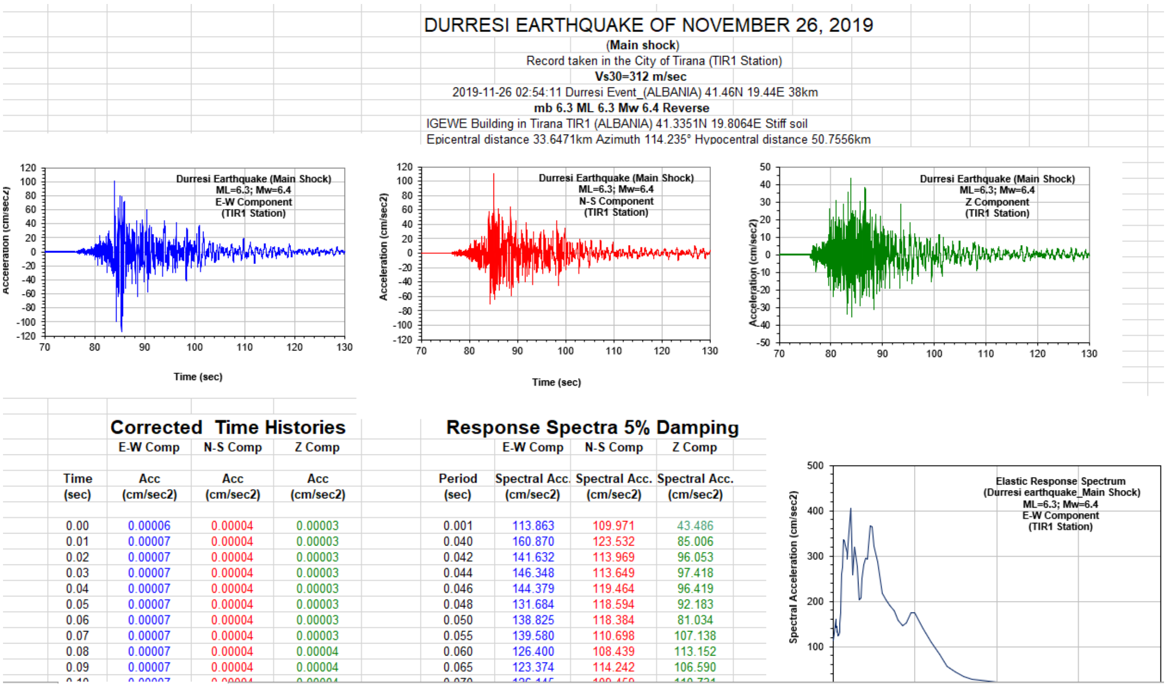


Figure 34 3 earthquakes considered for the time history analysis

Records of the Durres Earthquake are taken from Tirana station and the records were taken from the IGEWE (Institute of Geosciences, Energy, Water and Environment).



Time (sec)	E-W Comp Acc (cm/sec2)	N-S Comp Acc (cm/sec2)	Z Comp Acc (cm/sec2)	Period (sec)	E-W Comp Spectral Acc. (cm/sec2)	N-S Comp Spectral Acc. (cm/sec2)	Z Comp Spectral Acc. (cm/sec2)
0.00	0.00006	0.00004	0.00003	0.001	113.863	109.971	43.486
0.01	0.00007	0.00004	0.00003	0.040	160.870	123.532	85.006
0.02	0.00007	0.00004	0.00003	0.042	141.632	113.969	96.053
0.03	0.00007	0.00004	0.00003	0.044	146.348	113.649	97.418
0.04	0.00007	0.00004	0.00003	0.046	144.379	119.464	96.419
0.05	0.00007	0.00004	0.00003	0.048	131.684	118.594	92.183
0.06	0.00007	0.00004	0.00003	0.050	138.825	118.384	81.034
0.07	0.00007	0.00004	0.00003	0.055	139.580	110.698	107.138
0.08	0.00007	0.00004	0.00004	0.060	126.400	108.439	113.152
0.09	0.00007	0.00004	0.00004	0.065	123.374	114.242	106.590

Figure 35 Durres Earthquake records from Tirana station (https://www.geo.edu.al/)

Records are taken for E-W, N-S and Z components in a time period 291.81 seconds, from the exel file time and acceleration values are saved in .txt form in order to use the data for the ETABS time-history analysis. For this case study are used only N-S and E-W components since the Z component has not that big of an impact in the overall analysis of the building. 7500 header lines are skipped since the acceleration values that correspond to time are very small and almost in linear form.

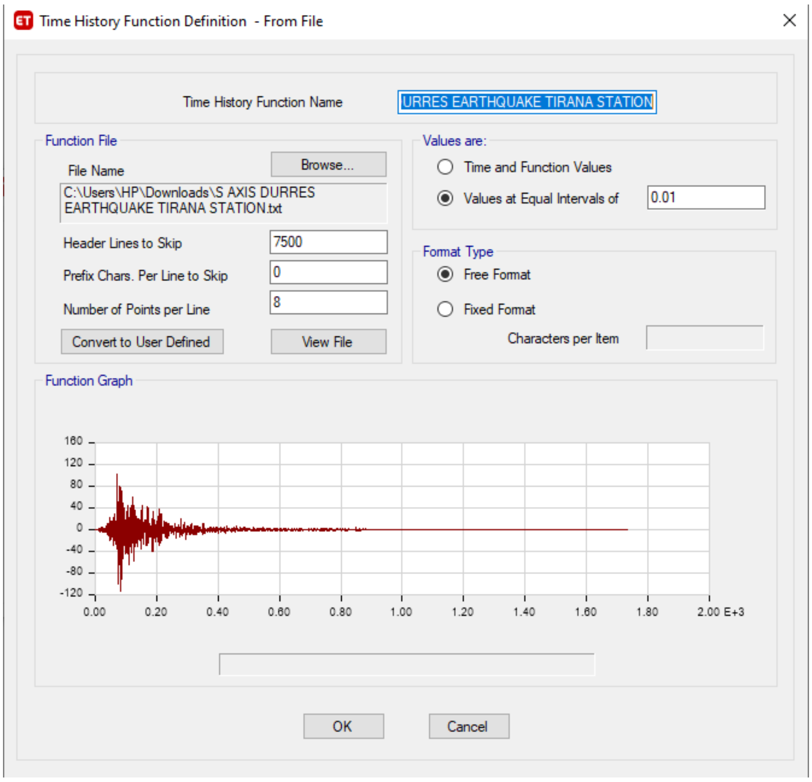


Figure 36 X component of Durres earthquake (Tirana station)

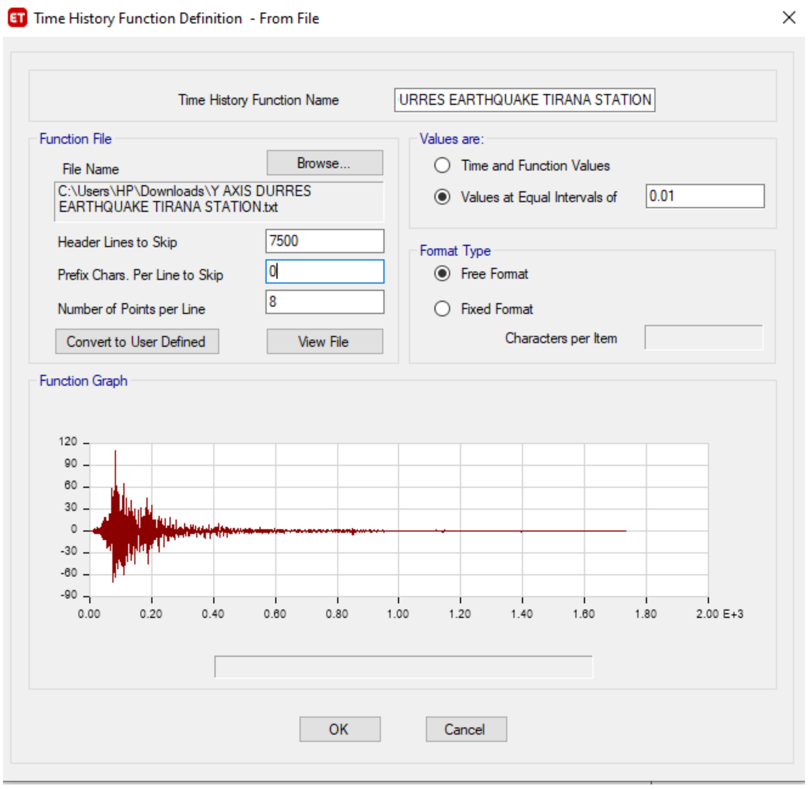


Figure 37 Y components of Durres Earthquake (Tirana Station)

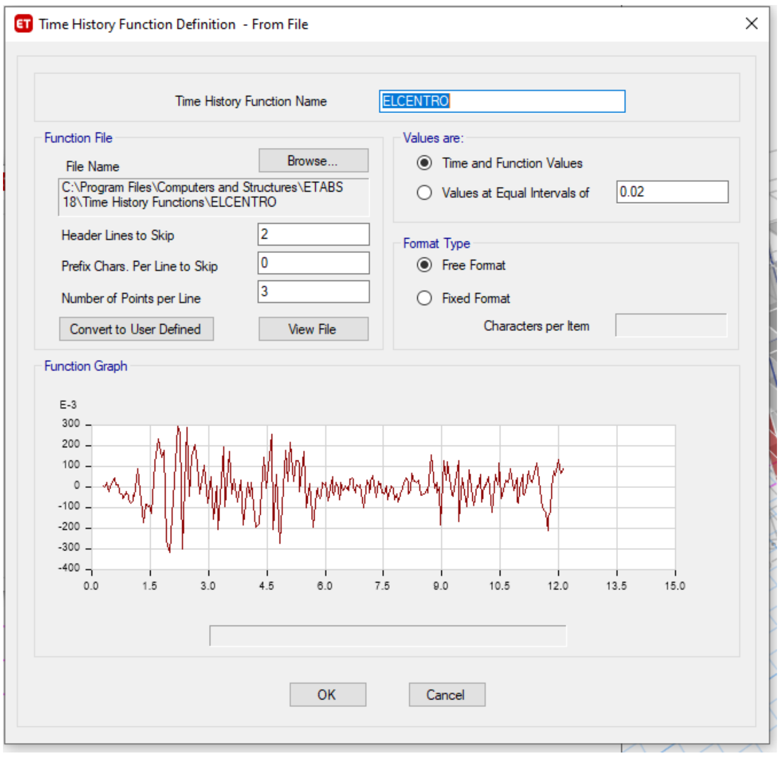


Figure 38 El centro earthquake

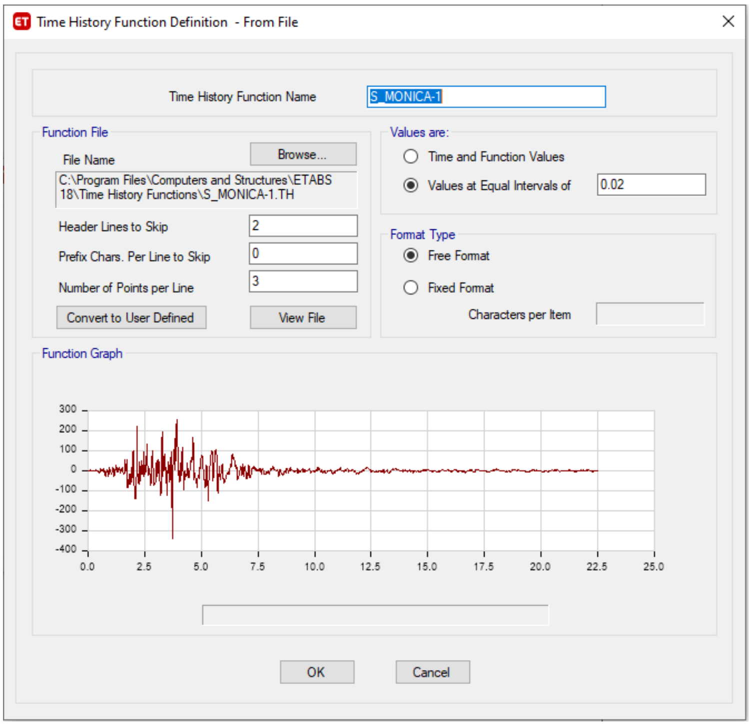


Figure 39 Santa Monica Earthquake

After putting all 3 earthquakes data in th time-history function, I have defined the time history as a load case.

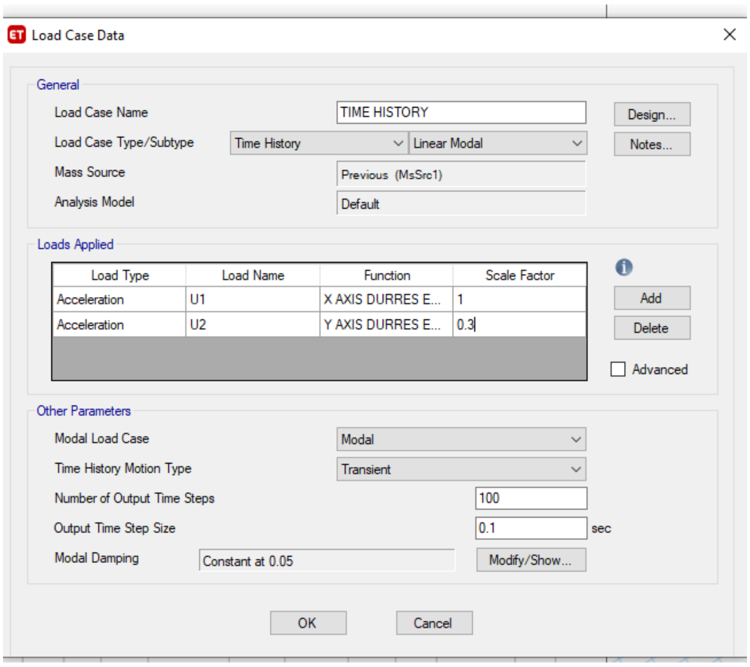


Figure 40 Time history as a load case definition

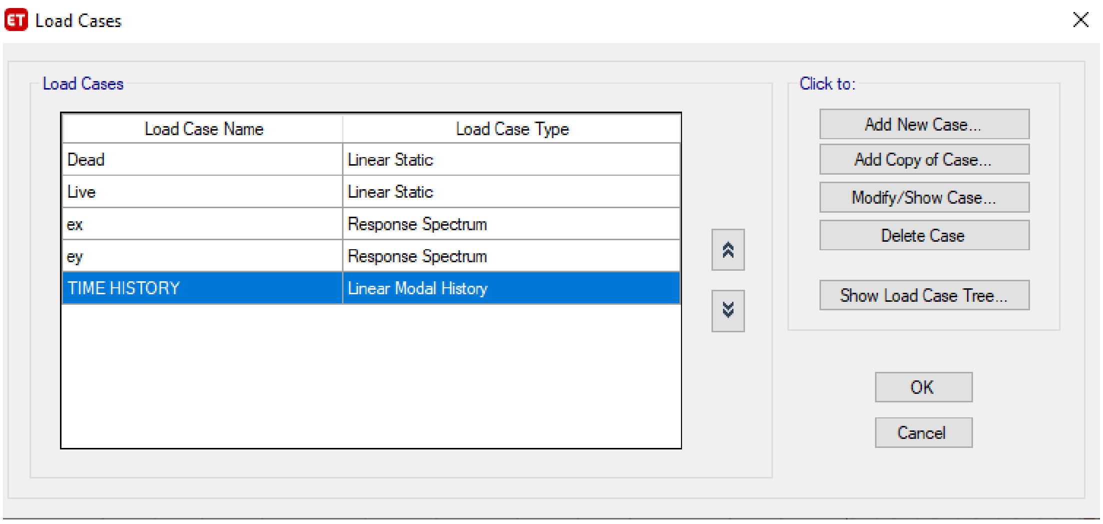


Figure 41 Time history defined as a load case

As shown in the picture above after defining the time history as a load case it will appear on the load cases tab among other load cases such as dead load, live and earthquake load.

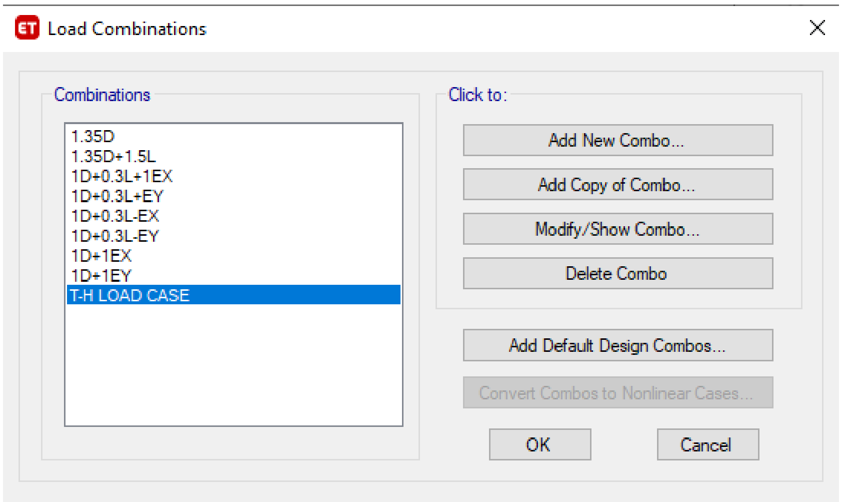


Figure 42 Load combinations for time-history analysis

For the time-history analysis the same load combinations as in the response spectrum analysis are used, with the time-history load case addition.

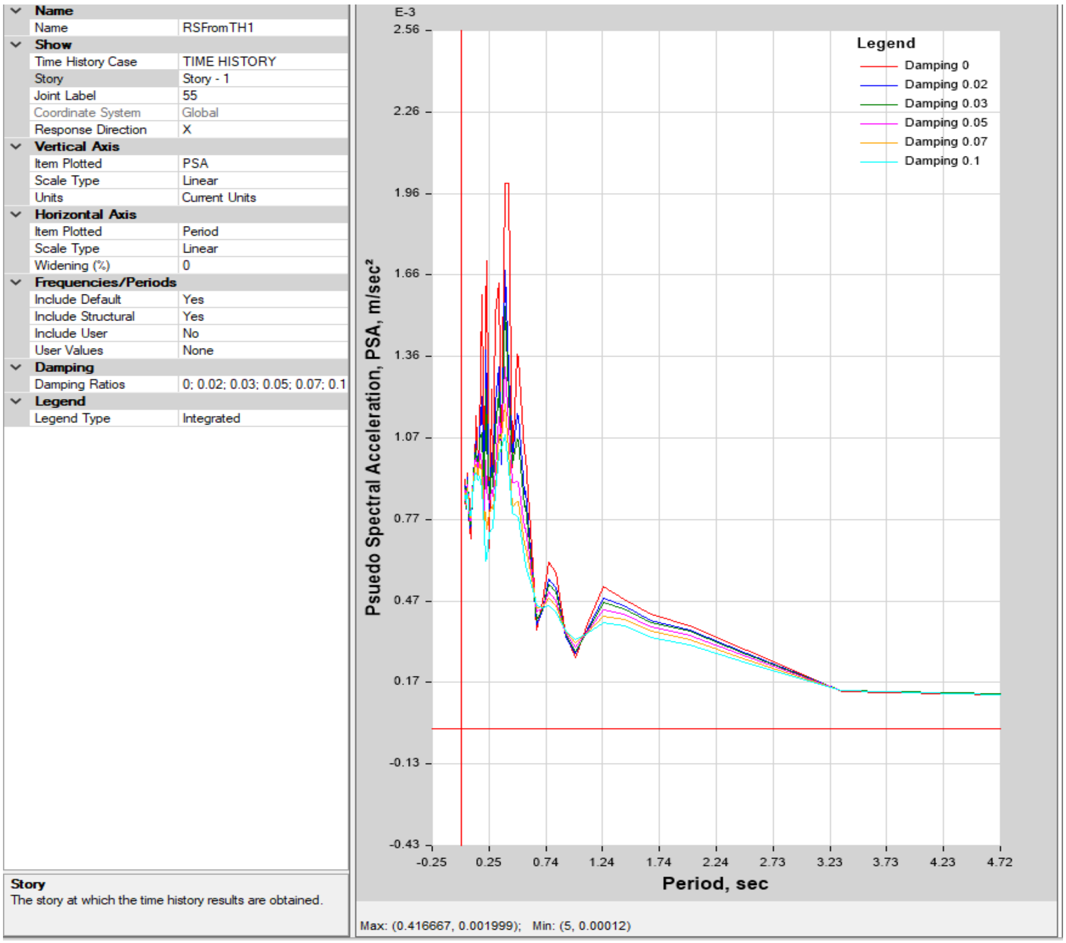


Figure 43 Time history response spectrum for story -1, Joint 55

4.5.1 Deformed hape for 6 modes and corresponding periods

Deformed shapes for 6 modes and periods are shown in the table below:

Mode	Period	Structure movement
1	1.0289	X displacement
2	0.6381	Y displacement
3	0.5497	Torsion
4	0.2703	
5	0.1360	
6	0.1348	

Table 7 6 mode deformed shape periods

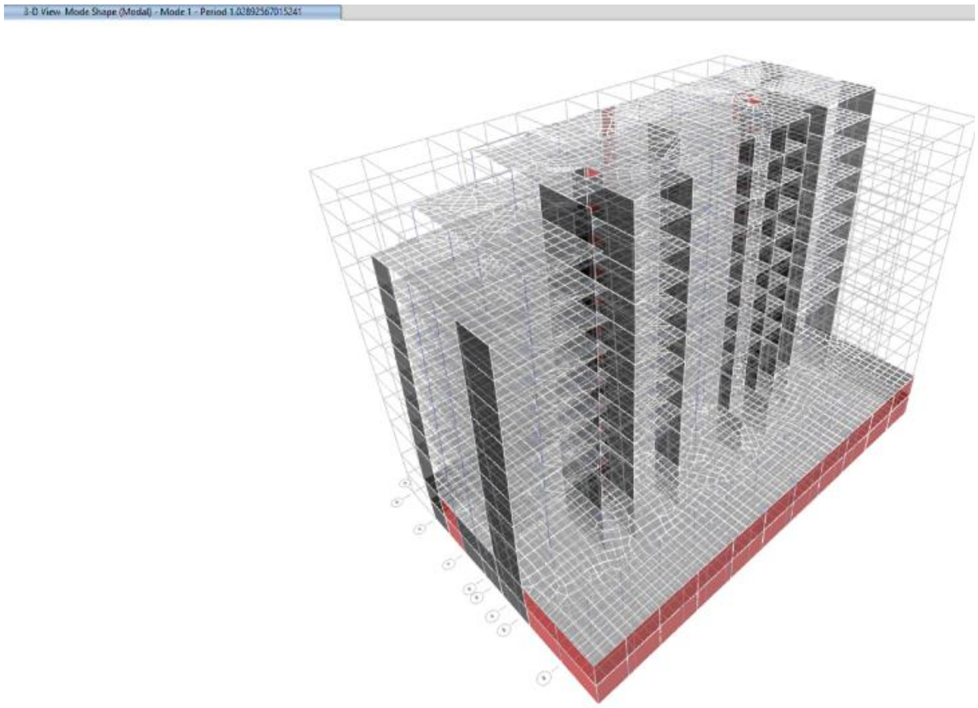


Figure 44 1st mode deformed shape and period

Figure 45 2nd mode deformed shape and period

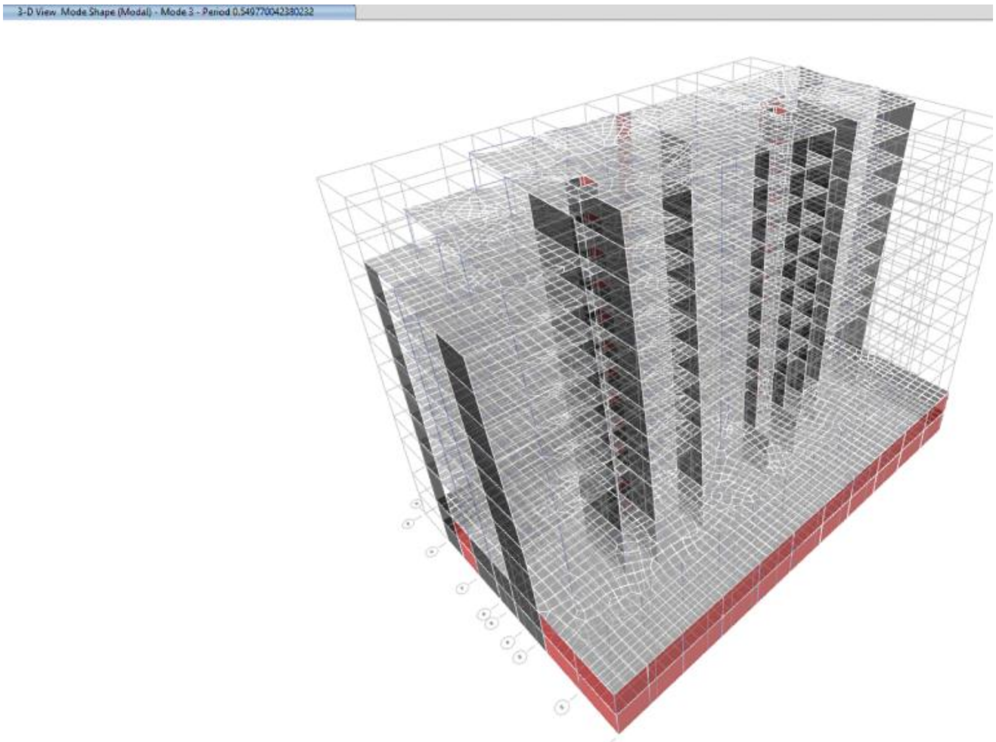


Figure 46 3rd mode deformed shape and period

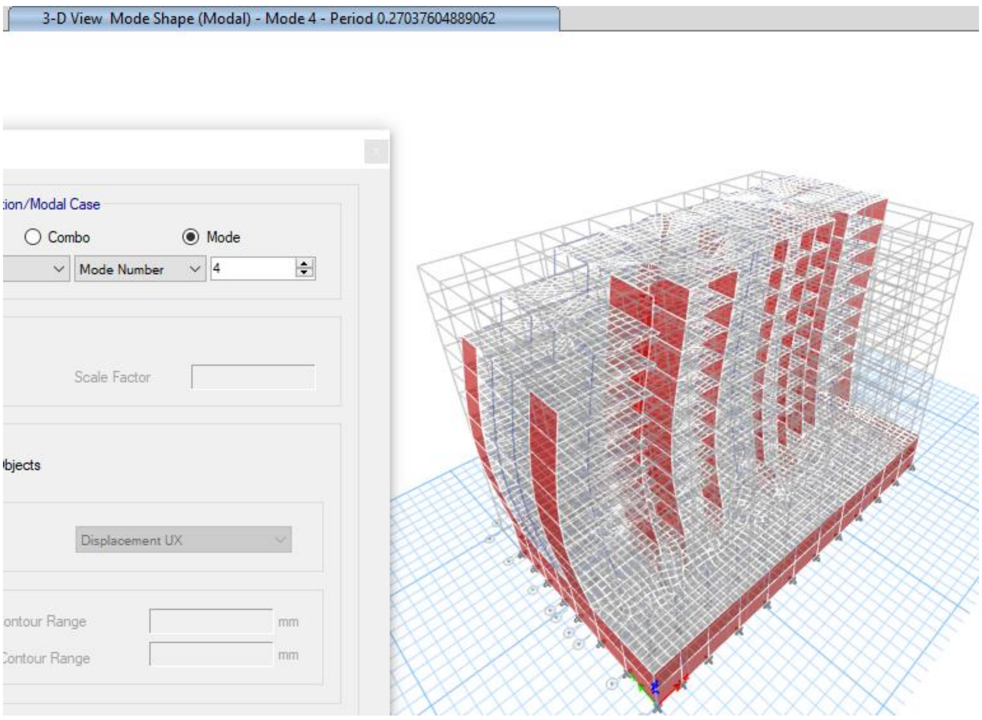


Figure 47 4th mode deformed shape and period

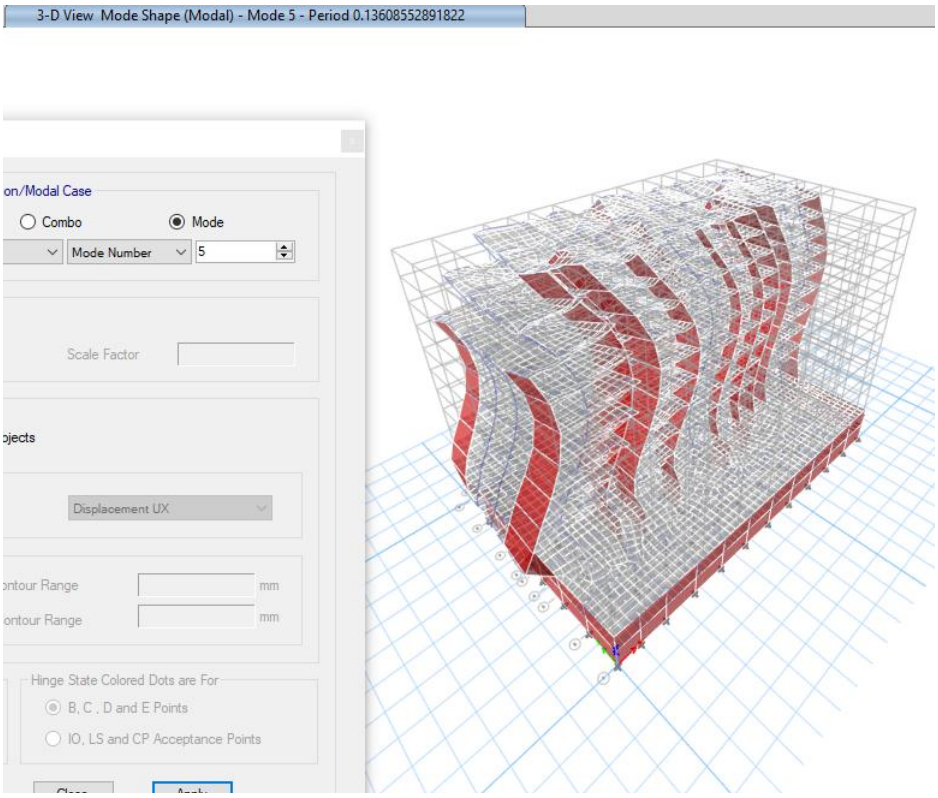


Figure 48 5th mode deformed shape and period

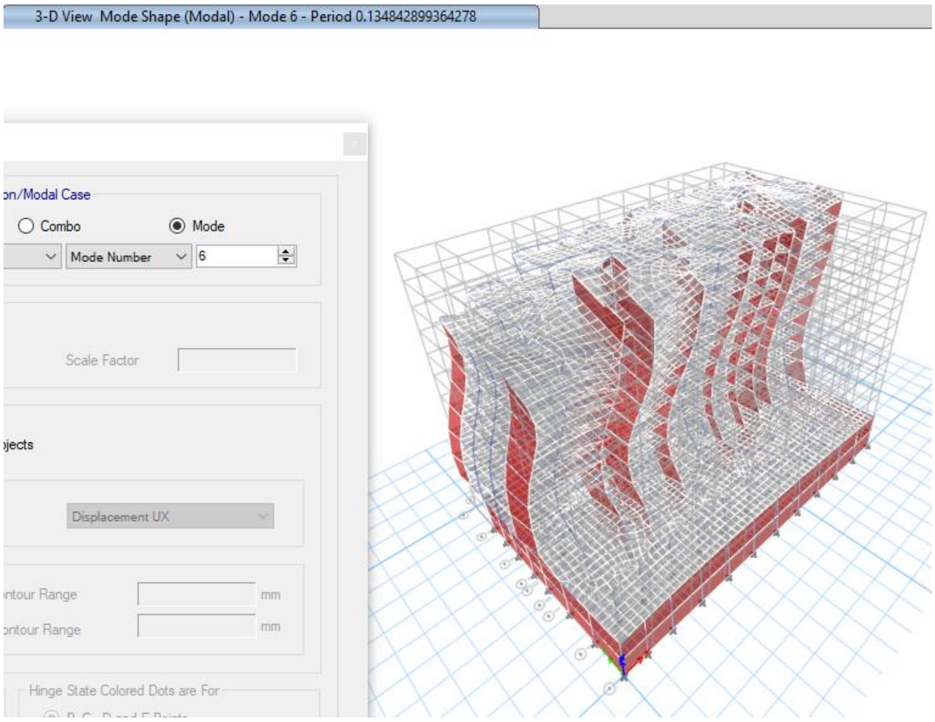


Figure 49 6th mode deformed shape and period

4.3 Results comparison for joint displacement, joint drift and story drift for RSA and Time-history analysis

4.3.1 Response spectrum analysis results

JOINT DISPLACEMENT						
1.35D	**UX**		**U Y**		**UZ**	
MAX	0.291	STORY 1	3.316	STORY 10	0	BASE
MIN	-0.764	STORY 10	-0.046	STORY -2	-8.56	BASE
0.35D+1.5L						
MAX	0.331	STORY 1	4.344	STORY 10	0	BASE
MIN	-1.065	STORY 10	0.067	STORY -1	-10.543	STORY 1
D+0.3L+EX						
MAX	42.729	STORY 10	11.594	STORY 10	1.663	STORY 2
MIN	-43.216	STORY 10	-7.775	STORY 10	-12.674	STORY 1
D+0.3L+EY						
MAX	12.699	STORY 10	28.988	STORY 10	3.652	STORY 6
MIN	-13.275	STORY 10	-24.779	STORY 10	-8.556	STORY 1
D+0.3L-EX						
MAX	42.729	STORY 10	11.594	STORY 10	1.663	STORY 2
MIN	-43.216	STORY 10	-7.775	STORY 10	-12.674	STORY 1
D+0.3L-EY						
MAX	12.699	STORY 10	28.988	STORY 10	3.652	STORY 6
MIN	-13.275	STORY 10	-24.779	STORY 10	-8.556	STORY 1
RESPONSE SPECTRUM						
MAX	42.97	STORY 10	9.845	STORY 10	5.937	STORY 1
MIN	0	BASE	0	BASE	0	BASE

Table 8 Joint displacements obtained from each load case and RSA

STORY DRIFTS							
MAX		X(m)		Y(m)		Z(m)	
0.001572	STORY 2	54.42	STORY -2	29.4	STORY 9	40.74	STORY 10
MIN		X(m)		Y(m)		Z(m)	
2.40E-05	STORY -2	0	STORY 2-7	10.6	STORY 8	3.24	STORY -2

Table 9 Story drift obtained from RSA

JOINT DRIFTS								
	DISP X		DISP Y		DRIFT X		DRIFT Y	
MAX	42.97	STORY 10	9.845	STORY 10	0.001572	STORY 2	0.00039	STORY 5
MIN	0.015	STORY -2	0.021	STORY -2	5.00E-06	STORY -2	6.00E-06	STORY -1

Table 10 Joint drift obtained from RSA

4.3.2 Time-history analysis results

JOINT DISPLACEMENT						
RESPONSE SPECTRUM	UX (mm)		U Y (mm)		UZ	
MAX	0.005	STORY 10	0.001	STORY 10	0.00032	STORY 2
MIN	-0.001	STORY 10	-0.002	STORY 10	-0.001	STORY 1

Table 11 Joint displacements obtained from T-H analysis

STORY DRIFTS							
MAX		X(m)		Y(m)		Z(m)	
1.80E-07	STORY 2	54.42	STORY -2	29.55	STORY 2	40.74	STORY 10
MIN		X(m)		Y(m)		Z(m)	
0.00E+00	STORY -2	0	STORY 2-7	10.6	STORY 10	3.24	STORY -2

Table 12 Story drift obtained from T-H anlysis

JOINT DRIFTS								
	DISP X		DISP Y		DRIFT X		DRIFT Y	
MAX	0.005	STORY 10	0.001	STORY 10	1.80E-07	STORY 2	8.06E-08	STORY 3
MIN	-0.001	STORY 10	-0.002	STORY 10	ZERO	STORY 1	ZERO	STORY 3

Table 13 Joint drift obtained from T-H analysis

4.4 Concrete frame elements design and detailing using CSI detailing

CSI Detailing is a software that generates detailing outputs such as detailed views, drawings and bill of materials, quantities from ETABS model. Using ETABS reinforcement results as presentet in the previous sections it is possible to design the concrete frames using also Eurocode requirements and rules for design. Designing a multistorey builinding takes time and is a challenging process but using different engineering softwares such as ETABS and CSI Detailing makes the process easier, less time consuming and more fun. While doing this study and designing this building I learned a lot about ETABS program, different analysis and results you can take from the program and CSI Detailing was an addition to this study since I found it really interesting. CSI Detailing designs the concrete frame elements using the code you want (in my case Eurocodes) and you can specify your wanted rebar sections to be used, mximum and minimum bar size, bar spacing etc. CSI Detailing gives you the opportunity to design the building using the seismic or non-seismic design option. For my case study I have used seismic design option since it is the topic I am focusing on this thesis.

CSI Detailing uses the ETABS modeling data of the builing for the design. Firstly we make sure that the modeling is correct, the results are checked with Eurocode standards and than we can proceed with the designing process. In order for CSI Detailing to detail the elements firstly analysis has to be runned on the ETABS model of the building and concrete frame element design and check needs to be done. To detail and design the slab we analyze the slab by using the strip method. Strip method consists on deviding the slab in strips in both X and Y direction with autospacing or self chosen ones.

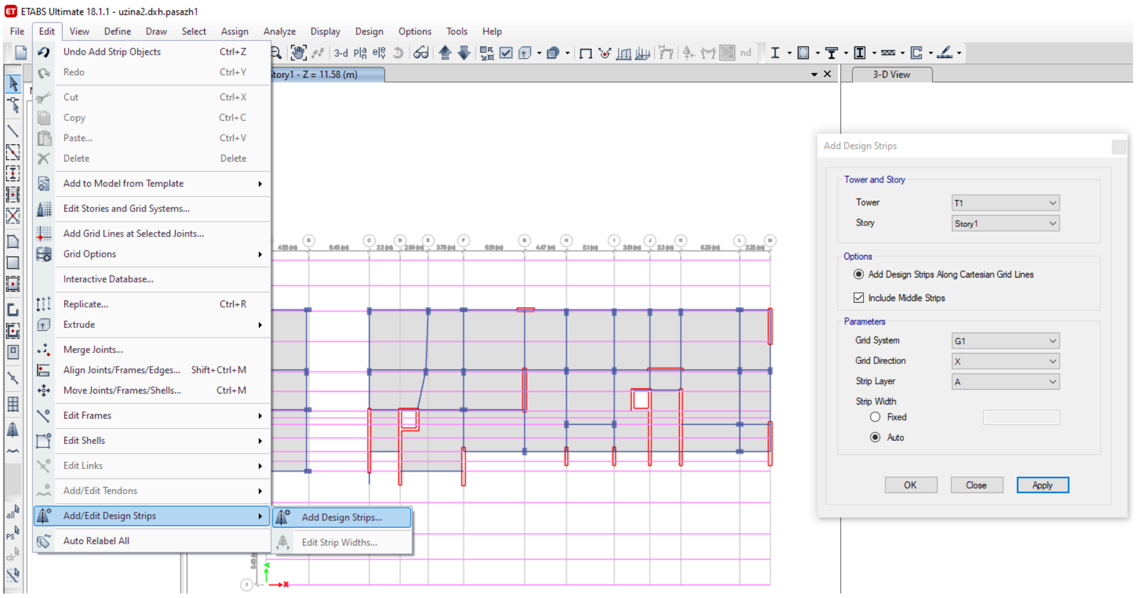


Figure 50 slab design strips

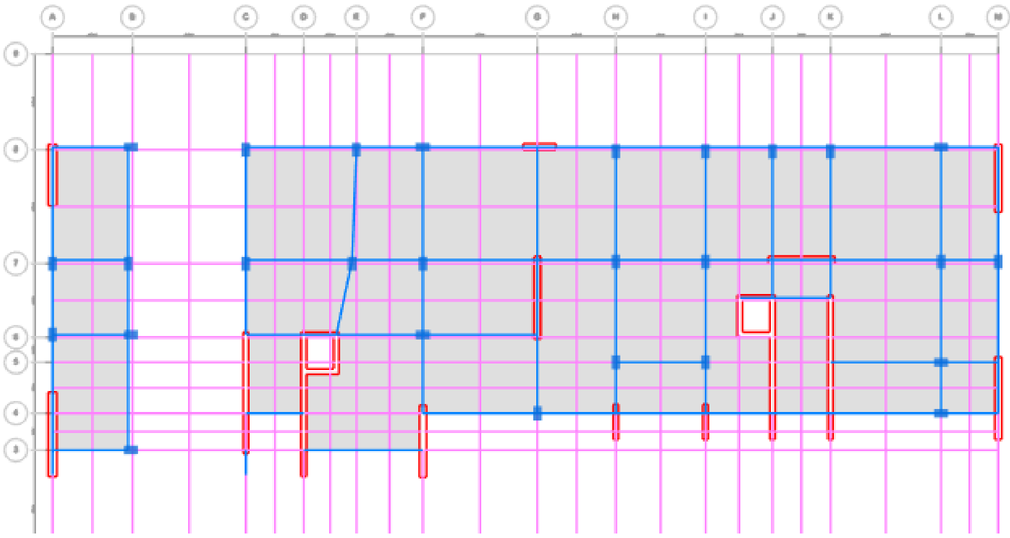
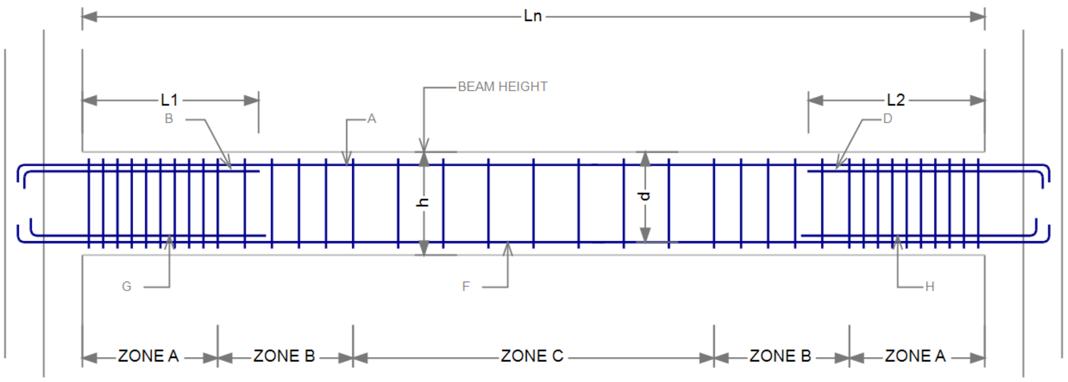


Figure 51 slab strip design division

We can include middle stripes as well and we can also remove the unwanted t=strips by deleting them. This step is done for the 1^{st} storey since I intend to interpret the results and detail the concrete frame elements of the 1^{st} storey only, but the same procedure may be done for each floor you want to study and design.After this step we can proceed with CSI Detailing.

4.4.1 Standard details

In the figures below the standard details of slab, beam, wall and column are presented.



NOTES:
1. SEE BEAM SCHEDULE FOR STIRRUP TYPE AND SPACING.
2. ZONE A LENGTH SHALL BE TWICE THE BEAM DEPTH, d.
3. THE FIRST STIRRUP IN ZONE A SHALL BE LOCATED 50mm MAXIMUM FROM THE FACE OF THE SUPPORT.
4. LAP SPLICES SHALL NOT BE LOCATED WITHIN THE BEAM/COLUMN JOINT, NOR WITHIN A DISTANCE OF 2H.
5. LAP SPLICE LENGTH SHALL NOT BE LESS THAN 300mm.

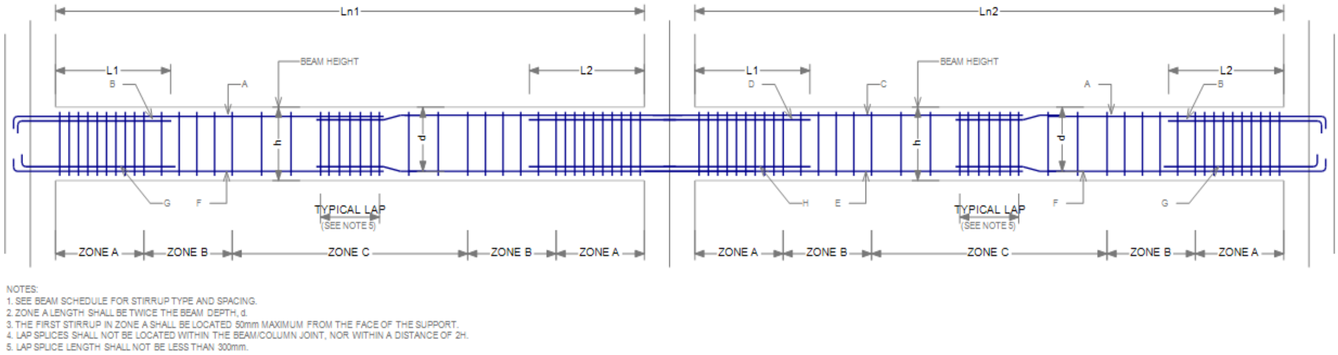


Figure 52 Typical concrete beam seismic design

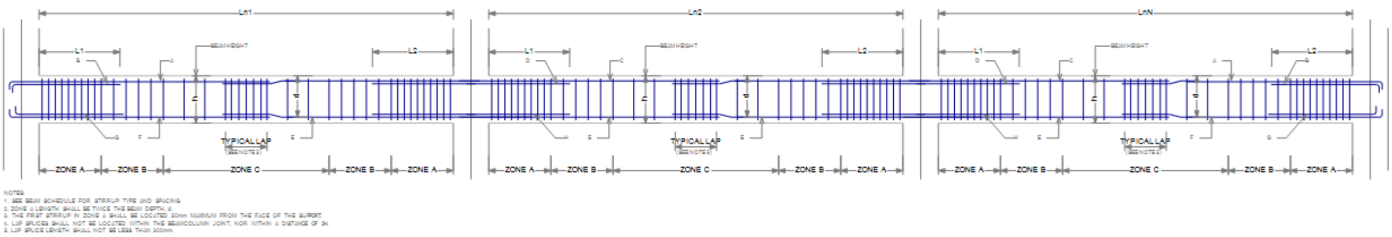


Figure 53 Typical concrete beam seismic design

Figure 54 Typical concrete beam seismic design

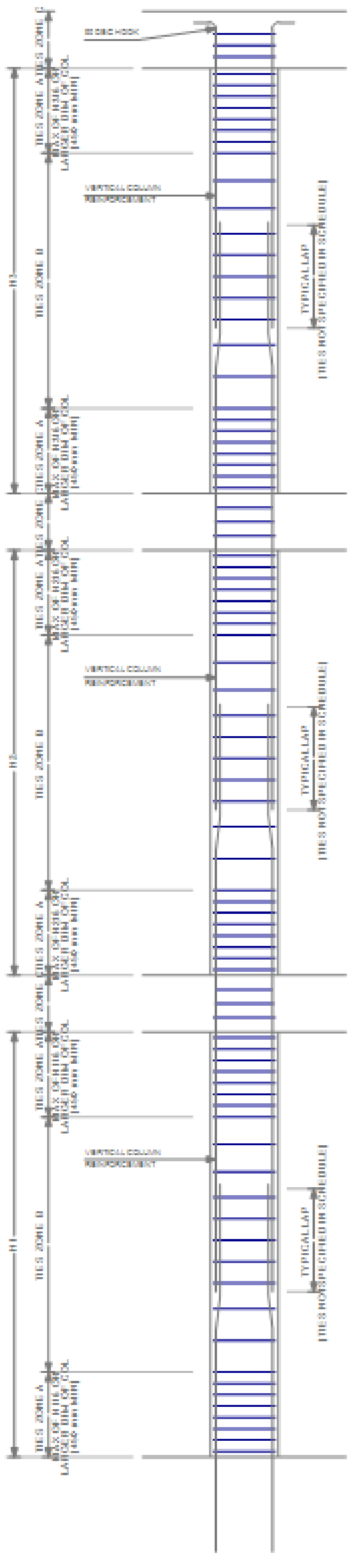


Figure 55 Typical column seismic design

CHAPTER 5

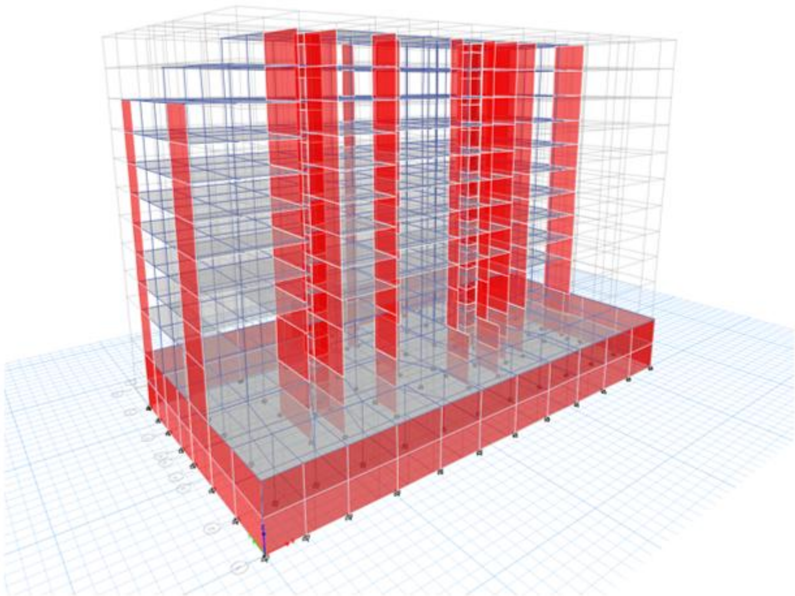
CONCLUSIONS

5.1 Conclusions

Designing a multistorey building is a challenging, difficult and time consuming process. Using engineering softwares such as ETABS and CSI Detailing as in this study makes the process easier and more complete. Using softwares to model and then analyze and design a building also saves time and give you the chance to interpret many results and output taken from the software. Besides designing the building you gain a lot of insight in the building you are modeling, learn to run different analysis and gain different results and compare them.

Earthquake resistant design method does not attempt to design a building that will not be damaged during very strong earthquakes that rarely happen but the main focus is to design a building that will be earthquake resistant. Designing a building that fully resists and does not damage at all during ground shaking is too robust and expensive. Earthquake resistant buildings are the ones that earthquake can cause damages but not collapse. Seismic design codes predict exactly that and the avoidance of life loss, collapse, and disaster during an earthquake. The risen question is rather if we must design a building that fully resists and does not damage during a strong earthquake that may come once in 500-2000 years or design a building that will damage but not collapse and does not cause a disaster? The best option while completing the design of a structure is designing it answering both questions risen above.

Many existing reinforced concrete frame buildings located in seismic zones have poor seismic resistance. Insufficient lateral strength and poor reinforcement details are the main reasons for insufficient seismic performance. The shear wall system is one of the most widely used anti-lateral load technologies in high-rise buildings. Shear walls have very high strength and rigidity on the plane, and can be used to resist large horizontal and gravity loads at the same time. In high-rise buildings, it is very important to ensure sufficient lateral rigidity to resist lateral loads. The goal of this project is to determine a solution for the location of the shear wall in a multi-storey building. Positioning the shear walls of the structure I have analyzed in a different way can increase the earthquake resistance of the structure.



As seen in the picture above this building has shear walls in the same direction and combining the shear walls in both x and y directions will results on a better resistance of the overall structure during an earthquake.

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C60 International Conference, 7-9 November, Cluj-Napoca, ROMANIA ”Tradition and Innovation - 60 Years of Civil Engineering Higher Education in Transilvania” Fig. 1: Scaled structure loading pattern and orientation Fig. 2: Steel bar weights Earthquake Resistant Multi Storey Structures Cristian Miculas, Marius Moldovan, Adela Ciocan;

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APPENDIX

APPENDIX A

a) Internal forces for Response spectrum analysis

In this section diagrams of shear forces and bending moments obtained by the modal response spectrum analysis will be presented. Results are shown for selected frames and walls.

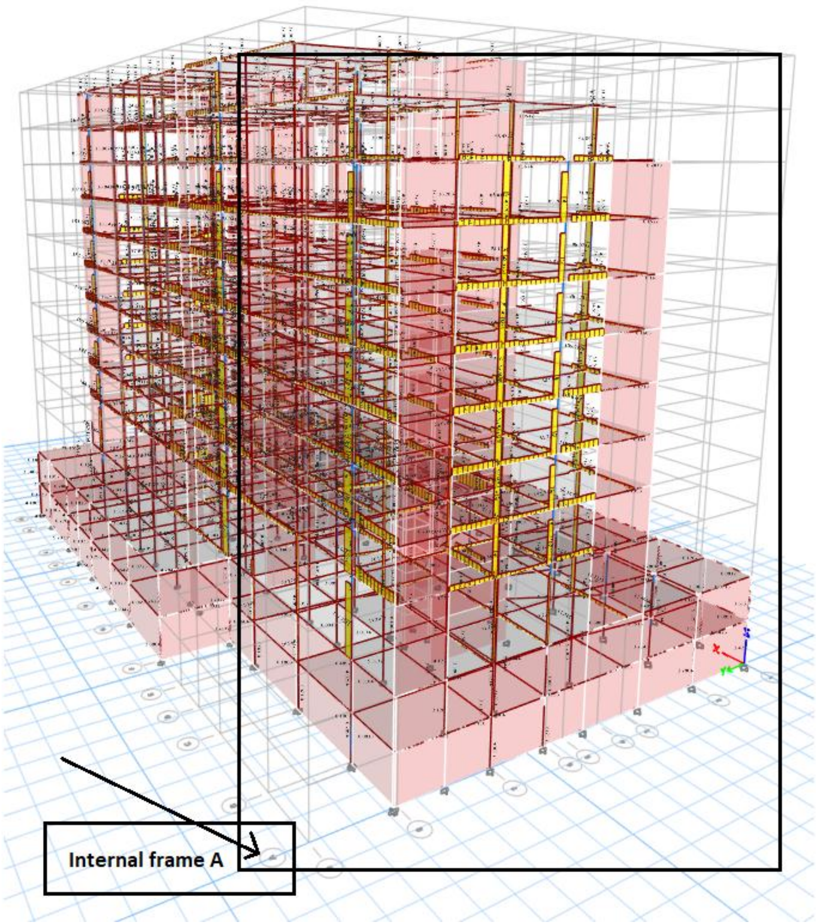


Figure 56 Internal frame A

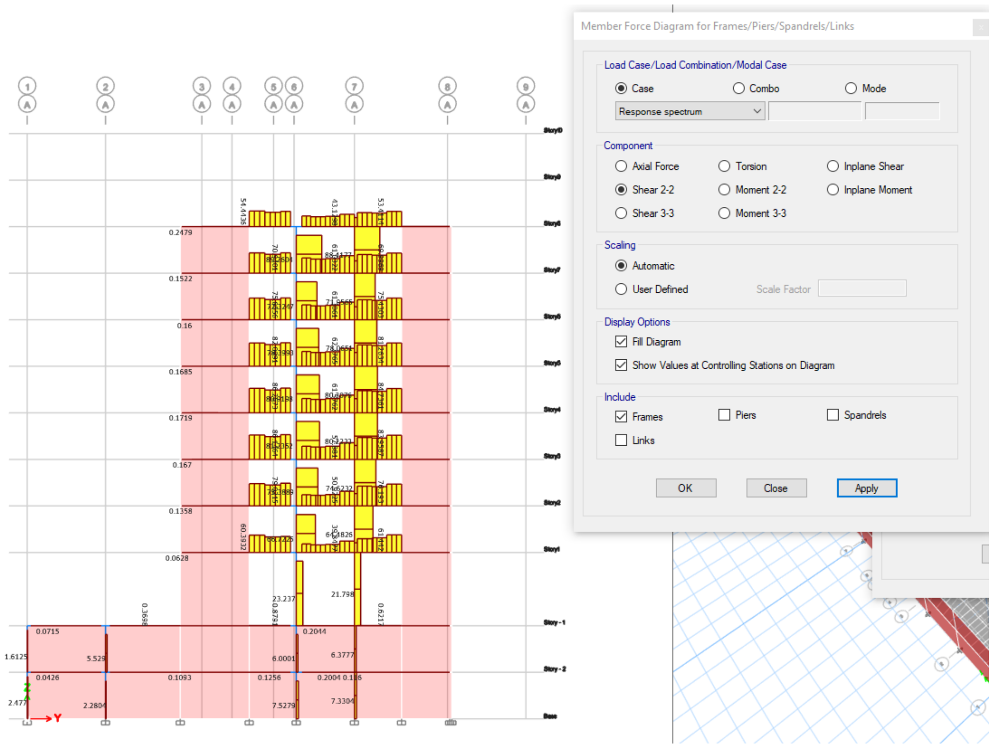


Figure 57 Shear force (Shear 2-2) for internal frame A

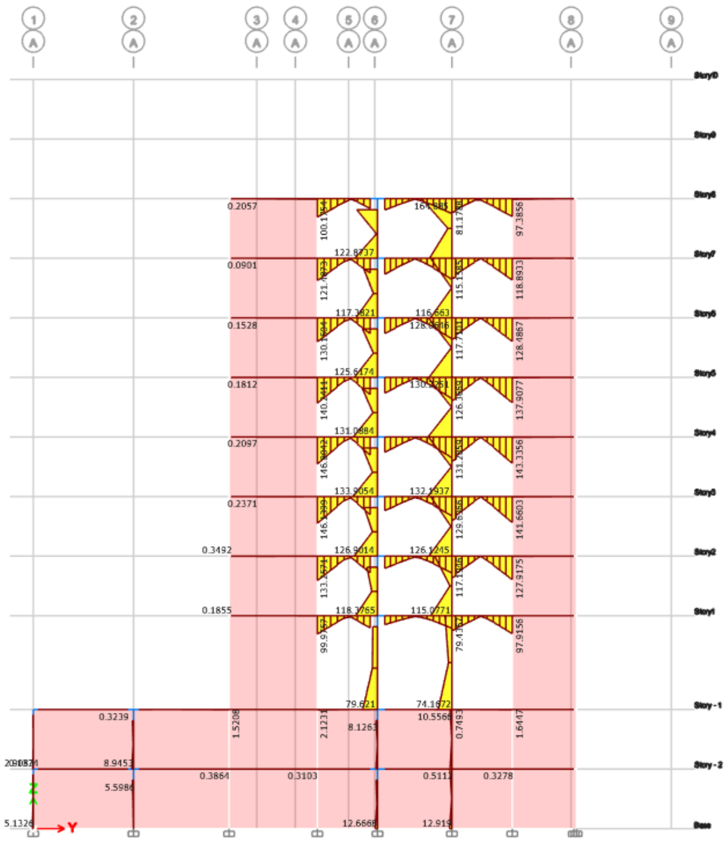


Figure 58 Moment 3-3 for internal frame A

Figure 59 Shear force (Shear 2-2) for internal frame C

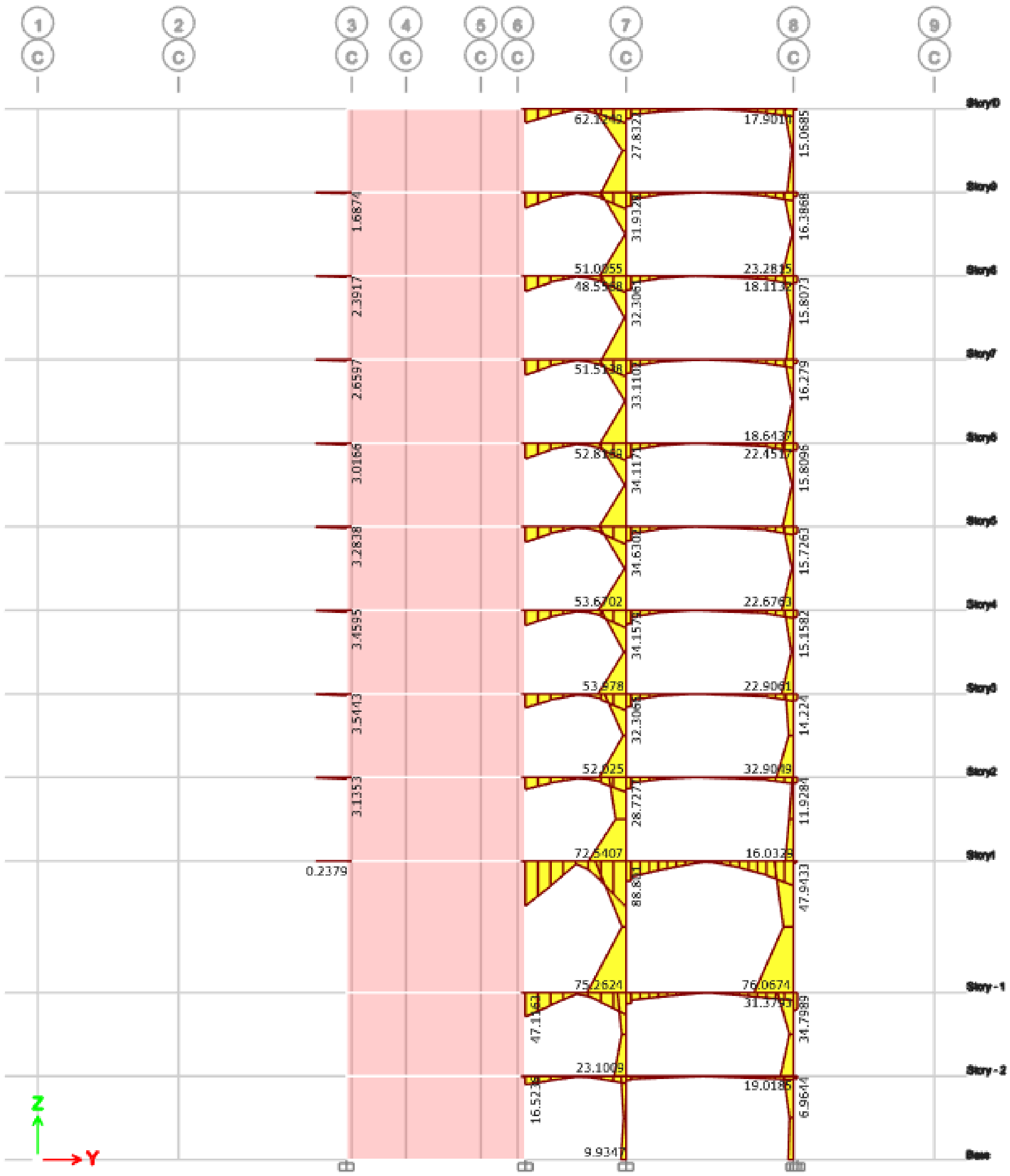


Figure 60 Moment 3-3 for internal frame C

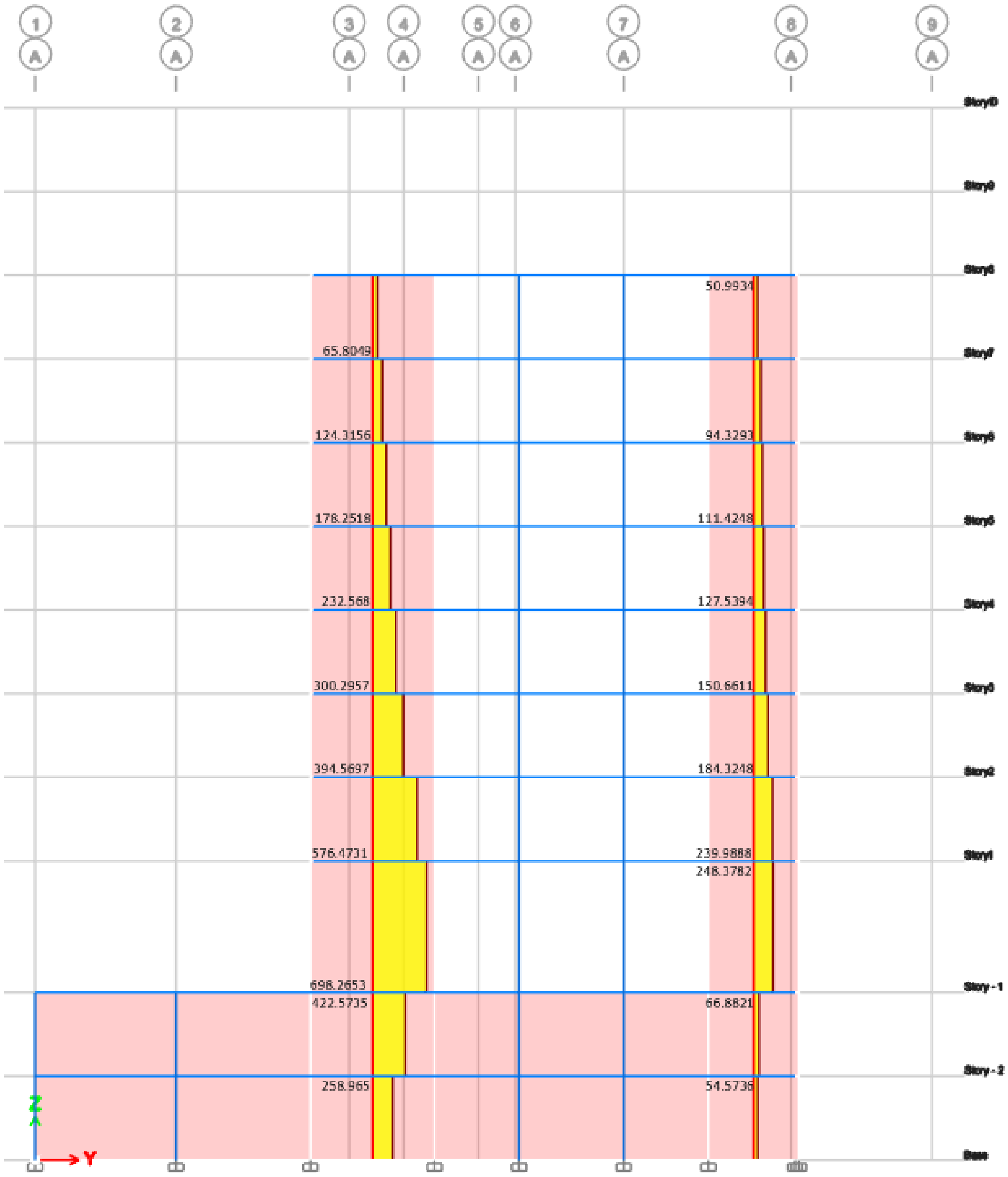


Figure 61 Pier shear forces (Shear 2-2) on frame A

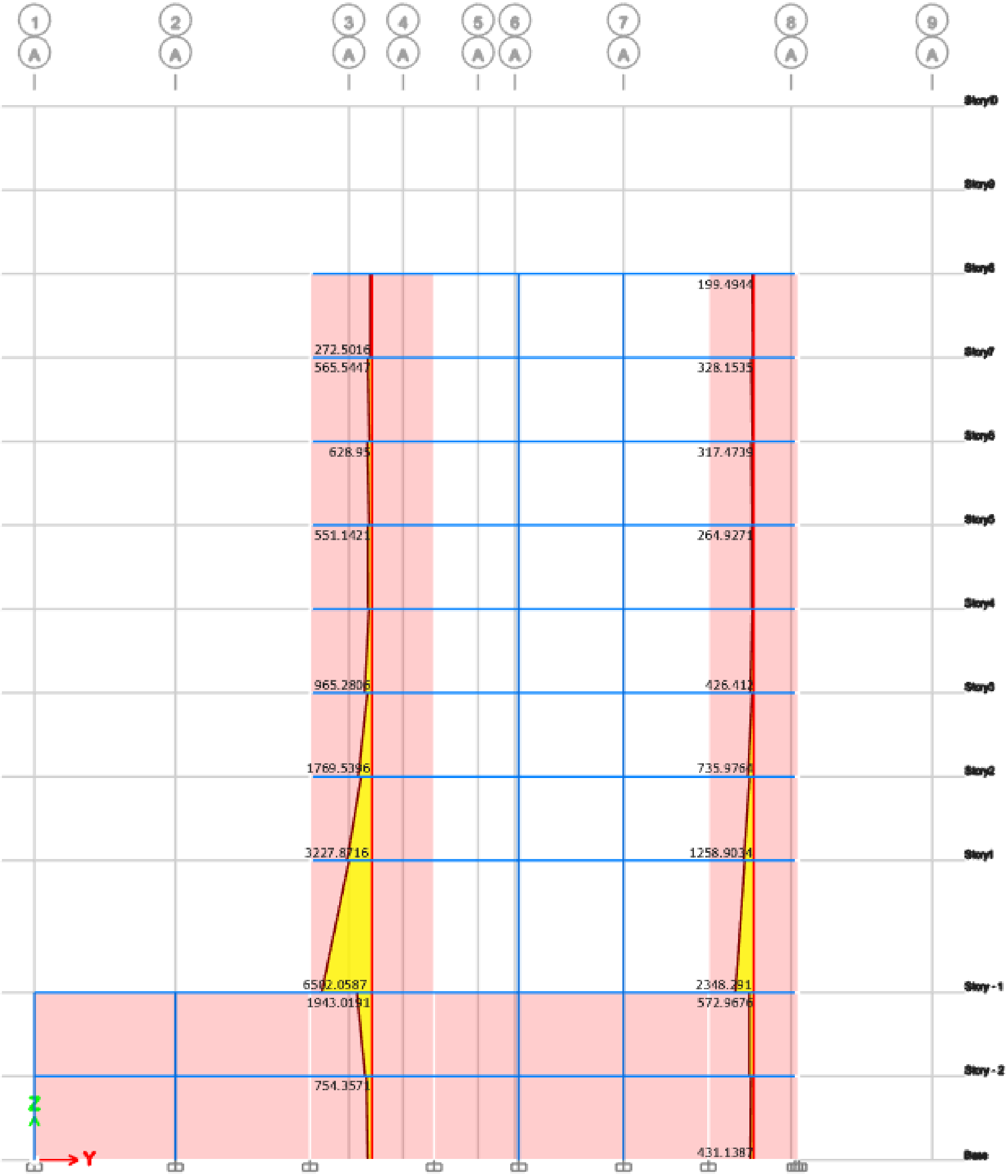


Figure 62 Pier Moment (Moment 3-3) on frame A

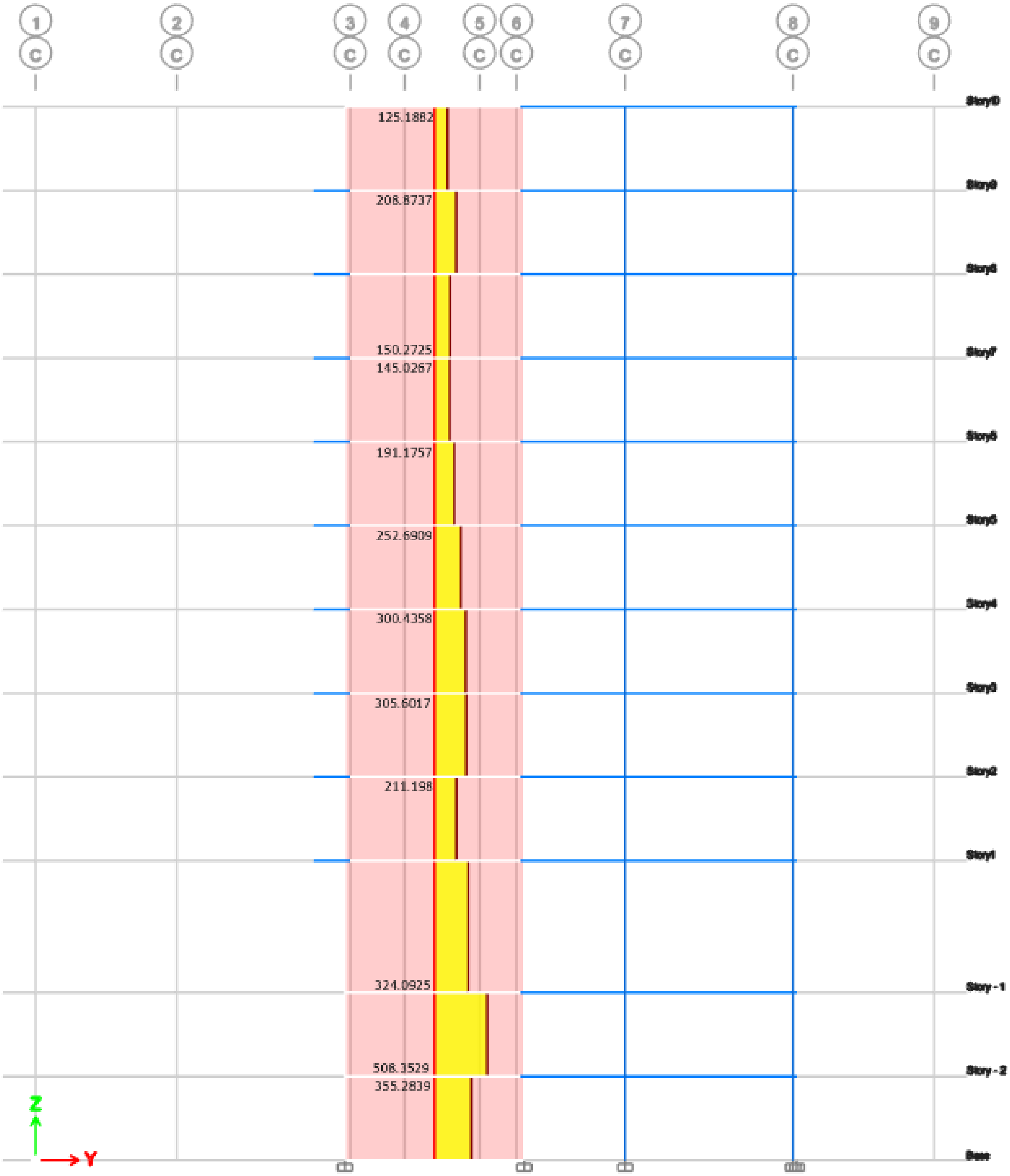


Figure 63 Pier shear forces (Shear 2-2) on frame C

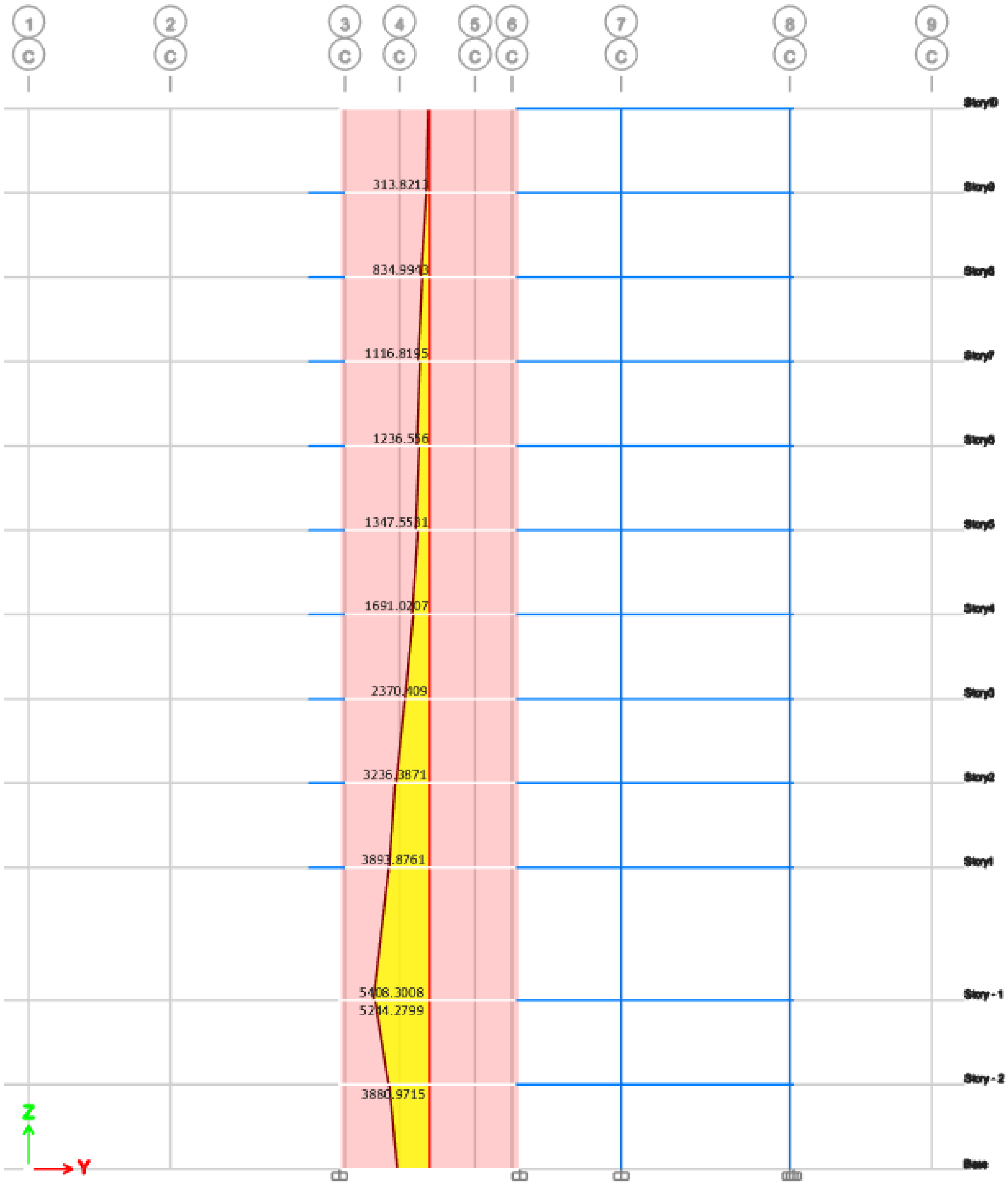


Figure 64 Pier Moment (Moment 3-3) on frame C

b) Result tables

Joint Displacements

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Joint Displacements

Filter: ([Case Type] = 'LinRespSpec')

Story	Label	Unique Name	Output Case	Case Type	Step Type	Ux mm	Uy mm	Uz mm	Rx rad
Story10	34	1261	Response sp...	LinRespSpec	Max	40.496	8.666	0.592	0.00021
Story10	79	657	Response sp...	LinRespSpec	Max	39.123	9.832	1.184	0.000272
Story10	89	905	Response sp...	LinRespSpec	Max	42.59	9.845	0.756	0.00028
Story10	90	904	Response sp...	LinRespSpec	Max	41.617	9.838	0.877	0.00026
Story10	93	957	Response sp...	LinRespSpec	Max	42.531	8.699	1.392	0.000263
Story10	94	1225	Response sp...	LinRespSpec	Max	42.532	8.376	1.365	0.000252
Story10	104	478	Response sp...	LinRespSpec	Max	42.935	8.352	1.334	0.000261
Story10	231	1267	Response sp...	LinRespSpec	Max	39.151	8.671	0.121	0.000256
Story10	232	1269	Response sp...	LinRespSpec	Max	39.166	8.149	0.783	0.000226
Story10	234	1273	Response sp...	LinRespSpec	Max	40.501	7.946	0.27	0.000195
Story10	241	182	Response sp...	LinRespSpec	Max	42.97	7.955	0.723	0.000246
Story10	242	181	Response sp...	LinRespSpec	Max	42.149	7.952	0.368	0.00024
Story10	248	1811	Response sp...	LinRespSpec	Max	41.299	8.669	1.279	0.000265
Story10	249	430	Response sp...	LinRespSpec	Max	41.346	8.331	0.724	0.000224
Story10	250	428	Response sp...	LinRespSpec	Max	40.564	8.065	0.725	0.000273

Record: << < 1 > >> of 1653 Add Tables... Done

Figure 65 Joint displacement obtained from RSA

Joint Drifts

File Edit Format-Filter-Sort Select Options

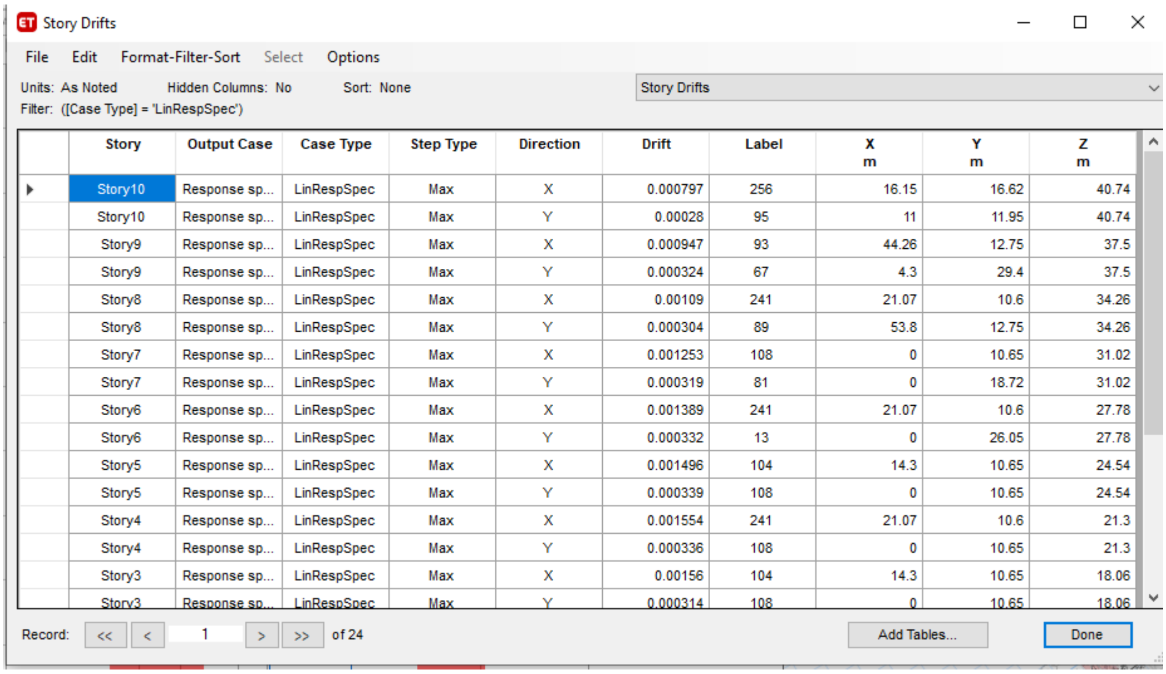
Units: As Noted Hidden Columns: No Sort: None Joint Drifts

Filter: ([Case Type] = 'LinRespSpec')

Story	Label	Unique Name	Output Case	Case Type	Step Type	Disp X mm	Disp Y mm	Drift X	Drift Y
Story10	34	1261	Response sp...	LinRespSpec	Max	40.496	8.666	0.000735	0.000275
Story10	79	657	Response sp...	LinRespSpec	Max	39.123	9.832	0.000699	0.000275
Story10	89	905	Response sp...	LinRespSpec	Max	42.59	9.845	0.00079	0.000278
Story10	90	904	Response sp...	LinRespSpec	Max	41.617	9.838	0.000767	0.000275
Story10	93	957	Response sp...	LinRespSpec	Max	42.531	8.699	0.000767	0.00026
Story10	94	1225	Response sp...	LinRespSpec	Max	42.532	8.376	0.000775	0.00025
Story10	104	478	Response sp...	LinRespSpec	Max	42.935	8.352	0.000784	0.000262
Story10	231	1267	Response sp...	LinRespSpec	Max	39.151	8.671	0.000685	0.000276
Story10	232	1269	Response sp...	LinRespSpec	Max	39.166	8.149	0.000688	0.000251
Story10	234	1273	Response sp...	LinRespSpec	Max	40.501	7.946	0.000732	0.000242
Story10	241	182	Response sp...	LinRespSpec	Max	42.97	7.955	0.000794	0.000247
Story10	242	181	Response sp...	LinRespSpec	Max	42.149	7.952	0.000778	0.000246
Story10	248	1811	Response sp...	LinRespSpec	Max	41.299	8.669	0.000764	0.000276
Story10	249	430	Response sp...	LinRespSpec	Max	41.346	8.331	0.000769	0.000261
Story10	252	431	Response sp...	LinRespSpec	Max	41.343	8.357	0.000769	0.000262
Story10	254	433	Response sp...	LinRespSpec	Max	41.81	8.356	0.000797	0.000262

Record: << < 1 > >> of 849 Add Tables... Done

Figure 66 Joint drift obtained from RSA



Story Drifts

File Edit Format-Filter-Sort Select Options

Units: As Noted Hidden Columns: No Sort: None Story Drifts

Filter: ([Case Type] = 'LinRespSpec')

Story	Output Case	Case Type	Step Type	Direction	Drift	Label	X m	Y m	Z m
Story10	Response sp...	LinRespSpec	Max	X	0.000797	256	16.15	16.62	40.74
Story10	Response sp...	LinRespSpec	Max	Y	0.00028	95	11	11.95	40.74
Story9	Response sp...	LinRespSpec	Max	X	0.000947	93	44.26	12.75	37.5
Story9	Response sp...	LinRespSpec	Max	Y	0.000324	67	4.3	29.4	37.5
Story8	Response sp...	LinRespSpec	Max	X	0.00109	241	21.07	10.6	34.26
Story8	Response sp...	LinRespSpec	Max	Y	0.000304	89	53.8	12.75	34.26
Story7	Response sp...	LinRespSpec	Max	X	0.001253	108	0	10.65	31.02
Story7	Response sp...	LinRespSpec	Max	Y	0.000319	81	0	18.72	31.02
Story6	Response sp...	LinRespSpec	Max	X	0.001389	241	21.07	10.6	27.78
Story6	Response sp...	LinRespSpec	Max	Y	0.000332	13	0	26.05	27.78
Story5	Response sp...	LinRespSpec	Max	X	0.001496	104	14.3	10.65	24.54
Story5	Response sp...	LinRespSpec	Max	Y	0.000339	108	0	10.65	24.54
Story4	Response sp...	LinRespSpec	Max	X	0.001554	241	21.07	10.6	21.3
Story4	Response sp...	LinRespSpec	Max	Y	0.000336	108	0	10.65	21.3
Story3	Response sp...	LinRespSpec	Max	X	0.00156	104	14.3	10.65	18.06
Story3	Response sp...	LinRespSpec	Max	Y	0.000314	108	0	10.65	18.06

Record: << < 1 > >> of 24 Add Tables... Done

Figure 67 Story drift obtained from RSA

c) Concrete frame design results

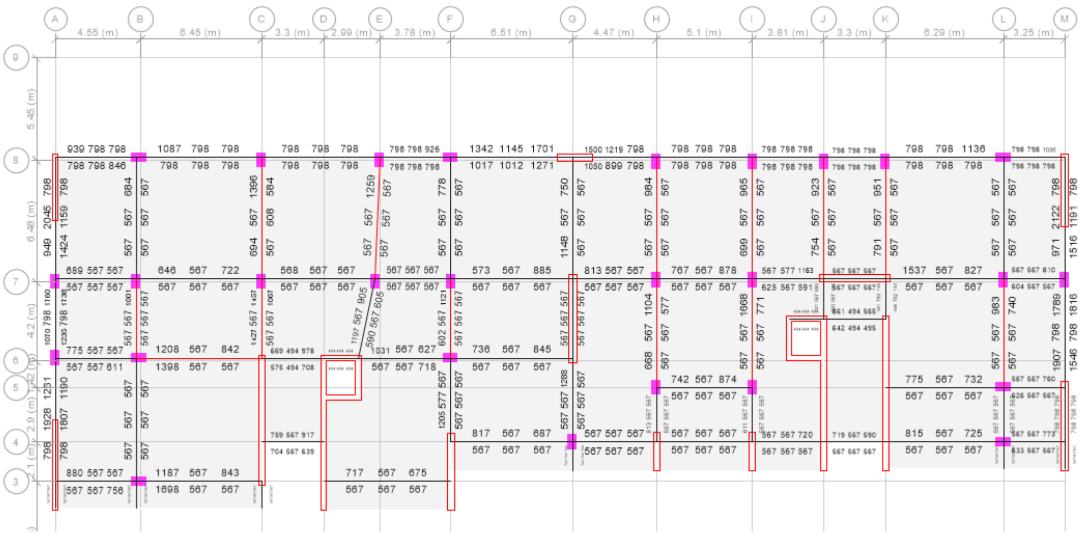


Figure 68 7th storey concrete frame design (ETABS output)

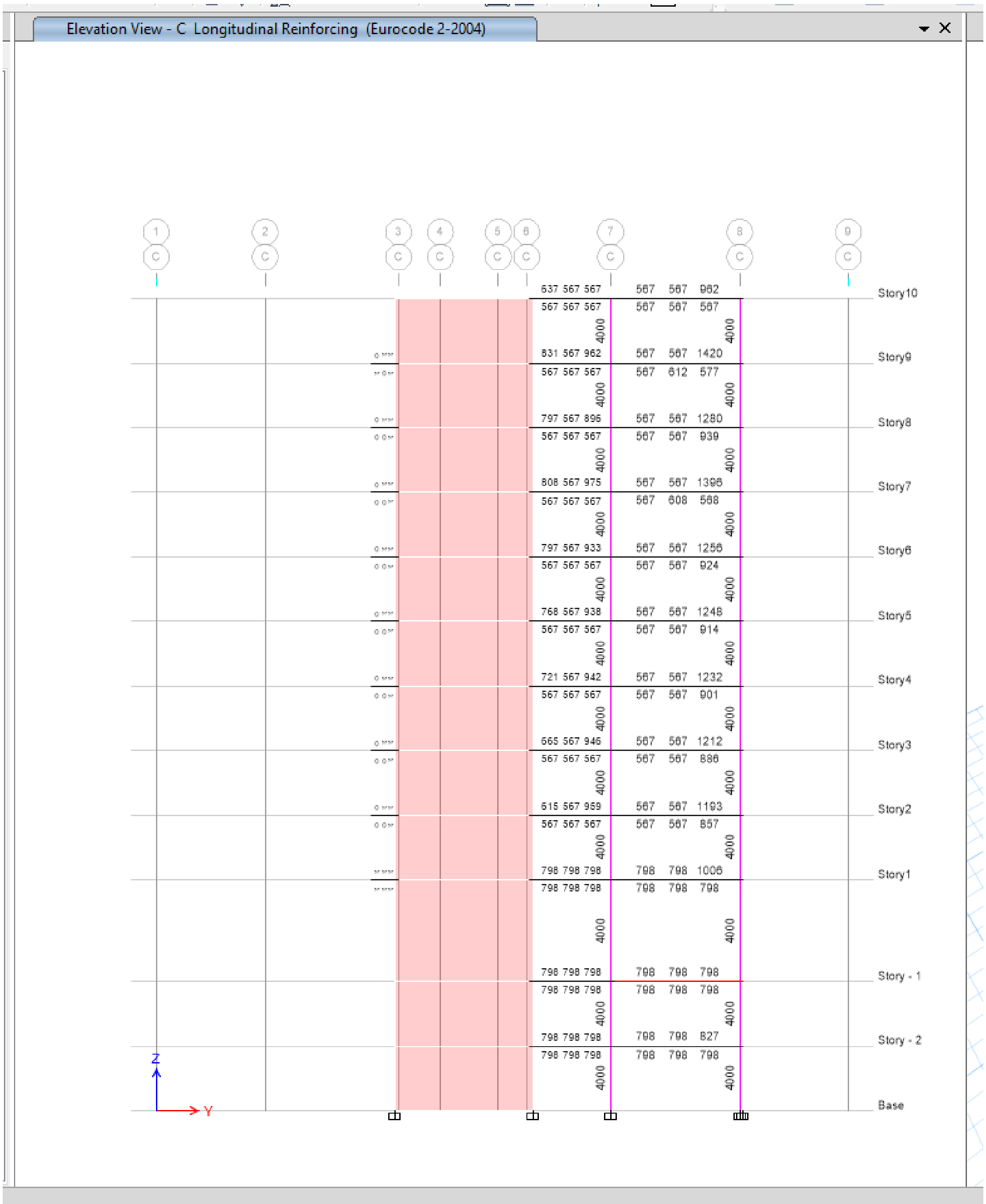


Figure 69 Concrete frame design ETABS output (Longitudinal reinforcement,Elevation view C-AXIS)

Time History Analysis Results

a) Maximum story displacement

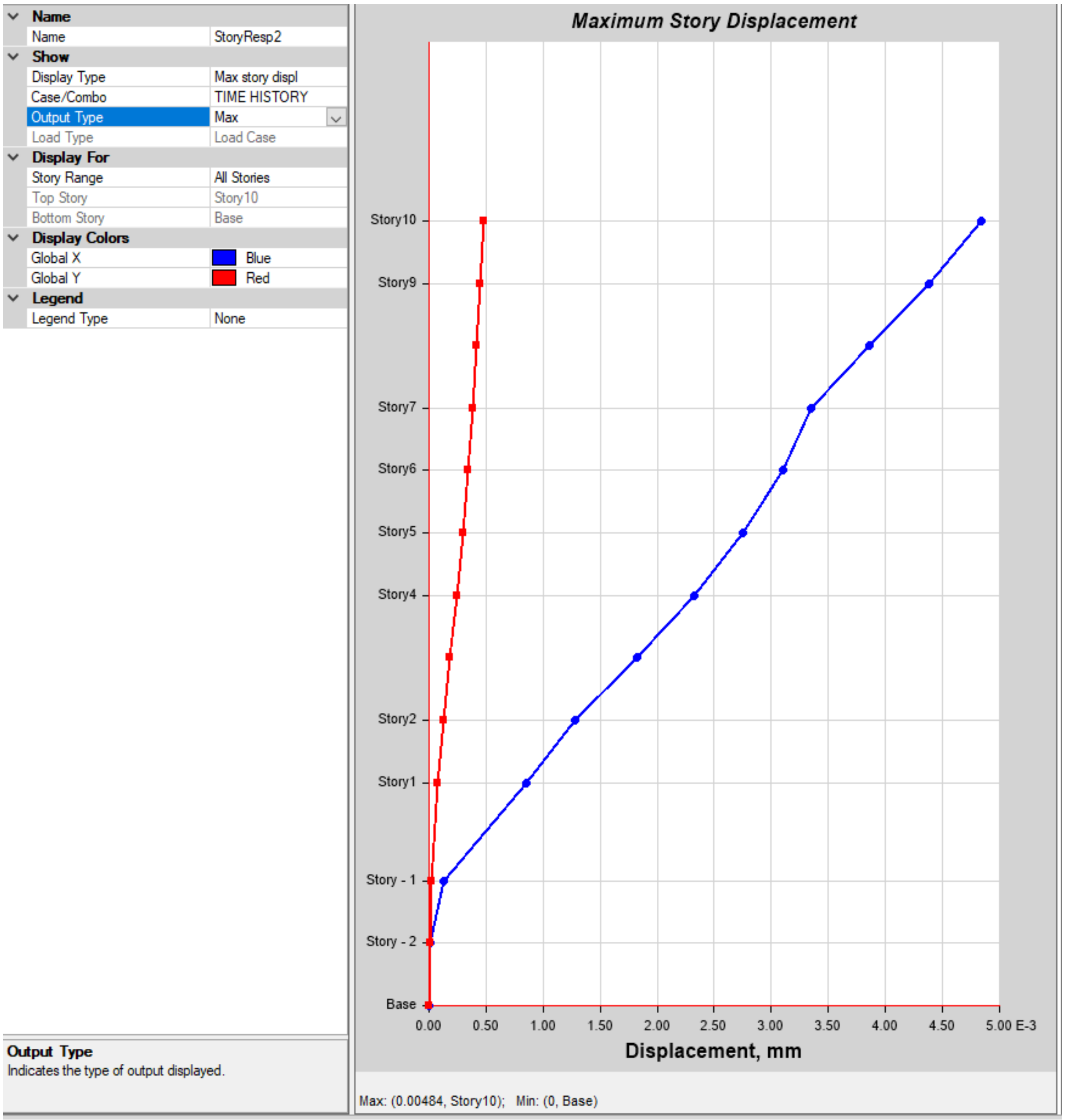


Figure 70 Maximum story displacement for max time

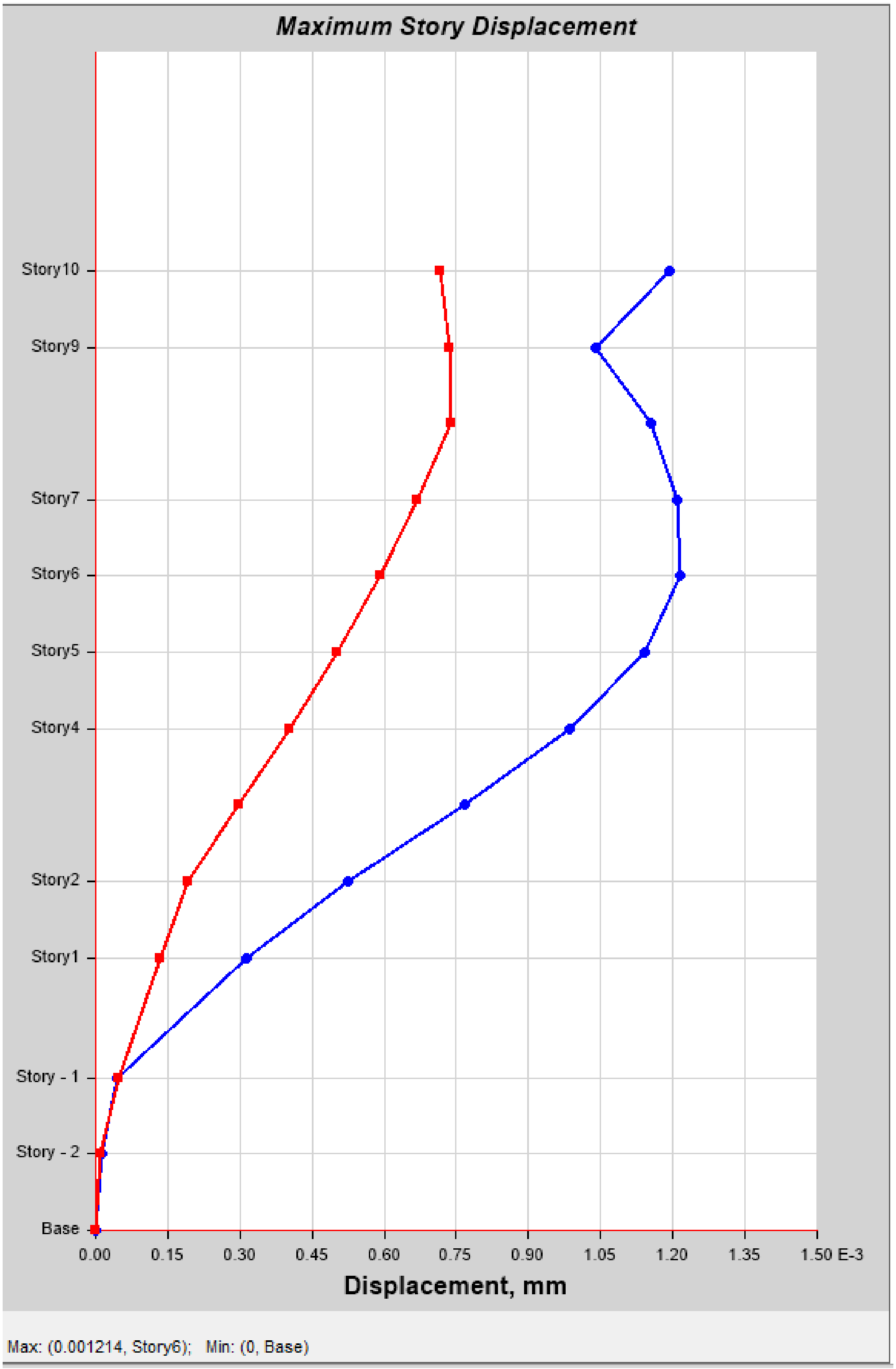


Figure 71 Maximum story displacement for min time

b) Story shears

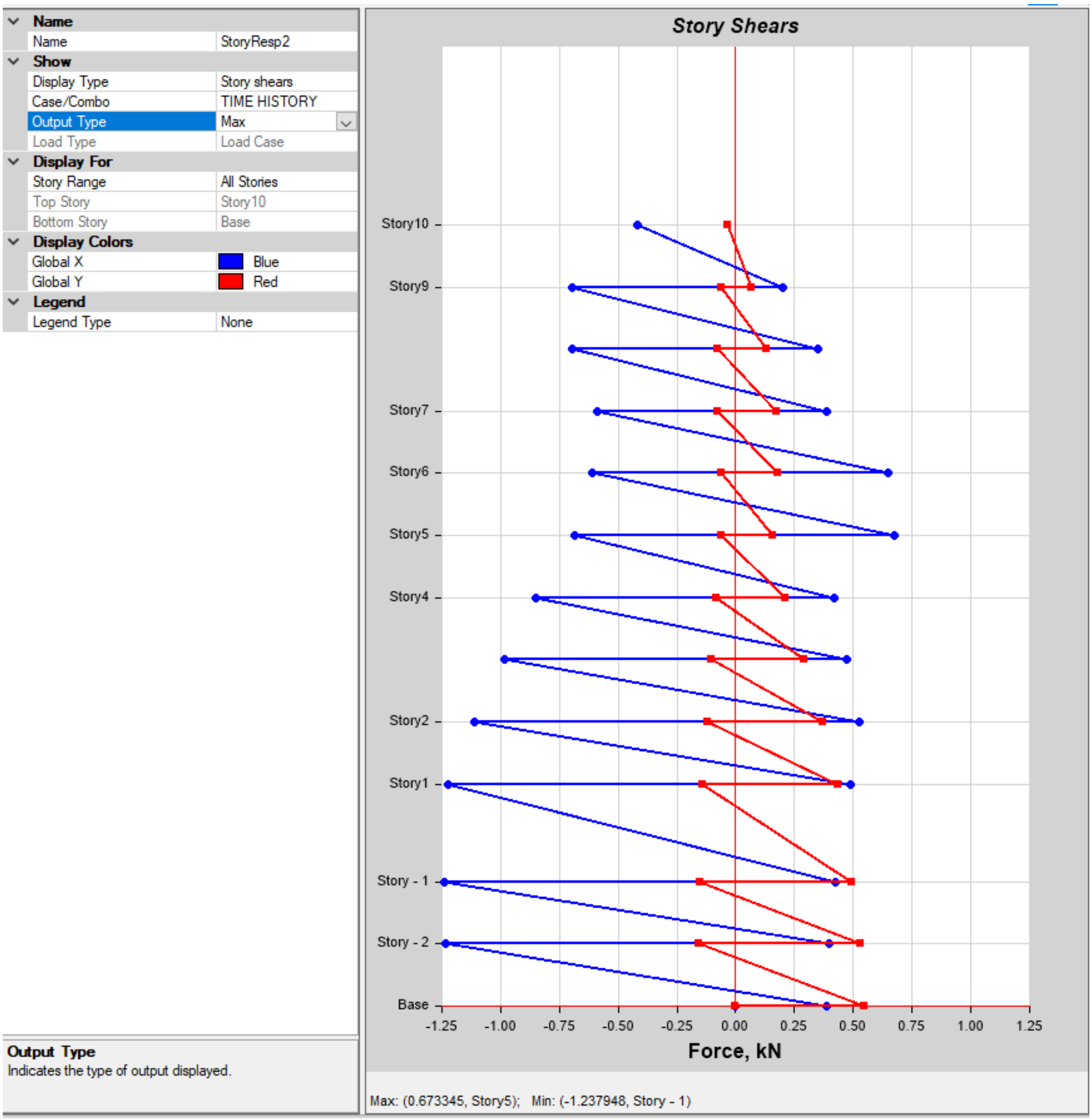


Figure 72 Maximum story shears for max time

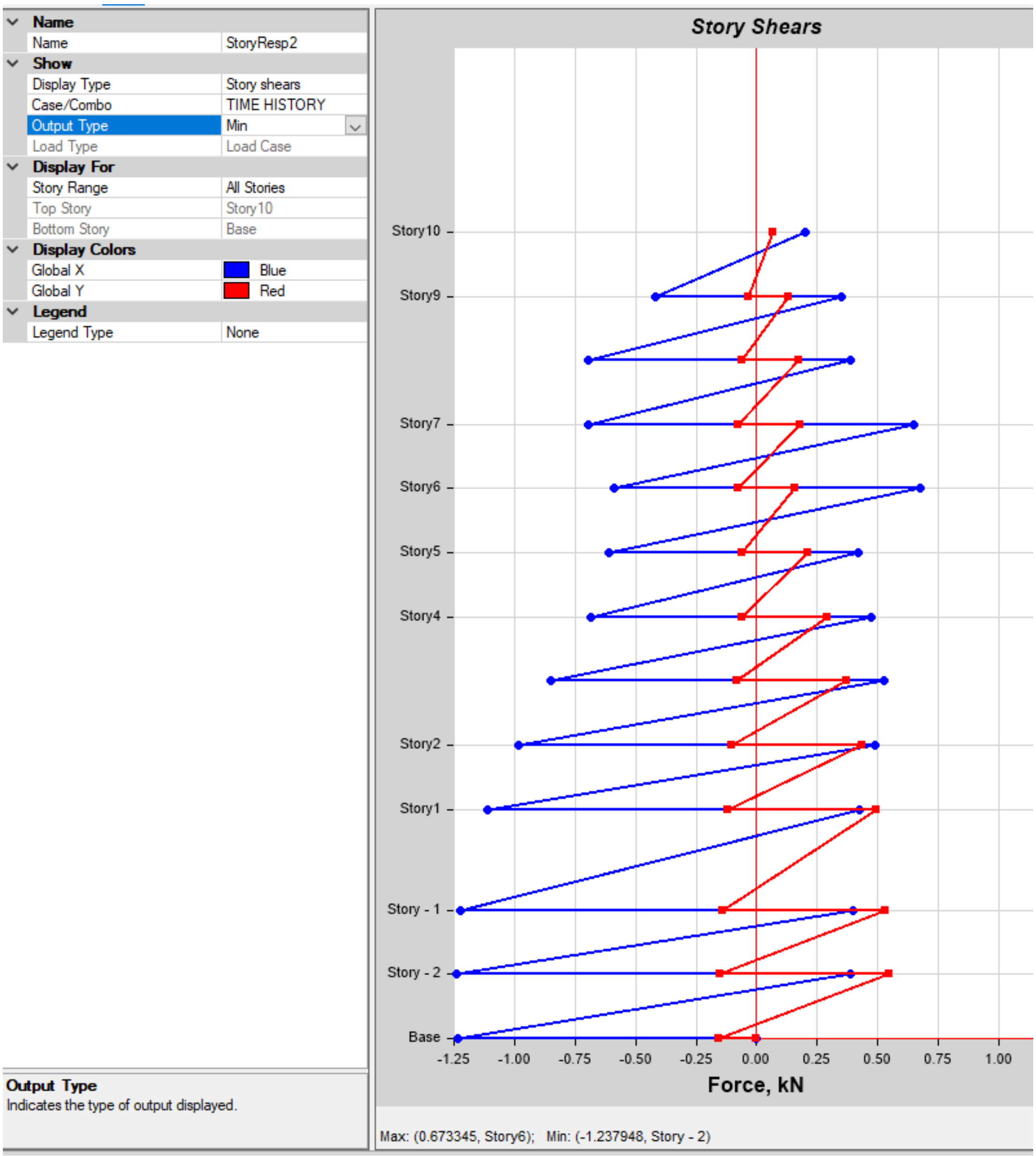


Figure 73 Story shears for min time

c) Story overturning moment

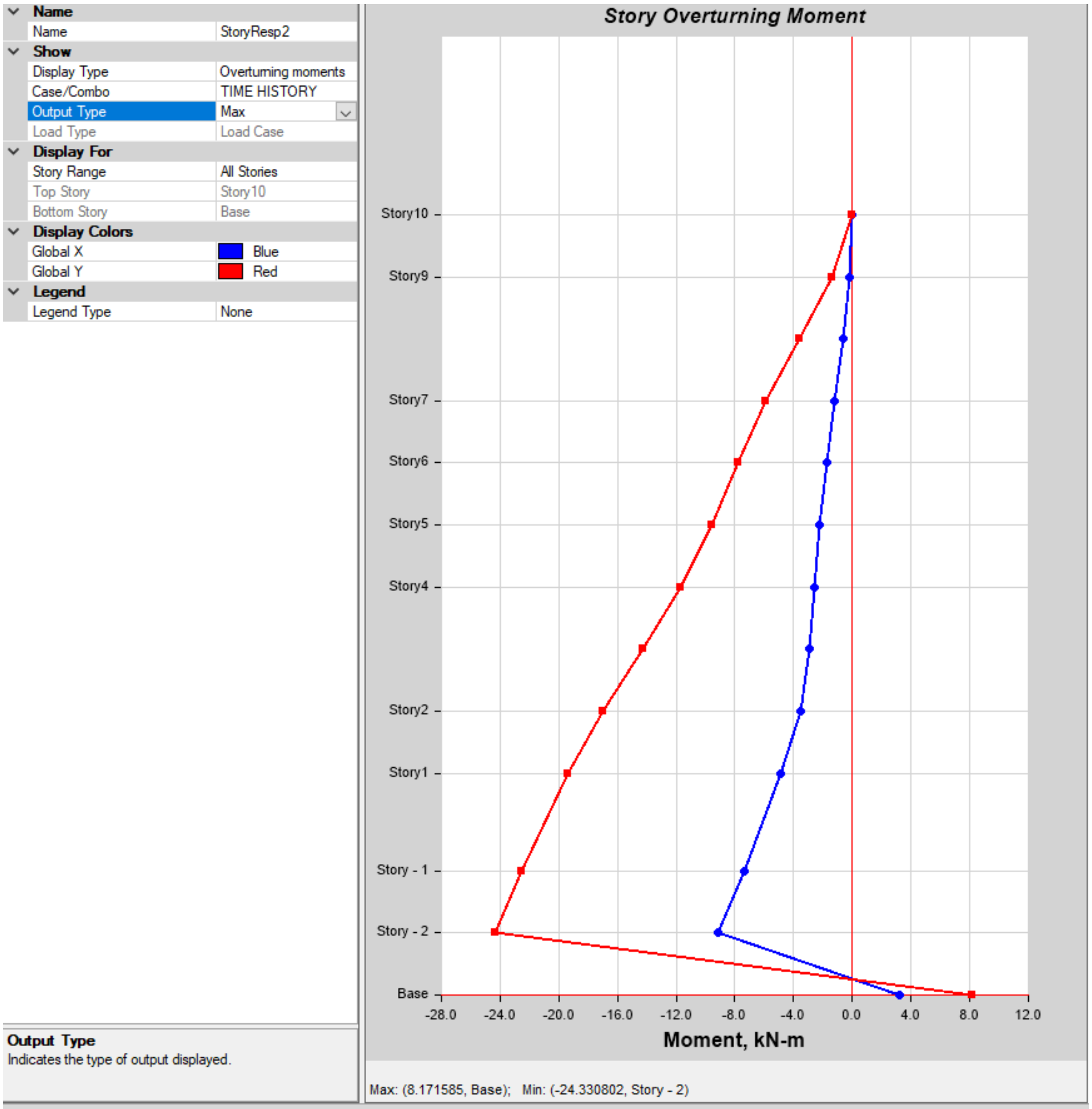


Figure 74 Story overturning moment for max time

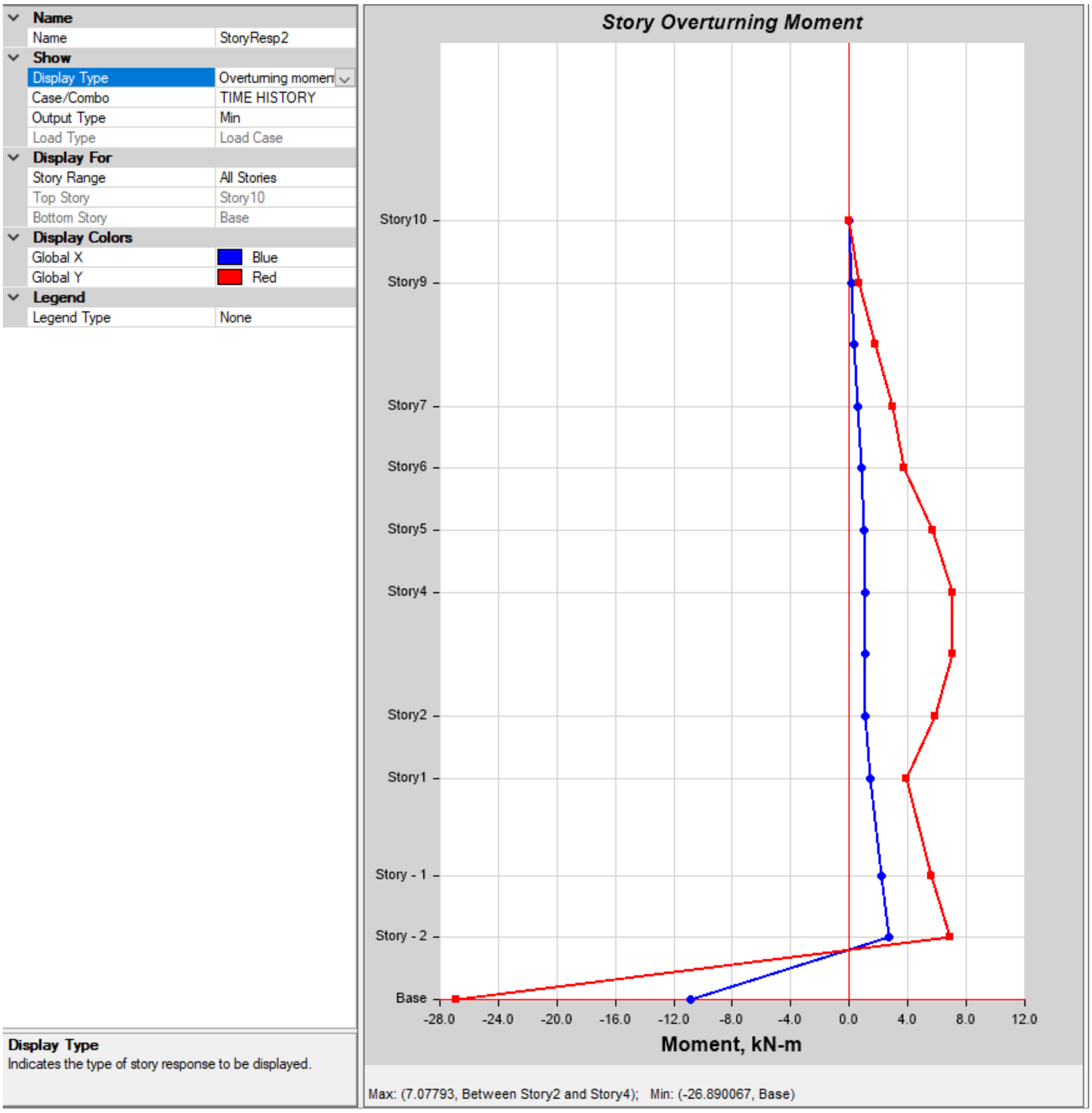


Figure 75 story overturning moment for min time

d) Internal forces

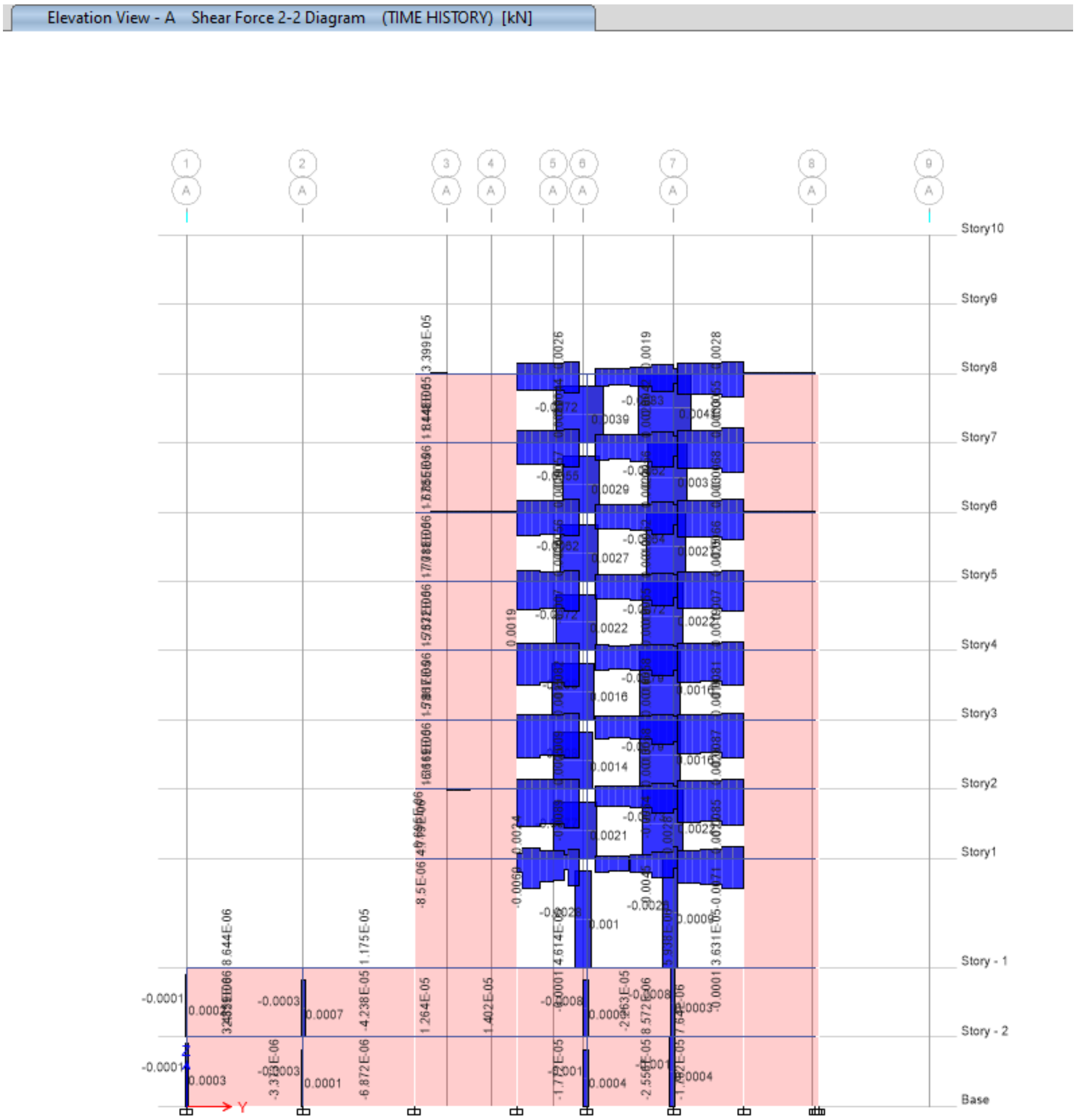


Figure 76 Shear force (shear 2-2) for frame A

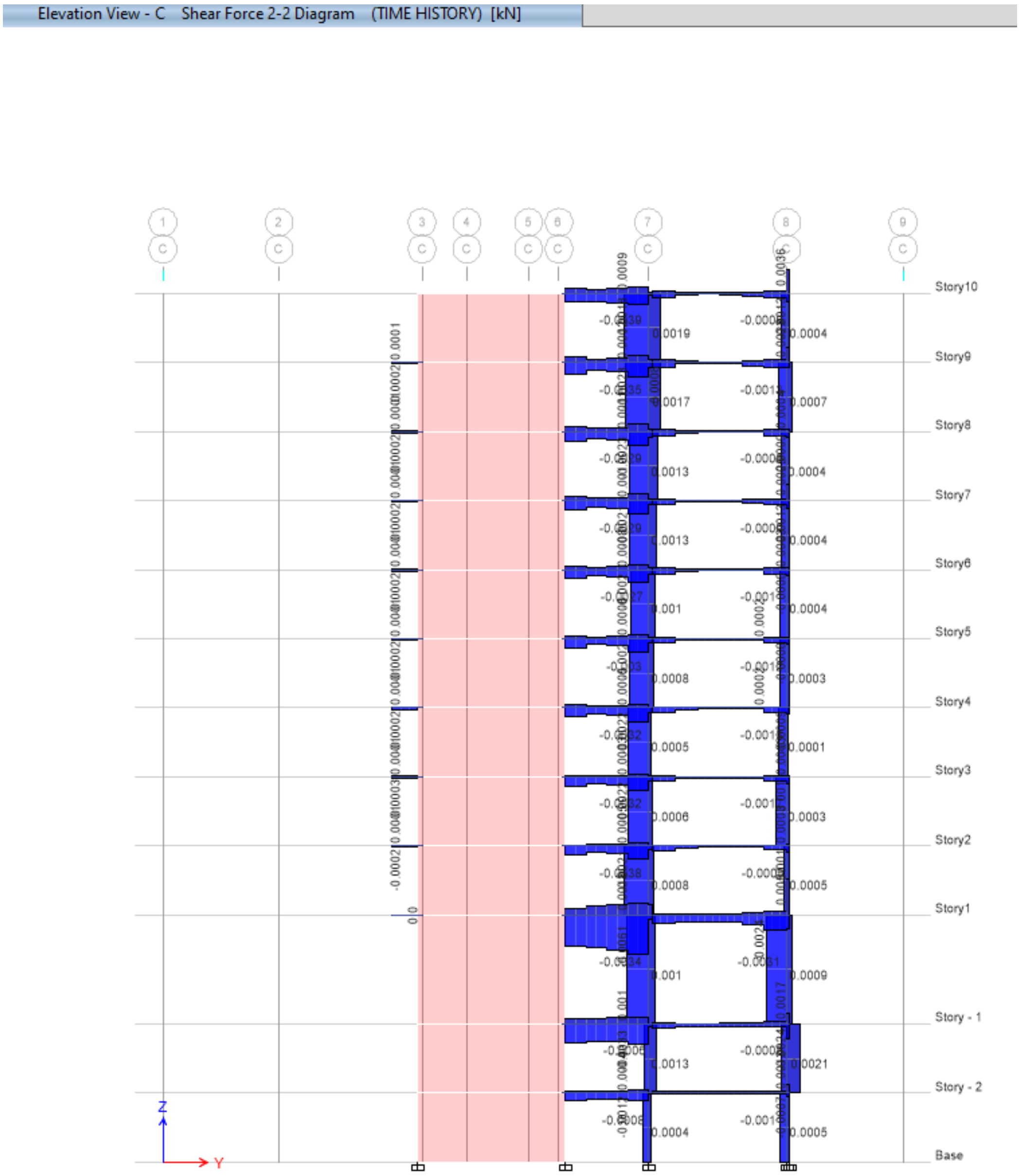


Figure 77 Shear force (shear 2-2) for frame C

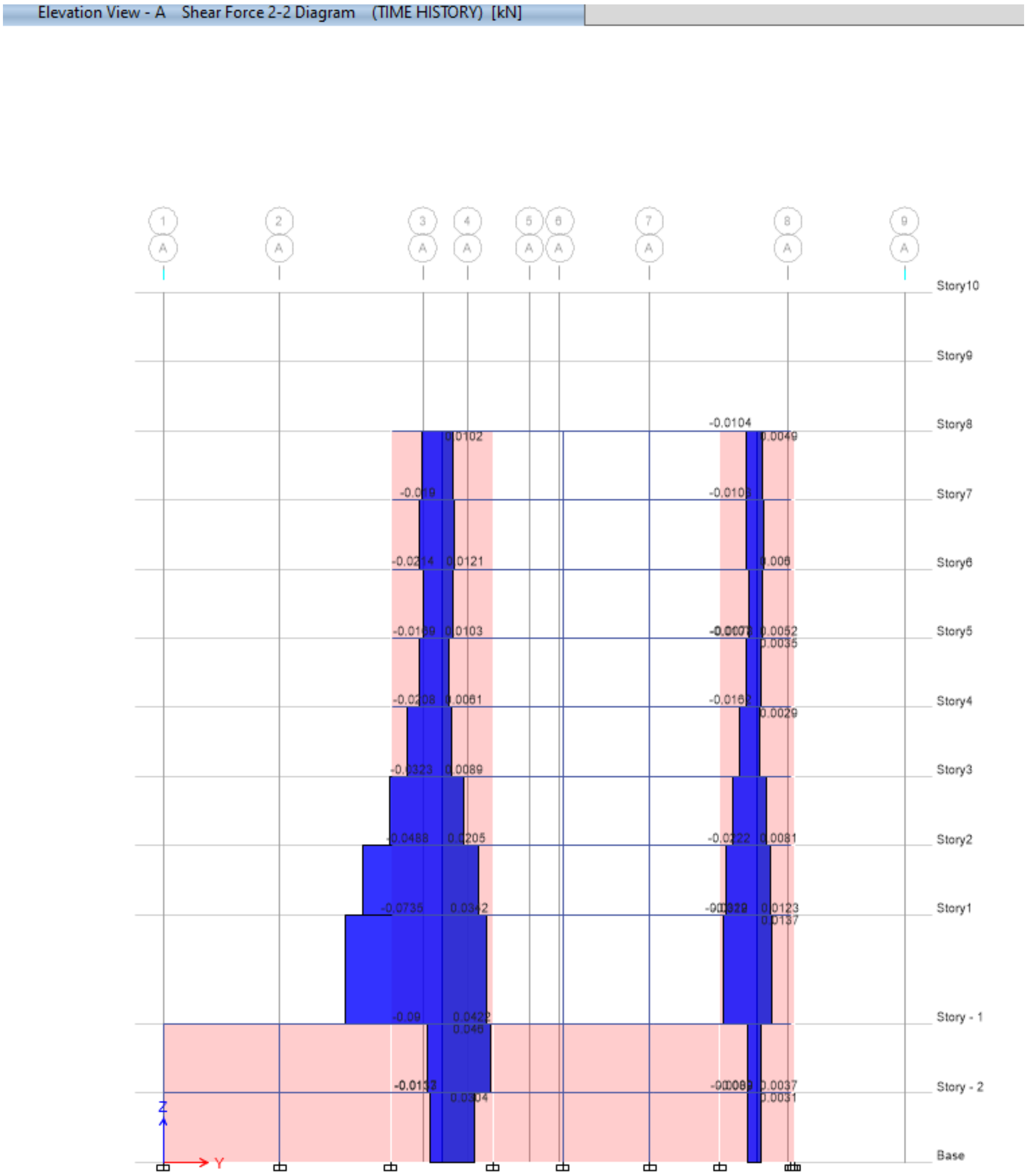


Figure 78 Shear force (shear 2-2) for pier A

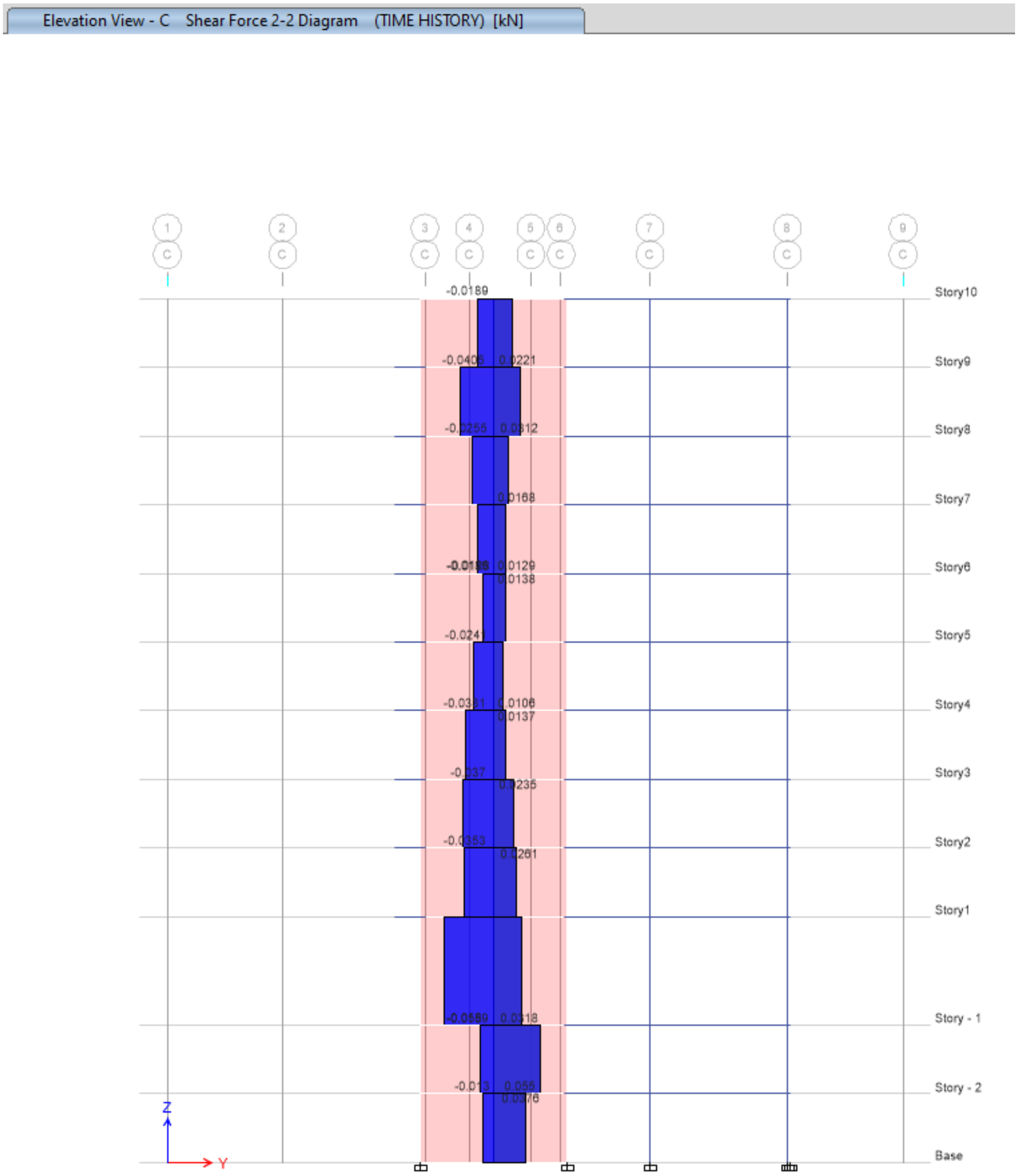


Figure 79 Shear force (shear 2-2) for pier C

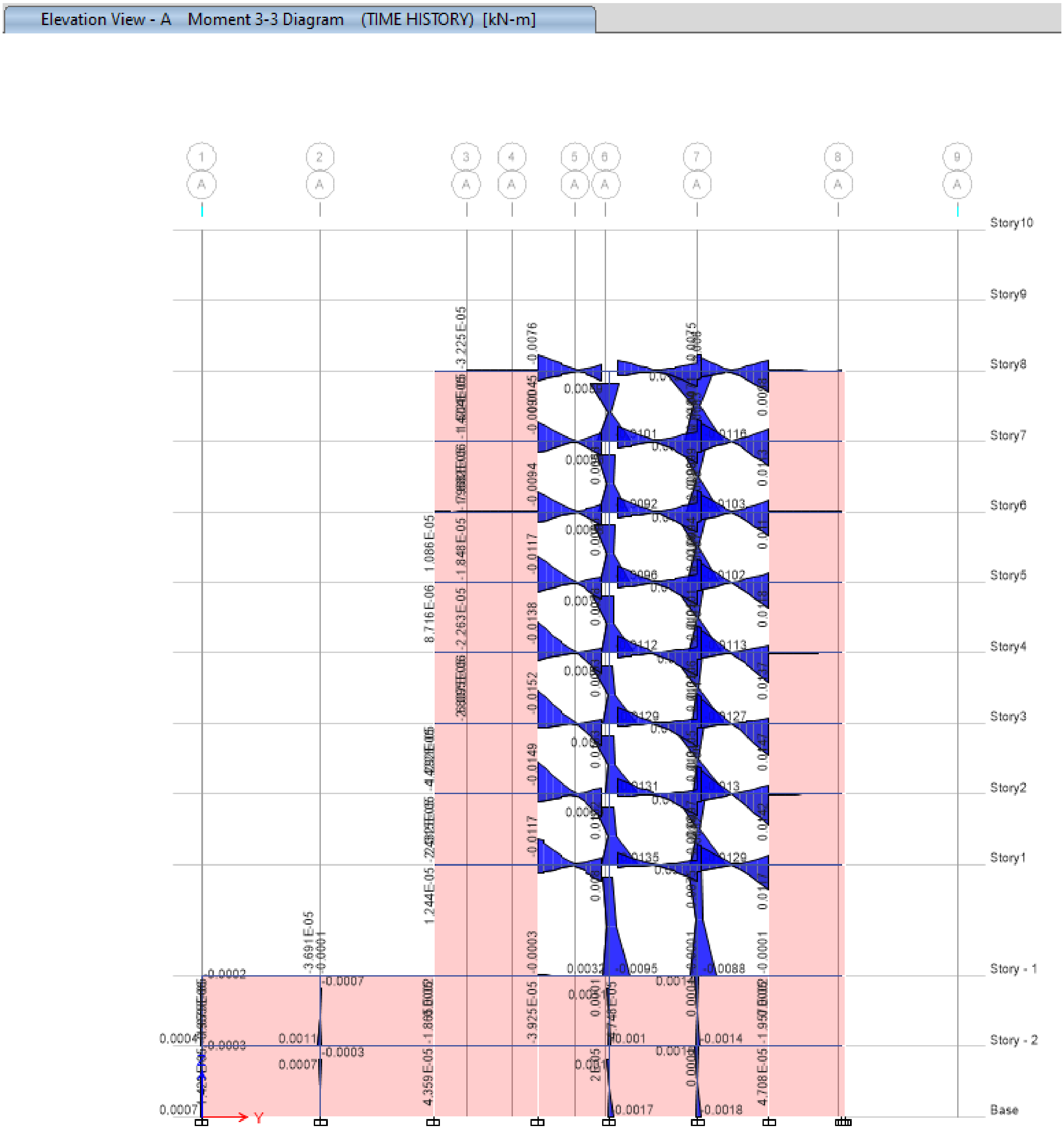


Figure 80 Moment (Moment 3-3) for frame A

\

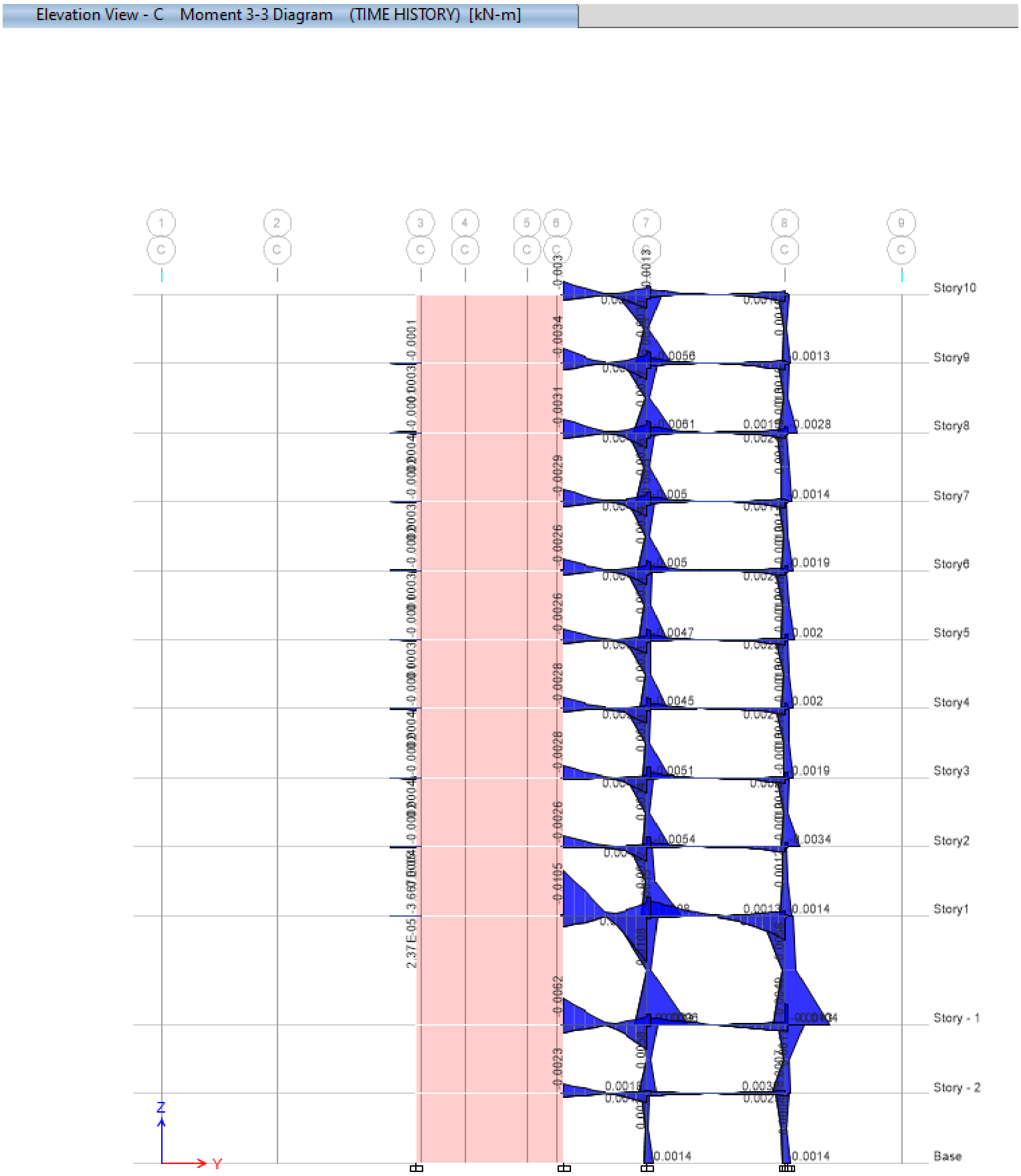


Figure 81 Moment (Moment 3-3) for frame C

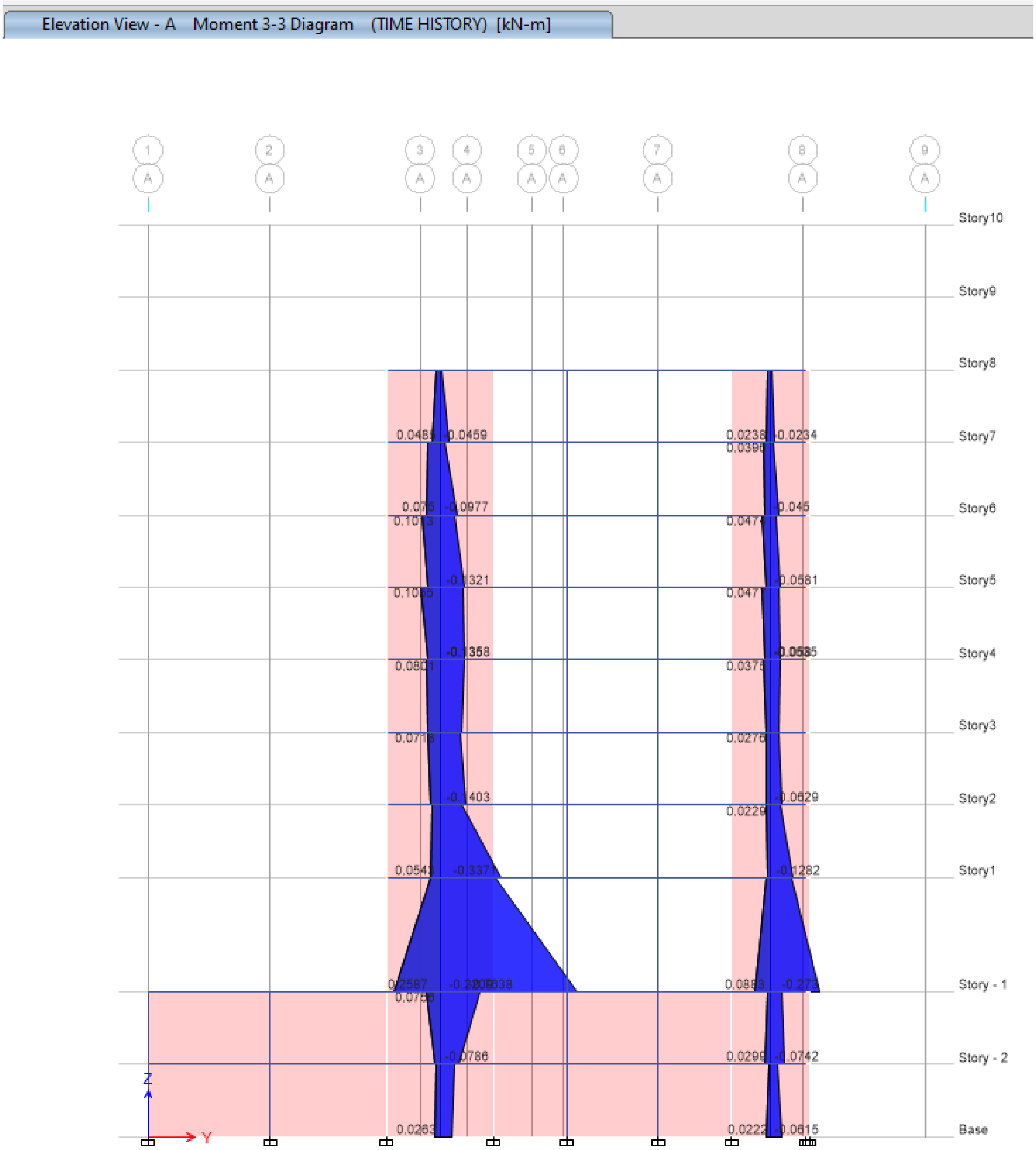


Figure 82 Moment (Moment 3-3) for Pier A

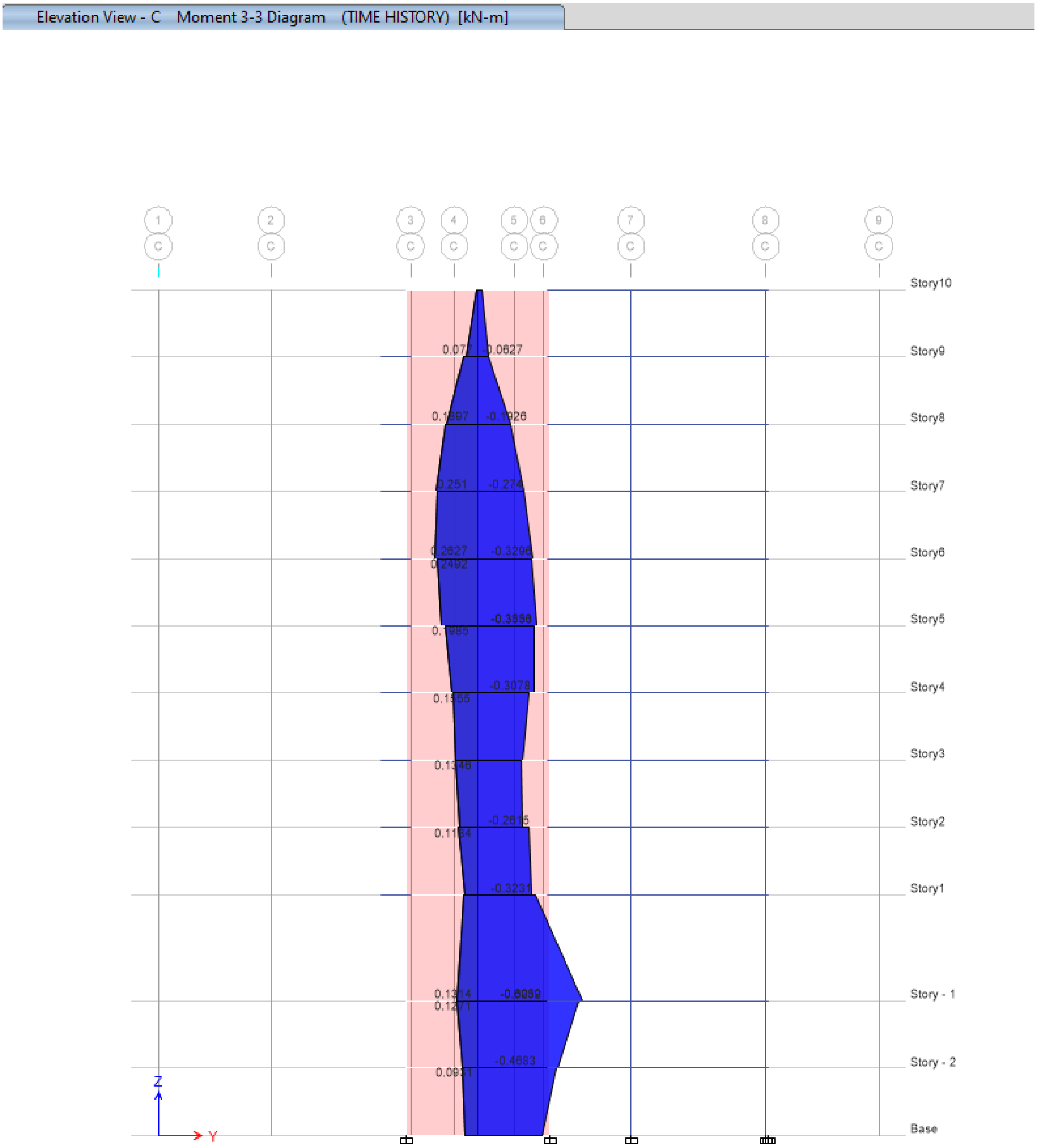


Figure 83 Moment (Moment 3-3) for Pier C

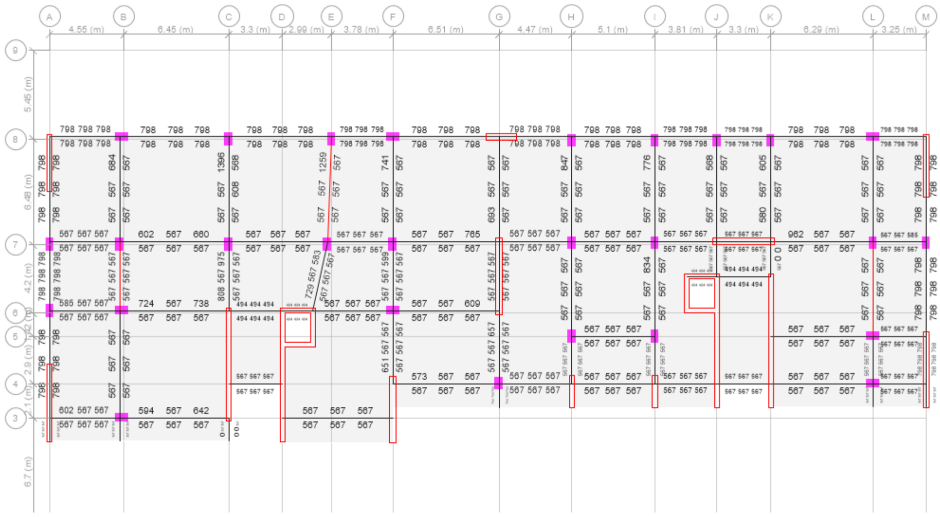


Figure 84 Concrete frame design ETABS output (Longitudinal reinforcement, plan view 7th storey level)

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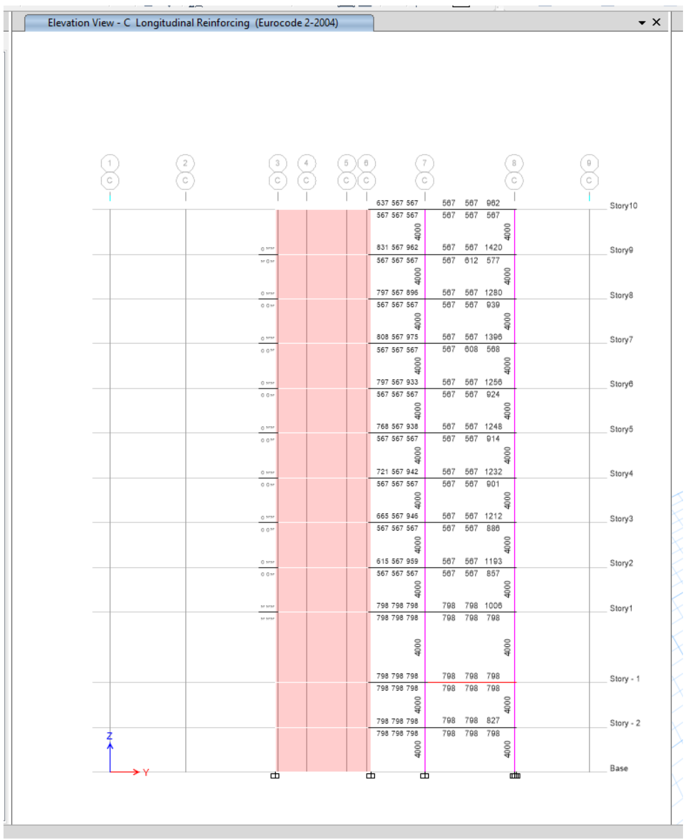

Figure 85 Concrete frame design ETABS output (Longitudinal reinforcement,Elevation view C-AXIS

APPENDIX B

a) Slab Details (Storey 1) with CSI Detailing

SR. NO.	ITEM	QUANTITY	UNIT
1	TOTAL AREA, A	4,282.48	SQ M
2	TOTAL VOLUME, V	856.495	CU M
3	AVERAGE THICKNESS, T=V/A	200	MM
4	TOTAL REBARS WEIGHT, W	16,331	KG
5	REBARS PER AREA, W/A	3.813	KG/SQ M
6	REBARS RATIO, W/V	19.0669	KG/CU M

Figure 86 Bill of materials: Floor slab

SR. NO.	BAR SIZE	LENGTH (M)	WEIGHT (KG)
1	10	2,608.1	1,607
2	12	4,237.4	3,762
3	14	2,160.6	2,610
4	16	2,926.8	4,620
5	18	1,868.4	3,732

Figure 88 Bill of quantities: Floor slab

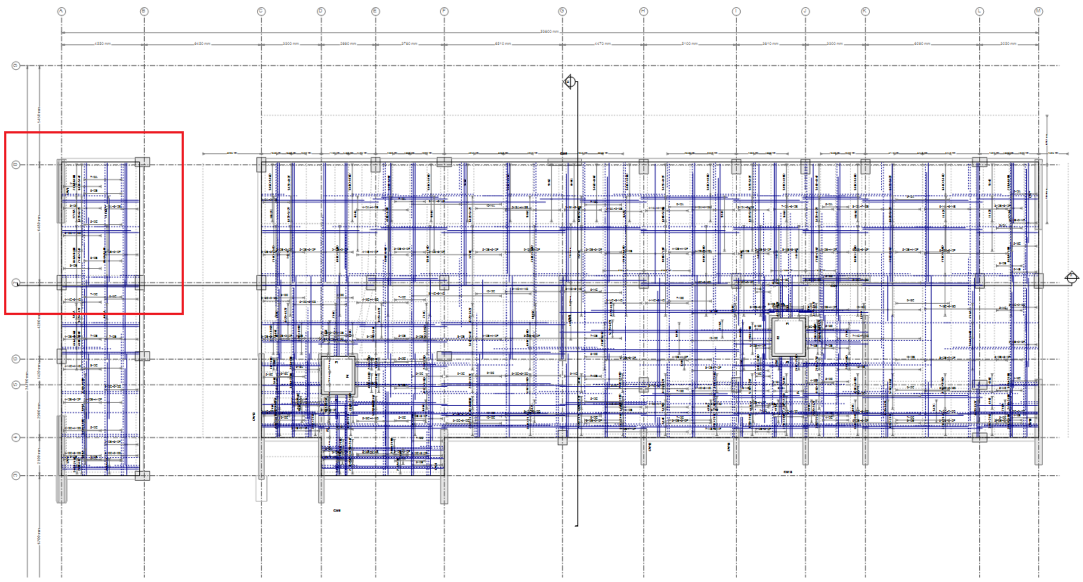

Figure 87 Rebar plan (top and bottom)

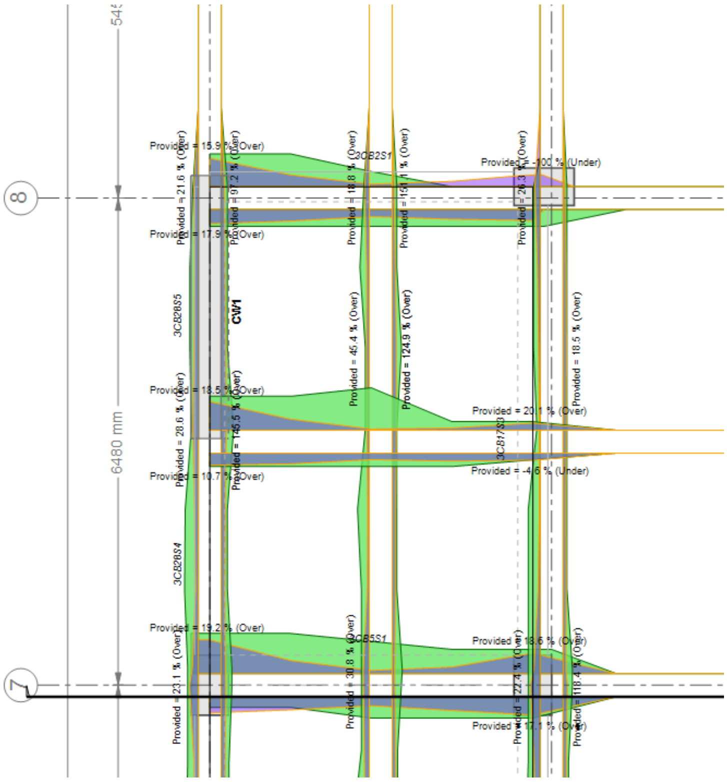

Figure 89 Reinforcement profile of the slab

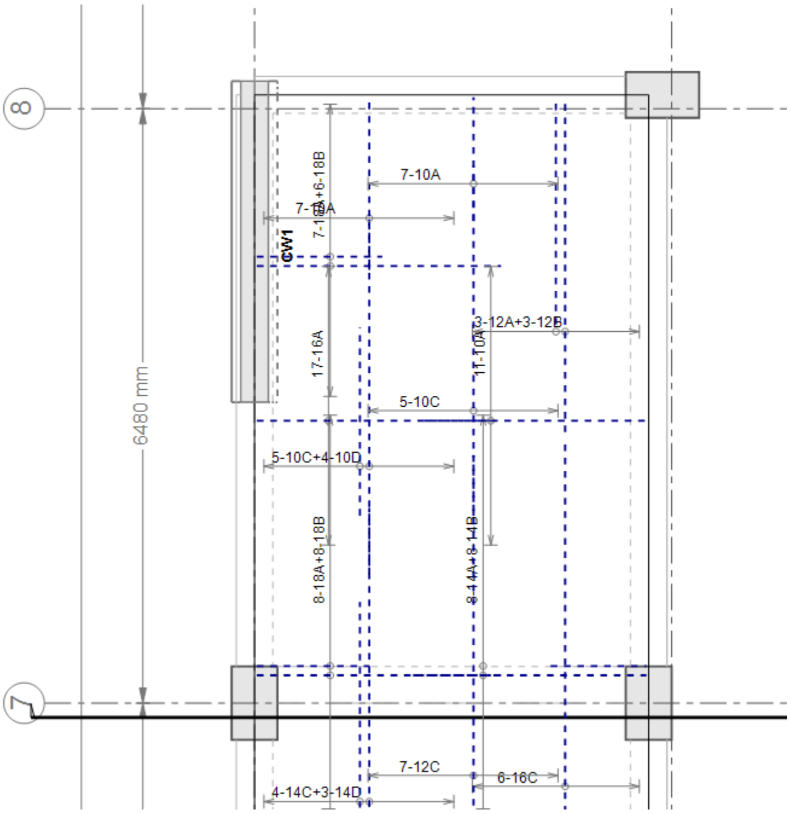

Figure 90 Top rebar plan (A-B-7-8 area)

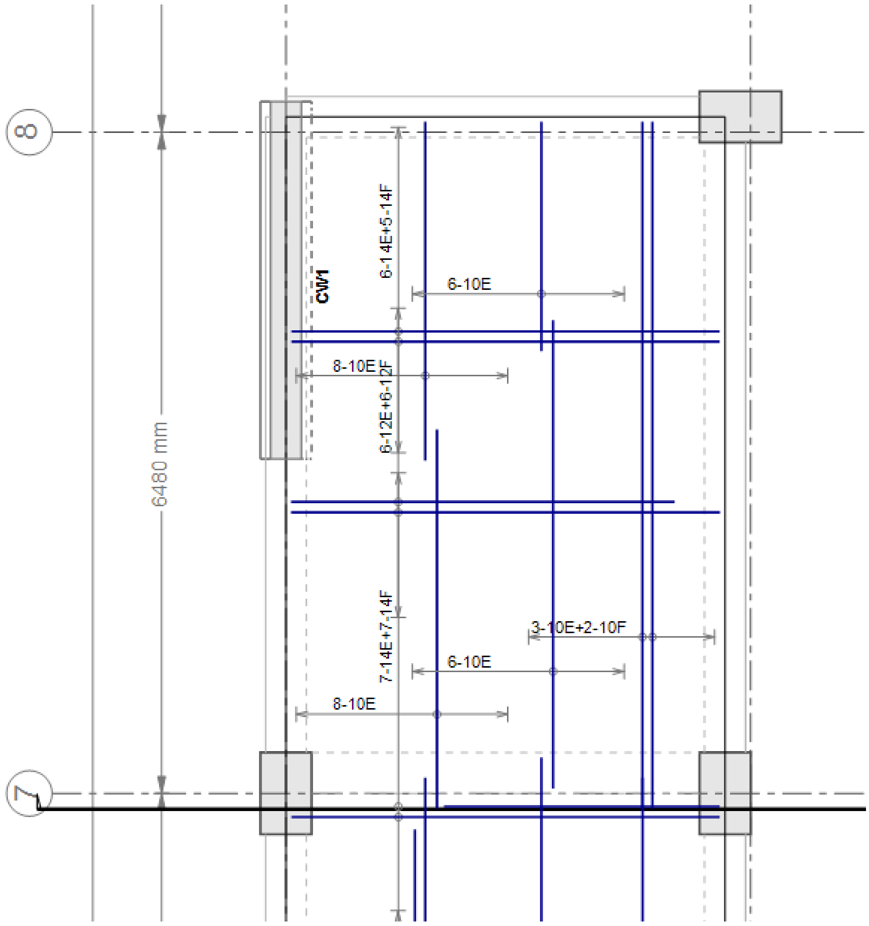

Figure 91 Bottom rebar plan (A-B-7-8 area)

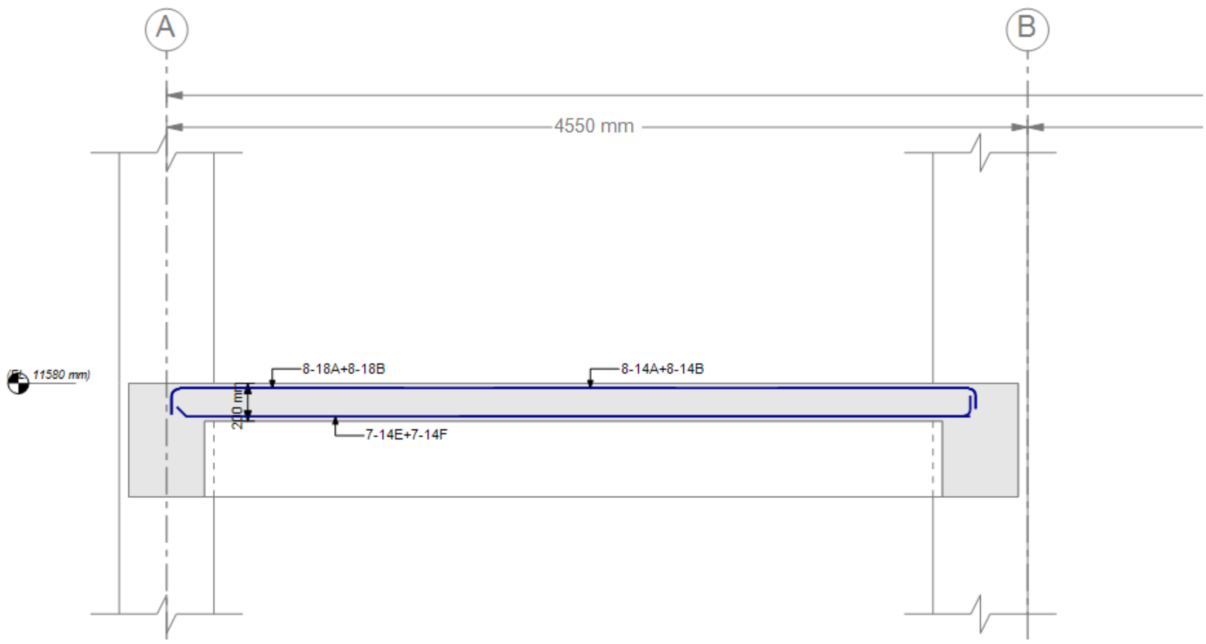

Figure 92 Slab detail storey 1 slab detail(A-B axes)

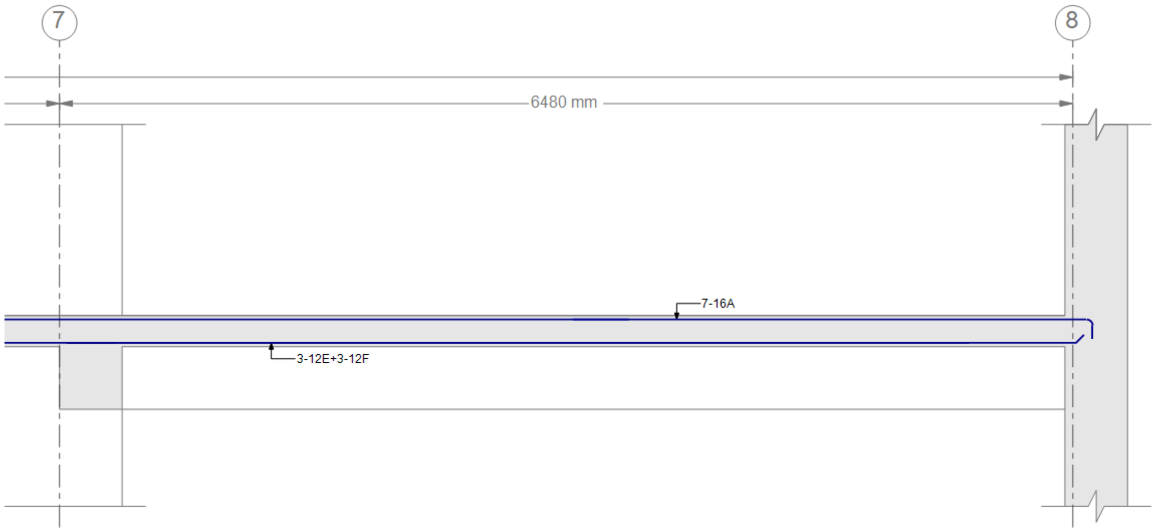

Figure 93 Slab detail storey 1 (7-8 axes)

b) Beam 3CB2 details (Storey 1)

SR. NO.	ITEM	QUANTITY	UNIT
1	CONCRETE VOLUME, V	1.068	CU M
2	REBARS WEIGHT, W	177	KG
3	REBARS RATIO, W/V	165.3677	KG/CU M

Figure 94 Bill of materials for beam 3CB2

SR. NO.	BAR SIZE	LENGTH (M)	WEIGHT (KG)
1	10	49.2	30
2	20	59.3	146

Figure 95 Bill of quantities for beam 3CB2

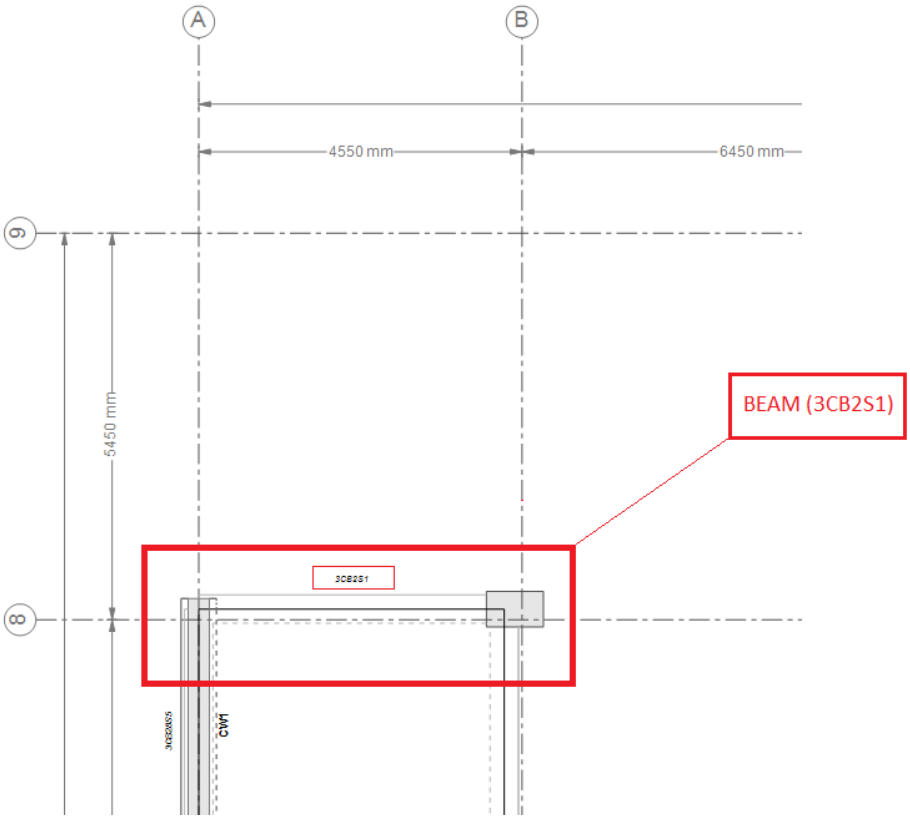

Figure 96 3CB2 beam location

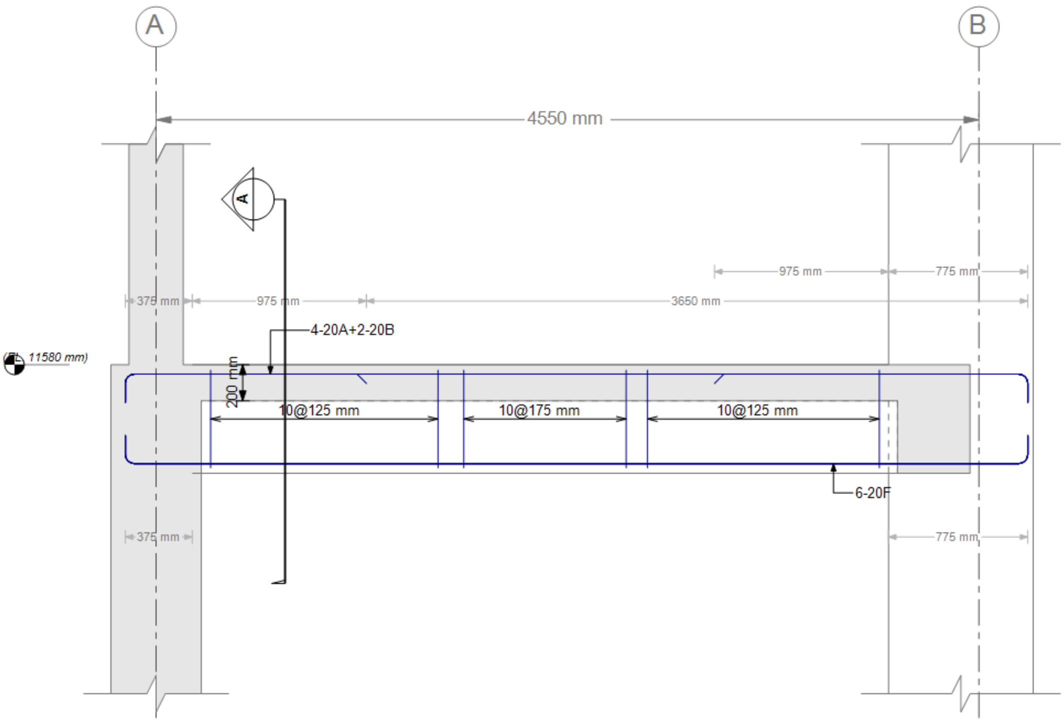

Figure 97 3CB2 beam design

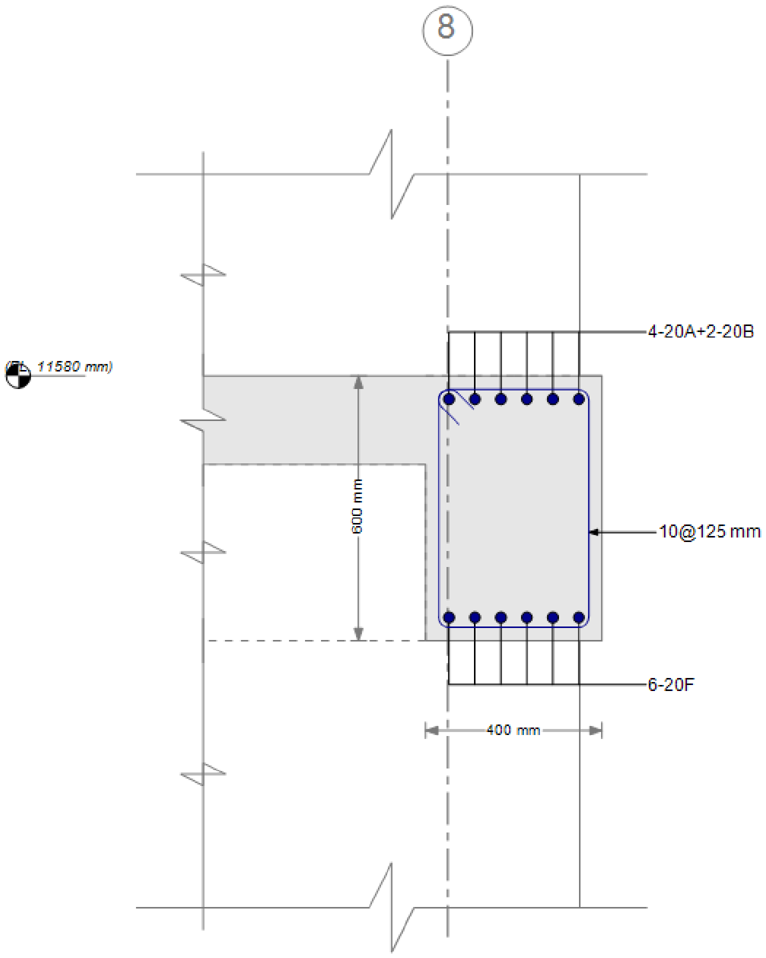

Figure 98 Section A of CB2 beam

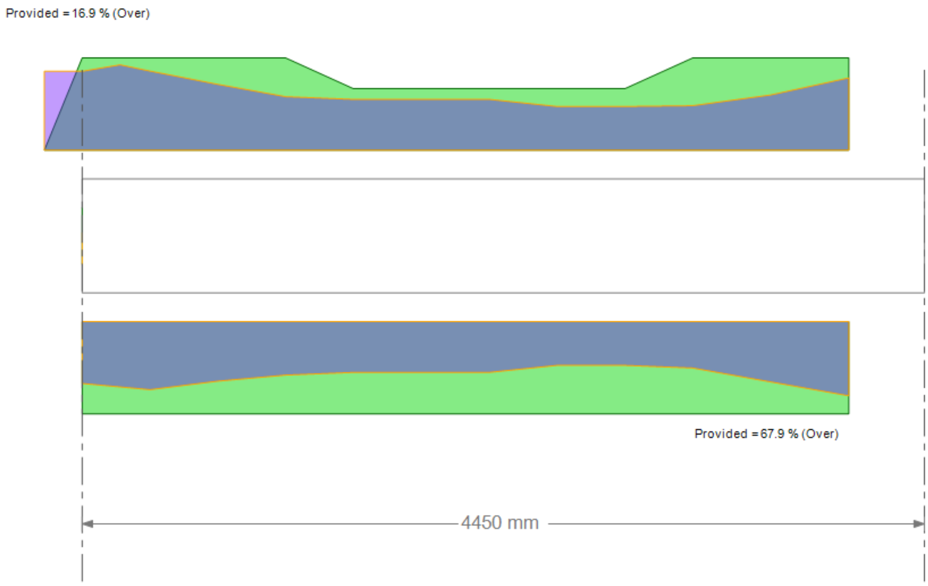

Figure 99 CB2 beam reinforcement profile

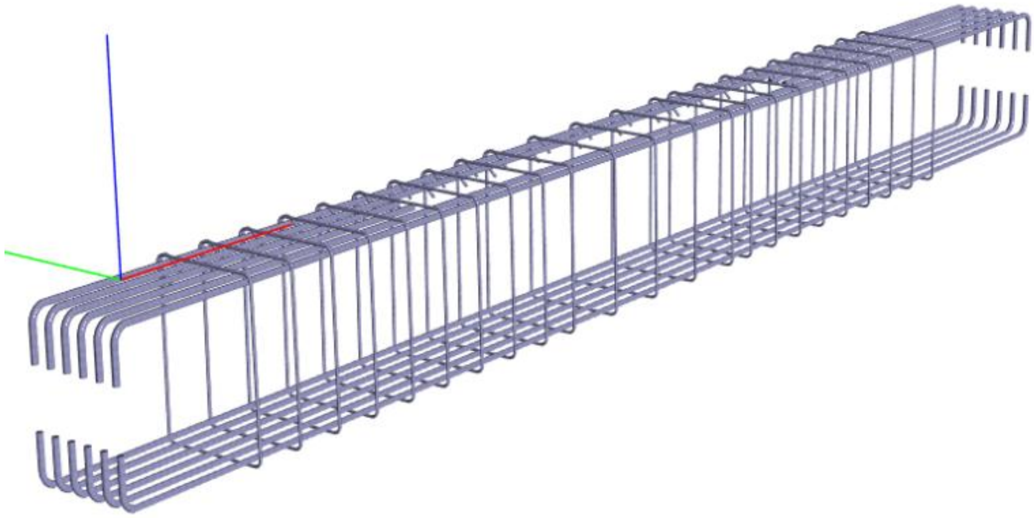

Figure 100 Rebar cage of CB2 beam

c) Beam 3CB1 details (Storey 1)

SR. NO.	ITEM	QUANTITY	UNIT
1	CONCRETE VOLUME, V	10.272	CU M
2	REBARS WEIGHT, W	1,718	KG
3	REBARS RATIO, W/V	167.2488	KG/CU M

Figure 101 Figure 102 Bill of materials for beam CB1

SR. NO.	ITEM	QUANTITY	UNIT
1	CONCRETE VOLUME, V	10.272	CU M
2	REBARS WEIGHT, W	1,718	KG
3	REBARS RATIO, W/V	167.2488	KG/CU M

Figure 103 Figure 104 Bill of quantities for beam CB1

SR. NO.	BAR SIZE	LENGTH (M)	WEIGHT (KG)
1	10	484.6	299
2	16	4.1	6
3	20	572.9	1,413

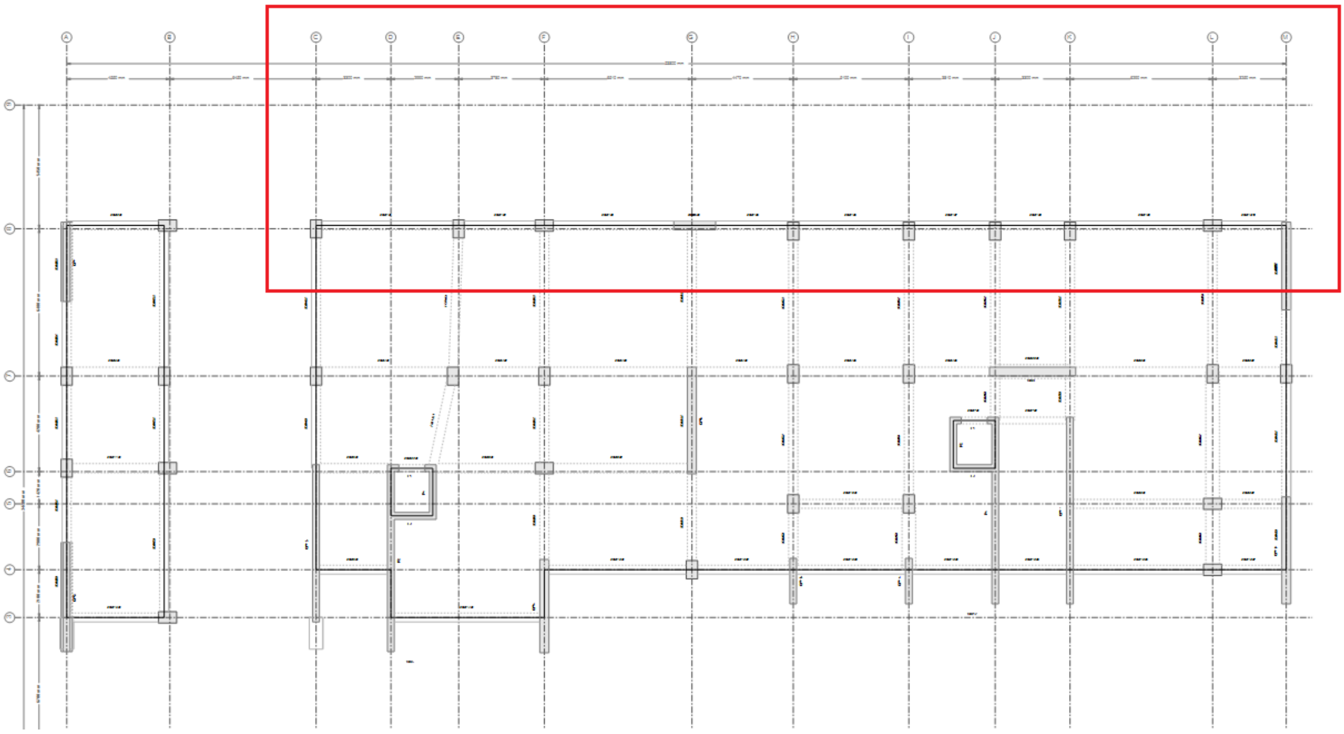

Figure 107 Beam CB1 specification in the case study planimetry

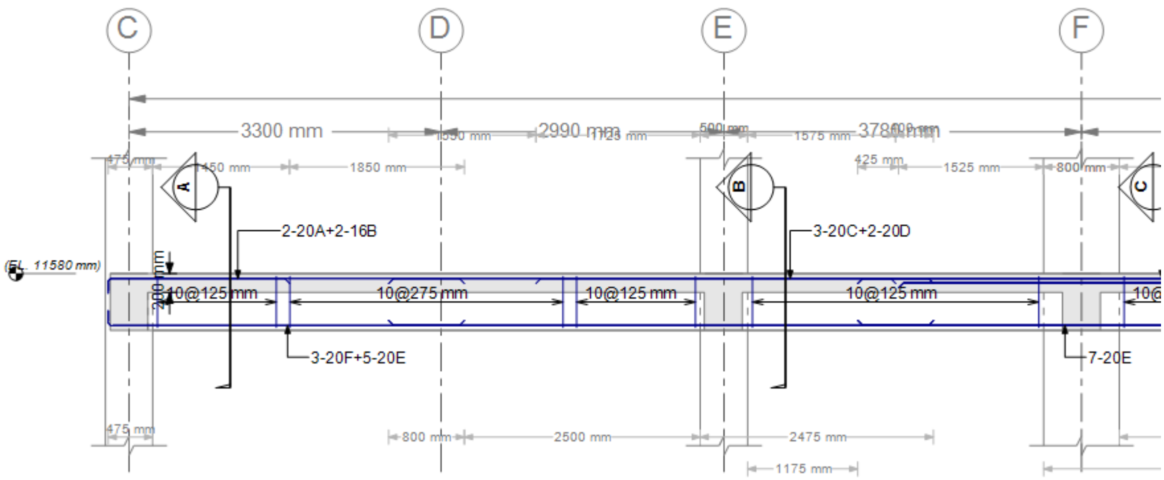

Figure 105 3CB1 beam design

Figure 106 Rebar quantities for beam CB1

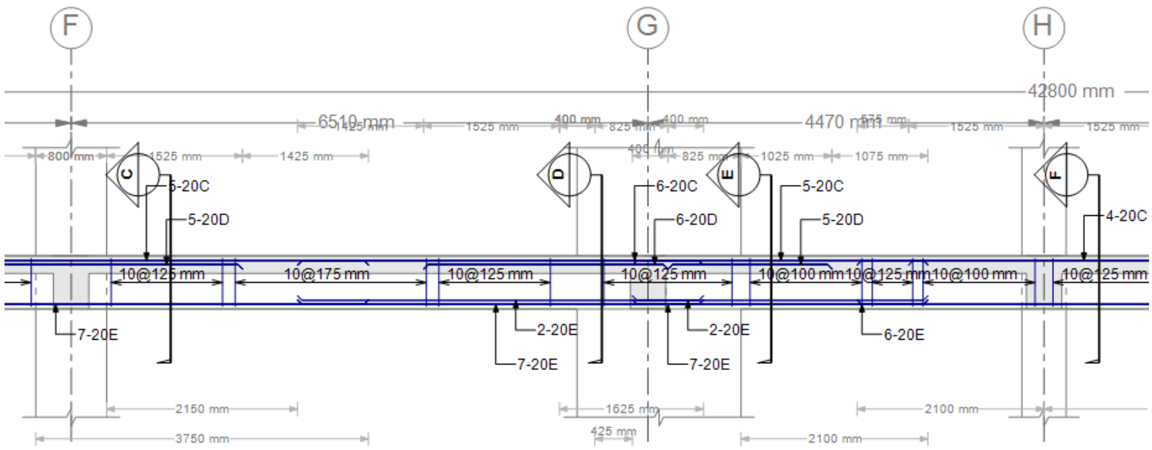

Figure 108 3CB1 beam design

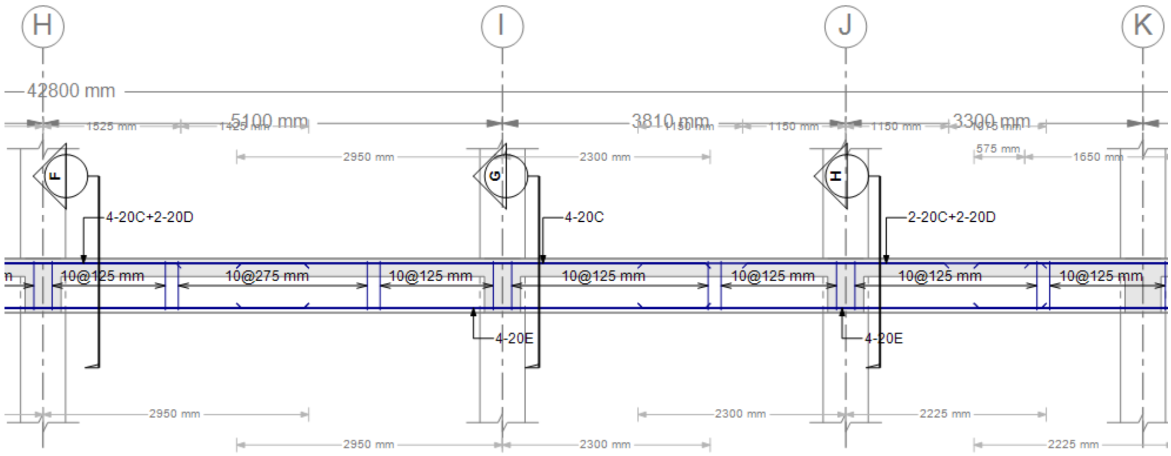

Figure 109 3CB1 beam design

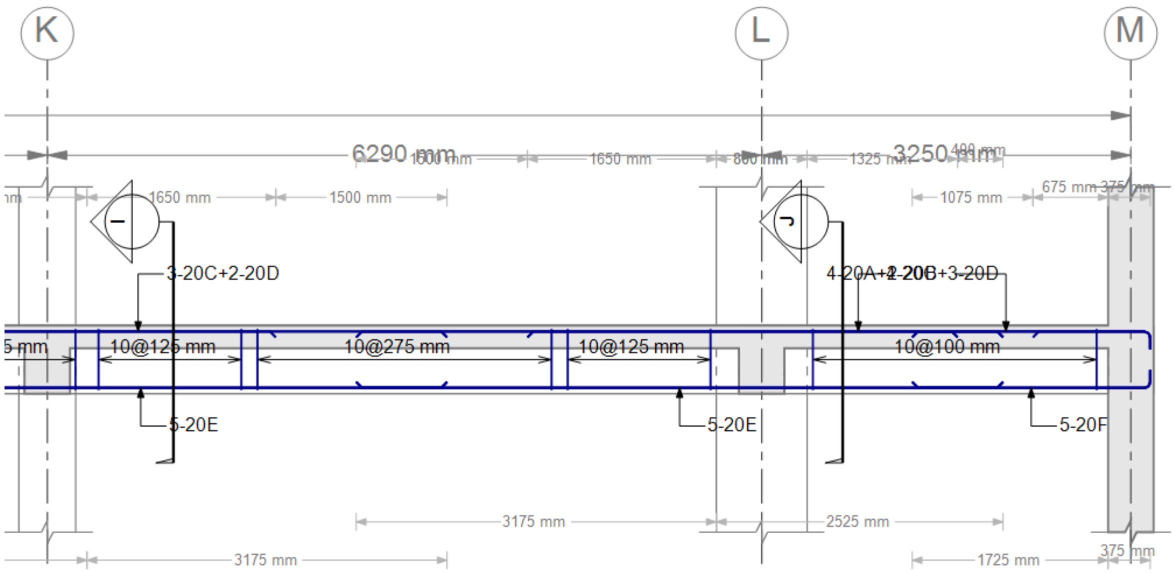

Figure 110 3CB1 beam design

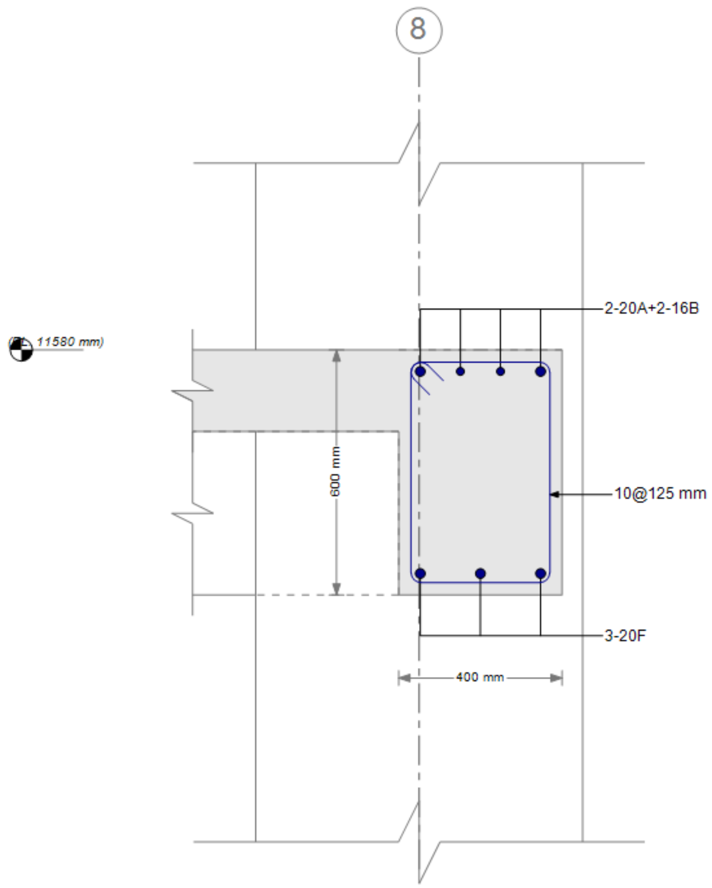

Figure 111 Section A CB1 Beam design

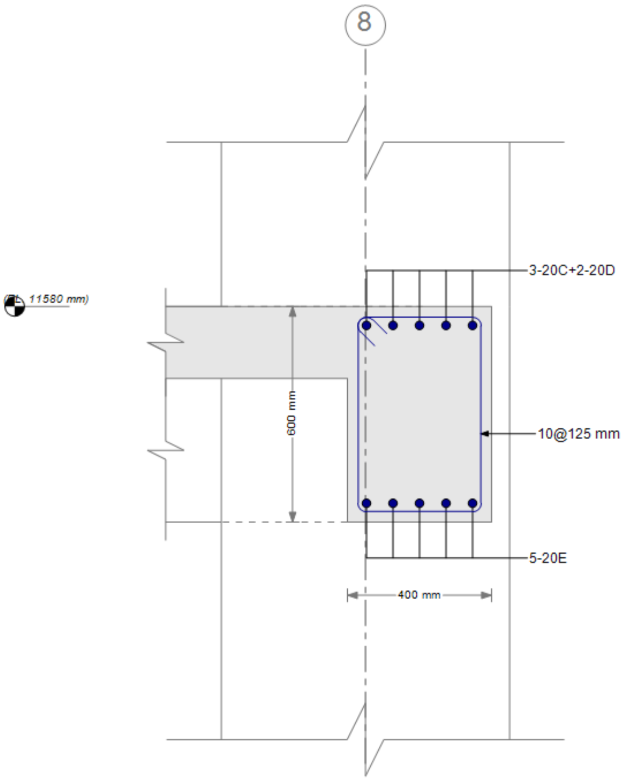

Figure 112 Section B CB1 Beam design

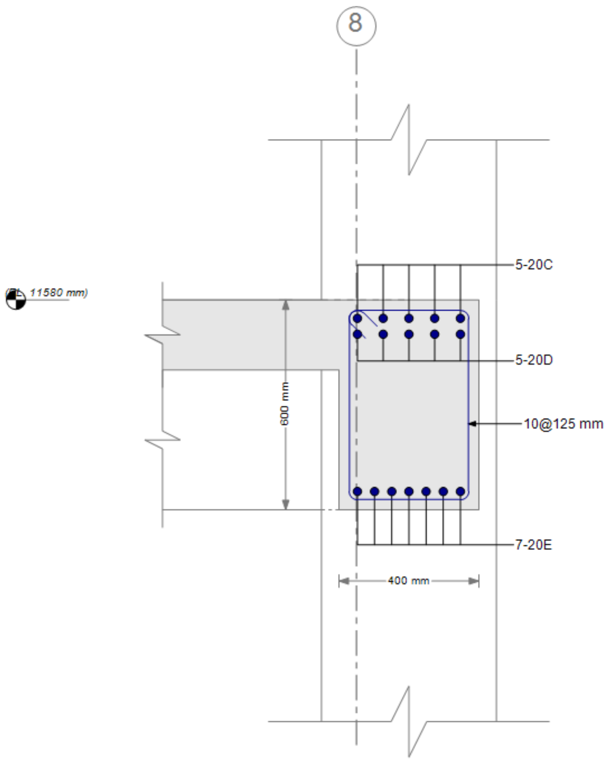

Figure 113 Section C CB1 Beam design

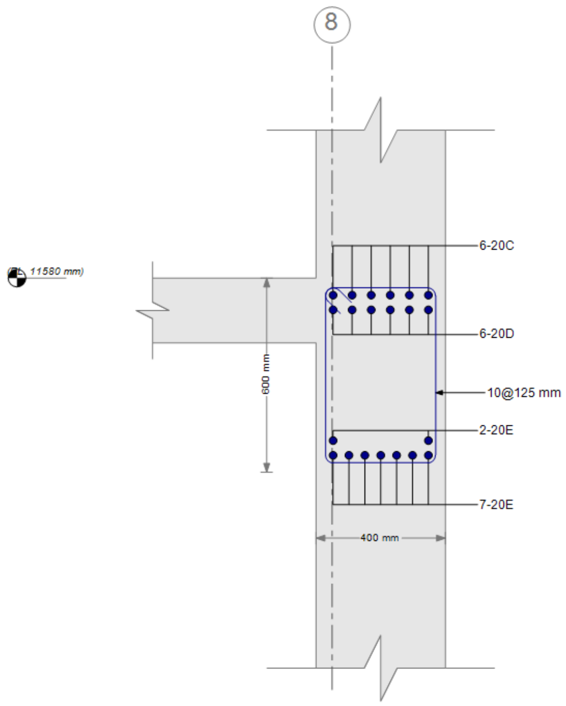

Figure 114 Section D CB1 Beam design

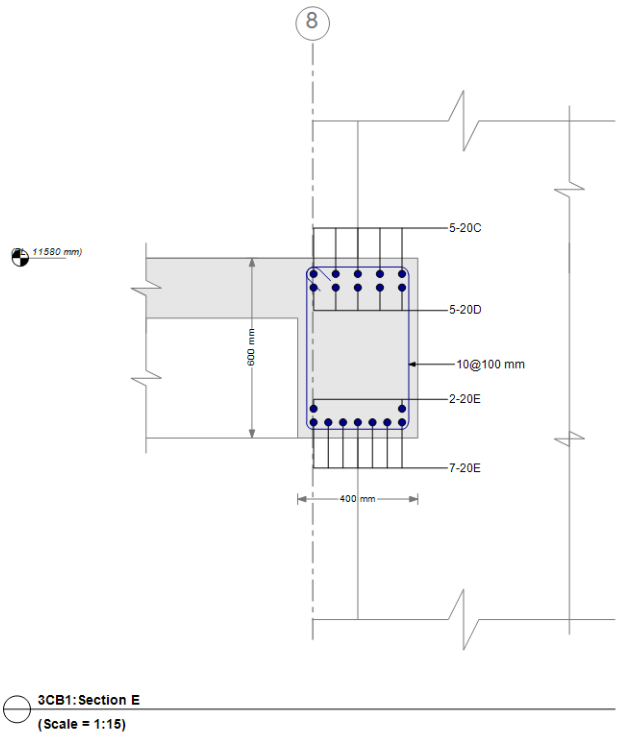

Figure 115 Section D CB1 Beam design

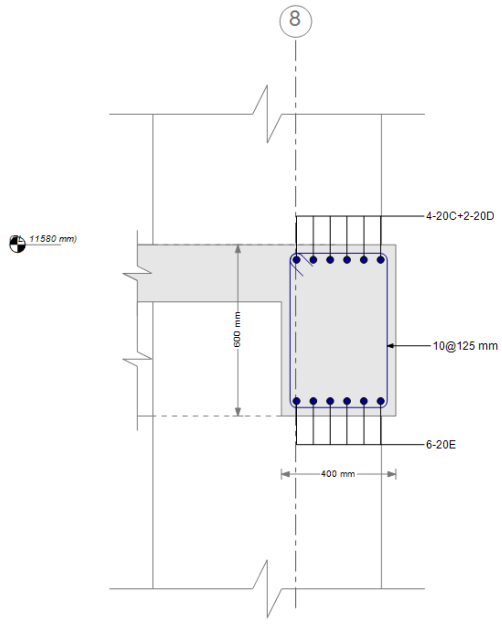

Figure 116 Section F CB1 Beam design

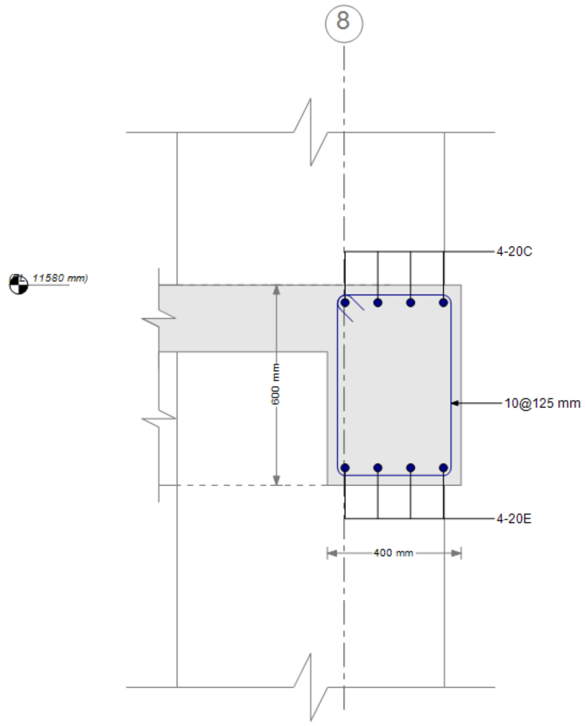

Figure 117 Section G CB1 Beam design

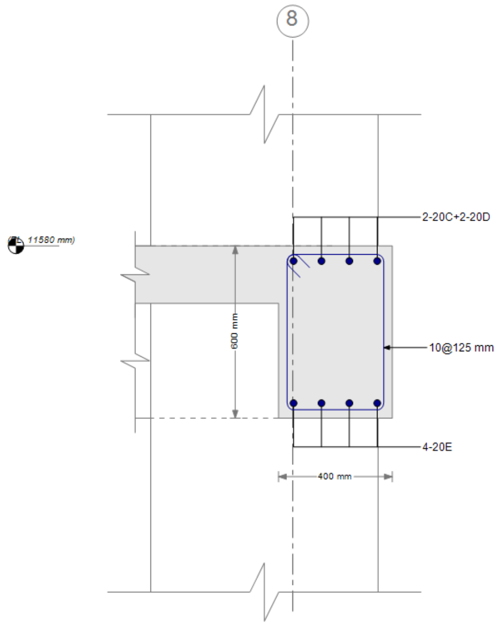

Figure 118 *Section H CB1 Beam design*

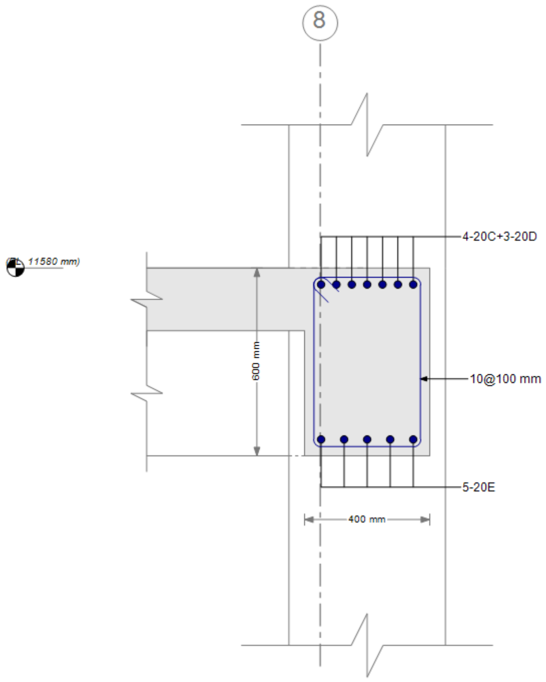

Figure 119 Section J CB1 Beam design

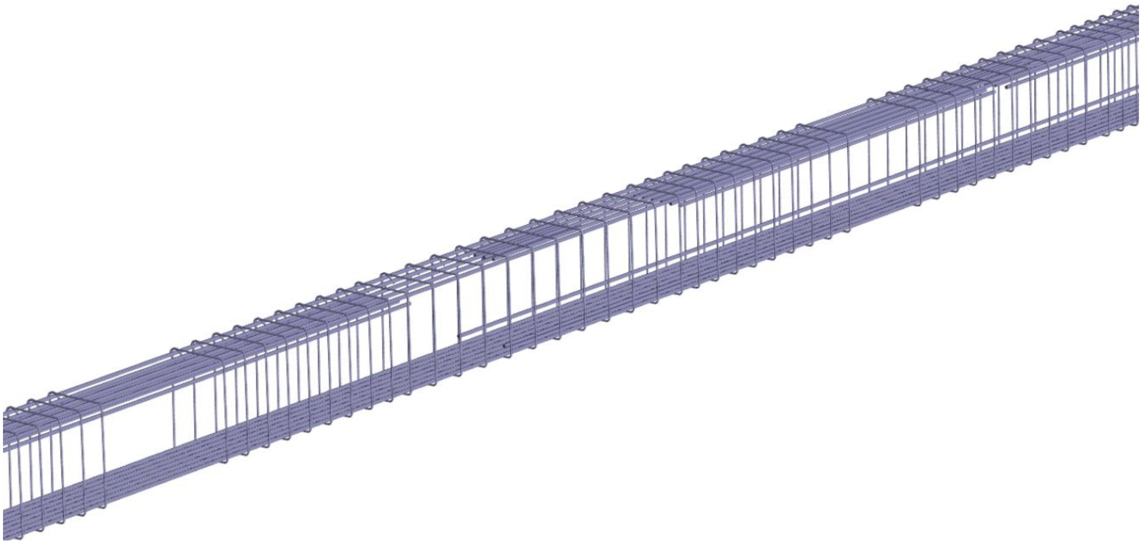

Figure 120 CB1 Beam rebar cage

c) Column Details

- **CC1 Column**

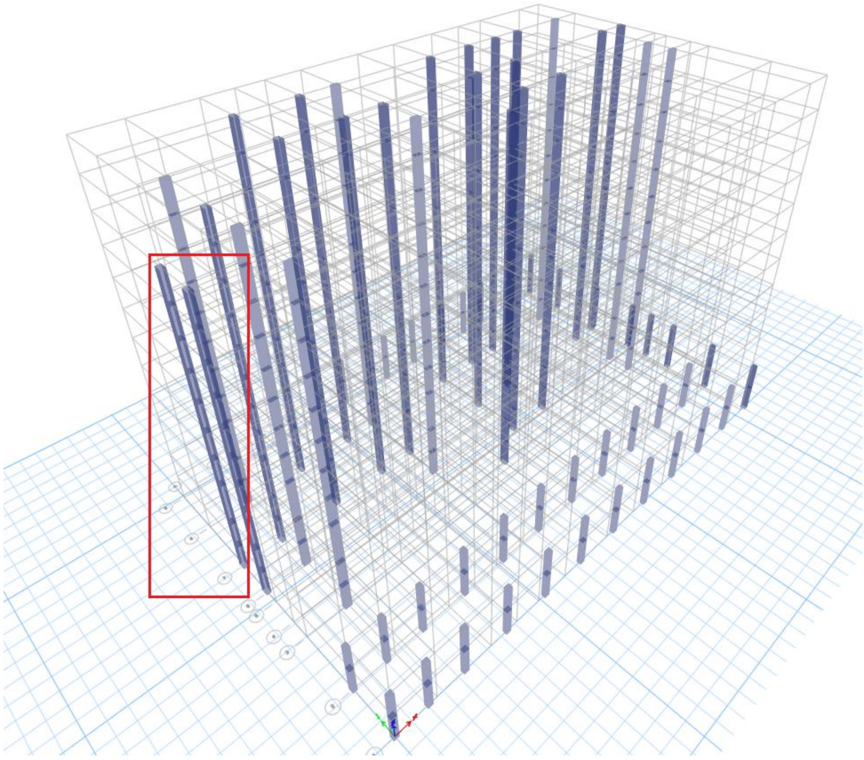

Figure 121 CC1 Column

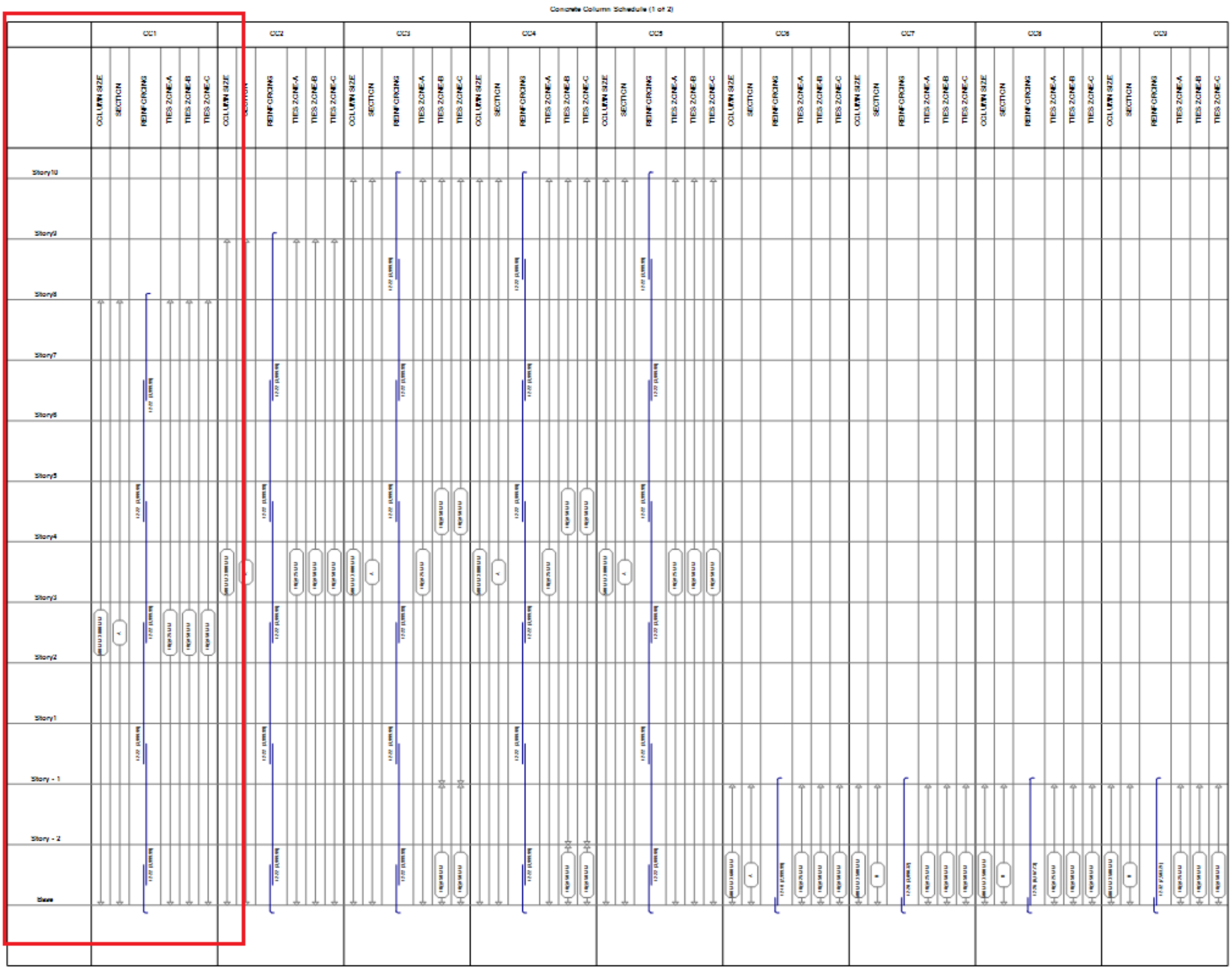

Figure 122 Column schedule (CC1 specified)

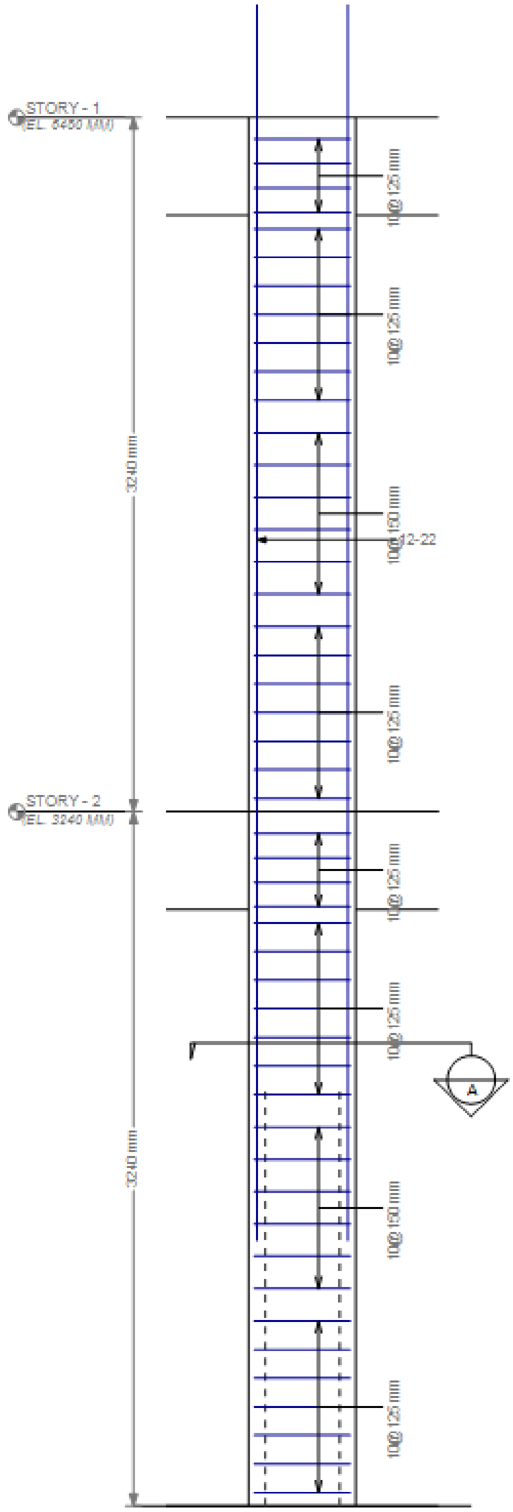

Figure 123 CC1 Column design (Elevation 1)

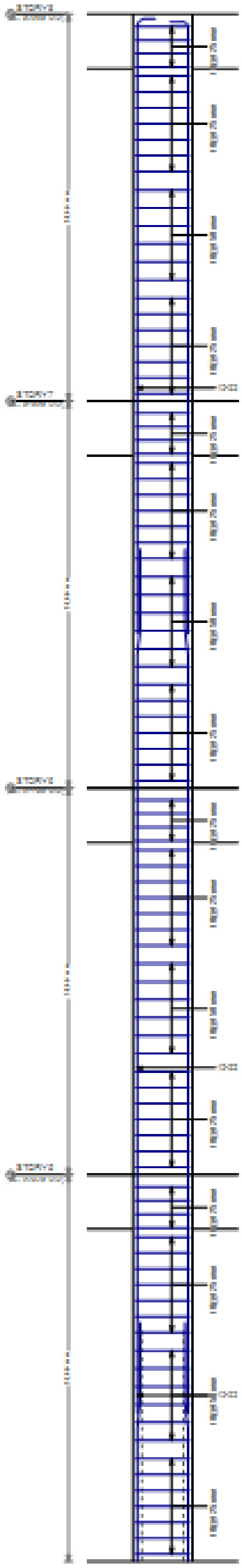

Figure 124 CC1 Column design (Elevation 2)

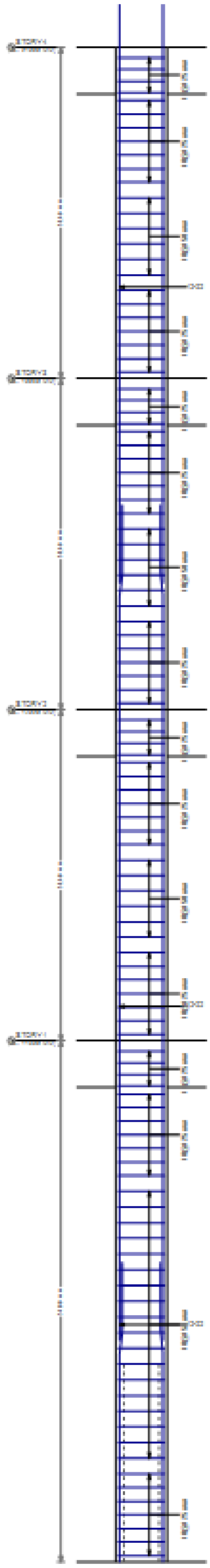

Figure 125 CC1 Column design (Elevation 3)

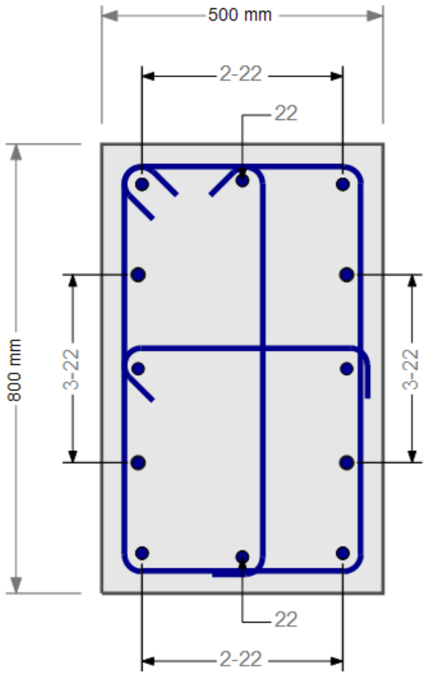

Figure 126 Section A of CC1 Column detail

- Underground column design (CC7 Column)

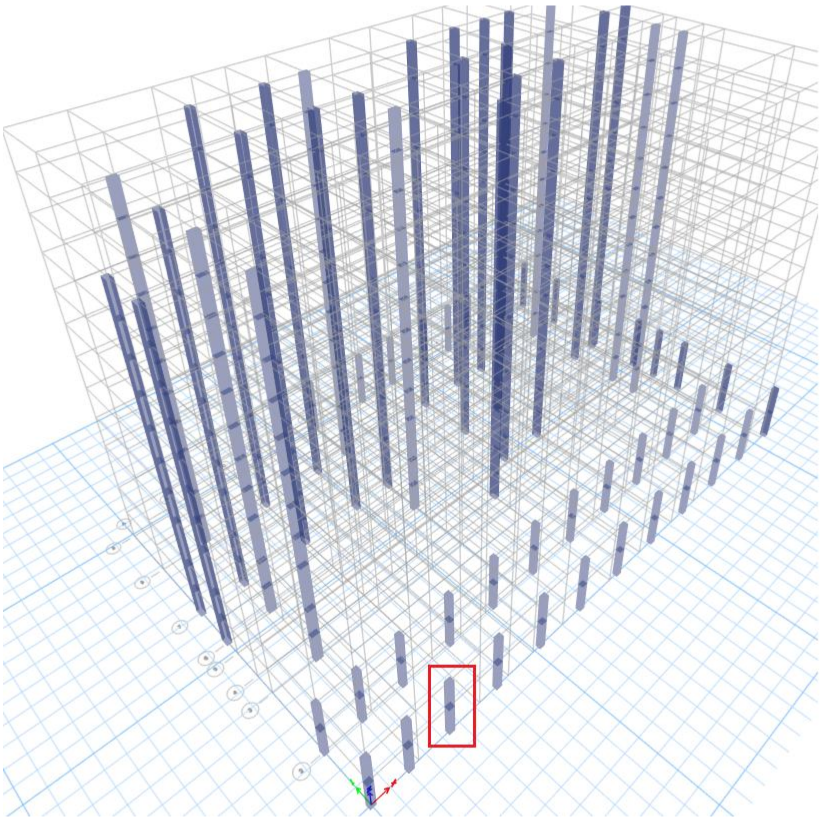

Figure 127 CC7 Column (Underground stories)

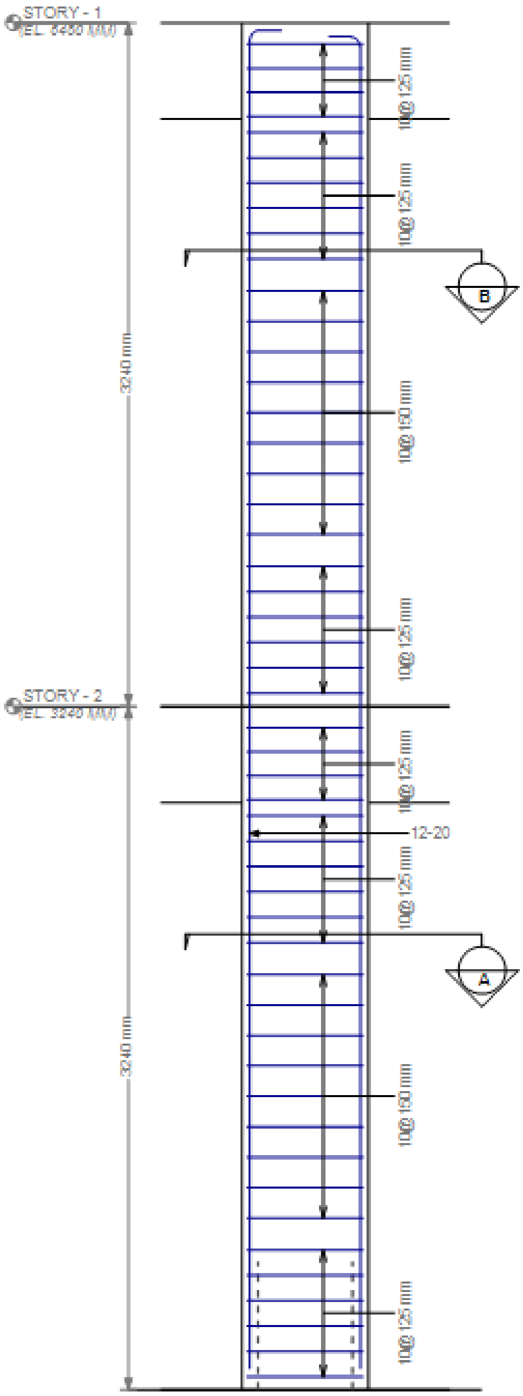

Figure 128 CC7 Column design

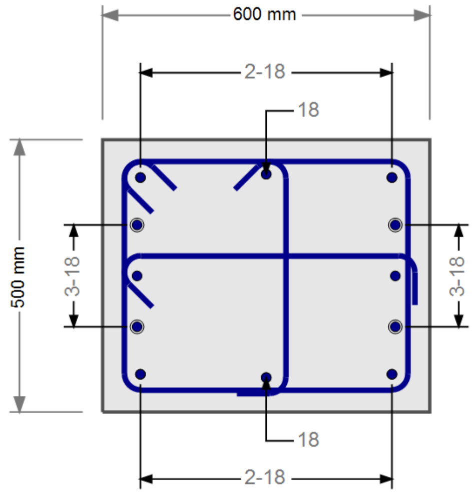

Figure 129 Section A of CC1 Column detail

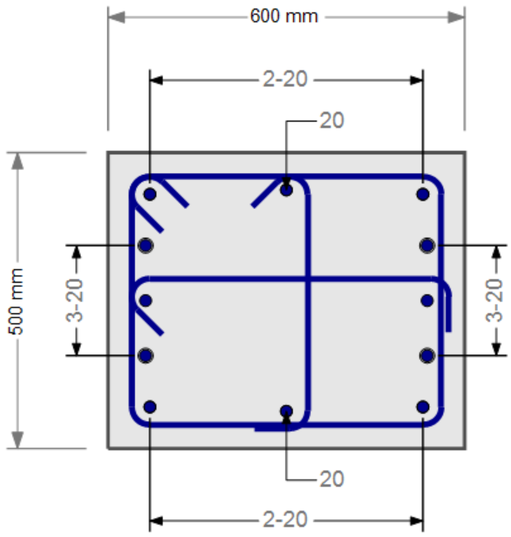

Figure 130 Section B of CC7 Column detail

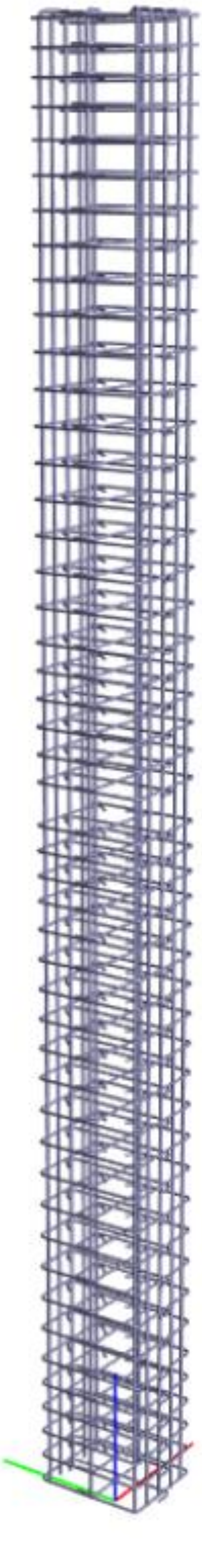

Figure 131 CC7 Rebar cage

d) Wall details

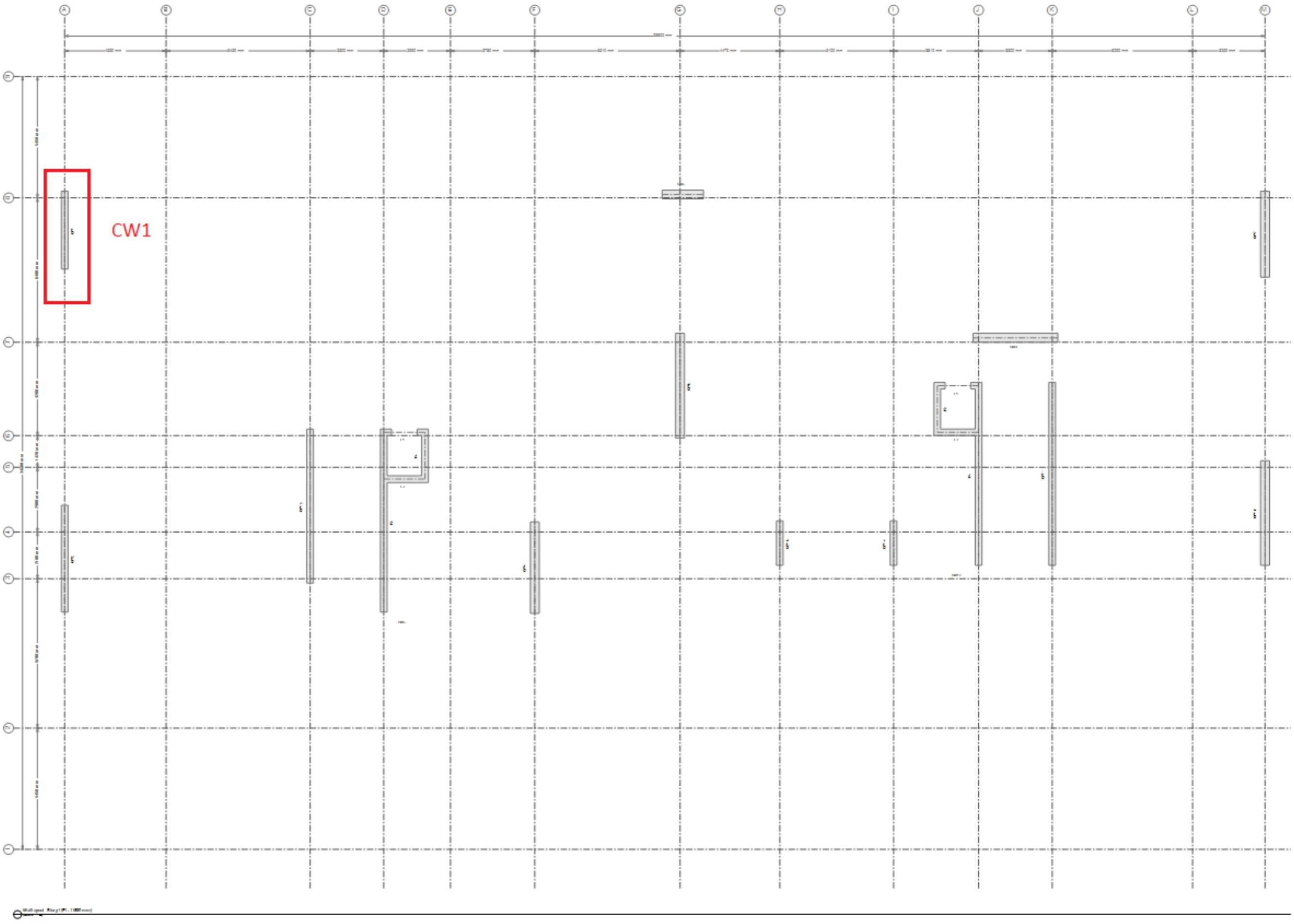

Figure 132 Storey 1 wall layout

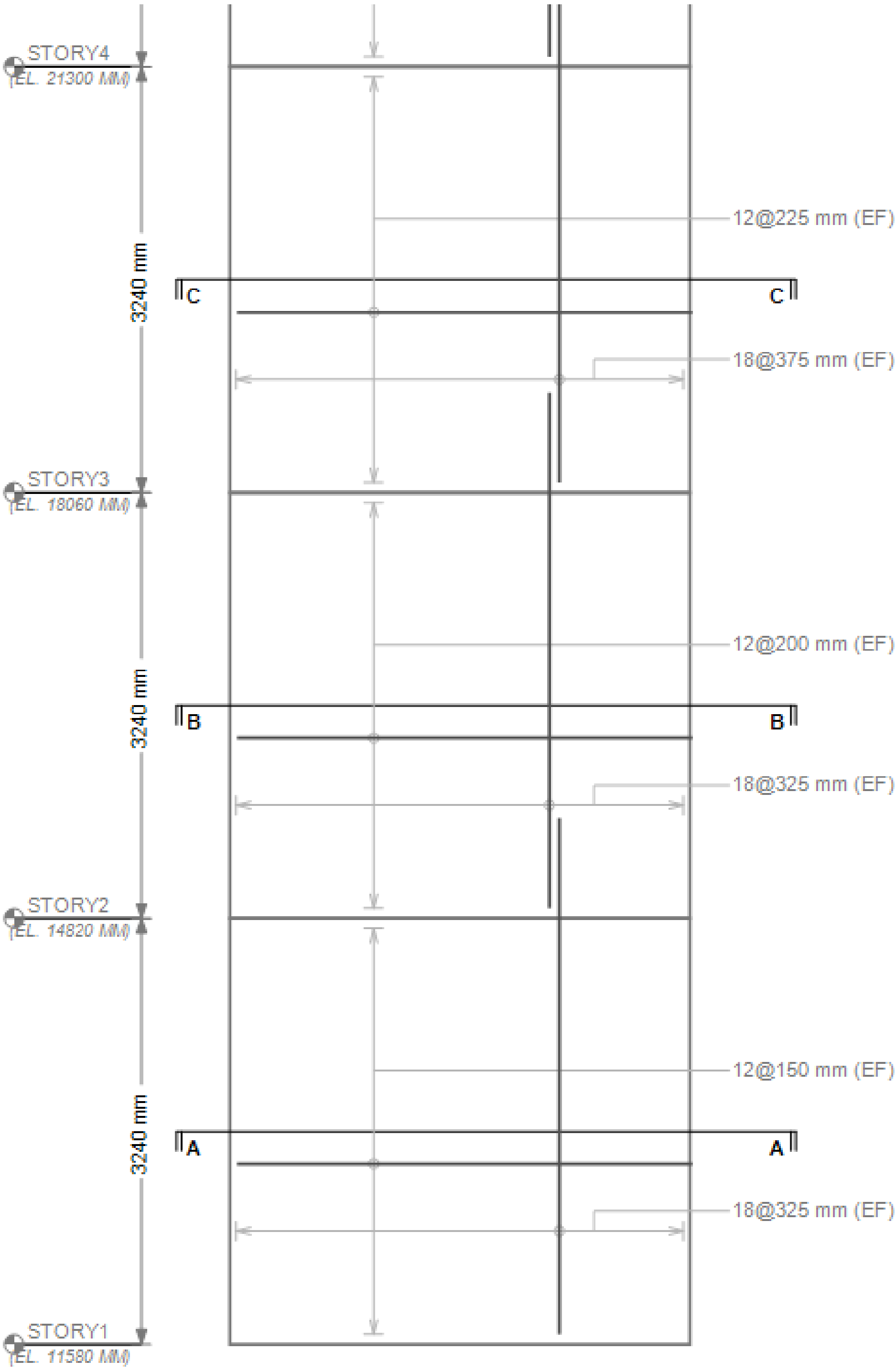

Figure 133 Storey 1-4 wall design

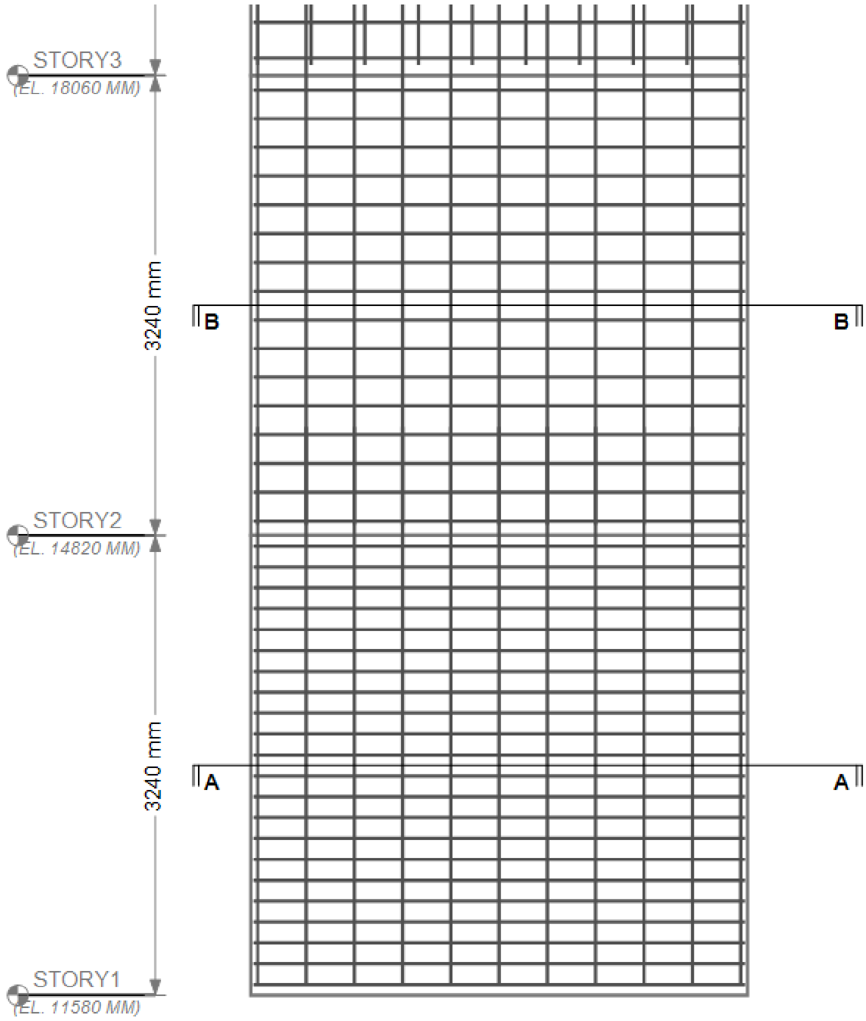

Figure 134 Wall design (Storey 1-3) All bars

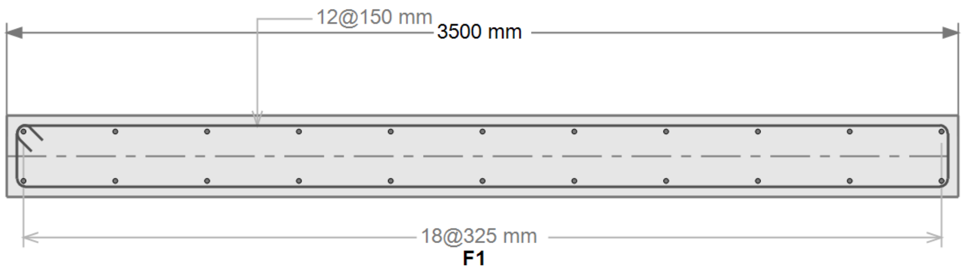

Figure 135 CW1 Section A detail

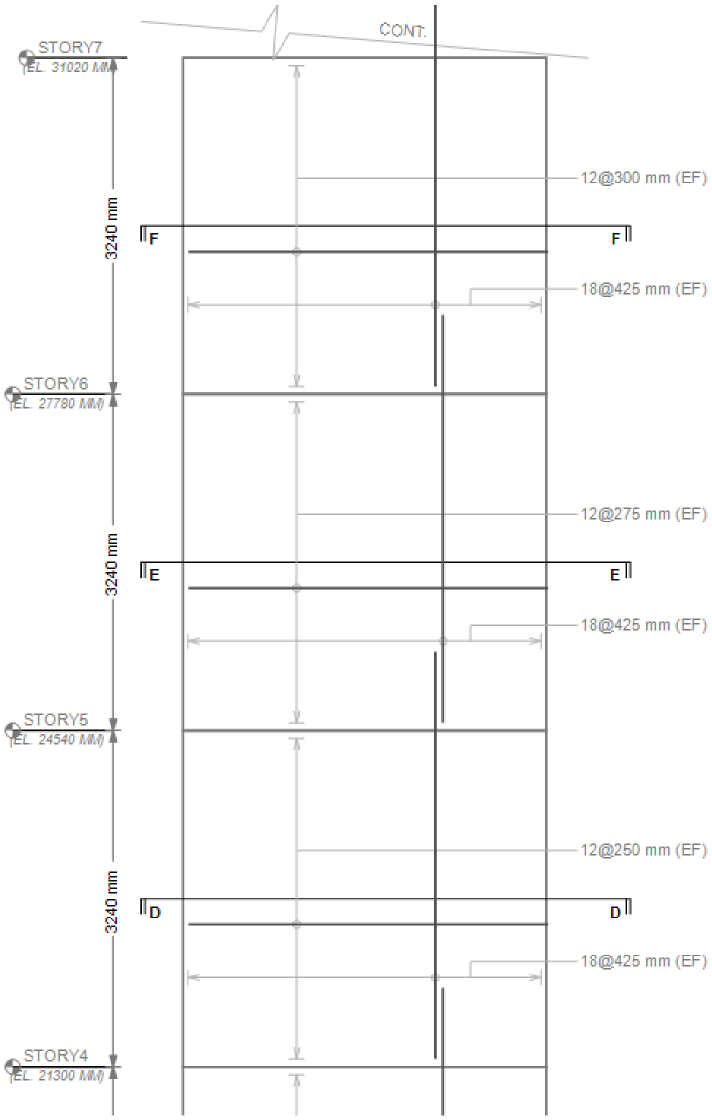

Figure 136 Strey 4-7 wall design

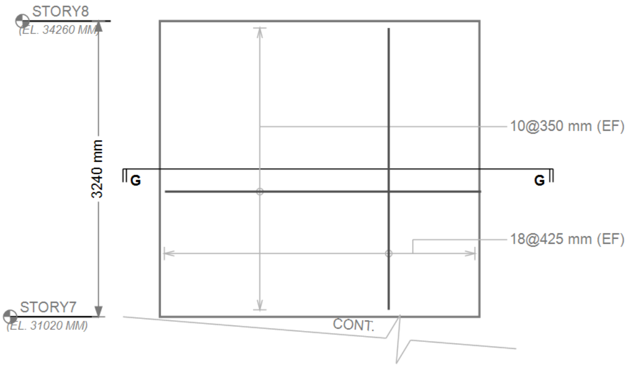

Figure 137 storey 8 wall design