ALMA MATER STUDIORUM
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SECOND CYCLE DEGREE

MULTI-CODE RESPONSE SPECTRUM ANALYSIS OF TEN-STOREY REINFORCED-CONCRETE BUILDING

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1. INTRODUCTION

1.1 Background of the Study

The construction industry in Albania has been developing rapidly during the last decades, especially in urban areas where the demand for tall buildings has increased significantly. With this expansion comes the need for safer and more reliable structures, particularly in regions with high seismic risk. Albania, situated in the Western Balkans, lies in one of the most seismically active zones in Europe. This reality has shaped the way engineers and architects approach building design, where seismic resistance becomes not just a regulatory requirement but a matter of public safety and resilience.

1.2 Definition of the Problem

Although Albania has its own seismic code (KTP89), many projects in the country are influenced by foreign practices, particularly Eurocode 8 and the Italian NTC2018, due to collaborations with international companies and joint ventures. However, limited comparative studies exist to show the practical implications of using different design codes for the same structure. This leaves a gap in understanding how different standards affect design outcomes such as base shear, structural displacements, and reinforcement requirements.

1.3 Objectives of the Study

The main objective of this thesis is to perform a comparative seismic analysis of a reinforced concrete tower building using three different seismic design codes: Eurocode 8, Italian NTC2018, and Albanian KTP89. Specific objectives include:

- Reviewing the seismic history and hazard characteristics of Albania.
- To generate design response spectra according to the three codes and apply them to the same structural model.
- To model and analyze a 10-storey tower with 2 underground floors under response spectrum analysis.
- To generate design response spectra according to the three codes and apply them to the same structural model.
- To compare the outcomes in terms of displacements, base shear, internal forces (axial, shear, moment), and reinforcement demands.
- To highlight the similarities, differences, and potential advantages of each code in the Albanian context.

1.4 Study Significance

This study is significant because it bridges academic research with real design practice. By comparing the outcomes of three codes, it offers insight into how structural safety and economy can vary depending on the adopted standard. For Albania, which is currently undergoing infrastructure growth, such a study can provide valuable information for engineers, policymakers, and companies engaging in design competitions and real projects. Furthermore, the work contributes to ongoing discussions about updating national codes and aligning them with international practices.

2. HISTORICAL AND SEISMIC CONTEXT IN ALBANIA

2.1 Historical Development of Construction in Albania

Construction in Albania has gone through several distinct phases, reflecting the country's political, social, and economic history. During the pre-communist period, building practices were primarily traditional, relying heavily on masonry structures, timber roofs, and stone foundations, typical of Mediterranean architecture. These buildings, while culturally rich, had limited resistance to seismic loads due to the lack of engineering-based design principles.

In the years following the collapse of communism in the early 1990s, Albania experienced a profound shift in the way construction was approached. The opening of the country to international influence introduced new actors into the sector, with private companies taking the lead in development. Partnerships with foreign professionals gradually brought in modern engineering techniques and design standards that had not been widely applied before. Cities such as Tirana soon faced a surge in demand for high-rise buildings and large infrastructure projects, reflecting the country's rapid urban transformation.

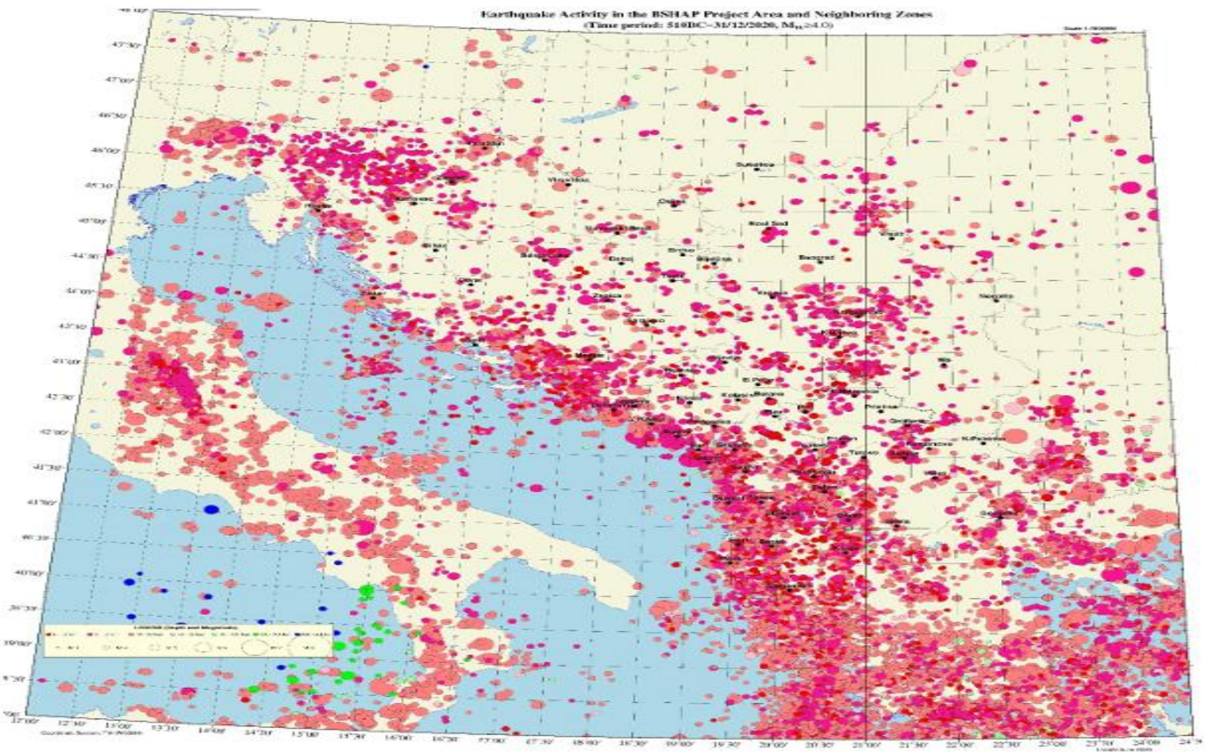
2.2 Seismicity of Albania

Albania is located in the Mediterranean–Alpine seismic belt, one of the most active tectonic regions in Europe. The country lies near the convergence zone between the African and Eurasian plates, where compressional forces have shaped the Dinaric Alps and surrounding geological structures. This tectonic environment makes Albania highly prone to earthquakes.

Historical records show that Albania has experienced several strong earthquakes with magnitudes exceeding 6.0 on the Richter scale. Notable events include the **1967 Dibër earthquake (M 6.5)**, which caused widespread damage in northeastern Albania, and the **1979 Montenegro earthquake (M 6.9)**, which severely affected northern Albania as well. More recently, the **2019 Durrës earthquake (M 6.4)** resulted in significant casualties, economic losses, and highlighted once again the seismic vulnerability of many urban structures.

These events demonstrate not only the seismic hazard but also the urgent need for stringent seismic design practices. They have shaped both public awareness and professional practice in structural engineering in Albania.

Figure 2.2.1 Spatial distribution of the earthquake epicenters included in the BSHAP [1] catalogue (Time period: 510 BCE-31/12/2019; MW≥4.0).[1]



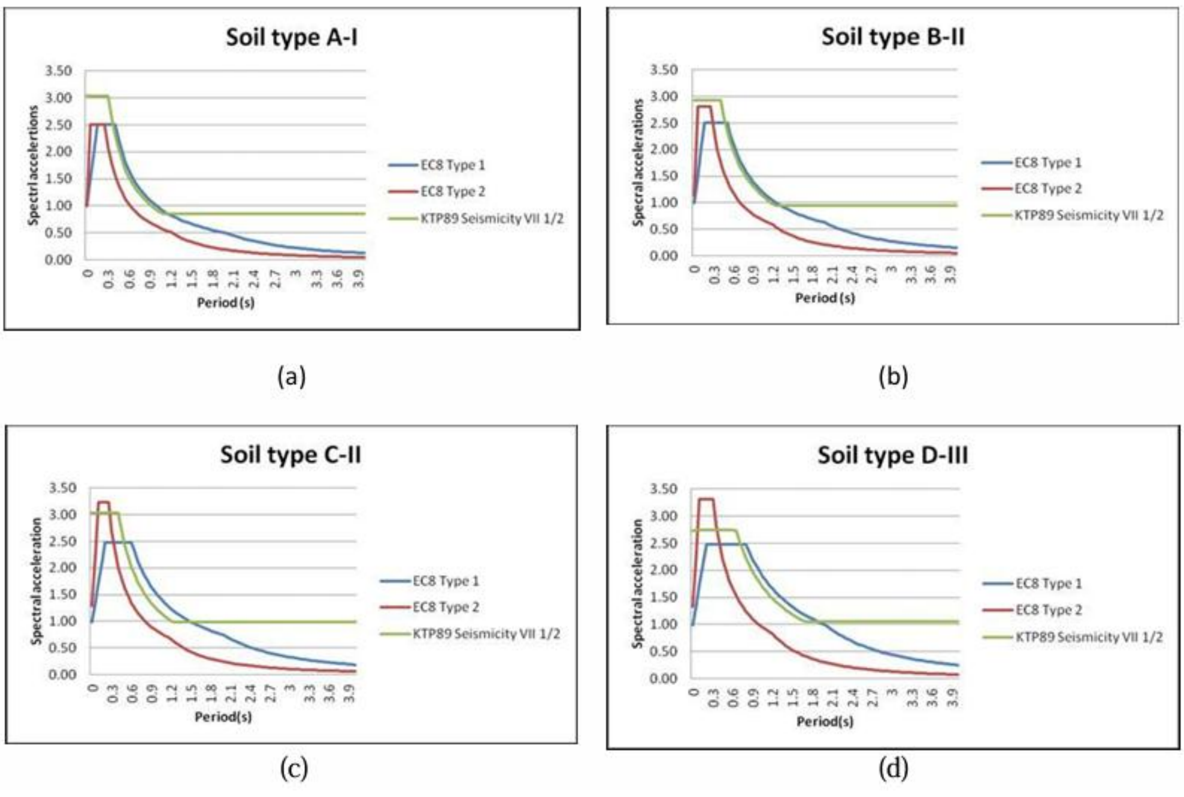


Figure 2.2.2. Seismic Response Spectrum Example Comparison EC8 with KTP89.[3]

2.3 Seismic Hazard Assessment and Mapping

Seismic hazard in Albania is not uniform. The western coastal region, particularly around Durrës, Tirana, and Vlora, is considered one of the most dangerous zones due to active faults and dense population. Hazard maps developed over the years (including those embedded in KTP89 and more recent international studies) assign high values of peak ground acceleration (PGA), often in the range of 0.25g to 0.35g for design purposes.

Modern probabilistic seismic hazard assessments (PSHA), many of them carried out through European projects like SHARE and ESHM20, suggest that the seismic risk in Albania is comparable to some of the most hazardous regions in Southern Europe, such as central Italy or western Greece. These maps are essential in calibrating response spectra for design purposes and highlight the importance of using up-to-date scientific data in building codes.[1]

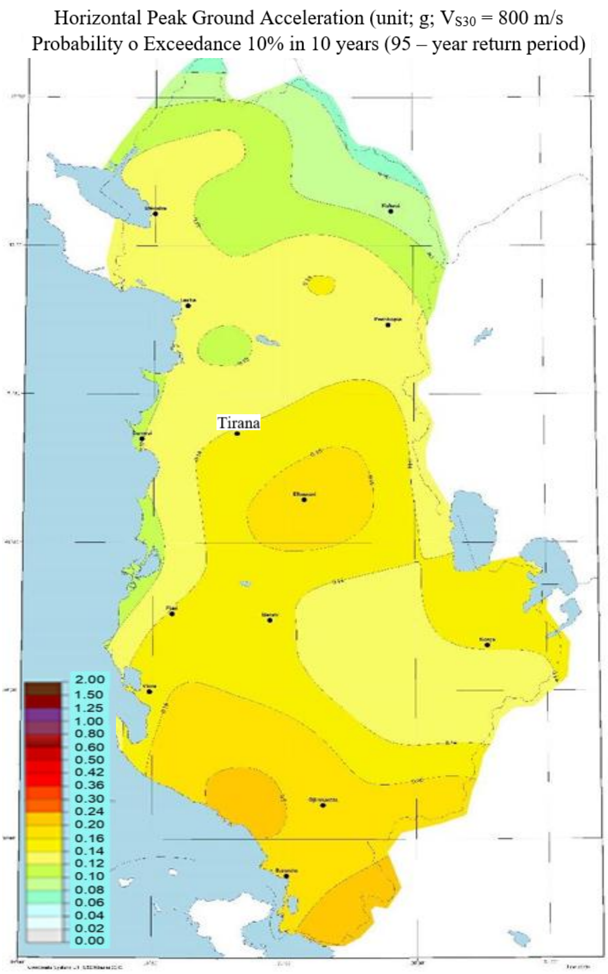


Figure 2.3.1. Seismic hazard map of Albania showing peak ground acceleration for 10-percent probability of exceedance in 10 years and VS30 site condition of 800 meters per second.[1]

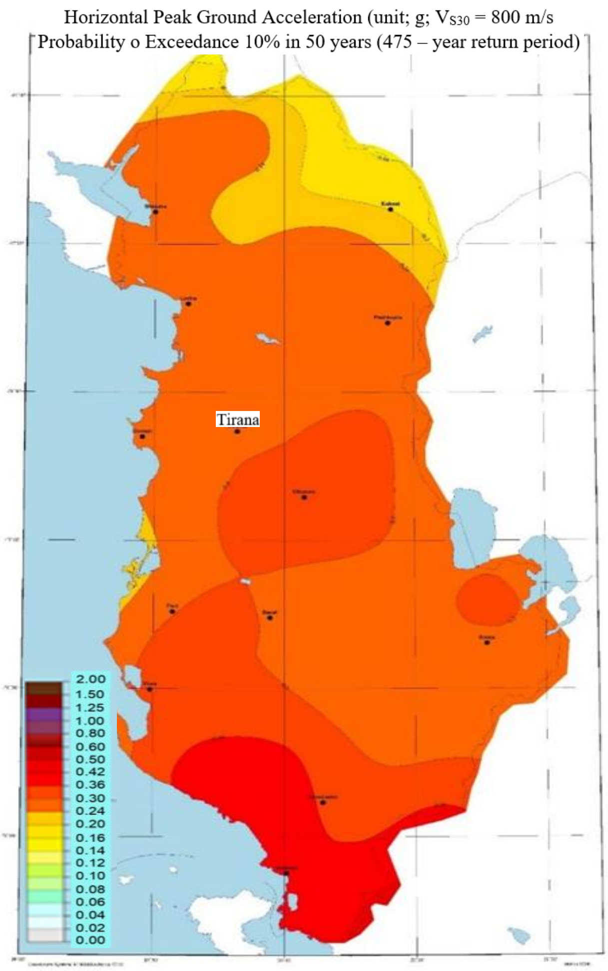


Figure 2.3.2. Seismic hazard map of Albania showing peak ground acceleration for 10-percent probability of exceedance in 50 years and Vs_{30} site condition of 800 meters per second.[1]

2.4 Evolution of Seismic Codes in Albania

Albania's first official seismic design regulations were introduced in the 1970s, evolving into **KTP89** (Code of Technical Provisions, 1989). This code was based largely on empirical approaches and deterministic seismic hazard assumptions, typical of codes of its era. While it represented a significant step forward at the time, its limitations became evident as international standards advanced toward probabilistic hazard models and performance-based design.

Over the past few decades, Albania has had to navigate a difficult balance between relying on its long-standing national code (KTP-89) and moving toward international standards. On one side, Eurocode 8 has gained prominence, especially under the influence of EU integration efforts; on the other, Italy's NTC-2018 has been widely applied by companies engaged in cross-border work. For practicing engineers, this overlap has created a complicated setting: the choice of which code to follow can affect not only the technical outcomes of a design but also how a project is perceived and approved by local authorities and international collaborators.

Figure 2.4. Picture captured after the Earthquake of the magnitude 6.4 near Durres, Albania (11.2019).

3. OVERVIEW OF SEISMIC DESIGN CODES

3.1 Introduction to Seismic Design Codes

Seismic design codes translate scientific knowledge of earthquakes into engineering standards that ensure structural safety and functionality. Although all codes aim to protect human life and minimize damage, their methodologies, assumptions, and parameters vary internationally. In Albania, where international collaboration is common, the primary codes are **Eurocode 8 (EC8)**, **Italian NTC2018**, and the **Albanian KTP89**. Every structural code reflects the scientific understanding, probabilistic models, and engineering approaches that were dominant at the time of its development. By comparing different codes, engineers are able to identify not only the technical differences between them but also the practical consequences that these distinctions have when applied in design work.

3.2 Eurocode 8 (EC8)

Eurocode 8 is the reference standard across Europe when it comes to seismic design. It was gradually developed during the 1990s and officially adopted in the 2000s, aiming to provide a harmonized framework for all EU countries.The code is built on a **probabilistic approach** to seismic hazard, meaning that ground motions are defined for specific return periods, most commonly 475 years (10% chance of exceedance in 50 years). One of its main tools is the **elastic response spectrum**, which is later reduced to a **design spectrum** using behavior factors (q-factors) that account for energy dissipation through ductility.

Another strength of EC8 is its **soil classification system** (A to E), which links ground type to amplification factors, ensuring that local soil effects are included in the design. Structural systems are divided into frames, walls, or dual systems, with different ductility and detailing rules for each.In practice, EC8 is becoming increasingly important in Albania, partly because of the country's path toward EU integration and partly because international companies already require it in many design competitions.

Figure 3.2. Eurocodes subdivision of the chapters.

3.3 Italian Code NTC2018

Italy's **Norme Tecniche per le Costruzioni 2018** shares many concepts with Eurocode 8, but it is adapted to Italy's particular seismicity and construction culture.

NTC2018 is based on **probabilistic seismic hazard maps like the EC8**, but the Italian maps are more detailed, reflecting the complex tectonics of the peninsula. The design spectra have a similar shape to EC8, but the parameters are calibrated to Italian ground motions, and designers can obtain **site-specific spectra** directly from a national online database.

What distinguishes NTC2018 is its **performance-based framework**. Instead of focusing only on life safety, it requires that buildings be checked at different limit states: Immediate Occupancy, Damage Limitation, Life Safety, and Collapse Prevention. This layered approach gives a clearer picture of how a building will behave not just in rare earthquakes but also in more frequent, moderate ones.

The code also gives strong weight to **importance classes** (especially for hospitals, schools, and infrastructure) and dedicates an entire section to **existing structures and retrofitting**, which is highly relevant in a country with a large historical building stock.

Because Albania and Italy have close professional ties, and many Albanian firms collaborate directly with Italian studios, NTC2018 has a significant influence on practice in Albania as well.

Figure 3.3. NTC 2018 – Norme Techniche per le Costruzioni.

3.4 Albanian Code KTP89

Albania's **KTP89** is the national seismic code introduced in 1989. It was prepared during the communist era, largely inspired by Soviet and Eastern European approaches at the time. While it was a major step forward when introduced, it now appears outdated compared to modern standards.

Albanian code divides the country into seismic zones, assigning each a **fixed seismic coefficient** or peak ground acceleration value. Unlike EC8 and NTC2018, it does not use probabilistic hazard models. The response spectrum is simplified, with fewer site categories and less flexibility to adapt to real ground conditions.

In terms of structural philosophy, KTP89 is more **strength-based** than ductility-based. The idea of reducing elastic forces through ductility factors is not fully developed, and detailing rules for energy dissipation are limited. Material assumptions are also lower, reflecting the concrete and steel strengths available in the 1980s.

While many projects in Albania now reference EC8 or NTC2018, KTP89 is still the only **nationally recognized code**, which places engineers in a difficult position: should they strictly follow it, or rely on modern European provisions? This dilemma is one of the reasons why comparing the three codes in practice is so important.

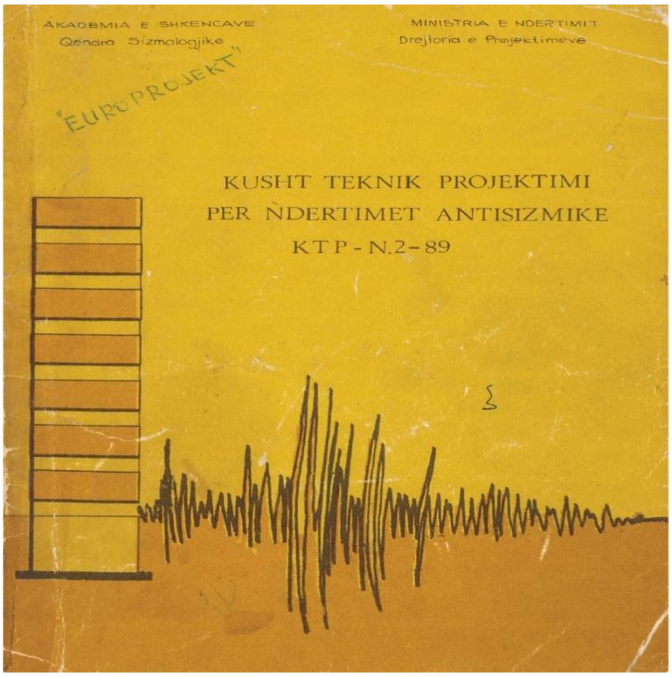


Figure 3.4. KTP89 – Kusht Teknik Projektimi per Ndertimet Antisizmike 1989.

4. Materials Used in Structural Design

4.1 Introduction

When designing reinforced concrete structures, the materials chosen play a decisive role in how the building performs. Concrete and steel are not just numbers on paper; their real-life properties determine strength, ductility, and long-term safety. Since this thesis compares three different seismic codes, it is useful to see how each one treats materials. Even though all three codes work with concrete and reinforcement, the level of detail and the assumptions behind them are not always the same.

4.2 Concrete

4.2.1 General Properties

Concrete is the backbone of reinforced concrete structures. It carries compressive forces effectively but is weak in tension, which explains why it always needs reinforcement. Important aspects that engineers care about are compressive strength, stiffness (elastic modulus), and the shape of the stress–strain curve. These factors directly affect how the structure behaves, especially during an earthquake.

4.2.2 Eurocode 8 (EC8) and Concrete

EC8 refers to Eurocode 2 (EN 1992) for the definition of concrete properties. Concrete is grouped into strength classes such as C25/30, C30/37, and so on, where the first number is cylinder strength and the second is cube strength. For seismic design, EC8 recommends not using grades lower than C20/25, because higher-quality concrete is more reliable under cyclic loading. The code also provides clear rules for stress–strain curves and design values obtained after applying safety factors.

4.2.3 Italian Code NTC2018 and Concrete

NTC2018 is fully aligned with European practice but, in many cases, is stricter. It uses the same strength classifications as EC2 but introduces additional detailing rules in seismic areas. For example, the Italian code requires stronger confinement of concrete in zones where plastic hinges may form. This approach highlights the Italian philosophy: the building should not only be strong enough but also capable of dissipating energy through ductile behavior.

4.2.4 Albanian Code KTP89 and Concrete

KTP89 was written in a different time, when material technology and design philosophy were more limited. Concrete classes are fewer, and the strength values are lower than those used today. The code places more emphasis on compressive strength and less on ductility. This means that if a structure were designed strictly under KTP89, it might use lower concrete grades compared to modern standards, which in turn could affect its seismic resilience.

4.3 Reinforcing Steel

4.3.1 General Properties

Reinforcing steel provides what concrete lacks: tensile strength and ductility. The key parameters are yield strength, ultimate strength, elongation capacity, and how well the bars bond with the surrounding concrete. In seismic zones, the ductility of steel is just as important as its strength, because it determines how the structure will behave under repeated cycles of loading and unloading.

4.3.2 Eurocode 8 (EC8) and Steel

EC8, following Eurocode 2, uses reinforcing steel classes such as B500, which has a yield strength of 500 MPa. The code is very strict about ductility requirements: the ratio between yield and ultimate strength must stay within certain limits, and minimum elongation values are defined to ensure that reinforcement does not break in a brittle way.

4.3.3 Italian Code NTC2018 and Steel

NTC2018 is again consistent with the Eurocode but often introduces more prescriptive detailing rules, especially for high-risk seismic zones. For example, the Italian code emphasizes the use of seismic-grade steel with controlled mechanical properties, ensuring that the bars can deform without sudden fracture.

4.3.4 Albanian Code KTP89 and Steel

In contrast, KTP89 offers a more basic classification of steel. Yield strength is defined, but there is much less discussion about ductility or cyclic performance. The rules for detailing, splicing, and confinement are also minimal compared to modern standards. This mirrors the context of the late 1980s, when ductility-based design was not yet a central concern in Albanian engineering practice.

5. RESPONSE SPECTRA

5.1 Introduction

In seismic design, the response spectrum is one of the most powerful tools engineers use to represent the effects of an earthquake. Instead of simulating every possible ground motion, the spectrum provides a simplified but effective way of describing the maximum expected response of structures with different natural periods. In practical terms, it tells us how much acceleration, velocity, or displacement a structure might experience depending on its stiffness and dynamic characteristics. Since this work compares three different codes—Eurocode 8 (EC8), the Italian NTC2018, and the Albanian KTP89—it is important to understand how each of them defines the response spectrum, what assumptions are made, and how the curves differ.

5.2 The Concept of a Response Spectrum

A response spectrum plots the maximum response (typically acceleration) of a series of single-degree-of-freedom oscillators subjected to the same ground motion, against their natural periods. Each oscillator represents a simplified model of a structure with a particular stiffness. This way, engineers can estimate how a real building, with its own dynamic properties, might respond without having to simulate every detail of the earthquake.

For design purposes, codes usually provide **elastic spectra** (assuming no inelastic behavior) and then modify them using reduction factors that account for ductility and energy dissipation.

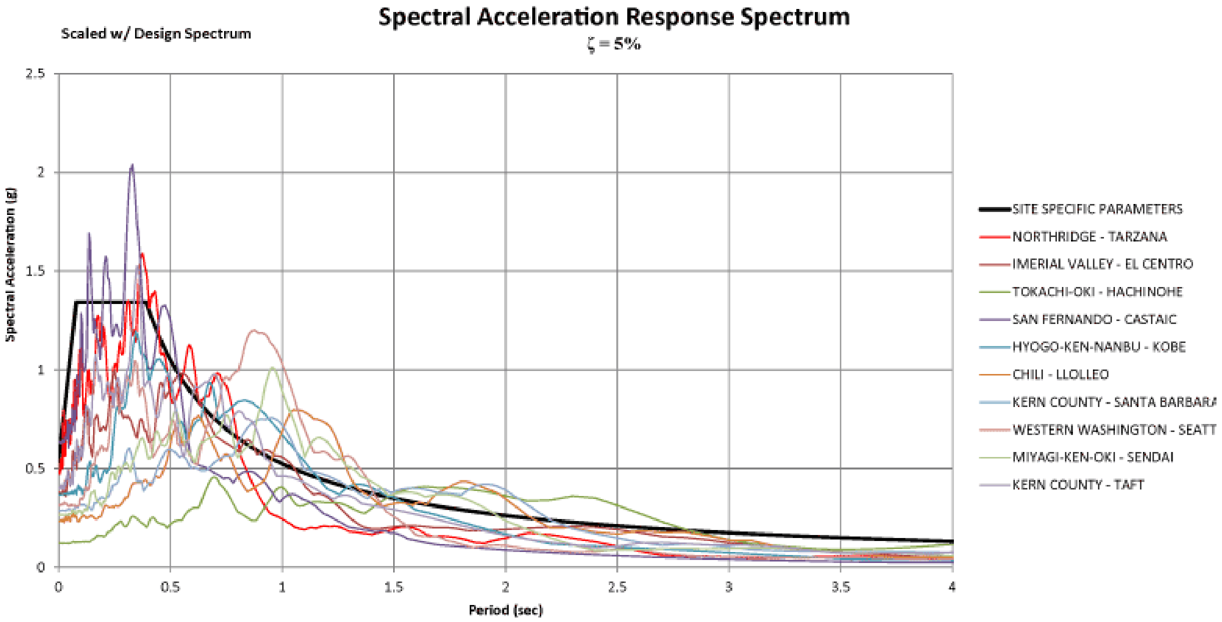


Figure 5.2. Examlpe of the Spectral Acceleration Response Spectrum (damping = 5%).

5.3 Eurocode 8 (EC8) Spectrum

Eurocode 8 offers a general framework that can be adapted to different European countries. The spectrum is defined using a few key parameters:

- **Peak ground acceleration (ag)**: the reference acceleration at the ground surface.
- **Importance factor (γI)**: modifies the seismic action depending on the building's function (e.g., hospitals vs. residential).
- **Soil factor (S)**: accounts for local ground conditions, amplifying or reducing seismic effects.
- **Spectral shape**: defined by corner periods (T1, T2, T3), which control the plateau and decay of the spectrum.

The design spectrum is obtained by dividing the elastic spectrum by a **behavior factor (q)**, which reflects the ductility of the structural system. The philosophy is that structures designed with ductility in mind can resist large earthquakes without necessarily being designed for full elastic response.

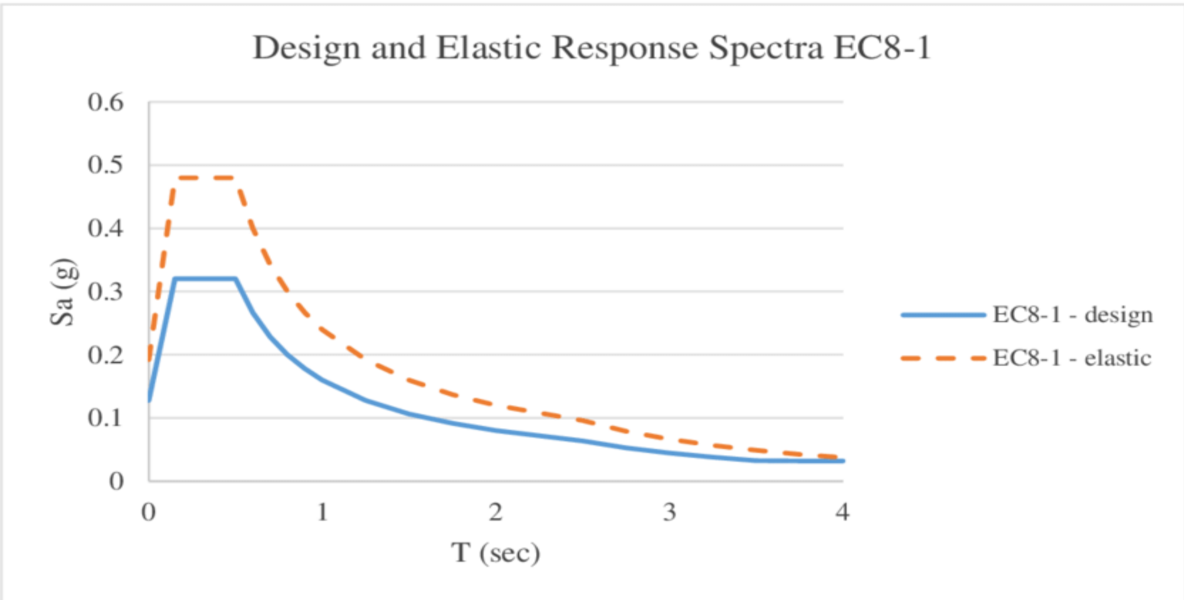


Figure 5.3. Design and Elastic Example of Response Spectra EC8-1

5.4 Italian Code NTC2018 Spectrum

NTC2018 adopts the same fundamental framework as EC8 but introduces more detail in certain areas, especially regarding seismic hazard characterization. Instead of using a single reference spectrum for the whole country, the Italian code relies on a dense national database of seismic hazard maps. This allows engineers to obtain site-specific response spectra with different probabilities of exceedance (50, 10, 5, 2% in 50 years).

The spectrum itself has the same general shape as EC8, with a plateau followed by a decay, but NTC2018 tends to be stricter in defining parameters like corner periods and damping corrections. The Italian code also emphasizes the importance of soil conditions, often requiring detailed site investigations to choose the right soil factor.

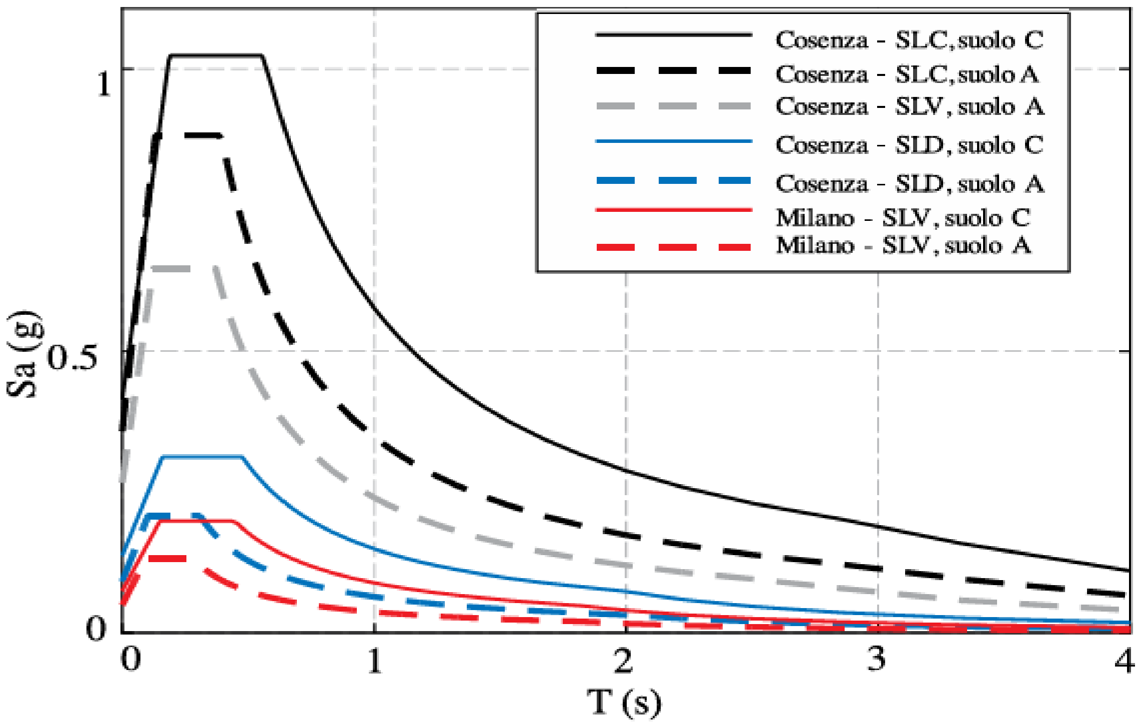


Figure 5.4. Elastic Example of Response Spectra NTC2018 [2]

5.5 Albanian Code KTP89 Spectrum

Its response spectrum is simpler, defined with fewer parameters and less emphasis on probabilistic hazard analysis. Instead of detailed seismic hazard maps, it provides reference values of seismic coefficients depending on seismic zones.

The spectral shape is more basic: a rising branch, a plateau, and then a decay. Unlike EC8 and NTC2018, it does not explicitly introduce a behavior factor in the same modern sense. Instead, the reduction of seismic forces is embedded indirectly through coefficients that account for building type and importance. While functional, this approach does not capture the nuances of modern seismic design and may underestimate or overestimate demands depending on the structural system.

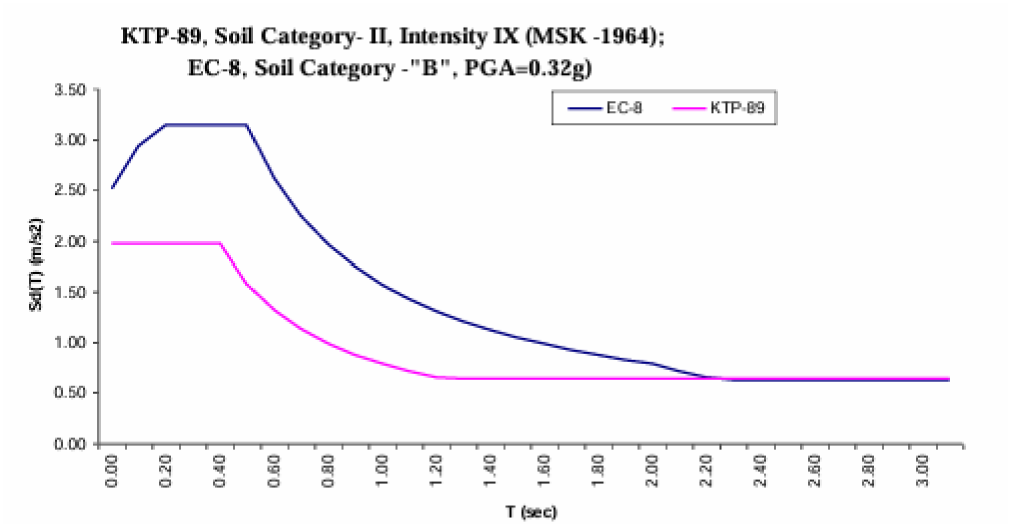


Figure 5.5. Example of Response Spectra KTP89 & EC8 [3]

5.6 Importance for This Thesis

The differences between these spectra will directly influence the structural response of the tower building analyzed in this thesis. Base shear, displacements, and reinforcement demands will vary depending on which spectrum is applied. By comparing them, the study aims to show how much the choice of seismic code affects the design outcome and whether older codes like KTP89 are still adequate for modern structures.

6. BUILDING GEOMETRY AND STRUCTURAL MODEL

6.1 Introduction

A clear definition of the building's geometry and intended function is the first step in any structural design process. The building selected for this thesis is a reinforced concrete tower with 10 stories above ground and 2 underground levels, designed for mixed-use purposes. The project is located in Tirana, Albania, a city that has experienced rapid urban growth in recent decades and lies within a seismically active region of the Balkan Peninsula.

The choice of this structure is motivated by its relevance: mid- to high-rise reinforced concrete buildings are increasingly common in Albania's urban centers, and our seismic performance has become a critical concern following recent earthquakes in the region (notably the 2019 Durrës earthquake).

6.2 Location and Urban Context

The proposed building is assumed to be located in a dense urban zone of Tirana, where space efficiency and vertical development are priorities. The presence of underground floors reflects the demand for parking space in modern urban projects, while the above-ground levels are intended to accommodate both commercial and residential functions.

This urban context also brings specific structural challenges: limited plot dimensions restrict the lateral spread of the building footprint, which often leads to taller, more slender structures. In seismic zones, this makes careful control of lateral displacements and torsional effects essential.

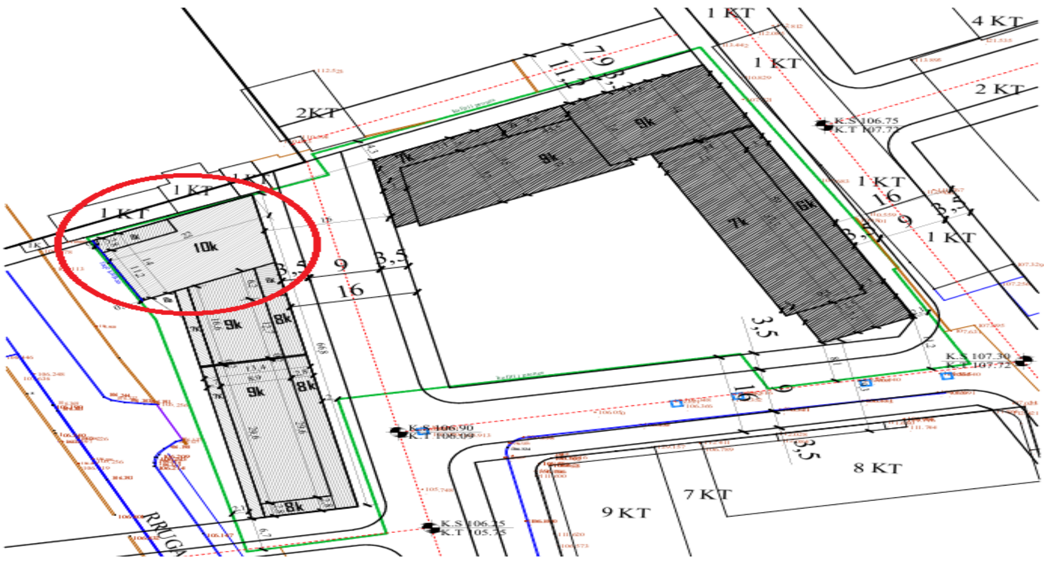


Figure 6.2. Layout plan of the Structure, Tirana, Albania "St. Karl Gega"

6.3 Purpose and Functional Layout

The tower serves a mixed-use function:

Two underground levels: parking and technical areas (mechanical/electrical rooms, storage).

Ground floor: commercial spaces such as shops, cafes, or offices, requiring open layouts with fewer columns.

Floors 2 to 10: primarily residential apartments, where a more regular structural grid is compatible with partition walls.

This mixed use is typical for modern developments in Albania, where a single building must serve multiple economic and social needs. Structurally, this variation in use influences load assumptions, floor heights, and the placement of vertical elements.

Figure 6.3. Layout plan of the Structure, Tirana, Albania “St. Karl Gega"

6.4 Structural System Selection

The building is designed as a **reinforced concrete frame–wall system**. This choice balances flexibility and stiffness:

- **Columns and beams** form the moment-resisting frame, providing ductility and redundancy.
- **Shear walls**, located around the staircases and elevator shafts, provide additional lateral stiffness and help control displacements.
- **Floor slabs** are designed as cast-in-place reinforced concrete, contributing to diaphragm action and ensuring horizontal load transfer to the vertical resisting elements.

This dual system is common practice in seismic design because it combines the energy dissipation capacity of frames with the stiffness of walls, thus avoiding excessive drift under earthquake loading.

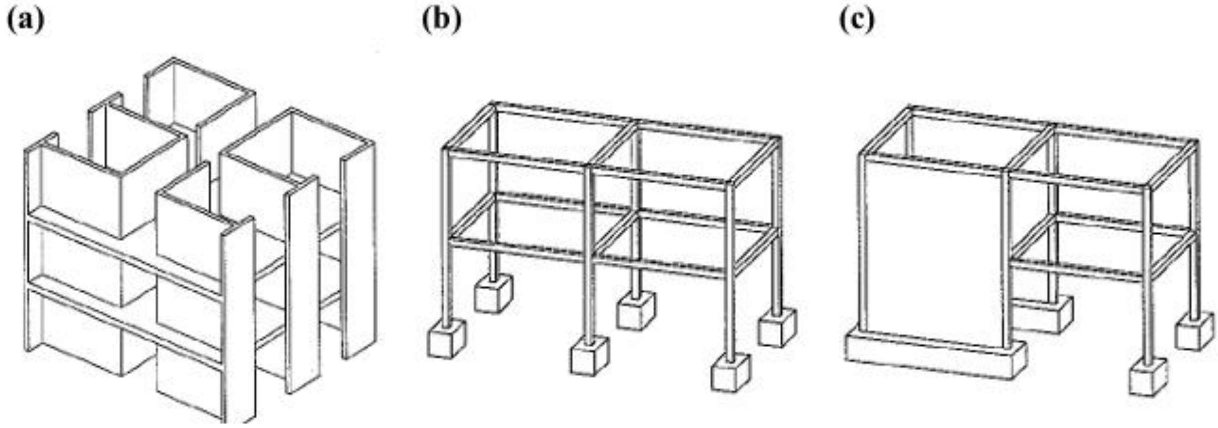


Figure 6.4. a) Wall System Building, b) Frame System Building, c) Dual System Building; figures from Arie, G (2003).[4]

Dual system structures are structural configurations that combine two different load-resisting systems, which work together to carry both gravity and seismic forces. These systems may also be made from different materials, such as reinforced concrete paired with masonry, or reinforced concrete combined with steel. In this study, the focus is on reinforced concrete frame–wall systems; however, the approach presented can also be extended to other types of dual systems. Figure 8 illustrates examples of a wall system building, a frame system building, and a frame–wall dual system building, respectively.

6.5 Geometric Parameters

- **Total above-ground height**: approx. 32–34 m.
- **Story heights**: 3.23 m for residential floors, 3.75 m for ground floor (to accommodate commercial spaces).
- **Underground levels**: each about 3.06 m high.
- **Footprint**: roughly rectangular plan, roughly 34 m × 20 m.
- **Grid spacing**: ~5–7 m in both directions, optimized for apartment layouts and open commercial areas at the base.

The structure is roughly regular in plan and elevation, which minimizes irregular torsional effects and is strongly recommended by seismic codes for good performance.

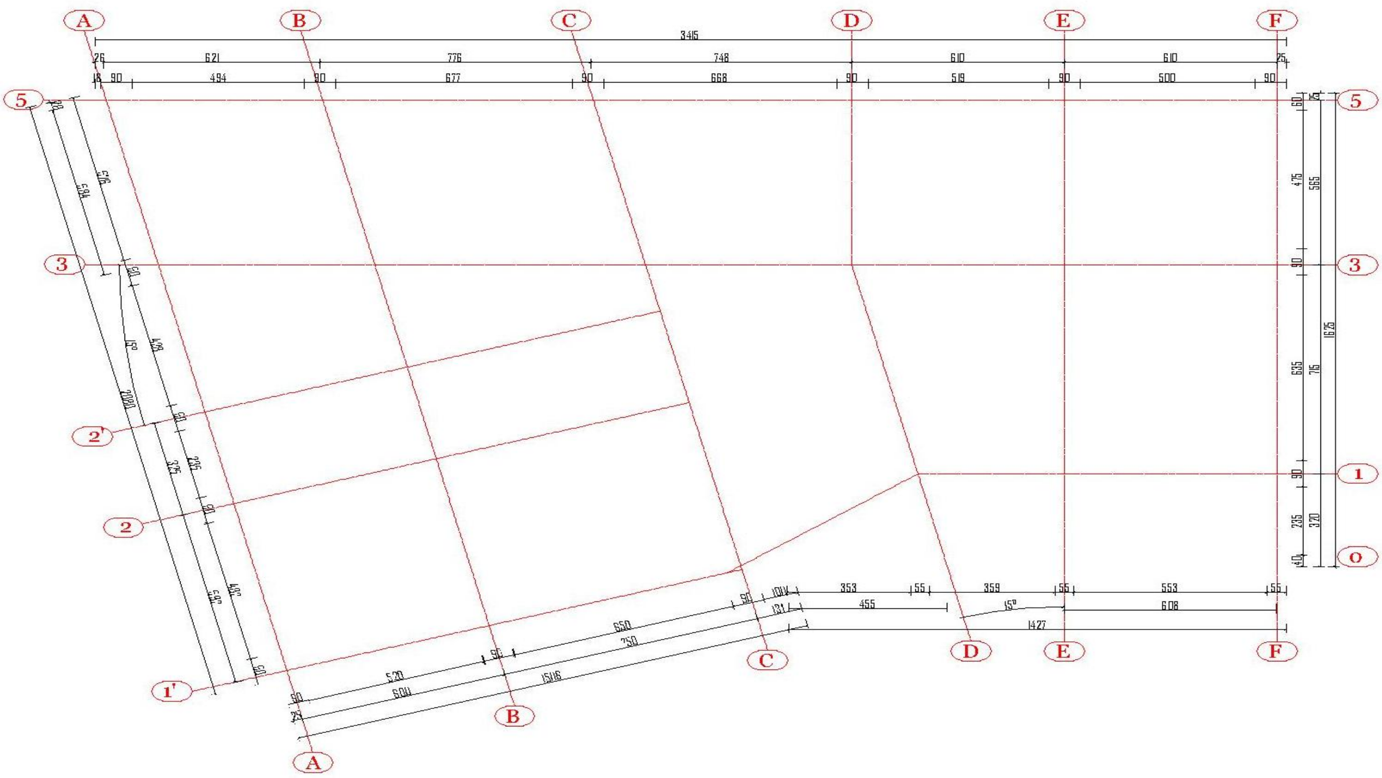


Figure 6.5. Axis system of the Reinforced Concrete Structure.

6.6 Modeling Approach in SAP2000

The numerical model is created in **SAP2000**, with the following assumptions:

- **Columns and beams**: modeled as frame elements with their cross-sectional properties.
- **Shear walls**: modeled using shell elements, continuous along the building height.
- **Slabs**: modeled as rigid diaphragms to simulate in-plane stiffness.
- **Foundation**: represented through clamped hinges to maximize the seismic forces acting on the structure.

The modeling process follows a progressive approach. Initially, a bare structural model is created based solely on the architectural plan. This model is then refined step by step until the final detailed model is obtained, which includes realistic material properties, cracked section behavior, drifts, deflections and reinforcement checks.

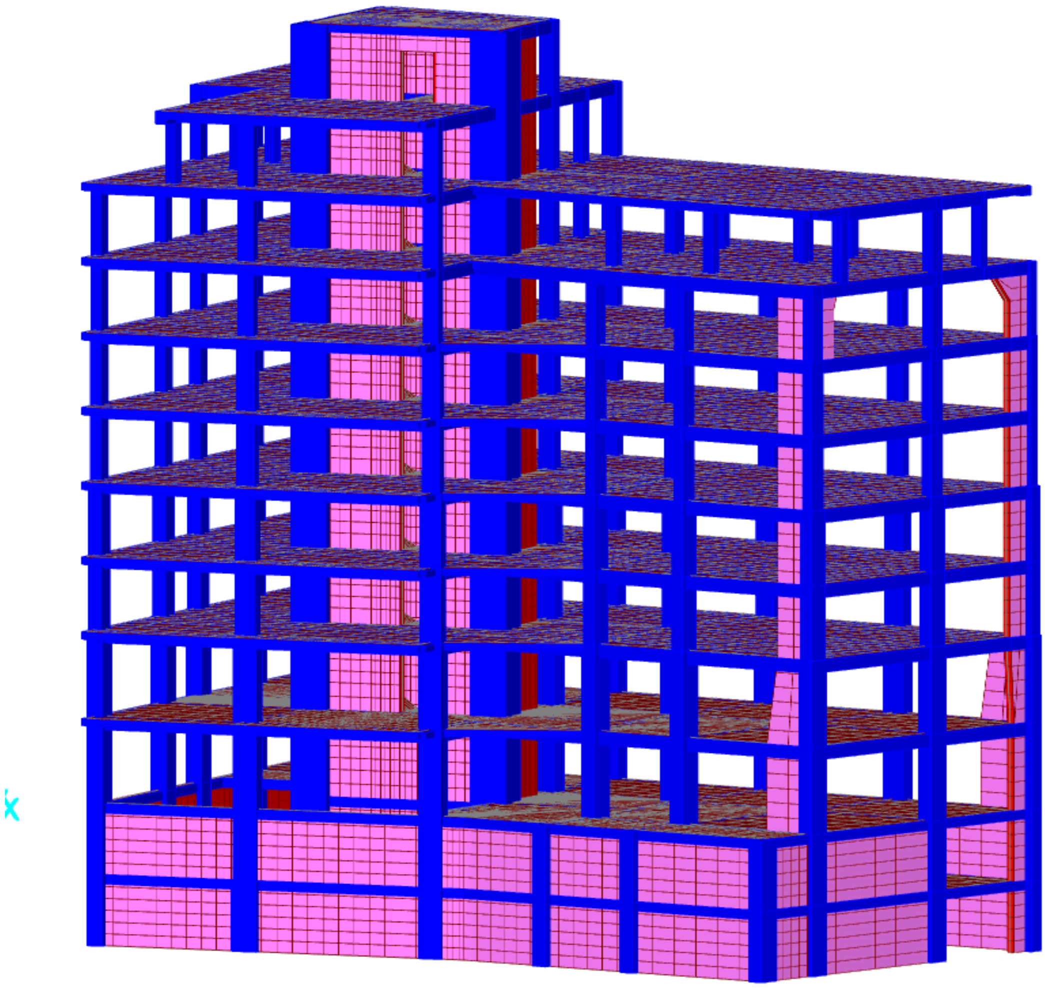


Figure 6.6. 3D View of the Final Model.

6.7 Mass, Load Distribution and Modelling Rules.

Accurate mass modeling is vital for seismic analysis. For this building:

- **Dead loads**: self-weight of structural elements, permanent finishes, and partitions.
- **Live loads**: commercial areas have higher live loads (offices/shops), while residential areas have lighter values. Parking levels are defined separately.
- **Seismic weight**: calculated according to each code's rules, ensuring consistency with the response spectrum definitions.
- **Cracking**: stiffness reductions are applied to account for concrete cracking under seismic action.
- **Damping**: a constant 5% damping ratio is assumed for the response spectrum analysis, as per international practice.

The placement of heavy elements, such as stair cores and elevator shafts, is carefully considered so as not to introduce torsional irregularities.

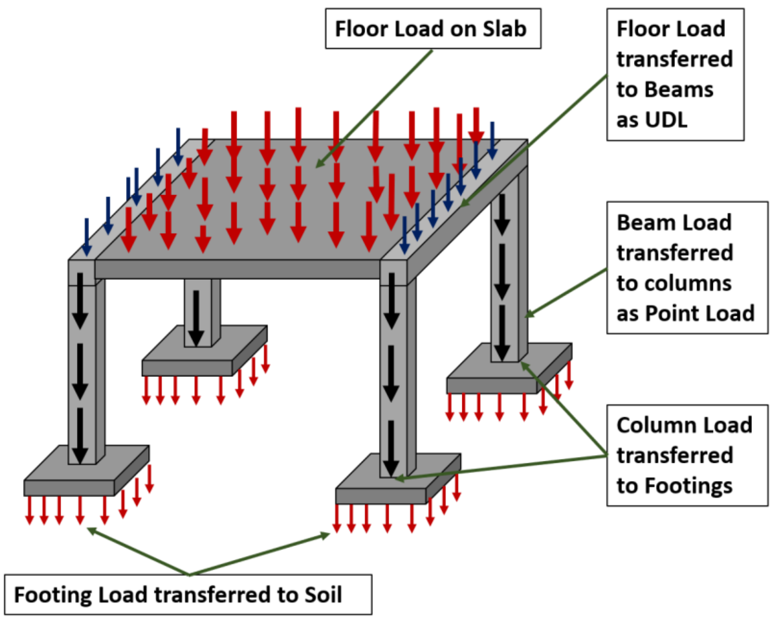


Figure 9.1. Load Transfering according to Eurocodes.[7]

7. MODELING APPROACH

1. Software used

The RC tower was modeled in **SAP2000**, selected for its reliability and ability to perform response spectrum analyses according to different seismic codes.

2. Geometric representation

- The model geometry was based directly on the architectural drawings.
- **Floor slabs** were defined as rigid diaphragms, simplifying the in-plane behavior without losing accuracy.
- **Beams, columns, and shear walls** were modeled using frame and shell elements to capture structural response realistically.

3. Material properties

- Concrete grade: **C30/37**, **C25/30**, according to Eurocode 2.
- Reinforcement steel: **B450C**, commonly used in Albania and Italy.
- Uniform properties were applied in all models to maintain comparability.

4. Boundary conditions

- The structure was supported on **fully fixed bases** at the foundation level.
- Soil–structure interaction was not included, ensuring the study focused only on spectrum differences.

5. Loading conditions

- **Dead loads** from self-weight and permanent partitions.
- **Live loads**: office occupancy on upper floors and parking usage in basements.
- **Seismic actions**: three spectra applied separately (EC8, NTC2018, KTP89).
- Gravity load combinations were identical in each case.

6. Mesh refinement

- Slabs and walls were meshed with a density that balanced precision and computational cost.
- Additional refinement was used around the **shear core**, where higher stress concentrations were anticipated.

7. Dynamic assumptions

- A **5% damping ratio** was applied across all analyses.
- This assumption reflects standard practice for RC structures and ensures uniformity in the comparison.

8. Principle of consistency

- All modeling decisions—geometry, materials, loads, supports, and damping—remained constant.
- The only variation across cases was the definition of the **response spectrum**, keeping the comparison fair and objective.

To carry out the seismic comparison in a meaningful way, the building needed to be translated into a consistent and realistic structural model. For this purpose, the analysis was conducted using **SAP2000**, a software widely recognized in both academic and professional settings for its capacity to handle response spectrum analyses according to different seismic codes. The decision to use SAP2000 was also influenced by its flexibility in modeling reinforced concrete structures with combined frame and wall systems, which corresponds well to the actual design of the tower.

The **support conditions** were idealized as fully fixed at the foundation level. While this assumption neglects the role of soil–structure interaction, it was considered acceptable for this study, since the goal is to highlight differences caused by seismic spectra rather than geotechnical effects. To maintain fairness across the three cases, this simplification was applied uniformly

The **loading conditions** included the building's self-weight, permanent partitions, live loads determined by occupancy (offices for the above-ground floors and parking for the basement levels), and seismic actions based on the three spectra under consideration: Eurocode 8, NTC2018, and KTP89. For comparability, gravity load combinations were identical in each case, with only the seismic definition changing.

In short, the modeling approach was designed around the principle of **consistency**. All parameters—geometry, materials, supports, loads, and damping—were kept identical across the three cases. The only intentional variation was the seismic action itself, which allows the comparison of EC8, NTC2018, and KTP89 to be both technically sound and fair.

8. MATERIAL PROPERTIES

8.1 Concrete Properties – Grade, Compressive Strength

The reinforced concrete used in this project is **C30/37** for vertical elements such as **columns and shear walls**, providing the strength needed to resist both gravity and seismic forces. For horizontal elements, including **slabs and beams**, a slightly lower grade of **C25/30** has been used, which is sufficient to carry self-weight and imposed loads while being more economical. Proper attention to curing, uniformity, and quality control ensures that all concrete elements perform reliably under both gravity and seismic actions.

8.2 Steel Reinforcement Properties – Yield Strength, Ductility

The **reinforcing steel** selected is **B450C**, with a yield strength of 450 MPa and a ductility class suitable for seismic applications. High ductility is essential in earthquake-prone regions, allowing the structure to dissipate energy without brittle failure. The steel meets all bending, anchorage, and lap-splice requirements for columns, beams, slabs, and walls.

8.3 Durability Considerations – Exposure Class, Cover

Durability is ensured through proper **exposure classification** and **concrete cover**. Environmental factors like humidity, rainfall, and carbonation are considered. Minimum concrete cover is **30–35 mm** for beams and columns exposed to moderate conditions, in line with Eurocode and Italian standards.

8.4 Material Safety Factors in EC8 – γC, γS Values

For Eurocode 8, the partial safety factors are:

- **γC = 1.5** for concrete
- **γS = 1.15** for steel reinforcement

These account for material variability, construction quality, and uncertainties in design assumptions.

8.5 Material Safety Factors in NTC2018 – Slight Variations from EC8

Italian NTC2018 defines similar safety factors, reflecting modern European design alignment:

- **γC = 1.5** for concrete
- **γS = 1.10** for steel

8.6 Material Safety Factors in KTP-89 – Historical Values Used

The Albanian **KTP-89** code uses slightly higher safety factors to account for historical design conservatism:

- **γC = 1.5** for concrete
- **γS = 1.2** for steel

9. LOADING CONDITIONS

9.1 Dead Loads – Self-Weight, Finishes [7]

Dead loads include the **self-weight of structural elements** (beams, columns, slabs, and walls) calculated based on material densities, as well as **permanent finishes** such as floor tiles, ceiling finishes, and partition walls. These loads are applied uniformly on slabs and transferred through beams and columns to the foundations. Accurate modeling of dead loads is critical, as they influence both gravity effects and the building's dynamic response under seismic actions.

Category	Specific Use	Example
A	Domestic and Residential	Residential Buildings, houses, bedrooms, bedroom in hotels, kitchens, toilets
B	Office Areas	
C	Areas where People may congregate	C1: Areas with Tables
		C2: Areas with fixed Seats
		C3: Museums , acces areas in public
		C4: Areas with physical activities
		C5: Large crowds, public events
D	Shopping Areas	D1: Retail Shops
		D2: Department Stores

Figure 9.1. Load Definition according to Eurocodes.

9.2 Live Loads – Building Occupancy [7]

Live loads are defined according to the intended **usage of each floor**. For the office floors above ground, a live load of **2 kPa** is applied, while for the underground parking and technical areas, a higher load of **3–4 kPa** is used to account for vehicle weight and storage. Live loads are variable, representing transient occupancy and use, and are combined with dead loads and seismic actions for ultimate and serviceability checks.

9.3 Environmental Loads – Snow, Wind [7]

Environmental actions are considered where relevant. **Snow loads** are applied to roof slabs based on local meteorological data as live loads.
Wind loads, however, are **not included in this analysis**, as Tirana is not exposed to strong winds and the **seismic forces dominate the design** of the structure. Therefore, wind effects are considered negligible compared to earthquake actions and do not influence member sizing or lateral system selection up to structures of 60m height from the zero level.

9.4 Seismic Loads per EC8 – Definition of Input [5]

For Eurocode 8, seismic loads are defined using the **design response spectrum** based on:

- **Peak ground acceleration (a_g)** at the building site,
- **Soil classification** to account for amplification effects, and
- **Importance factor (γI)** reflecting building function.
  The loads are applied in **both X and Y directions**, with modal combination rules used to combine effects from multiple vibration modes. [5]

9.5 Seismic Loads per NTC2018 – Spectrum Parameters [6]

In NTC2018, seismic actions are applied using:

- **a_g,30**, the design acceleration at 30% probability,
- **F0**, the spectral amplification factor, and
- **Tc***, the corner period distinguishing short- and long-period behavior. These parameters define the response spectrum applied to the SAP2000 model and capture Italian code-specific assumptions. [6]

9.6 Seismic Loads per KTP-89 – Older Assumptions [3]

The Albanian KTP-89 code uses older spectral definitions, typically based on **historical PGA values** and simplified soil classifications. While less refined than modern codes, these assumptions reflect the design practice in Albania during that period and are included for comparative purposes.[3]

9.7 Load Combinations for ULS – Governing Combinations [7]

Ultimate Limit State (ULS) combinations follow code requirements to account for the **worst-case scenario**:

$$1.35G+1.5Q(\text{gravity-dominated})$$

$$G+\psi Q+\gamma E(\text{seismic combination})$$

These ensure that the building remains safe under extreme events, including maximum seismic effects.

9.8 Load Combinations for SLS – Service Checks [7]

Serviceability Limit State (SLS) combinations are used to check **deflections, drifts, and vibrations** under normal use:

- **1.0G + ψQ** for long-term deflection checks
- Lower seismic factors to ensure comfort and operational functionality
  SLS checks are crucial for human comfort and long-term structural performance.

10. DEVELOPMENT OF THE DESIGN RESPONSE SPECTRA

10.1 Input Parameters for EC8 Spectrum – a_g, Soil Class, γI

For Eurocode 8 (EC8), the design response spectrum is defined using:

- **Peak ground acceleration (a_g)** at the site.
- **Soil classification (A–E)**, which modifies spectral amplification.
- **Importance factor (γI)**, reflecting building function:
    - Ordinary buildings: γI = 1.0
    - Important buildings: γI = 1.2–1.5

The spectral acceleration for a structure with damping ~5% is given by:

For T ≤ TB:

S(T) = a_g × γI × [1 + (T / TB) × (η − 1)]

For TB < T ≤ TC:

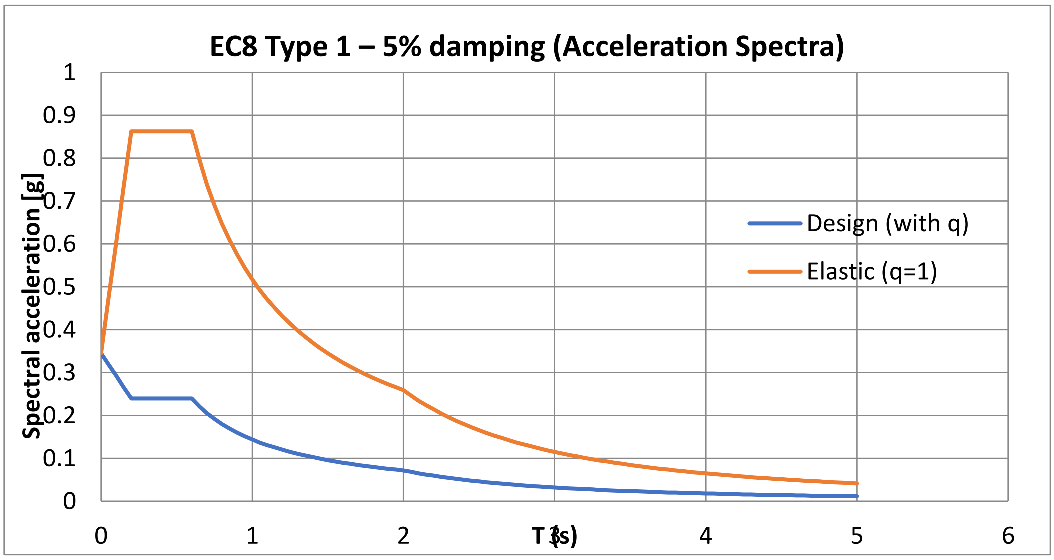
S(T) = a_g × γI × η

For T > TC:

S(T) = decreasing according to EC8 formula

- T = natural period of the building (s)
- TB, TC = characteristic periods depending on soil type
- η = damping correction factor (≈1.0 for 5% damping)

10.2 Spectrum PLOT for EC8



10.3 Input Parameters for NTC2018 Spectrum – a_g,30, F0, Tc*

- **a_g,30**: design acceleration at 30% probability over 50 years
- **F0**: short-period amplification factor
- **Tc***: corner period separating short- and long-period behavior

The NTC2018 spectrum for 5% damping is:

For $T \leq TB$:

$Se(T) = a_{g,30} \times F0 \times (T / TB)$

For $TB < T \leq TC$:

$Se(T) = a_{g,30} \times F0$

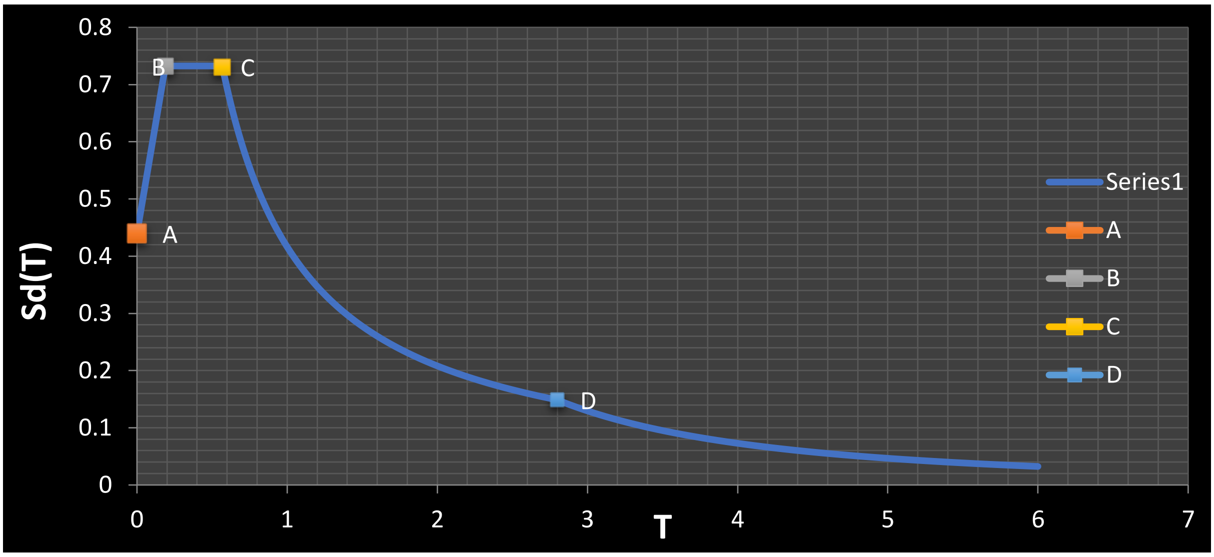
For $T > TC$:

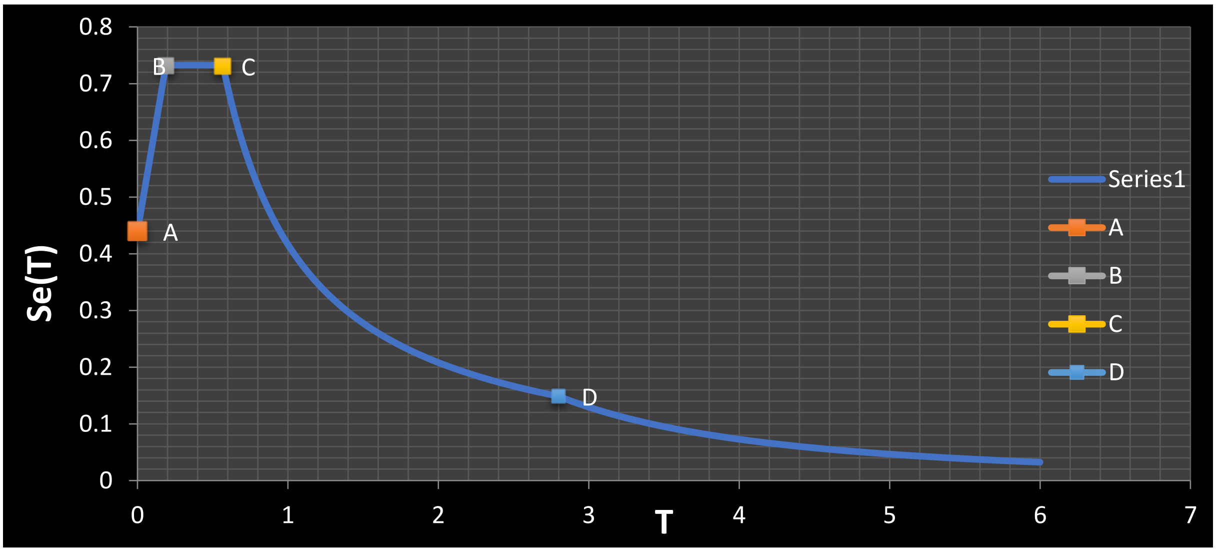
$Se(T) = a_{g,30} \times F0 \times (TC / T)$

- **Damping correction factor:**

  $\eta = \sqrt{[10 / (5 + \xi)]}$ ξ = actual damping (%)

10.4 Spectrum PLOT for NTC2018





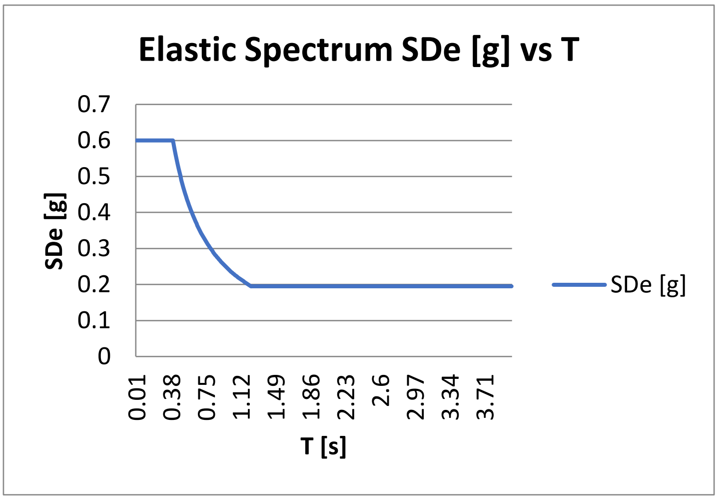
10.5 Input Parameters for KTP-89 Spectrum – PGA, Soil Type

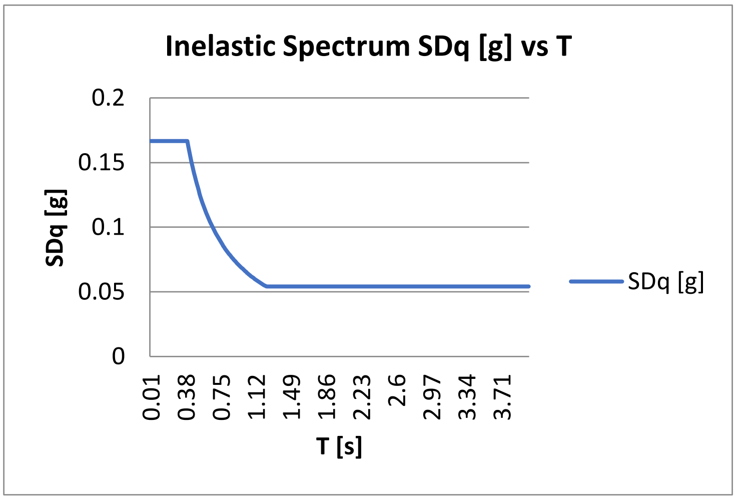
- **PGA**: historical peak ground acceleration for Tirana (≈0.25–0.3g)
- **Soil type**: simplified classification (rock, stiff soil, soft soil)

The KTP-89 spectrum uses a simplified **tri-linear shape**:

For T ≤ T1: S(T) = linear increase to S_max
For T1 < T ≤ T2: S(T) = constant S_max
For T > T2: S(T) = linear decrease

10.6 Spectrum PLOT for KTP-89





10.7 Discussion of Differences – Key Observations

- **EC8 and NTC2018**: higher accelerations for short-period buildings due to modern hazard assessment.
- **KTP-89**: underestimates accelerations for tall or flexible buildings; simpler soil assumptions.
- Differences in **corner periods, damping, and amplification factors** influence base shear, displacements, and reinforcement requirements.

11. STEP-BY-STEP MODELLING AND ANALYSIS

11.1 Introduction

The way we approach a structural model usually follows a logical sequence. As structural engineers, the first thing we do is try to understand the building as a whole — its geometry, its function, and the way it will behave under loads. Only after this do we translate it into a numerical model. The steps below describe this process in a practical manner, showing how the design evolves from architectural drawings to a complete analysis.

11.2 Project Setup

The first step is always setting up the project environment. We select the unit system, decide how the model will be organized, and prepare a clean grid layout to match the architectural drawings. A well-structured start makes the later stages easier to handle.

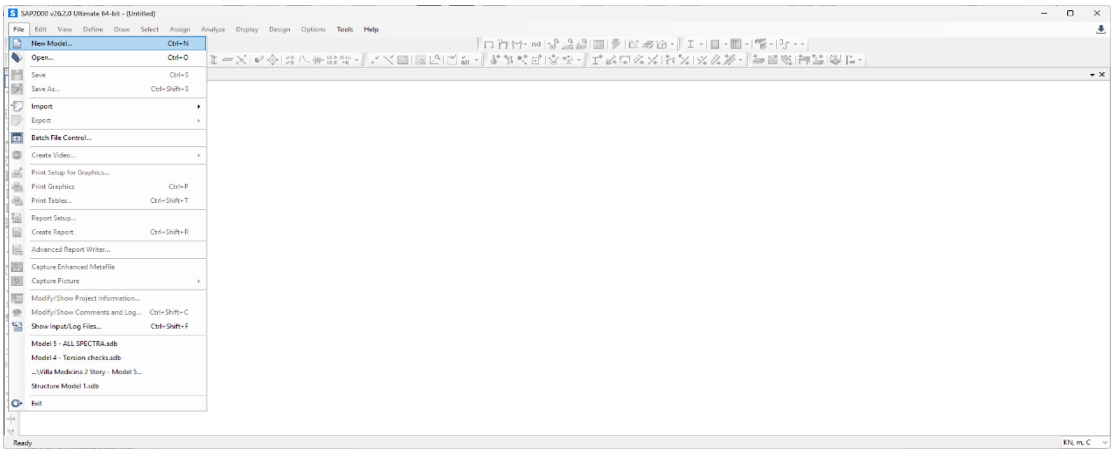


Figure 11.2.1. Starting the SAP2000 software.

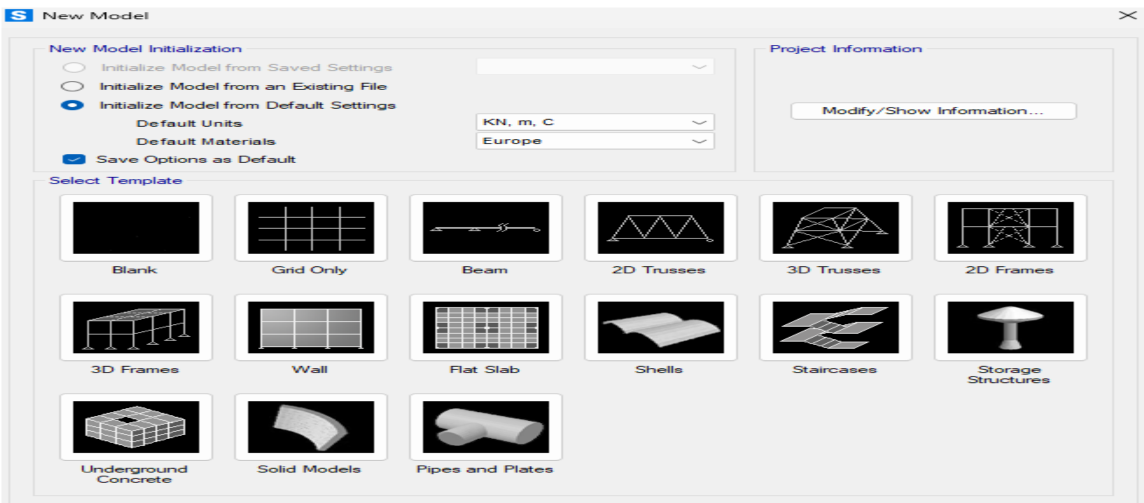


Figure 11.2.2. New Model Units/Materials/Template definition.

11.3 Geometry Definition

Once the base is ready, we move on to the geometry. The first thing we draw is the grid and the story levels, since these define the building's skeleton. On these grids, we place the columns, beams, and walls, making sure that they align with the architecture. Then we model the slabs, usually assigning them as rigid diaphragms to capture in-plane stiffness. At this stage, the model starts to look like the actual structure.

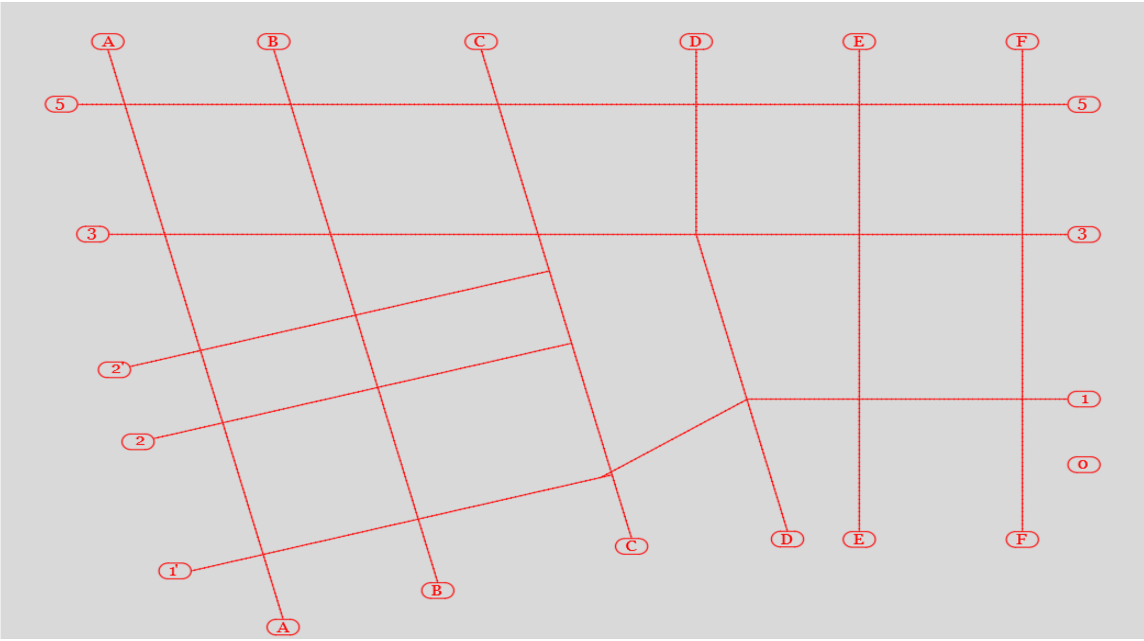


Figure 11.3.1. From Architectural Axis to Structural Axis.

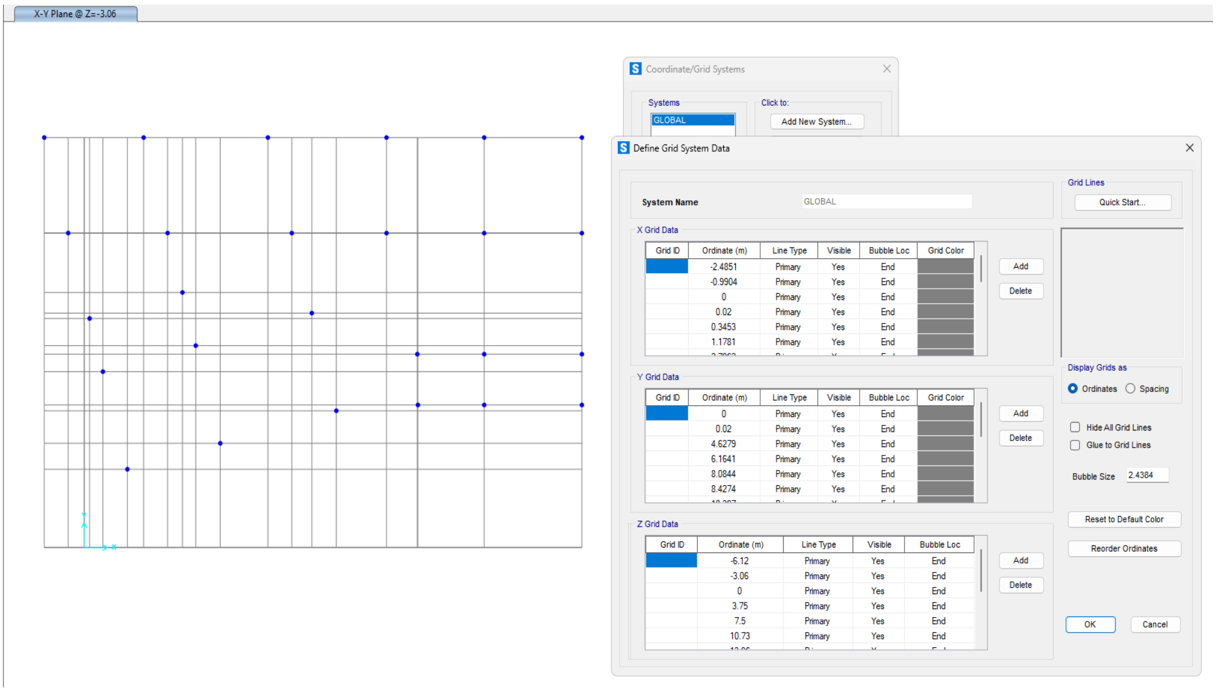


Figure 11.3.2. Structural Axis definition in the FEM Software SAP2000.

11.4 Materials and Sections

After the skeleton is in place, we assign materials. In our case, vertical elements such as columns and walls are designed in **C30/37 concrete**, while beams and slabs use **C25/30 concrete**. For reinforcement, we take steel grade B450C, which is the standard in both EC8 and NTC2018. Dimensions are chosen according to the preliminary design of beams and columns using excel sheets program.

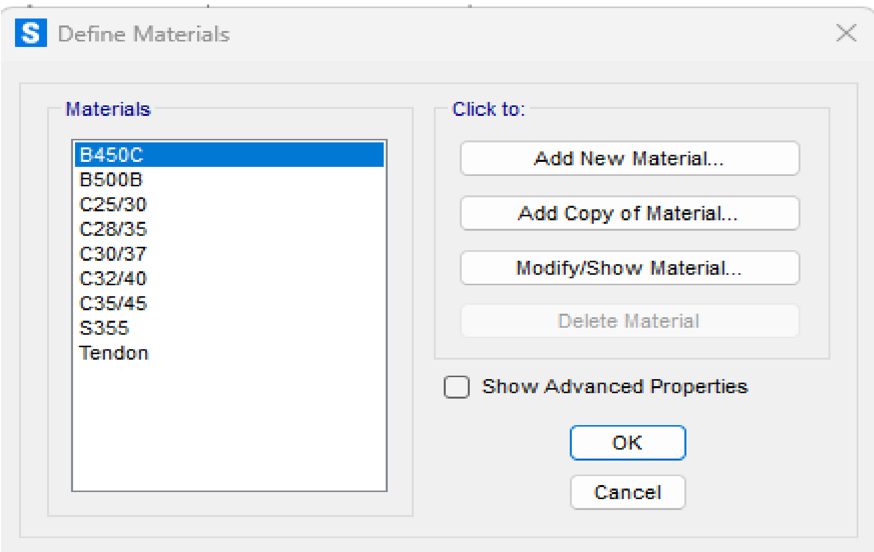


Figure 11.4.1. Structural Materials definition in the FEM Software SAP2000.

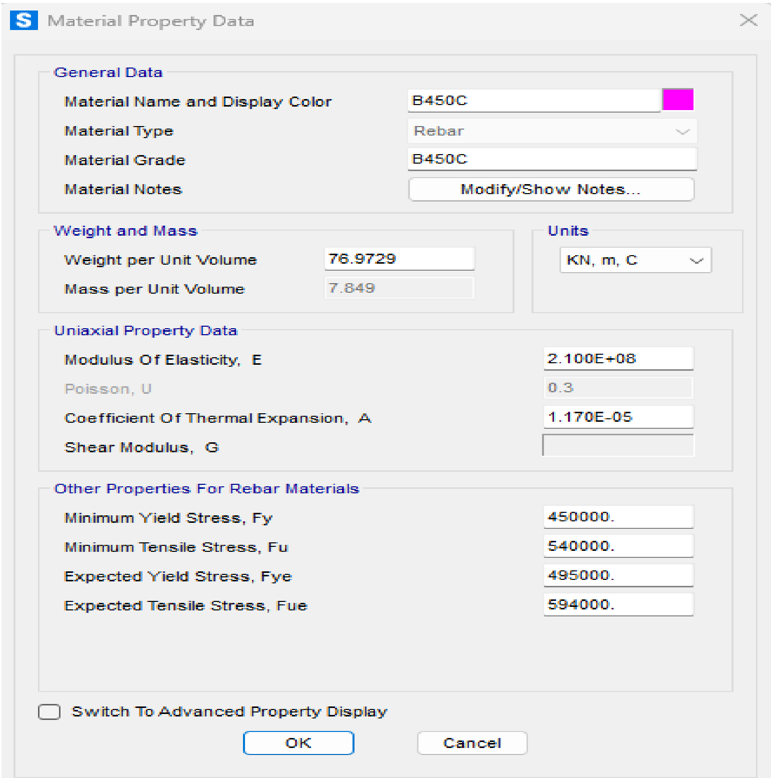


Figure 11.4.2. B450C rebar definition in the FEM Software SAP2000.

Figure 11.4.3. C25/30 Concrete (slabs and beams) definition in the FEM Software SAP2000

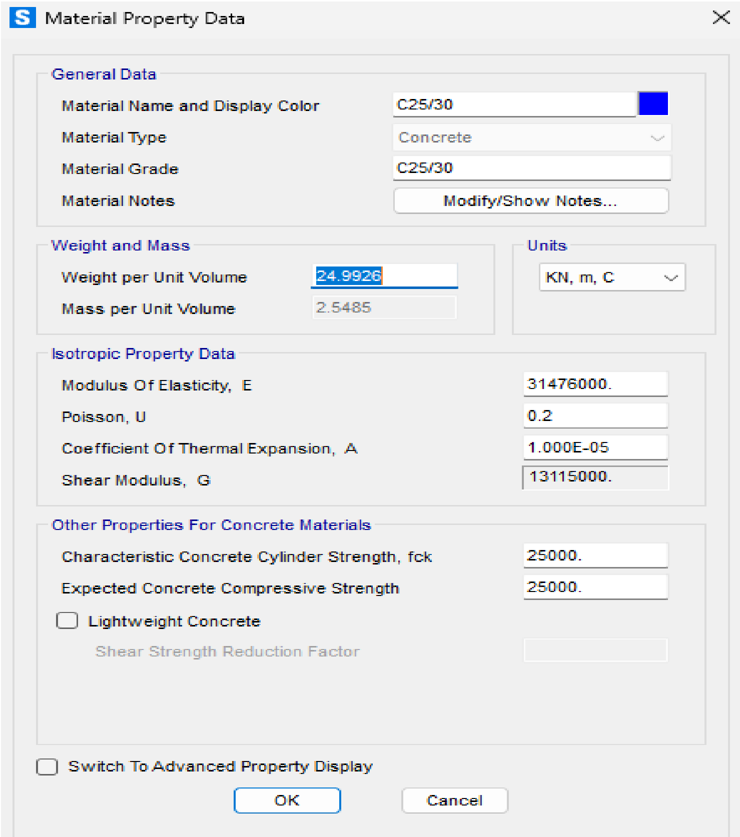
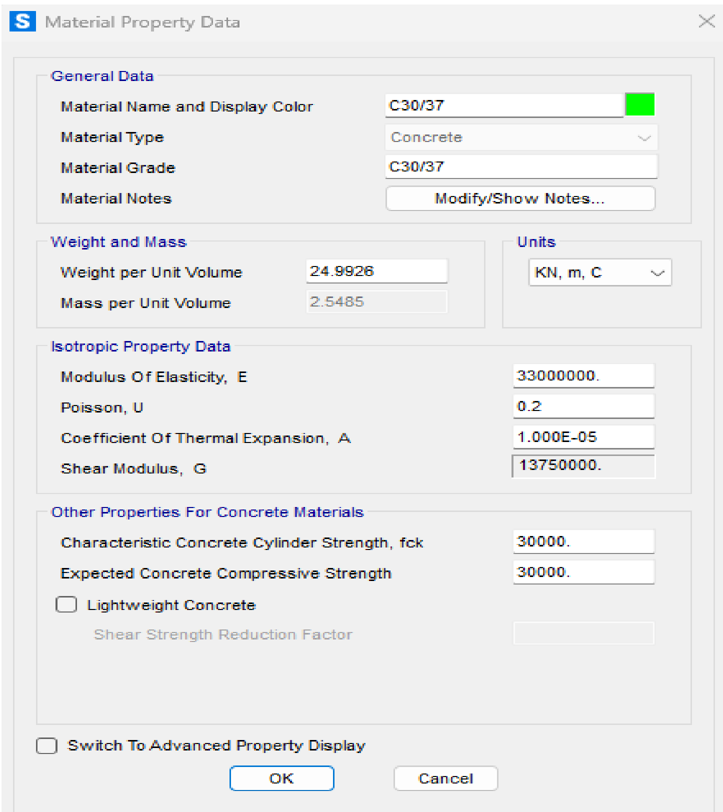


Figure 11.4.3. C30/37 Concrete (Columns and Shear Walls) definition in the FEM Software SAP2000



11.4.1 Preliminary Design of the Columns

Preliminary Design of the Columns is done based on the formulas provided in the Eurocodes, this is a general excel sheet containing the dimensioning of the centered columns, the same setup is followed for the edged and the cornered columns. In this proogram only static loads are taken into account and for that reason during the analysis the columns dimensions had to be changed after applying the seismic conditions.

When we design reinforced concrete columns, the process is really about balancing three things:

1. **The loads the column has to carry** (axial force + bending moments).
2. **The size of the section** (width and depth).
3. **The amount of reinforcement** (how many bars and their arrangement).

Eurocodes give us the exact rules, but in preliminary design we want something quick, safe, and practical — then later we refine it with detailed checks.

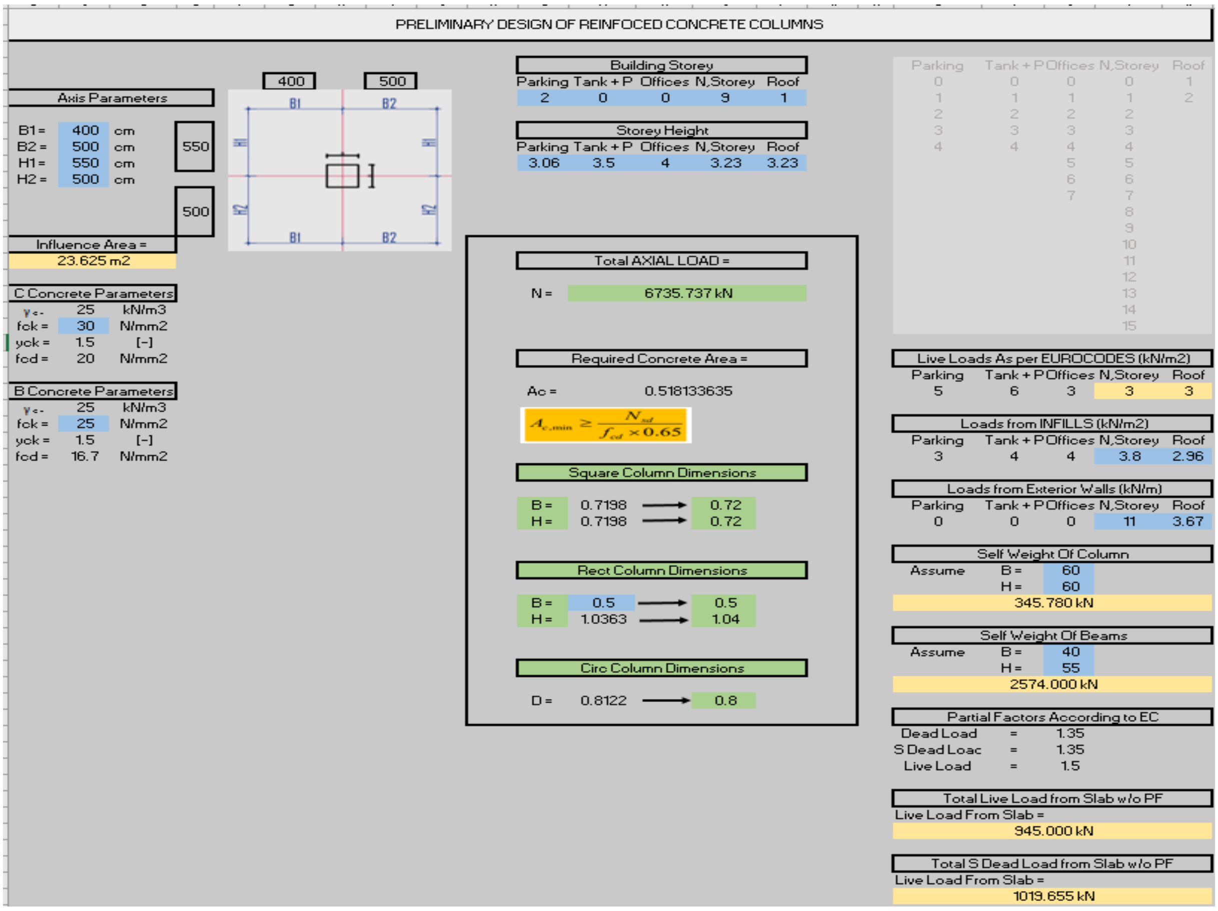


Figure 11.4.1.1. General Example of the Preliminary Dimensioning of the Concrete Columns.

11.4.2 Preliminary Design of the Beams

When we design beams, we mainly care about:

1. **Stiffness** (deflections and vibrations),
2. **Strength** (bending and shear),
3. **Practical reinforcement layout** (bars and stirrups).

Pre-dimensioning is about finding good **beam depth and width** before we run detailed analysis.

Span–depth ratios

Eurocode gives limits for slenderness (EN 1992 7.4.2)[8]. A safe way:

Simply supported beam:	**Continuous beam**:	**Cantilever**:
H ≈ L / 12 to 15	H ≈ L / 15 to 20	H ≈ L / 8 to 10

Width of the beam

Usually **b ≈ 0.4–0.6 h**.

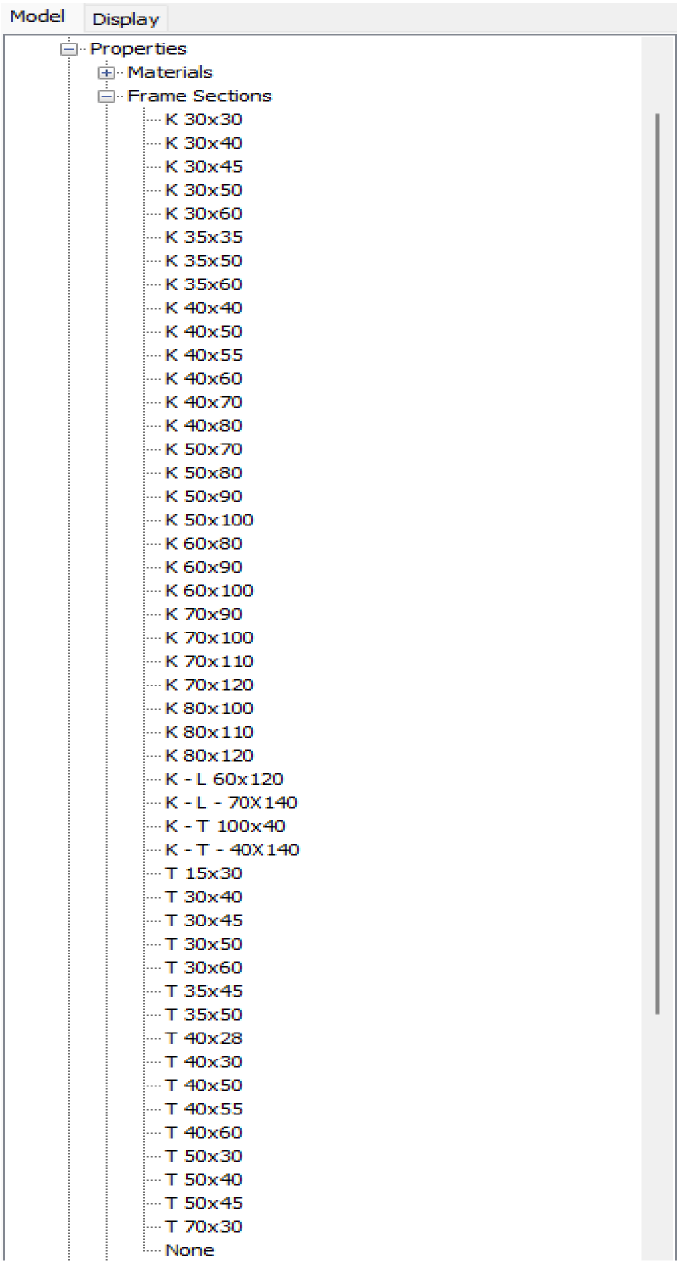
Practical limits: **25–30 cm** minimum width for normal spans.

Wider beams if they support thick slabs or need to carry big shear forces.

After the preliminary dimensioning of the beams we have created their sections in the FEM Software.

11.5 Materials and Sections

After the skeleton is in place, we assign materials. In our case, vertical elements such as columns and walls are designed in **C30/37 concrete**, while beams and slabs use **C25/30 concrete**. For reinforcement, we take steel grade B450C, which is the standard in both EC8 and NTC2018. Dimensions are chosen as in the following figures, where with T- mark we have defined the Beams and with the K- mark we have defined different columns sizes. The same logic is used to define the slabs using Sol- Mark, Walls using M- mark, Ramps and Stairs using (Shkalla [ALB])



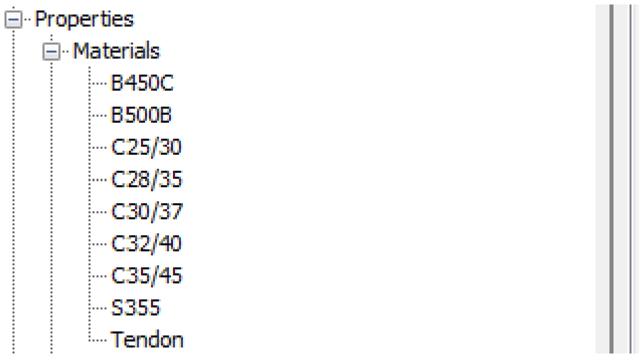


Figure 11.5.2. Material Properties Definition

Figure 11.5.2. Section Definiton for Slabs, Walls, Stairs, Ramps.

Figure 11.5.1. Section Definiton for Columns And Beams.

11.6 Stiffeness Modifiers Eurocodes Approach

When we model a structure in software (like SAP2000, ETABS, etc.), the computer normally assumes that the beams, columns, and slabs are perfectly rigid in bending, shear, and torsion, based on their geometric properties (like EI), but in reality, this is not completely true: **Cracks form in concrete** under load, reducing stiffness.

Shear deformations are often larger than what "perfect" theory predicts.

Long-term effects like creep and shrinkage make concrete more flexible over time.

The Eurocodes (especially **EN 1992-1-1**)[8] recognize this and allow us to modify stiffness values to better reflect reality. In Practice we introduce **stiffness modifiers** – basically reduction factors that we apply to the theoretical stiffness:

For beams and slabs: Flexural stiffness EI is often reduced to about **30–50%** of the uncracked value. **For columns**: Because they are more compressed and less cracked, their flexural stiffness may be reduced to about **70–100%**. **For shear and torsion**: Usually taken lower, depending on detailing, sometimes down to **50% or less**.

It avoids **overestimating stiffness** and therefore **underestimating deflections**. It gives a **more realistic distribution of internal forces**, especially in seismic design where cracked stiffness is critical.

Element	Axial	Shear Area in 2 direction	Shear Area in 3 direction	Torsional Constant	Moment of Inertia about 2 axis	Moment of Inertia about 3 axis
Columns	1	0.8	0.8	0.5	0.7	0.7
Beams	1	0.8	0.8	0.5	0.5	0.5
Foundation Beams	1	1	1	0.4	0.5	0.5
Coupling Beams	1	1	1	0.1	0.2	0.2

Element	Membrane f11 Modifier	Membrane f22 Modifier	Membrane f12 Modifier	Bending m11 Modifier	Bending m22 Modifier	Bending m12 Modifier	Shear v13 Modifier	Shear v23 Modifier
Basement Wall	0.5	0.5	0.5	0.5	0.5	0.5	0.7	0.7
Shear Wall	0.5	0.5	0.5	0.5	0.5	0.5	0.7	0.7
Normal Slab	0.4	0.4	0.4	0.3	0.3	0.3	0.7	1
Waffle Slab	0.4	0.4	0.4	0.3	0.3	0.3	0.7	1
Stairs	0.4	0.4	0.4	0.3	0.3	0.3	0.7	1

Table 11.6.1. Table of stiffness modifiers for different materials (EC2)[8]

Figure 11.6.1. Definition of the stiffness modifiers for Frame type Column.

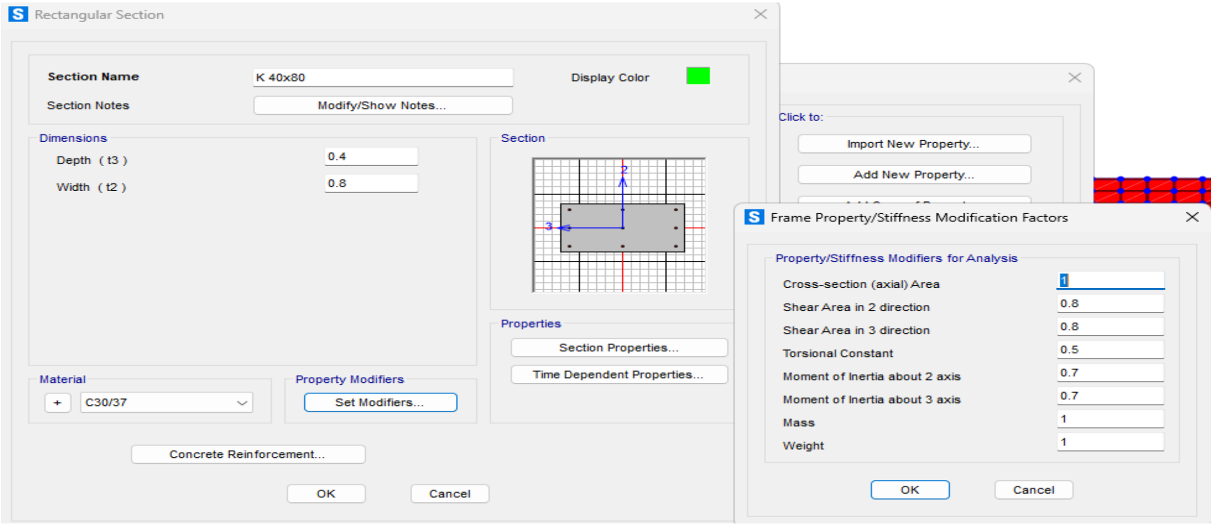


Figure 11.6.2. Definition of the stiffness modifiers for Frame type Beam.

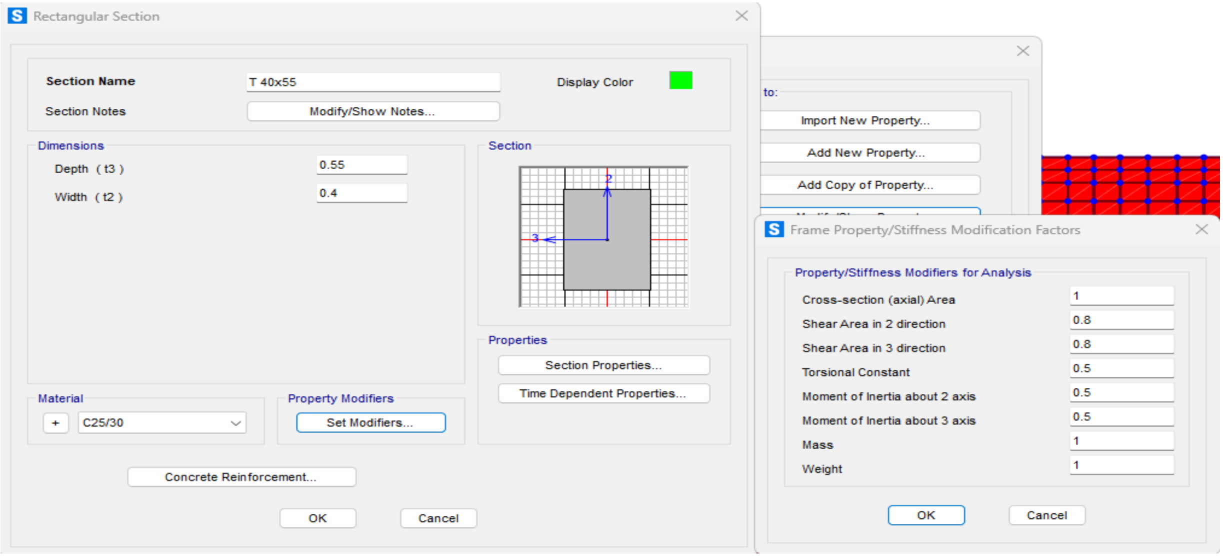
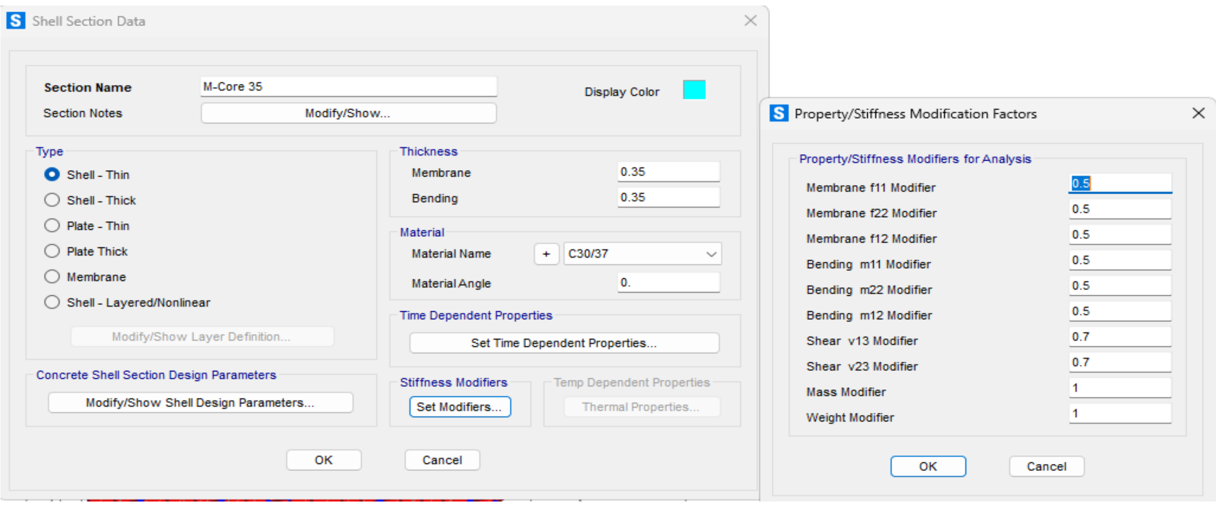


Figure 11.6.3. Definition of the stiffness modifiers for Shell type Wall.



11.7 3D Modelling of the Elements and the Structure

After defining all the parameters we have to draw the 3D concept of our Model based on the Architectural and the Structural Grids. After we created the so called grids in SAP2000 we have to draw the elements starting with columns and beams which are handled by the software as Frame elements, then we can model the Shell elements in our case walls, slabs, stairs and ramp are modelled.

For the Slab system we have used two types of slabs which are usually used in the construction technology in Albania, which are the flat slab with deep beams, and the waffle slab filled with hollow lightweighted brics.

The Flat Slab with deep beams is used up to the level of +3.75 and all the above slabs have been concepted as Waffle Slabs. The equivalent Heights are chosen as following:

H of Waffle Slab = 30cm (25cm + 5cm) | H of Normal Slab = 28cm

After the Static Load Analysis the Normal Slab was chosen at the height of 28cm because of the deflection caused by the office and the parking loads.

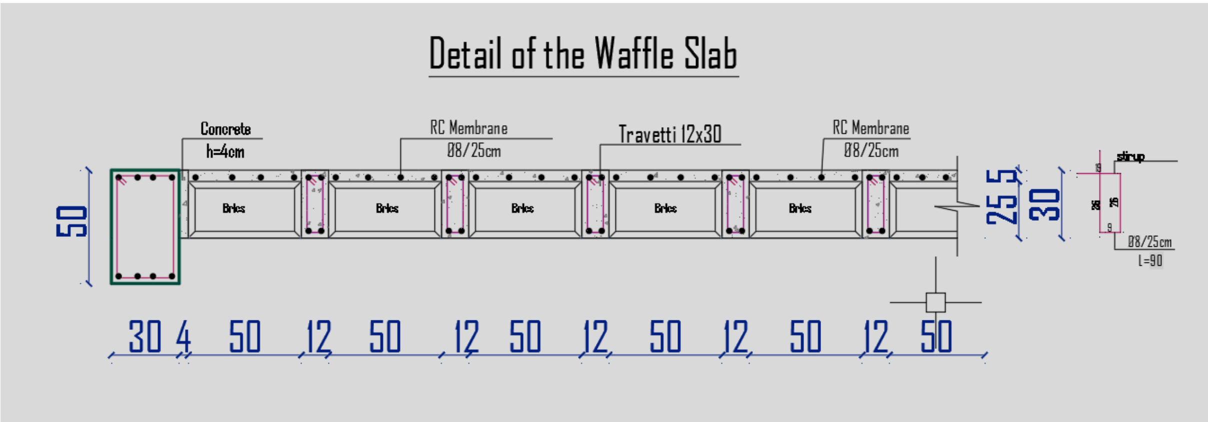


Figure 11.7.1. Detail of the Waffle Slab system used commonly in Albania.

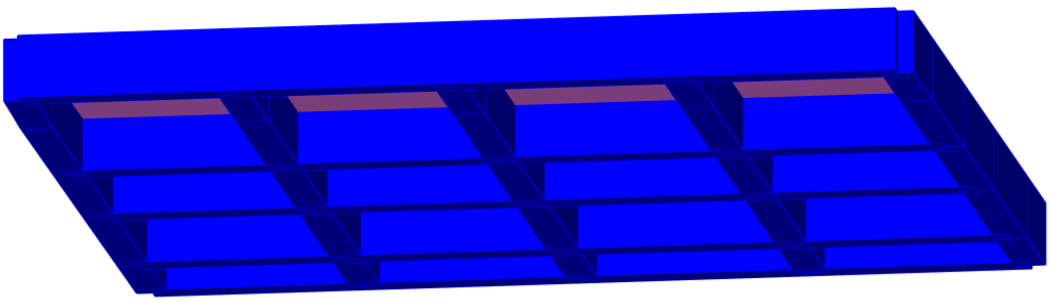


Figure 11.7.2. Idealization of a portion of the slab in SAP2000 FEM Software

Figure 11.7.3. Idealization of a of the Normal slab with deep Beams in SAP2000 FEM Software.

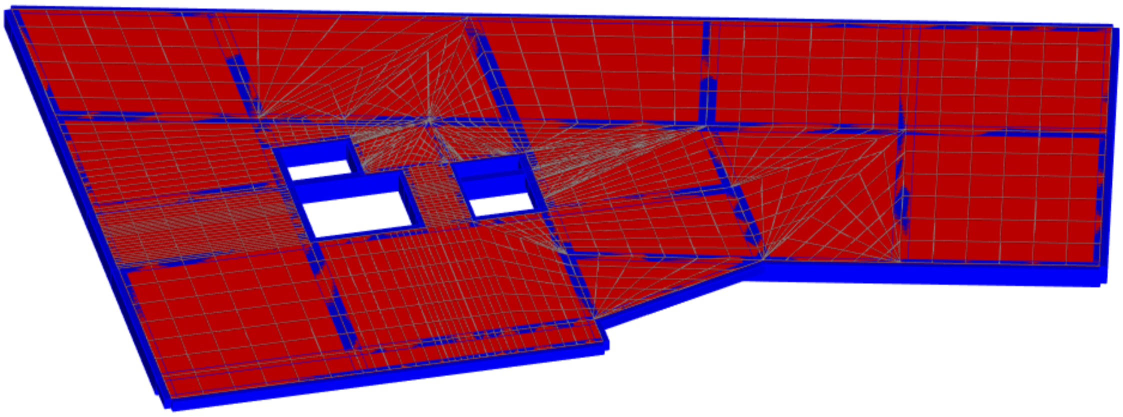
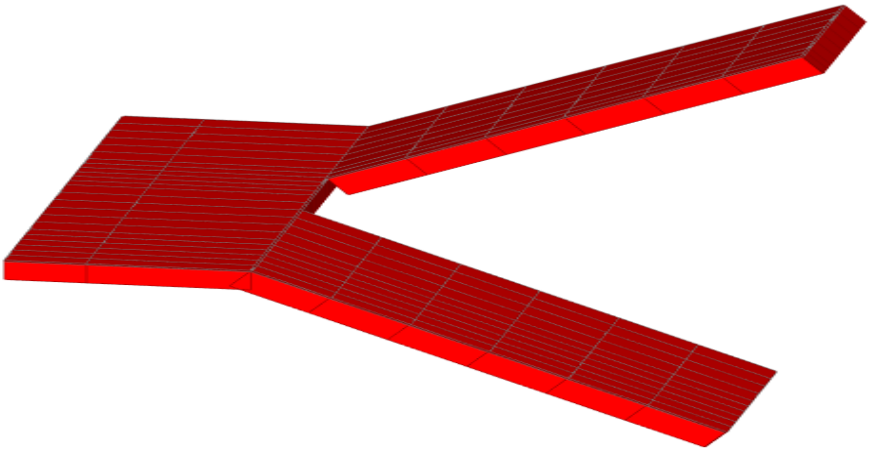


Figure 11.7.4. Idealization of a of a part of the Stairs in SAP2000 FEM Software.



For the Stairs and the Ramp it has been used a Shell elment with H = 20cm

Figure 11.7.5. Idealization of a of the Ramp in SAP2000 FEM Software.

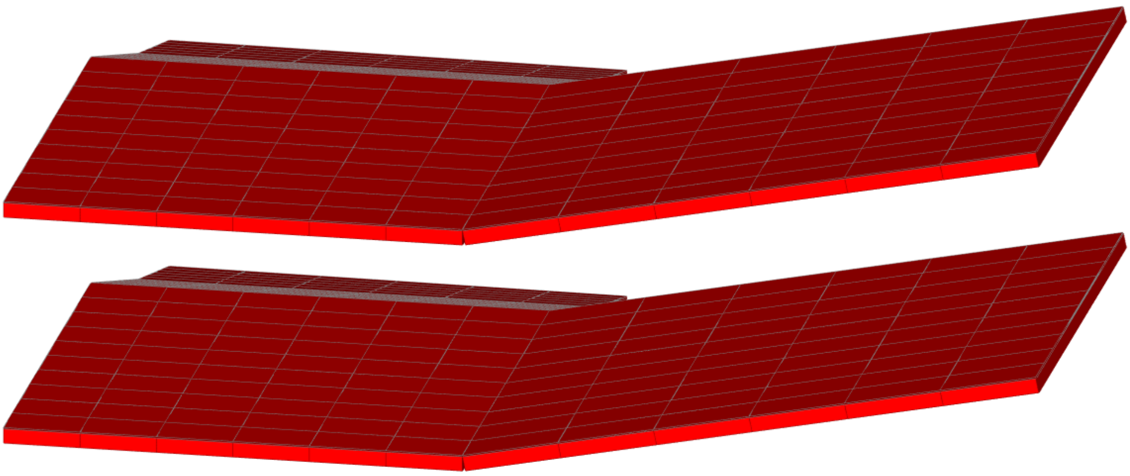
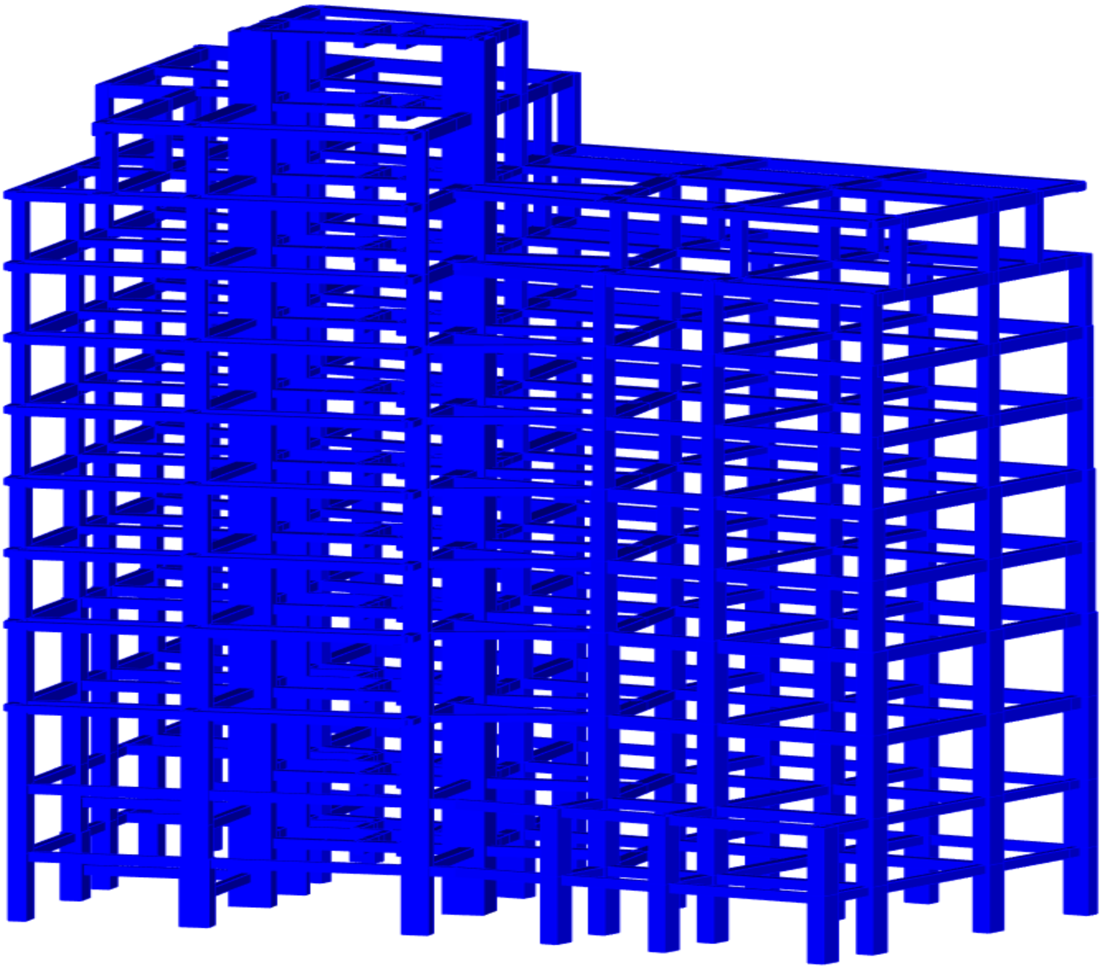


Figure 11.7.6. 3D Modelling of the Frame Elements (Columns + Beams) in the SAP2000 FEM Software.



Every 3 stories the Section of the Columns have been reduced to comply with the economical, architectural and the Structural Loading conditions.

The Corners of the Lift Core have been idealized as L or T Columns for a better P-M-M interaction between the Shell Elements and the Frame Elements.

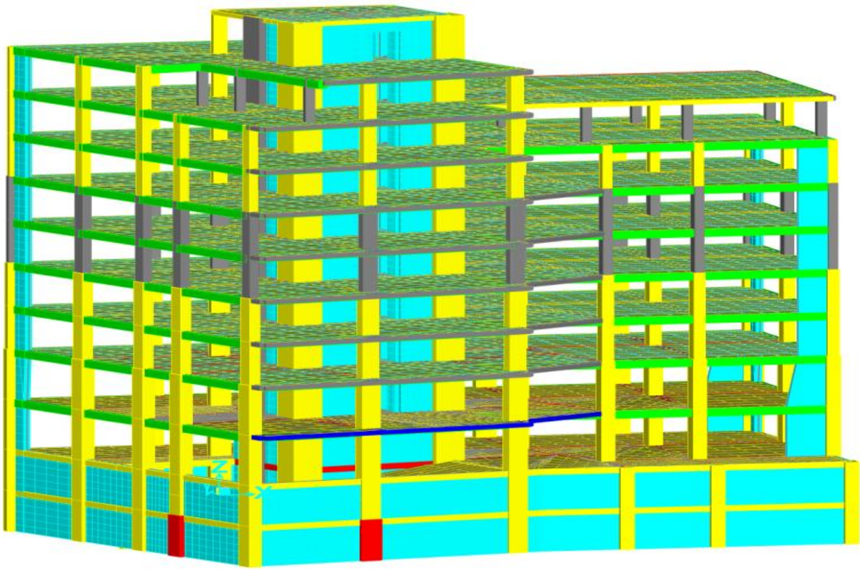


Figure 11.7.6. 3D Modelling of the Full Structure in the SAP2000 FEM Software

- L section Columns used in the Elevator Core.

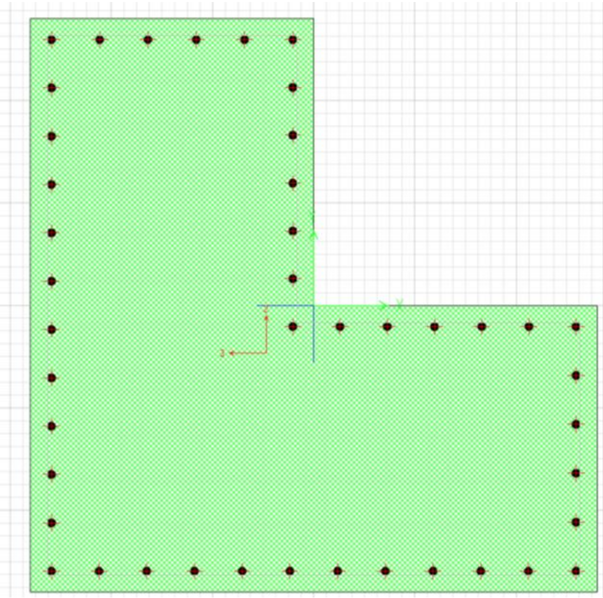


Figure 11.7.7. Modelling of the L shaped Column using the Section Designer Tool inside SAP2000.

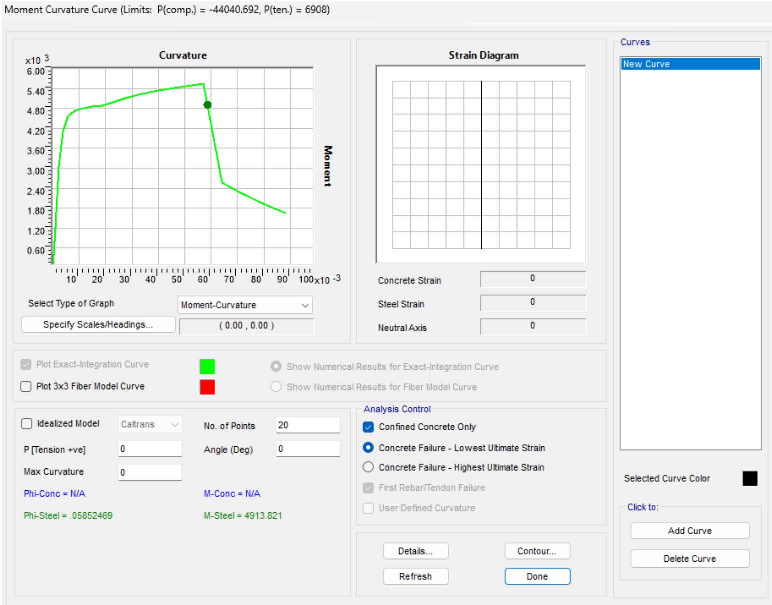


Figure 11.7.8. Moment Curvature Relationship Curve for the L shaped Column.

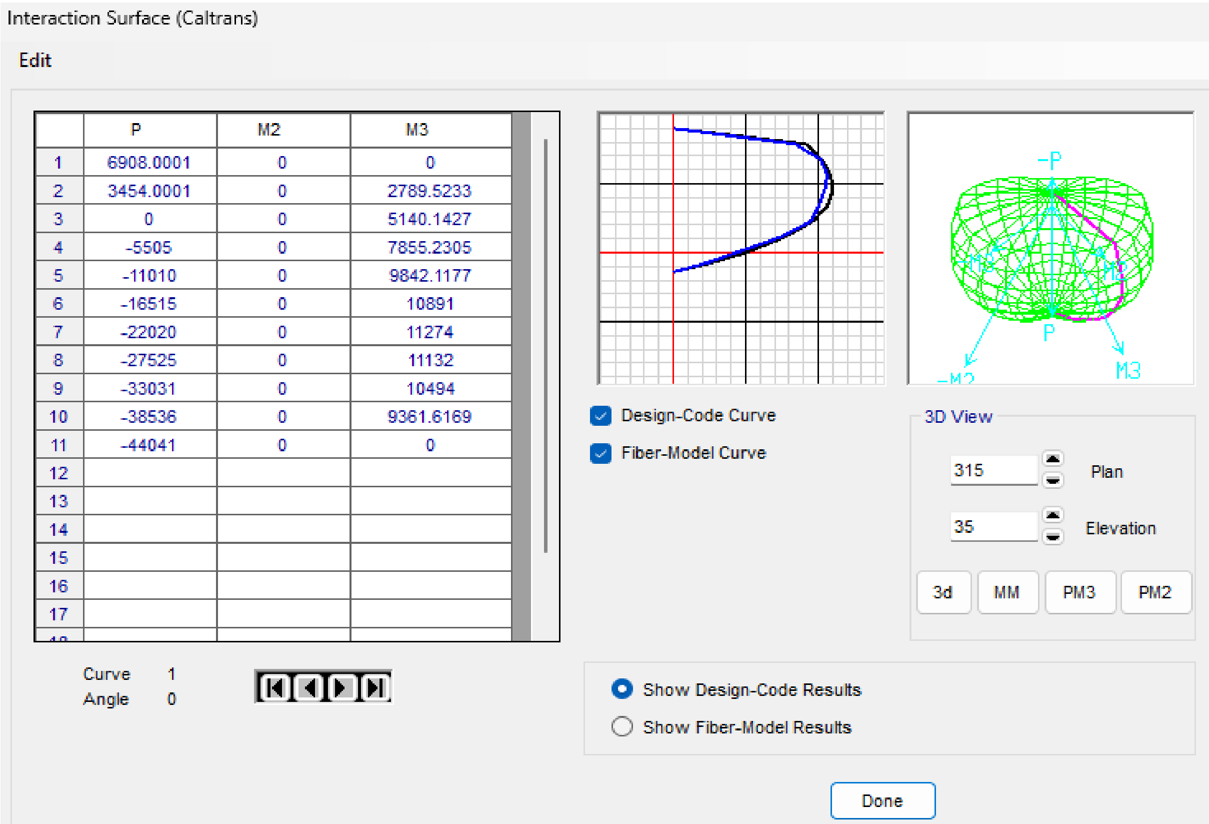
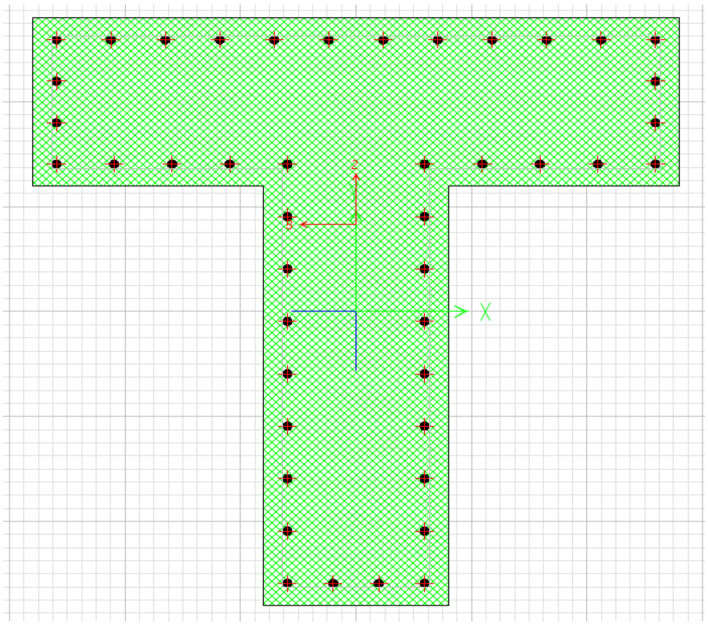


Figure 11.7.9. Interaction Surface of the L shaped Column.

- T section Columns used in the Elevator Core.



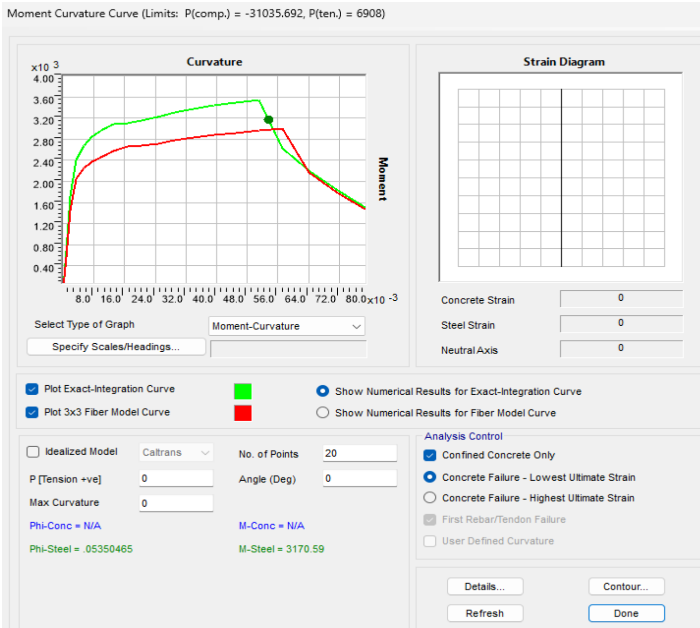
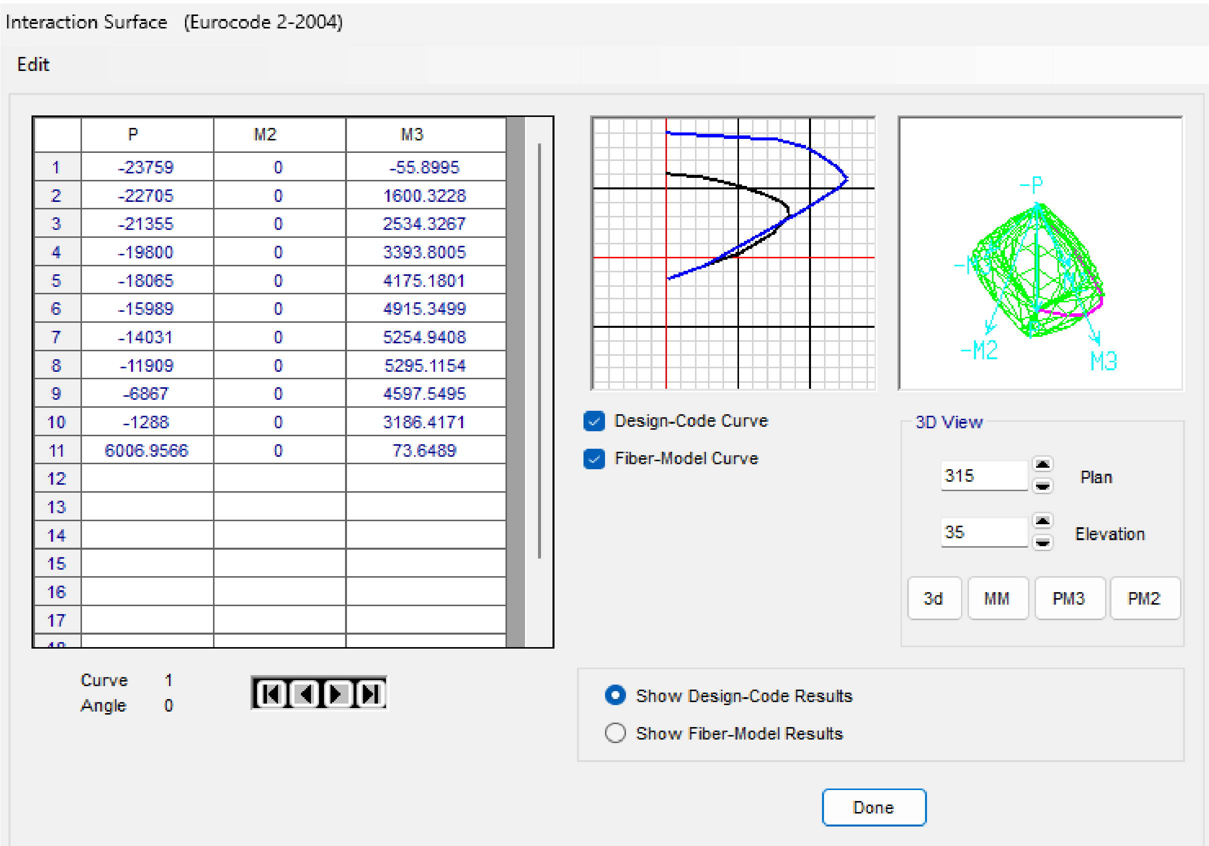


Figure 11.7.7. Modelling of the T shaped Column using the Section Designer Tool inside SAP2000.

Figure 11.7.8. Moment Curvature Relationship Curve for the T shaped Column.



	P	M2	M3
1	-23759	0	-55.8995
2	-22705	0	1600.3228
3	-21355	0	2534.3267
4	-19800	0	3393.8005
5	-18065	0	4175.1801
6	-15989	0	4915.3499
7	-14031	0	5254.9408
8	-11909	0	5295.1154
9	-6867	0	4597.5495
10	-1288	0	3186.4171
11	6006.9566	0	73.6489
12			
13			
14			
15			
16			
17			

Figure 11.7.9. Interaction Surface of the L shaped Column.

Figure 11.7.10. The Perimetral Wall T = 25cm modelled in the SAP2000 Software.

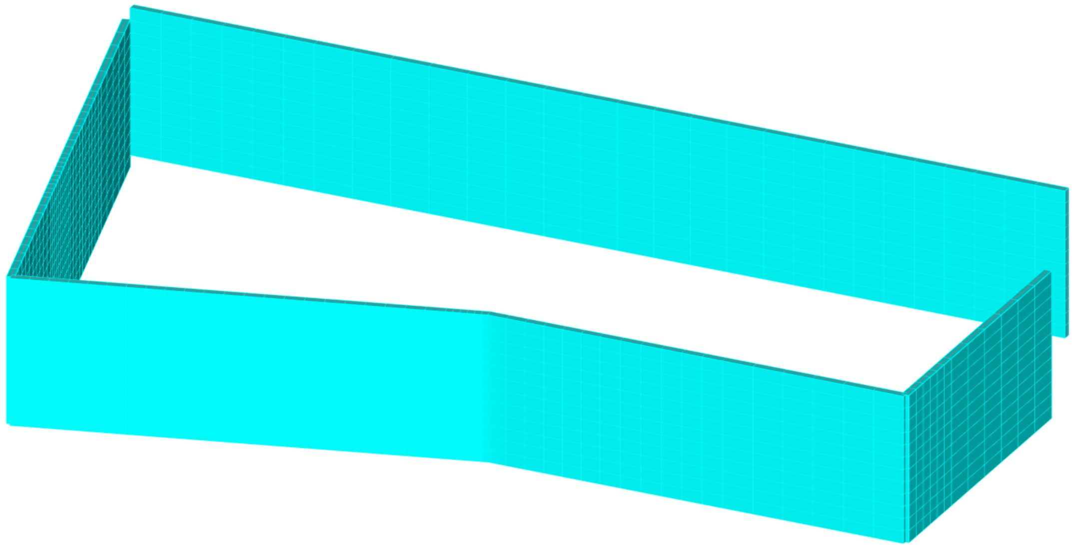
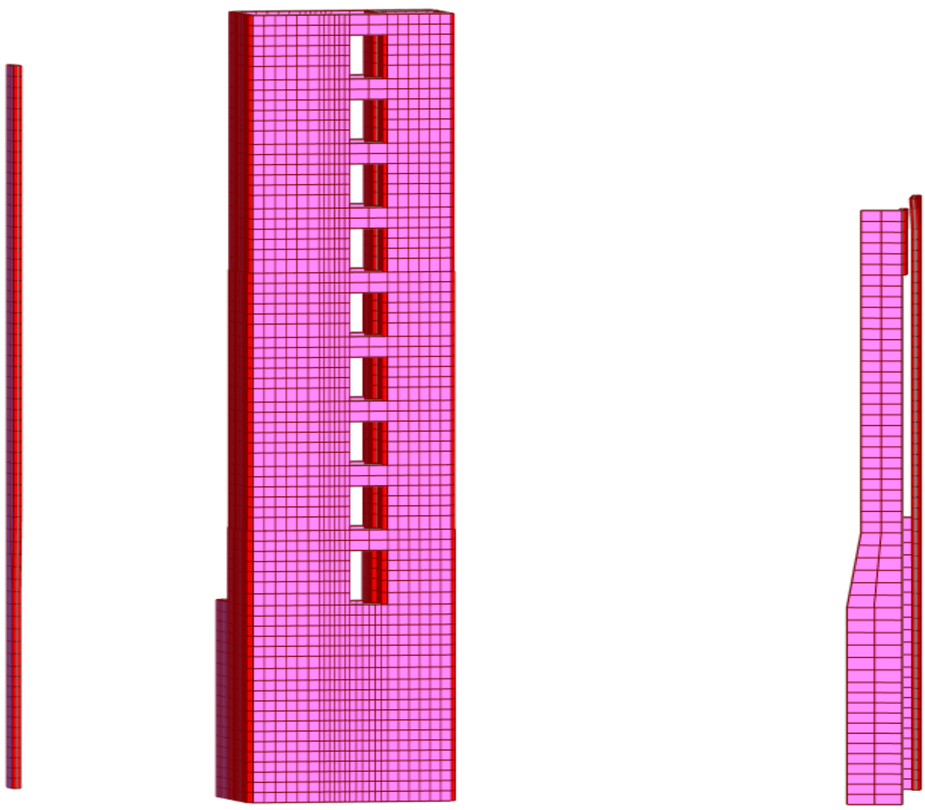


Figure 11.7.11. Shear Walls and the Elevator Core modelled in the SAP2000 Software,



The Shear Walls have been created using Shell elements and their thickness ranges from 30cm in the lower stories up to 25cm in the higher stories, and for the Elevator Core Walls, they have been created using Shell elements and their thickness ranges from 40cm in the lower stories up to 25cm in the higher stories.

11.8 Load Analysis & Loads applied to the 3D Model

The Self Weight of the Structural Elements it is automatically calculated by the Software.

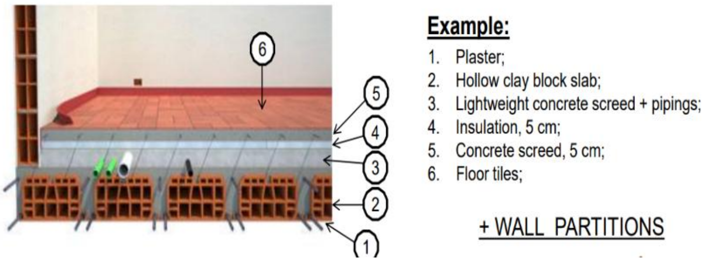
Loads applied to the Normal Slabs H = 28cm

Parking	
Load Definition	Value kN/m2
Live Loads =	5
Dead Loads =	3
Car Loads =	5

Offices	
Load Definition	Value kN/m2
Live Loads =	5
Dead Loads =	3.5

Loads applied to the Waffle Slabs H = 25+5cm

Apartaments	
Load Definition	Value kN/m2
Live Loads =	3
Dead Loads =	2.5
Infill Loads =	1.5



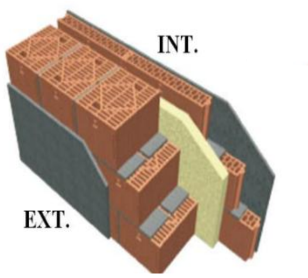
Loads applied to the Stairs H = 20cm
Loads applied to the Ramp H = 20cm

Stairs	
Load Definition	Value kN/m2
Live Loads =	5
Dead Loads =	3

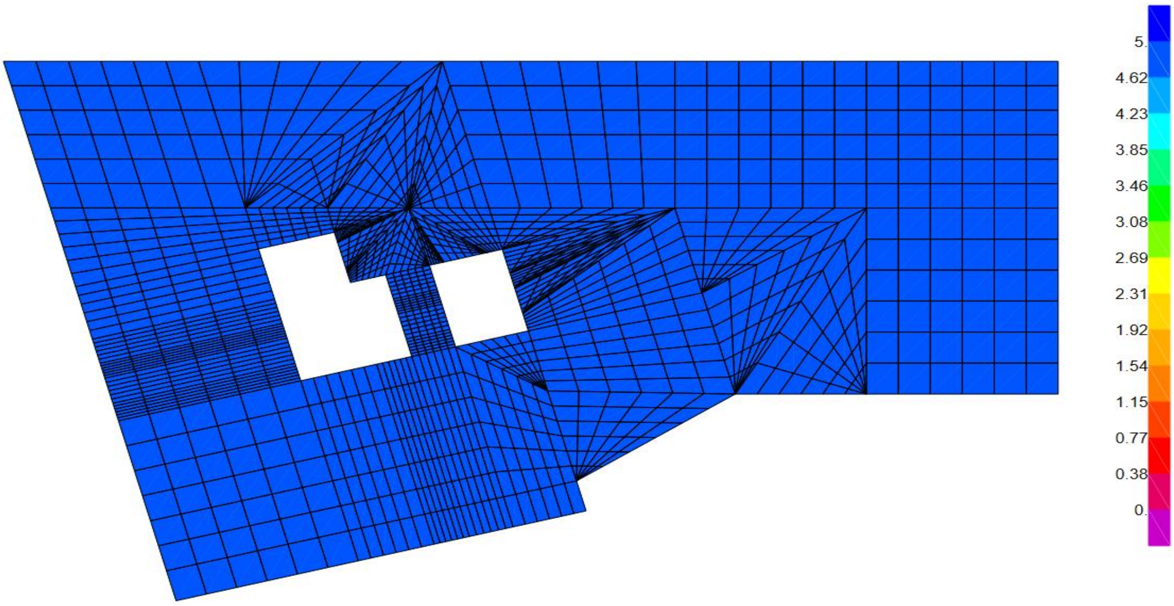
Ramp	
Load Definition	Value kN/m2
Live Loads =	5
Dead Loads =	5
Car Loads =	5

Loads applied to the Perimeter Beams (Exterior Walls) in kN/m

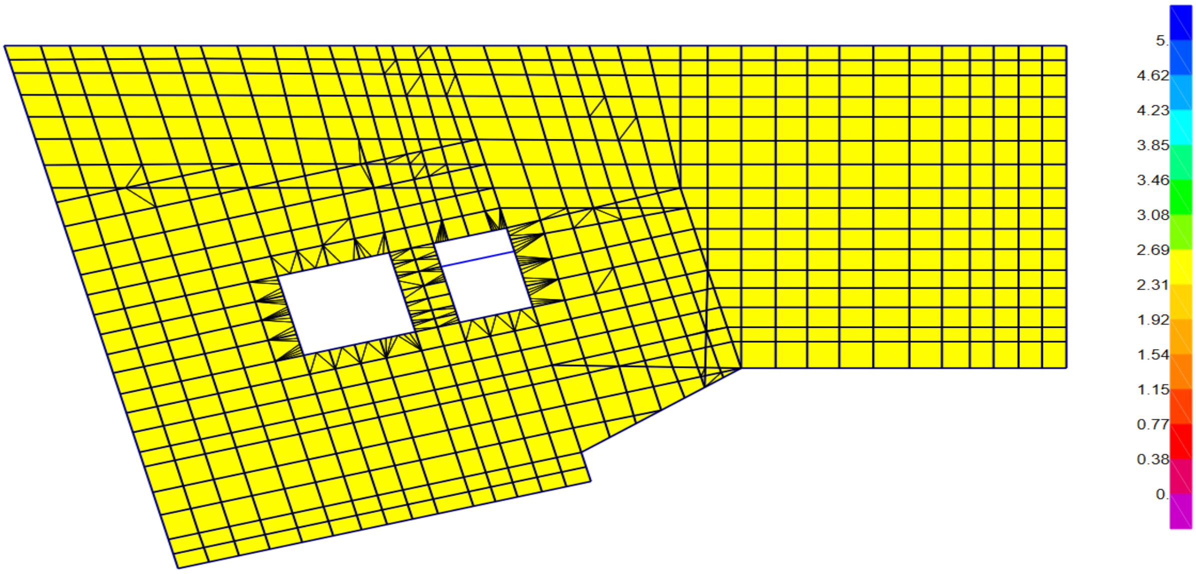
Non Structural Loads	Thickness [cm]	Density [kn/m3]	Weight [kN/m2]	H [m]
Plaster	2	20	0.4	3.23
Hollow Blocks	10	5	0.5	
Steam barrier			0.04	
TH Insulation panel	8	1	0.08	
Hollow Blocks	25	8	2	
Plaster	2	20	0.4	
TOTAL G2			3.42	11.05



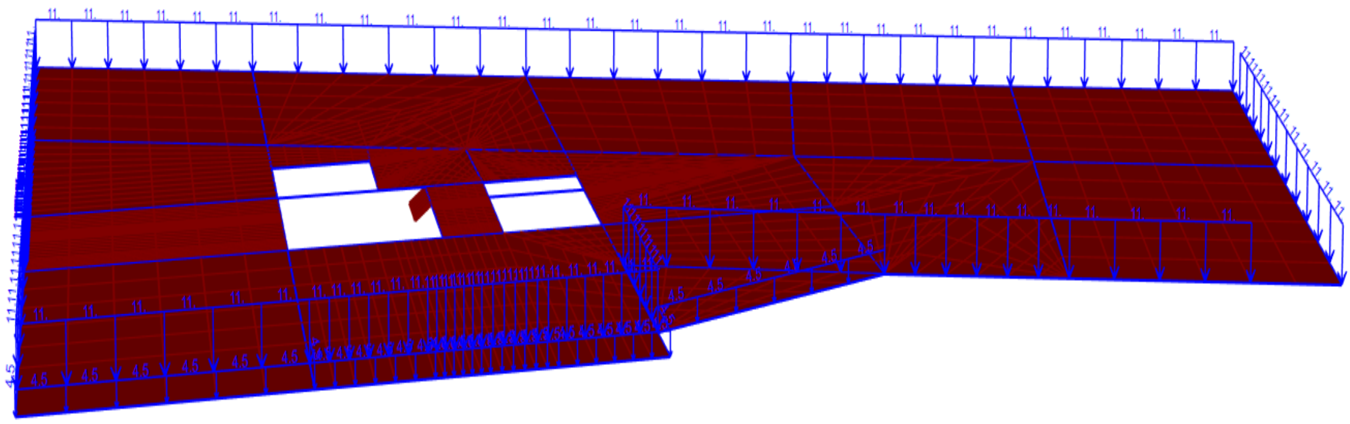
- Example of Live Loads being applied to the Normal Slab.



- Example of Dead Loads being applied to the Waffle Slab.



- Example of Dead Loads being applied to the Perimeter Beams.



11.9 Step by Step Definition of the Analysis Options in Sap2000

11.9.1 Load Patterns

When setting up the analysis model, one of the first things we define are the **load patterns**. These patterns represent the different types of actions that act on the structure, and they form the basis for load cases and combinations. In my model, I defined five principal load patterns:

Dead - This pattern accounts for the **self-weight of the structure**. It includes the weight of beams, slabs, columns, and walls. Since SAP2000 can automatically calculate self-weight based on material density and geometry, the Dead load pattern ensures that the most fundamental and unavoidable load is always included.

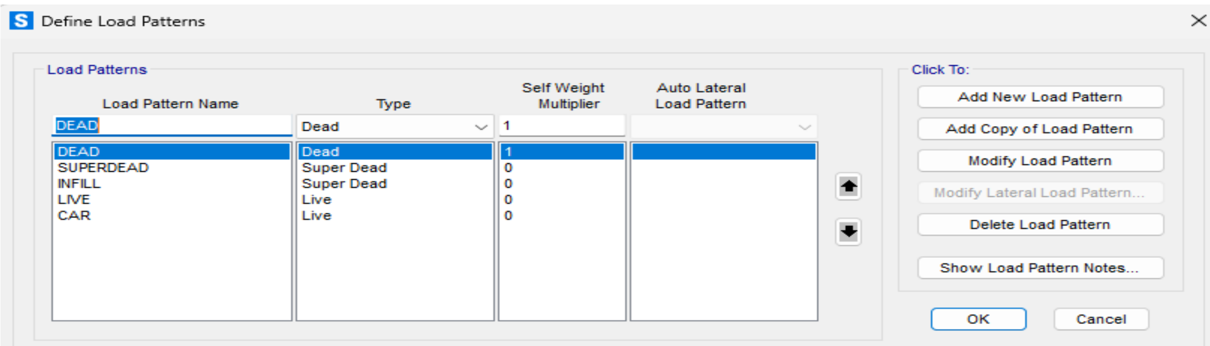
SuperDead - represents the **superimposed permanent loads**, which are not automatically included by the software. These are things like floor finishes, screeds, ceiling layers, and façade elements. They are permanent in nature but applied separately so they can be clearly distinguished from the structural self-weight.

Live - represents **occupancy loads**. These are variable actions due to people, furniture, and general usage of the building. Their magnitude depends on the building's function — offices, residential, parking, etc. Live loads are not always present at maximum intensity, so only a fraction of them is usually included in the seismic mass, following code rules.

Infill - Infill accounts for the **non-structural partition walls and masonry infills**. Even though these elements are not explicitly modeled as structural components, their weight contributes to the total mass of the building and therefore to its seismic response. Defining them as a separate pattern makes it easier to manage and apply them floor by floor.

Cars - used in the **basement and parking levels**. It represents the imposed load of vehicles, which is typically heavier than standard live load in residential or office areas. The model can capture the difference between normal occupancy loads and concentrated loads from vehicles.

Figure 11.9.2. Mass Source defined in the SAP2000 Software.



11.9.2 Mass Source

When we talk about dynamic analysis in programs like SAP2000, the first thing we have to define is **where the building's mass comes from**. This is important because the seismic forces in a response spectrum analysis are directly proportional to the mass of the structure. If the mass source is defined incorrectly, the entire seismic response will be unreliable.

In practice, the **mass source** is made up of:

Self-weight of structural elements (concrete beams, slabs, columns, and walls).

Superimposed dead loads (finishes, partitions, facade elements).

A **portion of the live load**, since not all live loads are likely to be present during an earthquake. Codes such as EC8 and NTC2018 specify this reduction using a factor ψ (often between 0.3–0.5 depending on occupancy).

If the mass source is too small (e.g., you forget to include partitions or part of the live load), the program will underestimate the seismic forces. If it's too large, it will overestimate them. Both situations lead to a design that doesn't reflect reality. For this reason, defining the mass source is one of the most critical steps before running modal or spectrum analyses.

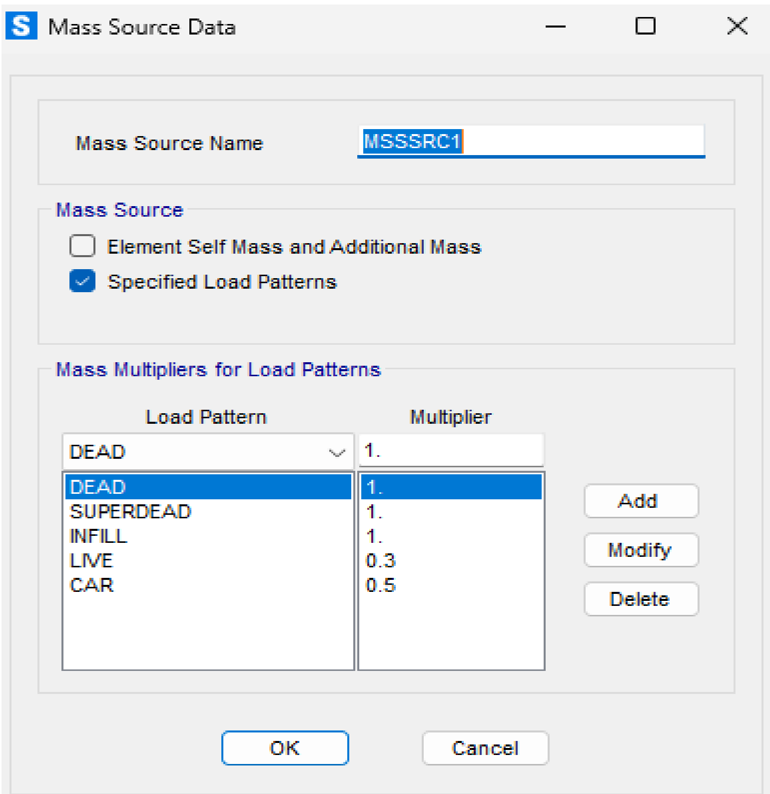


Figure 11.9.2. Mass Source defined in the SAP2000 Software.

11.9.3 Response Spectrum

Eurocode 8 Response Spectrum Definition in SAP2000

In Eurocode 8, the **design response spectrum** is the starting point for seismic analysis. Instead of applying a single static load, the spectrum defines how different structures — depending on their natural period — will respond to the same ground motion. In SAP2000, we translate this mathematical definition into a **tabular function** (period vs spectral acceleration), which the software then uses for modal and response spectrum analyses.

When defining the spectrum, we start with the input parameters prescribed by Eurocode 8:

- **Peak Ground Acceleration (a_g)**: site-specific acceleration value.
- **Soil Class (A–E)**: determines amplification factors and corner periods.
- **Behavior Factor (q)**: used later for design, not in the elastic spectrum itself.

The spectrum is piecewise, with three main branches:

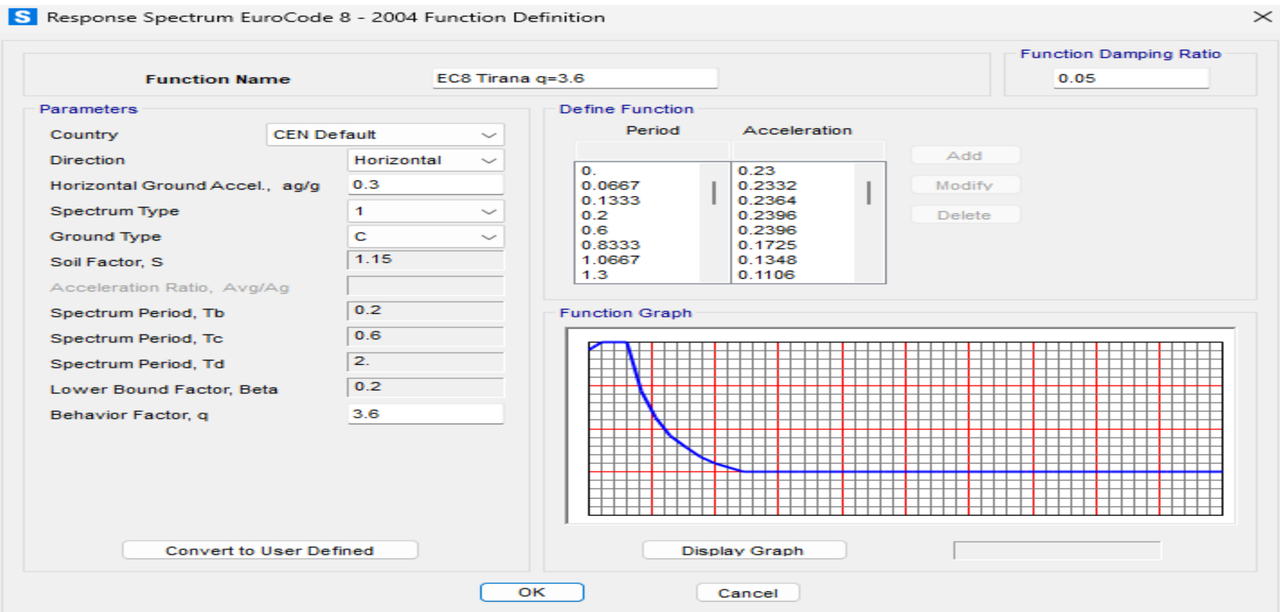
Rising branch (0 ≤ T ≤ TB): acceleration increases linearly with period.

Plateau (TB < T ≤ TC): constant maximum spectral acceleration.

Descending branch (T > TC): spectral acceleration decreases with period.

This gives a realistic shape: stiff buildings (short periods) feel strong accelerations, while flexible buildings (long periods) see reduced forces but larger displacements.

Figure 11.9.3.1. EC8 Response Spectrum defined in the SAP2000 Software.



Italian NTC 2018 Response Spectrum Definition in SAP2000

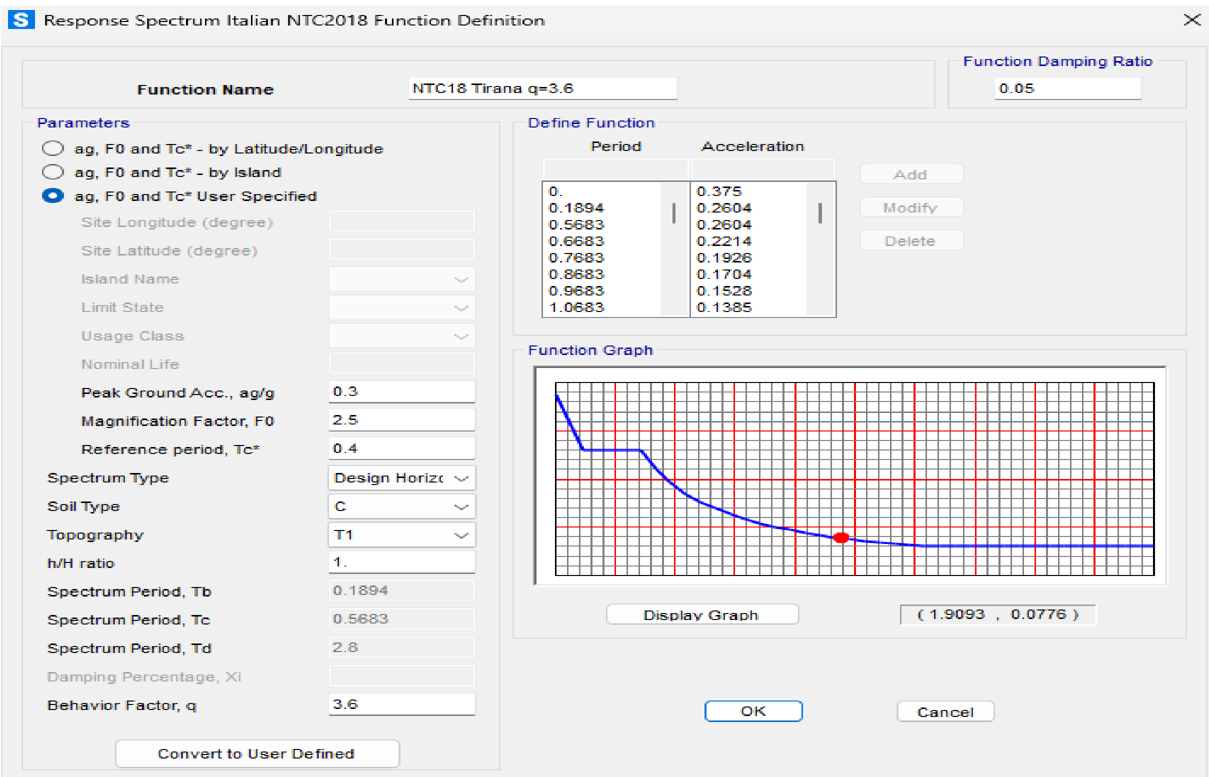
The Italian building code (**NTC2018**) defines the design response spectrum in a way that is very similar to Eurocode 8, but with some national parameters that make it more site-specific. Instead of a single generic curve, NTC2018 requires engineers to use hazard values taken directly from the official national seismic hazard maps, ensuring that the spectrum reflects local seismicity with more precision.

When defining the response spectrum, the following inputs are required:

- **a_g,SLV (or a_g,30)**: the reference peak ground acceleration for the site, depending on the selected return period (e.g., 30% probability in 50 years).
- **F0**: maximum spectral amplification factor at short periods.
- **Tc***: the period where the spectrum transitions from the constant plateau to the decreasing branch.
- **Soil category**: determines values of TB, TC, TD, and F0.
- **Damping factor η**: usually 1.0 for 5% damping, adjusted otherwise.

These parameters are obtained directly from hazard maps and soil classification tables given in NTC2018.

Figure 11.9.3.2. NTC2018 Response Spectrum defined in the SAP2000 Software.



Albanian KTP-89 Response Spectrum Definition in SAP2000

The Albanian code **KTP-89** was developed in the late 1980s and reflects the seismic design philosophy of its time. Compared to EC8 and NTC2018, its response spectrum is much simpler, with fewer parameters and a more conservative, strength-based approach. Although it does not capture site-specific seismic hazard with the same detail, it provides an important benchmark, since many existing buildings in Albania were designed under KTP-89.

ag/g	0.3	Peak ground acceleration ratio (kE = ag/g)
q	3.6	Behavior factor (use q ≈ 1/ψ)
kr	1	Importance factor (kr)
Soil_KTP	II	KTP soil category (I, II, III) — EC8 C ≈ KTP II

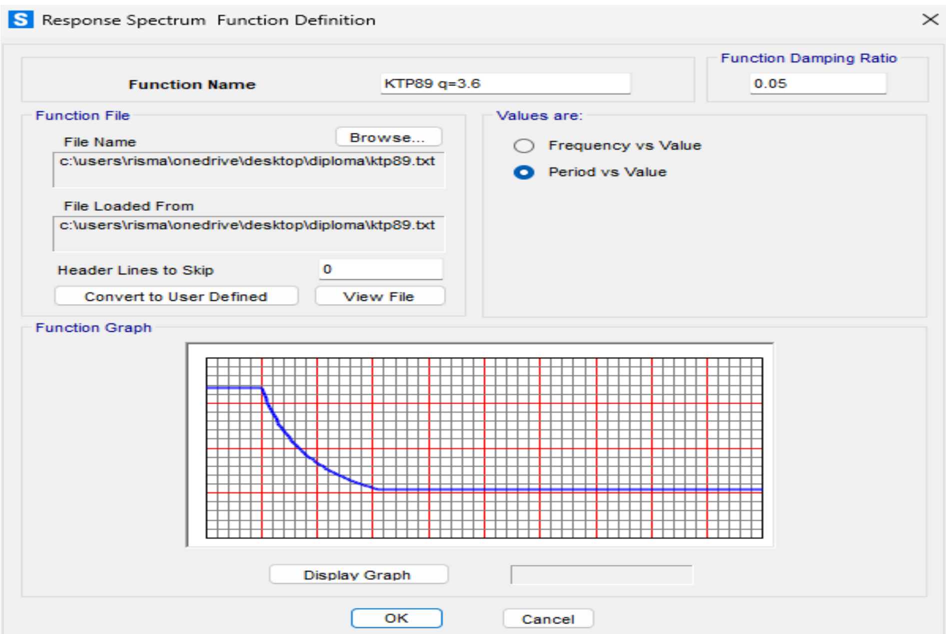
Soil_KTP	**c**	**beta_max**	**Tc = c/beta_max**	**Td = c/0.65**
I	0.7	2.3	0.304347826	1.07692308
II	0.8	2	0.4	1.23076923
III	1.1	1.7	0.647058824	1.69230769

Selected soil parameters	
c	0.8
beta_max	2
Tc	0.4
Td	1.230769231

Tables 11.9.3.1. KPT89 Soil parameters selection for our Spectra.

T [s]	**SDq [g]**
0.01	0.166667
0.02	0.166667
0.03	0.166667
0.04	0.166667
0.05	0.166667
0.16	0.166667
0.17	0.166667
0.18	0.166667
0.19	0.166667
0.2	0.166667
0.21	0.166667
0.22	0.166667
0.23	0.166667
0.24	0.166667
0.25	0.166667
0.26	0.166667
0.27	0.166667
0.28	0.166667
0.29	0.166667
0.3	0.166667
0.31	0.166667
0.32	0.166667
0.33	0.166667
0.34	0.166667
0.35	0.166667
0.36	0.166667
0.37	0.166667
0.38	0.166667
0.39	0.166667
0.4	0.166667
0.41	0.162602
0.42	0.15873
0.43	0.155039
…	…

Figure 11.9.3.2. KTP-89 Response Spectrum defined in the SAP2000 Software



$$\underline{Sa = k_E \times k_r \times \psi \times g}$$

k_r – building importance coefficient (Table 5 of KTP) k_E – Soil Parameters

ψ – structural coefficient (Table 4 of KTP-89) g – gravitational constant

In **Eurocode 8**, the value of the design ground acceleration (*ag*) is obtained by taking the reference peak ground acceleration for the site and multiplying it by the importance factor (γI). This simple relationship allows the seismic demand to be scaled depending on the significance of the building — for example, ordinary residential structures use γI = 1.0, while critical facilities such as hospitals or emergency centers may require higher values.

The **Albanian seismic code KTP-N2-89** approaches the problem differently. Instead of a direct formula, the code divides the country into seismic zones based on the **MSK-64 intensity scale**, which reflects the expected severity of ground shaking. For design purposes, only zones with intensities between VII and IX are considered relevant. These are described as:

- **Zone VII** – low seismicity,
- **Zone VIII** – moderate seismicity,
- **Zone IX** – high seismicity.

To each zone, KTP-89 assigns a **zone acceleration coefficient**, which varies between **0.08 and 0.42** depending on both the seismic intensity and the soil conditions at the site. This coefficient effectively translates the qualitative intensity scale into a numerical value that can be used in design calculations.

KE table	Seismic Intensity (MSK-64)		
Soil type	VII	VIII	IX
I	0.08	0.16	0.27
II	0.11	0.22	0.36
III	0.14	0.26	0.42

Table 11.9.3.4. KPT89 Soil Type to Seismic Intensity conversion.

Soil Category I:	Soil Category II:	Soil Category III:
$0.65 \leq \beta_i = 0.7/T_i \leq 2.3$	$0.65 \leq \beta_i = 0.8/T_i \leq 2.0$	$0.65 \leq \beta_i = 1.1/T_i \leq 1.7$

When comparing **Eurocode 8 (EC8)** with the older **Albanian code KTP-N2-89**, several key differences become apparent in both the **shape** of the spectra and their **amplitudes**. One of the most significant distinctions is the treatment of soil conditions. The KTP-89 code does not include a soil factor, meaning that the same peak spectral amplitudes are applied regardless of whether the ground is rock, stiff soil, or soft soil. By contrast, EC8 introduces soil classification directly into its formulation, producing different spectral amplitudes depending on ground type. It also distinguishes between near-field and far-field seismic conditions, resulting in a more refined and site-specific spectrum.

This contrast highlights a fundamental divergence in methodology: EC8 seeks to reflect the variability of actual ground motion, while KTP-89 applies a uniform framework that simplifies the problem but may overlook important local effects. As a result, EC8 spectra provide engineers with a tool that can adapt more precisely to different site conditions, whereas KTP-89's generalized approach may lead to unconservative or overly conservative results depending on the geological setting.

The role of soil conditions in seismic response has been extensively studied, and modern research consistently emphasizes their importance in seismic design. EC8's soil classification system, supported by empirical data and site investigations, divides soils into classes with distinct amplification effects. These categories directly modify the shape of the response spectrum, ensuring that buildings designed on soft soils, for example, account for the stronger amplification typically observed in earthquakes.

By contrast, KTP-89's uniform spectrum ignores these differences, which could mean that in areas with highly variable geology, the design does not truly reflect the actual seismic hazard. This limitation illustrates why modern codes like EC8 have moved towards site-specific seismic design, ensuring that the structural response accounts not only for the intensity of shaking but also for the ground conditions that strongly influence it.

KTP Soil Type	Coefficient (c)	βmax	Typical Ground Conditions	Comparable to Eurocode Soil Type
I	0.7	23	Hard rock very stiff deposits	A-B
II	0.8	2	Medium-stiff soils, gravel, dense sand	C
III	1.1	1.7	Soft day. loose sand	D

Table 11.9.3.5. KPT89 Soil Type definition and its coefficents.

11.9.4 Load Cases definition for Seismic Response Spectrum Analysis

When setting up the seismic analysis, the first step after defining the spectra is to apply them to the building model. Since the study compares three different seismic codes — Eurocode 8, the Italian NTC2018, and the Albanian KTP-89 — each spectrum has to be applied in both of the main building directions. This leads to a total of six load cases: one in the X direction and one in the Y direction for each spectrum.

For **Eurocode 8**, two cases were created: one with the spectrum applied in the global X direction (**EX-EC8**) and another in the Y direction (**EY-EC8**). These represent the building's response when ground shaking acts along each of its principal axes.

Figure 11.9.4.1. EC8 Load Case EX defined in the SAP2000 Software.

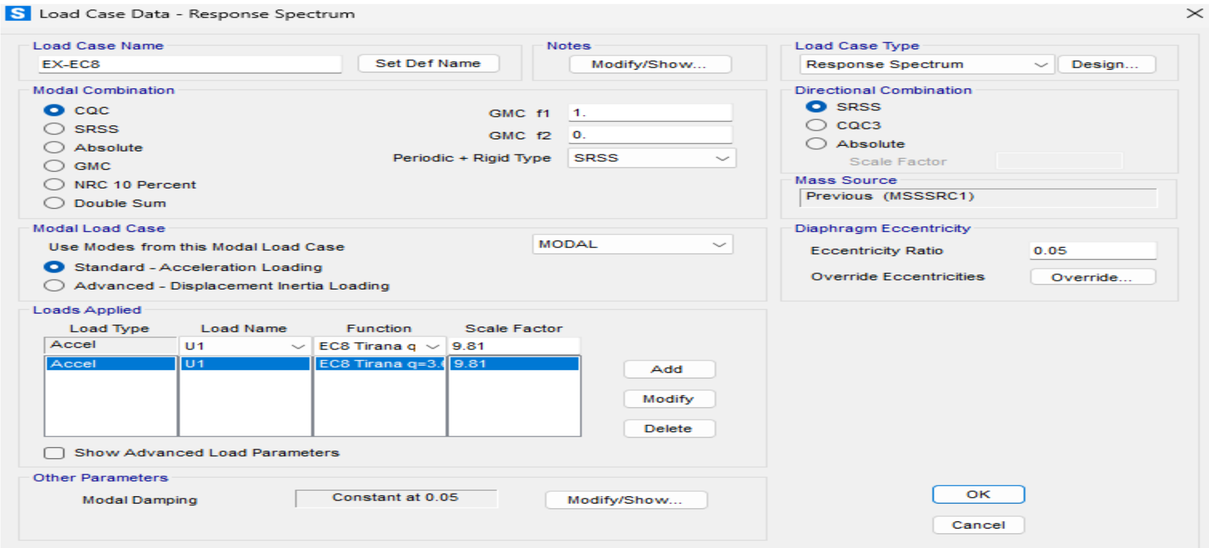
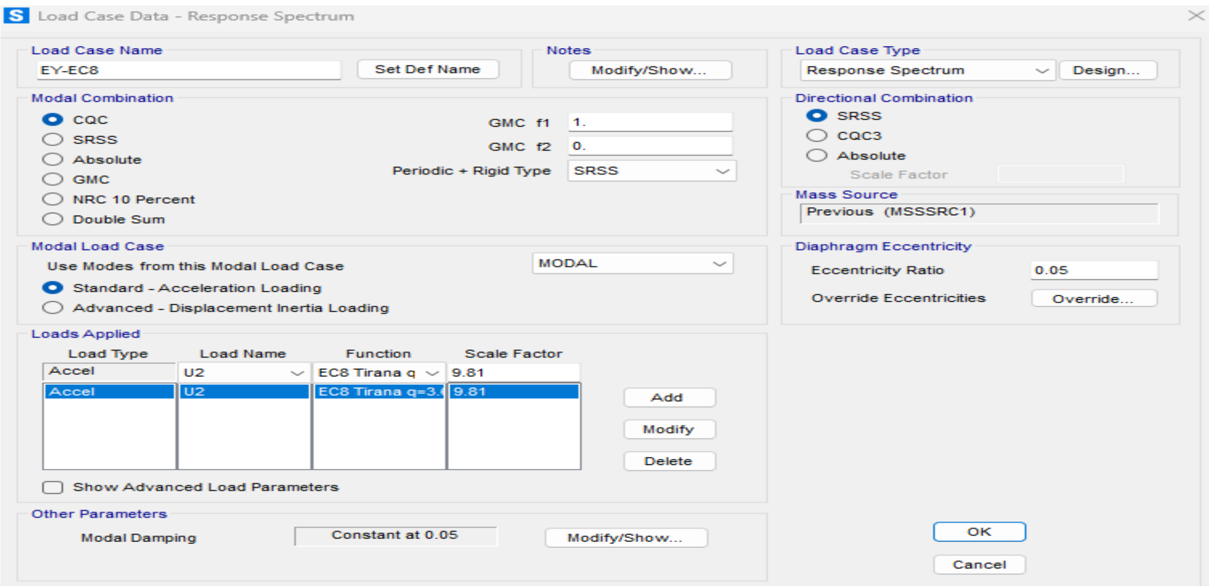


Figure 11.9.4.2. EC8 Load Case EY defined in the SAP2000 Software



The same approach was used for **NTC2018**, resulting in **EX-NTC18** and **EX-NTC19**. Although the procedure is identical, the Italian spectrum has its own parameters — such as a_g,30, F0, and Tc* — which shape the curve differently from Eurocode 8. Running both directions separately makes it possible to capture these differences in structural response.

Figure 11.9.4.3. NTC18 Load Case EX defined in the SAP2000 Software.

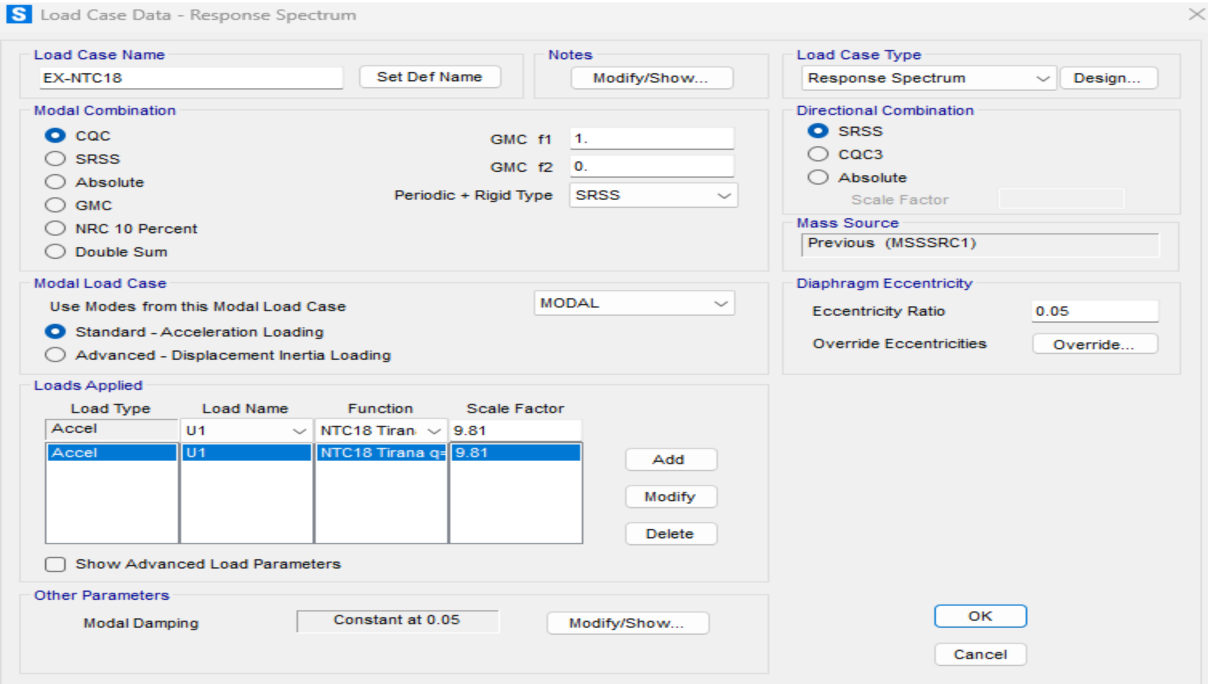
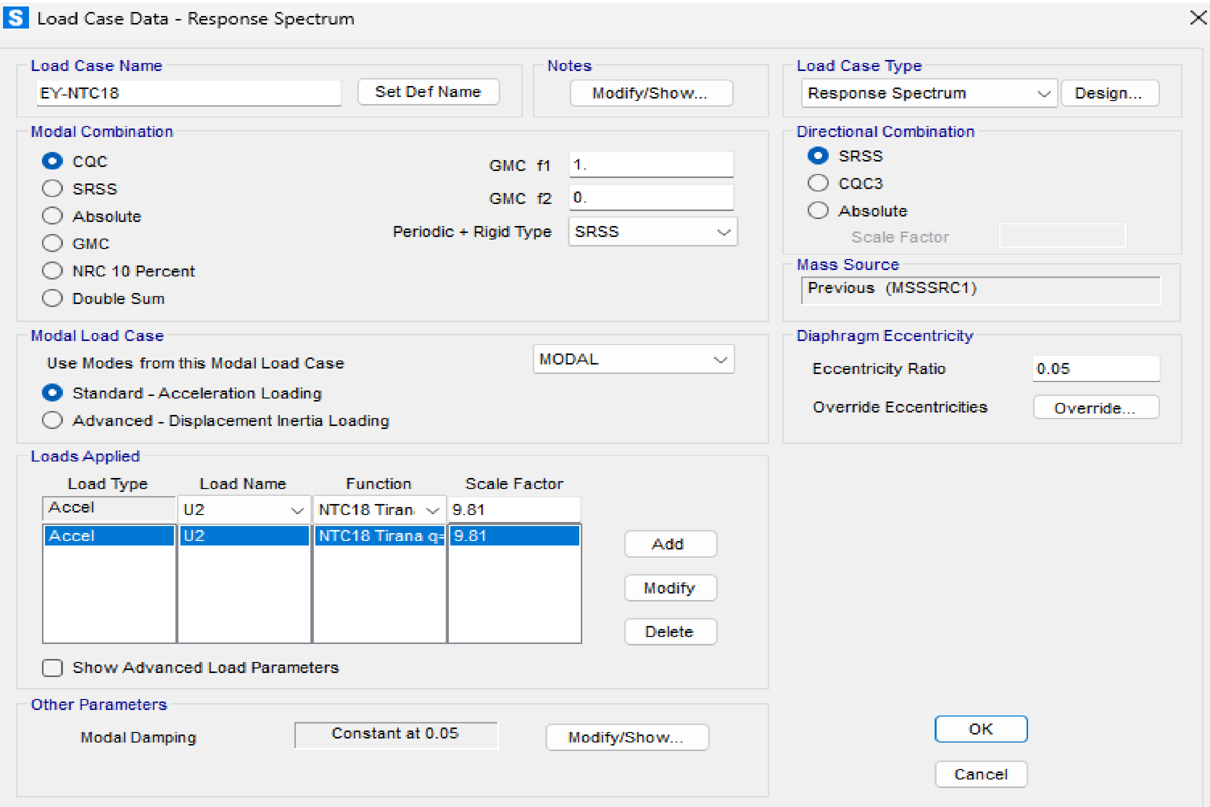


Figure 11.9.4.4. NTC18 Load Case EY defined in the SAP2000 Software.



Finally, for the **Albanian KTP-89 code**, two additional cases were defined: **EX-KTP89** and **EY-KTP89**. Even though the KTP-89 spectrum is simpler in its formulation, it still needs to be applied in both directions, since buildings can be affected by earthquakes coming from any orientation.

Figure 11.9.4.5. KTP89 Load Case EX defined in the SAP2000 Software.

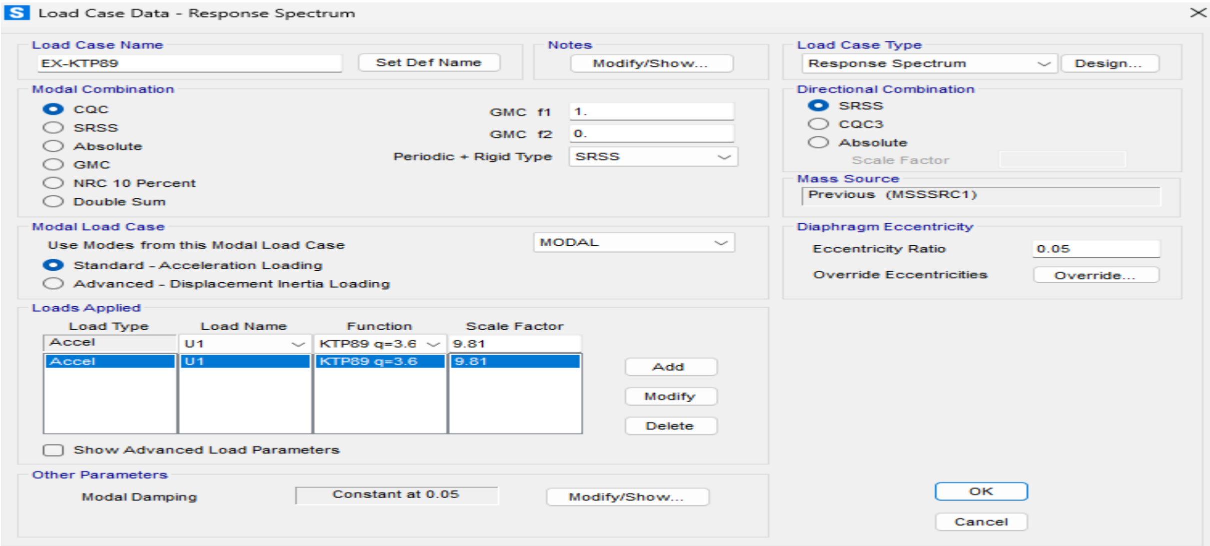
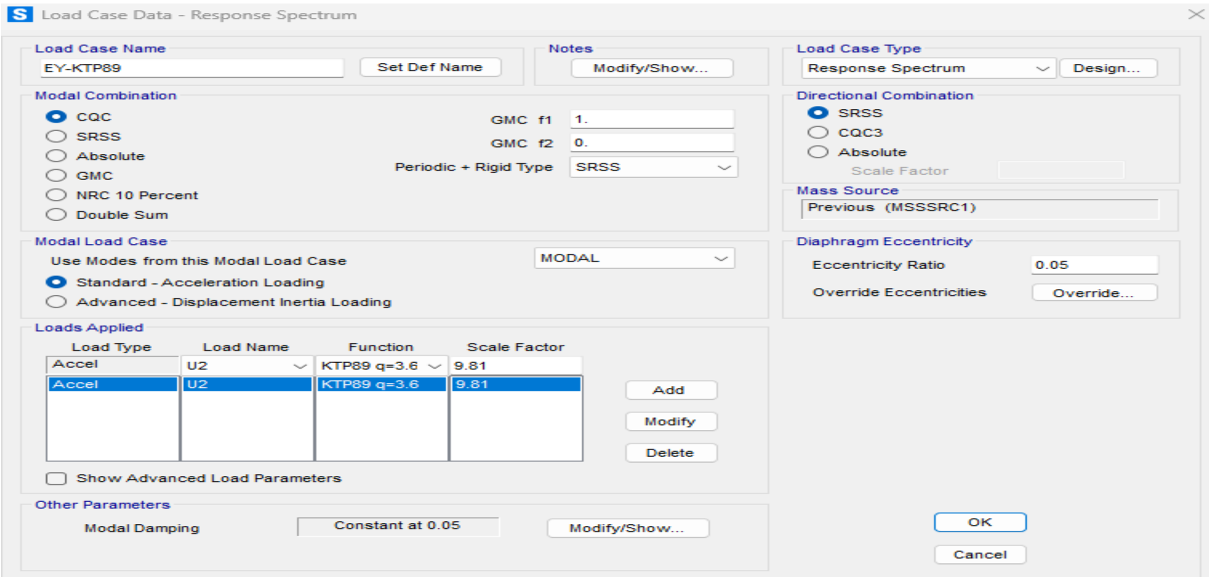


Figure 11.9.4.5. KTP89 Load Case EY defined in the SAP2000 Software.



In total, these six load cases give a consistent framework for comparison. Each code is applied in the same way, and both principal directions are considered. This ensures that when results such as base shear, displacements, drifts, or member forces are compared later, the differences reflect the influence of the seismic codes, not inconsistencies in how the load cases were applied.

11.9.5 Modal Analysis

When performing a response spectrum analysis, one of the first steps is to carry out a **modal analysis** of the structure. This process is crucial because seismic response is not governed by a single vibration mode, but rather by the combined effect of multiple modes of vibration. Each mode represents a unique way the building can deform under dynamic loading, with its own **natural period, frequency, and shape**.

By decomposing the structure into these modes, the software can later "weight" them according to the response spectrum and then combine them to estimate the overall seismic demand. Without this step, the dynamic behavior of the building would be oversimplified and unrealistic.

Modal analysis is therefore the bridge between the mathematical definition of a response spectrum and the actual behaviour of the structure.

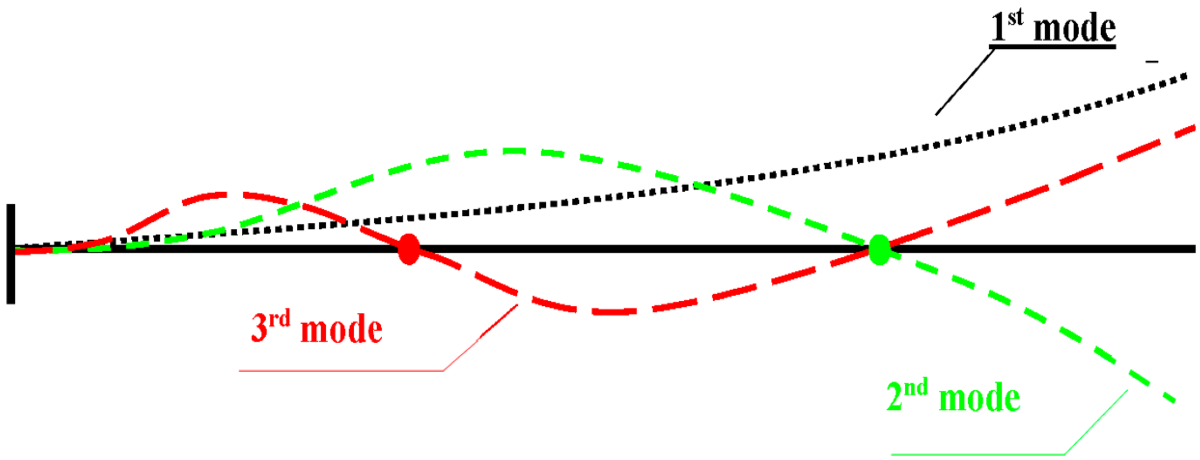


Figure 11.9.5.1. Modal Analysis general representation.

When we run a response spectrum analysis in SAP2000, the program first needs to know how the building naturally vibrates. This is done through a **modal analysis**, and SAP2000 gives us two main ways to do it: the **Eigenvalue method** and the **Ritz vector method.** Both are valid, but they reflect different philosophies in how vibration modes are extracted and used in seismic design.

Eigenvalue Modal Analysis

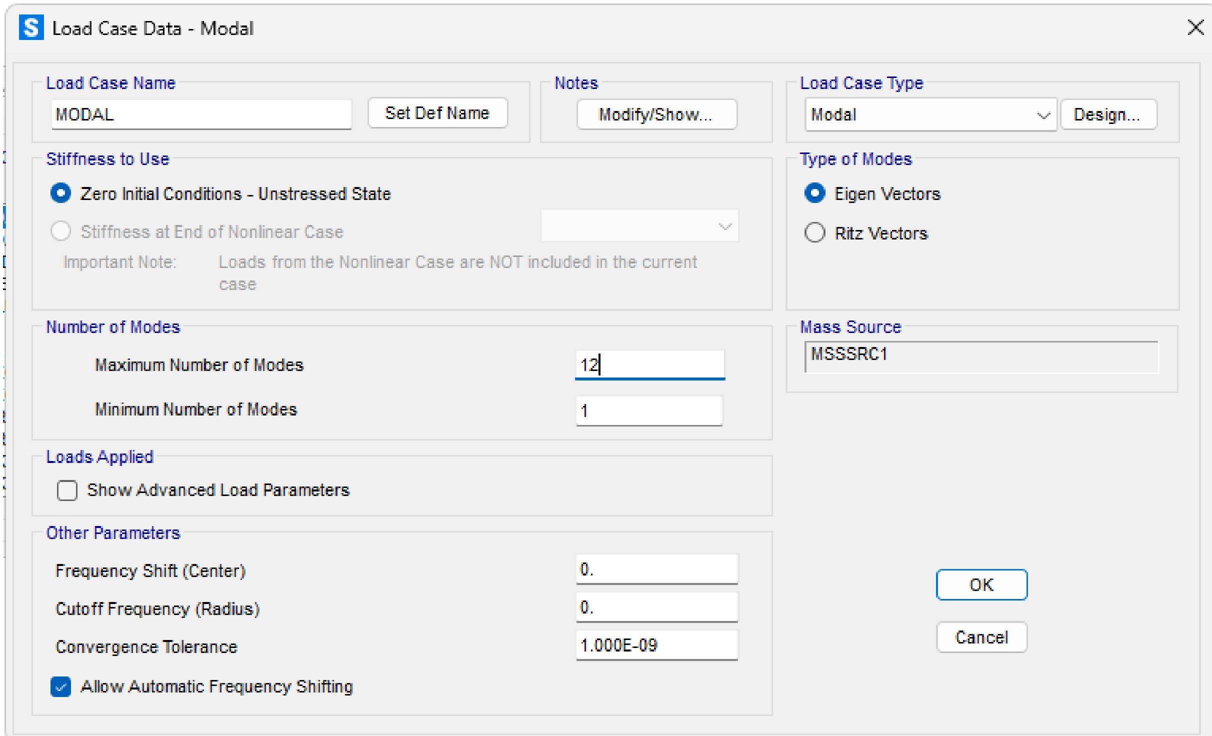
The eigenvalue method is the **classical approach**. It solves the mathematical eigenvalue problem of the structure's stiffness and mass matrices, providing the *exact natural frequencies, mode shapes, and mass participation factors*. These results are independent of the type of loading and represent the "true" dynamic properties of the structure. Because of its accuracy, eigenvalue analysis is widely used in both research and design, especially when the goal is to understand the fundamental periods and dynamic behaviour of a building in detail.

This method provides a precise picture of the building's dynamic behaviour. The results include:

- A list of natural frequencies (or their inverses, the natural periods).
- The shape of each mode (i.e., how the structure deforms in that mode).
- The percentage of mass each mode activates in the X, Y, and Z directions.

For seismic design, it is not enough to know just the first mode. Codes such as EC8 and NTC2018 require that at least 90% of the total mass is represented in both horizontal directions. This usually means including not only the fundamental mode but also higher modes, which capture more complex patterns of vibration, especially in tall or irregular buildings.

Figure 11.9.5.2. Eigen Modal Case definition in Sap2000 software.

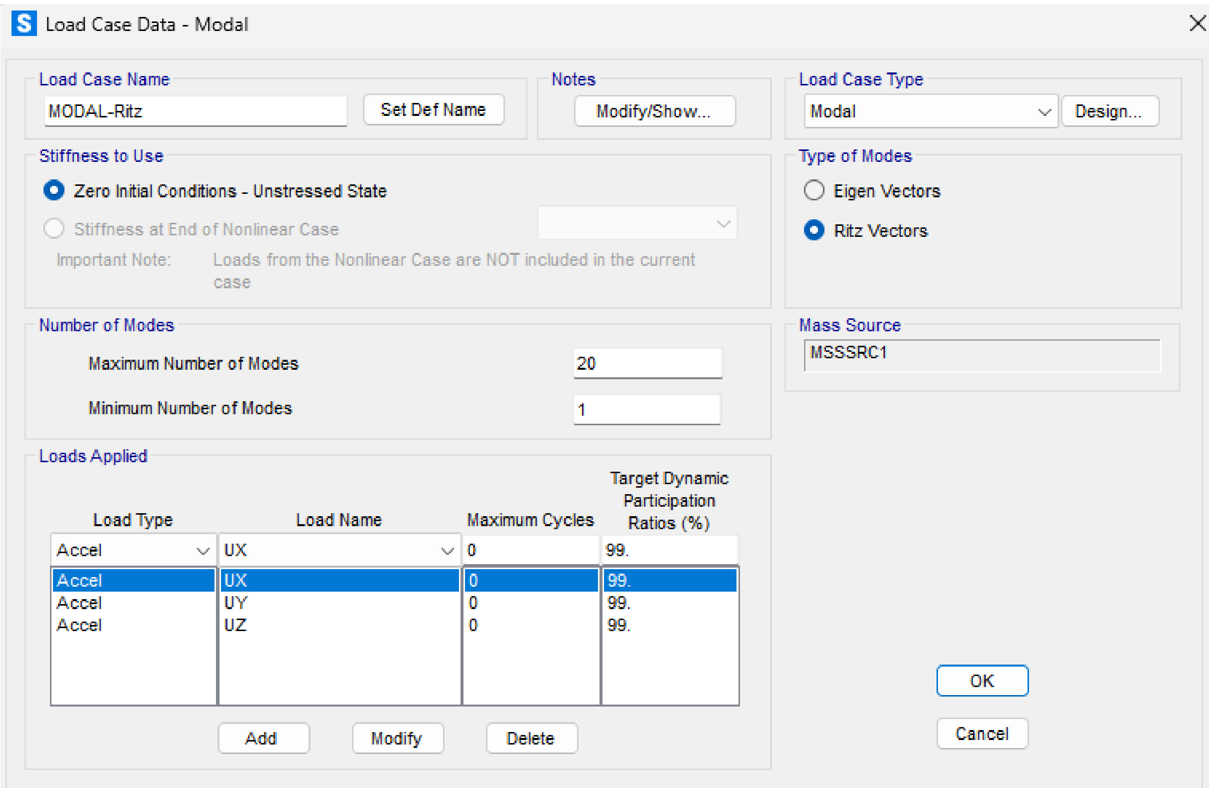


Ritz Vector Modal Analysis

The Ritz method, on the other hand, is a **load-dependent approach**. Instead of solving for all possible vibration modes, it generates vectors that are **biased towards the type of loading applied**, typically lateral earthquake forces. This makes it more efficient in capturing the seismic response with fewer modes, particularly for large or complex models. Ritz analysis may not provide the complete set of "exact" natural modes, but it usually gives a practical and reliable basis for response spectrum analysis.

This makes the Ritz method particularly efficient for seismic design. Since it "guides" the analysis towards lateral load shapes, it often captures the seismic response with fewer modes compared to the eigenvalue method. For very large or complex structures, this efficiency can save considerable computational effort while still producing reliable results.

Figure 11.9.5.2. Eigen Modal Case definition in Sap2000 software



Modal Periods and Frequencies

Every structure has its own natural rhythm of vibration. When subjected to dynamic actions such as wind or earthquakes, it tends to oscillate in specific ways called **modes of vibration**. Each mode is characterized by a **period** and a **frequency**, which describe how fast or slow the structure vibrates in that particular shape.

- The **modal period** is the time (in seconds) it takes for one full cycle of vibration. A long period means the structure vibrates slowly and tends to be more flexible, while a short period indicates faster oscillations and a stiffer response.
- The **modal frequency** is simply the inverse of the period, expressed in Hertz (cycles per second). Higher frequencies correspond to quicker, smaller-scale oscillations, whereas lower frequencies represent large, global movements of the building.

Eurocode Perspective

Eurocode 8 emphasizes the importance of correctly identifying the fundamental period of the structure, since this parameter has a direct influence on the seismic design forces. The shape of the design response spectrum is linked to the vibration period: shorter periods correspond to higher accelerations, while longer periods shift the response into lower acceleration ranges. This makes an accurate estimation of periods and frequencies crucial for determining the seismic demand on the structure.

TABLE: Modal Periods And Frequencies					
Case	Mode	Period	Frequency	CircFreq	Eigenvalue
		sec	cyc/sec	rad/sec	rad^2/sec^2
Modal	1	0.903	1.108	6.9588	48.4251
Modal	2	0.755	1.325	8.3224	69.2629
Modal	3	0.701	1.426	8.9611	80.3014
Modal	4	0.277	3.611	22.6878	514.735
Modal	5	0.238	4.205	26.4208	698.0569
Modal	6	0.201	4.97	31.227	975.1264
Modal	7	0.177	5.638	35.4268	1255.0595
Modal	8	0.139	7.193	45.1943	2042.527
Modal	9	0.124	8.044	50.5447	2554.7696
Modal	10	0.085	11.831	74.3387	5526.2435
Modal	11	0.08	12.442	78.1765	6111.5707
Modal	12	0.067	15.011	94.3191	8896.0939

Table 11.9.5.2. Modal Peridos and Frequencies for our Structure from the SAP2000 Software.

Modal Participation Mass Ratios

When a structure is analyzed dynamically, its response to an earthquake is not controlled by a single vibration shape but by a combination of many vibration modes. Each mode represents a particular way in which the structure can oscillate, and not all of them are equally important. Some modes carry a large portion of the total building mass, while others contribute very little. The **modal participation mass ratio** is a measure of how much of the building's total mass is activated by each vibration mode in a given direction. In simple terms, it tells us "how strongly this mode participates in moving the building" when seismic forces are applied. Lower modes (like the fundamental one) usually mobilize most of the mass and therefore dominate the response, while higher modes often contribute only small corrections.

Eurocode Perspective

Eurocode 8 requires that the dynamic analysis captures a sufficiently large portion of the total effective mass of the structure. The code states that enough modes should be included until the **cumulative effective modal mass** reaches at least **90% of the total mass** in each principal direction of vibration. This requirement ensures that the chosen set of modes provides a realistic representation of how the whole building moves during an earthquake, and that no significant response is left out.

TABLE: Modal Participating Mass Ratios								
Case	Mode	Period	UX	UY	UZ	SumUX	SumUY	SumUZ
		sec						
Modal	1	0.903	0.0112	0.5539	0.000007624	0.0112	0.5539	0.000007624
Modal	2	0.755	0.2014	0.013	0	0.2126	0.5669	0.000007692
Modal	3	0.701	0.3445	0.0037	0	0.5571	0.5706	0.000008135
Modal	4	0.277	0.0132	0.1045	0.00002584	0.5703	0.6751	0.00003397
Modal	5	0.238	0.0057	0.0464	0.00004946	0.576	0.7216	0.0001
Modal	6	0.201	0.1407	0.0028	0.0001	0.7167	0.7244	0.0002
Modal	7	0.177	0.0012	0.003	0.0315	0.7179	0.7273	0.0317
Modal	8	0.139	0.0017	0.048	0.0748	0.7196	0.7753	0.1065
Modal	9	0.124	8.543E-07	0.0189	0.4035	0.7196	0.7942	0.5101
Modal	10	0.085	0.0854	0.0609	0.073	0.805	0.8551	0.583
Modal	11	0.08	0.0974	0.0656	0.0272	0.9024	0.9207	0.6103
Modal	12	0.067	0.0024	0.0324	0.2483	0.9048	0.953	0.8586

Table 11.9.5.2. Modal Participating Mass Ratios for our Structure from the SAP2000 Software.

11.9.6 Load Combinations

In practice, a structure is never subjected to a single type of action in isolation. At any given moment, a building carries its own self-weight, while also experiencing variable influences such as the presence of people, furniture, wind, snow, or even an earthquake. Because these actions often act together, the structural engineer must ensure safety under the most unfavorable yet realistic situations.

To achieve this, design codes introduce the concept of **load combinations**. These combinations are essentially predefined scenarios that bring together different types of loads with appropriate safety factors. The factors are not arbitrary: they reflect the uncertainty of each load and the level of reliability required. Permanent loads are treated differently from variable loads, and seismic actions are handled with their own dedicated rules, since not all variable loads are likely to be fully present during an earthquake.

For **ultimate limit states (ULS)**, permanent loads are amplified by a factor (typically 1.35), while variable loads are amplified more strongly (often 1.50), acknowledging their greater variability.

For **seismic design situations**, permanent loads usually enter without amplification, while variable loads are reduced by specific combination factors, since full live loads are unlikely during an earthquake. At the same time, the seismic action is applied in orthogonal directions, one as the main component and the other reduced, to account for uncertainty in earthquake direction.

In essence, load combinations provide a systematic way to represent the **worst credible conditions** that a structure may face. They strike a balance between safety and economy, ensuring that the design is neither excessively conservative nor unrealistically optimistic.

Figure 11.9.6.1. Ultimate Limit States Combination.

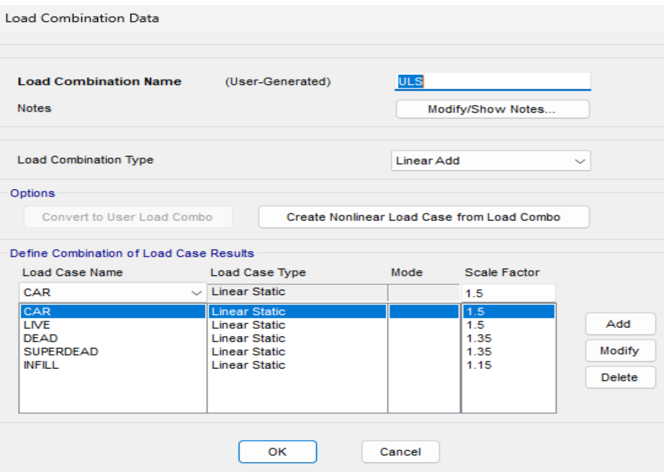


Figure 11.9.6.2. Servicability Limit States Combination.

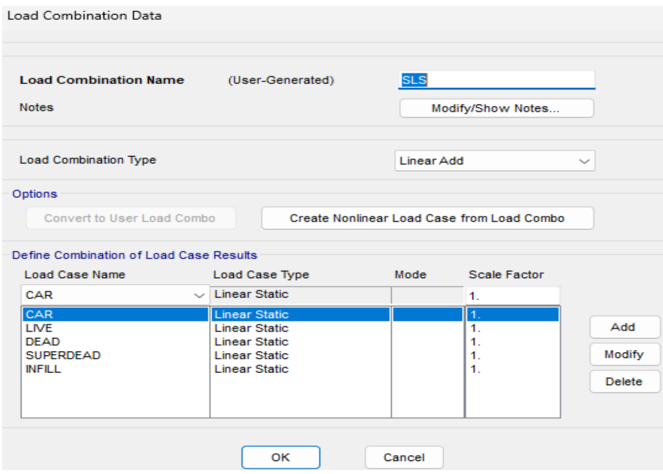


Figure 11.9.6.3. General Eurocode 8 Seismic Combination.

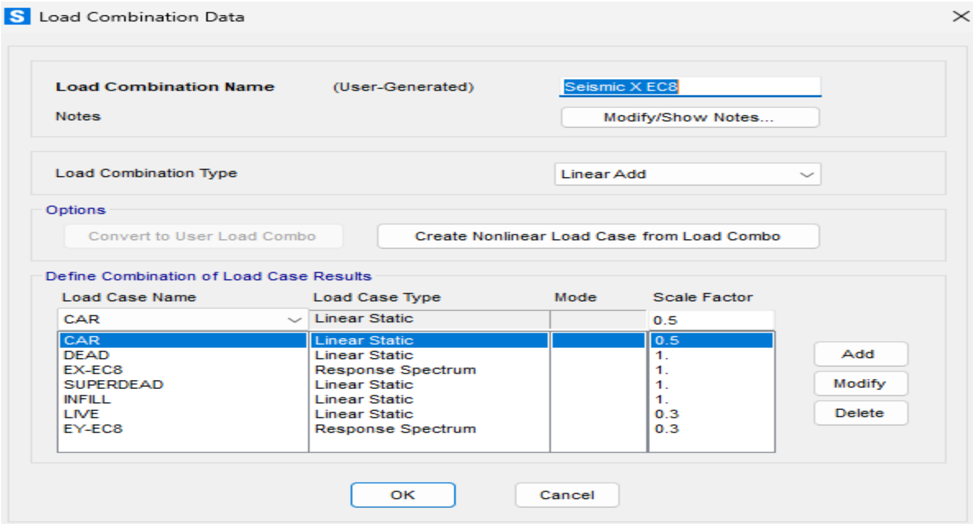


Figure 11.9.6.4. General NTC18 Seismic Combination.

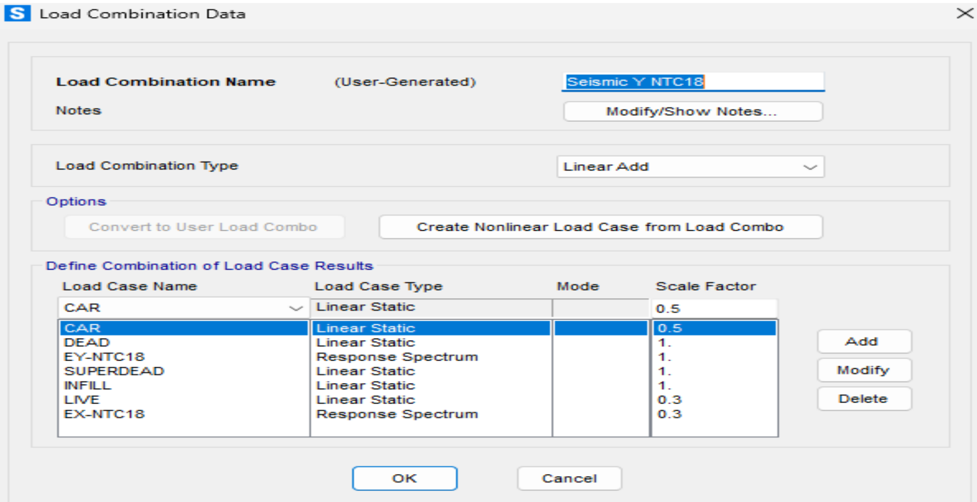
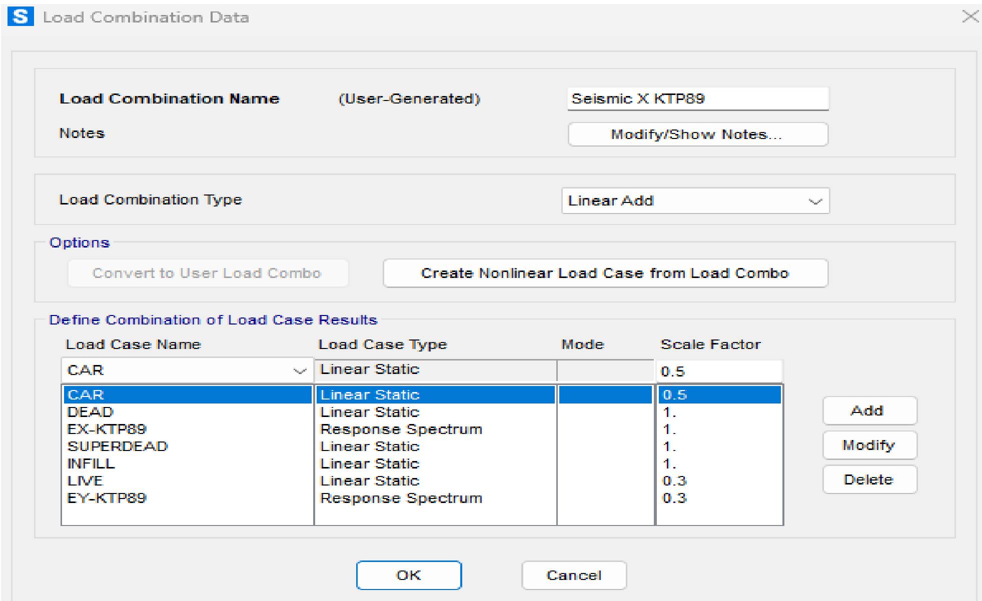


Figure 11.9.6.5. General KTP89 Seismic Combination.



12. RESPONSE SPECTRUM ANALYSIS RESULTS – EC8

The response spectrum analysis was first performed using the **Eurocode 8 design spectrum**. This provided the baseline results against which the Italian NTC2018 and Albanian KTP-89 analyses will later be compared. The following subsections present the main outcomes of the EC8-based analysis, including global building response (base shear and displacements), local element forces including Moments and Shear Forces.

12.1 Maximum Story Displacement – EC8 Seismic Combinations

The diagram above shows how the building responds laterally when the seismic action is applied in the **X direction and in the Y direction**, following the Eurocode 8 combination rules. On the horizontal axis we have the displacement values (in millimeters), while the vertical axis indicates the building stories.

Figure 12.1.1. Maximum Story Displacements for EY Seismic Combination EC8**.**

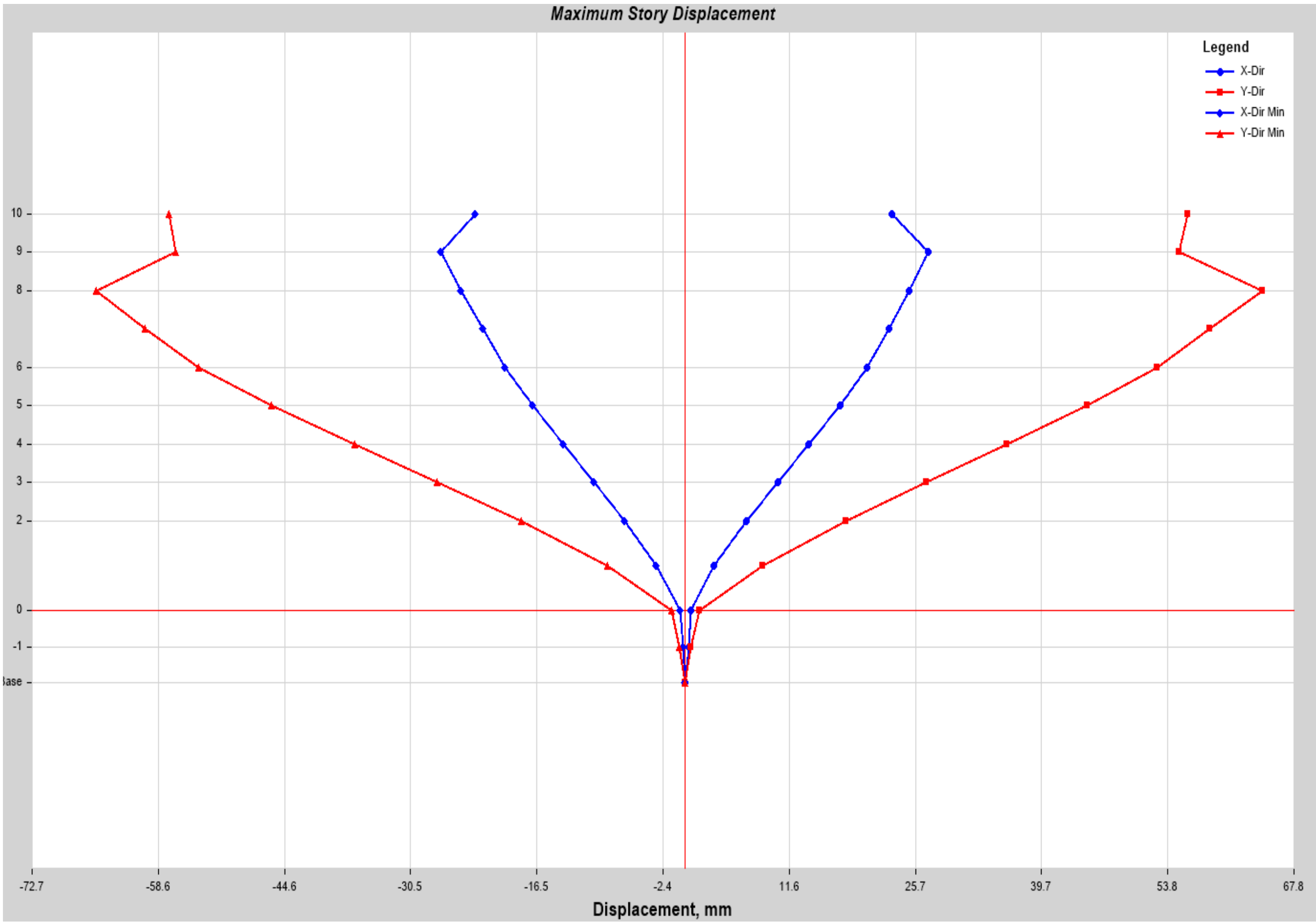
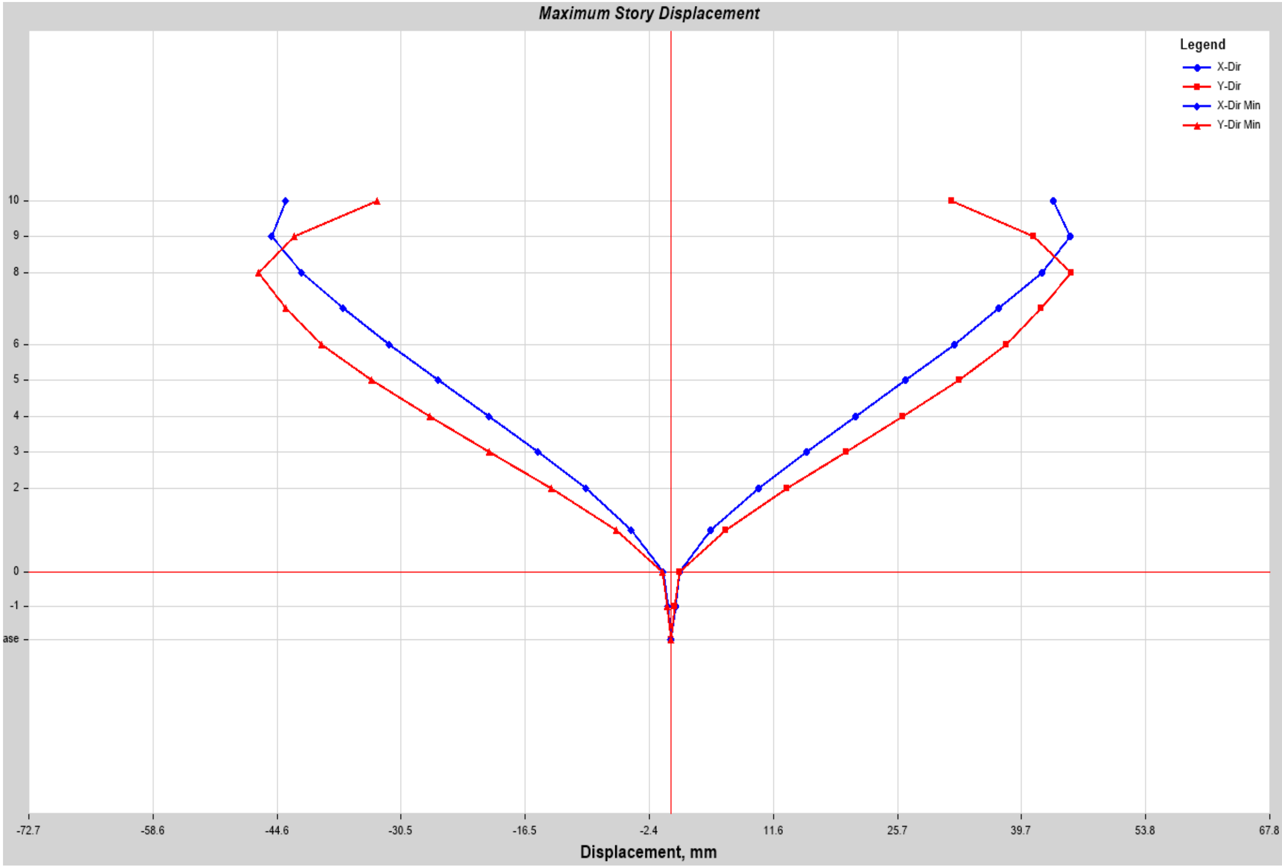


Figure 12.1.2. Maximum Story Displacements for EX Seismic Combination EC8**.**



What the graph really tells us is how the movement increases step by step as we go higher in the structure. At the base the displacements are very small, almost negligible, and then they gradually build up until they reach the maximum value at the roof. This is the typical way a multi-storey frame or wall system reacts under horizontal loads: it behaves somewhat like a cantilever, with the top moving the most.The two curves on either side represent the extreme values in both positive and negative directions. They are fairly symmetrical, which is expected since the analysis records the envelope of maximum displacements.

This type of result is important not just as a picture of the structural response, but because it is directly linked to design checks. Eurocode 8 requires us to control the lateral drift of each story so that damage is limited and the building can remain functional even after an earthquake. In other words, the figure is more than a graph—it's a way to judge if the structure is flexible enough to absorb seismic energy but still stiff enough to stay within safety limits.

12.2 Story Base Shear Results – Vx, Vy – EC8 Seismic Combinations

The total seismic base shear was obtained in both principal directions of the building. Under the EC8 spectrum, the structure developed a high story base shear in the X direction and a moderate story base shear in the Y direction. These values reflect the combined contribution of all significant vibration modes, with modal results combined using the Complete Quadratic Combination (CQC) method. As expected, the base shear magnitudes were directly influenced by the fundamental period of the building and the soil amplification factors associated with the chosen soil category.

Figure 12.2.1. Maximum Story Shear for EX Seismic Combination EC8

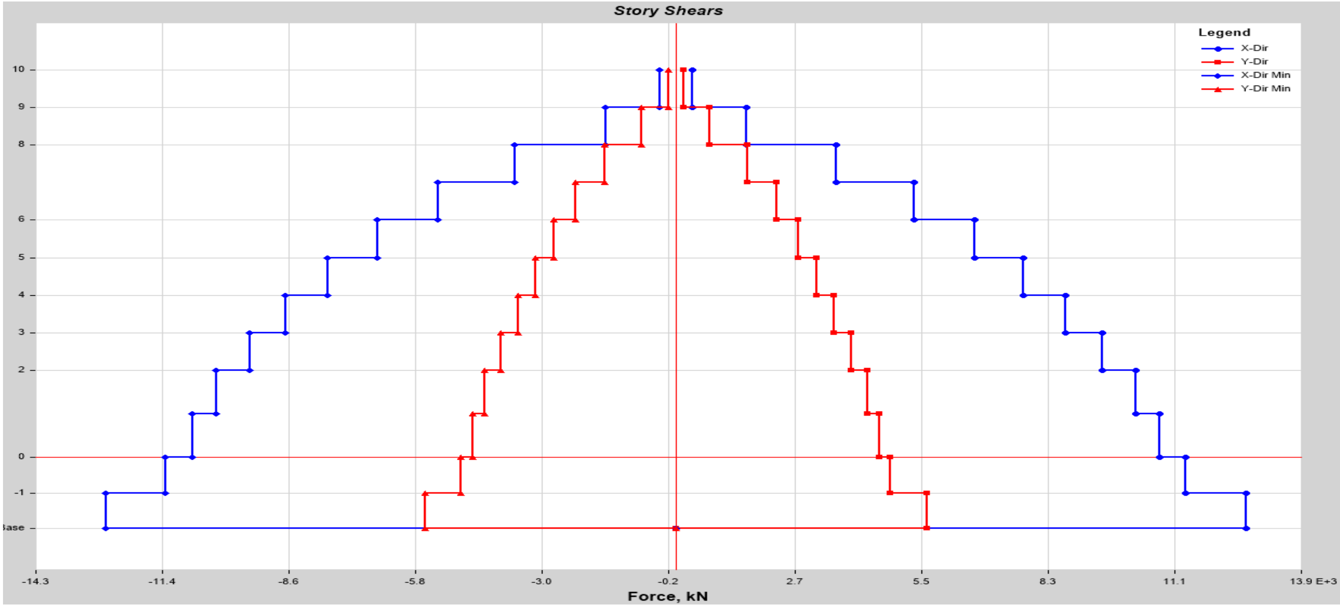
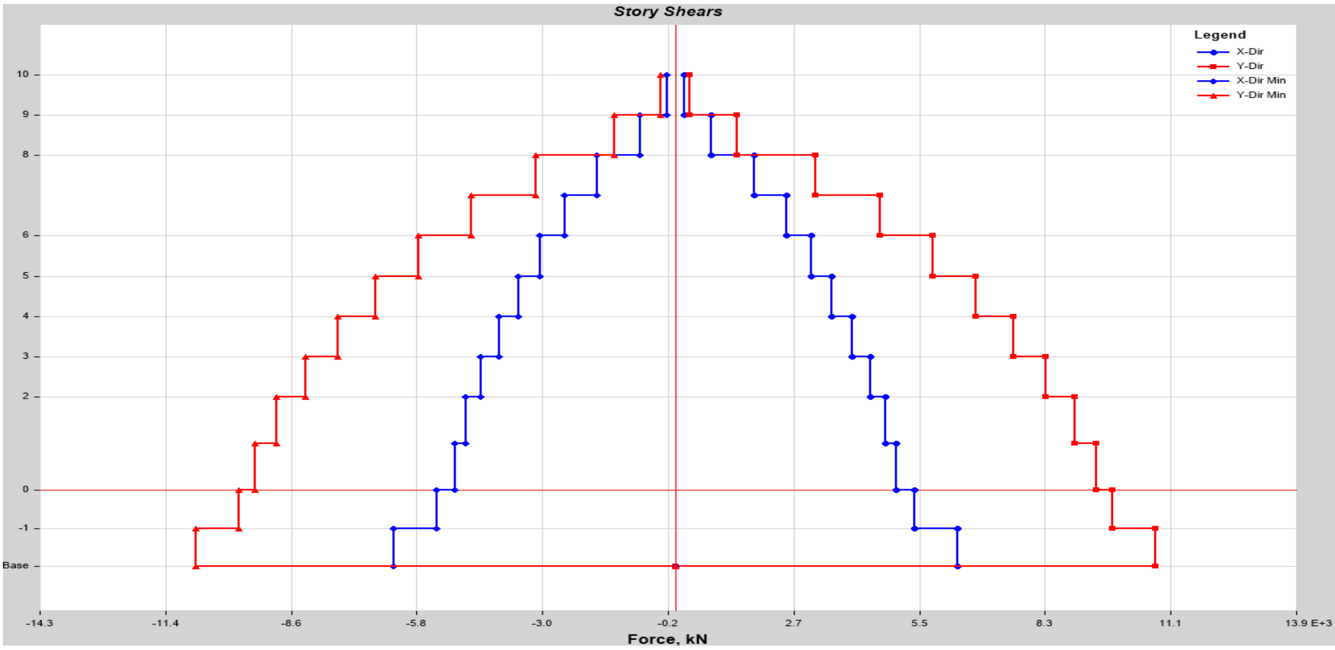


Figure 12.2.2. Maximum Story Shear for EY Seismic Combination EC8.



12.3 Maximum Story Drifts – EC8 Seismic Combinations

When a building is shaken by an earthquake, its floors do not move uniformly. Instead, each level displaces relative to the one below it, creating what is known as a **storey drift**. This drift is a direct measure of how much the building deforms laterally under seismic loading, and it is one of the key indicators of potential damage to both structural and non-structural elements.

Eurocode 8 introduces specific limits on these drifts to ensure that buildings not only remain standing but also maintain their serviceability and avoid excessive damage. The code sets maximum allowable inter-storey drift ratios, usually expressed as a fraction of the storey height (for example, **0.004 × storey height** for buildings under seismic design). This limit is intended to prevent issues like cracking in partitions, malfunction of doors and windows, or even structural instability if deformations become too large.

Figure 12.3.1. Maximum Story Drifts for EX Seismic Combination EC8.

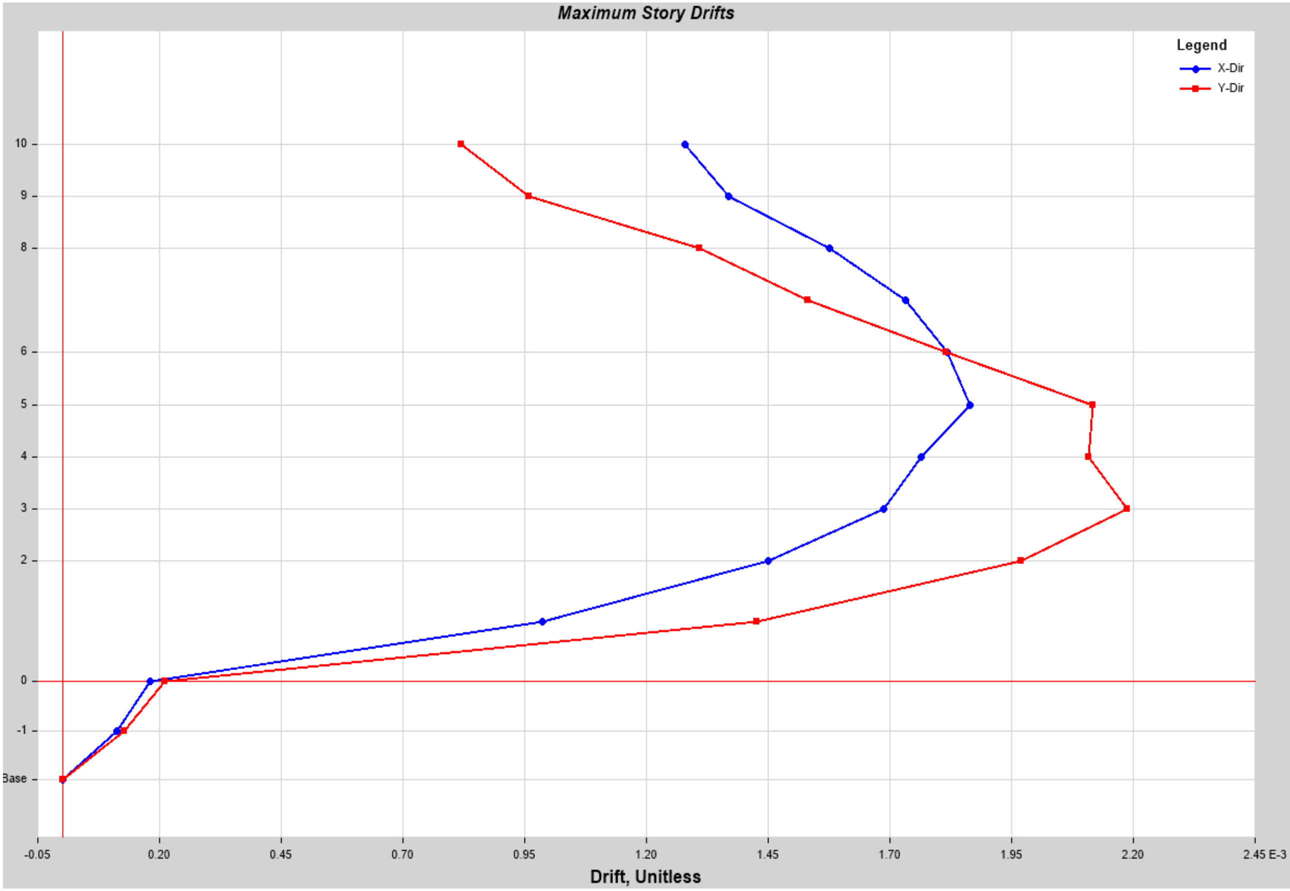
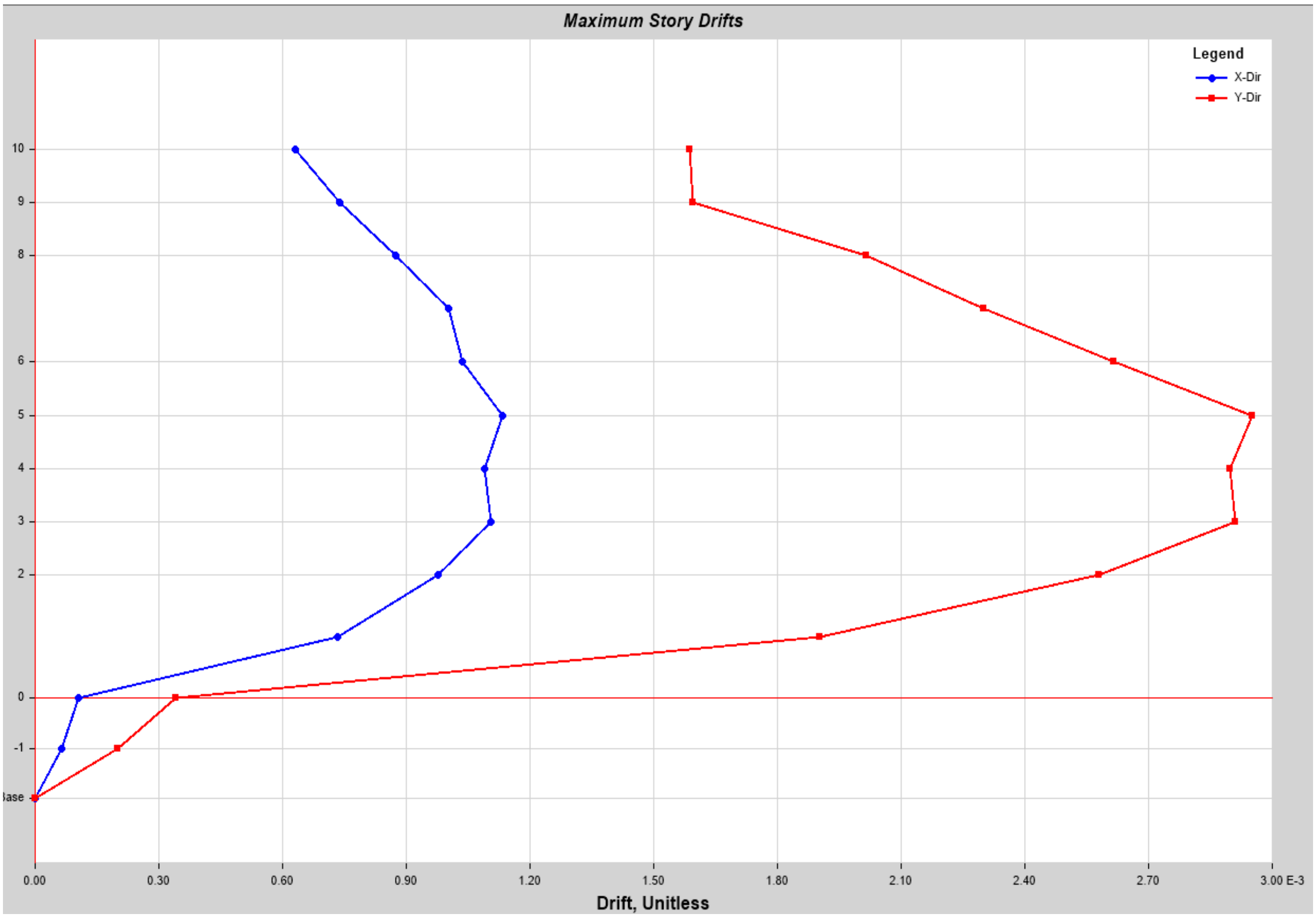


Figure 12.3.2. Maximum Story Drifts for EY Seismic Combination EC8.



By checking both **EX** and **EY** cases, Eurocode ensures that the building is verified for the full range of possible earthquake directions. This systematic approach provides confidence that the structure can deform safely within code-defined limits, without excessive risk of non-structural damage or loss of functionality.

12.4 Axial Forces in Columns – EC8 Seismic Combinations

Column axial forces were extracted to evaluate the combined effect of gravity and seismic actions. The maximum axial load recorded was **3352 kN**, concentrated in the centered columns of the lower storeys, where both gravity and lateral forces accumulate. This distribution is consistent with expectations: seismic overturning effects increase axial compression in some columns while reducing it in others, particularly at the building perimeter.

Figure 12.4.1. Maximum Axially Loaded Column
EX Seismic Combination EC8.

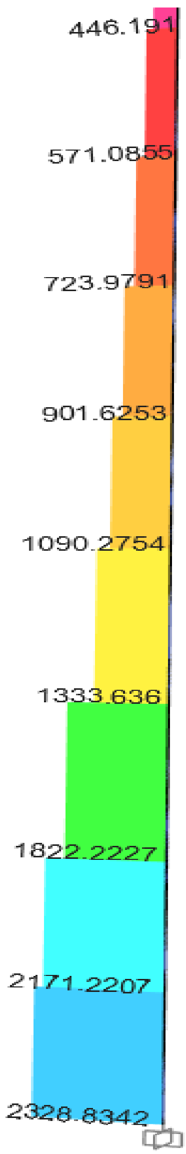
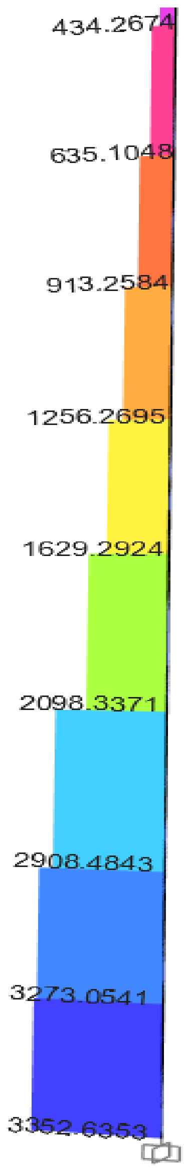


Figure 12.4.2. Maximum Axially Loaded Column
EY Seismic Combination EC8.



12.5 Shear Forces in Columns and Beams – EC8 Seismic Combinations

Shear force envelopes were generated for both beams and columns. Columns at the base exhibited maximum shear forces of **256 kN**, while beams at intermediate floors experienced shear peaks of **409 kN**. These results are important because shear governs the detailing of transverse reinforcement.

Figure 12.5.1. Maximum Shear 3-3 Loaded Column EX Seismic Combination EC8.

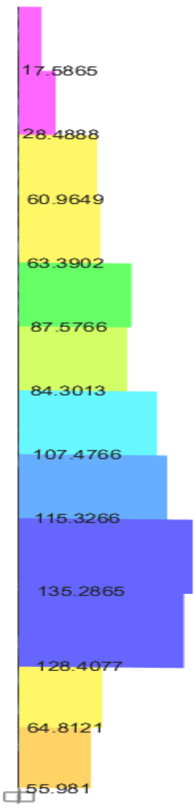


Figure 12.5.2. Maximum Shear 2-2 Loaded Column EY Seismic Combination EC8.

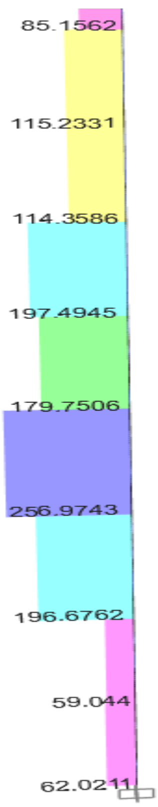


Figure 12.5.3. Maximum Shear 3-3 Loaded Beam EX Seismic Combination EC8.

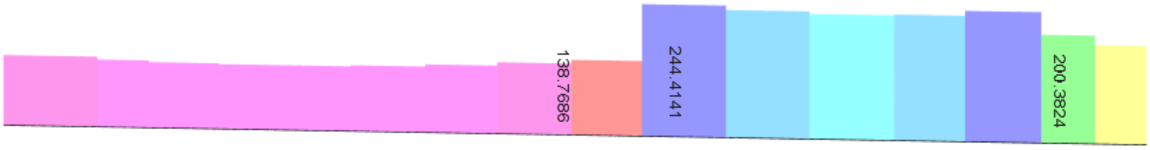
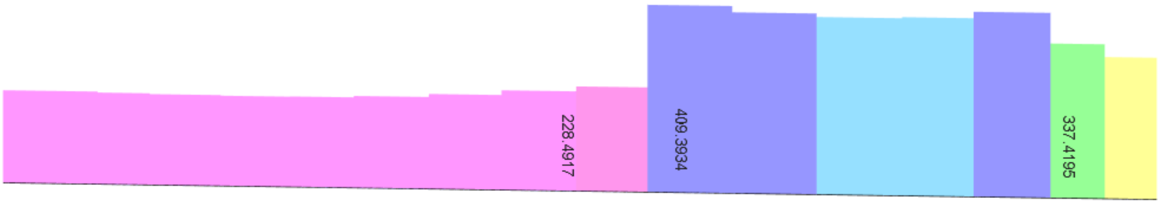


Figure 12.5.4. Maximum Shear 3-3 Loaded Beam EY Seismic Combination EC8.



12.6 Bending Moments in Columns and Beams – EC8 Seismic Combinations

The bending moment distribution followed the typical seismic pattern, with maximum positive and negative moments at beam–column joints. In beams, the largest moments were observed at the supports, reaching **298 kNm**. In columns, the largest bending moments occurred at the base, with peak values of **620 kNm**.

Figure 12.6.1. Maximum Moment 3-3 Loaded Column EX Seismic Combination EC8.

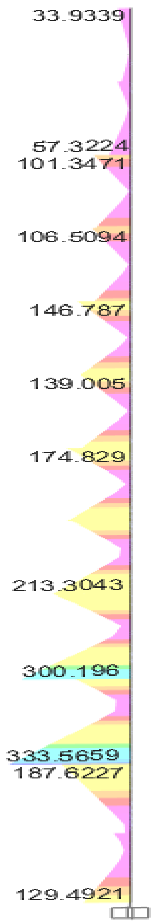


Figure 12.6.2. Maximum Moment 2-2 Loaded Column EY Seismic Combination EC8.

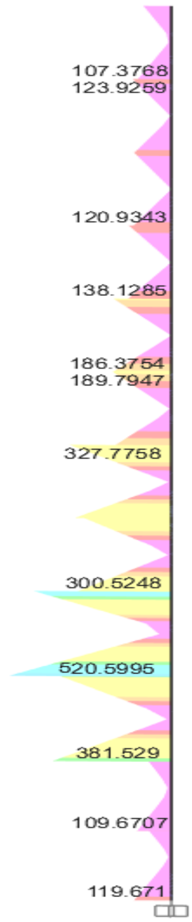


Figure 12.6.3. Maximum Moment 3-3 Beam EX Seismic Combination EC8.

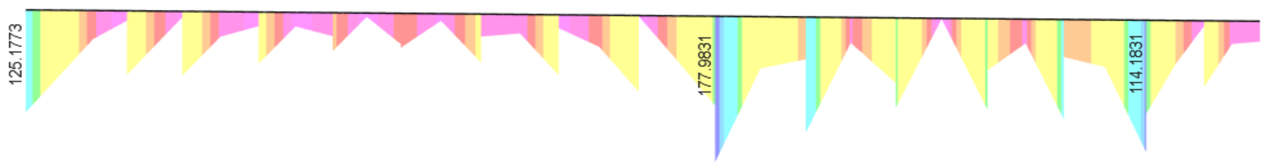
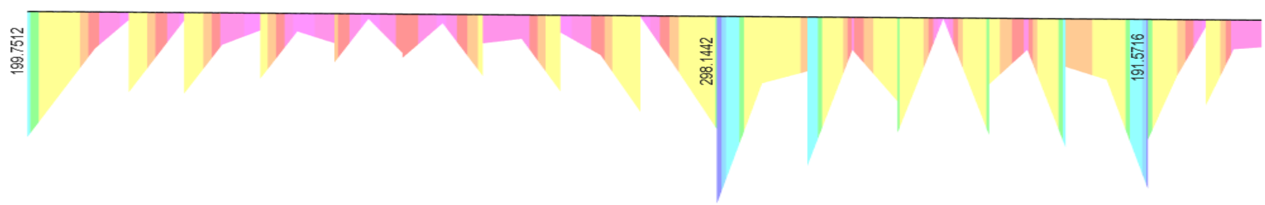


Figure 12.6.4. Maximum Moment 3-3 Beam EY Seismic Combination EC8.



13. RESPONSE SPECTRUM ANALYSIS RESULTS – NTC 2018

The second response spectrum analysis was performed using the **NTC2018 design spectrum**, to provide the comparison against which the Eurocode 8 and Albanian KTP-89 analyses. The following subsections present the main outcomes of the NTC 2018-based analysis, including global building response (base shear and displacements), local element forces including Moments and Shear Forces.

13.1 Maximum Story Displacement – NTC 2018 Seismic Combinations

The diagram above shows how the building responds laterally when the seismic action is applied in the **X direction and in the Y direction**, following the NTC2018 combination rules. On the horizontal axis we have the displacement values (in millimeters), while the vertical axis indicates the building stories.

Figure 13.1.1. Maximum Story Displacements for EY Seismic Combination NTC2018**.**

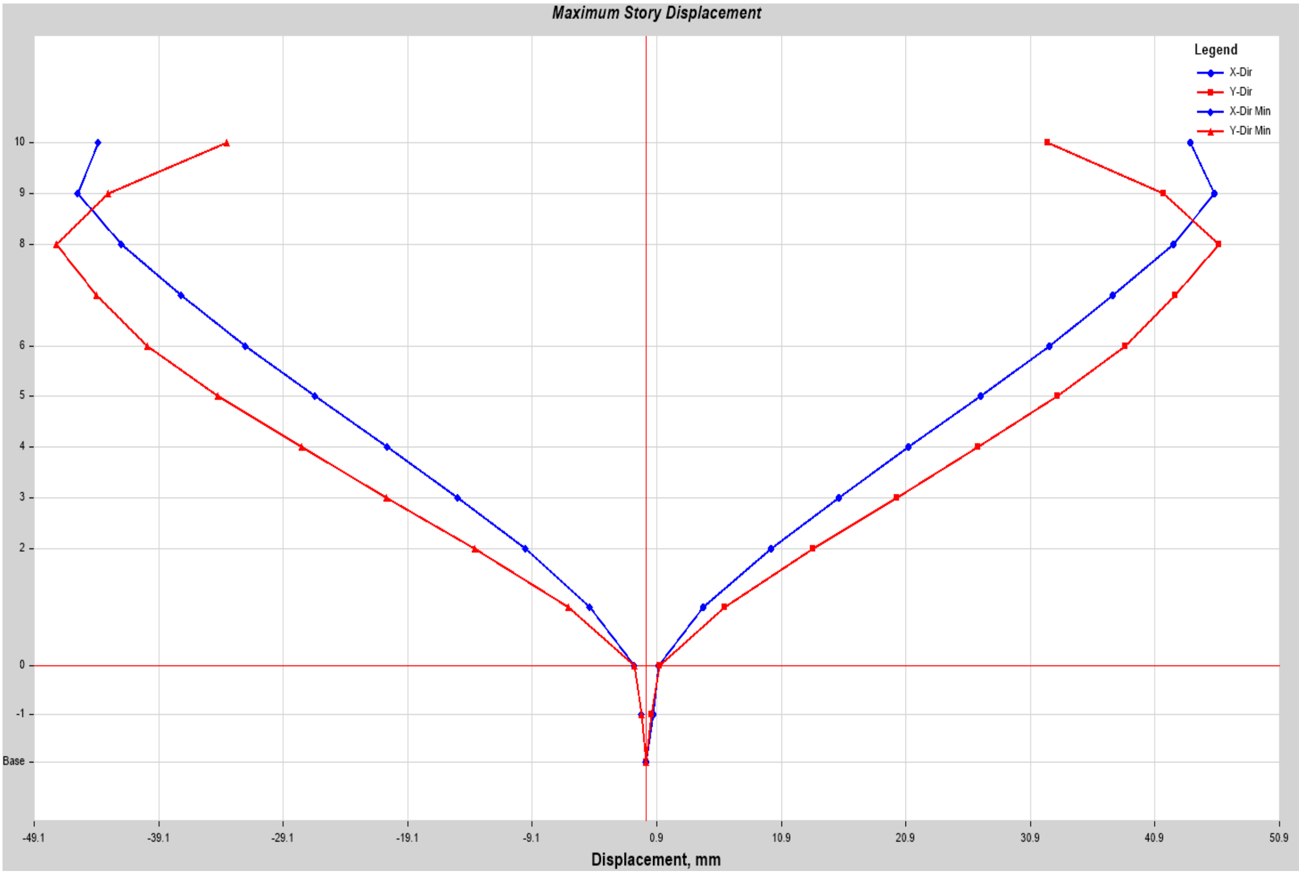
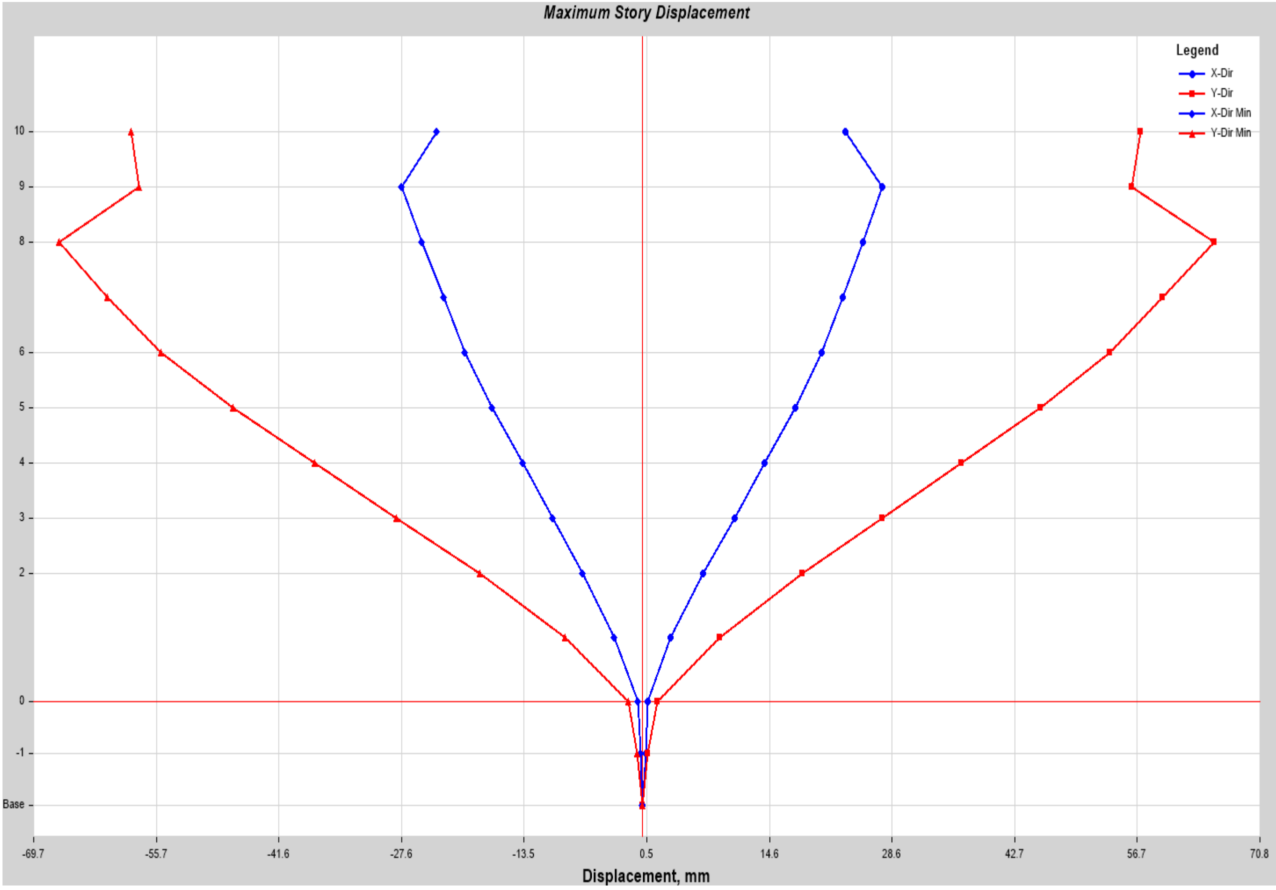


Figure 13.1.2. Maximum Story Displacements for EX Seismic Combination NTC2018**.**



What the graph really tells us is how the movement increases step by step as we go higher in the structure. At the base the displacements are very small, almost negligible, and then they gradually build up until they reach the maximum value at the roof. This is the typical way a multi-storey frame or wall system reacts under horizontal loads: it behaves somewhat like a cantilever, with the top moving the most.The two curves on either side represent the extreme values in both positive and negative directions. They are fairly symmetrical, which is expected since the analysis records the envelope of maximum displacements.

This type of result is important not just as a picture of the structural response, but because it is directly linked to design checks. NTC2018 requires us to control the lateral drift of each story so that damage is limited and the building can remain functional even after an earthquake. In other words, the figure is more than a graph—it's a way to judge if the structure is flexible enough to absorb seismic energy but still stiff enough to stay within safety limits.

13.3 Story Base Shear Results – Vx, Vy – NTC2018 Seismic Combinations

The total seismic base shear was obtained in both principal directions of the building. Under the NTC2018 spectrum, the structure developed a high story base shear in the X direction and a moderate story base shear in the Y direction. These values reflect the combined contribution of all significant vibration modes, with modal results combined using the Complete Quadratic Combination (CQC) method. As expected, the base shear magnitudes were directly influenced by the fundamental period of the building and the soil amplification factors associated with the chosen soil category.

Figure 13.2.1. Maximum Story Shear for EX Seismic Combination NTC2018

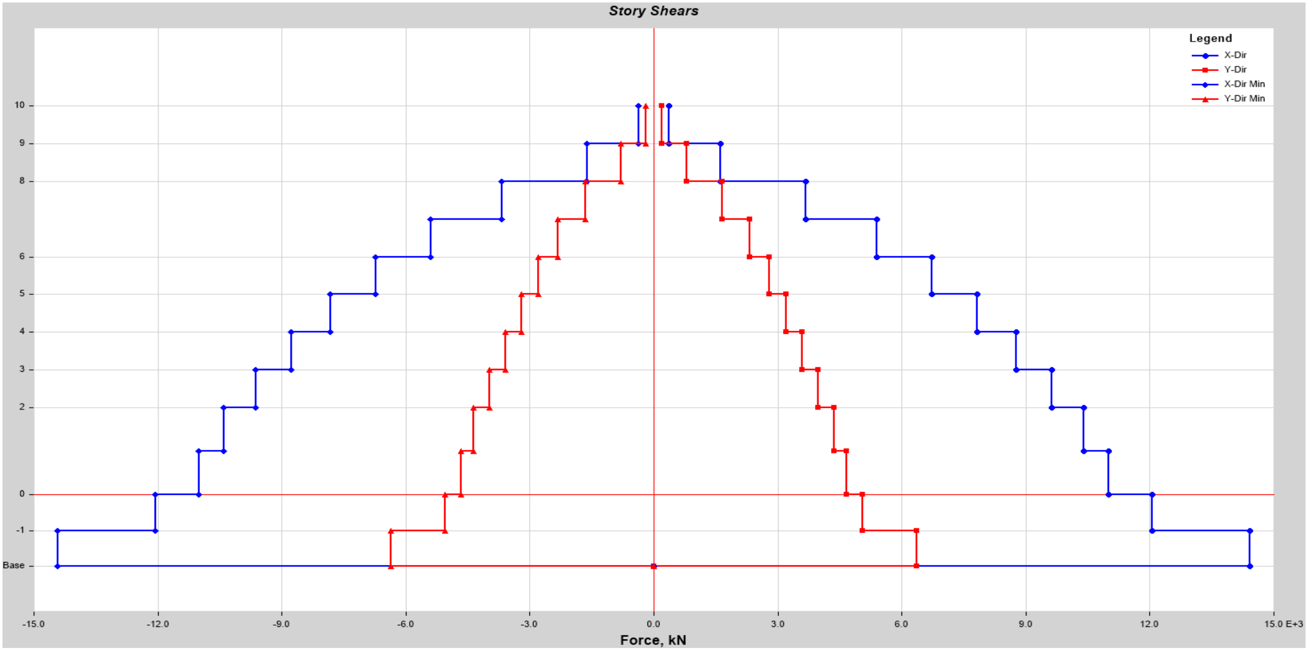
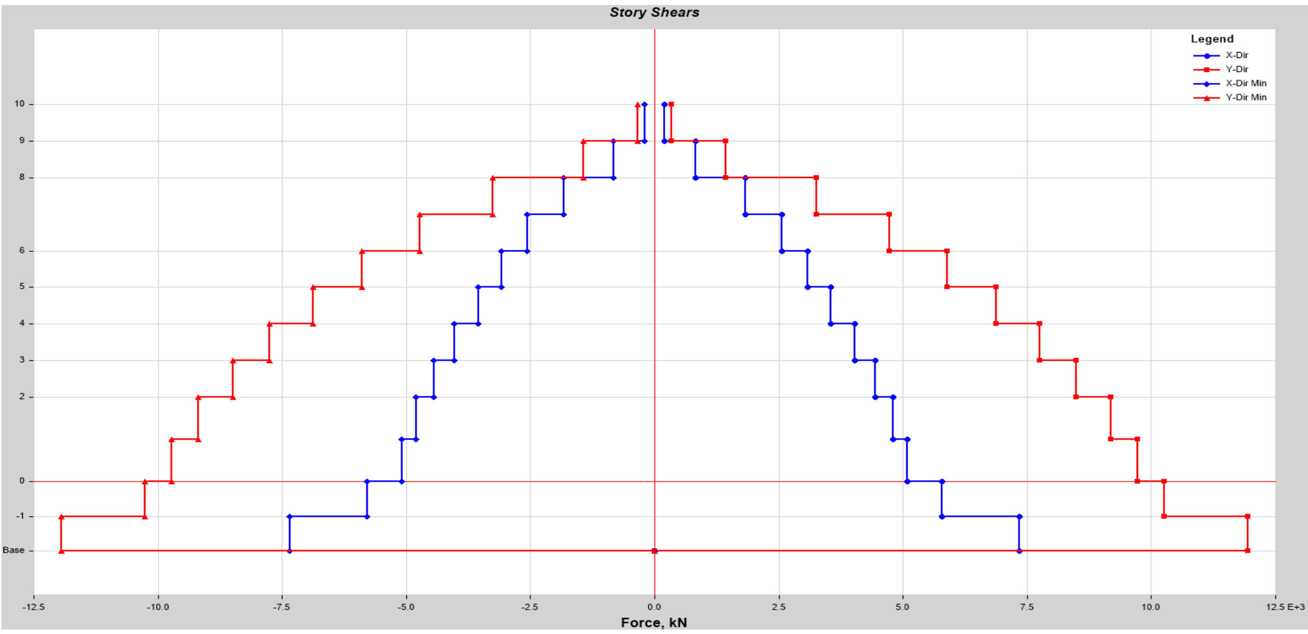


Figure 13.2.2. Maximum Story Shear for EY Seismic Combination NTC2018.



13.3 Maximum Story Drifts – NTC2018 Seismic Combinations

When a building is shaken by an earthquake, its floors do not move uniformly. Instead, each level displaces relative to the one below it, creating what is known as a **storey drift**. This drift is a direct measure of how much the building deforms laterally under seismic loading, and it is one of the key indicators of potential damage to both structural and non-structural elements.

NTC2018 introduces specific limits on these drifts to ensure that buildings not only remain standing but also maintain their serviceability and avoid excessive damage. The code sets maximum allowable inter-storey drift ratios, usually expressed as a fraction of the storey height (for example, **0.005 × storey height** for buildings under seismic design). This limit is intended to prevent issues like cracking in partitions, malfunction of doors and windows, or even structural instability if deformations become too large.

Figure 13.3.1. Maximum Story Drifts for EX Seismic Combination NTC2018.

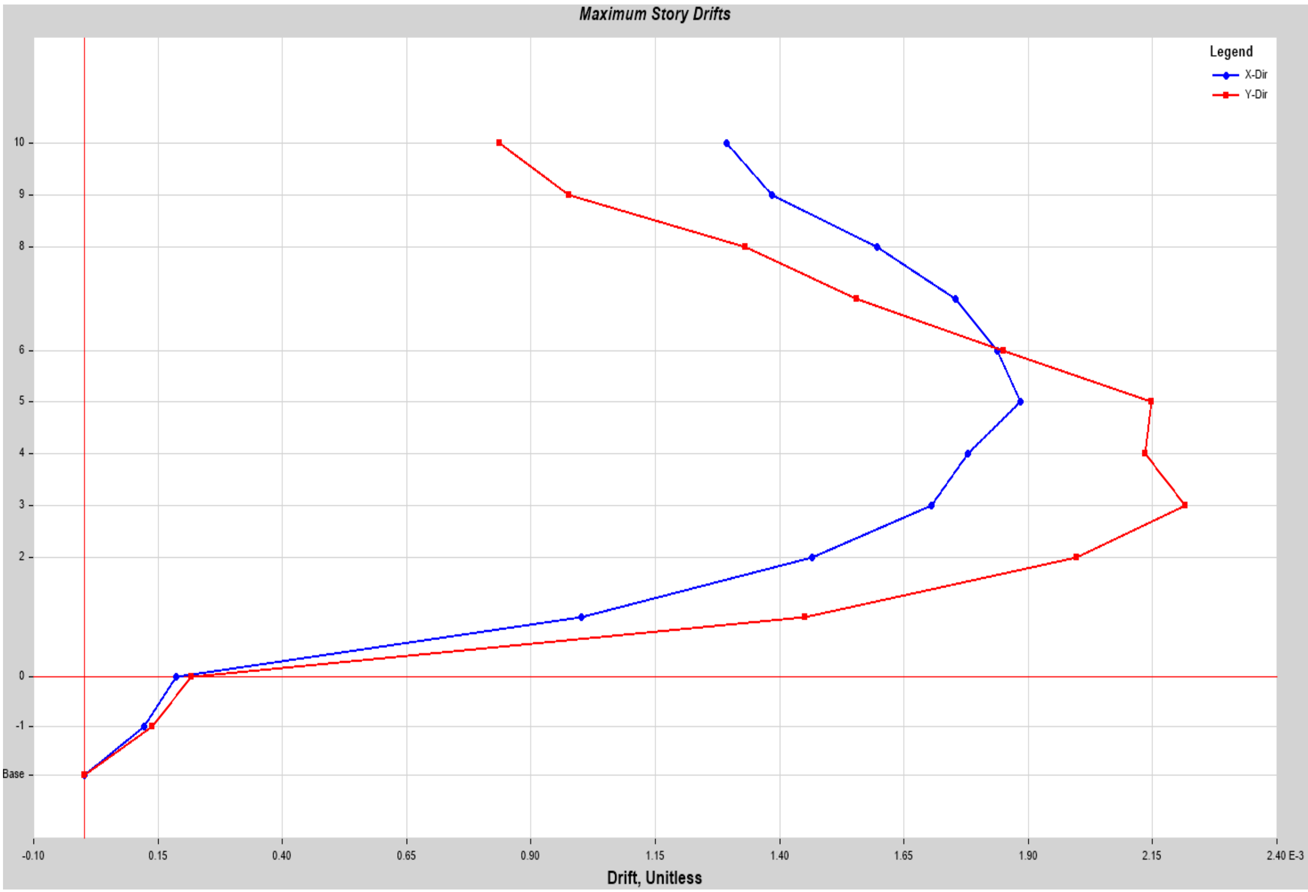
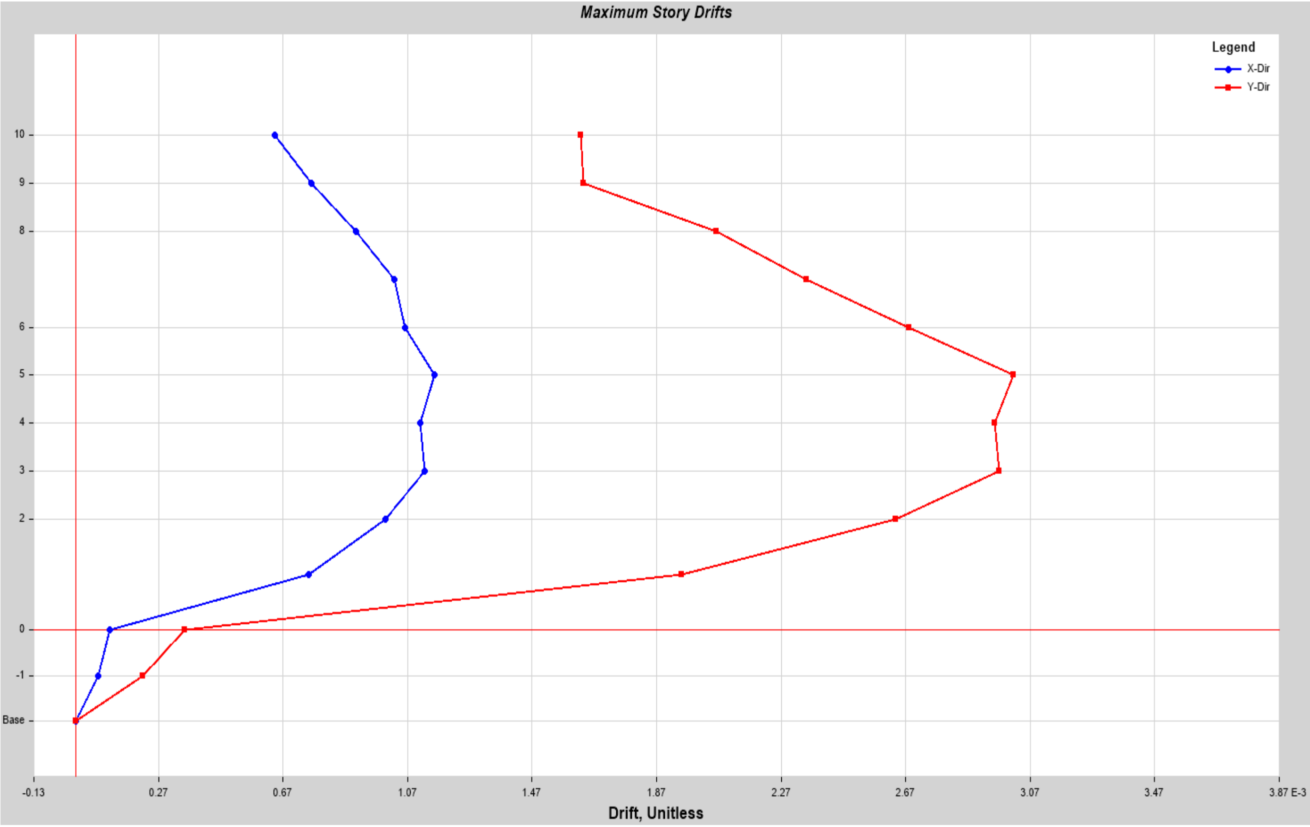


Figure 13.3.2. Maximum Story Drifts for EY Seismic Combination NTC2018



By checking both **EX** and **EY** cases, NTC2018 ensures that the building is verified for the full range of possible earthquake directions. This systematic approach provides confidence that the structure can deform safely within code-defined limits, without excessive risk of non-structural damage or loss of functionality.

13.4 Axial Forces in Columns – NTC2018 Seismic Combinations

Column axial forces were extracted to evaluate the combined effect of gravity and seismic actions. The maximum axial load recorded was **3420 kN**, concentrated in the centered columns of the lower storeys, where both gravity and lateral forces accumulate. This distribution is consistent with expectations: seismic overturning effects increase axial compression in some columns while reducing it in others, particularly at the building perimeter.

Figure 13.4.1. Maximum Axially Loaded Column
EX Seismic Combination NTC2018.

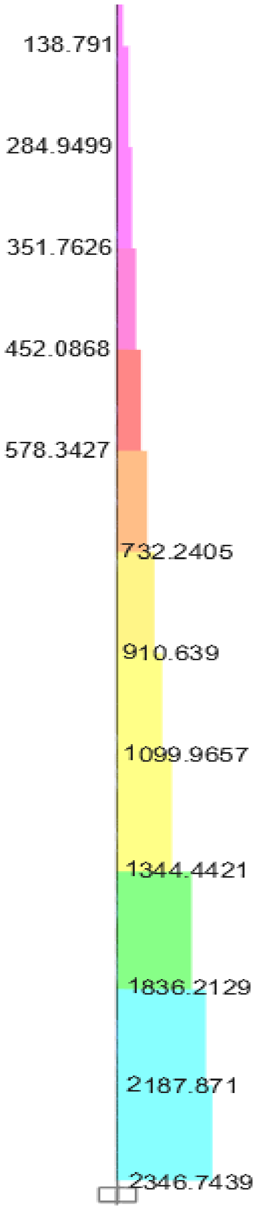
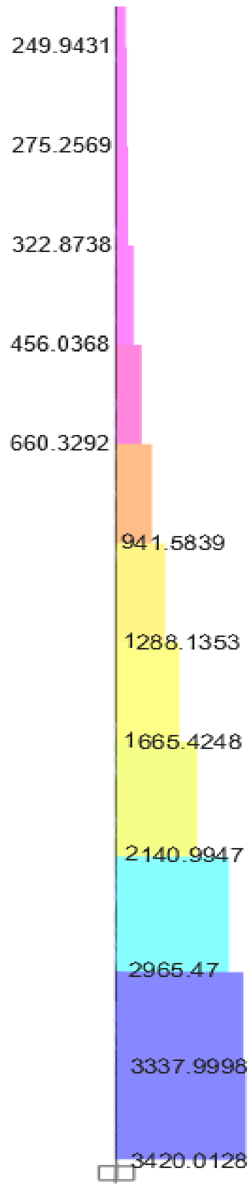


Figure 13.4.2. Maximum Axially Loaded Column
EY Seismic Combination NTC2018.



13.5 Shear Forces in Columns and Beams – NTC2018 Seismic Combinations

Shear force envelopes were generated for both beams and columns. Columns at the base exhibited maximum shear forces of **262 kN**, while beams at intermediate floors experienced shear peaks of **418 kN**. These results are important because shear governs the detailing of transverse reinforcement.

Figure 12.5.1. Maximum Shear 3-3 Loaded Column EX Seismic Combination NTC2018.

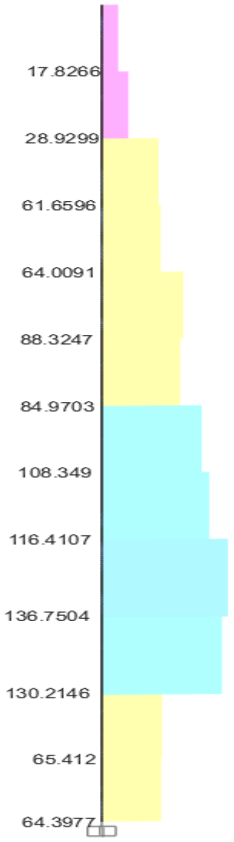


Figure 12.5.2. Maximum Shear 2-2 Loaded Column EY Seismic Combination NTC2018.

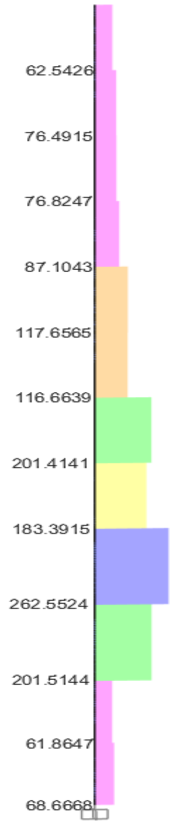


Figure 12.5.3. Maximum Shear 3-3 Loaded Beam EX Seismic Combination NTC2018.

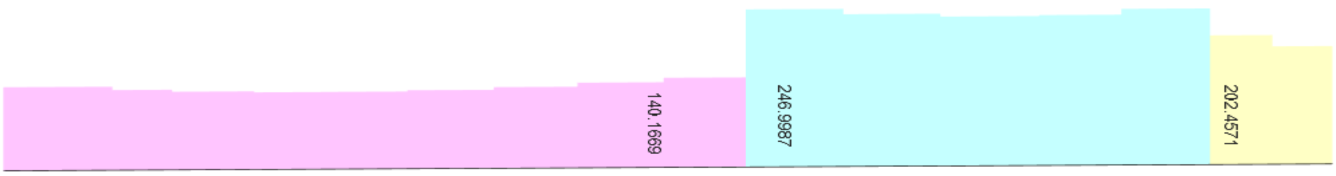
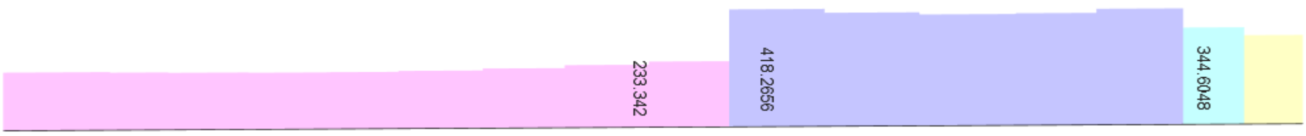


Figure 13.5.4. Maximum Shear 3-3 Loaded Beam EY Seismic Combination NTC2018.



13.6 Bending Moments in Columns and Beams – NTC18 Seismic Combinations

The bending moment distribution followed the typical seismic pattern, with maximum positive and negative moments at beam–column joints. In beams, the largest moments were observed at the supports, reaching **304 kNm**. In columns, the largest bending moments occurred at the base, with peak values of **531 kNm**.

Figure 13.6.1. Maximum Moment 2-2 Loaded Column EX Seismic Combination NTC2018.

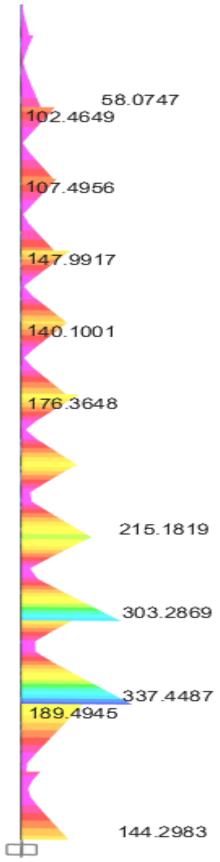


Figure 13.6.2. Maximum Moment 3-3 Loaded Column EY Seismic Combination NTC2018.

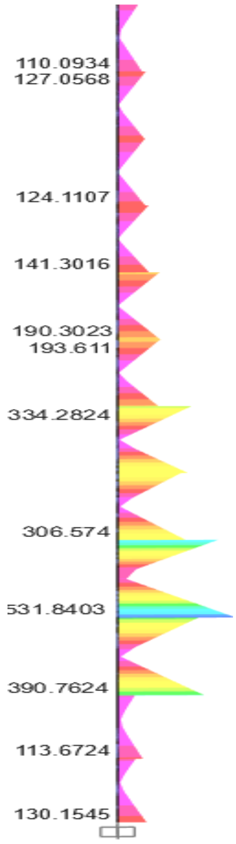


Figure 13.6.3. Maximum Moment 3-3 Beam EX Seismic Combination NTC2018.

Figure 13.6.4. Maximum Moment 3-3 Beam EY Seismic Combination NTC2018.

14. RESPONSE SPECTRUM ANALYSIS RESULTS – KTP89

The last response spectrum analysis was performed using the **KTP89 design spectrum**. This provided the results against the Italian NTC2018 and Eurocode 8 analyses that will later be compared. The following subsections present the main outcomes of the KTP89-based analysis, including global building response (base shear and displacements), local element forces including Moments and Shear Forces.

14.1 Maximum Story Displacement – KTP89 Seismic Combinations

The diagram above shows how the building responds laterally when the seismic action is applied in the **X direction and in the Y direction**, following the KTP89 combination rules. On the horizontal axis we have the displacement values (in millimeters), while the vertical axis indicates the building stories.

Figure 14.1.1. Maximum Story Displacements for EY Seismic Combination KTP89**.**

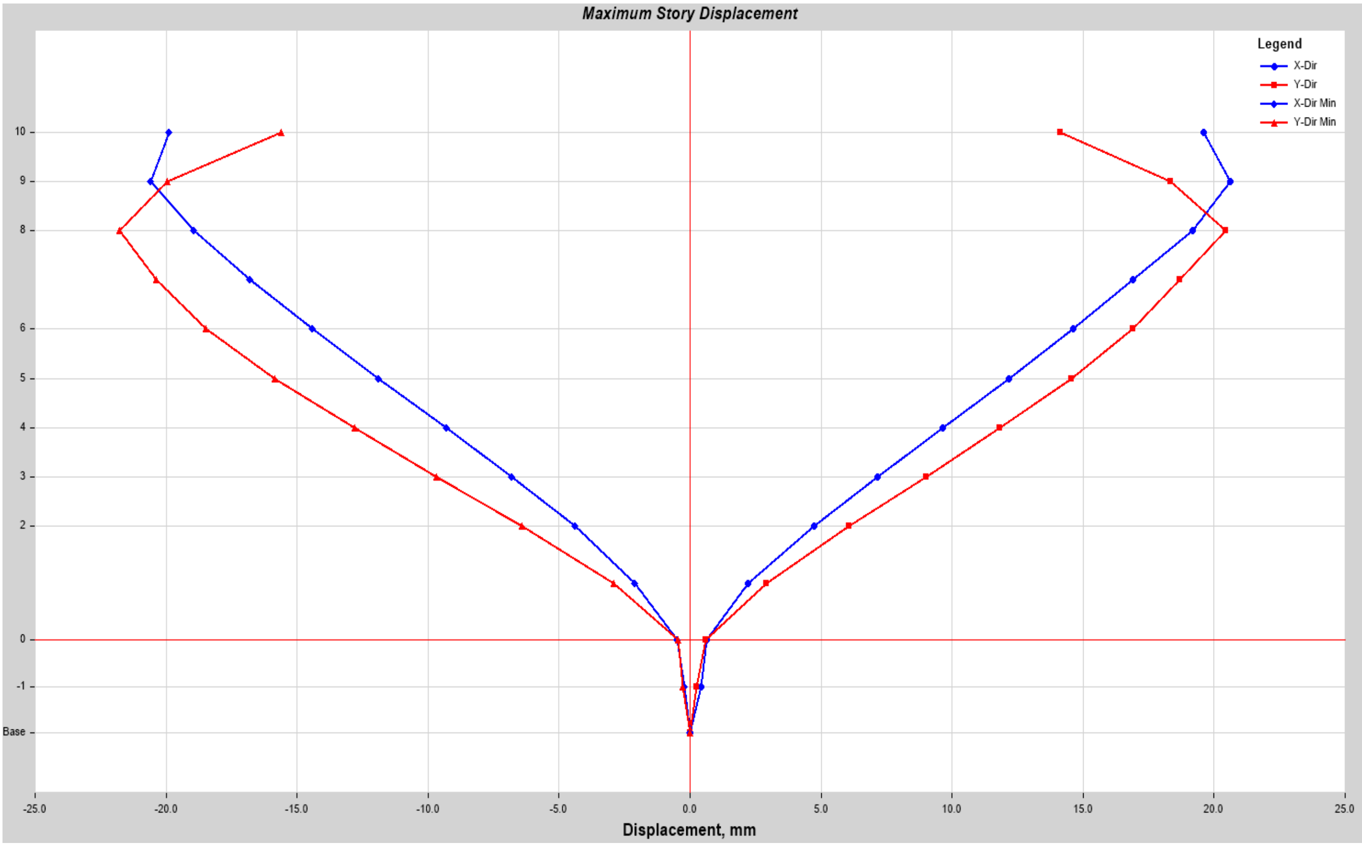
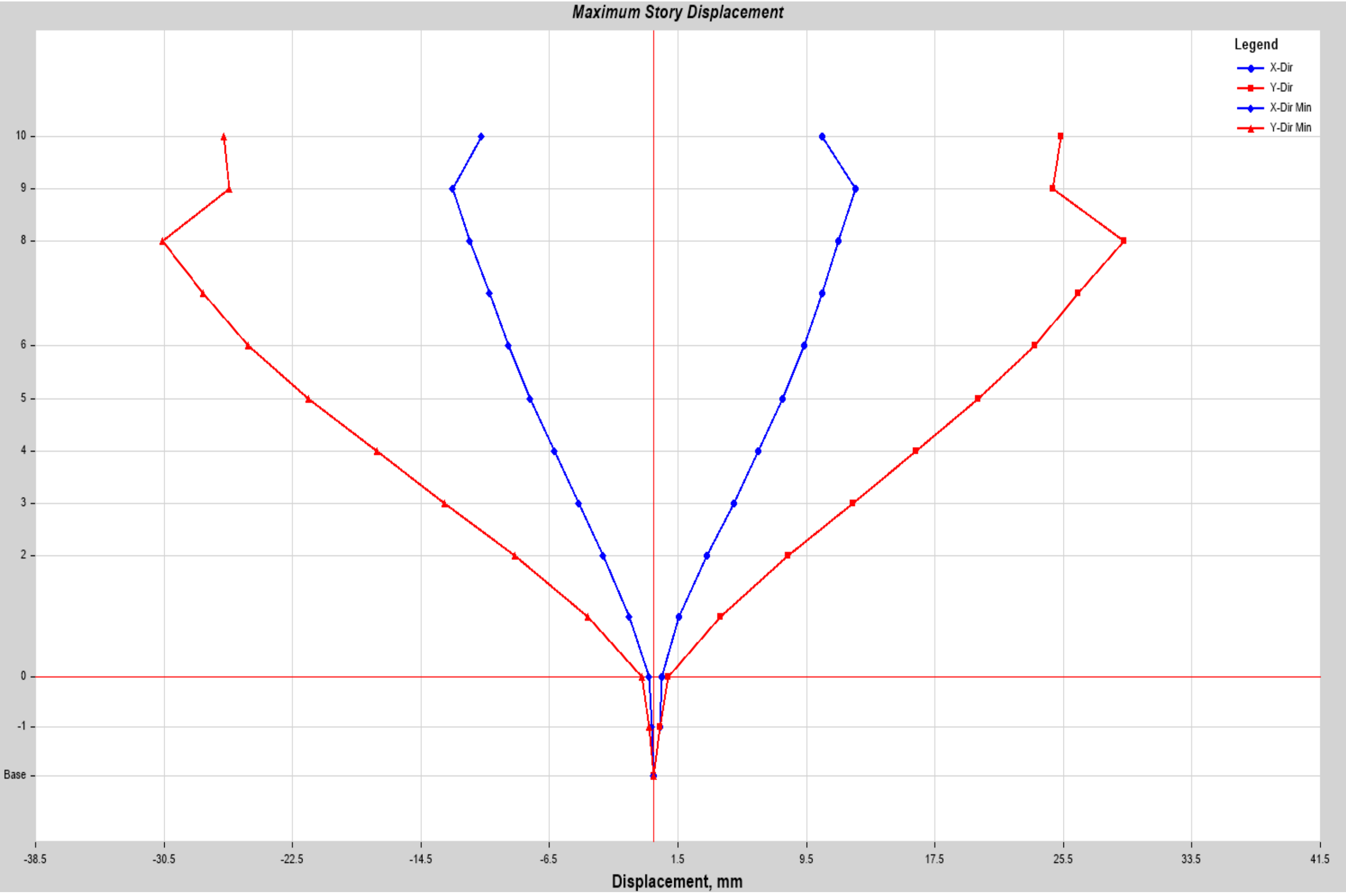


Figure 14.1.2. Maximum Story Displacements for EX Seismic Combination KTP89**.**



What the graph really tells us is how the movement increases step by step as we go higher in the structure. At the base the displacements are very small, almost negligible, and then they gradually build up until they reach the maximum value at the roof. This is the typical way a multi-storey frame or wall system reacts under horizontal loads: it behaves somewhat like a cantilever, with the top moving the most.The two curves on either side represent the extreme values in both positive and negative directions. They are fairly symmetrical, which is expected since the analysis records the envelope of maximum displacements.

12.2 Story Base Shear Results – Vx, Vy – KTP89 Seismic Combinations

The total seismic base shear was obtained in both principal directions of the building. Under the KPT89 spectrum, the structure developed a high story base shear in the X direction and a moderate story base shear in the Y direction. These values reflect the combined contribution of all significant vibration modes, with modal results combined using the Complete Quadratic Combination (CQC) method. As expected, the base shear magnitudes were directly influenced by the fundamental period of the building and the soil amplification factors associated with the chosen soil category.

Figure 14.2.1. Maximum Story Shear for EX Seismic Combination KTP89

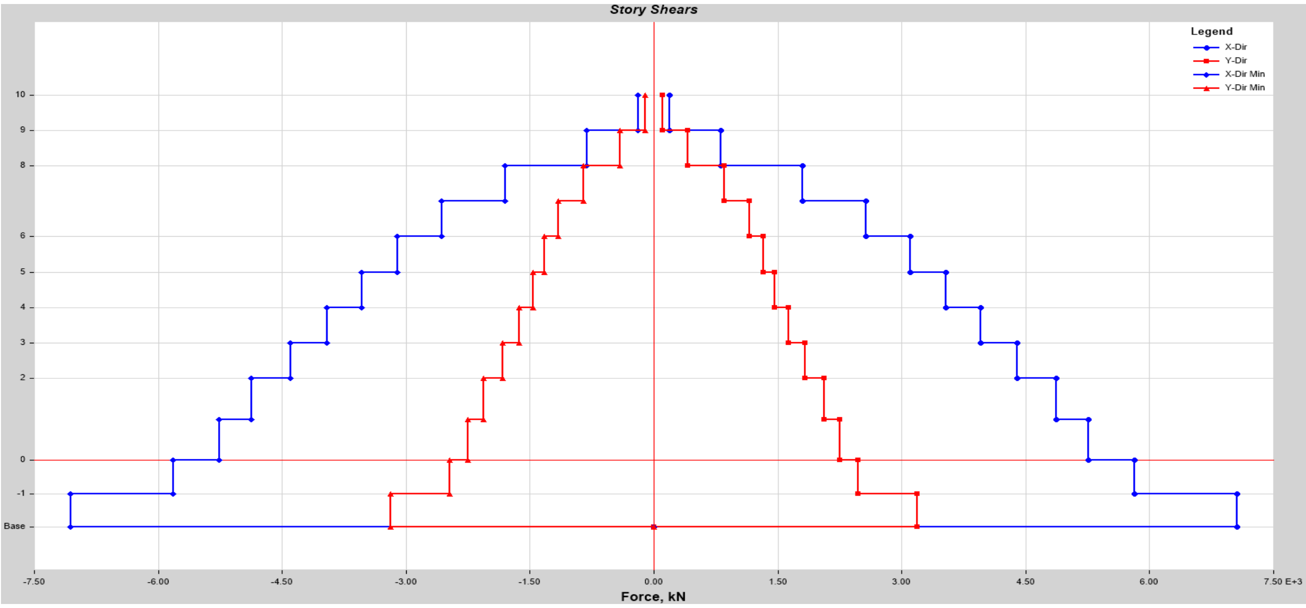
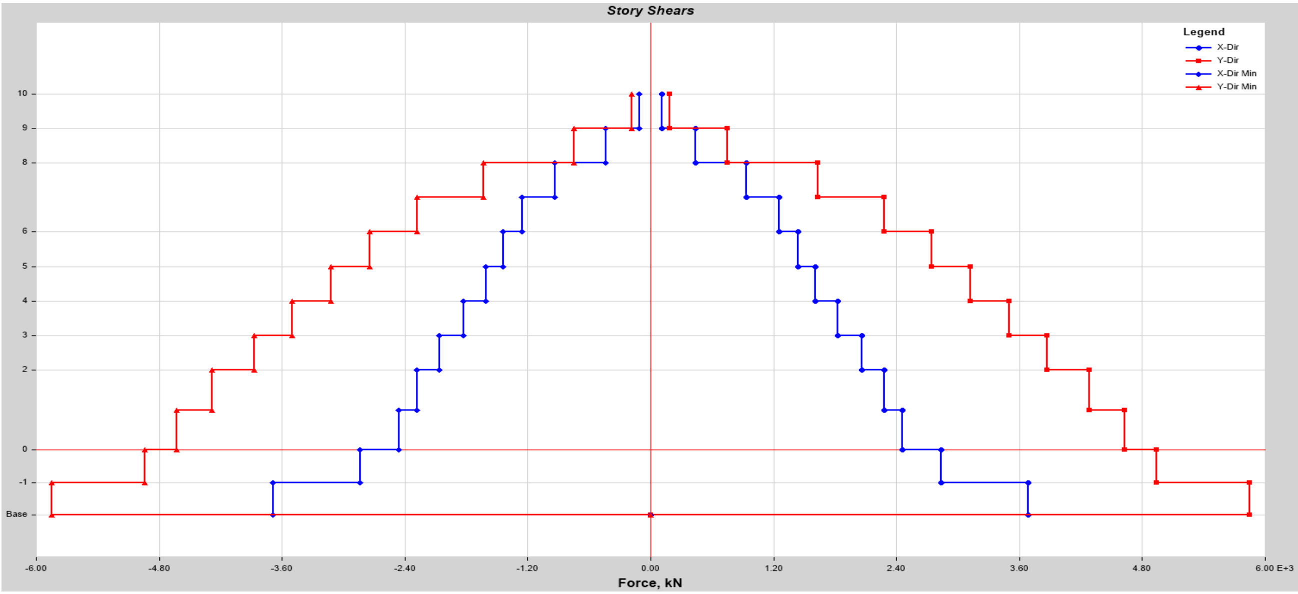


Figure 14.2.2. Maximum Story Shear for EY Seismic Combination KTP89.



14.3 Maximum Story Drifts – KTP89 Seismic Combinations

When a building is shaken by an earthquake, its floors do not move uniformly. Instead, each level displaces relative to the one below it, creating what is known as a **storey drift**. This drift is a direct measure of how much the building deforms laterally under seismic loading, and it is one of the key indicators of potential damage to both structural and non-structural elements.

KTP89 introduces specific limits on these drifts to ensure that buildings not only remain standing but also maintain their serviceability and avoid excessive damage. The code sets maximum allowable inter-storey drift ratios, usually expressed as a fraction of the storey height (for example, **0.0065 × storey height** for buildings under seismic design). This limit is intended to prevent issues like cracking in partitions, malfunction of doors and windows, or even structural instability if deformations become too large.

Figure 14.3.1. Maximum Story Drifts for EX Seismic Combination KPT89.

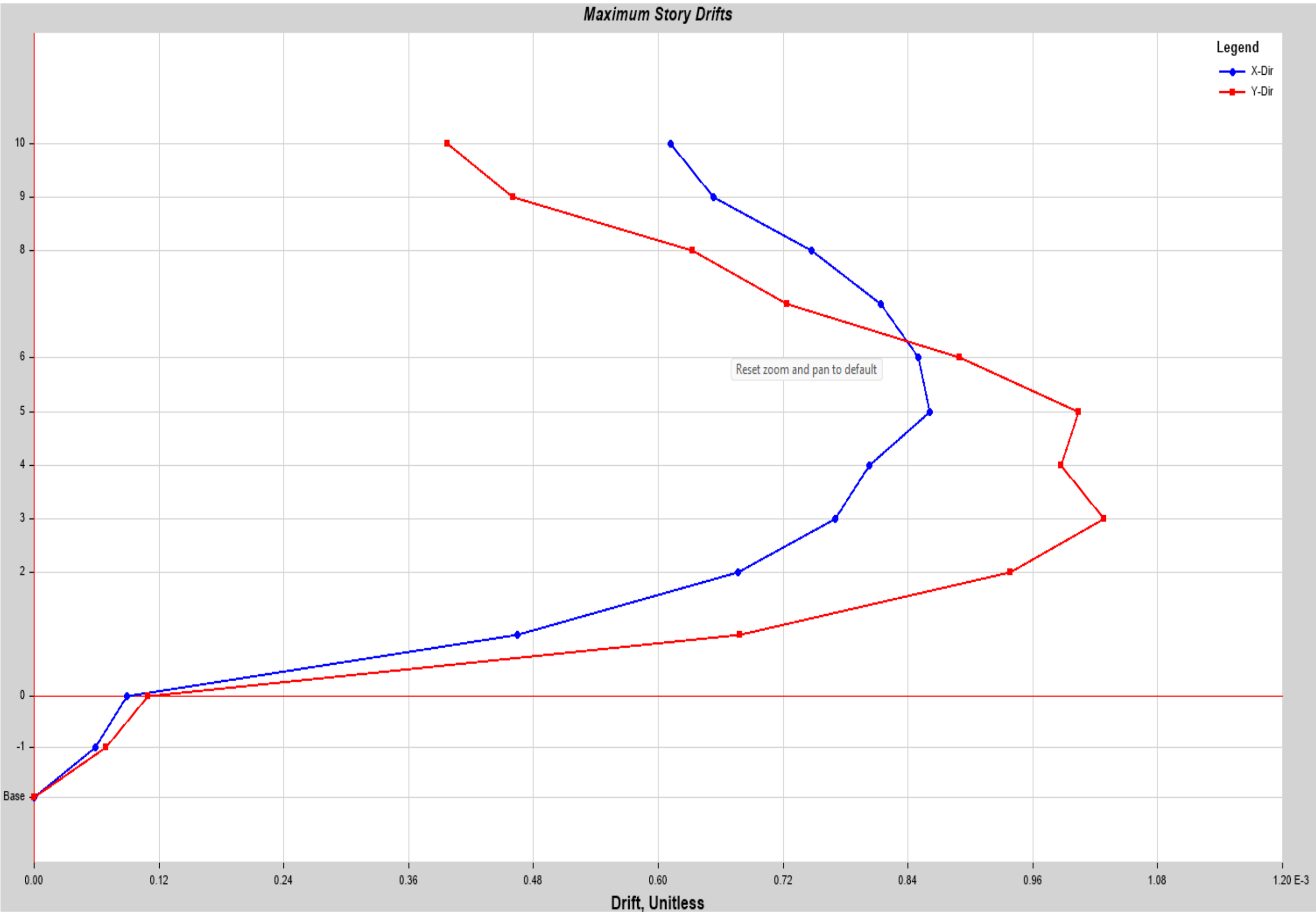
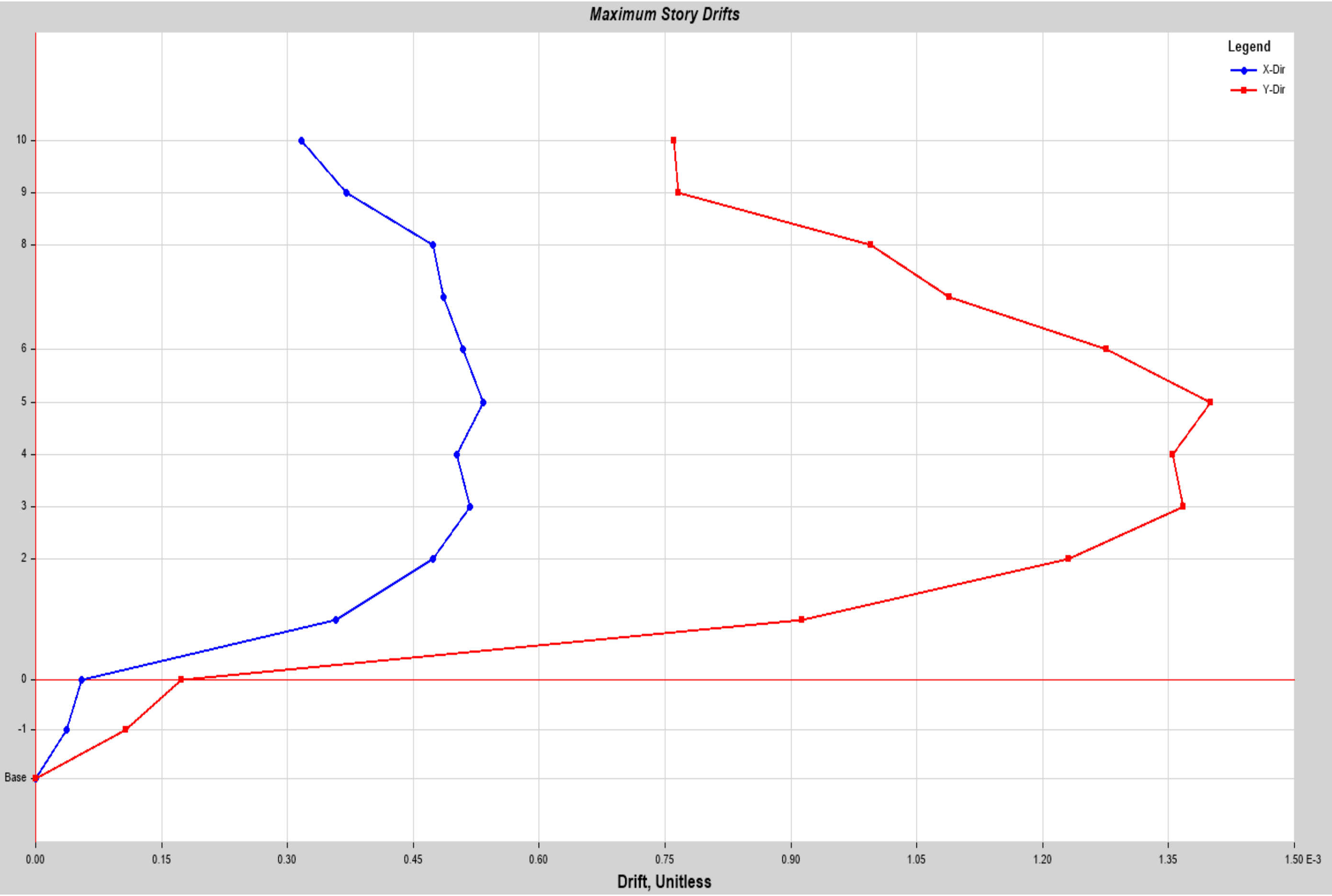


Figure 14.3.2. Maximum Story Drifts for EY Seismic Combination KTP89.



By checking both **EX** and **EY** cases, KTP89 ensures that the building is verified for the full range of possible earthquake directions. This systematic approach provides confidence that the structure can deform safely within code-defined limits, without excessive risk of non-structural damage or loss of functionality.

14.4 Axial Forces in Columns – KTP89 Seismic Combinations

Column axial forces were extracted to evaluate the combined effect of gravity and seismic actions. The maximum axial load recorded was **1541 kN**, concentrated in the centered columns of the lower storeys, where both gravity and lateral forces accumulate. This distribution is consistent with expectations: seismic overturning effects increase axial compression in some columns while reducing it in others, particularly at the building perimeter.

Figure 14.4.1. Maximum Axially Loaded Column
EX Seismic Combination KTP89.

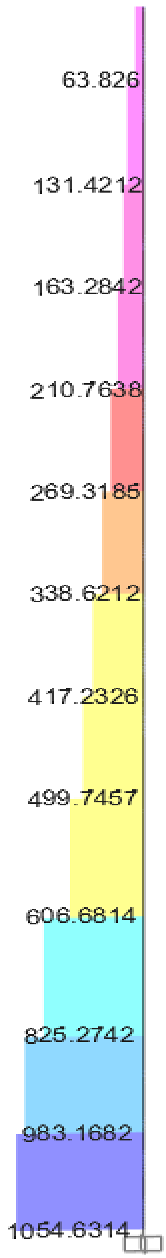
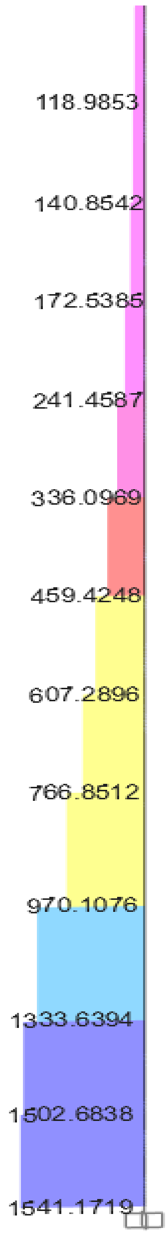


Figure 14.4.2. Maximum Axially Loaded Column
EY Seismic Combination KTP89.



14.5 Shear Forces in Columns and Beams – KTP89 Seismic Combinations

Shear force envelopes were generated for both beams and columns. Columns at the base exhibited maximum shear forces of **121 kN**, while beams at intermediate floors experienced shear peaks of **191 kN**. These results are important because shear governs the detailing of transverse reinforcement.

Figure 14.5.1. Maximum Shear 3-3 Loaded Column EX Seismic Combination KTP89.

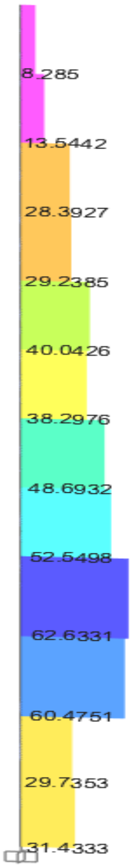


Figure 14.5.2. Maximum Shear 2-2 Loaded Column EY Seismic Combination KTP89.

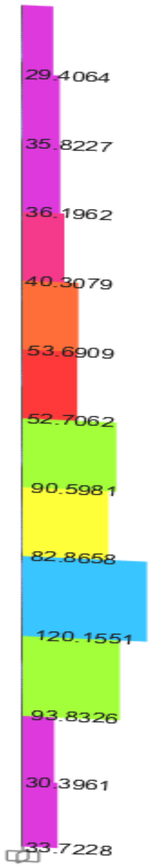


Figure 14.5.3. Maximum Shear 2-2 Loaded Beam EX Seismic Combination KTP89.

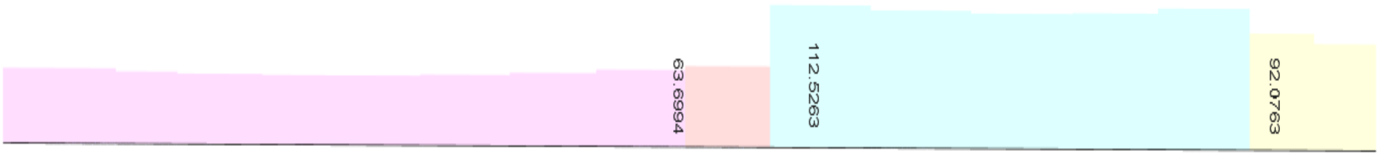
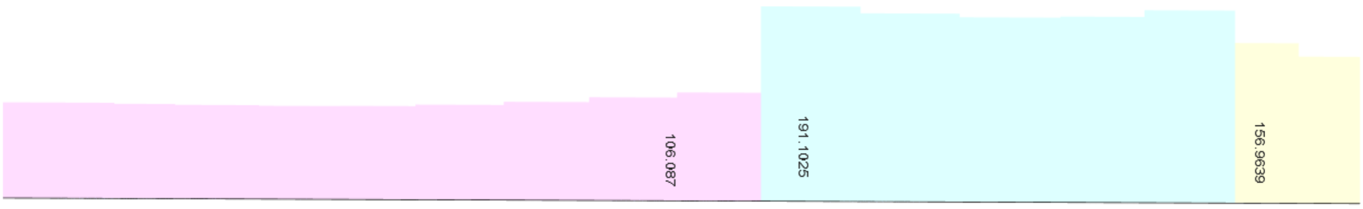


Figure 14.5.4. Maximum Shear 3-3 Loaded Beam EY Seismic Combination KTP89.



14.6 Bending Moments in Columns and Beams – KTP89 Seismic Combinations

The bending moment distribution followed the typical seismic pattern, with maximum positive and negative moments at beam–column joints. In beams, the largest moments were observed at the supports, reaching **139.2 kNm**. In columns, the largest bending moments occurred at the base, with peak values of **243 kNm**.

Figure 12.6.1. Maximum Moment 3-3 Loaded Column EX Seismic Combination KTP89.

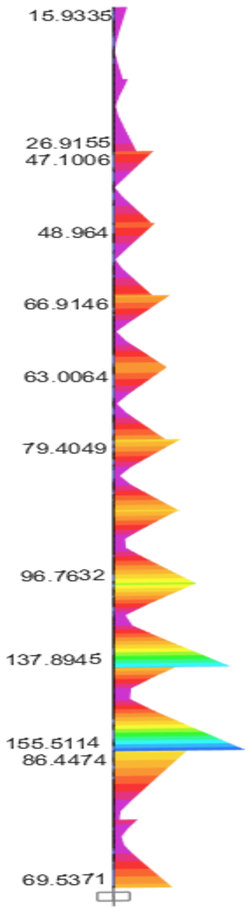


Figure 12.6.2. Maximum Moment 2-2 Loaded Column EY Seismic Combination KTP89.

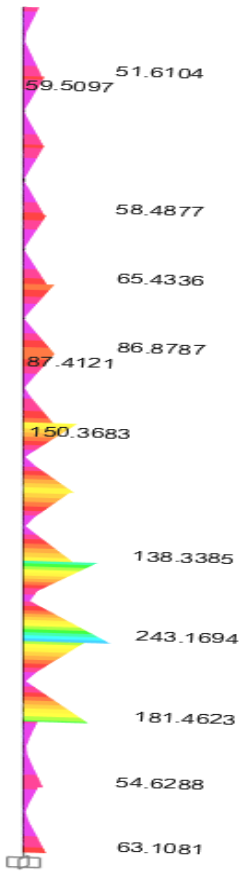


Figure 12.6.3. Maximum Moment 3-3 Beam EX Seismic Combination KTP89.

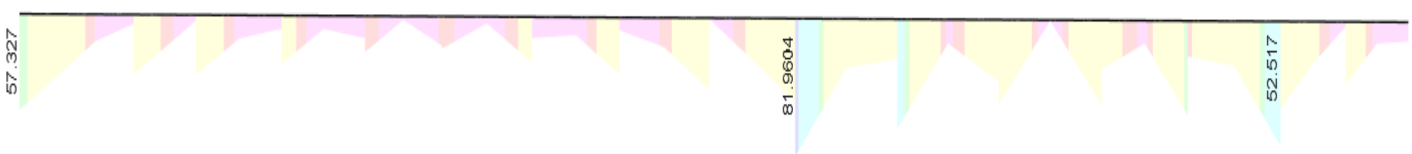


Figure 12.6.4. Maximum Moment 3-3 Beam EY Seismic Combination KTP89.

15. COMPARATIVE ANALYSIS OF THE RESULTS

The three response spectrum analyses - Eurocode 8, Italian NTC2018, and Albanian KTP-89 provide different perspectives on the seismic demand for the same structure. By comparing the results side by side, it becomes clear how the choice of code influences design quantities such as story base shear, displacements, drifts and internal forces. The following subsections present the main comparisons.

15.1 Story Displacements Comparison – Codes vs Each Other

The maximum story displacements obtained from the EC8 for the X direction Earthquake Combination is **66.3 mm**, from NTC2018 it is **66.8 mm**, and from KTP-89 **21.9 mm**.

The maximum story displacements obtained from the EC8 for the Y direction Earthquake Combination is **46.1 mm**, from NTC2018 it is **47.1 mm**, and from KTP-89 **30.5 mm**.

The results show that EC8 and NTC2018 are generally consistent, while KTP-89 tends to produce either lower values depending on the soil assumptions. This reflects the more simplified formulation of KTP-89 compared with the refined, soil-specific approaches of the modern codes.

MAX Story Displacements Comparison		
Code	Seismic Combination	
	EX	EY
EC8	66.3mm	46.1 mm
NTC2018	66.8 mm	47.1 mm
KTP89	21.9 mm	30.5 mm

Table 15.1. Maximum Story Displacements Comparison.

15.2 Story Base Shear Comparison – Codes vs Each Other

The maximum base shear obtained from the EC8 for the X direction Earthquake Combination was **12698 kN**, from NTC2018 **14430 kN**, and from KTP-89 **7054 kN**.

The maximum base shear obtained from the EC8 for the Y direction Earthquake Combination was **10770 kN**, from NTC2018 **11941 kN**, and from KTP-89 **5582 kN**.

The results show that EC8 and NTC2018 are moderately consistent, while KTP-89 tends to produce either lower values. This reflects the more simplified formulation of KTP-89 compared with the refined, soil-specific approaches of the modern codes.

MAX Story Shear Comparison		
Code	Seismic Combination	
	EX	EY
EC8	12698 kN	10770 kN
NTC2018	14430 kN	11941 kN
KTP89	7054 kN	5582 kN

Table 15.2. Maximum Story Shear Comparison.

15.3 Drift Profile Comparison – Storey Drift Curves

Interstorey drift ratios, plotted storey by storey, highlight how each code influences the lateral flexibility of the building. EC8 and NTC2018 both enforce drift checks against limits (commonly 0.4%–0.5% of storey height), while KTP-89 does not explicitly provide such limits. The comparison shows that drifts remain **within limits for EC8 and NTC2018**, whereas under KTP-89 the values may not always reflect the true deformation demand.

The maximum drift obtained from the EC8 for the X direction Earthquake Combination was **0.002**, from NTC2018 **0.0022**, and from KTP-89 **0.001**.

The maximum drift obtained from the EC8 for the X direction Earthquake Combination was **0.0029**, from NTC2018 **0.003**, and from KTP-89 **0.0014**.

MAX Story Drifts Comparison		
Code	Seismic Combination	
	EX	EY
EC8	0.0020	0.0029
NTC2018	0.0022	0.0030
KTP89	0.0010	0.0014

Table 15.3. Maximum Story Drifts Comparison.

15.4 Member Force Comparison – Axial, Shear, Bending Moment

Columns at the base carried maximum axial loads for the Seismic Combinations of **3352 kN under EC8**, **3420 kN under NTC2018**, and **1541 kN under KTP-89**. Similarly,

Columns at the base carried maximum shear forces for the Seismic Combinations of **256 kN under EC8**, **262 kN under NTC2018**, and **121 kN under KTP-89**

Columns carried maximum bending moments for the Seismic Combinations of **620 kNm under EC8**, **541 kNm under NTC2018**, and **243 kNm under KTP-89**

Beams at the carried maximum shear forces for the Seismic Combinations of **409 kN under EC8**, **418 kN under NTC2018**, and **191 kN under KTP-89**

Beams carried maximum bending moments for the Seismic Combinations of **298 kNm under EC8**, **304 kNm under NTC2018**, and **140 kNm under KTP-89**

MAX Column Internal Forces Comparison			
Code	Max Column Axial Loads	Max Column Shear Forces	Max Column Bending Moments
EC8	3352 kN	256 kN	620 kNm
NTC2018	3420 kN	262 kN	541 kNm
KTP89	1541 kN	121 kN	243 kNm

Table 15.4. & **Table 15.5** Maximum Internal Forces Comparison.

MAX Beam Internal Forces Comparison		
Code	Max Beam Shear Forces	Max Beam Bending Moments
EC8	409 kN	298 kNm
NTC2018	418 kN	304 kNm
KTP89	191 kN	140 kNm

These values shows that the internal forces and bending moments followed the same trend as global base shear: modern codes produced more conservative design forces, while KTP-89 gave simplified and often lower demands.

16. DISCUSSION OF COMPARATIVE RESULTS

The comparative analysis highlights clear differences between the modern seismic codes (EC8 and NTC2018) and the older Albanian standard KTP89. While EC8 and NTC2018 show remarkable consistency across almost all parameters, KTP89 systematically underestimates seismic demands. These differences are not accidental; they stem from the evolution of seismic design philosophy, changes in spectrum formulation, and a deeper understanding of soil–structure interaction.

In terms of displacements and drifts, EC8 and NTC2018 predict nearly identical values, demonstrating their close alignment. Both codes adopt a modern elastic response spectrum that accounts for soil type and distinguishes between short- and long-period ranges. This ensures that flexible structures, such as taller buildings, are properly penalized for their higher deformation demands. KTP89, by contrast, uses a much simpler spectrum formulation, without explicit soil amplification factors, and applies uniform peak values regardless of soil category. This explains why it consistently produces lower displacements and drift ratios: the influence of softer soils and long-period behavior is essentially ignored, leading to an unrealistic picture of structural performance.

The differences become even more pronounced in terms of story shear and internal forces. NTC2018 tends to be slightly more conservative than EC8, especially in shear forces, due to national choices that reflect the higher seismicity of Italy and the desire for additional safety margins. KTP89, however, produces forces that are often less than half of those predicted by the modern codes. From a design perspective, this means columns and beams sized according to KTP89 would require far less reinforcement. While this may appear economical, it undermines essential ductility and overstrength principles. Modern seismic design is based not only on strength but also on ensuring controlled inelastic behavior, something KTP89 does not explicitly address.

Another important aspect is the role of soil classification. EC8 and NTC2018 both incorporate detailed soil categories (A–E) and modify the spectral shape accordingly. Soft soils lengthen the fundamental period of the structure and amplify spectral accelerations, directly increasing displacements, drifts, and shear demands. KTP89 makes no such distinctions, which again contributes to the systematic underestimation of demands in more flexible soil conditions. For a country like Albania, where soft soils are common in many urban areas (e.g., Tirana, Durrës), this omission is particularly critical.

From a broader perspective, the comparison illustrates the relevance of using updated codes. KTP89 reflects the knowledge and assumptions of the late 1980s, when ductility-based design, soil amplification, and performance-based principles were still emerging. The lessons from this study confirm that relying on such an outdated code would risk underestimating true seismic vulnerability. In contrast, EC8 and NTC2018 not only provide more realistic structural demands but also embed modern design strategies such as capacity design, control of drifts, and consideration of non-structural elements.

The practical implications for Albanian designers are significant. Continuing to apply KTP89 would result in buildings that may not meet modern safety expectations, especially under strong earthquakes. On the other hand, adopting EC8 or a harmonized national version would ensure designs are consistent with European practice, improve structural resilience, and facilitate integration into the broader engineering market.

In conclusion, these findings strongly reinforce the main theme of this thesis: the response of a ten-storey reinforced concrete building varies significantly depending on the design code applied. EC8 and NTC2018 produce results that are realistic, consistent, and safety-oriented, while KTP89 consistently underestimates demands due to its simplified spectrum and outdated philosophy. The multi-code comparison carried out in this work not only quantifies these differences but also demonstrates why Albania should align its national provisions with Eurocode 8, ensuring safer, more resilient structures for the future.

17. CONCLUSIONS

17.1 Summary of Findings – Key Takeaways

This thesis set out to evaluate the seismic response of a ten-storey reinforced concrete tower with two underground levels, using response spectrum analysis based on three different seismic codes: Eurocode 8 (EC8)**,** Italian NTC2018**,** and Albanian KTP-89**.** The main findings can be summarized as follows:

Base shear and forces: EC8 and NTC2018 produced broadly consistent base shear values, while KTP-89 generally underestimated the seismic demand due to its simplified spectrum.

Displacements and drifts: Modern codes predicted larger displacements, reflecting their consideration of soil amplification and spectrum shape. KTP-89 results were less representative of real site conditions.

Code philosophy: EC8 and NTC2018 reflect modern, performance-based design philosophies, while KTP-89 follows an older, zonation-based approach that is less site-specific.

Materials and Safety Factors

Aspect	EC8	NTC2018	KTP-89
Concrete grades commonly used	C25/30, C30/37 (with γC)	Same as EC8	Mark-based (e.g., M25)
Reinforcement grade	B450C (γS)	Same as EC8	Lower strength steels used historically
Safety factors	γC ≈ 1.5, γS ≈ 1.15	Slight national variations (γC = 1.5, γS = 1.15)	Not harmonised, values lower or implicit

Loads and Combinations

Aspect	EC8	NTC2018	KTP-89
Dead loads	Fully included	Fully included	Fully included
Live loads in seismic mass	ψ2 factor (0.3–0.5)	Similar to EC8	Not explicitly defined
Load combinations	ULS: 1.35G + 1.5Q; Seismic: G + ψQ + E	Very close to EC8	Simpler, fewer cases

Seismic Hazard Definition

Aspect	EC8	NTC2018	KTP-89
Hazard basis	Probabilistic seismic hazard analysis (PGA from maps)	National hazard maps (PGA, a_g,30)	Zonation by seismic intensity (MSK-64 scale)
PGA values	Site-specific, depends on hazard level (10%/50 yrs)	From official maps, return periods defined	Fixed by zone (0.08–0.42 g)

Response Spectrum Shape

Aspect	EC8	NTC2018	KTP-89
Soil categories	A–E	Same categories, with Tc* variations	Rock, stiff, soft (simplified)
Plateau shape	TB–TC plateau	TB–TC plateau with national Tc*	Tri-linear, simplified
Near- vs far-field	Considered	Considered	Not considered

Behaviour Factors (q)

Aspect	EC8	NTC2018	KTP-89
q definition	Depends on ductility class (DCM, DCH) and system	Similar to EC8 with Italian adjustments	General reduction factors, less refined
Typical values for DCM	Frames: 2-3, Dual: 3-4, Walls: 2-3	Nearly same	Often lower, less specific

17.2 Contributions of the Thesis – Original Value

The work contributes to understanding how different seismic codes influence design outcomes for the same structure. By applying all three spectra within the same model, the study highlights the practical consequences of code selection in terms of forces, displacements, reinforcement demand, and overall safety margins. It also provides a comparative framework that can be useful for engineers and policymakers in Albania as the country transitions from older codes to modern European standards.

Closing Statement

This work has shown how different seismic design codes — Eurocode 8, NTC2018, and the Albanian KTP-89 — can lead to very different outcomes when applied to the same structure. Beyond the numbers, the study reinforces a simple but essential idea: **the safety and resilience of our buildings depend not only on how we design them, but also on the standards we choose to follow.**

As Albania continues its path toward harmonization with European norms, the insights from this research underline the importance of adopting modern, site-specific approaches to seismic design. Ultimately, the goal is not just to satisfy code requirements, but to ensure that the built environment is prepared for the earthquakes that will inevitably come, protecting both structures and the people who depend on them.

Final Acknowledgement

Writing this thesis has been both a professional and personal journey. It represents not only the culmination of my master's studies, but also the effort, support, and patience of many people around me. I am grateful to my professors for their guidance, to my colleagues and friends for their encouragement, and to my family for their unwavering support throughout this demanding process. This work is as much theirs as it is mine, and I hope that the lessons learned here will contribute in some small way to safer and more resilient structures in the future.

18. REFERENCES

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