RC BUILDING TYPOLOGIES AND FAILURE MODES OBSERVED DURING THE 2019 ALBANIA EARTHQUAKE

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Approval sheet of the Thesis

This is to certify that we have read this thesis entitled **"RC Building Typologies and Failure Modes observed during the 2019 Albania Earthquake"** and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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ABSTRACT

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Introducing the thesis approach to the reader about the 2019 earthquake in Albania highlighted the vulnerability of many building typologies, particularly load-bearing masonry buildings and unreinforced brick, stone buildings, and RC frame buildings. The damage to masonry and historic buildings also emphasized the importance of preserving these structures and implementing proper seismic retrofitting measures. Is also important to note that the 2019 earthquake in Albania resulted in a significant loss of life and injury. Overall, the 2019 earthquake in Albania serves as a reminder of the importance of building resilient communities that are prepared to face the challenges of natural disasters. This paper consists of an information processing about building typologies, analysing why these buildings failed after the earthquake, where the main problem and what measures should be taken so that the same consequences do not repeat themselves. It is based on research works and information discovered on the ground in the Durres area which was the most affected. The methodology will continue with the comparison in (the ETABS model) of two RC Buildings with different periods of time (1970-1980, 1980-1990) to see the difference in earthquake resistance.

Keywords: *Building RC, earthquake, seismic safety, assessment methods, building typologies, ETABS model, seismic safety assessment, earthquake resistance*

ABSTRAKT

TIPOLOGJITE E NDERTESAVE BETONARME DHE MENYRAT E SHKATERRIMIT TE TYRE NGA TERMETI I VITIT 2019

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Master Shkencor, Departamenti i Inxhinierisë së Ndërtimit

Udhëheqësi: Prof. Dr. Hüseyin Bilgin

Prezantimi i temes tek lexuesit rreth termetit te vitit 2019 në Shqipëri vuri në pah cenueshmërinë e shumë tipologjive të ndërtesave në vend, veçanërisht të ndërtesave me muraturë që mbajnë ngarkesë dhe ndërtesave të papërforcuara me tulle, gurë dhe struktur betoni. Dëmtimi i muraturës dhe ndërtesave historike theksoi gjithashtu rëndësinë e ruajtjes së këtyre strukturave dhe zbatimin e masave të duhura të rikonstruksionit sizmik. Kjo tezë bazohet në punimet kërkimore dhe informacionin e zbuluar në terren në zonën e Durrësit e cila ishte më e prekura. Për arsye të tilla si mosha e ndëtresave , ndërhyrjet e bëra nga njerëzit dhe kodi i projektimit të kohës, këto lloj ndërtesash kane rrezikshmeri ndaj tërmetet është e me prioritet rishikimi dhe vleresimi i performances sizmike e ketyre ndërtesave dhe duhet te zhvillohen tekina duke pasur pasasysh kete vleresim per të forcuar këto ndërtesa në mënyrë që ato t'i rezistojnë dëmtimeve të mundshme nga tërmeti. Ai konsiston në një përpunim informacioni për tipologjitë e ndërtesave, duke analizuar pse këto ndërtesa dështuan pas tërmetit, ku është problemi kryesor dhe çfarë masash duhen marrë që të njëjtat pasoja të mos përsëriten. Në metodologji janë përzgjedhur dy ndërtesa Betonarme të cilat do të krahasohen në (modeli ETABS) dhe do të analizohet pse nuk i rezistuan tërmetit.

Fjalët kyçe: Ndërtesa Betonarme, tërmeti, siguria sizmike, metodat e vlerësimit, tipologjitë e ndërtesave, modeli Etabs, vlerësimi i sigurisë sizmike, rezistenca ndaj tërmetit, KTP-89; EC-8

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CHAPTER 1

INTRODUCTION

1.1 General information

The 2019 earthquake in Albania, a seismic event that registered a magnitude of 6.4, resulted in significant damage and loss, casting a spotlight on the crucial role of building resilience in areas prone to such natural disasters.

Particularly, this devastating event underscored the criticality of understanding the interplay between Reinforced Concrete (RC) building typologies and the ensuing failure modes that manifested during the earthquake. This thesis endeavours to explore this nuanced relationship further, examining the extent to which RC building typology influences failure modes in seismic events. To analyse these phenomena that are in the buildings affected by the earthquake are selected two buildings that were damaged during the earthquake in the city of Durres and were then demolished by the decision of the government and experts.

1.2 Methodology

The methodology - with the comparison with (Etabs model software) of residential stock of the existing building in different periods of time 1970- 1980 and 1980- 1990.

The type project 82/2 VIP Building versus the type project 88/2 Baron's Buildings - with the old design codes KTP-N2-89 [1] , in accordance with the new code design Eurocode 8 [2]

The aim of this study is to use the non-linear static approach (Pushover) to investigate how this type of structure responds to seismic loads in two scenarios involving six storey buildings from two different typologies.

This analysis we will estimate Deformed shape of the building in two direction X,Y with load bearing and the Response of Seismic spectrum Load case

These results will be compared with the actual building damages that were seen, and their primary causes are subsequently addressed.

The percentage of reinforcement required for the indicated section exceeds the guidelines of design standards (Eurocode-2 and KTP-2-89) by almost 30% although a reduction in the amount of reinforcement from the first floor to the ground level of 35.5%.

Furthermore, the use of smooth rebar and the weak concrete that was used in the construction of some of these buildings (the compressive strength that results in 50% less than permitted) contribute to these failures.

Some pivotal questions guiding this research are:

- How does the typology of RC buildings contribute to the observed failure modes during the 2019 Albania Earthquake?
- How do existing RC buildings in the region react to the earthquake?
- What can we understand about their behaviour by the response spectrum?
- What should we have in mind to consider a building safe during earthquake?

These questions emerge from the hypothesis that RC building typology is a determinant characteristic of building performance during seismic events.

1.3 The objective and structure

The primary objective of this thesis is to explore the reaction times to RC-framed structures that were damaged during the 2019 Durres earthquakes.

- In terms of the structure, this thesis is organized as follows:

- Chapter 1…. Introduction with the topic
- Chapter 2 …. Literature review
- Chapter 3 …. Analytical model of RC buildings with ETABS software
- Chapter 4 …. Analyses of the results
- Chapter 5 …. Conclusions

CHAPTER 2

LITERATURE REVIEW

2.1 Building Classification

- Single family rural
- Multi-story RC buildings [6]

  a) Pre-1990 large-panel prefabricated buildings (were known for their lack of upkeep and indicators of degradation).

  b) Pre-1990 masonry buildings (high buildings - was characterized by strong motions between wall, with good seismic performance due to the ability to work together UMR and CM buildings as a whole up a structure under seismic load

- Historical Buildings
- Modern Buildings

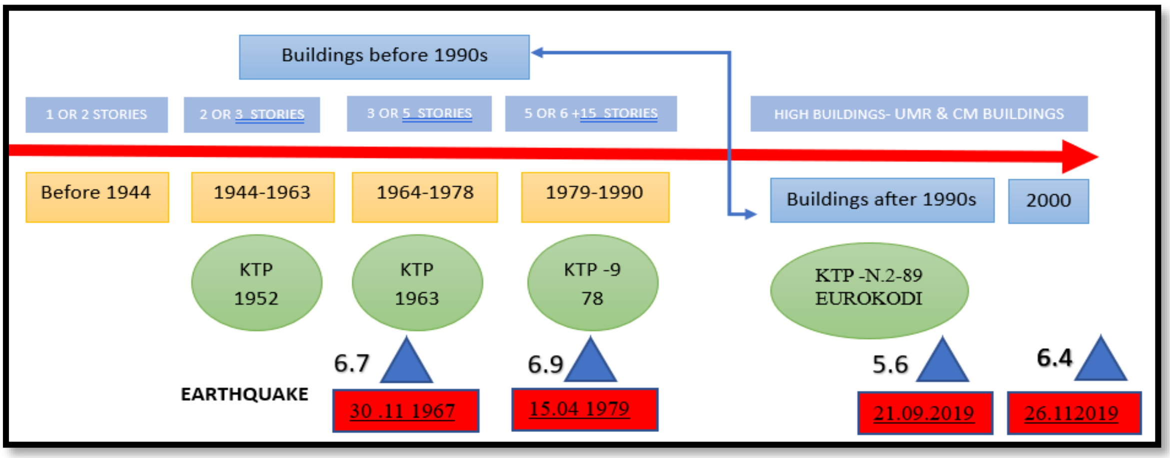


Figure 1. The horizontal diagram for the building classification before 1944 since 2000

2.2 Building Categories

2.2.1 Buildings constructed before the 1990s

a. Low rise buildings built before 1944, generally 1 or 2 floors high and based on traditional techniques This type of construction process uses walls made of stone or brick that are 38–50 cm thick.
b. Low-rise structures based on KTP 1952 that were typically 2 or 3 storeys tall (the walls of these masonry structures were supported by spread footing) were built between 1944 and 1963. Prefabricated clay and concrete slabs served as the basis for the floor and roof structures.
c. Low to midrise buildings, usually 2 to 3 or 5 floors high, were built between 1964 and 1978 based on KTP 1963 unreinforced masonry (the buildings were intended for seismic zone of intensity 7-8). Although the structure was made of load-bearing masonry, some RC construction materials, such as LINTELS, were used.)
d. Mid-rise buildings with RC columns and slabs, 5 or 6 floors high, were constructed on KTP -9 - 78 between 1979 and 1990. These novel foundation, slab, and RC frame typologies all make use of modern building materials. To improve the performance of structures against seismic stress, reinforced concrete technology initially developed shortly thereafter became the most utilized material.
e. The joints were improved, anti-seismic RC columns were inserted, and any-seismic belt was interconnected with anti-seismic RC columns at building corners and wall intersections. The first high -rise reinforced concrete building was designed in Tirana with 15 stories.
    - In 1979, prefabricated reinforced slabs were used to construct standard buildings that were suitable for earthquake zones with intensities between VII and VIII. In the 1980s, every building was prefabricated, apart from the foundation, which was constructed as a single monolithic block.
    - In 1982, the new constructions could be distinguished from the prefabricated ones through their slightly smaller spaces, with column spans from 3.4 and 4.2 meters.

- Substandard resources and inefficient methods of construction were used to create buildings of this quality.

2.2.2 Buildings constructed after the 1990s

KTP-N.2-89 and Eurocode were used to design mid- to high-rise building designs, which usually consisted of RE frames with masonry infills 6 to 12 stories high (RC construction was a popular choice during this new age in terms of structural typologies). When the Eurocodes went into effect in 2000, significant changes were made to the structural typologies of buildings that have been designated as RC structures. After 1991, political and economic upheavals had a significant impact on architecture and urban development; there was no governmental oversight and no building code, even in ALBANIA, where construction was thriving.

2.2.3 The main structural typologies based on Eurocodes 2000

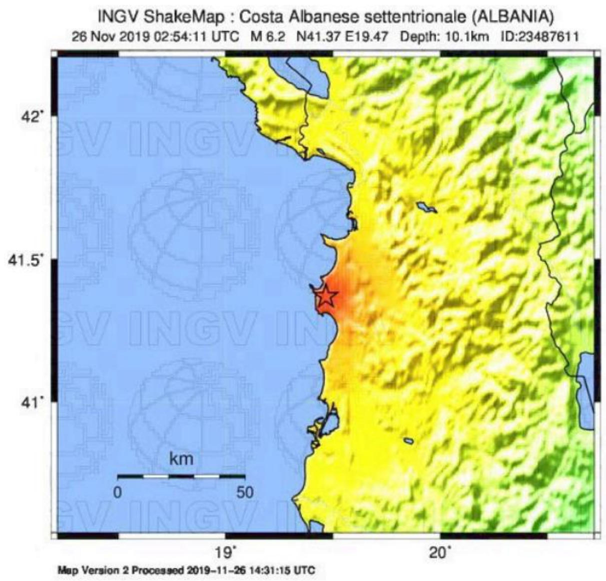
- Frame system -Beams and columns are joined together and form an RC skeleton. The reinforced concrete skeleton - Frame is a geometrically constant and statically stable unity of columns and beams. The effect of lateral and gravity loading is resisted by the spatial frame. The beams must be able to withstand the vertical forces from the slab. [7]
- Shear Wall System: This system consists of vertical RC walls that serve as shear walls and are strengthened on both the vertical and horizontal axes. It is suggested that you put these walls along the outside of buildings to prevent torsional moments.
- A dual system composed up of cores, beams, walls, and columns.

2.3 Reinforced concrete (RC) frames

In Albania, concrete reinforced (RC) frame buildings incorporating masonry infill are typical, particularly in urban settings. [5]

These structures are built using RC frames and masonry walls in between the columns. However, these structures are vulnerable to destruction during earthquakes, especially if the RC frame and masonry walls are not joined properly or if there are problems with the foundation.

Numerous of these structures were damaged by the 2019 earthquake, including foundation issues, soft story and weak story failures, and damage to staircases, walls, and stone buildings either in-plane and out-of-plane.



PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.06	0.2	0.8	2.0	4.8	12	29	70	>171
PEAK VEL.(cm/s)	<0.02	0.08	0.3	0.9	2.4	6.4	17	45	>120
INSTRUMENTAL INTENSITY	I	II–III	IV	V	VI	VII	VIII	IX	X+

Scale based upon Faenza and Michelini, 2010, 2011

Figure 2. The earthquake maps

Up until the collapse of the communist administration in 1990, residential and government buildings were constructed using masonry typologies. Due to the low cost at the time, they were developed and constructed, and after 1990, RC building typologies took their place. The building was built using low-strength masonry materials such burnt clay or silica bricks, blocks, or stone. Today, these buildings are still in use.

2.4 Classification of walls based on load bearing in Types of Masonry Wall

2.4.1 Load Bearing Masonry Wall:

One of the limitations of load-bearing masonry construction is that it is difficult to modify the layout of the building once the walls have been constructed. As a result, major changes or renovations to a load-bearing structure are often difficult and costly.

Load-bearing masonry construction is also not well-suited for regions that are prone to earthquakes, as it does not perform very well in seismic events. In addition, load-bearing construction is labour and material intensive, which can make it more expensive than other types of construction methods.

For these reasons, load-bearing masonry construction is less commonly used in modern building construction.

2.4.2 Non-Load Bearing Masonry Wall:

In modern building construction, most buildings are constructed using a combination of structural frames and non-load bearing walls. Structural frames are made of lightweight yet sturdy materials that can support the weight of the building, and non-load bearing walls are used to divide the interior space of the building.

This type of construction allows for more flexibility in terms of the interior layout of the building, as non-load bearing walls can be easily moved or removed without affecting the structural integrity of the building. This can be particularly useful for commercial or residential buildings where the interior space needs to be adapted to changing needs over time.

In addition, non-load bearing structures are typically less labour and material intensive than load-bearing structures, making them more cost-effective to build. They also perform very well in earthquakes because the structural frames can flex and absorb the energy from the seismic waves without compromising the integrity of the building. For these reasons, non-load bearing masonry is becoming increasingly popular in modern construction.

2.4.3 Solid Block Masonry:

Solid concrete blocks are best used for low rise construction where the load-carrying capacity of the blocks is necessary. They are often used for the construction of foundations, basement walls, retaining walls, and boundary walls. Their high compressive strength makes them suitable for areas with high wind or seismic activity. They are also ideal for projects where fire resistance is important, as they have excellent fire-resistant properties. However, due to their weight, solid concrete blocks may not be suitable for high-rise buildings.

2.4.4 AAC BLOCK – AUTOCLAVED

AAC blocks, also known as autoclaved aerated concrete blocks [4], have several pros and cons.

Pros:

- AAC blocks are light in weight, making them easier to handle and reducing the dead load on structures. This also helps optimize reinforcement consumption.
- Installation of services such as electrical and plumbing is easy due to the chiselling properties of the blocks.
- These blocks have very high dimensional accuracy.
- AAC blocks have superior thermal and acoustic properties compared to other blocks or brick masonry.
- They require less plaster per square foot compared to other blocks or brick masonry.

Cons:

- The compression strength of AAC blocks is lower than some other options, at 3-4 MPa.
- Improved quality control is required during production to ensure consistent quality, and they must be handled properly to prevent wear and tear.
- AAC blocks cannot be manufactured on-site, so they must be transported to the construction site.
- The unit cost of AAC blocks is higher compared to other types of blocks.

2.5 Factors affecting masonry constructions' vulnerability to earthquakes

Table 1. The factors that influence the seismic vulnerability of masonry buildings

Greater susceptibility	Lower susceptibility
• Materials of poor quality, poor mortar, and poor internal connections • Multi leaf masonry with no transverse connections • Irregular stones	• Regular and robust units • Interlock of units • Good bond • Masonry behaves monolithically though the whole thickness of the wall
• Out of plan instability	• Resistance of out of plane failures
Lack of efficient connections • among walls • between walls • horizontal walls Lack of structural redundancy	Wall intersections with good interlocking • Presence of ring beams and of tie rods in each floor and roof • Efficient floor to wall connection which reduce stress concentrations.
• Floors do not provide diaphragm actions	• Higher structural redundancy • Resistant out of plane vibration of the walls • Internal force redistribution from sufficiently stiff and resistant

	diaphragm
• Irregularly and widely spaced walls • Excessive unsupported floor spans	• Regularly spaced shear walls in at least two orthogonal directions • Limited floor spans
• Horizontal thrusts – equilibrated only by out of plane resistance of structural walls	• Horizontal thrusts are reacted by in plane action of strong walls - self equilibrating systems - by suitable structural elements - ties floor diaphragms
• High structural and non-structural masses and low material strength	• Masses and weight – in low ratio stress/strength
In plan • Torsional effects and stress concentrations for structural irregularity.	Regular structure -from Upper structure to foundation • Sufficient torsional resistance , • Regular path of forces

2.6 Unreinforced brick buildings and their damage

The damage of this structural building during the aftershock took for a few weeks until the major shock, registering Mw=6.4, with an epicentre about 15.6 km southwest of Mamurras, was recorded on November 26. [8]

This structure had showed a poor seismic response despite no causalities but was caused by flaws that result from bad seismic design, abnormalities brought on by unapproved interventions, inadequate reinforcing detailing, poor material quality, and subpar construction quality.

Figure 3. Unreinforced brick building with damage

The Openings placed too close to a building corner decreased the restraining effect of crossing walls. Openings for doors and windows weaken the bond among crossing walls. Due to the return walls' inadequate connectivity, the front facade and return walls often separate after failure.

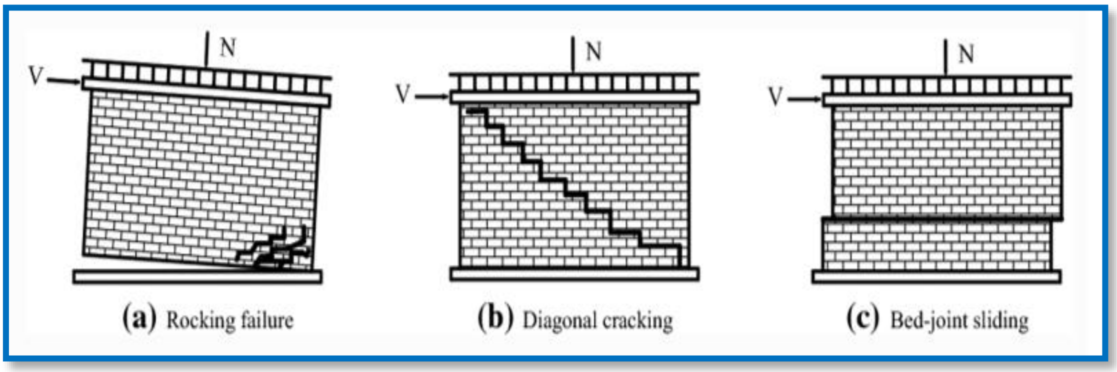


Figure 4. The wall failed out of plan due to (a) rocking failure (b) diagonal cracking (c) bed-joint sliding

Masonry units sometimes not properly overlapped to create an earthquake-resistant interconnection at the corner points of the constructions.

At the start of the earthquake, this causes walls to separate. Due to poor resistant conditions, the wall failed out of the plane.

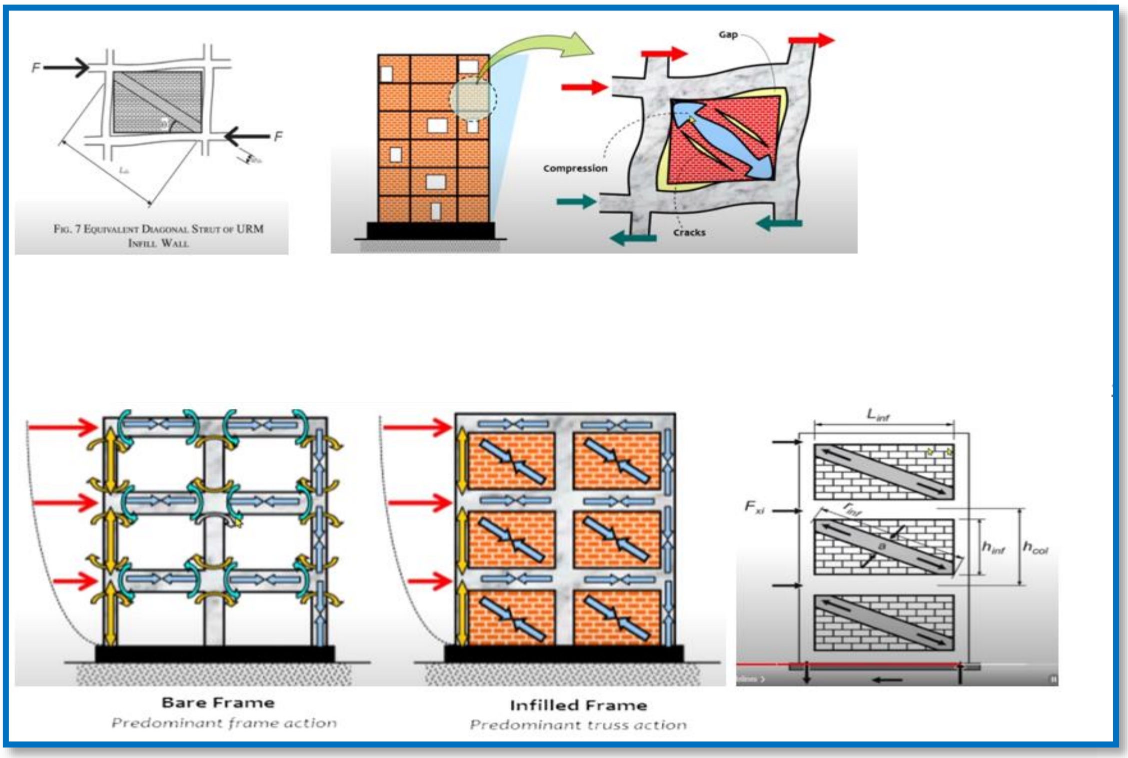


Figure 5. The behaviour of unreinforced brick masonry infill masonry under earthquake loading

2.7 Masonry religions and historic building (churches, mosques, bridges)

The 26th of November 2019 earthquake's seismic impact had involved several heritages building in Durres, Kruje, Preze.

a. Durrës Castle - Tower 'C' and fortification walls.
b. Gate walls at Porto Romano; Durrës
c. Hammam in the school yard 'September 16' Durres

We had observed several damage and partial collapse of the castles, in addition to in the components that follow, the susceptibility of their structural elements and the reasons for their pre-earthquake structural conditions.

Figure 6. Durrës Castle-Tower 'C' and fortification walls (a)before earthquake , (b) collapse wall , (c) the structure of steel reinforced

Clay brick walls adorned by multiple leaves and external leaf collapses are evident. The Durres Castle and some collapsed sections are still visible., the structure of steel reinforced.

- Gate walls at Porto Romano.
- Toilet (Hammam) in the school yard 'September 16'

Figure 7. Gate walls at Porto Romano; (a) the wall before earthquake , (b) the gate wall damaged (c) the wall under reconstruction

Figure 8. Heritage building of Hammam in Durres

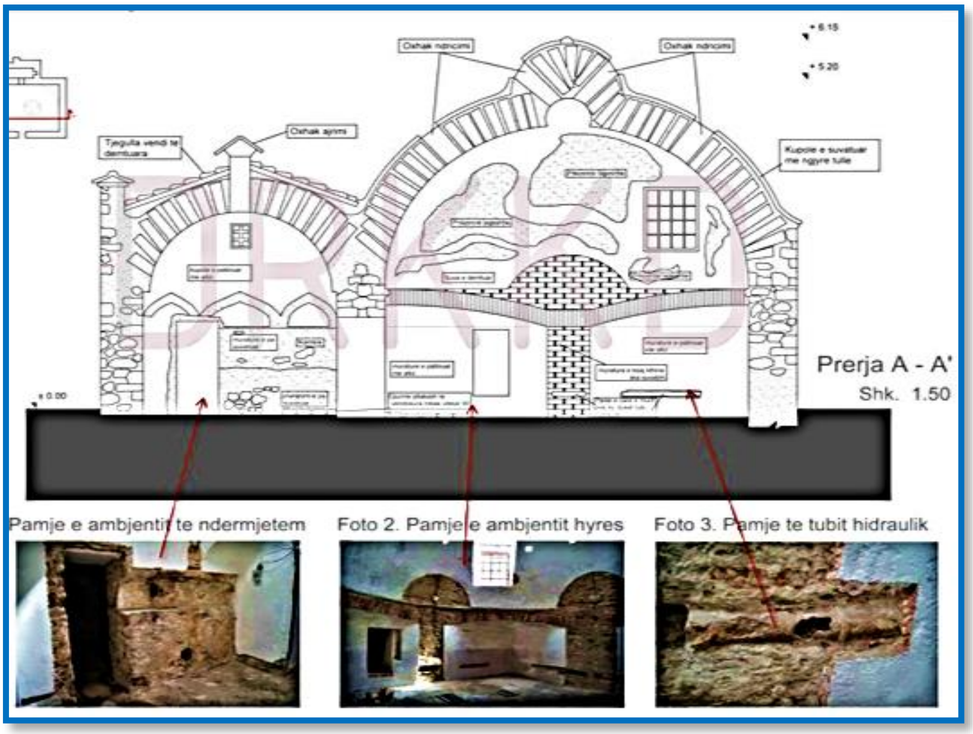


Figure 9. The damage of the inside masonry walls of the heritage building of Hammam in Durres

The conservation status of historical structures was altered by the presence of inadequate structural retrofitting regulations which - coupled with the presence of pre-existing damage - altered the reported failures and severe damage. The risk to their legacy is elevated by the significant earthquake vulnerability that has been observed. As a result, Albania's extensive built legacy and rich history must be maintained to safeguard social and historic values and further the nation's economic development.

The earthquake influenced both mid- and high-rise RC structures, generating structural and/or non-structural degradation and, in some cases, even collapse. The study also offers the findings of a statistical analysis of damaged RC buildings in the Durres municipality in in addition to observations pertaining to physical damage to RC structures. In the study, the topic is centred on damage patterns and failure mechanisms that are pertinent to the seismic response of RC structures. After the earthquakes, masonry infill walls were the most often observed damage pattern. These walls suffered damage and failure in some cases, and because of the interaction between the infill and the frame, they had a bearing on the performance of nearby RC columns. Although RC shear walls were missing from the damaged buildings of this type, they were predicted to be present in taller RC framed buildings (10 storeys and higher). In some instances, masonry infill walls were used in place of RC shear walls. Based on post-earthquake field observations and an accurate seismic assessment of two earthquake-damaged buildings, a case study has been presented that emphasizes the seismic behaviours of mid-rise and high-rise cast-in-place RC buildings in the November 2019 earthquake. Considering the observed performance of RC buildings, pertinent lessons and recommendations are offered in the last section..

2.8 The comparison between Eurocode and Albanian low for construction

During the periods of 1970-1980 and 1980-1990, the Eurocode standards were not applicable as they were still under development and not yet widely adopted. Instead, the construction rules and standards used in the Balkan countries during that time were influenced by the political and economic systems in place, particularly communism.

Under communist rule, the construction industry in the Balkan countries operated under centralized planning and state control. The state had a significant role in the design, construction, and approval processes of buildings and infrastructure projects. The focus was often on fulfilling the needs of the state and promoting ideological goals rather than following internationally recognized standards.

The construction practices during that period varied across the Balkan countries, but some common features can be identified. These include:

- Standardization: The construction industry operated based on standardized designs and construction techniques. These standardized approaches aimed to streamline the construction process and ensure efficiency in the use of resources.
- Socialist Realism: Architecture and urban planning often followed the principles of socialist realism, which emphasized monumentalism, grandeur, and a representation of socialist ideals. Large-scale public buildings and housing complexes were commonly constructed with an emphasis on functionalism rather than aesthetic diversity.
- Prefabrication: Prefabrication techniques were commonly employed in construction projects. Buildings were often constructed using precast concrete elements manufactured in factories and assembled on-site. This approach allowed for faster construction and mass production of standardized building components.
- State Control: The state had a dominant role in the construction industry, controlling the allocation of resources, labour, and materials. The planning and approval processes were typically centralized and governed by state authorities.

It is important to note that the specific construction practices and standards varied among the Balkan countries, as each country had its own unique political and economic circumstances during that period.

During the 2019 earthquake, the performance of buildings that implemented Eurocode (a set of European standards for the design of structures) varied depending on several factors. It is crucial to remember that the specific performance of buildings during an earthquake depends on a number of factors, including the earthquake's size and intensity, the design and construction practices implemented, the soil conditions, and other local factors.

Eurocode is a set of design standards that aims to ensure the structural integrity and safety of buildings. It provides guidelines for designing structures to withstand various loads, including seismic forces. Eurocode incorporates advanced engineering principles and knowledge gained from past earthquakes to improve the seismic resilience of buildings.

Some reasons why buildings designed using Eurocode may have performed better during the 2019 earthquake are:

- Seismic Design Considerations: Eurocode includes provisions for seismic design, considering the specific characteristics of seismic activity in different regions. It incorporates factors such as ground motion, soil conditions, and expected seismic forces into the design process. By considering these aspects, buildings designed according to Eurocode are better equipped to withstand seismic forces.
- Enhanced Structural Analysis: Eurocode emphasizes the use of advanced structural analysis techniques to assess the behaviour of buildings during earthquakes. These techniques allow engineers to model and predict the response of the structure under seismic loads more accurately. By conducting a comprehensive structural analysis, potential weaknesses can be identified and addressed in the design phase, leading to more robust buildings.
- Ductility and Redundancy: Eurocode encourages the use of ductile materials and the incorporation of redundancy in the structural design. Ductility refers to a structure's ability to undergo significant deformation without losing its overall strength and stability. Redundancy means that buildings have multiple load paths and backup systems, ensuring that the structure remains intact even if one component fails. These design principles enhance the resilience of buildings during seismic events.
- Quality Control and Construction Standards: Eurocode also emphasizes quality control and construction practices. It provides guidelines for the proper implementation of design specifications, including material selection, construction techniques, and inspection procedures. Adhering to these standards ensures that buildings are constructed to the required specifications, improving their overall performance during earthquakes.

While buildings designed according to Eurocode may exhibit improved seismic performance, you should note that how structures fare during an earthquake can still vary depending on the specific design, construction quality, and other site-specific factors. Local regulations and guidelines may also influence seismic design and performance requirements.

Prominent researchers have conducted studies and investigations on seismicity in the Balkan areas, including Albania, to understand the characteristics of earthquakes, assess hazard levels, and improve seismic risk mitigation. Some key topics of interest in these studies include:

- Seismic Hazard Assessment: Researchers analyse historical earthquake records and geological data to assess the seismic hazard in the region. This involves determining the frequency, magnitude, and location of past earthquakes and using this information to estimate the likelihood and intensity of
- Future seismic events: Hazard maps are created to identify areas with higher seismic risk.

2.9 Response of RC structures to the November 26, 2019, Mw 6.4 earthquake in Albania:

Prefabricated RC technology was widely employed in Albania between 1960 and 1990 to create homes and public structures, but since communism fell in the early 1990s, cast in-situ RC construction has taken its place. Besides the change in construction technology (prefabricated vs cast in-situ), there was also a change in structural system. Prefabricated RC buildings were mostly in the form of large panel wall structures, while cast in-situ buildings have moment frame structural system in which frame members resist the effects of both gravity and seismic loading. Masonry infills in these buildings are used to enclose interior spaces and act as façade elements at the exterior. In case of taller buildings structural RC walls are provided to resist seismic effects jointly with a RC frame (dual system Around 40% of the country's total building stock was constructed after 1990, and all structures, from low-rise single-family homes to multi-story residential and office buildings, were created using cast in-situ RC technology. According to an examination of the buildings affected by the earthquake that occurred on November 26, 2019, it was discovered that all collapsed and/or seriously damaged RC structures were constructed after 1990 and used an RC frame structural system. Field applications of this construction system started on a larger scale after 1989, when the Albanian seismic design code KTP-N.2-89 was released (Science Academy, 1989). Floor system is usually in the form of RC waffle slabs in which hollow clay blocks or lightweight Styrofoam are used as filling, thereby creating a system of shallow RC beams in the interior and in some cases deeper edge beams at the exterior. These shallow beams have a width that is higher than or equal to the cross-sectional dimension of a column and a depth that is equivalent to the overall slab thickness (30 cm thickness was specified by KTP-N.2-89).

Even in the case of taller buildings with ten storeys or more, these buildings were typically constructed without RC structural walls or with a small number of walls, so their seismic resistance depends on a 3-D moment frame system, which may be significantly flexible in the case of taller buildings. Most RC buildings in Durres were unaffected by the September 2019 earthquake (Mw 5.6), despite a number of design and construction flaws (such as the absence of RC structural walls, strong columns, and weak beams). However, some of these buildings sustained significant damage during the more intense November 2019 earthquake (Mw 6.4). It is believed that the main cause of damage in RC buildings in the November 2019 earthquake was high seismic demand.

Based to the microzoning map, the Durres geographical area has a seismic intensity of IX, therefore the seismicity coefficient is 0.42g, which corresponds to land category III. The following figure displays elastic spectra in accordance with EC-8 [10] and KTP-2-89 (Albanian Technical Design Code, published in 1989), as well as actual spectra in accordance with the two main directions of E-W and N-S recorded by Institute of Geosciences, Energy, Water, and Environment (IGJEUM) at the Durr's station.

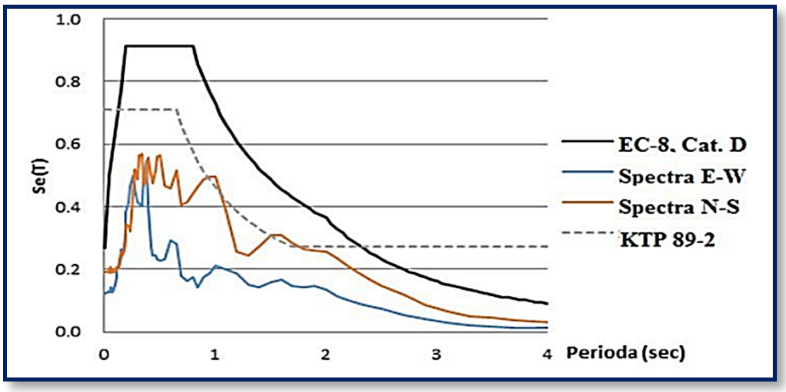


Figure 10. Peak ground acceleration map in (g%)

KTP-2 (Cat.III, ao = 0.42g), EC-8 (Cat.D, PGA 0.27g), and oscillation spectra collected for both directions at Durr's station. [9]

For example, peak ground acceleration (PGA) recorded in Durrës was 0.2 g, while spectral accelerations (0.5 g) were about equal to the elastic spectral accelerations outlined in KTP-N.2-89 for buildings with period ranges that fall within the range of mid- to high-rise RC frame systems. The basic periods of taller RC structures were predicted to be in the range of 1.0 sec. Case studies that show how RC buildings responded to the earthquake in November 2019 have been examined elsewhere. [10]

Masonry infills significantly affected how well Albanian RC frame constructions performed during the earthquake because of their flexibility. Buildings in Albania built after 1990 are frequently built using modular blocks with holes that are horizontally aligned, which lowers the compressive strength of the masonry.. Also, due to mixed use of some buildings, there is frequently an "open" ground floor and first floor which are used for commercial purposes, while masonry infills exist only at higher floor levels which are usually intended for residential purposes.

Masonry infills have frequently been constructed in an arbitrary manner, which could have torsional consequences that increase seismic demand. It is important to emphasize that the impact of masonry infills on the seismic performance of these structures depends on the relative ratio of the lateral stiffnesses of the RC frame structure and the masonry infills, as well as the capacity of the infills under compression and shear. As a result, RC buildings of various heights showed various damage patterns. Masonry infills were found to significantly alter the behaviour of structures in low- and mid-rise buildings, resulting in considerable overall damage or collapse. Most RC structures that were harmed by the earthquake were constructed between 1990 and 2010.

More recently built, taller RC frame buildings (10 storeys and up) had a stronger structural capability of the frame system, but these buildings also experienced significant damage to the masonry infill due to large interstorey drifts.

In the earthquake-affected region, irregularities in RC buildings were mostly caused by the interaction of flexible RC frames with hard masonry infills or stairs. The most frequent anomalies were "soft storey", "short column", and an uneven stiffness distribution in plan that had adverse torsional consequences. Examples of buildings that collapsed because of the "soft storey".

In the present case, buildings with open spaces on the ground floor (garages, stores, restaurants, etc.) and masonry infills on the upper floors that were used for residential purposes were what gave the illusion of a "soft storey" (vertical irregularities).

The seismic demand in the RC columns at the ground floor level was significantly raised because of the masonry infills and the increased stiffness at the upper floors, which led to damage and collapse of these structures.

Figure 11. Collapsed buildings in Durrës, Albania due to the "soft storey" irregularity (McKenney, 2019)

Those two figures illustrate significant damage of an RC frame structure due to torsional effects caused by masonry infills and a staircase, which were likely neglected in seismic design. The building has an open ground floor and additional five floors (Figure 11-a).

The building has a regular column grid; however, it was noticed that columns in the exterior frames in longitudinal direction were aligned such that the column's smaller cross-sectional dimension is aligned in longitudinal direction, which results in lower stiffness of these exterior frames compared to interior frames with a different column alignment.

An eccentric position of the staircase, with regards to the building's centre of mass, caused eccentricity of the centre of stiffness in relation to the centre of mass and the corresponding torsional effects.

High torsional demand contributed to excessive damage of an exterior masonry infill adjacent to the staircase, as seen in Figure 11-b. Seismic vulnerability of the staircase was compounded by the "short column" effect at the abutment of the staircase.

Figure 12. Damage to RC building in Durrës, Albania due to torsional effects: a) a view of the building after the earthquake; b) damaged masonry infill at the ground floor level (adjacent to the staircase) and c) damaged staircase area

Figure 13. Damage of RC buildings due to the "short column" effect in the November 2019 Albania earthquake: a) impact of the stair support on supporting columns and b) impact of partial masonry infill due to the openings

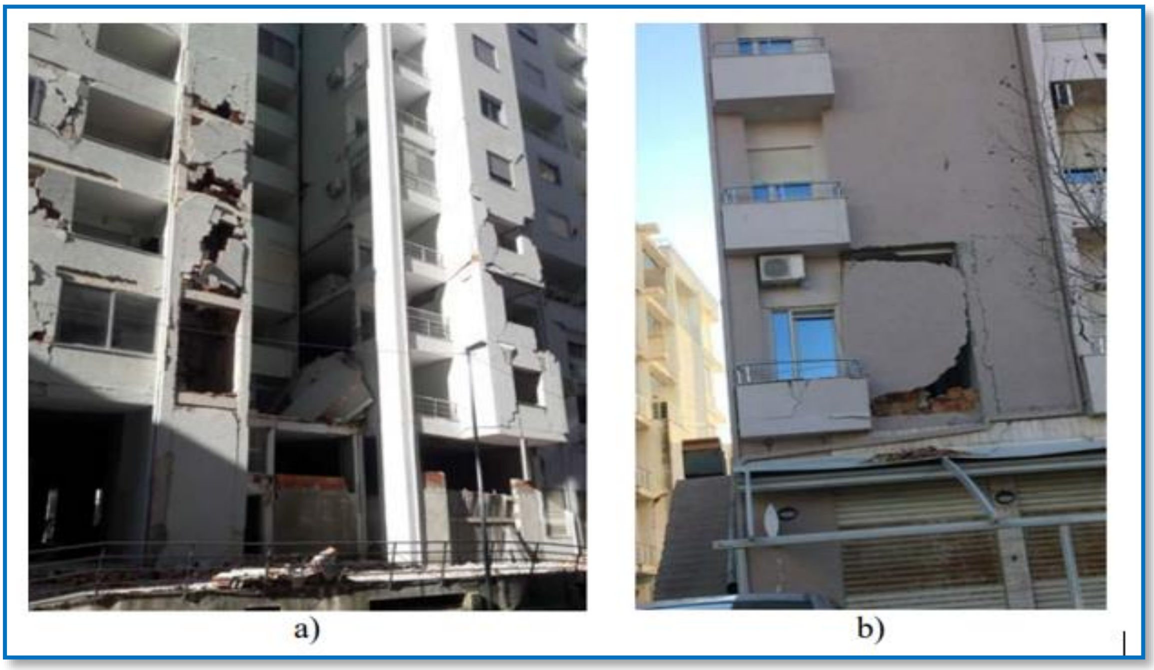


Figure 14. Severe damage and failure of masonry infills in RC frame buildings in Durrës, Albania due to the November 2019 earthquake

2.10 Performance of RC buildings during the November earthquake

Performance of RC buildings during the November earthquake (Mw 6.4) Masonry buildings account for approximately 25% of the urban building stock in Croatia, while the remaining 75% are RC structures (moment frames or shear wall systems). [11]

However, 76% of dwellings in rural areas are masonry buildings, while only 12% are RC buildings (Hadzima-Nyarko et al., 2020). Since the epicentre of the December 2020 Croatia earthquake was close to Petrinja, a small town (population of approximately 20,000) surrounded by villages, majority of buildings affected by the earthquake were loadbearing masonry structures. However, a few RC structures were also damaged during the earthquake, and relevant observations related to those buildings are presented next.

The Faculty of Education in Petrinja was built in 1962, and it was constructed before the first seismic design code in former Yugoslavia was issued in 1964 (PTP, 1964), hence the building was not designed and detailed for seismic effects. The building has a RC moment frame system with masonry infill walls. The building has a T-shaped plan and consists of two rectangular-shaped wings. There is a single-storey wing and an educational wing which is 3-storey high. The building did not experience visible damage at the exterior, however widespread damage occurred in the interior infill walls. Interior damage could be attributed to torsional effects and/or irregularity in elevation due to different height of building wings. Several walls experienced minor damage characterized by the spalling of plaster, which caused damage to building contents. It is expected that the RC frame structure was designed only for gravity loads and was excessively flexible for the effect of in-plane seismic loads, hence diagonal cracks developed in masonry infills due to excessive lateral drifts. In some cases, cracks were observed at the interface between the masonry infill walls and adjacent RC frame.

The Centre is a RC frame structure with masonry infill walls with RC frames constructed only in one direction – there is no lateral load-resisting system in the other direction. Note that the frames were constructed at variable spacing, thereby causing an eccentricity between the centre of stiffness and centre of mass. The absence of lateral load-resisting system in one direction, combined with eccentric frame layout in other direction, caused significant torsional effects and heavy damage of RC columns. Damage of RC columns can also be attributed to the setbacks in the columns along the height (an architectural feature), which was unfortunately not addressed by adequate stirrup spacing. Interior columns were robust (cross-sectional dimensions 55x30 cm) for a two-storey building; however, the reinforcement detailing was inadequate, with stirrup spacing at 200 mm. [12]

Frame-infill interaction in the exterior frames caused “short column” effect, and shear cracks in the columns. Majority of the columns experienced either severe damage or failure, while upper floor moved downwards by approximately 2 cm, showing a loss of capacity for resisting gravity loads. Beside the damage of structural elements, majority of masonry infill walls completely failed. These infills either experienced cracking or separation from the frame (along the infill-frame interface), and subsequently tilted in out-of-plane. Additionally, the brick masonry façade and the glazing at the entrance doors were also damage.

CHAPTER 3

ANALYTICAL MODEL OF RC BUILDINGS WITH ETABS

3.1 Object description and analysis (VIP Building)

VIP building number 14 on "Dyrrah" boulevard, which was damaged during the earthquake of November 26, 2019, was evaluated based on the detailed inspection of the building site, considering modern building codes on seismicity such as EC-6 and EC-8.

From the observations and inspections made in this building, it was observed that the building suffered major damage on the upper floor and moderate damage on the ground floors. Also, the area in which the building is located is one of the areas with high seismicity in our country according to the seismic risk map of Albania.

This structure with reinforced concrete bearing construction has suffered high and moderate damages for the poor quality of the structure materials and unqualified workers, the lack of the building code of the time if they have been applied, and the lack of inadequate repair after the events' previous seismic damage.

The biggest structural damages are in the eastern part of the building, where the concrete is broken and some of these iron columns have lost their stability and solidity. This term is significantly smaller than the design term according to Eurocode 8. considering the status of the structure, the material quality of the construction, and the bearing capacity of the structural support system, as well as the building codes of the time, make the structural reinforcement of building in its damaged parts should not be possible, according to the expertise done.

The examined building has been identified and its current structural capacity has been analysed. This building suffered not only heavy structural damage but also high damage to non-structural elements.

The photographs were taken in the field to describe the current condition of the building. Current performance has been assessed based on on-site inspection and observation of general building damage, in accordance with modern seismic codes.

Figure 15. Vip RC frame buildings in Durrës

Figure 16. Vip building façade in Durrës

Figure 17. Vip building – CT demolition

Figure 18. Vip building collapse

According to the technical report the project is designed to present horizontal terrains at a depth of 1.3m from natural ground.

A ground base resistance of 2kg/cm2 is anticipated. In terrain with a loading charge, the piles should be inserted 50cm below the maximum load level. The foundations should be applied with a concrete grade not less than 200kg/cm2 for water-resistant soil.

For the construction of the foundations, use concrete grade M-15. For soil with water retention, use concrete grade M-25. For soil with the presence of water, use a cement mix ratio of 1:2. In the case of perennial groundwater levels, it is recommended that the initial settlement of the foundation, as well as part of its trunk, be made with lightweight concrete M-100.

This height of the foundation construction with lightweight concrete is determined by the Projection office of the Government Department. - based on hydro-geological studies. The dimensions of the foundations specified in the project are suitable for a height of up to 1.5m.

For the height of the floor over 1.5m. from natural soil and for foundation insertion depth greater than forecast and for [σ] different from 2kg/cm2 - the static foundation calculations should be re-evaluated by Z.U.P. (the projection office of Government Department.)

For sloped terrain, the foundations should be applied with stairs with a slope of no greater than 50cm. respecting the minimum ratio #2 as well as the first stage (the initial settlement) should be at least from the corner of the foundation's edge - minimum 1m.

When the height of the foundation together with the plinth is 1.2m, every 0.9m to 1.2m, concrete rings of M-100 with a thickness of 10cm should be placed. In seismic zones 7-8, in strong soil with an allowable soil resistance of not less than 2 kg/cm^2, the M-150 concrete ring with a height of 15cm should be used.

For other types of soil, the M-150 reinforced concrete ring should be used, with a longitudinal reinforcement area of no less than $4.5cm^2$ (6 θ 10) for a width of the plinth up to 60cm, and $6.8cm^2$ (6 θ 12) for a width of the plinth greater than 60cm. The dimensions of the reinforcement are taken from the construction brochure and specifications from 1976.

Corners and places where load-bearing external and internal walls are interrupted with a thickness of 25cm should be reinforced according to the drawing on the respective sheet.

Figure 19. Damages made in the building by the earthquake

In the plan of the foundation rings, the cushions made of M-150 concrete are shown, from which the columns' rebars emerge. The columns K-1 are constructed with M-150 concrete.

These columns are connected to the masonry through the recesses in the masonry, which are cleaned of debris and moisture before pouring the concrete. Iron bars (refer to the sketch) are placed for the connection.

The floor is waterproofed with two layers of bitumen + asphalt with a softening point of 70°C, on top of the clean concrete layer.

The masonry for seismicity level 7 is applied as follows:

- With red bricks, M-75 --- mortar M-25.
- With silicate bricks, M-75 ---- mortar M-50

The support of partition walls on the intermediate floor is specified in A-3 12 of the brochure, regarding the types of elements for roofing with tiles and construction mortar, from 1976.

Architraves above doors and windows are constructed using precast reinforced concrete beams, whether load-bearing or non-load bearing, according to the brochures "Load-bearing Precast Architraves" from 1974 and "Non-load-bearing Architraves" from 1977, but they must be supported by a minimum of 30cm in the masonry (without grout).

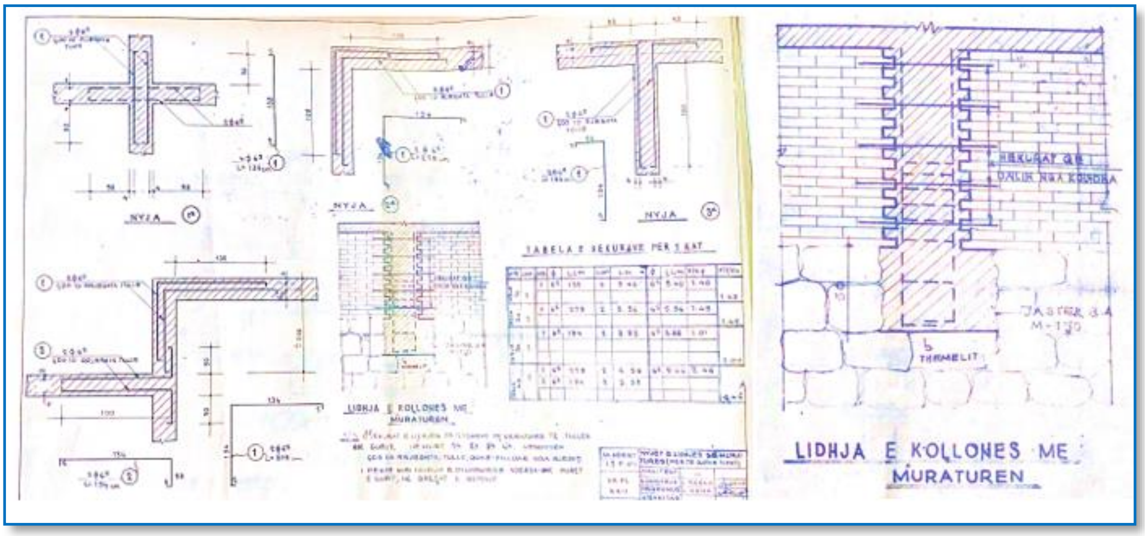


Figure 20. Construction project details taken by Technical Institution

Non-load-bearing precast architraves with M-200 concrete are also used on the spaces between walls, measuring 12cm.

The structures of the intermediate floor coverings are designed according to Album A 79. Prefabricated reinforced concrete floor slabs, approved by the technical-scientific council of the Ministry of Construction under decision no. 19.4.26-6-1979, should be used. Details can be obtained from the same brochure. Anti-seismic measures are taken from the brochure on types of elements for roofing with tiles and construction mortar from 1976 but using $4\theta12$ (as per the norms in seismic areas).

Ramps of stairs and square platforms are designed for the type with two ramps with panels, following the intermediate floor slabs, while for the type with a single ramp with panels, the details are as follows.

For the roofing of the staircase cage, a parapet is left for the exit to the terrace (refer to the corresponding drawings), and the holes thickness is taken from the brochure on types of elements for roofing with tiles and construction mortar, series no. 12.1.76.

The floor layers are taken from the brochure on types of elements for roofing with tiles and construction mortar from 1976 as follows:

Slab floor for the intermediate floor in brochure A-1/2.

Slab floor for the other floors in brochure A-1/7.

Parquet floor in brochure A 1-2/19. The calculation of the reinforced concrete structures is done using this theory. The thickness of the mortar layer is estimated to be 2cm.

The parapets of the terraces in seismicity level 7 should be constructed using solid brick walls measuring 25cm in height up to 1.2m.

Table 2. The table of the structural analyses for both type of buildings

VIP BUILDINGS	BARON'S BUILDINGS
The pile foundations H = 1.6m dimension 200 x200 cm and 250 x 250 cm	The pile foundations H = 3.20m dimension 100 x100 cm / 160 x 80 cm
Continuous Beam Foundation are only at the outside of the piles with dimension / 40 x 40 cm / 30 x 40 CM The other Piles are non-connected with the other structure at the foundation	Continuous Beam Foundation had connected all the piles of the foundation with the dimension 80c80 cm/ 70x80 cm / 40x40cm
The columns are symmetric with dimension 30 x 40 cm (in total 21 columns)	The columns are with dimension 51x38cm, 38x38, 38x25 cm (in total 14 columns)
Structure plan for the typical floor • LONGITUDINAL BEAMS ACCORDING TO THE AXIS and concrete BEAMS AT console • TRANVERSE BEAMS ACCORDING TO THE AXIS & concrete BEAMS AT console	Structure plan for the typical floor • TRANVERSE BEAMS • LONGITUDINAL BEAMS ACCORDING TO THE AXIS

For seismicity level 8, the parapets should be constructed using concrete/steel columns connected with anti-seismic reinforcement (refer to the respective sketch).

3.2 Object Etabs Modelling (VIP BUILDING)

- **Step 1 Begin a New Model**

In this Step, the basic grid that will serve as a template for developing the model will bedefined.

A. Click the **File menu** > **New Model** command or the **New Model** button. Theform shown in Fig. 21 will display. Verify that the default units are set to *KN, m, C.*

B. The New Model formallows for the quick generation of numerousmodel types using parametric generation techniques. However, in this tutorial the model will be started using only the grid generation. When laying out the grid, it is important that the geometry defined accurately represents the major geometrical aspects of the model, so it is advisable to spend time carefully planning the number and spacing of the grid lines. Select the **Grid Only** button, and the form shown in Fig. 21 will display.

C. The grids and spacing in the X, Y, and Z axes are specified using the Quick Grid Lines form (Fig. 21). Eight grid lines should be used in the X and Y directions, and seven grid lines should be used in the Z direction. In the X, Y, and Z directions spacing edit boxes, enter 8, 6, and 3.1, respectively. For the sake of this tutorial, make sure that all the values in the First Grid Line Location area are set to zero. This area's values identify where the grid lines' origin is. To proceed, press the OK button

D. The grids and spacing in the X, Y, and Z axes are specified using the Quick Grid

Lines form (Fig. 21). Eight grid lines should be used in the X and Y directions, and seven grid lines should be used in the Z direction. In the X, Y, and Z directions spacing edit boxes, enter 8, 6, and 3.1, respectively. For the sake of this tutorial, make sure that all the values in the First Grid Line Location area are set to zero. This area's values identify where the grid lines' origin is. To proceed, press the OK button.

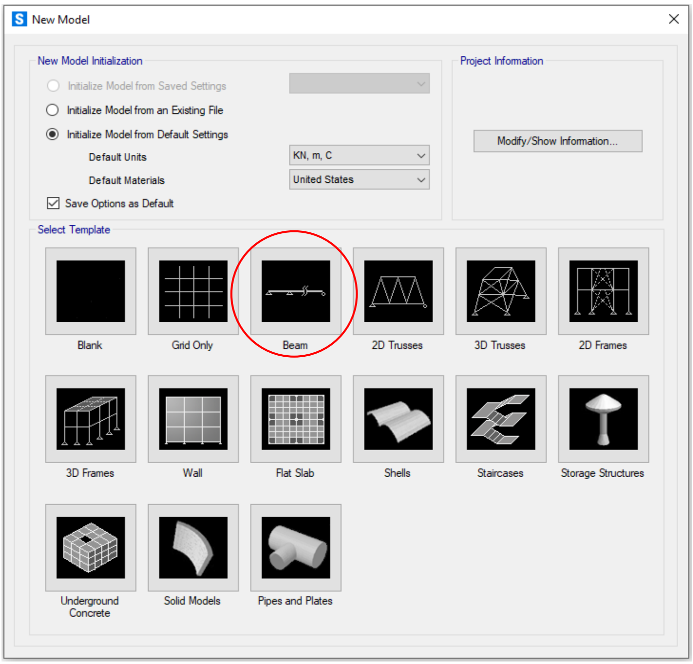


Figure 21. ETABS New Model form

1. Select Coordinate **Systems/Grids** from the Define menu. It will show the Coordinate/Grid Systems form.
2. Select Global on the Coordinate/Grid Systems form by clicking the Modify/Show System button. It will then show the Define Grid Data form (Fig. 22).
3. The uneven spacing in the X, Y, and Z directions is specified using the Define grid data type. Set the spacing of the display grid.
4. For Grid ID 2 in Y Grid Data, set the spacing to two, and for Grid ID 1 in Z Grid Data, put it to 1.2.
5. To close the Define Grid Data form, click the OK button.

E. Fig. 22 will show up once you click the OK button to close the Coordinate/Grid Systems form. As seen in Fig. 22, the grids are displayed in two vertically tiled view windows: an X-Y "Plan" View on the left and a 3-D View on the right. By choosing the Options menu > Windows command, the quantity of view windows can be altered.

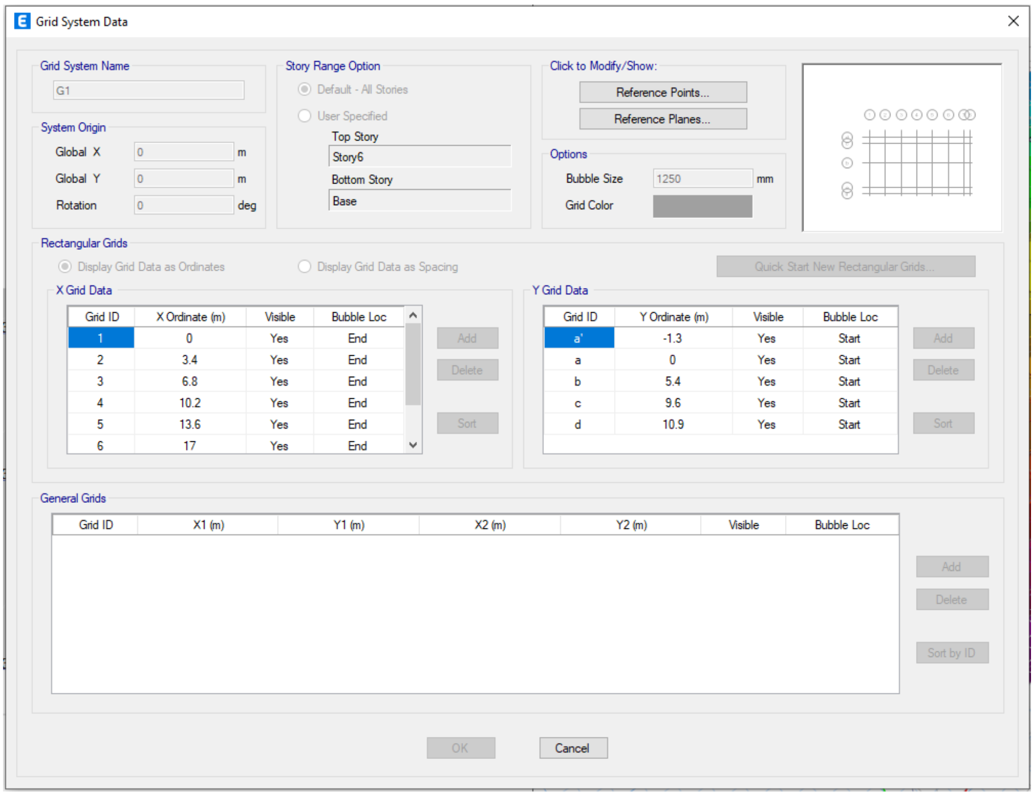


Figure 22. ETABS Define Grid Data form

In Fig. 22, you will see that the "Plan" view is open. The title bar of the display is highlighted when the window is open. By clicking anywhere in the view window, you can make a view active.

Keep in mind that Z positive is pointing "up" and that the Global Axes are also visible. When ETABS speaks about gravity's direction, it means "down" or in the negative Z direction.

- **Step 2 Define Material**

To create, change, or remove a material property definition, use the Define menu > Materials command. The structural objects (frame sections, cable sections, tendon sections, area sections, solid properties) are then defined using the definitions of the material properties.

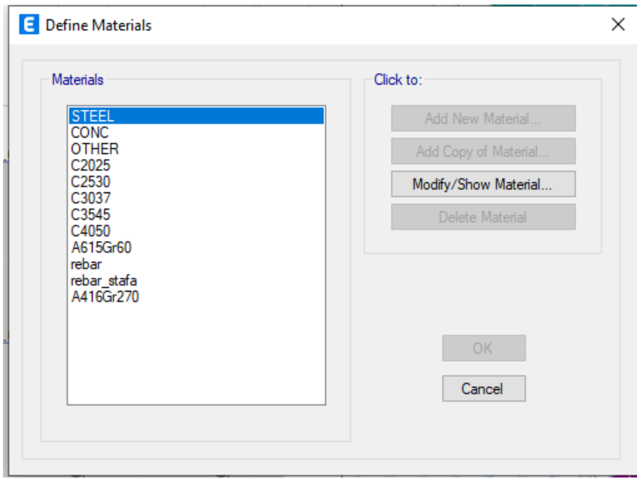


Figure 23. ETABS Define Materials form

The Define Material form (Fig. 23) will appear when you select the Materials command from the Define menu.

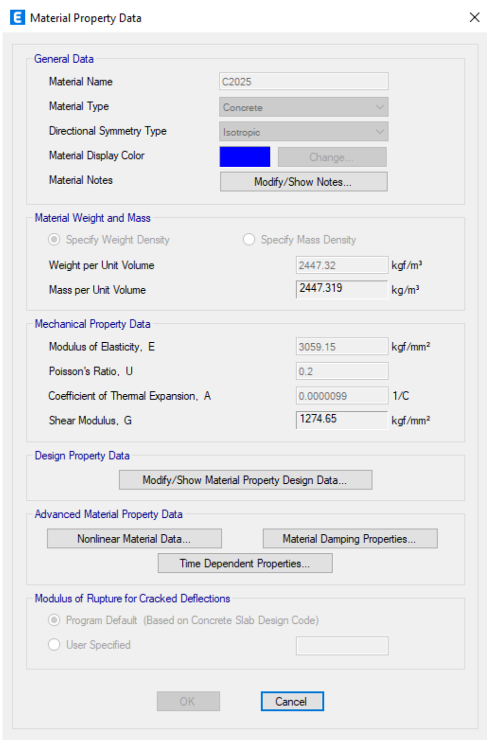


Figure 25. ETABS Material Property Data form -Concrete

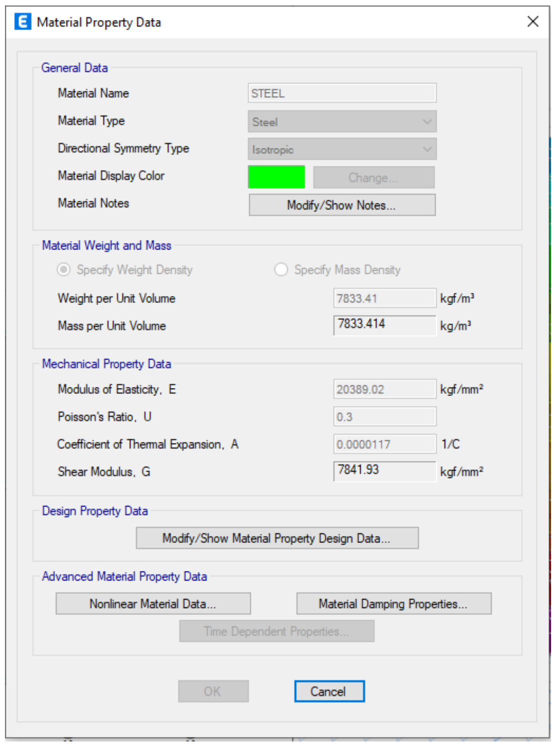


Figure 24. ETABS Material Property Data form -Rebar

A 4000Psi should be highlighted in the Materials display list. Next, click the Modify/Show Material option to bring up the form in Fig. 24.

A Type C20 in the Material Name and Display Colour edit boxes and choose concrete from the drop-down list under Material Type.

A. Select 25 as the Weight per Unit Volume. Modulus of Elasticity at Set

B. Set Weight per unit Volume as 25. Set Modulus of Elasticity to 4351.1324

C. Set Poisson's Ratio to 0.2. Set Specified Concrete compression strength at 29000, then push the OK button.

D. In the Materials display list, select an A992fy50. The form shown in Fig. 23 will appear once you click the Modify/Show Material button.

E. Type "Rebar" in the "Material Name" and "Display Colour" edit boxes and choose "Rebar" from the "Material Type" drop-down list.

F. Set 72.5189, 106.7783, 72.5189, and 108.7783 for the minimum yield stress, fy, minimum tensile stress, fu, expected yield stress, fye, and expected tensile stress, fue. To exit all forms, click the buttons on the Material Property Data form and the Add materials form.

- **Step 3 Define Frame Section**

When a frame section is defined, it can be assigned to frame objects.

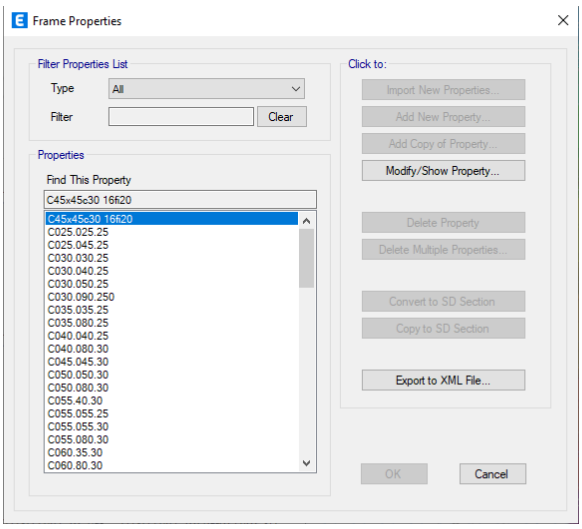


Figure 26. ETABS - Frame properties table

You can define frame section properties based on a section's dimensions using the Define menu > Frame Sections command,

- import sections from preconfigured databases, review and amend section properties,
- and delete section properties.

- **Step 4 Establish Load Patterns**

Dead, Dead Wall, Dead Slab, Dead FF (Floor Finish), Dead RT (Roof Treatment), Live, and Live Roof loads operating in the direction of gravity are the loads used in this problem.

To open the Define Load Patterns form (Fig. 27), select the Define menu > Load Patterns command. It should be noted that only one default load case—a Dead Load scenario with self-weight [DEAD]—has been defined.

Keep in mind that the default case's self-weight multiplier is set to 1. This means that the self-weight of all members will be automatically included in this load pattern at 1.0 times. Both load patterns and load cases, which may vary, are present in ETABS. When a load pattern is defined, the program still automatically generates a corresponding load case, and the load cases are accessible for inspection when the analysis is run.

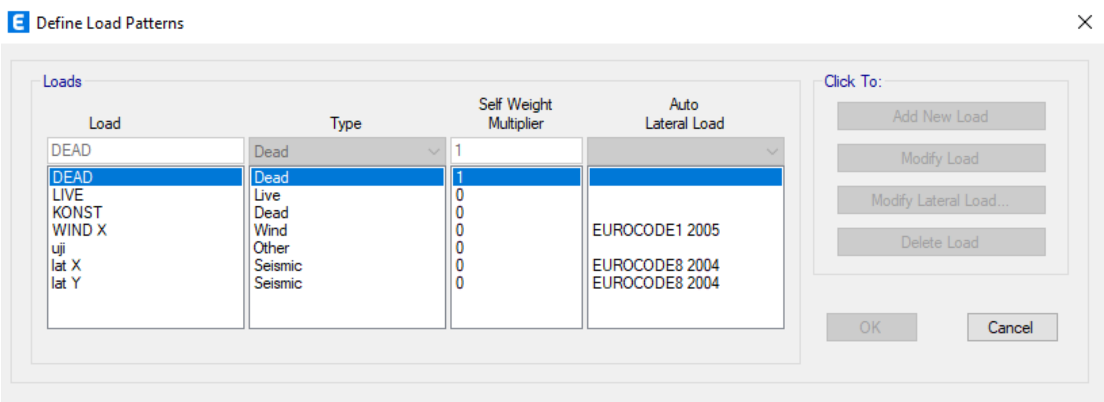


Figure 27. ETABS Define load Patterns form

Click on the Load Pattern Name column edit box. Enter the new load pattern's name, DEAD Wall. Choose a load type from the drop-down menu; in this instance, choose SUPER DEAD. The Self Weight multiplier should be set to 0. To add the Dead Wall load to the list of loads, click the Add New Load Pattern option.

- Add Dead Slab, Dead FF, and Dead RT load cases by repeating item B.
- Enter the new load pattern’s name, LIVE. Choose the LIVE load type from the drop-down menu. The Self Weight Multiplier should be set to 0. To add the Live load to the load list, click the Add New Load Pattern option.
- Enter the new load pattern's name, LIVE ROOF. From the drop-down list, choose the type of load ROOF LIVE. The Self Weight Multiplier should be set to 0. To add the Live load to the load list, click the Add New Load Pattern option.
- **Step 5 Define the Response Spectrum Function**

Define Response Spectrum Function. A collection of time versus spectral-acceleration values is all that makes up a response-spectrum function. Since the functions themselves are not assumed to have units in ETABS, the acceleration values in the function are presumed to be normalized. Instead, a scale factor that multiplies the function and is supplied when the response-spectrum analysis scenario is constructed is linked to the units.

1. To view the Define Response Spectrum Function form, select the Define menu > Functions > Response Spectrum Functions command.
2. From the drop-down selection in the "Choose the Function Type to Add" section, select Spectrum from File.
3. The Response Spectrum Function Definition form will appear when you click the Add New Function button (Fig. 28).
4. Enter the function name in the edit box.
5. In Function Damping Ratio area, type 0.05.

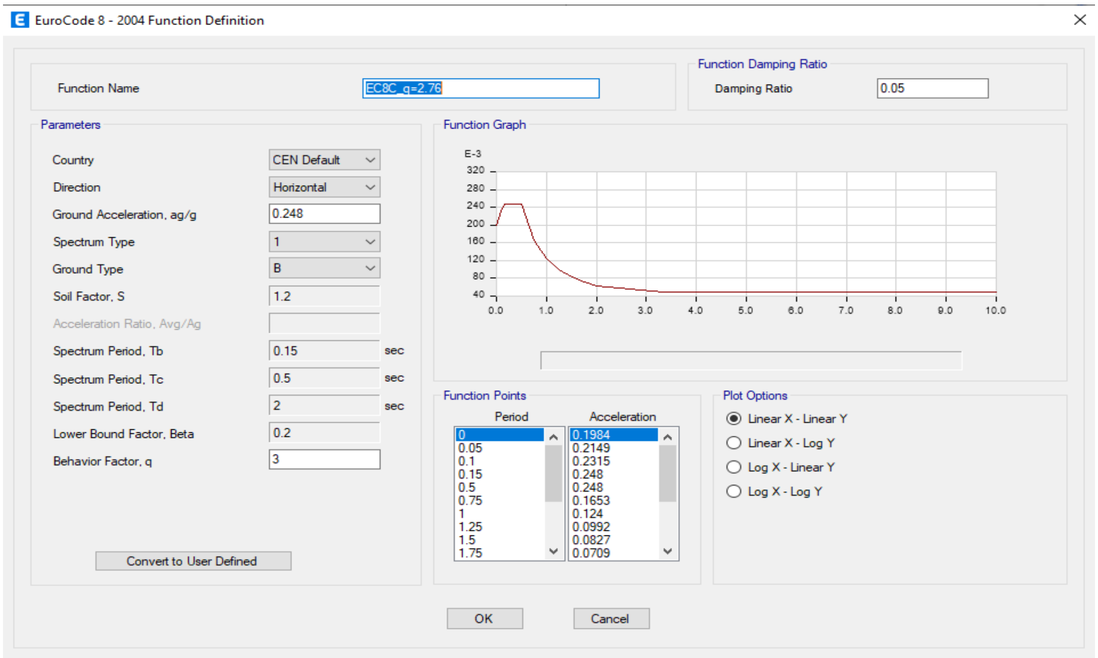


Figure 28. ETABS Define Response Spectrum Functions

- **Step 6 Definition of Response Spectrum Load scenario**

1. To open the Define Load Cases form, select the Define menu > Load Cases option.

2. The Define Load case data will appear when you click the Add New Load Case button. (Fig. 29)

3. Enter EQ - X in the Load Case Name Area.

4. From the drop-down box in the Load Case Type Area, choose Response Spectrum.

5. Choose the CQC option in the modal combination area.

6. In the area of applied load

7. To accept the EQ- X analysis case, click OK on the Load Case Data - Response Spectrum box.

8. On the Define Load Cases form, click the Add New Load Case button.

9. Enter EQ-Y in the Load Case Name Area.

10. Repeat Items D through E.

11. Applied to Load

12. Load applied region

- From the drop-down box in the Load Type field, choose Accel.
- Choose U2 from the drop-down list in the load name field.
- Choose RS IS 1893 II (the function defined in step 7) in the function field.

13. Click the **Add** button

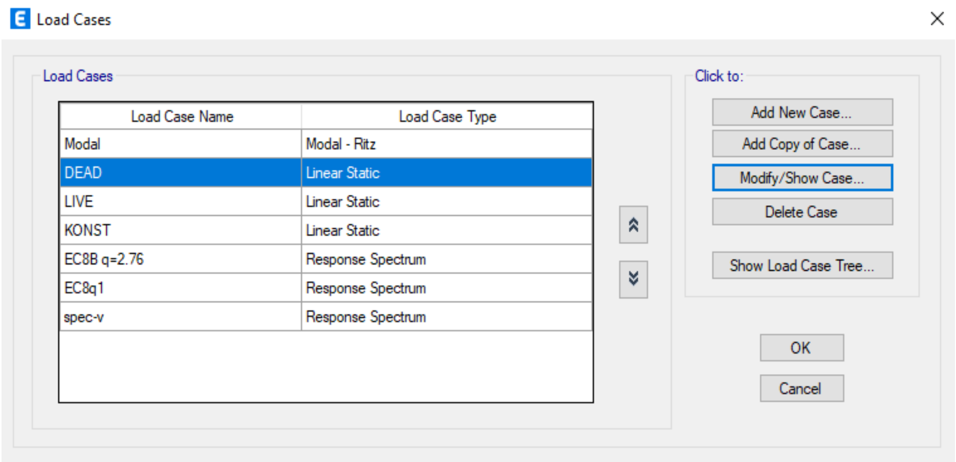


Figure 29. ETABS Define Response Spectrum Load case

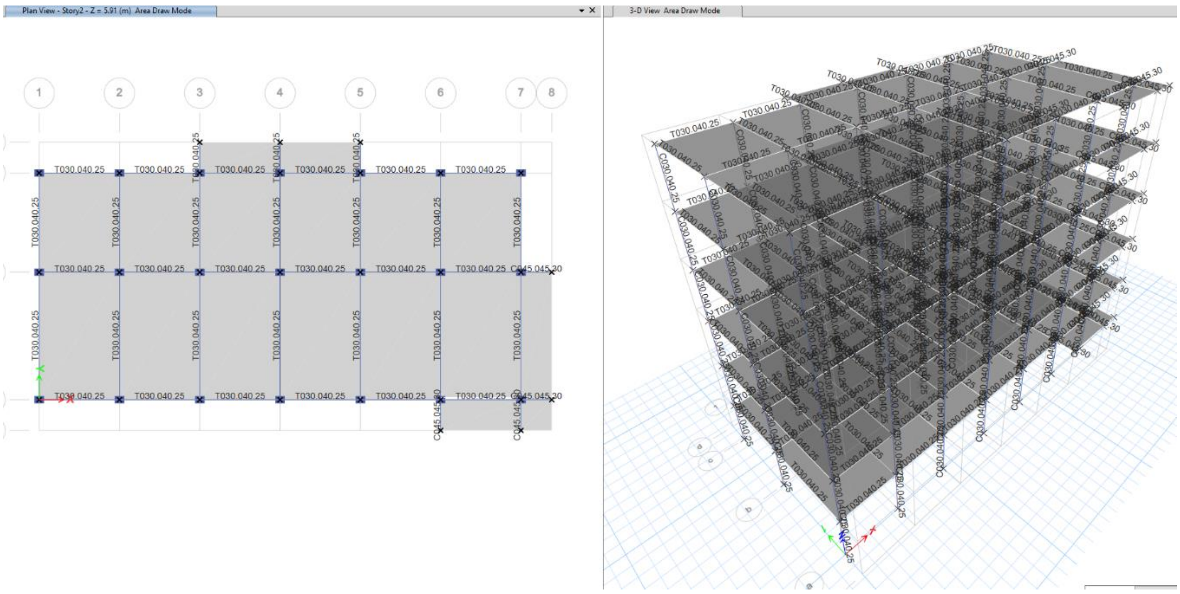


Figure 30. ETABS Undeformed shape of the building

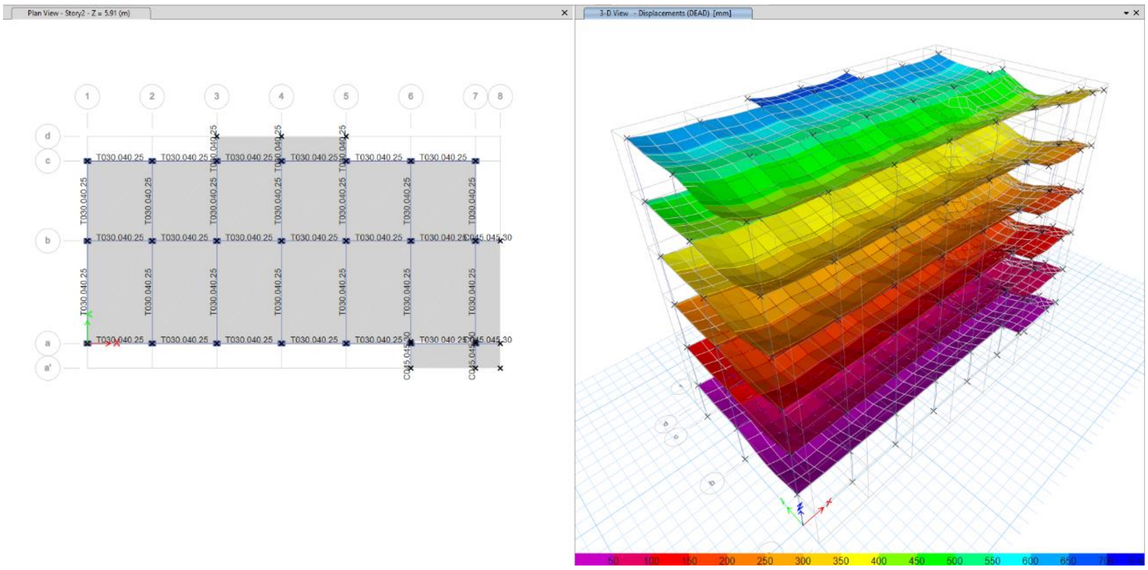


Figure 31. ETABS Deformed shape of the building after analyses

3.3 Object Etabs Modelling (BARON'S BUILDING)

- **Step 1 Begin a New Model**

In this Step, the basic grid that will serve as a template for developing the model will bedefined.

A. Click the **File menu > New Model** command or the **New Model** button. The form shown in Fig. 32 will display. Verify that the default units are set to KN, m, C.

B. The New Model form allows for the quick generation of numerous model types using parametric generation techniques.

   When you select the Grid Only button, the form in Figure will appear.

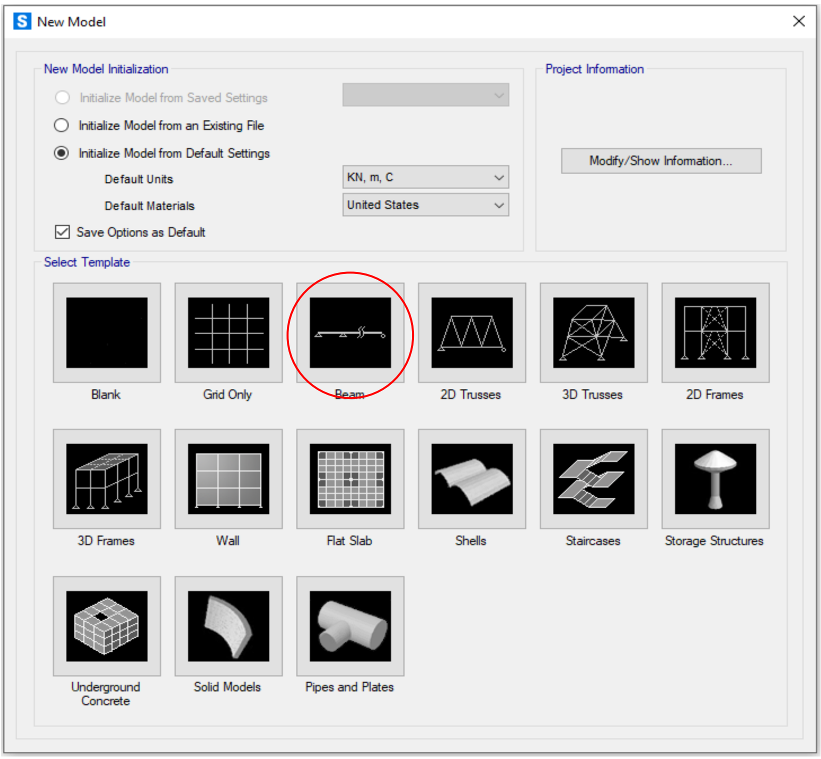
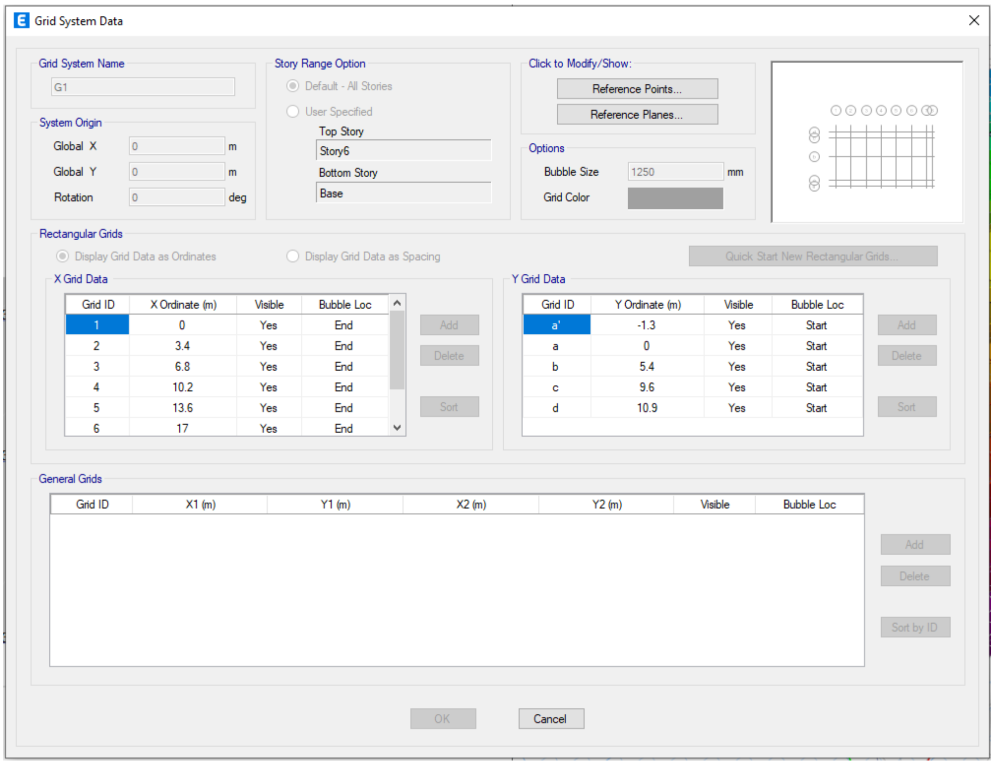


Figure 32. New model form

C. The grids and spacing in the X, Y, and Z directions are specified using the Quick Grid Lines form (Fig. 33). Eight grid lines should be used in the X and Y directions, and seven grid lines should be used in the Z direction. In the X, Y, and Z directions spacing edit boxes, enter 8, 6, and 3.1, respectively. For the sake of this tutorial, make sure that all the values in the First Grid Line Location area are set to zero. This area's values identify where the grid lines' origin is. To go further, press the OK button..



Grid ID	X Ordinate (m)	Visible	Bubble Loc
1	0	Yes	End
2	3.4	Yes	End
3	6.8	Yes	End
4	10.2	Yes	End
5	13.6	Yes	End
6	17	Yes	End

Grid ID	Y Ordinate (m)	Visible	Bubble Loc
a'	-1.3	Yes	Start
a	0	Yes	Start
b	5.4	Yes	Start
c	9.6	Yes	Start
d	10.9	Yes	Start

Grid ID	X1 (m)	Y1 (m)	X2 (m)	Y2 (m)	Visible	Bubble Loc

Figure 33. ETABS Define Grid Data form

1. Select Coordinate Systems/Grids from the Define menu. It will show the Coordinate/Grid Systems form. Ensure that Global is highlighted for the Systems item on the Coordinate/Grid Systems form will display. The uneven spacing in the X, Y, and Z directions is specified using the Define grid data form. Set the spacing of the display grid.

2. For Grid ID 2 in the Y Grid Data, set the spacing to 2, and for Grid ID 1 in the Z Grid Data, put it to 1.2.

3. To close the Define Grid Data form, click the OK button. Figure 33 will appear after you click the OK button to close the Coordinate/Grid Systems box. As seen in Fig. 33, the grids are displayed in two vertically tiled view windows: an X-Y "Plan" View on the left and a 3-D View on the right. By choosing the Options menu > Windows command, the quantity of view windows can be altered.

In Fig. 33, you will see that the "Plan" view is open. The title bar of the display is highlighted when the window is open. By clicking anywhere in the view window, you can make a view active. Keep in mind that Z positive is pointing "up" and that the

Global Axes are also visible. When ETABS speaks about gravity's direction, it means "down" or in the negative Z direction.

- **Step 2 Clarify Material**

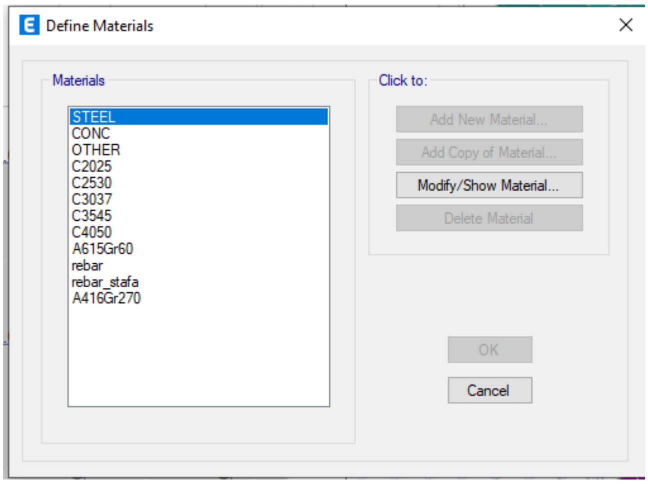


Figure 34. ETABS Define Materials form

To create, change, or remove a material property definition, use the Define menu > Materials command. The structural objects (frame sections, cable sections, tendon sections, area sections, solid properties) are then defined using the definitions of the material properties.

The Define Material form (Fig. 34) will appear when you select the Materials command from the Define menu.

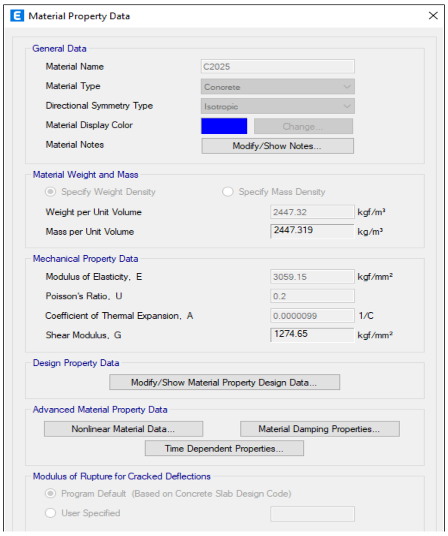


Figure 35. ETABS Material Property Data form -Concrete

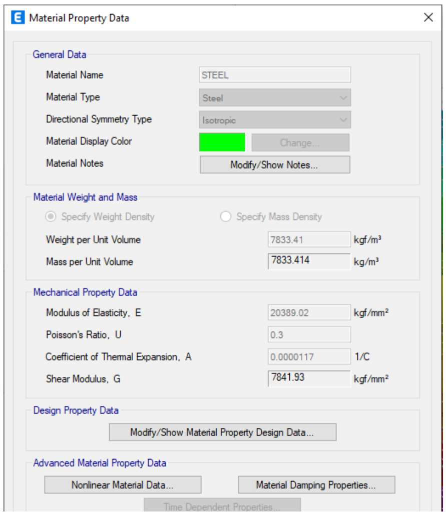


Figure 36. ETABS Material Property Data form -Rebar

A 4000Psi should be highlighted in the Materials display list. Next, click the Modify/Show Material option to bring up the form in Fig. 35.

A Type C20 in the Material Name and Display Colour edit boxes and choose concrete from the drop-down list under Material Type.

A. Select 25 as the Weight per Unit Volume. Modulus of Elasticity at Set

B. Set Weight per unit Volume as 25. Set Modulus of Elasticity to 4351.1324

C. Set Poisson's Ratio to 0.2. Set Specified Concrete compression strength at 29000, then push the OK button.

D. In the Materials display list, select an A992fy50. The form shown in Fig. 34 will appear once you click the Modify/Show Material button.

E. Type "Rebar" in the "Material Name" and "Display Colour" edit boxes and choose "Rebar" from the "Material Type" drop-down list.

F. Set 72.5189, 106.7783, 72.5189, and 108.7783 for the minimum yield stress, fy, minimum tensile stress, fu, expected yield stress, fye, and expected tensile stress, fue. To exit all forms, click the buttons on the Material Property Data form and the Add materials form.

- **Step 3 Define Frame Section**

When a frame section is defined, it can be assigned to frame objects.

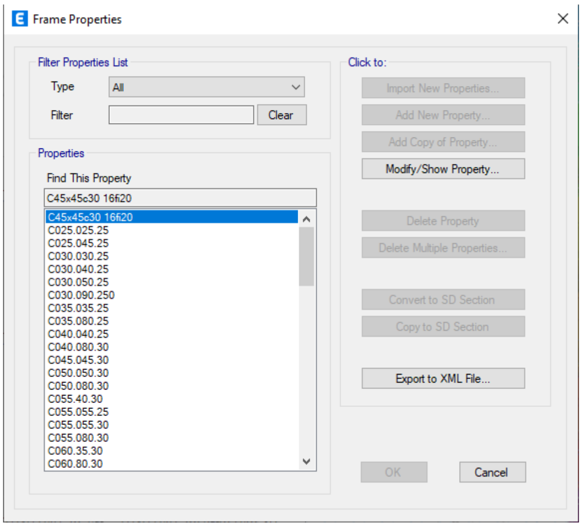


Figure 37. ETABS - Frame properties table

You can define frame section properties based on a section's dimensions using the Define menu > Frame Sections command,

- import sections from preconfigured databases, review and amend section properties,
- and delete section properties.

- **Step 4 Establish Load Patterns**

A. Define Load Patterns

B. Dead, Dead Wall, Dead Slab, Dead FF (Floor Finish), Dead RT (Roof Treatment), Live, and Live Roof loads operating in the direction of gravity are the loads used in this problem.

C. A. To open the Define Load Patterns form (Fig. 37), select the Define menu > Load Patterns command. It should be noted that only one default load case—a Dead Load scenario with self-weight [DEAD]—has been defined.

D. Keep in mind that the default case's self-weight multiplier is set to 1. This means that the self-weight of all members will be automatically included in this load pattern at 1.0 times. Both load patterns and load cases, which may vary, are present I ETABS. However, when a load occurs, the application immediately creates a corresponding Load case.

E. Select the Edit button next to the

F. Activate the Pattern Name column. Enter the new load pattern's name, DEAD Wall. Choose a load type from the drop-down menu; in this instance, choose SUPER DEAD. The Self Weight multiplier should be set to 0. To add the Dead Wall load to the list of loads, click the Add New Load Pattern option.

G. Add Dead Slab, Dead FF, and Dead RT load cases by repeating item B.

H. Enter the new load pattern's name, LIVE. Choose the LIVE load type from the drop-down menu. The Self Weight Multiplier should be set to 0. To add the Live load to the load list, click the Add New Load Pattern option.

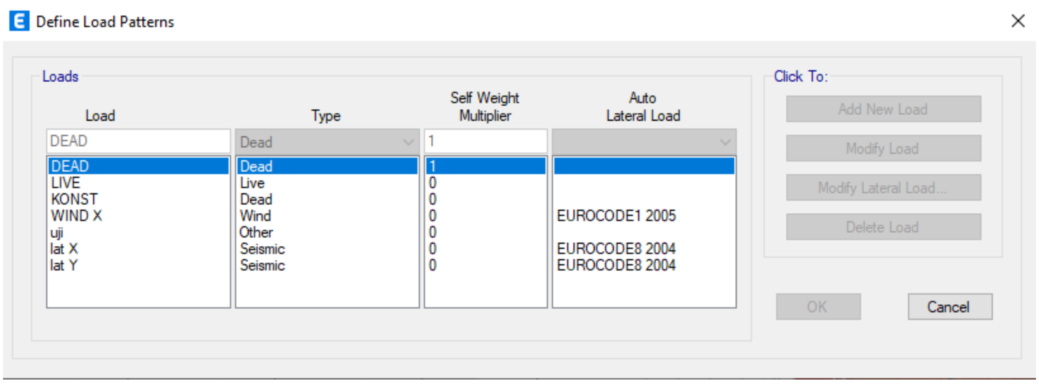


Figure 38. ETABS Define load Patterns form

I. Enter the new load pattern's name, LIVE ROOF. From the drop-down list, choose the type of load ROOF LIVE. The Self Weight Multiplier should be set to 0. To add the Live load to the load list, click the Add New Load Pattern option

- **Step 5 Define the Response Spectrum Function**

A collection of time versus spectral-acceleration values is all that makes up a response-spectrum function. Since the functions themselves are not assumed to have units in ETABS, the acceleration values in the function are presumed to be normalized. Instead, a scale factor that multiplies the function and is supplied when the response-spectrum analysis scenario is constructed is linked to the units.

1. To view the Define Response Spectrum Function form, select the Define menu > Functions > Response Spectrum Functions command.
2. From the drop-down selection in the "Choose the Function Type to Add" section, select Spectrum from File.
3. The Response Spectrum Function Definition form (Fig. 38) will appear when you click the Add New Function button.
4. In Function Nameedbox, type RES IS 1893 II.
5. In Function Damping Ratio area, type 0.05.

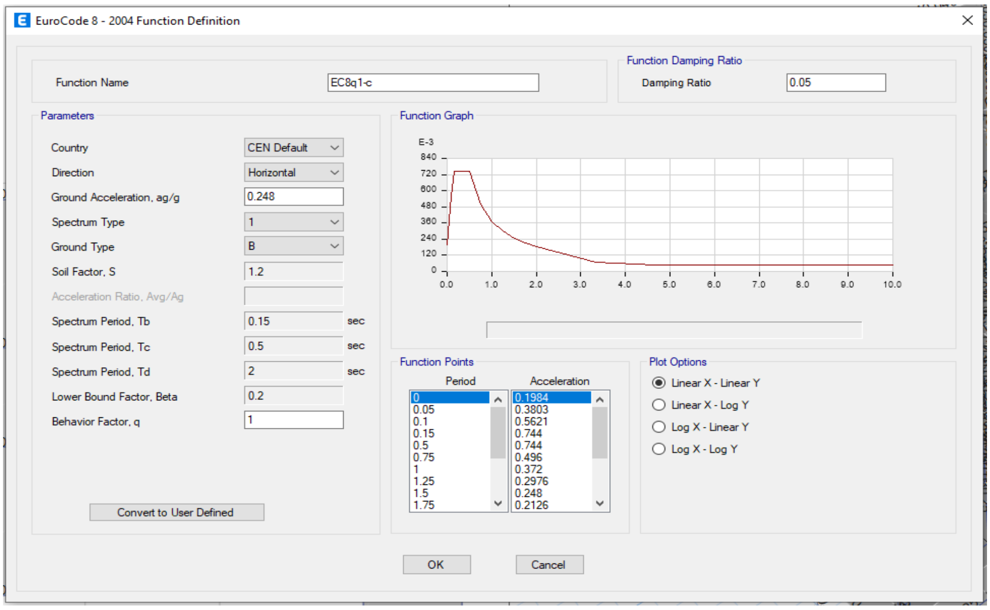


Figure 39. ETABS Define Response Spectrum Functions

1. Select the text file containing the response spectrum data by clicking the Browse button in the Function File section (see Appendix). The File Name display box will show the path of the selected file. To view the selected file in WordPad, click the View File button.

2. Type 5 in the Header lines to skip area.

3. Choose the Period vs Values option under the Values are: section.

4. To view the Response spectrum, click the Display graph button.

5. The Response Spectrum Definition form (Fig. 39) will appear when you click the Convert to User Defined button.

6. To close all forms, click the OK buttons on the Define Response spectrum functions and Response Spectrum Function Definitions forms.

- **Step 6 Definition of Response Spectrum Load scenario**

1. To open the Define Load Cases form, select the Define menu > Load Cases option.
2. The Define Load case data form (Fig. 39) will appear when you click the Add New Load Case button.
3. Enter EQ - X in the Load Case Name Area.
4. From the drop-down box in the Load Case Type Area, choose Response Spectrum.
5. Choose the CQC option in the modal combination area.
6. In the area of applied load
7. . To accept the EQ- X analysis case, click OK on the Load Case Data - Response Spectrum page.
8. On the Define Load Cases form, click the Add New Load Case button.
9. Enter EQ - Y in the Load Case Name field.

10. Repetition of Items D-E

11. In Load Applied Area

- From the drop-down box in the Load Type field, choose Accel.
- From the drop-down list, choose U2 in the Load Name field.
- From the drop-down list in the Function section, choose RS IS 1893 II (Function defined in step 7).

12. Select Add from the menu.

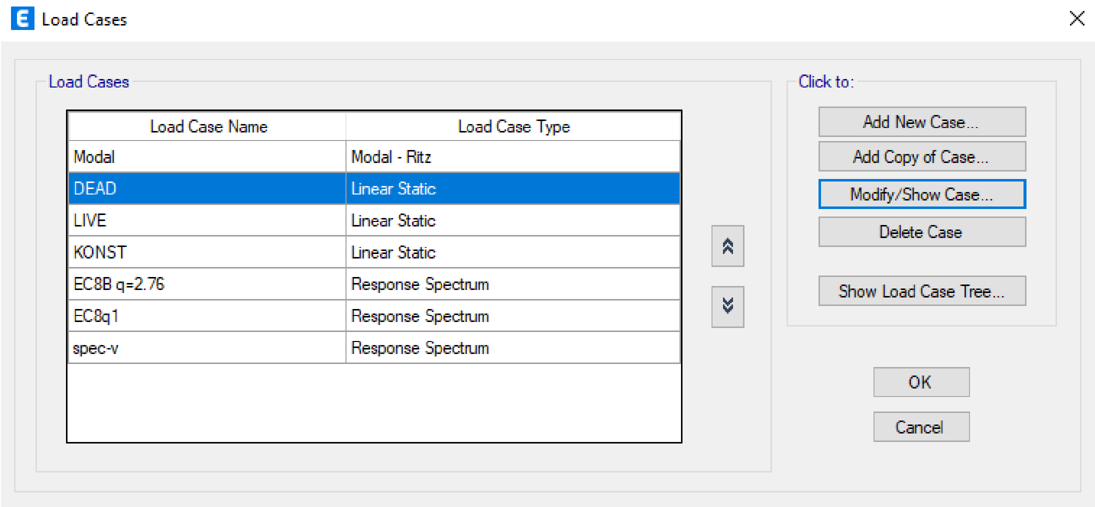


Figure 40. ETABS Define Response Spectrum Load case

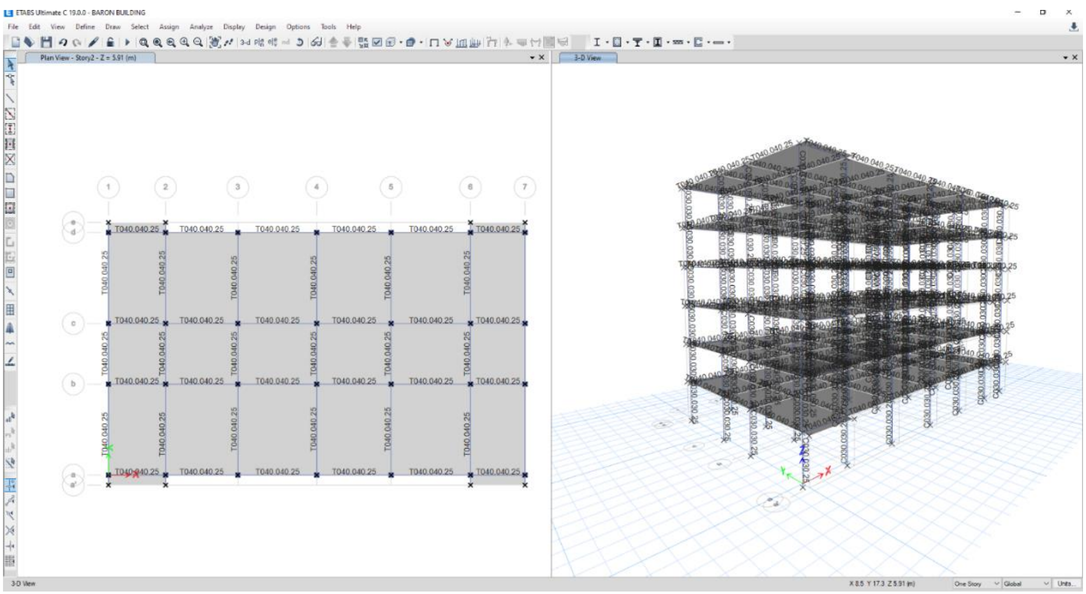


Figure 41. ETABS Undeformed shape of the building

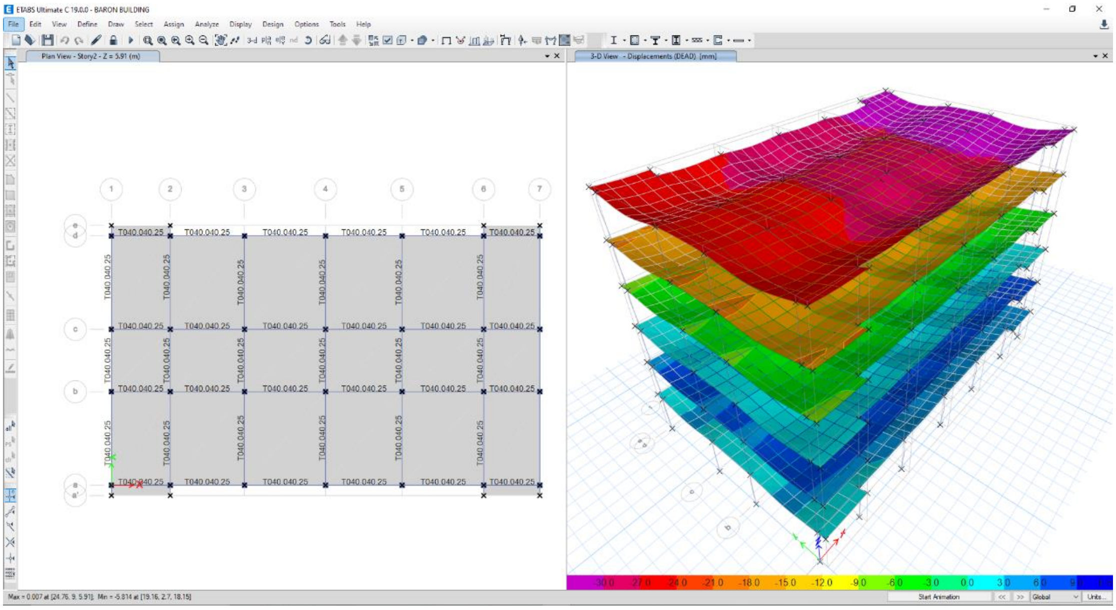


Figure 42. ETABS Deformed shape of the building after analyses

CHAPTER 4

ANALYSES OF THE RESULTS

4.1 Technical specifications and loads

The Central Technical Archive of Structure (AQTN) office in Tirana has taken the construction plan and technical specifications for the 82/2 building from the technical archives. The Research and Design Institute No. 1 Tirana (ISP) created them.

This project's design was for a structure with prefabricated reinforced concrete frames. This building is intended for areas with a seismic intensity of IX, according to the design notes. This project causes a delay in the beginning of the RC Frame Structure's development.

The district's Research and Design Bureau (BSP) was tasked with reconstructing the project in accordance with the technical project, considering the geography, surrounding natural environment, climate, and tradition.

The Durres building typology 82/2 is being built in accordance with this BSP-created design. Regarding this project's redesign, the following modifications have been made to the structural project developed by ISP (Research and Design Institute, Tirana): Assessment of Damages

Prior to 1990, Albanian Technical Codes KTP-2-89, which also incorporated seismic design standards, served as the main reference for building designs. RC frames, brick masonry (clay/silicate), and prefabricated constructions all had varying degrees of failure during the Durres Earthquake.

- **The main causes of RC frame damage:**

Both structural and non-structural elements had visible damage.

- The construction of high-rise buildings with reinforced concrete frames rather than diaphragms or cores to produce highly flexible structures. Non-structural components, such as out-of-plan walls and walls with horizontal and diagonal fissures, have been found to be damaged.

- Differential settlement of the foundation brought on by seismic activity and irregularities in plan and height.
- Using less structural stiffness, a structure can be designed with hidden beams in slabs of lower height, particularly the perimeter beams.
- Improper stair beam design that results in "short columns" by lowering the height of the columns.
- The lack of piles, even though they are required for seismic activity and the transfer of vertical stresses.
- The seismic junction is inadequately designed when two pieces are built on the same foundation slab or pile, which causes a collision.
- Because columns' strength is inferior to that of beams, plastic hinges have been developed for them.
- Poorly constructed beam-column joints.
- The earthquake that occurred on November 26, 2019, caused two of the analysed buildings (type 82/2) to collapse, while the other structures have sustained major structural damage.

The following Figure from a field investigation shows damages to structural and non-structural parts for various RC structures in Durres city.

Table 3. Properties of steel Ç-3 at the type 82/2

		
Figure 43. Spalling of concrete	***Figure 44.*** Longitudinal reinforcement not fastened by stirrup	***Figure 45.*** Out-of-plane wall failure

4.1 Materials' Qualities

Referencing the technical notes in the structural design of the type 82/2 structures

- Concrete is classified as M-200 (C 16//20).
- According to the KTP-Albania design code, the reinforcing steel is -3.

Table 4. Properties of concrete for the building type 82/2

Characteristics of Reinforcing steel	Ç-3
Characteristic yield strength	f_{yk} =250 MPa
Characteristic tensile strength	f_{tk} =320 MPa
Modulus of elasticity	E_s = 210 000 MPa =210GPa
Partial factor	γ_s = 1,15
Design yield strength	f_{yd} = 215 MPa
Design yield for shear	F_{ywd} = 180 MPa
Poisson's ratio	v = 0.30

Table 5. Concrete test for one of the building type 82/2

Characteristics of concrete	C16/20 MPa
Characteristic compressive cylinder strength	f_{ck} = 20 MPa
Characteristic cubic strength	R_{ck} = 16 MPa (fck,cube)
Mean value of concrete cylinder compressive strength (28 days)	f_{cm} = 28 MPa
Mean value of axial tensile strength of concrete	f_{ctm} = 2,2 MPa
Characteristic axial tensile strength of concrete	$f_{ctk}(5\%)$ = 1,5 MPa
Characteristic axial tensile strength of concrete	$f_{ctk}(95\%)$ = 2,9 MPa
Secant modulus of elasticity of concrete	E_{cm}=30GPA
Design value of modulus of elasticity of concrete	E_{cd}= 25GPa
Partial factor for concrete	γ_c = 1,5 α=0.85
Design value of concrete compressive strength	$f_{cd} = \alpha * f_{ck} / \gamma_c$ = 11,3 MPa
Poisson's ratio	v = 0.20

Table 6. Real Steel tests for one of the building type 82/2

Sampling Date / **Data e marrjes se mostres:** 12/12/19
Date of test / **Data e Proves:** 14/12/19

Specimen Nominal Diameter / **Diametri Nominal / Mostrave:** Ø = 14,16,22 mm (Hekura pa vlaska) (No Ribbed steel bars)

TEST RESULTS / REZULTATET E TESTIT

NR.	Diametri Nominal Ø, [mm]	Diametri Efektiv Ø, [mm]	Linear Weight Masa Lineare [kg/m]	Seksioni Terthor (Ao) [mm²]	Rezistenca ne Terheqje (Re) [N/mm²]	Rezistenca ne Kepujte (Rm) [N/mm²]	Raporti Kepujte/Rrjedhshmeri (Rm/Re)	Zgjatimi Relativ [%]	Note Shenime
1 2	14 1.4	14.05 9.50	1.377 0.514	175.40 78.27	333.6 333.9	402.1 444.0	1.500 1.314	32.14 -5.00	
1 2	15 16	16.27 15.57	1.031 1.552	207.73 199.66	269.6 369.4	413.7 400.2	1.429 1.416	33.75 35.00	
1 2	22 22	21.87 21.91	2.948 2.958	375.49 376.75	330.5 331.6	471.9 469.4	1.428 1.415	32.27 30.00	

Source: Republic of Albania, VKM, "For the Implementation of In-Depth Expertise for the objects damaged," Council of Ministers, Tirana, 2020

The results of the laboratory tests on the steel's qualities show that they are acceptable considering the design definition and the technical situation.

We only discovered a small number of tests that are conducted, specifically two tests for concrete and six for steel. in unnamed components

Laboratory tests and sample collection for analysis of concrete and steel. The concrete sample's compressive strength is lower, per tests done in one of the 82/2 building types (for the item in Tables 4 and 5). Figures 43, 44 and 45 illustrate the KTP (Albanian design code) criteria and less than 50% of the design requirements. While the results of the laboratory tests on the steel show that they are acceptable given the design specification and the technical requirements

4.2 Model and Examination

- The ETABS Ultimate 19 program is used to model the Type 82/2 structure in two examples for the 6-story building. The project's technical notes are followed when modelling the structure.
- Slabs serve as the shell pieces in a 3D frame section that makes up the model.
- The material proprieties (concrete and steel) are taken as stated in section 5. Dead, live, and seismic loads in accordance with KTP-6-78 (Albanian design code) in the following ways:
- Slabs serve as shell elements in a 3D frame part of the object.
- The following loadings (dead, live, and earthquake) follow KTP-6-78 (Albanian design code): [13]

Table 7. Test results

Coring Date / Dt. E Karrotazhit: 12.12.2019
Testing date / Data e testimit: 13.12.2019

/ REZULTATET E TESTIMIT

Sample Mostra No.	Position Pozicioni	Height for test Lartesia per test H [mm]	Diameter for test Diameter per test D [mm]	H/D ratio Raporti H/D H/D	Weight Masa [g]	Density Masa Volumore [g/cm³]	Load Ngarkesa [kN]	Compr. Strength Rez. Shtypje Cubic [MPa]	Note * Shenime
K1	Element beton/arme marre nga mbetjet e objektit te siperpermendur te depozituara prane Rajonit te Policise Shijak	77.0	75.0	1.03	778	2.287	27.9	6.32	
K2		77.5	75.0	1.03	797	2.328	35.6	8.06	

$$Sv = Sg1 + 0.8\Sigma Sp1 + Sp2 \quad \text{(Equation 1)}$$

Where:

$g_1 - dead\ load$
$p_1 - live\ loads$
$p_2 - live\ loads$

Live loads are applied as follows:

$Apartment\ p_n = 150\ daN/cm^2$
$Hallway\ p_n = 300\ daN/cm^2$
$Terraces\ p_n = 200\ daN/cm^2$

Response spectrum analysis and non-linear static analysis (Pushover), two methods of analysis are carried out using the ETABS software [14]

To estimate how much reinforcement is needed to comply with this design code, the structure is also simulated in accordance with design code guidelines. The spectra captured at Durrës station during the earthquake on November 26, 2019, are used to calculate the seismic loading. below

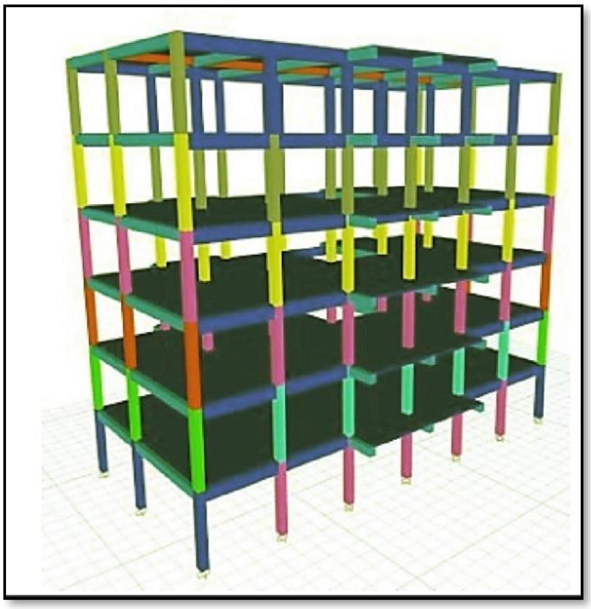


Figure 46. Geometric model six storey

4.3 Results

The results of the two methods analysis, such as non-linear static analysis and response spectrum analysis (Pushover), are displayed below. The interstorey drift is a crucial factor to consider when estimating the damage caused by aspects both structural and non-structural during earthquake activity

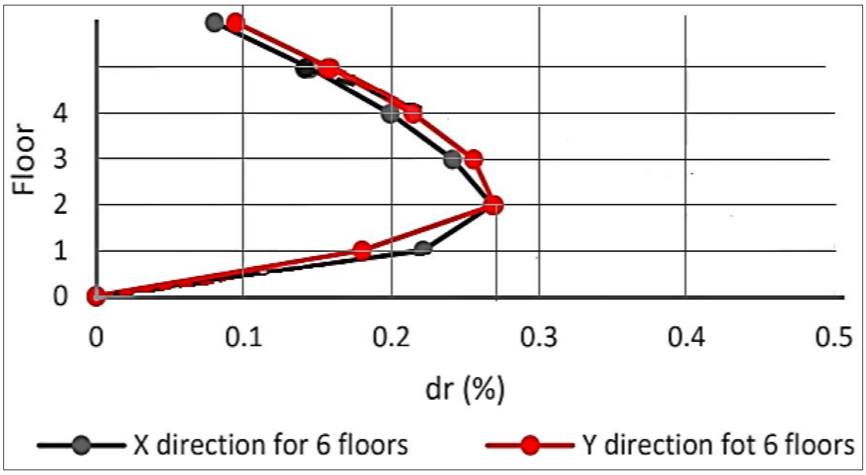


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Figure 47. Maximal drifts for the 82/2 type of building

As shown in the Graphic in Figure 46 of the Maximum Drift, the 6-storey building's interstorey drift can reach as high as 0.0043.

The criterion of EC-8 and KTP 89-2 are satisfied for the circumstances under study (the maximum drift allowed by the code is 0.0087). Because there is less reinforcing for the columns in the second through third floors, the inter storey drifts are greater.

With reference to the columns on the ground floor are strengthened with 1 ϕ 20+2 ϕ 16. The reduction is from 6 ϕ 20+2 ϕ 16 on the first floor to 4 ϕ 20+2 ϕ 16 in the 2-nd floor (up to the top floor), which is a reduction of 27.4%. However, the columns section's size remains constant.

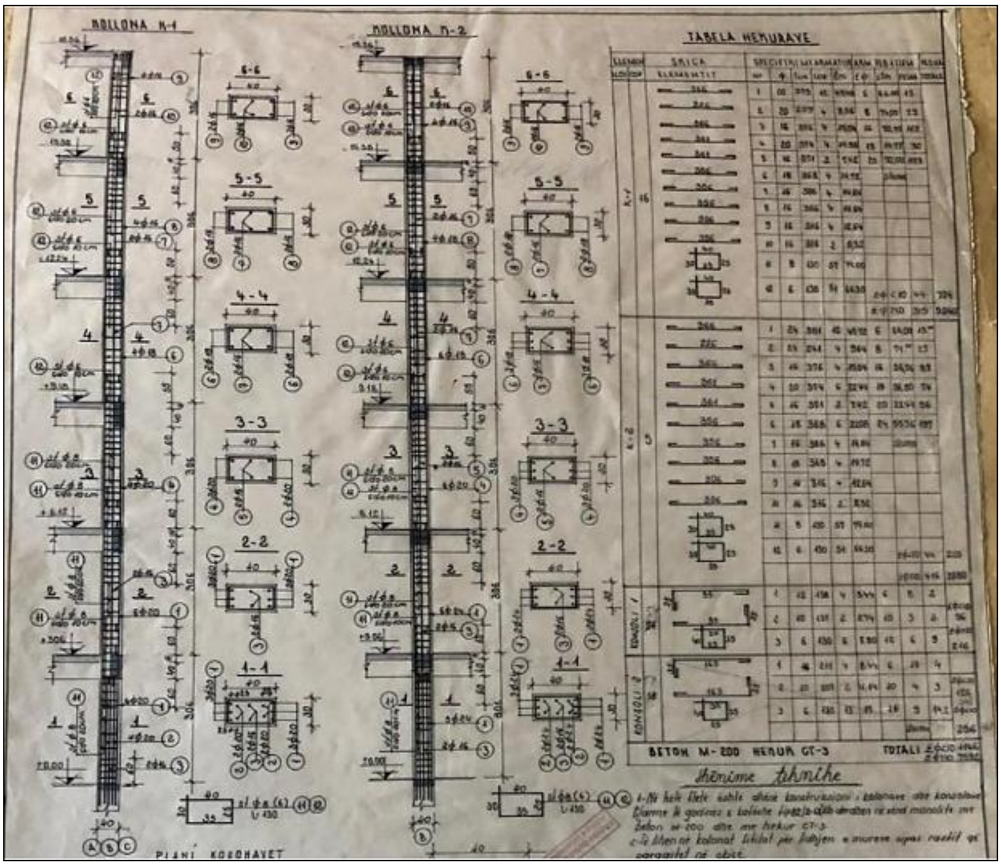

Figure 48. Details for reinforcement of columns (as constructed)

The six-story building exhibits a seismic capacity that is insufficient to meet the seismic demand (for y-direction) according to the nonlinear static analysis (Pushover)

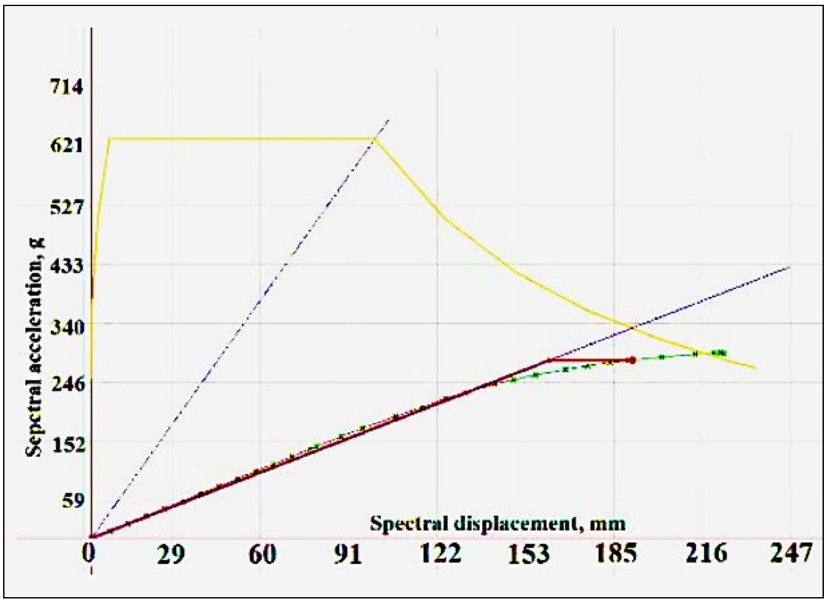

Figure 49. Capacity diagrams for 6-storey building X direction

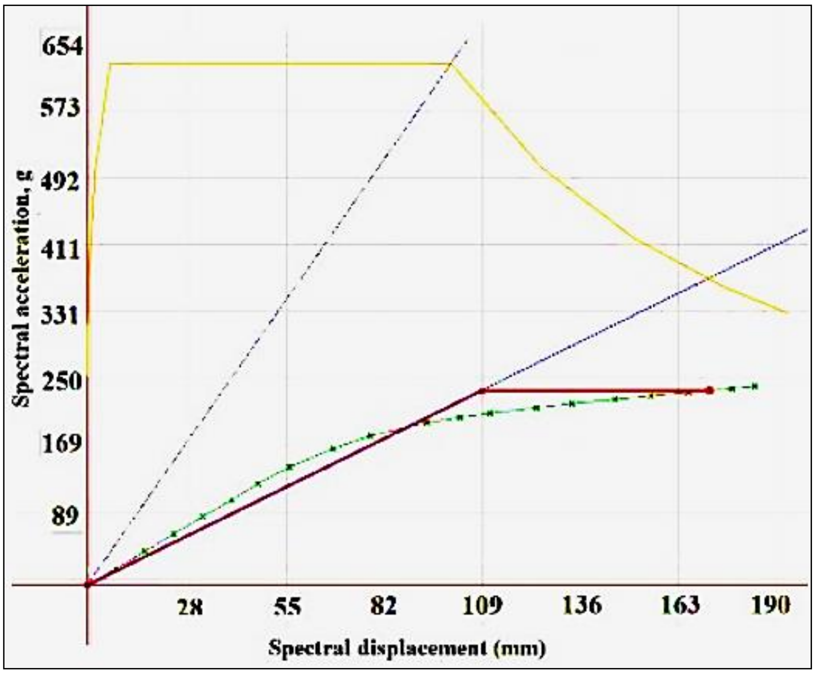

Figure 50. Capacity diagrams for 6-storey building Y direction

The P-M2-M3 interaction determined from the response spectrum is represented by a point outside the interaction curve (Figs. 50 and 51).

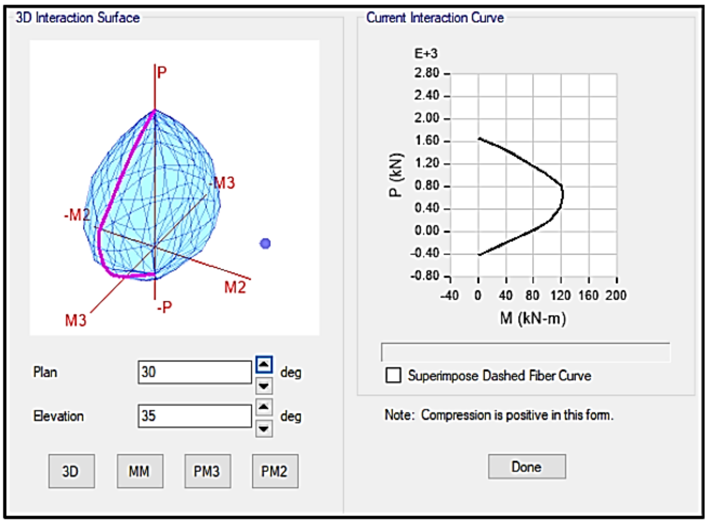

Figure 51. Interaction curves for column in axes A1 (specified in Fig. 47)

The ground floor and first floor columns' reduced ability to flex is evident from their ends appeared to have plastic hinges and were in a ruined state.

It is demonstrated the predicted mechanism of hinges made possible by the invention of plastic collapse develops on the first floor as opposed to the first floor (Figure 21).

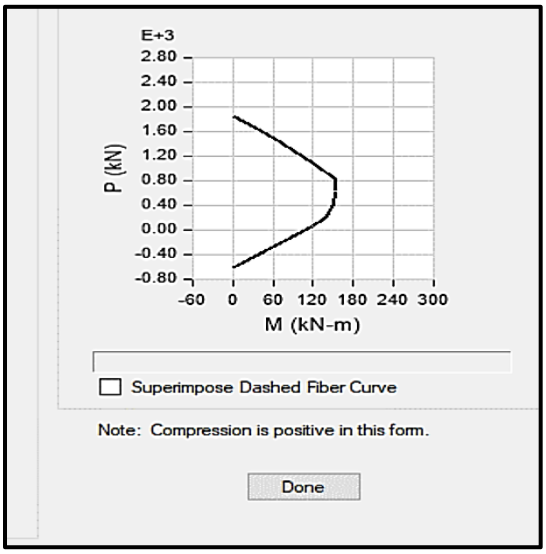

Figure 52. Interaction curves for column in axes B2 (specified in Fig.6) of 6-storey building

Given the sudden altering the columns on this level's longitudinal reinforcement, this is a plausible explanation.

The matching damage limit state is mentioned while describing the performance level. A typical plastic hinge's force-deformation (moment-rotation) curve is shown in Figure 52.

The first floor is where plastic hinges are produced; they correlate to points B (which denotes yielding) and, in some cases, E (which represents complete failure). The first floor of the six-story building also develops plastic hinges, some of which symbolize the points C (ultimate capacity) and E.

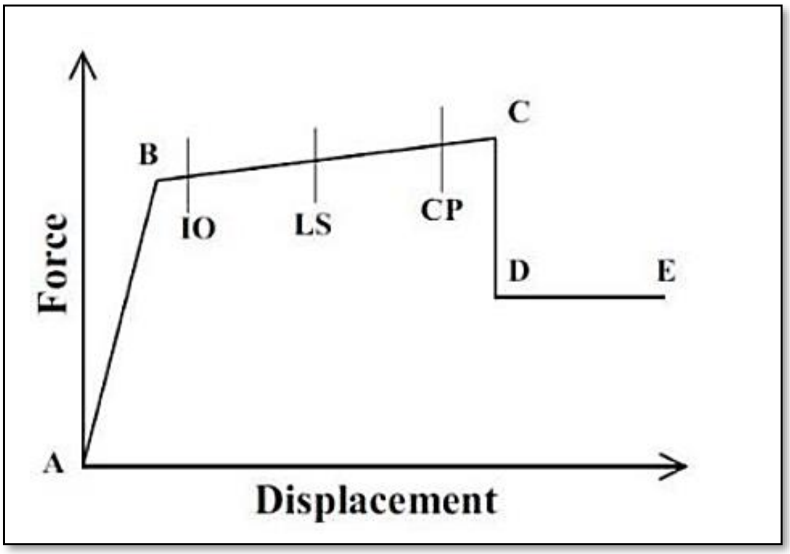

Figure 53. Force-Deformation relationship of a typical plastic hinge

The plastic hinge for the five-story building develops on the first level and corresponds to points B (which denotes yielding), as shown in Figure 53 (the distribution of plastic hinges) and E (which shows complete failure) in some elements.

The first floor of the six-story structure additionally has plastic hinges that depict the points C (ultimate capacity) and E in some parts.

Development of plastic hinges from the nonlinear static analysis (Pushover) (mostly in states a few in the extremely plastic, rupture-prone D area, and the regions A through B on the plastic hinge growth curve)

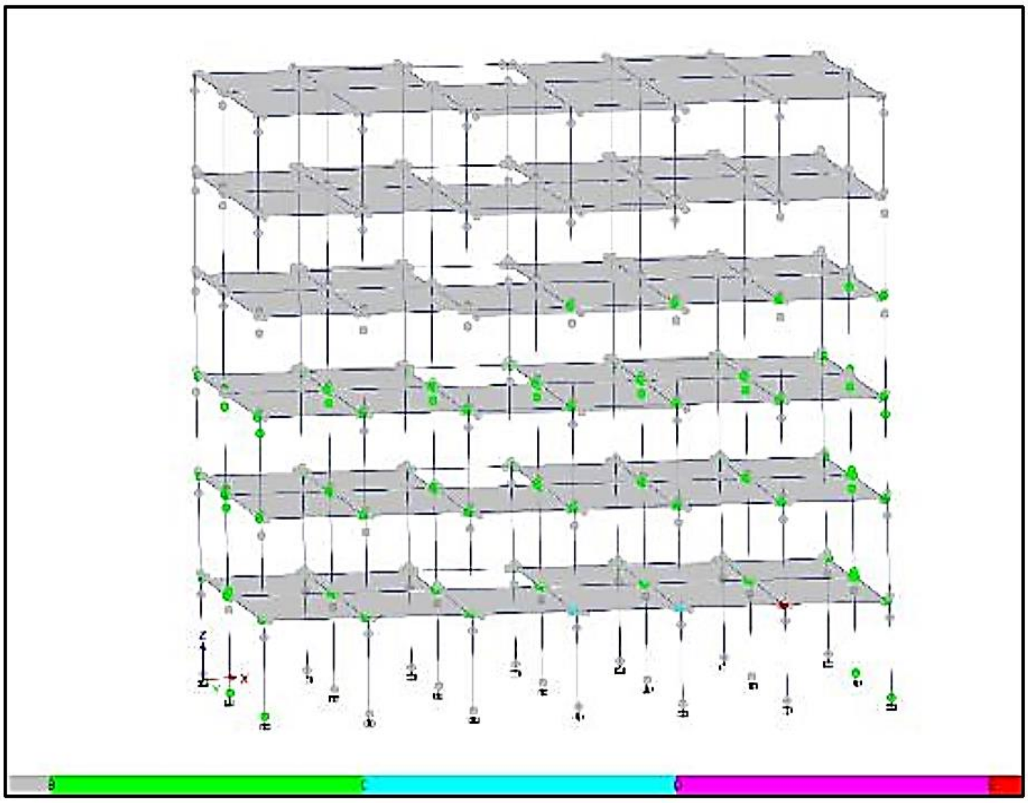

Figure 54. Development of plastic hinges with reference to Figure 53 (the distribution of plastic hinges), for the six-story building

Since ground floor columns in reinforced concrete structures are the most stressed components during seismic activity, a comparison of reinforcement according to the project and structural modeling is done in this work

It is noted that the quantity of reinforcement needed (as assessed by response spectrum analysis) exceeds the project's specifications, even for the six storey model. The project's modeling in accordance with current design codes like Eurocode -8 reveals that these components (with the same precise divisions as those offered in the design documentation) require extremely heightened defense, with respective values of 5.88% and 6.96% (> 4%) that are disallowed by this code.

Table 8. Ground-floor columns' reinforcing areas. Comparison of the two types of ground-floor columns' reinforcing areas and percentages (as-built project vs. design model).

#	6-storey Reinforcement (cm2) Reinforcement (%)	
	As-build project	Design model
K-1	35.43 (2.95%)	70.56 (5.88%)
K-2	49.26 (3.88%)	83.52 (6.96%)

4.3 Conclusions of the specific analysis.

The goal of this study is to compare the findings from response spectrum analysis (used to design potential reinforcements and assess the current project) and non-linear static analysis (Pushover) in order to evaluate the Type 82/2's behaviour and as well of the Type 88/2 - designed structure following the earthquake that struck the city of Durres on November 26, 2019.

The following is a summary of the study's findings:

- Comparing the behaviour of our two buildings - and 6-story buildings reveals that their collapse/failure mechanisms are identical.
- Plastic hinge development provides proof so rather than the ground floor, the predicted collapse/failure mechanism occurs on the first floor.
- Rather than being tied to displacement and drift, the structural problems appear Consequently, the projected collapse/failure mechanism happens on the first floor rather than the first floor.
- The percentage of reinforcement needed for the given part of the 6-story buildings is around 30% more than what is required by KTP and Eurocode 2 [15] while the amount of reinforcement already present is only about 56% of what is called for by design analysis. As a result, the concrete part cannot withstand loading.

- The increase in the ground floor columns' capacity ratio has had an impact on the reduction in the sixth floor's bearing capacity, thus reducing the five-story building's bearing capacity.
- The first floor's reinforcement area for columns is down 35.5% from the ground floor, while the second story's is down 27.4% from the first floor. This explains why the second through third floor has large inter storey drifts.
- This structure's poor performance during the Durres earthquake was caused by a few flaws, which also resulted in damage to the buildings' load-bearing components. This fact is made even clearer when considering the mistakes made During construction, mistakes were made that were excused by the conditions and construction standards in place during the years these structures were built (1983–1993), such as the use of transversal reinforcement with a diameter and distance in the seismic zone that does not meet design requirements, the use of smoothed rebar as longitudinal reinforcement, or rather poor quality of the concrete used.

CHAPTER 5

CONCLUSIONS

5.1 Conclusions

The September and November earthquakes were generated under the same geological conditions. The energy partially discharged by the September 21 earthquake triggered the mainshock of November 26. The statical analysis of its aftershock does not follow seismological law.

These earthquakes revealed and gave us a lesson on how important is to identify in a short time damaged objects and declared uninhabitable objects, to save lives and avoid injuries before another earthquake occurs. The best measure to be taken in zones with high seismic activity is to build according to Eurocode, based on the best seismic hazard map.

The extent of damage can be attributed to:

1. Existence of a large inventory of structures with deficient capacity and ductility
2. Location of the building: buildings in high seismic areas are more vulnerable.
3. Interventions during their lifespan
4. The area of construction (design code of the time, construction techniques)
5. Degradation of material: time, climate changes
6. Poor workmanship-lack of qualified workers, volunteer collaboration
7. The abusive intervention in the building's structure design:
   - At the VIP Building- the damage to the columns due to the poor material properties
   - At Barons Building- the damage to the beam of the foundation.

- **Response-Spectrum Analysis**

To estimate the maximum seismic response of an elastic structure, response-spectrum analysis (RSA), a linear-dynamic statistical analytic method, evaluates the contribution from each natural mode of vibration.

By measuring pseudo-spectral acceleration, velocity, or displacement as a function of structural period for a specific time history and damping level, response spectrum analysis sheds light on dynamic behaviour. The peak response for each realization of the structural period can be represented as a smooth curve by enclosing the response spectrum.

Response-spectrum analysis links the choice of structural type to dynamic performance, which is helpful for design decision-making. Longer structures undergo greater displacement, and shorter structures have greater acceleration.

Prioritized design and response-spectrum analysis should consider structural performance objectives. The various load situations shown in Figure 18 are applied to the models. The results are created for both X and Y directions because the model's plan is asymmetrical in both directions. Below is a list of the conclusions drawn from several analyses, along with a discussion of them in terms of the Response Spectrum analysis load case.

The outcomes of earthquake-resistant models are contrasted. The figures reveal that VIP Building exhibits lower displacements while Baron Building exhibits higher displacements. Storey displacement lowers by 40.24% in the F1 roof level and 18.17% in the F2 roof level in the VIP building model.

- **Base shear**

It can be observed that VIP building model (SW) demonstrates a base shear value of 5733 kN, which is higher than all other models. The lower base shear value for the Baron building model (LRB) is 1531 kN.

This might be the case because, in model 1 (SW), the presence of a shear wall causes the model's weight and base shear to increase.

- **Time Period**

The time period vs model graphs for mode 1 (the first of five modes), according to the Mode Eigen & Ritz load example. The graphs demonstrate that the time period of the VIP model (SW) is reduced by 50.5%, indicating that the model is stiffer, while the time period of the Baron model (LRB), which symbolizes flexibility, is raised by 52.4%.

Lower displacements, drifts, time period, acceleration (4.7%), and high base shear (175%) are all characteristics of the Vip Model (SW), a shear wall model, which all contribute to the model's viability as an earthquake-resistant model. Increased model weight may lead to increased steel content, which indirectly raises the cost of the construction.

Baron Model has high drifts, low displacements, and high. It does, however, have the advantage of being able to offer wherever in the story that we need specifically, which is not possible with other models.

The 2019 earthquake in Albania highlighted the vulnerability of many RC building typologies. The most common design and/or construction deficiencies observed in RC buildings after these earthquakes were:

1. Irregularities in height and/or plan of RC frame structure was often due to the layout of masonry infills or stairs, which caused "soft storey" failure mechanism and/or torsional effects in these structures. The “soft storey” effect occurred due to variation of stiffness in the building height, while torsional effects caused damage of infills in RC frames located at the building perimeter. Therefore, it is important to ensure a symmetric layout of infills in plan and regular distribution in elevation, whenever possible.
2. Poorly detailed RC frame components for local ductility resulted in poor seismic performance of Beam-column couplings, RC columns, and significantly contributed to damage and collapse of RC frame structures.
3. Interaction of RC frames with masonry infills, which was not considered in the design, sometimes resulted in a "short column" effect and failure of RC elements.

Additionally, this interaction caused cracking of infill walls due to in-plane seismic effects. It was observed that significant number of infills collapsed out-of-plane, which can be assigned to the relationship between in-plane and out-of-plane seismic effects.

Buildings ought to plan and built in conformity with the current seismic design standards and rules, as stated in point a. These codes outline the minimal specifications for structural components, building materials, and construction.

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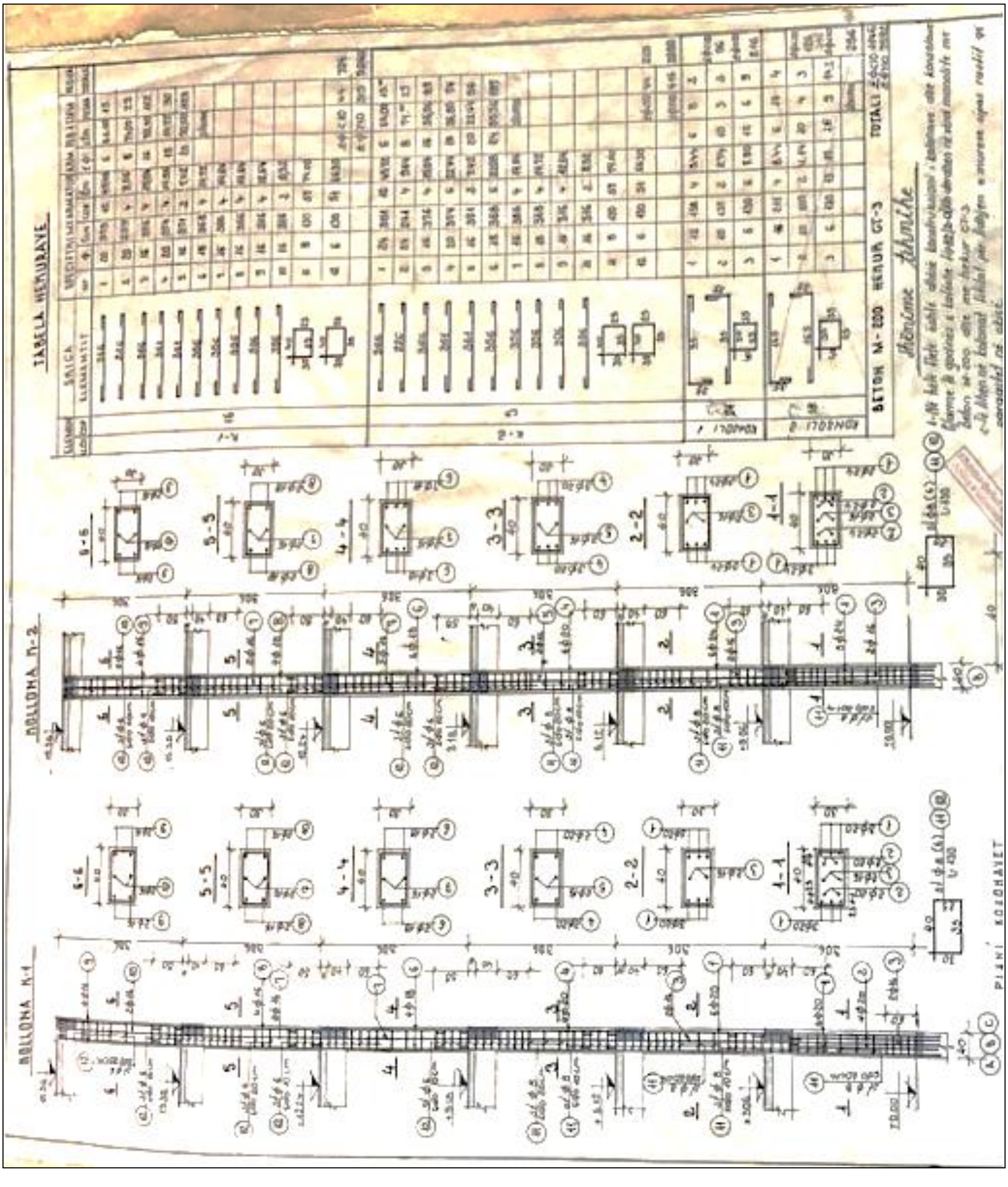
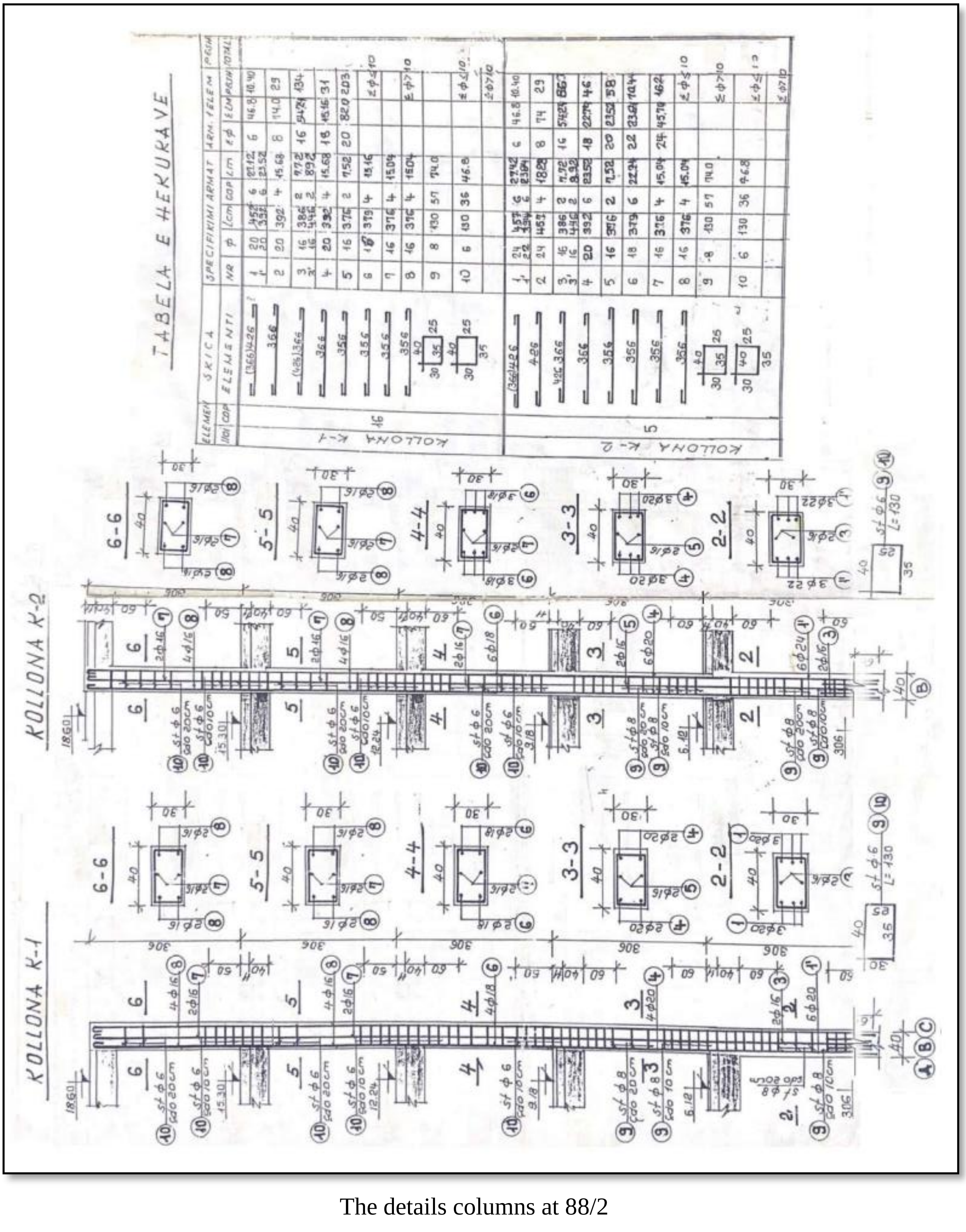
APPENDIX



The details columns at 82/2



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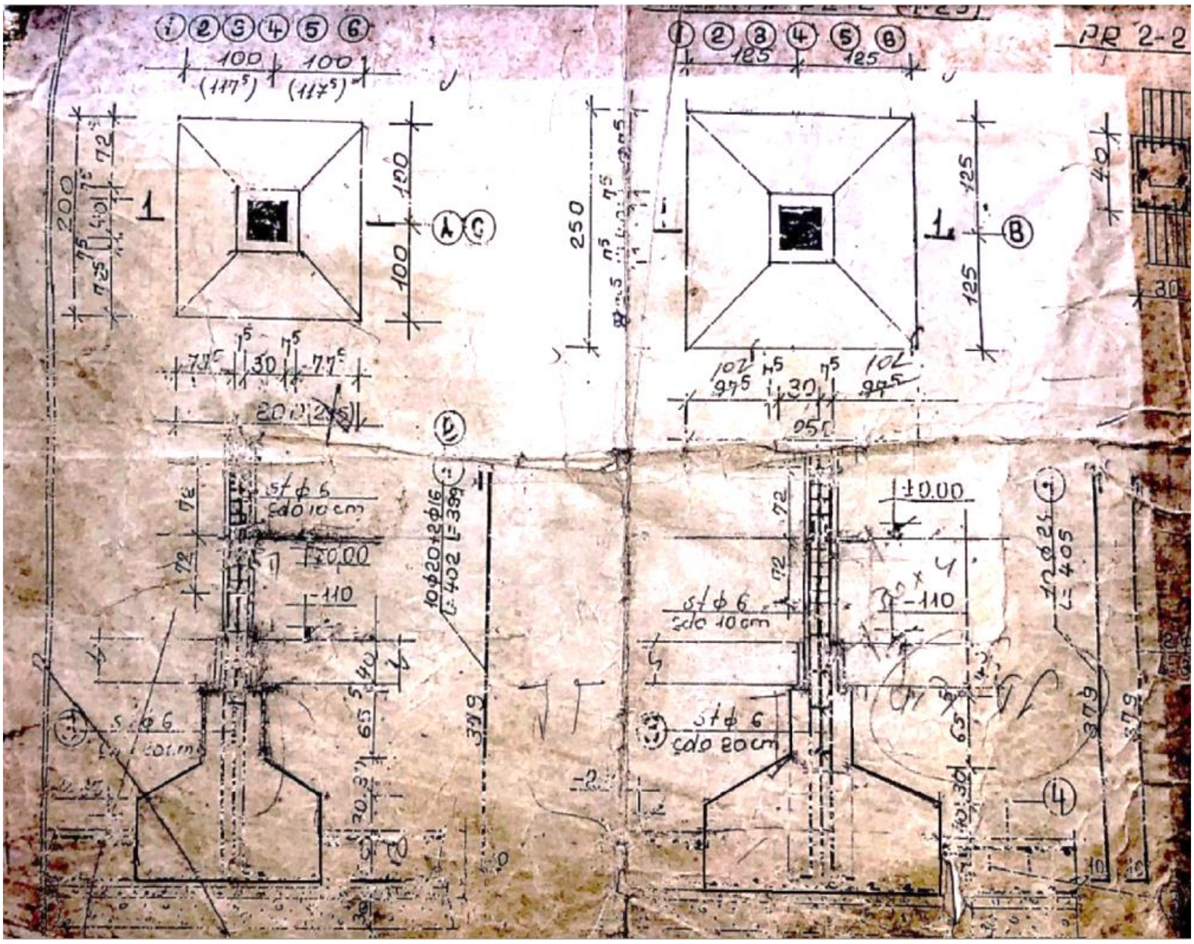

The foundation of the type buildings 82/2