SEISMIC ASSESSMENT AND UPGRADING OF MASONRY BUILDING:
A CASE STUDY FROM ALBANIAN PRACTICE

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Approval sheet of the Thesis

This is to certify that we have read this thesis entitled **“Seismic assessment and upgrading of masonry building: a case study from albanian practice”** 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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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

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ABSTRACT

SEISMIC ASSESSMENT AND UPGRADING OF MASONRY BUILDING:
A CASE STUDY FROM ALBANIAN PRACTICE

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Tirana and Durres city, along with the surrounding areas, were hit by two earthquakes in September and November 2019. The first earthquake with a magnitude of 5.8 on the Richter scale, dated 21.09.2019, and the second even a more powerful one on 26.11.2019 with a magnitude of 6.4 on the Richter scale. The earthquakes caused a series of damage to many residential, social, cultural, educational, and industrial facilities. Among all these objects is also the "Building No. 152", in "Haki Stermilli" street, Kombinat area, Tirana.The building is made of silicate brick with wall thickness varying from 38cm to 25cm over the five floors. The foundation consists of butoconcrete and socle silicate brick.The object of the study will be the assessment of the damages caused by the earthquakes (especially the one dated 26.11.2019), the analysis of the object's constructive stability, and the project of the necessary reinforcing interventions. For the evaluation of the supporting structure of the building we will rely on numerical calculations with finite elements (M.E.F) of 3-dimensional models, realized by commercial software which offers the possibility of calculating the structures with supporting masonry. More specifically, it will be used the numerical calculations performed in the software 3Muri-STA DATA, which offers calculation possibilities with special specifications for such structures. According to the requirements of Eurocodes in general the linear and nonlinear analyzes performed for the purpose of this in-depth analysis, the bearing capacity of the structures is insufficient. That's why structural improvements are necessary. Referring to the results, at the end of this in-depth study, the most likely interventions for the building are: structural rehabilitation against seismic action, adaptation and/or improvement of secondary structural elements. Similar buildings with comparable design, construction features, and soil characteristics can use these intervention strategies to reinforce their structures. Specifically, a 5-floor masonry building with silicate bricks to enhance its safety and durability.

Keywords: *masonry structure, structural damage, seismic capacity, earthquake resistance, pushover analysis, assessment, estimation, intervention.*

ABSTRAKT

SEISMIC ASSESSMENT AND UPGRADING OF MASONRY BUILDING: A CASE STUDY FROM ALBANIAN PRACTICE

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Master Shkencor, Departamenti i Inxhinierisë së Ndërtimit

Udhëheqësi: Prof.Dr. Huseyin Bilgin

Qyteti i Tiranës, se bashku me ate te Durresit dhe rrethinat perreth tyre, ne muajin shtator dhe nentor te 2019, jane goditur nga dy termete. Termeti i pare me magnitude 5.8 i shkalles Rihter, i dates 21.09.2019 dhe nga nje termet i dyte akoma me i fuqishem i dates 26.11.2019 me magnitude 6.4 i shkalles Rihter. Termetet kane shkaktuar nje sere demtimesh ne shume objekte banimi, sociale, kulturore, arsimore dhe industriale. Nder keto objekte eshte edhe “Pallati Nr.152”, ne Rrugen “Haki Stermilli” Kombinat, Tirane.” Ndërtesa është bërë me tulla silikate me trashësi muri që varion nga 38cm në 25cm në pesë katet. Themeli përbëhet nga butokoneton dhe tulla silikati.Objekti i studimit do të jetë vlerësimi i dëmeve të shkaktuara nga tërmetet (veçanërisht ai i datës 26.11.2019), analiza e qëndrueshmërisë konstruktive të objektit, si dhe projekti i ndërhyrjeve të nevojshme përforcuese. Per vleresimin e struktures mbajtese te objektit, do mbeshtetem ne llogaritjet numerike me elemente te fundem (M.E.F) te modeleve 3-dimensionale, te realizuar ne nga softet komerciale te cilat ofrojne mundesi llogaritje te strukturave me murature mbajtese. Konkretisht do shfrytezohen llogaritjet numerike te realizuara ne softin 3 Muri-STA DATA, i cili ofron mundesi llogaritjeje me specifika te vecanta per struktura te tilla.Përkundrejt kërkesave të Eurokodeve në përgjithësi në analizat lineare dhe jolineare të kryera për qëllimin e këtij studimi të thelluar, aftësia mbajtëse e strukturave rezulton e pamjaftueshme. Për rrjedhojë permiresime strukturore jane te domosdoshme.Referuar rezultateve, në përfundim të këtij studimi të thelluar, rezultatet tregojnë që ndërhyrjet më të mundshme për ndërtesën janë :

Riaftësim të plotë strukturor kundrejt veprimit sizmik, përshtatje dhe/ose përmirësim i elementeve dytësor strukturor. Ndërtesa me karakteristika te ngjashme ndërtimi dhe të tokës mund të përdorin këto strategji ndërhyrjeje për të përforcuar strukturat e tyre. Konkretisht, një ndërtesë murature 5-katëshe me tulla silicate, për të rritur sigurinë dhe qëndrueshmërinë e saj.

Fjalët kyçe: *Struktura e muraturës, dëmtimet strukturore, kapaciteti sizmik, rezistenca ndaj tërmetit, analiza e shtytjes, vlerësimi, ndërhyrja.*
12 pt,

DEDICATION

To my family and partner..

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A very special thank you goes out to all the people who contributed to the realization of this thesis and helped me in concluding this circle of studies. Initially I would like to thank my dissertation leader Dr.Huseyin Bilgin for the help and motivation in achieving the objectives of this topic.Also I would like to thank all the professors of Civil Egineering Departament of Epoka University who have contributed to my university education and intellectual development during these years of study.A special thank you is for my family,and partner the contributions and sacrifices of which have made it possible to successfully complete this thesis. I thank them for their continuous support, for believing in me and helping to achieve my dreams. Nothing would be possible without them.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 Overview

In September and November 2019, Albania experienced two significant earthquakes. On 21 September 21st, a magnitude 6.4 earthquake struck the country, killing 51 people and injuring more than 3,000. The epicenter of the earthquake was in the Adriatic Sea, about 20 kilometers (12 miles) northwest of the city of Durres and 34 kilometers (21 miles) west of the capital, Tirana. The earthquake caused extensive damage to buildings, roads, and infrastructure in the affected areas.

On November 26th 2019, a magnitude 5.6 earthquake struck the same region, causing significant damage and injuring 51 people. The epicenter of the earthquake was in the town of Thumane, about 30 kilometers (18.6 miles) northwest of Tirana.

Both earthquakes caused widespread panic and destruction and resulted in many people being left homeless. The Albanian government declared a state of emergency and called for international assistance to help with relief efforts.

After the two earthquakes in September and November 2019, many buildings in Albania were left severely damaged or destroyed. The September 21st earthquake caused widespread destruction, with many buildings in the affected areas collapsing or becoming structurally unsound. The November 26th earthquake also caused significant damage, with many buildings in the town of Thumane and surrounding areas left uninhabitable.

The destruction of buildings was particularly severe in the cities of Durres and Tirana, where many older buildings were not able to withstand the force of the earthquakes. Residential buildings, schools, and hospitals were among the structures that were destroyed or heavily damaged. Among all these objects is also the "Building No. 152", in "Haki Stermilli" street, Kombinat area, Tirana. The Albanian government, with the help of international organizations and donors, undertook a massive effort to inspect and repair damaged buildings, but many of the buildings were not rebuilt, and people were urged to move to safer places.

1.2 Scope of the study

This study includes on site work as well as comprehensive reports for the above mentioned building in order to be able to make an adequate expertise in regards to damages from the earthquake as well as the proper and necessary interventions.

For the preparation of this are taken into consideration:

a) Analysis of existing documents of the initial building (from the few materials provided), as well as various interventions in time (referring to the documents, analysis of physical findings on-site, information received from residents, etc.).
b) Analysis of damages caused by the earthquake and the existing condition of the building, focusing on possible changes and other deteriorations in the structural elements caused by time.
c) Construction of numerical computer models, representing as realistically as possible the geometry, loads, properties of materials, the impact of non-structural elements, and the bearing capacity of the structural elements of the building.
d) Assessment of seismic action according to KTP and the requirements of Eurocode.
e) Performing linear and nonlinear analyzes to assess the bearing capacity of the structure and verify the primary seismic elements.
f) Determining the performance/behavior of the structure against earthquakes and seismic code requirements.

1.3 Problem Statement

Many buildings in Albania suffer from structural deficiencies, and lack of maintenance, especially after the hit of both earthquakes in 2019. If left in these conditions, their service life will be short and most importantly they will impose danger to the surroundings and citizens. Therefore, there is a need for inspections and urgent intervention. In this study, damages from the earthquake as well as the proper and necessary interventions are documented.

1.4 Objective

The object of the study will be the assessment of the damages caused by the earthquakes (especially the one dated 26.11.2019), the analysis of the object’s constructive stability, and the project of the necessary reinforcing interventions if it will be estimated that it can be realized.

1.5 Organization of thesis

The organization of this thesis consists in 6 chapters. In Chapter 1 there are introduction scope of study, problem statement and objectives. Chapter 2 contains literature review which consists in reviewing some aspects of Eurocode 1,6,8 also some previously done studies related to masonry structures. In chapter 3 is explanied methodology used in this study that includes the technical project, conditions of the building and the push over analyses and more performed by 3Muri software. In the next chapter are included the results of these analyses. Chapter 5 contains suggested interventions and verification analysis. Conclusions and further studies are included in Chapter 6.

CHAPTER 2

LITERATURE REVIEW

2.1 History of Eurocodes

The European Union (EU) launched the Eurocodes in the mid-1980s as a uniform system of building codes to be used across all member countries. Prior to the creation of the Eurocodes, building regulations in each member state were distinct, leading to inconsistencies and confusion in the design and construction of structures. The goal of the Eurocodes was to harmonize the standards and procedures used in building construction across the EU, and to promote a single market for construction products and services.

The first phase of the Eurocodes project involved the development of a comprehensive set of codes and standards, covering all aspects of building design, construction, and safety. The codes were based on the best practices and standards used in the EU, and were reviewed and updated by experts from each member state. The final codes were then adopted by the EU and became legally binding across all member countries.

The Eurocodes have since become the foundation for the design and construction of all buildings in the EU, and have been widely adopted beyond the EU, in countries like Switzerland, Norway, and Iceland. The codes have been instrumental in promoting safety, quality, and efficiency in building construction, and have led to greater confidence in the performance and durability of structures. The codes have also helped to reduce the administrative burden associated with cross-border construction, as well as the cost of construction, through the reduction of unnecessary duplication and red tape.

Despite their many benefits, the Eurocodes have faced some criticism and resistance from some quarters, who argue that the codes are too prescriptive and bureaucratic, and that they stifle innovation and creativity in building design. However, the majority of experts and stakeholders believe that the Eurocodes are an essential tool in ensuring the safety, quality, and sustainability of buildings, and that they have played a critical role in shaping the construction industry in Europe and beyond.

The Eurocodes remain an essential tool for the future development of the construction industry, and will continue to play a key role in shaping the future of building construction in Europe and beyond.

2.2 Introduction to Eurocodes 1,6,8

Eurocode 1, 6, and 8 form the backbone of the design and construction of various structures, and they play a crucial role in ensuring the safety and stability of these structures under different loads and environmental conditions. These codes provide a thorough and systematic approach to the design and construction process, taking into account all relevant factors such as material properties, structural configurations, load combinations, and environmental effects.

2.2.1 Eurocode 6 :

Eurocode 6, also known as the Eurocode for masonry structures, is a set of technical standards that defines the design, calculation, and construction requirements for masonry structures in the European Union. It was adopted by the EU in 2002 and has since become the cornerstone of the design and construction of masonry structures in Europe.

Masonry structures, including brick, stone, and concrete block buildings, are widely used in construction due to their durability, strength, and cost-effectiveness. The Eurocode 6 sets out the requirements for the design and calculation of masonry structures, including load-bearing and non-load-bearing walls, chimneys, and arches. The code also covers the design and calculation of masonry structures subjected to seismic loads, fire resistance, and durability. The Eurocode 6 is an extensive and comprehensive code, which provides detailed information on the materials, construction methods, and design procedures for masonry structures.

One of the key objectives of the Eurocode 6 is to promote safety and quality in the construction of masonry structures. The code sets out the minimum requirements for the design and calculation of masonry structures, and provides guidance on the choice of materials and construction methods. It also covers the structural design and assessment of existing masonry structures, and provides guidance on the retrofitting and upgrading of these structures to improve their safety and performance.

The Eurocode 6 is based on a risk-based approach to the design and calculation of masonry structures. The code defines the different levels of risk associated with different types of masonry structures and sets out the minimum requirements for their design and calculation. It also provides guidance on the use of alternative design methods and construction techniques, including the use of advanced materials and technologies.

The Eurocode 6 has been widely adopted by the construction industry across the EU, and has become the benchmark for the design and construction of masonry structures. The code has helped to promote safety, quality, and efficiency in the construction of masonry structures, and has facilitated cross-border trade and investment in the construction industry. It has also helped to reduce the administrative burden associated with cross-border construction, as well as the cost of construction, through the reduction of unnecessary duplication and red tape.

It has become an integral part of the construction landscape in Europe and beyond, and will continue to play a key role in shaping the future of masonry construction in Europe and beyond.

2.2.2 Characteristic compressive strength of masonry:

The compressive strength of masonry should be established either through:
Testing as per EN 1052-1 guidelines, which can be conducted specifically for the project or obtained from a database of previous tests. The results of the tests must be presented in a table or through equation (Eurocode 6)

$f_k = K \times f_b^{\alpha} \times f_m^{\beta}$

Equation 1

fk is the characteristic compressive strength of the Masonry, in N/mm2

K is a constant and, where relevant, modifiedα , β. are constants

$f_b^a$is the normalised 1ean compressive strength of the units

f_m^β is the compressive strength of the mortar, in N/mm^2

Table 1. Values of K for use with general purpose, thin layer and lightweight mortars

Masonry unit		General Purpose mortar	Thin layer mortar (bed joint ≥ 0.5 mm and ≤ 3mm)	Lightweight mortar of density	
				600 ≤ pd ≤800 kg/m^3	800 < pd ≤1300 kg/m^3
Clay	Group 1	0.55	0.75	0.30	0.40
	Group 2	0.45	0.70	0.25	0.30
	Group 3	0.35	0.50	0.20	0.25
	Group 4	0.35	0.35	0.20	0.25
Calcium Silicate	Group 1	0.55	0.80		
	Group 2	0.45	0.65		
Aggregate Concrete	Group 1	0.55	0.80	0.45	0.45
	Group 2	0.45	0.65	0.45	0.45
	Group 3	0.40	0.50		
	Group 4	0.35			
Autoclaved Aerated Concrete	Group 1	0.55	0.80	0.45	0.45
Manufactured Stone	Group 1	0.45	0.75		
Dimensioned Natural Stone	Group 1	0.45			

2.2.3 Eurocode 8

Eurocode 8 follows a performance-based design philosophy aimed at safeguarding tructures and maintaining their functional continuity during and after earthquakes.

Seismic hazard assessment: Eurocode 8 offers a detailed process for assessing seismic hazards, which encompasses determining the level of seismic risk and calculating the seismic forces to be taken into consideration

-Design of earthquake-resistant structures: It offers comprehensive instructions on designing structures that can withstand earthquakes, covering aspects such as material selection and specification, calculation of structural strength and flexibility, and evaluation of a structure's seismic performance.

-Comprehensive framework: Eurocode 8 outlines a thorough approach for securing the stability and robustness of buildings and related structures in earthquake-prone areas..

-Determination of seismic actions: The code outlines procedures for determining the seismic forces acting on structures, including calculation of seismic loads and estimation of a structure's response to these loads.

-Seismic resistance and ductility: Eurocode 8 offers guidance on calculating the resistance and flexibility of structures against earthquakes, including the choice of suitable materials and design details for seismic-resistant structures.

-Seismic behavior of structures: It provides guidance on the assessment of the seismic behavior of structures, including the assessment of their dynamic response and the prediction of their seismic performance.

-Consideration of soil-structure interaction: Eurocode 8 considers the soil-structure interaction, a crucial factor that can greatly impact a structure's seismic behavior. The code acknowledges the importance of this interaction and incorporates it in its guidelines.

2.2.4 Ground types:

The classification of ground types in EC8 takes into account important factors such as seismicity, geology, and soil conditions to provide a comprehensive understanding of a site's seismic behavior. The different characteristics of each ground type, such as shear strength and compressibility, can have a significant impact on a structure's seismic response, and thus it is important to consider these factors in the design processIn EC8, ground types are defined based on their seismic site classification, which takes into account the site's seismicity, geology, and soil conditions. There are four main ground types defined in EC8:

Type A: Stable rock: Characterized by very low seismic wave velocity, high shear strength, and low compressibility.

Type B: Rock or stiff soil: Characterized by low seismic wave velocity, high shear strength, and moderate compressibility.

Type C: Soft soil: Characterized by high seismic wave velocity, low shear strength, and high compressibility.

Type D: Very soft soil: Characterized by very high seismic wave velocity, very low shear strength, and very high compressibility.

Table 2. Ground types via EC8

Ground Type	Description of stratigrahic profile	Parameters		
		$V_{s\ 30}$(m/s)	N_{spt}	C_u(kPa)
A	Rock or other rock-like geological formation including at most 5 m of weaker material at the surface.	>800	-	-
B	Deposits of very dense sand,gravel or very stiff clay at least several tens of metres in thickness,characterized by a gradual increase of mechanical properties with depth.	360-800	>50	>250

C	Deep deposits of dense or medium-dense sand,gravel or stiff clay with thickness from several tens to many hundreds of meters.	180-360	15-50	70-250
D	Deposits of loose-to-medium cohesionless soil(with or without some soft cohesive layers)or of predominantly soft-to-firm cohesive soil.	<180	<15	<70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5m and 20m underlain by stiffer material with v_s >800 m/s	-	-	-
S_1	Deposits consisting or containing a layer at least 10m thick of soft clays/silts with a high plasticity index(PI >40) and high water content.	<100(indicative)	-	10-20
S_2	Deposits of liquefiable soils of sensitive clays or any other soil profile not included in types A-E or S_1	-	-	-

2.2.5 Horizontal elastic response spectrum:

For the horizontal components of the seismic action the elastic response spectrulll SeT is defined by the following expressions

$0 \leq T \leq T_B: S_C(T) = a_g \; x \; S \; x \; (1 + \frac{T}{T_B} \; x \; (\eta \; x \; 2.5 - 1))$ Equation 2

$T_B \leq T \leq T_C: S_e(T) = a_g \; x \; S \; x \; \eta \; x \; 2.5$ Equation 3

$T_D \leq T \leq 4s: S_c(T) = a_g \; x \; S \; x \; \eta \; x \; 2.5 \; x \; (\frac{T_C \; x \; T_D}{T^2})$ Equation 4

$T_C \leq T \leq T_D: S_c(T) = a_g \; x \; S \; x \; \eta \; x \; 2.5 \; x \; \left(\frac{T_C}{T}\right).$ Equation 5

Se(T) is the elastic response spectrum;

T is the vibration period of a linear single-degree-of-fredom system;

ag is the design ground acceleration on type A ground

Tb is the lower limit of the period of the constant spectral acceleration branch;

Tc is the upper limit of the period of the constant spectral acceleration branch;

Td is the value defining the beginning of the constant displacement response range of the spectrum;

S is the soil factor

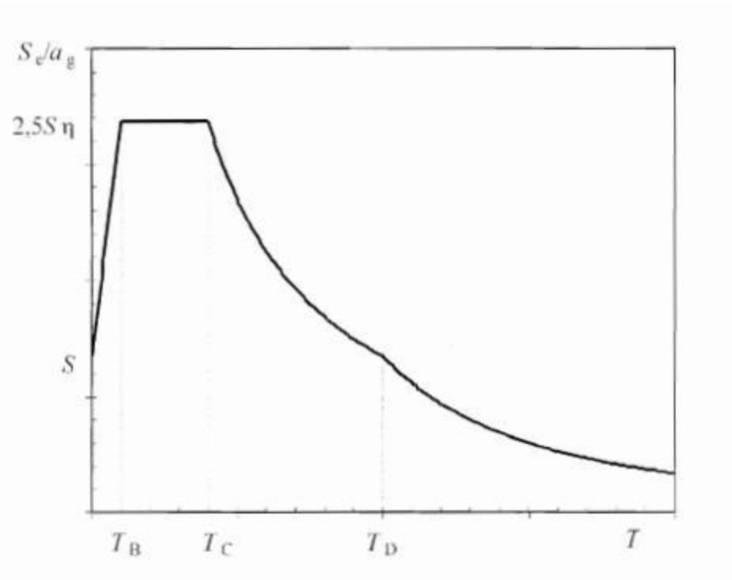


Figure 1 Shape of the elastic response spectrum

The values of the periods Tb, and Td and of the soil factor S describing the shape of the elastic response spectrum depends upon the ground type

Table 3. Values of the parameters describing the recommended Type 1 elastic response spectra

A	1	0.15	0.4	2
B	1.2	0.15	0.5	2
C	1.15	0.20	0.6	2
D	1.35	0.20	0.8	2
E	1.4	0.15	0.5	2

Table 4. Values of the parameters describing the recommended Type 2 elastic response spectra

Ground Type	S	T_B (s)	T_C(s)	T_D (s)
A	1	0.05	0.25	1.2
B	1.35	0.05	0.25	1.2
C	1.5	0.10	0.25	1.2
D	1.8	0.10	0.30	1.2
E	1.6	0.05	0.25	1.2

2.2.6 Eurocode 1:

Eurocode 1, commonly referred to as the European standard for action on structures, is a comprehensive code for the design and construction of buildings and other structures subjected to various loads and environmental conditions. This code plays a critical role in ensuring the safety, functionality, and longevity of structures, and is widely used by engineers and designers across Europe and the world.

The code outlines the procedures for determining the loads that structures must endure, including gravity loads, wind loads, snow loads, and seismic loads, and also takes into account the interactions between the structure and its surroundings, such as soil-structure interaction, to guarantee that the structure is capable of resisting these loads. The code provides guidance on load combinations, load distributions, and load effects calculation, as well as outlines the procedures for verifying the stability, stiffness, and strength of structures.

Eurocode 1 also covers structures subjected to accidental loads, such as explosions or impact, and provides a systematic methodology for their design. This includes the calculation of accidental loads and the determination of the structure's resistance to these loads.

This code is applicable to a wide range of structures, including buildings, bridges, retaining walls, and tanks, and includes provisions for special structures such as prestressed concrete, composite structures, and shell structures. Adhering to a performance-based design philosophy, Eurocode 1 focuses on ensuring that the structure performs as required during its service life, regardless of the loads or environmental conditions it may encounter.

The design process is comprehensively covered in Eurocode 1, from determining design loads and load combinations to calculating structural resistance and stability. The code also provides guidance on the selection of materials and detailing, such as reinforcement, concrete mixes, and prestressing systems, and includes provisions for durability and fire resistance to ensure that the structure maintains its integrity and performance over time.

Eurocode 1 is continuously updated to incorporate new developments in technology and the field of structural engineering. By following the guidelines outlined in this code, engineers and designers can guarantee that their structures are safe, durable, and able to meet the demands of their intended use.

2.2.7 Characteristic values of Imposed loads

This standard provides guidelines for the design of buildings and structures to ensure their safety and reliability when subjected to expected or typical loads over their lifetime. These loads include variable loads such as the weight of people or furniture, as well as exceptional loads such as wind or snow.

The characteristic values of imposed loads are determined through load tests or by using data from previous similar projects. EC1 provides minimum values for each type of load, but these can be adjusted based on the specific circumstances of the project. The use of these characteristic values in the design of the structure is important as it ensures that the structure will be capable of safely and reliably supporting these loads over its lifetime.

Areas in residential, social, commercial and administrative buildings shall be divided into categories according to their specific uses.

Table 5. Categories of use via EC1

Category	Specific use	Example
A	Areas for domestic and residential activites	Rooms in residential buildings and houses ;bedrooms and wards in hospitals ;bedrooms in hotels and hostels kitchens and toilets.
B	Office Areas	
C	Areas where people may congregate (with the exception of areas defiend under category A,B and D[1])	C1:Areas with tables etc. e.g areas in schools,caffes,restaurants,dinning halls ,reading rooms,receptions.
		C2:Areas with fixed seats. e.g areas in churches, theatres or cinemas, conference rooms, lecture halls,assembly halls, waiting rooms, railway waiting rooms.
		C3:Areas without obstacles for moving people. e.g areas in museums, exhibition rooms etc. and acces areas in public and administration buildings, hotels, hospitals, railway station forecourts.

		C4: Areas with possible physical activities, e.g dance halls, gymnastic rooms, stages.
		C5 : Areas susceptible to large crowds e.g in building for public events like concert halls, sports halls including stands, tarraces and access areas and railway platforms.
D	Shopping Areas	D1: Areas in general retail shops.
		D2: Areas in department stores.

Table 6. Imposed loads on floors , balconies and stairs in buildings via EC1

Categories of loaded areas	q_k kN/m^2	Q_k kN/m^2
Category A		
Floors	1.5 – 2.0	2.0 – 3.0
Stairs	2.0 – 4.0	2.0 – 4.0
Balconies	2.5 – 4.0	2.0 – 3.0
Category B	2.0 – 3.0	1.5 – 4.5
Category C		
C1	2.0 – 3.0	3.0 – 4.0
C2	3.0 – 4.0	2.5 – 7.0(4.0)
C3	3.0 – 5.0	4.0 – 7.0
C4	4.5- 5.0	3.5 – 7.0
C5	5.0 – 7.5	3.5 – 4.5
Category D		
D1	4.0 – 5.0	3.5 – 7.0 (4.0)
D2	4.0 – 5.0	3.5 – 7.0

Where necessary qk and Qk should be increased in the design for stairs and balconies depending on the occupancy and on dimensions. For local verifications a concentrated load Qk acting alone should be taken into account. For concentrated loads from storage racks or from lifting equipment, Qk should be determined for the individual case.The concentrated load shall be considered to act at any point on the floor, balcony or stairs over an area with a shape which is appropriate to the use and form of the floor.

2.2.8 KTP

They are a domestically relevant set of guidelines and regulations that work in conjunction with the Eurocodes to dictate the design and construction of buildings and other structures. These standards, which are maintained and monitored by the Ministry of Infrastructure and Energy, cover various aspects of building design such as structural design, fire protection, accessibility, energy efficiency, among others.

During various structural calculations and analyses, it is necessary to know some other very important characteristics such as tensile strength $\mathbf{f_t}$.

If we refer to the Albanian norms, KTP-9-78, the design resistance of the masonry in pressure, concerning the brand of bricks and mortar, is displayed in the table below.

Table 7. Relation between class of bricks and mortar

Nr.	Class of Brick	Class of Mortar N/mm2						
	N/mm^2	10.0	7.5	5.0	2.5	1.5	0.4	0.0
1	15	2.2	2.0	1.8	1.5	1.35	1.2	0.8
2	10	1.8	1.7	1.5	1.3	1.1	0.9	0.6
3	7.5	1.5	1.4	1.3	**1.1**	**0.9**	0.7	0.5
4	5.0	-	1.1	1.0	0.9	0.75	0.6	0.35

Regarding the calculation of the modulus of elasticity of masonry, we will refer to the expression:

In Limit State

E = 0.5 x $\alpha\ x\ R_n$ Equation 6

Deformation calculation

Where Rn is the compressive design resistance of the wall and

In KTP-9-78, the factor "α" is named the "elastic characteristic of masonry" and is derived from the relation σ-ε.

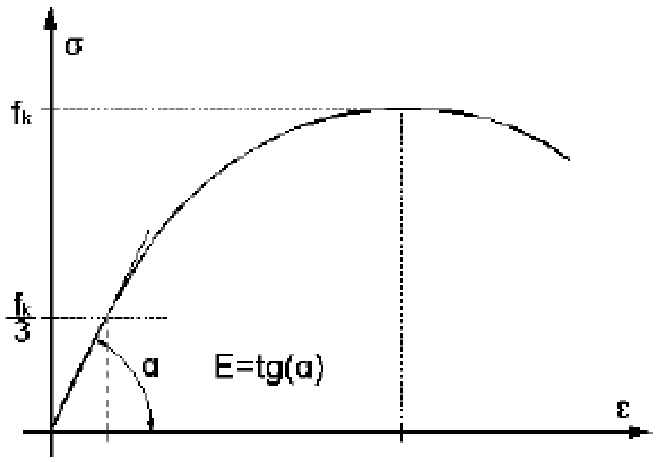


Figure 2. Elastic characteristic factor of masonry

The value of this coefficient is taken from the brand of mortar and the type of wall

Table 8. Values of Elastic characteristic of masonry factor

Nr	Type of Wall	Elastic characteristic α factor of masonry			
		100-50 kg/cm2	25 kg/cm2.	4 kg/cm2	0 kg/cm2
1	Brick walls, concrete blocks volumetric weight up to 1800 kg/m3	1000	750	500	350

2	Brick walls with vertical holes	2000	1500	1000	-
3	Brick walls with horizontal holes	1500	1000	750	-
4	Brick walls, concrete blocks volumetric weight up to 1800 kg/m3	2000	1000	750	-

2.3 Literature review

There are different types of structural analysis we can use for masonry structures and one of them is stability analysis. This involves evaluating the structure's ability to resist external loads and forces, such as wind, earthquakes, and lateral loads. Stability analysis can be performed using a variety of methods, including linear and nonlinear analysis, finite element analysis, and limit analysis.

Nonlinear static procedures, also known as nonlinear pushover analysis, is a type of structural analysis used to evaluate the behavior of masonry structures under extreme loads, such as earthquakes. These methodologies are simplified procedures in which the problem of estimating the maximum expected reaction (response), arising from the occurrence of a given seismic event, is traced back to the study of a nonlinear system with a single degree of freedom, equivalent to the model with “n” degree of freedom., which represents the real structure (A.Shibata dhe Mete A, Sozen 1976).

A common feature of these procedures is that they rely on the use of nonlinear (push) static analysis to then characterize the seismic resistant system through capacity curves: "static" analysis as external forces are applied statically to the

structure and "nonlinear" due to the accepted pattern of behavior for resistant elements of the structure.
These curves are intended to represent the overlap of the "hysterexis" cycles produced during the earthquake and can be considered as an indicator of the pos-elastic behavior of the structure.

While for the methods of elastic analysis, nonlinear behavior is taken into account by introducing the structure factor "q", nonlinear static analysis allows understanding of the evolution of the structural response as the only elements evolve in the nonlinear field, providing information on the distribution of inelasticity demand.

The curve obtained from the thrust analysis (which will then be converted to a capacity curve, taking into account the characteristics of the system equivalent to a degree of freedom) conventionally reports the resulting shear force trend at the base, concerning the horizontal displacement of the structure checkpoint. Each point of the curve can be associated with a specific damage status of the whole system, and therefore it is possible to associate the expected degree of functionality and the corresponding damage at certain levels of movement. The curve is obtained by applying the "pushover" analysis, which predicts the determination of a predetermined distribution of forces, increased statically and monotonously.
The distribution is kept unchanged even beyond the breaking point. Analyzes can be performed in force control or through a mixed force-displacement control. The distribution of the applied load is intended to represent the distribution of inertial forces caused by the seismic event. The profiles proposed by the technical norms are those of distribution in proportion to the static forces (first way), and those proportional to the masses (second way). In particular, in the case of regular structures, the distribution according to the first method is accepted, to better capture the response (reaction) of the structure in the elastic field and the second in the non-linear field. The capacity" provided by the structure must be compared, for the purpose of seismic control, with the "demand" required by the external force, i.e. by a specific seismic event.
There is the use of nonlinear procedures for the seismic assessment of the former Italian Embassy (Angjeliu , Baballeku 2013) . A study that investigated the seismic behavior of a building using a nonlinear static procedure and finite element

modeling. The building was subjected to increasing horizontal loads in two different directions, and the results showed that the out-of-plane behavior was critical in both cases, with most of the walls in the central part of the building being highly sensitive to this behavior. The sensitivity was found to depend on the type of floor and floor-wall connection, with properly connected masonry buildings having the capacity depend on in-plane behavior. The study proposed that fixing the out-of-plane behavior through measures such as ring beams, concrete floor, better anchorage, etc. may not be enough to ensure the building can sustain the expected peak ground acceleration. The new improvements in the finite element model allowed for the non-linear behavior of the structure to be described by the relationship between the base shear force and roof displacement.

Alongside softwares like SAP2000 we also have another programs like 3-Muri software used by (Korini, Bilgin 2012) in their study for a new modeling approach in the pushover analysis for structure the same as ours . The proposed line finite element software uses an axis to represent the line element. It models a wall with a width of "b" and thickness of "s", which is comprised of three parts: the axial deformability is located in the two end elements (1 and 3) with infinitesimal thickness "D", and they are infinitely rigid against shearing actions. The central body, with height "h", contains the tangential deformability and is non-deformable in terms of axial and flexural behavior. To fully represent the cinematic model for the macroelement, the three degrees of freedom for the nodes i and j and the interface 1 and 2 must be taken into account.

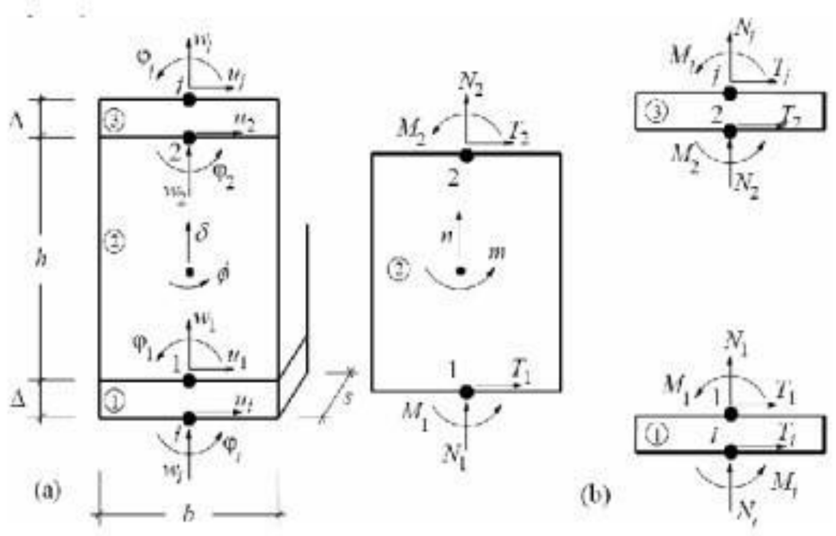


Figure 3. 3 Muri finite element view

Retrofitting refers to the upgrading of existing buildings to enhance their ability to withstand various loads, including seismic, wind, and thermal forces. This process involves making improvements to both the structural and non-structural components of the building, with the aim of extending its service life and improving its performance during adverse conditions. Over time, a wide range of retrofitting techniques have been developed to address the shortcomings of older structures and enhance their overall safety and durability. These techniques can range from straightforward modifications to more complex strengthening methods, and they vary based on the type of building and the loads it is exposed to.

Some of them are mention in different studies, one of them is the structural assessment of the ottoman mosques in our country Albania (Mustafaraj 2012).The technique involves injecting mortar or fluid resin into the wall through holes drilled in the external parameters of the wall. The aim of the injection is to fill existing cavities and internal voids, seal possible cracks, and increase the continuity of the masonry, which enhances its mechanical properties. The technique is usually applied to walls that present a diffuse presence of voids, incoherence of the rubble filling material, and visible cracks in the external parameters. There is another technique which involves adding high-performance materials such as FRP (fiber reinforced polymer), steel, wood, or plastic to the exterior of masonry structures to improve their strength and resilience against earthquakes and compressive stress. This technique is applied locally (in strips) or to the entire surface of the structure (grid reinforcement), with the masonry being connected to the reinforcement using epoxy resins or mortar. However, it is important to note that the technique requires a regular surface of the masonry to be effective. In arches and vaults, reinforcement can be added between the extrados and an additional masonry layer. The main goal of this method is to increase the structure's ductility and resistance to tension by adding a material that can withstand tension

There is also a study done by (Roselena Sullaa , Michele D’Amatoa, Rosario Gigliottib, Domenico Liberatoreb).The paper presents a concise overview of strength models for piers and spandrels using nonlinear analysis techniques, with a specific emphasis on the Italian design code. The study aims to explore the influence of masonry strength and floor modeling on the overall response to lateral loads. To conduct the research, a case study featuring three stories was analyzed, considering three different masonry strength values and three diaphragm behaviors. Vertical loads were adjusted accordingly to each type of floor.The research findings indicate that all pushover analyses showed a mixed response mechanism, with bending moment being the main cause of failures occurring in both the spandrels and piers at all levels. The study highlights that masonry strength plays a vital role in determining the lateral response of the case study. Specifically, in the case of ISM, the floor behavior has no effect on the lateral response, while in the case of BM, diaphragm behavior does influence the response if the masonry is of good quality. Additionally, the impact of floor stiffness on the global response increases with higher masonry strength values. Furthermore, in all cases, the global failure is not concentrated at one story but distributed across the walls.

Another case of reinforcing techniques is presented by (Ervin Paçi, Altin Bidaj, Hektor Cullufi) that highlights a significant issue with the flexural strength of beams in a structure. To address this problem and rehabilitate the structure, the authors suggests four different approaches. The first approach is to increase global capacity, which can be achieved through the addition of cross braces or new structural walls. The second approach is to reduce seismic demand by using supplementary damping and/or base isolation systems. The third approach is to increase the local capacity of structural elements, which involves selectively upgrading the deformation/ductility, strength, or stiffness of individual components while acknowledging their existing capacity. The fourth approach is selective weakening retrofit, which changes the inelastic mechanism of the structure.The authors then goes on to explain why the third approach is the most suitable in this particular case. The structure has limitations in terms of its stiffness for acceptable performance allowed drifts, making the first and fourth approaches unfeasible. Architectural requirements also make it impossible to add walls or braces, ruling out the first approach. Meanwhile, the use of seismic base isolation systems is too expensive, making the second approach unsuitable.To reinforce the beams and columns, the author has opted for longitudinal carbon fiber strips in the middle and supports of the beams, and carbon fiber web confinement for the columns.

There are studies for the characteristics of unreinforced masonry structure more specifically by (Ornela Lalaj, Yavuz Yardim).Unreinforced material comprises two phases, namely bricks and mortars, and their properties collectively determine the quality of the masonry. The quality of bricks is primarily influenced by the type of raw material and production process. Conversely, evaluating the quality of mortars is more complex as it is dependent on its mix design, which can vary significantly from batch to batch. Standards have been established for assessing the properties of bricks and prisms, but determining the compressive strength of current mortars has always presented a challenge. The experimental program involved determining the compressive strength of both bricks and mortars using a compressive strength machine. From these results, the properties of masonry prisms were inferred. The study involved obtaining brick samples from five cities and mortar samples from three locations. As anticipated, variations in results were observed for both the brick and mortar tests, even within the same sample location. According to the theoretical values the compressive strength of bricks is expected to be within the range of 7.5 to 10 MPa. However, empirical evidence shows that the actual compressive strength of bricks is considerably higher, ranging from 12.71 to 28.36 MPa. This suggests that the quality of bricks used in construction is better than what is assumed in the design. Nevertheless, it should be noted that the series had a relatively high standard deviation, which could have implications for design and analysis. The compressive strength of mortars typically exhibit a moderate range of compressive strength, with values typically falling between 2.42 to 6.21 MPa.The Eurocode 6 equations are commonly used to determine the compressive strength, elastic modulus, and shear modulus of masonry. According to these equations, the compressive strength of masonry is generally low to medium, with a range of 4.25 to 7.55 MPa.

There is also the study done by (Turgay Cosgun Oguz, Uzdil Baris, Sayin Kamil, Kerem Zengin). The purpose of it is to evaluate the seismic performance of a masonry load-bearing building through numerical analyses. Linear, nonlinear, and kinematic methods were used to identify the failure mechanisms and seismic strength of the structure. Results showed that the building had insufficient seismic strength, leading to a proposed strengthening practice that utilized FRP composites for the structural members. The strengthened structure was analyzed using linear analysis, which indicated that stress and damage levels were within the local earthquake code's limit values for three different earthquake levels. Shear stresses were far below the critical limit for a DD-3 level earthquake. Nonlinear analysis revealed that the strengthened building met performance criteria for target ground motion levels, as measured by story drift ratios. Kinematic failure mechanisms were also identified, with overturning observed under DD-3 class ground motion and lateral bending and vertical bending mechanisms observed for all target ground motions. The study demonstrated the effectiveness of pushover analysis for evaluating masonry structures and suggested that comprehensive analyses, including kinematic analyses, were necessary for identifying local behaviors in these structures. Linear and nonlinear analyses were both found to be important for accurately assessing the behavior of masonry materials, which are generally assumed to be elastic in linear analysis and plastic in nonlinear analysis. This study provides a practical and efficient methodology for evaluating and retrofitting masonry structures, and highlights the need for careful analysis to ensure these structures can withstand seismic events

CHAPTER 3

3.METHODOLOGY

3.1 Introduction

The methodology used in this study was chosen in order to determine if the building may need reinforcing intervention, in certain areas of it such as the walls or foundations. During the modeling process of the building, the dimensions of it, the type of materials used were taken into consideration in order to carry out the analysis. They were carried out with 3Muri software and consists of analysis of own oscillations, statistical load analysis and the most important non linear analysis (push over).The results of which will determine if the building will need reinforcing intervention or not.

3.2 Technical Project

Below are presented the geometric dimensions of the floors (relevant floor plans), referring to the measurements (on-site updates). The structural support scheme of the building is with silicate brick retaining masonry, with a thickness of 38cm on the first three floors and 25cm on the lasttwo floors**.** The foundations of the building are continued with butoconcrete and with socle silicate brick. The floor slabs of the building are pre-prepared, with joist/ slabs thrown in one direction. In the areas of support of the floor slab in the retaining walls are placed belts as high as the height of the floor slab and as wide as the width of the wall. The maximum distance between the transverse retaining walls is about 8m, which is greater than the 4.5m condition of KTP or 7.0m of Eurocode.

The building is geometrically regular both in height and plan. At the height, the stiffness undergoes an immediate decrease as a result of the passage of the walls from 38cm to 25cm.

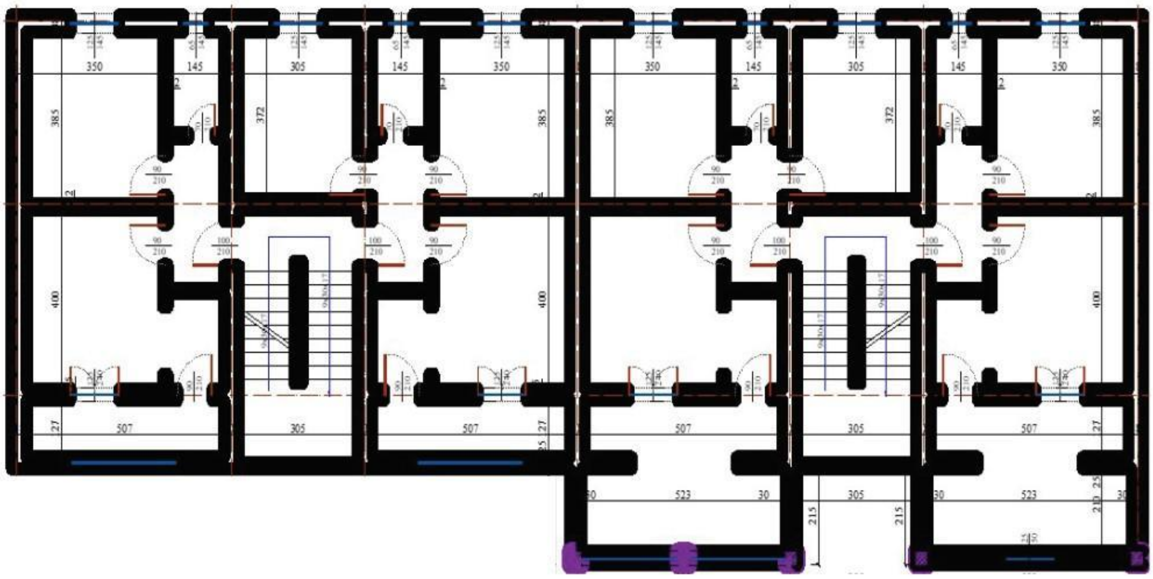


Figure 4. Ground floor planimetry

3.3 Existing Conditions

A. Stural interventions in the building have generally been few and they have been made in some areas. On the ground floor they consist of turning two or three windows into doors, although few in number (mainly at the entrance of the west side, which result in two additional volumes). In almost all floors there are interventions in closing the existing balconies with masonry positioned in the front part of the building.

B. The masonry of the building is walled with silicate bricks with poor cohesion of the mortar bond with the silicate brick.;

C. From the on-site verifications and measurements performed with topographic equipment (Total Station), no sagging and deformations were found in the building. Evaluations have been made through various engineering-technical interpretations. In the absence of information on periodic measurements or measurements at the stage of completion of works on the actual condition of the building, it is very difficult to judge exactly on this assessment.

D. Dimensions of the structural elements: From the on-site verifications, the retaining walls are 38cm thick on the first three floors and 25cm on the last two floors. The layers on the floor slab, referred to investigations on-site, have a thickness of 7cm. In different environments of the building, their relocation is evidenced and in a few other cases, they are rebuilt on the existing layers.

3.4 Evidenced Structural problems

During the on-site inspection it was found that there are structural damages in the building, caused mainly by the recent seismic actions. The abovementioned building consists of 5 floors without a basement. Access to the apartments, on each of the floors, is provided by two stairs, positioned on the north side of the building.

The supporting structures of the existing building are made of solid brick (silicate) masonry, the thickness of which varies according to the floors; specifically 38cm on the ground floor, the first, the second and 25cm on the two other floors. The interfaces are made of pre-prepared elements (with pre-made reinforced concrete panels), which are contoured on the perimeter by a connecting strip of reinforced concrete. In the upper levels of the openings (doors-windows) lintels are made of reinforced concrete, with width according to the respective widths of the walls. The organization of the structural elements is the same on almost all floors. The ground floor has a small addition in volume compared to other floors, realized with the beam-column system.

Being essential for the stability of the building, are damages of structural character and ongoing we have briefly presented some of them:

a. **Large cracks (openings > 5mm) in retaining masonry**.

Such damages are noticed both in the internal masonry of the building and from the outside perimeters. The cracks in the masonry are transcendent and in many cases of considerable length.

There are noticed two categories of cracks, those that have a diagonal direction (caused by shear force) and the horizontal ones (caused by bending moment and normal force). Cracks in the masonry in certain places reach up to 10-12mm.

Figure 5. Cracks in the building with dimension more than 5 mm (1-4)

b. Cracking in the elements of reinforced concrete

Cracks were mainly found in the stair rampsas well as in some of the reinforced concrete lintels of the doors or the interior spaces of the building, etc. In most cases, what is noticed is that the size of these cracks is moderate.

Figure 6. Presence of cracks in ramps

c. Cracks in small size (opening t <5mm)

Damages of moderate size, are found spread in almost all structural and non-structural elements of the building. Also, cracks are noticed in the joints which are made with pre-prepared panels reinforced concrete (as a result of their low stiffness).

Figure 7. Cracks with dimension less< 5 mm near doors, windows

3.5 Numerical Analysis

For the evaluation of the supporting structure of the building *"Building 5kT, P.152",* we will rely on numerical calculations with finite elements (M.E.F) of 3-dimensional models, realized by commercial software which offers the possibility of calculating the structures with supporting masonry. More specifically, will be used the numerical calculations performed in the software *3Muri-STA DATA*, which offers calculation possibilities with special specifications for such structures. It should be noted that the software *3Muri STA DATA*, is known as one of the "leader" calculation codes for seismic (pushover) and static analysis of masonry structures.

3.6 Construction Materials

Since the object taken into study includes a complex structure in terms of typology, in the use of construction materials, before the realization of numerical models, an analysis of the physical-mechanical characteristics of construction materials was made, based on the preliminary information provided. The core information is provided by the "Laboratory Analysis of Construction Elements", conducted by Altea & Geostudio 2000.

Referring to the above, the physical-mechanical characteristics of construction materials according to the relevant structural elements, are accepted:

Concrete for structure :

Lintels and ramp ladders are made of reinforced concrete, made of concrete M-200 with characteristic minimum cubic resistance Rck = 20 N / mm2.

Connecting lintels and beams, made of concrete M-150-200, with characteristic minimum cubic resistance Rck = 15-20 N / mm2

Steel for RC :

Structures of reinforced concrete are made of Ç-3 steel, with calculated resistance

Rs = 210N / mm2 .

Other :

According to the materials test report, the structure of solid silicate bricks has the following characteristics:

Compressive strength between. of brick: fb = 5.9 N / mm2

Compressive strength between. of mortar: fj = 1.77N / mm2

Referring to the recommendations of the Eurocode:

The compressive strength of the masonry is f_m=19Mpa

The characteristic shear strength of masonry is f_{vko}=0.1 Mpa

I will finally accept:

The characteristic compressive strength of masonry is: fm=17.5daN/cm2

Elasticity Module: E= 2800 N/mm2

Cutting Module: G= 1100 N/mm2

The characteristic shear strength of masonry is: fvko=1daN/cm2

3.7 Loads and Actions

During the calculations of the numerical model with finite elements, the following loads have been taken into account:

3.7.1 Permanent loads Gi

Personal weight of structural elements which are automatically taken into account by the program referring to the volume of the element and the respective volume weight; respectively of concrete *γb = 25 kN / m3*, solid brick masonry *γm = 18 kN / m3*, brick masonry with holes γm = *8-12 kN / m3,* etc. The own weight of the pre-prepared panels, erected on the roof of the building, is taken a load from the own weight of about 4-5 kN / m2 (in the absence of accurate detailed information, it is estimated with reserve, accepting an equivalent weight for the panel of 16-18cm);

The load from its own weight of the leveling layers in the meantime has been taken in advance at about 2.5 kN / m2 (estimated with reserve in the absence of accurate information)

3.7.2 Temporary loads Qi

The existing structures are 5kT building, the standardized loads of which, for the function that they have, referring to the norms, is 1.5kN / m2. Terrace area, according to the norms, is foreseen to be a normalized load of about 0.75 kN / m2.

3.7.3 Special loads (Seismic Actions Ed)

The design codes of our country took into consideration that seismic actions have changed over the years, and this is in reference to the necessity for increasing safety. Although the use of "design spectra" for seismic analysis has previously existed in the design codes of our country, the values in them have been much lower compared to today. If we compare the elastic spectra for the design codes, it can be seen that the increase in the values of the spectral accelerations between periods is distinct.

Based on the above, in the absence of new norms (revised), also referring to the purpose of this study, the given recommendations in the seismic study should also take into account the increase in seismic demand.

In order to meet the contemporary requirements in the field of safety against seismic actions, we have accepted that the assessment of the supporting structure for the object in question will be based on the response spectrum according to Eurocode 8 (EN 1998-1 - Design of structures for earthquake resistance), considering the following characteristics :

PGA = 0.25g (0.248g)Factor of importance γ_i = 1.0, The spectrum type is Type. 1 (M>5.5),

The behavioral factor is usually taken commensurate to the post-elastic behavior of the ductile elements of the structure and the presence of the largest group of elements that mostly contribute to the seismic resistance. Given that the normative recommendations

(EN 1998-1) display a value of 1.5-2.5 for structures with unreinforced masonry and 2-3 for structures with tight masonry. We have accepted a value of the behavior factor q = 2.0 given the complexity of the structure.

During the structural analysis of this unit, two seismic loads were taken into account: seismic load according to Eurocode (from the seismic study) and seismic load according to September and November earthquakes (based on IGJEUM records and obtained in spectral form). For the purposes of this report the earthquake spectra are represented by the type 2 spectrum according to Eurocode, with ground acceleration a_g = 0.15g, and type B ground.

3.8 Load Combinations

Main combination (normal situation)

$$F_d = \sum \gamma_g G_k + \gamma_p P_k + \psi_{1Q1} \cdot Q_{k1} + \sum \gamma_q \psi_{0i} Q_{ki}$$

Equation 7

Special combination (seismic situation)

$$F_d = \sum G_k + P_k + A_{Ed} + \psi_{21} \cdot Q_{k1} + \sum \psi_{2i} Q_{ki}$$

Equation 8

For the borderline condition of the service, the following combinations have been considered:

Characteristic combination

$$F_d = \sum G_k + P_k + Q_{k1} + \sum_{2}^{n} Q_{ki}$$

Equation 9

Frequent combination

$$F_d = \sum G_k + P_k + \psi_{1k} \cdot Q_{k1} + \sum_{2}^{n} \psi_{2i} Q_{ki}$$

Equation 10

Almost permanent combination

$$F_d = \sum G_k + P_k + \sum_{1}^{n} \psi_{2i} \cdot Q_{ik}$$

Equation 11

Relevant combination coefficients are taken with reference to normative recommendations.

3.9 Geometry of the model

As mentioned above, for the assessment of seismic response for buildings, are used the numerical calculations performed in software 3 Muri-STA DATA v10.0.2, which offers the calculation possibilities with special specifications. In this software, the 3-dimensional numerical model is subjected to static and nonlinear static analysis (pushover).

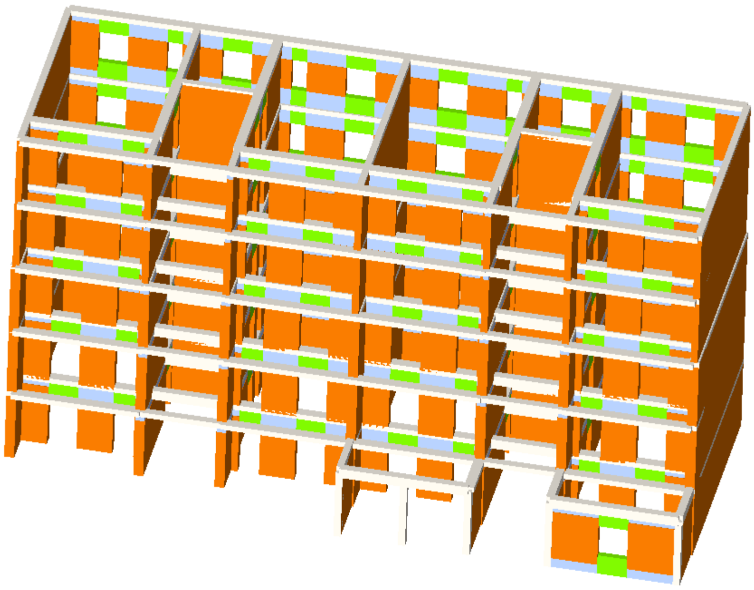


Figure 8. Pushover 3-dimensional model in 3D Wall, for nonlinear static Pushover analysis.

The modeling of buildings is performed by inserting walls that are discretized in macro elements, representatives of the "walls" (elements in orange color) of the masonry and "architectural walls" (green elements, above and below the window space) deformable; rigid joints are shown in the part of the masonry which are usually less prone to the seismic damage.

"walls" and " architraves walls" are usually adjacent to the openings, and the solid joints represent the connecting elements between the walls and the architraves.

Model nodes are three-dimensional nodes with 5 degrees of freedom (the three components of displacement in the global reference system and rotations around the X and Y axes) or two-dimensional nodes with 3 degrees of freedom (two displacement and rotation in the wall plan). The three-dimensional ones are used to allow the transfer of actions, from one wall to the second, positioned transversely to the first. The two-dimensional joints have degrees of freedom only in the wall plan, allowing the transfer of forced states between different points of the wall.

Horizontal structures are modeled with three-node shell elements connected to three-dimensional nodes, they can be loaded perpendicular to their plan from accidental and permanent loads; seismic actions load the floor along the direction of the middle plan. For this reason, the plate element is given with an axial (membrane) stiffness, but without bending stiffness, as the main mechanical behavior to be evaluated is under horizontal load due to the earthquake.

3.10 Analysis of own oscillations

This section presents the results of the analysis of own oscillations for the building. In order to have the highest possible mass participation, 22 oscillations of oscillations were taken into account during the analysis. The following are the tables of the sum of the effective modal measures taken into account in the calculation and of the mass participation for each oscillation tone.

Table 9. Sum of the effective modal measures

Modo	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
1	0.41967	1,490,324	67.21	4,970	0.22	100	0.00
2	0.33391	26,033	1.17	1,558,556	70.29	5,106	0.23
3	0.28087	249,982	11.27	55,509	2.50	63	0.00
4	0.17747	201,143	9.07	10	0.00	12	0.00
5	0.16745	740	0.03	109	0.00	3,236	0.15
6	0.14979	33,291	1.50	9	0.00	89	0.00
7	0.13063	168	0.01	363,983	16.42	154,038	6.95
8	0.12520	9,983	0.45	3,999	0.18	1,563	0.07
9	0.11701	10,172	0.46	145	0.01	519	0.02
10	0.11432	59,923	2.70	153	0.01	166	0.01
11	0.11303	926	0.04	3,695	0.17	24,347	1.10
12	0.10939	561	0.03	32,588	1.47	53,160	2.40
13	0.10460	2,038	0.09	452	0.02	6,457	0.29
14	0.09900	142	0.01	31,400	1.42	1,276,908	57.59
15	0.09613	1,856	0.08	2,365	0.11	28,747	1.30
16	0.09565	2,580	0.12	270	0.01	19,420	0.88
17	0.09373	4,977	0.22	2,436	0.11	32,839	1.48
18	0.09220	11,478	0.52	137	0.01	2,013	0.09
19	0.09153	102	0.00	2,096	0.09	16,827	0.76
20	0.08670	51,749	2.33	3	0.00	3	0.00
21	0.08632	2,428	0.11	917	0.04	18,348	0.83
22	0.08593	57	0.00	2,602	0.12	50,223	2.27

The above periods are compared with the periods calculated according to SSH EN 1998-1

Referring to SH EN 1998-1 the periods of oscillations in each direction are calculated by the following formula:

$T = C_t \times H^{3/4}$, (Equation 12) where H- is the height of the building while C_t for retaining masonry buildings is calculated according to the expression $C_t = 0.075 \sqrt{A_c}$ (Equation 13) where A_c is the surface of the retaining walls according to the direction of action of the load.

Periods of building oscillation models calculated with the reference expression differ from those based on numerical-computer models. This change relies mainly on the assumptions made while considering the reduction of stiffness for the calculation and the structural scheme of the building. The stiffness accepted with cracks in the computer model is closer to the existing condition of the building and gives a more realistic estimation to the displacements.

3.11 Statistical Load Analysis

Verification at the final limit for static loads is performed according to the provisions set out in the norms.

The following load combination is:

1.5 Gk + 1.5 Ψ Qk Equation 14

where: Gk: permanent loads Qk: accidental loads Ψ: combination coefficient for variable loads: Ψ = 1 for roofs and the first two floors most loaded; Ψ = 0.9 - 0.8 - ... - 0.5 for subsequent floors . Bending of masonry the bend of masonry is defined as the ratio "h0 / t" in which:

h0: free length of free wall equal to “r • h”;

t: wall thickness.h: internal height of the floor;

ρ: lateral limiting factor.Tenderness verification is satisfactory if the following is proven:

H0 / t <20

3.12 Non-linear Analysis (Push-Over)

To carry out the necessary checks about the building in question, it was decided to proceed with the execution of a non-linear static analysis

The required verifications take the form of a comparison between the capacity curve for the different conditions provided and the displacement requirement, provided by the norms.

The capacity curve is identified using a displacement diagram with the maximum shear force at the base. According to the norm [Eurocode 8], the loading conditions to be examined are of two types:

• Distribution of force in proportion to the mass

$$F_i = \frac{m_i}{\sum_i m_i}$$ Equation 15

Distribution of forces proportional to the product of the masses with the deformation for the first vibration mode. This calculates the value of the maximum displacement at the base of the building generated by that force distribution.

This displacement value constitutes the final value of the building. The displacement considered for tracing the capacity curve is that of a point in the building called the control node. The norm requires the tracing of a bi-linear capacity curve of an equivalent system (SDOF).

Determining the curve about the equivalent system allows us to determine the period by which the maximum displacement required by the earthquake can be obtained, according to the spectra presented in the standard

The norm (Eurocode8) determines an accidental eccentricity of the center of mass, equal to 5% of the maximum size of the building in a direction perpendicular to the earthquake.

Based on the type of building and design solutions, the seismic load condition to be considered will be:

Seismic load: identifies which of the two types of distributions (proportional to the mass or the first oscillation mode) is taken into consideration.

Direction: Identifies the direction along which the structure (X or Y of the global system) is loaded by the seismic load. To identify the most unfavorable state of seismic load, it was decided to perform separate analyzes according to the type of load, earthquake direction and accidental eccentricity.The analysis table is given below .

Table 10. Analysis results according to the type of load, earthquake direction and accidental eccentricity.

Nr. analysis	Direction of seismic action	Proportional seismic load	Eccentricity a [cm]	Floor	Control node
1	+X	Mass	0.0	5	82
2	+X	1° mode	0.0	5	82
3	-X	Mass	0.0	5	82
4	-X	1° mode	0.0	5	82
5	+Y	Mass	0.0	5	82
6	+Y	1° mode	0.0	5	82
7	-Y	Mass	0.0	5	82
8	-Y	1° mode	0.0	5	82
9	+X	Mass	59.7	5	82
10	+X	Mass	-59.7	5	82
11	+X	1° mode	59.7	5	82
12	+X	1° mode	-59.7	5	82
13	-X	Mass	59.7	5	82
14	-X	Mass	-59.7	5	82
15	-X	1° mode	59.7	5	82
16	-X	1° mode	-59.7	5	82
17	+Y	Mass	143.3	5	82
18	+Y	Mass	-143.3	5	82
19	+Y	1° mode	143.3	5	82
20	+Y	1° mode	-143.3	5	82
21	-Y	Mass	143.3	5	82
22	-Y	Mass	-143.3	5	82
23	-Y	1° mode	143.3	5	82
24	-Y	1° mode	-143.3	5	82

CHAPTER 4

4.RESULTS AND DISCUSSIONS

4.1 Statistical load analysis results

These checks were performed on each wall of the structure, in three main sections (bottom, center, and top).

The values of normal resistance stresses will only be calculated if the bending controls and the eccentricity of the loads are met

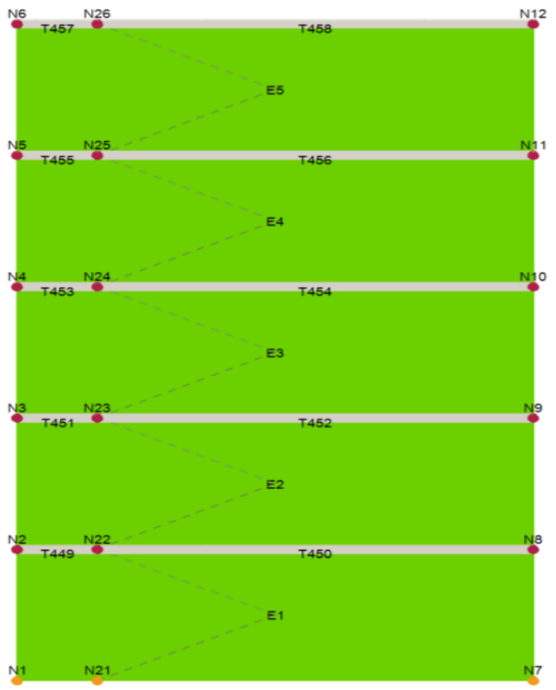
In the following images are the details of the verification for some of the walls.

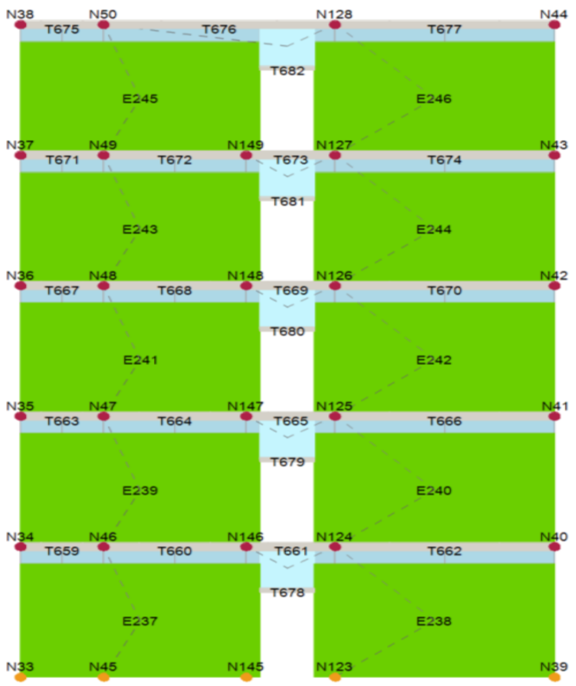
For wall 4,2,8,6,3,5 not all verifications are satisfied .

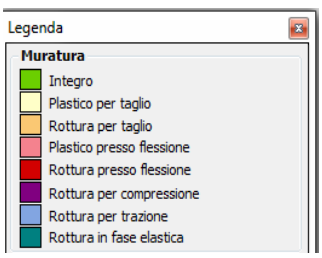
Table 11. Verifications for static loads, for each of the walls (EC requirements)

pianta

Parete	Maschi rotti	Nd/Nr Max	h0/t Max	e1/t Max	e2/t Max
4	2	1.13	12.60	0.063	0.063
2	10	1.27	12.60	0.181	0.079
8	1	1.12	12.60	0.186	0.076
6	1	1.06	12.60	0.185	0.075
3	0	0.66	12.60	0.190	0.077
5	0	0.96	12.60	0.184	0.076
1	0	0.78	12.60	0.190	0.077
7	0	0.69	12.60	0.063	0.063
9	0	0.79	12.60	0.187	0.079
10	0	0.77	12.60	0.063	0.063
11	0	0.74	12.60	0.063	0.063
13	0	0.61	12.60	0.063	0.063
14	0	0.49	12.60	0.063	0.063


N6
N26
N12
T457
T458
E5
N5
N25
N11
T455
T456
E4
N4
N24
N10
T453
T454
E3
N3
N23
N9
T451
T452
E2
N2
N22
N8
T449
T450
E1
N1
N21
N7


N38
N50
N128
N44
T675
T676
T677
T682
E245
E246
N37
N49
N149
N127
N43
T671
T672
T673
T674
T681
E243
E244
N36
N48
N148
N126
N42
T667
T668
T669
T670
T680
E241
E242
N35
N47
N147
N125
N41
T663
T664
T665
T666
T679
E239
E240
N34
N46
N146
N124
N40
T659
T660
T661
T662
T678
E237
E238
N33
N45
N145
N123
N39


Legenda
Muratura
Integro
Plastico per taglio
Rottura per taglio
Plastico presso flessione
Rottura presso flessione
Rottura per compressione
Rottura per trazione
Rottura in fase elastica

Intact
Plasticfor cutting
Breakage by cutting
Bending plastic model
Breaking at bending
Breakage due to compression
Breaking by traction
Breakage in the elastic phase

Figure 9. Legend of masonry statistical load analysis results

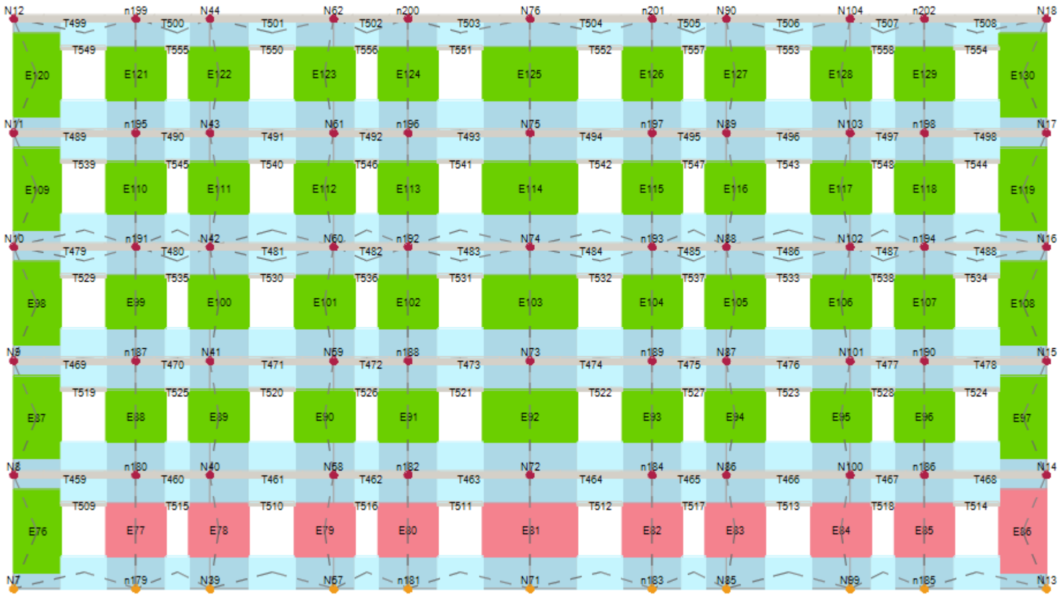


Figure 10. Verification status, for wall P.6 + P.7 (requirements according to EC)

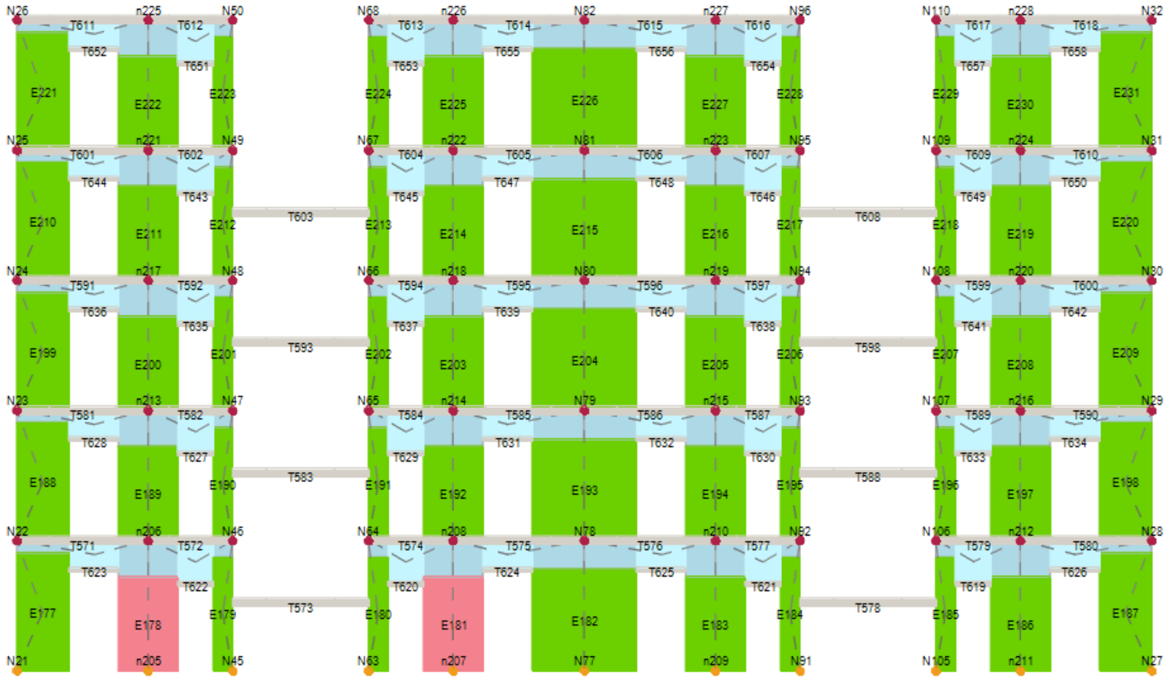


Figure 11. Verification status, for wall P.4 (EC requirements)

What is noticed in the above results regarding the static load verifications is that generally there are small problems in the structural verifications referring to the requirements of EC-06. In the case of verifications according to (approximate requirements) KTP, the results are positive in all cases.

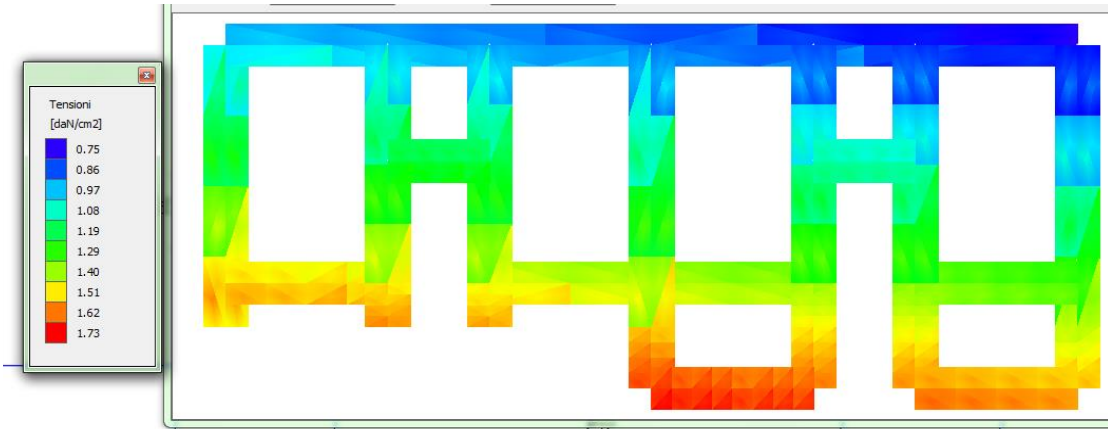


Figure 12. Results for the foundation floor

Although we do not have detailed information about the current dimensions of the foundations, based on similar projects of the same category, for accepted foundation width of 140-150cm, the values of the stresses in the foundation floor are generally presented within the allowed values.

4.2 Non-linear analysis (push-over)

According to the normative recommendations, two different controls should be performed:

Final Status (SLU):

$$D_{\max} \leq D_u \qquad \text{Equation 16}$$

Dmax: The maximum displacement required by the standard identified by the elastic spectrum.

Du: Maximum movement provided by the structure.

q * <q * lim. Equation 17

q *: the ratio between the elastic reaction force and the flow strength of the systemequivalent.

Damage limit state (SLD):

$Dsld_{max} \leq D_{d.}$ Equation 18

$Dsld_{max}$: Maximum displacement required by the rate, calculated as for Dmax assuming ag [SLD].

Dd: the maximum displacement in the SLD, which corresponds to the minimum value between that of the maximum shear force and that which causes the max value of the "drift" of the floor to exceed.The parameter (alpha-**u**) αu is considered an indicator of the risk of collapse. It is an indicator of the risk that the building will be unusable. These parameters are calculated as shown below:

PGADS: estimated acceleration for severe damage

PGADL: estimated acceleration for minor damage

(alfa- u) $\boldsymbol{\alpha_u}$ = PGADS / PGA. Equation 19

(alfa- e) $\boldsymbol{\alpha_e}$ = PGADL / PGA Equation 20

N.	Direction of seismic action	Seismic prop.	Ecc. [cm]	dt SLU [cm]	dm SLU [cm]	qu SLU	SLU ver.	dt SLD [cm]	dm SLD [cm]	SLD ver.	α SLU	α SLD
1	+X	Mass	0.0	5.45	3.16	5.78	No	3.16	1.34	No	0.52	0.48
2	+X	1° mode	0.0	6.61	2.74	7.11	No	3.91	1.43	No	0.42	0.40
3	-X	Mass	0.0	5.36	2.77	5.67	No	3.10	1.35	No	0.53	0.49

4	-X	1° mode	0.0	6.62	2.76	7.08	No	3.92	1.44	No	0.42	0.40
5	+Y	Mass	0.0	3.36	3.13	2.26	No	1.63	1.59	No	0.95	0.98
6	+Y	1° mode	0.0	4.34	6.69	3.08	No	2.33	1.26	No	0.98	0.65
7	-Y	Mass	0.0	3.34	3.76	2.30	Yes	1.63	2.87	Yes	1.10	1.47
8	-Y	1° mode	0.0	4.36	4.56	3.10	No	2.35	2.33	No	0.97	0.99
9	+X	Mass	59.7	5.37	3.57	5.88	No	3.11	1.25	No	0.51	0.46
10	+X	Mass	-59.7	5.55	2.84	5.66	No	3.22	1.43	No	0.53	0.50
11	+X	1° mode	59.7	6.40	2.76	7.12	No	3.78	1.25	No	0.42	0.37
12	+X	1° mode	-59.7	6.72	2.82	7.08	No	3.98	1.41	No	0.42	0.39
13	-X	Mass	59.7	5.35	2.68	5.67	No	3.10	1.36	No	0.53	0.50
14	-X	Mass	-59.7	5.37	2.76	5.62	No	3.11	1.43	No	0.53	0.52
15	-X	1° mode	59.7	6.61	2.68	7.04	No	3.91	1.46	No	0.42	0.41
16	-X	1° mode	-59.7	6.80	2.64	6.98	No	4.03	1.52	No	0.41	0.41
17	+Y	Mass	143.3	3.32	2.43	2.24	No	1.60	1.39	No	0.79	0.92
18	+Y	Mass	-143.3	3.44	2.97	2.49	No	1.72	1.31	No	0.89	0.85
19	+Y	1° mode	143.3	4.32	2.98	3.09	No	2.32	1.27	No	0.74	0.66

20	+Y	1° mode	-143.3	4.39	5.43	3.14	No	2.37	1.27	No	0.96	0.65
21	-Y	Mass	143.3	3.33	3.37	2.36	Yes	1.63	3.37	Yes	1.01	1.67
22	-Y	Mass	-143.3	3.42	2.99	2.45	No	1.70	2.31	Yes	0.90	1.23
23	-Y	1° mode	143.3	4.31	3.34	3.22	No	2.33	3.04	Yes	0.81	1.24
24	-Y	1° mode	-143.3	4.41	4.05	3.22	No	2.39	2.06	No	0.93	0.89

Figure 4. 6 Results for SLD and SLU

Analyzes number 16 and 19 are the most *unfavorable analyzes* because the combinations of values *αSLU, αSLD are the lowest* compared to the combinations of these values in other analyses.

For some of the analysis, the following are presented, some of the capacity curves generated by the numerical model (for the most unfavorable cases,):

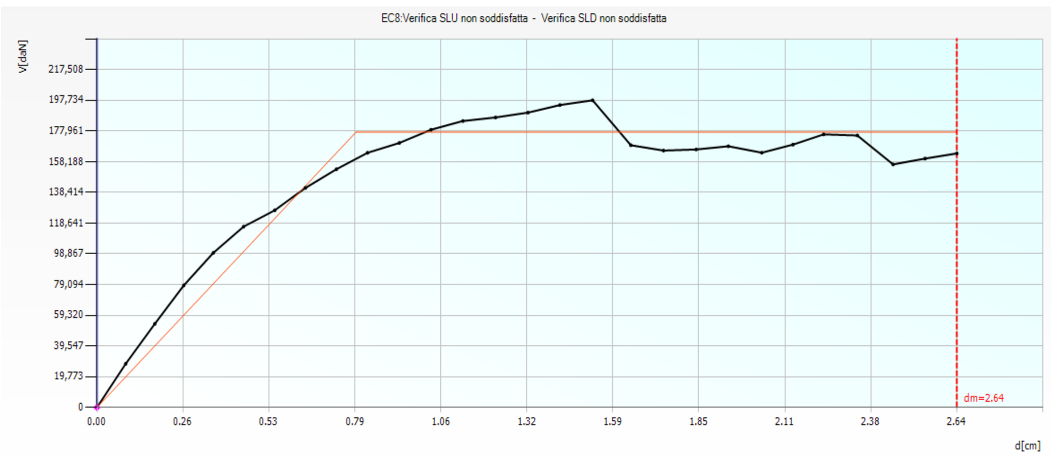


Figure 13. Capacity curve , Analysis No.-16 (Sx) 1-5

Figure 14. Pushover nonlinear static analysis verification , Analysis No.-16 (Sx) 1-5

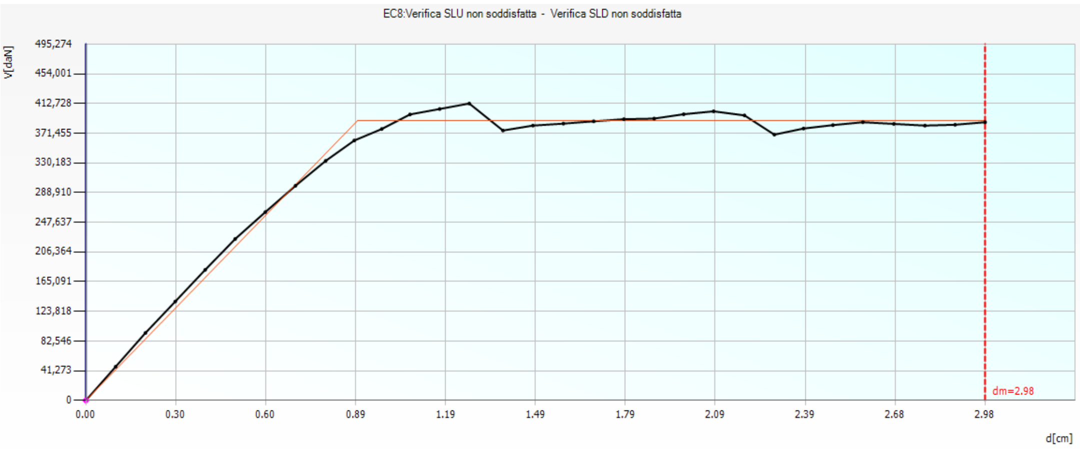


Figure 15. Capacity curve , Analysis No.-16 (Sy) 1-5

Figure 16. Pushover nonlinear static analysis verification , Analysis No.-16 (Sy) 1-5

In the case of analysis, the diagram of the capacity curve is interrupted at low values, influenced by the low physical-mechanical characteristics of the masonry.

Referring to the above results, what is noticed is that ***"Verifications are negative"***,as:

Displacements provided by the structure are smaller than those required (by standard)

Behavioral factor q *> (q * lim = 3)

Parameter (alpha-u) $\alpha_{u\,/\,e}$ <1, which is considered an indicator of the risk of collapse.

In the following, some of the walls of the building are presented in graphic form, with possible interpretations of the form of damage or destruction .

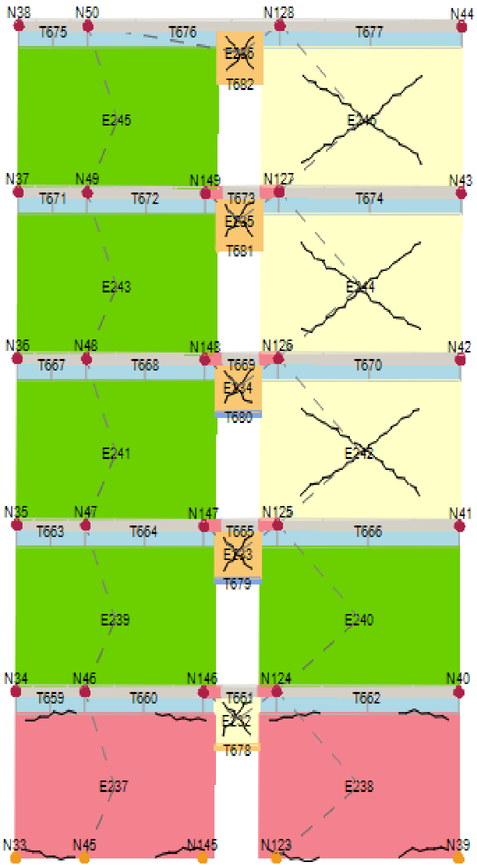


Figure 17. Damage / destruction form for wall P-2 Analysis No.-16 (Sx)

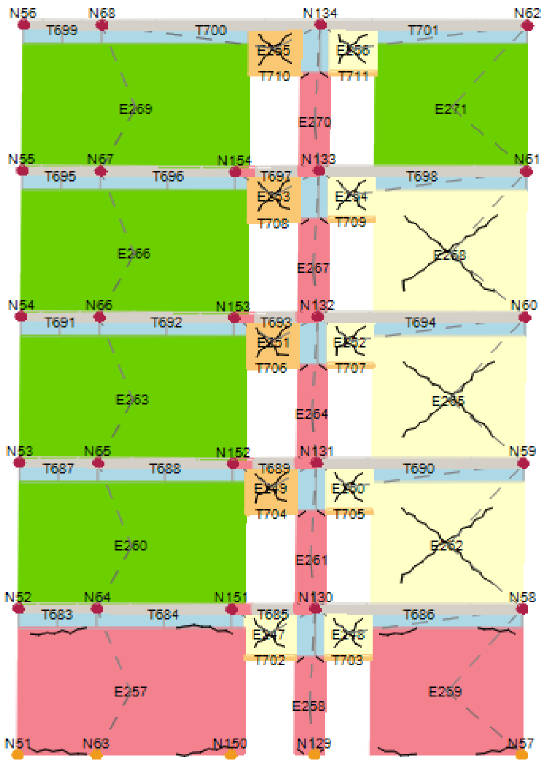
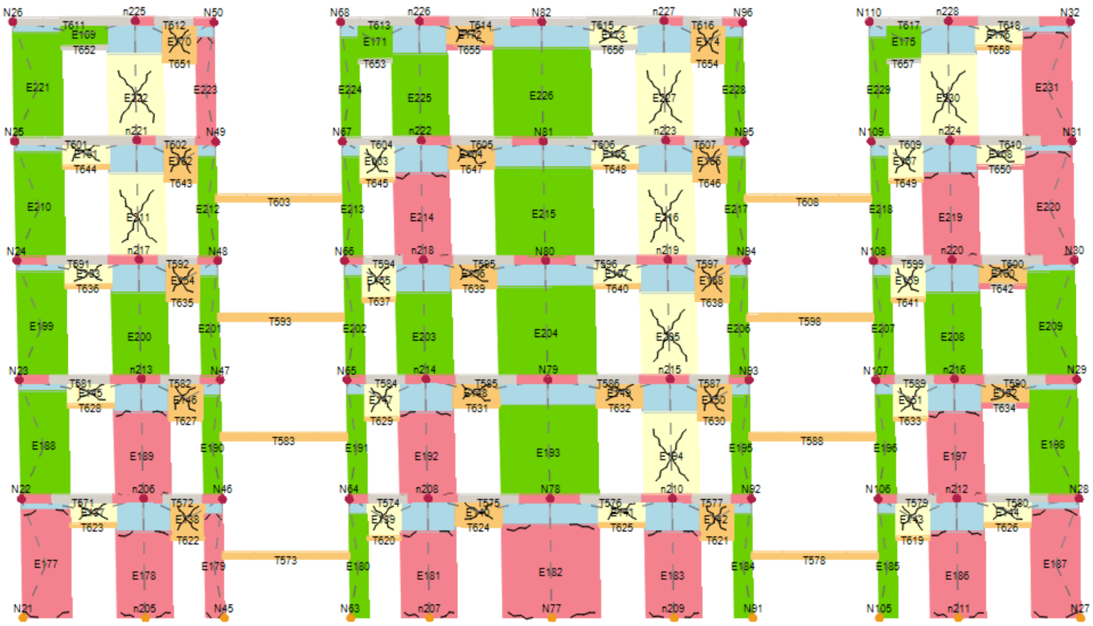


Figure 18. Damage / destruction form for wall P-5, P6 (Analysis N-19y)



CHAPTER 5

5. SUGGESTED STRENGTHENING METHODS AND VERIFICATION ANALYSIS.

5.1 Proposals for reinforcement-rehabilitation interventions .

While taking into account the fact that the building has met or is within the limit of its 50-year service period, and in addition to seismic rehabilitation is required to improve the overall longevity of the structure based we need to do the intervention "Structural renovation of the building"

The purpose of the rehabilitation is to modify the Ed seismic requirement, and / or the capacity of the structure, so that the structural elements of the rehabilitated structure meet the general Ed ≤ Rdverifications for the boundary condition and the relevant seismic action. This goal can be achieved using one of the following strategies, or by combining them :

- By increasing the capacity of the elements and the structure as a whole
- Reducing the consequence of seismic action in the structure.

5.1.1 Reinforcement -rehabilitation interventions:

a. ***Walls interventions***.

As the quality of the mortar is poor and the connection between it and the brick is poor, it is necessary to clean the masonry joints (at a depth of 1.5-2cm) and fill them with new mortar, to create in better bond between mortar and brick. Filling the joints with suitable mortar also envisages coating the masonry with plaster and galvanized mesh with a diameter of 1-2 mm and a step of 10x10-20x20 mm. Such interventions will significantly improve the physical mechanical characteristics of the masonry.

In certain areas, reinforcements are foreseen to be made through the overlaying technique on both sides of the wall with concrete thickness 4-5cm and Ø6 / Ø8 nets every 10 / 15cm.

In this intervention it is foreseen to improve the structural condition of the foundations through the technique of injection and coating with structural mortar or reinforced concrete.

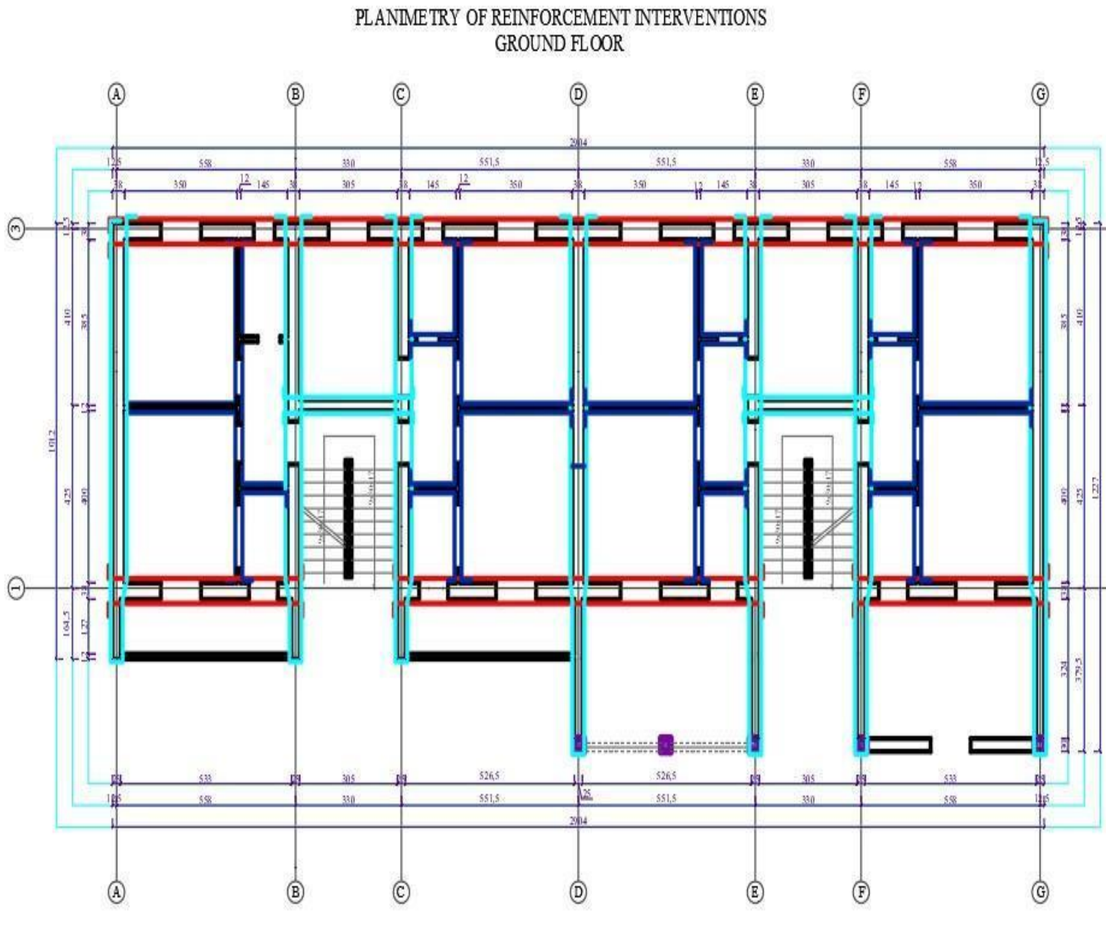


Figure 19. Planimetry of reinforcement interventions ground floor

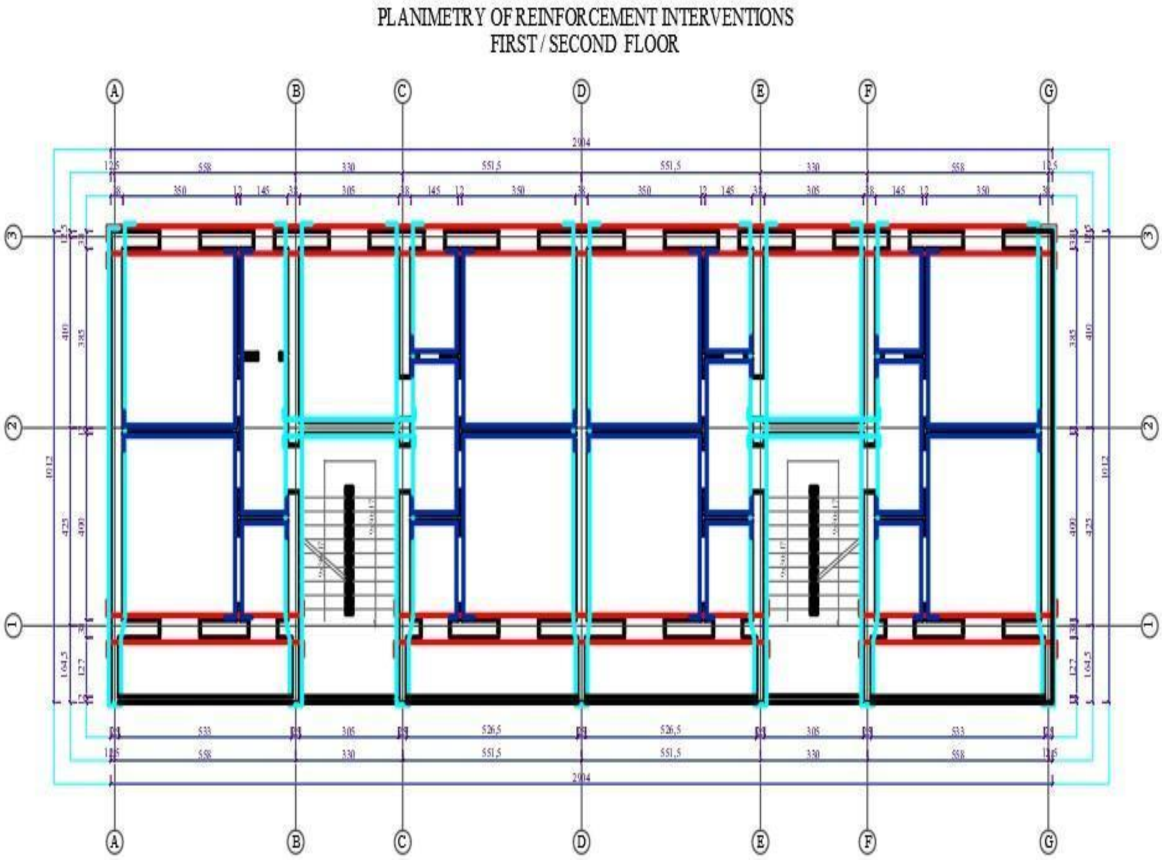


Figure 20. Planimetry of reinforcement interventions first /second floor

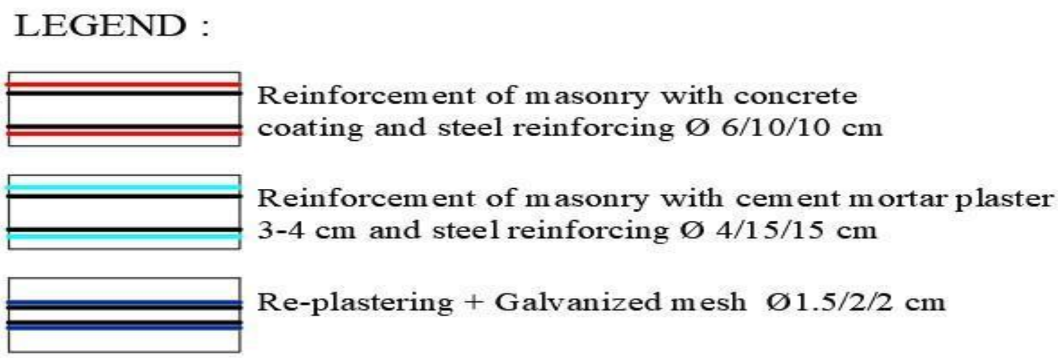


Figure 21. Legend of reinforcement interventions ground /first /second floor

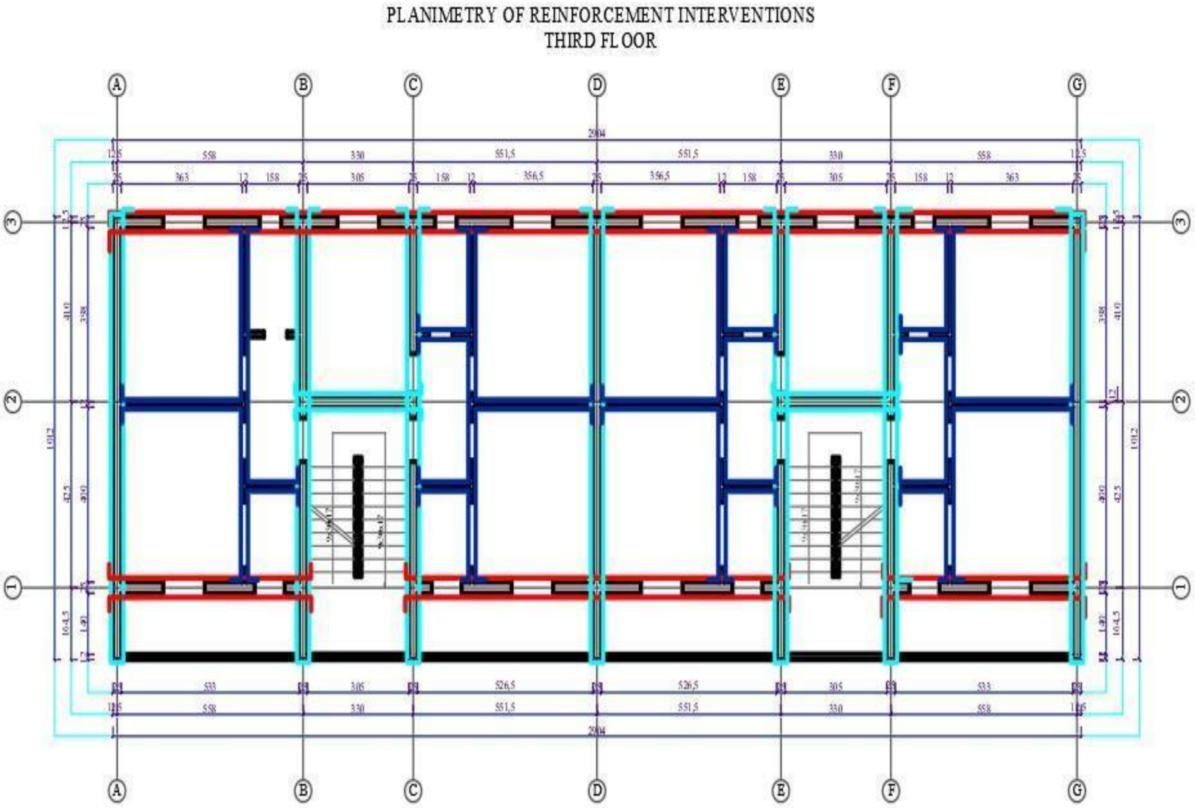


Figure 22. Planimetry of reinforcement interventions third floor

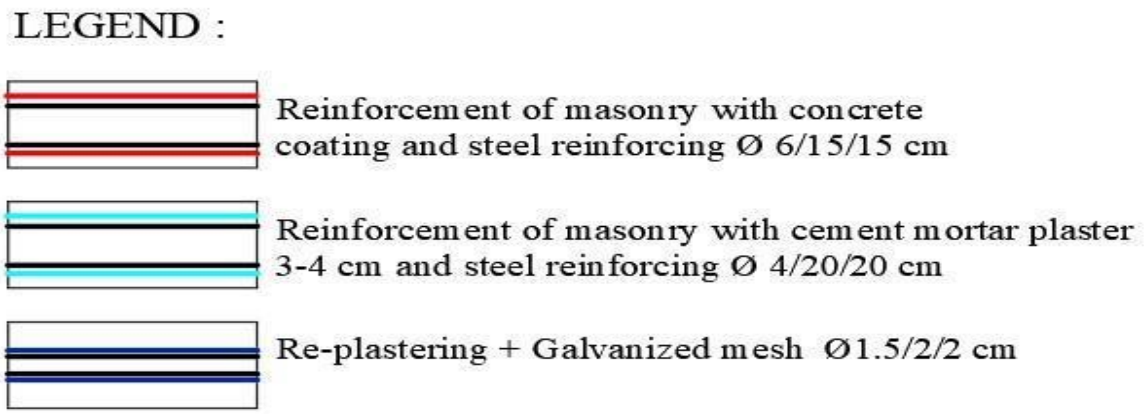


Figure 23. Legend of reinforcement interventions third floor

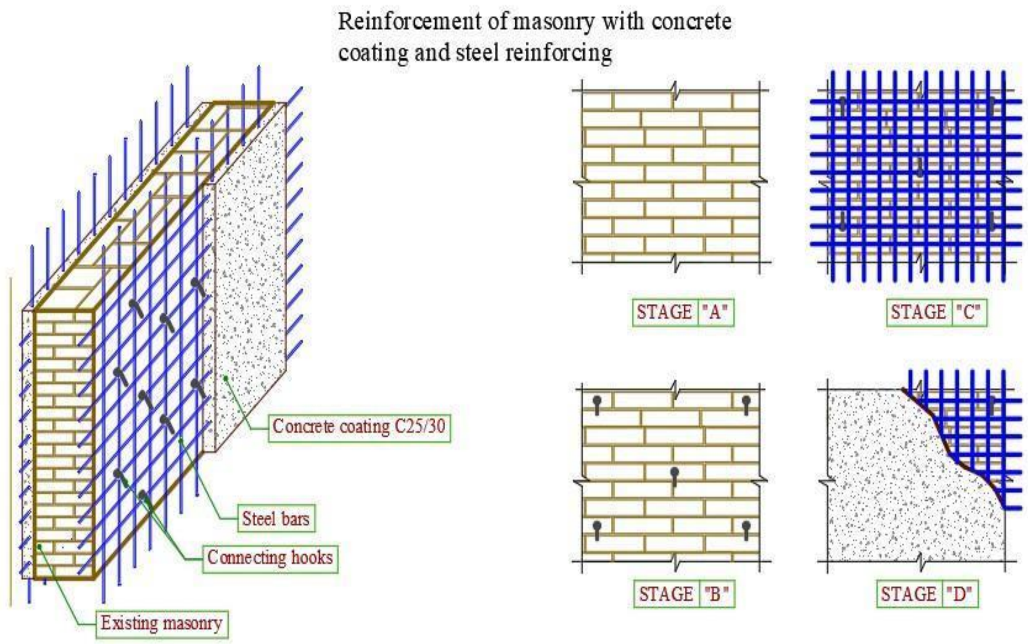


Figure 24. Reinforcement of masonry with concrete coating and steel reinforcing

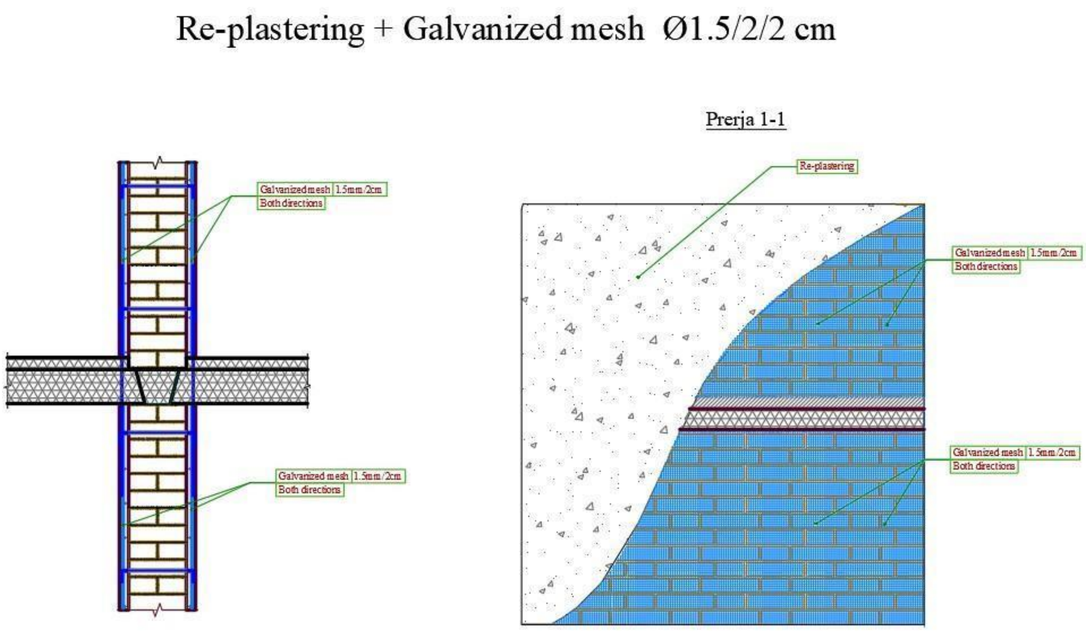


Figure 25. Re-plastering + galvanized mesh

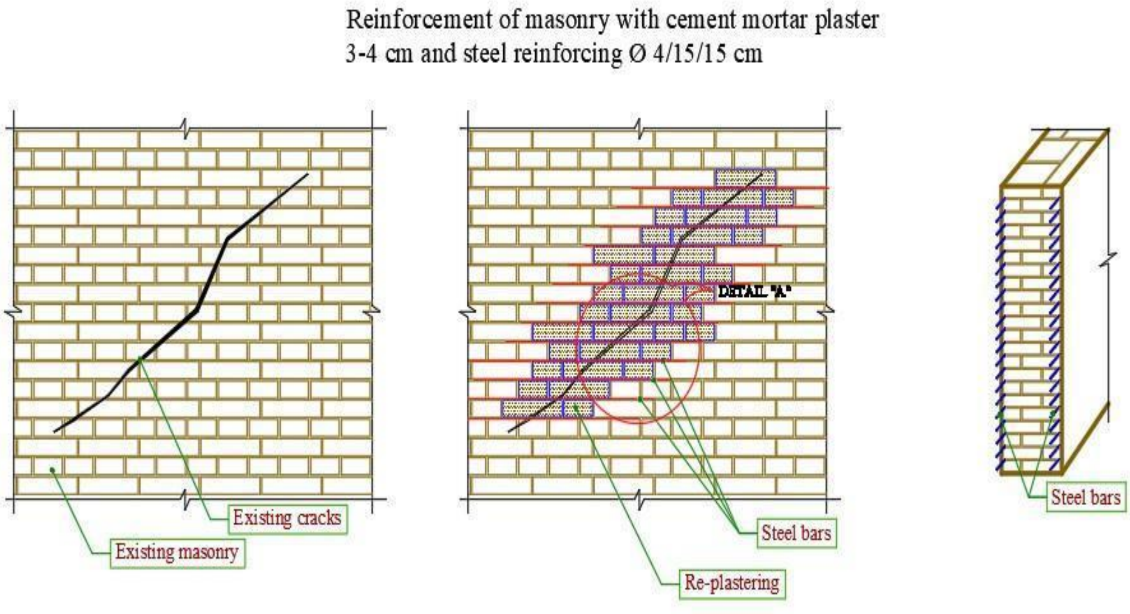


Figure 26. Reinforcement of masonry with cement mortar plaster and steel reinforcing

b. ***Intervention in the reconstruction of the terrace structure***

As the terrace is under the permanent influence of environmental influencing factors, its structural condition is significantly deteriorated. Moreover, the occasional repairs as well as the interventions in the placement of water tanks, have increased its weight and affect the bearing capacity of the building. Floor slab and hydro / thermal isolation layers of the terrace need to be rebuilt.

c. Interventions in the foundations

The foundations on the perimeter of the building will be reinforced by cementing with structural mortar. After cementing, it is planned to coat the foundation with a layer of reinforced concrete . Also the reinforced concrete walls added in the center of the building require additional excavation works and in the parts where the existing foundations are excavated also cemented and anchored wicks in them using resin for their connection with the new foundations.

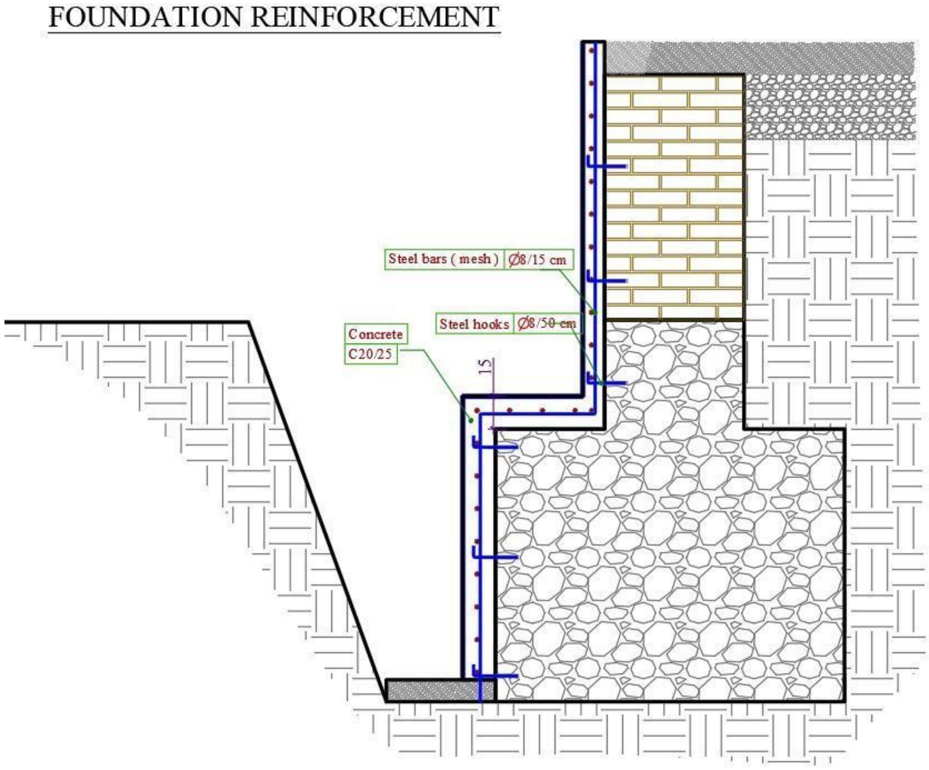


Figure 27. Reinforcement of foundation

d. ***Intervention in the demolition and remodeling of the floor slabs layers.***

It is foreseen to demolish the floor slabs layers as it is foreseen to make a reinforced concrete floor (constructive lightweight concrete) with a thickness of 4-5cm, reinforced with grill

d8 /15 / 15cm and connected to the existing floor slab with 4 ÷ 5 wick Ø10-12 / m2. This floor will ensure the increase of stiffness in the plan of the existing floor by also making a load distribution. This intervention will be realized in all the floor slabs, with the exception of the terrace floor which, as mentioned above, is foreseen to be reconstructed.

The whole process of replacing the floor layers will help eliminate excess weights as well as unify the loads on the existing floor.

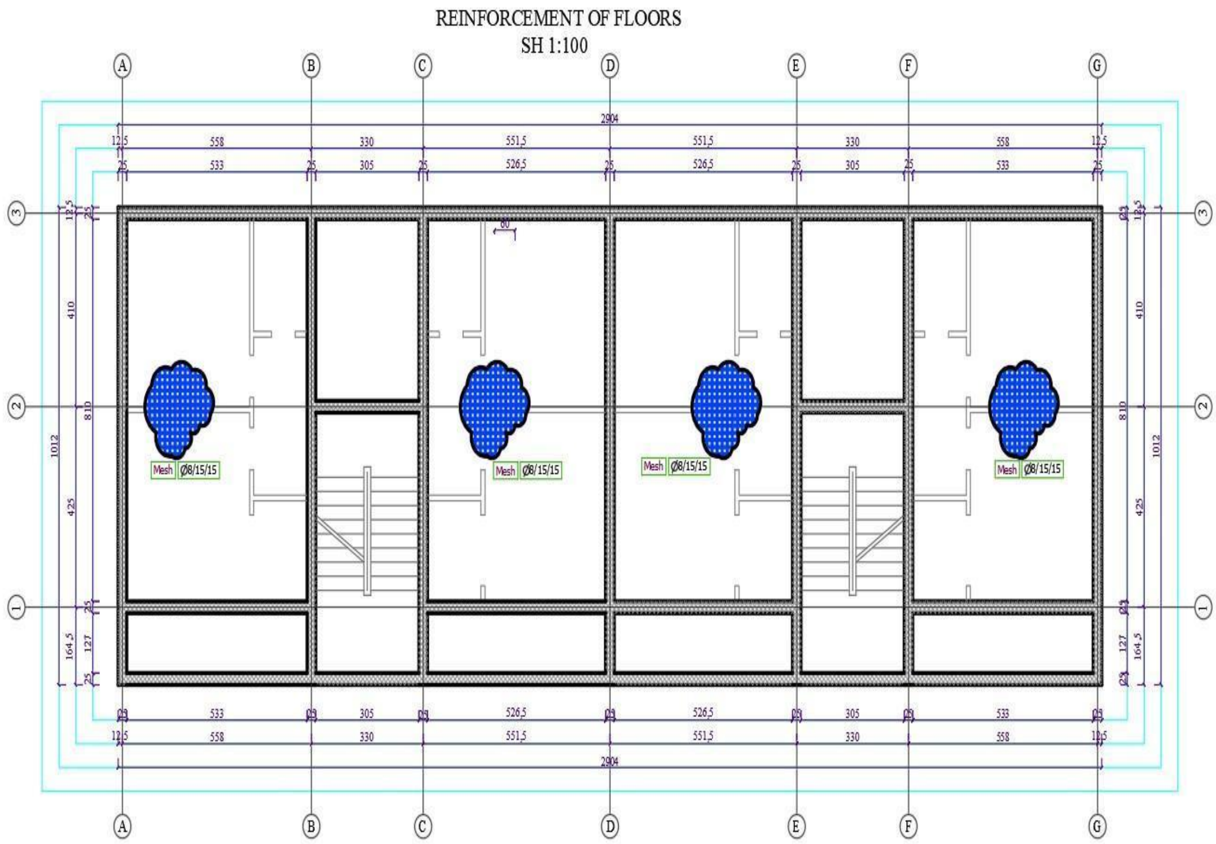


Figure 28. Reinforcement of floor

5.2 Verification Analysis

After we have done the rehabilitation interventions it is performed a second verification in

3Muri-STA DATA v10.0.2, which offers the calculation possibilities with different special

specifications. In this software,the 3 dimensional numerical model is subjected to static and nonlinear static analysis (pushover). This time i have taked in consideration while modeling the changes (interventions) made before .

5.2.1 Analysis of own oscillations:

This section presents the results of the analysis of own oscillations for the building. In order to have the highest possible mass participation, 22 oscillations of oscillations were taken into account during the analysis

Table 12. Sum of the effective modal measures

Modo	T [s]	mx [kg]	Mx [%]	my [kg]	My [%]	mz [kg]	Mz [%]
1	0.23411	1,289,209	56.45	13,616	0.60	179	0.01
2	0.20466	24,445	1.07	1,696,177	74.27	7,188	0.31
3	0.16162	505,670	22.14	10,236	0.45	23	0.00
4	0.08832	245,404	10.74	0	0.00	3	0.00
5	0.07932	13	0.00	365,849	16.02	160,180	7.01
6	0.06914	45,790	2.00	73	0.00	3	0.00
7	0.06544	198	0.01	33,727	1.48	22,292	0.98
8	0.05890	4	0.00	7	0.00	2,394	0.10
9	0.05806	297	0.01	28,479	1.25	1,386,973	60.73
10	0.05679	13,713	0.60	384	0.02	7,232	0.32
11	0.05524	67,569	2.96	282	0.01	2,159	0.09
12	0.05420	1,414	0.06	2,117	0.09	22,871	1.00
13	0.05230	11	0.00	1,583	0.07	3,315	0.15
14	0.05097	2,634	0.12	17	0.00	218	0.01
15	0.04858	583	0.03	2,411	0.11	25,909	1.13
16	0.04798	976	0.04	34	0.00	6,343	0.28
17	0.04755	3,442	0.15	2,478	0.11	30,132	1.32
18	0.04439	27,837	1.22	695	0.03	8,646	0.38
19	0.04340	480	0.02	13,202	0.58	60,346	2.64
20	0.04180	1	0.00	8,229	0.36	3,617	0.16
21	0.04170	221	0.01	22,033	0.96	103,370	4.53
22	0.04066	322	0.01	18,496	0.81	16,830	0.74

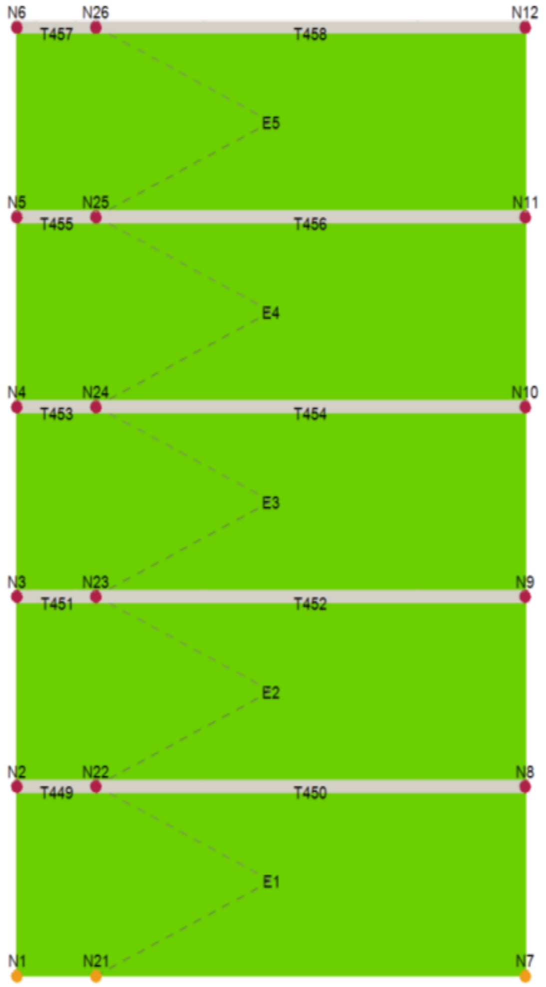
5.2.2 Statistical Load Analysis

These checks were performed on each wall of the structure, in three main sections (bottom, center, and top).

The values of normal resistance stresses will only be calculated if the bending controls and the eccentricity of the loads are met.All verifications shown in the table below are satisfied after the interventions.

Table 13. Verifications for static loads, for each of the walls (EC requirements)

Parete	Maschi rotti	Nd/Nr Max	h0/t Max	e1/t Max	e2/t Max
1	0	0.36	12.60	0.198	0.077
2	0	0.46	12.60	0.175	0.078
3	0	0.41	12.60	0.198	0.077
4	0	0.47	12.60	0.063	0.063
5	0	0.44	12.60	0.191	0.076
6	0	0.50	12.60	0.194	0.075
7	0	0.42	12.60	0.063	0.063
8	0	0.52	12.60	0.194	0.076
9	0	0.40	12.60	0.194	0.079
10	0	0.36	12.60	0.063	0.063
11	0	0.35	12.60	0.063	0.063
13	0	0.40	12.60	0.063	0.063
14	0	0.40	12.60	0.063	0.063



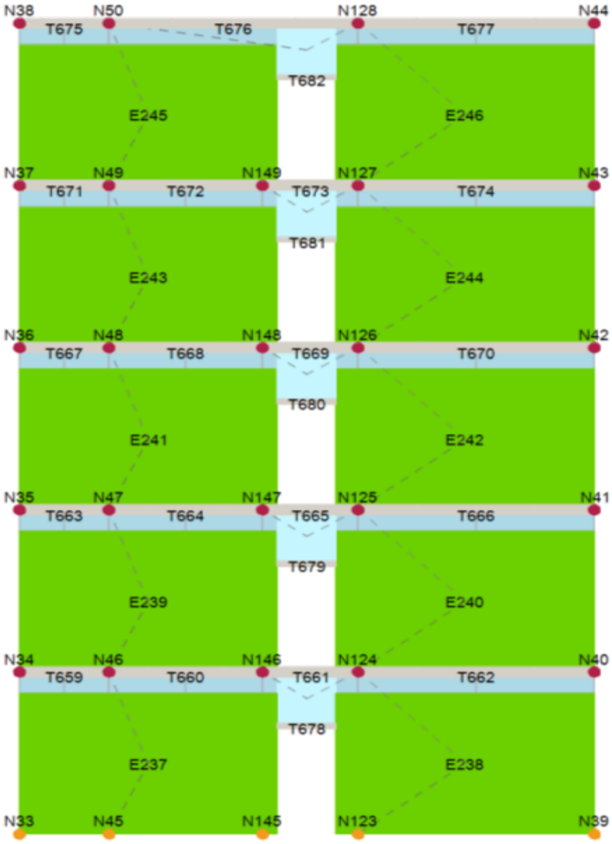


Figure 29. Verification status, for wall P.1 + P.5 (requirements according to EC)

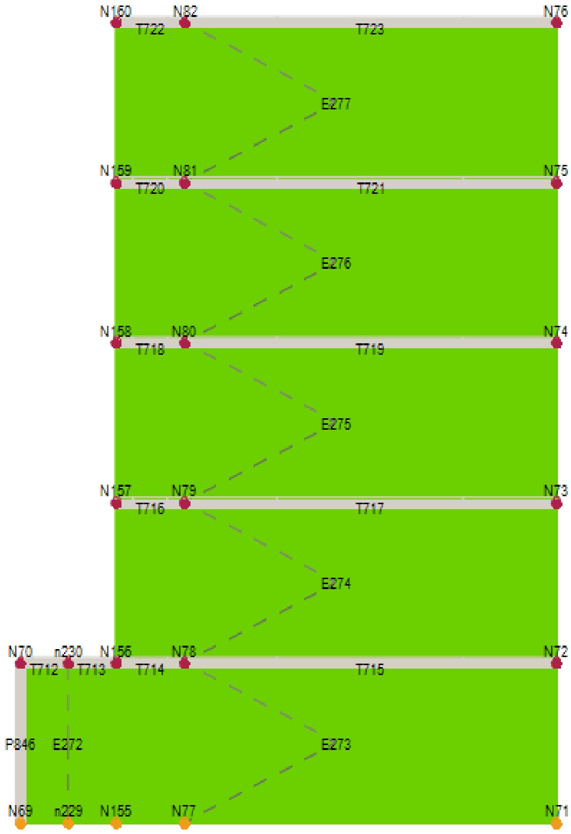


Figure 30. Verification status, for wall P.6 + P.7 (requirements according to EC)

5.3 Non-linear analysis (push-over)

Referring to the above results, what is noticed is that ***"Verifications are positive"***, as:

Displacements provided by the structure are smaller than those required (by standard)

Behavioral factor q *> (q * lim = 3)

Table 14. 3 of verifications, for SLU and SLD

N.	Dir. sisma	Car. sismico prop.	Ecc. [cm]	dt SLU [cm]	dm SLU [cm]	qu SLU	SLU ver.	dt SLD [cm]	dm SLD [cm]	SLD ver.	α SLU	α SLD
1	+X	Mass	0.0	2.80	3.58	2.31	Yes	1.32	2.06	Yes	1.21	1.33
2	+X	1° mode	0.0	3.71	3.78	2.87	Yes	1.93	2.26	Yes	1.01	1.12
3	-X	Mass	0.0	2.72	3.47	2.19	Yes	1.25	1.86	Yes	1.20	1.27
4	-X	1° mode	0.0	3.38	3.56	2.39	Yes	1.67	2.35	Yes	1.03	1.26
5	+Y	Mass	0.0	1.78	2.10	1.88	Yes	0.67	1.20	Yes	1.11	1.31
6	+Y	1° mode	0.0	2.26	4.91	2.03	Yes	0.96	2.20	Yes	1.48	1.62
7	-Y	Mass	0.0	1.69	2.29	1.71	Yes	0.57	1.79	Yes	1.21	1.71
8	-Y	1° mode	0.0	2.18	4.07	1.86	Yes	0.87	2.57	Yes	1.57	1.85
9	+X	Mass	59.7	2.69	3.69	2.31	Yes	1.26	2.08	Yes	1.27	1.37
10	+X	Mass	-59.7	2.67	3.57	2.24	Yes	1.23	2.16	Yes	1.25	1.42
11	+X	1° mode	59.7	3.16	3.19	2.89	Yes	1.67	1.67	Yes	1.02	1.01
12	+X	1° mode	-59.7	3.38	3.37	2.74	No	1.90	2.05	Yes	0.99	1.06
13	-X	Mass	59.7	2.60	3.87	2.10	Sì	1.16	2.07	Yes	1.35	1.41

14	-X	Mass	-59.7	2.62	3.96	2.38	Sì	1.23	1.95	Yes	1.26	1.34
15	-X	1° mode	59.7	3.11	3.38	2.61	Sì	1.55	1.67	Yes	1.11	1.05
16	-X	1° mode	-59.7	3.22	3.17	2.56	No	1.63	1.85	Yes	0.98	1.09
17	+Y	Mass	143.3	1.81	1.90	1.93	Yes	0.70	1.20	Yes	1.03	1.30
18	+Y	Mass	-143.3	1.79	2.30	1.88	Yes	0.67	1.20	Yes	1.18	1.31
19	+Y	1° mode	143.3	2.27	4.00	2.08	Yes	0.97	2.40	Yes	1.44	1.72
20	+Y	1° mode	-143.3	2.30	4.61	2.07	Yes	0.99	1.80	Yes	1.45	1.40
21	-Y	Mass	143.3	1.74	1.89	1.78	Yes	0.62	1.49	Yes	1.05	1.51
22	-Y	Mass	-143.3	1.70	2.49	1.72	Yes	0.58	2.09	Yes	1.28	1.88
23	-Y	1° mode	143.3	2.22	3.47	1.93	Yes	0.91	2.48	Yes	1.38	1.78
24	-Y	1° mode	-143.3	2.21	4.47	1.89	Yes	0.89	2.08	Yes	1.59	1.59

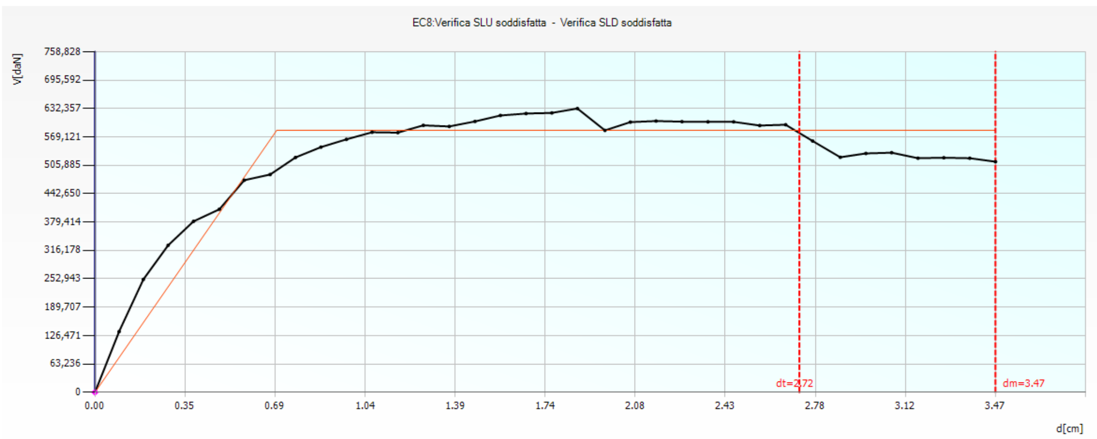


Figure 31. Capacity curve from satisfied static nonlinear Pushover analysis, Analysis No-3(Sx)

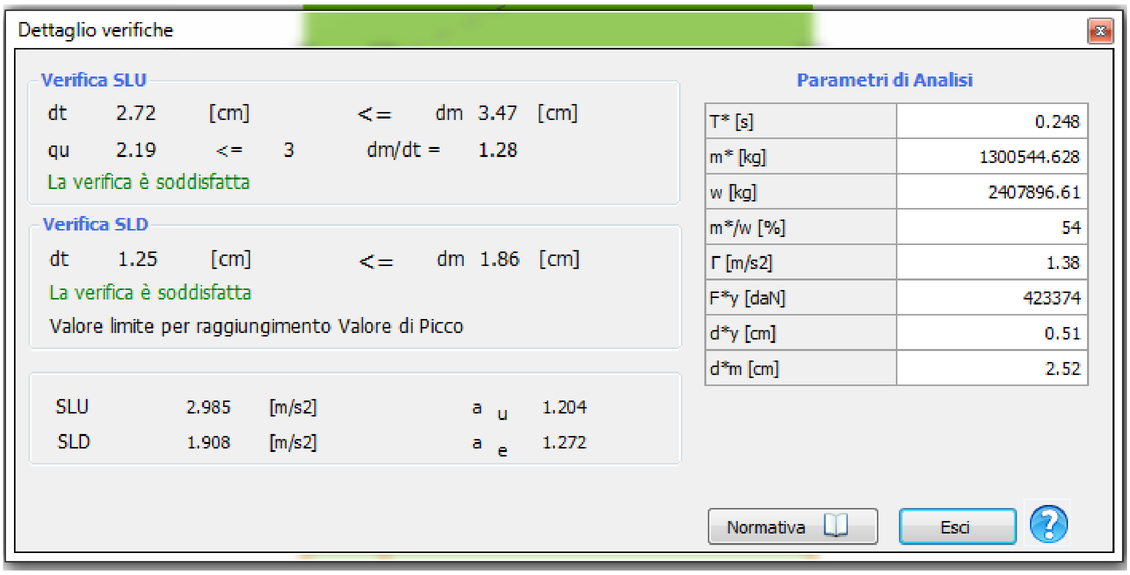


Figure 32. 14 Verifications from satisfied static nonlinear Pushover analysis, Analysis No-3(Sx)

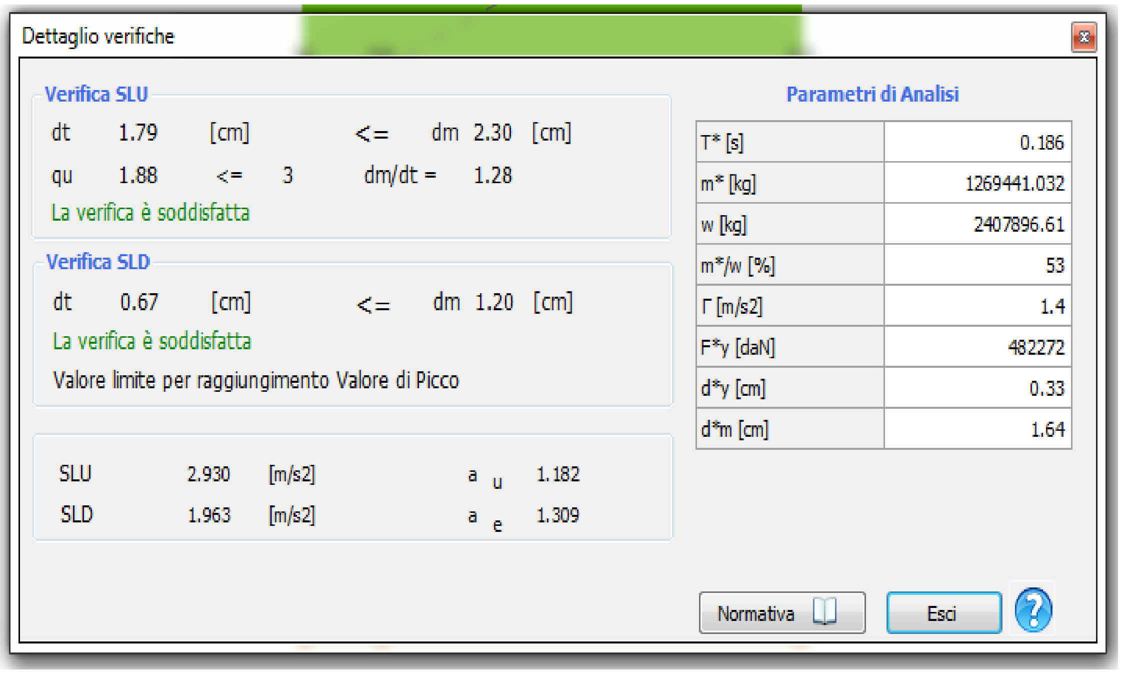


Figure 33. Verifications from satisfied static nonlinear Pushover analysis, Analysis No-19(Sy)

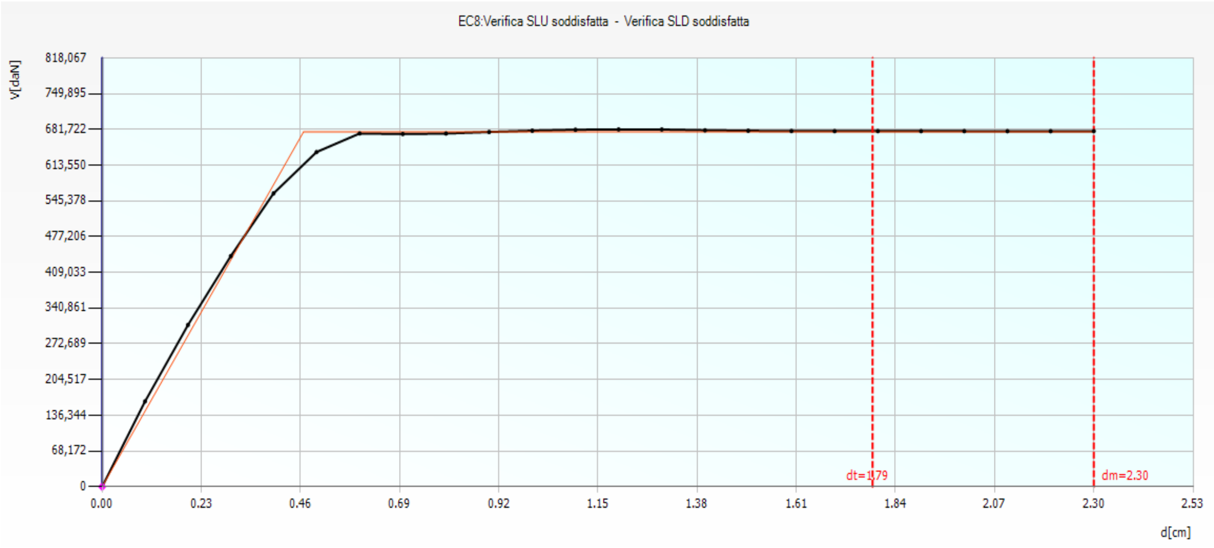


Table 15. Capacity curve from satisfied static nonlinear Pushover analysis, Analysis No-19(Sy)

CHAPTER 6

6. CONCLUSIONS

6.1 Conclusions

The conclusions are based on the analysis of existing materials, geometric surveying, assessment of damage from recent earthquakes, on-site and laboratory tests, numerical-structural calculations, assessment of bearing capacity and damage based on seismic actions of magnitude earthquake 5.8 of the Richter scale (dated 21.09.2019) and the one with magnitude 6.4 of the Richter scale (dated 26.11.2019).

Referring to the KTP with which the building is designed and implemented, the structure satisfies bearing capacity to withstand seismic action. Referring to the factual situation - on the one hand geometry and regular distribution of structural elements and on the other hand the deterioration to a significant degree of the condition of the masonry - it can be said that the actual structural bearing capacity is reduced by 10-20% compared to the initial one. So, from the time of the start of the service until today, although it has withstood at various times significant seismic actions (has passed at least 4 significant seismic events and beyond its design values) and has suffered occasional interventions in the structural scheme, it can be said that the seismic bearing capacity of the building turns out to be reduced but again acceptable.

In the impossibility of national definitions and specific requirements in terms of reference as well as referring to Eurocode guidelines the level of protection of the building is accepted by checking it according to the border condition "significant damage" - (SLD) and that border of close to collapse (SLU).For requirements of the specific seismic requirements of Eurocode 8, Part 3 (EN 1998-3), in the linear and nonlinear analyzes performed for the purpose of this in-depth expertise, the bearing capacity of the structures is insufficient. More concretely:According to the transverse direction the building has sufficient capacity and is limited to the border condition of significant damage; According to the longitudinal direction, the targeted displacement does not meet the criterion that it is smaller than the border displacement for the borderline condition of significant damage, even in this regard the intended displacement is even greater than the boundary displacement for the close to collapse boundary condition.

Consequently, structural improvements and complete structural rehabilitation are necessary. The importance of these interventions is to inncrease the capacity of the structure Increasing the capacity of structural elements and the structure as a whole can be achieved through the following strategies:

1.Overlaying of structural elements

2.Transformation of non-structural elements into structural elements

3.Addition of new structural elements.

These interventions made possible that afer the verification analysis the results for statistical load analysis and push over analysis are both satisfied for KTP and Eurocode guidelines .Specifically for the push over analysis the displacements provided by the structure are found to be smaller than the required standard, and the behavioral factor q * is greater than the limit of 3.These positive results indicate that the structure is performing well and is meeting the criteria for stability and strength.These successful intervention strategies can be used to reinforce similar buildings with comparable design and construction features and similar soil characteristics. Specificall, a masonry structure with silicate bricks and five floors in the area. By implementing these interventions, the safety and durability of the building can be enhanced, allowing it to withstand external forces and maintain its stability in the long term.

6.2 Further studies.

Future research in pushover analysis via 3Muri software or other programs for masonry buildings build in Albania many years before and affected by the earthquakes that hit our country could aim at enhancing accuracy by including more comprehensive information about the materials used and their behavior. This could be done by performing more laboratory tests to determine the strength and deformational properties of masonry materials, as well as utilizing sophisticated numerical modeling techniques, such as finite element analysis, to simulate the response of masonry structures to various loads.

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