THE STRUCTURAL ASSESSMENT OFA TYPICAL UNREINFORCED MASONRY BUILDING, IN ALBANIA

A THESIS SUBMITTED TO
THE FACULTY OF ARCHITECTURE AND ENGINEERING
OF
EPOKA UNIVERSITY

BY

NORBERTA FRROKU

IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF MASTER OF SCIENCE
IN
CIVIL ENGINEERING

MARCH, 2023

Approval sheet of the Thesis

This is to certify that we have read this thesis entitled **"The structural assessment of a typical unreinforced masonry building, in Albania "**and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

____

Assoc.Prof. Dr. Mirjam NDINI
Head of Department
Date: March, 06, 2023

Examining Committee Members:

Prof. Dr. Huseyin Bilgin	(Civil Engineering)	_________________
Dr. Begmurat Kulmedov	(Civil Engineering)	_________________
Dr. Marsed Leti	(Civil Engineering)	_________________

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.

NameSurname: Norberta Frroku

Signature: _______________

ABSTRACT

THE STRUCTURAL ASSESSMENT OF A TYPICAL UNREINFORCED MASONRY BUILDING, IN ALBANIA

Frroku, Norberta

M.Sc., Department of Civil Engineering

Supervisor: Prof.Dr. Huseyin Bilgin

Masonry buildings are some of the most spread construction objects in Albania. They were built mostly before the year 1990. In the early years of the communist period in our country, stone masonry buildings were built, but some years later, especially during 1944-1990 brick masonry buildings continued to be constructed. These years, in order to design according to Albanian Code (KTP) guidelines, template projects of the residential building having masonry load-bearing walls were designed.

This study highlights the importance of studying masonry buildings. Many people in our nation live and work in the masonry construction industry. Masonry buildings make up more than 60% of Albania's building stock [Bilgin and Hysenlliu 2020]. To determine whether repairs, retrofits, or merely upkeep are required, they must be thoroughly evaluated.

The thesis presents some designing Codes about masonry like Eurocode and Albanian Code provisions. Engineers must adhere to these criteria in order to identify issues with the buildings, maintain them, retrofit them, or construct new masonry structures that are safer than those constructed in accordance with outdated technical codes.

Albania is a country that is at the meeting point of the Euro-Asia and Adria tectonic plates. Several sized earthquakes have been caused by the crushing of these plates. The Earth's crust has suffered significant damage as a result.Buildings, people, and the environment are all affected by these impacts on the earth's crust. So, it is essential to determine the impacts of the earthquake in Albania in order to prevent deaths and minimize economic losses from such issues.

Earthquake design according to Eurocode is important. An applied example will be included in the thesis to provide readers with a thorough understanding of seismic design. A three-story, conventional masonry building will be examined. The analysis will be carried out utilizing the current Eurocodes guidelines and CDS-Win software. The structure will be analyzed utilizing the N2 and ATC-40 methodologies. To determine whether or not this building is safe to use, a nonlinear dynamic analysis will be conducted, and software findings will be obtained and analyzed. These analyses led to the decision that the masonry structure should be destroyed. This thesis is merely a highlight for aspiring engineers that work with these kinds of structures.

This research is just an entrance into the study of masonry buildings. Future engineers in Albania should be aware when intervening in these types of objects. First of all the load-bearing capacity of the building should be revealed in this case and then should be taken intoconsideration the depreciation of the building, since the construction culture of our country, there is little to do with the maintenance of buildings over the years.

Keywords:*masonry buildings, earthquake , Albanian Code, Eurocode, Seismicity, Pushover analyse*

ABSTRAKT

VLERESIMI STRUKTUROR I NJE NDERTESE MURATURE TE PAPERFORCUAR, NE SHQIPERI

Frroku, Norberta

Master Shkencor, Departamenti i Inxhinierisë sëNdërtimit

Udhëheqësi: Prof.Dr. Huseyin Bilgin

Objektet me tulla jane nder ndertesat me te perhapura ne Shqiperi.Ato jane ndertuar kryesisht para vitit 1990. Ne vitet e para te periudhes se komunizmit, ne vendin tone, ndertesa me mur guri u ndertuan, por vite me vone 1944-1990 filluan te ndertoheshin dhe ndertesa me mure mbajtese prej tulle. Ne keto vite, me qellim qe te ndertohej sipas Kushteve Teknike te Projektimit, shembuj te caktuar projektesh te objekteve civile me sistem konnstruktiv mure mbajtes aplikoheshin ne terren .

Ky studim ve ne qender rendesine e studimit te ndertesave me murature , mbajtese. Shume njerez ne vendin tone jetojne dhe punojne ne ndertesa te tilla. Ndertesat prej me mure mbajtese perbejne rreth 60% te ndertesave te mbetura stok, ne Shqiperi.[Bilgin dhe Hysenlliu 2020].Per te percaktuar nese nevojitet riparim, perforcim apo thjeshte mirembajtje ato duhet dosmosdo te rivleresohen.

Ne kete teme perfshihen Kodet e projektimit per ndertesat prej murature si Eurokodet dhe Kushtet teknike te projektimit. Inxhnieret duhen mbeshtetur ne keto kritere per te gjetur ne menyren e duhur problemet me keto objekte, per t'i mirembajtur, perforcuar ose per te ndertuar ndertesa te reja me mure mbajtese qe jane me te sigurta se sa ato te ndertuara me kodet e projektimit te meparshme.

Shqiperia eshte nje shtet qe ndodhet ne piken e bashkimit te dy pllakave tektonike, pllakes Euro-Aziatike dhe plakes tektonike Adria. Shume termete te madhesive te ndryshme gjenerohen nga perplasja e ketyre pllakave .Si rezultat siperfaqja e tokes ka pesuar deformime te shumta. Ndertesa , njerez dhe mjedisi

perreth kane qene nen ndikimin e ketij impakti mbi koren tokesose. Keshtu qe eshte e rendesishme per te njohur deme te termeteve ne Shqiperi qe te ndalohen vdekjet dhe te pakesohen humbjen ekonomike nga probleme te tilla..

Projektimi kundrejt termeteve eshte i rendesishem sipas Eurokodeve.Nje shembull aplikativ do te perfshihet ne kete teme per t'ju paraqitur lexuesve nje njohje me te mire te projektimit sizmik. Do te shqyrtohet nje ndertese konvencionale prej murature tulle me 3 kate. Analiza do te behet duke perdorur rregullat e Eurokodit dhe programin CDS-Win. Struktura do analizohet sipas metodes N2 dhe ATC-40.Per te percaktuar nese kjo ndertese eshte e sigurt apo jo, do behet nje analize dinamike jolineare e struktures duke futur te dhenat ne program dhe nxjerre rezultatet e nevojshme. Kjo analize ka nxjerre si perfundim qe objekti ka pesuar shume demtime dhe do te shembet. Kjo teme eshte thjeshte nje reference per ata qe aspirojne ne inxhinieri mbi punen me strukturat me mure mbajtese.

Ky studim eshte vetem nje hyrje ne studimin e objekteve me murature tulle. Inxhinieret e ardhshem ne Shqiperi duhet te kene kujdes gjate nderhyrjeve ne keto objekte. Mbi te gjitha duhet te vleresohet fillimisht aftesia mbajtese e ketyre ndertesave, dhe menyra e deformimit te objektit deri ne shkaterrim total perpara cdo nderhyrje te mundshme. Ne Shqiperi mungon kultura e mirembajtjes se objekteve, prandaj ato duhen vleresuar me kujdes.

Fjalët kyçe: *Ndertesa murature, termeti , Kodi Shqiptar I projektimit , Eurokodi, Sizmicitet.Analiza Pushover*

This thesis is dedicated to my family, especially to my father, who motivated me to continue to follow my studies and research for years.

ACKNOWLEDGEMENTS

The author is very thankful and grateful for the work done by the supervisor of this study Dr.Huseyin Bilgin. He was very kind and patient during the guidance of the study. His work was very valuable and helped me to have a deep understanding of how masonry structures work.The author is also thankful to all colleagues and friends for their assistance and motivation, during the working process of this study.

.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 General information about masonry buildings

Ten thousand years ago, they built for the first time. Building and creating in stone, clay, brick, or concrete blocks are known as masonry. Stone, brick, and concrete blocks are used to make masonry structures. It has many benefits as a well-liked construction technique. Three of the most famous masonry buildings in history that have weathered strong earthquakes are the Colosseum, Taj Mahal, and Pyramids in Cairo, Egypt [Korini,2011].

Brick is admired by architects for its size, color, pattern, permanency, and scale. The resistance to fire, control of sound, and insulation from daily temperature variations are some additional advantageous qualities of masonry.Masonry is also a well-known technique even in Albania[Korini,2011].

Between 1945 and 1990, in Albania were created new guidelines for architectural design. Masonry buildings were mostly employed for private and public spaces. The first buildings made of masonry had one or two stories and were constructed from a variety of bricks, including clay and stone ones. Between 1945 and 1963, a great number of masonry structures were constructed with the use of engineers' prior knowledge and streamlined calculations.[Korini,2011].

The Albanian government authorities approved the first standardized design template for a two-story adobe building in 1949. Conventional design templates for buildings with three to five masonry stories were used between 1963 and 1978. After the 1979 earthquake in Shkodra, publications of KTP-N.2-89 were added to the previous Code [Bilgin and Hysenlliu 2020]. Nowadays, RC buildings and masonry structures are both in use. These structures make up a stock of the country's residential construction stock. Both of these types of constructions load-bearing walls in URM structures and load-bearing walls in CM buildings, coupled with RC tie elements are presented nowadays.

In the previous Albanian Codes was used the force-based design. The force-based approach designed for buildings without seismic code requirements needs to be reevaluated. It is necessary to assess these buildings' seismic performance while taking into account their non-linear response during powerful earthquakes. After conducting extensive research in the field of earthquake engineering, today's rules are the same as those in Eurocode 6 for masonry structures. With time, it has become possible to construct safer buildings because of developments in technology, the usage of new guidelines, and contemporary studies in the construction industry. These new rules are necessary because our country is in a very high seismic zone.

Earthquakes frequently strike our nation, Albania. Two earthquakes struck the city of Durres and its surrounds in September and November of 2019. The first earthquake, measuring 5.8 on the Richter scale, occurred on September 21, 2019, while the second, more violent earthquake, measuring 6.4 on the Richter scale, occurred on November 26, 2019. (without mentioning several shakings from smaller earthquakes as an effect of aftershocks). After the first earthquake, there were a total of 300 aftershocks, and after the second, there were nearly 1400 [Data from the Institute of Geoscience]. These earthquakes help us know their effects on the construction industry today.

1.2 Thesis Objective

This thesis provides a thorough grasp of the operation of masonry constructions (through the inspection of an unreinforced masonry school as an applied example). Schools in Albania are frequently built of masonry. Since these facilities contain a large number of students and children who are learning there, it is important to examine them. We can easily see how many different masonry schools are constructed and how they respond to seismic activity by looking at this type of structure. Also, if we want to improve the stability of numerous structures of this type, this research is essential.

An objective of this research is to have a deeper knowledge of the masonry construction materials in accordance with both Albanian Technical Codes and Eurocodes. This is an important part of the study of a building because without knowing the materials that make a structure we can not find out its behavior under the effect of external or internal forces. The materials should be tested according to nowadays standards which will be followed by further research.

This study uses the Finite Element Method and Pushover analysis along with the CDS program, in order to gain a thorough understanding of the structural behavior of unreinforced masonry buildings. The building under investigation, was built after 1960 and had numerous wall breaches during the 2019 earthquake, and was decided to be demolished.We can better understand the causes of these damages at this institution by conducting its investigation.After dealing with this structure, we can clearly see how other structures of a similar type will respond to earthquake shocks.

1.3 The Scope of works

Masonry is the craft of building a structure with brick, stone, or similar material, which is often laid in and bound together by mortar. Load-bearing walls, non-load-bearing walls, or other structures built of brick, cinderblock, tiles, adobe, or another form of masonry material that is not braced by reinforcing material such as rebar in concrete or cinderblock are known as "unreinforced masonry buildings."

Masonry buildings were commonly employed in our country due to their affordability. They hold a significant amount of the residential stock. Many are employed in residential unreinforced buildings. One problem stated in this study is masonry’s material properties. For this, samples from an unreinforced masonry building were obtained. They have been brought to Epoka's research facility. Results from tests performed in accordance with Eurocode have been recorded. Then, we can draw a conclusion about the sorts of bricks, concrete, and steel produced in the communist period.

In order to determine the best method to utilize while analyzing our unreinforced masonry building, researchers have looked through the most recent Albanian designing codes and Eurocodes. Following much research, we came to the conclusion that pushover analysis was the most appropriate method for determining the stability of our building.It uses the finite element method while modeling the structure in CDS-Win Software and getting the results we need.Earthquake designaccording to Eurocodes is an important part of Pushover analysis.Some earthquake data from the November 2019 earthquake in Albania will be displayed. This all helps to the primary scope of this study.

The primary scope of this study is the evaluation of the behavior of masonry buildings for various damage limit states through the study of unreinforced masonry schools under designing earthquakes.Their behavior is crucial to study because there are many people who operate inside them.

At the end of this study, we should have found which material types were used in the communism period and their properties. The Albanian Code and Some Eurocode guidelines will be known. Earthquakes in Albania and their damages in years will be clear. The reader should also get help in understanding how to make a pushover analysis for an unreinforced masonry building and get results from it.

1.4 Organization of the thesis

This thesis is divided in 6 chapters. The organization is done as follows:

In Chapter 1, general information, thesis objective, and scope of work are presented. Chapter 2, includes the literature review.Chapter 3,consists of the methodology followed in this study. In Chapter 4, the experimental results in an existing structure are presented. In Chapter 5 recommendations for further research and conclusions are shown and the references.

CHAPTER 2

LITERATURE REVIEW

2.1 Problem statement

Brickmasonry constructions became popular later, in 8350-7350 B.C. at Jericho in Palestine, with some examples of round and oval houses. The first bricks were made of mud or clay, shaped in the desired form, and dried in the sun. After sunburned, bricks were laid on the walls using mud mortar. [O.Korini 2011].

Masonry construction in past years are studied from different engineers of our country . After the 26 November 2019 it became a wide topic to study because the masonry building had major damages as one of the oldest kind of construction of our country and Albania became an experimental territory in the engineering field. Engineers from Italy as Fabio Freddi, Viviana Novelli, Roberto Gentile and others like Enes Veliu, Stoyan Andreev ,Anton Andonov, Federica Greco, Emiljano Zhuleku were in mission for the observations from the 26th November 2019 Albania earthquake.The damages and performance evaluation of masonry buildings constructed in 1970 during this earthquake were studied from Huseyin Bilgin, Neritan Shkodrani, Marjo Hysenlliu, Hayri Baytan Ozmen, Ercan Isik , Ehsan Harirchian. Papers based on the data taken from the damages of this earthquake were written as “The effect of material strength and discontinuity in RC structures according to different site-specific design spectra“ by Ercan Isik , Ehsan Harirchian,Huseyin Bilgin ,Kirti Jadhav ,”Influence of interventions on the seismic performance of URM buildings designed according to pre-modern codes” by Nertitan Shkodrani , Huseyin Bilgin , Marjo Hysenlliu, “Influence of material properties on the seismic response of masonry buildings” by Marjo Hysenlliu , Altin Bidaj, Huseyin Bilgin, “Seismic performance of existing low-rise URM buildings considering the addition of new stories” by Neritan Shkodrani and Huseyin Bilgin. “Comparison of near fault and far fault ground motion effects on low and mid-rise masonry buildings” by Huseyin Bilgin and Marjo Hysenlliu.

Based on this papers and other data taken from other authors and engineers we have studied the behavior of a 3-story building made of brick masonry under the effect of the 26th November earthquake . This study as the above mentioned studies will help on understanding more masonry buildings.

2.2 Definitions of masonry

Masonry is one of the most ancient kinds of human construction. It was first used around ten thousand years ago. Masonry was used to build the Colosseum, the Pantheon, and the Egyptian Pyramids, which are among the most famous monuments in the world. Masonry buildings are also possible because of technological advancements.Masonry can refer to a variety of materials such as tile, brick, stone, concrete blocks, and so on, as well as a mixture of these materials with mortar [1].

Masonry, according to Eurocode 6 [EN 1996-1-1,Section 1.5] , is an assembly of masonry components laid out in a specific design and mortared together. Unreinforced masonry is masonry that does not have enough reinforcement.[4]

2.3 Masonry unit

While talking about masonry units we consider the :

-clay units which will be according to EN 771-1,

-calcium silicate units according to EN 771-2,

-aggregate concrete units (dense and lightweight aggregate) according to EN 771-3;

-autoclaved aerated concrete units according to EN 771-4,

-manufactured stone units according to EN 771-5,

- manufacture, dimensioned natural stone units according to EN 771-6.

Masonry is grouped into several groups according to masonry units. Here are shown the geometrical requirements for grouping masonry units according to Eurocode 6 [EN 1996-1-1,Section 3.1]. There are 4 groups of masonry.

Table.1 Geometrical requirements according to the Groups of masonry [Eurocode 6 , Section 3.1]

<table>
<tr><th rowspan="3"></th><th colspan="8">Materials and limits for Masonry Units</th></tr>
<tr><th rowspan="2">Group 1 (all materials)</th><th></th><th colspan="2">Group 2</th><th colspan="2">Group 3</th><th colspan="2">Group 4</th></tr>
<tr><th>Units</th><th colspan="4">Vertical holes</th><th colspan="2">Horizontal holes</th></tr>
<tr><td rowspan="3">Volume of all holes (% of the gross volume)</td><td rowspan="3">≤ 25</td><td>clay</td><td colspan="2">> 25; ≤ 55</td><td colspan="2">≥ 25; ≤ 70</td><td colspan="2">> 25; ≤ 70</td></tr>
<tr><td>calcium silicate</td><td colspan="2">> 25; ≤ 55</td><td colspan="2">not used</td><td colspan="2">not used</td></tr>
<tr><td>concrete [b]</td><td colspan="2">> 25; ≤ 60</td><td colspan="2">> 25; ≤ 70</td><td colspan="2">> 25; ≤ 50</td></tr>
<tr><td rowspan="3">Volume of any hole (% of the gross volume)</td><td rowspan="3">≤ 12,5</td><td>clay</td><td colspan="2">each of multiple holes ≤ 2 gripholes up to a total of 12,5</td><td colspan="2">each of multiple holes ≤ 2 gripholes up to a total of 12,5</td><td colspan="2">each of multiple holes ≤ 30</td></tr>
<tr><td>calcium silicate</td><td colspan="2">each of multiple holes ≤ 15 gripholes up to a total of 30</td><td colspan="2">not used</td><td colspan="2">not used</td></tr>
<tr><td>concrete[b]</td><td colspan="2">each of multiple holes ≤ 30 gripholes up to a total of 30</td><td colspan="2">each of multiple holes ≤ 30 gripholes up to a total of 30</td><td colspan="2">each of multiple holes ≤ 25</td></tr>
<tr><td rowspan="4">Declared values of thickness of webs and shells (mm)</td><td rowspan="4">No require-ment</td><td></td><td>web</td><td>shell</td><td>web</td><td>shell</td><td>web</td><td>shell</td></tr>
<tr><td>clay</td><td>≥ 5</td><td>≥ 8</td><td>≥ 3</td><td>≥ 6</td><td>≥ 5</td><td>≥ 6</td></tr>
<tr><td>calcium silicate</td><td>≥ 5</td><td>≥ 10</td><td colspan="2">not used</td><td colspan="2">not used</td></tr>
<tr><td>concrete [b]</td><td>≥ 15</td><td>≥ 18</td><td>≥ 15</td><td>≥ 15</td><td>≥ 20</td><td>≥ 20</td></tr>
<tr><td rowspan="3">Declared value of combined thickness [a] of webs and shells (% of the overall width)</td><td rowspan="3">No require-ment</td><td>clay</td><td colspan="2">≥ 16</td><td colspan="2">≥ 12</td><td colspan="2">≥ 12</td></tr>
<tr><td>calcium silicate</td><td colspan="2">≥ 20</td><td colspan="2">not used</td><td colspan="2">not used</td></tr>
<tr><td>concrete [b]</td><td colspan="2">≥ 18</td><td colspan="2">≥ 15</td><td colspan="2">≥ 45</td></tr>
</table>

[a] The combined thickness is the thickness of the webs and shells, measured horizontally in the relevant direction. The check is to be seen as a qualification test and need only be repeated in the case of principal changes to the design dimensions of units.

[b] In the case of conical holes, or cellular holes, use the mean value of the thickness of the webs and the shells.

2.3.1Brick

In the world of construction, brick is the most popular masonry component. Its measurements cannot be larger than 337.5 x 225 x 112.5 mm.[1] A block is what is called when an element's dimensions are exceeded. This kind of masonry unit (brick and blocks) is constructed from calcium silicate, concrete, or baked clay. The most popular material for general construction projects is brick. Face bricks, which come in a variety of textures and hues, are used for both interior and exterior walls. Engineering bricks, also known as dense, strong, and having set limitations of compressive strength and absorption, are used in engineering construction. The brick that is utilized must not have any large, deep cracks or other damage to the corners or edges. They should be free of any lime expansion particle.[1]

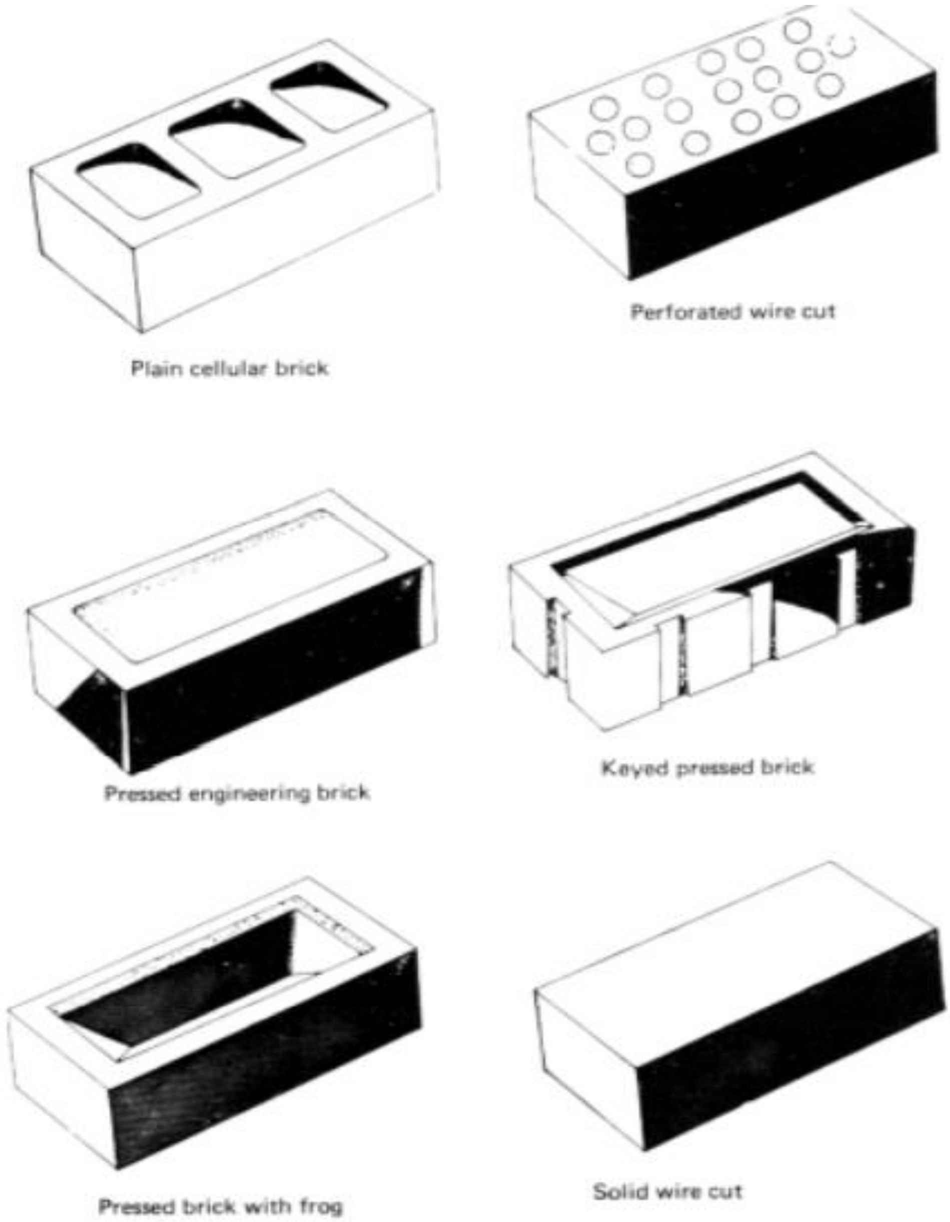


Fig.1 Types of bricks referred to [1]

2.3.2Mortar

The second component in brickwork is the mortar. The cement lime: sand mix should be as shown in the table :

Table 2 Requirements for mortar according to [KTP-89, addition norms]

The designation of the mortar	Types of mortar (proportion by volume)			Mean compressive strength at 28 days (M/mm2)	
	Cement lime sand	Masonry cement sand	Cement sand with plasticizer	Preliminary (laboratory) test	Site tests
I	1:0 to 1/4:3	-	-	16.0	11.0
II	1:1/2:4 to 4 ½	1:21/2 to 3 1/2	1:3 to 4	6.5	4.5
III	1:1:5 to 6	1:4 to 5	1:5 to 6	3.6	2.5
IV	1:2:8 to 9	1: 51/2 to 61/2	1:7 to8	1.5	1.0

According to Eurocode 6 Section 3.2.2 mortar is classified based on their compressive strength in N/mm2. For example M3 means 1:1:3 are the components cement: lime : sand by volume .

When choosing the type of mortar to use, we should consider its short- and long-term aesthetic appeal, workability, water retentivity (the ability to retain water against the suction of the brick), resistance to rain penetration and cracking, proper development of the bond with the bricks, and resistance to frost and chemical attack.[1]

The two types of cement used in mortar are Portland cement and masonry cement. Both non-hydraulic and semi-hydraulic limes are possible. To improve workability, water retention, and bonding properties, lime is added to cement mortar. Useful sand must be crystal clear, razor sharp, and free of salt and organic contaminants. Sand can be found in nature, along with very little clay and silt. Furthermore, contaminants shouldn't be dissolved or suspended in the mortar's mixing water.[1]

For this application, regular drinking is appropriate. Plasticizers can be added to reduce the mortar's cement content or improve its workability. They inhale air. For architectural purposes, tinted mortar is required. When using the pigments, the directions from the fabric are correctly followed. An excessive use of pigment degrades the quality of the construction piece (reducing the compressive strength of the mortar and making weaker the strength between the mortar and masonry unit)[1].

2.4 Earthquakes in Albania and their history

The collision of tectonic plates in the earth's crust causes rocking of the Earth's surface.This phenomenon is called Earthquake. Earthquakes are accompanied by the release of seismic waves that can be longitudinal or transverse waves.

Albania is a complicated area from a geologically and seismotectonically point of view. It is an area that is characterized by a microseismicity developed with small earthquakes, with an average number of earthquakes (M=5.5÷5.9) and large earthquakes (M>6.5). These earthquakes occur generally according to these zones :

-the Ionian Adriatic belt which extends northwest to northwest and coincides with the boundary between the European plate and the Adria microplate.

-the Peshkopia-Korça belt, which extends north-south in the eastern part of the country.

-the Elbasani-Dibra-Tetova transverse belt which extends southwest-northeast across the former two belts. [Bilgin, 2021].

Here is given the seismic map of Albania and its neighborhood.

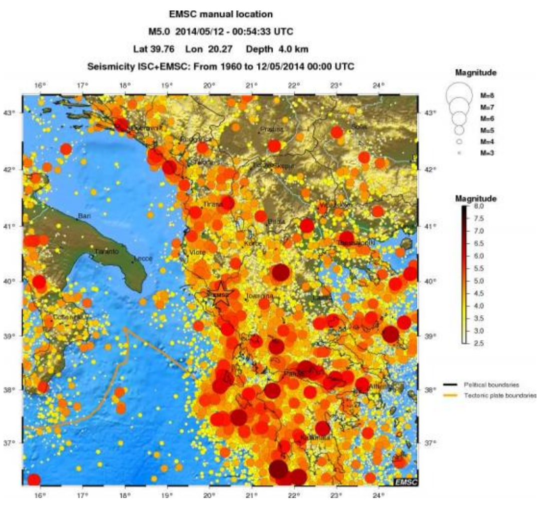


Fig 2.The map of earthquakes in Albania according MSK-64 scale

Albania is a nation situated at the intersection of two tectonic plates, the Euro-Asian one and Adria. As a result of the crash of these 2 plates, an active seismogenic belt has been created which often has generated many earthquakes. The damages caused by earthquakes in most cases have been catastrophic. The evocation of very large damages is related to the fact that these earthquakes have occurred in areas where the population density was high and the constructions made in these areas did not consider enough the seismic risk or the seismic risk of the area has not been properly calculated.

According to the investigations, it is established that along " the seismic zone" Jonian –Adriatic in the northern part of the transversal zone Shkoder-Peje, can occur earthquakes with maximum magnitude M= 7-7.5, but in the south in the frontal zone can occur earthquakes with magnitude M=6.0-7.0. In the direction of Tirana, in the eastern part can occur earthquakes with magnitude M=5.5-6.0. According to studies, only in Albania, the returning period of an earthquake with M=5.0 is 3.6 years, of an earthquake with M=6.0 is 29.1 years, and one of the earthquakes with M=6.5 is 93.9 years and of an earthquake with M=7.0 is 505.6 years.[Marku 2022]

2.4.1Intensity and seismic acceleration

The term 'Intensity' is used to indicate the degree of consequences that the earthquake causes on the population in construction structures and construction sites. Actual or subsequent damages are reflected differently in seismic intensity scales. Such are, e.g. scales MSK-64, EMS-92, and EMS-98, which were conceived and used more in Europe. Also, very famous is the degree of modified Merkal - MM, conceived in 1931 .Seismic scales are divided into 12 scales of intensity, according to MSK-64 (Medvev –Sponheuer –Karnik scale). Referring to European and American engineering, appear interesting in the anti-seismic design, the earthquakes with intensity scale I >7. This is because of major structural damage that starts to appear in construction after this intensity scale.

The conception of seismic scales is based first on the assessment of the intensity according to the descriptions of the surface effects and of the earthquake effects respectively on people, buildings and land or more broadly on the environment.From an engineering point of view, it is very important to correlate these effects with the size of seismic ground accelerations estimated on the basis of instrumental data. This correlation makes more objective the descriptive macroseismic assessment of seismic intensity.

Since the earthquake is a possible event, to make an assessment as accurate as possible, is necessary that statistically to take into consideration a large number of seismic events. For this engineers and scientists built seismic risk maps by contouring the same values of accelerations or PGA of the ground. The value of PGA is known and does not exceed during a certain period of exposure. Engineers consider these seismic risk maps during the designing process. The acceleration in this map is called designing acceleration. More specifically designing acceleration is the acceleration caused by the earthquake with a repeating period of 475 which is also called earthquake design. This acceleration is referred to as seismic ground acceleration which does not exceed 90 % probability, within a period of time t=50 years. The conclusions of each analysis of a given region are collected forming seismic hazard maps.

For the territory of our country, the map of seismic regionalization on the scale exists and is at scale 1:500 000. This map assesses seismic risk based on intensities I according to the MSK-64 scale. Based on it, the maximum expected surface effect of earthquakes, for average conditions can be estimated for the land. (Fig.3)

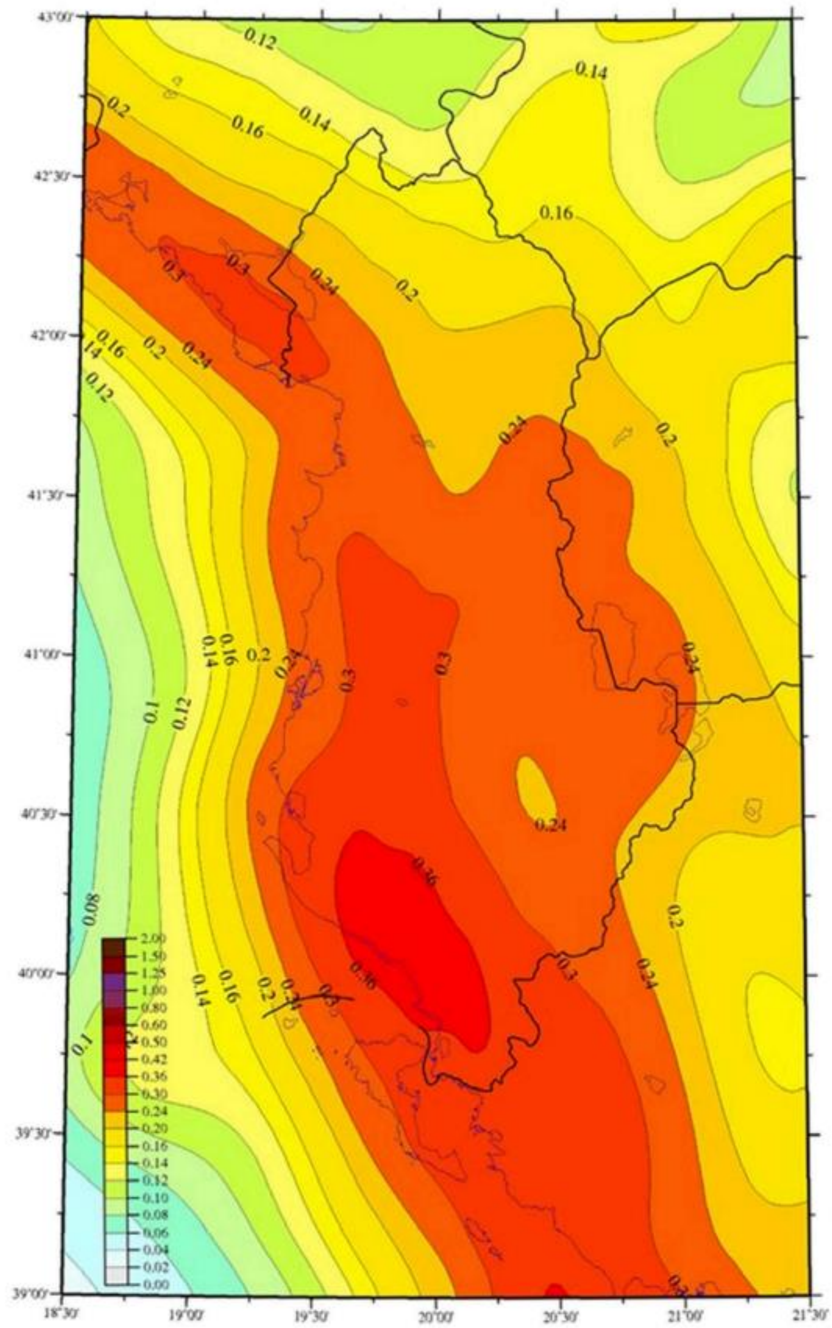


Fig 3. Probalistic seismic hazard map for PGA in “g” with a returning period ,rock soil, [Bilgin, Hysenlliu 2021]

On the map, three main categories of areas with shaking intensity of 6, 7 and 8 can be distinguished. The base intensity given in the seismic map of Albania is specified on the basis of knowledge of the engineering-geological, hydrological and geomorphological conditions of the construction sites.

The territory of the country is divided into seismic zones, which depend on the local risk. By definition, it is assumed that the risk within each zone is constant. For most applications of these Technical Rules risk is described in terms of a single parameter, i.e. of the maximum acceleration value of the ground reference point in ground type A, PGAr.

The ground reference peak acceleration in ground type A, PGAr, can be derived from zoning maps found in the National Annex (Map of Seismic Regionalization of Albania, on a scale of 1:200 000, with recurrence period T = 475 years and the Seismic Regionalization Map of Albania on a scale of 1:200 000, with recurrence period T=90 years).

The reference ground peak acceleration, selected for each seismic zone, corresponds to the reference period of repetition (T_{NCR}) of the seismic action referred to the non-collapse requirement selected. An importance factor γI equal to 1.0 is set for this reference repetition period. For other iteration periods other than the reference one, the design acceleration of the plot in type A, PGA, is equal to the product PGAr with γI (PGA= γI PGAr).

To be considered, cases with very low seismicity are those cases in which the design acceleration of ground type A, PGA, is not greater than 0.4g (0.39 m/s2), completing simultaneously, also the condition that the PGA product is not greater than 0.05g (0.49 m/s2) [Luca 2021, internet data]

2.4.2 Ground condition

In order to design properly engineers study also the ground conditions of the site. They study the site conditions and the nature of the supporting land. There should not be any risk of the nature of the ground crack, continuous sinking caused by liquefaction, slope instability, or compression (densified in the event of an earthquake). We should also determine the seismic action depending on the structure, its importance class and special conditions to conduct studies on the land (geological studies).

Engineers to consider the influence of local ground conditions on seismic action use the ground types A, B, C, D and F and their properties as shown in table

3according to Eurocode.These results are taken considering the impact of deep geology (seismo-tectonics) in seismic actions.

The ground classification table taking into account deep geology (tectonics) can be specified in the relevant technical documents, including the values of parameters S, T_B, T_C and T_D that define elastic, horizontal and vertical spectra.

Table 3The characteristics of ground types [Eurocode 8 section 3.1.2]

Type of ground	The description of the stratigraphic profile	Parameters		
		$v_{s,30}$(m/S)	T_{ncr} (strike/30 cm)	Cu (kPa)
A	Rock or another similar geological structure rock, with no more than 5 meters of weak surface material	> 800		
B	Deposits with extremely compacted sand, gravel, or very hard clays, at least several tens of meters deep, that exhibit a steady rise in mechanical properties with depth	360-800	> 50	> 250
C	Deep deposits containing hard clays, gravel, or half-compacted sand with a thickness of a few dozen to several hundred meters	180-360	15-50	70-250
	Deposits of unrelated lands that are up to half unbound (with or			

D	without some coherent soft binding layers) or lands that are primarily soft (weak) to hardbound	< 180	< 15	< 70
E	A soil profile (land) that is supported by a solid material with vs > 800 m/sec and has an alluvium surface layer with a value of type C and D that ranges in thickness from around 5 m to 20 m			
S_1	Deposits with or containing a layer of soft clays or loams with indications (index), high plasticity (PI>40), and high level subterranean water that is at least 10 meters thick	100 (indicated)		10-20
S_2	deposits of liquefiable soils made of delicate (weak) clays or other types of land (land) excluding S1 or A-E			

Special studies are required in construction sites with land conditions that correspond to two separate types S1 and S2, for the determination of seismic action. For this land especially for S2 must be considered the possibility of destruction (loss of bearing capacity) of the land during the seismic action. If the deposit is of type S1 of the ground special attention is shown.

This types of ground have very low Vs(velocity speed of earthquake) values, a non–ordinary extended order of linear behavior, and internal attenuation. Seismic amplification can be induced with anomalies of the building sites as effects of interaction between the site and the structure. In this case special studies are required.

2.4.3 Earthquake on 26 November 2019, in Albania

An earthquake on 26 November 2021 with epicenter in the north of the city of Durres and with macro seismic effects mostly in the Shijak municipality hit Albania. Its magnitude was Mw 6.4 and the focal depth was 10 km. According to observations and data from various seismological institutes, the primary shock was brought on by the activation of a NW-SE striking reverse fault. Both the primary shock and its aftershocks caused damage to the structures in Durres, Tirana, and other nearby settlements. While Durres (near the epicenter) recorded a value of about 0.20 g, Tirana's horizontal Peak Ground Acceleration (PGA) was only about 0.12 g. [Bilgin,Hysenlliu 2021].

Because of this earthquake,51 people were dead and 193 were injured (255 individuals were hurt during aftershocks of the earthquake)The fatalities were caused mostly by the collapse of 10 buildings in Durres and Thumane. [Marku 2022]

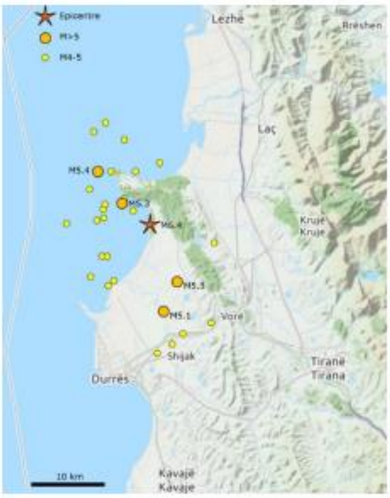


Fig 4. The epicenter and location of aftershocks in the first month of the 26 November 2019 earthquake [Bilgin, Hysenlliu 2021]

Before, on 21 September this zone was hit by an earthquake of magnitude Mw 5.6 with a focal depth of 10 km, injuring 108 people. More than 2000 buildings and 47 educational facilities were damaged, These earthquakes are very important because they increase the vulnerability of buildings and of communities. Most severe damage was in residential buildings where 18% of them in the impacted area were predicted to require reconstruction or restoration. Here are presented some damage statistics in table 4 related to single-family houses, apartment blocks and nonresidential buildings.[Marku 2022]

Table 4 Building damage statistics from three municipalities in the epicentral region [Marku, 2022]

	Durres M	**Kruja M**	**Shijak M.**	**Combined**
Inspected	2112	2499	1670	6281
Safe	1369	1533	346	3247
Unihabitable	651	921	900	2472
Demolition	93	45	424	562
Demolished	34	12	0	46
Safe	64.80%	61.20%	20.70%	51.70%
Uninhabitable	30.80%	36.90%	53.90%	39.40%
Demolition	4.40%	1.80%	25.40%	8.90%

These statistics omit unreinforced buildings. In these conditions, it is challenging to come to firm judgments. From an overall review, engineers get to as conclusion that buildings built before 1992 suffered more damage than the ones constructed after this year. More damages were discovered in low-rise buildings than in high-rise ones (6 or more floors), This happened because low-rise buildings were made of adobe or clay brick and the tall buildings were made of reinforced concrete. The mid-rise buildings (3 to 5 floors) consist of a mixture of structural types and were constructed throughout the two construction eras under examination (communism and democracy era).

Table 5 Tirana Municipality building damage statistics analysis [Marku,2022]

Buildings Characteristics	**Safe**	**Review**	**Evacuate**
Pre-1992	56.30%	23.60%	20.00%
Post-1991	71.60%	15.30%	13.10%
Unclassified	47.00%	16.60%	36.50%
1-2 floor	29.80%	22.10%	48.00%
3-5 floors	60.60%	2340%	16.10%
6-floors	78.00%	14.00%	8.10%
Unclassified	66.30%	12.50%	21.20%
Adobe walls	17.40%	17.40%	65.20%
Brick masonry	56.10%	20.90%	23.00%
Conrete Block masonry	9.10%	9.10%	81.80%
Prefabricated	86.20%	10.30%	3.40%
Reinforced concrete	71.70%	16.00%	12.30%
Structural Masonry	62.80%	21.90%	15.40%
Unclassified	4.00%	36.00%	60.00%

The earthquake that hit on 21 September affected areas such as Tirana, Durres and the surrounding area of these cities causing damage to buildings. The buildings damaged were made of reinforced concrete (RC) and unreinforced masonry, having concrete walls or infill baked clay. They were built according to KTP-Albania Technical Codes, those of the year 1978 then reviewed in the year 1989. The main reasons why these buildings were damaged were: the poor quality of construction. their age, poor workmanship, human intervention, the time period's design code, the absence of maintenance, and poor repair. These buildings had both structural and damage that is not structural, such as the partial or total collapse of masonry load-bearing walls.

2.5 Earthquakes design and other levels of seismic action

In European normative practice (EC8) as a recommended value for the frequency of seismic action of the design or the design earthquake is given its recurrence period RP=475 years. This number corresponds to a probability of 10% of exceeding the design earthquake intensity over a 50-year span.

The designing acceleration PGA design, used as a "input function" in the verification of sufficient resistance and ductility to meet the fundamental design requirements of buildings, which are primarily related to the ultimate limit state ULS, is used as a measure of the intensity of this seismic action in EC8. The earthquake that is handled considerably more lightly is the one that corresponds to the serviceability limit state SLS.

The prospect of a very big seismic activity, sometimes known as the "maximum possible" earthquake, is discussed in European technical literature. Naturally, this is seen as being far rarer than the "design earthquake." Some evidence suggests that the largest earthquake that might occur would have a recurrence period of 1000 years. Additionally, it is advised to select it for an earthquake that is twice as intense as the design earthquake. According to these suggestions, the maximum earthquake acceleration would be 0.5g if the latter corresponds to, for instance, an acceleration of 0.25g. There are no precise definitions in EC8 for this earthquake or the accompanying design requirements.

It may be considered that the following of the specific supplementary constructive rules contained in this Eurocode, help for a possible similar eventual earthquake. Since there is little rare seismic action that is likely to occur during the lifetime of the structure, in cases where this is addressed, act as proof of structural safety, serious structural damage may be justified since the collapse of the building is again avoided and people's lives are guaranteed.

2.5.1 Horizontal elastic response spectrum

These equations for the horizontal seismic motion components yield the elastic response spectrum Se (T):

$0 \leq T \leq TB : Se(T) = ag \,.\, S \,.(1+\frac{T}{Ta}$ ‘ (ŋ*2.5-1)) Equation 2.5.1

TB≤T≤TC: Se(T)=ag. S . η. 2,5Equation 2.5.2

$TC \leq T \leq TD: Se(T) = ag \,.S \,.\, \eta.\ 2{,}5\ [\frac{Tc}{T}]$ Equation 2.5.3

$TD \leq T \leq 4s : Se(T) = ag \,.\, S \,.\, \eta.\ 2{,}5[\frac{Tc*Td}{T}]$ Equation 2.5.4

Se(T) →elasticresponse spectrum

T →period of oscillation of linear system with one degree of freedom

a_g → the design acceleration of the land in type A of the land ($a_g = \gamma_I * a_{gR}$)

T_B, T_C→boundaries of constant spectral acceleration

T_c→values that determine the beginning of the reaction order with constant displacement at spectrum

S→theterrain factor

H →5% viscous damping reference value for the damping correction factor is 1

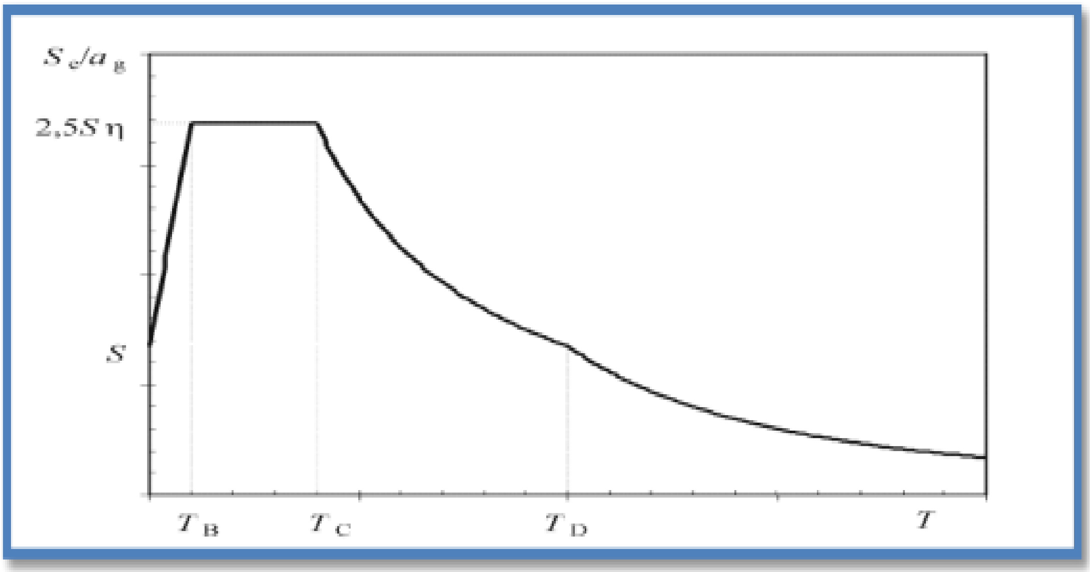


Fig 5. The shape of the elastic response spectrum [Eurocode 8, Section 3.2.2.2.2]

The kind of soil affects the values of the TB, TC, and TD periods as well as the terrain factor S, which determine the form of the elastic response spectrum.

Tab 6. Values for the parameters defining the suggested Type 1 elastic response spectrum [Eurpocode 8, section 3.2.2.2]

Type of soil	S	TB (S)	TC(S)	TD(S)
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.20	0.6	2.0
D	1.35	0.20	0.8	2.0
E	1.4	0.15	0.5	2.0

Tab- 7-Parameter values for the advised Type 2 elastic response spectrum[Eurocode 8 section 3.2.2.2]

Type of soil	S	TB (S)	TC(S)	TD(S)
A	1.0	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

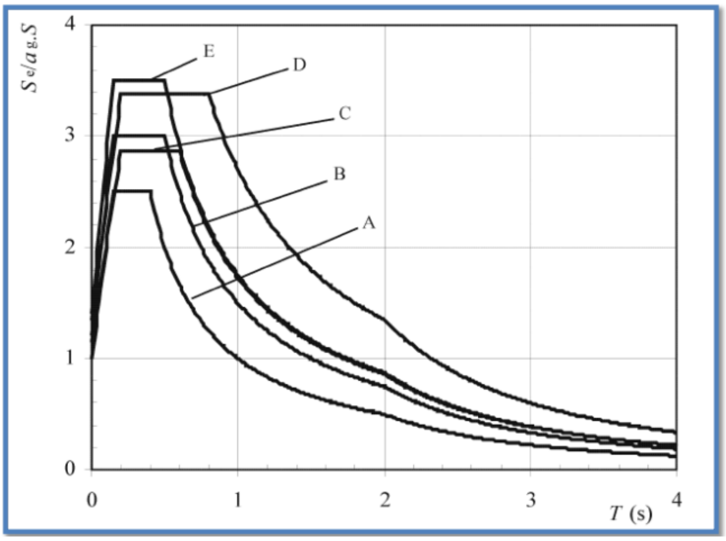


Figure 6For soil types A through E, the Type 1 elastic response spectrum is advised (5% damping). [Eurocode 8 , Section 3.2.2.2]

To determine the values corresponding to size S, TB, TC, and TD for soil types S1 and S2, separate experiments must be conducted.The value of the extinction correction factor η can be determined by the expression:

$$\eta = 10 / (5 + \xi) \geq 0.55 \qquad \text{Equation 2.5.5}$$

where: ξ is the structure's viscous damping ratio, represented as a percentage.

By directly translating the elastic response spectrum of accelerations, Se(T), into the displacement elastic response spectrum, $S_{DE(T)}$, use the following expression:

$$S_{De(T)} = S_{e(T)}\left[\frac{T}{2\Pi}\right]^2 \text{Equation 2.5.6}$$

2.5.2 Vertical elastic response spectrum

An elastic response spectrum, $Sve_{(T)}$, derived using formulae, should serve as a representation of the vertical component of the seismic action:

$$0 \leq T \leq TB : S_{ve(T)} = avg\ .\ [1 + \frac{T}{TB}(\eta \cdot 0{,}3 - 1)] \qquad \text{Equation 2.5.2.1}$$

$$TB \leq T \leq TC : S_{ve(T)} = avg\ .\ \eta\ .\ 3{,}0 \text{Equation 2.5.2.2}$$

$$TC \leq T \leq TD : S_{ve(T)} = avg\ .\ \eta\ .\ 3{,}0\ [\frac{Tc}{T}] \qquad \text{Equation 2.5.2.3}$$

$$TD \leq T \leq 4s : S_{ve(T)} = avg.\ \eta\ .3{,}0\ [\frac{Tc * TB}{T2}] \qquad \text{Equation 2.5.2.4}$$

Two types of vertical spectra are used: Type 1 and Type 2

When earthquakes have a magnitude of surface waves, Ms, not larger than 5.5, as estimated for the construction site, for the purpose of probabilistic risk assessment, the Type 2 Spectrum is employed. The spectra that characterize the component's horizontal seismic action should likewise experience this. The recommended values of the parameters S, TB, TC, and TD characterizing the vertical spectra are provided in Table 8 for each of the five soil types A, B, C, D, and E. The specific land soil types S1 and S2 are not covered by these suggested values.

Tab 8 Recommended values of parameters that describe the vertical elastic response spectrum [Eurocode 8, Section 3.2.2.3]

The specter	Avg/ag	TB(S)	TC(S)	TD(S)
Type 1	0.9	0.05	0.15	1.0
Type 2	0.45	0.05	0.15	1.0

2.5.3 The elastic spectrum for the elastic analysis

Structures should typically be constructed on the basis of forces that are lower than those that correspond to linear elastic analysis due to structural systems' ability to resist seismic activities in the nonlinear phase.

The ability of the structure to dampen the energy is taken into consideration while doing an elastic analysis based on a reduced elastic response spectrum, which will be referred to as the "design spectrum," in order to avoid the explicit structural inelastic analysis during design. By looking at the behavioral factor q's analysis, this reduction is achieved.

The behavior factor, or q, is an approximation of the ratio of seismic forces that might be experienced by a structure if its response were entirely elastic with 5% viscous damping, compared to the lowest seismic forces that could be applied during design, while still ensuring that the structure responds satisfactorily. This analysis is based on the analysis of a conventional elastic model. The behavior factor q values can vary in the various horizontal directions of the structure despite the requirement that the ductility classification be the same in all directions.

The following expression defines the design spectrum Sd(T) for the horizontal components of the seismic action:

$$0 \leq T \leq TB : Sd(T) = ag \cdot S \cdot [\frac{2}{3} + \frac{T}{TB} \cdot (\frac{2.5}{q} - \frac{2}{3})] \qquad \text{Equation 2.5.3.1}$$

$$TB \leq T \leq TC : Sd(T) = ag \cdot S \cdot \frac{2.5}{q} \qquad \text{Equation 2.5.3.2}$$

$$TC \leq T \leq TD : Sd(T) \quad = ag \cdot S \cdot \frac{2.5}{q} \cdot [\frac{TcTD}{T2}] \qquad \text{Equation 2.5.3.3}$$

$$TC \leq T \leq TD : Sd(T) \quad \geq \beta \,.\, ag \quad \text{Equation 2.5.3.4}$$

ag, S , TC, TD   as defined above

Sd(T): design spectrum;

q = behavioral factor;

β = factor of the lower bound on the horizontal spectrum of the design

The value for the β factor is =0.2

The formulae above, using PGA_d to replace size PGA and 1.0 as the design acceleration of the ground in the vertical direction, give the design spectrum for the vertical component of seismic action.

For all materials and structural systems, a factor q of 1.5 will often need to be adjusted for the vertical component of the seismic action.

Relevant analyses must be used to support the adaptation of values of q greater than 1.5 for the vertical direction.

2.6 Albanian designing Code for masonry structures

2.6.1 Albanian designing Code KTP-78 for masonry

In these technical codes is presented the method of calculating the foundations and wall sections. The calculation of these two elements is specified in this technical code using the ultimate limit states. Loads and their combination are taken according to the instruction techniques that are defined in KTP 6-78. All masonry cases are specified in this code. The following list of recommendations is most crucial. By multiplying the standardized resistance by the homogeneity coefficient, the characteristic resistance of the masonry in the final product calculated. This value fluctuates depending on the element's stress state and the kind of materials used to build the wall.

Table9 Compressive strength for brick masonry (Marku.2022)

Nr. Brick Class Kg/cm2		**Mortar class kg/cm2**						
		100	75	50		15	4	0
1	150	22	20	18	15	13.5	12	8
2	100	18	17	15	13	11	9	6
3	75	15	14	13	11	9	7	5
4	50	**-**	11	10	9	7.5	6	3.5

Table 10 Compressive strength for concrete block

Nr.	**Brick Class Kg/cm2**	**Mortar class kg/cm2**						
		100	75	50	25	15	4	0
1	100	20	18	17	16	14.5	13	9
2	75	16	15	14	13	11.5	10	7
3	50	12	11.5	11	10	9	8	5

Table 11Compressive strength for stone wall and foundation

Nr.	**Stone class kg/cm2**	**Concrete class kg/cm2**		
		100	75	50
1	Above 100	20	18	17
2	Under 200	16	15	14

2.6.1.1 Seismic design according to KTP-78

Engineers should use limit state design while designing in concrete, steel, wood and masonry. The design should be done using dynamic theory. It takes into account the deformed shape of the structure, the free and imposed vibrations ecc. While designing with the seismic design we should consider these principles :

-the proper resistance distribution

-the proper distribution of the structural mass and stiffness

-the design should be made for the most favorable mechanisms during plastic deformations

-it should be designed for stability even after the partial collapse of the structure

-the design of the seismic joint should beas :

The seismic junction must be at least 3 cm broad and up to 5 m high.

When it comes to taller structures, the width should be increased by 2 cm for every 5 m increase in height.

-the load combination includes :

1) Seismic force is regarded as a unique force.

2) Variable load in a seismic combination will be compounded by 0.8.

2.6.2Albanian designing Code KTP-1989 for masonry buildings

This code is the next Technical design code for seismic resistance after the one in 1978.It is a major upgrade compared to the previous code and includes the first designing codes based on contemporary European Codes.Some of the guidelines this Code includes are as follows :

1)For retaining walls,with seismic intensity VII ,VIII and II magnitude are used this type of materials:

-bricks with a class lower than 75

-concrete blocks not lower than 100 with a net surface not lower than 60%

-have stones not lower than 200

-masonry mortar with a class not lower than 15

2) The use of bricks with longitudinal gaps that pass horizontally or across the wall is not allowed in the retaining walls.

3) For partition and non-bearing walls, the use of bricks with longitudinal gaps is allowed.

The masonry of the 3-d category is allowed only for buildings and works of group > IV according to the importance scale.

4) The categorization of retaining walls

Retaining walls are considered the walls that have a thickness minimum of 25 cm; when made of bricks, 50 cm when made of stone, and 25 cm when made of concrete blocks. For the masonry with concrete blocks with hollows over 25 %, in calculations is taken the actual neto surface of the cross-section.

According to the center of the seismic action, the retaining walls are divided into 3 categories:

Table 12 Types of masonry (retaining walls)

NR	Type of masonry			
1	Masonry with clay bricks, full, or hollow bricks with vertical holes	I	II	III
2	Masonry with silicate bricks	II	III	-
3	Masonry with concrete blocks of a grade not lower than 100	II	III	-
4	Masonry with gravestones of the brand not lower than 200 a) With regular form	II	III	-
	b) With unregular form	III	-	-

The distance between the axes of the transverse walls or the frames that replace them is checked based on the corresponding calculations made to the walls between those axes.

For buildings with retaining walls with non-complex construction, these distances should not be greater than the values in table 13.

Table 13 the limit height in (m) depending on the seismic intensity in magnitude

The seismic intensity in magnitude	The limit height in (m) ofthe transverse walls when the longitudinal walls are of the same category		
	I	II	III
VII	11	10	4.5
VIII	9	8	4.5
II	7	5	4.5

For buildings with retaining walls with complex construction, the boundary distances given in table 13 are multiplied by the coefficient 1.5.

2.6.2.1Some border dimensions in buildings with retaining walls.

In buildings with retaining walls, the dimensions in (m) of the wall elements must be determined in the calculation.

In any case, the above-mentioned dimensions or their ratios must meet the requirements of the table14.

Table 14 Dimensions of the retaining walls depending on the seismic intensity in magnitude.

Nr	The dimensions of the wall elements	The seismic intensity in magnitude			Notes
		VII	VIII	II	
1	The width between doors and windows should be: - for walls of category I not	1.0	1.0	1.25	These limit dimensions are not valid when the wall element is bordered by

2	greater than: - for walls of categories II and III not greater than The distance from the corner of the building to the space closest to it, not less than	1.0 1.25	1.25 1.5	1.5 1.75	reinforced concrete reinforcing columns These limit dimensions are not valid when the wall element is bordered by reinforced concrete reinforcing columns
3	The ratio of the length of the wall between two spaces (doors or windows) against the greater width must not be smaller than:	0.33	0.50	0.75	
4	The width of the door and window openings will not be greater than	3.5	3.0	2.5	For greater width, the spaces must have reinforced concrete frames

					connected to the walls. In height, the anti-seismic bands of the floors should not be interrupted
5	The exit of the walls in the plan should not be greater than	2	1		At the exit of the walls is understood the continuation of the internal walls.
6	The exit from the wall side of the frames should be: - for brick frames no larger than: - for reinforced concrete frames connected to the anti-seismic belt no larger than: - for wooden frames plastered in metal nets not larger than:	0.2 0.4 0.75	0.2 0.4 0.75	0.2 0.4 0.75	The exit of unplastered wooden frames is allowed up to 1.0 m.

	the distance between the inner wall and the face (from the side of the space) of one of its transverse walls, must not be smaller than:	0.25	0.25	0.25	

2.6.2.2Floor height of buildings with retaining walls.

Table 15 Floor heights of buildings with retainingwalls

Limit values of floor heights, h (m)			
The seismic intensity in magnitude	Category of wall	Wall thickness m (cm)	
		25	38
VII	I , II and III	3	4.5
VIII	I	3	4.0
	II and IIII	3.0	3.8
II	I	2.8	3.6
	II and III	2.8	2.9

The ratio between the height of the floor and the thickness of the wall should not be greater than 12.The brick columns must be built in category I masonry and with a height not greater than 4.0 m. The brick columns must not be weakened as a cross-section in their height.

2.6.2.3Method of calculation according to KTP-1989

Calculation of buildings from engineering works to seismic actions isdone :

-based on the modal analysis with the response spectrum method, according to paragraph 2.6 of KTP-1989 (calculated seismic loads determined according to this method are accepted as equivalent static loads and are applied instead of concentrated measures)

-by direct use (through the integration of the equations of motion) by choosing the computational accelerograms on the basis of studies of the seismicity of the construction site, its geomorphological and geotechnical features, meanwhile, the largest acceleration amplitudes in the selected accelerograms should be taken not less than k_E, where K_E-Coefficient of the seismicity of the building site, while g-the acceleration of free fall.

-calculations based on modal analysis with the response spectrum method must be performed for all buildings and engineering works.

-calculations using direct dynamic analysis can be made for the design of buildings and engineering works of particular importance. The stress (deformation) values that result according to this method should not be accepted as less than 70% of the values determined based on the modal analysis with the reaction spectrum method.

2.6.2.4Calculation of structures with retaining walls with complex and non-complex construction

Buildings with retaining walls are calculated for the simultaneous action of seismic forces, according to the horizontal and vertical direction (seismic actions should be taken into account separately)

Horizontal seismic forces are determined :

$E_{ki}=K_E* K_R*\psi*ßi*\eta_{ki}*Q_k$ Equation 2.6.2.4

K_E,K_R,ψ-are coefficient determined in paragraph 2.6.4 of KTP-89

ßi –the dynamic coefficient

η_{ki} – The coefficient of the distribution of the seismic load

Vertical seismic forces are determined as follows:

a)For the zones with seismic intensity VI ½ and VII magnitudes as much as 15% of the corresponding vertical static load

b) For the zones with seismic intensity VII magnitudes as much as 30% of the corresponding vertical static load

2.6.3 Eurocode 6 for masonry buildings

Eurocode 6 is a part of European contruction normatives. It includes the design of buildings, and civil engineering works in unreinforced, reinforced, prestressed and confined masonry. So the design of masonry structures, should be according to EN 1990. Eurocode 6 is made up of the following components:

Part 1-1, General guidelines for both reinforced and unreinforced masonry constructions

Part 1–2, Structural fire design

Part 2 Design factors, material choices, and masonry work execution

Part 3 For unreinforced masonry constructions, more straightforward calculation techniques

Also, each section has a National Annex (NA) that lists the Nationally Determined Parameters (NDPs) to be used when using Eurocode 6.Moreover, PD 6697 offers helpful advice that is a complement to Eurocode 6.The purpose of Eurocode 6 is to be used in conjunction with

Eurocode: Basis of structural design,

Eurocode: Actions on structures, and, where necessary, other Eurocodes and other European Standards.

The European Committee for Standardization (CEN), committee TC/125, has published European standards for these materials as part of a range of masonry-related standards.

Masonry buildings must be created in compliance with the broad guidelines stated in Eurocode, which mandates that:

$$E_d \leq R_d \qquad \text{Equation 2.6.3.1}$$

Where E_d= design value of the actions' effects

R_d=resistance specified in the design

Whenthe following criteria are met, the fundamental requirements of Section 2 of the Eurocode are regarded to be satisfied for masonry structures:

- Combining limit state design with the partial factor approach as defined in Eurocode.
- Actions in accordance with Eurocode 1
- Combination guidelines as stated in the Eurocode
- The values and guidelines provided in Eurocode 6 for implementation

Thedesign value for a material property is then determined using the partial

factor approach by multiplying its characteristic value by the appropriate partial factor for materials, as shown below:

$$R_d = R_k / \gamma_M \qquad \text{Equation 2.6.3.2}$$

Where R_d= design value of resistance

R_k= characteristic value of resistance

γ_M=partial factor for a material property

Table 16 Values of γ_M for the ultimate limit state

Values of γ_M for ultimate limit state

Material	Class of execution control γ_M	
	1[a]	2[a]
Masonry		
When in a state of direct or flexural compression		
Unreinforced masonry made with:		
Units of category I	2.3[b]	2.7[b]
Units of category II	2.6[b]	3.0[b]
Reinforced masonry made with mortar M6 or M12:		
Units of category I	2.0[b]	–[c]
Units of category II	2.3[b]	–[c]
When in a state of flexural tension		
Units of category I and II	2.3[b]	2.7[b]
but in laterally loaded wall panels when removal of the panel would not affect the overall stability of the building	2.0[b]	2.4[b]
When in a state of shear		
Unreinforced masonry made with:		
Units of category I and II	2.5[b]	2.5[b]
Reinforced masonry made with mortar M6 or M12:		
Units of category I and II	2.0[b]	–[c]
Steel and other components		
Anchorage of reinforcing steel	1.5[d]	–[c]
Reinforcing steel and prestressing steel	1.15[d]	–[c]
Ancillary components – wall ties	3.0[b]	3.0[b]
Ancillary components – straps	1.5[e]	1.5[e]
Lintels in accordance with EN 845-2[12]	See NA to BS EN 845–2[f]	See NA to BS EN 845–2[f]

Key

A*When the work is completed in accordance with the workmanship recommendations in BS EN 1996-2, including suitable supervision and inspection, Class 1 of execution control should be assumed.*

i*The construction is suitable with the application of the relevant partial safety factors listed in BS EN 1996-1-1 because of the specification, monitoring, and control.*

ii*If the mortar was manufactured, it complies with BS EN 998-2. If the mortar is site-mixed, preliminary compressive strength tests are conducted in accordance with BS EN 1015-2 and 1015-11 on the mixture of sand, lime (if any), and cement that is intended to be used. The proportions given in Table NA.2 of Eurocode 6 may be initially used for the tests to confirm that the strength requirements of the specification can be met; the proportions may need to be changed to achieve the* *required strengths, and the new proportions are then to be used. To ensure that the required strengths are being met, samples of the site mortar are subjected to regular compressive strength testing.*

Assume Class 2 execution control whenever the job is completed in accordance with the BS EN 1996-2 recommendations for craftsmanship, including adequate supervision.

b*These values might be cut in half when taking into account the effects of abuse or accidents.*

c*For reinforced masonry, Class 2 execution control is not thought to be appropriate and should not be applied. Nevertheless, bed joint reinforcement was employed to reinforce brick wall panels.*

i*to increase the masonry panel's lateral strength*

ii*to limit or regulate shrinkage or expansion of the brickwork, can be regarded to be unreinforced masonry for the purpose of class of execution control and the unreinforced masonry direct or flexural compression The use of* γ_M *values is suitable.*

d*These numbers should be treated as 1.0 when assessing the impacts of abuse or accident.*

e*Unless otherwise stated, the reported ultimate load capacity for horizontal restraint straps relies on the brickwork having a design compressive stress of at least 0.4 N/mm2. When using autoclaved aerated concrete or lightweight aggregate concrete masonry, for example, when a lower stress from design loads may be acting, you should seek the manufacturer's advice and utilize a partial safety factor of 3.*

f*Not yet released.*

Themasonry unit manufacturer shall announce the two recognized levels of attestation of conformance, Category I and Category II. Moreover, there are two recognized classes of execution control: 1 and 2.

According to the Eurocodes**,** an action is a collection of forces, deformations, or accelerations operating on a structure, this includes horizontal and vertical loads.The ultimate limit state, STR (which denotes an internal failure or excessive deformation of the structure or structural member), will typically be used for masonry design (with the exception of retaining structures), though there are many other combinations of actions that are described in Eurocode.The STR limit state Expression (6.10), which is always conservative, or the most difficult Expression (6.10a), or both, can be employed in one of three ways (6.10b).Expression (6.10) alone will be sufficient for laterally loaded masonry walls, where self-weight is typically advantageous. Expression (6.10b) is the most cost-effective of the three expressions for members supporting one variable action (aside from storage loads) and can be used for vertically loaded walls, provided that the permanent actions are not more than 4.5 times the variable actions.[Introduction to Eurocode 6]

When a variable action acts in conjunction with another variable action (i.e., when it is an accompanying action), a factor called ψ_0, shown in Table 18, lowers the design value of the variable action. Table 18 can be used to determine the value of ψ_0.

Expression (6.10)can be implemented using the UK NA to Eurocode values, as indicated in Table 17 along with the variables to be employed when wind loads interact with imposed loads. Keep in mind that both imposed loads and wind loads are regarded as variable activities.

Table 17 Design values of actions , ULS (Table A1,2(B) of Eurocode)

Combination Expression reference	Permanent actions		Leading variable action	Accompanying variable action
	Unfavourable	Favourable		
Exp. (6.10)	$\gamma_{G,j,sup} G_{k,j,sup}$	$\gamma_{G,j,inf} G_{k,j,inf}$	$\gamma_{Q,1} Q_{k,1}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$
Exp. (6.10a)	$\gamma_{G,j,sup} G_{k,j,sup}$	$\gamma_{G,j,inf} G_{k,j,inf}$	$\gamma_{Q1} \psi_{01} Q_{k1}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$
Exp. (6.10b)	$\xi\gamma_{G,j,sup} G_{k,j,sup}$	$\gamma_{G,j,inf} G_{k,j,inf}$	$\gamma_{Q,1} Q_{k,1}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

Notes

G_k = characteristic value of a permanent load
Q_k = characteristic value of the variable action
$\gamma_{G,sup}$ = partial factor for permanent action upper design value
$\gamma_{G,inf}$ = partial factor for permanent action lower design value
γ_Q = partial factor for variable actions
ψ_0 = factor for combination value of a variable action
ξ = reduction factor/distribution coefficient
Where a variable action is favourable Q_k should be taken as 0.

Table 18 Recommended combination values of variable actions (ψ_o) buildings(from UK National Annex to Eurocode)

Action	ψ_0
Imposed loads in buildings (see BS EN 1991–1–1)	
Category A: domestic, residential areas	0.7
Category B: office areas	0.7
Category C: congregation areas	0.7
Category D: shopping areas	0.7
Category E: storage areas	1.0
Category F: traffic area, vehicle weight < 30 kN	0.7
Category G: traffic area, 30 kN < vehicle weight < 160 kN	0.7
Category H: roofs [a]	0.7
Snow loads on buildings (see BS EN 1991–3)	
For sites located at altitude H > 1000 m above sea level	0.7
For sites located at altitude H < 1000 m above sea level	0.5
Wind loads on buildings (see BS EN 1991–1–5)	**0.6**

Key

a See also 1991–1–1: Cl 3.3.2

While prescribed mortars employ predetermined proportions, designed mortars use the compressive strength of the mortar to manage the quality of the hardened mortar. The compressive strength of a mortar developed in accordance with BS EN 1015-214 should not be less than the declared compressive strength when samples are taken and tested in accordance with BS EN 1015-1115.

Simplified calculation techniques for unreinforced masonry structures are found in Eurocode 6, Part 3. These techniques are not to be confused with straightforward rules derived from experience; rather, they are founded on the ideas presented in Part 1.

Table 19 Maximum horizontal distance between vertical movement joints in walls (in the absence of other guidance from the manufacturer)

Type of masonry	l_m (m)
Clay masonry – unreinforced	15[a]
Calcium silicate masonry	9 [b]
Aggregate concrete and manufactured stone masonry	9 [b]
Autoclaved aerated concrete masonry	9 [b]
Natural stone masonry	20 [c]

Key

a The value for clay masonry walls containing bed joint reinforcement may be greater than 15 m subject to expert advice.

b This value applies when the ratio, length to height of panel, is 3 to 1 or less. It should be reduced for long horizontal panels of masonry which lie outside this ratio.

c When using this figure, movement joints should be located at not more than 8 m from the corner.

Table 20 Permissible deviatioons for structural design purposes

Position	Maximum deviation
Verticality	
In any one storey	± 20 mm
In total height of building of three storeys or more	± 50 mm
Vertical alignment	± 20 mm
Straightness [a]	
In any one metre	± 10 mm
In 10 metres	± 50 mm
Thickness	
Of wall leaf[b]	± 5 mm or ± 5 % of the leaf thickness, whichever is the greater
Of overall cavity wall	± 10 mm

Key

a Deviation from straightness is measured from a straight reference line between any two points.

b Excluding leaves of single masonry unit width or length, where the dimensional tolerances of the masonry units govern the leaf thickness.

Results of tests conducted in accordance with BS EN 1052-15 are used to estimate the typical compressive strength of masonry (other than shell bedded masonry). Instead of the storey-height panels utilized in the past, the testing are now conducted on small wallette examples.

The designer might choose to use values obtained from a database or test the units that are intended to be used in a project. The constants to be utilized in the calculation below are values from a huge database that are published in the UK NA to Eurocode 6, Part 1-1:

$$f_k = K\, f_b^{\alpha} f_m^{\beta} \qquad \text{Equation 2.6.3.3}$$

where :

f_k= The masonry's typical compressive strength, expressed in N/mm2,

K= constant

α ,β = constant

f_b = N/mm2 is the units normalized mean compressive strength when applied in the direction of the action effect.

f_m= mortar's compressive strength, expressed as N/mm2

A precise value for K to be utilized in Equation (2.6.3.3) of Eurocode 6, Part 1-1 for blocks laid flat can be found in Table 8 of the National Appendix to Eurocode 6, Part 1-1.

Equation (2.6.3.3) is restricted in the following ways:

-The masonry is designed and built in line with BS EN 1996-1-1, section 8 criteria.

-When laying units in general-purpose mortar,f_b is assumed to be no more than 110 N/mm2, and 50 N/mm2 when laying units in thin layer mortar (f_b is calculated in the typical loading direction).

-When units are laid in general purpose mortar or lightweight mortar, f_m is taken to be no more than f_b or 12 N/mm2, or 10 N/mm2, respectively.

-The masonry unit's strength has a coefficient of variation no more than 25%.

Adjustments are made to the value of K for masonry constructed with all-purpose mortar.Also it is important to remember the following:

-The value of f_b should be obtained by assuming the units to be Group 1 having a compressive strength corresponding to the compressive strength of the units or of the concrete infill, whichever is the lesser, for masonry made of general purpose mortar where Group 2 and Group 3 aggregate concrete units are used and the vertical cavities are completely filled with concrete.

-The unit shape factor correction for collar joined aggregate concrete masonry made with general purpose mortar, with or without the collar filled with mortar, should use the breadth of the wall as the unit width and the height of the masonry units to obtain the normalized strength.

-When action effects are parallel to the direction of the bed joints and the direction of the load applied to the test specimens coincides with the direction of the action effect in the masonry, the characteristic compressive strength may be calculated using Equation (2.6.3.3) with the shape factor, d, as given in BS EN 772-1 taken to be no greater than 1.0. K should then be increased by 0.5 for Group 2 and Group 3 units.

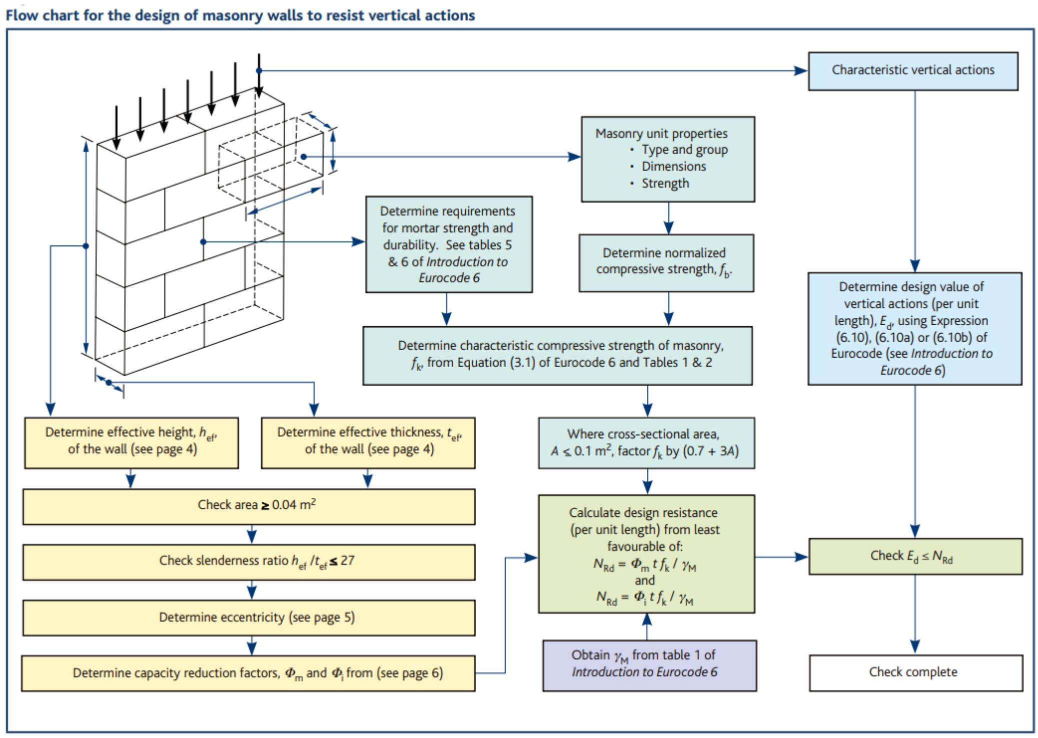


Fig.7 Flow chart for the design of masonry walls to resist vertical actions

-Two methods are provided by Eurocode 6 for designing lateral loaded panels. The first approach uses yield line analysis to calculate the bending moment coefficients and depends on the masonry's flexural strength. The second technique takes the stance that a three-pinned arch will form within the wall and is based on the arching principle. In this manual, both approaches are presented.

The following are the bending moments per unit length (M_{Ed}) for panels without openings:

$$M_{Ed1} = \alpha_1 W_{Edl}^2 \qquad \text{Equation 2.6.3.4}$$

when the bed joints are parallel to the plane of failure:

Where :

α_{1} = indicator of the bending moment parallel to the bed joints

α_2= the bed joints' perpendicular bending moment coefficient

W_{Ed}= calculated wind load per area ($\gamma_Q W_k$)

l = panel's distance between supports

μ = orthogonal ratio (f_{xk1}/f_{xk2})

A panel’s flexural strength increases in the direction parallel to the bed joints when there is a vertical force present. The following factors determine the design moment of resistance within the wall's height :

$$M_{RD} = (f_{xk1}/\gamma_M+\sigma_d)Z \qquad \text{Equation 2.6.3.5}$$

Where:

f_{xk1}= Masonry's typical flexural strength when bent around an axis parallel to bed joints

γ_M= the right partial factor for the materials

σd= Create a vertical load per region. ($<0.2\ f_k/\gamma_M$)

Z= section modulus of the wall's plan form

f_k=characteristic compressive strength

Flow chart for the design of masonry walls to resist lateral actions

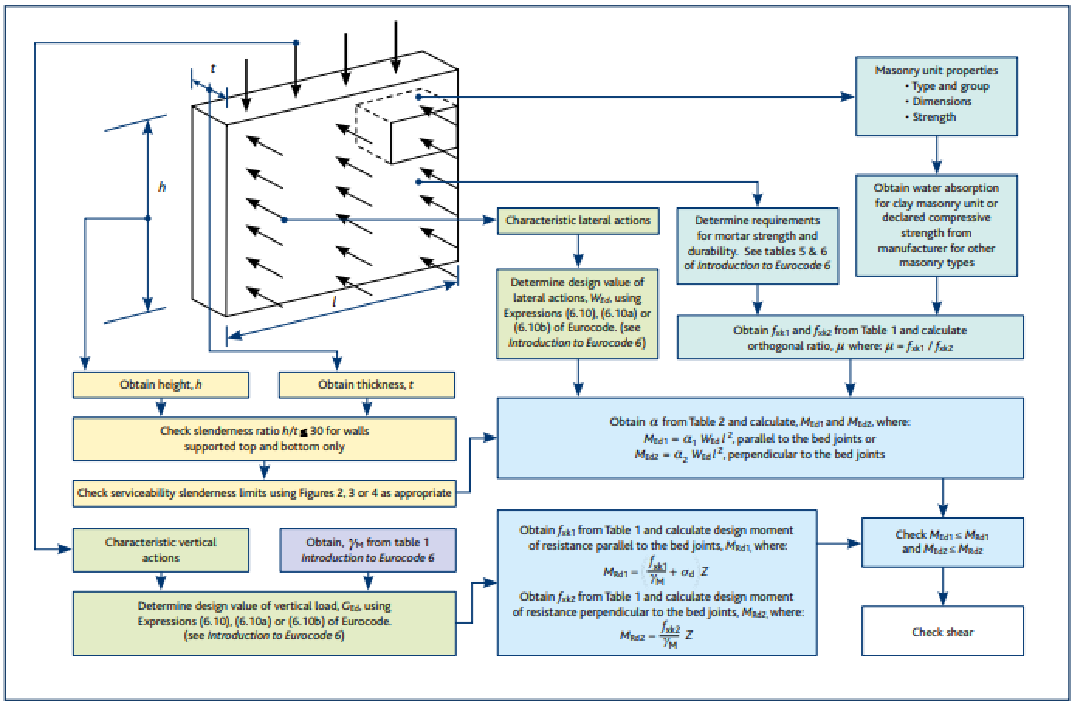


Fig.8 Flow chart for the design of masdonry walls to resist laterat actions

The iterative design process can be summed up as follows:

1. Assume the support condition initially.
2. Establish assumptions regarding the strength and thickness of the masonry unit that is needed; the thickness of a cavity wall's minimum leaf is 100mm.
3. Check serviceability slenderness limits. For wall panels supported top and bottom only, h should be limited to 30t.
4. Find the orthogonal ratio,, and bending moment coefficient that are suitable for the panel shape.
5. Calculate the applied moment's design value, M_{Ed}
6. Verify the moment of resistance's design value.M $_{Rd}$
7. If M_{Ed}> M_{Rd}, the wall is okay; otherwise, go back to step 1 or step 2 and make changes.
8. Shear check

The structure should be verified in the ultimate limit state and serviceability limit state.

The USL (The Ultimate limit state) is a physical situation of a structure that has excessive deformations. These deformations lead to the collapse of the element studied or the structure as a whole. Deformations here exceed the pre-agreed values.

The Serviceability Limit State (SLS)is a state of design of a structure or a building element beyond which the element or the structure loses operationally its serviceability for the actual service load that the structure is designed to.

The structure should remain undamaged under the shaking from the designated earthquake while calculated for the ultimate limit state and serviceability of the limit state.

CHAPTER 3

METHODOLOGY

3.1Finite element modeling

Masonry is a type of building material comprised of masonry units (brick, block, and stone) and mortar, where mortar serves as the bonding agent and significantly affects how the wall behaves. When analyzing, there are two masonry modeling methods: micro and macro modeling (Lourenco,1996).

Brick and mortar as well as the interaction between them are precisely detailed by micro modeling. Large computer models are needed for this technique, and their development, calculation, and processing all take a lot of time. The macro modeling methodology, which is faster but less precise than the prior one, is seen to be more practical. It frequently delivers acceptable outcomes in less time.

For the sake of computer computations, "homogenization" is the process usedto combine the three components (masonry unit, mortar, and their interface) into a single unit.[Marku, 2022]

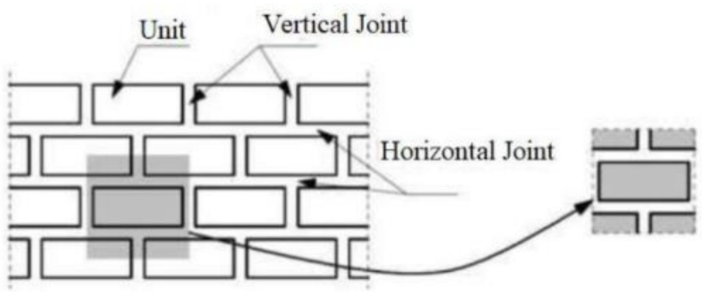


Fig 9. Homogenization procedure [Marku,2022]

In engineering, it is essential to determine a building's ability to withstand earthquakes and how it will react to ground movement. Engineers, therefore, utilize nonlinear processes in accordance with national standards such as ATC-40, developed in 1996, FEMA-356, 2000, FEMA-440, 2005, the N2 Method, and Eurocode in order to get the right results.

We can predict the performance of the structure by utilizing nonlinear analysis in the form of a capacity curve.

The complexity of masonry's behavior, which differs in different directions, has made modeling difficult for many engineers over the years. The wall's vertical direction can support static vertical loads (the higher static loads). Making the best use of the bricks' shearing capacity requires alternating the heights and placement of the bricks. Due to a lower bearing capacity on the cohesive bond between brick and mortar than in the vertical direction, this does not occur in the horizontal direction. Tensile stress values from bending at the on and outside plan of the wall are lower. Due to these structures' significant rigidity, bending is typically not sensitive. The greatest risks to masonry structures are earthquakes. If not correctly constructed and built, a structure can collapse during an earthquake in a matter of seconds. Shear strength is the masonry's ability to support weight in direct opposition to seismic forces.

We have three options for modeling masonry constructions with computers: 1) using linear components 2) using planar components 3) containing items in three dimensions. The engineers give the structure the rigidity it needs to accurately mimic the masonry's working conditions by using linear parts. [Marku, 2022].

In laboratories, it was tested on smaller models. Engineers create models in finite element programs with appropriate coefficients based on test results, making it feasible to compare the outcomes of computer analysis with those of laboratory experiments. These modeling parameters enable the analysis of real structures.We have countless modeling options with plan elements thanks to CDS Win software with finite elements. Methods and regulations in accordance with Eurocode that employ a nonlinear behavioral plan of elements are put into practice.

Due to the time required, the three-dimensional finite element method is less commonly used. This model uses the nonlinear behavior in each direction and divides the wall into bricks.

3.2Non linear modeling of masonry in CDS Win principle

CDS Win Software is a powerful calculation software, which uses the Eurocode normative and allows to carry out the analysis of structures while using the most sophisticated FEM techniques. (Finite Element Method).

Due to the time required, the three-dimensional finite element method is less commonly used. This model uses the nonlinear behavior in each direction and divides the wall into bricks.

To replicate brickwork, a shell element with nonlinear behavior layers is employed. The layers will show masonry characteristics in axial compression and shear.The planning element in an elastic analysis has three or four nodes and is stiff both inside and outside of the plane, in bending, compression, tension, and shear. This kind of composition might be layered or homogeneous. This plan element will not be used since the masonry is not homogeneous or isotropic.

The layered shell element specifies how many layers there are in the width direction, each with its own position, thickness, behavior, and material. Materials may not behave linearly. The degree of torsion freedom in a plan is not employed and should not be taken into account while determining bearing capacity. The rotations perpendicular to the plane of the element is fixed to avoid instability. Displacements outside of the plane are consistent with those in the plan. Layers can be used to represent the degrees of freedom of a model, with "Shell" layers typically.[Marku,2022]

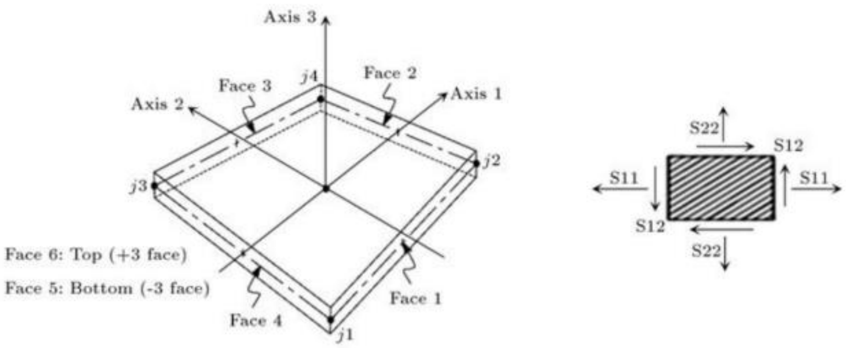


Fig 10. The plan element with 4 nodes and stresses in plan (Marku 2022)

We shall combine parts of the "Shell" type with nonlinear analysis. Two different stresses-strain graphs will be utilized to model the behavior of the masonry. The shear stress is denoted by S12, the horizontal stress by S22, and the vertical stress by S11. The most crucial stresses of the brickwork's behavior are those mentioned above. We can accurately estimate the stresses-strain graph in each direction using this method. So that earlier studies can be ready, we should look into masonry strengthening. The following describes a few of them.

Graphs of stresses and strains for directions S11 and S22

Researchers like Kaushik defined the stress-strain relationship based on laboratory measurements in 2007.[Marku,2022]

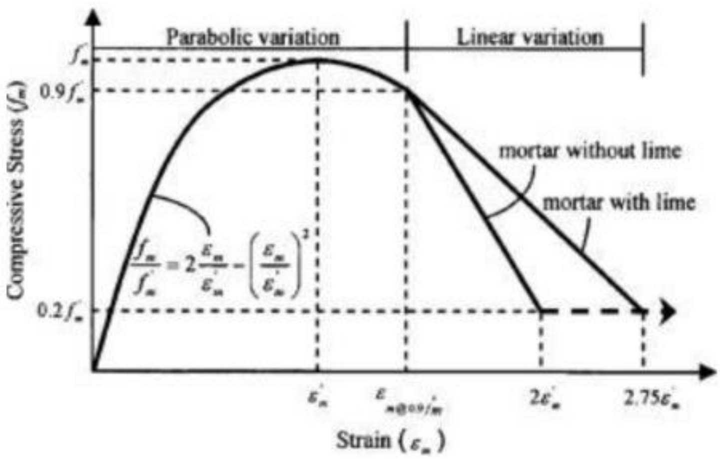


Fig 11 Stresses –strain graphs for directions S11 and S22

In this instance, Eurocode 6 does not consider the tensile strength. Calculations will be performed using a straight line with a zero value on the opposite side of the diagram.The nonlinear behavior of a masonry component up to the point of horizontal destruction is shown in this graph. When masonry is exposed to horizontal ground shaking, the cohesiveness and roughness between the bricks and mortar are described as the horizontal resistance force in the literature. Mohr-Columbus shear stresses are the following:

$\Gamma = c + \sigma tg\ \Phi$Equation 3.2.1

The vertical strain and the friction between the elements are depicted in these expressions as tg. The cohesiveness between the brick and mortar is present when the sliding friction is triggered by outside forces. The link between vertical stresses and friction is expressed by the equation above. For a nonlinear plan element in CDS Win, it is not possible to realize this relationship. An ideal behavior curve is calculated, where the cohesiveness between mortar and brick is the value of the bearing shear stresses in the plasticity zone.

The nonlinear analysis in CDS Win will be carried out once static loads are applied vertically.This is done in an effort to simulate the building's actual behavior as closely as feasible. The building's collapse was caused by the seismic shear force, which acts in the structure's two primary axes.
After that, the model will be subjected to a pushover analysis. The thing is pushed directly into the collapse. For each floor, a specific push pattern is utilized. It might be a modal vibration or life produced by horizontal forces. The seismic impact is described in the first modal forms.[Marku,2022].

The masonry will be stressed due to the strains in the plan. The strained states shown in the diagram are caused by static loading and pushover in the schematic:

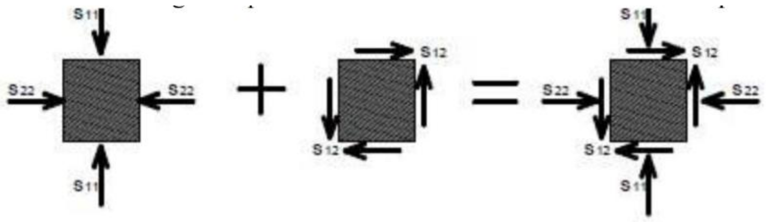


Fig.12 Stresses states caused by schematic loading and pushover (Source :Guri ,2016)

3.3Laboratory tests

They are made according to European Standard Codes. So Test standards are part of the comprehensive system of European standards relating to construction. They are intended to be used for the determination of material and product properties required for the design of buildings and other civil engineering structures. In particular, test standards related to the EN Eurocodes comprise testing for materials, e.g. concrete, masonry, timber and metallic materials, non-destructive test methodsecc.

We use EN 13791 for testing concrete, EN 10002 for testing metallic materials, and tensile testing, EN 772 for Methods of test for masonry units, and EN 1015 for Methods of test for mortar for masonry.

3.3.1 Concrete test

EN 13791 is the part of Eurocode that discusses methods for compressive strength toin-situ concrete.It gives helps to engineers (the methods and the procedure) to determine the quality of concrete on the site while following the rules included in Eurocode. To estimate the compressive strength and characteristics of concrete in site both direct method (core testing) and indirect methods , ultra-sonic pulse velocity, and rebound number can be used. According to EN 206-1, the table below lists the specifications for the minimum characteristic in situ compressive strength with respect to the compressive strength class.[EN 13791, January 2007]

Table 21 The EN 206-1 compressive strength classes' minimal characteristic in-situ compressive strength.

Compressive strength class according to EN 206-1	Ratio of in-situ characteristic strength to characterictic strength of standard specimens	Minimum characteristic in-situ strength N/mm^2	
		$f_{ck,,crl}$	$f_{ck,cube}$
C8/10	0.85	7	9
C12/15	0.85	10	13
C16/20	0.85	14	17
C20/25	0.85	17	21
C25/30	0.85	21	26
C30/37	0.85	26	31
C35/45	0.85	30	38
C40/50	0.85	34	43
C45/55	0.85	38	47
C50/60	0.85	43	51

C 55/67	0.85	47	57
C60/75	0.85	51	64
C70/85	0.85	60	72
C80/95	0.85	68	81
C90/105	0.85	77	89
C100/115		85	98
Note: The in- situ compressive strength may be less than that measured on standart test specimens taken from the same hatch of concrete Note 2 : The ratio 0.85 is part of γ_t = in EN 1992-1-1,2004			

Where cores are used to determine in-situ strength:

-A core of the same length and nominal diameter as a 150 mm cube produced and cured under the identical conditions is tested, and the results are equivalent in terms of strength.

-The strength of a 150 mm by 300 mm cylinder made and cured under the same conditions is determined by testing a core having a nominal diameter of at least 100 mm and not more than 150 mm and with a length to diameter ratio of 2.0.

-The conveyance of test findings from cores with diameters between 50 and 150 mm and other length-to-diameter ratios shall be based on established suitable conversion factors.

-The amount of concrete involved and the goal of the testing cores will dictate how many cores should be taken from one test zone. Every test site offers any cores.For each test region, an evaluation of in-situ compressive strength must be based on at least three cores.Any structural ramifications of adopting cores must be taken into account; for more information, see EN 12504-1.

Either procedure A or approach B is used to determine the in situ characteristic compressive strength.

Approach A

The test region's estimated in-situ characteristic strength is equal to the lesser of:

$f_{ck,is} = f_{m(n)\ is} – k_2 x\ s$ Equation 3.3.1

or $f_{ck,is} = f_{is,\ lowest} +4$ Equation 3.3.2

The test results' standard deviation, or s, is 2.0 N/mm2, whichever is higher.

If no value is specified, 1.48 is used as the value for k2, which is the given national provision.The estimated in situ characteristic strength is used to determine the stress strength class in table 21.

Note 1 The estimate of characteristic strength based on the lowest core result should reflect the degree of certainty that the lowest core result corresponds to the component's structure's lowest strength.

Note 2 The region may be divided into two separate regions where the distribution of the core strength looks to derive from two populations.

Approach B

The lower value of the estimated in situ characteristic strength of the test location is:

$f_{ck,is} = f_{m(n0,is}–k$ Equation 3.3.3

or $f_{ck,is} = f_{is,lowest} + 4$ Equation 3.3.4.

The suitable value is determined from the table below, where the margin k depends on the quantity n of test results:

Table 22 Margin k associated with small numbers of test results

n	k
10 to14	5
7 to 9	6
3 to 6	7

NOTE This approach provides estimates of characteristic strength that are typically lower than those obtained with more test results due to the uncertainty associated with small numbers of test results and the requirement to achieve the same level of reliability. More cores should be taken or a combined technique approach should be employed to get more test results when these estimations of in-situ characteristic strength are deemed to be too cautious. Because of this, this method should not be applied where there is a disagreement on the quality of concrete based on results from accepted tests.

In our case, we have prepared 2 concretecubics from the damaged object. The two samples taken from the damaged object are of dimensions 10x10x8 cm and 10x11x9 cm.

They are put in the apparatus and were loaded an axial force. This load is called the ultimate force of collapse of the material. It is measured.Its' values for the first sample are Fu=19.59 KN, and for the second sample Fu=210 KN

Here are shown a group of photos from the testing of concrete:

Fig 13. Preparing the concrete samples (Creating concrete cubics)

Fig 14. Samples created in the laboratory from the the concrete taken from structure and prepared for testing

Fig 15. Testing the prepared concrete cubics into the apparatus and measuring the compressive strength

Table 23 Results of tests for concrete cubics

Dimensions of prepared concrete cubic	Theultimate force of collapse	The bearing capacity of each sample
10 x10x8	19.59 KN	1959 KN/m2
10x10x9	21 KN	1909KN/m2

3.3.2 Tensile test of steel

EN ISO 6892 (DIN EN 10002) is the part of Eurocode that serves for the determination of strength and deformation characteristics of steel.ISO 6892-1 is one of the most generally adopted testing standards for the tensile testing of metallic materials at ambient temperature. The most recent iteration of a metals testing standard that has undergone numerous revisions is ISO 6892-1:2016. The tensile characteristics of metallic materials in any form and at room temperature are measured by ISO 6892-1. The temperature must be 23 degrees Celsius, plus or minus 5 degrees, for controlled-environment tests. Many different tensile qualities are measured by ISO 6892-1, with the following being the most popular:

The tension at which a material permanently deforms is known as the yield strength. According to the yielding phenomenon, ISO 6892-1 provides both upper and lower yield strength criteria for material that yields discontinuously, and the offset yield technique for material that yields continuously.

The Yield Point Elongation, which is only appropriate for discontinuously yielding material, is the difference between the specimen's elongation at the beginning and end of the discontinuous yielding process (the area in which an increase in strain occurs without an increase in stress).

The maximal force or stress that a material can withstand during a tensile test is known as its tensile strength.

A measurement of a material's ductility is the reduction of area. This is the difference, typically represented as a percentage decrease in original cross section, between the area of a specimen's original cross section and the area of its smallest cross section following testing. At or after fracture, the smallest cross section can be determined.

According to these rules, the tensile test of steel is made. For this purpose, a tensile test specimen is put in the apparatus and is slowly loaded in the axis direction. The load can be applied mechanically or hydraulically. In order to ensure a comparison of the results, the sample shapes are defined in respective standards. The samples taken are loaded with a defined strain rate until breakage. The execution, sample size, sample materials and testers of tensile tests are exactly defined in EN ISO 6892 (DIN EN 10002, part 1). The addition "part 1" in the name of the standard refers to the execution of the tensile test at room temperature. The measured and recorded tensile forces are referred to as the initial cross-section of the tensile test sample. The resulting nominal tensile forces are applied in a tension-yield diagram.

There are taken three samples of steelof different dimensions in our case.

Table 24 Results taken from steel laboratory test in tension

Φ5	l=440mm	σ_u=633.00N/mm2
Φ8	l=420 mm	σ_u=250 N/mm2
Φ10	l=530 mm	σ_u=377.90 N/mm2

Figure 16 The prepared steel testing element taken from the damaged object

Fig 17 Testing process of the steel element in the apparatus

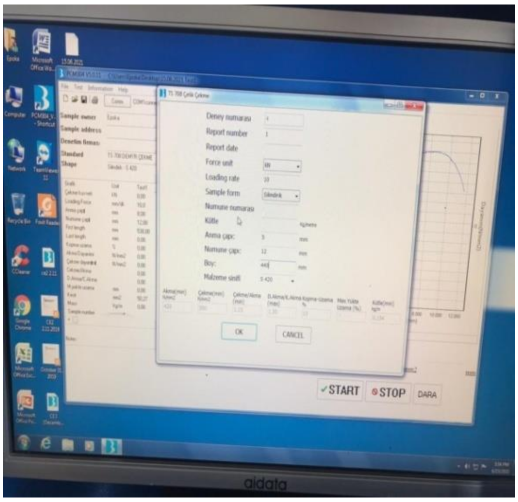
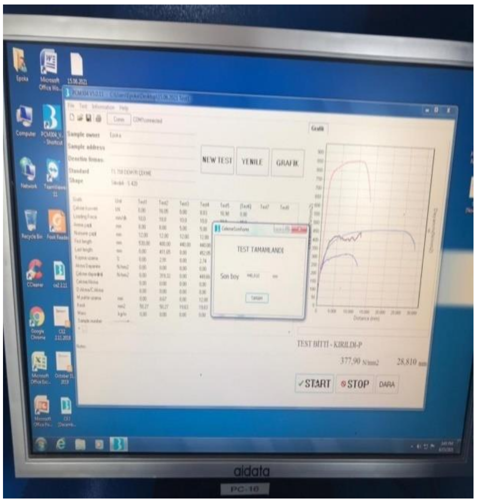



Fig 18 Testing results and the relationship stress –strain of the element given from the apparatus

3.3.3Solid brick test according to EN 772-1

Compressive strength test on bricks are carried out to determine the load carrying capacity of bricks under compression with the help of compression testing machine. Bricks are generally used for construction of load bearing masonry walls, columns and footings. These load-bearing masonry structures experience mostly compressive loads. Thus, it is important to know the compressive strength of bricks to check for its suitability for construction.

3.3.3.1Apparatus for testing of solid bricks

Table 25 Requirements for testing machine

Maximum permissible repeatability of forces as percentage of indicated force %	Maximum permissible mean error of forces as percentage of indicated force %	Maximum permissible error of zero force as percentage of maximum force of range %
2,0	±2,0	±0,4

The testing apparatus must be powerful enough to crush all test specimens, but the scale must contain a load-pacer. Two steel-bearing platens must be included in the testing apparatus. The platens' stiffness and the method of load transfer must be such that, at failure load, the deflection of the platen surfaces must be less than 0.1 mm measured over 250 mm. Either the faces of the platens or the through-hardened plates must be used. When tested according to EN ISO 6507-1, the testing faces must have a Vickers hardness of at least 600 HV.

One of the machine's platens must be able to freely align with the specimens as contact is established, but it must be prevented from tilting during loading by friction or another mechanism. The opposite platen must be a block that is level and non-tilting. Both platens' bearing faces must be larger than the largest specimen being tested. Auxiliary platens must match the main platens' hardness, stiffness, and planarity when used. They must also be suitably positioned. The platens' bearing surfaces cannot deviate from a plane by more than 0.05 mm. When measured according to the guidelines in ISO 468, the surface texture cannot be higher than 3.2 µm Rs.

Weighing device with a precision of 0.1% of the mass of the specimens.

Sufficient firm steel strips for use on ground units that are shell- or strip-bedded.

3.3.3.2Preparation of specimens

Sampling

The sampling procedure must follow the appropriate section of EN 771. Six specimens are the required minimum quantity, however if a larger minimum number is specified in the product specification, it must be used. According to the relevant section of EN 771, representative sections, or cubes, may be cut from big masonry units in a variety of positions.

Fig 19 Samples for laboratory tests of bricks from the studied object (Solid bricks 25x12.5x6 mm)

Table 26 The data for making the laboratory test of bricks

Brick N°=1 25.5x6x12.5	3452 kg
Brick N°=2 25.5x6.5x12	3263.5 kg
Brick N°=3 25.5x7x12	3378.5 kg
Brick N°=4 24x13x13.5	2960.5 kg
Brick N°=5 25x13x6	3322.5 kg

Surface preparation

Test specimens must be created in accordance with the applicable section of EN 771. Specimens must be tested in the designated orientation, which must be noted in the test report. It will be necessary to test the masonry units in more than one orientation for some types of construction.

The faces of the specimen, whether a whole mechanical unit or a piece cut from a larger unit, through which the load is to be applied, shall be to a tolerance of 0.1 mm in any 100 mm and such that the top surface lies between two parallel planes that are parallel to the bottom surface and not more than 1 mm apart for every 100 mm. If the test faces of the manufactured masonry unit or the piece taken from a larger unit don't meet this specification, prepare the surfaces by capping or grinding as directed by the applicable product standard.

Removal of tounges and grooves

If pieces are to be cut from bigger units, plan the cutting such that any tounges and/or grooves are limited. Remove any tounges and/or grooves from the test faces of the units before testing.

Preparation of masonry unit containing frogs and which are not to be capped

Test masonry units without removing or filling the frogs if they are determined to have a net loaded area of more than 35% of the bed surface. The frogs must be filled with mortar of the same kind as used for capping when the net loaded area of masonry units with frogs is less than or equal to 35% of the gross area, and the curing must be done in compliance with the storage of capped specimens criteria.

Grinding

Grind the specimen's surfaces until they meet the criteria for planeness. Keep any frogs, intended lettering, cavities, perforations, internal holes, or exterior holes present in the masonry units, however. The capping procedure of capping outlined below must be conducted if the grinding operation would considerably change the contact area of the tested faces. Make a composite specimen by stacking the specimens on top of one another without using mortar, binding material, or separating layer(s) between them if the specimens' remaining height after being ground is less than 40 mm or the height/width ratio is less than 0.4.

NOTE When a composite specimen is constructed up of more than one ground unit, it should be regarded as a single specimen that yields a single test result. Therefore, in order to provide the appropriate number of test results, more masonry units than those listed in the relevant part of EN 771 will be needed.

Capping

Employ a cement/sand capping mortar that will, when tested in accordance with EN 1015-11, achieve a minimum compressive strength that is at least equal to the estimated masonry unit strength or 30 N/mm2, whichever is less.

If necessary, moisten the surfaces to be capped beforehand, such as for units with strong water absorption qualities. Each specimen should be placed on a flat, smooth plate made of stainless steel or ground glass that does not deviate from a true plane surface by more than 0.1 mm every 100 mm.

Using a spirit level, level the plate firmly in two directions at right angles with the machined face upmost. Apply a thin layer of mold release oil or a piece of paper or plastic that is about 25 mm longer than the unit and 10 mm broader to the plate. In order for the specimen's vertical axis to be perpendicular to the plate's plane, press the specimen's bed face firmly into the layer. Use a square or spirit level and place it against each of the specimen's four vertical faces to examine this condition.

Make sure the mortar bed is at least 3 mm thick over the entire area and that any gaps in the bed face that are typically filled with mortar when the masonry units are installed in the wall are entirely filled with mortar. Except for those that are supposed to be filled during construction, avoid filling voids. Trim any extra mortar flush with the masonry units' sides. Put a wet cloth over the specimen and mortar. Do not dry out the cloth.After it has adequately dried, check the mortar bed. the second bed face in the same manner as the first, using mortar mixed with materials drawn from the same batches of cement and sand and using the same mix proportions, if free from faults such as lack of compaction, lack of adhesion to the masonry unit, and/or cracking. Check that the mortar bed is defect-free once the specimen has been removed from the plate. If necessary, tiny holes could be cut in the capping to let water out of cavities.

Conditioning of specimens before testing

Specimens must be prepared utilizing a predetermined regime of moisture conditions or, if necessary, to a prescribed moisture condition. The conditioning procedure must follow the guidelines outlined in this section. The procedure must follow the guidelines laid out in the pertinent section of EN 771 for each product type. Free air calculation surrounding each specimen must be ensured during conditioning in all circumstances, with the exception of conditioning by immersion.

3.3.3.3Loaded area

By multiplying the length by the width of each specimen determined in accordance with EN 772-16, the gross area of the loaded surface is to be calculated in square millimeters. When using units with compressive forces that are not normal to the bed face, the gross area must be calculated similarly but using the appropriate width and height or length and height.

3.3.3.4.Procedure

Placing specimens in the testing machine.

Fig 20 Testing of the solid bricks procedure according to EN 772

By dividing each specimen's length by its width, as determined in accordance with EN 772-16, the gross area of the loaded surface is to be calculated in square millimeters. The gross area must be calculated similarly when units are to be used with compressive forces that are not normal to the bed face, but using the appropriate width and height or length and height measurements.

Loading

Use any practical loading rate at first, but once half the anticipated maximum load has been applied, change the rate so that the maximum load is attained in no less than one minute.Table 27 serves as a guide for selecting the proper loading rate.

Fig 21 Loading the solid bricks until their failure according to EN 772

Table 27 Loading rate

Expected compressive strength (N/mm²)	Loading rate (N/mm²)/s
< 10	0,05
11 to 20	0,15
21 to 40	0,3
41 to 80	0,6
>80	1,0

Compressive Strength of Bricks = Maximum Load at Failure (N)/Average area of bed face (mm^2)

Table 28 The results of brick masonry

Brick Nr.	Half 1 dimension (cm)	The force of collapse KN	Half 2 dimensionscm	The force of the collapse KN	The bearing capacity of the brick in tension **σ**KN/cm2
Brick N=1	12.75x12	30	12.75x12.5	33	1.66
Brick N=2	12.75x12	28	12.75x12	28	0.75
Brick N=3	12.75x12	23.6	13x13	18	1.28

Brick N=4	9.5x12	29	9.5x12.5	29	1.84
Brick N=5	12.5x13	24	12.5x13	59	2.18

3.4 Finite element analysis

The use of calculation, models, and simulation to predict and for a deep understanding of how an object behaves under physical conditions is called finite element analysis (FEA). It is used by engineers to find vulnerabilities in their design prototypes. Finite element analysis uses the finite element method, which is a numerical technique that cuts the structure into several elements, then reconnects them at points called nodes. Engineers use the finite element method (which consists of a set of algebraic equations) nowadays.Through finite element analysis, we can predict the linear or nonlinear physical experiences of a product.While using finite element analysis we can reduce the number of physical prototypes created. There are many types of FEA tests used during the finite method analysis as follow:

-linear statics (LSA –Linear Static Analysis)

-linear dynamics (LDA –Linear Dynamic Analysis)

-nonlinear statics (NLSA-Non linear static Analysis –Pushover)

-nonlinear dynamics (NLDA-Non Linear Dynamic Analysis)

3.4.1Nonlinear frame analysis methods in Eurocode 8

The Eurocode 8 nonlinear pushover procedures are presented in this section. The definition of pushover analysis is found in EC8 Part 1 4.3.3.4.2. Pushover analysis may be used, in accordance with EC8

-to assess the structural performance of both freshly constructed and existing buildings.

- to check or update the α_u/α_1 overstrength ratio values. With the help of fig 23, the overstrength ratio's definition is remembered. α_u/α_1 is the ratio between the base shear$\alpha_u V_b$ corresponding to the construction of a mechanism and the base shear $\alpha_1 V_b$ corresponding to the formation of the first plastic hinge when the structure is pushed with a lateral load distribution of constant shape and increasing intensity. In linear analysis, the overstrength ratio is utilized to calculate the behavior factor q (EC8 Part 1 3.2.2.5, 5.2.2.2, 6.3.2, and 7.3.2), which enables the creation of a nonlinear design spectrum for inelastic analysis beginning with a linear design spectrum;

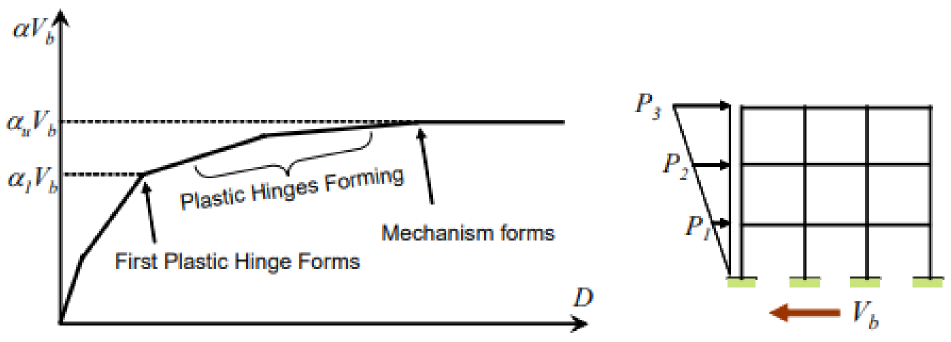


Fig 22 Overstrength ratio

-to calculate the damage distribution and anticipated plastic processes;

-to evaluate the structural performance of already-existing or renovated structures in accordance with EN 1998-3 (EC8 Part 3)

-offers an alternative to the design that makes use of the behavior factor q and is based on a linear-elastic analysis. In this instance, the pushover analysis's goal displacement should serve as the design's starting point.

-Buildings that do not meet the regularity requirements of EC8 must also be examined using a spatial (3D) structural model, according to EC8 Part 1 4.3.3.4.2.1. It is possible to conduct two separate evaluations with lateral loads applied just in one direction. It is assumed here that in a pushover analysis the structure is pushed with loads applied in one horizontal direction at a time because there are no instructions in EC8 or the published literature on how to perform such an analysis with two loads distributions applied simultaneously in two orthogonal directions. (the vertical seismic is typically neglected in buildings). According to EC8 Part 1 4.3.3.5, instructions are provided on how to combine the results of operations applied in two horizontal directions.Two planar models, one for each primary direction horizontal direction, may be used in the analysis for structures that meet EC8's regularity criteria.

For pushover and nonlinear time-history analysis, the structural element models and the final structural model of the entire building are quite similar. The requirement for cyclic models for the time-history analysis is the only distinction.

Building the nonlinear frame model and applying the gravity loads are the first phases in both nonlinear methods. The nonlinear analysis does not change the gravity loads. (both static and dynamic). In Fig 23, the application of the gravity loads is depicted schematically. EC8 provides the value of the constant gravity loads.This first step is crucial since it has the potential to alter the structure's initial state.For instance, gravity loads frequently cause cracking in beams and impart strong axial strains to columns in a building made of reinforced concrete.

$$G_k+P_k+\sum_I (\psi_{2i}Q_{ki}) \qquad \text{Equation 3.4.1.1}$$

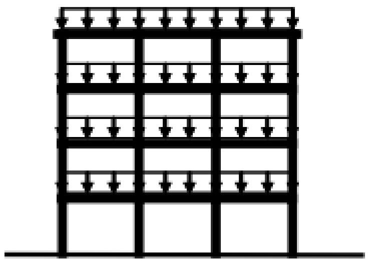


Fig .23 Application of constant gravity load

The seismic action in nonlinear methods must be applied in both positive and negative directions, according to EC8 Part 1 4.3.3.4.1. (depending on the symmetry of the structure).

3.4.2 Nonlinear static pushover analysis according to Eurocode 8

The Nonlinear Static Pushover Procedure in EC8 follows the N2 method developed by Fajfar (1999). The technique involves giving the building model fixed load forms. The lateral loads imposed by the ground motion are represented by the load forms. A pseudo-static increase in load intensity occurs. Depending on the building's regularity features, the structure model may be spatial (3D) or planar (2D). On the other hand, the load pattern is always applied in a single direction. Combination rules are provided by EC8 for studies involving input ground motion in more than one direction, such as input ground motion in the x and y dimensions.

Applyingmonotonically rising constant shape lateral load distributions to the structure under consideration constitutes the nonlinear pushover analysis. Either a 2D or 3D structure model is possible. In example, EC8 notes that whereas a complete 3D model is required for buildings with plan irregularity, 2D analysis of single plane frames can be undertaken for buildings with plan regularity. A 3D model is typically needed because existing buildings, which are rarely regular, are particularly intriguing for nonlinear algorithms.

A shear building model-a frame model with floors rigid in their planes—was used to construct the N2 approach. Additionally, the approach normally ignores vertical displacement and only takes into account the two x and y components of the horizontal ground motion. It is simple to extend this result to the general case of a fully deformable frame. Two load distributions are applied to the frame as part of the N2 method:

-.a "modal" pattern, which is the result of multiplying the first elastic mode shape by the mass matrix to get the load form:

$$P^1 = M\psi_1 \qquad \text{Equation 3.4.2.1}$$

-a mass proportional load shape in a "uniform" pattern.

$$P^2 = MR \qquad \text{Equation 3.4.2.2}$$

where M is the mass matrix, φ1 is the first mode shape and R a vector of 1s corresponding to the degrees of freedom parallel to the application of the ground motion and 0s for allother dofs. In the N2 method φ1 is normalized so that the top floor displacement is 1, i.e. φ1 ,n = 1 Fig 24 , the two load distributions are depicted schematically. The response is plotted as base shear Vb vs. top floor displacement D as the lateral load distributions are increased. (for example center of mass of the top floor)1. This is the so-called capacity or pushover curve. (also shown schematically in Fig.24

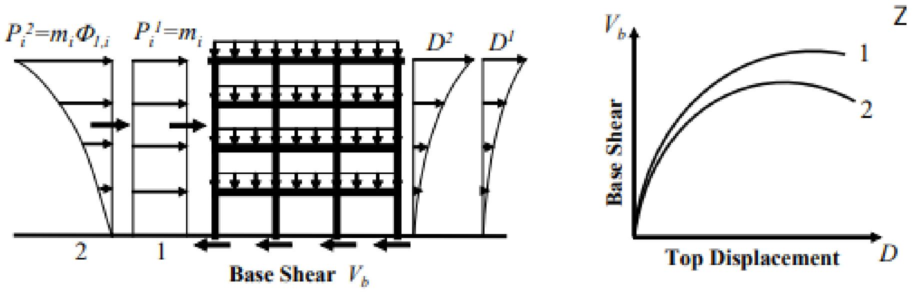


Fig 24 Load distribution of pushover analysis according to EC 8 and pushover response curve

The MDOF system's response is converted into an equivalent SDOF system's response via the N2 process. This is required in order to compare the demand stated in the design codes by the design spectra, which correspond to SDOF systems, with the building capacity curve of Fig 24.

3.4.2.1 Equivalent SDOF model and capacity diagram in Eurocode 8

Fajfar (1999), as previously mentioned, makes the assumption that the structure is a shear frame, meaning that the floors are stiff in their own plane. The three degrees of freedom indicated in Fig 25 can be used to calculate the floor displacements if the building's vertical displacements are ignored. Commonly, the center of mass is used to measure the degrees of freedom. Keep in mind that the nodes contain rotational degrees of freedom that are not within the floor plane, allowing the beams to deform outside of the floor plane.

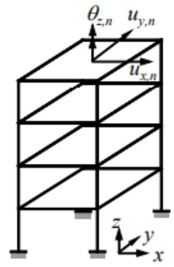


Fig.25 Rigid slab degrees of freedom in 3D shear frame (for simplicity column axial deformability is neglected)

The transformation procedure's theoretical justification is given below. A MDOF building according to the basic ground motion model has the following equations of motion: $M\ddot{U}+F(U) = - MR\alpha$ Equation 3.4.2.1.1

If damping is disregarded, M is the mass matrix, U and F are vectors indicating relative displacements and internal forces, respectively, R is the influence vector, and a(n) is the ground acceleration as a function of time, i.e. a = a(t). A is only provided in one direction. In the nonlinear situation, F depends on the displacement history while in the linear elastic case F = KU (where K is the structure stiffness matrix).The influence vector R for unidirectional ground motion, for instance in the direction x, is composed of 0s for all other dofs and 1s for the dofs in the x direction.

The N2 method's initial presumption (and approximation) is that the displacement U has a fixed form that does not alter throughout the response to the ground motion:

U = Dt or U(x,t) = (x)Dt(t), Equation 3.4.2.1.2

where Dt(t) is the intensity of the displacement shape at the pseudo-time t and x denotes that the displacement shape relies on the location of the degree of freedom. For convenience, n is set to 1 and is normalized such that the top-storey displacement is equal to 1. D(t) thus provides the top floor displacement at time t. Has nonzero components in the six dofs of each node in the general situation of a 3D building.

According to the aforementioned equations, the lateral force in the i-th storey of a shear frame is proportional to the component i of the assumed displacement, weighed by the storey mass mi. If the x direction is used to apply the ground motion

$P_i = pm_i + \phi_{xi}$ Equation 3.4.2.1.3

The internal forces Fare equivalent to the external forces P that are considered to be pseudo-static as a result of statics, which means that P = F. The result is obtained by pre-multiplying by ϕ and combining the aforementioned formulae.

$$\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}\ddot{D}_t + \mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}p = \mathbf{\Phi}^T\mathbf{M}\mathbf{R}a$$ Equation 3.4.2.1.4

Following that , the left term of the equation is divided and multiplied by $\phi^T MR$ to produce:

$$\underbrace{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}_{m^*}\underbrace{\frac{\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}}{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}}_{\frac{1}{\Gamma}}\ddot{D}_t + \underbrace{\frac{\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}}{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}}_{\frac{1}{\Gamma}}\underbrace{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}p}_{V_b} = \underbrace{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}a}_{m^*}$$

Equation 3.4.2.1.5

Where m* is the mass of the SDOF equivalent to the MDOF building :

$m^* = \phi^T MR$ Equation 3.4.2.1.6

When a building is sheared and a ground motion is applied in the x direction:

$m^* = \sum mi\phi x,I$ Equation 3.4.2.1.7

where $\phi x.i$ is x component of the modal shape vector for node i.The constant г controls the transformation from MDOF to SDOF and back:

$$\Gamma = \frac{\mathbf{\Phi}^T\mathbf{M}\mathbf{R}}{\mathbf{\Phi}^T\mathbf{M}\mathbf{\Phi}}$$

Equation 3.4.2.1.8

For a shear building and ground motion in the x direction :

$$\Gamma = \frac{\sum m_i \Phi_{x,i}}{\sum m_i \Phi_{x,i}^2}$$

Equation 3.4.2.1.9

The mode participation factor is a factor that, for a value of equal to the mode shape of one of the building's modes, is equal to. Any logically deformed shape can be used in the development of the N2 technique.

Vb is the base shear of the MDOF building in the direction of the ground motion, equal to: $V_b = \phi^T$ MRp Equation 3.4.2.1.10

For a building under shear and a ground motion in the x direction:

V_x= pΣ mi$\phi_{x,i}$=Σ$P_{x,I}$ Equation 3.4.2.1.11

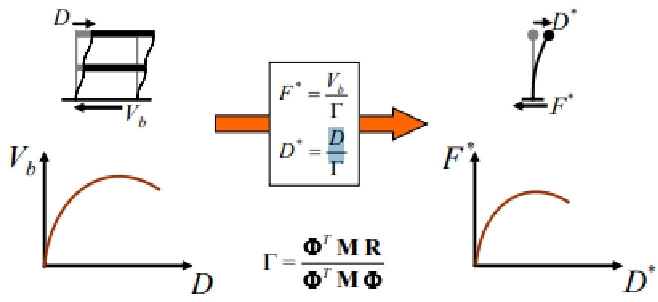


Fig.26 Capacity curve : transformation from response of MDOF to equivalent SDOF

The equation the SDOF equivalent to the MDOF building is thus obtained:

D*=Dt/Γ Equation 3.4.2.1.12 F*=V_b/Γ Equation 3.4.2.1.13

The MDOF pushover capacity curves of Fig.26 can be converted into pushover curves for the analogous SDOF system using the aforementioned derivation, as illustrated in Fig 26.

The scaling factor is applied to both the force and displacement axes. The system's rigidity stays the same. Be aware that the transformation factor varies depending on the assumed displacement shape's shape and is therefore dependent on the choice of. In EC8, two types of loadings are recommended, as seen in Fig.26:

a)$\phi = \phi_1$thus $\Gamma = \phi_1^T MR/\phi_1^T M\phi_1$ Equation 3.4.2.1.14

b) ϕ=R thus Γ=1 Equation 3.4.2.1.15

3.4.2.2 Linearizationof the capacity curve and comparison to demand spectrum (Linearization of the capacity curve)

The nonlinear pushover curves of the SDOF are approximated by elastic-perfectly plastic (or bilinear) curves in order to contrast the capacity curve and the demand curve provided by the design spectrum. The equal energy concept may be the foundation for this transformation, per Annex B of the draft EC 8.0. Assumptions are made regarding a target displacement and an energy balance between bilinear and nonlinear pushover curves. This simple procedure is illustrated in Fig 27.

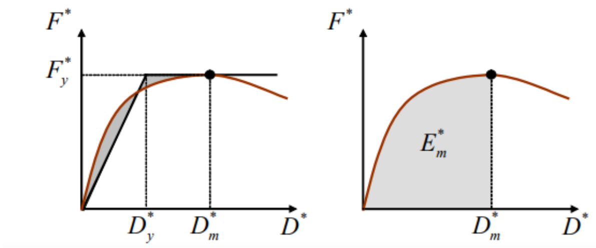


Fig .27 Bilinearization of the capacity curve of SDOF

The yield force and the yield displacement are obtained from the bilinearization of Fig.28 .$D_y^* = 2\,(D_m^* - E_m^*/F_y^*)$ Equation 3.4.2.2.1

which make it possible to calculate the initial elastic period as:

$$T^* = 2\Pi\frac{\sqrt{m*Dy*}}{Fy*} \quad \text{Equation 3.4.2.2.2}$$

Secondly, by normalizing the force with respect to the SDOF weight, the capacity curve is converted into a capacity spectrum. In Fig.28, the resulting capacity spectrum is displayed.

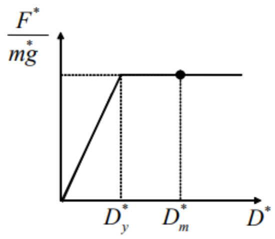


Fig.28 SDOF capacity spectrum

3.4.2.3 Seismic demand

The design spectrum made available by the design codes determines what is required of the building.

The first step in comparing capacity and demand is to change the design spectrum's format from the traditional Acceleration a vs Period T format to the ADRS format, which is Acceleration a vs. Displacement D. Due to the fact that displacement and acceleration are related by:

$S_D=(T/2\Pi)^2\ S_A$ Equation 3.4.2.3.1

Fig,29 displays the transformation into the ADRS spectrum. Constant periods are shown by lines that start at the origin.

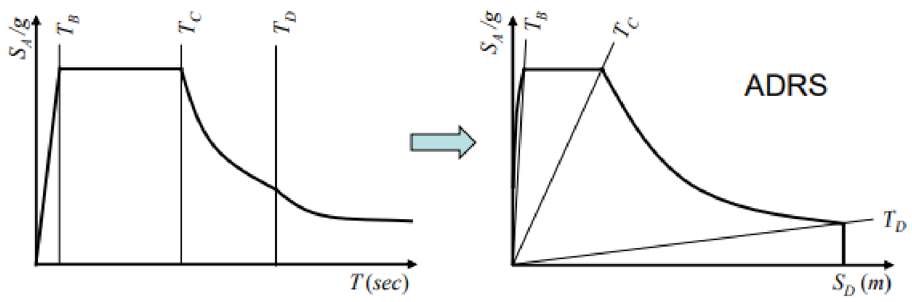


Fig 29 the transformation of response spectrum into ADRS format

3.4.2.4 Transorformation to ADRS linear spectrum

Now, the Fig.28 capacity spectrum is compared to the Fig.29 ADRS demand spectrum, but the comparison is not straightforward because the capacity spectrum is nonlinear and the ADRS spectrum provided by the design codes is linear.

The acceleration spectrum S_A and the displacement spectrum S_D for an SDOF system with a bilinear plastic behavior can be calculated as:

$$S_A = \frac{S_{Ae}}{R_\mu}$$ Equation 3.4.2.4.1

$$S_D = \frac{\mu}{R_\mu} S_{De} = \frac{\mu}{R_\mu} \frac{T^2}{4\pi^2} S_{Ae} = \mu \frac{T^2}{4\pi^2} S_A$$ Equation 3.4.2.4.2

where R is the ductility factor, which equals the maximum inelastic displacement / yield displacement, and R is the reduction factor brought on by ductility, subscript e denotes elastic. There are several ways to find the reduction factor R, some analytical and some approximative. The following approximated expressions are provided for the simple N2 method:

$R_\mu=(\mu-1)T/T_C +1$ $\quad T<T_C$ Equation 3.4.2.4.3

$R_\mu=\mu$ $\quad T \geq T_C$ Equation 3.4.2.4.4

where the soil type-dependent characteristic period of the ground motion, T_C, is determined by EC8. It typically corresponds to the change in the response spectrum's constant velocity range (medium-period range) from the constant acceleration range (short-period range). According to the aforementioned equations, the equal energy principle is used in the medium- and long-period ranges while the equal displacement principle is used in the short- and long-period ranges (elastic and inelastic SDOFs have the same maximum displacement). The principles in Fig 30 are displayed.

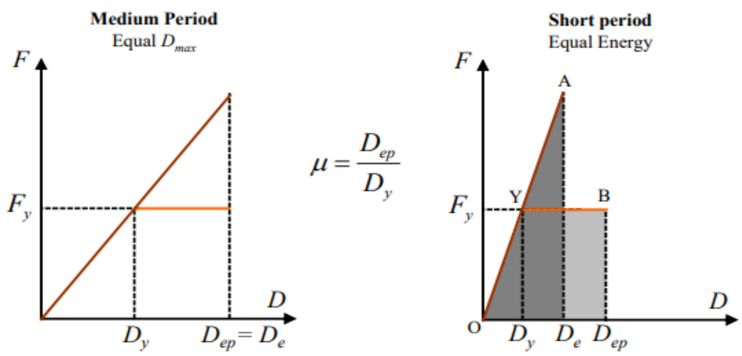


Fig.30 Transformation elastic response – bilinear response:

Equal maximum displacement and equal energy assumption

The inelastic demand spectra of constant ductility are obtained using the aforementioned equations, as shown in Fig 31. It should be noted that this method is approximate and that nonlinear dynamic analysis can be used to determine inelastic demand spectra in a rigorous (but more difficult) manner.

The seismic demand on the equivalent SDOF can be calculated using the methods shown in the section. Fig.31, the steps are schematically shown for a bilinear oscillator with a medium or long elastic period T. The target displacement $D_t$is theoretically determined by locating the inelastic demand spectrum of ductility that intersects the capacity spectrum in a point corresponding to a capacity ductility, given the elastic demand spectrum and the bilinear capacity spectrum. In other words, the point with equal demand and capacity ductility serves as the design point. Utilizing the general process outlined in Fig.31, this is accomplished in practice very quickly.

Capacity and Demand spectra for short period T*

a) $T^* < T_c$ (Short periods)

a1) $F^*_y / m^* \geq S_A(T^*)$ the response remains linear elastic (case a)$D_t^* = D_{et}^*$

a2) $F^*_y / m^* \leq S_A(T^*)$ the response enters the nonlinear plateau (case b)

$$D_t^* = D_{et}^* / q_\mu \left(1 + (q_\mu - 1) T_c / T^*\right) \qquad \text{Equation 3.4.2.4.5}$$

Where $q\mu = S_{Ae}(T^*) / F_y^* / m^*$ is the reduction factor

b)$T^* \geq T_c$ (Medium and long perids)

$D_t^* = D_{et}^*$Equation 3.4.2.4.6

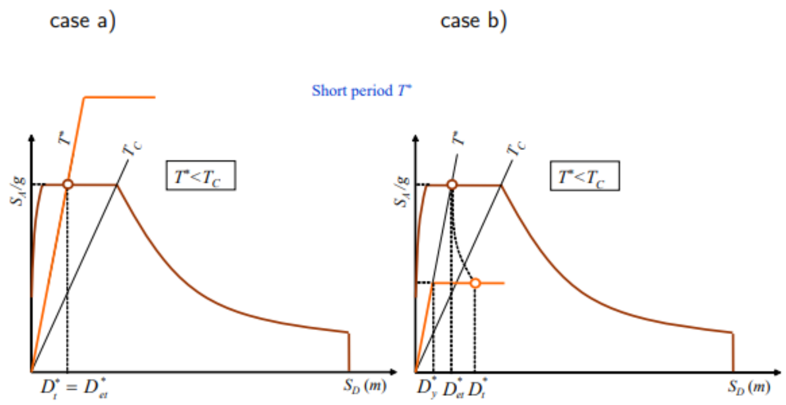


Fig.31 Capacity and demand spectra for short period T*

NB: The above procedure is based on the assumption of a tentative target displacement D^*_m (from which the equal energy principle is used to obtain the bilinear capacity curve in Fig.32). If the target displacement $D*$ t is very different from the assumed value $D*$ m, then the procedure must be repeated, setting for instance $D^*_m = D^*_t$. This is a simple iterative procedure that converges very rapidly.

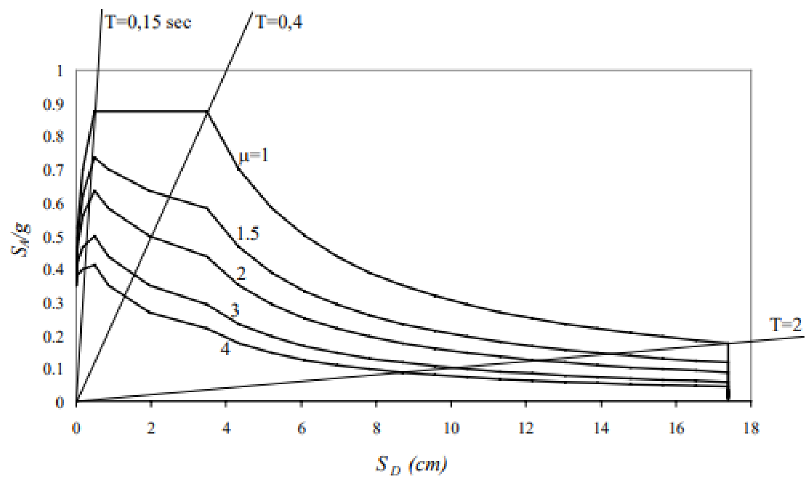


Fig.32 Demand spectra for constant ductilities in AD format (based on EC 8 spectrum for Zone 1, Soil type A)

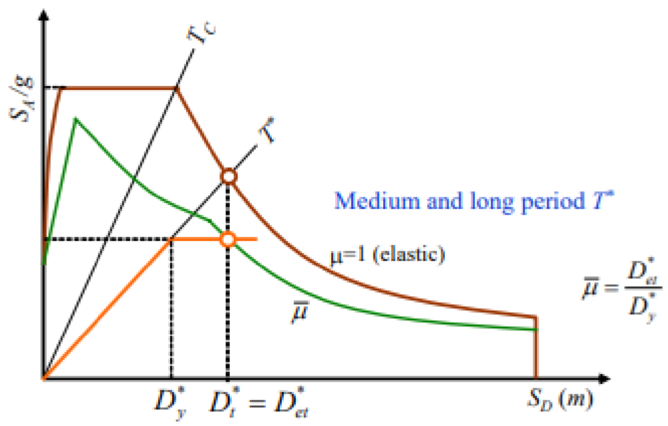


Fig.33 Capacity and demand spectra for long and medium period T* determination of the target displacement

By flipping the transformation in Fig 33, the desired displacement at the top of the building is obtained.i.e. $D_t = \Gamma D^*_t$

The location where the pushover curve is stopped is one small problem. Although it is not necessary to perform the pushover analysis until the top floor displacement reaches absurd values (doing so would result in lengthy computation times and convergence issues at high displacement values), there is no set rule for when to stop the pushover curve. This indicates that it is possible for the pushover curve to end at a displacement level that is less than the calculated target displacement. The pushover analysis must be performed again in this situation and stopped at higher top displacement values. In order to perform a pushover analysis at top-displacements of the order of 2% to 3% h, where h is the height of the building as a whole,the Ultimate and Collapse Limit States are used.

3.5 Service limit states

A limit point on the capacity curve known as the "service limit states" is used to categorize building deterioration. These depend on the sort of construction used and the materials used in its creation. Masonry building limit states are influenced by the regularity of the plan, the amount and density of openings, and the thickness of the walls. Calvi suggested determining service states based on the distance between floors (inter-story drifts). The shear stress generated by the seismic force that each floor has absorbed is correlated with relative displacement. This shear force causes the building to be destroyed. It is practical to apply the limit states to all masonry building scenarios. So, in accordance with Calvi, we shall use these limit states.

The building can be used after an earthquake with just little structural damage or considerable non-structural damage, necessitating little to no strengthening or structural element replacement. The upper limit of relative displacement, according to Sds2, is 0.1%.

Sds3 Significant structural and non-structural damage. Buildings need extensive repairs before they can be used again following the earthquake. Rebuilding and fortification are possible. The upper limit of relative displacement, according to Sds3, is 0.3%.

Sds4Complete collapse; it is neither feasible nor cost-effective to repair the structure. As a result of the earthquake, the building crumbles. The upper limit of relative displacement, according to Sds4, is 0.5%.

The limit states according to Calvi are depicted below (Calvi G,1999). Not every structure meets these performance requirements because some of them fall before reaching Sds4 or Sds3. It depends on how the load-bearing walls are set up and how the cracks look after an earthquake.

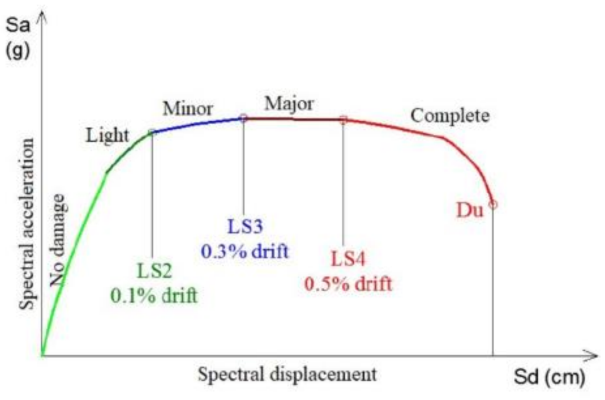


Fig 34.Service states graph

A greater loss ratio and a lower annual probability are projected to be associated with a more severe damage condition because lower losses are linked to lower seismic levels of intensity that are more likely to be exceeded.

CHAPTER 4

APPLIED EXAMPLE

Fig 35 School “Gjergj Kastrioti”

Location School “Gjergj Kastrioti “ was located in the Street “ Beselidhja “-Road “Rruga e Kosovareve”

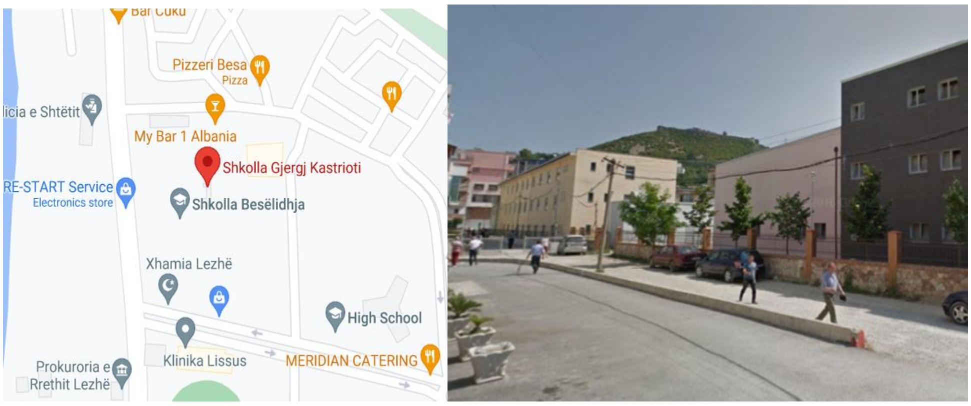


Fig.36 The location of the school “Gjergj Kastrioti”

For the alalysis of these building we have followed the following guidelines :

1)Eurocode 0,1,2,6,5,7,8,

2)Decreto 17 gennaio 2018 , Aggiornamento delle <Norme tecniche per le construzioni> NTC 2018

3)NTC 2018 Circ 21/01/2019 Istruzioni per l' applicazione dell'Aggiornamento delle "Norme tecniche per le construzioni" di cui al decreto ministerial 17 gennaio 2018

3)Albanian Technical designing guidelines KTP-9-78

4)Albanian Technical designing guidelines KTP-2-89

4.1 General information about the studied school

The school was built in the early of 60 years. It had 3 floors. The construction system was of the type of bearing masonry with a thickness that varies from 25 to 51 cm. The slabs were supported in the contour and were with a thickness of 10 cm.There were no antiseismic belts and antiseismic columns. The foundation beams were made of stone and in form T overturned. It had a regular form in plan and elevation.

The geological data are taken from in situ geological study. A new construction will be built so the information was taken from the studies for this new reinforced concrete building.

The layer where the foundations were laid is slit gravel with gray color with a lot of moisture until fully hydrated It contains thin layers of sand. The dimensions of gravel vary from 7-8 cm. They are just a few rounded-up particles,with sand and carbonatic compounding, They are averagely compacted.The soil type according Eurocode 8 is Siol C as mentioned in Chapter 2 .

For the granulation components, there are shown some important parameters:

Clay fractions < 0.002 mm 11.50 % Dust fractions 0.002-0.075 mm 18.70 % Sand fractions < 4.75 mm 27.40 % Gravel fractions > 4.75mm 52.40 %	Plasticity The upper value of plasticity Wu = 28.70 % The lowest value of plasticity Wl = 21.20 % Number of plasticity Ip=7.50 The weighted volume in natural state Δ=2.09 T/m3
Natural moisture Wn = 18.40 % Specific weight δ=2.64 T/m3 The porosity coefficient ε= 0.67	Modulus of elasticity E = 140 kg/cm2 Internal friction Ø = 30° Cohesion C = 0.12 kg/ cm2 Poisson ratio μ= 0. 24 Allowable compressive stress σ= 2.40 kg/cm2

4.2 Survey as an important part of studying of the existing structure

Shear sliding failure of masonry

Diagonal cracks on masorny

Sliding shear failure of masonry

Non structural damages in masonry

Sliding shear failure of masonry

Fig 37.Photos taken from the damaged object

Accordingto NTC 2018 Circ. 21/01/2019 section 8.5.2.1 in the survey are included, visual examinations of the structure, the studying of its plans, its architectonic sections over all the floors, the examination of the floor, wall components, slabs direction, types of foundation and the geometry of structural elements. The composition of the materials used and their connection have a significant influence on the structure built with bearing walls when considering various types of materials and construction technology.

In survey, depending on the degree of recognition over elements geometry has 3 types of examinations:

1-Limited examinations, which are generally based on the visive examinations, where after the removal of the plaster in different parts, the type of the masonry is defined, its thickness, etc. Also, it is examined the way of the support of the slab and the wall, the connection and the elimination of the pushes of the plaster if the cover is arched.

2-Extended examinations,all controls according to the following point plus the ones determining the characteristics of the constructive materials.

3-All inclusive(General)A technical person will have a clear understanding of the quality of walls in surface and interior, the connection of slats, the appliances that eliminate push (from arch covers), and the quality of horizontal elements over walls, in addition to what was mentioned in the previous point.Tests are made in laboratory , and the samples are taken from the structure and will include tests in compression , in flexural bending. Test will be made over all structural elements or over the one based in a preliminary analysis resulting more stressed. In masonry constructions will be considered other materials that are part of the structure as concrete slabs. This tests are often costly in time and in monetary value .

There are removed parts from the plaster for determining the compounding materials , and the type of examination resulting to be a limited examination.

4.2.1 The scale of recognition and the factor of confidence

The norm NTC 2018 in base of the data collected is determined in three level of recognition of the structure (a.k.aLivelli di conoscenza / knwoledge level).

LC 1 (the scale of limited recognition) is reached when is done the minimum , historical analysis according to C8.5.1 , the complete geometric survey and examinations for construction details according to C 8.5.2 limited tests for the properties of materials according to C 8.5.3 and exactly the factor of confidence FC=1.35

LC 2 (the scale of appropriate recognition) which is reached when there are done as minimum , the historical analyses according to C8.5.1. Completed geometric survey and the examinations for the construction details according to C8.5.2, expanded tests for the properties of the materials according to C8.5.3 and respectively the factor of confidence FC=1.2

LC 3 (the fully recognition scale) is reached when there are done the minimum , historical analyses according to C.8.5.1 completed geometric survey and examinations for the constructive details according to C.8.5.2 general tests for the properties of materials according to C 8.5.3 and respectively the factor of confidence FC=1.0

4.2.2 Surveying of the existing building

For this analysis, we have a recognition LC 1. The geometry of the object must be established in order to continue the survey. Limited in-situ assessments are conducted in accordance with the constructional details and material characteristics; the factor of confidence is FC=1.35.There are observed 3 types of failing in the masonry building. The first is the sliding shear failure as seen in the previous photos of the damaged object.There is dislocation of a lightly attached roof .Tha causes of this type of failure are low vertical load and poor mortar. The second type of failure is diagonal cracks. They happen when the tensile stresses, developed in the wall under a combination of vertical and horizontal load, exceed the tensile strength of the masonry material. The third type of failure is nonstructural failure.This type of failure may not lead to building collapse, but still constitutes danger for occupants and requires costly replacements or repair.

In this point and without expanded examinations in-situ (Circolare 2019)is given in table C8.5.1 with masonry types most used associated with resistance values.

Tabella C8.5.I -*Valori di riferimento dei parametri meccanici della muratura, da usarsi nei criteri di resistenza di seguito specificati (comportamento a tempi brevi), e peso specifico medio per diverse tipologie di muratura. I valori si riferiscono a:* **f** *= resistenza media a compressione,* τ_0 *= resistenza media a taglio in assenza di tensioni normali (con riferimento alla formula riportata, a proposito dei modelli di capacità, nel §C8.7.1.3),* **fv0** *= resistenza media a taglio in assenza di tensioni normali (con riferimento alla formula riportata, a proposito dei modelli di capacità, nel §C8.7.1.3),* **E** *= valore medio del modulo di elasticità normale,* **G** *= valore medio del modulo di elasticità tangenziale,* **w** *= peso specifico medio.*

Tipologia di muratura	f (N/mm²)	τ_0 (N/mm²)	f_{V0} (N/mm²)	E (N/mm²)	G (N/mm²)	w (kN/m³)
	min-max	min-max		min-max	min-max	
Muratura in pietrame disordinata (ciottoli, pietre erratiche e irregolari)	1,0-2,0	0,018-0,032	- -	690-1050	230-350	19
Muratura a conci sbozzati, con paramenti di spessore disomogeneo (*)	2,0	0,035-0,051	- -	1020-1440	340-480	20
Muratura in pietre a spacco con buona tessitura	2,6-3,8	0,056-0,074	- -	1500-1980	500-660	21
Muratura irregolare di pietra tenera (tufo, calcarenite, ecc.,)	1,4-2,2	0,028-0,042	- -	900-1260	300-420	13 ÷ 16(**)
Muratura a conci regolari di pietra tenera (tufo, calcarenite, ecc.,) (**)	2,0-3,2	0,04-0,08	0,10-0,19	1200-1620	400-500	
Muratura a blocchi lapidei squadrati	5,8-8,2	0,09-0,12	0,18-0,28	2400-3300	800-1100	22
Muratura in mattoni pieni e malta di calce (***)	2,6-4,3	0,05-0,13	0,13-0,27	1200-1800	400-600	18
Muratura in mattoni semipieni con malta cementizia (es,: doppio UNI foratura ≤40%)	5,0-8,0	0,08-0,17	0,20-0,36	3500-5600	875-1400	15

Fig 38. Reference parameters of the masony according it type (Norme techniche per le costruzioni)

The symbols signed in this table present:

f - average resistance in compression

τ0 - average shearing resistance

E – average value of modulus of elasticity (normal)

G – average value of modulus of elasticity (tangential)

w – average weight per unit

We have in disposition the values of experiments of masonry , According the laboratory tests we have these data :

Fm= 1.542 Mpa

E= 737.71 Mpa

The values of resistanceconverge against the values of tables C8.5.1

In favor of the resistance there are chosen the values of table C 8.5.1 and the values for LC 1 definitelyare :

f = 2.6Mpa - The average compression resistance

τ0=0.05Mpa - The average resistance in shearing for diagonal rift of masonry

fv0 =0.13Mpa - The shearing resistance in the horizontal slide of masonry

E = 1200Mpa –The average value of the modulus of elasticity (normale)

G =400Mpa – The average value of the module of elasticity (tangential)

w =18kN/m3 - The average weight for unit

Tabella C8.5.II -*Coefficienti correttivi massimi da applicarsi in presenza di: malta di caratteristiche buone; ricorsi o listature; sistematiche connessioni trasversali; consolidamento con iniezioni di malta; consolidamento con intonaco armato; ristilatura armata con connessione dei paramenti.*

Tipologia di muratura	Stato di fatto			Interventi di consolidamento			
	Malta buona	Ricorsi o listature	Connessione trasversale	Iniezione di miscele leganti (*)	Intonacoarmato (**)	Ristilatura armata con connessione dei paramenti (**)	Massimo coefficiente complessivo
Muratura in pietrame disordinata (ciottoli, pietre erratiche e irregolari)	1,5	1,3	1,5	2	2,5	1,6	3,5
Muratura a conci sbozzati, con paramenti di spessore disomogeneo	1,4	1,2	1,5	1,7	2,0	1,5	3,0
Muratura in pietre a spacco con buona tessitura	1,3	1,1	1,3	1,5	1,5	1,4	2,4
Muratura irregolare di pietra tenera (tufo, calcarenite, ecc.,)	1,5	1,2	1,3	1,4	1,7	1,1	2,0
Muratura a conci regolari di pietra tenera (tufo, calcarenite, ecc.,)	1,6	-	1,2	1,2	1,5	1,2	1,8
Muratura a blocchi lapidei squadrati	1,2	-	1,2	1,2	1,2	-	1,4
Muratura in mattoni pieni e malta di calce	(***)	-	1,3 (****)	1,2	1,5	1,2	1,8
Muratura in mattoni semipieni con malta cementizia (es,: doppio UNI foratura ≤40%)	1,2	-	-	-	1,3	-	1,3

(*) I coefficienti correttivi relativi alle iniezioni di miscele leganti devono essere commisurati all'effettivo beneficio apportato alla muratura, riscontrabile con verifiche sia nella fase di esecuzione (iniettabilità) sia a-posteriori (riscontri sperimentali attraverso prove soniche o similari).

(**) Valori da ridurre convenientemente nel caso di pareti di notevole spessore (p.es. > 70 cm).

(***) Nel caso di muratura di mattoni si intende come "malta buona" una malta con resistenza media a compressione f_m superiore a 2 N/mm². In tal caso il coefficiente correttivo può essere posto pari a $f_m^{0,35}$ (f_m in N/mm²).

(****) Nel caso di muratura di mattoni si intende come muratura trasversalmente connessa quella apparecchiata a regola d'arte.

Fig.39 . Corrective maximum coefficients to apply in the above cases

The mortar is not considered good(malta buona).According to the upper table, there are no applicable corrective coefficients over the parameters resisting the masonry.

According to the observations, it consists that the beams of the stairs have important damage from the last earthquake. The cover is 3-4 cm. The concrete taking into consideration the construction is considered M-150 or C 20//25. The project uses slabs with a 10 cm thickness. They are supported by the reinforcement in two directions as well as the contour. The concrete is considered of class C20/25.The steel is taken into account with Rs = 2100 kg/cm2 and is not rebared.Additionally, the concrete in this construction is not a component that is of secondary importance due to its function in mitigating seismic loads. Preliminary data for the category of testing materials is typically the concrete results of weak building materials in classes C 20/25.

Note: The information connected with existing concretecharacteristic elements has been limited.

Here are the plans of the structure taken from the municipality of Lezha:

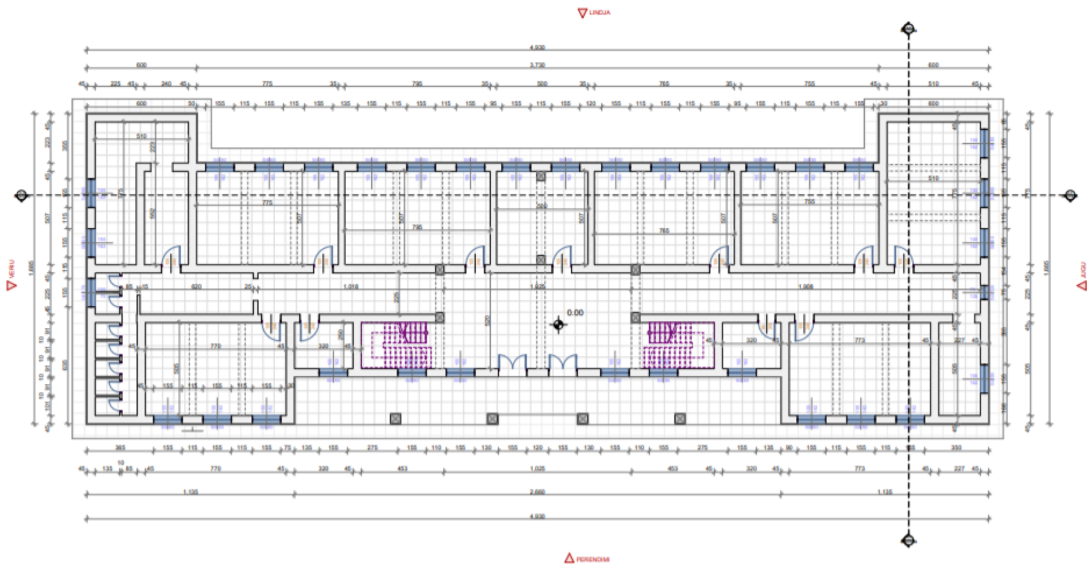


Fig 40.Ground floor plan of the existing building

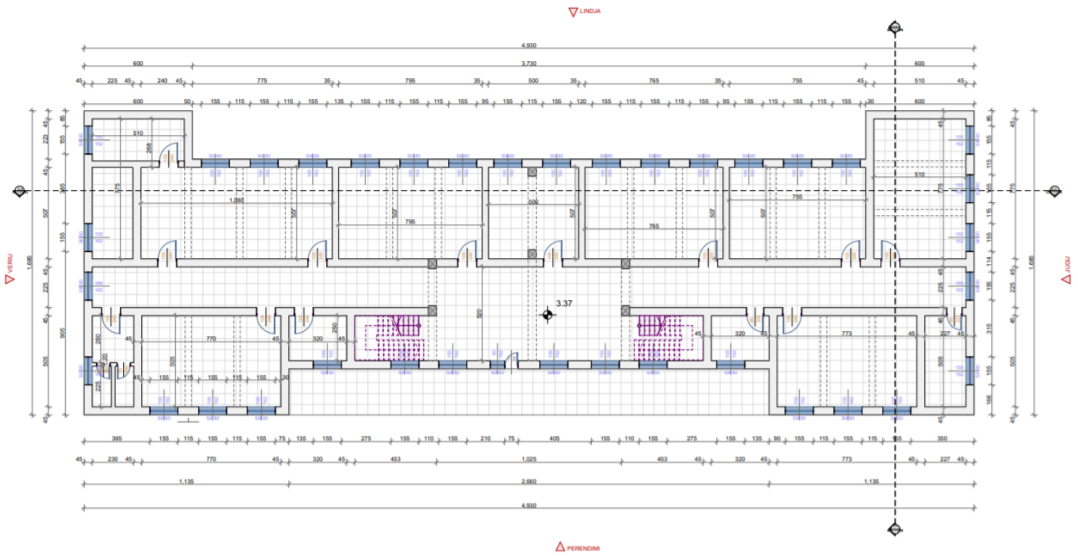


Fig 41.First floor plan of the existing building

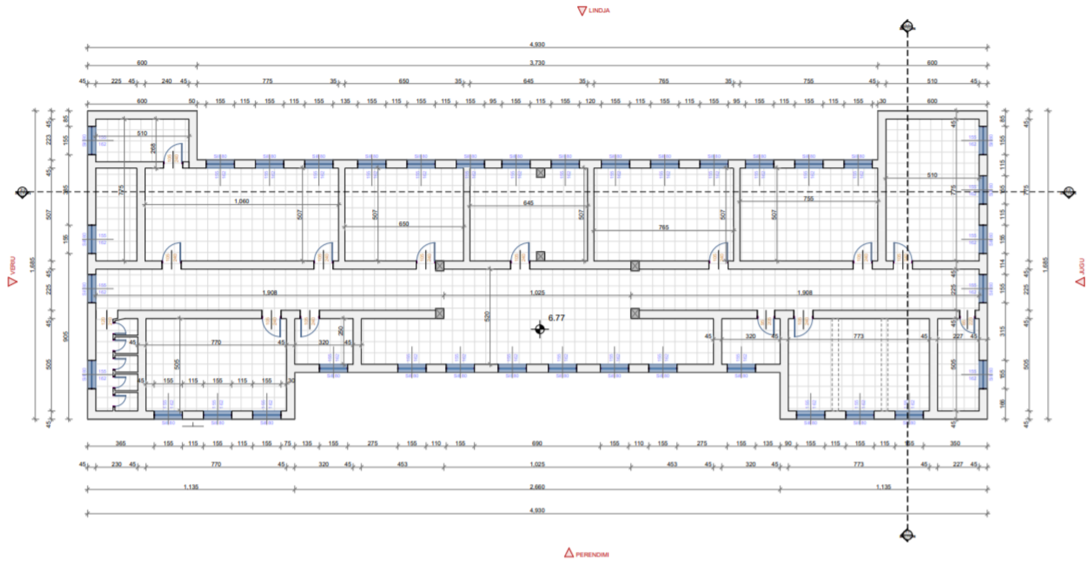


Fig 42.Second floor plan of the existing building

4.3 Data for modeling and analyzing the existing structure

1- Permanent loadings

For the masonry wall ρ =1800kg/m3)

Concrete slabs ρ =2500kg/m3)

Layers over the slab +plaster =250kg/m2

2-Variabel loadings

School =300kg/m2

Wind loadIt is neglected because it is not dimensioned for this kind of objects.

Snow loadIt is neglected because it is not calculated for this kind of objects.

Seismic loadsBelow we can give the graph of elastic spectra

3-According to the descriptions given in NTC 2018 we have :

Operational limit state that is the state when the structure is in full function, without problems.

Damage limit state that is the state when the structure has some small damages but they do not influence its use.

Life–saving limit state is the state when the structure has cracking on its elements or its element may be damages by different phenomens , but during its use the lives of the people are not concerned.

Ultimate limit state (state limit of collapse) is the state when the structure has several structural damages and need to be demolished.

Following there is shown the elastic spectra which helps the designing process (γM=1)

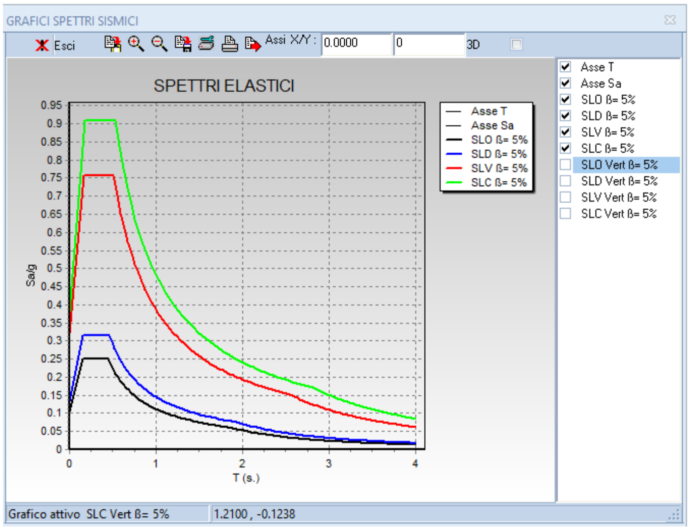


Fig 43. The elastic specters for the importance coefficient is γ_I=1

By using the upper spectrum it can be continued the dynamic analysis of the structure that consists in determining the mass of matrix , the stiffness matrix and than the determination of main forms of shakings through the modal analysis.

Load combination

According the descriptions of NTC 2018 section 7.3.4.2 the distributions of forces over the structure are two : one inproportion with static forces and the other in proportion with the masses. In section 7.2.5 of the same normative is shown that three of components of seismic actions are combined as below :

1*Ex+0.3*Ey+0.3*Ez Equation 4.4.1

For this analysis it is negleted the vertical component of the seismic action . Whereas the horizontal component is combined within (example Ex+0.3Ey) alternating the signs.

Regarding the choice of the point of control it is chosen the center of mass of the last floor. The vertical component of seismic action isnegleted in this analysis .

The most important parameters of the seismic hazard for the construction site are:

Seismic studies

According KTP –Nr-2-89	According EC 8
Intensity VIIIballe (MSK-64) Category of soil II KE =0.22 KR =1 For the category II of soils $0.65 < \beta = 0.8/Ti < 2.0$ TC=0.4 TD=1.23 The elastic designing spectrum is given : $Sa(T) = kE\ \beta(T)\ g$	$0 \leq T \leq TB$: $Se(T) = ag.S.[1 + (T/TB).(\eta.2,5 - 1)]$ $TB \leq T \leq TC$: $Se(T) = ag.S.\eta.2,5$ $TC \leq T \leq TD$: $Se(T) = ag.S.\eta.2,5. [TC/T]$ $TD \leq T \leq 4s$: $Se(T) = ag.S.\eta.2,5. [TC.TD/T2]$ ag=0.248 For contructionsoils type B S=1.2 TB=0.15s TC=0.5s TD=2s

Here are shown graphically the elastic spectra according KTP-89 and EC-8

EC8

ag	0.248
S	1.2
TB	0.15
Tc	0.5
TD	2
β	0.2
q	1

KTP-89

Ke	0.22
Tc	0.4
TD	1.23

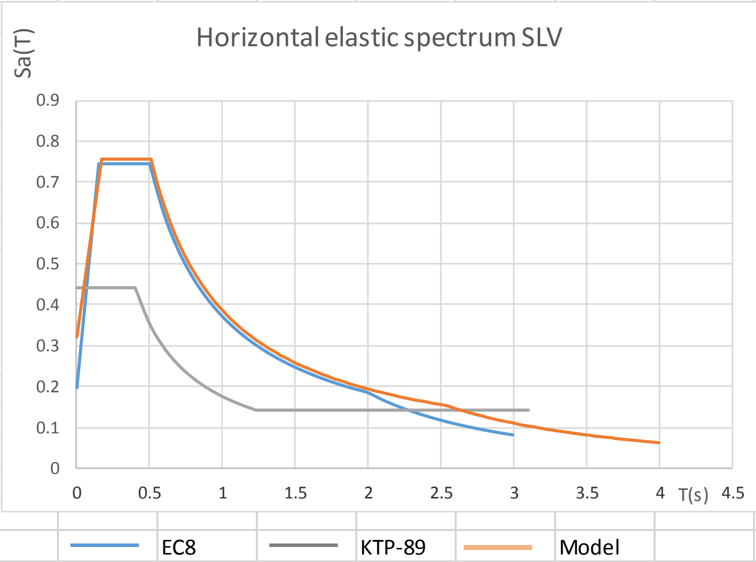


Fig .44 The elastic spectra according KTP-89 and EC-8

It is obvious that the specter according EC8 gives results in favor of the reserve. From the library of CDS is chosen a specter that covers the one of EC 8 in all the values.

For the progression of the work the specter according EC 8 is choosen over the one on KTP -89

4.4 Modeling of the structure in CDS –Win

For the structural assessment of this building the analysis according the actual state (stato di fatto)or (ante operam)need to be done. The modeling of this structure with bearing masonry walls means that the wall take an important part in the seismic resistance , which is very important. The model of the building is simplified because it is hard to approach the real model considering having a dis-homogen material compared it with concrete or steel . The method of finite elements (FEM finite element method) is essential for the final model in order to pass from the physic model into the numerical one , neglecting some non important factors and evaluatinng some that are determining. The choice of these factors according their importance should be made carefully , in a such way that the numerical model to approach the physic one .

4.4.1 The modeling scheme for the structure with bearing walls

In buildings with bearing walls, firstly we should evaluate the behavior of brickswalls considering the ones as wall slats (setti), and from which is obtained the one called the global structural behavior.

METHODS OF EQUIVALENT FRAMES (SAM)

Sam method (simplified analysis of masonry) is formulated in two dimensional and after converted in that three dimensional. It serves to valuate the global reaction of structures in which the resistance is determined from the reaction in the plan of wall slats.

To make it clearer, it refers to a multi-story masonry wall that is subjected to constant vertical and horizontal loads that are applied in ascending order according to the quotas of the floors. When the geometry is sufficiently regular, as it is in this case, the wall can be converted into an equivalent frame formed by vertical elements and horizontal beams, considering the joints as rigid. The elements of the frame are modeled as elements (beams) deformable by the normal force, bending and shear, while applying the hypothesis of absolute stiffness to the joints, which means that the part from the beam or column to the joint is considered non-deformable (rigid offset)

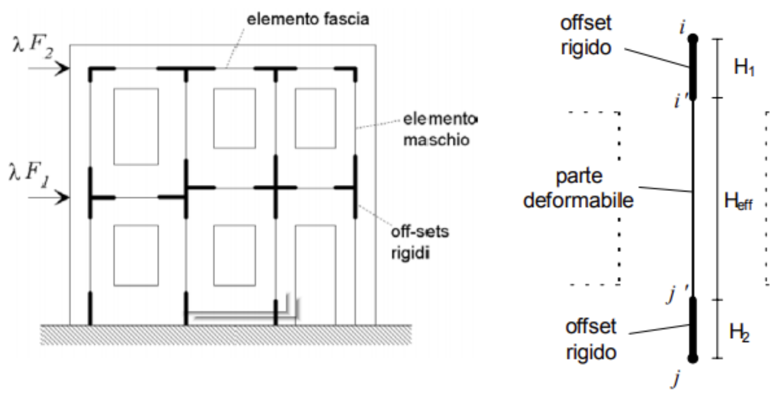


Fig 45. The Sam method example

The wall column is composed of a deformable central part and two completely rigid extremes. The deformable height, the so-called effective height, is determined according to the model given by Dolce; and takes into account the presence of two windows and parts of the masonry that are not continuously generated precisely from the latter.

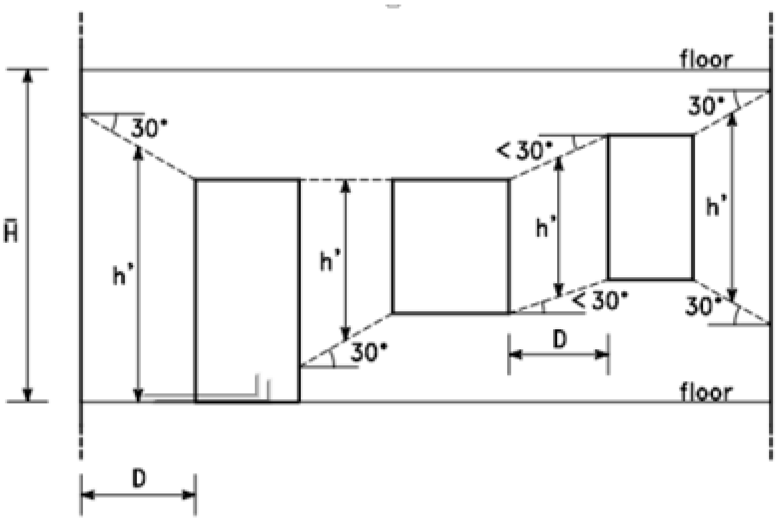


Fig 46.The definitions of effective height of the wall (Dolce 1989)

The force-deformation diagram for the vertical elements of the masonry is considered elastoplastic type, where the forms of destruction are from external compression in the diagonal view or horizontal view, in the bed of the mortar joint. As for the horizontal elements, the hypothesis remains the same as for the forms of destruction from eccentric compression or shearing.

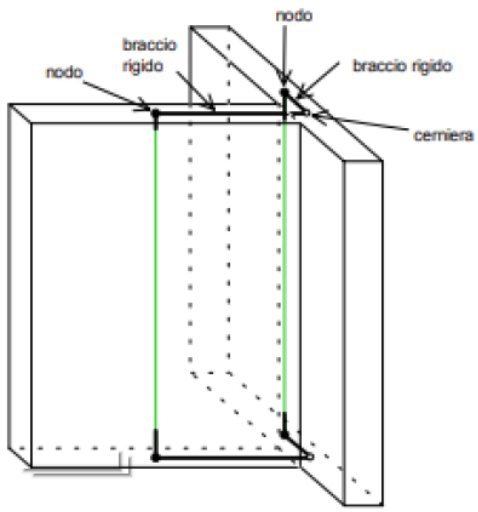


Fig 47.Rigid join (Dolce 1989)

For this analysis, the equivalent frame method was used and the structure model was made through the CDS win software that implements the aforementioned theories.

CDS is a module for the 3-dimensional calculation of masonry structures.

The modeling begins by setting the set for each, where the latter represents the masonry walls. In each access break, a fili is marked at the beginning, which is the beginning and end of the wall. Then each wall is attributed to the physico-mechanical properties of the masonry based on the type of wall and the degree of LC recognition described earlier

For this analysis, two types of walls were used according to table C8.5.1 of the 2019 Circular, where the latter is also in the CDS materials archive. Signed as masonry wall 13 are all the materials included in the walls, in height of the building and with masonry 14 is signed the material that constitutes the structure under the quote 0.00. Both these materials are subject to the confidence coefficients that belong to the LC1 level of recognition.

After finishing the modeling of the walls, the door and window openings are placed in them. The position of the windows and doors and their dimensions are referred to the actual condition measured on site.

Basically, they are also modeled as beams in the shape of an inverted T, where the component material is masonry 14. The slabs of the type floor and in 0.00 are 10 cm thick slabs where the temporary and permanent loads are acting.

After that, the 3D model is generated where the program itself determines the equivalent frames according to both directions. For all mid-plan slabs, it is attributed (piano-sizmico), which means that the floor slab is absolutely stable. This criterion is fulfilled by the fact that the slab is a monolithic plate with a thickness of over 7cm.

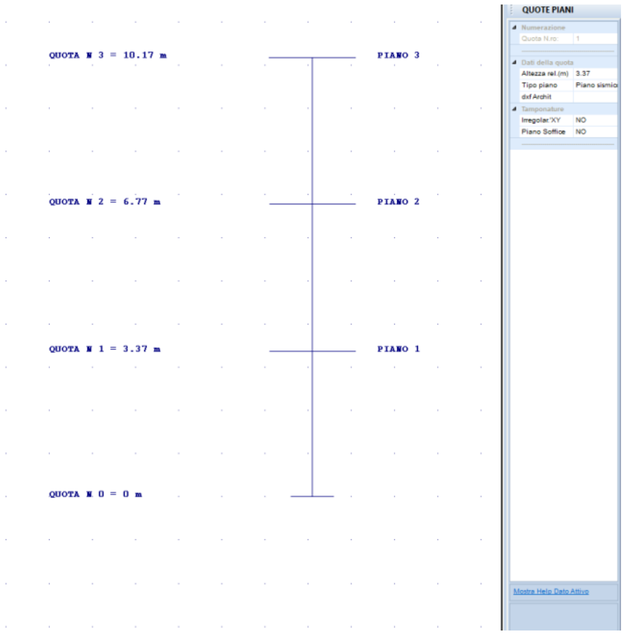


Fig 48. The vertical frame of one axes while modeling with CDS-Win software

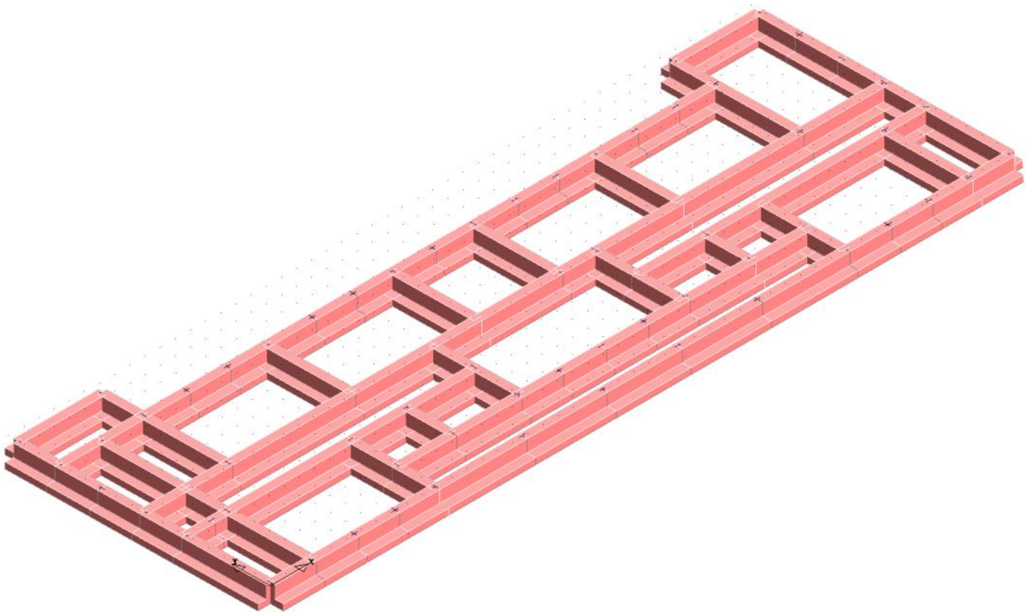


Fig 49. Foundations of the structure while modeling with CDS-Win software

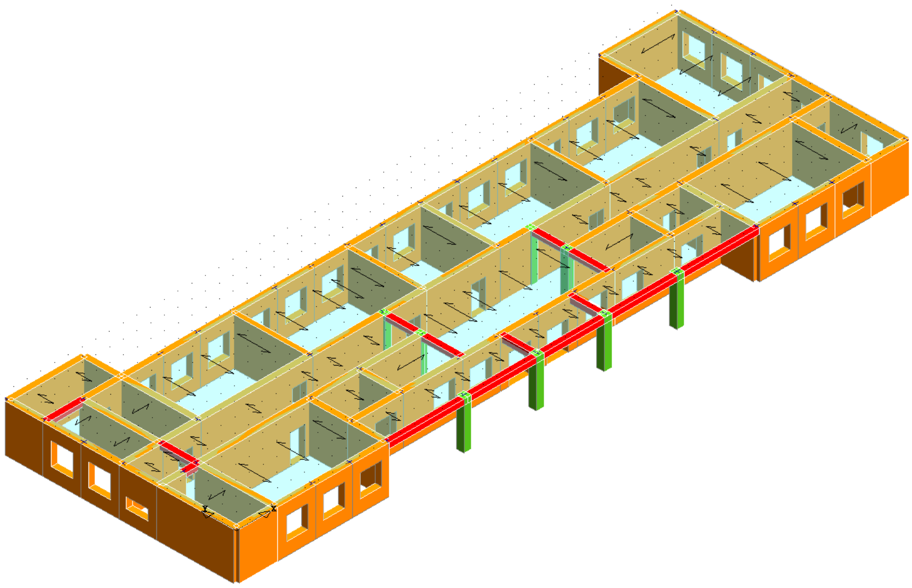


Fig 50. Floor, quota + 0.00 of the structure while modeling with CDS-Win software

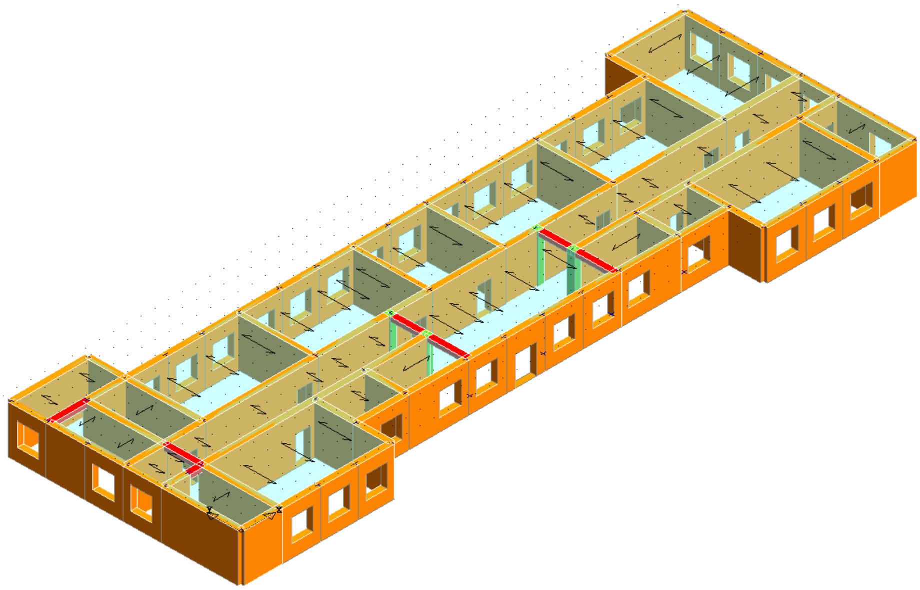


Fig 51.Typical floor, of the structure while modeling with CDS-Win software

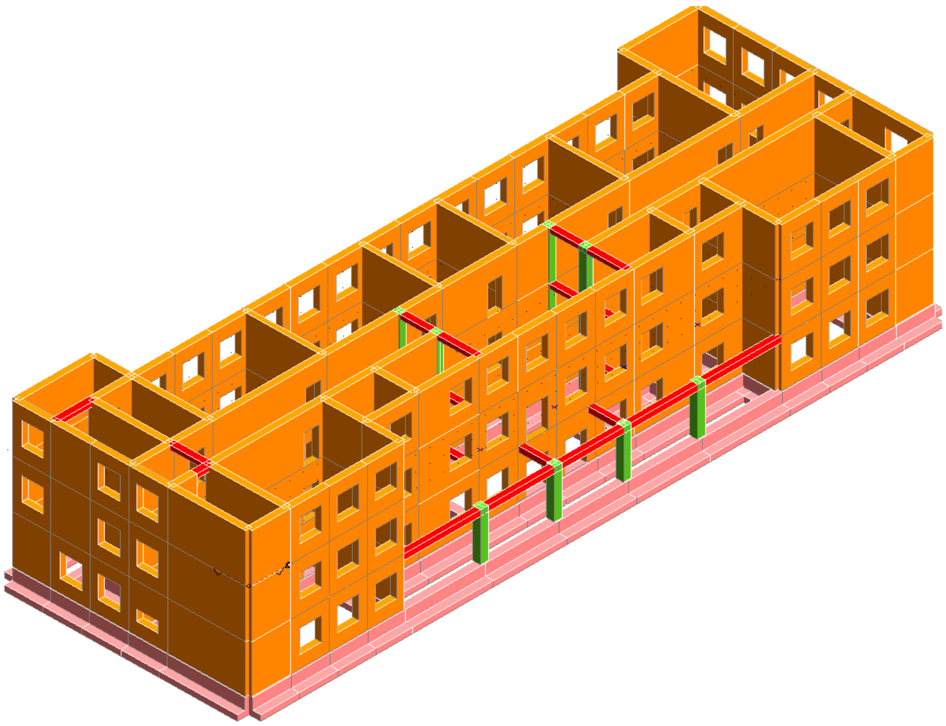


Fig 52. 3d-view, of the structure without slabs while modeling with CDS-Win software

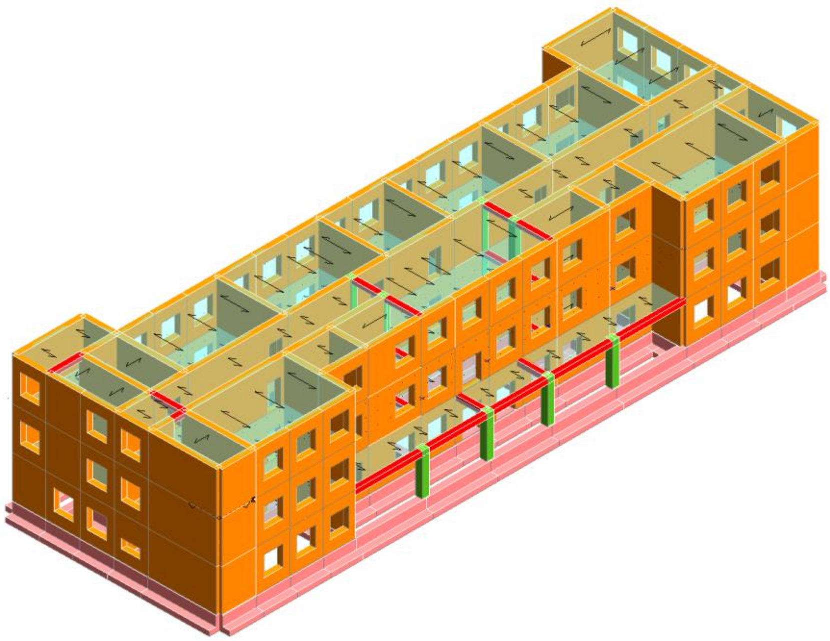


Fig 53. 3d-view, of the structure with slabs while modeling with CDS-Win software

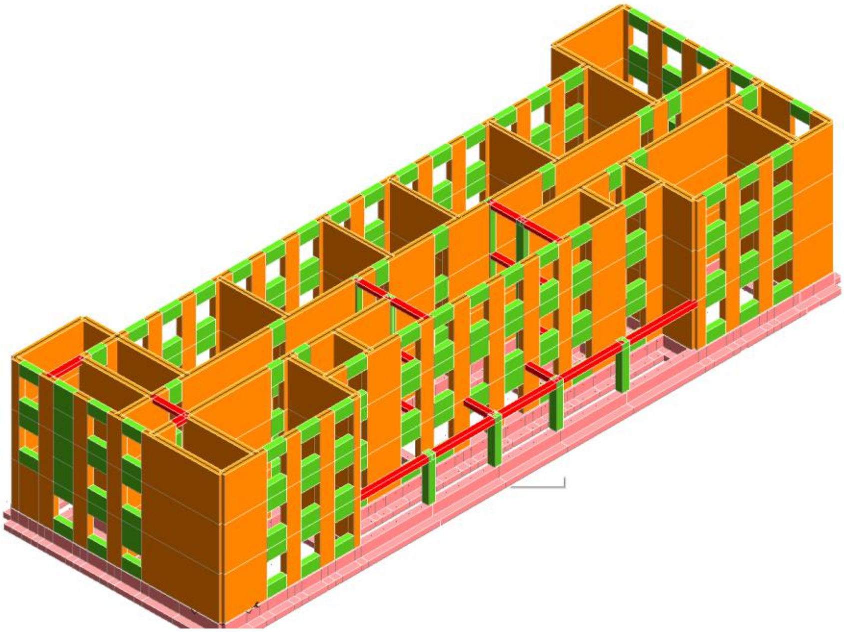


Fig 54. 3d-view, of the structure generalization with slabs while modeling with CDS-Win software

4.5 Methods of structural analysis

Referred to the normatives the types of analyses are devided in two main groups ,in linear and non linear , addopted in the function of the structure . There are predicted four ways for the seismic analyses that can be followed :

Linear static analyses   (a.k.a Livelli di conoscenza / knwoledge level):It is based in the idealization of structures as a linear elastic system and the seismic action as the system of static forces which acts in the center of mass in every rigid diaphragm (floors).

Linear dynamic analyses (LDA-Linear dynamic analysis): It is based on the determination of the modes of vibrations of the structure idealized in the elastic field

Non linear static analyses (NLSA-NON Linear static analysis): Consists of the placement of the structure under the action of gravitational load , and one system of horizontal forces that are raised in a monotonic way until the achievment of a deformation that consist of the collapse

Non linear dynamic analyses (NLDA-Non linear dynamic analyses): Typology that estimates, through the integration of the equation of motion, seismic reaction of the structure,under the behaviour of non-linear hypothesis.

For the dissipative systems as the masonry contructions, the methods of linear analyses (LSA and LDA) are more limited, until they underestimate the resisting capacity and deforming of the materials. The methods of non-linear statics (NLSA and NLDA) are more adapted but according to NLDA they are more difficult to be applicable in practice considering the great volume of calculations, from the computer mostly.

There is nothing else left until to be limited to the NLSA called the pushover analysis. The normatives allow the use of the pushover analysis in the masonry buildings and in the buildings where the mass according to the first mode of vibration is less than 75 %.

4.5.1 The dynamic analyses

The dynamic analysis is made following the steps:

The modal analysis (the determination of periods of free shaking)

The determination of the calculated specter

The combination of effects

The modal analysis presents a procedure for the assessment of the seismic action effects and it is done referring the definitions in the way of vibrations of the structure considering the elastic field.

The analysis should be taken in consideration that all the ways of vibration give an important contribution for the dynamic reaction of the structure considering the elastic field.

The analysis should be taken in consideration that all the way of vibrations that give an important contribution important for the dynamic reaction of the structure, according the participant mass. Connected to it, the section 7.3.3.1 of

NTC18 specifies that it is important to take in consideration all the modes in the participant mass higher than 5 % and in every case a number mode the total mass participant of them is higher than 85 %.

Combining the effects and individual modes of vibrations, the combination most used are: SRSS (the square root of the sum of the square) and CQC (full quadratic combination), which are expressed as :

SRSS $E = \sqrt{\Sigma E_i^2}$ Equation 4.5.1

CQC $E = \sqrt{\Sigma_i \Sigma_j \rho_{ij} E_i E_j}$ Equation 4.5.2

Where

Ej is the effect according to mode J

ρ ij is a corrective coefficient between the modes i and j

It is done the first control of the structure by performing a linear dynamic analysis. The structure is regular in plan and in height where the behavior factor is taken :

q=1.75*αu/α1 NTC 2018 tab 7.3.II

q=1.75*1.7=2.9 NTC 2018 § 7.8.1.3

Here is shown the spectra that the structure can afford in different limit states: The computed spectra are as follows :

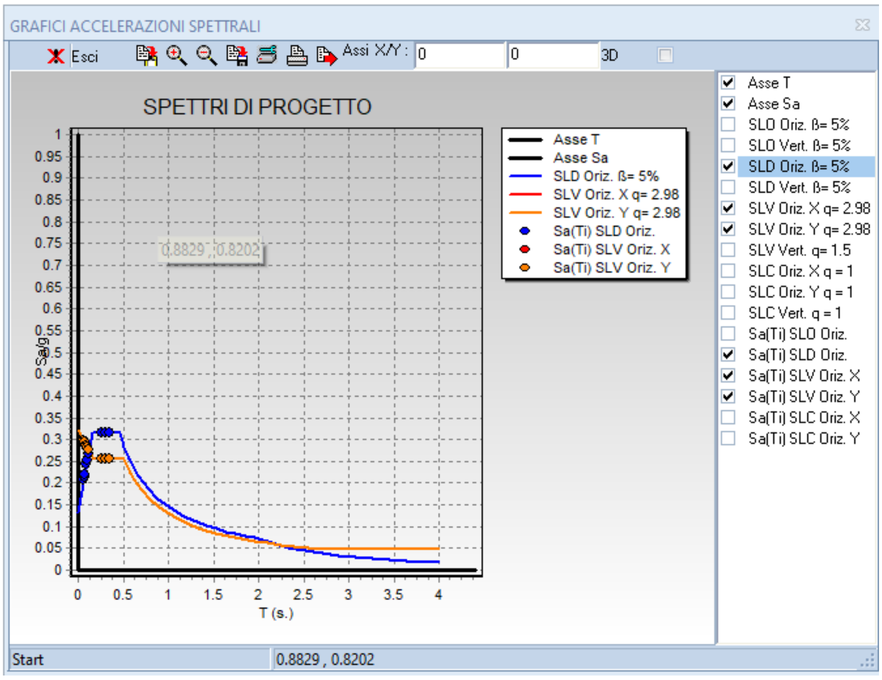


Fig 55. The computed spectra for the masonry structure

The results of the modal linear analysis are shown in the tables below:

Table 30

PULSATIONS AND MODES OF VIBRATIONS													
ModE N.ro	Pulsation (rad/sec)	Period (sec)	Damping Mod(%)	Sd/g SLO	Sd/g SLD	Sd/g SLV X	Sd/g SLV Y	Sd/g SLC X	Sd/g SLC Y	Floor N.ro	X (m)	Y (m)	Rot (rad)
1	17.814	0.35270	5.0		0.316	0.255	0.255			1	0.001263	0.005466	0.000128
										2	0.002728	0.011504	0.000262
										3	0.003876	0.016356	0.000365
2	20.494	0.30658	5.0		0.316	0.255	0.255			1	-.003126	0.014806	-.000535
										2	-.006623	0.030475	-.001096
										3	-.009285	0.042271	-.001516
3	23.850	0.26345	5.0		0.316	0.255	0.255			1	0.009774	-.001613	0.000040
										2	0.019598	-.003528	0.000099
										3	0.025863	-.004872	0.000142
4	56.479	0.11125	5.0		0.267	0.278	0.278			1	0.004240	0.009113	0.000481
										2	0.002259	0.004420	0.000240
										3	-.003901	-.007775	-.000447
5	62.691	0.10022	5.0		0.254	0.282	0.282			1	0.007958	-.038028	0.001254
										2	0.003743	-.017845	0.000593
										3	-.007462	0.035109	-.001168
6	68.508	0.09171	5.0		0.243	0.285	0.285			1	0.022692	-.003003	0.000073
										2	0.009721	-.001066	0.000032
										3	-.021111	0.002797	-.000074
7	88.081	0.07133	5.0		0.219	0.293	0.293			1	0.003267	0.007060	0.000413
										2	-.003638	-.009268	-.000474
										3	0.001838	0.004974	0.000247
8	96.064	0.06541	5.0		0.212	0.296	0.296			1	-.006336	0.031333	-.001042
										2	0.007855	-.037350	0.001231
										3	-.004502	0.020818	-.000686
9	101.998	0.06160	5.0		0.207	0.297	0.297			1	0.018113	-.001981	0.000065
										2	-.022856	0.003107	-.000101
										3	0.012903	-.001939	0.000062

Table 31

CENTERS OF GRAVITY , MASSES AND RIGIDITY														
IDENTIFIER		CENTER OF GRAVITY MASSES AND RIGIDITY							RIGIDITY FLEXIBLE AND TORSIONAL					
FLOOR N.ro	QUOTE (m)	WEIGHT (t)	XG (m)	YG (m)	XR (m)	YR (m)	DX (m)	DY (m)	floor (m)	Bfloor (m)	Rig.FleX (t/m)	Rig.FleY (t/m)	RigTors. (t*m)	(r/ls)²
1	3.37	1115.74	23.91	7.42	22.78	7.19	-1.13	-0.22	16.40	48.85	345469	237490	75493632	
2	6.77	1064.50	23.89	7.75	22.81	7.43	-1.08	-0.32	16.40	48.85	272536	166032	55834996	
3	10.17	833.47	24.08	7.80	22.78	7.50	-1.29	-0.29	16.40	48.85	233449	121679	41929432	

Table 32

VARIATIONS MASSES AND RIGIDITY OF THE FLOOR															
				D I R E C T I O N X						D I R E C T I O N Y					
FLOOR N.ro	Quote (m)	Weight (t)	Variac. (%)	Shearing Comb.(t)	shearing modal(t)	Shiftt. (mm)	Klat. (t/m)	Variaz (%)	Teta	Shearing Comb.(t)	Shearing modale(t)	shiftt. (mm)	Klat. (t/m)	Variac (%)	Teta
1	3.37	1115.74	0.0	726.45	669.53	2.14	313273	0.0	0.008	678.29	621.67	3.26	190585	0.0	0.011
2	6.77	1064.50	-4.6	552.78	531.40	2.11	251559	-19.7	0.011	523.51	501.90	3.54	141786	-25.6	0.015
3	10.17	833.47	-21.7	283.76	269.54	1.34	201591	-19.9	0.006	276.48	263.88	2.80	94198	-33.6	0.010

Table 33

PERCENTAGES RIGIDITY PILLARS AND SETTS						
	RAPORT OF RIGIDITY IN X DIRECTION			RAPORT OF RIGIDITY IN Y DIRECTION		
Floor N.r	Rigidity Pillars ----------------- Rig.Pil+Rig.Setti	Rigidity Setts ----------------- Rig.Pil+Rig.Setti	Rigid.Elem.Second ----------------- Rig.Pil+Rig.Setti	Rigidity Pillars ----------------- Rig.Pil+Rig.Setti	Rigidity Setts ----------------- Rig.Pil+Rig.Setti	Rigid.Elem.Second ----------------- Rig.Pil+Rig.Setti
1	0.02	0.98	0.00	0.02	0.98	0.00
2	0.01	0.99	0.00	0.01	0.99	0.00
3	0.01	0.99	0.00	0.01	0.99	0.00

Table 34

STRUCTURAL REGULARITY												
				SEISMIC FORCE 1				SEISMIC FORCE 2				
FLOOR N.ro	QUOTE (m)	Res X t	Res Y t	Dem X t	Dom Y t	Res/Dem	Var.R/D	Dem X t	Dem Y t	Res/Dem	Var.R/D	Verifications
1	3.37			744.38	-55.35			54.52	696.44			VERIF
2	6.77			554.40	-40.85			41.70	524.43			VERIF
3	10.17			283.76	-20.80			21.59	276.48			VERIF

Table 35

FACTORS AND FORCES OF MODAL FLOORS S.L.V.									
DIRECTION OF SEISMICITY : 0°									
Exited mass (t): 3013.7 Total mass (t): 3013.7 Raport:1									
Mode N.ro	Factor Modale	Fmod/Fmax (%)	Mass Mod Eff. (t)	Mmod/Mtot %	Floor N.ro	FX (t)	FY (t)	Mt (t*m)	Mom.Ecc. 5% (t*m)
1	1.947	3.80	3.79	0.13	1	0.17	4.72	24.13	128.72
					2	0.37	9.38	47.78	195.94
					3	0.42	10.40	53.12	210.67
2	5.051	9.85	25.51	0.85	1	1.21	2.89	-201.13	
					2	2.57	5.88	-404.49	
					3	2.72	6.20	-421.00	
3	51.255	100.00	2627.11	87.17	1	138.14	-9.60	112.71	
					2	261.86	-16.25	272.57	

FACTORS AND FORCES OF MODAL FLOORS S.L.V.									
DIRECTION OF SEISMICITY : 0°									
Exited mass (t): 3013.7 Total mass (t): 3013.7 Raport:1									
Mode N.ro	Factor Modale	Fmod/Fmax (%)	Mass Mod Eff. (t)	Mmod/Mtot %	Floor N.ro	FX (t)	FY (t)	Mt (t*m)	Mom.Ecc. 5% (t*m)
					3	269.54	-15.85	306.93	
4	0.833	1.63	0.69	0.02	1	0.17	5.32	38.95	
					2	0.10	2.50	18.79	
					3	-0.08	-3.58	-27.32	
5	1.044	2.04	1.09	0.04	1	0.44	2.64	-106.49	
					2	0.27	1.15	-49.61	
					3	-0.40	-1.72	73.36	
6	17.682	34.50	312.67	10.37	1	124.71	-7.02	74.01	
					2	50.91	-1.67	28.09	
					3	-86.37	4.27	-51.06	
7	0.195	0.38	0.04	0.00	1	0.01	1.08	8.20	
					2	0.00	-1.25	-9.21	
					3	0.00	0.52	3.77	
8	0.466	0.91	0.22	0.01	1	0.21	0.99	-41.52	
					2	-0.25	-1.16	48.15	
					3	0.10	0.49	-20.16	
9	6.526	12.73	42.59	1.41	1	38.16	-0.93	27.62	
					2	-45.57	1.43	-40.59	
					3	20.07	-0.71	19.68	

Table 36

FACTOR AND FORCES ANF MODAL FLOORS S.L.V.									
DIRECTION OF SEISMICITY: 90°									
Exited mass (t): 3013.7 Total mass (t): 3013.7 Raport:1									
Mode N.ro	Factor Modale	Fmod/Fmax (%)	Mass Mod Eff. (t)	Mmod/Mtot %	Floor N.ro	FX (t)	FY (t)	Mt (t*m)	Mom.Ecc. 5% (t*m)
1	49.389	100.00	2439.29	80.94	1	4.40	119.77	612.15	383.43
					2	9.33	238.02	1212.26	583.65
					3	10.77	263.88	1347.68	627.52
2	11.630	23.55	135.25	4.49	1	2.79	6.65	-463.09	
					2	5.92	13.54	-931.32	
					3	6.26	14.28	-969.34	
3	3.192	6.46	10.19	0.34	1	-8.60	0.60	-7.02	
					2	-16.31	1.01	-16.97	
					3	-16.78	0.99	-19.11	
4	18.339	37.13	336.32	11.16	1	3.83	117.14	857.41	
					2	2.17	55.02	413.49	
					3	-1.76	-78.73	-601.43	
5	7.058	14.29	49.81	1.65	1	2.99	17.87	-720.22	
					2	1.82	7.78	-335.48	
					3	-2.73	-11.60	496.15	
6	0.875	1.77	0.77	0.03	1	-6.17	0.35	-3.66	
					2	-2.52	0.08	-1.39	
					3	4.27	-0.21	2.53	
7	6.072	12.29	36.87	1.22	1	0.40	33.66	255.96	
					2	0.07	-39.05	-287.57	
					3	-0.13	16.21	117.53	
8	2.281	4.62	5.20	0.17	1	1.05	4.82	-203.07	
					2	-1.21	-5.70	235.52	
					3	0.48	2.41	-98.62	
9	0.109	0.22	0.01	0.00	1	-0.63	0.02	-0.46	
					2	0.76	-0.02	0.68	
					3	-0.33	0.01	-0.33	

Graphically below there are three modes of deformations of the structure.

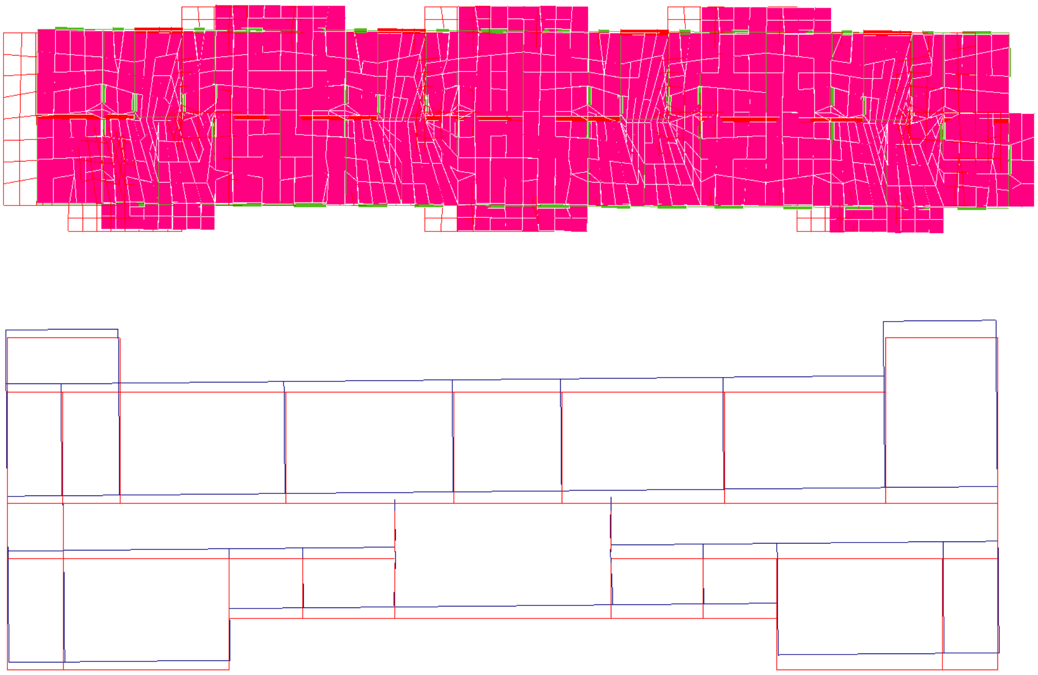


Fig.56 First mode of shaking

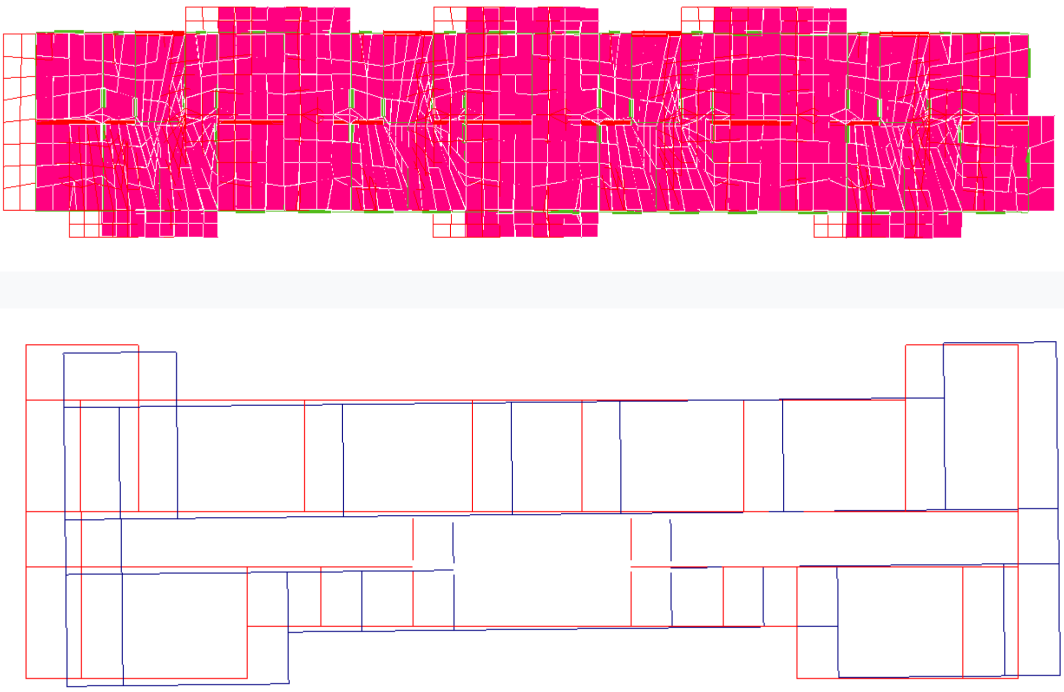
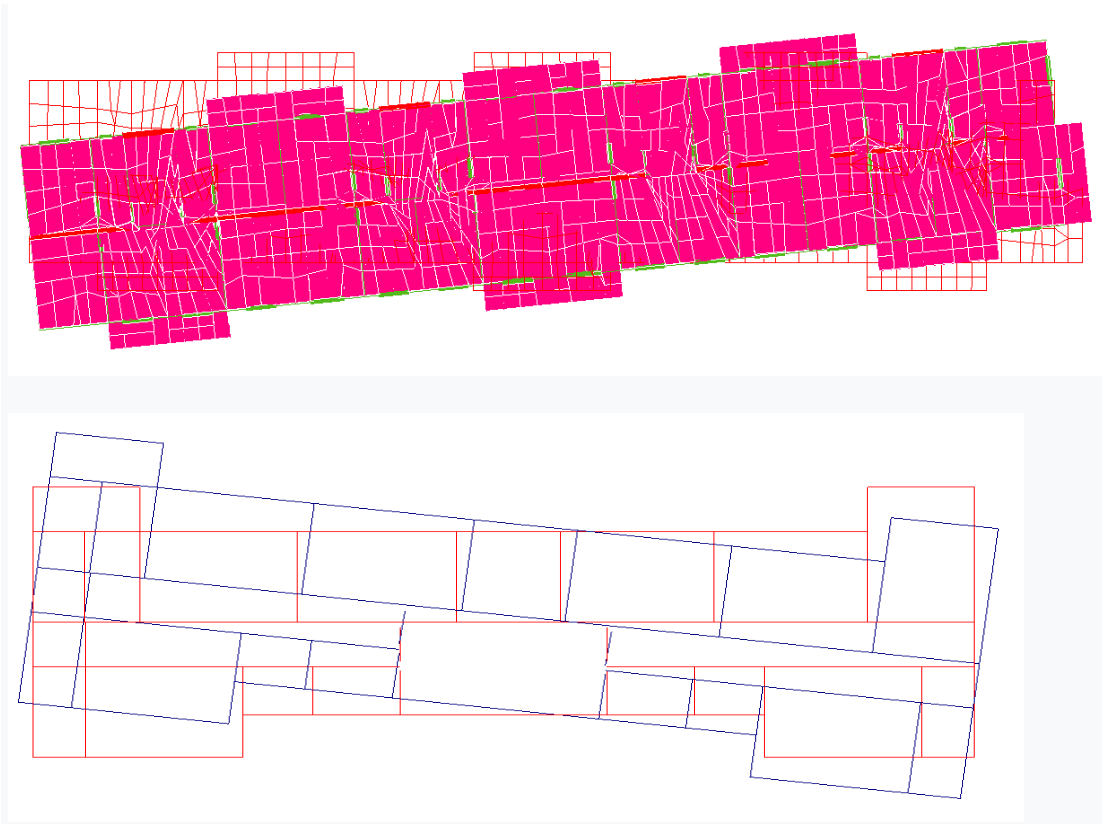


Fig.57 Second mode of shaking



Fig,58 Third mode of shaking

CDS highlights the elements that are not verified according to the colormap , as the type of the control for the one which is being exercised.

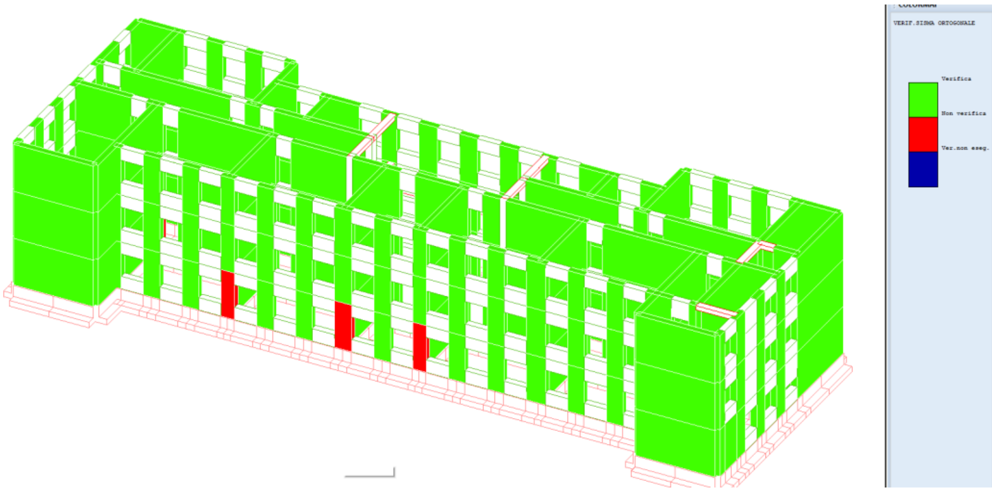


Fig 59. The results of 3-d modeling under the seismic load (in plane motion)

A type of finite element analysis known as a modal analysis enables us to look at the natural frequencies and mode shapes of a building or component.Understanding these natural frequencies is crucial for attempting to anticipate how a structure or component will react when subjected to vibrational frequencies in the field. All structures and components have natural frequencies.From the above tables we can found the natural frequencies of shaking for each mode of vibration ad the displacements of the building in each direction.For example according to the first mode of shape the natural period of shaking is 0.35270 sec and the deformations in X direction is 0.1263 cm , in Y direction 0.566 cm and the rotation of the building is 0.000128 rad. We can find the center of mass and the center of the rigidity of the structure from the above information (for each floor and for the structure as a whole element). By combining the modes, from the results of modal dynamic analysis we found out that the building itself is regular in plan and have enough stiffness and rigidity. The first mode of shake is in translation Y direction , the second is translation in X direction and the third is Torsional effect. This is a desiderable effect of our buildings' performance.

4.5.2 The Pushover analysis

According the data given in "Metodi di calcolo e tecniche di consolidamento per edifici in muratura " the push over analysis is the one adeguate for masonry buildings. The method consists of the application of a series of forces on every floor that rises gradually until the total collapse of the structure. It must be at the end of the analysis that the capacity of the structure to be deformed u_{max}will be higherthan the deformation demanded d max. In this type of analysis is done the conversion of a system with more degrees of freedom (MdoF multi-degree of freedom) in the one with one degree of freedom (SDOG single degree of freedom) considering the mechanical behavior, not more as elastic but as elasto-plastic. In other words, the pushover analysis consists in loading the structure until collapse, while acting external horizontal force, in a monotonic way called the loaded profile (the term pushover means pushing more). The result is expressed through the graph force-displacement(Vb-dc).In this graph, it is expressed the shearing force in the base and in the X-axis is the displacementof the controlling point (dc) or (the performance point) PP. According to the normative the calculation of masonry buildings is developed as below :

The determination of the curve of the capacity of the system MDOF

The determination of equivalent SDOF

The calculation of the displacement capacity Umax

The calculation of the displacement demand dmax

By confronting two last results and the results is positive if

$$S = U_{max}/d_{max} > 1 \qquad \text{Equation 4.5.2.1}$$

The pushover analysis serves to control the building in seismic conditions, the loads acting that act in it are combined :

$$E + G1 + G2 + \Sigma\, \psi 2i\, Qki \qquad \text{Equation 4.5.2.2}$$

As mentioned before the reaction of the structure SDOF is expressed through the curve of capacity that in the X-axis has a horizontal displacement dc that in general is accepted the one of the centers of mass of the last floor and in ordinate is the shearing force in base. By increasing the load until collapse it is obtained the capacity curve, in which the dependency between the force and displacement is not linear (the effect that comes from the plastification of elements includes the structure, and the reduction of the stiffness, because of the static scheme of the structure is modified.

The curve for one structure depends on a variety of factors, overall by the choice of the center of control and by the load in the profile that applies.

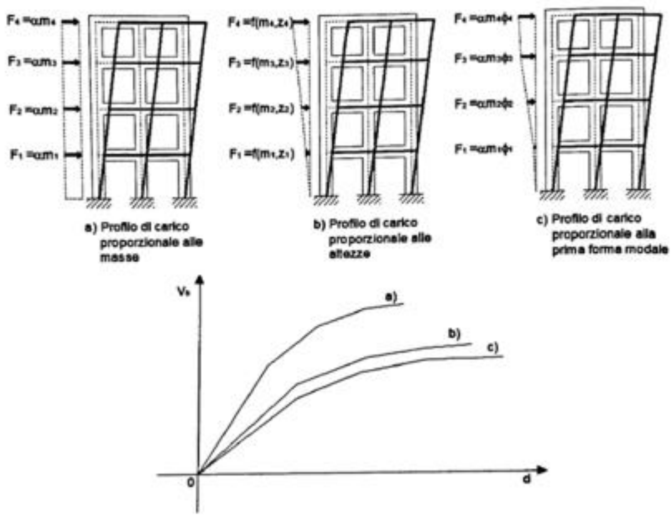


Fig.60 The capacity curve in the function of the loading force (Metodi di calcolo di consolidamento per edifice in muratura)

For the calculation of the internal forces, in difference with linear methods, because of the continued change of the static scheme of the structure, it is noticed that the graph force-displacement is not linear. Because of the voluminous calculations, this method is not applicable manually, so for the calculation is used the software, CDS Win

The curve of capacity is very important for the pushover analysis of a structure, In X-axis there are determined the deformations of a point of control for example the center of gravity of the last floor. By observing the graphs that are given below seems that they are not linear , reducing in this way the stiffness of the structures. According to the laws should be considered two loading profiles , one proportional to the masses and the other proportional to the forms of the shakings. As it is mentioned the new normative present the coefficient ζ_E that should be greater than one, for the assessment of the security of the existing objects. This coefficient represents the proportion between the seismic actions searching as this building was new. For the reason of the time that this time takes in the table below, there are shown results only for two directions in the seismic action, X and Y with the loading profile proportional to the mass.

Table 37 The Check of pushover analysis

Nr Pushover	Direction	Loading	Demamd (mm)	Capacity (mmm)	PGA	ζ	Check
1	X+0.3Y	Modo	37	28	0.174	0.74	No
3	Y+0.3X	Modo	40	24	0.136	0.58	No
1	X-0.3Y	Modo	35	28	0.192	0.82	No
3	Y-0.3X	Modo	37	23	0.14	0.60	No

No one of the controls doesn't resist positive for none of the directions (for more as it was combined (fx+0.3fy), and for more as there were included also the accidental eccentricity. Below we should represent graphically the curves of capacity.

Below it is given graphically the capacity curves for the limited state of the not collapse SLV.

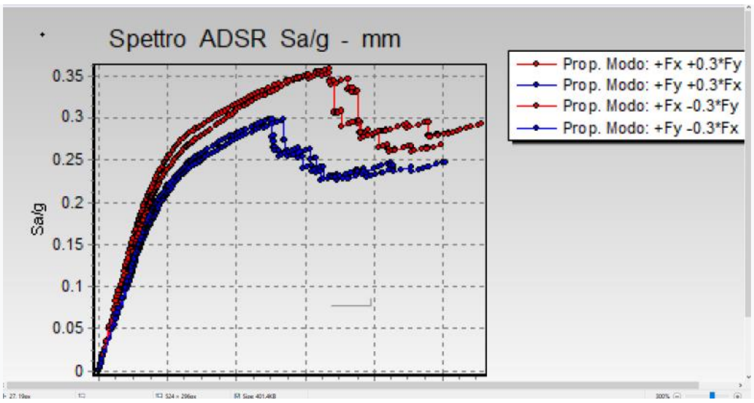


Fig.61 the capacity curves for the limited state of the not collapse SLV

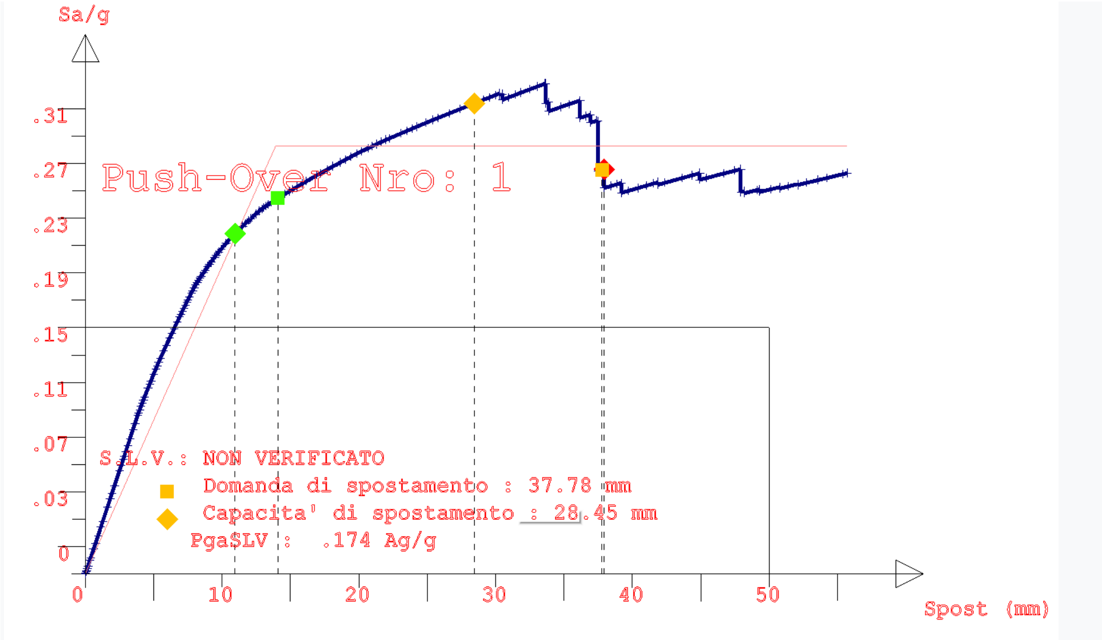


Fig.62 the capacity curves for the limited state of the not collapse SLV (Node 1)

Table 38

GENERAL RESULTS OF PUSHOVER ANALYSIS			
PUSH-OVER N.ro	1 -	Distrib.Forces Fx(+) Prop.Mode	
Angle of seismic force (Grd) Number of passes Resist.Max.	0 138	Numerber of total collapse Numerber of significant passes	30 193
Mass SDOF (t) Coef. of participation Rigidity SDOF (t/m) Period SDOF (sec)	2070.34 1.28 46520.84 0.42	Shear base max. (t) Resistence SDOF (t) Deformation . SDOF mm Raport of work hardening	952.70 647.52 14 0.000
Raportαu/α1 Coef of equivalent damping .(%)	17469.662 28	Behavior factor Ductility	3.494 4.003
LIFE SAFE LIMIT STATE			
D E M A N D		C A P A C I T Y	
Deformation mm S.L. Life safety limit state	37.788 NON VERIFIED	Deformation mm Number of previous passes	28.459 124
PgaLV/g Raport q*=Fe/Fy >3 Time of intervention (years) ------------------------	0.174 2.42 23	ZetaE=PgaLV/Pga 10% Asta3D Nro TrCLV (years) (TrCLV/TDLV)^a	0.742 218 0.726

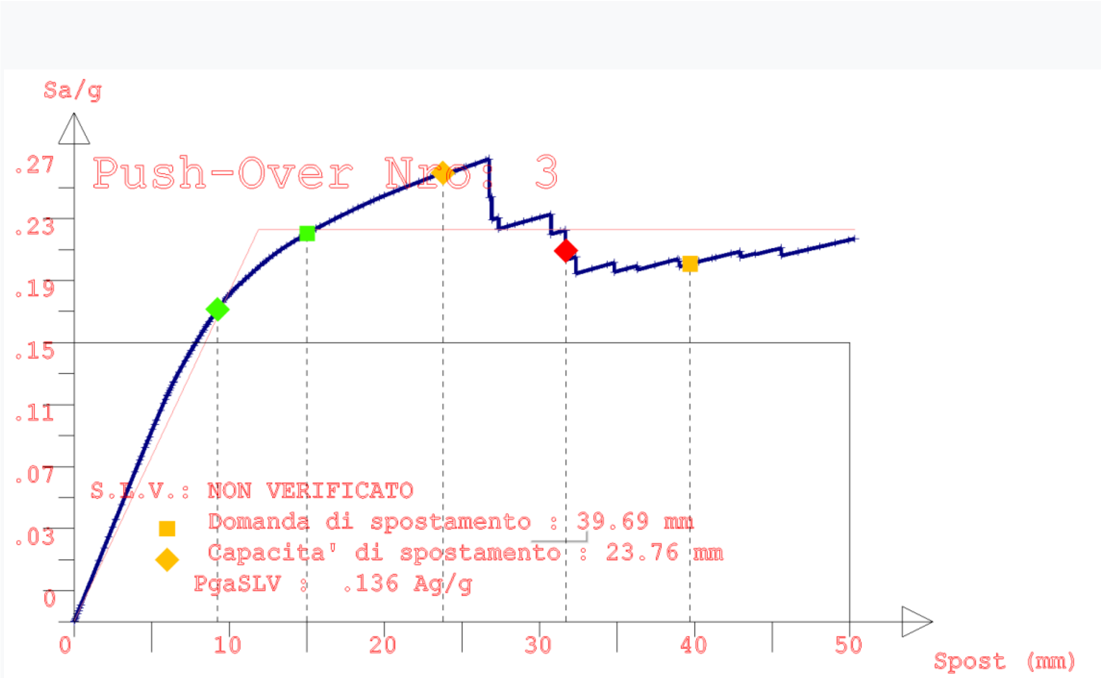


Fig.63 the capacity curves for the limited state of the not collapse SLV (Node 1)

Table 39

GENERAL RESULTS OF PUSHOVER ANALYSIS			
PUSH-OVER N.ro	3 -	Distrib.Forces Fx(+) Prop.Mode	
Angle of seismic force (Grd) Number of passes Resist.Max.	90 113	Numerber of total collapse Numerber of significant passes	30 162
Mass SDOF (t) Coef. of participation Rigidity SDOF (t/m) Period SDOF (sec)	1963.52 1.32 41807.32 0.43	Shear base max. (t) Resistence SDOF (t) Deformation . SDOF mm Raport of work hardening	771.19 496.93 12 0.000
Raportαu/α1 Coef of equivalent damping .(%)	17847.662 29	Behavior factor Ductility	3.758 4.233
LIFE SAFE LIMIT STATE			
D E M A N D		C A P A C I T Y	
Deformation mm S.L. Life safety limit state	39.693 NON VERIFIED	Deformation mm Number of previous passes	23.768 104
PgaLV/g Raport q*=Fe/Fy >3 Time of intervention (years) ------------------------	0.1364 3.00 13	ZetaE=PgaLV/Pga 10% Asta3D Nro TrCLV (years) (TrCLV/TDLV)^a	0.578 125 0.577

Fig.64 the capacity curves for the limited state of the not collapse SLV (Node 9)

Table 40

GENERAL RESULTS OF PUSHOVER ANALYSIS			
PUSH-OVER N.ro	9 -	Distrib.Forces Fx(+) Prop.Mode	
Angle of seismic force (Grd) Number of passes Resist.Max.	90 132	Numerber of total collapse Numerber of significant passes	30 180
Mass SDOF (t) Coef. of participation Rigidity SDOF (t/m) Period SDOF (sec)	2070.34 1.28 52970.61 0.40	Shear base max. (t) Resistence SDOF (t) Deformation . SDOF mm Raport of work hardening	944.74 637.20 12 0.000
Raportαu/α1 Coef of equivalent damping .(%)	15717.595 29	Behavior factor Ductility	3.441 4.136
LIFE SAFE LIMIT STATE			
D E M A N D		C A P A C I T Y	
Deformation mm S.L. Life safety limit state	34.649 NON VERIFIED	Deformation mm Number of previous passes	28.532 124
PgaLV/g Raport q*=Fe/Fy >3 Time of intervention (years) ------------------------	0.192 2.46 30	ZetaE=PgaLV/Pga 10% Asta3D Nro TrCLV (years) (TrCLV/TDLV)^a	0.819 282 0.807

Fig.65 the capacity curves for the limited state of the not collapse SLV (Node 11)

Table 41

GENERAL RESULTS OF PUSHOVER ANALYSIS			
PUSH-OVER N.ro	11 -	Distrib.Forces Fx(+) Prop.Mode	
Angle of seismic force (Grd) Number of passes Resist.Max.	90 111	Numerber of total collapse Numerber of significant passes	30 160
Mass SDOF (t) Coef. of participation Rigidity SDOF (t/m) Period SDOF (sec)	1963.52 1.32 44964.36 0.42	Shear base max. (t) Resistence SDOF (t) Deformation . SDOF mm Raport of work hardening	772.97 505.95 11 0.000
Raportαu/α1 Coef of equivalent damping .(%)	17805.498 28	Behavior factor Ductility	3.320 3.821
LIFE SAFE LIMIT STATE			
D E M A N D		C A P A C I T Y	
Deformation mm S.L. Life safety limit state	37.810 NON VERIFIED	Deformation mm Number of previous passes	23.202 106
PgaLV/g Raport q*=Fe/Fy >3 Time of intervention (years) ------------------------	0.140 2.94 14	ZetaE=PgaLV/Pga 10% Asta3D Nro TrCLV (years) (TrCLV/TDLV)^a	0.595 133 0.592

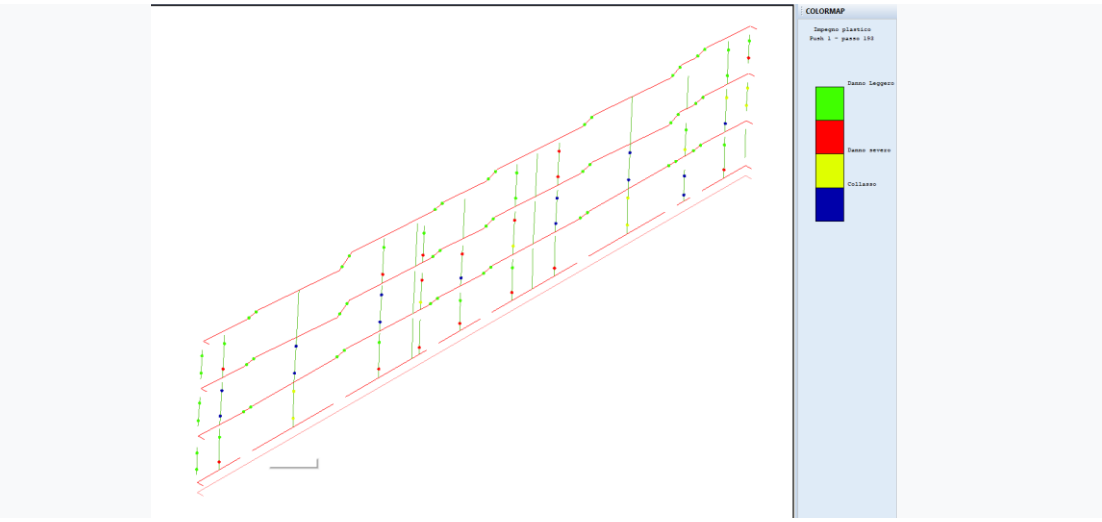


Fig.66 The deformation of push over the X-axis (in blue there is the formation of the plastic hinges , the collapse)

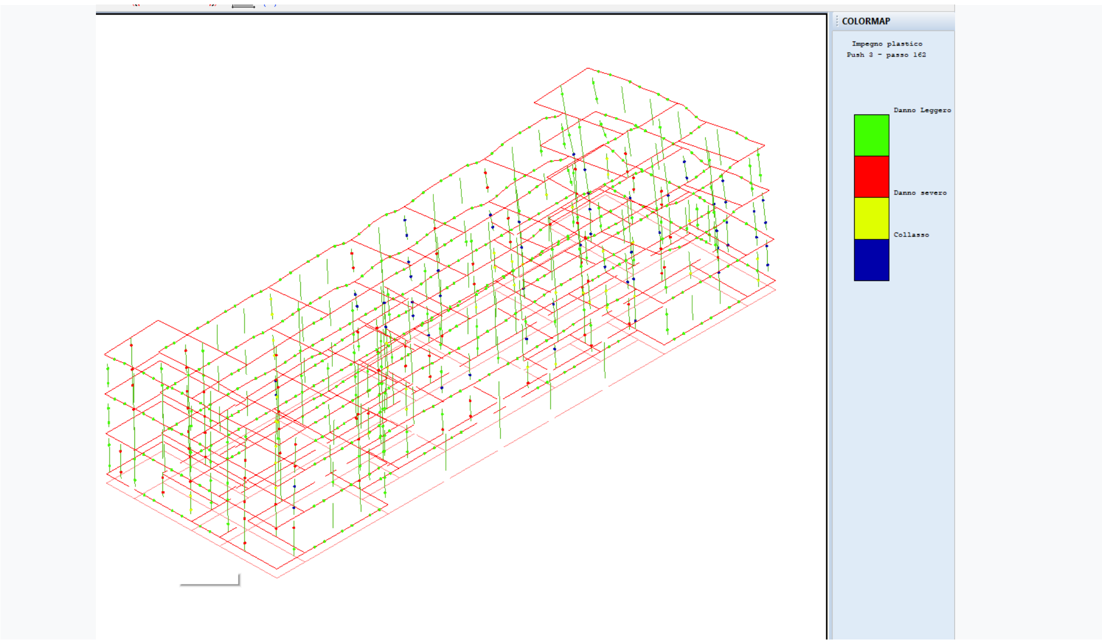


Fig.67 The deformation of push over the 3-D structure (in blue there is the formation of the plastic hinges , the collapse)

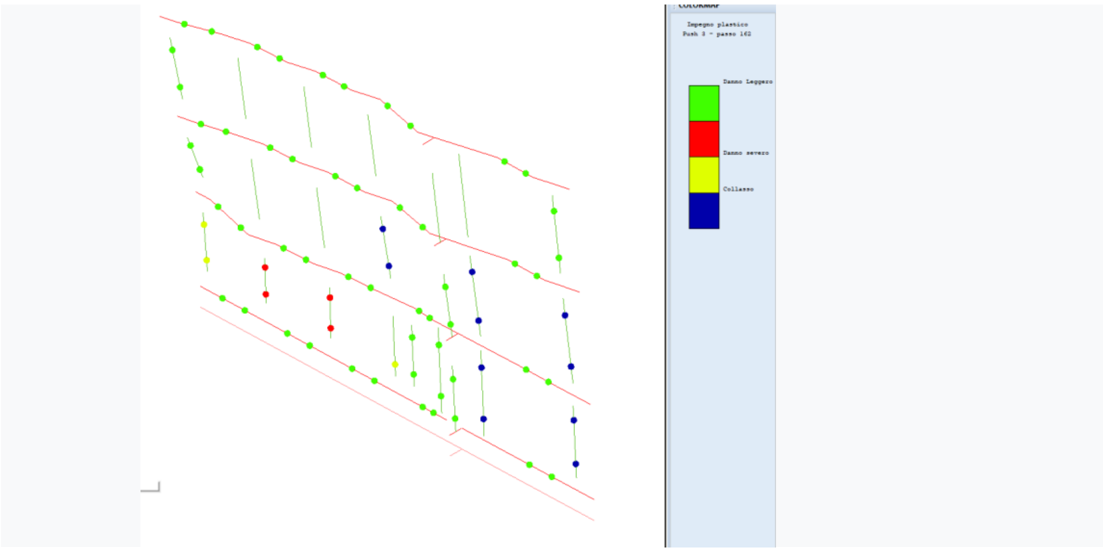


Fig.68 The deformation of push over , Y direction (in blue the creation of plastic hinges , the collaps)

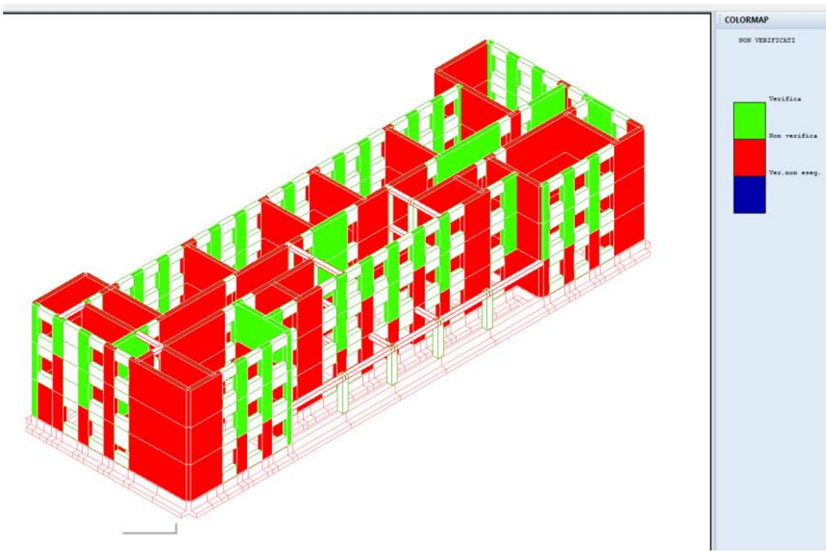


Fig 69. Non- verified = not passing at least one of controlled (eccentric compressions in plan and out of plan , diagonal shearing and horizontal)

The pushover analysis does not result positive for the factic state. Observing the curves it results that the structure has a plastic behavior with a considerable difference between the two directions. According to the non plastic behavior, coming from the fact that the structure is not in conditions to answer in terms of capacity,maximal allowable displacement is lower in value than the seismic demand. The difference highlighted according to the directions X and Y come from the fact that the direction X has only two wall slats and between has some small slats that are near the case of stairs, but in the direction of Y the wall slats are more complete, most frequent or near each other, in a short way in the Y direction has bigger presence of the walls than compared according to X.

After doing all this work the school was decided to be demolished because of having structural damages(more than 60% of this buildind had damages), and it was in the ultimate limit state (near collapse).

CHAPTER 5

CONCLUSIONS

In Albania, the masonry building stock was created using old building codes that contemplate a lower seismic demand than EC-8. In the current research, the seismic response of masonry structures damaged by the 2019 earthquakes in Albania has been examined. In order to achieve this goal, a three-story masonry building with a typical template design project that was used by the Albanian government's Ministry of Public Works and Settlement in a number of regions of the nation was examined. This investigation was done in order to contribute to studies on the evaluation and strengthening of existing masonry buildings situated in high seismic regions of the nation. This project was examined using nonlinear static analysis techniques and the Eurocode 8, Part 3 earthquake performance evaluation principles. Nonlinear static analyses were used to determine the building's seismic deformation capabilities. The results demonstrate a high degree of masonry stock vulnerability and a high level of anticipated damage for this class of buildings during powerful earthquake shakings. The examinations conducted on masonry structures following the earthquake on November 26, 2019, have confirmed these findings. The comparison of post-earthquake survey work with numerical analysis revealed that it is now possible to evaluate the seismic safety levels of masonry buildings and avoid catastrophic damage by implementing the proper retrofit interventions. In particular, it has been verified that the adopted approach, which the authors had previously suggested as an analytical method capable of estimating the actual response of masonry structures to seismic actions, is reliable.

In this research, CDS Win software was used to model the building, which employs a similar frame macro-model methodology. To assess the building's capability, nonlinear pushover analysis was used. The analysis's findings were compared to actual damage sustained on November 26, 2019, and there is a strong correlation between the predicted damage based on performance-based evaluation and the estimated magnitude of the local earthquake.

The following additional significant findings are provided regarding the seismic behavior of the masonry building:

- According to tests conducted at the Epoca facility, the materials used in this form of template project design were concrete C 20/25, clay brick f=5-8 N/mm2, stone 2.6-4.3 N/mm2, and steel with Rs=2100 kg.cm2.

- Due to the examined building's inadequate lateral load capacities and stiffness under the envisioned earthquake, its primary flaw is its high desplacement demands.

- It is noteworthy that the earthquake had an intensity comparable to that required by the Albanian Building Code, and the structure displayed damage at a level comparable to that anticipated based on the life safety limit state, as required by the same regulations.

-It was determined to demolish the building because it performed poorly on November 26, 2019, and more than 60% of the building had damage.This helps to understand how similar structures behave in other areas when there is a seismic impact.

In order to save the lives of those who live and work in masonry buildings in our nation and to enhance their job, it is crucial to understand the seismic behavior of those structures. Given that they are historical structures(load bearing wall buildings are some of the oldest building in Albania), they require thorough study.This study give a highlight on masonry structures , bur further studies need to follow.

REFERENCES

[1] A.W.Henrdry.B.PSihnaand S.R Davies, "Design of masonry structures ", United Kindom, : Departament of civil engineering ,2004 .

[2] A.Dogangun,A.Uraland R.Livaoglu,"Seismic performance of masonry buildings during recent Earthquake in Turkey",Bejing China: The 14th World Conference of earthquake Engineering,October 2008

[3] W.G Curtin. G.Shaw. J.K Beek.W.A Bray,"Structural Masonry Designers's manual",Vol.3 ,2006

[4] JJ Roberts,O.Brooker ,"Introduction to Eurocode 6",Published by The concrete center, London,2013

[5] Miha Tomasevic,"Earthquake -Resistant design of Masonry buildings",Slovenian National Building and Civil Engineering Institute,London, 1999

[6] Ergys Huta , "Earthquake performance of low and mid-rise masonry buildings in Albania"Epokas' Department of Civil Engineering,December 2015

[7] L.Pasticier,C.Amadioand M.Fragiacomo,"Nonlinear seismic analysis and vulnerability evaluation of a masonry building by means of SAP 2000 V.10 code,Italy, November 2007,published in Wiley InterScience.

[8] Christopher Potter, "Seismic Analysis of a typical Masonry Building from Barcelona'sExa,ple District",Polythechnic University of Ctalunya , July 2011

[9] Miha Tomasevuc "Seismic Behavior of plain and reinforced masonry buildings",Journal of structural engineering, February ,1994

[10] Otion Korini, "Seismic safety assessment of Albanian Masonryresidential buildings using nonlinearprocedures",Epoka's Department of Civil Engineering, June, 2012

[11] U.Hancilar,E.Cakti,M.Erdik,"Earthquake performance assessment and rehabilitation of two historical unreiforced masonry buildings",published on Speringer Science+Business Media B.V.2011

[12]H.Bilgin, N.Shkodrani,M. Hysenlliu, H.Ozmen,E.Isik,E.Harirchian,"Damage and performance evaluation of masonry buildings contructed in 1970s during the 2019 Albania earthquakes"pushblished in Engineering Failure Analysis, 2021

[13]F.Freddi,V.Novelli,R.Gentile,E.Veliu,S.Andreev,A.Andonov,F.Greco,E.Zhuleku ,"Observations from the 26 th November 2019 Albania Earthquake :the earthquake engineering field investigation team(EEFIT) mission ",published in the Bulletin of Earthquake Engineering(2021)19:2013-2044 , 4 March 2021

[14]H.Crowley,V.Despotaki,V.Silva,J.Dabbeek,X.Romao, N.Pereira,J.M.Castro,J.Danieli,E.Veliu,H.Bilgin,Ch.Adam,M.Deyanova, N.Ademovic,J.Atalic,E.Riga,A.Karatzetzou,B.Bessason,V.Shendova, A/Tiganescu,D.Toma-Danila,Z.Zugic, S.Akkar,U.Hancilar ,"Model of seismic design lateral force levels for the existing reinforced concrete European building stock",published in Bulletin of Earthquake engineering, 08April,2021.

[15] H. Bilgin , M.Hysenlliu ,"Comparison of near and far fault ground motion effects on low and mid-rise masonry buildings",Department of Civil Engineering Epoka University, Tirana , Albania,6 February 2020, published in Journal of Bilding Engineering.

[16]N.Shkodrani, H.Bilgin,"Seismic performance of existing low-rise URM buildings considering the addition of new stories", Department of Civil Engineering, Epoka University , Tirana , Albania, August 2021 published in Structural Engineering and Mechanics.

[17]M.Hysenlliu, A.Bidaj,H.Bilgin,"Influence of material properties on the seismic response of masonry buildings",Tirana , Albania, 15April 2020, published in "Research on Egineering structures and materials"

[18]N.Shkodrani, H.Bilgin, M.Hysenlliu,"Influence of interventions on the seismic performance of URM buildings designed according to pre-modern codes",Tirana ,Albania,May 2021, published in "Research on Engineering Structures and materials"

[19] E.Isik, E.Harirchian, H.Bilgin, K.Jadhav,"The effect of material strength and discontinuity in RC structures according to different site-specific design spectra.",Tirana , Albania, published in "Research on Engineering Structures and materials"

[20]A.Marku Pdh thesis:"Sustainability, Low-cost strategies for renovation of residential building stock of the 70s-80s in Albania", Department of Civil Enginnering,PolisUniversity,Tirana,Albania, https://issuu.com/polisuniversity/docs/phd_thesis_aguljeln_marku/72

[21] "Eurocode 6:Design of masonry structures, Part 3 Simplified calculation methods and simple rules for masonry" ,Natianal Application Document, 1996-3:2001

[22] A.Elghazouli,"Seismic design of buildings to Eurocode 8",second edition ,Imperial college London, 2016

[23] "Eurocode 8:Design of structures for earthquake resistance-Part 3 :Assessment and retrofitting of buildings", European Standart, June 2005.

[24]"Eurocode 8:Design of structures for earthquake resistance-Part 1 :General rules, seismic actions and rules for buildings", British standarts BS EN 1998-1:2004

[25]M.Hysenlliu, H.Bilgin,"Seismic performance assessment of a typified school building damaged during the 2019 Albanian earthquakes",Tirana , Albania 2020 published in IOP Conference Series:Materials Science and Engineering.

[26] EN 13791 "Assessment of in-situ compressive strength in structures and precast, concrete components", European standard, January 2007

[27]EN ISO-6891-1 "Metallic materials-Testing-Part1:Method of test at room temperature" International standard" Third edition , November 2019

[28]"Method of test for masonry units, Part 1:Determination of compressive strength" ,British Standars BS EN 771-1:2000, 2000