Structural and Seismic Behavior of Typical Masonry Buildings from Bosnia and Herzegovina

Naida Ademovic

Master’s Thesis

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DECLARATION

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Year: 2011

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God thank you for giving me the strength and the wisdom.

MSc. Naida Ademovic

I will use the same comparison as I did in my Master of Science in Computational Engineering in Bochum, Germany. This year has proved once again the quotation that I have used. Repetition of a wise thing is never too much.

“A teacher came to class with an empty glass mason jar and began filling it with large rocks. When the rocks reached the top of the jar he asked the class if the jar was full. A student answered yes. The teacher then added gravel to the jar and asked the same question, “Is the jar full?” Another student, wising up to the situation said, “No.” Smiling from the correct answer, the teacher added sand to the jar, and upon finishing asked the question, “Is this jar now full?” The class responded with another no. The teacher then brought out a pitcher of water and filled the jar. He then asked if the jar was full. Not knowing what else could be placed in the jar, the class could only answer with silence. The teacher interrupted the silence by saying that the jar was indeed full. Then the teacher asked his class what the moral of the demonstration was. An eager student quickly responded that the demonstration shows that you can always find more room for responsibilities. Shaking his head no, the teacher explained that in life you have many different size responsibilities and that you must first put in the big rocks, or they will never fit in the jar.”

This work is dedicated to the “big rocks” of my life, to my devoted parents, Ademovic Nihad, BSc. Civ. Eng. whose footsteps I followed and who is always giving me engineering advices and leading me the way and Ademovic-Filipovic Meliha, attorney, who is always teaching me diplomacy and the way that that matter could be done from the lawyer point of view. My brother, Ademovic Kenan, Master in International Law and Master in European Studies, an attorney, working with my mom, who has always, even in the most difficult situations, made me laugh. Thank you for always being there for me. They have always been my greatest support and the bright star lighting my way to success.
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ABSTRACT

Throughout history earthquakes have shown their strength, devastation and unpredictability. It has been seen that masonry structures are rather vulnerable to seismic actions. Recent earthquakes once again have warned us about the necessity of assessing existing residential masonry structures built without any seismic guidelines.

In order to conduct a good assessment of existing structures, recommendations given by ICOMOS should be followed. The knowledge about the structure, its geometry, characteristics of materials, history is of the utmost importance. This combined with the adequate numerical tools either FEM of macro-elements leads us to quite good indications regarding the behavior of the structure to earthquake actions.

This thesis elaborated a behavior of a typical masonry building in Bosnia and Herzegovina built in the 60's without any seismic guidelines. A brief state of the art is given regarding the seismicity of Bosnia and Herzegovina and its surroundings, indicating several major earthquakes in the region. Data regarding experimental tests is further elaborated with a clear indication about the necessary additional experimental in-situ and lab test to be conducted. The thesis proceeds with the numerical modeling using non-linear time integration in two software programs, namely DIANA and 3MURI. Comparison of the crack pattern (damage pattern) as well as capacity curves is done and good agreement between the methods has been found. It is recognized that a simpler software 3MURI could be used for assessment of this type of building, however if a more detailed analysis is required DIANA or some other FEM should be utilized. It the engineering practice it is necessary to find a compromise between accuracy and simplicity. As well behavior of the structure is compared with experiments done by Tomaževič in Slovenia. The thesis finished with the recommended strengthening methods, and possible future works.
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RESUMO

Ao longo da história, os sismos têm mostrado a sua força, poder de devastação e imprevisibilidade. Tem-se observado que as estruturas de alvenaria são bastante vulneráveis a acções sísmicas. Os sismos recentes têm mostrado mais uma vez a necessidade de avaliar as estruturas de alvenaria residenciais construídas sem quaisquer orientações sísmicas.

A fim de realizar uma adequada avaliação de estruturas existentes, deve-se seguir as recomendações providenciadas pelo ICOMOS. O conhecimento da estrutura, a sua geometria, características de materiais e a sua história revestem-se da maior importância. Estes aspectos, combinados com ferramentas numéricas adequadas, FEM ou macro-elementos, conduzem a indicações muito boas a respeito do comportamento estrutural às acções sísmicas.

Nesta tese aborda-se o comportamento de um edifício de alvenaria típico da Bósnia e Herzegovina, construído na década de 1960, sem quaisquer orientações sísmicas. Apresenta-se o estado da arte sobre a sismicidade da Bósnia e Herzegovina e regiões adjacentes, indicando-se os grandes terramotos ocorridos na região. A informação sobre ensaios experimentais é aprofundada, com uma indicação sobre os testes necessários adicionais a desenvolver. A tese prossegue com a modelação numérica não-linear usando dois programas de cálculo, DIANA e 3MURI. A comparação do padrão de dano bem como das curvas de capacidade mostra uma boa concordância entre as duas ferramentas numéricas. É reconhecido que o software 3MURI, mais simples, poderá ser usado para a avaliação sísmica deste tipo de edifícios. No entanto, se for necessário uma análise mais detalhada, devem ser utilizados programas como DIANA ou outros similares.

O comportamento numérico da estrutura é ainda comparado com resultados experimentais desenvolvidos Tomaževič na Eslovénia. A tese termina com uma abordagem aos métodos de reforço mais adequados e possíveis trabalhos futuros a desenvolver.
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SAŽETAK

Kroz historiju zemljotresi su pokazali svoju jačinu, razornu moć i neizvjesnost. Mnogobrojni primjeri su pokazali da su zidane konstrukcije dosta "osjetljive" na djelovanje seizmičkih aktivnosti. Zamljotresi koji su se desili u skorijoj prošlosti još jednom su nas upozorili o neophodnosti procjene postojećih zidanih stambenih objekata koji su izgrađeni bez primjene ikakvih smjernica vezanih za aseizmičko građenje.

Kako bi se izvršila dobra procjena postojećih konstrukcija, preporuke koje su date od strane ICOMOS-a bi se trebale primjenjivati. Znanje o konstrukciji, njena geometrija, karakteristike materijala, historija su od izuzetnog značaja. Ovo u kombinaciji sa adekvatnim numeričkim alatima, bilo da se radi o FEM-u (metodi konačnih elemenata) ili metodi koja se zasniva na makro-elementima dovodi nas do dobrih naznaka vezanih za ponašanje konstrukcije uslijed djelovanja zemljotresa.

Ova teza elaborira ponašanje tipične zidane stambene zgrade u Bosni i Hercegovini koja je izgrađena u 60-tim godinama bez upotrebe ikakvih seizmičkih smjernica. Kratki uvod je dat vezano da seizmičnost Bosne i Hercegovine i njenog okruženja, te su dati primjeri nekoliko značajnih zemljotresa koji su pogodili ovo područje. Nakon toga, proračunate su karakteristike materijala na osnovu eksperimentalnih podataka sa jasnom naznakom vezanom za neophodnost dodatnih in-situ eksperimentalnih i laboratorijskih ispitivanja koji bi se trebali napraviti. U nastavku slijedi numeričko modeliranje primjenom nelinearnih vremenskih integracija, koristeći dva software-ska programa, DIANA i 3MURI. Izvršena je usporedba propagacije pukotina (rasprostiranje oštećenja) kao i krivih kapaciteta i pri tome je uočena dobra suglasnost. Uočeno je da bi se jednostavniji software 3MURI mogao koristiti za procjenu ovog tipa konstrukcije, no ukoliko se zahtijeva detaljniji proračun onda je neophodno da se koristi jedan od programa koji su zasnovani na konačnim elementima, bilo da je to DIANA ili neki drugi. U inženjskoj praksi neophodno je da se nađe kompromis između tačnosti i jednostavnosti. Također, ponašanje konstrukcije upoređeno je sa eksperimentima koje je izvršio Tomažević u Sloveniji. Na koncu, date su moguće metode za ojačanje konstrukcije kao i preporuke za daljni rad.
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Erasmus Mundus Programme

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Chapter 1

INTRODUCTION

1.1 Motivation

Former Yugoslavia experienced great destruction after the II World War leading to expansion in the use of brick as the basic material for construction of the new facilities. This was justified by a rapid and efficient construction on one hand, and on the other hand by cheap and durable material. Apartment buildings up to 6 stories high were constructed with load-bearing walls in the transverse direction only (Tomaževič, 1999), and without any rules related to seismic regulations in the ex-Yugoslavian region. The catastrophic earthquakes that occurred in Skopje, Macedonia (1963), then in Banja Luka, Bosnia and Herzegovina (herein after B&H) (1968), followed by an earthquake in Petrovac, Monte Negro (1979), indicated the deficiencies and errors in design and construction of those structures.

Several buildings suffered minor damage after the 31, March 2009 earthquake in Sarajevo. The magnitude was of M=3.8 on the Richter’s scale. This opened a question on the safety and vulnerability of buildings made of brick masonry build before the adoption of the technical standard for construction of buildings in seismic areas (Pravilnik, 1981). The devastating earthquake that struck this region emphasized the need for verification of typical brick-masonry buildings. A typical brick masonry building was chosen for verification in this thesis, designed in the year of 1957. Utilizing experimental tests and combining them with the Finite Element Method (FEM), calculations based on a non-linear approach provides with the behavior of the building under seismic action. On the basis of
this data, decisions regarding strengthening methods can be obtained. The importance of experimental testing is more than evident as this represents an input for the analytic calculations done using FEM Program (DIANA) and macro-element program 3MURI.

1.2 Objectives of the Dissertation

The main objective of this thesis is to study the seismic resistance of a typical masonry building (hereinafter referred as the case study), in B&H, this one located in Sarajevo. A destruction of such a building was evident in the 1963 Skopje earthquake. These building were designed without taking to account seismic actions and their seismic performance (although poor) is unknown. Structural and dynamic analysis was done by two software package (DIANA and 3MURI). The objective was to quantify the seismic vulnerability of the structure. On the basis of the calculations elaboration of damage pattern can be investigated, the most damage and vulnerable parts of the structure can be identified. This will lead to a better understanding of the seismic performance and real vulnerability of these structures.

1.3 Outline of the Dissertation

The dissertation is organized in 8 (eight) chapters as follows:

- **Chapter 1** provides a brief introduction to the dissertation topics and defines the motivation and importance of the thesis, along with an outline of the dissertation.

- **Chapter 2** begins with general data regarding earthquakes. Further provides a stated of the art regarding the seismic context of the Balkan region and B&H. Brief data is given regarding the seismicity of B&H and its surroundings, followed by the major earthquakes that struck this area. Then some detail is given regarding the three major earthquakes that struck this region. The chapter closes with a brief history of seismic codes and finishes up with the present seismic zonations of B&H.

- **Chapter 3** explores the methodology that is to be employed during the structural assessment of existing buildings. Brief data is provided about both the knowledge and analysis phases, and in such a way improving the knowledge and understanding the behavior of the structure. This procedure given by ICOMOS in 2001 is followed in the analysis of the typical residential structure in B&H referred to as case study. It starts with the first part of the assessment methodology, the historical investigation and first section is closed by the laboratory tests conducted by the Institute for Materials and Structures, Faculty of Civil Engineering in Sarajevo (hereinafter IMK). The obtained data received by these very scarce tests is used as input for the numerical model. As experimental in-situ and lab test have been lacking for this case study the chapter closes with
several remarks regarding some tests that should be done in order to get a more realistic view of the structure in question.

- **Chapter 4** starts with general information about numerical methods and proceeds with very brief information regarding the methods utilized in this thesis. It continues with the definition of the parameters necessary for creation of the model. The structure is modeled in DIANA software. Firstly the modal analysis is performed in order to grasp the behavior of the structure and then the pushover is conducted in "+Y" and "-Y" directions. Elaboration of the data is presented in both directions, following by discussion, conclusion and some comparisons. The chapter continues with the brief introduction to time history analysis and the parameters that were used in this calculation. The building was exposed to the Petrovac scaled earthquake having the ground acceleration of $a_g = 0.1g$ corresponding to the requirement for the location of Sarajevo. Results of the analysis are presented and discussed. The chapter finished with the conclusions obtained from these analysis.

- **Chapter 5** is devoted to calculation done by the 3MURI software. It has been decided to make calculations with a more simple software (3MURI), compare the results and indicate if simpler software programs could be used for analysis of this structure having an acceptable degree of accuracy. Firstly the modal analysis is done in order to verify the frequencies and modes with DIANA calculations. The chapter proceeds with the nonlinear static pushover analysis where in this case the entire structure was analyzed in order to prove the hypothesis regarding the weak "X"-direction. Then, nonlinear pushover was done in "±X", "±Y" utilizing the "uniform" as well as "first mode" force distribution. The chapter closes with conclusions regarding the capacity curve and damage pattern.

- **Chapter 6** is dedicated to the comparison of the results obtained by different methods and utilizing different software packages. On the basis of the comparison conclusion is obtained that for this type of structure simpler model can be used for the analysis, however if detailed damage propagation is required software based on FEM should be utilized.

- **Chapter 7** suggests possible strengthening methods for upgrading the structure with the aim of having a more resistant structure to seismic actions. Methods that were proposed refer to the global strengthening of the structure due to weak "X" direction and to local strengthening due to shear in-plane damage. This is not elaborated further as it is outside of the scope of this thesis.

- **Chapter 8** presents the main conclusions of the thesis and proposes future issues to be addressed.
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Chapter 2
SEISMIC CONTEXT

2.1 Introduction

Besides floods, volcano eruptions, and fires, earthquakes are one of the natural processes that can be defined as frightening and disastrous phenomena. Return period is taken as 475 years which is much shorter in respect to the average life of man. Remembrance on the past earthquakes is usually something that remains in written notes (where ones have been retrieved), but the collective memory of seismic events disappears after 2 to 3 generations. Interest in the earthquake engineering increased in the late 19th and 20th centuries, in the UK, Japan, USA, Italy and other countries around the world as major earthquakes struck these regions.

2.2 Causes of Earthquakes

The most scientifically accepted explanation of the occurrence of earthquakes is connected to the model of plate tectonics. It describes the large scale motions of Earth's lithosphere. The lithosphere is broken up into what are called "tectonic plates"; 6 continental-size plates and 14 sub continental-size plates, as shown in Figure 2.1. These plates move in relation to one another, and during their movement they slide next to the other or one plate goes under the other plate (subduction).
The occurrence of earthquakes is mostly, but not always, connected to a location where plates meet, called plate boundary. Earthquakes occur along breaks within the earth's crust known as faults. Most earthquakes occur along pre-existing faults, but a new fault can be created during an earthquake. However, when the plates have been locked for a long period of time, a large amount of energy has been accumulated and stores, so a sudden release would cause breaking of the plates and an enormous amount of energy would be released. Release of energy is manifested by body and surface waves. Energy is usually released within 10 to 15 seconds (even though in some situations periods of 30 to 60 seconds have been registered). Up to now the largest release of energy was 1x1025 ergs (earthquake in Chile) which happened in 1960.

2.3 Earthquake Historic Data in the Balkans

It is interesting to note that the first written documents about earthquake occurrence are connected to China and 1177 B.C. The first data regarding earthquakes in Europe goes back to the 16th Century, even though earthquakes as a phenomena have been mentioned in 580 B.C. Detailed and expanded investigations on earthquakes start only in the XVII century (Polyakov, 1985).

In the Balkans the first written historical data dates back to 6th Century B.C. The first instrumental earthquake registration in the Balkans was done in 1882. In average, annually this region is struck by an earthquake of a 6.3 magnitude by Richter scale. In the last century eighty destructive earthquakes struck this region. This data cannot leave anyone indifferent.

However, historical data regarding earthquakes in the Balkan's far past is rather poor. Knowledge is limited due to the lack or instrumental recordings and as well most of the devastating earthquakes happened outside of Europe. Around 80% of the seismic energy is released in the Pacific Ring, while only 15% in the Secondary Ring (Alpine Belt) as indicated in Figure 2.2, which starts in Mediterranean region and in the eastern direction extends over Asia. The accuracy of the data depends as well upon the net of the installed seismological devices.
2.4 Seismicity of Balkans

Tectonic activities that cause the occurrence of earthquakes cannot be understood without a previous knowledge and making certain correlations between the geological structure of the earth, geodynamics and geodetic data. In order to evaluate the seismic hazard of a given site or a region it is necessary to identify all possible sources of seismic activity. The identification of possible seismic sources is connected to geological and tectonic evidence, historical seismicity and instrumental seismicity.

2.4.1 Tectonic Activity of the Region

In the newsletter of Gisdevelopment (Gisdevelopment, 2008) it is stated: "The Balkan peninsula, Figure 2.3, has one on the most complex tectonics in Europe".

When talking about the Balkans and B&H it has to be kept in mind that it is not affected only by the movement of the "big" tectonic plates, Euroasian and African plate, but as well by the movement of the Arabian and micro Adriatic plate, as indicated in Figure 2.4. The micro Adriatic plate is moving northeastwards and in the way sub-ducts under the Eurasian plate. A new study has discovered an existence of a fault (200km) in the Adriatic Sea, northwest of Dubrovnik. Richard Bennett from the University of Arizona, in Tucson, believes that due to the movement of the Italian peninsula towards the Republic of Croatia in the value of 4 mm/year, that new islands will be formed.
in the Adriatic Sea and in the 50 to 70 million years total disappearance of the Adriatic Sea will occur. And he states: “All indications are there that this fault will cause an occurrence of a very large earthquakes, but additional investigations are necessary in order for this to be confirmed (Joshi, 2008). This all indicates that B&H cannot be looked at locally, but a global view is required.

However, there is no unique accepted hypothesis regarding the creation and kinematic of the Adriatic micro plate as well as regarding the time of its creation. Different hypothesis are given by (Nocquet, et al., 2001), (Mele, 2001), (McClusky, et al., 2000) and (Pribićević, et al., 2002). This is just one segment of the complexity of the region affecting the occurrence of earthquakes in this region.

![Adriatic Micro Plate](https://example.com/adiatric-micro-plate.png)

**Figure 2.4 - Adriatic Micro Plate (Oldow, et al., 2002)**

### 2.4.2 Geological Evidence and the Fault Types of the Region

On this relatively small area different types of faults are evident. Existence of a normal fault is characteristic for the Apennine peninsula; Adriatic coast and Dinarides are characterized by reverse faults. The most important earthquakes that happened in this region with the fault solution are shown in Figure 2.5. Intensity of the seismic activity for the past 33 years of the region is shown in Figure 2.6.

![Fault solutions for major earthquakes on the Adria micro plate](https://example.com/fault-solutions.png)

**Figure 2.5 - Fault solutions for major earthquakes on the Adria micro plate (black cycles representing earthquake of magnitude larger than 6, and gray of magnitude larger than 5.5) (Slejko, et al., 1999)**
This region has been affected by numerous earthquakes. Already from 1905 a magnitude of M=6.6 by Richter's scale, and the intensity X at the epicenter, struck Albania. During this destructive earthquake total devastation of the north-west part of Albania and south-east part of Montenegro was observed. Further, there are earthquakes that struck Zagreb in 1905, Republic of Croatia, and later B&H in 1923 and 1942. The most significant earthquakes that hit this region are: Skopje in 1963, Macedonia; Kotor (Petrovac) in 1979, Montenegro and Banja Luka in 1969, B&H.

2.5 Seismicity of Bosnia and Herzegovina (B&H)

Seismic activity in B&H is connected to the existence of deep lateral and reverse faults. The fact that the second biggest belt (Alpine Belt), going from the Himalayas over Iran, Turkey and Greece, passes through B&H verifies the tectonic activity of this region.

2.5.1 Geomorphologic Texture, Geology and Tectonic Activity

B&H is one of the active seismic zones in the Balkans which makes a part of the Trans-Mediterranean-Asian Seismic belt (Figure 2.2). The most frequent earthquakes are present in the area of Outer Dinarides indicated in Figure 2.7 with no. 1, while the strongest, but seldom, are manifested within the Sarajevo Thrust in the south and southwest of B&H representing the Central Dinarides (no.2), and the Inner Dinarides denoted as no.3.
Until present, the most complete picture of the tectonic structure in B&H was done by (Papeš, 1988) who has identified deep faults passing through B&H as well as 30 tectonic units. This is shown in Figure 2.8.

The longest is the Sarajevo Fault spreading in the direction NW-SW in the length of 300km, meaning across the entire B&H. The second longest is the Banja Luka Fault, and the third Konjic Fault (Papeš, 1988). All transversal faults are under-passing the Sarajevo Fault. High seismic activity is evident along the transversal deep faults, while low to moderate seismicity along the Sarajevo Fault is noted. According to Papeš (Papeš, 1988), Sarajevo and Gradiška Faults may experience series earthquakes of magnitude M6 by Richter's scale or even higher. After the earthquake that struck Haiti, January 12, 2011 (Glavatović, 2010), the director of the Seismological Institute of Montenegro stated that after a long silence period of 15 to 20 years, intensified seismic activity in the Balkans is to be expected.

On the basis of activities that happened in the last 100 years, B&H was divided into five seismic zones and fifty-seven potential seismic structures. According to the tectonic characteristics of the ground and earthquake occurrence, connection and mutual dependency has been detected. It has been concluded (Papeš, 1988) and (Hrvatović, 2006) that the major seismic activity is located on the border belts of the geotechnical units (direction NW-SE), then in the direction of the longitudinal dislocations (direction NW-SE) and finally at the crossing of the transversal faults (directions NE-SW and N-S) as seen in Figure 2.9.
In B&H three significant epicentral regions can be detected; Region of North Bosnia (Banja Luka, Tuzla, Derventa and Skelani); Central and Outer Region of the Dinarides Mountains. On the basis of earthquake data (historical and instrumental recordings from 1991 to 2004 the following data was obtained (Hrvatović, 2006) and presented in Table 2.1.

Table 2.1 - Number of earthquake vs. Magnitude by Richter scale (1991 to 2004)

<table>
<thead>
<tr>
<th>Number of earthquakes</th>
<th>Magnitude by Richter scale</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>&gt; 6</td>
</tr>
<tr>
<td>10</td>
<td>5.6-6.0</td>
</tr>
<tr>
<td>14</td>
<td>5.1-5.6</td>
</tr>
<tr>
<td>78</td>
<td>4.6-5.0</td>
</tr>
<tr>
<td>162</td>
<td>4.1-4.5</td>
</tr>
<tr>
<td>406</td>
<td>3.6-4.0</td>
</tr>
<tr>
<td>412</td>
<td>3.1-3.5</td>
</tr>
</tbody>
</table>

It is interesting to state that for 82% of all those earthquakes the focal depth is only 10km; focal depth in the range from 11 to 20km (12%); around 5% the depth is at 21 to 30 km; and only 1% the focal depth is greater than 30km. The destructive strength of these earthquakes is characterized by the shallow focus as well. Seismicity of B&H as per 1964 to date is shown in Figure 2.10.
As per Euro Mediterranean Seismic Hazard Map, as indicated in Figure 2.11, B&H falls in the Moderate Seismic Hazard having the PGA in the range of 0.08 to 0.24g, while a south-west part of the country experiences a High Hazard (PGA>0.24g).

Figure 2.11 - European-Mediterranean Seismic Hazard Map (European-Mediterranean-Seismic-Hazard-Map)

It is important to emphasize there are no sufficient data regarding the geodynamic soil characteristics of B&H. Investigations including GPS (Global Positioning System) started only in 1998, giving the main directions of compression and tension movement as indicated in Figure 2.12. In B&H there is just one permanent GPS station, and that being in Sarajevo. Of course this is insufficient to determine the movements in this region. In 2000 a GPS network has been created with 23 points (Mulić, et al., 2006) in B&H; however the data has not been processed yet. It is important to emphasize the importance of this data due to the fact that it will give more information about the movement of the plates and prediction of the possible earthquakes in this region (Yüksel, et al., 2006). Currently only three seismological stations (Sarajevo, Banja Luka and Mostar) and located in B&H out of 262 stations located in southeastern Europe (Mulić, et al., 2006).

Figure 2.12 - Main directions of dilatations and three deformation zones (Yüksel, et al., 2006)
2.6 Major Earthquake in the Past of ex-Yugoslavia and B&H

According to the seismological data, annually 1100 earthquakes of intensity lower than III by Mercalli-Cancani-Sieberg (here in after MCS) were registered, while in the last 104 years 1084 earthquakes of the Richter's magnitude greater than 3 were registered as well. In Table 2.2 some of the most destructive earthquakes in the region and on the territory of B&H are listed.

By analyzing the data presented in Table 2.2 it can be seen that B&H was hit by several devastating earthquakes with the highest intensity and magnitude located in Ljubinje, Treskavica (mountain near Sarajevo) and Banja Luka. The strongest earthquake that struck Sarajevo region (30km to the south) was in 1962 at the location of the biggest fault line near the city of Sarajevo. The Region of Banja Luka represents one of the most seismic active areas in B&H. Seismological investigations reveal that earthquakes in Banja Luka occur in series. Until now there have been four series being in the years of 1888, 1935, 1969 and 1981 (Trukulja, 1999).

Table 2.2 - Most destructive earthquakes in Bosnia and Herzegovina

<table>
<thead>
<tr>
<th>Date</th>
<th>Place</th>
<th>Magnitude by Richter's Scale (M)</th>
<th>Intensity in the epicenter (Io) by MCS</th>
</tr>
</thead>
<tbody>
<tr>
<td>07.04.1905.</td>
<td>Pertovac</td>
<td>5.0</td>
<td>VII</td>
</tr>
<tr>
<td>01.08.1907.</td>
<td>Poćitelj</td>
<td>5.7</td>
<td>VII-VIII</td>
</tr>
<tr>
<td>25.12.1908.</td>
<td>Vlasenica</td>
<td>5.3</td>
<td>VI-VII</td>
</tr>
<tr>
<td>12.03.1916.</td>
<td>Bihać</td>
<td>5.0</td>
<td>VII</td>
</tr>
<tr>
<td>06.02.1923.</td>
<td>Jajce</td>
<td>5.0</td>
<td>VII</td>
</tr>
<tr>
<td>1923</td>
<td>Tihanjina</td>
<td>6.5</td>
<td>No data</td>
</tr>
<tr>
<td>14.02.1927.</td>
<td>Ljubinje</td>
<td>6.0</td>
<td>VIII</td>
</tr>
<tr>
<td>17.12.1940.</td>
<td>Derventa</td>
<td>5.1</td>
<td>VII</td>
</tr>
<tr>
<td>31.12.1950.</td>
<td>Drugovići (Prijedor)</td>
<td>5.7</td>
<td>VIII</td>
</tr>
<tr>
<td>11.06.1962.</td>
<td>Treskavica</td>
<td>6.0</td>
<td>VIII</td>
</tr>
<tr>
<td>26.07.1963.</td>
<td>Skopje - Vitina (Macedonia)</td>
<td>6.1 (depth 5km)</td>
<td>IX</td>
</tr>
<tr>
<td>07.03.1967.</td>
<td>Srebrenica</td>
<td>5.1</td>
<td>VII</td>
</tr>
<tr>
<td>27.10.1969.</td>
<td>Banja Luka</td>
<td>6.6</td>
<td>IX</td>
</tr>
<tr>
<td>25.08.1970.</td>
<td>Gacko</td>
<td>5.0</td>
<td>VII</td>
</tr>
<tr>
<td>29.10.1974.</td>
<td>Lukavac</td>
<td>5.0</td>
<td>VII</td>
</tr>
<tr>
<td>15.04.1979.</td>
<td>Kotor-Bar-Petrovac (Montenegro)</td>
<td>6.9 (depth 11km)</td>
<td>IX/X</td>
</tr>
<tr>
<td>23.05.2004.</td>
<td>Grude</td>
<td>5.5</td>
<td>VI-VII</td>
</tr>
<tr>
<td>31.03.2009.</td>
<td>Region Sarajevo</td>
<td>4.2 (depth 2 km)</td>
<td>VI</td>
</tr>
<tr>
<td>21.06.2009.</td>
<td>Posušje, western Herzegovina</td>
<td>4.6 (depth 10km)</td>
<td>VI</td>
</tr>
<tr>
<td>18.03.2010.</td>
<td>55 km from Sarajevo</td>
<td>3.9 (depth 2 km)</td>
<td>V</td>
</tr>
</tbody>
</table>

In the following sub-sections some general data will be given about the three highlighted earthquakes in a chronological order as these had the largest impact on the region of ex-Yugoslavia, and B&H.
2.6.1 Skopje -Vitina Earthquake (26.07.1963)

One of the most devastating earthquakes that struck the Balkan region was the 26, July 1963 Skopje, Macedonia earthquake. The magnitude of the earthquake was 6.1 by the Richter's scale and the Intensity by MSC was of level IX. The strength of the earthquake was so high that it was felt in Greece and Bulgaria, 195km and 173km far from the epicenter respectively with IV intensity. The epicenter of the earthquake was located in the center of the city causing large devastations. The clock at the railway station due to the intensity of movement stopped working at the exact time of the quake, exactly at 05:17 am as seen in Figure 2.13.

![Figure 2.13 - Devastated railway station (Skopje-images-earthquake1963)](image)

Destroyed masonry residential buildings made of masonry, representing of the typical structures built in the 1950-1960 is shown in Figure 2.14. It is interesting to denote that this is the same type of building that will be analyzed in this thesis.

![Figure 2.14 - Destroyed residential masonry building (five stories) (Petrovski, 2003)](image)

Casualties in this earthquake rose to 1300, and 4000 people were injured. Around 80.7% of the buildings were destroyed; 200,000 people were homeless (76%); and only 19.7% of buildings could be used (Milutinović, 2007). Most of the buildings were masonry structures with load bearing walls in only one direction which caused the destruction of the building (Petrovski, 2003). Besides residential buildings, many category I objects were destroyed (hospitals, school and public institution etc.). This can be connected to the fact that at that time there were no regulations or guidelines for design of structures in seismic areas. The degree of devastation is confirmed by financial, medical, engineering facilities offered by 78 countries.
2.6.2 Banja Luka Earthquake (27.10.1969)

Enormous devastation was experienced after the earthquake that struck Banja Luka on the 27.10.1969. The magnitude by the Richter's scale was 6.6, while the intensity ranged from VII to IX by the MCS greatly depending on the soil characteristics (Stojanković, 1999). It was a shallow earthquake with a focal depth at 25km and this earthquake was proceeded by a foreshock of M=5.6 at a depth of 15km. Maximum horizontal acceleration of soil was $a_h = 0.5g$ as shown in Figure 2.15.

![Figure 2.15 - Horizontal soil acceleration (Aničić, et al., 1990)](image)

Devastation of the 27, October 1969 is shown in Figure 2.16 and Figure 2.17. Two months later the same region was struck by a 5.4 magnitude earthquake with a focal depth of 13km. Schools, cultural and heritage structures, hospitals and other objects were destroyed and damaged during this earthquake. Additionally, some soil deformations were observed near the epicenter location, landslides were activated, liquefaction was observed in small amounts, ground water regime has been alternated, new thermal mineral springs were formed, water temperature has increased for three degrees and so on. Changes that can be done by an earthquake are fascinating. Number of casualties was 15, while 1117 people were hurt severely. According to the experts, it was determined that the area of Banja Luka could experience earthquakes of a maximum 6.9 magnitude (Skoko, et al., 1975).

![Figure 2.16 - Damage caused by the earthquake and a clock which stopped at the time of earthquake (Banja-Luka-Earthquake)](image)

![Figure 2.17 - Real Gymnasium before and after the 27, October 1969 earthquake (Banja-Luka-Earthquake)](image)
2.6.1 Petrovac Earthquake (15.04.1979)

After the Friuli earthquake that struck northern part of Italy in 1979, an earthquake of 6.9 Richter’s magnitude struck Montenegro. This earthquake was accompanied by several very strong aftershocks of 6.1 magnitude by Richter’s scale. Maximum acceleration was in the North-South direction in the amount of $a_g = 0.4g$ as shown in Figure 2.18.

![Figure 2.18 - NS Component measured in Petrovac](image)

This earthquake covered a large part of ex-Yugoslavia, and the entire B&H had the intensity of V by MCS, as it can be seen on Figure 2.19.

![Figure 2.19 - Isoseismic lines and the registered accelelogram in Petrovac NS component (Montenegro-earthquake, 1979)](image)

Casualties due this earthquake rose to 101 and 100,000 people were left homeless. One of the causes of structure damage were foundation failures as with the Hotel Slavija in Budva resulting in a "sandwich" effect shown in Figure 2.20 and a characteristic failure of stone masonry structures indicated in Figure 2.22.

![Figure 2.20 - Hotel Slavija (Montenegro-earthquake, 1979)](image) ![Figure 2.21 - Stone masonry building](image)
2.7 Seismic Codes in Bosnia and Herzegovina

In ex-Yugoslavia the first temporary codes for Construction in Seismic Regions were passed only in 1964. Even though the seismic activity in this region was high, severe earthquake actions were not considered in the design of structures. Seventeen years later, in 1981, Technical Regulations for Design and Construction of Buildings in Seismic Regions was enforced (Pravilnik, 1981) with much stricter conditions in respect of the first ordinance. The catastrophic earthquakes that occurred in 1963 in Skopje (Macedonia) and in 1968 in Banja Luka (B&H) were the impetus for drafting the new rules. Rulebook for Technical Standards for Masonry Walls were made available only in 1991 (Pravilnik 1991) ten years after the adoption of the Technical Regulations for Design and Construction of Buildings in Seismic Regions, however never enforced. So, many structures were built without any seismic regulations. Currently B&H is using Eurocode 8 - Design of structures for earthquake resistance and Eurocode 6 - Design of Masonry Structures.

2.8 Current Seismic Zones in B&H

According to the current Seismological map of B&H (Figure 2.22) the county is divided into four (4) zones. Most of the country lies in the zone six (6) and seven (7), while zones eight (8) and nine (9) of the MCS cover only small parts of the country. Hazard Seismic Map of the Mediterranean Region as indicated in (Figure 2.11), which gives the Peak Ground Acceleration (PGA) with a 10% probability exceedance in 50 years (or 475 years return period). So, for the largest part of the country the Peak Ground Acceleration (PGA) is in the range of 0.03g to 0.12g (zones 6 and 7), while small part of the country has the PGA value in the range of 0.12g to 0.35g (zones 8 and 9). This is indicated in Table 2.3

<table>
<thead>
<tr>
<th>Seismic Intensity</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>PGA (m/s²)</td>
<td>0.03-0.06g</td>
<td>0.06-0.12 g</td>
<td>0.12-0.24 g</td>
<td>≥ 0.24 g</td>
</tr>
</tbody>
</table>

Table 2.3 - Correlation between PGA and Seismic Intensity
Figure 2.22 - Seismological Map of Bosnia and Herzegovina
3.1 Introduction

The importance of gathering information and increasing knowledge about existing masonry structures was clear after the Umbria and Marche earthquake of 1997, in Italy. It was evident that the lack of knowledge and inadequate intervention methods contributed to major collapses and damages of the existing buildings. These damages provided proof that knowledge about the existing masonry structures had to be increased in order for suitable interventions to be developed and implemented.

3.2 General Aspects

Proper investigation and diagnosis are the key factors for the design of a suitable intervention. The importance of investigations for structural diagnosis (including historical aspects, materials and structures) is emphasized in the Venice-Charter (1964), (ICOMOS, 2010). Investigations must take into account a variety of different aspects including: the typology of the building, the type of masonry elements, connections between elements, characteristics of materials, etc. Each structure has unique characteristics and the investigations must be planned and executed to ensure that an adequate understanding of the structure is obtained.

Of critical importance is that through investigation and diagnosis determination of the likely causes and consequences of damages to the structure could be done. Interventions are then designed to ensure the structural safety. Once the likely causes of damage have been defined, it is
necessary to interpret the damages. This is only possible with a clear idea of the structural and material characteristics and requires thorough investigations including historical and geometrical surveying, non-destructive and minor-destructive testing, damage characterization and analysis.

The purpose, scope and extent of the investigations must be clearly defined before the works proceed and must be regularly reviewed during the works. This will ensure that the relatively short period of investigations, in terms of the life of the structure, will provide the essential information to ensure interventions and maintenance works are suitable for the safety and serviceability of the structure. It is clear that when investigations are inadequate, poor interventions can result in tasks that do not ensure stability; may not have been required, or even can worsen the safety of the structure.

Characterization of an existing structure is a very complex task, which requires a specific multidisciplinary evaluation methodology for its assessment, as stated in the ICOMOS-Recommendations for the Analysis, Conservation and Structural Restoration of Architectural Heritage (ICOMOS, 2003) presented in Error! Reference source not found. and Error! Reference source not found.

With adherence to this methodology, a clear assessment of the current physical state of the structure can be obtained. With this knowledge numerical models can be developed and calibrated to accurately define the structural form and mechanisms. As seen from Error! Reference source not found., the methodology can be divided into two main phases:

1. **Knowledge phase** (historical research; description of the structure - geometry and materials; damage survey; in-situ and laboratory tests); and,

2. **Numerical Analysis phase** (definition of the type of analysis, models and tools).

The knowledge phase is characterized by several steps with the ultimate goal of accurately defining the structural system. The knowledge phase incorporates both qualitative and quantitative approaches. Qualitative approaches are in the collection of all available historical data about the structure and the structural description. Quantitative data includes measurement of the observed damage and geometrical information. It is on the basis of these details that the on-site and laboratory
testing regime can be planned. This phase essentially comprises of four steps as indicated in Table 3.1.

<table>
<thead>
<tr>
<th>Phase 1 - Knowledge phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>II</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>III</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>IV</td>
</tr>
<tr>
<td></td>
</tr>
</tbody>
</table>

In the analysis phase a modeling strategy for the numerical assessment of the structures is defined based on the previously gathered information. In this phase it is necessary to define various aspects, as indicated in Table 3.2.

<table>
<thead>
<tr>
<th>Phase 2 - Numerical phase</th>
</tr>
</thead>
<tbody>
<tr>
<td>The most effective modeling type (e.g. limit analysis, analysis with numerical models such as finite element models, etc.)</td>
</tr>
<tr>
<td>Type of analysis that may be conducted (linear or nonlinear) based on the objectives defined for the structure.</td>
</tr>
</tbody>
</table>

The most suitable analysis tool will also be related to the type and accuracy of results obtained from the investigation works and those required from the analysis. It is always necessary to review the adequacy and accuracy of the results retrieved from the knowledge phase throughout the analysis phase. In this way the phases are not to be completed in sequence, but should be completed concurrently. The analysis phase should be initiated when there is sufficient information to obtain initial estimates of the structural mechanics. It is with the results of the knowledge phase that complex analysis tools such as Finite Element Analyses are calibrated. In addition to the knowledge and analysis phases described, monitoring of the structure is now also understood as a key activity to fully appreciate and review the behavior of the structure.

The use of this phased multidisciplinary procedure is essential for an in-depth understanding of structures. This kind of analysis allows performing of knowledge based structural analysis and thus defines with more confidence the strengthening interventions. This is also important as it can prevent
the execution of intrusive repair works. Its application gains special interest when these structures are located in areas with high seismic risk (Casarin, 2006).

3.3 Historical Investigations of this Case Study

As stated in Section 3.2, the first step is gathering historical information about the structure. The building analyzed in the thesis represents a typical masonry structure built between the 1950's and 1970's in the entire ex-Yugoslavia, an example of the such a building destroyed during Skopje earthquake is presented in Section 2.6.1. These buildings are mainly located in the urban regions of the cities as isolated buildings or several of them are attached together making a block of buildings. This type of a building is a characteristic example of a large building stock of residential unreinforced masonry buildings that do not satisfy the latest code provisions leading to the necessity of strengthening methods.

The building that will be analyzed is shown in Figure 3.3, located in Sarajevo, in the part of the city called Grbavica. It is a five stories (basement + ground floor + 5 storeys) building with a basement underneath the entire structure. The load bearing system is made of masonry walls, while the floors are made of semi-assembly slabs of the construction type "Herbst "(pre-stressed reinforced-concrete joists with hollow concrete block) and a flat roof. The roof is of the same construction as the floor, proving continuity of the construction, and later on a possibility for additional storeys to be added. This has been the case on several buildings of this type; however this is out of the scope of this thesis. The external walls have an additional façade layer made of non-bearing hollow masonry bricks.

![Figure 3.3 - Considered Building, built in the year of 1957](image)

3.4 Description of the Building

Following the historical survey, the next step in the investigation is the visual inspection of the building. The visual inspection includes three phases, the geometry survey, the material survey, and the damage survey.
Original design was obtained from the authorities. It was noted that there were no structural changes on the building as to the original design. Verification of the geometric data was done with laser distancemeters and total stations, on the basis of which AutoCAD drawings were performed as shown in Figure 3.4, and Figure 3.5. Some AutoCAD drawings are given in the Appendix 1.

![Figure 3.4 - East façade](image)

![Figure 3.5 - Plan of the floor and adopted axes system](image)

The structure is composed of load bearing walls only in the transverse direction (y direction) with slabs made of semi-prefabricated elements. The longitudinal walls are not considered as load-resisting elements as they have been made weak by many openings as shown in Figure 3.4. In the plan the structure is of dimensions 38.0m x 13.0m with 7 levels (basement, ground floor and five stories). The transversal bearing walls are composed of brick masonry walls of 0.25m thick and a non-bearing façade wall made of hollow bricks of 0.12m thick, while the inner bearing walls are made of solid brick walls of 0.25m thick. Bricks are of standard dimensions 25x12x6.5cm, connected by cement mortar. The slabs are made out of semi-prefabricated elements "Herbst" concrete hollow elements as shown in Figure 3.6. Construction of these blocks was regulated by Yugoslavian
standards B.D1.030-1965 (Peulić, 2002). The basement concrete inner perpendicular walls have a 0.38m thick, while the outer (longitudinal) walls are 0.30m thick, and two inner walls are 25cm thick. As the span is larger than 3.0m it was foreseen, as per above mentioned standards at that time, to construct a transversal beam of 25cm width with the same height as the slab see Figure 3.6. Large openings of a regular sequence are found on both west and east façades of the building, as indicated in Figure 3.4. Visual inspection revealed that there were no major damages on the structure.

3.5 Experimental In-situ Tests

In order to obtain data regarding the mechanical and physical properties of material, it is important to conduct experimental test. Due to the lack of instrumentations only tests for determination of the compressive test on brick units and compressive tests for concrete walls were done by the IMK. The results of the tests are given in the following sub-sections.

3.5.1 Compressive Strength of Brick Units

Experimental tests were conducted by the IMK in order to obtain the compressive strength of the solid bricks. Five series of two bricks were taken out from the representative locations in the structure and their compressive strength was tested in accordance with the standards for brick investigation. Locations of the conducted tests are marked by numbers from 1 to 5 and are shown in Figure 3.8.
All samples were photographed before, during and after testing, this is shown in Figure 3.9.

![Samples before testing](image1)
![Samples after preparation](image2)
![Samples during testing](image3)
![Samples after testing](image4)

**Figure 3.9 - (a) Samples before testing  (b) Samples after preparation  (c) Samples during testing  (d) Samples after testing**

Calculation of the compressive strength of bricks was conducted on the basis of Yugoslavian Standards (JUSB.D1.011, 1995). It was determined that the compressive strength of bricks corresponds to the class M150 (new M15) and fulfills the requirement for load-bearing walls. Individual data is given in Table 3.3

<table>
<thead>
<tr>
<th>DETERMINATION OF THE SOLID BRICK CLASS (JUSB.D1.011)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. DIMENSIONS OF THE BRICK 25x12x6,5 cm</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>No.</th>
<th>Structural elements</th>
<th>Mark of the sample</th>
<th>Sample length (cm)</th>
<th>Sample width (cm)</th>
<th>Fracture force (kN)</th>
<th>Compression strength f (N/cm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>WALLS OF THE BUILDING</td>
<td>1</td>
<td>23,50</td>
<td>11,50</td>
<td>73500</td>
<td>271,97</td>
</tr>
<tr>
<td>2</td>
<td></td>
<td>2</td>
<td>23,50</td>
<td>11,70</td>
<td>63000</td>
<td>229,13</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>3</td>
<td>24,40</td>
<td>12,20</td>
<td>40000</td>
<td>134,37</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>4</td>
<td>24,00</td>
<td>12,10</td>
<td>53000</td>
<td>182,51</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>5</td>
<td>24,80</td>
<td>12,00</td>
<td>47300</td>
<td>151,37</td>
</tr>
<tr>
<td></td>
<td><strong>average</strong></td>
<td></td>
<td></td>
<td></td>
<td><strong>193,87</strong></td>
<td></td>
</tr>
</tbody>
</table>

| fmin | 134,37 | >120   |
| fsr  | 193,87 | >150   |

**SOLID BRICK CLASS**

M 150

### 3.5.1 Compressive Strength of Concrete

In order to determine the mechanical characteristics of the concrete, IMK has tested cylindrical concrete samples of 100mm diameter. Coring and testing of the six samples was done in
accordance with the regulations defined in standard (U.M1.048, 1987) and the results are presented in Table 3.4.

Table 3.4 - Compressive strength of concrete walls

<table>
<thead>
<tr>
<th>No</th>
<th>Structural element</th>
<th>Mark of sample</th>
<th>Sample diameter (mm)</th>
<th>Sample height (mm)</th>
<th>Sample weight (g)</th>
<th>Sample density (kg/m³)</th>
<th>Fracture force (N) on a cylinder (D:mm)</th>
<th>Reduction to a cube a=200 mm (N/mm²)</th>
<th>Compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Basement Wall</td>
<td>W1</td>
<td>94.4</td>
<td>102.3</td>
<td>1671.57</td>
<td>2336</td>
<td>133000</td>
<td>19.00</td>
<td>19.36</td>
</tr>
<tr>
<td>2</td>
<td>Basement Wall</td>
<td>W2</td>
<td>94.5</td>
<td>100.5</td>
<td>1490.75</td>
<td>2400</td>
<td>146000</td>
<td>21.10</td>
<td>21.52</td>
</tr>
<tr>
<td>3</td>
<td>Basement Wall</td>
<td>W3</td>
<td>94.3</td>
<td>102.1</td>
<td>1800.73</td>
<td>2677</td>
<td>147000</td>
<td>21.04</td>
<td>21.47</td>
</tr>
<tr>
<td>4</td>
<td>Basement Wall</td>
<td>W4</td>
<td>94.4</td>
<td>102.2</td>
<td>1711.32</td>
<td>2394</td>
<td>149000</td>
<td>21.29</td>
<td>21.71</td>
</tr>
<tr>
<td>5</td>
<td>Basement Wall</td>
<td>W5</td>
<td>94.4</td>
<td>101.1</td>
<td>1666.51</td>
<td>2342</td>
<td>135000</td>
<td>19.29</td>
<td>19.67</td>
</tr>
<tr>
<td>6</td>
<td>Basement Wall</td>
<td>W6</td>
<td>94.3</td>
<td>102.2</td>
<td>1727.37</td>
<td>2425</td>
<td>166000</td>
<td>23.62</td>
<td>24.16</td>
</tr>
</tbody>
</table>

On the basis of this data it was concluded that the basement walls are made out of concrete grade MB25 which is equivalent with C20/25 in Eurocode 2, and a reinforcement ø=14mm, type of steel GA240/360. As no other data was provided, it was decided to take the values from the EC2 and proceed with its mean values in the numerical calculations.

3.6 Analytical Formulations

In the absence of adequate experimental tests, analytical formulations can be used for determination of characteristics of materials. This is presented below and this data was used as input in the numerical modeling. However, due to the importance of in-situ and laboratory testing for obtaining precise data few sections are devoted to this subject.

3.6.1 Masonry Walls

3.6.1.1 Compressive Strength of Masonry

In the case when no experimental data is available, the characteristic compressive strength of masonry made with general purpose mortar, can be calculated as per Eurocode 6 (CEN, 2005) giving the dependency by following equation:

\[ f_k = K \times f_D^{0.70} \times f_m^{0.30} \]
It can be seen that it depends upon the normalized compressive strength of masonry units $f_b$ and compressive strength of mortar $f_m$, while K depends on masonry type and type of mortar with values varying from 0.4-0.6

### 3.6.1.2 Modulus of Elasticity

One can refer to the Young's modulus of elasticity only when there is linear elastic material behavior. In the nonlinear $\sigma$-$\varepsilon$ diagram a question rises regarding determination of this value. As for a large strength variation of different masonry units, it was decided to define the modulus of elasticity for the value of 33% (Drysdale, et al., 1999) or 35% (Rots, 1997) of the fracture load.

If there are no experimental data, Eurocode 6 (CEN, 2005) recommends to take the modulus equal to $E = 1000f_k$ [MPa]; Pauley (Paulay, et al., 1997) recommends a value of $E = 750f_k$ [MPa]; Binda (Binda, et al., 2007) recommends $E = 900 N/mm^2$ for rural poor buildings, for civil building and palaces $E = 900 - 1500 N/mm^2$; and Tomažević (Tomažević, 1999) proposes $200f_k \leq E \leq 2000f_k$.

### 3.6.1.3 Possible Non-linear Properties

Experiments on the uni-axial post-peak behavior of compressed bricks do not exist, so recommendations regarding the compressive fracture energy $G_f$ cannot be given. Tensile strength is usually taken as 10% of compressive strength (Sorić, 2004). Values of tensile strength are in the range of 1.5-3.5N/mm². For the fracture energy $G_f$ of solid clay, Pluijm (Pluijm, 1992) found values in the range from 0.06 – 0.13 Nmm/mm² and for the tensile strength values $f_t$ in the range from 1.5 – 3.5 N/mm². As per Lourenço (Lourenço, 2004) and based upon Model Code90 (CEB-FIP, 1993) for concrete, the fracture energy can be calculated by the expression:

$$G_f = 0.025 (2f_t)^{0.7} \frac{N}{mm}$$

assuming the ratio between tensile and compressive strength as 5%. The ductility index defined as $d = \frac{\varepsilon_f}{f_t}$ for the brick has a recommended value of 0.029mm (Lourenço, 2010). The determination of the compressive fracture energy is as well based on the Model Code90 (CEB-FIP, 1993) for a peak strain of 0.2%, as shown in Figure 3.10.
Applicability of this curve is in the range from 12 to 80 N/mm² and it is calculated from the equation:

\[ G_{fc} = 15 + 0.43f_c - 0.0036f_c^2 \left( \frac{N}{mm} \right) \]

Whereas for \( f_c < 12 \text{ N/mm}^2 \), a value of \( d=1.6\text{mm} \) and for \( f_c > 80 \text{ N/mm}^2 \), a value of \( d=0.33\text{mm} \) is recommended (Lourenço, 2010).

### 3.6.2 Calculation of the Material Characteristics for this Case Study

#### 3.6.2.1 Masonry Walls

As per Eurocode 6, compressive strength of masonry made out of solid bricks units with longitudinal mortar joints, taking into account the \( \delta \)-coefficient (shape coefficient), the compressive strength is obtained by the following formula:

\[ f_k = 0.45 \times 15.71^{0.70} \times 2.5^{0.30} = 4.07 \text{N/mm}^2 \]

Tensile strength is taken as 5% of the compressive strength as per Lourenço (Lourenço, et al., 2005). For determination of the compressive fracture energy the recommendations given by Lourenço (Lourenço, et al., 2010) are used as stated in Sub-title 3.6.1.3, respectfully giving the fracture energy \( G_c = 1.6 \times 4.07 = 6.51 \text{N/mm} \). Several values were taken for tensile fracture energy for masonry, as this is a very sensitive parameter leading to convergence problems, at the end finally a value of \( G_t = 0.10 \text{ N/mm} \) was adopted. Modulus of elasticity is taken as proposed by Eurocode 6 (CEN, 2005) and Paulay (Paulay, et al., 1997) see 3.6.1.2. Finally, the data that is used as the input in the calculations is given in Table 3.5. On the basis of experimental test the value of density of this type of masonry is 1900 kg/m³ (Sorić, 2004), however in order to take into account the non-bearing façade walls in respect of the mass the value has been proportionally increased, while keeping the thickness of \( d=25\text{cm} \) enabling the stiffness to remain intact (shown in Table 3.5 with *).
### Table 3.5 - Masonry Data used as Input for Modeling

<table>
<thead>
<tr>
<th>Element</th>
<th>Compressive strength $f_k$ [N/mm²]</th>
<th>Compressive fracture energy $G_{fs}$ [N/mm]</th>
<th>Tensile strength $f_t$ [N/mm²]</th>
<th>Tensile fracture energy $G_{ft}$ [N/mm]</th>
<th>Shear strength $E$ [N/mm²] as per EC 6</th>
<th>E [N/mm²] as per Pauley</th>
<th>Poisson ratio $\nu$</th>
<th>Density $\rho$ [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Façade Masonry walls (25+12.5cm) thick</td>
<td>4.07</td>
<td>6.51</td>
<td>0.20</td>
<td>0.10</td>
<td>1.02</td>
<td>4070</td>
<td>0.20</td>
<td>2700*</td>
</tr>
<tr>
<td>Façade Masonry walls (25+12.5cm) (second model)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner masonry walls 25cm thick</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Inner masonry walls 25cm thick (second model)</td>
<td>4.07</td>
<td>6.51</td>
<td>0.20</td>
<td>0.10</td>
<td>1.02</td>
<td>4070</td>
<td>0.20</td>
<td>1900</td>
</tr>
</tbody>
</table>

When compared to mechanical properties of existing buildings, provided by Tomaževič (Tomaževič, 1999), and indicated in Table 3.6, the values give a good correspondence.

### Table 3.6 - Mechanical Properties of Existing Masonry (Tomaževič, 1999)

<table>
<thead>
<tr>
<th>Element</th>
<th>Compressive strength $f_k$ [N/mm²]</th>
<th>Tensile strength $f_t$ [N/mm²]</th>
<th>Modulus of elasticity $E$ [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Brick masonry</td>
<td>1.5-10</td>
<td>0.10-0.70</td>
<td>1500-3800</td>
</tr>
</tbody>
</table>

3.6.2.2 **Floors and Roof of the Structure**

The slabs are made out of prefabricated (joist and block with topping) elements, as shown in Figure 3.6. The reinforced topping is of 60mm, made out of a grade C20/24 and a reinforcement ø=8mm, type of steel GA240/360. As there was no experimental data, mean values were taken from the Eurocode 2.

### Table 3.7 - Slab Data used as Input for Modeling

<table>
<thead>
<tr>
<th>Element</th>
<th>Mean compressive strength $f_{cm}$ [N/mm²]</th>
<th>Mean tensile strength $f_{ctm}$ [N/mm²]</th>
<th>E [N/mm²]</th>
<th>Poisson ratio $\nu$</th>
<th>Density $\rho$ [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floors 26.5cm thick</td>
<td>24</td>
<td>2.2</td>
<td>27000</td>
<td>0.20</td>
<td>2190</td>
</tr>
<tr>
<td>Roof 43.5cm thick</td>
<td>24</td>
<td>2.2</td>
<td>27000</td>
<td>0.20</td>
<td>2050</td>
</tr>
</tbody>
</table>

3.6.2.3 **Concrete Basement Walls**

As per experimental test it was determined that the equivalent class for MB25 is C20/25, so mean values from Eurocode2 are taken in the calculation. The valued taken for calculation are shown in Table 3.8.

### Table 3.8 - Concrete Walls Data used as Input for Modeling

<table>
<thead>
<tr>
<th>Element</th>
<th>Mean compressive strength $f_{cm}$ [N/mm²]</th>
<th>Mean tensile strength $f_{ctm}$ [N/mm²]</th>
<th>E [N/mm²]</th>
<th>Poisson ratio $\nu$</th>
<th>Density $\rho$ [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete walls 38, 30 and 25 cm thick</td>
<td>24</td>
<td>2.2</td>
<td>30000</td>
<td>0.20</td>
<td>2400</td>
</tr>
</tbody>
</table>
It is foreseen that the roof and floors act as stiff diaphragms in the process of distribution of lateral loads to the walls, which is to be proven during the calculation. In this way, the shear forces are distributed to the walls according to their stiffness.

### 3.6.3 Calculation of the Loads for this Case Study

As the non-bearing walls are distributed equally throughout the structure it was decided to distribute this load equally over the floors in order to simplify the model. The total load was determined on the basis of Eurocode0 (CEN, 2001) and the inertial effects are to be calculated taking into account the masses associated with the gravity loads appearing in the following combinations of actions (CEN, 2003):

\[
\sum G_{k,ij} + \sum \psi_{E,i} Q_{k,i}
\]

where \( G_{k,ij} \) are permanent actions; \( Q_{k,i} \) represents variable actions; \( \psi_{E,i} \) is the combination for variable action, and it takes into account the likelihood of the loads \( Q_{k,i} \) not being present over the entire structure during the earthquake. The combination factor is calculated by the formula:

\[
\psi_{E,i} = \varphi \psi_{2i}
\]

the value for \( \varphi \) was taken to be equal to 0.5 representing independently occupied storeyes, as per Table 4.2 of Eurocode 8 (CEN, 2003). From Table A1.1-Recommended values of \( \psi \) factors for buildings from Eurocode0 (CEN, 2001) for Category A; domestic, residential areas \( \psi_{2,i} = 0.30 \), and as per Eurocode 1 (CEN, 1995) from Table 6.2-Imposed loads on floors, balconies and stairs in buildings the value for Category A, floors the value amounts to \( q_k = 2.0 \frac{kN}{m^2} \).

### 3.6.4 Further Experimental Data

Unfortunately, due to lack of finance very scarce experimental data is available. As it was seen after the completion of the historical and visual survey a general idea about the structure is obtained. On the basis of this data the choice of appropriate Non Destructive and Minor Destructive Tests (NDT/MDT) is made. It is important here to emphasize that these tests are the basis for obtaining indispensable information (compressive strength, modulus of elasticity) necessary to be used in the phase 2 of the assessment methodology.

In the future, in order to obtain more data about this case study and in order to calibrate the model several tests should be done. Here only some of them will me mentioned. For example, Single Flat jack Test should be conducted for determination of the state of stress. In order to determine the deformability characteristics (tangent and secant modulus of elasticity) of the masonry a Double Flat jack Test can be used. As the structure is made with regular masonry brick no major problems should arise during these tests. These tests identify the characteristics of materials of the structure locally, while dynamic identification is the only technique that can experimentally measure the global behavior of the structure. Dynamic identification tests are used for characterization of the modal response of
the building measuring the dynamic response of a structure under a vibration excitation. These characteristics can then be related to physical and mechanical characteristics of the building as the mass, stiffness, and energy dissipation, as well as boundary conditions. Results obtained by these tests would enable to get more realistic information about the structure and conduct the calibration of the model.

In the case that strengthening of the structure is to be done, then tests after the strengthening should be done as well, which will enable the comparison of the tests results before and after the strengthening, leading to the information regarding the effectiveness of the strengthening technique.

3.6.5 Conclusion

This available experimental data was used as an input data for the numerical modeling by Finite Element Method (DIANA Software) and "Equivalent frame" Macro-model (3MURI Software) in the following chapters. As it has been seen, one of the major drawbacks is the unavailability of enough experimental data. This has led to analytical calculations of the material characteristic which is in a sense "guessing" of the parameters, caused by lack of equipment. The importance of the experimental laboratory and on-site testing in order to increase the knowledge about the structure cannot be emphasized enough. This represents an essential input data for calibration of numerical models and for decisions regarding retrofitting. It is evident that the constitutive law of masonry, indicating a nonlinear behavior, can be obtained only by conducting deformation controlled tests, experiments in large-scale masonry tests, small masonry samples and masonry components (Lourenço, 1998). Knowledge of numerical modeling has to go hand in hand with the thorough material description, leading to accurate and reliable models (Lourenço, 1998). Further sections will be devoted to in-situ and laboratory experiments that have been proven as indispensible in the analysis of existing masonry structures.
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Chapter 4

SEISMIC ANALYSIS OF THE CASE STUDY USING DIANA

4.1 Introduction

Following numerical modeling and simulation of masonry structures, interpretation of results can be understood after a detailed description of material characteristics and cross-checking with a large amount of experimental data. As it has been seen from the previous chapter this regards investigations of "big" elements of a structure (walls), wallets, and its components (brick, mortar), as well as the entire structure. When conducting the experimental testing of the entire structure it is important that the prototype and the model have as much as similar dynamic behavior (distribution of mass and stiffness), and similarities regarding the failure mechanism which requires equivalency in the stress state. The characteristics of "softening" and "dilatancy" have a major importance in the non-linear process (Lourenço, 2010). Non-linear methods enable to conduct analysis and make control of ultimate limit state (ULS) and serviceability limit state (SLS).

What is of the utmost importance is to check the safety of the existing structure due to possible earthquake actions. Reliable numerical models are necessary in order to estimate the state of the existing structure and its eventual strengthening. Introduction of sophisticated models is a major step towards understanding of structure behavior, from linear state, through the crack formation, and propagation, through degradation to eventual collapse.
Depending on the structural characteristics of the building, linear or non-linear analysis can be used. In the simple linear analysis proposed by the codes, the structure is subjected to the seismic force determined from the elastic spectra, or by the design spectra. In order to account for the non-linear behavior of material, the forces are reduced by the behavior factor "q", while displacement is kept. However, the reduction of the force proposed by the codes is approximate; global reduction factor "q" cannot represent the change of force and deformation distribution once the structure enters into an inelastic range, mechanism leading to collapse cannot be represented. In order to overcome these shortcomings, inelastic analyses were utilized, either static or dynamic. The difference between the two being that in the static the variable is the value of the displacement or force, while in dynamic it is time. The model codes for seismic analysis of buildings as Eurocode 8 (CEN, 2003), Italian Codes (NT08, et al., 2008) and American Codes (FEMA-356, 2000) propose four basic methods for calculations as shown in Table 4.1.

<table>
<thead>
<tr>
<th>Action Analysis</th>
<th>Static</th>
<th>Dynamic</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>Equivalent Force Method</td>
<td>Response Spectrum Method</td>
</tr>
<tr>
<td>Non-linear</td>
<td>Pushover</td>
<td>Non-Linear Dynamic</td>
</tr>
</tbody>
</table>

### 4.2 Modeling - Global Analysis of Masonry Structures

#### 4.2.1 Introduction

Global analysis is mainly used for calculation and analysis of structures. In practice mainly linear method and non-linear static methods are used, whereas the non-linear dynamic analysis is mainly used for research purposes or for very important buildings (e.g. nuclear power plants).

At the beginning of the 80’s a linear mathematical model for the seismic in-plane behavior of brick masonry walls was developed. This method was based on replacement of heterogeneous brick masonry walls with homogeneous material and orthotropic symmetry (Meng, et al., 1984). It was observed that these linear models are valid in the linear region; mainly when the acceleration of ground is in the range from 0.05-0.10g. Clough, Mayes, Gulkan, Bazan, Meli and other researchers conducted thorough investigations regarding the behavior of masonry structures under earthquake actions. According to data given by Benedetti (Benedetti, et al., 1984) suggestion is given for the usage of a non-linear numerical model for seismic analysis of masonry buildings. It is a finite element approach, using an equivalent homogenous material based on experimental data.

#### 4.2.2 Modal Analysis

Modal Analysis, a linear analysis, is used for determination of the vibration characteristics (eigen-frequencies and mode shapes) of the structure. These characteristics are important for understanding the dynamic behavior of the structure under earthquake actions. Basis of the calculation will be explained in the subsequent sections.
4.2.3 Non-linear Pushover Method

In the last twenty years there has been a large improvement in the development and usage of non-linear methods for calculation and analysis of masonry structures, so today rather reliable non-linear pushover methods are utilized. The need for such an analysis dated back from the 70’s after the destructive 1976 Friuli earthquake. Simple equivalent static, non-linear method for seismic analysis of masonry buildings was firstly developed by Tomaževič in 1978 (Tomaževič, 1999). Types of non-linear pushover methods proposed by Eurocode 8 will be explained later in the thesis, as they will be used in the software packages DIANA and 3MURI.

4.2.4 Non-linear Dynamic Procedure (Dynamic Inelastic Time-History Analysis)

The dynamic inelastic time-history analysis represents the most sophisticated level of analysis of buildings to earthquake actions. This kind of method provides a stepwise solution in the time domain of multi-degree-of-freedom equations of motion representing a response of the modeled structure. Non-linear properties of the structure are taken into account in the time-domain analysis. As an input several ground motion records (accelerograms) are necessary, giving an accurate evaluation of the seismic response of structures. One of the drawbacks of this method is that is requires a lot of computational effort.

4.3 Description of the Numerical Model - DIANA

By the FEM an accurate modeling of the building (geometrical and material) can be obtained however the computational time is very long. On the other hand, simplification regarding geometrical and material characteristics can be done in the equivalent frame approach.

The numerical model was made in DIANA 9.4 (DIANA 9.4, 2009) utilizing the geometrical data obtained from the original design conducted in 1957, and cross checked by laser measurements on the site, with some simplifications, as shown in Figure 4.1.

![Figure 4.1 - (a) General view of the building and (b) numerical model performed in DIANA](image)

The structure was modeled by one type of curve shell elements, corresponding to the quadrilateral element CQ40S type, as indicated in Figure 4.2. This kind of element is characterized by 8 nodes and 5 degrees of freedom for each node (40 DOF per element) and was used to model the walls and the floors of the structure. Two shell hypotheses are connected to these elements: shear
deformation is included as defined by Mindli-Reissner theory and the normal stresses component perpendicular to the surface element is equal to zero (DIANA 9.4, 2009). The same type of elements was used for the slab as no information regarding the details of the slab is available, and there is a high possibility of the existence of concrete sections in the perpendicular direction of the pre-assembled elements, as indicated by standards of construction at that time. Rigidity of the floors depends not only on the type of the floor, but also upon the rigidity of the shear walls and the type of the structure. As a rough guide, thickness of 63mm concrete layer produces a rigid diaphragm (Drysdale, et al., 1999). So, rigid floors were assumed, enabling the distribution of the lateral loads to the walls in respect to their stiffness. As indicated before this is to be verified by the analysis.

![Figure 4.2 - Elements used for the FE model (CQ40S)](image)

The first analysis (modal) was done with the entire structure, as shown in the Figure 4.3 (c) and a refined mesh, consisting of 84523 nodes and 28522 CQ40S elements. The unit length of the element is 0.25 m, and the total 422615 degrees of freedom (DOF) were obtained. Different material and physical properties were taken into account and appropriate constrains were applied, as shown in Figure 4.3 respectively.

![Figure 4.3 - (a) Different material properties; (b) Different physical properties; (c) Constraints](image)
For the static non-linear analysis and dynamic time history analysis the mesh was kept the same, however due to a very large number of elements and limited time, half of the structure was calculated. This was enabled by the fact that the structure is symmetric, so half of the structure was modeled having 45443 nodes and 15759 elements. Adequate boundary conditions were used as shown in Figure 4.4 and indicated in Table 4.2.

![Figure 4.4 - Constrains for half of the model](image)

<table>
<thead>
<tr>
<th>Support</th>
<th>Translation</th>
<th>Rotations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axis of symmetry</td>
<td>X</td>
<td>Y; Z</td>
</tr>
<tr>
<td>Base of structure</td>
<td>X; Y; Z</td>
<td>X; Y; Z</td>
</tr>
</tbody>
</table>

### 4.4 Linear Models used in this Case Study

#### 4.4.1 Introduction

In order to evaluate the seismic behavior of a masonry structure, different modeling techniques can be utilized. The first approach used a detailed model based on Finite Element Method (meso-scale) conducted by DIANA (DIANA 9.4, 2009) software, where firstly a linear static analysis was done and then the modal response analyses. In the modal response analysis the modulus of elasticity was changed in order to the implication on the natural frequencies and mode shapes of the building. As there were no experimental results, this would provide information regarding the range of these values. If once experimental test are to be conducted, which is hoped to be done after the finalization of this thesis, its comparison with the experimental results would be done. This would further lead to the calibration of the model.

#### 4.4.2 Linear Elastic Analysis

In the first step a linear static analysis was performed in order to check the reactions, stresses, strains, displacements due to self weight of the structure and the overall behavior.
4.4.3 Modal Response Analysis

By Modal Response Analysis the natural frequencies and mode shapes of the building are determined. On the basis of this analysis, the contribution of each mode can be obtained. The modal response analysis is based on solving the eigenvalue problem as stated in (Chopra, 2007). By solving the equation:

$$\det[K - \omega_n^2 M] = 0$$

eigen frequencies are obtained, where $K$ is the stiffness matrix, $M$ is the mass matrix, and $\omega_n$ is the eigen frequency of each mode. Once the eigen frequencies are obtained the mode shapes are calculated by the following equation:

$$[K - \omega_n^2 M] \phi_n = 0$$

where $\phi_n$ is the eigenvector (mode shape). The mass participation $\Gamma_n$ (modal participation factor) is each mode is obtained by the formula:

$$\Gamma_n = \frac{L_n}{M_n}$$

where $M_n$ is the normalization of the mass in respect to the modes, defined as:

$$M_n = \phi_n^T M \phi_n$$

and $L_n$ is the modal earthquake-excitation factor defined by:

$$L_n = \phi_n^T M \ i$$

where $i$ is the influence vector representing the displacements of the masses resulting from static application of a unit ground displacement. Finally, the modal mass per vibration mode and direction is obtained by:

$$m_{eff,n} = \frac{L_n^2}{M_n}$$

4.4.3.1 Introduction to this Case Study

As stated previously, two models were done with different modulus of elasticity and compared. For this building 100 modes were calculated for which around 80% mass participation was obtained. For this calculation the entire structure was modeled with having an element size of 0.25m, type CQ40S, and total 84523 nodes and 28522 elements. The base was fixed at the bottom of the basement. The quality of the mesh was checked and all it passed all the tests.

4.4.3.2 Model 1 - Modulus of Elasticity of Masonry $E_1=4070 \text{ N/mm}^2$

For the first model the value of the Modulus of Elasticity was calculated as proposed by Eurocode2 and the following results have been obtained. Here, only the three (3) modes will be presented, while the calculation was done for 100 modes. The values of the first ten frequencies as
well as periods are presented in Table 4.3 and the cumulative percentage of the effective mass in directions X and Y is given in Table 4.3.

Table 4.3 - Frequencies and Period for Model 1

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency [Hz]</th>
<th>Period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.19</td>
<td>0.46</td>
</tr>
<tr>
<td>2</td>
<td>3.85</td>
<td>0.26</td>
</tr>
<tr>
<td>3</td>
<td>4.00</td>
<td>0.26</td>
</tr>
<tr>
<td>4</td>
<td>6.71</td>
<td>0.15</td>
</tr>
<tr>
<td>5</td>
<td>11.50</td>
<td>0.09</td>
</tr>
<tr>
<td>6</td>
<td>11.79</td>
<td>0.06</td>
</tr>
<tr>
<td>7</td>
<td>11.94</td>
<td>0.06</td>
</tr>
<tr>
<td>8</td>
<td>12.11</td>
<td>0.06</td>
</tr>
<tr>
<td>9</td>
<td>12.49</td>
<td>0.06</td>
</tr>
<tr>
<td>10</td>
<td>13.47</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Table 4.4 - Cumulative Percentage of the EM

<table>
<thead>
<tr>
<th>Mode</th>
<th>X</th>
<th>Y</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>67.33</td>
<td>0.60</td>
</tr>
<tr>
<td>2</td>
<td>67.39</td>
<td>0.60</td>
</tr>
<tr>
<td>3</td>
<td>67.39</td>
<td>88.79</td>
</tr>
<tr>
<td>4</td>
<td>76.21</td>
<td>88.79</td>
</tr>
<tr>
<td>5</td>
<td>78.42</td>
<td>88.79</td>
</tr>
<tr>
<td>6</td>
<td>78.42</td>
<td>88.79</td>
</tr>
<tr>
<td>7</td>
<td>79.39</td>
<td>88.79</td>
</tr>
<tr>
<td>8</td>
<td>79.45</td>
<td>88.80</td>
</tr>
<tr>
<td>9</td>
<td>79.45</td>
<td>72.93</td>
</tr>
<tr>
<td>10</td>
<td>79.46</td>
<td>72.94</td>
</tr>
</tbody>
</table>

As it can be seen in Table 4.3 only 79.46% of the effective mass is utilized in the 10 modes, in order to take into account 90% of the effective mass, as per (CEN, 2003), it is necessary to perform the analysis with a larger amount of modes. Including 100 modes the effective mass was increased to 84.46%, in the x direction, 79.97% in the y direction. It is clear that the first mode is the prevailing mode of the response with the largest effective mass contribution in the x-direction. This is a "typical" box behavior of a masonry structure with stiff connections between the walls and the slab (diaphragm effect). In Figure 4.5, Figure 4.6, and Figure 4.7 the first 3 modes are presented.
Participation factor showed that the translation in the X direction has the major influence in the 1st and 4th mode, and in the Y direction the largest influence is observed in the 3rd and 9th mode, while the rotation around the Z axis is mostly manifested in the 2nd and 8th mode.

4.4.3.3  **Model 2 - Modulus of Elasticity of Masonry $E_2=3052.5$ N/mm$^2$**

For the second model the value of the Modulus of Elasticity was calculated as proposed by Pauley (Paulay, et al., 1997). The values of the first ten frequencies and corresponding periods are presented in Table 4.5. The same sequence regarding the mode shapes, effective mass participation is observed in this model, as expected, so it will not be repeated.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency [Hz]</th>
<th>Period [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.53</td>
<td>0.52</td>
</tr>
<tr>
<td>2</td>
<td>3.41</td>
<td>0.29</td>
</tr>
<tr>
<td>3</td>
<td>3.53</td>
<td>0.28</td>
</tr>
<tr>
<td>4</td>
<td>5.93</td>
<td>0.17</td>
</tr>
<tr>
<td>5</td>
<td>10.10</td>
<td>0.10</td>
</tr>
<tr>
<td>6</td>
<td>10.29</td>
<td>0.10</td>
</tr>
<tr>
<td>7</td>
<td>10.47</td>
<td>0.10</td>
</tr>
<tr>
<td>8</td>
<td>10.62</td>
<td>0.09</td>
</tr>
<tr>
<td>9</td>
<td>10.94</td>
<td>0.09</td>
</tr>
<tr>
<td>10</td>
<td>11.78</td>
<td>0.08</td>
</tr>
</tbody>
</table>

4.4.3.4  **Comparison between the two Models**

The frequencies from the two models having different modulus of elasticity were compared in order to see the influence on the behavior of the structure due to these changes. The values are given in Table 4.6.

<table>
<thead>
<tr>
<th>Modulus of Elasticity $E$ [N/mm$^2$]</th>
<th>Mode</th>
<th>Frequency [Hz]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>4070</td>
<td>1</td>
<td>2.19</td>
<td>12.94</td>
</tr>
<tr>
<td>3025</td>
<td>2</td>
<td>3.85</td>
<td>13.10</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>4.00</td>
<td>13.40</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6.71</td>
<td>13.12</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>11.50</td>
<td>13.85</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>11.78</td>
<td>14.46</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>11.94</td>
<td>14.01</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>12.11</td>
<td>14.00</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>12.49</td>
<td>14.18</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>13.47</td>
<td>14.38</td>
</tr>
</tbody>
</table>

The difference in the first ten (10) modes, as indicated in Table 4.6, is in the range from 12.94 % to 14.38%. As well the value of the frequencies were compared with the data provided by Tomaževič (Tomaževič, 1999), indicating that for higher structures, "even up to 11-storeys the values are close to 2Hz even though buildings have been built with different materials". This is very important in order to be able to "verify" the model as there are no experimental data. Additionally, the first eigen-
frequency is within the limits of the flat part of the design response spectra as per Eurocode 8. Model 1 was adopted for further analysis, as Eurocode is used as the basis for calculations.

4.5 Non-Linear Static (Pushover) Analysis

4.5.1 Introduction

The non-linear static analysis (Pushover), a performance-based methodology, is based on an incremental increase of the horizontal force distribution on a structure and constant gravity loads, and as a result envelope of all the responses derived from the non-linear dynamic analysis represents the structural behavior. This reduces the problem to a SDOF (single degree of freedom system). This method enables to track the yielding sequences as well as the progress of the overall capacity curve of the structure. Previously elements that have to be defined are the lateral load and its distribution pattern. Once this is defined the lateral load is applied on the numerical model and the amplitude is increased in a stepwise fashion. At each step a non-linear static analysis is conducted, until the structure becomes unstable or until it reaches a specific limit that has been previously stated. Then, a capacity curve is plotted; usually it gives the dependency of base shear (vertical axis) on the displacement (horizontal axis). This curve is then combined with the demand (earthquake) curve usually represented in ADES (Acceleration-Displacement Response Spectra) for determination of the top displacement under the design earthquake - the target displacement.

Two basic things define different methods, one being the choice of the load pattern and the second the procedure utilized for simplification of the pushover curve for design purposes. Drawback of this procedure is that for masonry structures until now best pattern of loads is not yet determined. Additionally, it gives a time-independent displacement shape. This indicates that it is not able to take into account higher modes; but as for masonry structures, if it is the first, at least three modes, (Tanrikulu, et al., 1992) that should be regarded, makes this method applicable.

Advantage of this procedure is that it is able to locate the most vulnerable parts of the structure. It as well provides data that cannot be obtained by elastic analysis, being strength and ductility of the structure. In the following subsection proposal given by Eurocode 8 will be presented, as the "uniform" pattern will be utilized in DIANA and both, the "uniform" and "modal" pattern in 3MURI.

4.5.2 Proposal by EC8 for Pushover Analysis

First it is necessary to choose a load pattern. Eurocode 8 (CEN, 2003) defines two load patterns that are to be applied for seismic verification of structures.

The first load pattern that is proposed is a "uniform" pattern, based on lateral forces that are proportional to the mass distribution among the structure.

The second load pattern that is proposed is a "modal" pattern, meaning that the load pattern is assumed to be proportional to the fundamental mode shape $\phi_1$. Then the inertial force on the mass $k$ is equal to:
\[ F_k = F_b \frac{\phi_k m_k}{\sum_{j=1}^{m} \phi_j m_j} \quad \text{and} \quad F_b = S_d(T_1)m\chi \]

where \( F_b \) is increased steadily from zero until failure, \( S_d(T_1) \) is the acceleration of the design spectra and \( \chi \) is the correction factor.

On the basis of this data pushover curve can be plotted indicating the maximum displacement \( d_m \). As a second stage of the procedure it is necessary to convert the pushover curve into an equivalent SDOF system using the following relations:

\[ F^* = \frac{F_b}{\Gamma}, \quad \Gamma = \frac{\sum_{j=1}^{n} \phi_j m_j}{\sum_{j=1}^{m} \phi_j m_j} \]

where \( \Gamma \) is the transformation factor. Thirdly, capacity curve is simplified by an elastic-perfectly plastic pushover curve as indicated in Figure 4.8.

\[ d^* = s \left( d_m^* - \frac{E_m^*}{F_y^*} \right) \]

where \( E_m^* \) is the actual deformation energy up to the formation of the plastic mechanism (point A), as marked in Figure 4.8. Then the maximum load is set to be equal to \( F_y^* \) and the value of the displacement \( d_y^* \) is chosen in such a way that the areas under the actual and idealized curves are equal. The forth step consists of determination of the idealized equivalent SDOF system by using the formulation:

\[ T^* = 2\pi \sqrt{\frac{m^* d_y^*}{F_y^*}}, \quad \text{where} \quad m^* = \sum_{j=1}^{n} \phi_j m_j \]

As the next step it is necessary to determine the target displacement with period \( T^* \) of the SDOF system due to the design earthquake. Equations for determination of the target displacement for the structures in the short-period range and for the ones in medium and long-period ranges differ.

For the short-period range the corner period is defined as stated in Figure 4.9.
So, if $T^* < T_c$, and if $\frac{F_{y}}{m^*} \geq S_e(T^*)$, the response is elastic leading to $d_e^* = d_{et}^*$.

However, if $\frac{F_{y}}{m^*} < S_e(T^*)$, the response is nonlinear leading to:

$$d_e^* = \frac{d_{et}^*}{q_u} \left[ 1 + (q_u - 1) \frac{T_c}{T^*} \right] \geq d_{et}^*,$$

where $q_u = \frac{S_e(T^*)m^*}{F_{y}}$ and $m^* = \sum_{j=1}^{n} \phi_j m_j$.

Indicating that $q_u$ represents a relation between the acceleration of the structure with unlimited elastic behaviour $S_e(T^*)$ and the structure with the limited strength $\frac{F_{y}}{m^*}$.

In this way the target displacement is obtained in the SDOF, meaning it is necessary to transform it back to the MDOF system, and that is done by the transformation factor. So, the target displacement, corresponding to the control node, for MDOF system is obtained by:

$$d_e = d_e^* T^*$$

The value of the target displacement has to be checked that it satisfies the following $d_e \leq \frac{d_{m}}{1.5}$.

Finally it is necessary to check that member strength and story drift are acceptable at the value of $d_e$ (CEN, 2003).

4.5.3 Definition of the Masonry Constitutive Law and Non-linear Material Properties

Physical non-linear behavior of the masonry walls is defined through the total strain fixed crack model detailed in Diana (DIANA 9.4, 2009). In this way the cracks are fixed in the direction of the principal strain vectors being unchanged during the loading of the structure.

The complexity of the masonry model behavior is given by the anisotropic continuum model defined by Lourenço (Lourenço, 1998) and presented in Figure 4.10. In this model the internal damage due to two failure mechanisms is characterized by two internal parameters, one in tension and one in compression being related to the inelastic strain as indicated in Figure 4.10. This is used only as a reference model, and further details can be obtained in (Lourenço, et al., 1998), but due to the fact that dynamic analysis is performed it is necessary to use a more "simplified" model as shown in Figure 4.11.
For hysteretic behavior of masonry as indicated in Figure 4.11 parabolic stress-strain relation for compression, based on Hill-type yield criterion, was chosen with no lateral confinement and no lateral crack reduction, with the compressive strength $f_c = 4.07 \, \frac{N}{mm^2}$, and the corresponding compressive fracture energy $G_c = 6.51 \, \frac{N}{mm}$. If lateral confinement had been chosen, then the compressive strength of the masonry would have increased because masonry elements within the section would be strengthened by the added compression of the masonry elements around them. No lateral confinement is a more realistic assumption for masonry because it is easy for the masonry elements to separate at mortar joints and provide no added confinement strength to other elements. Lateral crack reduction refers to the reduction in strength of the material when it has cracked in tension and then been reloaded in compression. Since there are no cyclical loads in the Pushover analysis, there is no need to make the model more complex by considering lateral crack reduction.

The post-cracked shear behavior was defined by taking into account the retention factor of its linear behavior, which reduces its shear capacity according to the following equation:

$$G' = \beta \cdot G$$

where $\beta$ is the retention factor $0 < \beta \leq 1$, and $G$ is the shear modulus of the uncracked material. Tension path, based on Rankine-type yield criterion, was described by an exponential tension-softening diagram having a tensile strength of $f_t = 0.2 \, \frac{N}{mm}$, and respectively the tensile fracture energy being $G_f = 0.1 \, \frac{N}{mm}$. Finally, constant shear retention was chosen and the values for the tensile and compressive strength and fracture energy were input. The shear retention factor, $\beta$, was left at the default value of 0.01. This means that the shear strength of the material will be reduced to one percent of the original shear strength when cracks form. Crack bandwidth, $h$, was left as the default value as well. The default value is:

$$h = \sqrt{A}$$

where $A$ is the total area of the two-dimensional element.
The calculation procedures for non-linear material characteristics, are shown in section 3.6.2 and are used for the non-linear pushover analysis. It is assumed that the slabs and concrete walls in the basement stay in the linear elastic range, due to which control of the tensile stresses has been done. Due to consistency, material properties are repeated once again in Table 4.7 and Table 4.8.

Table 4.7 - Masonry Data used as Input for Modeling

<table>
<thead>
<tr>
<th>Element</th>
<th>Thickness [m]</th>
<th>Compressive strength f_cm [N/mm²]</th>
<th>Compressive fracture energy G_cm [N/mm]</th>
<th>Tensile strength f_tm [N/mm²]</th>
<th>Tensile fracture energy G_tm [N/mm]</th>
<th>E [N/mm²] as per EC6</th>
<th>Poisson ratio ν</th>
<th>Density ρ [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Façade Masonry walls</td>
<td>0.375</td>
<td>4.07</td>
<td>6.51</td>
<td>0.20</td>
<td>0.10</td>
<td>4070</td>
<td>0.20</td>
<td>2700*</td>
</tr>
<tr>
<td>Inner Masonry walls</td>
<td>0.250</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>1900</td>
</tr>
</tbody>
</table>

Table 4.8 - Concrete Data used as Input for Modeling

<table>
<thead>
<tr>
<th>Element</th>
<th>Thickness [m]</th>
<th>Mean compressive strength f_cm [N/mm²]</th>
<th>Mean tensile strength f_tm [N/mm²]</th>
<th>E [N/mm²]</th>
<th>Poisson ratio ν</th>
<th>Density ρ [kg/m³]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Floor</td>
<td>0.265</td>
<td>24</td>
<td>2.2</td>
<td>30000</td>
<td>0.20</td>
<td>2190</td>
</tr>
<tr>
<td>Roof</td>
<td>0.435</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2050</td>
</tr>
<tr>
<td>Walls</td>
<td>0.380</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>2400</td>
</tr>
</tbody>
</table>

4.5.4 Integration Scheme

Numerical integration scheme is utilized for determination of the stiffness matrix. Basically, a function is integrated over predefined specific points that are named integration points. The function values are weighted as defined in these integration points and added up in order to obtain the value of the integral. It is important to determine the minimum integration requirement permitting convergence and the requirements of the integration scheme have to be such to preserve the rate of convergence which would result if exact integration were used (Zienkiewicz, 2005). For the chosen CQ40S quadrilateral elements the in-plane Gauss integration scheme was selected with 3x3 integration points on the sides, minimum as per (Zienkiewicz, 2005), while thought the thickness in order to
capture the non-linear behavior 5 points were selected, and this is defined by the Simpson rule, as shown in Figure 4.12.

As per (DIANA 9.4, 2009) for more than 2 points in thickness direction the Simpson rule is applied which always has integration points at the upper and lower surface of the element. As the structure is large, one of the things that had to be kept in mind is the computation time, so that was an additional parameter that had to be followed for the choice of the integration points.

4.5.5 Iteration Method - Regular Newton-Raphson Method

The Regular Newton-Raphson method was chosen as the iteration method. It is necessary to do an iteration process for each increment. Meaning that by linearizing the problem, it is necessary to calculate the stiffness matrix in each point. Given the external forces \( f_{ext} \), the value of the displacement is \( d^{(1)}, d^{(2)}, d^{(3)} \ldots \) is determined in such a way that \( f_{int}(d^{(n)}) = f_{ext} \) for \( n = 1, 2, 3 \ldots \), then Regular Newton-Raphson method reads:

\[
K^{(n,t-1)} \delta d^{(n,t)} = f_{ext}^{(n)} - f_{int}^{(n,t-1)}
\]

where the residual value is defined as \( r_{n}(d^{(n)}) = f_{ext}^{(n)} - f_{int}^{(n,t-1)} \), and the incremental displacement then reads \( \delta d^{(n,t)} = \frac{r_{n}(d^{(n)})}{K^{(n,t-1)}} \), giving finally the displacement \( d^{(n,t)} = d^{(n,t-1)} + \delta d^{(n,t)} \) as shown in Figure 4.13.
The stiffness matrix is updated after each iteration; and it has a rather quick convergence to the exact solution.

The equations for the time-stepping methods reads:

\[
\begin{align*}
\ddot{u}_{i+1} &= \dot{u}_i + [(1 - \gamma)\Delta t] \ddot{u}_i + (\gamma \Delta t) \dddot{u}_{i+1} \\
u_{i+1} &= u_i + (\gamma \Delta t) \dot{u}_i + [(0.5 - \beta) (\Delta t)^2] \ddot{u}_i + [\beta (\Delta t)^2] \dddot{u}_{i+1}
\end{align*}
\]

where the time steps are taken to be equal at the same step, meaning \(\Delta t = t_{i+1} - t_i\), and the parameter \(\beta\) and \(\gamma\) are used for defining the change of acceleration over the time step, as well as stability and the accuracy of the methods. Parameter \(\gamma\) gives a linearly varying weighting between the influence of the initial and final acceleration on the change of velocity; and the factor \(\beta\) is connected to the displacement.

Applied to equations of motions for the earthquake (Chopra, 2007):

\[
M \dddot{u}_{i+1}(t) + C \dot{u}_{i+1}(t) + K u_{i+1}(t) = p_{\text{eff}}(i+1)(t)
\]

where:

- \(M\) - mass matrix of the structure
- \(C\) - damping matrix of the structure
- \(K\) - stiffness matrix of the structure
- \(\dddot{u}(t)\) - relative acceleration vector of the structure
- \(\dot{u}(t)\) - relative velocity vector of the structure
- \(u(t)\) - relative displacement vector of the structure
- \(p_{\text{eff}}(t) = -m1\dddot{u}_{\gamma(i+1)}(t)\) - effective earthquake force

and for \(t=0\) representing the initial conditions displacement and the velocity are defined as:

\[
u(0) = u_o \text{ and } \dot{u}(0) = \dddot{u}_o
\]

it then reads:

\[
[M + \gamma \Delta t C + \beta (\Delta t)^2 K] \ddot{u}_{i+1}(t) = p_{\text{eff}(i+1)}(t) - C[\dot{u}_i + [(1 - \gamma)\Delta t] \dddot{u}_i] - K[u_i + (\gamma \Delta t) \dot{u}_i + (0.5 - \beta)(\Delta t)^2 \dddot{u}_i]
\]

where:

- \(\hat{K} = [M + \gamma \Delta t C + \beta (\Delta t)^2 K]\) is the effective stiffness matrix
- \(\hat{p}_{i+1} = p_{\text{eff}(i+1)}(t) - C[\dot{u}_i + [(1 - \gamma)\Delta t] \dddot{u}_i]\) is the effective load vector

and then one obtains the system of coupled equations

\[
\hat{K} \dddot{u}_{i+1}(t) = \hat{p}_{\text{yield}(i+1)} \dddot{u}_{i+1}(t)
\]
Depending on the values of parameter $\beta$ and $\gamma$, the method can be conditionally or unconditionally stable. For the values of $\beta = 0.25$ and $\gamma = 0.50$ which corresponds to an average acceleration method it is unconditionally stable (does not depend on the size of the time steps), while for the value of $\beta = \frac{1}{6}$ the method is conditionally stable. Usually recommended value of the time step is $\Delta t = \frac{\pi}{10}$, meaning 10th of the shortest period of interest.

The Newton-Raphson method has a “quadratic” convergence rate, the error is essential squared, meaning that in each iteration the error is reduced by a factor of four, and its convergence is local (Chopra, 2007). Local means that the quadratic convergence is reached in the vicinity of the solution, what practically means in the last one or two steps. It is clear from the formula for Newton's method that it will fail in cases where the derivative is zero. Similarly, when the derivative is close to zero, the tangent line is nearly horizontal and hence may overshoot the desired solution. Problems occur when there are multiple solutions, points of inflection, zero or near zero slope, here the convergence is not guaranteed. For the Full Newton-Raphson method it is important that the starting point (vector) is close to the solution, for it to converge. If this is not the case, then a smaller step should be used. In order to control the load step, arch-length controlling method was utilized. The direction of the solution found by the Newton-Raphson method is correct but a smaller size step is needed. Model of energy convergence was adapted for this model with the tolerance of 1.0E-03.

4.5.6 Application of the load "±Y" Direction

The structure was exposed only to the horizontal acceleration in the "± Y" direction, as shown in Figure 4.14, as it would not be able to resist earthquakes where the predominant ground motion would be in the weak direction of the building. The horizontal load was applied in a stepwise fashion proportional to its mass. A control node was chosen in the line of symmetry at the roof level, node no. 44014, as shown in Figure 4.14, and two additional nodes in the same line where selected (node 44035 and 43935) in order to verify the behavior of the slab.

![Figure 4.14 - Location of the nodes; wall labeling; and direction of the horizontal force](image)
4.5.7 Presentation of the Results for Acceleration in "+Y" Direction

The capacity curves are obtained by connecting the load factor (coefficient) and the horizontal displacement by the following formulation:

\[ \alpha = \frac{\sum F_{\text{Horizontal}}}{\sum F_{\text{Vertical}}} \]

where:

- \( \sum F_{\text{Horizontal}} \) - Total sum of reactions at the base of the structure of increment horizontally (y direction)
- \( \sum F_{\text{Vertical}} \) - Total sum of reactions at the base of the structure due to vertical loads (z direction)

Firstly, movement of the nodes at the top of the structure in the same horizontal line (43935, 44035 and 44014) as shown in Figure 4.14 were investigated in order to prove the assumption of the rigid floors. The movements of these nodes were the same indicating a rigid behavior of the slabs, as seen in Figure 4.15.

Additionally, the displacement of nodes 38979 and 39057 was checked and it has seen that there is a very small rotation less than 5 degrees, which in this case can be neglected. This may be connected to the "not completely" symmetric structure.

![Capacity Curves for Nodes 44014, 44055 and 43935](image.png)

*Figure 4.15 - Capacity Curves for Nodes 44014, 44055 and 43935*

The capacity curve for this direction is shown in Figure 4.16. The nonlinear behavior of the structure starts very early and the maximum load coefficient of \( \alpha = 0.518 \) was reached.
In order to comprehend the non-linear behavior of the structure it was decided to select three load coefficients in the capacity curve (Figure 4.16) and principal tensile strains at those stages were analyzed for the load bearing and non-load bearing walls. The first stage corresponds to the load coefficient of $\alpha = 0.243$ (starting of the visible non-linear behavior), the second to $\alpha = 0.397$ and the third and at the same time the final stage to $\alpha = 0.518$. The stages are almost at the same distances and in this way the damage pattern can be followed. The first stage corresponds to the load coefficient of $\alpha = 0.243$, and the damage pattern (principal strains) are shown in Figure 4.17.

As it can be seen from Figure 4.17 the formation of the cracks is located at the load bearing walls (parallel to the action of the force) and the first cracks form around the openings due to the concentration of the stresses and "weak" points of the structure and bending of the lintels above the doors is seen. Additionally, cracks are located at the ground level at the location of the opening as indicated in Figure 4.17 (f). The behavior of the structure is governed by the load bearing walls, while the first cracks appear on the façade walls at the upper floors. Then cracks start forming at the load bearing W-Y1, and then continue to the load-bearing W-Y3 and W-Y4.
Further development of the cracks is initiated in the walls W-Y2 and W-Y5 and then W-Y6. It should be noted that parallel to the formation of the shear cracks in the load-bearing walls, formation of the cracks in the façade walls (W-X1 and W-X3) starts as well, mainly caused by the compression. This was clearly seen from the principal compressive stresses, however due to limitation of space this will not be presented in the thesis. As stated, formation of the first cracks on the façade walls is seen at the location of the upper floor around the window openings as seen on Figure 4.18, and then later on at the ground level. From the above mentioned, it is obvious that in the plane of the walls shear causes the formation of the diagonal cracks while some horizontal cracks that are located around the opening at the upper floor could be addressed to the shear sliding due to low vertical load (Tomaževič, 1999).

Figure 4.17 - Load bearing walls W-Y1 to W-Y6 (\(\alpha=0.243\))
However, at the 2nd stage, with the load coefficient equal to $\alpha = 0.3973$, the creation of a large number of new cracks and their propagation is seen on the wall W-Y6, as shown in Figure 4.19. From this moment onward the formation of the cracks is more visible in the walls W-Y5, W-Y4, continuing with walls W-Y1, whereas 5 times smaller amount of cracks appear in the walls W-Y2 and W-Y3. This is evidently connected to the individual stiffness of the walls and there is an obvious redistribution of the stresses. Redistribution of seismic loads is possible due to the available ductility of the walls, which enables the redistribution of the seismic loads from the most damaged walls to the less damaged and even to the undamaged wall. In this way energy is being dissipated during the seismic response of the building. At this moment, as well, the concentration of the cracks in the façade wall (W-X1) at the level of the ground floor is seen. The formation of the vertical cracks at the corner and at the location of the openings is evident as seen in Figure 4.20.
The location of the major cracks is at the level between the basement and the ground floor. From the above, it can be seen that actually the basement is rigid and in the linear elastic range while the upper part of the structure is behaving like an “another” body having its movement. This kind of behavior can be connected to the large difference of stiffness between the basement made out of concrete and upper floors made out of masonry.
Finally, at the 3rd stage ($\alpha = 0.518$) the largest amount of cracks is located at the walls W-Y6 and W-Y5 caused by shear. Shear damage of transversal walls (parallel to the Y direction) is evident, which would further on lead to shear failure. Additionally, due to bending above the openings, bending damage and maybe later on even failure is to be expected in these locations. Of course this has implications on the development of the cracks on the facade wall W-X1 and W-X3, but to a smaller extent, where evident compression (seen from the principal compressive stresses, not presented here) damage and even failure at the ground level is noticed, with most probably later on local falling out of masonry. The appearance of the cracks at the last step equivalent to 51.8% of the force is shown in Figure 4.21. It was interesting to note that at this step more cracks have formed now in the wall W-Y5 than in the W-Y6, followed by the wall W-Y4, while the walls W-Y1, W-Y2 and W-Y3 had almost the same and the smallest amount of strains at this stage. The calculation was stopped here as with the usage of very small steps (additionally limitation of time) convergence after 50 iterations was not obtained. This all is clearly seen in Figure 4.21 and Figure 4.22.
As indicated at the beginning of the analysis, the floors are modeled as linear elastic and it was necessary to check that the principal stresses in the floors are lower than the tensile strength. As it can be seen from Figure 4.23 the major part of the slab has a tensile strength lower than 2.2 MPa, with few peaks at the locations of the walls which is caused due to the compatibility conditions. It can be stated that the proper hypothesis was chosen.

Figure 4.21 - Load bearing walls W-Y1 to W-X6 (α=0.518)

Figure 4.22 - Failure in the ground floor Façade Walls W-X1 and W-X3 (α=0.518)

Figure 4.23 - Tensile Stresses in the slab of the 1st floor and 2nd floor (α=0.518)
This kind of damage was seen on a structure of a similar type, as shown in Figure 4.24, after the earthquake that struck Skopje in 1963. As it can be seen the concentration of damage is located on the ground floor with diagonal cracks between the openings probably caused by shear and in one of the corners falling of the masonry is evident.

![Figure 4.24 - Concentrated damage at the ground floor (Petrovski, 2003)](image)

### 4.5.8 Presentation of the Results for Acceleration in "-Y" direction

The structure was now exposed to the horizontal movement proportional to the mass in a stepwise sequence in the "-Y" direction, as indicated in Figure 4.25, the same nodes as in the previous case where chosen in order to, firstly confirm the rigid behavior of the slabs, and secondly compare the movements, behavior of the structure in the "+Y" and "-Y" direction.

![Figure 4.25 - Location of the nodes; wall labeling; and direction of the horizontal force](image)

The capacity curve for this direction is shown in Figure 4.26, the nonlinear behavior of the structure starts early and the maximum load coefficient of $\alpha = 0.449$ was reached. The movement of the nodes confirmed the rigid behavior of the slabs, as the in the case of "+ Y" direction, due to repetition the graph is omitted in this direction.
Figure 4.26 - Capacity Curve for Pushover Analysis in "Y" direction

As in the previous case, the creation of the cracks at the wall W-Y1 starts early and then it continues on the walls W-Y3, W-Y5, W-Y4 and W-Y2. In order to compare the behavior of the structure with the previous direction, structure was examined in detail at the three stages (load coefficients). The first stage corresponds to the load coefficient of $\alpha = 0.24$, the second to $\alpha = 0.40$ and the third and at the same time the final stage to $\alpha = 0.449$. The largest amount of principal tensile strains after reaching the load coefficient of $\alpha = 0.24$ is located in the walls W-Y5 and W-Y1, and additionally wall W-Y6 is activated to a large extend at this stage. This is indicated in Figure 4.27. As it can be seen in Figure 4.28, the formation of the cracks starts at the facade wall W-X2 at the upper levels with the concentration of the cracks around the openings. However, in the façade wall W-X1, as there is a large opening, tensile strains spread over the entire height and concentration of cracks is seen in the ground level at the connection with the walls W-Y6 and W-Y5 in the perpendicular direction. Evident bending of lintels is seen with cracks concentrating at the corners of the openings.
In the same trend as in the "+Y" direction after reaching \( \alpha=0.279 \) large amount of new cracks form in the wall W-Y6, followed by the creation of the new cracks in walls W-Y5, W-Y1, W-Y4, W-Y3 and W-Y2 respectively. The redistribution of the forces among the walls due to their different stiffness and stiffness degradation due to crack development is noted.

At the 39.92% of the force, the concentration and growth of the cracks due to shear is located at the wall W-Y6, followed by the shear cracks in walls W-Y5, W-Y4, W-Y1, W-Y2 and W-Y3, as well...
as vertical cracks caused by bending, and major cracks at the South façade (W-X3) as shown in Figure 4.29 and Figure 4.30.

Figure 4.29 - Load bearing walls W-Y1 to W-Y6 (α=0.40)
As it can be seen from Figure 4.30, the development of the major cracks is located between the level of the basement and the ground floor, as well as propagation of the diagonal crack along the walls in the direction of the seismic action caused by shear. These cracks have a direct impact on the formation of the cracks on the façade walls, where in both façades the concentration of the cracks is located at the lower level of the ground floor, as seen in the "+Y" direction as well.

The iteration process has stopped at the 45% of the vertical force with the major cracks developed in the wall W-Y6 (3x more than in the walls W-Y5 and W-Y4); followed by the walls W-Y1 and W-Y2 having 50% less amount of cracks in respect to the W-Y5 and W-Y4, and the wall W-Y3 had the smallest amount of cracks (50% less than W-Y5 and W-Y4). This indicates that the concentration of the cracks is located to the walls next to the wall of symmetry W-Y6.

The difference in respect to the seismic action in "+Y" direction can be connected to the fact that the North Façade (W-X3) has additional perpendicular walls (W-X2) in comparison with the South Façade (W-X1) where the middle section of the structure is made of glass (a large opening stretching over the entire height of the structure).

Figure 4.31 and Figure 4.32 show the final stage of crack development (no convergence was possible after this point).
At this stage the stresses in the stabs were checked in order to verify the hypothesis of linear elastic behavior. As it can be seen from Figure 4.33 the stresses are mostly under 2.2MPa, while the peaks can be disregarded.
4.5.9 **Comparison Between "+Y" and "-Y" Direction**

Comparing the capacity curves of the two directions it is seen that the behavior trend of the structure is almost "the same" in both directions, as indicated in Figure 4.35, this can be attributed to the rather "symmetric" structure, the difference being the non-central position of the openings in the perpendicular wall as well as a bit different configuration of the longitudinal walls, as indicated in Figure 4.34 causing local redistribution of the forces.

![Tensile stresses in the slab of the 1st floor and 2nd floor (α=0.45)](image)

**Figure 4.33 - Tensile stresses in the slab of the 1st floor and 2nd floor (α=0.45)**

![Wall W-Y1 in "+Y" direction vs. wall W-Y1 in "-Y" direction at (α=0.45)](image)

**Figure 4.34 - Wall W-Y1 in "+Y" direction vs. wall W-Y1 in "-Y" direction at (α=0.45)**

![Comparison of Capacity Curves for "+Y" and "-Y" direction for control node 44014](image)

**Figure 4.35 - Comparison of Capacity Curves for "+Y" and "-Y" direction for control node 44014**
Additionally, the existence of the longitudinal wall (W-X2) might have caused some redistribution of the forces. In both directions the structure is able to sustain a force in the value of 45% to 52% of its weight, which could be connected to the rather good characteristics of masonry.

4.5.10 Conclusion

The behavior of the structure can be clearly connected to the behavior of masonry structures stated by Tomaževič (just schematic representation) as indicated in Figure 4.36. The diagonal cracks in the load-bearing wall in direction of the seismic action are caused by shear causing shear damage and probably later on shear failure of these walls, accompanied most probably by bending damage as well. Due to bending, cracks are developing at the corners of the openings represented by vertical cracks. The bending damage of the lintels will probably lead to the bending failure of these elements. The façade walls (W-X1 and W-X3) have the development of the cracks located around the openings in the piers which can be causing compression damage, as seen from the principal compressive stresses. Additionally, formation of some diagonal crack have been identified, so here a combination of the compression and shear failure could be expected. The lintels in the longitudinal (façade) walls W-X1 and WX-3 are experiencing bending damages. The major diagonal cracks are located at the walls in the direction of the load (Y-direction) clearly indicating a typical shear behavior of the structure. The behavior of the structure is similar in both directions due to its rather symmetrical composition. Some changes minor differences are observed mainly due to different position of the opening and in the longitudinal and transversal direction. The weakest point in the structure is the level between the basement and the ground floor, caused by a large change of stiffness.

Figure 4.36 - Behavior of masonry structure of a "box" type (Tomaževič, 1999)

4.6 Time History Analysis

4.6.1 Introduction

The dynamic inelastic time-history analysis represents the most sophisticated level of analysis of buildings to earthquake actions. With the recording of accelograms, the understanding of the ground motion was improved but far away from defined. It is the PGA (Peak Ground Acceleration-$a_g$) which is usually considered as a physical parameter that is used to determine the intensity of an earthquake. So, the forces acting on the structure due to earthquake are induced by ground motion and dependent on the intensity of motion. So, a correlation regarding the intensity $I$, and the PGA has
been proposed in the Modified Mercalli Scale by Murphy in 1977 (Tomažević, 1999). However, it is not only the intensity of the earthquake that is important, but the structure upon which it acts, in order words the response of the structure. However, the largest drawbacks of this method is that is requires a lot of computational effort. Additionally, velocity can be determined and this is very important as the damage of the building is connected to the amount of released energy and further the displacement; further, duration of the strong phase of the ground motion and predominant periods of ground oscillations.

This kind of method provides a stepwise solution in the time domain of multi-degree-of-freedom equations of motions representing a response of the modeled structure. Non-linear properties of the structure are taken into account in the time-domain analysis. The following equation of motion that needs to be satisfied in each step (Chopra, 2007):

\[ M\ddot{u}(t) + C\dot{u}(t) + Ku(t) = p_{eff}(t), \] where \( p_{eff}(t) = -M1\ddot{u}_g(t) \)

Only the relative components are taken into account on the left-hand side as they will be producing the internal forces. As an input several ground motion records (accelelograms) are necessary, giving an accurate evaluation of the seismic response of structures.

The case study was subjected to a nonlinear dynamic analysis. In this type of analysis the equilibrium and compatibility of the model are evaluated for each instant of time according to the equation of motion, resulting in structure's response also over time (displacement, velocity and acceleration among others).

4.6.2 Mechanical and Physical Characteristics

The same non-linear characteristics as for the Pushover Analysis were used in the non-linear time history analysis.

4.6.3 Petrovac Accelelogram

The structure was exposed to Petrovac short-period earthquake recorded during the earthquake at Petrovac on the April 15, 1979, (Montenegro) with peak acceleration of 0.43g, in order to verify the anticipated response of the structure. This is one of the accelerations being frequently used for different verifications throughout ex-Yugoslavia (Tomažević, et al., 1997),(Tomažević, et al., 2004). The differences in the soil conditions between Sarajevo and Petrovac in this case were not taken into account. The accelelogram was scaled in order to have a maximum PGA of 0.1g corresponding to the Zone VII (MCS-Scale) for the Sarajevo region, where the building is located, and filtered using the software Seismosignal (Seismosignal, 2010). The value for the damping ratio was taken as 5% (\( \xi = 5\% \)). The accelelogram, velocity and displacement in the duration of around 24s of the scaled and filtered Petrovac earthquake in the North-South direction, is presented in Figure 4.37.
As only half of the structure was modeled and investigated (only in the “Y” direction) it was decided to expose the structure only to an accelerogram in this direction, opposed to the standard recommendations by Eurocode 8 (CEN, 2003) that the structure should be exposed to simultaneously acting accelerograms in both horizontal directions. This was additionally omitted due to fact that the structure has only load bearing walls in the Y direction and enforcing an earthquake in the weak - “X” direction would probably cause a premature failure of the structure.

### 4.6.4 Material Damping

As it is well known, damping properties or materials are not well established, due to the large number of factors upon which it depends and uncertain relations among them. This is a phenomena still being investigated and until now fully not understood. For the existing buildings it would be ideal to determine this value through experiments; however due to lack of budget and time this was not done. In the absence of such data, it is recommended to use values from similar buildings (Chopra, 2007). When using this data one needs to be very cautious as for small structural motions the
sampling values are not representative for larger structure movement; structures which experience significant yielding during an earthquake give damping ratios that includes the energy dissipation caused by the yielding of the materials.

There are several possibilities for appropriate idealization of the damping matrix. From classical damping used when similar damping mechanisms are distributed throughout the structure. This would be a good approximation for this structure as it is a multistory building of the same structural system and material over its height. The methods that are usually utilized, when there is no availability of the experimental data are Rayleigh's and Caughey's Damping Method. As for both methods the damping matrix is diagonal due to inherent modal orthogonality.

Classical damping, and precisely Rayleigh damping, shown in Figure 4.38, has been chosen as the building is of a similar structural system and structural material over its height. Rayleigh damping composes of mass-proportional and stiffness proportional damping defined by the following equation (Chopra, 2007):

\[ c = a_0m + a_1k \]

where \( a_0 \) is the first Rayleigh coefficient and \( a_1 \) is the second Rayleigh coefficient.

![Figure 4.38 - Rayleigh damping (Chopra, 2007)](image)

It is assumed that the modes have the same damping ratio of \( \xi = 5\% \) (Tomažević, et al., 1997), (Chopra, 2007), (Flores, et al., 1996), (Morante, et al., 2006), (Anderson, et al., 2009), usually supported by experimental data, then the Rayleigh's coefficients are calculated by the following equations:

\[ a_0 = \xi \frac{2\omega_i\omega_j}{\omega_i + \omega_j} \text{ and } a_1 = \frac{\xi}{\omega_i + \omega_j} \]

where \( \omega_i, \omega_j \) are angular frequencies of the \( i \)-th and \( j \)-th mode.

Correct choice of modes \( i \) and \( j \) is essential in order that they can ensure reasonable values for the damping ratios in all the modes contributing to the response. Frequencies of the mode 1 and 3 were taken in the analysis, even though the cumulative mass participation was 73.05\%, as the
contribution of the other modes was rather low. In this way the contribution of the undesirable higher modes would be avoided.

### 4.6.5 Iteration and Integration Methods

Application of the same iteration method (Newton-Raphson) was used in this computation, for details see 4.5.5, while it was necessary to use an implicit Hilbert-Hughes-Taylor method (DIANA 9.4, 2009) for time integration.

Hilbert-Hughes-Taylor (HHT) method (known as \( \alpha \) methods) is actually a generalization of the Newmark \( \beta \) method. The equations are the same as the ones used in the Newmark \( \beta \) method, but an additional parameter \( \alpha \) is introduced. So, now the equation of motion reads:

\[
M\ddot{u}_{i+1}(t) + (1 + \alpha)Cu_{i+1}(t) - (a)Cu_i(t) + (1 + \alpha)Ku_{i+1}(t) - (a)Ku_i(t) = (1 + \alpha)p_{\text{eff}}(i+1)(t) - (a)p_{\text{eff}}(t)
\]

If \( \frac{1}{3} \leq \alpha \leq 0; \beta = \frac{1 - a^2}{4}; \text{and} \ \gamma = \frac{1}{2} - \alpha \), the method is second-order accurate and unconditionally stable. Decreasing \( \alpha \) means increasing the numerical damping. This damping is low for low-frequency modes and high for the high-frequency modes, meaning that influence of non-realistic modes of high frequencies is eliminated. By this method it is possible to introduce numerical dissipation without degrading the order of accuracy (DIANA 9.4, 2009). The value of \( \alpha = -0.1 \) was selected.

The reasoning behind this lies in the fact that masonry has a very low tensile strength, so there is a rapid transition from the elastic range to the fully cracked stage with development of a large number of distributed cracks, leading to almost no stiffness. This all leads to problems of convergence and additionally taking into account that the stiffness matrix has to be updated at each step making the calculation time consuming. Secondly, this method has proven useful in structural dynamic simulations with many degrees of freedom.

It is important to mention that the chosen method satisfies three important requirements:

- **convergence** - as the time steps are decreased the numerical solution approaches the exact solution;
- **stability** - in the presence of the numerical round-off errors the numerical solution is stable;
- **accuracy** - the result obtained by the numerical solution is close to the exact solution.

For an accurate dynamic response illustration it is important to make a correct choice of the time increment \( \Delta t \). As per (DIANA 9.4, 2009) the time increment should be small enough in comparison with the total duration of the accelerogram, defined as:

\[
\Delta t \ll t
\]

Secondly, as stated in Chopra (Chopra, 2007), the recommended time step depends on the shortest period of interest, and is defined as:
In order to satisfy the second recommendation it would be necessary to perform 14000 steps, which due to the limited time and very high computation time was not done. Due to the size of the structure, and the fine mesh (0.25m) with a large number of DoF, it was decided to select \( \Delta t = 0.01 \text{ sec} \), being much smaller that the duration of the accelelogram \( t = 23.87 \text{ sec} \), leading to 2387 steps.

### 4.6.6 Results of the Analysis

In order to view the development of the damage to the structure it was necessary to investigate the damage pattern (principal tensile strain distribution) in different time steps. The displacement vs. time of the Control Node 44014 (as in the pushover) was chosen and plotted as indicated in Figure 4.39. As it can be seen, the maximum displacement was -11.38mm at the time of 7.65s. In order to view the progression of the damage in the structure it was necessary to select several time instances during the earthquake action. The time instances that have been selected are indicated in Figure 4.39. Elaboration of the time steps equal to \( t_1 = 3.29s \) and \( t_3 = 7.79s \) is done in the thesis, as they show the two peak displacements in two opposite directions. Additionally, the damage pattern on the structure after the duration of the earthquake, meaning at the time \( t_5 = 23.87s \) is shown as well. The principal strains at the time instance \( t_2 = 3.41s \) and \( t_4 = 7.65s \) representing the opposite peaks in respect to the elaborated ones, as shown in Figure 4.39, will be given in the Appendix 2.

![Displacement of the Control Node 44014](image)

**Figure 4.39 - Displacement of the Control Node 44014 in respect to time**

As it can be seen from Figure 4.40 and Figure 4.41, the development of the cracks follows the same pattern as obtained in the pushover analysis. Development of the cracks is located around the openings with the cracks slowly concentrating at the ground level of the structure on the façade walls.
Figure 4.40 - Load bearing walls W-Y1 to W-Y6 (t = 3.29s)
The second step presented is equal to the $t_1 = 3.29s$ and in the opposite direction in respect to the previous illustrated step. The development of the crack pattern follows the same sequence as obtained in the pushover analysis for this direction, as it can be seen in Figure 4.42 and Figure 4.43.
The concentration of the damage can be correlated by the propagation of the acceleration shown in Figure 4.44 and Figure 4.45. It is clear that at the micro-locations of the "high" acceleration, bigger damages are evident.
Figure 4.44 - Acceleration distribution in the load bearing walls W-Y1 to W-Y6 (t_E = 7.79s)

Figure 4.45 - Acceleration distribution in the façade Walls W-X1 and W-X3 (t_E = 7.79s)

The final damage of the structure after the termination of the earthquake, after (t_E = 23.87s) is shown in Figure 4.46 and Figure 4.47. The concentration of the damage is located at the location of the openings as well as in the ground level and the first floor. The creation of the typical "X" cracks for shear behavior is evident in Figure 4.46 (f).
Figure 4.46 - Load bearing walls W-Y1 to W-Y6 ($t_s = 23.87s$)
Structural and Seismic Behavior of Typical Masonry Buildings from Bosnia and Herzegovina

Regarding the global behavior of this case study, it can generally be correlated with the experimental investigations conducted by Tomažević in 1991 (Tomažević, et al., 1991) where it has been seen that there is a concentration of the damage in the first floor as seen in Figure 4.48. In this case, the ground floor can be looked at as the first floor due to high stiffness of the basement made out of reinforced concrete. This indicates that this type of masonry structures can be modeled as a storey mechanism mode-shear wall with pier action(Tomažević, 1987),(Tomažević, et al., 1991).

The analysis of the collapse mechanism in respect to the maximum displacement and inter-storey drift, defined as difference of displacement at the top and the bottom of the storey under consideration as per formula:

\[ \Delta = \frac{d_n - d_{n-1}}{h_n - h_{n-1}} \]

is shown in Figure 4.49 and Figure 4.50. The maximum displacement is observed at the last floor in the value of 11.38mm. However, in order to correlate the damage it is necessary to evaluate the inter-storey drift. It was interesting to note that the maximum drift in floors 1, 2 and 4 is reached at the same time instance, whereas floors 5 and 6 reach their maximum drift at the same time, as shown in Figure 4.50. The biggest jump in the inter-storey drift is observed at the ground level, at the height of 2.8m, being equivalent to 0.51%, as shown in Figure 4.50. The cause of damage in walls is a high...
value of the drift (which is a "jump" between floors). The jump in the drift values is a sign of deep change in stiffness. The envelope shows the largest storey drift is 0.78% located at the second floor (8.4m), which is consistent to the damage pattern shown before. Smaller damage is observed at the upper floors and in the inter-storey drift is smaller, so a good correspondence is observed.

![Figure 4.49 - Maximum displacement](image)

The large drift imposes severe deformation and ductility demand at locations of the lower floors. The distribution of the storey drift over the height of the structure depends on how much the structure can deform into the inelastic range. Basically, it greatly depends upon the ductility factor, a measure of the degree of inelastic behavior.

![Figure 4.50 - Inter-story drift](image)

Additionally, hysteresis curve has been determined for the control node 44014. In this respect the relationship between the load coefficient and displacement has been obtained as shown in Figure 4.51. The maximum load coefficient of 0.23 was reached while the maximum displacement was 11.38mm.
4.6.7 Conclusion

On the basis of the results obtained from the THA it can be seen that the structure has a typical shear behavior. The walls parallel to the load experience diagonal cracks caused by shear, and due to the cyclic loading, an evident diagonal "X" type cracks are formed. At the location of the openings the concentration of the damage is evident due to the concentration of the stresses. As it can be noticed at the end of the earthquake action major damage, besides the load-bearing walls that are governing the behavior of the structure, are seen at the non-load bearing walls at the lower levels. The major concentration of the damage is located between the basement and the ground floor, which can be connected with the discontinuity and large difference of the stiffness. Large damage is observed in the lower floors where the largest inter-story drift was observed imposing severe deformation and ductility demand at these. This kind of behavior has been identified by the previous earthquakes on a similar structure in Skopje and by experiments conducted by Tomaževič in Slovenia. The propagation of the damage slowly expands to the upper floors on the façade walls. The earthquake action, as a cyclic loading causes the degradation of the stiffness of the structure.
5.1 3Muri Software program

The second calculation was done in 3MURI Program, which uses the Frame by Macro Elements (FME) method (S.T.A.-DATA, 2010) being one of the most practiced methods within this category available for the calculation of the masonry structures, inspired by the "equivalent frame" method. The non-linear response is determined in masonry macro-elements, composed of piers and spandrels and rigid elements that connect the piers and the spandrels as seen in Figure 5.1. This is something that came out of from observation of the structures after earthquake actions and imposed damages on the structure as shown in Figure 5.2.

![Figure 5.1 - Mesh composed of macro-elements](image)
Figure 5.2 - Shear failure mechanism and failure mechanism due to compression bending and edge crushing

This method allows a simplified and fast seismic assessment of the structural behavior and elaboration of damage, while computational effort is reduced in respect to FEM.

The macro-elements are able to capture the main characteristics of the non-linear behavior of masonry structure with regular openings, as is the one proposed in this work. As the slab is rigid, the structure behaves as a typical "box" structure indicating that the global seismic analysis can be conducted. So, in this way the seismic loads (horizontal forces) are distributed to the walls according to their stiffness, so the resistance of the whole structure depends on the lateral strength and deformation capacities of the corresponding macro-elements. In the macro-elements, an equivalent homogeneous material within these elements is assumed, and as its tensile strength is very low it is assumed that it is no-tensile resistant (NTR) and in this way the flexural strength of the macro-elements is defined.

The panels, connected by rigid nodes are of two types: the "piers" which represent the principal vertical elements that resist both dead and seismic loads; and the "spandrels" representing secondary horizontal elements, coupling piers in the case of seismic loads (Galasco, et al., 2004), (Roca, et al., 2005). The entire structure is modeled by assembling the masonry walls (2D frames) and a right horizontal floor.

5.2 Macro-element Modeling

Modeling of existing masonry structures under seismic actions is a very complex task owing to the constitutive characteristics of the structural material and its geometrical non-linear behavior when subjected to strong ground motion.

As it is well known, there are two main in-plane masonry modes, bending-rocking and shear-sliding mechanism. 3MURI software is based on non-linear macro-elements which represent an entire masonry model. By the utilization of the internal parameters, the macro-element takes into account the limited compressive strength of masonry. The limited crushing strength of masonry is usually developed in the bending-rocking mechanism through a toe crushing effect. A phenomenological nonlinear constitutive law is used to model this effect with stiffness deterioration in compression.
Additionally, by the internal variables, this model takes into account the shear-sliding damage evolution, controlling the strength deterioration and stiffness degradation (Galasco, et al., 2004).

The macro-element model is a representation of a continuous model but in a macro scale with the parameters, representing average characteristics of a masonry panel, directly correlated to the mechanical properties of the masonry elements. The macro element is composed of three substructures as indicated in Figure 5.3. It is made out of a central part 2, and two layers being 1 and inferior and 3 a superior layer where bending and axial effects are concentrate, while in the central part 2 shear-deformations are observed. In this way the shear failure modes are confined by controlling the average response parameters able to simulate stress/strain fields inside the whole masonry panel. This implies that the shear failure modes are not connected to the local conditions in respect to individual discontinuity surfaces. Each node has three degrees of freedom: axial displacement \( w \), horizontal displacement \( u \) and rotation \( \varphi \). Meaning that a complete 2D kinematic model has to take into account the DoF for the two nodes \((i \text{ and } j)\). The central zone has two degrees of freedom: the axial displacement \( \delta \) and rotation \( \psi \). (S.T.A.-DATA, 2010). This means that the macro-element is described with 8 degrees of freedom. So, it is the two terms of \((u, \varphi, w)\) that simulate the axial and flexural response of the panel; while a couple of \((\psi, \delta)\) are connected to the axial and shear response.

As it can be seen from Figure 5.3, the bending flexibility is taken into account at the ends (dimensions \( b \times a \times s \)); shear flexibility being assigned only at the central part (dimensions \( b \times h \times s \)); and axial flexibility is being assigned to all parts of the macro-element. The wall of certain height and width is taken into account, while the thickness of the inferior and superior layer is assumed to be zero \((d = 0)\) (Penna, 2002). The compatibility conditions at the edges require that the adjacent piers have the same vertical displacement; by connecting the floors and walls the compatibility conditions are automatically imposed by connecting the macro-element nodes directly to the floor elements.

![Figure 5.3 - Kinematic and static parameters for the macro element (3MURI)](S.T.A.-DATA, 2010)

Non-linear constitutive equations of the macro-elements are based on the hypotheses:

- zero tensile strength in the extreme parts
- sliding shear with damage in the central part
In this way the axial and flexural responses are uncoupled from the shear response. This is valid for their inelastic and damage mechanisms as well.

For the extreme parts of the macro-element, a relationship between the kinematic and static parameters is established by the constitutive equations. If the cross section of the panel is completely compressed then the equations are uncoupled, as the axial force is applied within the central core of inertia. This means that under this axial force, the rotation \( \varphi \) will increase linearly with the bending moment until the static and kinematic conditions denoting this stage are satisfied. So, the axial force and the bending moment can be expressed as:

\[
N = k \frac{b}{s} w \\
M = \frac{k}{12} b^3 s \varphi
\]

where \( k = \frac{2E}{b} \) is the axial stiffness for the unit area.

The kinematic and static conditions are being applied in such a sense that:

\[
|e| = \left| \frac{M}{N} \right| = \left| \frac{k}{12} b^3 s \varphi \right| \left| \frac{k}{b s w} \right| = \frac{k}{12} \frac{b^3 s |\varphi|}{b s w} = \frac{k}{12} \frac{b^2 |\varphi|}{w s w} \leq \frac{b}{6}
\]

and taking into account the kinematics, and according to the NTR masonry in bending \( w < 0 \), the equation reads

\[
|\varphi| \leq \frac{2w}{b}
\]

When these conditions are not satisfied this means that the cross-section will crack, inelastic contributions of the internal forces caused by the eccentric compression can be calculated due to the assumption of the small displacements, and the equations become

\[
N = k b s w - \frac{ks}{8|\varphi|} (|\varphi|b + 2w)^2
\]

\[
M = \frac{k}{12} b^3 s |\varphi| + \frac{ks}{24} \frac{(|\varphi|b + w)}{|\varphi|} (|\varphi|b + 2w)^2
\]

When the cross-section cracks, (geometrical non-linearity) at this time the axial force and the bending moment are being coupled and then the relation between \( w \) and \( \varphi \) become:

\[
w = \frac{|\varphi|b}{2} - \sqrt{\frac{-2|\varphi|N}{ks}}
\]

Compression damage is described by two parameters. When the translation \( w \) exceeds the limit value defined by \( w_R = \frac{3\varphi}{k} \) in a part of the cross-section identified by a damage parameter \( \zeta \). The second damage parameter is defined by \( \mu = \frac{w_{max}}{w_R} \) representing the ductility demand on the extreme fiber, and the resulting stiffness degradation. If \( w_{max} < w_R \) when the macro-element is unloaded, then
the stress state depends on the load history through the damage parameters $\zeta$ and $\mu$ (S.T.A.-DATA, 2010). (Galasco, et al., 2004).

Additionally parameters that are describing the behavior of the macro-element are the shear modulus of elasticity ($G$), axial stiffness ($K'$); the shear strength at zero confining stress ($f_{vo}$); flexibility coefficient ($c$) controlling the inelastic shear deformations; the global friction coefficient ($\mu$); and a factor ($\beta$) associated with toughness ($R$) which controls the softening phase (Galasco, et al., 2004).

5.3 In-plane Wall Model

The in-plane behavior of masonry walls is modeled by using the macro-elements as indicated in Figure 5.1 and forming a "frame" structure. The wall is divided into piers and spandrels being connected by rigid nodes. It has been seen that, in most cases, piers and spandrels are characterized by inelastic and damaging mechanisms and are related to both opening of crack and shear dissipative sliding, whereas the areas that are connecting them do not experience any damage or rather very low. In this way the structure is discretized into macro-elements as shown in Figure 5.1 (S.T.A.-DATA, 2010).

5.4 Model of the 3D Structure

Once the in-plane masonry walls are joined together, a 3D model is obtained. As the macro-elements are taking into account only the in-plane behavior of the walls the floors are distributing the horizontal load to the walls. In this case a rigid floor is assumed indicating a distribution of the forces to the walls in respect to their individual stiffness. The out-of-plane behavior is not taken into account as it can be disregarded when the global behavior of the structure is being controlled by the in-plane behavior. Different walls are connected in the corners and intersection by nodes with 5 DoF ($u_x, u_y, u_z, q_x, q_y$). $q_z$ can be disregarded as a membrane behavior is adopted between the walls and the floors.

As the case study is with regular opening pattern and well connected walls, it is recommended by Eurocode8, (CEN, 2003) to conduct an analysis by the "Equivalent Frame".

Advantages of this method are that modeling is less time consuming in respect to the FEM. Even though it is based on simplified assumptions based on the macro-element constitutive equations, it is able to estimate to a good degree the peak-load, quite accurately to illustrate the damage evolution and stiffness degradation.

5.5 Response Spectra as per Eurocode 8

The calculation is done on the basis of the Response Spectra defined in Eurocode 8 (CEN, 2003). The shape of the proposed elastic response spectra by Eurocode 8 is:
The response spectrum of the building is obtained from the following equations, resulting in a response spectrum with the setup shown in Figure 5.4.

\[
0 \leq T \leq T_B : S_e(T) = a_g \cdot S \cdot \left[1 + \frac{T}{T_B} \cdot (\eta \cdot \beta_0 - 1)\right] \\
T_B \leq T \leq T_C : S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \\
T_C \leq T \leq T_D : S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \cdot \left[\frac{T_C}{T}\right] \\
T_D \leq T \leq 4S : S_e(T) = a_g \cdot S \cdot \eta \cdot \beta_0 \cdot \left[\frac{T_C \cdot T_D}{T^2}\right]
\]

where:
- \( S_e(T) \): elastic response spectrum;
- \( T \): vibration period of a SDOF system;
- \( a_g \): design ground acceleration on type A ground;
- \( \beta_0 \): maximum normalized spectral value assumed constant between \( T_B \) and \( T_C \) (\( \beta_0 = 2.5 \))
- \( T_B \): lower limit of the period of the constant spectral acceleration branch;
- \( T_C \): upper limit of the period of the constant spectral acceleration branch;
- \( T_D \): value defining the beginning of the constant displacement response range of the spectrum;
- \( S \): soil factor;
- \( \eta \): damping correction factor with a reference value (\( \eta = 1 \) for 5\% viscous damping)

By the Eurocode8 (CEN, 2003) definition of soil effect and intensity of the earthquake are separated as the response spectra are normalized in respect to the maximum ground acceleration. So, in this way the response spectra indicates the amplification of the ground motion, whereas the design ground acceleration correspond to expected intensity of earthquakes taking into account the specific return period of occurrence (475 years).

In order to define the parameters for the creation of the elastic response spectrum for far field earthquakes (\( M_s > 5.5 \)) - Type 1, the data provided in EC8 (CEN, 2003) as indicated in Table 5.1.
According to the geological data of Sarajevo it corresponds to type C, and the value of the maximum ground acceleration, as per Table 5.1 amounts to 0.1g.

Table 5.1 - Value of parameters describing the recommended Type 1 elastic response spectra (CEN, 2003)

<table>
<thead>
<tr>
<th>Ground Type</th>
<th>S</th>
<th>$T_a$ [s]</th>
<th>$T_c$ [s]</th>
<th>$T_D$ [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1.00</td>
<td>0.15</td>
<td>0.4</td>
<td>2.0</td>
</tr>
<tr>
<td>B</td>
<td>1.20</td>
<td>0.15</td>
<td>0.5</td>
<td>2.0</td>
</tr>
<tr>
<td>C</td>
<td>1.15</td>
<td>0.20</td>
<td>0.6</td>
<td>2.0</td>
</tr>
<tr>
<td>D</td>
<td>1.35</td>
<td>0.20</td>
<td>0.8</td>
<td>2.0</td>
</tr>
<tr>
<td>E</td>
<td>1.40</td>
<td>0.15</td>
<td>0.5</td>
<td>2.0</td>
</tr>
</tbody>
</table>

5.6 Results Obtained by 3MURI

The structure was modeled in 3MURI having with the same material and geometrical characteristics as defined in Section 3. The model consists of 7 levels, 218 - 3D nodes, 34 - 2D nodes, and 506 elements. The view of the structure and elements is presented in Figure 5.5.

![Figure 5.5 - View of the structure and its elements in 3MURI](image)

First of all modal analysis was conducted and on the basis of this analysis the data regarding the periods as stated in Table 5.2 is obtained. The first three mode shapes are shown in Figure 5.6, Figure 5.6 and Figure 5.8. Figure 5.6 shows the first mode corresponding to translation in X direction, the second mode corresponds to the torsion along Z axes, as seen in Figure 5.6, and the third mode expressed as translation in Y direction is shown in Figure 5.8.

In order to verify the assumption of the weakness of the structure in the X-direction the structure in this program was exposed additionally to the seismic action in “± X” as well as in “± Y” in order to make the comparison with the DIANA calculations. Results of the analysis performed by 3MURI were compared with calculations done by DIANA and the comparison is provided in the Chapter 6.

Table 5.2 - Periods and eigen-frequencies obtained by 3MURI

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency [Hz]</th>
<th>Periods T [s]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.96</td>
<td>0.51</td>
</tr>
<tr>
<td>2</td>
<td>3.23</td>
<td>0.31</td>
</tr>
<tr>
<td>3</td>
<td>3.57</td>
<td>0.28</td>
</tr>
</tbody>
</table>
5.6.1.1 Results Obtained in the "± X" Direction

As assumed the structure is not resistant in the "± X" as indicated in Figure 5.9. During the calculation it has been seen that in the X direction the structure is not able to support the load, generalized failure is observed, due to its weak characteristics in that direction, as the walls are not load-bearing in this direction. Using the legend shown in Figure 5.10, the damages of the façade walls with compression failure at the ground level are shown in Figure 5.11 and Figure 5.12.
5.6.2 Results Obtained in the "+Y" Direction

The capacity curve of the structure while exposed to the load pattern proportional to the mass in "+Y" direction is shown Figure 5.13. The maximum displacement amounts to $D_{\text{max}} = 25.73 \text{ mm}$, while $D_u = 35.10 \text{ mm}$.

![Figure 5.13 - Capacity curve for "+Y" direction](image)

In order to understand better the behavior of the structure the individual walls have been elaborated. As it can be seen from Figure 5.14, the bending damage is located at the spandrels above the doors and windows, and even in some of the walls bending failure is observed. Shear damage is seen on all the perpendicular walls (Y direction) and especially on the wall W-Y6. The basement which is made out of concrete remains undamaged during the application of the load. The damage sequence is shown in the following figures (legend indicated in Figure 5.10) giving a description of the associated damages.
As it can be seen in Figure 5.15, the compression failure is concentrated on the ground level of the façade wall W-X1, and it propagates in the direction of the upper floors. The ground and the first floor are mainly affected and the same pattern was observed in the THA using FEM. The damage
at the line of symmetry propagates until the fourth level. Evident bending damages are observed around the openings.

![Figure 5.15 - Façade walls W-X1 and W-X3 at the failure stage](image)

5.6.3 Results Obtained in the "-Y" Direction

Capacity curve obtained for the load proportional to the mass in the "-Y" direction is shown in Figure 5.16. The maximum displacement amounts to $D_{max} = 14.33\,mm$, while the ultimate displacement is $D_u = 25.42\,mm$.

![Figure 5.16 - Pushover curve for "-Y" direction](image)

A similar behavior is observed in the "-Y" direction and due to consistently the results are shown in Figure 5.17 and Figure 5.18. The damage pattern is characterized by the bending damage and even bending failure at the spandrels in most of the walls. The largest shear damage is observed in the W-Y4 wall. The façade walls (W-X1 and W-X3) have mainly bending damage, whereas at the lower levels local compression failure is observed. The same sequence of behavior has been seen in the FEM calculations.
Figure 5.17 - Load bearing walls W-Y1 to W-Y6 at the failure stage.
5.7 Conclusion

On the basis of the calculations done in 3MURI, it could be seen that the structure has a typical shear behavior. As the structure is composed by load-bearing walls in only one direction, the weakness in the other direction ("X" direction - longitudinal walls) was evident. Additionally damage of the spandrels is due to stress concentration around the openings. Regarding the façade walls most of the damage concentrated on the lower floors with slow rising of the damage to the upper levels. On the basis of these calculations it could be concluded that the structure has a rather "good" behavior in the "Y" direction, however its evident weakness in the "X" direction in the global view makes the structure not capable of resisting earthquake actions of this range. Due to this strengthening of the structure is suggested.
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Chapter 6
COMPARISON OF RESULTS

6.1 Introduction

First of all it is important to note that the scale of the two models is rather different. Utilizing DIANA the structure is modeled as a non-linear continuum by finite elements, while in 3MURI the structure is formed from assembly of structural elements as blocks upon which the non-linear behavior is assigned. This has a direct implication on the mesh of the structure, definition of the constitutive law and mechanical properties.

In the FEM the non-linear constitutive models are formed by the characteristics of the material (stress - strain relation). While in the "equivalent frame" model the constitutive models are connected to the masonry panels (force-drift relationship). The calculation done by 3MURI requires less computation time, not so "complex" and one may say not so "detailed" regarding the location of the damages, as the calculation done by DIANA but the results are more than acceptable.

6.2 Dynamic Properties

In the comparison firstly the eigen-frequencies and eigen-modes were compared in order to verify the correctness of the model. It has been noticed that the difference in the weight is 8.5%, which due to different modeling scales is acceptable. As a consequence, the eigen-frequencies, (only first three will be presented) and mass participation is different as indicated in Table 6.1.
Table 6.1 - Comparison of periods and eigen-frequencies (DIANA vs. 3MURI)

<table>
<thead>
<tr>
<th>Periods T [s]</th>
<th>Difference [%]</th>
<th>Mass participation M [%]</th>
<th>Difference [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>DIANA</td>
<td>3MURI</td>
<td>DIANA</td>
<td>3MURI</td>
</tr>
<tr>
<td>0.46</td>
<td>0.51</td>
<td>10.9</td>
<td>67.33 (x)</td>
</tr>
<tr>
<td>0.26</td>
<td>0.31</td>
<td>19.2</td>
<td>67.39 (x)</td>
</tr>
<tr>
<td>0.25</td>
<td>0.28</td>
<td>12.0</td>
<td>58.79 (y)</td>
</tr>
</tbody>
</table>

As it can be seen from Table 6.1, the maximum difference for the first three modes is 19.2% while for mass participation is 9.2%. The mode shapes obtained by both modes are the same. It can be concluded that a good correlation between the two calculations has been obtained. Additionally, when looking at general behavior of the structure regarding the crack development and failure mechanism it can be seen that the two models are consistent in the application of the pushover method and even the THA in DIANA. The global view of the damage is visible as well as the propagation of a certain mechanism failure in 3MURI program, while DIANA gives a more detailed crack pattern with a clear manifestation of stiffness degradation, and localization of the cracks and damage in the structure.

6.3 Pushover Curves in 3MURI

Comparison between the pushover obtained by the uniform pattern and proportional to the first mode was conducted, as shown in Figure 6.1.

![Capacity Curves - Uniform vs. Modal Pattern "+Y" and "-Y" direction](image)

As it can be seen the uniform load distribution gives higher value of the load coefficient in respect to the modal load distribution, where as the damage pattern of the both pushover curves follows the same law. As the structure is rather uniform in height the modal load distribution leads to relatively higher inertial forces in the higher stories.

6.4 Pushover Curves DIANA and 3MURI

In Figure 6.2 and Figure 6.3 capacity curves for the "±Y" direction obtained by DIANA and 3MURI are compared. The capacity curve achieved by Finite Element Method (DIANA), once the maximum strength was obtained, stopped due to convergence issues. The capacity curve obtained by
the Equivalent Frame Model (3MURI) after reaching the strength continues on with a horizontal plateau and then reduction of strength is evidently seen. The difference in the stiffness can be attributed to rigid connection between the spandrel and the pier elements. Spandrels mainly influence the boundary conditions of the piers (Cattari, et al., 2008) as stated: "their characterization in terms of strength is thus more important than that in terms of stiffness". The difference regarding the maximum load coefficient is in the range from 6.39% to 6.94% which is less than 10%, so it is in the acceptable range.

![Figure 6.2 - C.C. DIANA vs. 3MURI '+Y' direction](image1)

![Figure 6.3 - C.C. DIANA vs. 3MURI '-Y' direction](image2)

### 6.5 Pushover and THA Curves in DIANA

Comparison between the calculation obtained in DIANA performed by the pushover analysis and the time history analysis can be done only in the pattern of damage. As it can be clearly seen from the damage figures, the pattern of the damage follows the same sequence. Firstly, the location of the cracks is located around the openings caused by the flexure. Additionally, the windows are only 75-100cm away from the corner of the wall connections, weakening the connection between the walls and causing additional concentration of the stresses and crack development.

The propagation of the cracks in the transverse walls follows the same pattern, with the cracks around the windows caused by flexure and diagonal cracks caused by the shear deformations. The major damage is concentrated on the ground and the first floor.

Comparison of the capacity curves from the pushover and THA for the earthquake action of $a_g = 0.1g$, as shown in Figure 6.4 gives information regarding the maximum load coefficient and displacement, however it is important to take into account the maximum inter-storey drift when assessment and verification of the structure is done as discussed in Section 4.6.6. It is also possible to observe the dynamic behavior causes a more pronounced stiffness degradation due to cyclic loading.
6.6 Conclusion

Several comparisons have been made, and it can be concluded that when talking about the non-linear pushover method that the choice of the load pattern has a direct implication on the results. Choice regarding the level of sophistication of the model has a direct impact on the accuracy of the results as well as on the degree of detail regarding the representation of the crack pattern. Model done in DIANA gives a very detailed crack pattern, however at the same time the computational time is much longer in respect to 3MURI. In this case study, the structure being rather regular, results obtained with 3MURI are in a very good correlation with the results obtained by DIANA.

In both calculations the structure showed a typical shear failure mode in the walls parallel to the action of the load. In the façade walls (along the x-axis) the concentration of the damage in both models is seen at the lower floors with a slow propagation over the height of the structure. The location of this concentrated damage can be connected to the large stiffness change at this location.

The structure showed a “good” behavior in the “±Y” direction, passing the verifications defined by Eurocode 8 in the 3MURI program, however the structure failed in the “±X” direction implying that in the global view the structure is not safe. Due to this some proposals will be given regarding strengthening which will not be elaborated in detail, as this is out of the scope of this thesis.
Chapter 7
POSSIBILITIES OF STRENGTHENING

7.1 Introduction

The structural risk that the existing structures represent may be enlarged due to several reasons: deterioration of structural elements due to bad maintenance, reduction the load-bearing capacity due to removal of inner elements for obtaining more space, construction of additional storey levels and further damages cause by earthquake actions.

Which type of strengthening method will be applied depends on type of the structure, quality of the material of the existing structure, availability of adequate equipment and workers' skills and knowledge, and above all its seismic resistance. The decision will be made according to the required degree of improving the structure's resistance. The state of the structure before the intervention greatly influences the selection of the technical procedure, its applicability and effectiveness. It is important to ensure a good global behavior of the structure due to earthquake action. So, the entire structure has to possess an adequate resistance, ductility and energy dissipation. The correct application of strengthening method is not only of a technical issue but of an economic justification as well.

Knowledge about the structure, its material, and its connections is of the utmost importance for a correct strengthening choice. Inappropriate interventions can lead to further damage and even collapse of the structure, as was the case after the Umbria earthquake (Penazzi, et al., 2000). Until today many techniques have been exploited for strengthening of the existing unreinforced masonry
structures. Decision of the adequate strengthening method has to be done for each structure individually.

### 7.2 Possible Strengthening Methods

The typical residential masonry structure in B&H that has been investigated in this thesis, indicated on the major deficiencies of these types of structures and that is the lack of load-bearing structures in the longitudinal direction as indicated in Figure 7.1.

![Figure 7.1 - Adequate and not adequate distribution of load-bearing walls (Tomaževič, 1999)](image)

One of the first repair methods would have to be done regarding the walls in the longitudinal direction, as they are not load bearing, they do not have enough strength and capacity to take over the seismic actions.

The check was only done in 3MURI as the entire structure was modeled there and it was noted that the structure in the X-direction does not have enough resistance as indicated in Section 5.6.1.1.

One possibility would be to make additional walls in this direction, the location of these walls should be chosen such that a uniform distribution is obtained in plane an in height, in order to avoid unwanted torsion effects. As the structure is occupied, this would be rather difficult to perform, especially construction of internal walls.

A better solution would be to construct a steel skeleton around the building and in that way improve the resistance of the structure. This kind of strengthening has been done on several structures of historical importance like a historical theater in Italy (Tomasi, et al., 2007), masonry old building (main walls) in Poland (Berkowski, et al., 2001). Introduction of steel braces allows for the introduction of shear walls with lattice scheme. In this way the internal rigidity in respect to the shear center will be balanced, avoiding big torsional effects and at the same time the resistance and the ductility of the structure to lateral loads would be increased.

Different possibilities of bracing elements can be used as shown in Figure 7.2 (a) and a practical application is shown in Figure 7.2 (b). This type of strengthening could be applied for the proposed structure with addition of the dampers for energy dissipation. In this way the global structural behavior would be improved.
Secondly, strengthening of the non-bearing walls could be done by reinforced cement coating, as indicated in Figure 7.3. This has been proposed by Tomaževič (WHE, 2002), for buildings of the same typology in Slovenia. It is recommended that for the inadequate lateral load resistance of the walls a reinforcement-cement coating could be applied as indicated in Figure 7.3. Application of this method would improve the lateral resistance and energy dissipation capacity of the system as stated by Tomaževič (WHE, 2002).

The old plaster should be removed and the surface of the wall is to be cleaned. Then a new reinforcement coating, made of two-layer cement coating with steel mesh, is placed on both wall surfaces. Steel anchors are used in order to connect the steel meshes on both sides of the wall. Adequate anchoring of the coating is essential for good performance of this method and as a result improvement of lateral resistance (Tomaževič, 1999).

![Figure 7.2 - (a) Different bracing systems (b) Practical application (Masonry building in Hungary)(ArcelorMittal)](image)

![Figure 7.3 - Seismic Strengthening Techniques-Reinforced Cement Coating (WHE, 2002)](image)
Possibilities of using ferro-cement, carbon fiber or polymer grids, see Figure 7.4, are another option which is being investigated and in particular polymer grids have been recognized by Eurocode 8 in 2003 (Ramiro, 2004). The grids made out of textile in the form of open meshes are named textile-reinforced mortars - TRM. The grid is organized in two perpendicular directions and cement is used as a connector opposed to resin which was previously used. A full composite action of the TRM material is achieved through mechanical interlock of the grip structure and the mortar that protrudes from the grid's openings (Papanicolaou, et al., 2010). Testing of TRM was firstly done for concrete and then it was extended to masonry structures. A large number of investigations were done by (Prota, et al., 2006), (Papanicolaou, et al., 2007), (Papanicolaou, et al., 2008). It is interesting to note that the experimental investigations conducted by Papanicolaou (Papanicolaou, et al., 2010) revealed that substantial gain in strength and deformation capacity is obtained by utilizing the TRM. It has been seen that even a better performance is seen for the out-of-plane behavior in respect to the FRP strengthening, conditioned that tensile fracture of the textile reinforcement does not occur. Regarding in-plane behaviour, it is more effective than FRP regarding deformation capacity, however lower performance was observed for effectiveness for strength (Papanicolaou, et al., 2010). It is easily applied and it does not require any special equipment. Another advantage of this method is that most of the works are to be done at the exterior part of the structure; just the windows have to be replaced, so the structure could be operational during the entire time. However, attention has to be paid regarding the possible unwanted distributions of seismic loads onto the structure elements.

![Figure 7.4 - Grid strengthening around the windows (Ramiro, 2004)](image)

As the floors are rigid, another possibility would be to introduce vertical confining elements (tie-columns) at all corners and wall intersections. In this way the integrity of the structural system would be upgraded, improving the resistance and energy dissipation capacity. This could be done in combination with other presented methods.

In order to conduct this type of strengthening it is necessary to remove the bricks on the part of the wall where the new tie-column will be located, having a new cogged shape of concrete, which
will enable a good connection of the existing brick wall and the new concrete column as indicated in Figure 7.5. Concrete needs to be removed from the horizontal tie-beam, and then the reinforcement of both elements are being connected. Usually, four 14-mm bars are used and anchored into the foundations. The depth of the tie-columns is usually less than the wall thickness, but it should not be less than 20cm (Tomaževič, 1999).

![Figure 7.5 - Placement of new tie-column in a brick-masonry wall(Tomaževič, 2004)](image)

### 7.3 Conclusion

Before applying any of the intervention methods to the structure it is important to take into consideration all the changes in the structural behavior that it can induce. It is necessary to conduct numerical calculations and experimental tests to prove the effectiveness of the strengthening method in a particular case. As well it is important to keep in mind that the structure is occupied, so the works which are to be conducted for the strengthening should have a minimum impact on the occupants. Availability of equipment and skillfulness of the workers to conduct certain strengthening techniques should be taken into account. Financial aspects as well as aesthetic implications that the strengthening would have on the structure and its surrounding should be carefully considered.

It can be clearly seen that the assessment of the safety of a structure is a multidisciplinary approach where many researchers from different fields have to work together in order to have a good and effective results and solutions.
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Chapter 8

CONCLUSIONS AND FUTURE WORK

8.1 Conclusions

The dissertation addresses the procedure for analysis of an existing masonry structure as prescribed by ICOMOS. Due to the lack of experimental data calculation of the material parameters was done on the basis of analytical formulations. However, due to a wide range of variations regarding strength and deformability characteristics of consistent materials of masonry conducting the experimental investigations is the most important aspect in order to conduct its reliable seismic resistance verification. On the basis of the experimental data calibration of the models will be conducted. Once again emphasis on the experimental tests necessary for the input data in the numerical model should be underlined.

Detailed assessment of the structure is done by numerical modeling using nonlinear analysis (Pushover and Time History Analysis) in two software programs DIANA, finite element based, and 3MURI, macro-element based. Comparison of the capacity curves, damage formation and propagation is conducted. Good correlation has been found between the different analyses conducted, especially in the damage pattern.

It has been noted that the major damage is located at the lower levels of the structure caused by discontinuity in strength and stiffness in the ground level. Due to a sudden change of stiffness between the basement and the ground floor severe concentration of stresses is observed, a big energy dissipation demand and as a consequence damage to those zones. The upper stories of the building had much smaller damage, damage decreasing towards the top. This kind of behavior has
been observed by post-earthquake observation and experiments for masonry structures with rigid horizontal diaphragm (Tomažević, 1999).

Redistribution of seismic loads has been seen and it is possible due to the available ductility of the walls, which enables the redistribution of the seismic loads from the most damaged walls to the less damaged and even to the undamaged ones. In this way energy is being dissipated during the seismic response of the building. The structure showed a typical shear behavior with the development of the diagonal cracks in the load-bearing walls. The analysis indicated that the size and position of the wall openings and its closeness to the corners of the walls has a strong effect on the in-plane resistance of masonry shear walls. Under seismic actions stress concentration is evident on opening zones, development and progression of cracks to a further deterioration of masonry structure.

It has been seen that the calculation done with a "less" sophisticated model are in a good correlation with the FEM calculations. As well it was able to "grasp" the damage pattern; not in the degree of DIANA calculations, but still quite good. On the basis of this it may be concluded that in this case calculation with 3MURI program could be recommended for future analysis of this type of structures, having quite good results with a less computation time. However, in the need for more precise data FEM should be utilized.

By the calculations done in 3MURI the structure is not capable of taking over the forces in the "±X" direction due to the weak non-load bearing walls in this direction. For this reason it would be necessary to strengthen the structure. Some proposals for strengthening were given, however they need to be checked (experimentally and numerically) before application. The strengthening methods have not been elaborated in detail, as this is out of the scope of this thesis.

8.2 Future Works

In order to have a more reliable input data regarding the material properties, it is planned to conduct experimental in-situ tests of the selected building. This data would be used in order to calibrate the model. As in this model all the structure elements were modeled as shell isotropic elements, another possibility would be to perform the analysis modeling by using shell elements with anisotropic behavior and comparing the results. This would be especially beneficial once the experimental tests including dynamic identification are conducted.

One of the major issues regarding the time history analysis was time limitation. For the earthquake analysis, applying the scaled Petrovac earthquake (step size of 0.01s) it took a whole week for the calculation. Second calculation was for the real Petrovac earthquake \( (a_g = 0.4g) \), unfortunately the convergence issue remained very present so the calculation was terminated. It would be interesting to conduct the analysis making the time steps smaller and see the implication. Additionally, the iteration method that was selected was Newton-Raphson, stiffness matrix being updated before every iteration, could be replaced with another iteration method and results could be compared.
It would be interesting to conduct and analyze the entire structure with the application of earthquake action in both directions as prescribed by the Eurocode 8 (CEN, 2003). On the basis of these results, appropriate strengthening methods should be proposed and calculation should be conducted in order to verify the strengthening procedure. Incremental dynamic analysis could be applied to the strengthened structure to understand better the behavior of the structure.
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Appendixes
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Appendix 1

Autocad drawings
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Principal strains for

time step $t_2 = 3.41 \text{s}$

and time step $t_4 = 7.65 \text{s}$